Confederation Line
Level 2 Proximity Study

Proposed Mixed Use Development

1047 Richmond Road – Ottawa, Ontario

Prepared for Fengate Asset Management

Report PG6108-1 Revision 2 dated September 3, 2024

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Fengate Asset Management (Fengate) to conduct a Confederation Line Level 2 Proximity Study for the proposed mixed-use development to be located at 1047 Richmond Road, in the City of Ottawa.

The objectives of the current study were to:

☐ Review all current information provided by the City of Ottawa with regards to the construction of the Confederation Line and New Orchard Station.

☐ Liaison between the City of Ottawa and the Fengate consultant team involved with the aforementioned project.

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

2.0 Development Details

The proposed development at 1047 Richmond Road will consist of two residential buildings. The first building is noted as Tower A, rising to 40 storeys. Details of Tower B were not available at the time of preparation of this report. It is further understood that both structures will share a common two-level underground parking structure. The underground parking structure will occupy the majority of the subject site, with the exception of a proposed park area located at the south-west corner of the site. The development will also include associated access lanes, amenity areas, and landscaped areas. The underground parking structure for the proposed building is to be set back approximately 1 m from the property line along Richmond Road. The design underside of the footing elevation is anticipated to be approximately 55.5 m and will be founded upon sound bedrock.

At the time of submission, it is understood that the City of Ottawa proposes that the Confederation Line and New Orchard Station will be constructed in proximity to the proposed development. Current design details regarding the Confederation Line and associated infrastructure were not provided to Paterson at the time of submission.

For purposes of the top of the tunnel and top of rail elevations, and footing levels for the station, City of Ottawa Confederation Line West LRT Extension drawings dated June 2, 2016, were used. For the purposes of the tunnel alignment, the rail implementation O-Train layer was referenced on GeoOttawa.

Therefore, several assumptions will be made assuming a ‘worst case’ scenario regarding the Confederation Line with respect to the proposed development. The following was assumed about the Confederation Line:

☐ The Confederation Line alignment will run in a north-east to south-west direction and will be located at the existing pathway and landscaped area between Richmond Road and Byron Avenue, approximately 19 m south-east of the subject site.

☐ The Confederation Line tunnel will be below ground, with the top of the tunnel located near the existing ground surface (65 m - geodetic elevation). The top of the rail elevation is anticipated to be approximately 58 m.

☐ Based on the subsurface profile at 1047 Richmond Road, bedrock is assumed to vary near the location of the rail line structure at approximate geodetic elevations of 61 and 64 m. Therefore, it is anticipated that the Confederation Line tunnel will be founded on bedrock.

☐ New Orchard Station is proposed to be located approximately 19 m south-east of the proposed development property line.

3.0 Construction Methodology and Impact Review

Paterson has prepared a construction methodology summary along with possible impacts on the adjacent segment of the Confederation Line and New Orchard Station based on the current building 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 bedrock blasting removal program. It is recommended that a vibration monitoring program be implemented to ensure 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 construction of the proposed Confederation Tunnel and New Orchard Station, 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 of the proposed development. The VMCP will also provide a guideline for assessing results against the relevant vibration impact assessment criteria and recommendations to meet the required limits.

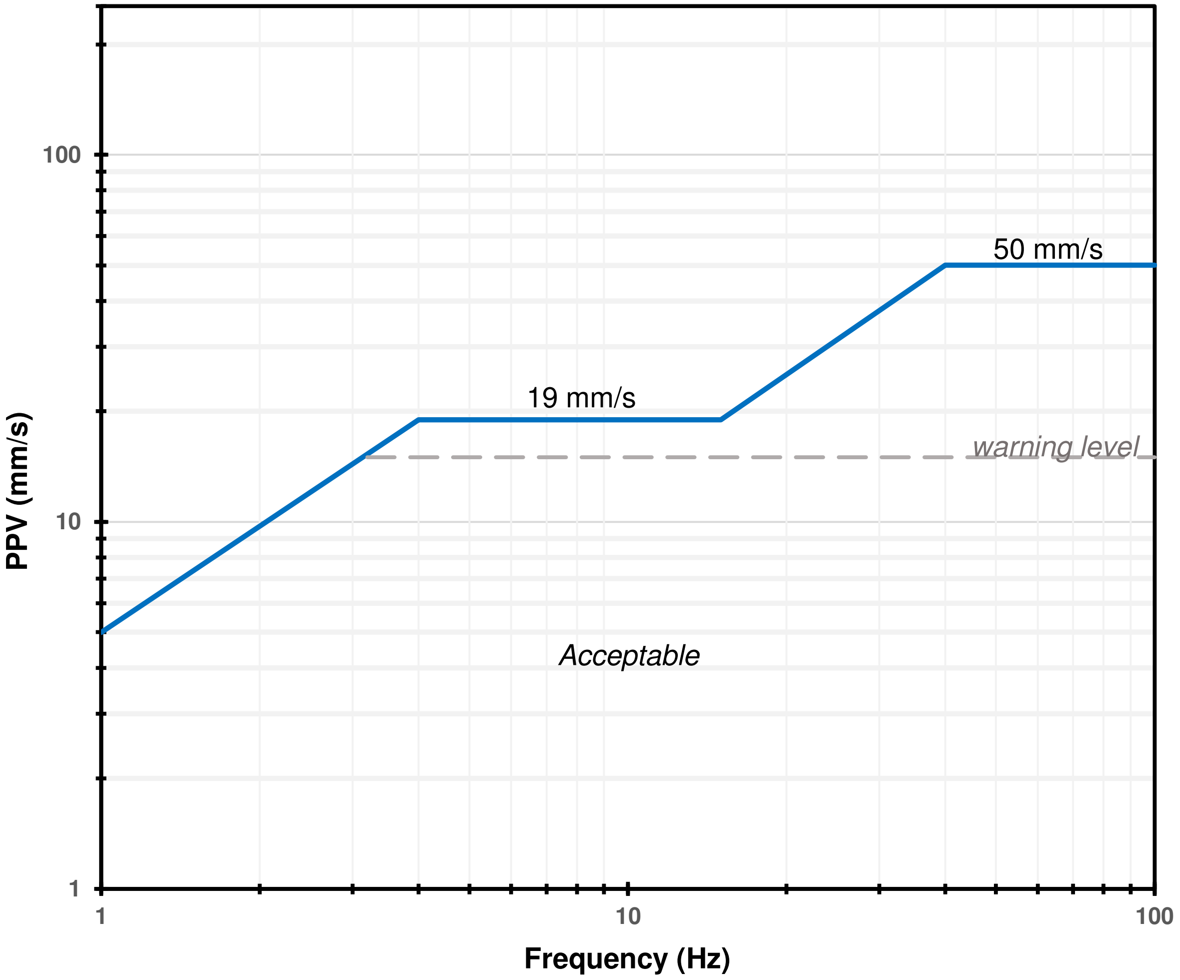
The monitoring program will incorporate real time results at the Confederation Tunnel and rail station, which is located in the general vicinity of the subject site. The monitoring equipment should consist of a tri-axial seismograph, capable of measuring vibration intensities up to 254 mm/s at a frequency response of 2 to 250 Hz. The monitoring equipment should be placed in the tunnel, if the tunnel has been constructed by the time blasting has commenced at 1047 Richmond Road. Otherwise, if the tunnel construction has not been completed at the time of blasting at 1047 Richmond Road, then the monitoring equipment should be placed at the ground surface at the nearest boundary of the Confederation Line alignment.

The location should be reviewed periodically throughout construction to ensure that the monitoring equipment remains within the closest radius to the construction activities. The vibration monitor locations should be approved by the project manager prior to installation. During construction, the vibration monitor will be relocated to 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 and New Orchard Station:

Figure 1 - USBM RI 8507 Vibration Limits



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 the 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 2 Confederation Line Proximity Study should be completed for the proposed development. A Level 2 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 and Level 2 study and the response location for each item:

Table 1 - List of Confederation Line Proximity Study Requirements

Level 1 Projects	**Response**
Site Plan (or Plan of Subdivision) of the development.	See Confederation Line Proximity Plan (Drawing No. PG6108-1 Revision 2 dated September 3, 2024) presented in Appendix A.
Floor Plans for the development.	See Floor Plans provided by RLA Architecture presented in Appendix D.
Development cross-section.	Refer to the Confederation Line Proximity Plan (Drawing No. PG6108-1 Revision 2 dated September 3, 2024) and Cross-Section A-A' (Drawing No. PG6108-2 Revision 1 dated September 3, 2023) presented in Appendix A.
A geotechnical Report prepared in accordance with the Geotechnical Investigation and Reporting Guidelines for Development Applications.	Refer to Geotechnical and Hydrogeological Investigation: prepared by Golder Associates Ltd. Report No. 21494078 Revision 2 dated June 27, 2023, presented in Appendix B.
Up to date property survey of existing and proposed property lines prepared to Strata Reference Plan Standards, signed and sealed by an Ontario Land Surveyor.	Refer to the property survey presented in Appendix A.
Utility Servicing Plan.	A Utility Servicing Plan will be provided prior to the Site Plan Agreement.
Stormwater Management Plan and Grading Plan.	A Stormwater Management Plan and Grading Plan will be provided prior to the site Plan Agreement.

Architectural Drawings and Landscape Plans.	Architectural Drawings and Landscape Plans will be provided prior to the Site Plan Agreement.
Noise and Vibration Study prepared in accordance with the City's Environmental Noise Control Guidelines.	Refer to the Roadway Traffic Noise and Vibration Feasibility Assessment Report No. 21-416 prepared by Gradient Wind Engineers & Scientists Addendum dated August 12, 2024, which is presented in Appendix C.
Level 2 Projects	**Response**
Fire/Life Safety and HVAC Report	A Fire/life safety and HVAC Report will be provided prior to the Site Plan Agreement.
Excavation Plan.	A temporary shoring system will be designed to at-rest earth pressures as required by the site Geotechnical Investigation Report. Temporary shoring drawings will be submitted once they are finalized.
Construction Plan.	A Construction Plan will be provided prior to the Site Plan Agreement Reference can further be made to the Construction Methodology and Impact Review presented in Appendix A.

We trust that this information satisfies your immediate request.

Best Regards,

Paterson Group Inc.

Nicole R.L Patey, B.Eng.

LICENSED PROFESSIONAL ENGINEER
Sept. 3, 2024
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Scott Dennis

Scott S. Dennis, P.Eng.

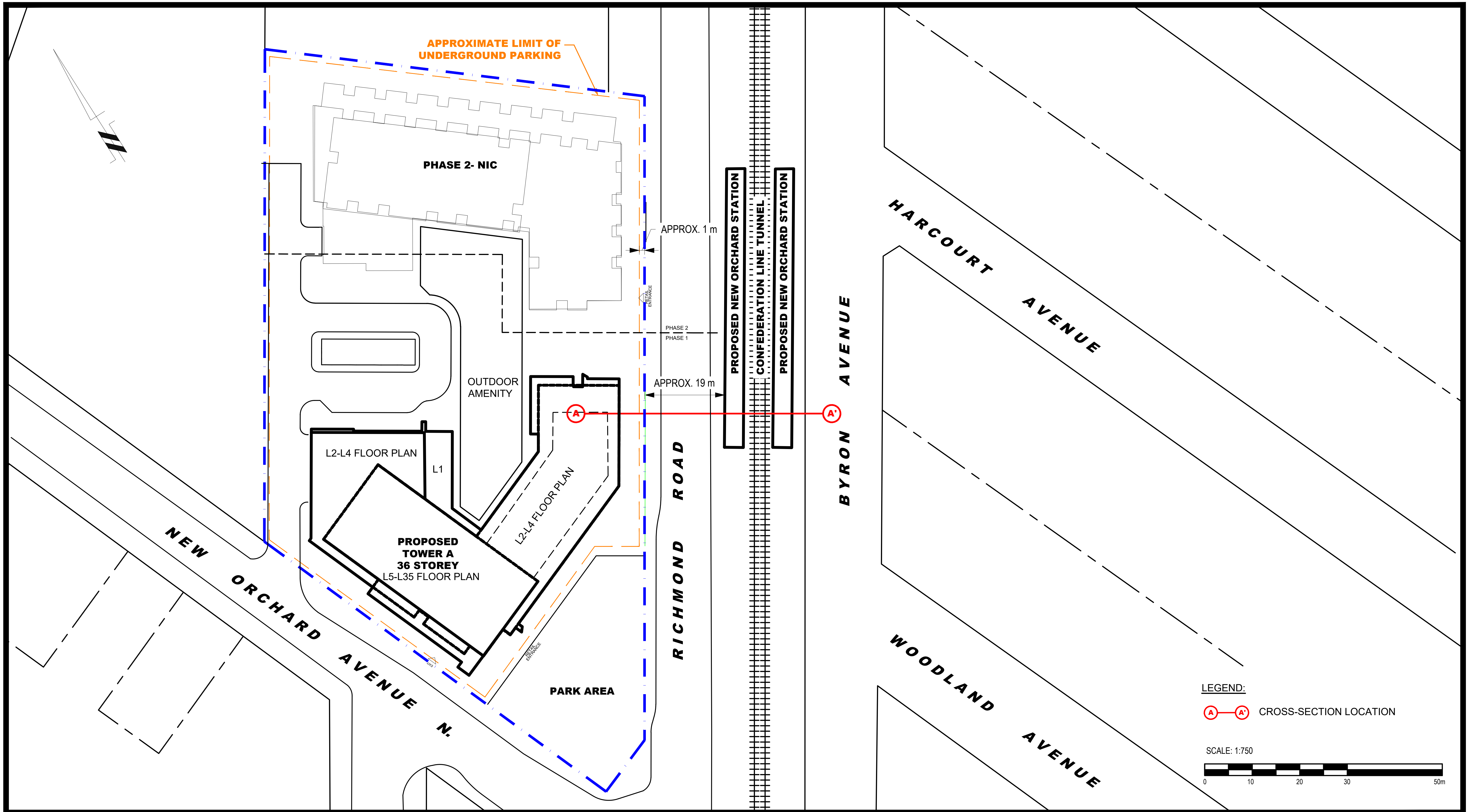
APPENDIX A

Confederation Line Proximity Plan

Cross Section A-A’

Topographic Survey Plan

Construction Methodology and Impact Review



PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
2	UPDATED CONCEPTUAL PLAN	09/03/2024	NP
1	UPDATED CONCEPTUAL PLAN	07/07/2023	NP

FENGATE ASSET MANAGEMENT

CONFEDERATION LINE PROXIMITY STUDY
PROPOSED MIXED USE DEVELOPMENT
1047 RICHMOND ROAD

OTTAWA, ONTARIO

Title:

CONFEDERATION LINE PROXIMITY PLAN

Scale: 1:750
Date: 01/2022
Drawn by: NFRV
Report No.: PG6108-1
Checked by: NP
Dwg. No.: PG6108-1
Approved by: DJG
Revision No.: 2

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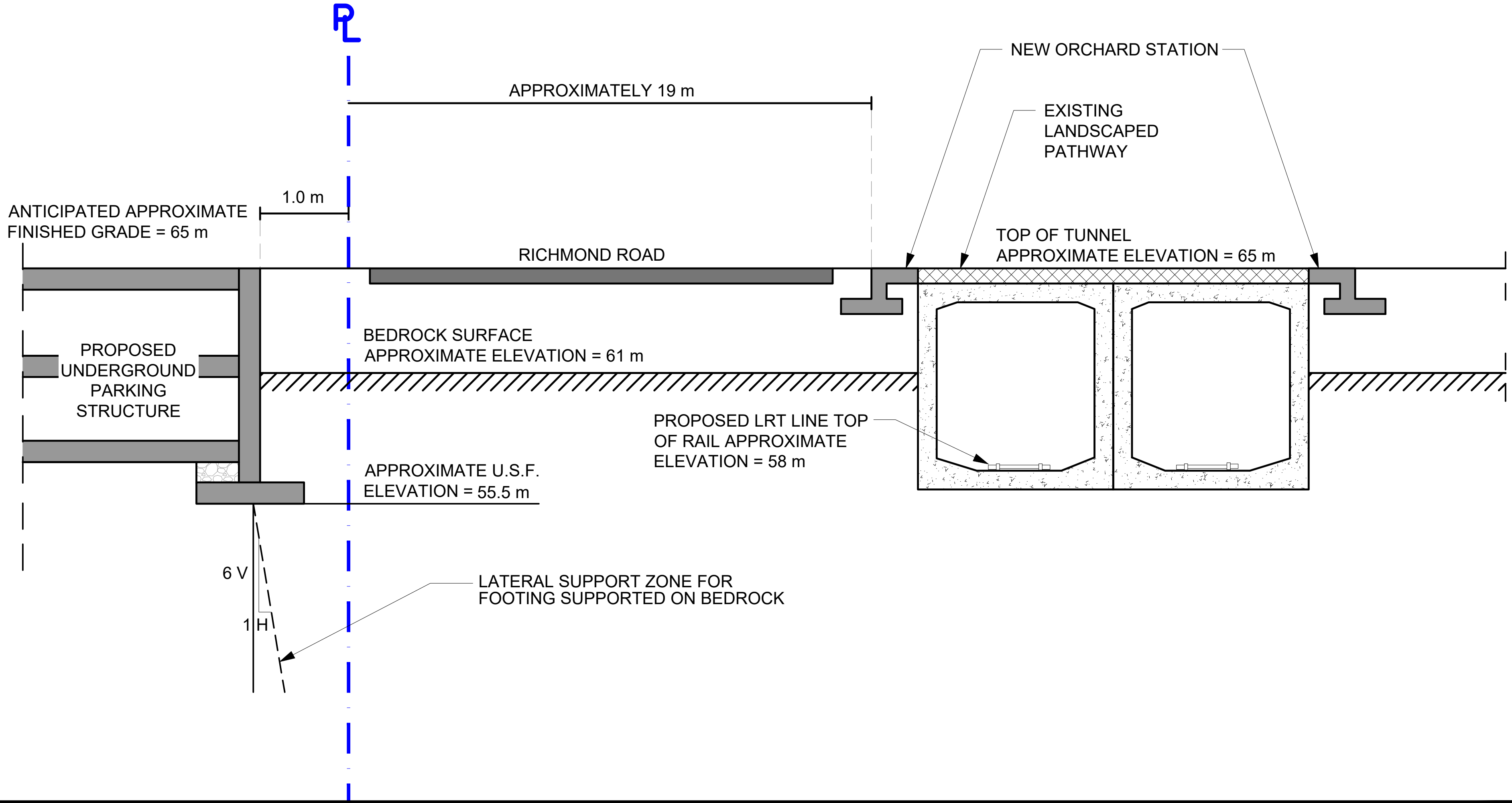
CROSS SECTION A-A'



PATERSON GROUP

FENGATE ASSET MANAGEMENT

CONFEDERATION LINE PROXIMITY STUDY
PROPOSED MIXED USE DEVELOPMENT
1047 RICHMOND ROAD

OTTAWA, **ONTARIO**

Title:

CROSS SECTION A-A'

Scale: N.T.S.	Date: 07/07/2023
Drawn by: NFRV	Report No.: PG6108-1
Checked by: NP	Drawing No.: PG6108-2
Approved by: DJG	Revision No.: 2

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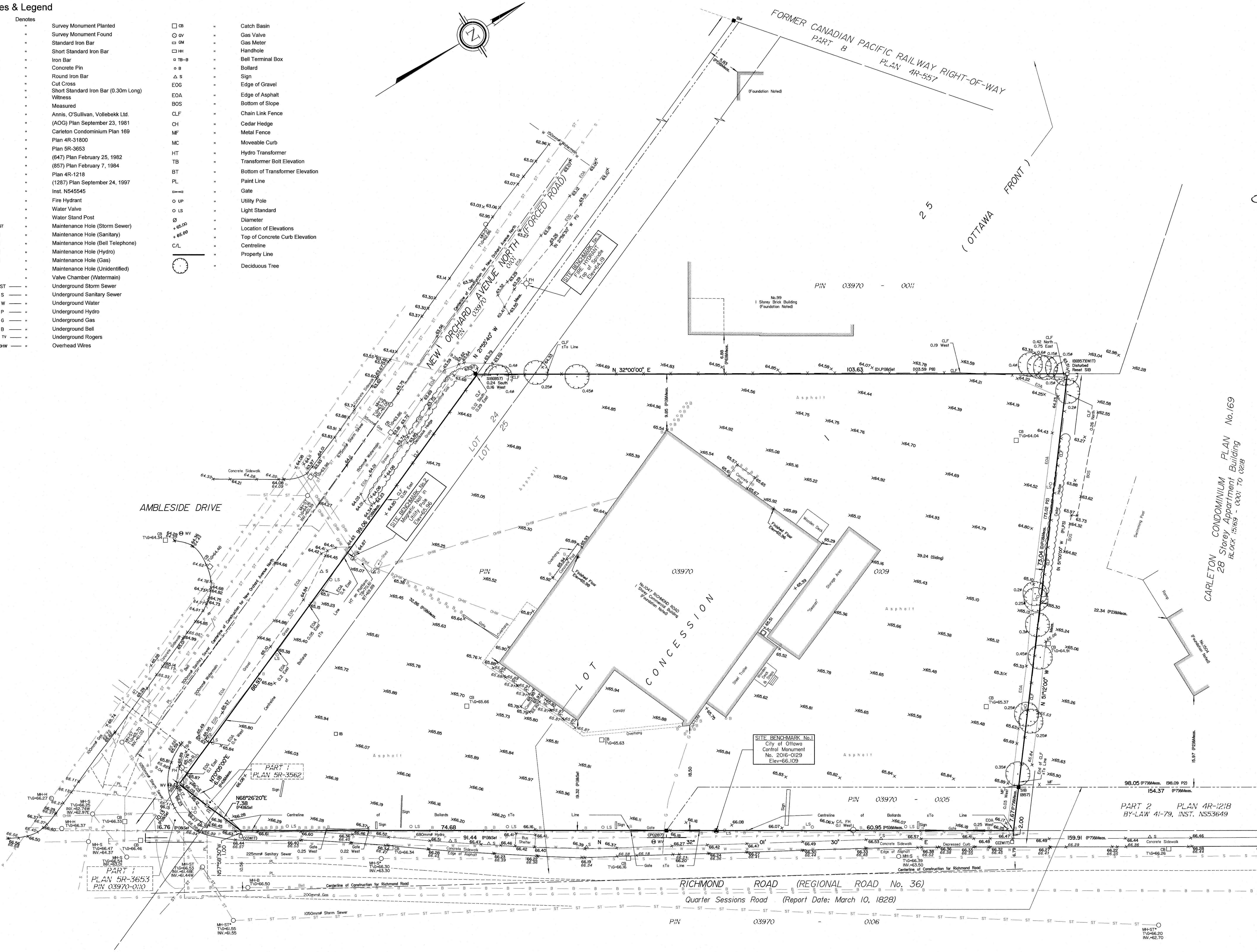
Notes & Legend

	Denotes	
─□─	"	Survey Monument Planted
─■─	"	Survey Monument Found
SIB	"	Standard Iron Bar
SSIB	"	Short Standard Iron Bar
IB	"	Iron Bar
CP	"	Concrete Pin
IBø	"	Round Iron Bar
CC	"	Cut Cross
SSIB*	"	Short Standard Iron Bar (0.30m Long)
(WIT)	"	Witness
Meas.	"	Measured
(AOG)	"	Annis, O'Sullivan, Vollebekk Ltd.
(P1)	"	(AOG) Plan September 23, 1981
(P2)	"	Carleton Condominium Plan 169
(P3)	"	Plan 4R-31800
(P4)	"	Plan 5R-3653
(P5)	"	(647) Plan February 25, 1982
(P6)	"	(857) Plan February 7, 1984
(P7)	"	Plan 4R-1218
(P8)	"	(1287) Plan September 24, 1997
(DI)	"	Inst. N545545
○ FH	"	Fire Hydrant
⊗ WV	"	Water Valve
∘ SP	"	Water Stand Post
○ MH–ST	"	Maintenance Hole (Storm Sewer)
○ MH-S	"	Maintenance Hole (Sanitary)
○ MH-B	"	Maintenance Hole (Bell Telephone)
○ MH-H	"	Maintenance Hole (Hydro)
○ MH-G	"	Maintenance Hole (Gas)
○ MH	"	Maintenance Hole (Unidentified)
⊖ VC	"	Valve Chamber (Watermain)
—— ST ——	"	Underground Storm Sewer
—— S ——	"	Underground Sanitary Sewer
—— W ——	"	Underground Water
—— P ——	"	Underground Hydro
—— G ——	"	Underground Gas
—— B ——	"	Underground Bell
—— TV ——	"	Underground Rogers
—— OHW ——	"	Overhead Wires

□ CB	"	Catch Basin
⊗ GV	"	Gas Valve
▭ GM	"	Gas Meter
▭ HH	"	Handhole
▫ TB–B	"	Bell Terminal Box
∘ B	"	Bollard
△ S	"	Sign
EOG	"	Edge of Gravel
EOA	"	Edge of Asphalt
BOS	"	Bottom of Slope
CLF	"	Chain Link Fence
CH	"	Cedar Hedge
MF	"	Metal Fence
MC	"	Moveable Curb
HT	"	Hydro Transformer
TB	"	Transformer Bolt Elevation
BT	"	Bottom of Transformer Elevation
PL	"	Paint Line
▻◅	"	Gate
○ UP	"	Utility Pole
○ LS	"	Light Standard
Ø	"	Diameter
+ 65.00	"	Location of Elevations
+ 65.00	"	Top of Concrete Curb Elevation
C/L	"	Centreline
━━━	"	Property Line
○	"	Deciduous Tree



TOPOGRAPHICAL PLAN OF SURVEY OF

PART OF LOTS 24 AND 25
CONCESSION 1 (OTTAWA FRONT)
GEOGRAPHIC TOWNSHIP OF NEPEAN
CITY OF OTTAWA

Surveyed by Annis, O'Sullivan, Vollebekk Ltd.

Scale 1 : 300

12 9 6 3 0 6 12 Metres

Metric

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

Surveyor's Certificate

I CERTIFY THAT :

1. This survey and plan are correct and in accordance with the Surveys Act and the Surveyors Act and the regulations made under them.
2. The survey was completed on the 29th day of September, 2021.

Oct. 1, 2021
Date

E. H.
E. H. Herweyer
Ontario Land Surveyor

Distances shown on this plan are ground distances and can be converted to grid distances by multiplying by the combined scale factor of 0.999982.

For bearing comparisons, a rotation of 0°01'00" counter-clockwise was applied to bearings on P1,P4 & P5.
For bearing comparisons, a rotation of 0°19'10" counter-clockwise was applied to bearings on P6.
For bearing comparisons, a rotation of 0°01'30" counter-clockwise was applied to bearings on P2 & P7.

Coordinates are derived from Can-Net 2016 Real Time Network GPS observations referenced to Specified Control Points 01919680005 and 01919750705, MTM Zone 9 (76°30' West Longitude) NAD-83 (original).

ASSOCIATION OF ONTARIO
LAND SURVEYORS
PLAN SUBMISSION FORM
V-16989
THIS PLAN IS NOT VALID UNLESS
IT IS AN EMBOSSED ORIGINAL
COPY ISSUED BY THE SURVEYOR
In accordance with
Regulation 1026, Section 29 (3).

ELEVATION NOTES

1. Elevations shown are geodetic derived from City of Ottawa Monument No.2016-0129 with a published elevation of 66.109m and are referred to the CGVD28 geodetic datum.
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.

UTILITY NOTES

1. This drawing cannot be accepted as acknowledging all of the utilities and it will be the responsibility of the user to contact the respective utility authorities for confirmation.
2. Only visible surface utilities were located.
3. Underground utility data derived from City of Ottawa utility sheet reference E-01-09.
4. Sanitary and storm sewer grades and inverts were derived from Plan 2353 (Sheet 4, 5 & 6 of 6) & Plan I-10-c-1.
5. A field location of underground plant by the pertinent utility authority is mandatory before any work involving breaking ground, probing, excavating etc.
6. Manholes noted with a " * " were not located in the field due to field conditions and construction work. The location of these manholes is derived per the noted drawings above.

Ontario Land Surveyors
ANNIS, O'SULLIVAN, VOLLEBEKK LTD.
14 Concourse Gate, Suite 500
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Email: Nepean@aovltd.com

Job No. 21985-21 FengateDvlpmnt PrtLts24 25 Con1 OF T F | DG

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 proposed building footprints will need to be shored in order to complete the construction of the underground parking levels. The shoring system is anticipated to consist of a soldier pile and lagging system.	Vibration issues during shoring system installation.	Design of the temporary shoring system, in particular vibrations during installation, will take into consideration the presence of the Confederation Line and New Orchard Station. Installation of the shoring system is not anticipated to have an adverse impact on the Confederation Line or New Orchard Station, 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 PG6108-1 Revision 2 dated August 30, 2024.
Item B - Bedrock Blasting and Removal Program - Blasting of the bedrock will be required for the proposed development and parking garage structure construction. It is expected that bedrock removal is required based on the current design concepts for the proposed development.	Structural damage of Confederation Line due to vibrations from blasting program.	Structural damage to the Confederation Line and New Orchard Station during bedrock blasting and removal is not anticipated, nonetheless, a vibration monitoring device is recommended to be installed in the tunnel 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 PG6108-1 Revision 2 dated August 30, 2024.
Item C - Construction of Footings and Foundation Walls - The proposed building will include 2 levels of underground parking. Therefore, the footings will be placed over a clean, surface sounded dolostone with interbedded shale, limestone, and sandstone bedrock bearing surface.	Building footing loading on adjacent Confederation Line, and excavation within the lateral support zone of the Confederation Line.	Due to the distance between the proposed building and the Confederation Line and New Orchard Station, the zone of influence from the proposed footings will not intersect the rail line structure and associated infrastructure. Further, although the underground parking levels for the proposed building will extend approximately 9.5 m below existing ground surface, due to the approximate 19 m distance between the proposed building and rail line infrastructure, the building excavation will not impact the lateral support zone of the Confederation Line and New Orchard Station.

APPENDIX B

Geotechnical and Hydrogeological Investigation Report:

Prepared By Golder Associates Ltd.

Report No. 21494078 Revision 2 dated June 27, 2023

REPORT

GEOTECHNICAL AND HYDROGEOLOGICAL INVESTIGATION

1047 RICHMOND ROAD, OTTAWA, ONTARIO

Submitted to:

1047 Richmond Nominee Inc.
2275 Upper Middle Road East
Suite 700, Oakville Ontario L6H 0C3

Submitted by:

Golder Associates Ltd.

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21494078_Rev2

June 27, 2023

Distribution List

1 e-copy - 1047 Richmond Nominee Inc.

1 e-copy - Golder

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FIGURES

APPENDICES

1.0 INTRODUCTION

This report presents the results of geotechnical and hydrogeological investigation carried out at the site of a proposed residential development located at 1047 Richmond Road in Ottawa, Ontario.

The purpose of this investigation was to assess the general subsurface conditions at the site by means of a limited number of boreholes. Based on an interpretation of the factual information obtained, a general description of the soil, bedrock, and groundwater conditions is presented. These interpreted subsurface conditions and available project details were used to provide engineering guidelines on the geotechnical design aspects of the project, including construction considerations which could influence design decisions.

The reader is referred to the "*Important Information and Limitations of This Report*" which follows the text but forms an integral part of this document.

2.0 DESCRIPTION OF PROJECT AND SITE

The site of the proposed development is located at 1047 Richmond Road in Ottawa, Ontario. The site is about 2.5 acres and is currently occupied by a single-story commercial building (a car dealership) which consists of a building located in approximately the middle of the site, surrounded by parking areas.

The site is bordered to the east by a residential tower, to the south by Richmond Road, to the west by New Orchard Avenue and the north by a low-rise residential building. The project limits and the location of the proposed development are shown on Figure 1.

Based on the updated design scheme provided to Golder, the site will be developed into three residential buildings including two 38 and 40 storey towers with 6 storey podiums (Towers A and B), and a smaller 6 storey building (Tower C). The proposed development also includes a new park, an outdoor amenity and various access roadways and parking areas. The development includes three levels of underground parking which will encompass the entire development site excluding the future park in the southwest corner.

3.0 PROCEDURE

The field work for the current geotechnical investigation was carried out between September 21 and 30, 2021, in conjunction with the Phase 2 Environmental Site Assessment (ESA). During that time, ten boreholes (numbered 21-01 to 21-10) were advanced at the approximate locations shown on Figure 1.

The boreholes were advanced with a track-mounted hollow stem auger drill rig supplied and operated by CCC Geotechnical and Environmental Drilling of Ottawa, Ontario. The boreholes were advanced to depths ranging from 7.6 m to 15.5 m below the existing ground surface. Refusal to augering was encountered at all of the boreholes at depths ranging from 1.6 to 4.8 m below the existing ground surface.

Upon encountering refusal to augering, boreholes 21-01 to 21-05 were further advanced to a depth of about 7.6 m into the bedrock using pneumatic hammer rock drilling. No rock cores were recovered from these boreholes. Boreholes 21-06 to 21-10 were further advanced for 7.5 and 13.9 m into the bedrock using rotary diamond drilling techniques while retrieving HQ sized core.

Standard Penetration Tests (SPTs) were carried out within the overburden at various intervals of depth in general conformance with ASTM D 1586. Soil samples were recovered using 35 mm inside diameter split-spoon sampling equipment.

Monitoring wells were sealed in all the boreholes (with the exception of 21-08) to allow for subsequent measurements of stabilized groundwater levels as well as to perform in-situ hydraulic conductivity testing. The monitoring wells consist of 51 mm inside diameter rigid PVC pipe with 3.0 m long slotted screen sections, installed within silica sand backfill and sealed by a section of bentonite pellet backfill. Measurement of the groundwater levels was completed on October 05, 2021.

At borehole 21-08, a 63.5 mm inside diameter rigid PVC casing was grouted over the full depth of the borehole to allow for Vertical Seismic Profile (VSP) testing to determine the shear wave velocity profile of the soil and rock.

The fieldwork was supervised by Golder staff who logged the boreholes, directed the in-situ testing, and collected the soil and rock samples retrieved in the boreholes. The samples obtained during the fieldwork were brought to our laboratory for further examination and laboratory testing.

The laboratory testing included determination of natural water content, grain size distribution on selected soil samples, and Uniaxial Compressive Strength (UCS) testing on selected bedrock samples.

Two samples of soil from boreholes 21-06 and 21-10 were submitted to Eurofins Environment Testing for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements.

The borehole locations were marked in the field and surveyed by Golder. The positions and ground surface elevations at the borehole locations were determined using a Trimble R8 GPS survey unit. The Geodetic reference system used for the survey is the North American Datum of 1983 (NAD83). The borehole coordinates are based on the Universal Transverse Mercator (UTM Zone 09) coordinate system. The elevations are referenced to Geodetic datum (CGVD28).

4.0 SUBSURFACE CONDITIONS

4.1 General

The following information on the subsurface conditions is provided in this report:

- Borehole records are provided in Appendix A
- Laboratory test results are provided in Appendix B, and on the relevant borehole records
- Rock core photographs are provided in Appendix C
- Results of the basic chemical analyses are provided in Appendix D
- Results of geophysical testing are provided in Appendix E
- Results of in-situ hydraulic conductivity testing are provided in Appendix F

In general, the subsurface conditions at this site consist of fill, or fill underlain by a deposit of glacial till which is in turn underlain by dolostone bedrock with shale, limestone, and sandstone interbeds.

The following sections present a more detailed overview of the subsurface conditions encountered during the field investigation.

4.2 Pavement Structure

Pavement structure was encountered in all of the boreholes. The pavement structure consists of 50 to 100 mm of asphaltic concrete overlying 110 to 540 mm thick granular base and subbase layers.

4.3 Fill

Fill was encountered below the pavement structure at all of the borehole locations. The fill consists of sand, silty sand to gravelly silty sand. The fill extends to depths ranging between 0.9 and 2.4 m below the existing ground surface at the borehole locations.

The results of SPT tests carried out within the fill gave ‘N’ values ranging from 1 to 35 blows per 0.3 m of penetration, indicating a very loose to dense (but typically compact) state of packing.

The measured natural water content of two samples of fill were about 10%.

The result of grain size distribution testing carried out on two sample of the fill is provided on Figures B-1 and B-2 in Appendix B.

4.4 Glacial Till

A discontinuous deposit of glacial till exists below the fill, and was encountered in the boreholes 21-04, 21-05, 21-08, and 21-10. The glacial till generally consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand. The glacial till deposit (where encountered) was fully penetrated to depths ranging from 3.1 to 4.8 m below the existing surface.

The results of standard penetration tests carried out within the glacial till gave SPT ‘N’ values ranging from 46 to greater than 50 blows per 0.3 m of penetration, indicating a dense to very dense state of packing. High SPT ‘N’ values can also be indicative of cobbles and boulders and not the density of the soil matrix.

The measured natural water content of eight samples of glacial till ranged from 7 to 14%.

The result of grain size distribution testing carried out on one sample of the glacial till is provided on Figure B-3 in Appendix B.

4.5 Bedrock

Refusal to augering was encountered in all of the boreholes at depths ranging from 1.6 to 4.8 m below the existing ground surface. The bedrock was cored in boreholes 21-06 to 21-10 to depths ranging between 9.4 and 15.5 m below the existing ground surface.

In boreholes 21-02, 21-03 and 21-06 to 21-09, a zone of weathered/disturbed bedrock (which could be penetrated by augering and SPT sampling) was encountered at depths ranging from 0.9 to 3.1 m. The thickness of this zone was about 0.5 to 1.7 m at these borehole locations.

The following table summarizes the ground surface elevations, depth to bedrock, bedrock surface elevations and core lengths at the borehole locations:

Borehole Number	Ground Surface Elevation (m)	Bedrock Depth (m)	Core Length (m)	Bedrock Surface Elevation (m)
21-01	65.7	1.8	N/A[1]	63.9
21-02	65.5	3.1	N/A[1]	62.4
21-03	65.2	3.1	N/A[1]	62.1
21-04	65.1	3.7	N/A[1]	61.4
21-05	65.5	3.7	N/A[1]	61.8
21-06	65.0	1.9	7.5	63.1
21-07	66.1	1.6	8.1	64.4
21-08	64.6	3.2	12.3	61.4
21-09	65.9	1.7	13.8	64.2
21-10	65.9	4.8	10.7	61.1

Note: [1] No bedrock core recovery due to pneumatic hammer rock drilling

The bedrock encountered in the cored boreholes typically consists of medium grey dolostone with shale, limestone, and sandstone interbeds to a depth ranging from 9.1 to 13.2 m below the existing ground surface.

In boreholes 21-08 to 21-10, light grey sandstone with thin partings of shale was encountered below the dolostone layer at depths of 9.1 and 13.2 m below the existing ground surface, respectively.

Rock Quality Designation (RQD) values for dolostone and sandstone bedrock measured in the boreholes range from about 0 to 100% but are more typically in the range of 75 to 100% indicating good to excellent quality rock. In general, the RQD values increase with depth.

Nine UCS tests were carried out on core specimens of the bedrock, and measured UCS values range from 86 to 144 MPa, indicating strong rock. The results of the UCS tests are included in Appendix B. The UCS values are also presented in Figures B-4 and B-5 in Appendix B.

Photographs of the recovered bedrock core are presented in Appendix C.

4.6 Groundwater Conditions

In-situ hydraulic conductivity testing was carried out in monitoring wells installed in Boreholes 21-01 through 21-07, 21-09 and 21-10. An insufficient amount of water was present at monitoring wells 21-01, 21-07 and 21-09 to allow for testing to occur. The testing method at monitoring well 21-06 involved the rapid removal of water from the well using a dedicated foot valve and tubing, and measurement of the recovery of the water level over time. At monitoring wells 21-02, 21-03, 21-04, 21-05 and 21-10, a solid cylindrical slug was lowered quickly into the well and the change of the water level over time was recorded.

The data collected during the falling-head tests on monitoring wells 21-02, 21-03, 21-04, 21-05 and 21-10 were analyzed using the Hvorslev method (Hvorslev, 1951) to provide an estimate of the horizontal hydraulic conductivity of the bedrock adjacent to the test intervals. During the rising head test on monitoring well 21-06, the groundwater level was drawn down into the monitoring well screen; therefore, these data were analysed using the Bouwer and Rice (1976)[1]. The relevant calculations are included in Appendix F.

Summary of In-situ Hydraulic Conductivity Testing Results

Borehole	Geologic Unit of Screened Interval	Depth of Screened Interval (m)	Ground Surface Elevation (m)	Groundwater Level		Date of Measurement	Hydraulic Conductivity (cm/s)
				Depth below ground surface* (m)	Elevation (m)		
21-01	Dolostone	4.57 - 7.62	65.73	7.60	58.13	Oct. 5, 2021	Insufficient water for testing
21-02	Dolostone	3.96 - 7.01	65.46	3.32	62.14	Oct. 5, 2021	$2x10^{-3}$*
21-03	Dolostone	4.57 - 7.62	65.24	3.22	62.02	Oct. 5, 2021	$1x10^{-4}$*
21-04	Dolostone	4.57 - 7.62	65.09	2.70	62.39	Oct. 5, 2021	$4x10^{-4}$*
21-05	Dolostone	4.57 - 7.62	65.47	3.94	61.53	Oct. 5, 2021	$2x10^{-4}$*
21-06	Dolostone	6.33 - 9.38	65.00	6.84	58.16	Oct. 5, 2021	$1x10^{-6**}$
21-07	Dolostone	6.68 - 9.73	66.07	9.34	56.73	Oct. 5, 2021	Insufficient water for testing
21-09	Dolostone	6.63 - 9.68	65.90	Dry	Dry	Oct. 5, 2021	Not tested
21-10	Sandstone	12.40 - 15.45	65.89	8.85	57.04	Oct. 5, 2021	$1x10^{-3}$*

Notes: *analysed using Hvorslev (1951) method
**analysed using Bouwer and Rice (1976) method

The groundwater level measurement results indicate that the groundwater level in the bedrock ranged from 2.7 m to 9.3 m below the existing ground surface. The results of the rising head test analyses indicate that the hydraulic conductivity (K) of the bedrock at the borehole locations ranged from about $1×10^{-6}$ to $2×10^{-3}$ cm/s.

