Rail Line
Level 1 Proximity Study

Proposed Development

299 West Hunt Club Road
Ottawa, Ontario

Prepared for the Pritec Management

Report PG7560-1 dated May 29, 2025

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Report No. 25012 dated March 24, 2025

1.0 Introduction

Paterson Group (Paterson) was commissioned by Pritec Management to prepare a Rail Line Proximity Study – Level 1 for the proposed development to be located at 299 West Hunt Club Road in the City of Ottawa.

The objectives of the current study were to:

- ❑ Review all current information available from the City of Ottawa with regards to the infrastructure of the Rail Line in the vicinity of the subject site.

- ❑ Liaison between the City of Ottawa and Pritec Management consultant team involved with the aforementioned project.

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

2.0 Development Details

Based on the current development plans, the proposed development at the subject site will consist of a multi-storey parking structure located at the northern end of the subject site. The reminder of site will be occupied by a fire access route, parking areas, and an existing commercial building with landscaped margins.

The following is known about the Rail Line in the vicinity of the subject site:

- ❑ The proposed development is anticipated to be located approximately 15 m south of the rail line.

- ❑ The rail line is located at-grade in the vicinity of the subject site,

- ❑ Based on the subsurface profile encountered within the boreholes at 299 West Hunt Club Road, the overburden soils consist of an approximate 4.5 m thickness of silty sand fill, underlain by silty clay. A dense silty sand was observed underlying the silty clay at a depth of about 6 m. Further, from available geological mapping, bedrock is present at an approximate depth 15 to 25 m below the existing ground surface at the subject site.

3.0 Construction Methodology and Impact Review

Paterson has prepared a construction methodology summary along with possible impacts on the adjacent rail line, 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 mitigation program recommended for the proposed development. The primary item will be vibrations associated with the installation of the temporary shoring system in the vicinity of the northern boundary of the site. It is recommended that a vibration monitoring program be implemented to ensure vibration levels remain below recommended tolerances. Details of the recommended vibration monitoring program are presented below.

3.1 Vibration Monitoring and Control Program

Proposed Vibration Limits

Due to the proximity of the existing rail line to the subject site, the contractor should take extra precaution to minimize vibrations. The monitoring program will be required for the full duration of the temporary shoring system installation. The purpose of the vibration monitoring and control program (VMCP) is to provide a description of the measures to be applied by the contractor to manage excavation operations and any other vibration sources during the construction for 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 rail line located in the 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.

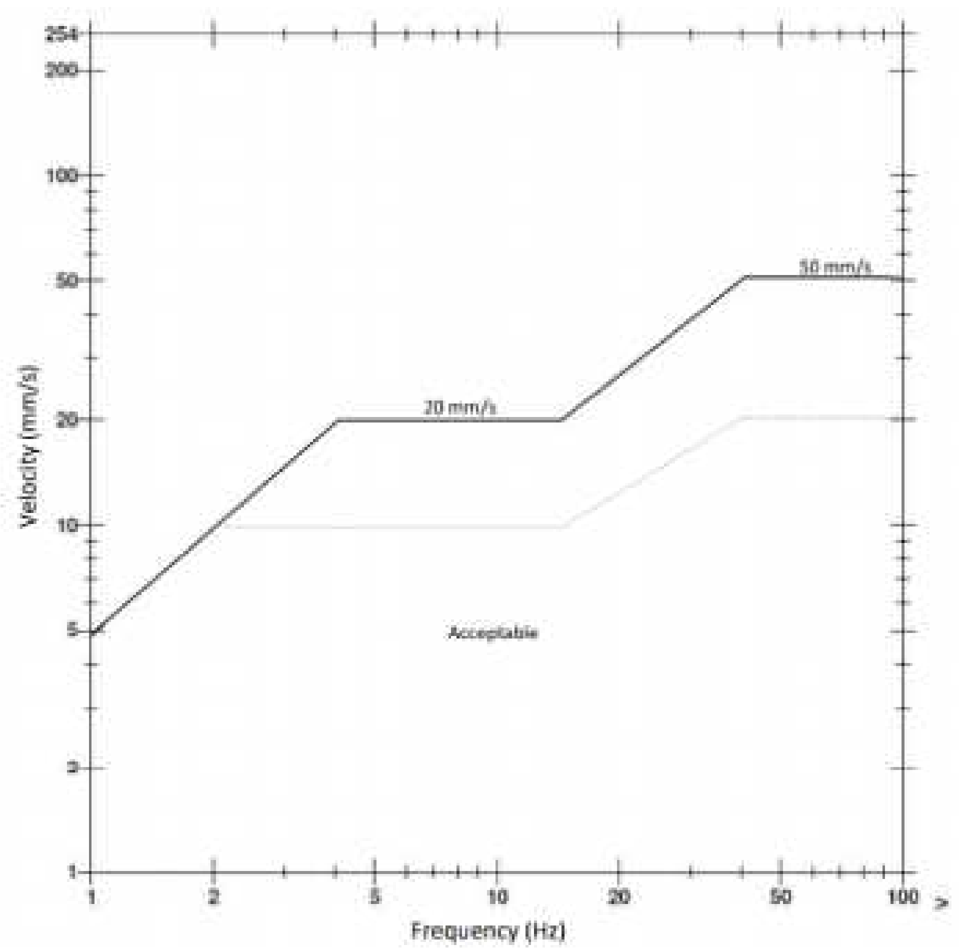
The location of the seismograph should be reviewed periodically throughout construction to ensure that the monitoring equipment remains along the alignment of the rail line with the closest radius to the construction activities. The seismograph locations should be approved by the project manager prior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

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

Proposed Vibration Limits

The excavation and shoring operations should be planned and conducted under the supervision of a licensed professional engineer who is an experienced excavation and shoring consultant. The following Figure 1 outlines the vibration limits for the rail line:

Figure 1 - Proposed Vibration Limits at the Rail Line



Monitoring Data

The monitoring protocol should include the following information:

Warning Level Event (indicated by the **light blue** line on Figure 1)

- ❑ Paterson will review all vibrations over the established warning level, and.
- ❑ Paterson will notify the contractor if any vibrations occur due to construction activities and are close to exceedance level.

Exceedance Level Event (indicated by the **black line** on Figure 1)

- ❑ Paterson will notify all the relevant stakeholders via email
- ❑ Ensure monitors are functioning
- ❑ Issue the vibration exceedance result
- ❑ Cease construction while means and methods are discussed with the Contractor in order to reduce vibration levels.

The data collected will include the following:

- ❑ Measured vibration levels
- ❑ Distance from the construction activity to monitoring location
- ❑ Vibration type

Monitoring should be compliant with all related regulations.

3.2 Incident/Exceedance Reporting

In case an 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:

- ❑ The identified location of the vibration exceedance
- ❑ The date, time and nature of the exceedance
- ❑ Purpose of the exceeded monitor and current vibration criteria
- ❑ The likely cause of the exceedance
- ❑ Description of the response action that has been completed to date
- ❑ Description the proposed measures to address the exceedance.

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

Based on the O-Train System Proximity Study Guidelines dated April 2022, a Level 1 Rail Line Proximity Study is considered to be required for the proposed development. A Level 1 Proximity Study is required where the proposed development is located within the City of Ottawa's Development Zone of Influence.

The following Table 1 lists the applicable requirements for Level 1 study for each item and our associated responses:

Table 1

List of Rail Line Proximity Study - Level 1 Requirements

Level 1 Projects	**Response**
A site plan of the development	See Site Plan prepared by Vandenberg & Wildeboer Architects and Rail Line Proximity Plan (Drawing PG7560-1) prepared by Paterson, both presented in Appendix A.
Floor Plan of the development	See floor plans prepared by Vandenberg & Wildeboer Architects presented in Appendix A.
Development Cross Section	See Cross-Section A-A (Drawing PG7560-1A) prepared by Paterson and presented in Appendix A.
Geotechnical Report prepared in accordance with the City's Geotechnical Investigation and Reporting Guidelines for Development Applications	Refer to Geotechnical Investigation prepared by others: Report No: 25012 dated March 24, 2025, 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	See Survey Plan prepared by Farley, Smith & Denis Surveying Ltd. and presented in Appendix A.
Utility Service Plan	The Servicing Plan will be provided once available for the proposed project
Stormwater Management Plan and Grading Plan	Stormwater Management Plan and Grading Plan will be provided once available for the proposed project

Architectural Drawings and Landscape Plans	Refer to the Architectural Drawings prepared by Vandenberg & Wildeboer Architects, presented in Appendix A.
Noise and Vibration Study prepared in accordance with the City's environmental noise control guidelines (required for all applications within 75m of light rail transit)	The Noise Feasibility Study will be provided once available for the proposed project.

We trust that this information satisfies your immediate request.

Best Regards,

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
May 29, 2025
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Deepak K Rajendran, E.I.T.

Scott S. Dennis P.Eng.

APPENDIX A

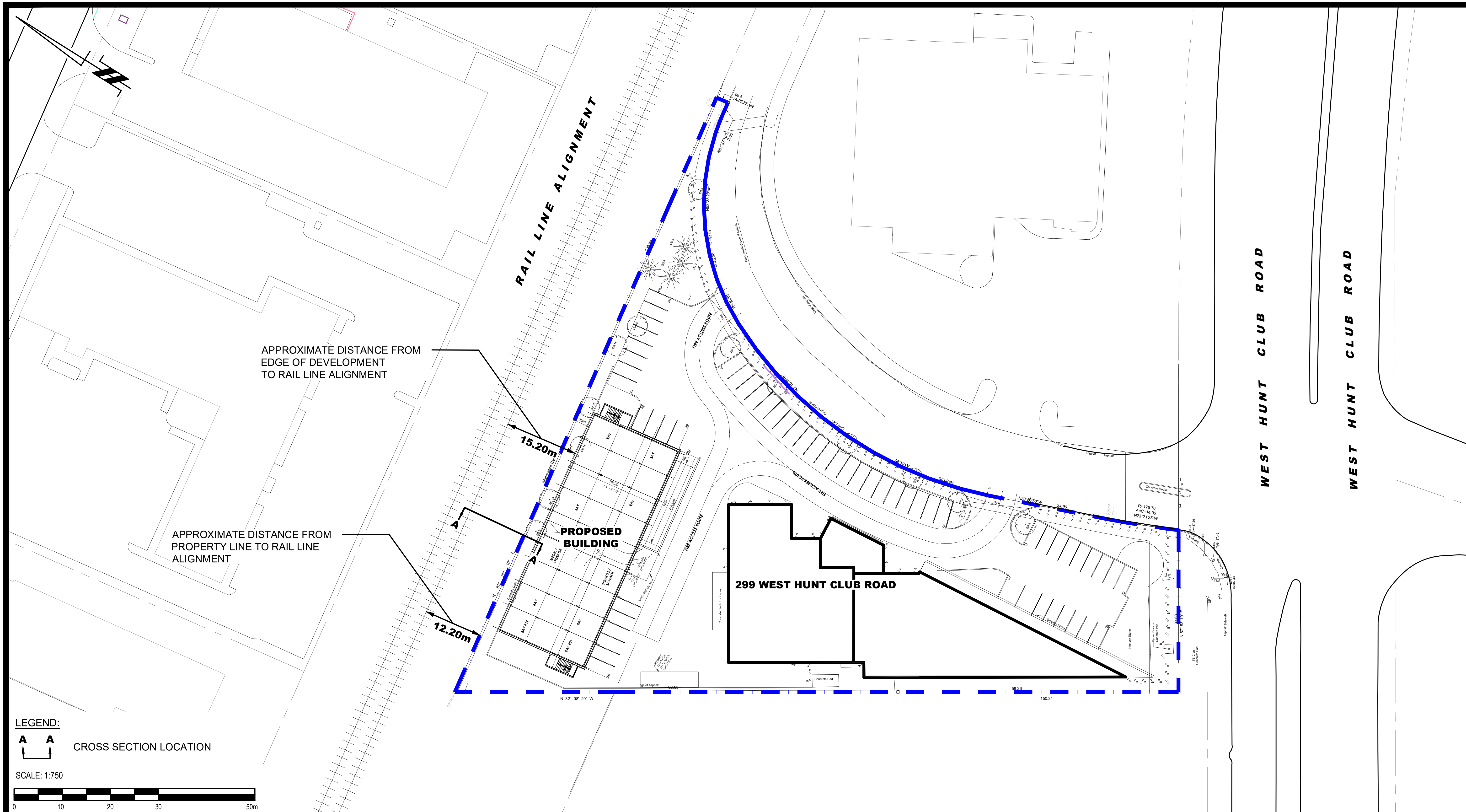
Proximity Assessment Plan

Cross Section A-A'

Topographic Plan of Survey

Architectural Drawings

Construction Methodology and Impact Review



PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

PRITEC MANAGEMENT
GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
299 WEST HUNT CLUB ROAD

OTTAWA, ONTARIO

Title:

RAIL LINE PROXIMITY PLAN

Scale: 1:750	Date: 05/2025
Drawn by: GK	Report No.: PG7560-1
Checked by: DR	Dwg. No.: PG7560-1
Approved by: SD	Revision No.:

11x17

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PATERSON GROUP

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

NO.	REVISIONS	DATE	INITIAL

PRITEC MANAGEMENT
GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
299 WEST HUNT CLUB ROAD

OTTAWA, ONTARIO

Title:

CROSS SECTION A-A'

Scale: 1:50	Date: 05/2025
Drawn by: GK	Report No.: PG7560-1
Checked by: DR	Dwg. No.: PG7560-1A
Approved by: SD	Revision No.:

11x17

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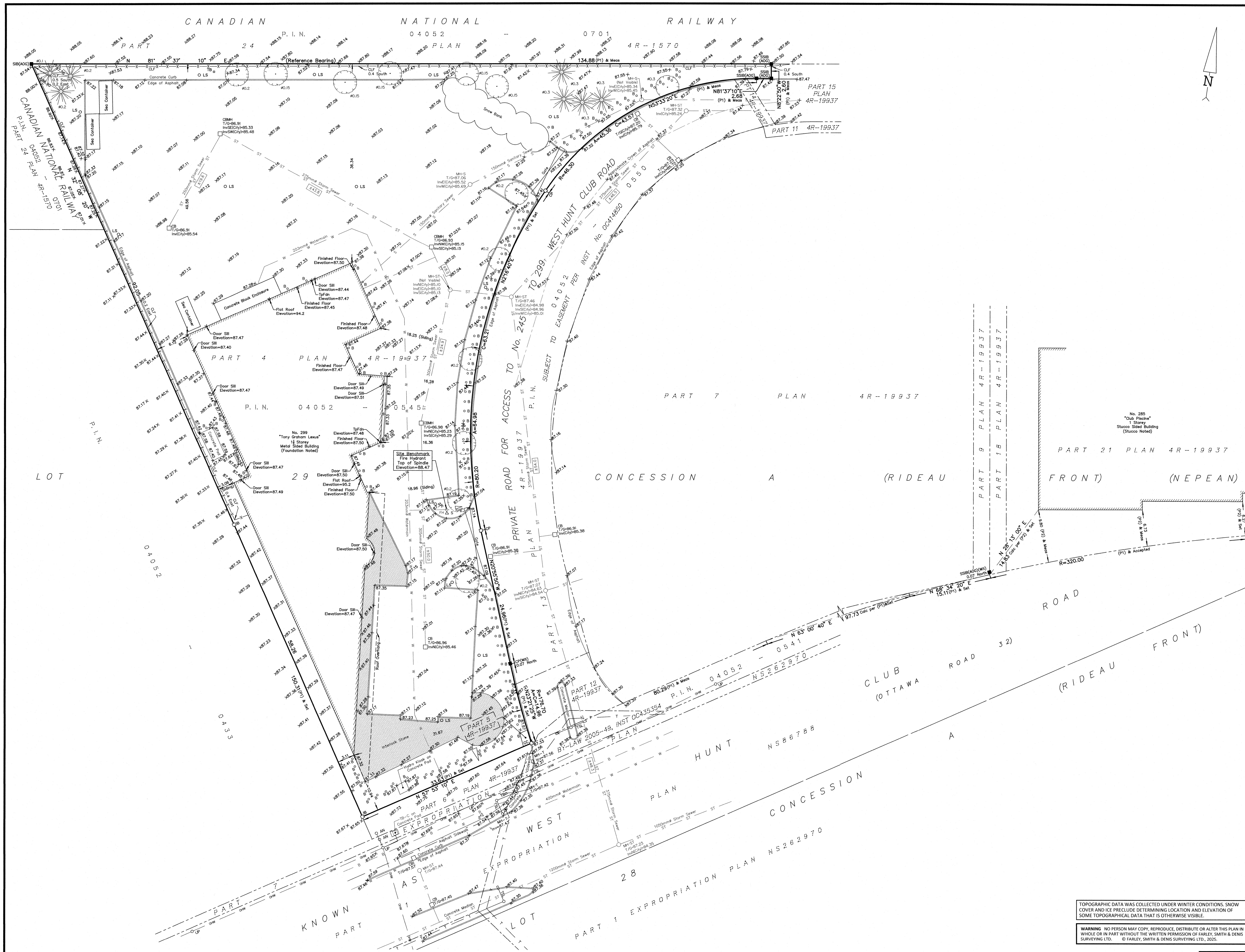


TOPOGRAPHIC PLAN OF SURVEY OF

PART OF LOT 29
CONCESSION A (RIDEAU FRONT)
GEOGRAPHIC TOWNSHIP OF NEPEAN
CITY OF OTTAWA

FARLEY, SMITH & DENIS SURVEYING LTD. 2025

Scale 1: 300

0 5 10 15 20 25 30 metres

Metric Note
Distances and/or coordinates on this plan are in metres and can be converted to feet by dividing by 0.3048.

Distance Note
Distances shown on this plan are ground distances and can be converted to grid distances by multiplying by the combined scale factor of 0.99994.

Bearing Note
Bearings hereon are grid bearings derived from the Can-Net Real Time Network and are referred to the Central Meridian of MTM Zone 9 (76°30' West Longitude) Nad-83 (Original).

Elevation Notes
1. Elevations shown are geodetic and are referred to Geodetic Datum CGVD-1928 :1978. (AOG Job No. 2175-00)
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: 7089p&p04, 10344p&p3, 10344p&p4, 12705p&p1 & 15125plan01.
4. Sanitary and storm sewer grades and inverts were compiled from: City of Ottawa Utility Plans.
5. A field location of underground plant by the pertinent utility authority is mandatory before any work involving breaking ground, probing, excavating etc.
6. Not all overhead wires/transformers adjacent to the property have been located, the nearest overhead wire locations are shown on the plan.

Notes & Legend

—□—	Denotes	Survey Monument Planted
—■—	"	Survey Monument Found
SIB	"	Standard Iron Bar
SSIB	"	Short Standard Iron Bar
IB	"	Iron Bar
CC	"	Cut Cross
CP	"	Concrete Pin
(Wit)	"	Witness
Meas	"	Measured
(P1)	"	Plan 4R-19937
(P2)	"	Plan by (AOG) dated December 18, 2006 (Job No. 7657-06)
○ MH-ST	"	Maintenance Hole (Storm)
○ MH-S	"	Maintenance Hole (Sanitary)
○ MH-T	"	Maintenance Hole (Traffic)
—— ST ——	"	Underground Storm Sewer
—— S ——	"	Underground Sanitary Sewer
—— W ——	"	Underground Water
—— T ——	"	Underground Traffic
—— B ——	"	Underground Bell
— — — —	"	Underground Street Lighting
OHW ——	"	Overhead Wires
○ UP	"	Utility Pole
○ AN	"	Anchor
○ LS	"	Light Standard
□CB	"	Catch Basin
□CBMH	"	Catch Basin Maintenance Hole
□CBI	"	Catch Basin Inlet
FH	"	Fire Hydrant
GM	"	Gas Meter
TB-C	"	Cable Terminal Box
TB	"	Unidentified Terminal Box
TSL	"	Traffic Light
○ B	"	Bollard
△ S	"	Sign
	"	Gate
Ø	"	Diameter
CLF	"	Chain Link Fence
Inv.	"	Invert
Inv(City)	"	Invert compiled from City of Ottawa Utility Plans
T/G	"	Top of Grate
TpFdn	"	Top of Foundation
C/L	"	Centreline
+65.00	"	Location of Elevations
+65.00	"	Top of Concrete Curb Elevation
——	"	Property Line
	"	Deciduous Tree - The Symbol shown denotes location and trunk diameter only. Size of its' root system/overhead canopy may be smaller/larger than the symbol size depicted on this plan.
	"	Coniferous Tree - The Symbol shown denotes location and trunk diameter only. Size of its' root system/overhead canopy may be smaller/larger than the symbol size depicted on this plan.

Site Area=7909.7 sq.m.

Surveyor's Certificate
I certify that :
1. This survey and plan are correct and in accordance with the Surveys Act, the Surveyors Act and the Regulations made under them.
2. The survey was completed on the 14th day of February, 2025.

March 10/2025
Date

Emad Alrefaai
Ontario Land Surveyor

This plan of survey relates to AOLS Plan Submission Form Number V-98731

TOPOGRAPHIC DATA WAS COLLECTED UNDER WINTER CONDITIONS. SNOW COVER AND ICE PRECLUDE DETERMINING LOCATION AND ELEVATION OF SOME TOPOGRAPHICAL DATA THAT IS OTHERWISE VISIBLE.

FILE No. : 16-25

FARLEY, SMITH & DENIS SURVEYING LTD.