It is expected that the groundwater levels will be subject to fluctuations both seasonally and as a result of precipitation events. Groundwater levels may also be currently influenced by the excavations currently taking place on the south side of Richmond Road and may change as construction in that area is completed.

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[1] Bouwer, H. and R.C. Rice, 1976. A slug test method for determining hydraulic conductivity of unconfined aquifers with completely or partially penetrating wells, Water Resources Research, vol. 12, no. 3, pp.423-428.

4.7 Corrosion Testing

Two samples of soil from boreholes 21-06 and 21-10 were submitted to Eurofins Environment Testing for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements. The results of this testing are provided in Appendix D and are summarized below:

Borehole Number	Sample Number	Depth Intervals (m)	Chlorides (%)	Sulphates (%)	pH	Resistivity (Ohm-cm)
21-06	2	1.5 – 1.9	0.007	<0.01	8.9	4,350
21-10	3	2.3 – 2.7	<0.002	0.01	8.4	6,670

5.0 DESIGN AND CONSTRUCTION CONSIDERATIONS

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the available information described herein and project requirements. The information in this portion of the report is provided for planning and design purposes for the guidance of the design engineers and architects. Where comments are made on construction, they are provided only to highlight aspects of construction which could affect the design of the project. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the factual information for construction, and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, safety, and equipment capabilities, costs, sequencing and the like.

The reader is referred to the "*Important Information and Limitations of This Report*" which follows the text of this report but forms an integral part of this document.

5.2 Seismic Considerations

5.2.1 Seismic Zone

The site falls within the Western Québec Seismic Zone (WQSZ) according to the Geological Survey of Canada. The WQSZ constitutes a large area that extends from Montréal to Témiscaming, and which encompasses the Ottawa area. Within the WQSZ recent seismic activity has been concentrated in two subzones; one along the Ottawa River and another more active subzone along the Montréal-Maniwaki axis. Historical seismicity within the WQSZ includes the 1935 Témiscaming event which had a magnitude (i.e., a measure of the intensity of the earthquake) of 6.2 and the 1944 Cornwall-Massena event which had a magnitude of 5.6. In comparison to other seismically active areas in the world (e.g., California, Japan, New Zealand), the frequency of earthquake activity within the WQSZ is significantly lower but there still exists the potential for significant earthquake events to be generated.

5.2.2 Site Class

Vertical Seismic Profiling (VSP) geophysical testing was carried out within borehole 21-08 to evaluate the average shear wave velocity of the upper 30 m of soil/bedrock at the site (see Figure 1 for the VSP testing location). The results of the shear wave velocity test are included in Appendix E.

The VSP test results indicate that the average shear wave velocity in the upper 30 m from the bedrock surface (V_{s30}) was about 1,700 m/s. Based on this value, it is considered that a Site Class "A" designation is appropriate for the site for all structures founded on rock.

5.3 Frost Protection

The presence of frost-susceptible soils within the frost penetration depth will require that isolated, unheated exterior footings adjacent to surfaces which are cleared of snow cover during winter months be provided with a minimum of 1.8 m of earth cover (or equivalent insulation). Exterior foundations of heated structures should be provided with a minimum of 1.5 m of earth cover (or equivalent insulation).

If sufficient earth cover cannot be provided, foundation insulation details can be provided during detailed design.

The foundations of the proposed residential towers and podiums with three underground parking levels are expected to be placed on or within the bedrock at depth, which is unlikely to be highly frost susceptible and will be below the depth of frost. As such, frost protection is not required for the footings founded on bedrock at depth.

5.4 Foundations

Based on our understanding of the proposed development (in particular the three levels of underground parking which cover the entire footprint of the buildings) it is assumed that the foundations for the high-rise towers as well as the mid-rise podiums would likely consist of spread footings placed on the relatively shallow bedrock.

5.4.1 Bearing Resistance

In general, subsurface conditions encountered during the investigation consist of fill, or fill underlain by glacial till over dolostone/sandstone bedrock. It is considered that the proposed residential towers and podiums can be supported on spread footings placed on or within the competent bedrock.

The factored bearing resistance at Ultimate Limit States (ULS) for spread footing foundations founded on bedrock may be taken as 4,800 kPa for all areas where the foundations are three stories below the existing grade (which includes all of the currently proposed buildings).

These values are applicable provided that the bedrock surface is acceptably cleaned of soil and loose bedrock (i.e., any bedrock that can be easily removed with a hydraulic excavator). The settlement of footings at the corresponding service (unfactored) load levels will be less than 25 mm. Serviceability Limit States (SLS) conditions generally do not govern foundation design in rock.

Should there be localized locations within the excavation where the bedrock surface, following excavation and removal of any weathered rock, is below the planned founding level, then the footing level may be lowered such that the footing will bear directly on the unweathered bedrock. Alternatively, the subgrade could be raised to the underside of the foundation using mass concrete.

The bedrock surfaces should be inspected by qualified geotechnical personnel to confirm that the expected bearing material has been exposed and that the bearing surface has been adequately prepared and cleaned.

5.4.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the clean surface of sound bedrock could be considered. For cast-in-place concrete footings bearing directly onto the bedrock surface, the coefficient of friction, tan φ', may be taken as follows:

- Cast-in-place concrete footing to clean sound bedrock: tan φ' = 0.70

The sliding resistance value is unfactored, and a resistance factor of 0.8 would need to be applied to the sliding resistance in accordance with limit states design.

The resistance to lateral loads could be increased by constructing a shear-key at the bottom of the footings if needed. The design of shear keys would require a specific analysis taking into consideration the magnitude of the horizontal loading, the magnitude of the vertical loading, and any variations in the bearing pressure due to overturning moments.

5.5 Rock Anchors

Rock anchors could potentially be used to resist uplift and/or overturning of the foundation. Rock anchors should consist of grouted anchors installed into the bedrock at depth.

Rock anchors are typically installed in a borehole that is drilled with air-percussion equipment or with rotary diamond drilling equipment with water circulation; those drilling methods can fairly penetrate through boulder/cobblery ground such as exists on this site. A cased hole would be drilled through the overburden (if present) with a socket drilled into the bedrock, the steel anchor inserted, and then the annular space around the bar filled with grout.

Because the rock anchors would be permanent elements of the foundations, a “double corrosion protection” system should be provided.

In designing grouted rock anchors, consideration should be given to four possible anchor failure modes.

i) failure of the steel tendon or top anchorage

ii) failure of the grout/tendon bond

iii) failure of the rock/grout bond

iv) failure within the rock mass, or rock cone pull-out

Potential failure modes i) and ii) are structural and are best addressed by the structural engineer. Adequate corrosion protection of the steel components should be provided to prevent potential premature failure due to steel corrosion.

For potential failure mode iii), the factored bond stress at the concrete/rock interface may be taken as *1,000 kPa* for ULS design purposes. This value should be used in calculating the resistance under ULS conditions. If the response of the anchor under SLS conditions needs to be evaluated, for a preliminary assessment it may be taken as the elastic elongation of the unbonded portion of the anchor under the design loading.

For potential failure mode iv), the preliminary resistance is calculated based on the unit weight (undrained) of the potential mass of rock and soil which could be mobilized by the anchor, and resistance to shear of the rock mass. This is typically considered as the mass of rock included within a cone (or wedge for a line of closely spaced anchors) having an apex at the tip of the anchor and having an apex angle of 60 degrees. For each individual anchor, the ULS factored geotechnical resistance can be calculated based on the following equation:

$$Q_r = \varphi \frac{\pi}{3} \gamma' D^3 \tan^2(\theta)$$

Where:

Qr = Factored uplift resistance of the anchor (kN)

φ = Resistance factor (use 0.4)

γ' = Effective unit weight of rock (use 16 kN/m^3 below the groundwater level)

D = Anchor length in metres

θ = One-half of the apex angle of the rock failure cone (use 30°)

Where the anchor load is applied at an angle to the vertical, the anchor capacity should be reduced as follows:

$$Q_r' = Q_r \cos(\alpha)$$

Where:

Qr' = Factored uplift resistance of the anchor subject to inclined load (kN)

Qr = Factored uplift resistance of the anchor (kN)

α = Angle between the load direction and the vertical

For a group of anchors or for a line of closely spaced anchors, the resistance must consider the potential overlap between the rock masses mobilized by individual anchors. In the case of group effects for a series of rock anchors in a rectangle with width “a” and length “b” installed to a depth “D”, the equation for the volume of the truncated trapezoid failure zone would be as follows:

$$V = \frac{4}{3} D^3 sin^2\Theta + aD^2 \sin\Theta + bD^2 \sin\Theta + abD$$

Where:

V = Volume of the truncated trapezoid failure zone (m3)

D = Depth of anchor group (m)

a = Width of anchor group (m)

b = Length of the anchor group (m)

θ = One-half of the apex angle of the rock failure cone, use 30°

The ULS factored geotechnical resistance for the truncated trapezoid failure formed by the group of anchors can then be calculated based on the following equation:

$$Q_r = \varphi \gamma' V$$

Where:

Q_r = Factored uplift resistance of the anchor (kN)

φ = Resistance factor, use 0.4

γ' = Effective unit weight of rock, use 16 kN/m^3

V = Volume of truncated trapezoid (m^3)

The method described above does not explicitly consider the tensile strength of the rock that must be overcome prior to mobilization of the weight of the rock mass. If required, the tensile strength of the rock mass can be assessed based on the unconfined compressive strength, recovery, and quality of bedrock core obtained. This assessment, however, requires a detailed understanding of the anchor lengths, geometry, loads, etc. and would need to be completed during detailed design.

It is recommended that proof load tests be carried out on the anchors to confirm their resistance. The proof load tests should be carried out in accordance with OPSS 942 *(Prestressed Soil and Rock Anchors).*

A geotechnical professional should be present during the installation and testing of the anchors. Care must be taken during grouting to ensure that the grouting pressure is sufficient to bond the entire length of the grouted area with minimum voids. Confirmation of sufficient embedment into the rock beneath the foundations should be carried out to make sure that the anchors are being installed in rock of adequate quality. The anchor holes must be thoroughly flushed with water to remove all debris and rock flour. It is essential that rock flour be completely removed from the holes to be grouted to promote an adequate bond between the grout and the rock. Prestressing of the anchors prior to loading will minimize anchor movement due to service loads.

5.6 Lateral Earth Pressure

Lateral earth pressures acting on the foundation walls of the underground parking are provided in the following sections for the portion of the underground parking within the overburden (or approximate elevations of between 65.5 and 62.5 m) and portion of the underground parking below and within the bedrock (or approximate elevations of between 62.5 and 56.5 m).

The following sections can also be used to estimate the lateral earth pressures on a temporary shoring system that might be required during the excavations.

5.6.1 Underground Parking – Within Overburden

Lateral earth pressures acting on the foundation walls (or temporary retaining system) above bedrock (i.e., within the overburden) will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading may also need to be considered in the design.

5.6.1.1 Static Lateral Earth Pressures

It is assumed that the foundation walls will be non-yielding, and therefore at-rest conditions will apply for those walls. It is assumed that the foundation walls will be drained but if the structures will not be drained, the earth pressure equation below the groundwater level should be used for the depth of the soil below groundwater level. The groundwater level was measured to be between 2.7 and 9.3 m depth at this site.

As a first, but likely conservative approximation, the static lateral earth pressure can be calculated as:

$$\sigma_h(z) = K\,(\gamma \cdot z + q) \text{ (Above the groundwater level)}$$

$$\sigma_{h(z)} = K\,[\gamma d_w + (\gamma - \gamma_w)(z - d_w) + q] + (z - d_w)\,\gamma_w \text{ (Below the groundwater level)}$$

Where:

$\sigma_{h(z)}$ = Lateral earth pressure on the wall at depth z (kPa)
K = Earth pressure coefficient, K_o for restrained structures or K_a for unrestrained structures
γ = Unit weight of retained soil (see table below)
γ_w = Unit weight of water (use 9.81 kN/m^3)
z = Depth below the top of wall (m)
d_w = Depth to groundwater level (see discussion above)
q = Uniform surcharge at ground surface behind the wall to account for traffic, equipment, or stockpiled soil (use 12 kPa)

The pressures are based on the existing fill and native materials behind the wall and the following parameters (unfactored) should be used to estimate the lateral earth pressures:

Material	Existing Fill	Glacial Till	Granular A / Granular B Type II	Clear Stone
Soil Unit Weight:	20 kN/m^3	21.5 kN/m^3	22 kN/m^3	18 kN/m^3
Internal Angle of Friction	Ø = 28°	Ø = 31°	Ø = 35°	Ø = 32 °
Coefficients of static lateral earth pressure: Active, K_a At rest, K_o Passive, K_P	0.36 0.53 2.77	0.32 0.48 3.12	0.27 0.43 3.70	0.31 0.47 3.25

The above lateral earth pressures have not been factored; factoring of these loads will be required if the foundation wall is being designed in accordance with Limit States Design.

Where the permanent structure is significantly smaller than the excavation and a wide backfilled gallery exists between the structure wall and an adjacent rigid shoring system, then the permanent structure walls should be designed to retain the granular backfill soils using the above formulas, and an at rest earth pressure coefficient given above.

A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the structure. Care must be taken during the compaction operation not to overstress the structure. Heavy construction equipment should be maintained at a distance of at least 1 m away from the structure while the backfill soils are being placed and the backfill should be uniformly raised around the structure. Hand operated compaction equipment should be used to compact the backfill soils within a 1 m wide zone adjacent to the walls. Other surcharge loadings should be accounted for in the design, as required.

5.6.1.2 Seismic Lateral Earth Pressures

Seismic loading will result in increased lateral earth pressures acting on the retaining and foundation walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake induced dynamic earth pressure. The earthquake-induced dynamic pressure distribution is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution).

If the foundation walls are backfilled with granular free draining fill either in a zone with width equal to at least half of the height of the wall or within the wedge-shaped zone defined by a line drawn at 1 horizontal to 1 vertical (1H:1V) extending up and back from the rear face of foundation wall, the following parameters (unfactored) provided in the table below may be used.

The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(z) = K \gamma z + (K_{AE} - K_a) \gamma (H\text{-}z)$$

Where:

$\sigma_{h(z)}$ = Static plus seismic lateral earth pressure at depth z, (kPa)
K_a = Static active earth pressure coefficient, (see table above)
K_o = Static at-rest earth pressure coefficient, (see table above)
K = Earth pressure coefficient, K_o for restrained structures or K_a for unrestrained structures
H = Total depth to the bottom of the foundation wall (m)
K_{AE} = Seismic active earth pressure coefficient (see table below)
γ = Unit weight of the backfill soil (kN/m^3) (see table above)
z = Depth below the top of the wall (m)

Seismic (earthquake) loading must be taken into account in the assessment of lateral earth pressures:

- The horizontal seismic coefficient (k_h) used in the calculation of the seismic active pressure coefficient is taken as 1.0 times the PGA. For structures which allow lateral yielding, (k_h) is taken as 0.5 times the PGA.
- The following seismic active pressure coefficients (K_{AE}) are for the fill, glacial till and granular backfills:

Seismic Active Pressure Coefficients, K_{AE}

Wall Behavior	Site PGA (2475-year Earthquake)	Existing Fill	Glacial Till	Granular A / Granular B Type II	Clear Stone
Yielding wall	0.244 g	0.44	0.40	0.34	0.38
Non-yielding wall		0.55	0.50	0.43	0.48

The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A mm, where A is the design zonal acceleration ratio of (0.244 g). This corresponds to displacements of up to approximately 40 mm at this site.

It should be noted that the above seismic earth pressure coefficients assume that the back of the wall is vertical and that the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

5.6.2 Underground Parking – Within Bedrock

It is considered that three design conditions could exist with regards to the lateral earth pressures that will be exerted on the foundation walls founded within the bedrock:

- Case 1: Walls cast directly against the bedrock face
- Case 2: Walls cast against formwork with a narrow-backfilled gallery provided between the foundation wall and the adjacent excavation bedrock face
- Case 3: Walls cast against formwork with a wide backfilled gallery provided between the foundation wall and the adjacent excavation face

<u>Case 1</u>

For the first case (wall cast against the bedrock), there will be no effective lateral earth pressures on the foundation wall. This assumes that any loose blocks or wedges of rock are removed from the face of the excavation or are stabilized prior to constructing the wall, and that any rock stabilization is designed for permanent use (i.e., with appropriate corrosion protection).

<u>Case 2</u>

For the second case, the magnitude of the lateral earth pressure depends on the magnitude of the arching which can develop in the backfill and therefore depends on the width of the backfill, its angle of internal friction, as well as the interface friction angles between the backfill and both the rock face and the foundation wall. The magnitude of the lateral earth pressure can be calculated as:

$$\sigma_h(z) = \frac{\gamma B}{2\tan\delta}\left(1 - e^{-2K\frac{z}{B}\tan\delta}\right) + \mathrm{K\,q}$$

Where:

$\sigma_{h(z)}$ = Lateral earth pressure on the foundation wall at depth z (kPa)

K = Earth pressure coefficient (use 0.6)

γ = Unit weight of retained soil (use 20 kN/m^3 for clear stone)

B = Width of backfill between foundation wall and bedrock face (m)

δ = Average interface friction angle at backfill-foundation wall and backfill-rock face interfaces (use 15 degrees)

z = Depth below top of formwork (m)

q = Surcharge at ground surface to account for traffic, equipment, or stock piled materials (use 12 kPa)

Case 3

For the third case, when the width of the backfill is equal to half the wall height or more (i.e., wide backfill), the foundation walls should be designed to resist lateral earth pressures calculated as outlined in Section 5.6.1.

The following should be considered in estimating the lateral earth pressure:

- Hydrostatic groundwater pressures would also need to be considered if the structure is designed to be water-tight.
- It has been assumed that the underground parking level will be maintained at minimum temperatures but will not be permitted to freeze. If these areas are to be unheated, additional guidelines for the design of the foundation wall will be required.

5.7 Excavation

Excavations for the underground parking and foundations will be made through the overburden and underlying dolostone and sandstone bedrock. It is expected that the excavation will extend up to approximately 9 or 10 m below the existing ground surface (to accommodate the three-storeys of underground parking).

5.7.1 Excavation in Overburden

No unusual problems are anticipated with excavating the overburden materials using large hydraulic excavating equipment.

In accordance with the Occupational Health and Safety Act (OHSA) of Ontario, the overburden materials above the groundwater table (i.e., fill and glacial till) would generally be classified as a Type 3 soil and therefore, the side slopes should be stable in the short term at 1 horizontal to 1 vertical. Below the water table, side slopes of 3 horizontal to 1 vertical (Type 4 soil in accordance with the OHSA) will be required.

Where site conditions (such as proximity of existing structures and utilities, or space restrictions) do not allow for the above noted side slopes then suitable safety and support measures must be undertaken according to the requirements of the OSHA. These measures include installation of a suitable shoring system to create and maintain positive support to the sidewalls of the excavation.

Guidelines on excavation shoring are provided in Section 5.9.

5.7.2 Excavation in Bedrock

The bedrock surface was encountered at depths ranging from about 1.6 to 4.8 m below the existing ground surface. Excavations into the bedrock will be required for the three levels of underground parking.

The bedrock encountered at this site, in general, consists of slightly weathered to fresh dolostone/sandstone. The thin upper portion of the bedrock, however, is highly weathered (as encountered in boreholes 21-02, 21-03 and 21-06 to 21-09). It will likely be possible to carry out the bedrock removal using mechanical methods (such as hydraulic excavators and hoe ramming) for the removal of the highly weathered portion of the bedrock or for shallow excavations into bedrock (such as for service installation).

Where deep excavation of the sound bedrock is required (for the underground parking), it is anticipated that the bedrock removal could be carried out using controlled blasting, potentially in conjunction with hoe ramming and closely spaced line drilling.

The borehole log information (such as bedding and jointing orientations and spacing) suggests that near-vertical excavation walls in the bedrock should stand unsupported for the construction period. The borehole data, however, provides only limited information of the bedding and jointing in the bedrock and therefore the exposed bedrock should be inspected regularly (as the bedrock excavation proceeds) by qualified geotechnical personnel to assess the exposed bedrock surface for potential localized instabilities. Additional temporary rock support system such as rock bolts or shotcrete and mesh might be required to secure localized unstable rock wedges or poor-quality rock. If rock bolts are used to secure the unstable rock wedges (on the rock faces against the foundation walls where they are relied on for long-term support of the rock walls), they should be designed as a long-term / permanent stabilization measure and should have adequate corrosion protection cover.

All loose rock should be removed from the sidewalls during excavation to ensure the safety of workers.

The blasting should be controlled to limit the peak particle velocities at all adjacent structures or services such that blast induced damage will be avoided. This will require blast designs by a specialist in this field (see Section 5.13).

5.8 Groundwater Management

5.8.1 Estimates of Groundwater Taking and Radius of Influence

5.8.1.1 Construction Condition

It is understood that three levels of underground garage parking are being considered. Accordingly, excavation will be through surficial fill and native glacial till, into the underlying bedrock. Based on the groundwater conditions observed in the monitoring wells, excavations will likely extend below the groundwater level. The rate of groundwater inflow to the excavation will depend on many factors, including: the contractor’s schedule and rate of excavation, the size of the excavation, and the time of year at which the excavation is made. Also, there may be instances where precipitation collects in an open excavation and must be rapidly pumped out.

According to O.Reg 63/16 and O.Reg 387/04, if the volume of water to be pumped from excavations for the purpose of construction dewatering is greater than 50,000 L/day and less than 400,000 L/day, the water taking will need to be registered as a prescribed activity in the Environmental Activity and Sector Registry (EASR) and requires the completion of a “Water Taking Plan”. Alternatively, a Permit to Take Water (PTTW) is required from the Ministry of the Environment Conservation and Parks (MECP) if a volume of water greater than 400,000 L/day is to be pumped from an excavation.

It is possible that groundwater elevations encountered during construction may be higher than those observed in October 2021, if, for example, construction occurs during the spring. Therefore, groundwater inflow estimates were completed using a groundwater elevation that is 0.5 m higher than the measured groundwater elevations. Incident precipitation could add approximately 700,000 L/day to the underground parking excavation, based on the proposed footprint, and assuming a 79.2 mm precipitation event (a 10-year event as observed at the Ottawa Airport weather station).

The Dupuit-Forcheimer analytical solution was used to estimate the potential groundwater inflow into the underground parking excavation using the geometric mean hydraulic conductivity measured in the wells screened above and to the depth of the underground parking (all monitoring wells except 21-10). The initial head elevation of the analytical model was assigned a value of 62.9 m (i.e., 0.5 m above the value recorded at monitoring well 21-04). It is assumed that construction dewatering activities would lower the groundwater level to an elevation of 56.0 m for the preliminary analysis. The hydraulic conductivity of the bedrock at this depth was estimated to be approximately 1x10^{-4} cm/s. The amount of dewatering needed for the excavation (including a safety factor of 2) is estimated to be between 105,000 (steady-state inflow) and 450,000 (initial inflow) litres per day (L/day). The radius of influence for the excavation is estimated to be approximately 25 m from the edge of the excavation. Groundwater inflow and dewatering radius of influence calculations are included in Appendix F.

Based on the groundwater conditions observed at the site and depending on how the excavation proceeds, water taking exceeding 400,000 L/day may be required to dewater groundwater from the excavation. As a result, a PTTW may be necessary for the water taking associated with the proposed work.

5.8.1.2 Permanent Condition

The Dupuit-Forcheimer analytical solution was used to estimate the potential groundwater inflow to the drainage system for the three levels of underground parking. The initial groundwater elevation was assumed to be 62.9 m (i.e., 0.5 m above the value recorded at monitoring well 21-04), and it was assumed that the drains would lower the groundwater elevation to elevation 56.5 m for the preliminary analysis. The analytical solution was run using the geometric mean hydraulic conductivity measured in the wells screened at the depth of the underground parking. The steady-state dewatering rate (including a safety factor of 2.0) for the drainage system is estimated to be approximately 92,000 L/day. The radius of influence for the drainage system for steady-state flow was estimated to be approximately 25.0 m from the underground parking (see Appendix F).

5.9 Temporary Shoring

The excavation through the overburden for underground parking will extend to depths of about 1.6 to 4.8 m below the existing ground surface. The contractor is fully responsible for the detailed design and performance of the temporary shoring systems. However, this section of the report provides some general guidelines on possible concepts for the shoring to be used by the designers for assessing the possible impacts of the shoring design and site works as well as to evaluate, at the design stage, the potential for impacts of this shoring on the adjacent properties. Temporary shoring can be used in combination with open cuts above the top of shoring, however, the earth pressure distribution must take into account the effects of the soil pressures from the upper open cut section.

The shoring method(s) chosen to support the excavation sides must take into account the soil and bedrock stratigraphy, the permissible movement of the shoring, the groundwater conditions, the methods adopted to manage the groundwater and construct the shoring systems, the potential ground movements associated with the excavation and construction of the shoring system, and their impact on adjacent structures and utilities.

For design purposes, a soldier pile and timber lagging system are considered a feasible shoring method that may be considered for the proposed excavations at the site. Due to the presence of shallow bedrock beneath the overlaying deposits, the soldier piles might need to be socketed into the competent bedrock to provide sufficient embedment for toe fixity, or the piles may need to be pinned to bedrock. Soldier pile and lagging walls are considered suitable for the sides of the excavations (provided that settlement-sensitive structures or utilities are not present in the zone of influence of the walls) where the objective is to maintain an essentially vertical excavation wall and the movements above and behind the wall need only be sufficiently limited so that relatively flexible features (such as roadways or sidewalks) will not be adversely affected.

Where foundations or settlement-sensitive infrastructure are present within the zone of influence of the shoring, the deflections may need to be greatly limited and therefore soldier pile and timber lagging system might not be feasible. Golder can provide further recommendations and guideline in the detailed design stage when the distance and extent of the excavations with respect to the sensitive structures are determined.

For a soldier pile and lagging system, some form of lateral support to the wall is typically required for excavation depths greater than about 3 to 4 m. Lateral restraint could be provided by means of tie-backs consisting of grouted soil or bedrock anchors. However, the use of rock/ground anchor tie-backs would require the permission of the adjacent property owners since the anchors would likely encroach on their properties. The presence of utilities beneath the adjacent properties, which could interfere with the tie-backs, should also be considered. Alternatively, interior struts can be considered, connected either to the opposite side of the excavation (if not too distant) or to raker piles and/or footings within the excavation.

5.10 Floor Slab

The floor slab of the underground parking will be cast on bedrock.

Provision should be made for at least 150 mm of OPSS.MUNI 1010 Granular A compacted to 100% of the material's standard Proctor Maximum Dry Density (SPMDD) to form the base for the floor slab. Any engineered fill required to raise the grade to the underside of the Granular A, should consist of OPSS.MUNI 1010 Granular B Type II, or the Granular A bedding can be thickened, as needed. The underslab fill should be placed in maximum 300 mm thick lifts and compacted to at least 100% of the material's SPMDD using suitable vibratory compaction equipment.

Provision should be made for drainage underneath the floor slab. The details on the permanent dewatering system are provided in Section 5.8.1.2, and subfloor drainage system should be designed to accommodate permanent groundwater inflow.

As a preliminary guideline, the subfloor drainage system may consist of a network of perimeter drains and sub-drain pipes conveying collected groundwater to a sump or sumps from which the groundwater can be pumped to a municipal sewer. The drainage system would consist of interconnected, perforated drain pipes (fully wrapped in non-woven geotextile and backfilled with free draining granular soils) installed around the perimeter and within the underground parking footprint. The capacity of the subfloor drainage system should be modified during construction as required if higher than anticipated inflows are observed. For preliminary design, the subdrains should be spaced no greater than 6.0 m apart.

Vertical drainage system should be provided to the exterior foundation walls. The drainage system must withstand the design horizontal earth pressures used for foundation wall design and should be connected to the underslab perimeter drainage system (see further discussions below).

5.11 Foundation Wall Backfill and Drainage

The existing fill and glacial till encountered at this site are potentially frost susceptible and should not be used as backfill against the foundation walls. To avoid problems with frost adhesion and heaving as well as to provide drainage, the foundation walls should be backfilled with non-frost susceptible sand or sand and gravel conforming to the requirements for Ontario Provincial Standard Specification (OPSS) Granular B Type I, Granular B Type II, or Granular A. The granular backfill should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% of the material's SPMDD using suitable vibratory compaction equipment. To reduce compaction induced stresses, only light compaction rollers or plate tampers should be used within 1.0 m of the wall.

If the basement walls will be backfilled, vertical drainage membrane such as Miradrain (or similar drainage board) should be installed prior to backfilling. If the wall will be cast against shoring/rock the drainage board should be installed prior to casting the wall.

Any narrow galleries between the foundation walls and shoring wall/exposed bedrock may be backfilled using clear stone (where too narrow for normal compacted granular fill). Where the clear stone is in direct contact with soil, it should be fully wrapped in non-woven geotextile.

The perimeter drainage of the basement wall backfill should be provided by means of a perforated pipe in a surround of 19 mm clear stone, fully wrapped in geotextile, which leads by gravity drainage to an adjacent storm sewer or sump pit.

In areas where pavement or other hard surfacing will abut the building, differential frost heaving could occur between the granular fill immediately adjacent to the building and the more frost susceptible backfill placed beyond the wall backfill. To reduce the severity of this differential heaving, the backfill adjacent to the foundation walls should be placed to form a frost taper. The frost taper should be brought up to pavement subgrade level from 1.5 m below finished exterior grade at a slope of 3 horizontal to 1 vertical, or flatter, away from the wall. The granular fill should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% of the material’s SPMDD using suitable vibratory compaction equipment.

5.12 Ground Movements

During the excavation of the underground parking area, lateral deformation and vertical settlement of the adjacent ground may occur as a result of installation and deflection of the excavation activities. The ground movements induced could affect the stability or performance of structures and buildings or underground utilities adjacent to the excavation. Therefore, the magnitude and extent of ground movement and potential impacts on surrounding infrastructure should be assessed prior to construction to confirm movements will be in tolerable limits and monitored during construction.

Protective measures such as temporary shoring for the excavations in soil may need to be adopted where the excavations interfere with the zone of influence of adjacent building foundations or other structures/utilities.

5.13 Vibration Monitoring

A pre-construction or pre-blast survey should be carried out for all of the surrounding structures. Selected existing interior and exterior cracks in the structures should be identified during the pre-construction survey and should be monitored for lateral or shear movements by means of pins, glass plate telltales and/or movement telltales.

The excavation contractor should be required to submit a complete and detailed blasting design and monitoring proposal prepared by a blasting/vibrations specialist prior to commencing blasting. This would have to be reviewed and accepted in relation to the requirements of the blasting specifications.

The contractor should be limited to only small, controlled shots. The following frequency dependent peak vibration limits at the nearest structures and services are suggested as typical vibration criteria commonly adopted for construction projects. If unusually sensitive receptors are identified during construction planning, then specific criteria may need to be adopted for those receptors.

Frequency Range (Hz)	Vibration Limits (mm/s)
< 10	5
10 to 40	5 to 50 (sliding scale)
> 40	50

It is recommended that the monitoring of ground vibration intensities (peak ground vibrations and accelerations) from the blasting operations be carried out both in the ground adjacent to the closest structures and services and within the structures themselves.

If practical, bedrock removal should commence at the furthest points from the closest structure or service to assess the ground vibration attenuation characteristics and to confirm the anticipated ground vibration levels.

Vibration monitoring should be carried out throughout all bedrock removal operations.

5.14 Site Servicing

5.14.1 Pipe Bedding and Cover

At least 150 mm of OPSS Granular A should be used as pipe bedding for sewer and water pipes. Where unavoidable disturbance to the subgrade surface occurs, or if fill material is located below the invert of the pipe, it will be necessary to remove the disturbed material or fill, and place a sub-bedding layer consisting of compacted OPSS Granular B Type II beneath the Granular A. The bedding material should in all cases extend to the spring line of the pipe and should be compacted to at least 95% of the material’s SPMDD. The use of clear crushed stone as a bedding layer should not be permitted anywhere on this project since fine particles from the sandy backfill materials or surrounding soil could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

Cover material, from spring line of the pipe to at least 300 mm above the top of pipe, should consist of OPSS Granular A or Granular B Type I with a maximum particle size of 25 mm. The cover material should be compacted to at least 95% of the material’s SPMDD.

5.14.2 Trench Backfill

Trench backfill may consist of approved excavated material such as the existing pavement granulars, inorganic fill and glacial till, where the services will be overlain by pavements or other hard surfacing.

It is important for frost heave compatibility that the trench backfill within the frost zone of 1.8 m depth below pavement grade matches the soil exposed on the trench walls. This will require some separation of materials upon excavation. In particular, where the watermain is to be installed beneath existing pavements, the trench backfill should match those existing granular layers.

Trench backfill should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% of its SPMDD using suitable compaction equipment.

5.15 Pavement Design

In preparation for pavement construction, all disturbed, or otherwise deleterious materials (i.e., those materials containing organic material) should be removed from the roadway areas. To minimize potential for disturbance, the general grade should not be cut to final subgrade level until all services have been installed.

Sections requiring grade raising to the proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow, OPSS Select Subgrade Material (SSM) or granular fill. These materials should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% of the material's SPMDD using suitable compaction equipment.

Below the pavement structure, frost compatibility must be maintained across any new service trenches.
Due to the variability of the soils within the project limits, the subsoil should be inspected by qualified geotechnical personnel to make sure that there is no potential for differential frost heaving.

5.15.1 Pavement Drainage

The surface of the pavement subgrade should be crowned to promote drainage of the roadway granular structure. The subgrade surface should be crowned or sloped to promote drainage of the roadway granular structure. Perforated pipe subdrains should be provided along the low sides of the roadway along the entire length.
The subdrains should be installed in accordance with the City of Ottawa Specification F-4050 "Pipe Subdrain" and as per City of Ottawa Drawing No. R1. The geotextile should consist of a Class I non-woven geotextile to OPSS 1860. The geotextile should have a maximum Apparent Opening Size A.O.S. of 212 μm. The subdrains should be connected to the catch basins such that the pavement structure will be positively drained and will intercept flows within the subbase.

Backfilling of catch basin laterals located below subgrade level should be completed using acceptable native soils or fill which match the material types exposed on the lateral trench walls. This will reduce potential problems associated with differential frost heaving.

5.15.2 Granular Pavement Materials

Good drainage significantly improves the freeze-thaw resistance of the asphaltic concrete and decreases the frequency of transverse cracking, thereby extending the life of the pavement. The granular base and subbase for new construction should consist of Granular A and Granular B Type II (City of Ottawa F-3147), respectively.

5.15.3 Pavement Design

The pavement structure for local roads or parking lots, which will not experience bus or truck traffic (other than school bus and garbage collection), should consist of:

Pavement Component	Thickness (mm)
Asphaltic Concrete	90
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	400

The pavement structure for roadways which will experience bus and/or truck traffic as well as fire routes should consist of:

Pavement Component	**Thickness (mm)**
Asphaltic Concrete	120
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	450

The granular base and subbase materials should be uniformly compacted to at least 100% of material's SPMDD using suitable vibratory compaction equipment. The asphaltic concrete should be compacted in accordance with Table 10 of OPSS 310.

The composition of the asphaltic concrete pavement with 90 mm thickness should be as follows:

- Superpave 12.5 mm Surface Course – 40 mm
- Superpave 19.0 mm Base Course – 50 mm

The composition of the asphaltic concrete pavement with 120 mm thickness should be as follows:

- Superpave 12.5 mm Surface Course – 50 mm
- Superpave 19.0 mm Base Course – 70 mm

The asphaltic cement should consist of PG 58-34 and the design of the mixes should be based on a Traffic Category B.

The above pavement design is based on the assumption that the pavement subgrade has been acceptably prepared (i.e., where the trench backfill and grade raise fill have been adequately compacted to the required density and the subgrade surface not disturbed by construction operations or precipitation). Depending on the actual conditions of the pavement subgrade at the time of construction, it could be necessary to increase the thickness of the subbase and/or to place a woven geotextile beneath the granular materials.

5.15.4 Pavement Structure Compaction

Adequate compaction of the granular materials will be essential to the continued acceptable performance of the roadway and parking areas. Compaction should be carried out in conformance with procedures outlined in OPSS 501 "Construction Specification for Compacting" with compacted densities of the various materials being in accordance with Subsection 501.08.02 Method A. The granular base and subbase material should be uniformly compacted to at least 100% of the material's SPMDD using suitable vibratory compaction equipment. Compaction of the asphaltic concrete should be carried out in accordance with OPSS 310, Table 10.

The placement and compaction of any engineered fill, as well as sewer and watermain bedding and backfill, should be inspected to ensure that the materials used conform to the specifications from both a grading and compaction viewpoint. In addition, compaction testing and sampling of the asphaltic concrete used on site should be carried out to make sure that the materials used, and level of compaction achieved during construction meet the project requirements.

5.15.5 Joints, Tie-ins with Existing Pavements, Pavement Resurfacing

At intersections with roadways at the project extents, the new pavement structure should be continued to the limits of construction. At connections to existing pavements, the existing pavement should be milled back beyond the curb return an additional 300 mm to the depth of the new surface course to accept the new surface course asphaltic concrete.

The granular courses and subbase level should be tapered between the new and existing pavements by using 10 horizontal to 1 vertical tapers up or down as required. At driveways and commercial entrances, butt joints may be used.

A tack coat should be provided on all and vertical and milled horizontal surfaces. The tack coat should consist of SS-1 emulsified asphalt diluted with an equal amount of water. The undiluted and emulsified asphalt shall be in conformance with OPSS 1103.

5.16 Site Grading

The subsurface conditions at this site generally consist of fill, or fill underlain by a deposit of glacial till, which are in turn underlain by bedrock.

No practical restrictions apply to the thickness of grade raise fill which may be placed on the site from a foundation design perspective.

5.17 Material Reuse

The existing fill and glacial till materials encountered at this site are not considered to be generally suitable for reuse as structural/engineered fill. Within foundation areas, imported engineered fill such as OPSS Granular B Type II should be used (if required). The existing fill and native overburden soils could however be reused in non-structural areas (i.e., landscaping).

5.18 Trees

The silty clay soils in Ottawa are sensitive to water depletion by trees of high-water demand during periods of dry weather. When trees draw water from the clayey soil, the clay undergoes shrinkage which can result in settlement of adjacent structures.

Based on the results of the geotechnical investigation, the site is not underlain by sensitive silty clay. Therefore, no restrictions on the types or sizes of trees that may be planted or tree to foundation setback distances need to be considered for this development.

5.19 Corrosion and Cement Type

The concentration of sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. The sulphate results (see Section 4.7) were compared with Table 3 of Canadian Standards Association Standards A23.1-14 (CSA A23.1) and generally indicate a low degree of sulphate attack potential on concrete structures at the locations of all tested samples. Therefore, concrete made with Type GU Portland cement is considered acceptable for all substructures.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. Generally, the results indicate a moderate potential for corrosion of exposed ferrous metal within the study area, which should be taken into consideration in the design of substructures.

6.0 ADDITIONAL CONSIDERATIONS

If construction is carried out during periods of sustained below freezing temperatures, all subgrade areas should be protected from freezing (e.g., by using insulated tarps and/or heating).

All footing and subgrade areas should be inspected by experienced geotechnical personnel prior to filling or concreting to document that the correct/expected strata exist and that the bearing surfaces have been properly prepared. The placing and compaction of any engineered fill, pipe bedding, and pavement base and subbase materials should be inspected to ensure that the materials used conform to the specifications from both a grading and compaction point of view.

At the time of the writing of this report, only conceptual details for the proposed development were available. Golder Associates should review the final drawings and specifications for this project prior to tendering to confirm that the guidelines in this report have been adequately interpreted and to review some of our preliminary recommendations.

7.0 CLOSURE

We trust that this report contains sufficient information for your present purposes. If you have any questions regarding this report or if we can be of further service to you on this project, please reach out us.

Signature page

Golder Associates Ltd.