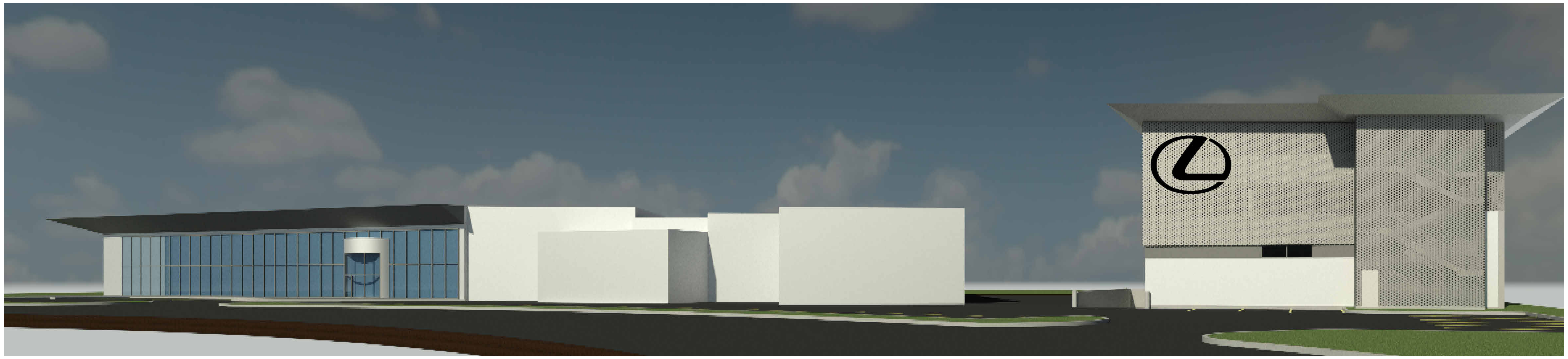
ONTARIO LAND SURVEYORS
CANADA LAND SURVEYORS

Unit 275, 30 COLONNADE ROAD, OTTAWA, ONTARIO K2E 7J6
TEL. (613) 727-8226 E-mail: info@fsdsurveys.ca

LEXUS DETAILING BAYS AND GARAGE BUILDING

299 WEST HUNT CLUB, OTTAWA, ON, K2E 1A6

TONY GRAHAM MOTORS INC.



PROJECT TEAM

CLIENT

TONY GRAHAM MOTORS INC.

A: 299 West Hunt Club, Ottawa, ON, K2E 1A6
T:

CONTRACTOR

PRITEC MANAGEMENT

A: 2405 March Rd, Carp, ON, K0A 1L0
T: 613 839 3462

ARCHITECT

VANDENBERG & WILDEBOER ARCHITECTS

A: 160 Flamborough Way, Ottawwa, ON, K2K 3H9
T: 613 287 0144

NO.	REVISION	DATE

CONSTRUCTION

Vandenberg & Wildeboer
A·R·C·H·I·T·E·C·T·S

www.vwarchitects.ca Telephone: 613.287.0144 Facsimile: 613.271.8609 mail@vwarchitects.ca
▪ THE OLD STONE LODGE ▪ 160 FLAMBOROUGH WAY ▪ OTTAWA (KANATA) ▪ ONTARIO ▪ K2K 3H9 ▪

PROJECT TITLE:
LEXUS DETAILING BAYS & GARAGE
299 West Hunt Club

DRAWING TITLE:
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DRAWN BY: Author
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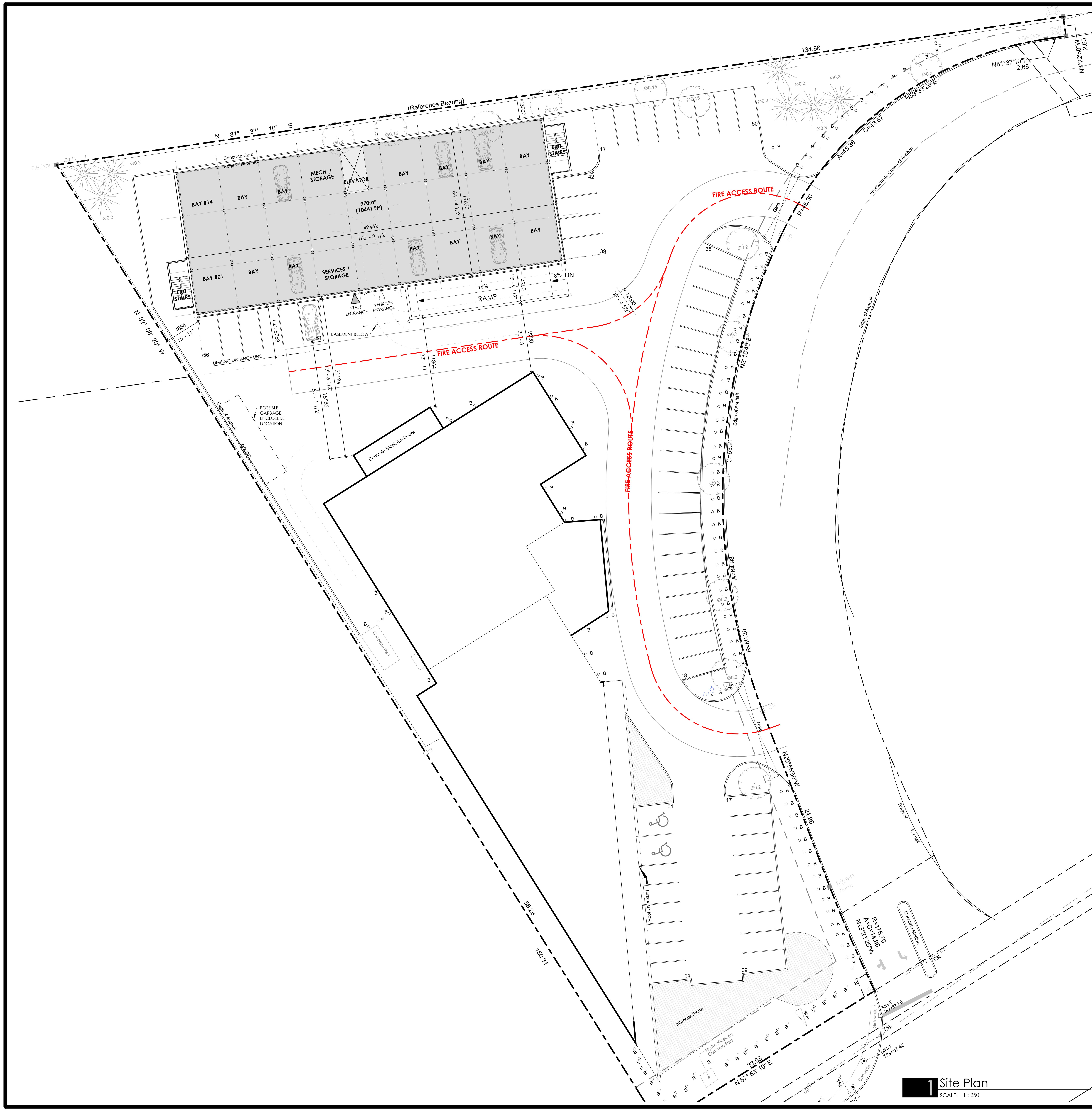

NO.	REVISION	DATE

CONSTRUCTION

Vandenberg & Wildeboer
A·R·C·H·I·T·E·C·T·S

www.vwarchitects.ca Telephone: 613.287.0144 Facsimile: 613.271.8609 mail@vwarchitects.ca
▪ THE OLD STONE LODGE ▪ 160 FLAMBOROUGH WAY ▪ OTTAWA (KANATA) ▪ ONTARIO ▪ K2K 3H9 ▪

PROJECT TITLE:
LEXUS DETAILING BAYS & GARAGE
299 West Hunt Club

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NO.	REVISION	DATE

CONSTRUCTION

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PROJECT TITLE:
LEXUS DETAILING BAYS & GARAGE
299 West Hunt Club

DRAWING TITLE:
ENLARGED SITE PLAN

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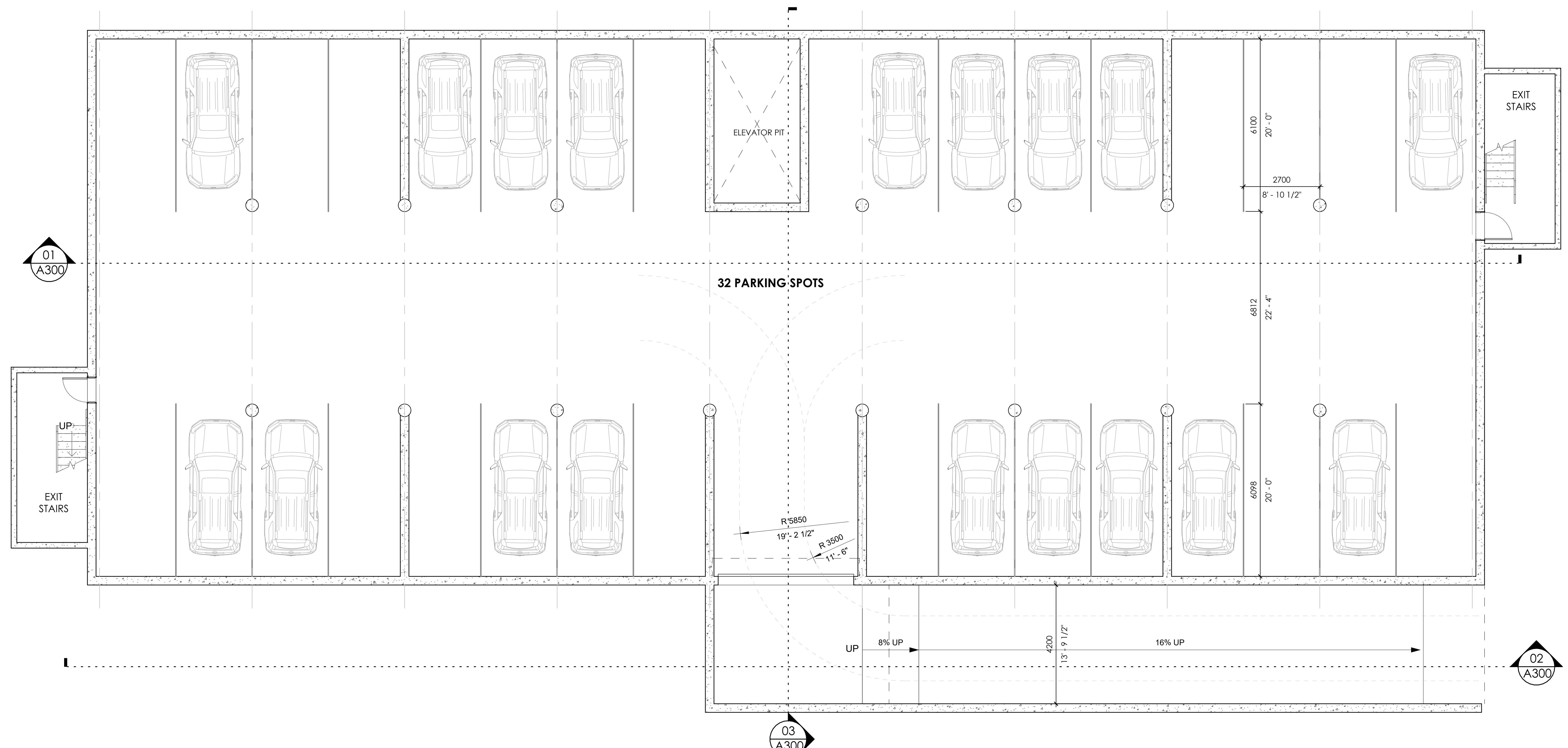

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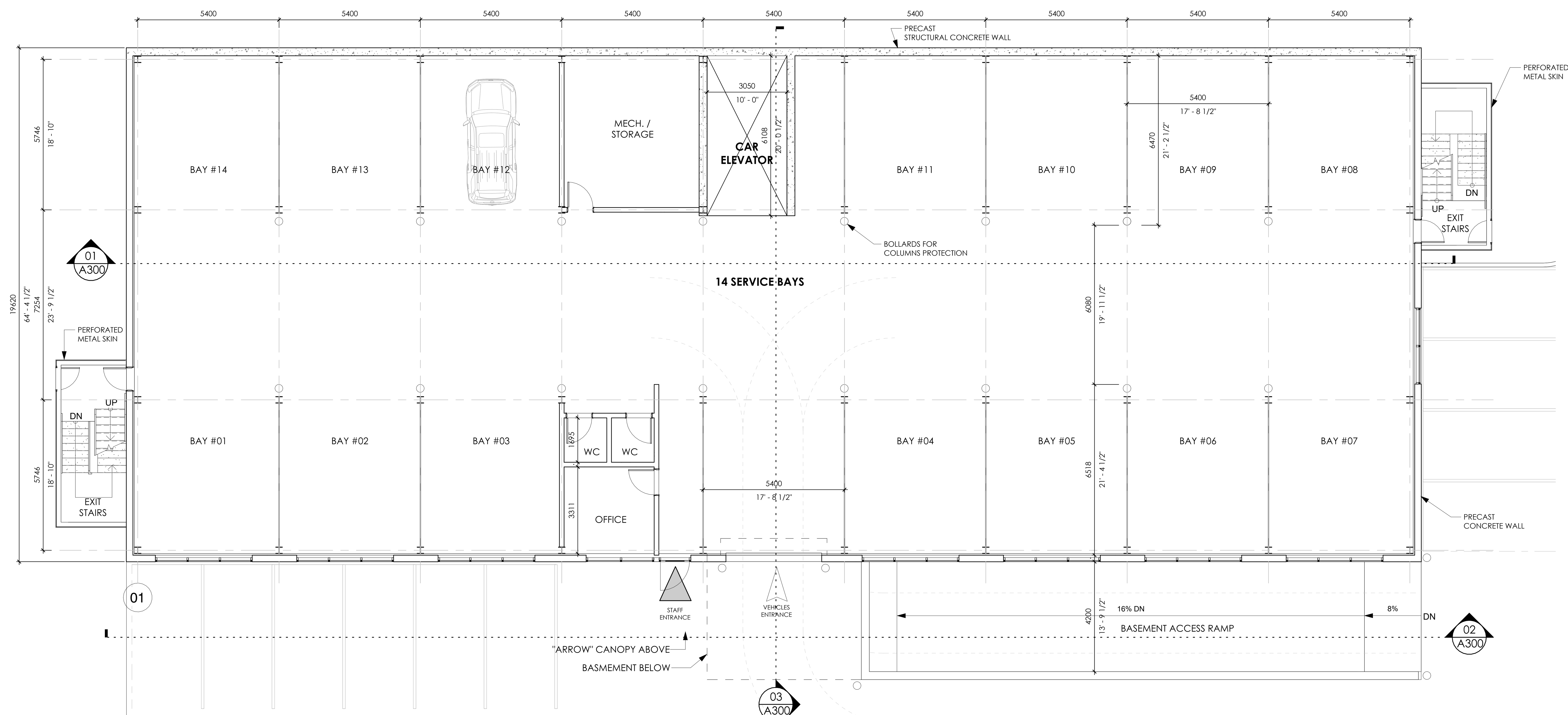

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NO.	REVISION	DATE

CONSTRUCTION

Vandenberg & Wildeboer
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PROJECT TITLE:
LEXUS DETAILING BAYS & GARAGE
299 West Hunt Club

DRAWING TITLE:
BASEMENT AND GROUND FLOOR PLANS

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DRAWN BY: Author
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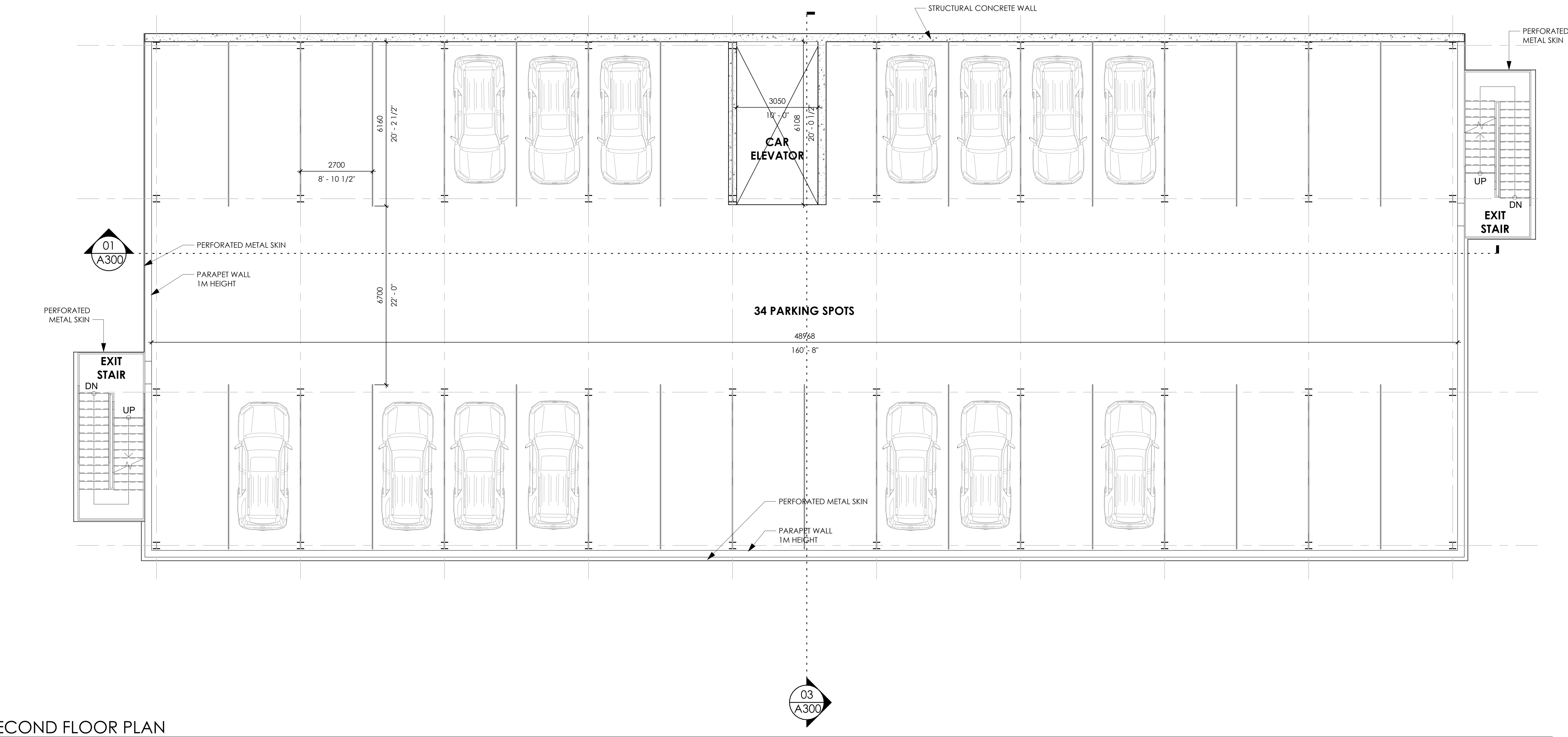

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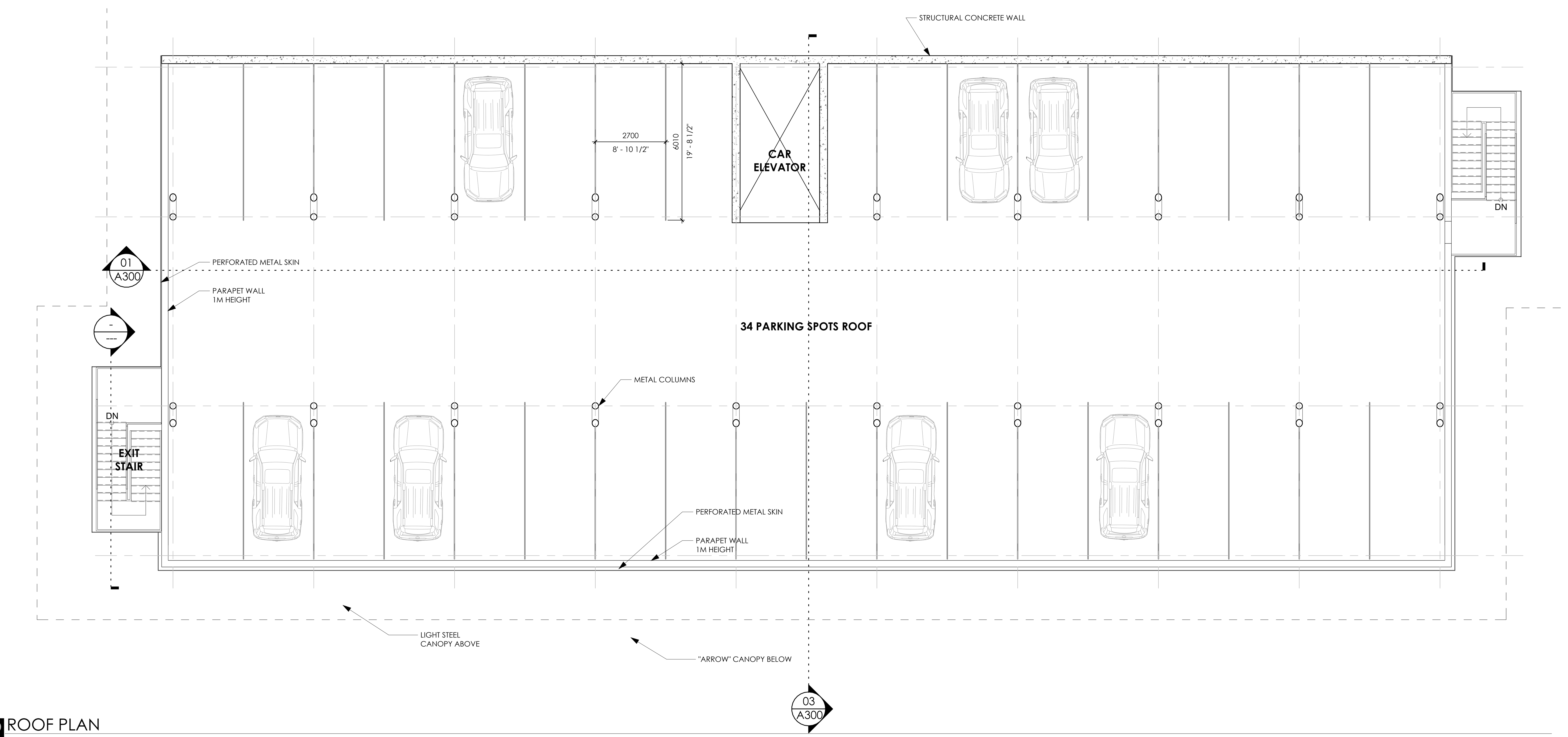

02 ROOF PLAN

SCALE: 1 : 100

NO.	REVISION	DATE

CONSTRUCTION

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A·R·C·H·I·T·E·C·T·S

www.vwarchitects.ca Telephone: 613.287.0144 Facsimile: 613.271.8609 mail@vwarchitects.ca
▪ THE OLD STONE LODGE ▪ 160 FLAMBOROUGH WAY ▪ OTTAWA (KANATA) ▪ ONTARIO ▪ K2K 3H9 ▪