LICENSED PROFESSIONAL ENGINEER
C. G. HENDRY
100011328
June 27, 2023
PROVINCE OF ONTARIO

Chris Hendry, P.Eng.
Associate, Senior Geotechnical Engineer

KG/AG/CH/hdw

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IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practicing under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client **LPF Development Fund 3 LP.** The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then the client may authorize the use of this report for such purpose by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process, provided this report is not noted to be a draft or preliminary report, and is specifically relevant to the project for which the application is being made. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client cannot rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder cannot be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont'd)

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

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KEY MAP

SCALE: 1:50,000 m

LEGEND

⊕ APPROXIMATE BOREHOLE LOCATION

REFERENCE(S)

1. PROJECTION: TRANSVERSE MERCATOR, DATUM NAD 83, COORDINATE SYSTEM: MTM ZONE 9, VERTICAL DATUM CGVD28

DRAFT

0 25 50

1:1,000 METRES

CLIENT

FENGATE DEVELOPMENT HOLDINGS LP

PROJECT

GEOTECHNICAL INVESTIGATION
1047 RICHMOND ROAD, OTTAWA, ONTARIO

TITLE

SITE PLAN

CONSULTANT

YYYY-MM-DD	2023-06-27
DESIGNED	----
PREPARED	ZS
REVIEWED	----
APPROVED	----

PROJECT NO.	CONTROL	REV.	FIGURE
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0 25 mm IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B

APPENDIX A

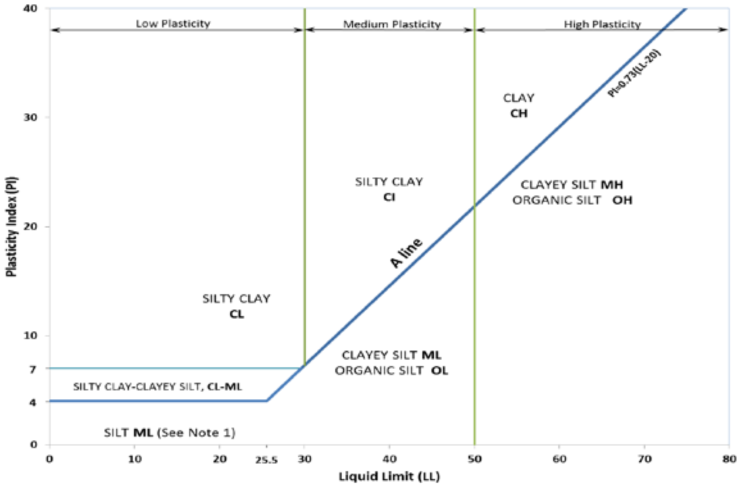
Borehole Records

METHOD OF SOIL CLASSIFICATION

The Golder Associates Ltd. Soil Classification System is based on the Unified Soil Classification System (USCS)

Organic or Inorganic	Soil Group	Type of Soil		Gradation or Plasticity	$Cu = \frac{D_{60}}{D_{10}}$	$Cc = \frac{(D_{30})^2}{D_{10} x D_{60}}$	Organic Content	USCS Group Symbol	Group Name
INORGANIC (Organic Content ≤30% by mass)	COARSE-GRAINED SOILS (>50% by mass is larger than 0.075 mm)	GRAVELS (>50% by mass of coarse fraction is larger than 4.75 mm)	Gravels with ≤12% fines (by mass)	Poorly Graded	<4	≤1 or ≥3	≤30%	GP	GRAVEL
				Well Graded	≥4	1 to 3		GW	GRAVEL
			Gravels with >12% fines (by mass)	Below A Line	n/a			GM	SILTY GRAVEL
				Above A Line	n/a			GC	CLAYEY GRAVEL
		SANDS (≥50% by mass of coarse fraction is smaller than 4.75 mm)	Sands with ≤12% fines (by mass)	Poorly Graded	<6	≤1 or ≥3		SP	SAND
				Well Graded	≥6	1 to 3		SW	SAND
			Sands with >12% fines (by mass)	Below A Line	n/a			SM	SILTY SAND
				Above A Line	n/a			SC	CLAYEY SAND

Organic or Inorganic	Soil Group	Type of Soil	Laboratory Tests	Field Indicators					Organic Content	USCS Group Symbol	Primary Name
				Dilatancy	Dry Strength	Shine Test	Thread Diameter	Toughness (of 3 mm thread)			
INORGANIC (Organic Content ≤30% by mass)	FINE-GRAINED SOILS (≥50% by mass is smaller than 0.075 mm)	SILTS (Non-Plastic or PI and LL plot below A-Line on Plasticity Chart below)	Liquid Limit <50	Rapid	None	None	>6 mm	N/A (can't roll 3 mm thread)	<5%	ML	SILT
				Slow	None to Low	Dull	3mm to 6 mm	None to low	<5%	ML	CLAYEY SILT
				Slow to very slow	Low to medium	Dull to slight	3mm to 6 mm	Low	5% to 30%	OL	ORGANIC SILT
			Liquid Limit ≥50	Slow to very slow	Low to medium	Slight	3mm to 6 mm	Low to medium	<5%	MH	CLAYEY SILT
				None	Medium to high	Dull to slight	1 mm to 3 mm	Medium to high	5% to 30%	OH	ORGANIC SILT
		CLAYS (PI and LL plot above A-Line on Plasticity Chart below)	Liquid Limit <30	None	Low to medium	Slight to shiny	~ 3 mm	Low to medium	0% to 30% **(see Note 2)**	CL	SILTY CLAY
			Liquid Limit 30 to 50	None	Medium to high	Slight to shiny	1 mm to 3 mm	Medium		CI	SILTY CLAY
			Liquid Limit ≥50	None	High	Shiny	<1 mm	High		CH	CLAY
HIGHLY ORGANIC SOILS (Organic Content >30% by mass)		Peat and mineral soil mixtures							30% to 75%	PT	SILTY PEAT, SANDY PEAT
		Predominantly peat, may contain some mineral soil, fibrous or amorphous peat							75% to 100%		PEAT



Note 1 – Fine grained materials with PI and LL that plot in this area are named (ML) SILT with slight plasticity. Fine-grained materials which are non-plastic (i.e. a PL cannot be measured) are named SILT.

Note 2 – For soils with <5% organic content, include the descriptor "trace organics" for soils with between 5% and 30% organic content include the prefix "organic" before the Primary name.

Dual Symbol — A dual symbol is two symbols separated by a hyphen, for example, GP-GM, SW-SC and CL-ML.

For non-cohesive soils, the dual symbols must be used when the soil has between 5% and 12% fines (i.e. to identify transitional material between "clean" and "dirty" sand or gravel.

For cohesive soils, the dual symbol must be used when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart (see Plasticity Chart at left).

Borderline Symbol — A borderline symbol is two symbols separated by a slash, for example, CL/CI, GM/SM, CL/ML.

A borderline symbol should be used to indicate that the soil has been identified as having properties that are on the transition between similar materials. In addition, a borderline symbol may be used to indicate a range of similar soil types within a stratum.

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

Percentage by Mass	Modifier
>35	Use 'and' to combine major constituents (*i.e.,* SAND and GRAVEL)
> 12 to 35	Primary soil name prefixed with "gravelly, sandy, SILTY, CLAYEY" as applicable
> 5 to 12	some
≤ 5	trace

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)
An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve frictions are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , w_L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, Gs)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

Term	SPT 'N' (blows/0.3m)[1]
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	>50

1. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.
2. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grainsize. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

COHESIVE SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N'[1,2] (blows/0.3m)
Very Soft	<12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Water Content

Term	Description
w < PL	Material is estimated to be drier than the Plastic Limit.
w ~ PL	Material is estimated to be close to the Plastic Limit.
w > PL	Material is estimated to be wetter than the Plastic Limit.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$ - u)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l – w_p)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index = $(w – w_p) / I_p$
I_C	consistency index = $(w_l – w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} – e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c′	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1 $\tau = c' + \sigma' \tan \phi'$

2 shear strength = (compressive strength)/2

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

PROJECT: 21494078

RECORD OF BOREHOLE: 21-01

SHEET 1 OF 1

LOCATION: N 5026314.5 ;E 361326.2

BORING DATE: September 24, 2021

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ ND = Not Detected	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	65.73						
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALT	0.00						Flush Mount Casing
		FILL - (SW) gravelly SAND, angular; brown (PAVEMENT STRUCTURE); non-cohesive, moist	0.10						
		FILL - (SM) gravelly SILTY SAND; grey to dark brown, trace sand (SP); non-coohesive, moist, compact to very loose	65.43 0.30	1	SS	19	ND	Metals	
1				2	SS	4	ND		
				3	SS	2	ND	PHCs, VOCs	
2	Air Hammer H Bit	BEDROCK (Auger Refusal) (Air hammer from 1.83 m to 7.62 m)	63.90 1.83						Bentonite Seal
4									SIlica Sand
6									50 mm Diam. PVC #10 Slot Screen
		End of Borehole	58.11 7.62						

HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] ☐ ND = Not Detected: 100 200 300 400

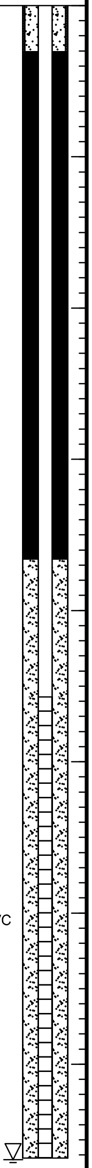
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ├──○W──┤ Wl: 20 40 60 80

Note(s):

1. Water level measured at a depth of 7.63 m (Elev. 58.13 m) on October 5, 2021

2. Borehole log not for geotechnical purposes.



DEPTH SCALE

1 : 50

GOLDER

LOGGED: DG

CHECKED: AG

PROJECT: 21494078

RECORD OF BOREHOLE: 21-02

SHEET 1 OF 1

LOCATION: N 5026359.3 ;E 361297.8

BORING DATE: September 21, 2021

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ *ND = Not Detected* 100 200 300 400

HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] ☐ *ND = Not Detected* 100 200 300 400

HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	HEADSPACE	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	65.46						
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALT	0.08						Flush Mount Casing
		FILL - (SW) gravellyl SAND, angular; grey (PAVEMENT STRUCTURE); non-cohesive, moist	65.16 0.30	1	SS	22	⊕ *ND*		
		FILL - (SP) SAND, fine to medium, trace silt; brown; non-cohesive, moist, compact to dense	64.24 1.22	2	SS	31	⊕ *ND*	Metals	
1		FILL - (SM/GP) SILTY SAND and GRAVEL; dark brown, contains brick fragments and rootlets; non-cohesive, moist, compact	63.63 1.83	3	SS	10	⊕ *ND*		
2		Highly weathered BEDROCK	62.41 3.05	4	SS	>84	⊕ *ND*		Bentonite Seal
				5	SS	>50	⊕ *ND*	PHCs, VOCs	
3	Air Hammer H Bit	BEDROCK (Auger Refusal) (Air hammer from 3.05 M TO 7.62 M)	57.84 7.62						▽ SIlica Sand 50 mm Diam. PVC #10 Slot Screen
4									
5									
6									
7									
		End of Borehole							
8									
9									
10									

Note(s):

1. Water level measured at a depth of 3.32 m (Elev. 62.14 m) on October 5, 2021

2. Borehole log not for geotechnical purposes.

DEPTH SCALE

1 : 50

GOLDER

LOGGED: DG

CHECKED: AG

PROJECT: 21494078

RECORD OF BOREHOLE: 21-03

SHEET 1 OF 1

LOCATION: N 5026355.1 ;E 361289.2

BORING DATE: September 21 & 22, 2021

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ ND = Not Detected (100, 200, 300, 400)

HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] ☐ ND = Not Detected (100, 200, 300, 400)

HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3})

WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20, 40, 60, 80)

BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	HEADSPACE	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	65.24						
Power Auger 200 mm Diam. (Hollow Stem)	ASPHALT	0.08						Flush Mount Casing
	FILL - (SW) gravelly SAND, angular; grey (PAVEMENT STRUCTURE); non-cohesive, moist	64.63 0.61	1	SS	43	⊕ ND	VOCs	
	FILL - (SP) SAND, fine to medium, trace silt; brown; non-cohesive, moist, dense	64.02 1.22	2	SS	31	⊕ ND	PHCs	
	FILL - (SM) SILTY SAND, some topsoil, trace gravel; dark brown, contains shale fragments; non-cohesive, moist, compact	63.41 1.83	3	SS	12	⊕ ND	Metals	
	Highly weathered BEDROCK	62.19 3.05	4	SS	>94	⊕ ND		Bentonite Seal
			5	SS	52/ 6"			
Air Hammer H Bit	BEDROCK (Auger Refusal) (Air hammer from 3.05 m to 7.62 m)	57.62 7.62						▽ SIlica Sand 50 mm Diam. PVC #10 Slot Screen
	End of Borehole							

Note(s):

1. Water level measured at a depth of 4.22 m (Elev. 62.02 m) on October 5, 2021

2. Borehole log not for geotechnical purposes.

DEPTH SCALE

1 : 50

GOLDER

LOGGED: DG

CHECKED: AG

PROJECT: 21494078

RECORD OF BOREHOLE: 21-04

SHEET 1 OF 1

LOCATION: N 5026369.7 ;E 361313.7

BORING DATE: September 21, 2021

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

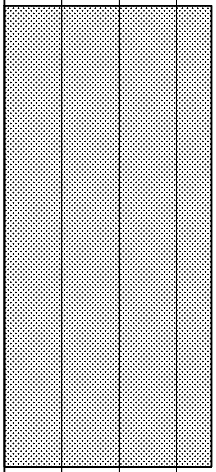
DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.30m	HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ ND = Not Detected	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	65.09						
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALT							Flush Mount Casing
		FILL - (SM) SILTY SAND, trace gravel; brown to grey brown, contains wood fragments; non-cohesive, moist, loose to compact	0.05	1	SS	9	ND	VOCs	
1				2	SS	10	ND		
				3	SS	7	ND		
2				4	SS	14	ND		Bentonite Seal
		(SM) gravelly SILTY SAND; grey brown, contains cobbles and boulders (GLACIAL TILL); non-cohesive, wet, dense	62.65 2.44	5	SS	49	ND		∇
3				6	SS	55/ 4"	ND	PHCs	
4	Air Hammer H Bit	BEDROCK (Auger Refusal) (Air hammer from 3.66 m to 7.62 m)	61.43 3.66						SIlica Sand
6									50 mm Diam. PVC #10 Slot Screen
		End of Borehole	57.47 7.62						

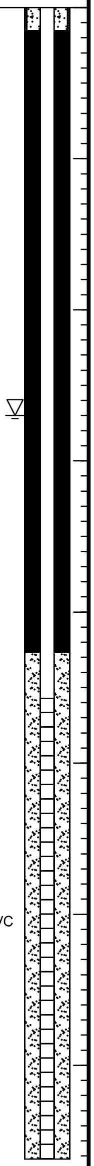
HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ ND = Not Detected: 100 200 300 400

HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □ ND = Not Detected: 100 200 300 400

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl: 20 40 60 80





Note(s):

1. Water level measured at a depth of 2.70 m (Elev. 62.39 m) on October 5, 2021

2. Borehole log not for geotechnical purposes.

DEPTH SCALE

1 : 50

GOLDER

LOGGED: DG

CHECKED: AG

PROJECT: 21494078

RECORD OF BOREHOLE: 21-05

SHEET 1 OF 1

LOCATION: N 5026358.2 ;E 361327.9

BORING DATE: September 22/24, 2021

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

SOIL PROFILE

BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)
	GROUND SURFACE	65.47
Power Auger 200 mm Diam. (Hollow Stem)	ASPHALT	0.08
	FILL - (SP) SAND, fine to coarse, some gravel, trace silt; brown; non-cohesive, moist, compact	64.86 0.61
	FILL - (SM/GW) SILTY SAND and GRAVEL; dark brown, contains wood fragments; non-cohesive, moist, compact	64.02 1.45
	Possible FILL - (SP) SILTY SAND, fine to coarse, trace silt, trace gravel; grey brown; non-cohesive, moist, compact to dense	62.73 2.74
	(SM) gravelly SILTY SAND, non-plastic fines; grey brown, contains cobbles (GLACIAL TILL); non-cohesive, moist, dense	61.82 3.65
Air Hammer H Bit	BEDROCK (Auger Refusal) (Air hammer from 3.65 m to 7.62 m)	57.85 7.62
	End of Borehole	

SAMPLES

NUMBER	TYPE	BLOWS/0.30m	HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ / HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □ (ND = Not Detected)	ADDITIONAL LAB. TESTING
1	SS	15	ND	
2	SS	20	ND	PHCs, VOCs
3	SS	52/ 0"	ND	PHCs, VOCs
4	SS	20	ND	
5	SS	39	ND	
6	SS	46	ND	
7	SS	34/ 10"	ND	

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl; 20 40 60 80

PIEZOMETER OR STANDPIPE INSTALLATION

- Flush Mount Casing
- Bentonite Seal
- ∇
- SIlica Sand
- 50 mm Diam. PVC #10 Slot Screen

Note(s):

1. Water level measured at a depth of 3.94 m (Elev 61.53 m) on October 5, 2021

2. Borehole log not for geotechnical purposes.

DEPTH SCALE 1 : 50

GOLDER

LOGGED: DG

CHECKED: AG

PROJECT: 21494078

RECORD OF BOREHOLE: 21-06

SHEET 1 OF 2

LOCATION: N 5026317.1 ;E 361275.1

BORING DATE: September 30, 2021

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.30m	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	65.00				
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALT					Flush Mount Casing
		FILL - (SW) gravelly SAND, angular; grey (PAVEMENT STRUCTURE); non-cohesive, moist	0.05				
		FILL - (SP) SAND, fine to medium, trace silt; brown; non-cohesive, moist, loose	0.20				
1		FILL - (SM) gravelly SILTY SAND; grey brown, contains organic matter, possible cobbles; non-cohesive, moist, loose	64.09 0.91	1	SS	37	Bentonite Seal
		Highly weathered BEDROCK	63.63 1.37	2	SS	>76	
2		Borehole continued on RECORD OF DRILLHOLE 21-06	63.12 1.88				
3							
4							
5							
6							
7							
8							
9							
10							

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ : 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl : 20 40 60 80

ADDITIONAL LAB. TESTING

DEPTH SCALE

1 : 50

GOLDER

LOGGED: RI

CHECKED: AG

PROJECT: 21494078

RECORD OF DRILLHOLE: 21-06

SHEET 2 OF 2

LOCATION: N 5026317.1 ;E 361275.1

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 30, 2021

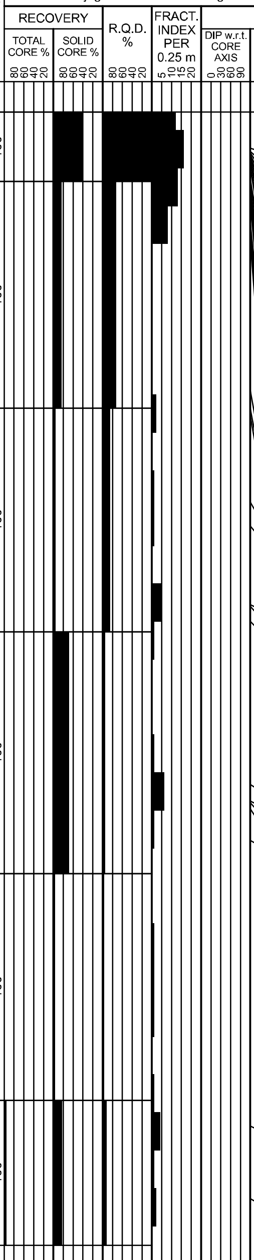
DRILL RIG: CME 55

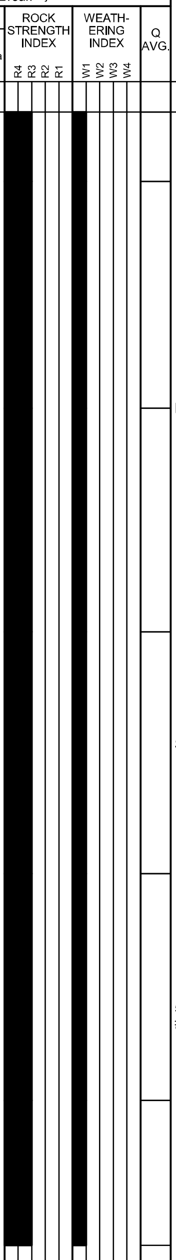
DRILLING CONTRACTOR: CCC Drilling

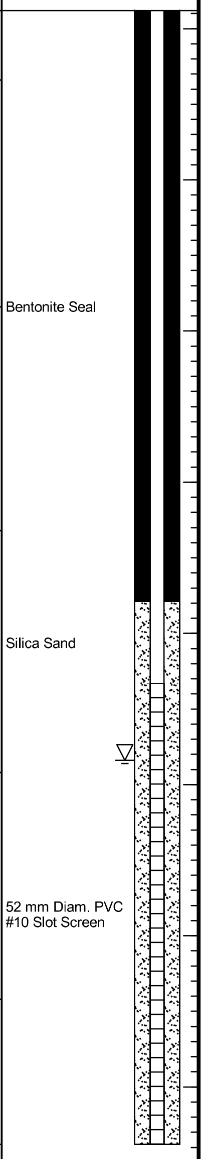
DATUM: Geodetic

JN - Joint	BD- Bedding	PL - Planar	PO- Polished	BR - Broken Rock
FLT - Fault	FO- Foliation	CU- Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR- Shear	CO- Contact	UN- Undulating	SM- Smooth	
VN - Vein	OR- Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB- Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN FLUSH	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION
		BEDROCK SURFACE		63.12			
2	Rotary Drill HQ3 Core	Slightly weathered to fresh, medium to thickly bedded, medium grey, fine grained, faintly porous, medium strong DOLOSTONE, interbedded with shale, limestone and sandstone - Broken core from 1.88 m to 2.07 m - Broken core from 2.34 m to 2.38 m - Broken core from 2.41 m to 2.43 m		1.88	1	100	BD,PL,SM SO BD,PL,SM SO BD,PL,SM SO BD,PL,SM SO BD,PL,SM SO BD,PL,SM SO BD,PL,SM SO BD,PL,SM SO BD,PL,SM SO
3					2	100	BD,PL,SM BD,PL,SM BD,PL,SM BD,PL,SM BD,PL,SM SO BD,PL,SM BD,PL,SM SO BD,PL,SM BD,PL,RO SO BD,PL,SM SO BD,PL,SM SO BD,UN,SM
4					3	100	BD,PL,SM SO BD,PL,SM SO BD,PL,SM SO BD,PL,SM SO BD,PL,SM SO BD,PL,SM DC,SI <1 mm BD,UN,SM DC,CL 5 mm BD,UN,RO DC,CL <1 mm BD,PL,SM SO BD,PL,SM
5		- Broken core from 5.11 m to 5.14 m					BD,PL,SM BD,PL,SM BD,UN,SM SO BD,PL,SM
6		- Broken core from 6.47 m to 6.49 m			4	100	AJN,PL,H IN,CA <1 mm BD,PL,SM BD,PL,SM DC,CL <1 mm JN,PL,SM DC,CL <1 mm BD,PL,SM BD,PL,SM DC,SI <1 mm JN,PL,SM
7					5	100	BD,PL,SM BD,PL,SM
8							BD,PL,SM
9		- Lost core from 8.56 m to 8.59 m		55.62 9.38	6	100	BD,PL,SM BD,PL,SM DC,SI,SA 2 mm BD,PL,SM BD,PL,SM BD,PL,SM
		End of Drillhole Note(s): 1. Water level measured at a depth of 6.84 m (Elev. 58.16 m) on October 5, 2021					
10							
11							







DEPTH SCALE

1 : 50

GOLDER

LOGGED: RI

CHECKED: AG

PROJECT: 21494078

RECORD OF BOREHOLE: 21-07

SHEET 1 OF 2

LOCATION: N 5026297.0 ;E 361328.4

BORING DATE: September 30, 2021

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20, 40, 60, 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20, 40, 60, 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	66.07							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALT	0.05							Flush Mount Casing
		FILL - (SW) gravelly SAND, angular; grey (PAVEMENT STRUCTURE); non-cohesive, moist	65.82 0.25							
		FILL - (SP) SAND, fine to medium, trace sand; brown; non-cohesive, moist	65.64 0.43							
		FILL - (SM) gravelly SILTY SAND; dark brown; non-cohesive, moist, loose	65.16 0.91							Bentonite Seal
1		Highly weathered BEDROCK		1	SS	92				
			64.45 1.62	2	SS	50/ 4"				
		Borehole continued on RECORD OF DRILLHOLE 21-07								
2										
3										
4										
5										
6										
7										
8										
9										
10										

DEPTH SCALE

1 : 50

GOLDER

LOGGED: RI

CHECKED: AG

PROJECT: 21494078

RECORD OF DRILLHOLE: 21-07

SHEET 2 OF 2

LOCATION: N 5026297.0 ;E 361328.4

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 30, 2021

DRILL RIG: CME 55

DRILLING CONTRACTOR: CCC Drilling

DATUM: Geodetic

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate

BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN FLUSH	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 m	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	ROCK STRENGTH INDEX	WEATH-ERING INDEX	Q AVG.
		BEDROCK SURFACE		64.45														
	Rotary Drill HQ3 Core	Slightly weathered to fresh, medium to thickly bedded, medium grey, fine grained, non to faintly porous, medium strong DOLOSTONE, interbedded with shale, limestone and sandstone		1.62														

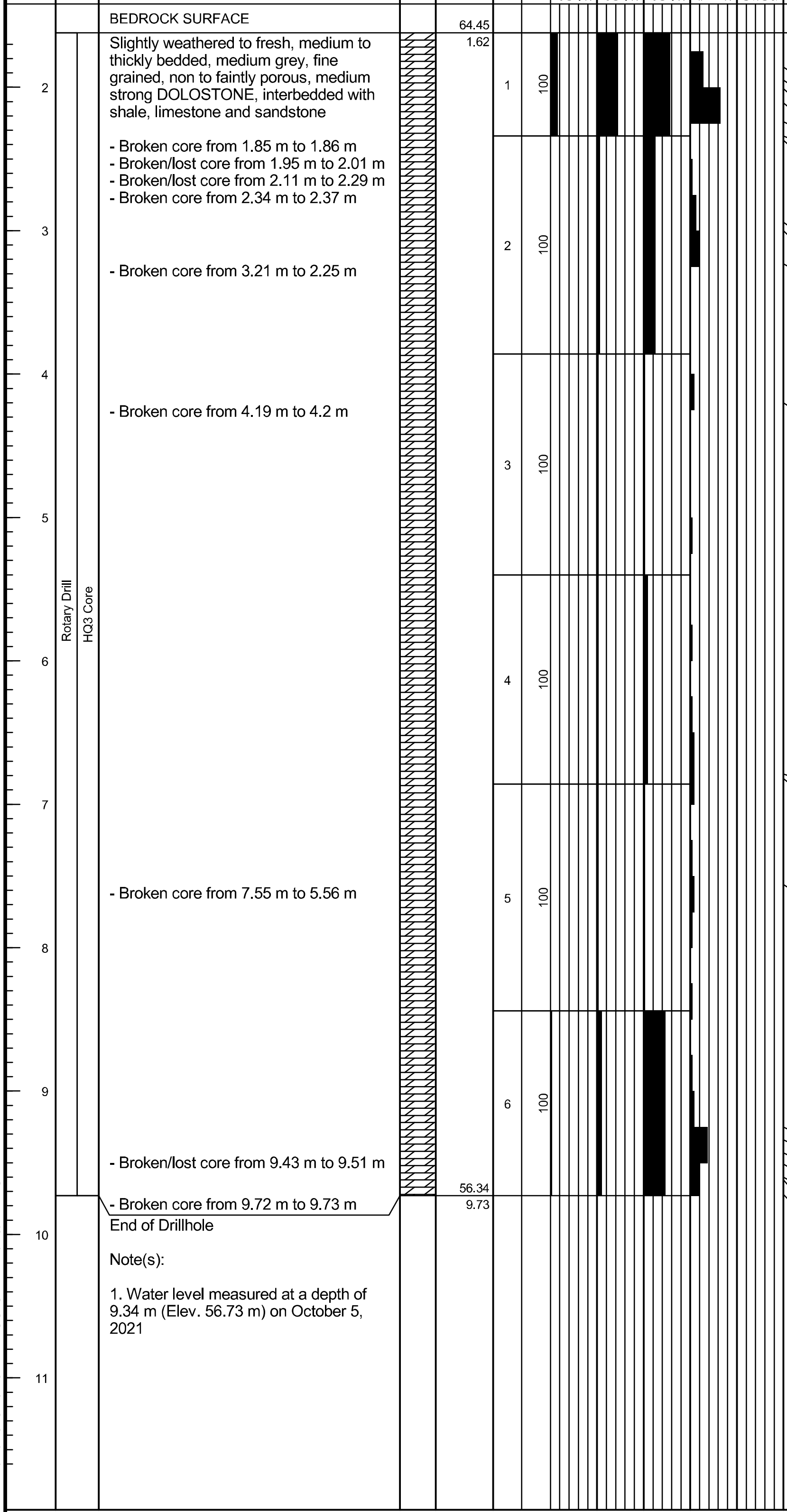
Slightly weathered to fresh, medium to thickly bedded, medium grey, fine grained, non to faintly porous, medium strong DOLOSTONE, interbedded with shale, limestone and sandstone

- Broken core from 1.85 m to 1.86 m
- Broken/lost core from 1.95 m to 2.01 m
- Broken/lost core from 2.11 m to 2.29 m
- Broken core from 2.34 m to 2.37 m
- Broken core from 3.21 m to 2.25 m
- Broken core from 4.19 m to 4.2 m
- Broken core from 7.55 m to 5.56 m
- Broken/lost core from 9.43 m to 9.51 m
- Broken core from 9.72 m to 9.73 m

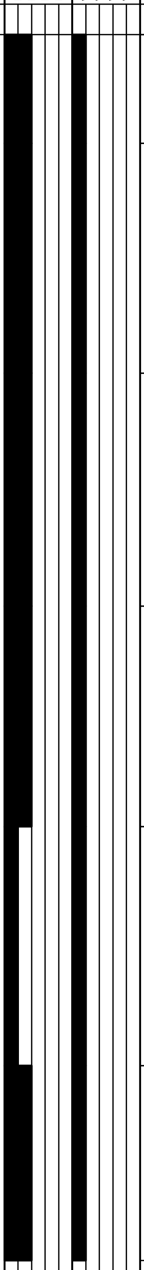
End of Drillhole

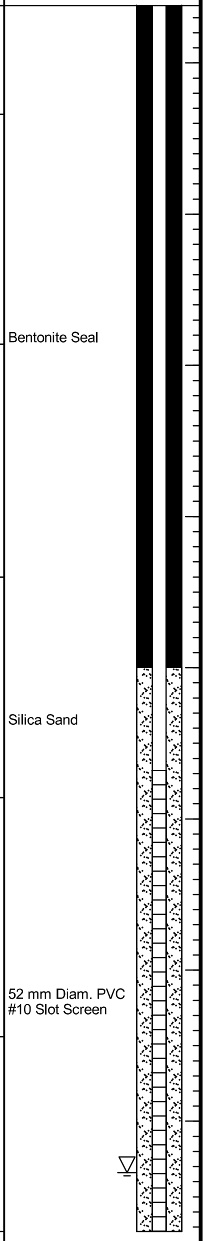
Note(s):

1. Water level measured at a depth of 9.34 m (Elev. 56.73 m) on October 5, 2021



BD,UN,SM SO
BD,UN,SM SO
BD,UN,SM SO <1 mm
BD,PL,SM SO
BD,UN,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,UN,SM
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO/DC,SI,SA <1 mm
BD,UN,SM SO
BD,UN,SM SO
BD,UN,RO
BD,PL,SM
BD,PL,SM SO
BD,UN,RO SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,SM SO
BD,PL,SM SO
BD,PL,RO SO
BD,PL,SM SO
BD,CU,SM SO
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM
BD,PL,SM
BD,UN,SM





DEPTH SCALE
1 : 50

GOLDER

LOGGED: RI
CHECKED: AG

PROJECT: 21494078

RECORD OF BOREHOLE: 21-08

SHEET 1 OF 3

LOCATION: N 5026385.1 ;E 361306.5

BORING DATE: September 28, 2021

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ├──○W──┤ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	64.64							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALT	0.05							
		FILL - (SW) gravelly SAND, angular; grey (PAVEMENT STRUCTURE); non-cohesive, moist	0.16							
		FILL - (SP) SAND, fine to medium, trace silt; brown; non-cohesive, moist	64.11 0.53							
1		FILL - (SM) gravelly SILTY SAND; dark brown, contains organic matter (rootlets); non-cohesive, moist, loose to compact		1	SS	6				
2		(SM) gravelly SILTY SAND; grey to grey brown, trace organic matter, weathered shale and thick laminations to thin beds of sand, fine to medium (GLACIAL TILL); non-cohesive, moist, compact to very dense	62.81 1.83	2	SS	23				
				3	SS	56				
3		Highly weathered BEDROCK	61.59 3.05	4	SS	50/ 6"				
		Borehole continued on RECORD OF DRILLHOLE 21-08	3.2							
4										
5										
6										
7										
8										
9										
10										

MIS-BHS 001 21494078.GPJ GAL-MIS.GDT 12/16/21 ZS

DEPTH SCALE

1 : 50

GOLDER

LOGGED: RI

CHECKED: AG

PROJECT: 21494078

RECORD OF DRILLHOLE: 21-08

LOCATION: N 5026385.1 ;E 361306.5
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 28, 2021
DRILL RIG: CME 55
DRILLING CONTRACTOR: CCC Drilling

DATUM: Geodetic

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate

BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

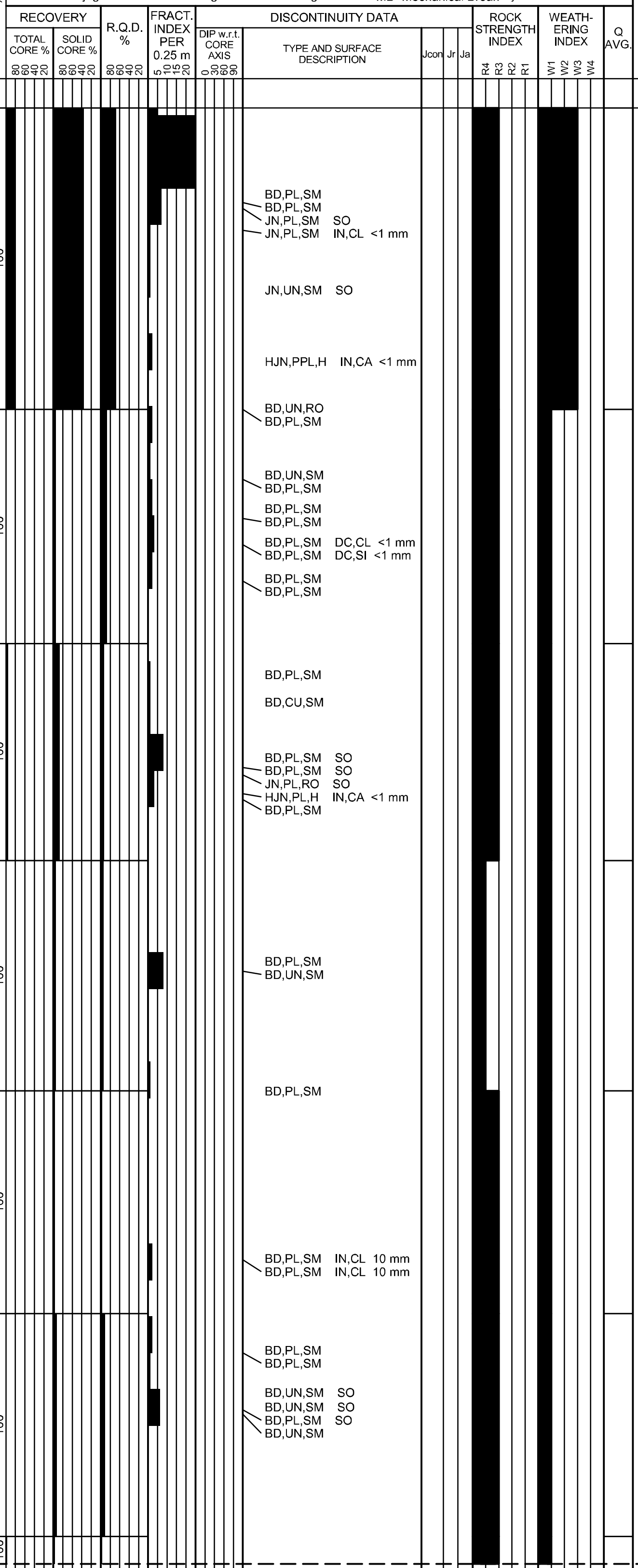
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	COLOUR / % RETURN (FLUSH)	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION
		BEDROCK SURFACE	61.44			
	Rotary Drill HQ3 Core	Slightly weathered to fresh, medium to thickly bedded, medium grey, fine grained, non to faintly porous, medium strong DOLOSTONE, interbedded with shale, limestone and sandstone	3.20	1	100	
4		- Broken/lost core from 3.2 m to 3.79 m				BD,PL,SM BD,PL,SM JN,PL,SM SO JN,PL,SM IN,CL <1 mm JN,UN,SM SO
5						HJN,PPL,H IN,CA <1 mm BD,UN,RO BD,PL,SM
6				2	100	BD,UN,SM BD,PL,SM BD,PL,SM BD,PL,SM BD,PL,SM DC,CL <1 mm BD,PL,SM DC,SI <1 mm BD,PL,SM BD,PL,SM
7				3	100	BD,PL,SM BD,CU,SM
8		- Broken/lost core from 7.66 m to 7.73 m				BD,PL,SM SO BD,PL,SM SO JN,PL,RO SO HJN,PL,H IN,CA <1 mm BD,PL,SM
9		- Broken core from 9.06 m to 9.13 m	55.51 9.13	4	100	BD,PL,SM BD,UN,SM
		Fresh, thinly to thickly bedded, light grey, fine to medium grained, non to faintly porous, medium strong SANDSTONE, with thin partings of shale				
10						BD,PL,SM
11		- Clay seam from 11.10 m to 11.11 m		5	100	BD,PL,SM IN,CL 10 mm BD,PL,SM IN,CL 10 mm
12		- Broken core from 11.73 m to 11.75 m		6	100	BD,PL,SM BD,PL,SM
		- Broken core from 12.14 m to 12.17 m				BD,UN,SM SO BD,UN,SM SO BD,PL,SM SO BD,UN,SM
13				7	100	



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DEPTH SCALE
1 : 50

GOLDER

LOGGED: RI
CHECKED: AG

PROJECT: 21494078

RECORD OF DRILLHOLE: 21-08

SHEET 3 OF 3

LOCATION: N 5026385.1 ;E 361306.5
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 28, 2021
DRILL RIG: CME 55
DRILLING CONTRACTOR: CCC Drilling

DATUM: Geodetic

JN - Joint	BD- Bedding	PL - Planar	PO- Polished	BR - Broken Rock
FLT - Fault	FO- Foliation	CU- Curved	K - Slickensided	**NOTE:** For additional abbreviations refer to list of abbreviations & symbols.
SHR- Shear	CO- Contact	UN- Undulating	SM- Smooth	
VN - Vein	OR- Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB- Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN FLUSH	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 m 5 10 15 20	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	ROCK STRENGTH INDEX R4 R3 R2 R1	WEATH-ERING INDEX W1 W2 W3 W4	Q AVG.
		--- CONTINUED FROM PREVIOUS PAGE ---																
14	Rotary Drill HQ3 Core	Fresh, thinly to thickly bedded, light grey, fine to medium grained, non to faintly porous, medium strong SANDSTONE, with thin partings of shale - Lost core from 13.59 m to 13.60 m			7	100						BD,PL,SM SO BD,UN,SM SO BD,UN,SM SO						
15				49.15	8	100						BD,UN,RO						
		End of Drillhole		15.49														
16																		
17																		
18																		
19																		
20																		
21																		
22																		
23																		

DEPTH SCALE
1 : 50

GOLDER

LOGGED: RI
CHECKED: AG

PROJECT: 21494078

RECORD OF BOREHOLE: 21-09

SHEET 1 OF 3

LOCATION: N 5026279.3 ;E 361293.7

BORING DATE: September 29, 2021

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ : 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl : 20 40 60 80

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE / SHEAR STRENGTH	HYDRAULIC CONDUCTIVITY / WATER CONTENT PERCENT	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		65.90							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALT									Flush Mount Casing
		FILL - (SW) gravelly SAND, angular; grey (PAVEMENT STRUCTURE)		0.05							
		FILL - (SP) SAND, fine to medium, trace to some silt; brown; non-cohesive, moist		65.65 0.25							
		FILL - (SM/ML) gravelly SILTY SAND to sandy SILT; brown to dark brown, contains weathered shale and organic matter; non-cohesive, moist, loose		65.34 0.56	1	SS	5				Bentonite Seal
		Highly weathered BEDROCK		64.38 1.52	2	SS	50/ 5"				
		Borehole continued on RECORD OF DRILLHOLE 21-09		1.65							
2											
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

GOLDER

LOGGED: RI

CHECKED: AG

PROJECT: 21494078

RECORD OF DRILLHOLE: 21-09

SHEET 2 OF 3

LOCATION: N 5026279.3 ;E 361293.7

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 29, 2021

DRILL RIG: CME 55

DRILLING CONTRACTOR: CCC Drilling

DATUM: Geodetic

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate

BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)
		BEDROCK SURFACE	64.25 1.65
2 3 4 5 6 7 8 9 10 11	Rotary Drill HQ3 Core	Fresh, medium to thickly bedded, medium grey, fine grained, non to faintly porous, medium strong DOLOSTONE, interbedded with shale, limestone and sandstone - Broken core from 1.65 m to 1.92 m - Broken core from 2.3 m to 2.41 m - Broken core from 3.37 m to 3.4 m - Broken/lost core from 8.09 m to 8.17 m - Broken/lost core from 9.86 m to 9.87 m - Broken core from 10.18 m to 10.26 m - Broken core from 10.73 m to 10.76 m	
		Fresh, thinly to thickly bedded, light grey, fine to medium grained, non to faintly porous, medium strong SANDSTONE, with thin partings of shale	54.73 11.17

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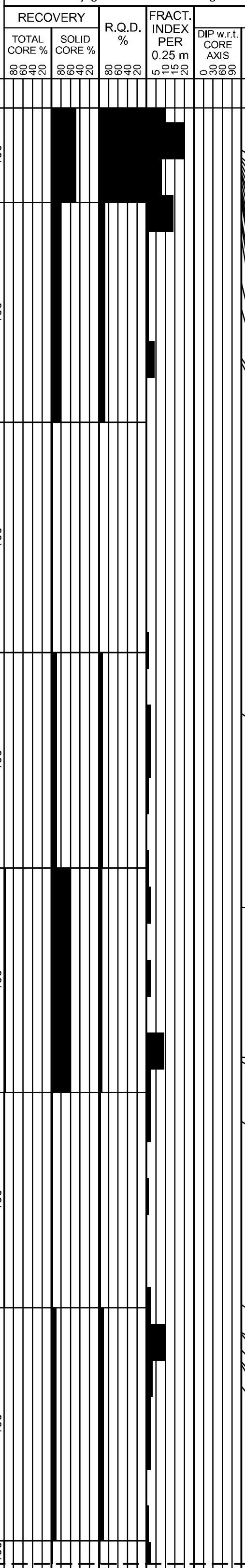
RUN No.	COLOUR % RETURN FLUSH
1	100
2	100
3	100
4	100
5	100
6	100
7	100
8	100



DISCONTINUITY DATA — TYPE AND SURFACE DESCRIPTION

BD,UN,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,UN,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
BD,PL,SM SO
JN,UN,RO SO
BD,PL,SM
BD,PL,SM
BD,UN,SM SO
BD,PL,SM SO

BD,PL,SM SO

BD,PL,SM
BD,CU,SM
BD,PL,SM
BD,PL,SM SO

JN,PL,SM SO

BD,PL,SM SO
JN,UN,RO SO
JN,PL,RO SO

BD,UN,SM SO

BD,PL,SM

BD,PL,SM
BD,PL,SM
BD,PL,SM SO

BD,PL,SM
BD,PL,SM SO
BD,PL,SM SO

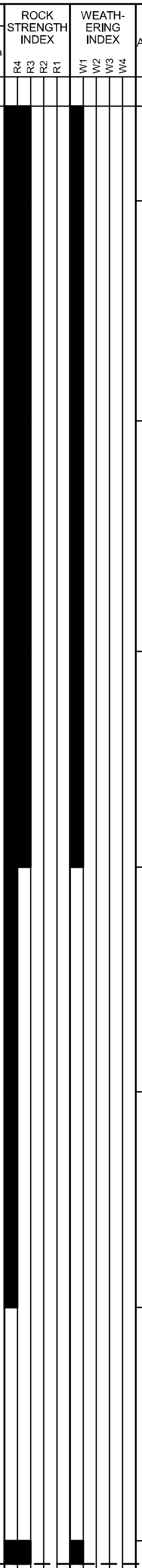
BD,PL,SM

BD,PL,SM SO
BD,PL,SM
BD,PL,SM
BD,PL,SM IN,CL 10 mm
BD,PL,SM IN,CL 10 mm
BD,UN,SM IN,CL 20 mm
BD,PL,SM SO
BD,PL,SM SO
BD,UN,SM SO

BD,PL,SM SO

BD,UN,RO

Jcon Jr Ja



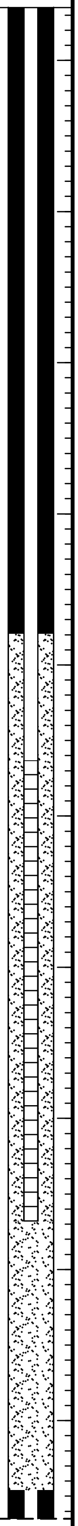
Bentonite Seal

Silica Sand

52 mm Diam. PVC
#10 Slot Screen

Silica Sand

Bentonite Seal



DEPTH SCALE
1 : 50

GOLDER

LOGGED: RI
CHECKED: AG

PROJECT: 21494078

RECORD OF DRILLHOLE: 21-09

SHEET 3 OF 3

LOCATION: N 5026279.3 ;E 361293.7

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 29, 2021

DRILL RIG: CME 55

DRILLING CONTRACTOR: CCC Drilling

DATUM: Geodetic

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate

BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage

PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular

PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	
		--- *CONTINUED FROM PREVIOUS PAGE* ---						
12	Rotary Drill HQ3 Core	Fresh, thinly to thickly bedded, light grey, fine to medium grained, non to faintly porous, medium strong SANDSTONE, with thin partings of shale - Broken core from 11.67 m to 11.68 m - Lost core from 12.42 m to 12.43 m			8	100	BD,UN,SM BD,UN,SM BD,UN,RO BD,UN,SM	
13								Bentonite Seal
14		- Broken core from 13.84 m to 13.85 m			9	100	BD,UN,SM BD,PL,SM SO	
15				50.40 15.50	10	100		
16		End of Drillhole Note(s): 1. Borehole was dry on October 5, 2021						
17								
18								
19								
20								
21								

RECOVERY: TOTAL CORE % 80 60 40 20; SOLID CORE % 80 60 40 20
R.Q.D. % 80 60 40 20
FRACT. INDEX PER 0.25 m 5 10 15 20
DIP w.r.t. CORE AXIS 0 30 60 90
Jcon Jr Ja
ROCK STRENGTH INDEX R4 R3 R2 R1
WEATHERING INDEX W1 W2 W3 W4
Q AVG.