PROJECT TITLE:
LEXUS DETAILING BAYS & GARAGE
299 West Hunt Club

DRAWING TITLE:
SECOND AND ROOF FLOOR PLANS

DESIGNED BY: Designer
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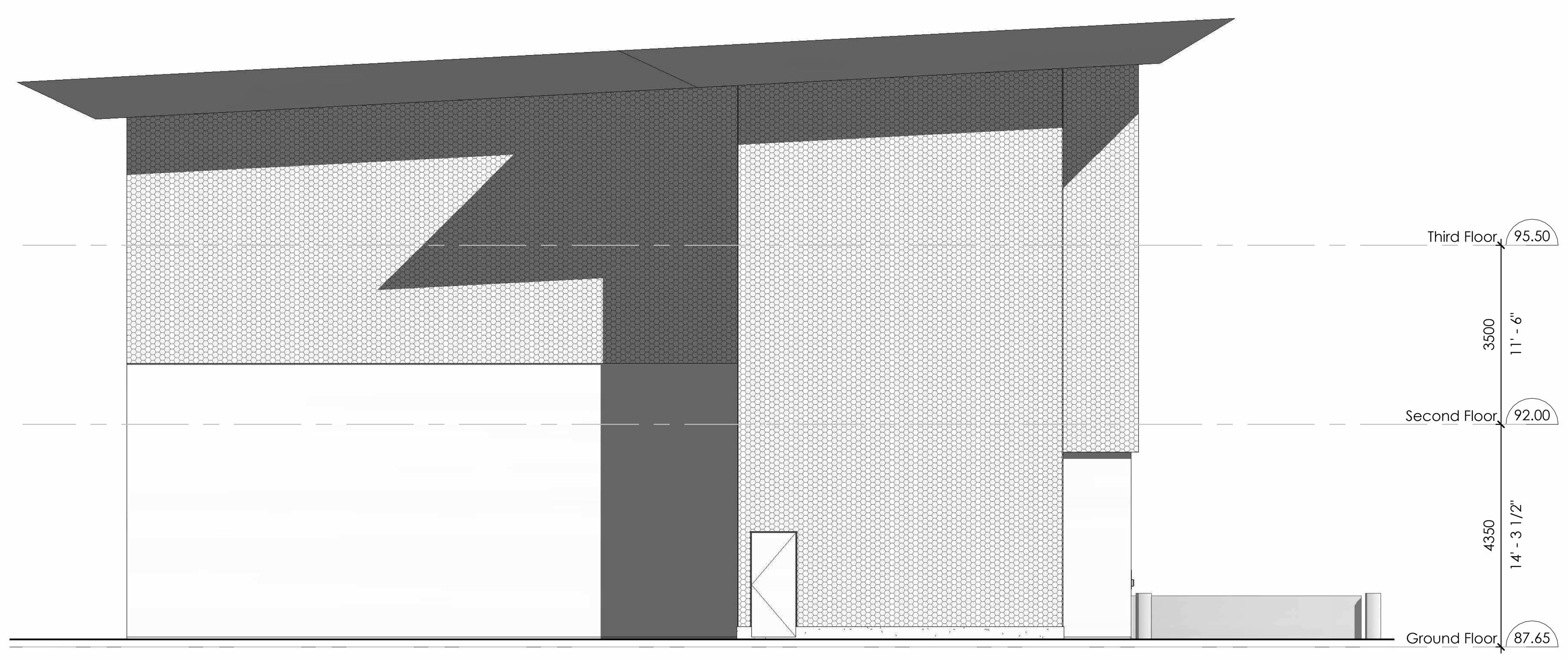

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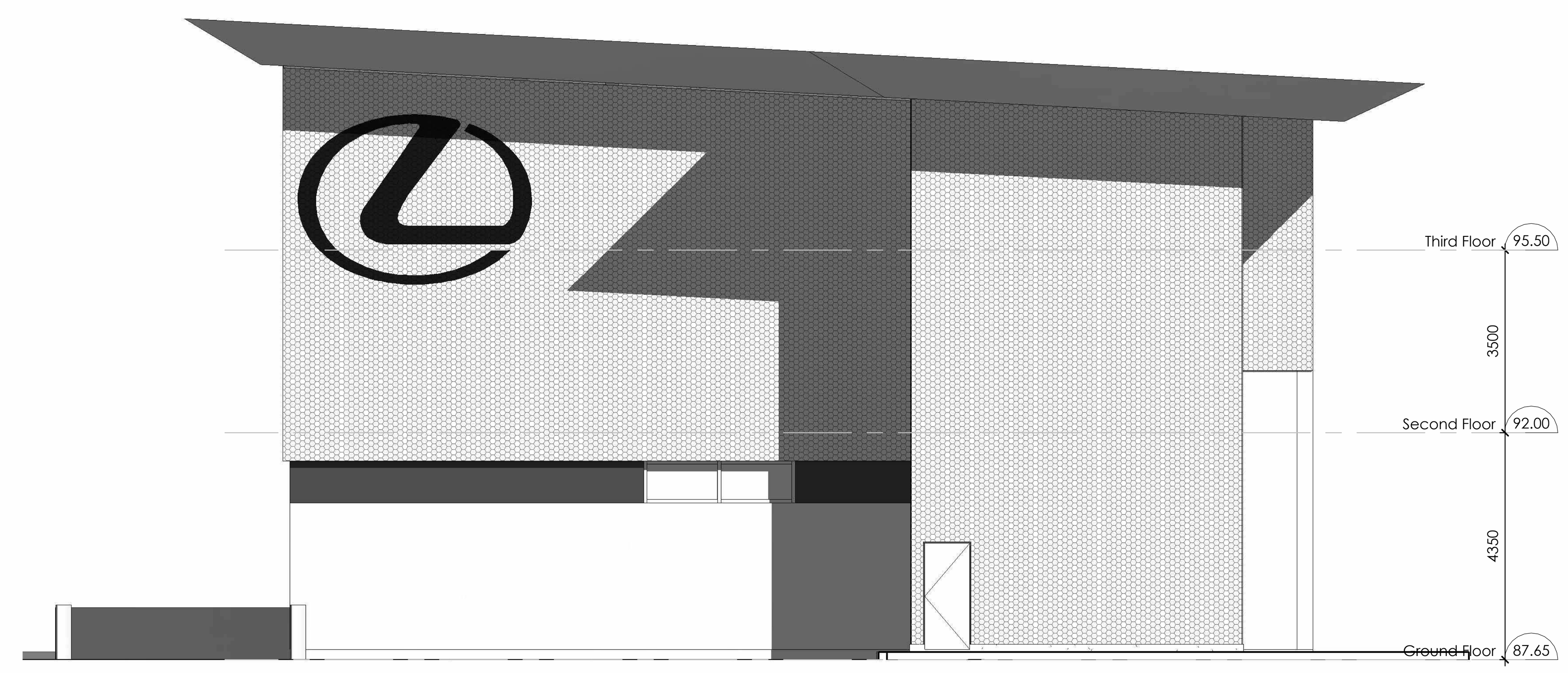

03 EAST ELEVATION
SCALE: 1 : 100

NO.	REVISION	DATE

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▪THE OLD STONE LODGE ▪ 160 FLAMBOROUGH WAY ▪ OTTAWA (KANATA) ▪ ONTARIO ▪ K2K 3H9▪

PROJECT TITLE:
LEXUS DETAILING BAYS & GARAGE
299 West Hunt Club

DRAWING TITLE:
ELEVATION

DESIGNED BY: Designer
DRAWN BY: Author
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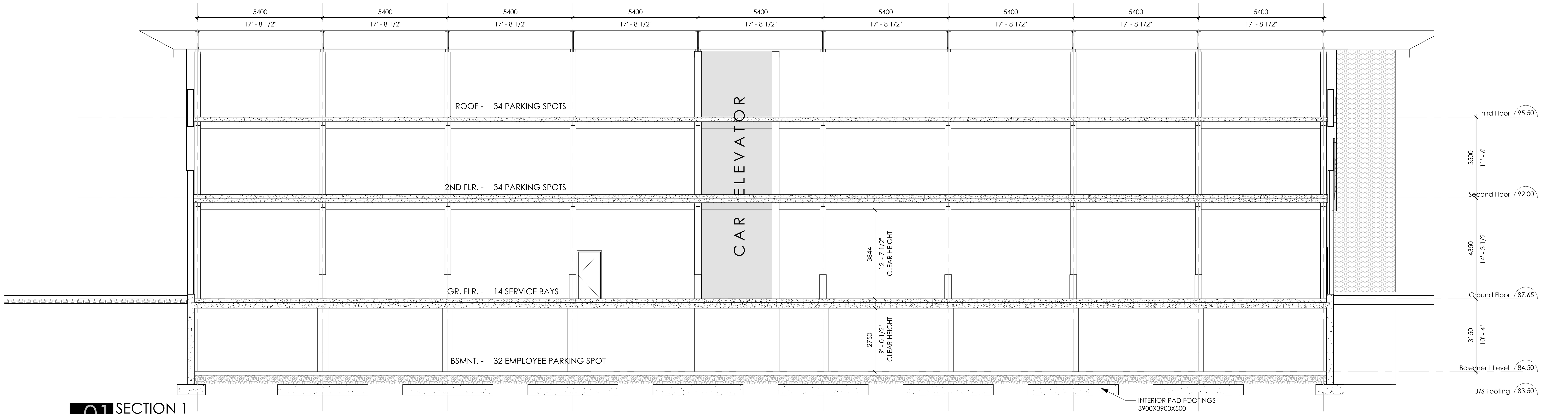

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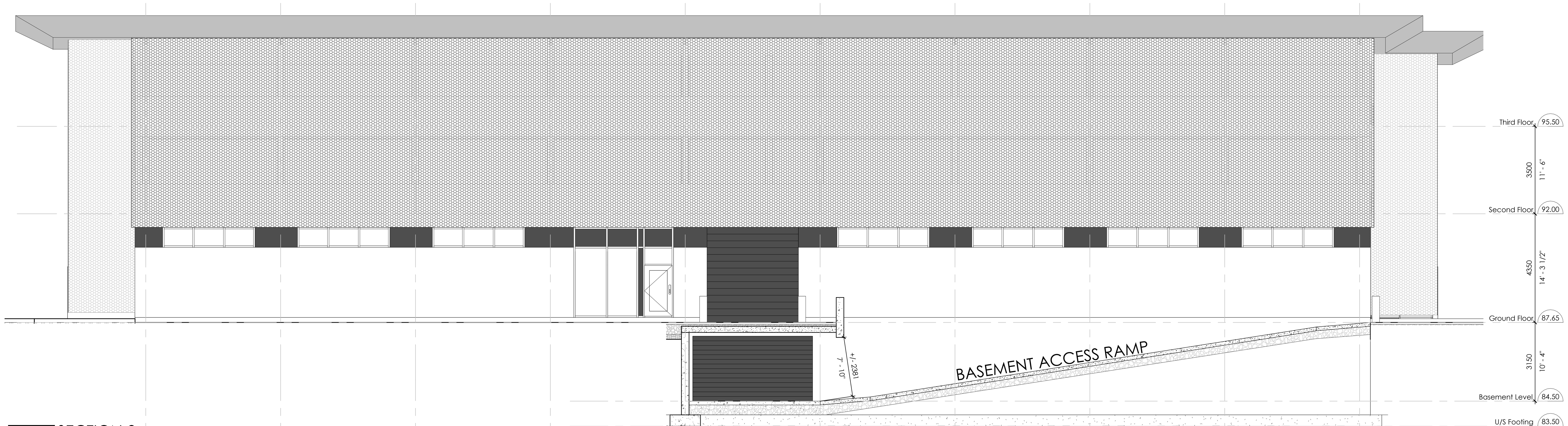