DEPTH SCALE
1 : 50

GOLDER

LOGGED: RI
CHECKED: AG

PROJECT: 21494078

RECORD OF BOREHOLE: 21-10

SHEET 1 OF 3

LOCATION: N 5026360.8 ;E 361363.7

BORING DATE: September 29, 2021

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20, 40, 60, 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ●; rem V. ⊕ U - ○; 20, 40, 60, 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}

WATER CONTENT PERCENT: Wp ├──○W──┤ Wl; 20, 40, 60, 80

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	65.89					
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALT	0.05					Flush Mount Casing
		FILL - (SM) gravelly SILTY SAND; brown; non-cohesive, moist	65.15 0.74					
1		FILL - (SM) gravelly SILTY SAND; grey brown, trace organic matter; non-cohesive, moist, compact	64.37 1.52	1	SS	10		
2		(SM) gravelly SILTY SAND; grey brown, contains cobbles and boulders (GLACIAL TILL); non cohesive, moist, dense to very dense		2	SS	46		
				3	SS	73		Bentonite Seal
3				4	RC	DD		
4				5	RC	DD		
			61.09 4.8	6	SS	>50		
5		Borehole continued on RECORD OF DRILLHOLE 21-10						
6								
7								
8								
9								
10								

DEPTH SCALE

1 : 50

GOLDER

LOGGED: RI

CHECKED: AG

PROJECT: 21494078

RECORD OF DRILLHOLE: 21-10

SHEET 2 OF 3

LOCATION: N 5026360.8 ;E 361363.7
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 29, 2021
DRILL RIG: CME 55
DRILLING CONTRACTOR: CCC Drilling

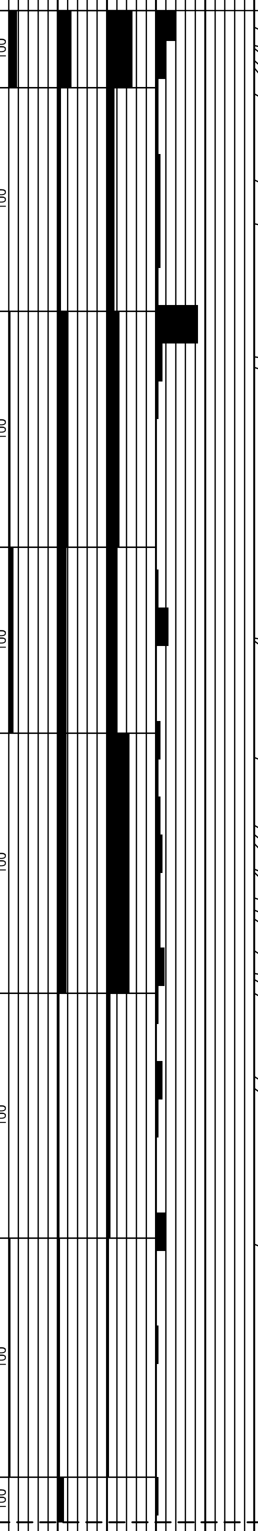
DATUM: Geodetic

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular
PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN FLUSH
		BEDROCK SURFACE	61.09 4.80		
5	Rotary Drill HQ3 Core	Fresh, medium to thickly bedded. medium grey, fine grained, non to faintly porous, medium strong DOLOSTONE, interbedded with shale, limestone and sandstone - Broken/lost core from 4.8 m to 4.88 m - Broken core from 5.03 m to 5.05 m		1	100
6				2	100
7		- Broken/lost core from 6.79 m to 7.02 m - Broken core from 7.09 m to 7.16 m		3	100
8					
9		- Broken/lost core from 8.72 m to 8.88 m - Broken core from 8.93 m to 8.97 m		4	100
10				5	100
11					
12				6	100
13		- Broken/lost core from 12.92 m to 12.96 m	52.73 13.16		
		Fresh, thinly to thickly bedded, light grey, fine to medium grained, non to faintly porous, medium strong SANDSTONE with thin partings of shale		7	100
14					
				8	100

CONTINUED NEXT PAGE



DISCONTINUITY DATA — TYPE AND SURFACE DESCRIPTION

BD,PL,SM
BD,CU,SM SO
BD,CU,SM SO
BD,UN,SM
BD,PL,SM SO
BD,PL,SM SO
BD,UN,SM CC,CA <1 mm
HJN,PL,H IN,CA <1 mm
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,SM
HJN,PL,H IN
BD,PL,SM Ca 3-5 mm
BD,PL,SM
BD,CU,SM
BD,PL,SM DC,CL <1 mm
BD,PL,RO
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,RO
BD,PL,SM
BD,UN,SM
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,SM
BD,PL,RO
BD,PL,SM
BD,CU,SM
BD,CU,SM
BD,UN,SM
BD,PL,SM
BD,CU,SM
BD,PL,SM
BD,UN,SM
BD,PL,SM
BD,UN,SM
BD,PL,SM
BD,PL,SM
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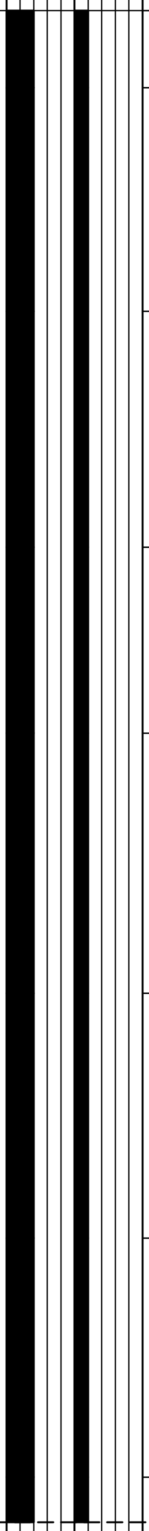
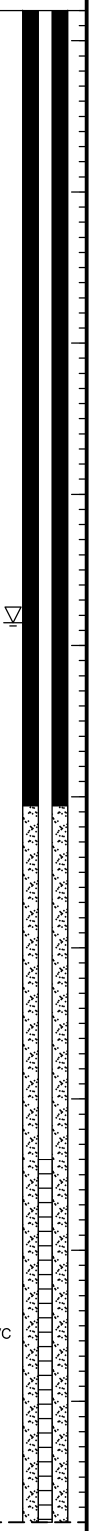
BD,UN,SM
BD,PL,SM
JN,PL,RO





DEPTH SCALE
1 : 50

LOGGED: RI
CHECKED: AG

PROJECT: 21494078

RECORD OF DRILLHOLE: 21-10

SHEET 3 OF 3

LOCATION: N 5026360.8 ;E 361363.7

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 29, 2021

DRILL RIG: CME 55

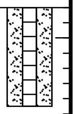
DRILLING CONTRACTOR: CCC Drilling

DATUM: Geodetic

JN - Joint
FLT - Fault
SHR- Shear
VN - Vein
CJ - Conjugate
BD- Bedding
FO- Foliation
CO- Contact
OR- Orthogonal
CL - Cleavage
PL - Planar
CU- Curved
UN- Undulating
ST - Stepped
IR - Irregular
PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break
BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	COLOUR / % RETURN	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	
		--- CONTINUED FROM PREVIOUS PAGE ---					
15	Rotary Drill HQ3 Core	Fresh, thinly to thickly bedded, light grey, fine to medium grained, non to faintly porous, medium strong SANDSTONE with thin partings of shale	50.44 15.45	8	100	JN,PL,RO	52 mm Diam. PVC #10 Slot Screen
		End of Drillhole					
		Note(s):					
16		1. Water level measured at a depth 8.85 m (Elev. 57.04 m) on October 5, 2021					



DEPTH SCALE

1 : 50

LOGGED: RI

CHECKED: AG

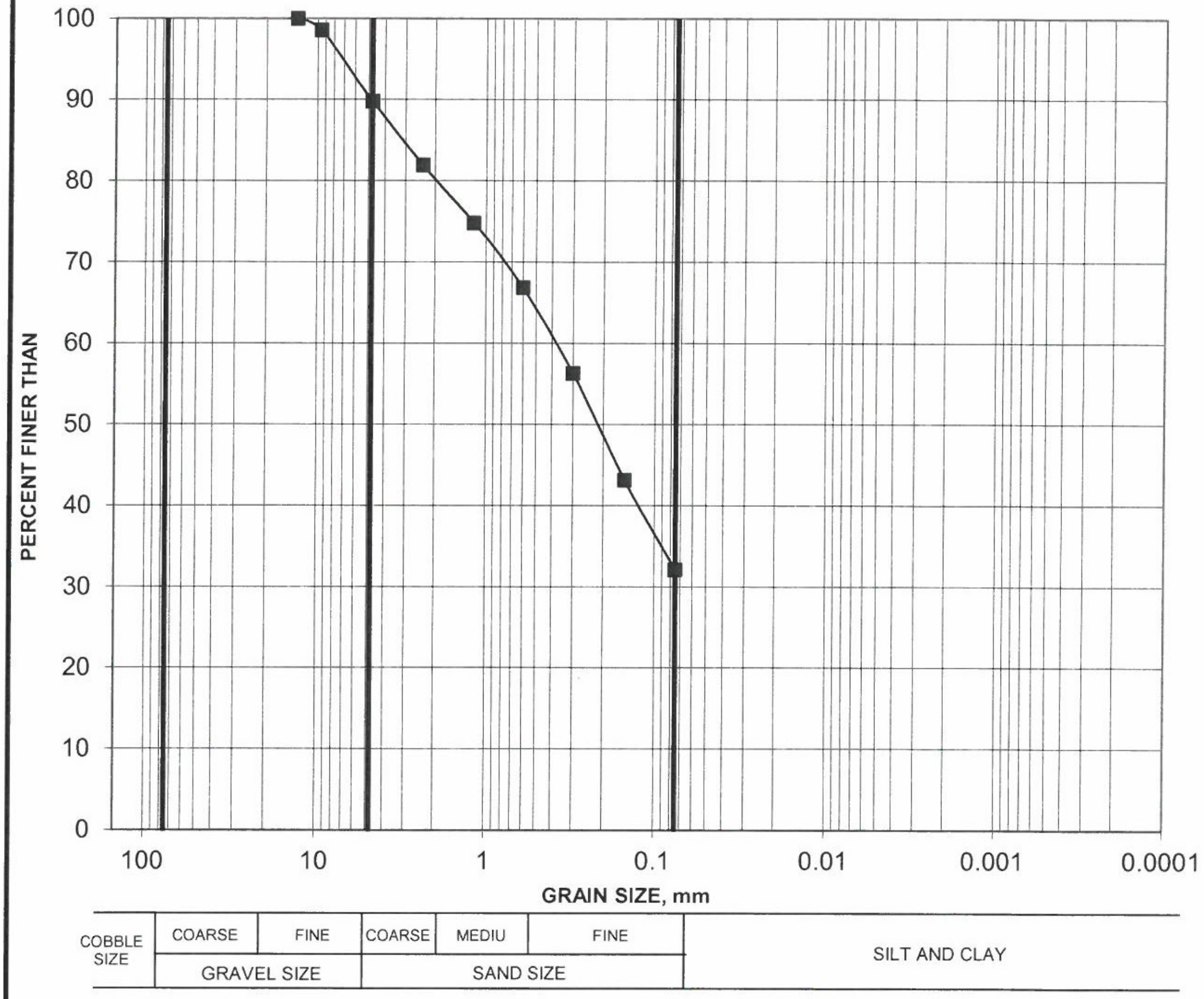
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

FIGURE B-1

SILTY SAND (FILL)



			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
—■— 21-01	2	0.61-1.22	10	58	32	

Project: 21494078/3000

GOLDER
MEMBER OF WSP

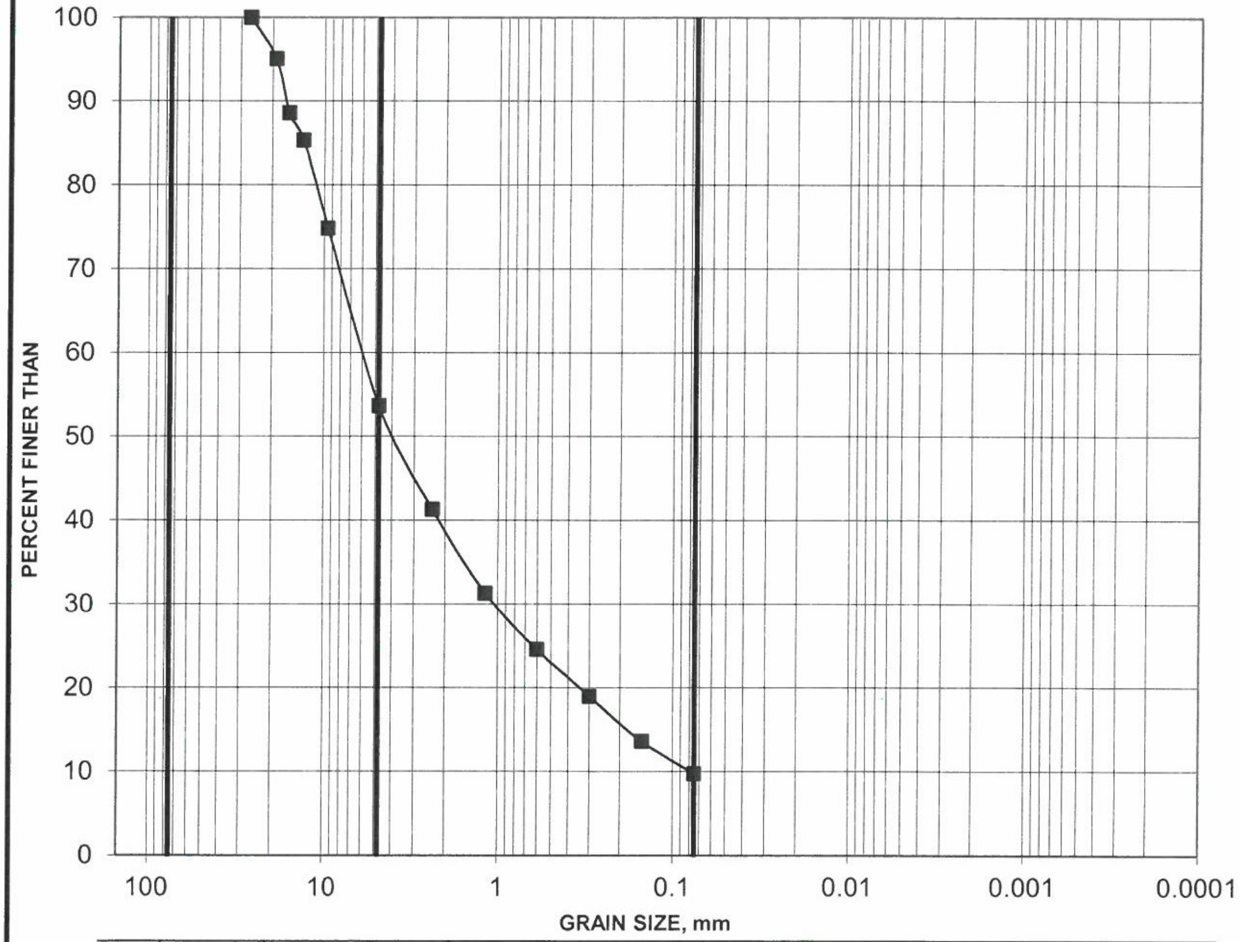
Created by:

Checked by: SB

GRAIN SIZE DISTRIBUTION

FIGURE B-2

GRAVELLY SAND (FILL)



COBBLE SIZE	COARSE	FINE	COARSE	MEDIU	FINE	SILT AND CLAY
	GRAVEL SIZE		SAND SIZE			

				Constituents (%)			
	Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
─■─	21-02	3	1.22-1.83	46	44	10	

Project: 21494078

GOLDER
MEMBER OF WSP

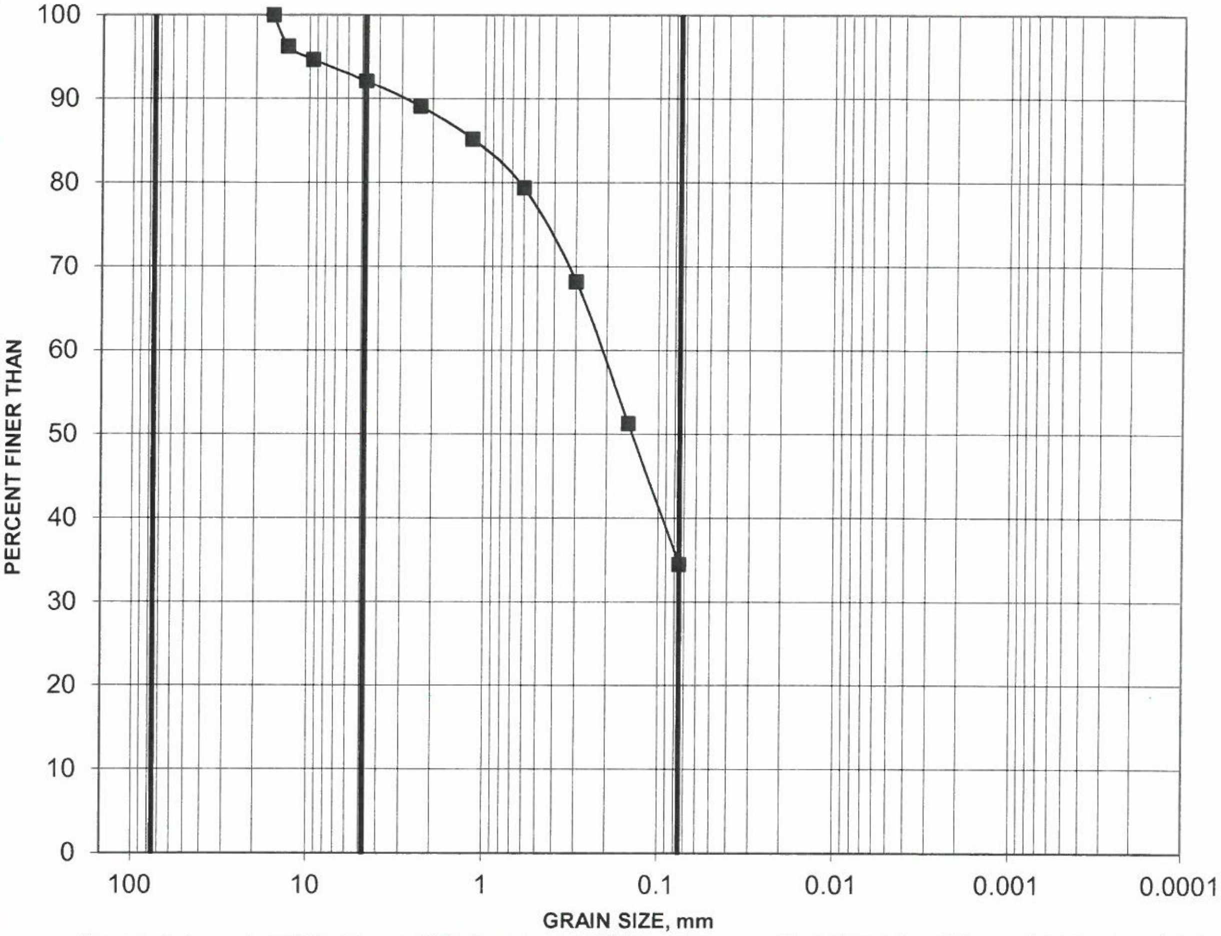
Created by: Cw

Checked by: JB

GRAIN SIZE DISTRIBUTION

FIGURE
B-3

SILTY SAND (GLACIAL TILL)



COBBLE SIZE	COARSE	FINE	COARSE	MEDIU	FINE	SILT AND CLAY
	GRAVEL SIZE		SAND SIZE			

				Constituents (%)			
	Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
■	21-08	3A	2.29-2.44	8	57	35	

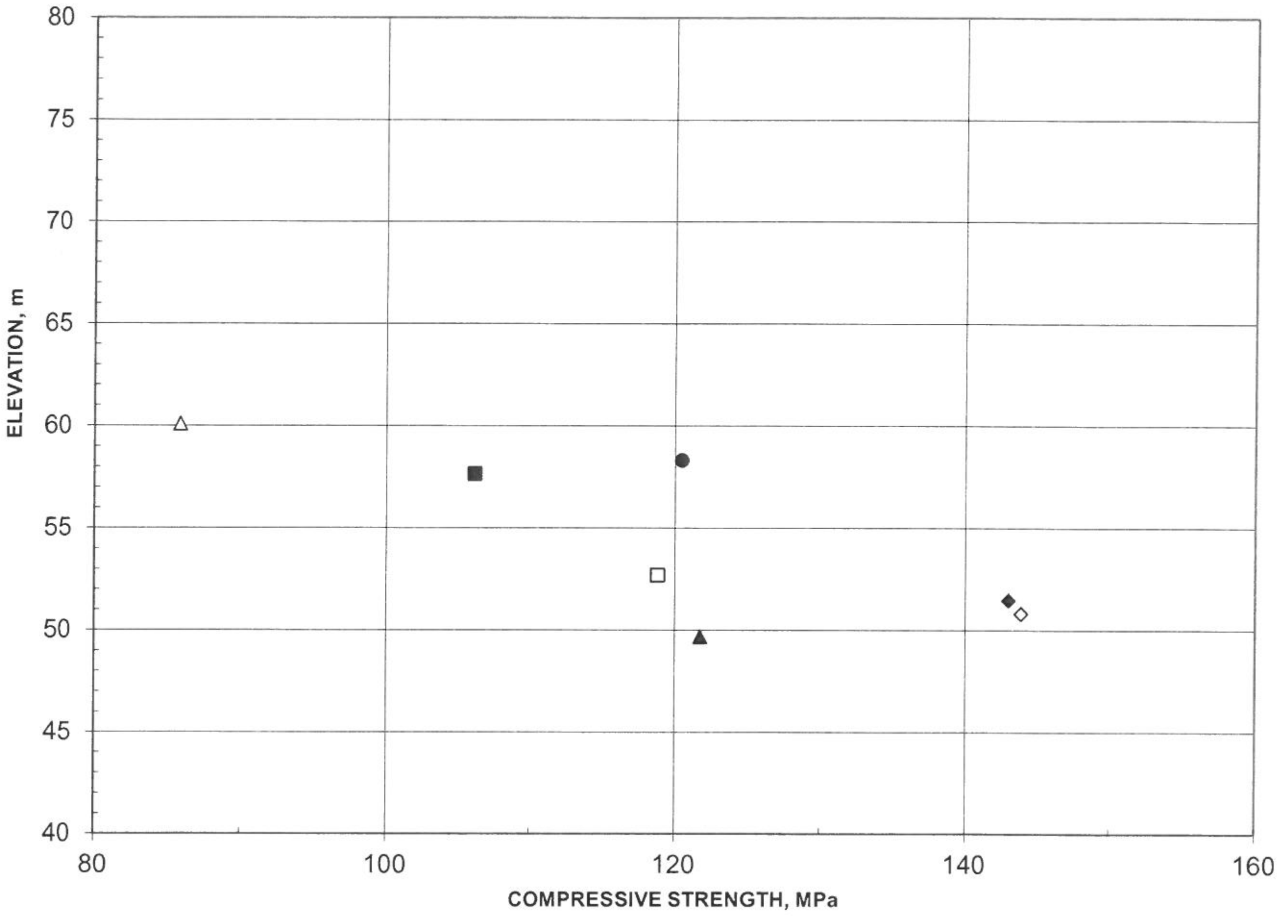
Project: 21494078

GOLDER
MEMBER OF WSP

Created by:
Checked by: JB

ASTM D7012 - Method C
UNCONFINED COMPRESSIVE STRENGTH OF ROCK CORE
SUMMARY OF LABORATORY TEST RESULTS

FIGURE
B-4



	Borehole	Depth (m)	L/D	Bulk Density (kg/m^3)	Lithology	UCS (MPa)	Failure Type
■	BH21-06 RC1	7.4	2.1	2669	shale/limestone	106	1
◆	BH21-08 RC1	13.2	2.1	2610	limestone	143	1
▲	BH21-08 RC2	15.0	2.1	2580	limestone	122	1
●	BH21-09 RC1	7.6	2	2640	limestone	120	1
□	BH21-09 RC2	13.2	2.0	2500	limestone	119	1
◇	BH21-09 RC3	15.1	2	2542	limestone	144	1
△	BH21-10 RC1	5.8	2.1	2671	shale/limestone	86	1

Notes:

Failure Types

1. Well formed cones on both ends
2. Well formed cones on one end, vertical cracks through cap
3. Columnar vertical craking through both ends
4. Diagonal fracture with no cracking through ends
5. Side fractures at top or bottom
6. Side fractures at both sides of top or bottom

Remarks

- Cores tested in vertical direction.
- Cores tested in air-dry condition.
- Time to failure > 2 and < 15 minutes.

GOLDER
MEMBER OF WSP

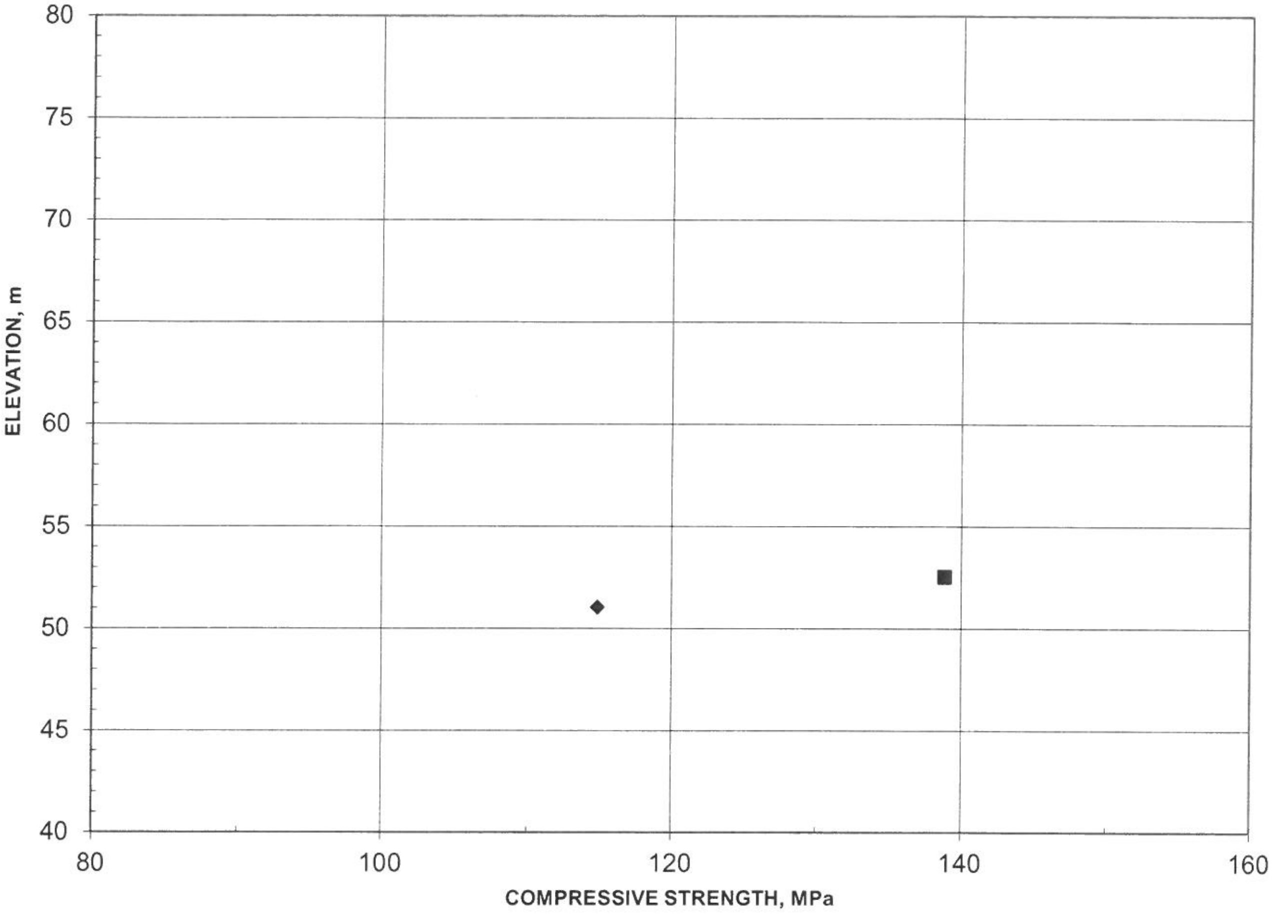
Created by:	CW
Checked by:	JB

Project: 21494078/3000

https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2021/21494078/

ASTM D7012 - Method C
UNCONFINED COMPRESSIVE STRENGTH OF ROCK CORE
SUMMARY OF LABORATORY TEST RESULTS

FIGURE B-5



	Borehole	Depth (m)	L/D	Bulk Density (kg/m^3)	Lithology	UCS (MPa)	Failure Type
■	BH21-10 RC2	13.3	2.2	2550	limestone	139	1
◆	BH21-10 RC3	14.8	2.2	2543	limestone	115	1

Notes:

Failure Types

1. Well formed cones on both ends
2. Well formed cones on one end, vertical cracks through cap
3. Columnar vertical craking through both ends
4. Diagonal fracture with no cracking through ends
5. Side fractures at top or bottom
6. Side fractures at both sides of top or bottom

Remarks

- Cores tested in vertical direction.
- Cores tested in air-dry condition.
- Time to failure > 2 and < 15 minutes.