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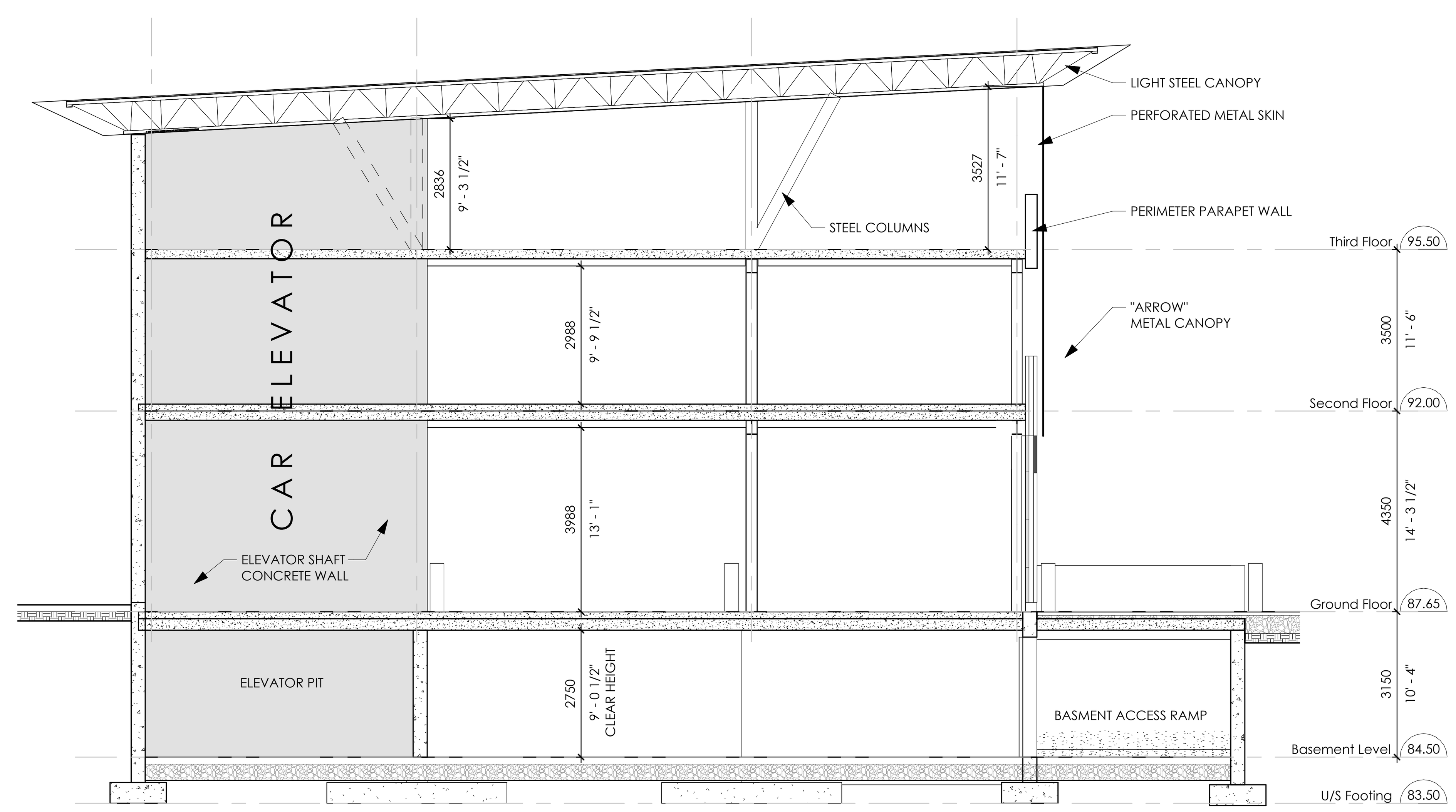

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SCALE: 1 : 100

NO.	REVISION	DATE

CONSTRUCTION

Vandenberg & Wildeboer
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www.vwarchitects.ca Telephone: 613.287.0144 Facsimile: 613.271.8609 mail@vwarchitects.ca
▪ THE OLD STONE LODGE ▪ 160 FLAMBOROUGH WAY ▪ OTTAWA (KANATA) ▪ ONTARIO ▪ K2K 3H9 ▪

PROJECT TITLE:
LEXUS DETAILING BAYS & GARAGE
299 West Hunt Club

DRAWING TITLE:
BUILDING SECTION

DESIGNED BY: Designer
DRAWN BY: Author
START DATE: Issue Date
SCALE: 1 : 100
PROJECT NO. Project Number

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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 buildings footprints will need to be shored in order to complete the construction of the basement level. 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 of the presence of the exisiting Rail Line. Installation of the shoring system is not anticipated to have an adverse impact on the Rail Line, nonetheless, a series of vibration monitoring devices are recommended to be installed to monitor vibrations. The vibration monitors would be remotely connected to permit real-time monitoring and a vibration monitoring program would be implemented as detailed in Section 3.1 - Vibration Monitoring and Control Plan.
Item B - Bedrock Blasting and Removal Program - Blasting of the bedrock is not anticipated for the proposed development. It is not expected that bedrock removal is required based on the current design concepts for the proposed development and the depth of bedrock at the site.	Structural damage of Confederation Line due to vibrations from blasting program.	Blasting is not anticipated as part of this development
Item C - Construction of Footings and Foundation Walls - The proposed building will include 1 basement level. Therefore, the footings will be placed over a soil bearing surface.	Building footing loading on adjacent Rail Line alignment, and excavation within the lateral support zone of the Rail Line.	Due to the distance between the proposed building and the Rail Line alignment , the zone of influence from the proposed footings will not intersect the rail line structure. Further, although the basement level for the proposed building will extend approximately 3.5 m below the existing ground surface, due to the approximate 15 m distance between the proposed building and rail line structure, the building excavation will not impact the lateral support zone of the Rail Line.

APPENDIX B

Geotechnical Investigation Report by Others

Report No. 25012 dated March 24, 2025

AllRock
Consulting Ltd

Geotechnical Investigation Report

299 West Hunt Club Road, Ottawa, Ontario

Prepared For:

Pritec Management
P.O Box 296
Carp, ON K0A 1L0

Date: March 24, 2025
AllRock FIle: 25012

AllRock Consulting Ltd.
174 Colonnade Road #35
Ottawa, ON K2E 7J5

Version 1.0

AllRock Consulting Ltd
174 Colonnade Road South,
Unit #35
Ottawa, ON K2E 7J5
1-844-440-7625
www.allrockconsulting.com

Geotechnical Investigation Report

Proposed Parking Garage – 299 West Hunt Club Road

Project No.: 25012

March 24, 2025

Prepared by:

Jeremy Milsom, B.Sc.
Geoscientist
613.608.4059
Jeremy.milsom@allrockconsulting.com

Reviewed by:

Greg Davidson, P.Eng.
President – Geotechnical Lead
343.542.2433
greg.davidson@allrockconsulting.com

Approved by:

Scott Allen, P.Eng.
President
416.452.8998
scott.allen@allrockconsulting.com

AllRock Consulting Ltd
174 Colonnade Road South,
Unit #35
Ottawa, ON K2E 7J5
1-844-440-7625
www.allrockconsulting.com

TABLE OF CONTENTS

1. INTRODUCTION 1

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Appendix A Record of Borehole Sheets

AllRock Consulting Ltd
174 Colonnade Road South,
Unit #35
Ottawa, ON K2E 7J5
1-844-440-7625
www.allrockconsulting.com

QUALITY CONTROL

Version No.	Date	Comments
1.0	March 24, 2025	Original Version

QUALITY MANAGEMENT

Issue/Revision	Version No. 1
Remarks	Issued for Draft
Date	March 24, 2025
Prepared By:	Jeremy Milsom, G.I.T
Signature:	
Check By:	Greg Davidson, P.Eng
Signature:	G. Davidson
Project No.	25012
Authorized By:	Scott Allen, P.Eng
Signature:	

AllRock Consulting Ltd
174 Colonnade Road South,
Unit #35
Ottawa, ON K2E 7J5
1-844-440-7625
www.allrockconsulting.com

1. INTRODUCTION

This report presents the results of a geotechnical investigation carried out for the proposed parking garage located at the Lexus Toyota dealership at 299 West Hunt Club Road in Ottawa, Ontario.

The purpose of the investigation was to identify the general subsurface conditions at the site by means of a limited number of boreholes and, based on the factual information obtained, to provide engineering guidelines on the geotechnical design aspects of the project, including construction considerations that could influence design decisions.

2. BACKGROUND

2.1 Project Description

It is understood that the proposed development includes the following aspects:

- A parking garage in the existing parking lot

2.2 Previous Reports

AllRock notes no previous investigations have been provided for review.

3. SUBSURFACE INVESTIGATION

3.1 Geotechnical Investigation

The field work for this investigation was carried out on the 25th of February 2025. At that time, three (3) boreholes, numbered BH1-25 to BH3-25, were advanced to depth of 8 meters below existing grade.

The borehole locations were selected and positioned on-site by AllRock. The field work was observed throughout by a member of our engineering staff who directed the drilling operations and logged the samples.

Following completion of the boreholes, the soil samples were returned to our laboratory for examination by a geotechnical / materials engineer. Selected samples were submitted for moisture content and grain size distribution testing.

The approximate locations of the boreholes are shown on the Borehole Location Plan, Figure 2. The results of the boreholes are provided on the Record of Boreholes Sheets in Appendix A. The results of the laboratory testing results are provided on the Record of Boreholes Sheets in Appendix B.

3.2 Methodology

Materials and soil description have been made with reference to the following documents:

- Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) – ASTM D2487-06
- Standard Practice for the Description and Identification of Soils (Visual-Manual Procedure) – ASTM D2488-06

4. SUBSURFACE CONDITIONS

4.1 General

As previously indicated, the soil and groundwater conditions identified in the boreholes are given on the Record of Borehole sheets in Appendix A. The logs indicate the subsurface conditions at the specific test locations only. Boundaries between zones on the logs are often not distinct but rather are transitional and have been interpreted. The precision with which subsurface conditions are indicated depends on the method of exploration, the frequency and recovery of samples, the method of sampling, and the uniformity of the subsurface conditions. Subsurface conditions at other than the borehole locations may vary from the conditions encountered in the boreholes.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves judgement and AllRock does not guarantee descriptions as exact but infers accuracy to the extent that is common in current geotechnical practice.

The groundwater conditions described in this report refer only to those observed at the place and time of observation noted in the report. It is noted that groundwater conditions can vary seasonally or as a result of construction activities in the area.

4.2 Subsurface Conditions

The following presents an overview of the subsurface conditions encountered in the borehole investigation

4.2.1 Asphalt Pavement

As all the boreholes were advanced in an existing parking lot, a layer of asphalt was encountered at all locations. The asphalt was found to have a thickness of approximately 0.15 – 0.25 meters.