Created by: CW
Checked by: JB

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MEMBER OF WSP

Project: 21494078/3000

https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2021/21494078/

APPENDIX C

Bedrock Core Photographs

BH 21-06 (Dry)
Rock core from a depth of 1.9 m to 9.4 m
Core Box 1 to 3 of 3

1.9 m

9.4 m

GOLDER

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-06
1 to 3 of 3

BH 21-06 (Wet)
Rock core from a depth of 1.9 m to 9.4 m
Core Box 1 to 3 of 3

1.9 m

9.4 m

GOLDER

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

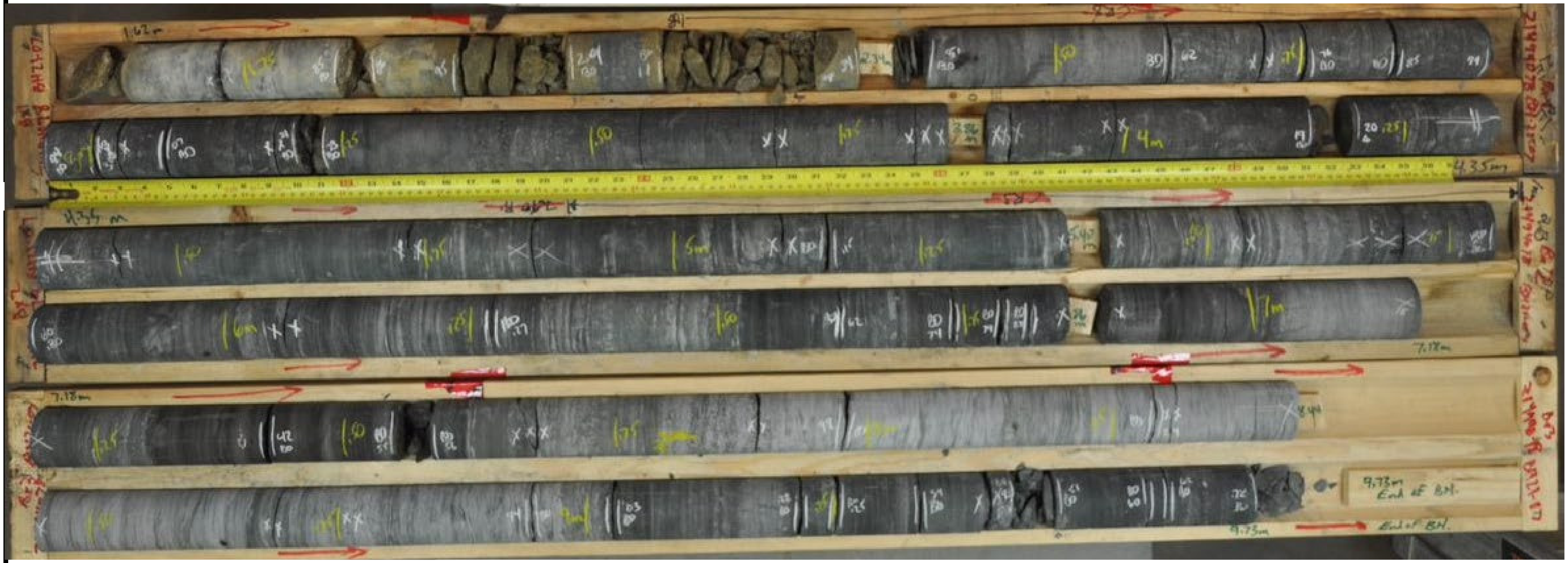
Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-06
1 to 3 of 3

BH 21-07 (Dry)
Rock core from a depth of 1.6 m to 9.7 m
Core Box 1 to 3 of 3

1.6 m



9.7 m

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-07
1 to 3 of 3

BH 21-07 (Wet)
Rock core from a depth of 1.6 m to 9.7 m
Core Box 1 to 3 of 3

1.6 m

9.7 m

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

Ottawa, ON

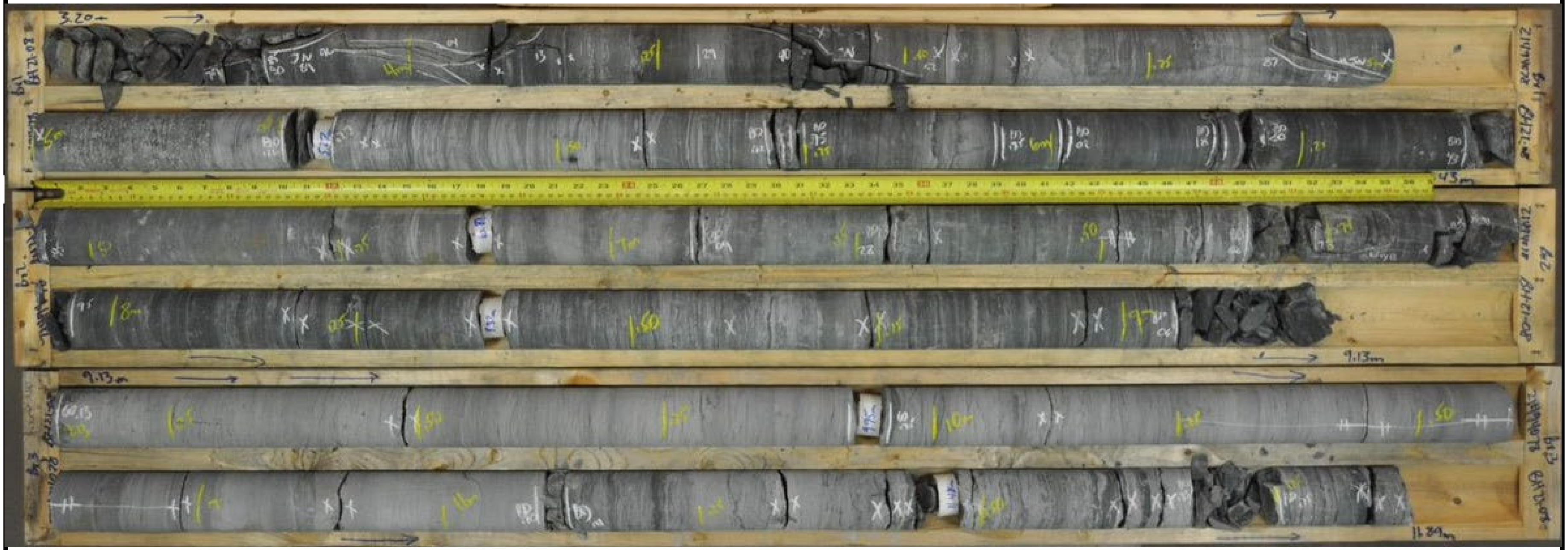
GOLDER

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-07
1 to 3 of 3

BH 21-08 (Dry)
Rock core from a depth of 3.2 m to 11.2 m
Core Box 1 to 3 of 5

3.2 m



11.2 m

GOLDER

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

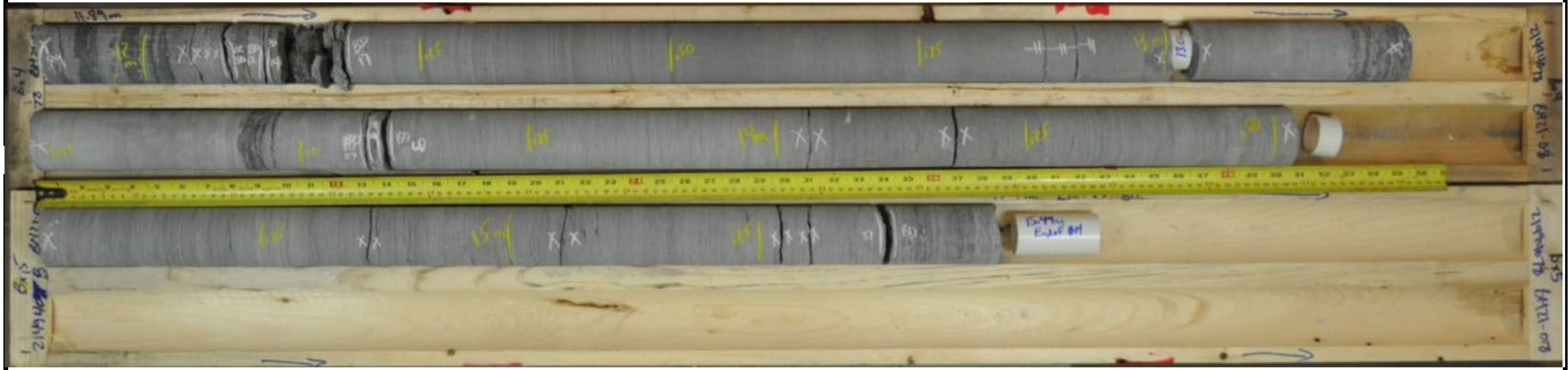
Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-08
1 to 3 of 5

BH 21-08 (Dry)
Rock core from a depth of 11.2 m to 15.5 m
Core Box 4 to 5 of 5

11.2 m



15.5 m

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

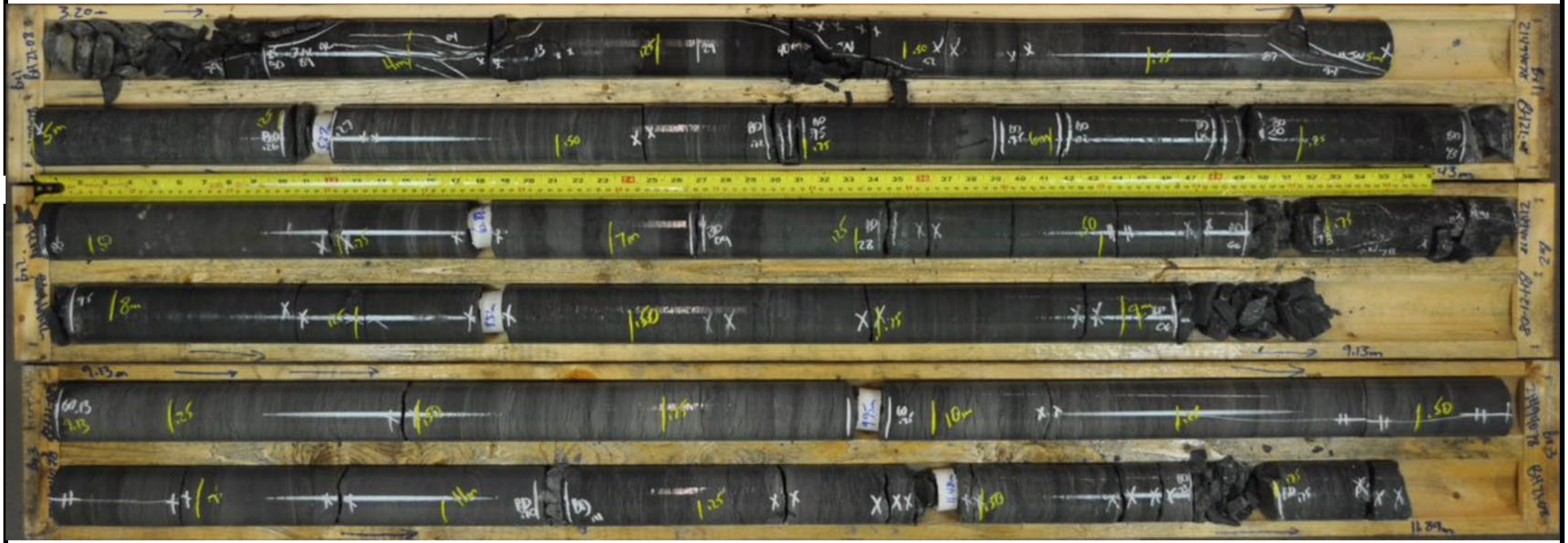
Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-08
4 to 5 of 5

BH 21-08 (Wet)
Rock core from a depth of 3.2 m to 11.2 m
Core Box 1 to 3 of 5

3.2 m



11.2 m

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-08
1 to 3 of 5

BH 21-08 (Wet)
Rock core from a depth of 11.2 m to 15.5 m
Core Box 4 to 5 of 5

11.2 m

15.5 m

GOLDER

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

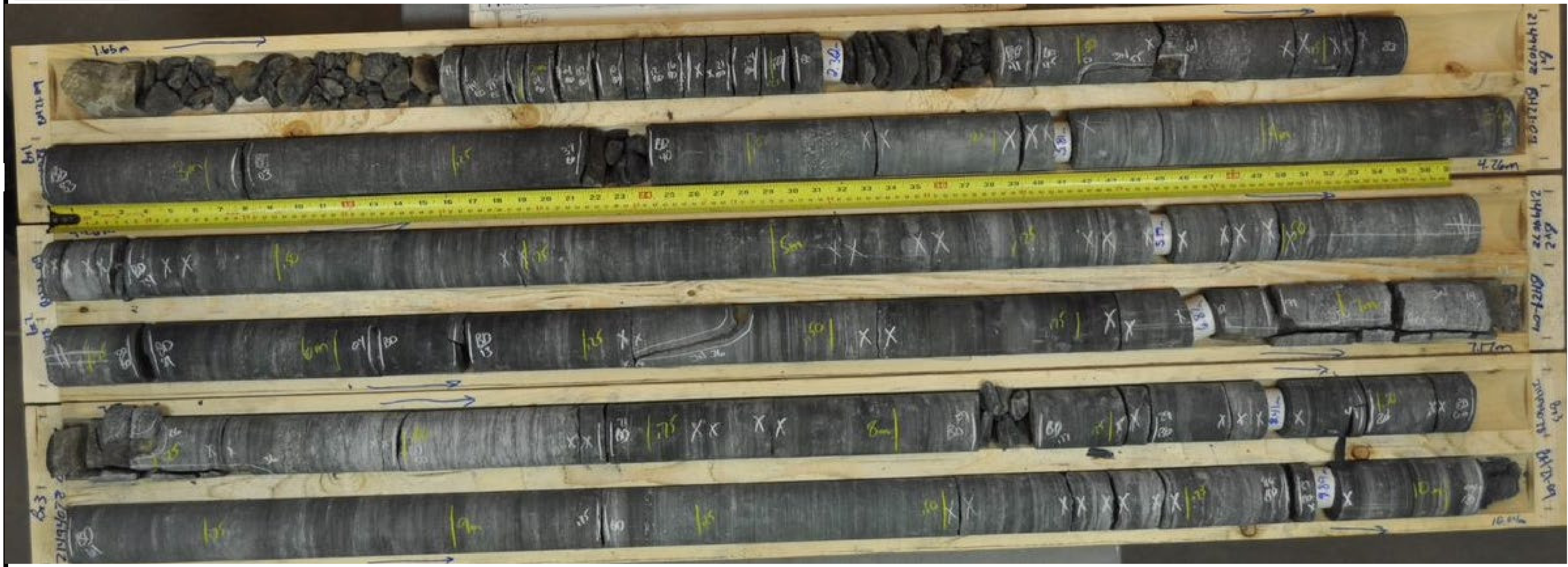
Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-08
4 to 5 of 5

BH 21-09 (Dry)
Rock core from a depth of 1.6 m to 10.0 m
Core Box 1 to 3 of 5

1.6 m



10.0 m

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Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

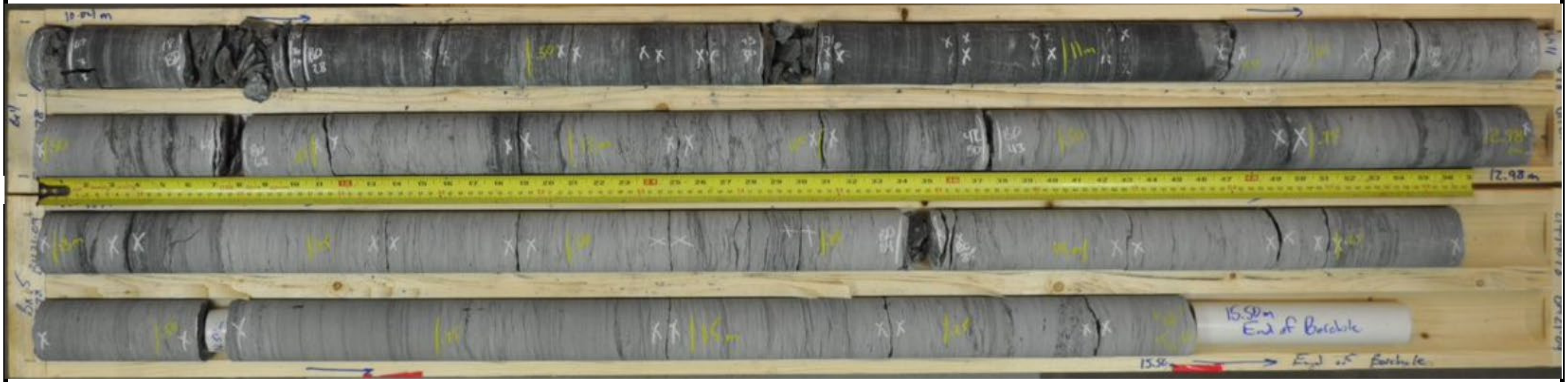
Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-09
1 to 3 of 5

BH 21-09 (Dry)
Rock core from a depth of 10.0 m to 15.5 m
Core Box 4 to 5 of 5

10.0 m



15.5 m

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

Ottawa, ON

BH 21-09
4 to 5 of 5

BH 21-09 (Wet)
Rock core from a depth of 1.6 m to 10.0 m
Core Box 1 to 3 of 5

1.6 m

10.0 m

GOLDER

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

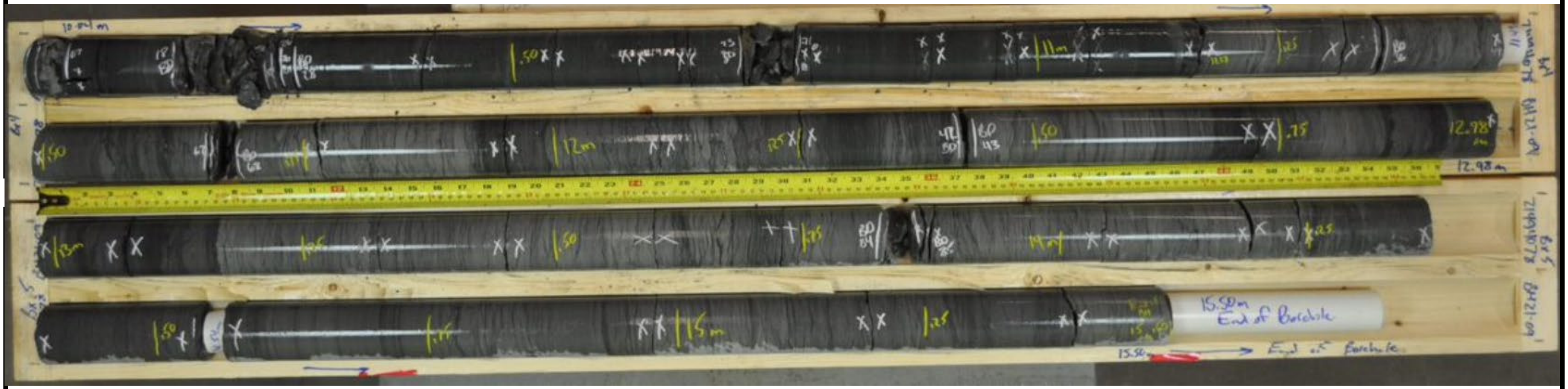
Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-09
1 to 3 of 5

BH 21-09 (Wet)
Rock core from a depth of 10.0 m to 15.5 m
Core Box 4 to 5 of 5

10.0 m



15.5 m

GOLDER

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-09
4 to 5 of 5

BH 21-10 (Dry)
Rock core from a depth of 2.7 m to 12.1 m
Core Box 1 to 3 of 5

2.7 m

12.1 m

GOLDER

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

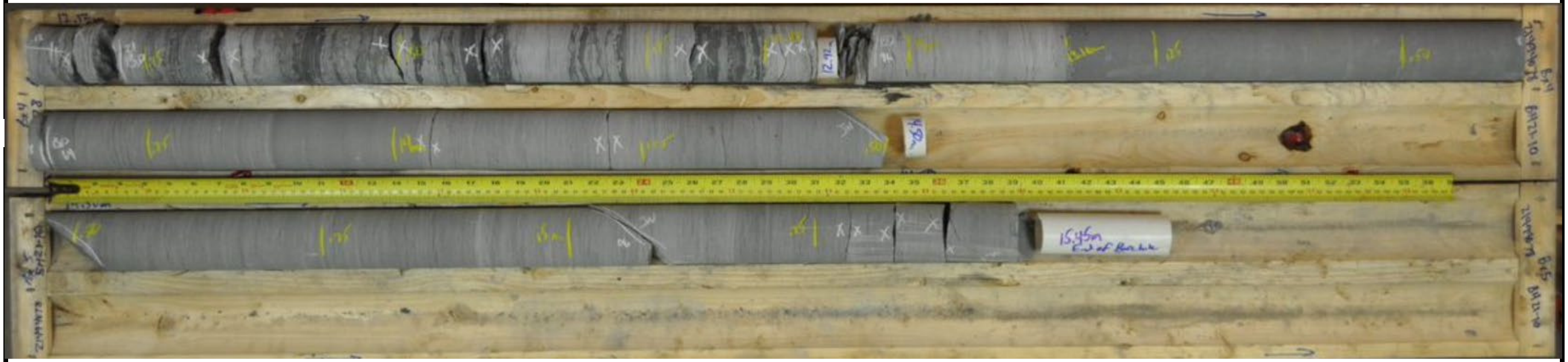
Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-10
1 to 3 of 5

BH 21-10 (Dry)
Rock core from a depth of 12.1 m to 15.4 m
Core Box 4 to 5 of 5

12.1 m



15.4 m

GOLDER

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

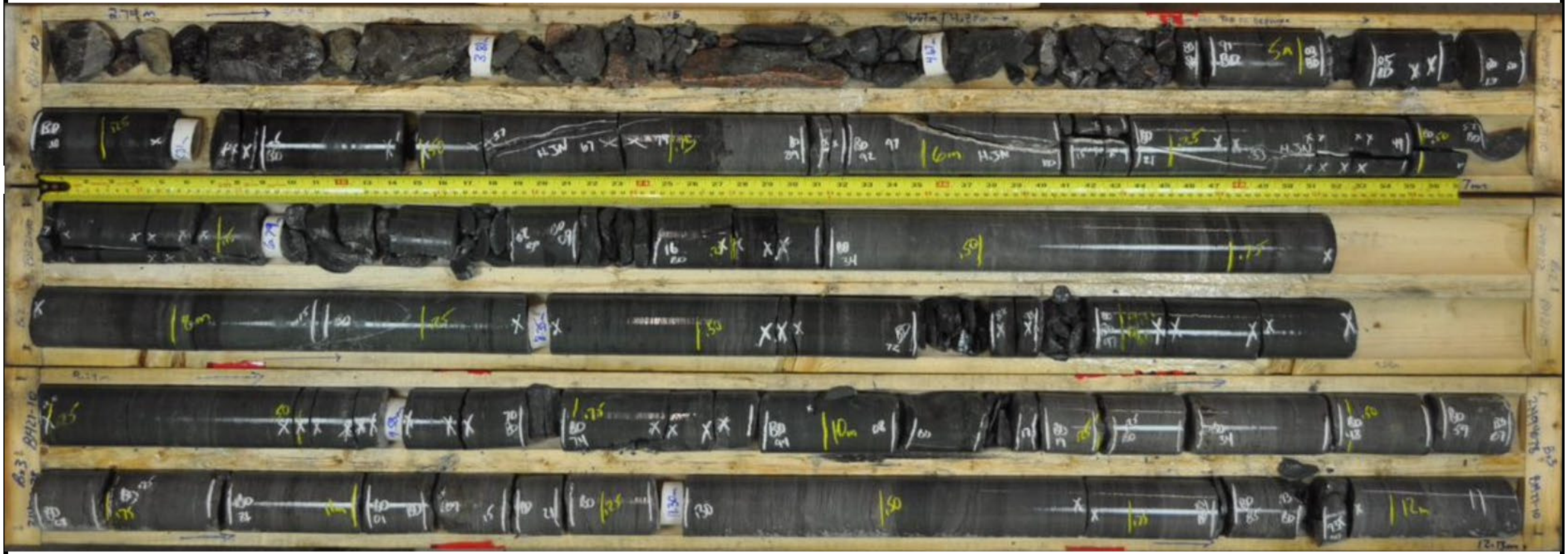
Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-10
4 to 5 of 5

BH 21-10 (Wet)
Rock core from a depth of 2.7 m to 12.1 m
Core Box 1 to 3 of 5

2.7 m



12.1 m

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

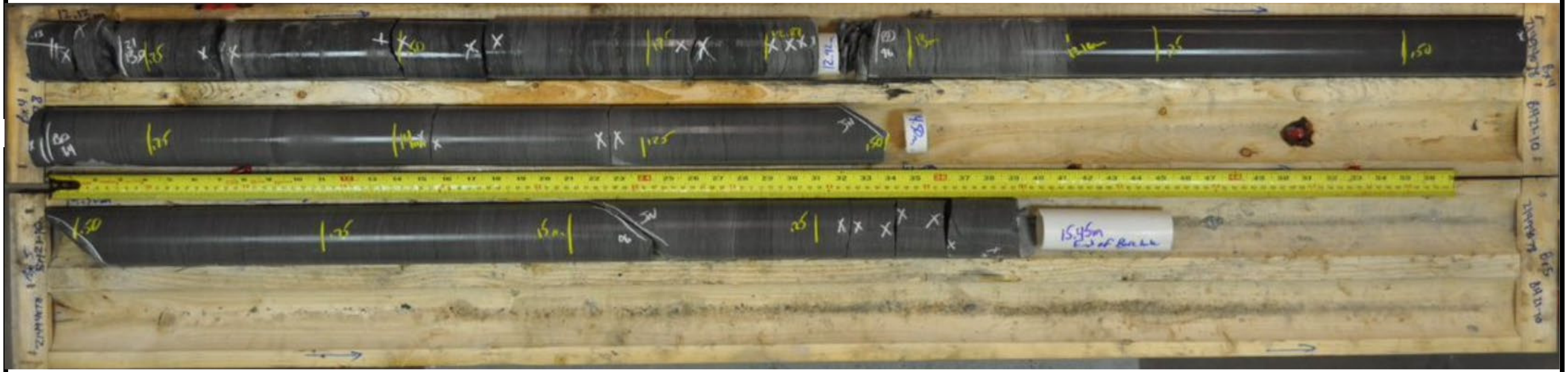
Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-10
1 to 3 of 5

BH 21-10 (Wet)
Rock core from a depth of 12.1 m to 15.4 m
Core Box 4 to 5 of 5

12.1 m



15.4 m

GOLDER

Environmental Assessment, Geotechnical and Hydrogeological Investigation

21494078- LPF Ph One Two RSC Richmond

Ottawa, ON

Project No.	21494078
Drawn:	AG
Date:	2021-10-08
Checked:	AG
Review:	WC

BH 21-10
4 to 5 of 5

APPENDIX D

Results of Basic Chemical Analyses

Certificate of Analysis

Client:	Golder Associates Ltd. (Ottawa) 1931 Robertson Road Ottawa, ON K2H 5B7	Report Number:	1964465
Attention:	Ms. Ali Ghirian	Date Submitted:	2021-10-12
PO#:		Date Reported:	2021-10-15
Invoice to:	Golder Associates Ltd. (Ottawa)	Project:	21494078
		COC #:	881198

Group	Analyte	MRL	Units	Guideline	1588443	1588444
				Lab I.D.	1588443	1588444
				Sample Matrix	Soil	Soil
				Sample Type		
				Sampling Date	2021-09-30	2021-09-27
				Sample I.D.	21-06 sa2	21-10 sa3
Anions	Cl	0.002	%		0.007	<0.002
	SO4	0.01	%		<0.01	0.01
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.24	0.15
	pH	2.00			8.88	8.39
	Resistivity	1	ohm-cm		4350	6670

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

APPENDIX E

Results of Geophysical Testing

GOLDER
MEMBER OF WSP

TECHNICAL MEMORANDUM

DATE October 27, 2021 21494078

TO Ali Ghirian
Golder Associates Ltd.

FROM Peter Giamou, Christopher Phillips **EMAIL pgiamou@golder.com; cphillips@golder.com**

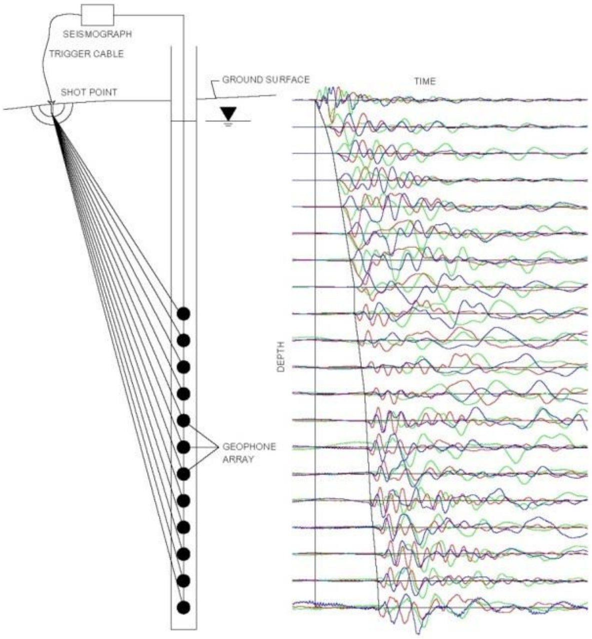
VERTICAL SEISMIC PROFILING RESULTS
1047 RICHMOND ROAD, OTTAWA, ONTARIO

This memorandum presents the results of two Vertical Seismic Profiling (VSP) testing carried out in Borehole 21-08 at 1047 Richmond Road, Ottawa, Ontario. VSP testing was carried out on October 6, 2021. Borehole 21-08 was drilled to an approximate depth of 15 m below the existing ground surface and then cased with a 2.5 inch PVC pipe grouted in place. The borehole consisted of approximately 3.2 m of sandy silt over dolostone and sandstone bedrock to the bottom of the borehole.

Methodology

For the VSP method, seismic energy is generated at the ground surface by an active seismic source and recorded by a geophone located in a nearby borehole at a known depth. The active seismic source can be either compression or shear wave. The time required for the energy to travel from the source to the receiver (geophone) provides a measurement of the average compression or shear-wave seismic velocity of the medium between the source and the receiver. Data obtained from different geophone depths are used to calculate a detailed vertical seismic velocity profile of the subsurface in the immediate vicinity of the test borehole.

The high-resolution results of a VSP survey are often used for earthquake engineering site classification, as per the National Building Code of Canada (NBCC).



Example 1: Layout and resulting time traces from a VSP survey.

Field Work

The field work was carried out on October 6, 2021, by personnel from the Golder Mississauga office.

At Borehole 21-08, compression and shear-wave seismic energy were generated from a sledge-hammer located 2.00 m from the borehole. The seismic source for the shear-wave test consisted of a 2.4-metre-long, 150 millimetre by 150 millimetre wooden beam, weighted by a vehicle and horizontally struck with a 9.9 Kg sledge-hammer on alternate ends of the beam to induce polarized shear waves. Test measurements started at ground surface and were recorded in the borehole with a 3-component receiver spaced at 1-metre intervals below the ground surface to the maximum depth of the casing (15 m).

The seismic records collected for each source location were stacked a minimum of three times to minimize the effects of ambient background seismic noise on the collected data. The data was sampled at 0.020833 millisecond intervals and a total time window of 0.341 seconds was collected for each seismic shot.

Data Processing

Processing of the VSP test results consisted of the following main steps:

1) Compilation of seismic records to present seismic traces for all depth intervals on a single plot for each seismic source and for each component;

2) Low Pass Filtering of data to remove spurious high-frequency noise;

3) First-break picking of the compression and shear-wave arrivals; and,

4) Calculation of the average compression and shear-wave velocity to each tested depth interval.

Processing of the VSP data was completed using the SeisImager/SW software package (Geometrics Inc.). The seismic records from Borehole 21-08 are presented on the following two plots and show the first-break picks of the compression wave (Figure 1) and shear wave arrivals (Figure 2) overlaid on the seismic waveform traces recorded at the different geophone depths. The arrivals were picked on the vertical component for the compression source and on the two horizontal components for the shear source.

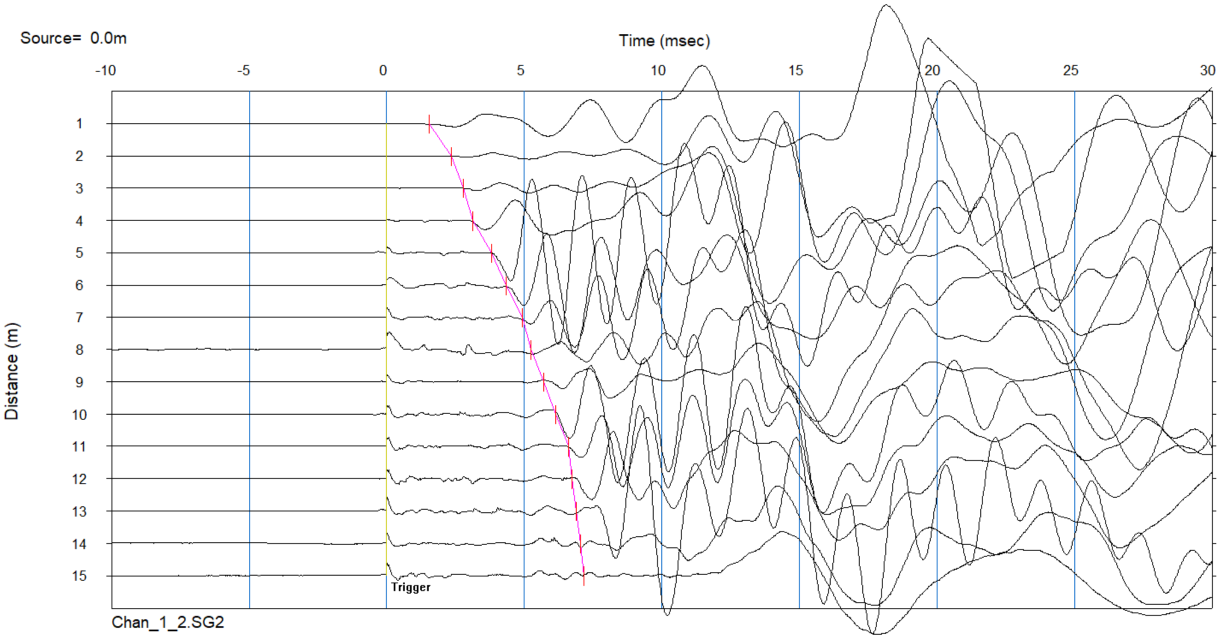


Figure 1: First-break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 21-08.

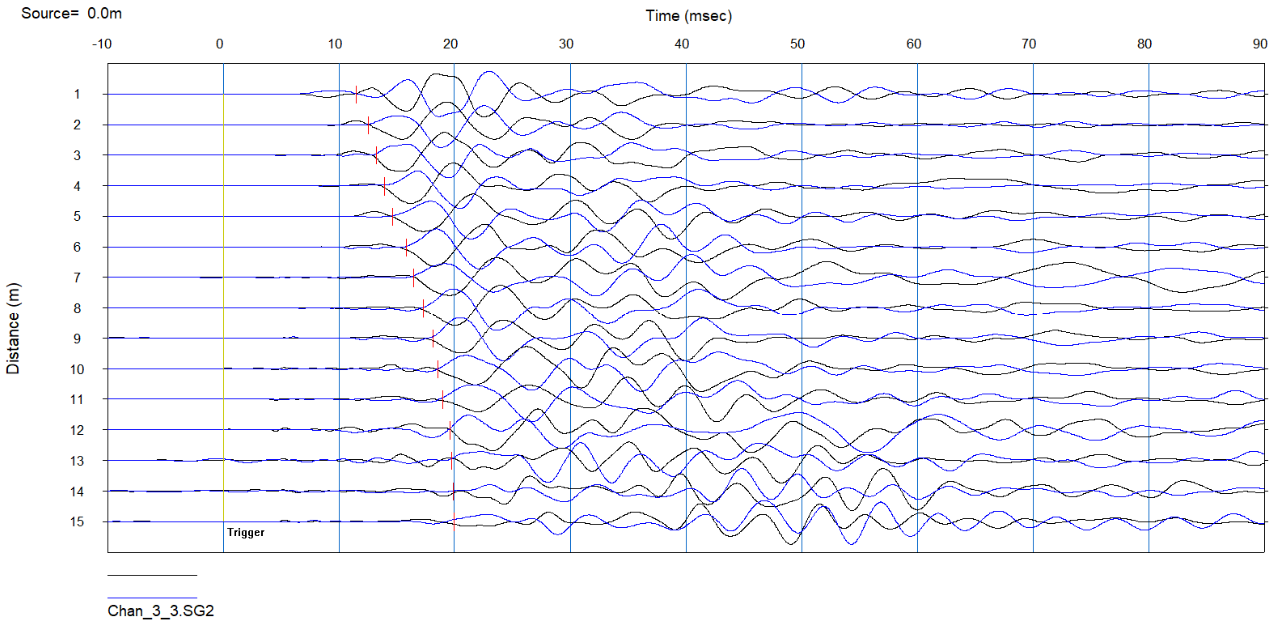


Figure 2: First-break picking of shear wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 21-08.

Results

The VSP results at Borehole 21-08 are summarized in Table 1. The shear wave and compression wave layer velocities were calculated by best-fitting a theoretical travel time model to the field data. The depths presented on the table are relative to ground surface.

The estimated dynamic engineering moduli, based on the calculated wave velocities, are also presented in Table 1. The engineering moduli were calculated using an estimated bulk density, based on the borehole log. An estimated bulk density of 2000 kg/m^3 was used for the overburden and an estimated bulk density of 2,600 kg/m^3 was used for the limestone bedrock.

At Borehole 21-08 the average shear wave velocity from ground surface to a depth of 30 metres was measured to be 1,171 metres per second. The average velocity at Borehole BH 21-08 was calculated assuming that the velocity from 15 metres to a depth of 30 metres was constant with an average shear-wave velocity value of 2,800 m/s which is equal to the velocity at the bottom of the borehole.

Limitations

This technical memorandum, which specifically includes all tables, figures and attachments, is based on data and information collected by Golder Associates Ltd. and is based solely on the conditions of the properties at the time of the work, supplemented by historical information and data obtained by Golder Associates Ltd. as described in this memo.

Golder Associates Ltd. has relied in good faith on all information provided and does not accept responsibility for any deficiency, misstatements, or inaccuracies contained in the reports as a result of omissions, misinterpretation, or fraudulent acts of the persons contacted or errors or omissions in the reviewed documentation.

The services performed, as described in this memo, were conducted in a manner consistent with that level of care and skill normally exercised by other members of the engineering and science professions currently practicing under similar conditions, subject to the time limits and financial and physical constraints applicable to the services.

Any use which a third party makes of this memo, or any reliance on, or decisions to be made based on it, are the responsibilities of such third parties. Golder Associates Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this memo.

The findings and conclusions of this memo are valid only as of the date of this memo. If new information is discovered in future work, including excavations, borings, or other studies, Golder Associates Ltd. should be requested to re-evaluate the conclusions of this memo, and to provide amendments as required.

Closure

We trust that these results meet your current needs. If you have any questions or require clarification, please contact the undersigned at your convenience.

Golder Associates Ltd.

Peter Giamou, B.Sc.,P. Geo
Senior Geophysicst
PG/CRP/jl

Christopher Phillips,M.Sc., P.Geo
Senior Geophysicist

Attachments: Table 1 – VSP Modeller BH 21-08

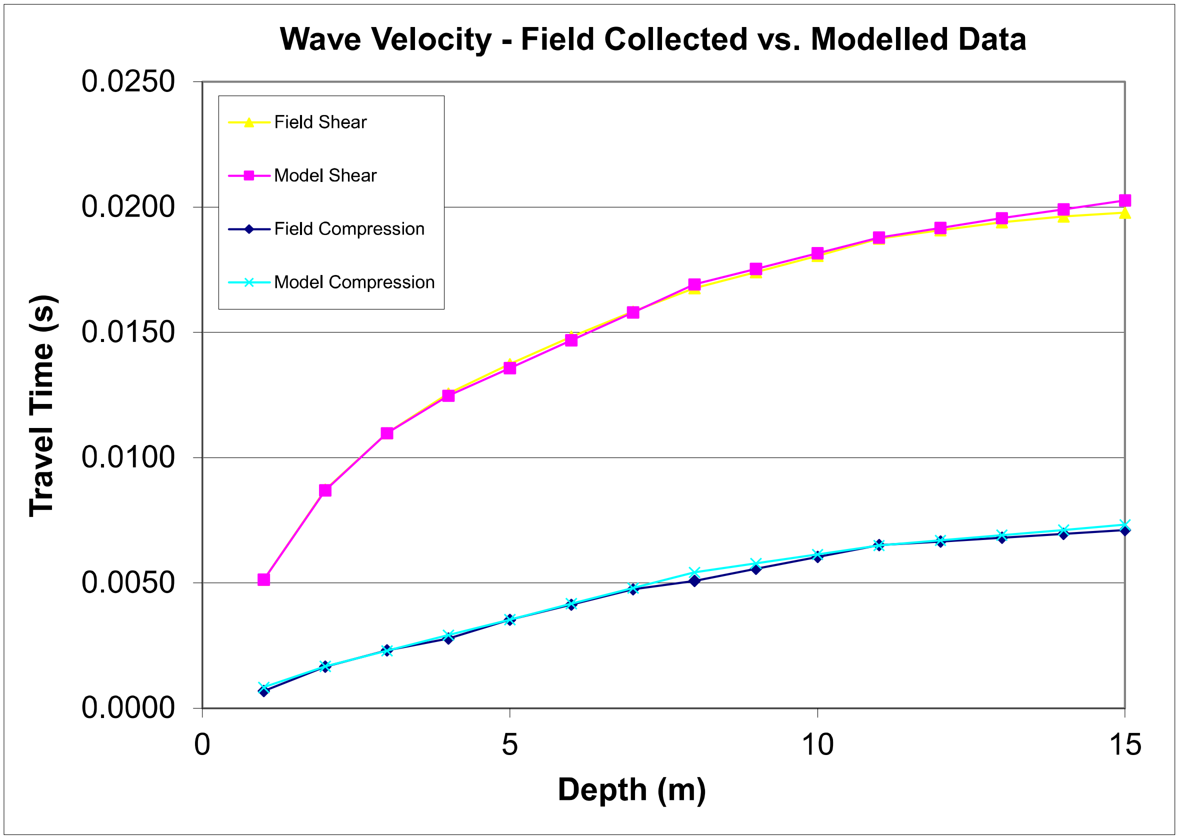
https://golderassociates.sharepoint.com/sites/152441/project files/5 technical work/geotechnical_1047 richmond rd/vsp survey/report/21494078 tech memo vsp model bh21-08 27oct2021.docx

TABLES

TABLE 1- VSP MODELLER BH21-08

TABLE 1
VSP VELOCITY PROFILE
BOREHOLE 21-08

Layer Depth (m)		Velocities (m/s)		Estimated Bulk Density (kg/m^3)	Dynamic Engineering Properties			
Top	Bottom	Compressional Wave	Shear Wave		Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0.0	1.0	400	195	2000	0.34	76	204	219
1.0	2.0	1200	280	2000	0.47	157	461	2671
2.0	3.0	1600	440	2000	0.46	387	1130	4604
3.0	4.0	1600	670	2600	0.39	1167	3253	5100
4.0	5.0	1600	900	2600	0.27	2106	5343	3848
5.0	6.0	1600	900	2600	0.27	2106	5343	3848
6.0	7.0	1600	900	2600	0.27	2106	5343	3848
7.0	8.0	1600	900	2600	0.27	2106	5343	3848
8.0	9.0	2800	1600	2600	0.26	6656	16741	11509
9.0	10.0	2800	1600	2600	0.26	6656	16741	11509
10.0	11.0	2800	1600	2600	0.26	6656	16741	11509
11.0	12.0	4800	2600	2600	0.29	17576	45430	36469
12.0	13.0	4800	2600	2600	0.29	17576	45430	36469
13.0	14.0	4800	2800	2600	0.24	20384	50638	32725
14.0	15.0	4800	2800	2600	0.24	20384	50638	32725



Notes
1. Depth presented is relative to the ground surface.
2. This table shall be analyzed in conjunction with the accompanying report.

APPENDIX F

Results of In-situ Hydraulic Conductivity Testing

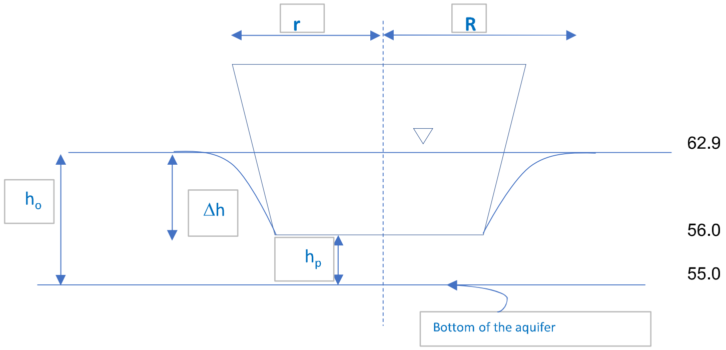
Inflow to Excavation

Dupuit-Forchheimer Equation: $Q=\pi K((h_o^2-hp^2)/\ln(R/r))$

1047 Richmond Road

K (m/sec)	1E-06	
h_0 (m)	7.9	**r - equivalent radius of pit**
h_p (m)	1.0	**R - radius of influence**
r (m)	**52.8**	**SF - safety factor**
SF	**2**	

	SF * Q (m^3/s)	R	Rad of Inf. from edge	m^3/day	L/day
Initial*	5.2E-03	57.8	5.0	450.11	450,108
	2.7E-03	62.8	10.0	234.68	234,677
	1.9E-03	67.8	15.0	162.75	162,751
	1.5E-03	72.8	20.0	126.69	126,687
Steady-State**	1.2E-03	77.8	25.0	104.97	104,972
	1.0E-03	82.8	30.0	90.44	90,437
	9.3E-04	87.8	35.0	80.01	80,008
	8.4E-04	92.8	40.0	72.15	72,148
	7.6E-04	97.8	45.0	66.01	66,005
	7.1E-04	102.8	50.0	61.06	61,065
	6.2E-04	112.8	60.0	53.60	53,596
	5.6E-04	122.8	70.0	48.20	48,202
	5.1E-04	132.8	80.0	44.11	44,110
	4.7E-04	142.8	90.0	40.89	40,891
	4.4E-04	152.8	100.0	38.29	38,286
	3.5E-04	202.8	150.0	30.23	30,232

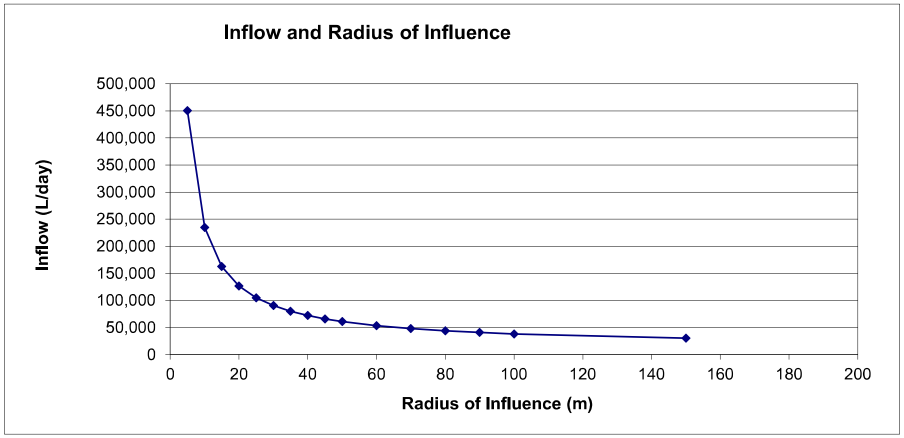


Excavation Assumptions

Width (m)	73	
Length (m)	120	
round surface elevation (masl)	65.50	
Groundwater elevation (masl)	62.89	*0.5 m higher than measured at 21-4*
Bottom of basement (masl)	56.5	*assumed 3 basement levels totalling 9 m depth*
Dewatered level (masl)	56.0	*0.5 m below depth of excavation*
Base of aquifer (masl)	55.0	*assumed 1 m below dewatered level*

Sichart and Kyrieleis Equation: $R=3000\Delta h(K^{1/2})$

Radius of Influence of Excavation (m) 23



Rainfall Amount - Based on a 79.2 mm precipitation event in 24 hours with a return of 10 years

Excavation Area (m^2)	8760
10 year Rainfall event (m)	0.0792
Max Vol Precipiation (L)	693,792

Notes

L - litres
m - metres
mbgs - metres below ground surface
Initial*: Potential worst-case inflow rate when trench is initially rapidly dewatered
Steady-State**: Steady state inflow rate

Prepared by: CAMC
Checked by: BH

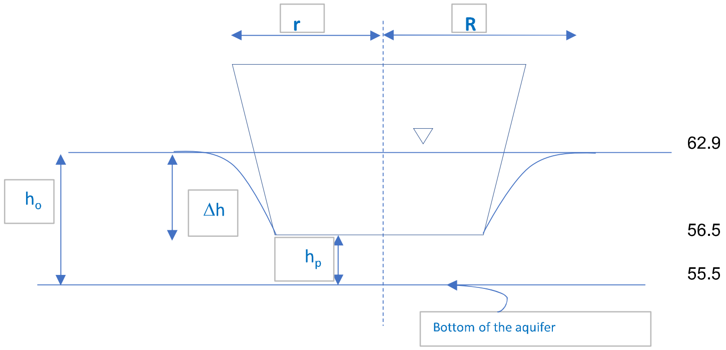
Inflow to Excavation

Dupuit-Forchheimer Equation: $Q=\pi K((h_o^2-hp^2)/\ln(R/r))$

1047 Richmond Road

K (m/sec)	1E-06	
h_0 (m)	7.4	**r - equivalent radius of pit**
h_p (m)	1.0	**R - radius of influence**
r (m)	**52.8**	**SF - safety factor**
SF	**2**	

	SF * Q (m^3/s)	R	Rad of Inf. from edge	m^3/day	L/day
	3.8E-03	58.8	6.0	331.12	331,121
	2.2E-03	63.8	11.0	188.25	188,249
	1.6E-03	68.8	16.0	134.57	134,568
	1.2E-03	73.8	21.0	106.37	106,366
Steady-State**	1.1E-03	77.8	25.0	91.88	91,879
	9.2E-04	82.8	30.0	79.16	79,156
	8.1E-04	87.8	35.0	70.03	70,028
	7.3E-04	92.8	40.0	63.15	63,149
	6.7E-04	97.8	45.0	57.77	57,772
	6.2E-04	102.8	50.0	53.45	53,448
	5.4E-04	112.8	60.0	46.91	46,911
	4.9E-04	122.8	70.0	42.19	42,190
	4.5E-04	132.8	80.0	38.61	38,608
	4.1E-04	142.8	90.0	35.79	35,791
	3.9E-04	152.8	100.0	33.51	33,511
	3.1E-04	202.8	150.0	26.46	26,461

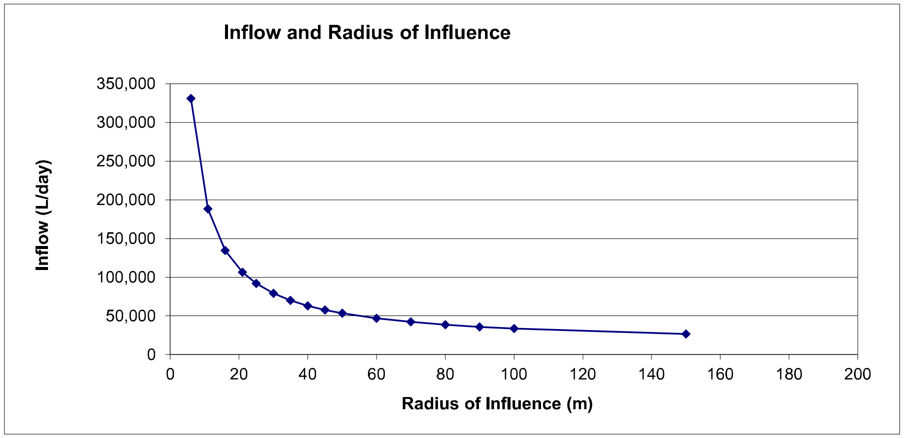


Excavation Assumptions

Width (m)	73	
Length (m)	120	
Ground surface elevation (masl)	65.50	
Groundwater elevation (masl)	62.89	*0.5 m higher than measured at 21-4*
Bottom of basement (masl)	56.5	*assumed 3 basement levels totalling 9 m depth*
Dewatered level (masl)	56.5	*same as the depth of excavation*
Base of aquifer (masl)	55.5	*assumed 1 m below dewatered level*

Sichart and Kyrieleis Equation: $R=3000\Delta h(K^{1/2})$

Radius of Influence of Excavation (m) 21



Notes

L - litres

m - metres

mbgs - metres below ground surface

Initial*: Potential worst-case inflow rate when trench is initially rapidly dewatered

Steady-State**: Steady state inflow rate

Prepared by: CAMC
Checked by: BH

HVORSLEV SLUG TEST ANALYSIS
RISING HEAD TEST 21-2

INTERVAL (metres below ground surface)

Top of Interval = 3.96
Bottom of Interval = 7.01

$$K = \frac{r_c^2}{2L_e} ln\left[\frac{L_e}{2R_e} + \sqrt{1+\left(\frac{L_e}{2R_e}\right)^2}\right]\left[\frac{ln\left(\frac{h_1}{h_2}\right)}{(t_2 - t_1)}\right]$$

where K = (m/sec)

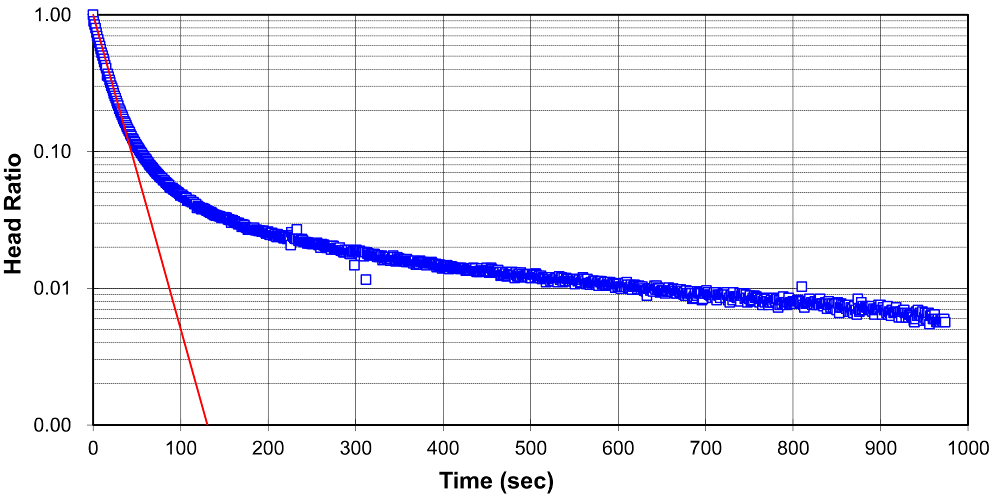
where:

r_c = casing radius (metres)
R_e = filter pack radius (metres)
L_e = length of screened interval (metres)
t = time (seconds)
h_t = head at time t (metres)

INPUT PARAMETERS	
r_c =	0.025
R_e =	0.05
L_e =	3.1
t_1 =	0
t_2 =	29
h_1/h_0 =	1.00
h_2/h_0 =	0.22

RESULTS

K=	2E-05	m/sec
K=	2E-03	cm/sec



Project Name: Fengate/Phase 1, 2 and RSC/Ottawa
Project No.: 21494078
Test Date: 2021-10-05

Analysis By: SPS
Checked By: BH
Analysis Date: 2021-10-06

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HVORSLEV SLUG TEST ANALYSIS
RISING HEAD TEST 21-3

INTERVAL (metres below ground surface)

Top of Interval = 4.57
Bottom of Interval = 7.62

$$K = \frac{r_c^2}{2L_e} ln\left[\frac{L_e}{2R_e} + \sqrt{1 + \left(\frac{L_e}{2R_e}\right)^2}\right]\left[\frac{ln\left(\frac{h_1}{h_2}\right)}{(t_2 - t_1)}\right]$$

where K = (m/sec)

where:

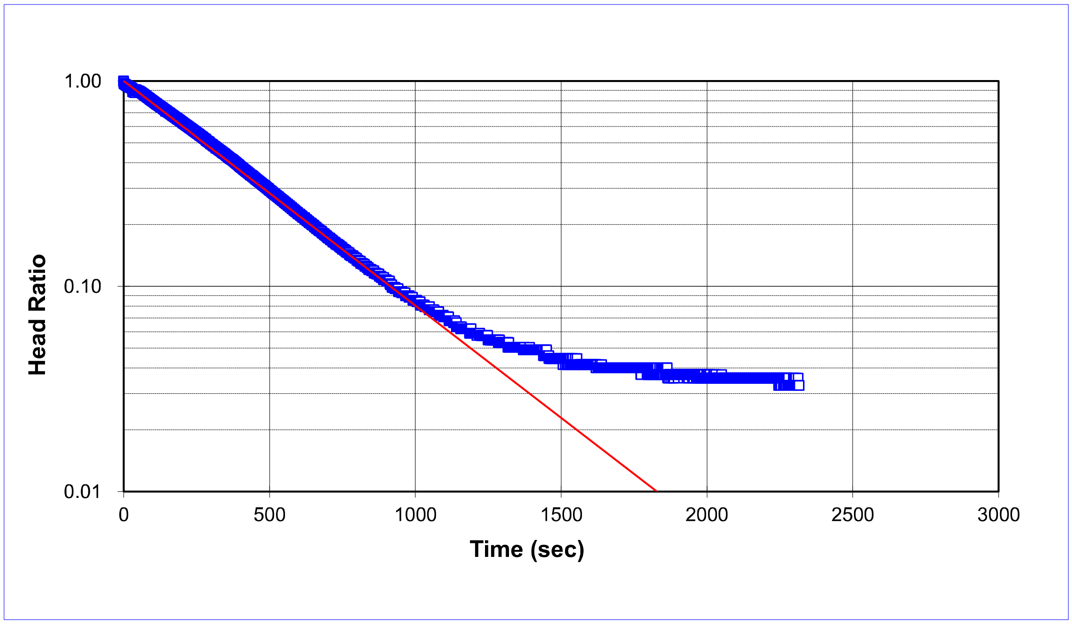
r_c = casing radius (metres)
R_e = filter pack radius (metres)
L_e = length of screened interval (metres)
t = time (seconds)
h_t = head at time t (metres)

INPUT PARAMETERS

r_c =	0.025
R_e =	0.05
L_e =	3.1
t_1 =	0
t_2 =	775
h_1/h_0 =	1.00
h_2/h_0 =	0.14

RESULTS

K=	1E-06	m/sec
K=	1E-04	cm/sec



Project Name: Fengate/Phase 1, 2 and RSC/Ottawa
Project No.: 21494078
Test Date: 2021-10-05

Analysis By: SPS
Checked By: BH
Analysis Date: 2021-10-06

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HVORSLEV SLUG TEST ANALYSIS
RISING HEAD TEST 21-4

INTERVAL (metres below ground surface)

Top of Interval = 4.57
Bottom of Interval = 7.62

$$K = \frac{r_c^2}{2L_e} ln\left[\frac{L_e}{2R_e} + \sqrt{1 + \left(\frac{L_e}{2R_e}\right)^2}\right]\left[\frac{ln\left(\frac{h_1}{h_2}\right)}{(t_2 - t_1)}\right]$$

where K = (m/sec)

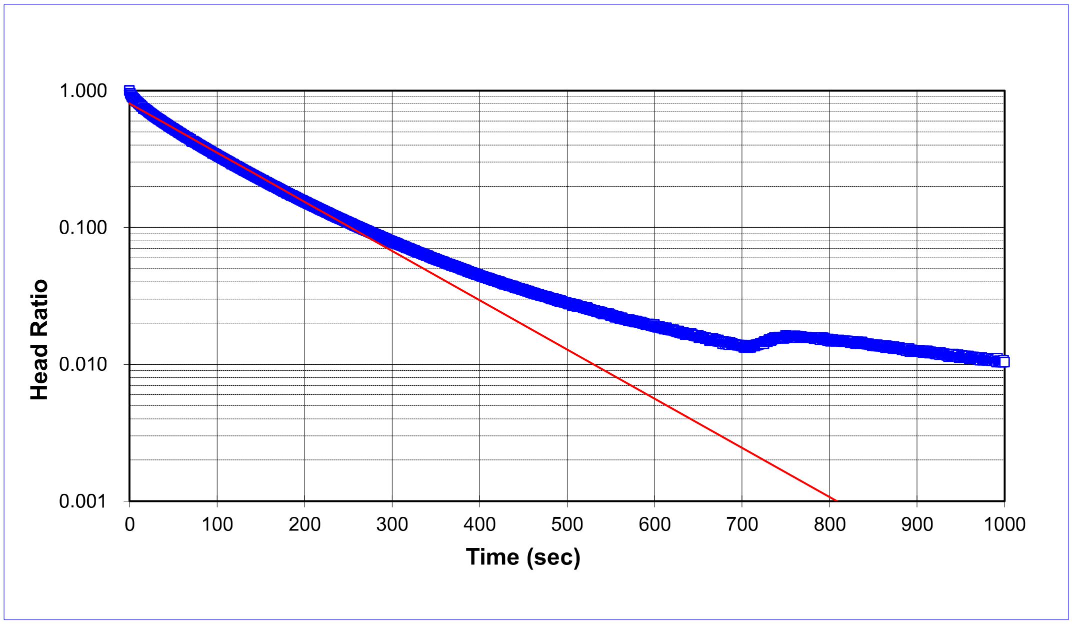
where:
r_c = casing radius (metres)
R_e = filter pack radius (metres)
L_e = length of screened interval (metres)
t = time (seconds)
h_t = head at time t (metres)

INPUT PARAMETERS

r_c =	0.025
R_e =	0.05
L_e =	3.1
t_1 =	41
t_2 =	199
h_1/h_0 =	0.57
h_2/h_0 =	0.16

RESULTS

K=	4E-06	m/sec
K=	4E-04	cm/sec



Project Name: Fengate/Phase 1, 2 and RSC/Ottawa
Project No.: 21494078
Test Date: 2021-10-05

Analysis By: SPS
Checked By: BH
Analysis Date: 2021-10-06

Golder Associates Ltd.

HVORSLEV SLUG TEST ANALYSIS
FALLING HEAD TEST 21-5

INTERVAL (metres below ground surface)

Top of Interval = 4.57
Bottom of Interval = 7.62

$$K = \frac{r_c^2}{2L_e} ln\left[\frac{L_e}{2R_e} + \sqrt{1 + \left(\frac{L_e}{2R_e}\right)^2}\right]\left[\frac{ln\left(\frac{h_1}{h_2}\right)}{(t_2 - t_1)}\right]$$

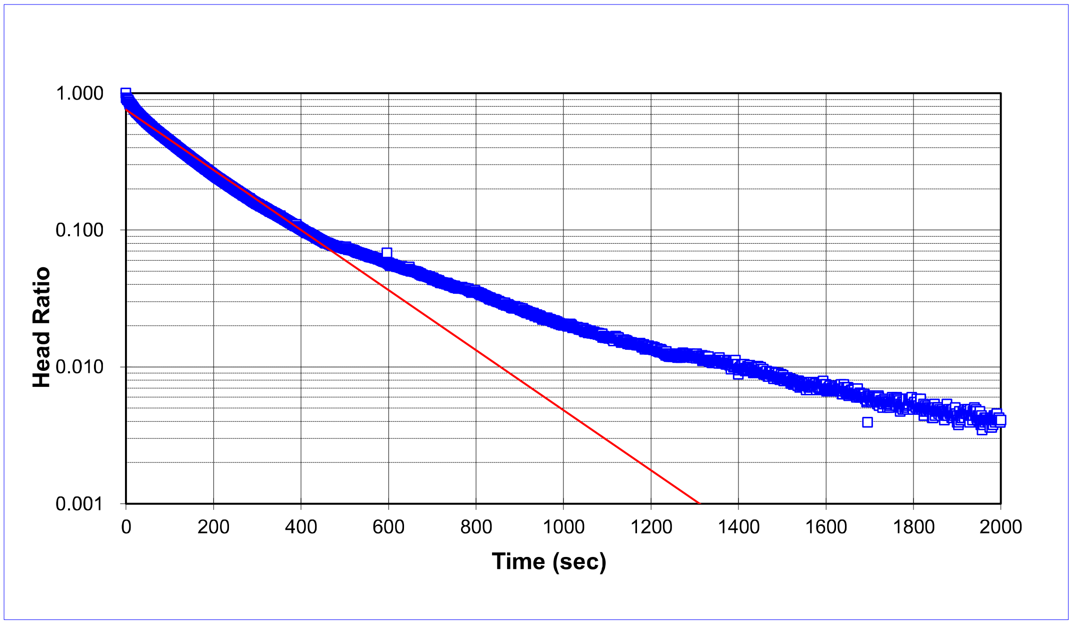
where K = (m/sec)

where:
r_c = casing radius (metres)
R_e = filter pack radius (metres)
L_e = length of screened interval (metres)
t = time (seconds)
h_t = head at time t (metres)

INPUT PARAMETERS	
r_c =	**0.025**
R_e =	**0.05**
L_e =	**3.1**
t_1 =	**69**
t_2 =	**376**
h_1/h_0 =	**0.53**
h_2/h_0 =	**0.11**

RESULTS

K=	**2E-06**	**m/sec**
K=	**2E-04**	**cm/sec**



Project Name: Fengate/Phase 1, 2 and RSC/Ottawa
Project No.: 21494078
Test Date: 2021-10-05

Analysis By: SPS
Checked By: BH
Analysis Date: 2021-10-06

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BOUWER AND RICE SLUG TEST ANALYSIS
RISING HEAD TEST 21-6

INTERVAL (metres below ground surface)

Top of Interval = 6.33
Bottom of Interval = 9.38

$$K = \frac{r_c^2 \ln\left(\frac{R_e}{r_w}\right)}{2L_e}\frac{1}{t}\ln\frac{y_0}{y_t}$$ where K=m/sec

where:

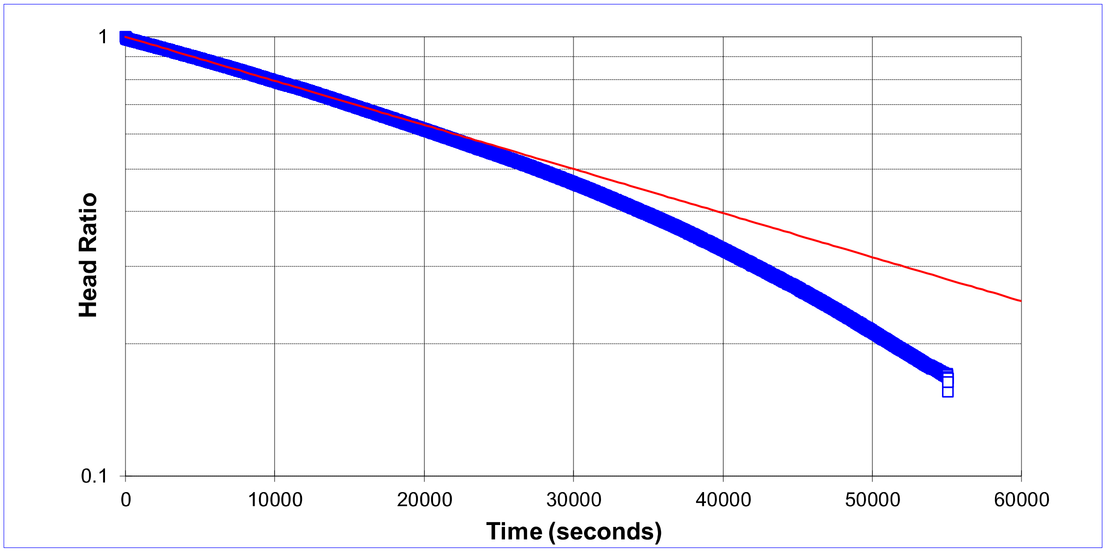
r_c = casing radius (metres);
R_e = effective radius (metres);
L_e = length of screened interval (metres);
r_w = radial distance to undisturbed aquifer (metres)
y_0 = initial drawdown (metres)
y_t = drawdown (metres) at time t (seconds)

INPUT PARAMETERS

r_c = 0.03
r_w = 0.05
L_e = 2.54
$\ln(R_e/r_w)$ 2.69
y_0 = 1.00
y_t = 0.63
t = 20000

RESULTS

K=	**1E-08**	**m/sec**
K=	**1E-06**	**cm/sec**



Project Name: Fengate/Phase 1, 2 and RSC/Ottawa
Project No.: 21494078
Test Date: 05-Oct-21
Analysis By: SPS
Checked By: BH
Analysis Date: 06-Oct-21

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https://golderassociates.sharepoint.com/sites/152441/Project Files/5 Technical Work/Hydrogeology/K-tests/
BH21-06_RHT_BH.xlsx

HVORSLEV SLUG TEST ANALYSIS
RISING HEAD TEST 21-10

INTERVAL (metres below ground surface)

Top of Interval = 12.40
Bottom of Interval = 15.45

$$K = \frac{r_c^2}{2L_e} ln\left[\frac{L_e}{2R_e} + \sqrt{1 + \left(\frac{L_e}{2R_e}\right)^2}\right]\left[\frac{ln\left(\frac{h_1}{h_2}\right)}{(t_2 - t_1)}\right]$$

where K = (m/sec)

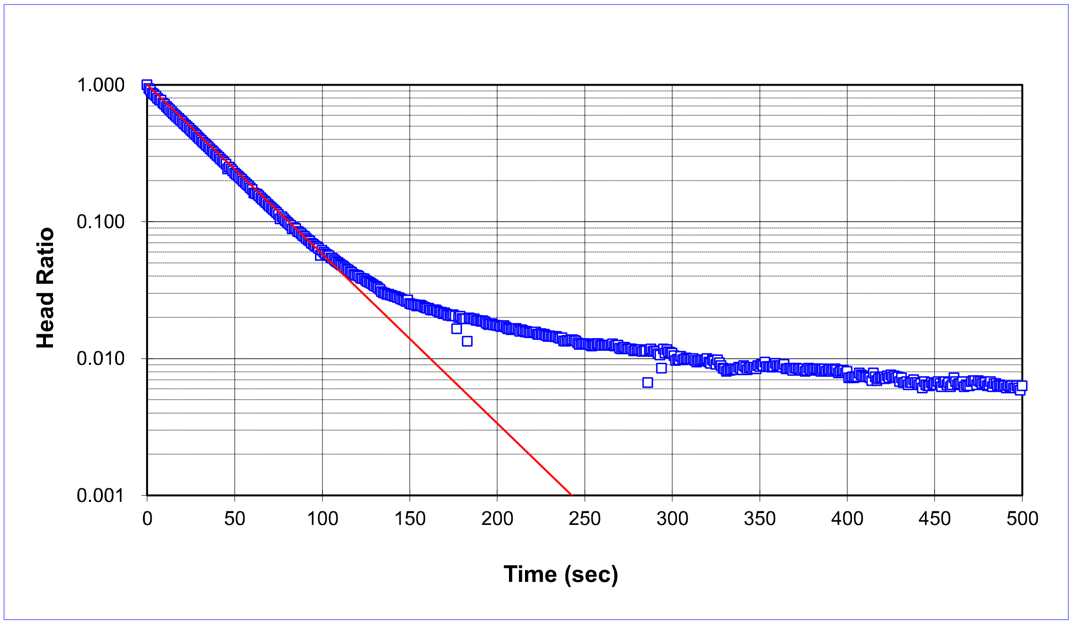
where:

r_c = casing radius (metres)
R_e = filter pack radius (metres)
L_e = length of screened interval (metres)
t = time (seconds)
h_t = head at time t (metres)

INPUT PARAMETERS	
r_c =	**0.025**
R_e =	**0.05**
L_e =	**3.1**
t_1 =	**0**
t_2 =	**87**
h_1/h_0 =	**1.00**
h_2/h_0 =	**0.08**

RESULTS

K=	**1E-05**	**m/sec**
K=	**1E-03**	**cm/sec**



Project Name: Fengate
Project No.: 21494078
Test Date: 2021-10-05

Analysis By: SPS
Checked By: BH
Analysis Date: 2021-10-06

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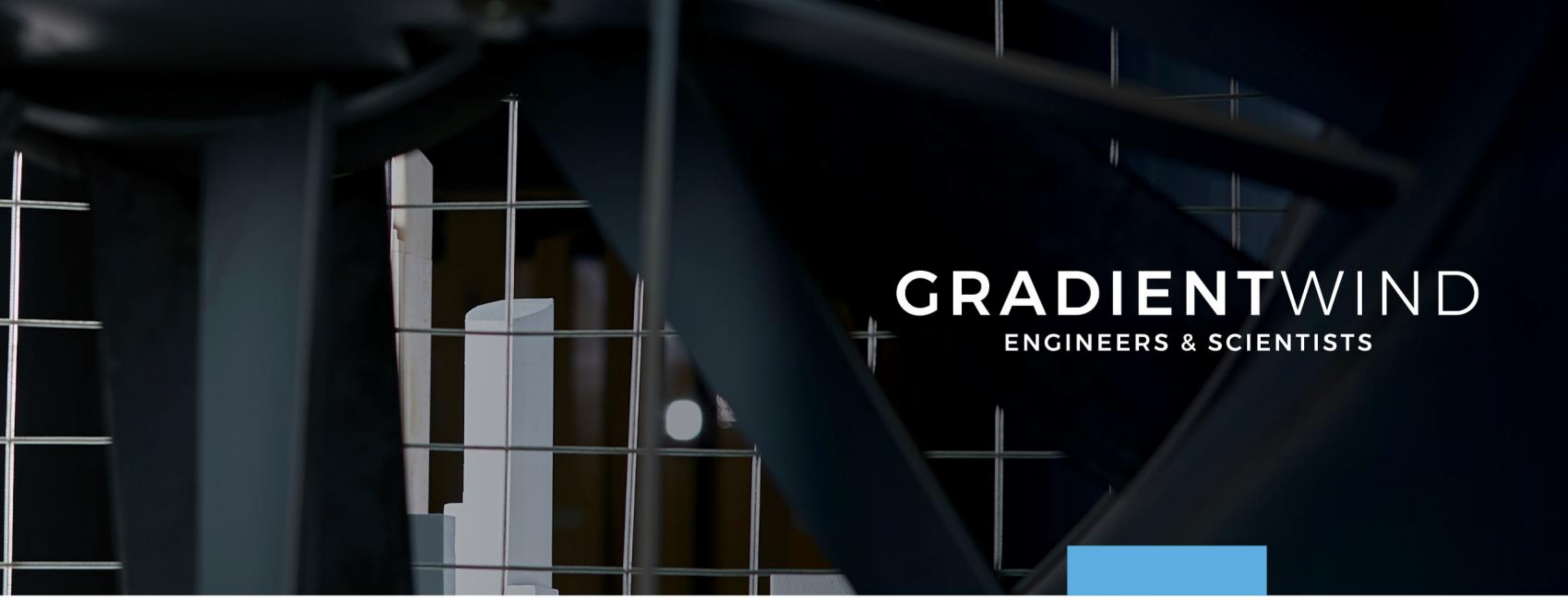
APPENDIX C

Traffic Noise and Vibration Feasibility Study:

Prepared by Gradient Wind Engineers and

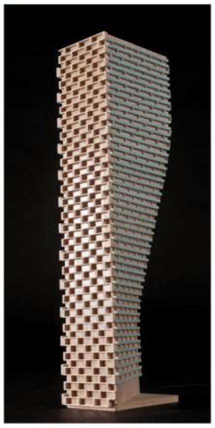
Scientists Report No. 21-416 Addendum dated August 12, 2024

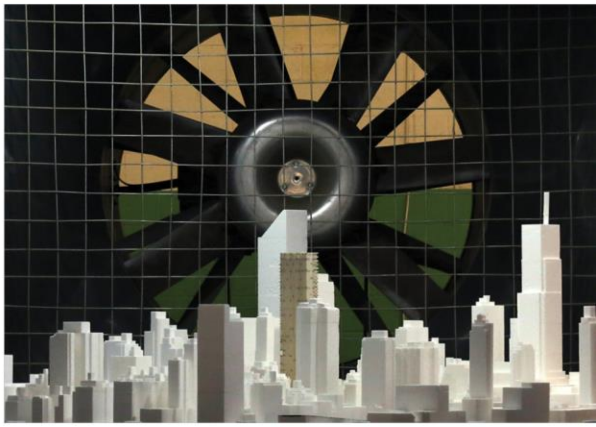


TRANSPORTATION NOISE AND VIBRATION ASSESSMENT

1047 Richmond Road
Ottawa, Ontario

Report: 21-416- Transportation Noise and Vibration





August 12, 2024

PREPARED FOR
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EXECUTIVE SUMMARY

This report describes a transportation noise and vibration assessment undertaken in support of a Site Plan Control (SPC) application for the proposed residential development located at 1047 Richmond Road in Ottawa, Ontario. The proposed development comprises two towers rising from two four-storey podia. The primary source of roadway traffic noise is Richmond Road to the south. As the site is in proximity to the future proposed Ottawa-Carleton Regional Transit Commission (OC Transpo) Light Rail Transit (LRT) Confederation Line, a ground vibration impact assessment from the proposed underground LRT system on the development was conducted following the procedures outlined in the Federal Transit Authorities (FTA) protocol. Figure 1 illustrates a complete site plan with the surrounding context.

The assessment is based on (i) theoretical noise prediction methods that conform to the Ministry of the Environment, Conservation and Parks (MECP) NPC-300, Ministry of Transportation Ontario (MTO), and City of Ottawa Environmental Noise Control Guidelines (ENCG) guidelines; (ii) future vehicular traffic volumes corresponding to roadway classification, roadway traffic volumes obtained from the City of Ottawa, and LRT information from the Rail Implementation Office; (iii) architectural drawings provided by Roderick Lahey Architect Inc. in August 2024; and (iv) ground-borne vibration criteria as specified by the Federal Transit Authority (FTA) Protocol.

The results of the current analysis indicate that noise levels will range between 48 and 60 dBA during the daytime period (07:00-23:00) and between 41 and 53 dBA during the nighttime period (23:00-07:00). The highest noise level (60 dBA) occurs at the south façade of Tower A, which is nearest and most exposed to Richmond Road. Figures 4 and 5 illustrate daytime and nighttime noise contours of the site 4.5 m above grade.

The results indicate that upgraded building components and central air conditioning will not be required for Tower A as noise levels predicted due to roadway traffic do not exceed the criteria of 65 dBA during the daytime listed in ENCG. However, noise levels fall between 55 dBA and 65 dBA during the daytime period. As such, Tower A will need forced air heating with provisions for central air conditioning, as a minimum requirement. These requirements will allow occupants to keep windows closed and maintain a comfortable living environment. A Type C Warning Clause will also be required in all Lease, Purchase and Sale Agreements, as summarized in Section 6.

The results also indicate that noise levels at the at-grade amenity area and the Level 4 amenity terraces are expected to be between 51 dBA and 56 dBA. As noise levels at the Level 4 outdoor amenity are slightly above 55 dBA, acoustic mitigation in the form of a noise screen is recommended but not required. If no mitigation is provided, a Type A Warning Clause will also be required in all Lease, Purchase and Sale Agreements, as summarized in Section 6.

Estimated vibration levels at the foundation nearest to the OC Transpo LRT Confederation Line are expected to be 0.044 mm/s RMS (65 dBV), based on the FTA protocol and an offset distance of 32 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 vibration levels are acceptable, correspondingly, regenerated noise levels are also expected to be acceptable.

With regard to stationary noise impacts from proposed mechanical systems on the building, they will be designed to ensure compliance with the ENCG sound level limits. Noise impacts can generally be minimized by judicious selection and placement of the equipment. Where necessary, noise screens and silencers can be placed into the design. It is recommended a stationary noise study be conducted once mechanical plans for the proposed building become available. This study would assess the impacts of stationary noise from rooftop mechanical units serving the proposed building on surrounding noise-sensitive areas.

TABLE OF CONTENTS

1. INTRODUCTION

Gradient Wind Engineering Inc. (Gradient Wind) was retained by Fengate Asset Management to undertake a transportation noise and vibration assessment, in support of a Site Plan Control (SPC) application for the proposed residential development located at 1047 Richmond Road in Ottawa, Ontario. This report summarizes the methodology, results, and recommendations related to the assessment of exterior noise and vibration levels generated by local transportation traffic.

This assessment is based on theoretical noise calculation methods conforming to the Ministry of the Environment, Conservation and Parks (MECP) NPC-300[1], Ministry of Transportation Ontario (MTO)[2], and City of Ottawa Environmental Noise Control Guidelines (ENCG)[3] guidelines. Noise calculations were based on architectural drawings provided by Roderick Lahey Architect Inc. in August 2024, with future traffic volumes corresponding to roadway classification and theoretical roadway capacities, and recent satellite imagery.

2. TERMS OF REFERENCE

The focus of this transportation noise assessment is "Tower A" of the proposed residential development located at 1047 Richmond Road in Ottawa, Ontario. The subject site is located on a nearly rectangular parcel of land north of the intersection of New Orchard Avenue North and Richmond Road.

The proposed development comprises two towers rising from two four-storey podia. The two towers are identified as "Tower A" (36 storeys) and "Tower B" (38 storeys) which are situated in the southwest corner and northeast corner of the subject site, respectively. A park is provided at the southwest corner of the subject site. Tower A and Tower B are topped with a mechanical penthouse and both buildings share two below-grade parking levels which are accessed by a parking ramp located to the north of Tower A via a loading/service laneway extending along the north elevation of the subject site from New Orchard Avenue North. A central drop-off courtyard is accessed from the noted laneway.

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

[2] Ministry of Transportation Ontario, "*Environmental Guide for Noise*", August 2021

[3] City of Ottawa Environmental Noise Control Guidelines, January 2016

Above the two levels of underground parking, Level 1 of Tower A includes retail space fronting a proposed park at the southwest corner of the site, a residential lobby along the east elevation, and a loading area and garbage room at the northwest elevation, with residential units and shared building support spaces throughout the remainder of the level. An at-grade outdoor amenity area is located east of Tower A. Level 2 of Tower A includes storage lockers to the northeast and residential units throughout the remainder of the level. At Level 3, the podium steps back towards Tower A in the east and north directions to incorporate private terraces. At Level 4, the podium steps back towards Tower A in the east direction to incorporate an outdoor amenity area. The remainder of Level 4 comprises of indoor amenity space. Tower A rises from the podium with a rectangular planform. All floors serving Towers A between Level 5 and Level 38 comprise residential units. Level 39 includes an indoor amenity space to the northeast with residential units throughout the remainder of the level.

The site is surrounded by Sir John A. Macdonald Parkway and the Trans-Canada Trail northeast, high-rise residential buildings to the northeast and to the southwest, and mostly low-rise residential buildings for the remaining compass directions. Additionally, the Ottawa-Carleton Regional Transit Commission (OC Transpo) Light Rail Transit (LRT) Confederation Line extension and the future New Orchard Station are currently under construction approximately 20 m to the south of the subject site. The primary source of roadway traffic noise is Richmond Road to the south. Figure 1 illustrates a complete site plan with the surrounding context.

The primary source of ground-borne vibration is the future OC Transpo LRT line located to the south of the subject site. As per the City of Ottawa’s Official Plan, the LRT system is situated within 75 m from the nearest property line. As a result, a ground vibration impact assessment from the underground LRT system on the proposed development was conducted following the procedures outlined in the Federal Transit Authorities (FTA) protocol. Airborne noise transmission from the LRT onto the development was considered to be negligible compared to surface transportation noise as the LRT is located entirely underground.

With regard to stationary noise impacts from proposed mechanical systems on the building, they will be designed to ensure compliance with the ENCG sound level limits. Noise impacts can generally be minimized by judicious selection and placement of the equipment. Where necessary, noise screens and silencers can be placed into the design. It is recommended a stationary noise study be conducted once mechanical plans for the proposed building become available. This study would assess the impacts of stationary noise from rooftop mechanical units serving the proposed building on surrounding noise-sensitive areas.

3. OBJECTIVES

The principal objectives of this study are to (i) calculate the future noise levels on the study building produced by local transportation sources, (ii) predict vibration levels on the study building produced from the LRT system, and (iii) explore potential noise mitigation where required.

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 Traffic Noise

4.2.1 Criteria for Roadway Traffic Noise

For surface roadway 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, 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. NPC-300 specifies that the recommended indoor noise limit range (that is relevant to this study) is 50, 45 and 40 dBA for retail/office/indoor amenity space, living rooms, and sleeping quarters, respectively, as listed in Table 1.

TABLE 1: INDOOR SOUND LEVEL CRITERIA (ROAD) [4]

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 reduction[5]. A closed window due to a ventilation requirement will bring noise levels down to achieve an acceptable indoor environment[6]. 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 attenuation[7].

The sound level criterion for outdoor living areas is 55 dBA, which applies during the daytime (07:00 to 23:00). When noise levels exceed 55 dBA, mitigation should be provided to reduce noise levels where technically and administratively feasible to acceptable levels at or below the criterion.

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[4] MOECP, Environmental Noise Guidelines, NPC 300 – Part C, Table C-9
[5] Burberry, P.B. (2014). Mitchell's Environment and Services. Routledge, Page 125
[6] MOECP, Environmental Noise Guidelines, NPC 300 – Part C, Section 7.8
[7] MOECP, Environmental Noise Guidelines, NPC 300 – Part C, Section 7.1.3

4.2.2 Roadway 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 Plan[8] 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. Table 2 (below) summarizes the AADT values used for each roadway included in this assessment.

TABLE 2: ROADWAY TRAFFIC DATA

Segment	Roadway Traffic Data	Speed Limit (km/h)	Traffic Volumes
Richmond Road	2-Lane Urban Arterial Undivided (2-UAU)	50	**15,000**

4.2.3 Theoretical Roadway 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 *Predictor-Lima* which utilizes the United States Federal Highway Administration's Traffic Noise Model (TNM) to represent the roadway line sources. The TNM model is also being accepted in the updated Environmental Guide for Noise of Ontario, 2021 by the Ministry of Transportation (MTO)[9]. This computer program can represent three-dimensional surfaces and first reflections of sound waves over a suitable spectrum for human hearing. A set of comparative calculations were performed in the current Ontario traffic noise prediction model STAMSON for comparisons to Predictor simulation results. The STAMSON model is, however, older and requires each receptor to be calculated separately. STAMSON also does not accurately account for building reflections and multiple screening elements, and curved road geometry. A total of 6 receptor locations were identified around the site, as illustrated in Figure 2.

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[8] City of Ottawa Transportation Master Plan, November 2013

[9] Ministry of Transportation Ontario, "*Environmental Guide for Noise*", August 2021, pg. 16

Roadway noise calculations were performed by treating each segment as separate line sources of noise, and by using existing and proposed 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 was taken to be 92% / 8%, respectively.
- Default ground surfaces were taken to be reflective due to the presence of hard (paved) ground.
- Topography was assumed to be a flat/gentle slope surrounding the study building.
- Noise receptors were strategically placed at 6 locations around the study area (see Figure 2).

4.3 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 (μin/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.

4.3.1 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*[10], which indicates 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 building.

4.3.2 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*[11] 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. The vibration impact on the building was 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 maximum operating speed of the LRT line is 43 mph (70 km/h) at peak.
- The setback distance between the development and the closest track is 32 m.