4.2.2 Fill Material

A natural fill layer was encountered at all borehole locations below the surficial asphalt. Fill material can be an assortment of grain size and textures. At this site, the fill can be described as a brown, medium grained, and medium dense silty sand. The layer extended to a depth of 4.5 meters below ground surface at all borehole locations

4.2.3 Silty Clay

Below the natural fill, a native silty clay layer was encountered at all borehole locations. The layer was described as grey, soft, medium plasticity, inorganic and very moist. The layer extended to a depth of approximately 6 meters below ground surface at all borehole locations.

Standard penetration tests carried out in the native silty sand gave N values ranging from 0 (weight of hammer) to 25 blows per 0.3 metres of penetration, which reflects a very loose to dense relative consistency.

4.2.4 Silty Sand

Underlaying the clay, a silty sand layer was encounter at all borehole locations. The layer was described as brownish/grey fine grained, and medium dense. The layer extended to the termination depth of 8 meters below ground surface at all borehole locations.

Standard penetration tests carried out in the native silty sand gave N values ranging from 50 to 7 blows per 0.3 metres of penetration, which reflects a medium to dense relative consistency.

4.2.5 Gradation Analysis and Moisture Content

Laboratory results will be released with the final report.

4.2.6 Groundwater Level

A return trip to site to measure water levels was conducted on March 20th, 2025. The measure depth was 6.0 meters below ground surface.

5. RECOMMENDATIONS AND GUIDELINES

5.1 General

The information in the following sections is provided for the guidance of the design engineers and is intended for the design of this project only. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.
The professional services retained for this project include only the geotechnical aspects of the subsurface conditions.

The National Building Code of Canada 2020 Guidelines (hereafter NBCC 2020), the 2012 Ontario Building Code (OBC 2012) and the 4th edition of the Canadian Foundation Engineering Manual, 2006 (hereafter CFEM 2006) were considered for these recommendations. Based on the collected information from the boreholes advanced as part of this investigation, the geotechnical recommendations are presented in the following sections.

5.2 Proposed Site Development

5.2.1 Excavation

The excavation for the proposed dwellings will be carried out through asphalt, silty sand and silty clay. The sides of the excavation should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. According to the act, soils at this site can be classified as Type 3. That is, open cut excavations within overburden deposits should be carried out with side slopes of 1 horizontal to 1 vertical, or flatter. Where excavation side slopes cannot be accommodated due to space constraints, a shoring system may be required. Additional guidelines for the design and selection of a suitable shoring system could be provided as the design progresses.

In the event that a granular pad is necessary below the foundations, the excavations should be sized to accommodate a pad of imported granular material which extends at least 0.6 metres horizontally beyond the edge of the footings and down and out from this point at 1 horizontal to 1 vertical, or flatter.

Depending on construction methodology, it may be necessary to the lower the groundwater level in the native deposits to about 0.3 metres below the base of the excavation. Below the groundwater level, sloughing of the sandy overburden soils into the excavation should be anticipated, along with disturbance to the soils in the bottom of the excavation. Sloughing of the excavation side slopes below the groundwater level could be reduced, where necessary, by a shoring system installed along the sides of the excavation to below the level of the excavation in combination with pumping from within the excavation.

5.2.2 Groundwater and Pumping Management

Groundwater inflow, if any, from the overburden deposits should be controlled by pumping from filtered sumps within the excavation. It is not expected that short term pumping during excavation will have a significant effect on nearby structures and services. It is anticipated that groundwater inflow from the overburden deposits into the excavations could be handled from within the excavations.

It is noted that groundwater levels and surface water flows can increase during wet periods of the year such as the early spring or following periods of precipitation.

The groundwater handling should be carried out in accordance with provincial and local regulations. Suitable detention and filtration will be required before discharging water. The contractor should be required to submit an excavation and groundwater management plan for review.

5.2.3 Subgrade Preparation and Placement of Engineered Fill

Any existing topsoil, organic material, fill, and/or weathered/disturbed soil should be removed from below the proposed structures.

Imported granular material (engineered fill) should be used to raise the grade in areas where the proposed founding level is above the level of the native soil, or where sub-excavation of material is required below proposed founding level. The engineered fill should consist of granular material meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular B Type II and should be compacted in maximum 200-millimetre-thick lifts to at least 99 percent of the standard Proctor maximum dry density. To allow spread of load beneath the footings, the engineered fill should extend horizontally at least 0.6 metres beyond the footings and then down and out from the edges of the footings at 1 horizontal to 1 vertical, or flatter. The excavations should be sized to accommodate this fill placement.

It is noted that engineered fill in excess of 1 metre thick can be expected to experience post-construction settlement in the order of 0.5 to 1 precent of the height of the soil placed (depending on the composition of the engineered fill). It is anticipated that if engineered soil is sourced from the native onsite soils, it may take 2 to 4 months for the majority of post-construction settlement to occur; however, if imported granular fill as such as that meeting the (OPSS) requirements for Granular B Type II, settlement will likely occur within 1 to 2 weeks of placement.

5.2.4 Footing Design

In general, the silty clay and the native silty sand are considered suitable to support the proposed structures founded on spread footings The existing topsoil/organic material and fill material are not considered suitable for the support of the proposed development and should be removed from the proposed development areas.

For preliminary design purposes, footings founded on the silty clay or on a pad of compacted engineered fill above the silty clay layer (depth of 4.6 meters below ground surface) should be sized using a geotechnical reaction at Serviceability Limit State (SLS) of 75 kilopascals and a factored geotechnical resistance at Ultimate Limit State (ULS) of 112.5 kilopascals.

Alternatively, footings founded on the native silty sand or on a pad of compacted engineered fill above the silty sand layer (depth of 6.1 meters below ground surface) should be sized using a geotechnical reaction at Serviceability Limit State (SLS) of 100 kilopascals and a factored geotechnical resistance at Ultimate Limit State (ULS) of 150 kilopascals.

The post construction total and differential settlement of footings should be less than 25 and 15 millimetres respectively, provided that all loose or disturbed soil is removed from the bearing surface and provided that any engineered fill material is compacted to the required density.

5.3 Proposed Site Development

Due to the thickness of fill material and depth to native soil (4.6 meters below existing grade) the excavation will be sufficiently deep and require shoring for conventional foundations on strip or spread footings. As an alternative, other foundation options as provided in section in 5.2.6 and 5.2.7 discuss considerations for caissons and helical piles.

5.3.1 Caissons (Augered Piers)

If the bearing capacities provided with the spread footings are not suitable, caisson foundations can be used as an alternative.

For the design of foundations, the passive resistance within the upper 1.7 metres below ground surface should be neglected to account for frost action. The unfactored lateral resistance should be calculated assuming an equivalent width equal to three times the caisson diameter. A resistance factor of 0.5 should be applied to the unfactored lateral resistance to obtain the factored lateral geotechnical resistance at Ultimate Limit State (ULS).

In the case of cohesive soils, the capacity of the caisson should be checked to determine where the drained or undrained case will govern. In this case, the lateral resistance for the length of the caisson within the cohesive soil should be calculated assuming an unfactored passive lateral pressure distribution varying from 2 Cu at the surface to 9 Cu at and below a depth equivalent to three caisson diameters, acting over the actual width of the caisson. A resistance factor of 0.5 should be applied to this calculated lateral resistance in order to obtain the factored lateral geotechnical resistance at ULS.

The factored unit soil weight should be used below the groundwater level, where applicable.
For design the full passive resistance will be mobilized only where the width of soil in front and behind the caissons is equal to or greater than eight caisson diameters. If there is lesser width of soil from development of passive resistance (i.e., if there is sloping ground adjacent to the culvert), the magnitude of the passive resistance may be determined by interpolating between zero passive resistance at ground surface and full passive resistance at the depth where the slope face is greater than eight caisson diameter way from the face of the caisson.

Where caissons will be installed below the groundwater table or in loose non-cohesive soils, the caissons should be installed inside temporary steel liners driven ahead of the drill head to prevent fill and soft soils in the caisson holes to become unstable.

The founding soils could be susceptible to disturbance by augering; therefore, the bases of the augered caisson should inspected by geotechnical personnel to confirm they are located in native, undisturbed and competent bearing soils which has been cleaned of any ponded water and loosened materials prior to pouring concrete. Concrete for the caisson should be poured as soon as practicable after augering. The bearing soils and fresh concrete must be kept from freezing during cold weather construction.

The ultimate shaft uplift resistances of a caisson within non-cohesive soils can be determined from the following expression:

$$Q_s = 0.5\pi D \sum_{z=0}^{L} \beta\sigma\Delta z$$

Where:

- Q_s = ultimate uplift shaft capacity (kN)
- D = diameter of caisson (m)
- Δz = thickness of soil layer (m)
- z = depth (m)
- L = length of caisson (m)
- β = shaft resistance factor (0.2 from fill, 0.3 for native soils)
- σ = vertical effective stress adjacent to the pile at depth z

The uplifting resistance in the upper 1.8 metres should be neglected.

A resistance factor of 0.3 should be applied to the calculated uplift resistance in order to obtain the factored shaft uplift geotechnical resistance at ULS. The weight of the concrete caisson can be assumed as 14 kN/m3 below the groundwater table. An appropriate factor of safety should be utilized in the structural analysis of uplift resistances of caissons.

The axial (compressive) loading for caisson should be relatively small compared to the lateral and uplift loads and it is anticipated that the foundation design will be governed by lateral loading and uplift resistance. Cave-in should be anticipated in non-cohesive soils and below the groundwater level. Based on the size of the caisson, proper cleanup of the caisson bottom may not be practical. As a result, axial bearing resistance is mainly mobilized from shaft resistances of caissons.

A resistance factor of 0.4 should be applied to the calculated axial bearing resistance in order to obtain the factored axial geotechnical bearing resistance ULS. An appropriate factor of safety should be utilized in the structural analysis.

5.3.2 Helical Piles

It is understood that the proposed foundation design will use helical piles.

The depth of penetration and required design of helices (single or multiple) will depend on the soil conditions and design loads. The shaft diameters, wall thicknesses and welds need to be designed by a structural engineer to meet the required installation stresses and the expected geotechnical conditions.

It should be noted that helical piles are a proprietary foundation system and the helical pile resistances are highly dependent on the pile design geometry and method of installation. It is therefore, generally accepted industry practice that the Piling Contractor designs and warrants the helical piles for the specified ULS design loads. Varying helix diameters and configurations may be required based on the loading requirements. Where installed in groups, helical piles should not be installed at spacing closer than three times the largest helix diameter, centre to centre.

The ultimate capacity of the screw pile (Q_{ult}) with a single helix in native silty sand may be expressed as follows:

$Q_b = [(N_C \times C_u) + (\gamma' \times H)] \times [\pi \times (D_2 - d_2)/4]$

Where:

- N_c = bearing capacity factor
- Use 9 for D < 0.5 m.
- Use 7 for D between 0.5 to 0.9 m
- Use 6 for D > 0.9 m
- C_u = undrained shear strength (kPa) at depth of the helix plate
- D = helix plate diameter
- d = pile shaft diameter (where applicable)
- H = depth of helix below ground
- γ' = Effective unit weight (use 19 kN/m^3 above the water table, 10 kN/m^3 below
- the water table).