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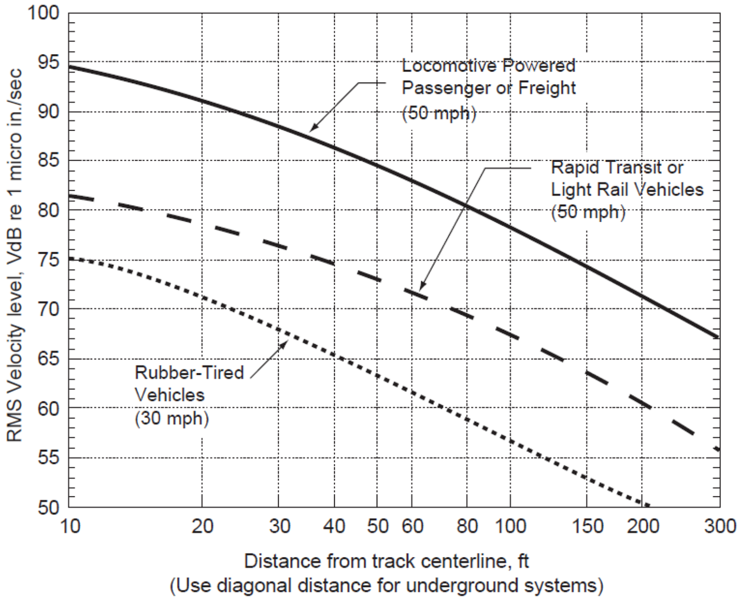
[10] Dialog and J.E. Coulter Associates Limited, prepared for The Federation of Canadian Municipalities and The Railway Association of Canada, May 2013

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

- 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.
- The building's foundation will bear on bedrock.
- Type of transit structure is Station.



**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 Traffic Noise Levels

The results of the transportation noise calculations are summarized in Table 3 below.

TABLE 3: EXTERIOR NOISE LEVELS DUE TO ROADWAY TRAFFIC SOURCES

Receptor Number	Receptor Height Above Grade/Roof (m)	Receptor Location	Roadway Noise Level (dBA)	
			Day	Night
R1	109.5	POW - Level 39 Tower A - South Façade	60	53
R2	109.5	POW - Level 39 Tower A - East Façade	58	51
R3	109.5	POW - Level 39 Tower A - North Façade	48	41
R4	109.5	POW - Level 39 Tower A - West Façade	52	45
R5	13.5	OLA - Level 4 Tower A - Outdoor Amenity	56	**N/A***
R6	1.5	OLA - At-Grade Outdoor Amenity	51	**N/A***

*Noise levels during the nighttime are not considered for OLAs

The results of the current analysis indicate that noise levels will range between 48 and 60 dBA during the daytime period (07:00-23:00) and between 41 and 53 dBA during the nighttime period (23:00-07:00). The highest noise level (60 dBA) occurs at the south façade of Tower A, which is nearest and most exposed to Richmond Road. Figures 4 and 5 illustrate daytime and nighttime noise contours of the site 4.5 m above grade.

Table 4 shows a comparison in results between Predictor-Lima and STAMSON. Noise levels calculated in STAMSON were found to have a good correlation with Predictor-Lima and variability between the two programs was within an acceptable level of ±0-3 dBA. STAMSON input parameters are shown in Appendix A.

TABLE 4: RESULTS OF STAMSON/PREDICTOR-LIMA CORRELATION

Receptor ID	Receptor Height (m)	Receptor Location	STAMSON 5.04 Noise Level (dBA)		PREDICTOR-LIMA Noise Level (dBA)	
			Day	Night	Day	Night
R1	109.5	POW - Level 39 Tower A - South Façade	63	55	60	53
R5	13.5	OLA - Level 4 Tower A - Outdoor Amenity	59	**N/A***	56	**N/A***

*Noise levels during the nighttime are not considered for OLAs

5.1.1 Noise Control Measures

The results indicate that upgraded building components and central air conditioning will not be required for Tower A as noise levels predicted due to roadway traffic do not exceed the criteria of 65 dBA during the daytime listed in ENCG. However, noise levels fall between 55 dBA and 65 dBA during the daytime period. As such, Tower A will need forced air heating with provisions for central air conditioning, as a minimum requirement. These requirements will allow occupants to keep windows closed and maintain a comfortable living environment. A Type C Warning Clause will also be required in all Lease, Purchase and Sale Agreements, as summarized in Section 6.

The results also indicate that noise levels at the at-grade amenity area and the Level 4 amenity terraces are expected to be between 51 dBA and 56 dBA. As noise levels at the Level 4 outdoor amenity are slightly above 55 dBA, acoustic mitigation in the form of a noise screen is recommended but not required. If no mitigation is provided, a Type A Warning Clause will also be required in all Lease, Purchase and Sale Agreements, as summarized in Section 6.

5.2 Ground Vibrations and Ground-Borne Noise Levels

Estimated vibration levels at the foundation nearest to the OC Transpo LRT Confederation Line are expected to be 0.044 mm/s RMS (65 dBV), based on the FTA protocol and an offset distance of 32 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 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 48 and 60 dBA during the daytime period (07:00-23:00) and between 41 and 53 dBA during the nighttime period (23:00-07:00). The highest noise level (60 dBA) occurs at the south façade of Tower A, which is nearest and most exposed to Richmond Road. Figures 4 and 5 illustrate daytime and nighttime noise contours of the site 4.5 m above grade.

The results indicate that upgraded building components and central air conditioning will not be required for Tower A as noise levels predicted due to roadway traffic do not exceed the criteria of 65 dBA during the daytime listed in ENCG. However, noise levels fall between 55 dBA and 65 dBA during the daytime period. As such, Tower A will need forced air heating with provisions for central air conditioning, as a minimum requirement. These requirements will allow occupants to keep windows closed and maintain a comfortable living environment. A Type C Warning Clause will also be required in all Lease, Purchase and Sale Agreements, as summarized below.

Type C:

> *"This dwelling unit has been designed with the provision for adding central air conditioning at the occupant's discretion. Installation of central air conditioning by the occupant in low and medium density developments 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."*

The results also indicate that noise levels at the at-grade amenity area and the Level 4 amenity terraces are expected to be between 51 dBA and 56 dBA. As noise levels at the Level 4 outdoor amenity are slightly above 55 dBA, acoustic mitigation in the form of a noise screen is recommended but not required. If no mitigation is provided, a Type A Warning Clause will also be required in all Lease, Purchase and Sale Agreements, as summarized below.

Type A:

> *"Purchasers/tenants are advised that sound levels due to increasing road traffic (rail traffic) (air traffic) may occasionally interfere with some activities of the dwelling occupants as the sound levels exceed the sound level limits of the Municipality and the Ministry of the Environment."*

As the development is adjacent to a future proposed LRT line and station, 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.'"*

Estimated vibration levels at the foundation nearest to the OC Transpo LRT Confederation Line are expected to be 0.044 mm/s RMS (65 dBV), based on the FTA protocol and an offset distance of 32 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 vibration levels are acceptable, correspondingly, regenerated noise levels are also expected to be acceptable.

With regard to stationary noise impacts from proposed mechanical systems on the building, they will be designed to ensure compliance with the ENCG sound level limits. Noise impacts can generally be minimized by judicious selection and placement of the equipment. Where necessary, noise screens and silencers can be placed into the design. It is recommended a stationary noise study be conducted once mechanical plans for the proposed building become available. This study would assess the impacts of stationary noise from rooftop mechanical units serving the proposed building on surrounding noise-sensitive areas.

This concludes our transportation noise and vibration 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,

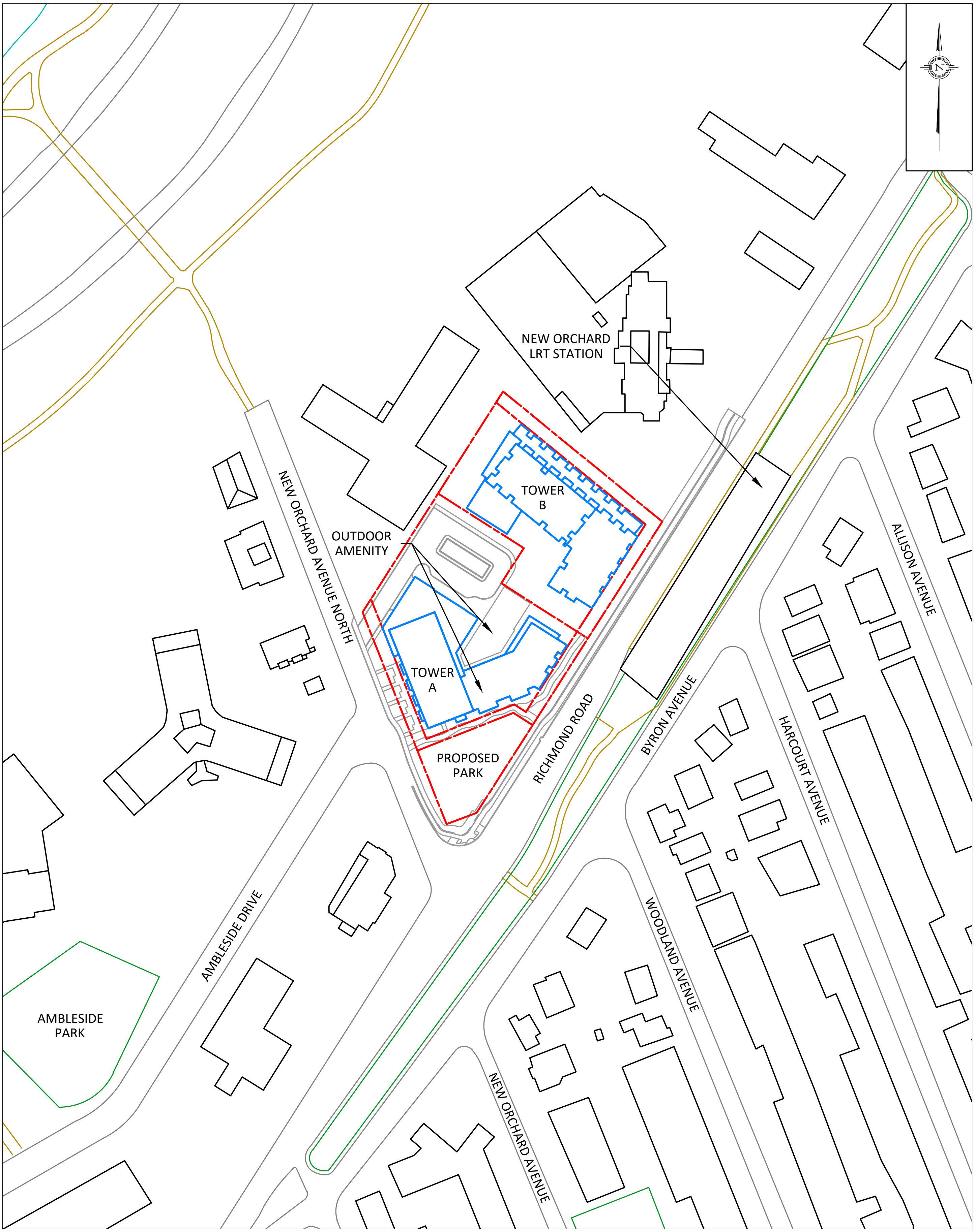
Gradient Wind Engineering Inc.

LICENSED PROFESSIONAL ENGINEER
J. R. FOSTER
100155655
Aug 12, 2024
PROVINCE OF ONTARIO

Benjamin Page, AdvDip.
Junior Environmental Scientist

Joshua Foster, P.Eng.
Lead Engineer

Gradient Wind File 21-416- Transportation and Vibration



GRADIENTWIND
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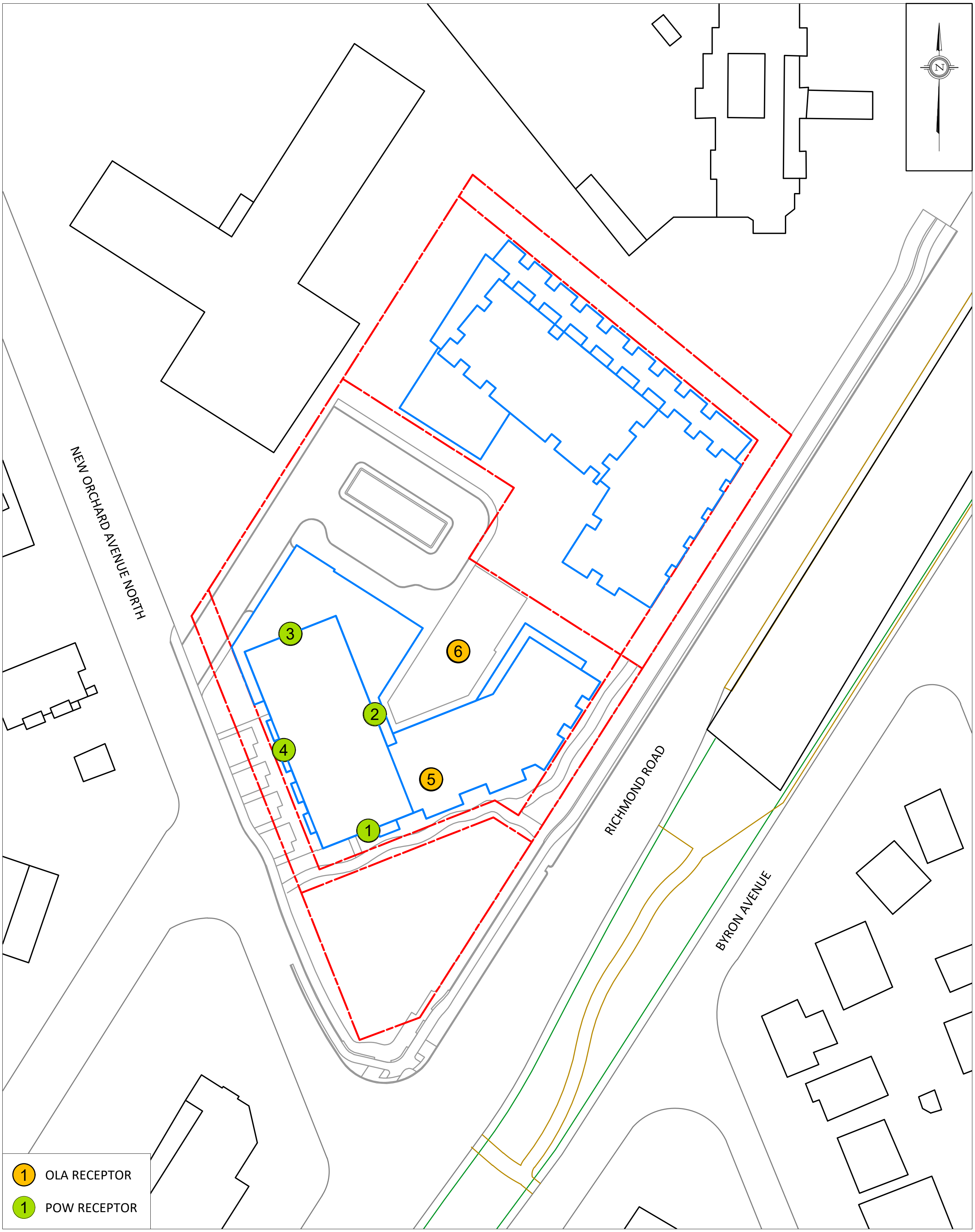
PROJECT	1047 RICHMOND ROAD, OTTAWA TRANSPORTATION NOISE AND VIBRATION ASSESSMENT		
SCALE	1:2000	DRAWING NO.	21-416-1
DATE	AUGUST 12, 2024	DRAWN BY	B.P.

DESCRIPTION

FIGURE 1:
PROPOSED SITE PLAN AND SURROUNDING CONTEXT



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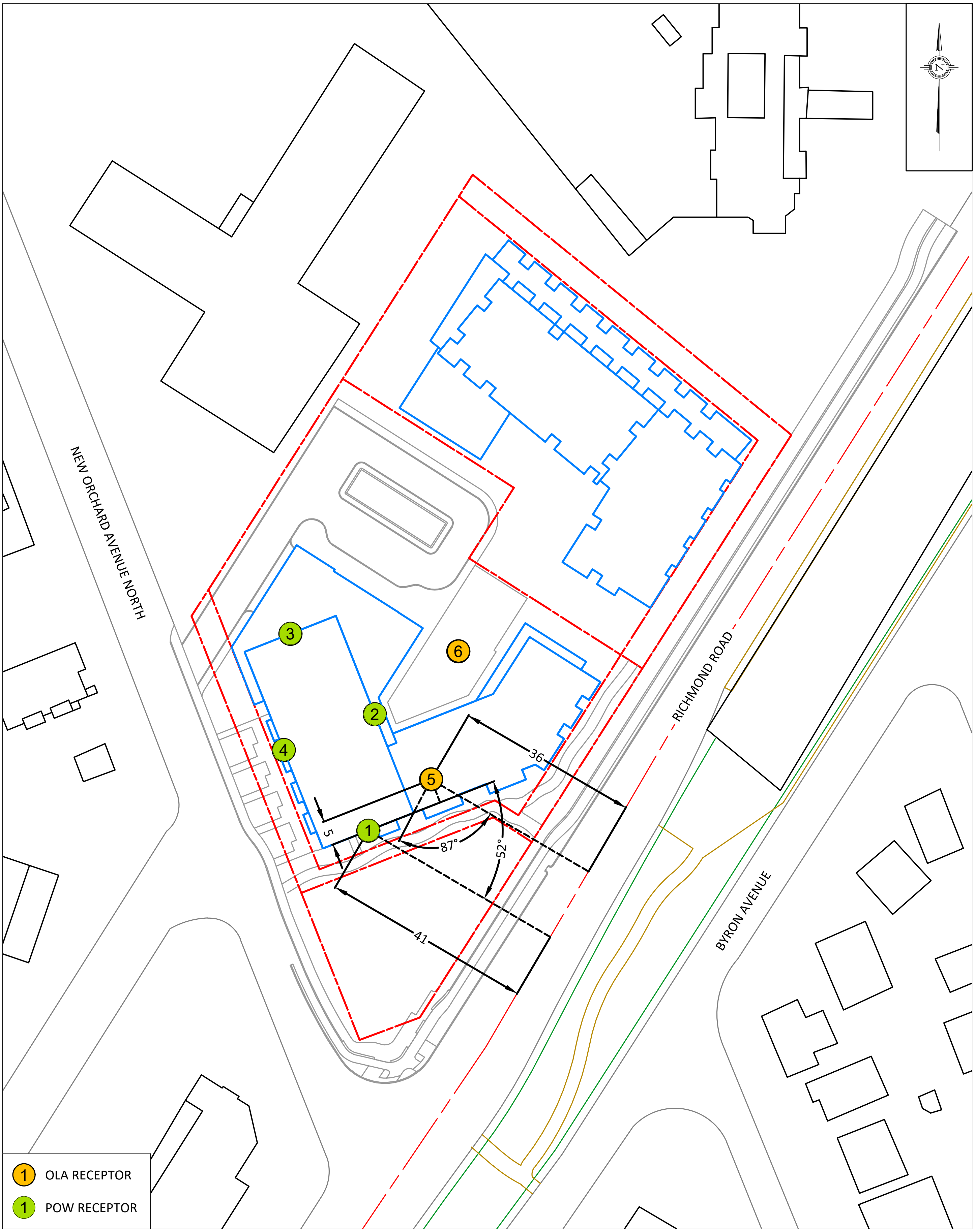
PROJECT	1047 RICHMOND ROAD, OTTAWA TRANSPORTATION NOISE AND VIBRATION ASSESSMENT		
SCALE	1:1000	DRAWING NO.	21-416-2
DATE	AUGUST 12, 2024	DRAWN BY	B.P.

DESCRIPTION

FIGURE 2:
RECEPTOR LOCATIONS



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PROJECT	1047 RICHMOND ROAD, OTTAWA TRANSPORTATION NOISE AND VIBRATION ASSESSMENT
SCALE	1:1000
DRAWING NO.	21-416-3
DATE	AUGUST 12, 2024
DRAWN BY	B.P.
DESCRIPTION	FIGURE 3: STAMSON 5.04 INPUT DATA

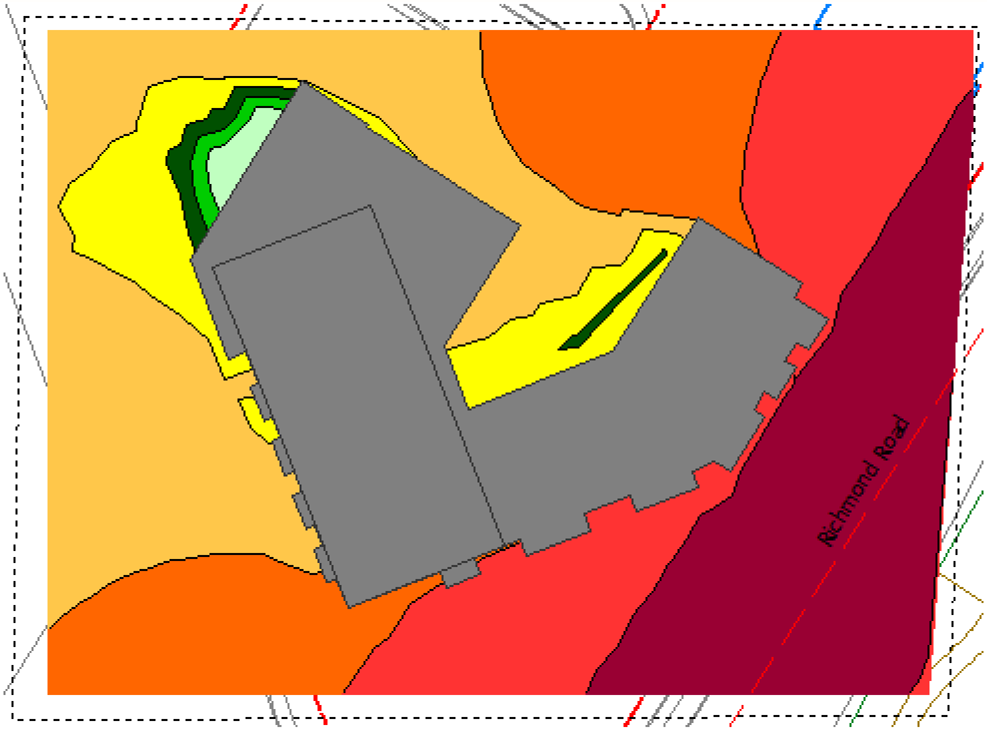


FIGURE 4: DAYTIME TRANSPORTATION NOISE CONTOURS
(4.5 M ABOVE GRADE)

🟦	*80 – 85 dB*
🟦	*75 – 80 dB*
🟪	*70 – 75 dB*
🟥	*65 – 70 dB*
🟥	*60 – 65 dB*
🟧	*55 – 60 dB*
🟧	*50 – 55 dB*
🟨	*45 – 50 dB*
🟩	*40 – 45 dB*
🟩	*35 – 40 dB*
🟩	*0 – 35 dB*

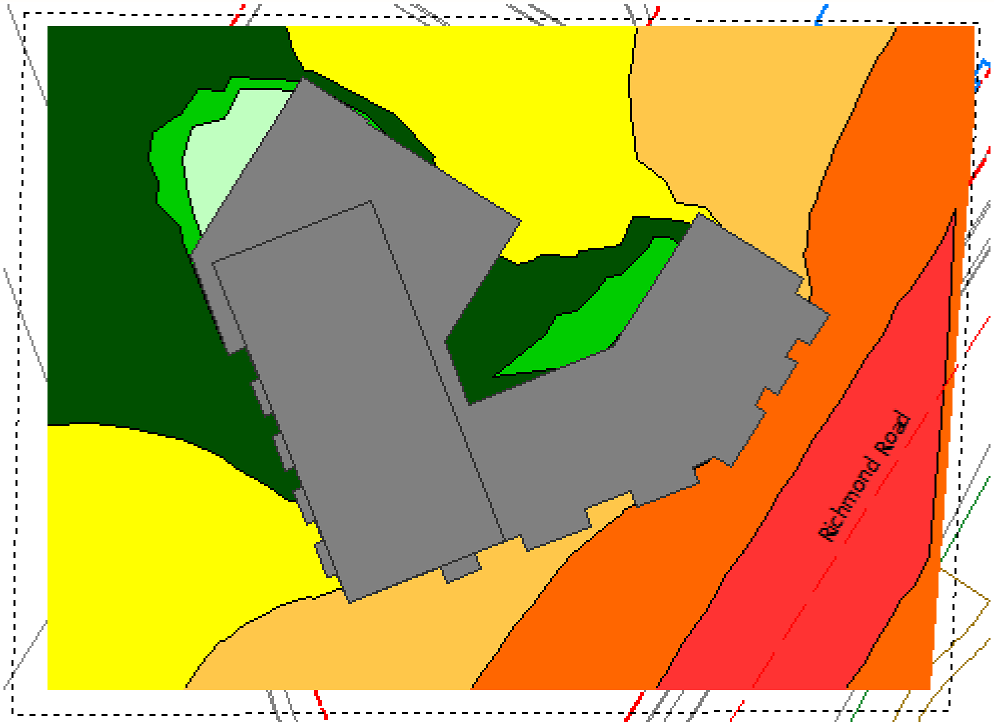


FIGURE 5: NIGHTTIME TRANSPORTATION NOISE CONTOURS (4.5 M ABOVE GRADE)

🟦 *80 – 85 dB*
🟦 *75 – 80 dB*
🟪 *70 – 75 dB*
🟥 *65 – 70 dB*
🟥 *60 – 65 dB*
🟧 *55 – 60 dB*
🟧 *50 – 55 dB*
🟨 *45 – 50 dB*
🟩 *40 – 45 dB*
🟩 *35 – 40 dB*
🟩 *0 – 35 dB*

APPENDIX A

STAMSON SAMPLE CALCULATIONS

```
STAMSON 5.0        NORMAL REPORT        Date: 24-07-2024 12:23:21
MINISTRY OF ENVIRONMENT AND ENERGY / NOISE ASSESSMENT

Filename: R1.te                Time Period: Day/Night 16/8 hours
Description:

Road data, segment # 1: Richmond Rd (day/night)
-----------------------------------------------
Car traffic volume  : 12144/1056  veh/TimePeriod  *
Medium truck volume :   966/84    veh/TimePeriod  *
Heavy truck volume  :   690/60    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):  15000
    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: Richmond Rd (day/night)
---------------------------------------------
Angle1   Angle2           : -52.00 deg   90.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0 / 0
Surface                   :      2       (Reflective ground surface)
Receiver source distance  :  41.00 / 41.00  m
Receiver height           : 118.50 / 118.50 m
Topography                :      1       (Flat/gentle slope; no barrier)
Reference angle           :   0.00

Results segment # 1: Richmond Rd (day)
--------------------------------------

Source height = 1.50 m

ROAD (0.00 + 63.08 + 0.00) = 63.08 dBA
Angle1 Angle2  Alpha RefLeq  P.Adj  D.Adj  F.Adj  W.Adj  H.Adj  B.Adj SubLeq
----------------------------------------------------------------------------
   -52     90   0.00  68.48   0.00  -4.37  -1.03   0.00   0.00   0.00  63.08
----------------------------------------------------------------------------

Segment Leq : 63.08 dBA

Total Leq All Segments: 63.08 dBA
```

```
Results segment # 1: Richmond Rd (night)
----------------------------------------

Source height = 1.50 m

ROAD (0.00 + 55.49 + 0.00) = 55.49 dBA
Angle1 Angle2  Alpha RefLeq  P.Adj  D.Adj  F.Adj  W.Adj  H.Adj  B.Adj SubLeq
----------------------------------------------------------------------------
   -52     90   0.00  60.88   0.00  -4.37  -1.03   0.00   0.00   0.00  55.49
----------------------------------------------------------------------------

Segment Leq : 55.49 dBA

Total Leq All Segments: 55.49 dBA

TOTAL Leq FROM ALL SOURCES (DAY): 63.08
                         (NIGHT): 55.49
```

```
STAMSON 5.0        NORMAL REPORT        Date: 24-07-2024 12:43:37
MINISTRY OF ENVIRONMENT AND ENERGY / NOISE ASSESSMENT

Filename: R5.te               Time Period: Day/Night 16/8 hours
Description:

Road data, segment # 1: Richmond Rd (day/night)
-----------------------------------------------
Car traffic volume  : 12144/1056  veh/TimePeriod  *
Medium truck volume :   966/84    veh/TimePeriod  *
Heavy truck volume  :   690/60    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):  15000
    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: Richmond Rd (day/night)
---------------------------------------------
Angle1   Angle2           : -90.00 deg   87.00 deg
Wood depth                :      0       (No woods.)
No of house rows          :      0 / 0
Surface                   :      2       (Reflective ground surface)
Receiver source distance  :  36.00 / 36.00  m
Receiver height           :  13.50 / 13.50  m
Topography                :      2       (Flat/gentle slope; with barrier)
Barrier angle1            : -90.00 deg   Angle2 : 87.00 deg
Barrier height            :  12.00 m
Barrier receiver distance :   5.00 / 5.00   m
Source elevation          :   0.00 m
Receiver elevation        :   0.00 m
Barrier elevation         :   0.00 m
Reference angle           :   0.00
```

```
Results segment # 1: Richmond Rd (day)
--------------------------------------

Source height = 1.50 m

Barrier height for grazing incidence
------------------------------------
Source      ! Receiver    ! Barrier     ! Elevation of
Height  (m) ! Height  (m) ! Height  (m) ! Barrier Top  (m)
------------+-------------+-------------+--------------
       1.50 !       13.50 !       11.83 !        11.83

ROAD (0.00 + 59.51 + 0.00) = 59.51 dBA
Angle1 Angle2  Alpha RefLeq  P.Adj  D.Adj  F.Adj  W.Adj  H.Adj  B.Adj SubLeq
----------------------------------------------------------------------------
   -90     87   0.00  68.48   0.00  -3.80  -0.07   0.00   0.00  -5.09  59.51
----------------------------------------------------------------------------

Segment Leq : 59.51 dBA

Total Leq All Segments: 59.51 dBA

Results segment # 1: Richmond Rd (night)
----------------------------------------

Source height = 1.50 m

Barrier height for grazing incidence
------------------------------------
Source      ! Receiver    ! Barrier     ! Elevation of
Height  (m) ! Height  (m) ! Height  (m) ! Barrier Top  (m)
------------+-------------+-------------+--------------
       1.50 !       13.50 !       11.83 !        11.83

ROAD (0.00 + 51.91 + 0.00) = 51.91 dBA
Angle1 Angle2  Alpha RefLeq  P.Adj  D.Adj  F.Adj  W.Adj  H.Adj  B.Adj SubLeq
----------------------------------------------------------------------------
   -90     87   0.00  60.88   0.00  -3.80  -0.07   0.00   0.00  -5.09  51.91
----------------------------------------------------------------------------

Segment Leq : 51.91 dBA

Total Leq All Segments: 51.91 dBA

TOTAL Leq FROM ALL SOURCES (DAY): 59.49
                         (NIGHT): 51.91
```

APPENDIX B

FTA VIBRATION CALCULATIONS

GW21-416 13-Aug-24

Possible Vibration Impacts
Predicted using FTA General Assesment

Train Speed	70 km/h	43 mph	

	Distance from C/L	
	(m)	(ft)
LRT	32.0	105.0

	Vibration		
From FTA Manual Fig 10-1			
Vibration Levels at distance from track	67	dBV re 1 micro in/sec	
Adjustment Factors FTA Table 10-1			
Speed reference 50 mph	-1.30	Speed Limit of 70 km/h (43 mph)	
Vehicle Parameters	0	Assume Soft primary suspension, Wheels run true	
Track Condition	0	None	
Track Treatments	0	None	
Type of Transit Structure	-5	Station	
Efficient vibration Propagation	0	None	
Vibration Levels at Fdn	61		
Coupling to Building Foundation	0	Bear on bedrock	
Floor to Floor Attenuation	-2.0	Ground Floor Occupied	
Amplification of Floor and Walls	6		
Total Vibration Level	64.7	dBV or	0.044 mm/s
Noise Level in dBA	29.7	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
		Reference Speed		
Speed	Vehicle Speed	50 mph	30 mph	Vibration level is approximately proportional to $20*\log(\text{speed}/\text{speed}_{ref})$. Sometimes the variation with speed has been observed to be as low as 10 to 15 $\log(\text{speed}/\text{speed}_{ref})$.
	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.

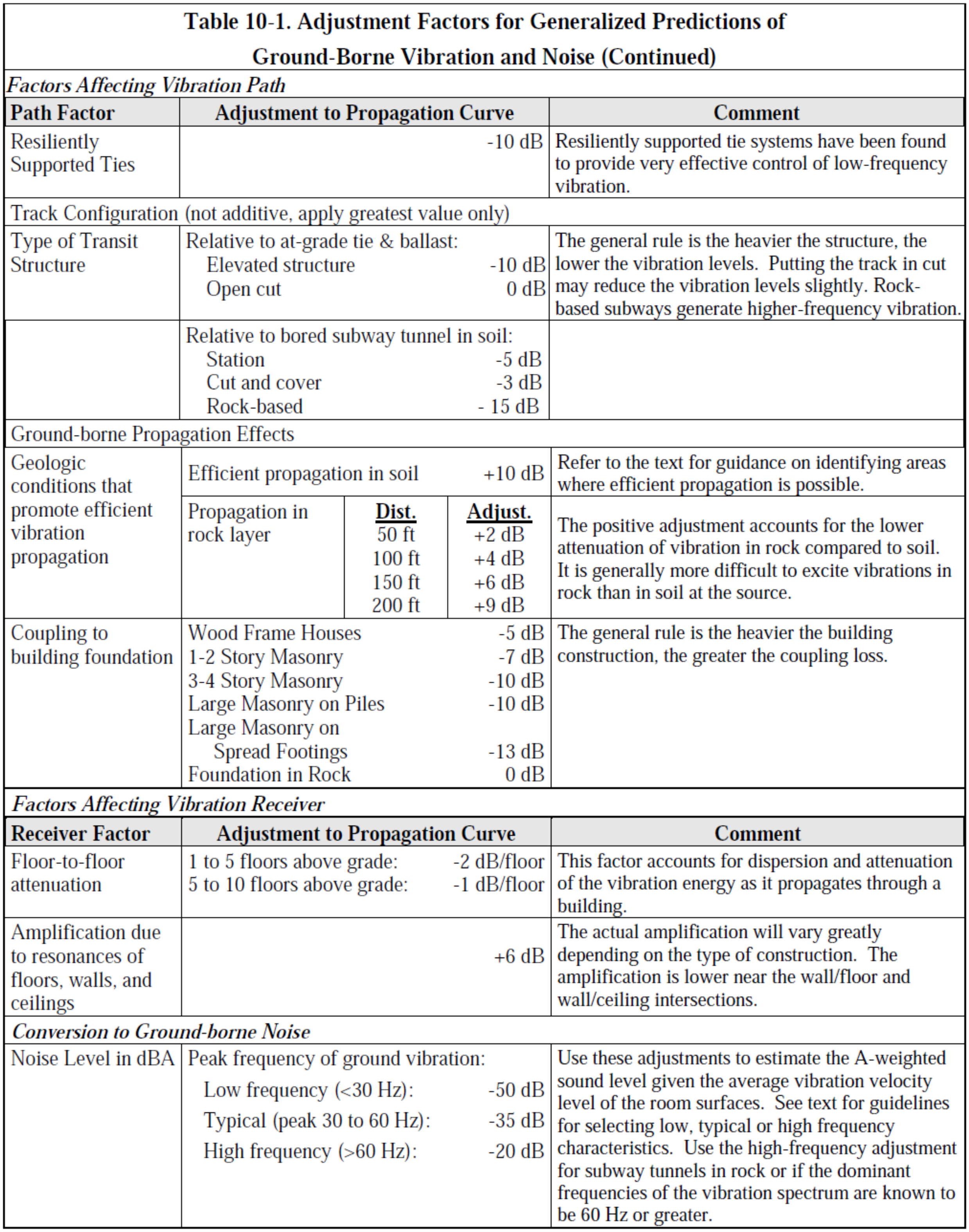


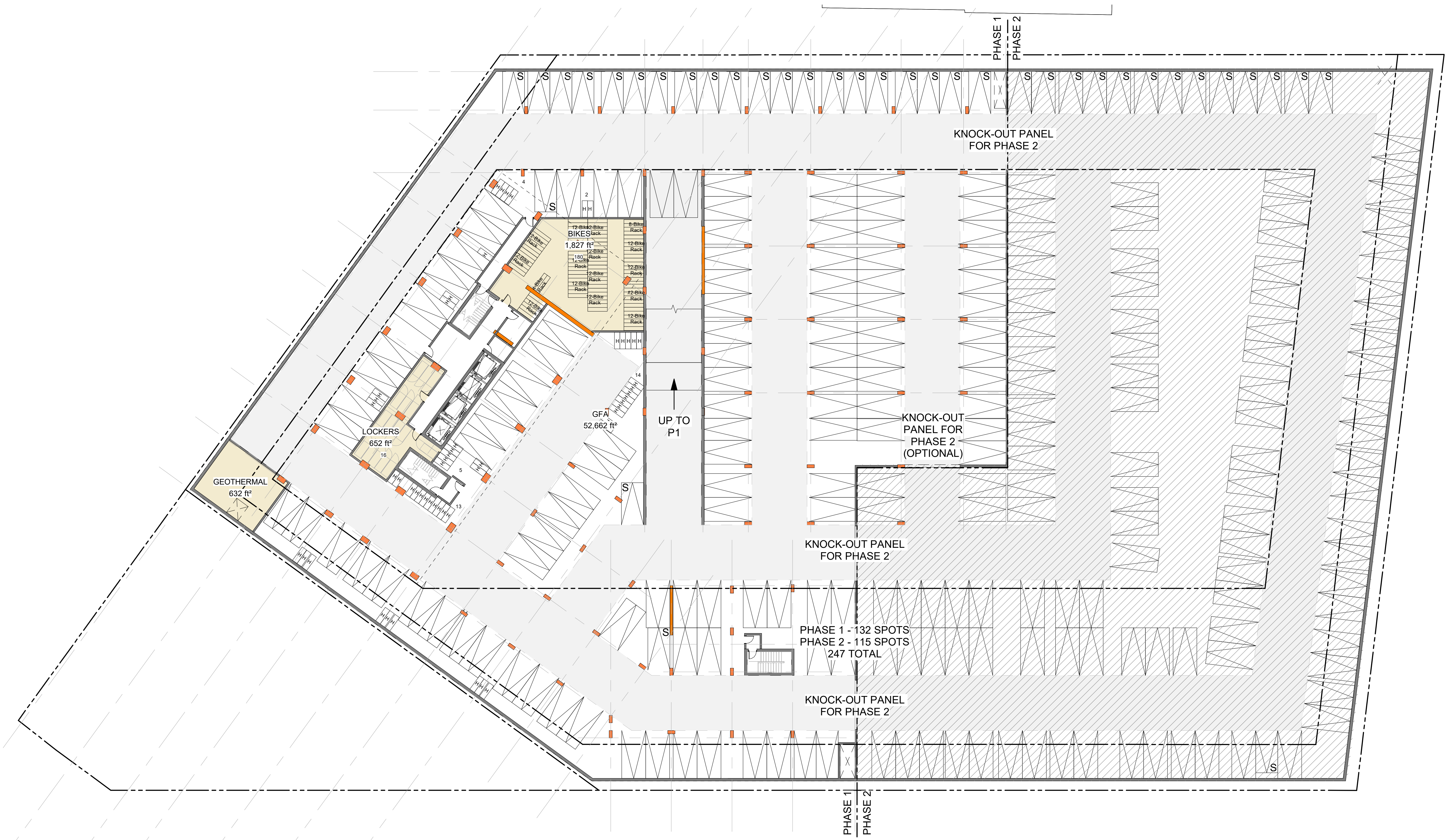
Table 10-1. Adjustment Factors for Generalized Predictions of Ground-Borne Vibration and Noise (Continued)

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 **Dist.** / **Adjust.** 50 ft +2 dB 100 ft +4 dB 150 ft +6 dB 200 ft +9 dB	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.
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

1047 Richmond Road Ottawa, ON – Floor Plans – Prepared by – RLA Architecture –

Project Number 2404



rla/architecture

P2 FLOOR PLAN

SCALE: 1 : 400

DATE: 2024-08-16

1047 RICHMOND ROAD

OTTAWA, ON

DRAWN BY
RIB

SHEET #
D100

PROJ. No. 2404

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PLOT SCALE: 1:1

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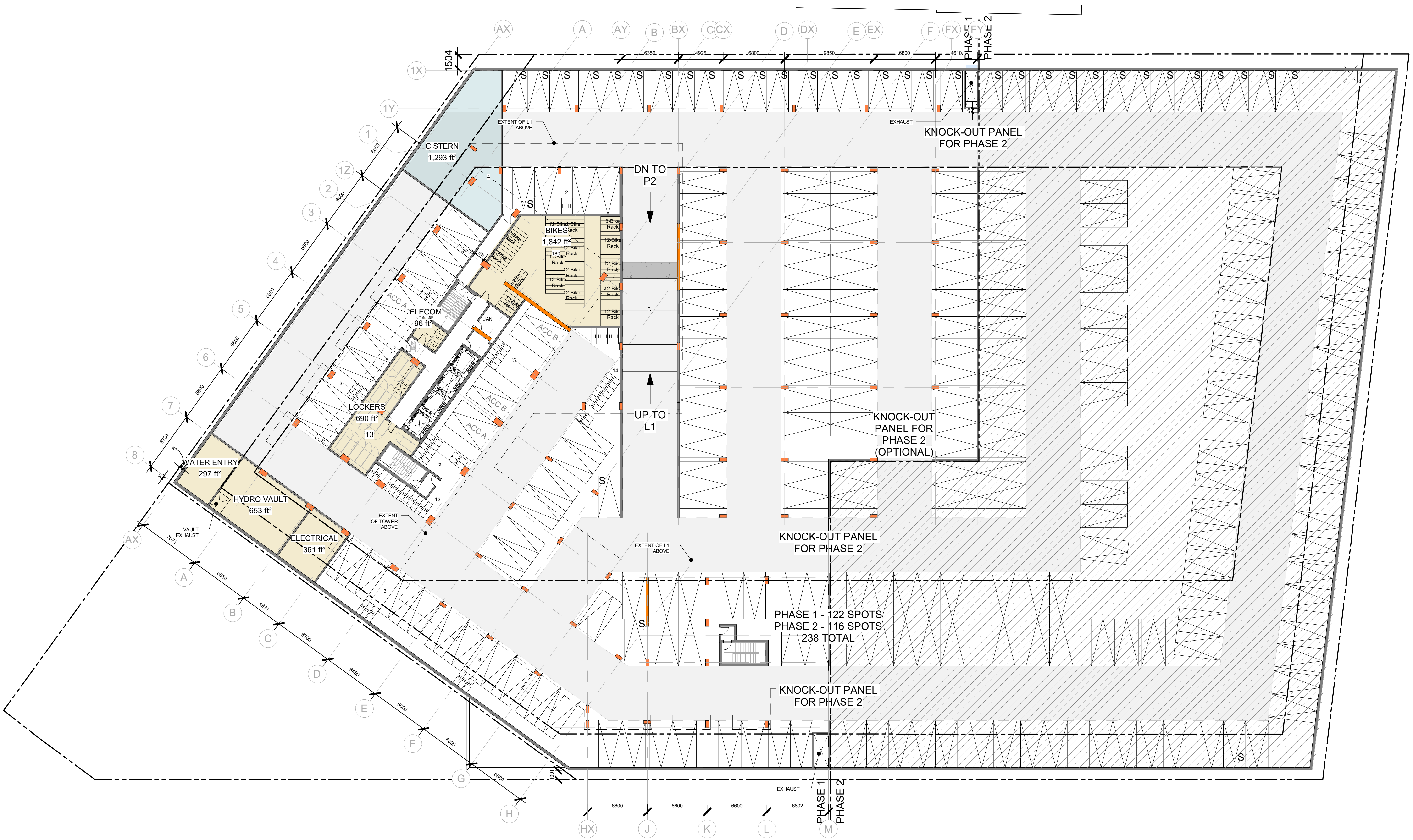


rla / architecture

P1 FLOOR PLAN

SCALE: 1 : 400

DATE: 2024-08-16

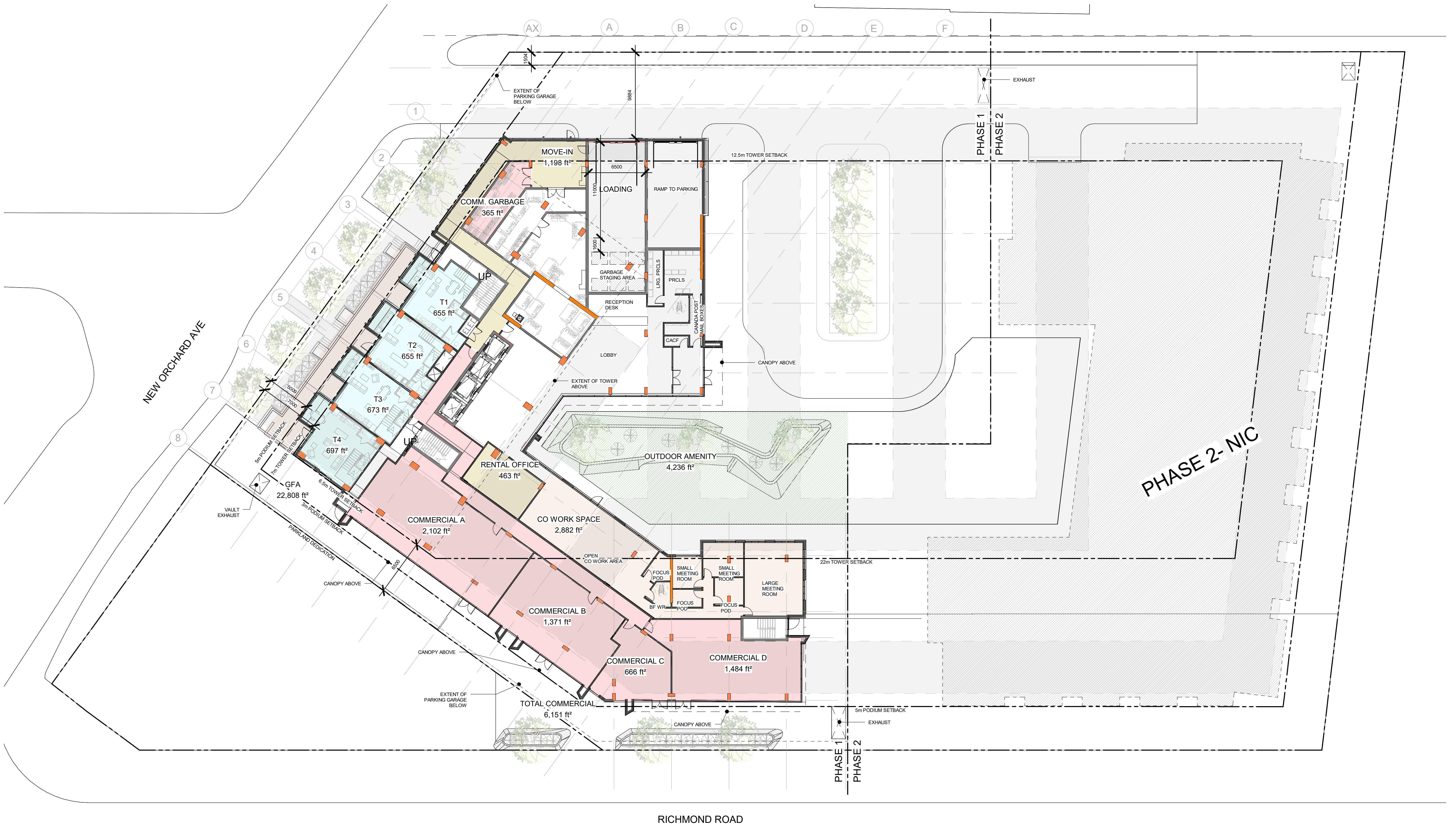
1047 RICHMOND ROAD

OTTAWA, ON

DRAWN BY
RIB

SHEET #
D101

PROJ. No. 2404



rla/architecture

L1 FLOOR PLAN

SCALE: 1 : 400

DATE: 2024-08-16

1047 RICHMOND ROAD

OTTAWA, ON

DRAWN BY
RIB

SHEET #
D102

PROJ. No. 2404

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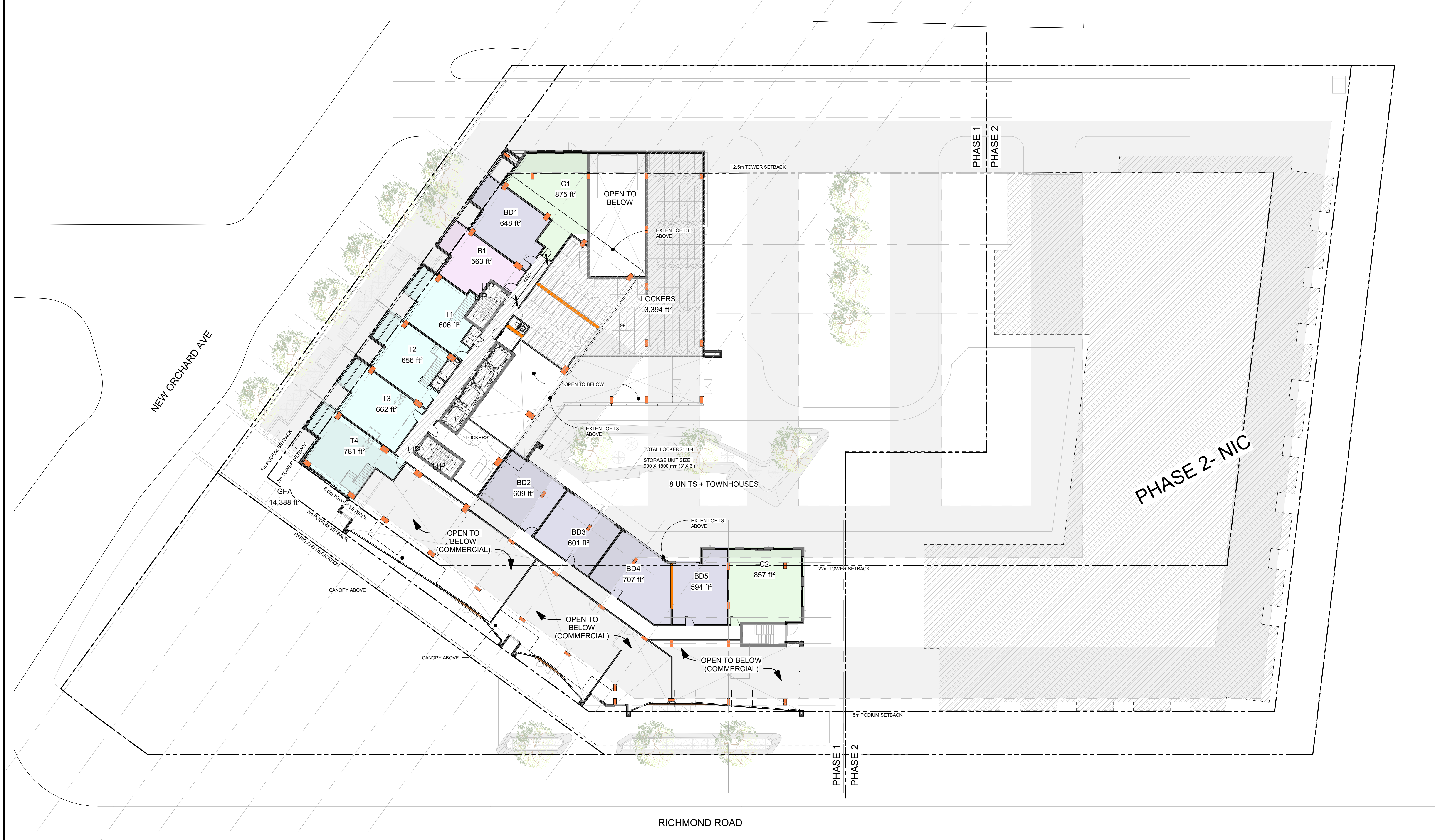

rla/architecture

L2 FLOOR PLAN

SCALE: 1 : 400

DATE: 2024-08-16

1047 RICHMOND ROAD

OTTAWA, ON

DRAWN BY
RIB

SHEET #
D103

PROJ. No. 2404

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PLOT SCALE: 1:1

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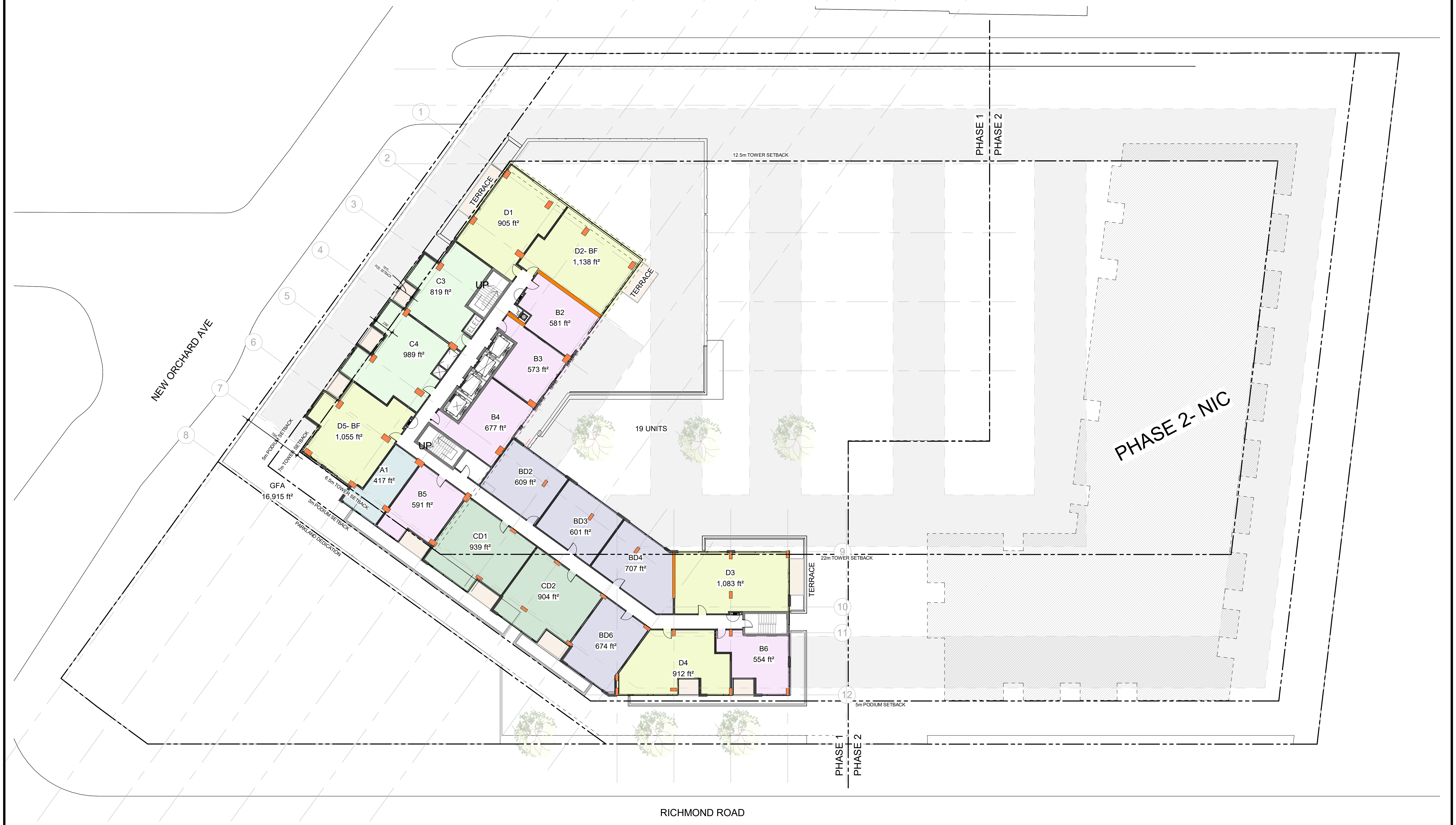

rla/architecture

L3 FLOOR PLAN

SCALE: 1 : 400

DATE: 2024-08-16

1047 RICHMOND ROAD

OTTAWA, ON

DRAWN BY
RIB

SHEET #
D104

PROJ. No. 2404

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PLOT SCALE: 1:1

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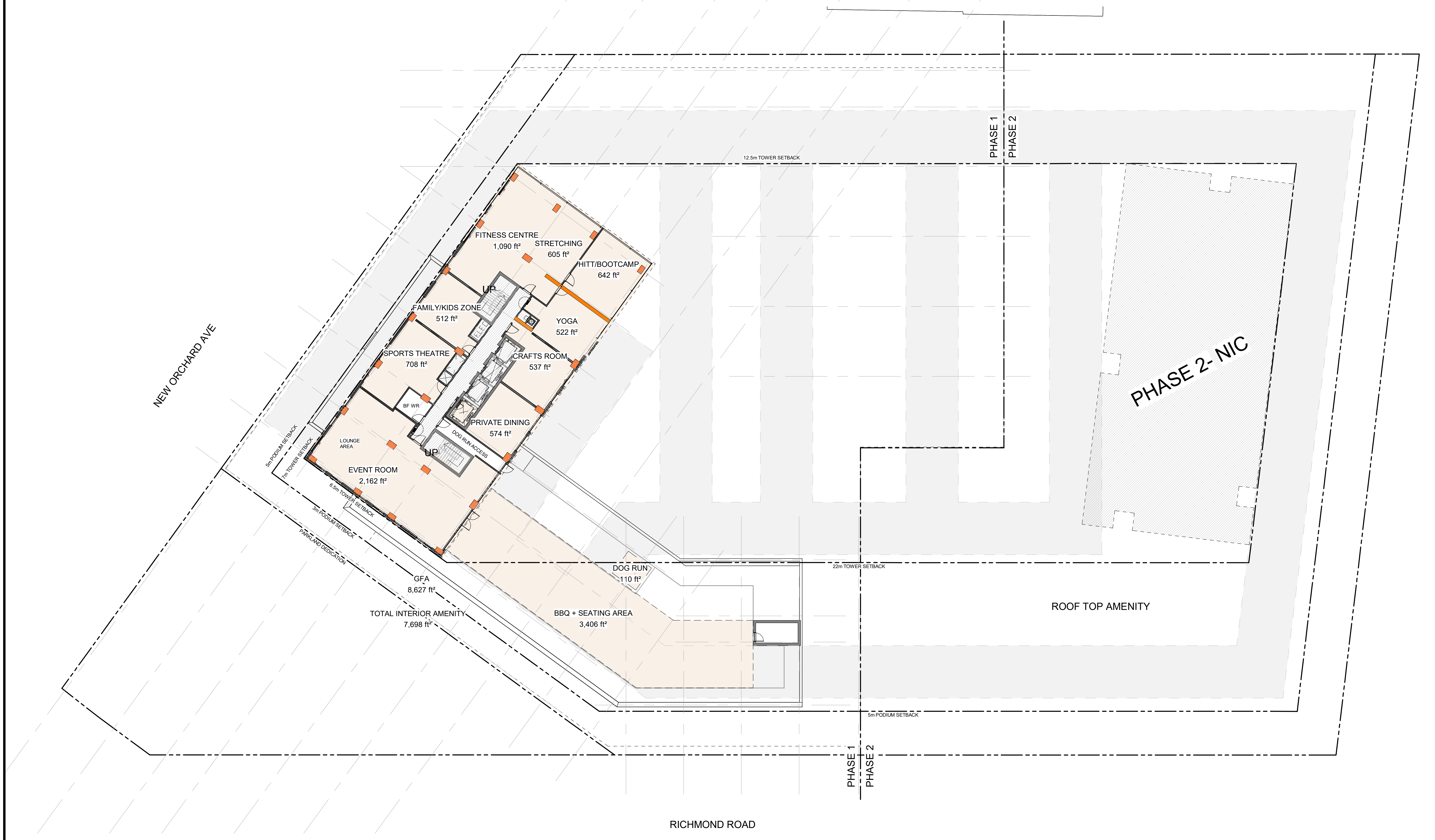

rla / architecture

L4 FLOOR PLAN- AMENITY FLOOR

SCALE: 1 : 400

DATE: 2024-08-16

1047 RICHMOND ROAD

OTTAWA, ON

DRAWN BY
RIB

SHEET #
D105
PROJ. No. 2404

PLOT DATE: 8/16/2024 10:55:03 AM

PLOT SCALE: 1:1

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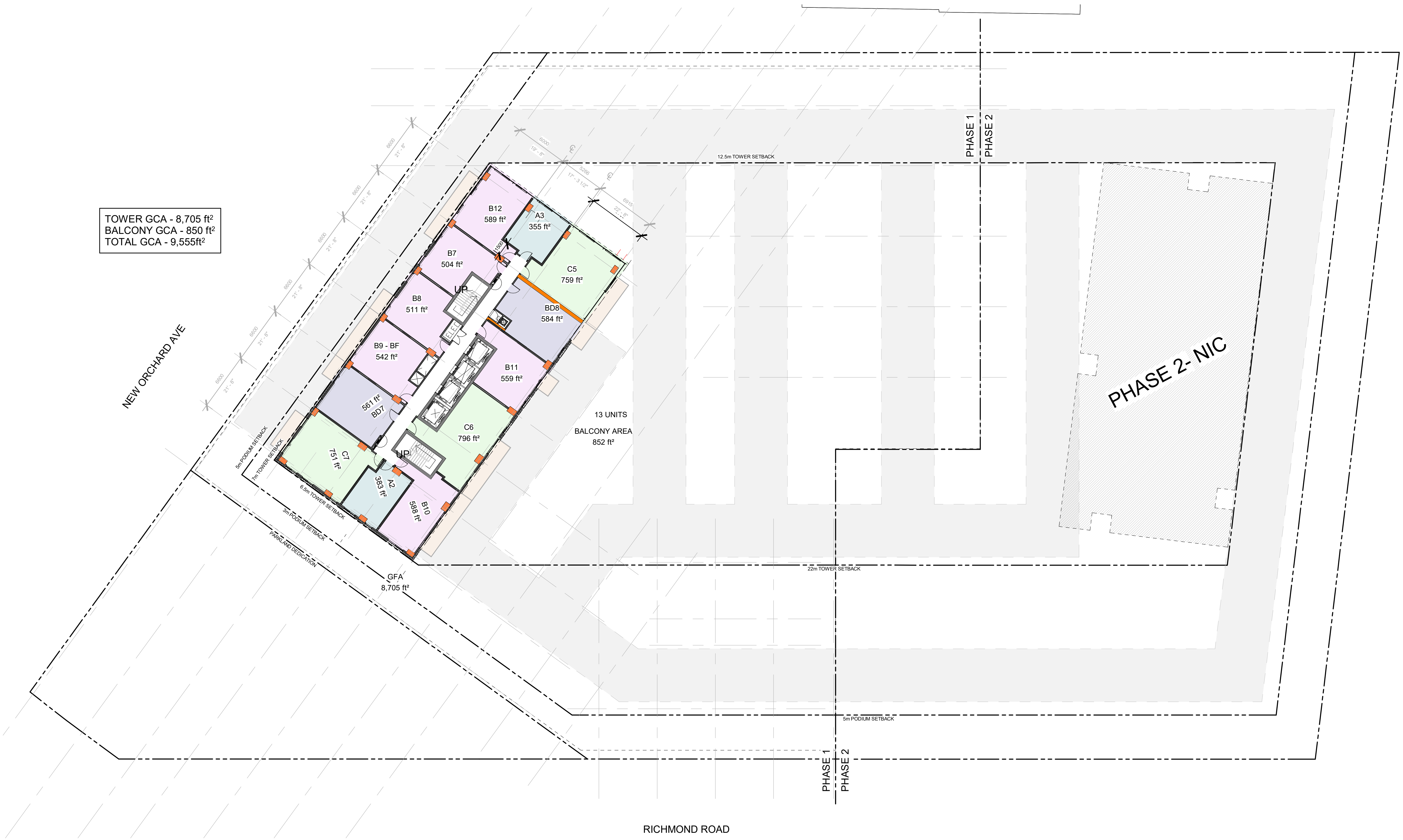

TOWER GCA - 8,705 ft²
BALCONY GCA - 850 ft²
TOTAL GCA - 9,555ft²

rla/architecture

TYP. TOWER FLOOR PLAN A (L5-L7)

SCALE: 1 : 400

DATE: 2024-08-16

1047 RICHMOND ROAD

OTTAWA, ON

DRAWN BY
RIB

SHEET #
D106

PROJ. No. 2404

PLOT DATE: 8/16/2024 10:55:06 AM

PLOT SCALE: 1:1

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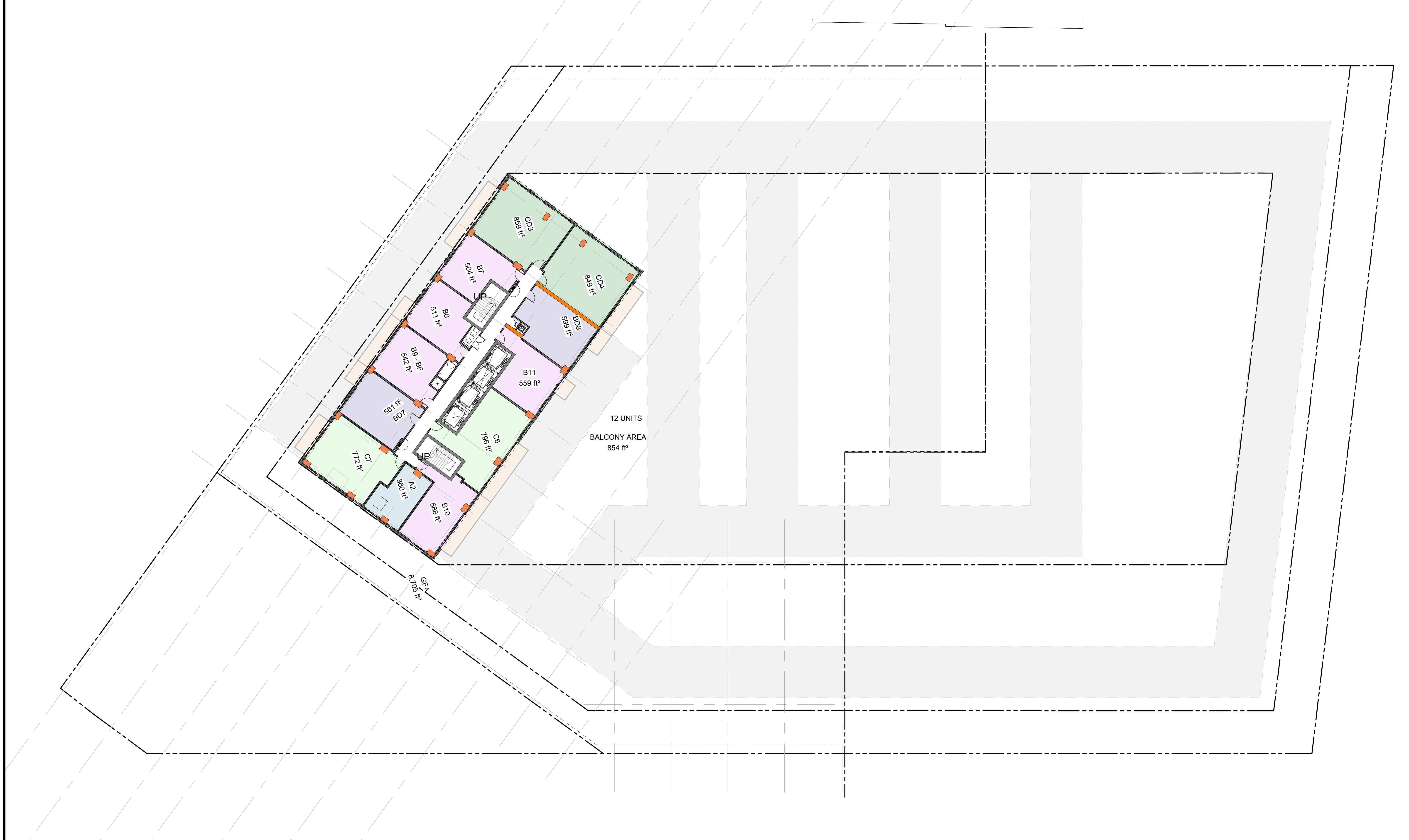


rla/architecture

TYP. TOWER FLOOR PLAN B (L15-35)

SCALE: 1 : 400

DATE: 2024-08-16

1047 RICHMOND ROAD

OTTAWA, ON

DRAWN BY
RIB

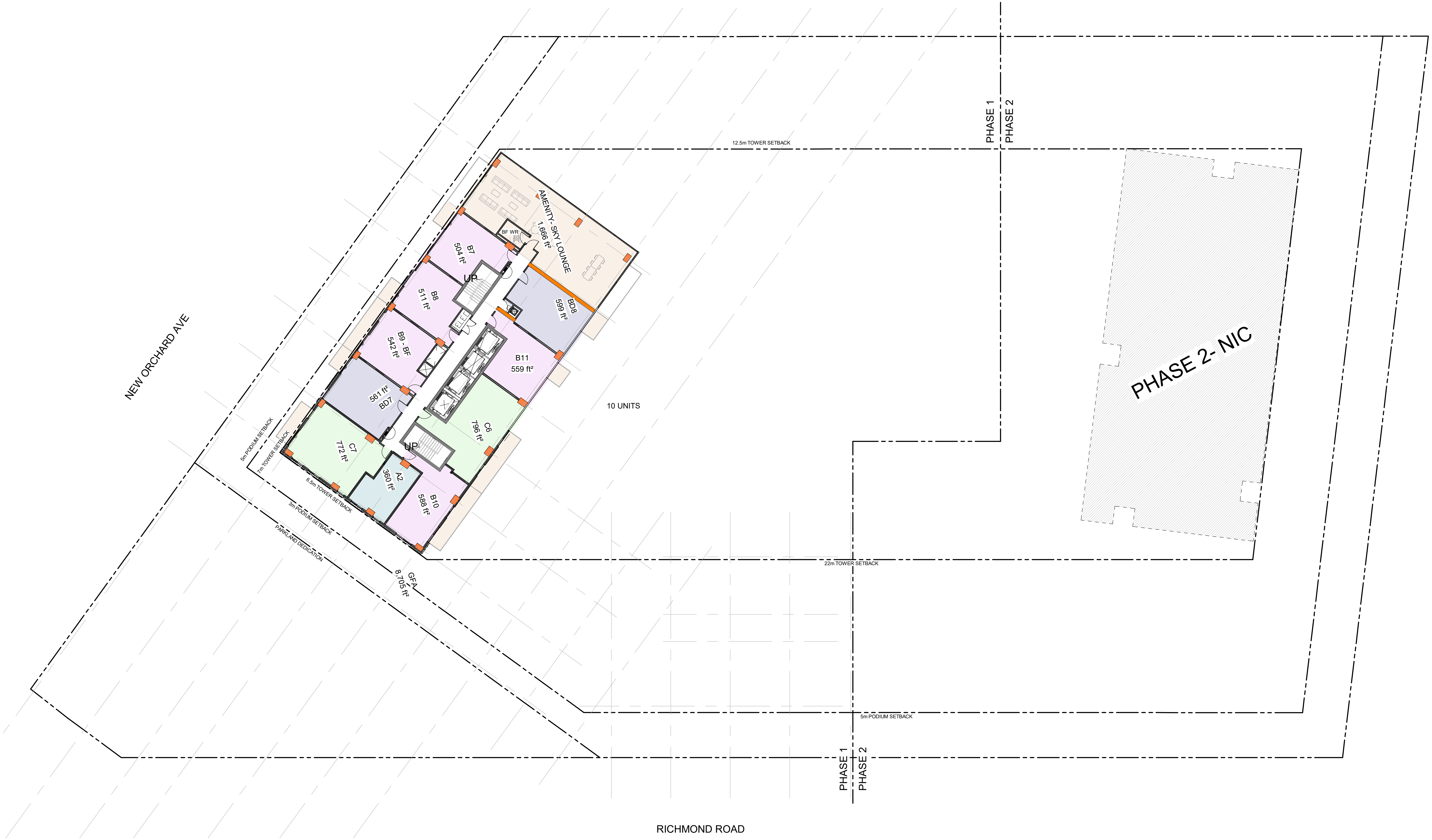
SHEET #
D107

PROJ. No. 2404

PLOT DATE: 8/16/2024 10:55:12 AM

PLOT SCALE: 1:1

PAPER SIZE: ANSI B Tabloid 11" x 17" (279.40 x 431.80mm)



rla/architecture

L36- AMENITY

SCALE: 1 : 400

DATE: 2024-08-16

1047 RICHMOND ROAD

OTTAWA, ON

DRAWN BY
RIB

SHEET #
D109

PROJ. No. 2404

PLOT DATE: 8/16/2024 10:55:15 AM

PLOT SCALE: 1:1

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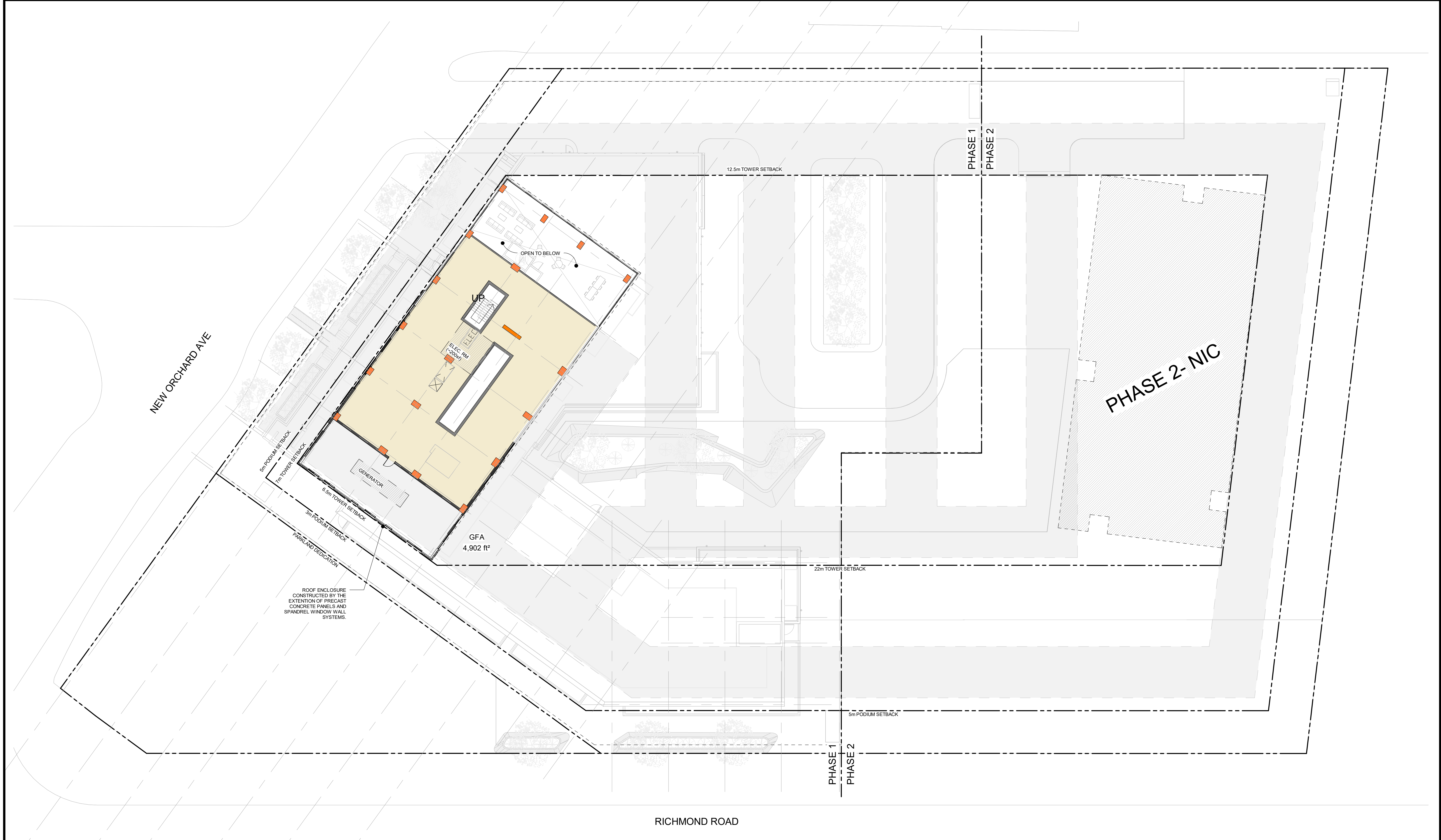

rla / architecture

MECHANICAL PENTHOUSE

SCALE: 1 : 400

DATE: 2024-08-16

1047 RICHMOND ROAD

OTTAWA, ON

DRAWN BY
Author

SHEET #
D110

PROJ. No. 2404

PLOT DATE: 8/16/2024 10:55:48 AM

PLOT SCALE: 1:1

PAPER SIZE: ANSI B Tabloid 11" x 17" (279.40 x 431.80mm)

APPENDIX E

Proximity Assessment:

Report PG6108-LET.01 Revision 2 dated September 3, 2024

PATERSON GROUP
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, 2024
File: PG6108-LET.01 Revision 2

Fengate Asset Management
TD North Tower
77 King Street West, Suite 3410
Toronto, Ontario
M5K 1H1

Attention: **Ms. Corina Sajewski**

Subject: **Proximity Assessment**
Proposed Mixed-Use Development
1047 Richmond Road, Ottawa, ON

Dear Madam,

Further to your request and authorization, Paterson Group (Paterson) prepared the current letter report to summarize construction issues which could occur due to the proximity of the proposed buildings with respect to the subject alignment of the proposed Confederation Line Light Rail project and New Orchard Station. The following letter should be read in conjunction with the Geotechnical Investigation Report (Report No. 21494078 Revision 2 dated June 27, 2023, prepared by Golder Associated Ltd).

1.0 Background Information

The proposed development at 1047 Richmond Road will consist of two residential buildings. The first building is noted as Tower A, rising to 36 storeys. Details of Tower B were not known at the time of preparation of this report. It is further understood that both structures will share a common two-level underground parking structure placed approximately 1 m away from the property boundary along Richmond Road. At the time of issuance of this report, drawings of the final alignment of the Confederation Line and New Orchard Station have not been provided to Paterson. However, it is understood that the subject tunnel alignment will be located below the landscaped area between Richmond Road and Byron Avenue.

The following sections summarize our existing soil information and construction precautions for the proposed building, 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, dewatering and discharge plans, temporary shoring design drawings, foundation and subsurface walls/structure design drawings, a Blast Assessment Report and field monitoring program as described in the application conditions.

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 is at a geodetic elevation of approximately 65 to 66 m.
- ☐ The overburden thickness is approximately 1.6 to 4.8 m.
- ☐ Bedrock surface elevation is at an approximate geodetic elevation of 61.1 to 64.4 m.
- ☐ The bedrock underlying the site consists of a good to excellent quality dolostone with interbedded shale, limestone, and sandstone. Unconfined compressive strengths, where tested, ranged from 86 to 144 MPa.

Tunnel Location

The GeoOttawa Rail Alignment O-Train tool indicates that an approximate setback of 19 m is present between the property line and the proposed Confederation Line and New Orchard Station. The rail tunnel runs parallel to the south-east property boundary. It is understood that the underground parking levels for the proposed building will be placed approximately 1 m away from the southeast property line adjacent to the Richmond Road Right-of-Way (ROW). Therefore, an approximate horizontal separation of 20 m is present between the subject alignment of the Confederation Line and New Orchard Station, and the proposed underground parking structure at 1047 Richmond Road.

Based on preliminary design drawings issued in 2016, the underside of the tunnel elevation will be at an approximate elevation of 58 m along the subject alignment. The founding elevation of the proposed building will be approximately 55.5 m (geodetic). Therefore, a vertical differential of approximately 3 m is present between the founding levels of the two structures with a horizontal separation of at least 20 m.

3.0 Construction Precautions and Recommendations

Influence of Proposed Development on Tunnel

Based on existing soil information and building design details, the footings of the proposed building will be founded on good-quality bedrock. Therefore, lateral loads due to the building footings will be transferred directly into the bedrock well within a conservative 1H:6V zone of influence from the outside face of the footing.

From the preliminary information provided for the subject alignment and the proposed building location, the proposed building at 1047 Richmond Road will not cause additional loading on the subject alignment of the Confederation Line or New Orchard Station.

Excavation and Temporary Shoring

The overburden along the perimeter of the proposed building footprint will need to be temporarily shored with a solder pile and lagging system in order to complete the construction of the underground parking structure for the proposed buildings. Bedrock removal is also anticipated, which will 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. It is understood that the Confederation Line LRT extension at Richmond Road is currently under construction and the bedrock removal for the proposed buildings may potentially be completed prior to the construction of the subject alignment of the proposed Confederation Line and rail station. In that case, there will be no impact of the building excavation on the subject alignment of the proposed Confederation Line and rail station.

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 draft Geotechnical and Hydrogeological Investigation Report (Report No. 21494078 Revision 2 dated June 27, 2023, prepared by Golder Associated Ltd.).

A seismograph is recommended to be installed either adjacent to or within the Confederation Line Tunnel as part of the Vibration Monitoring and Control Program to monitor vibrations during the bedrock removal program. 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 blasting operations, dewatering, backfilling and compaction, construction traffic and other construction activities.

Pre-Construction Survey

A pre-construction survey will be required for the tunnel structure and rail station. Any existing structures in the immediate area of the proposed building will also undergo a pre-construction survey as per standard construction practices, where bedrock blasting will be required.

Groundwater Control

Groundwater observations during the geotechnical investigation indicated groundwater levels within the bedrock between approximately 2.7 to 9.3 m below the existing ground surface. However, the Confederation Line is understood to be founded on bedrock. Therefore, no groundwater lowering effects due to the proposed development are anticipated with respect to the Confederation Line.

Tunnel Waterproofing System

Due to the separation between the proposed buildings at 1047 Richmond Road and the subject alignment of the Confederation Line and New Orchard Station, it is anticipated that the replacement or repair of the waterproofing systems for the tunnel structure and rail station will not be required during construction.

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 proposed tunnel alignment or rail station.

It should be noted that 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 and subsurface walls/structure design drawings, a Blast Assessment Report and field monitoring program as described in the application conditions.

We trust that this information meets your immediate request.

Best Regards,

Paterson Group Inc.

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