A resistance factor of 0.4 (compression) and 0.3 (tension) may be used to determine the factored ULS bearing resistance of the screw pile helix, Shaft friction should generally be ignored for small diameter shafts due to potential effects of disturbance and loss of shaft adhesion.

The undrained shear strength (s_u) of the soil up to the depth of investigation can be taken as 75kPa.

Piles should be founded with the upper helix at least one metre below the design frost depth which was given as 1.5 metres at this site. The pile designer should refer to the borehole logs and use the appropriate soil strength parameters for design. As noted above the piling contractor remains responsible for selection of appropriate pile design parameters and for design and installation of the piles.

It should be recognized that screw pile capacities are highly dependent on the pile design and method of installation. The installation method used to install helical piles can also cause significant soil disturbance (due to churning) and/or the development of voids around the pile shaft near the ground surface. The potential for disturbance may increase with multiple helix piles. This can have a significant impact on the lateral load deformation behavior of helical piles since good soil support in the upper few meters is critical for lateral support. Any voids formed around the pile shaft should be backfilled with sand or crushed gravel to maintain intimate contact between the pile shaft and the soil.

Installation of helical piles should be monitored by a geotechnical engineer, and the final torques should be recorded and used as a method of confirming the pile capacities.

Uplift loads can be resisted by the pile shaft and helices. Piles resisting uplift load should be installed at a minimum depth ratio (H/D) of 4, or at least 1 m below the frost depth of 1.8 metres, whichever is greater. The ultimate axial resistance should be multiplied by a GRF of 0.3 for piles subject to uplift loads. The upper 1.5 m of the pile shaft should be neglected when calculating the uplift resistance.

Screw piles should also be checked for frost uplift. An ultimate adhesion of 10 kPa may be used for a bare steel shaft within the frost depth. The resistance to frost action is provided by the helices and hence these need to be founded below the frost depth, as suggested above. A resistance factor of 0.8 may be applied to the ultimate helix capacity in resisting frost heave uplift forces.
It is recommended that the final screw pile design be reviewed by a geotechnical engineer. In addition, the structural capacity should be checked for the applied loading conditions.

Precautions should be taken to prevent heaving of the structure foundation due to frost penetration or seasonal moisture variation or swelling of the underlying soil. Adfreeze forces on the sides of pile and/or pile caps exposed to freezing should also be accounted for in foundation design.

5.3.3 Concrete Slab Support (only required for slab-on-grade)

Based on the results of the investigation, the area in the vicinity of the proposed structure is generally underlain by asphalt, fill material and native overburden deposits. The existing topsoil and fill material should be removed from the slab on grade areas. The grade below the concrete slabs on grade could be raised, where necessary, with granular material meeting NL DTI Specification book requirements for Granular B. The use of Granular B material is preferred under wet conditions. The granular base for the proposed slab on grade should consist of at least 150 millimeters of Granular A.

All imported granular materials placed below the proposed floor slab should be compacted in maximum 200-millimetre thick lifts to at least 99 percent of the standard Proctor maximum dry density value.

Proper moisture protection with a vapour retarder should be used for any slab on grade where the floor will be covered by moisture sensitive flooring material or where moisture sensitive equipment, products or environments will exist. The "Guide for Concrete Floor and Slab Construction", ACI 302.1R-04 should be considered for the design and construction of vapour retarders below the floor slab.

Underfloor drainage is not considered necessary provided that the floor slab level is above the finished exterior ground surface level.

Thermal protection of the concrete slab on grade is required in areas that will remain unheated during the winter period. The type of insulation used below the slabs will depend on the stresses imposed on the insulation. The stress on the insulation should not exceed about 35 percent of the insulation's quoted compressive strength due to the time dependent creep characteristics of this material. Further comments could be provided as the design progresses.

5.3.4 Frost Protection of Foundations

All exterior footings for heated buildings that consist of slab on grade construction or included basement should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated, unheated and/or exterior pier footings adjacent to surfaces which are cleaned of snow cover during the winter months should be provided with a minimum of 1.8 metres of earth cover. Alternatively, the required frost protection could be provided by means of a combination of earth cover and extruded polystyrene insulation. Further details regarding the insulation of foundations could be provided at the detailed design stage, if necessary.

5.4 Site Services

5.4.1 Excavation

Based on the investigation, the excavations for the services within the site will be carried out through asphalt, sub-base course and silty sand.

The sides of the excavations within overburden soils should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. According to the Act, the soils at this site can be classified as Type 3 soils. Therefore, for design purposes, allowance should be made for 1 horizontal to 1 vertical, or flatter, excavation slopes within the native soils at this site. As an alternative to sloping the excavations, all services installations could be carried out within a tightly fitting, braced steel trench box, which is specifically designed for this purpose.

The groundwater inflow should be controlled throughout the excavation and pipe laying operations by pumping from sumps within the excavation.

5.4.2 Groundwater Pumping

Possible groundwater inflow from the overburden deposits into the excavations could be controlled by pumping from filtered sumps within the excavations. It is not expected that short term pumping during excavation will have any significant affect on nearby structures and services. The groundwater handling should be carried out in accordance with provincial and local regulations. To reduce the groundwater pumping requirements, we suggest that the excavation be planned for the dry period of the year (i.e., June to September).

Suitable detention and filtration will be required before discharging water. The contractor should be required to submit an excavation and groundwater management plan for review.

5.4.3 Pipe Bedding and Cover

Based on the investigation, the excavations for the services within the site will be carried out through granular fill and silty clay.

The sides of the excavations within overburden soils should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. According to the Act, the soils at this site can be classified as Type 3 soils. Therefore, for design purposes, allowance should be made for 1 horizontal to 1 vertical, or flatter, excavation slopes within the native soils at this site. As an alternative to sloping the excavations, all services installations could be carried out within a tightly fitting, braced steel trench box, which is specifically designed for this purpose.

The groundwater inflow should be controlled throughout the excavation and pipe laying operations by pumping from sumps within the excavation.

5.4.4 Seismic Site Classification

According to Table 4.1.8.4.A of the NBCC 2020, Site Class D should be used for the seismic design of the structures bearing on native soils or on engineered fill material over native soils.

In our opinion the soils at this site are not considered to be liquefiable or collapsible under seismic loads.

6. ADDITIONAL CONSIDERATIONS

6.1 Effects of Construction Induced Vibration

Some of the construction operations (such as excavation, granular material compaction, etc.) will cause ground vibration on and off on the site. The vibrations will attenuate with distance from the source but may be felt at nearby structures. Assuming that any excavating is carried out in accordance with the guidelines in this report, the magnitude of the vibrations will be much less than that required to cause damage to the nearby structures or services in good condition but may be felt at the nearby structures.

6.2 Excess Soil Management Plan

This report does not constitute an excess soil management plan. The disposal requirements for excess soil from the site have not been assessed.

6.3 Design Review and Construction Observation

It is recommended that the final design drawings be reviewed by the geotechnical engineer to ensure that the guidelines provided in this report have been interpreted as intended.

The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed excavations do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design.

The subgrade surfaces for the proposed structures should be inspected by experienced geotechnical personnel to ensure that suitable materials have been reached and properly prepared. The placing and compaction of earth fill and imported granular materials should be inspected to ensure that the materials used conform to the grading and compaction specifications.

7. CLOSURE

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report, please do not hesitate to contact our office.

Jeremy Milsom, G.I.T.
Geoscientist
Jeremy.Milsom@allrockconsulting.com

G. Davidson

Greg Davidson, P.Eng.
President
greg.davidsonn@allrockconsulting.co

AllRock Consulting Ltd

174 Colonnade Road
Ottawa, Ontario K2E 7J5

Borehole Plan

Client No:	Job No: 25012
Client: Pritec	
Project: 299 Hunt Club	
Address: 299 West Hunt Club Road, Nepean, ON, Canada	

Legend:

- Borehole Locations
- Groundwater Monitoring Well Locations

Image Source: Google Maps		Viewed: 2025-03-12	
Drawn By: Jeremy Milsom	Checked By: Greg Davidson	Date: 2025-03-12	Figure: 1

AllRock Consulting Ltd
24 Brydon Drive, Unit #5
Toronto, ON M9W 5R6
1-844-440-7625
www.allrockconsulting.com

Appendix A

Record of Borehole Sheets

AllRock Consulting

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Geotechnical Log - Borehole

BH1-25

Latitude	: 45.34039	Drill Rig	: Truckmount Drill Rig	Job Number	: 25012
Longitude	: -75.71439	Driller Supplier	: Downing Drilling	Client	: Pritec
Ground Elevation	: 87.64 (m)	Logged By	: Jeremy Milsom	Project	: 299 Hunt Club
Total Depth	: 8m BGL	Reviewed By	: Greg Davidson	Location	: 299 West Hunt Club Road, Nepean, ON, Canada
		Date	: 25/02/2025	Loc Comment	:

Samples: SPT Sample	Blow Counts	Graphic Log	Elevation / Depth (m)	Material Description
SS1	50 (N=50blows/6Inches) R = 20		87.49 / 0.15	Pavement ASPHALT Fill material - medium grained silty sand, moist
SS2	9,11,10,12 (N=21) R = 100			
SS3	3,2,4,5 (N=6) R = 100			
SS4	WH,WH,1,1 (N=1) R = 24			
SS5	1,2,1,1 (N=3) R = 100			
SS6	WH,WH,WH,WH (N=2) R = 100		82.89 / 4.6	
SS7	WH,WH,1,1 (N=1) R = 24			Grey, very moist, firm, silty clay
SS8	1,1,1,23 (N=2) R = 20		76.79 / 6.1	
SS9	22,23,20,50 (N=43) R = 50			Silty Sand (SM): wet, grey/brown, dense,
SS10	7,3,15,15 (N=18) R = 60			

BH1-25 Terminated at 8m (Terminated)

Method
- EX excavator
- BH backhoe bucket
- NE natural exposure
- EE existing xcavation
- RP ripper

Water
- complete water less
- Water inflow
- water level
- Level during drilling
- P partial water loss
- N none encountered

USC Classification

GW	well graded gravels	SW	well graded sands
GP	poorly graded gravels	SP	poorly graded sands
GM	silty gravel	SM	silty sands
GC	clayey gravel	SC	clayey sands
ML	inorg silts low plastic	CL	inorg clay low plastic
MH	inorg clay high plastic	CI	inorg clay med plastic
OL	org silts low plastic	CH	inorg clay high plastic
OH	org sills high plastic	Pt	peat of high org soils

Consistency
- VS Very soft
- S Soft
- F Firm
- St Stiff
- VSt Very stiff
- H Hard

Density
- VL Very loose
- L Loose
- MD Medium dense
- D Dense
- VD Very dense

Moisture
- D Dry
- M Moist
- W Wet
- PL plastic limit
- LL liquid limit

Soil Samples
- B bulk
- D disturbed
- U(63) U(63) push tube
- U(50) U(50) push tube
- WS water

In Situ Testing
- PP pen penetrometer
- VS vane shear
- DCP dynamic cone penetrometer

Laboratory Results
- UC undrained unconsol cohesion
- UF undrained unconsol friction angle
- MC moisture content
- DD dry density
- LL liquid limit
- PL plastic limit
- LS linear shrinkage
- CC undrained console cohesion
- CF undrained console friction angle
- FH falling head permeability
- CH constan head permeablity
- CBR californian bearing ratio

AllRock Consulting

Geotechnical Log - Borehole

BH2-25

UTM	**: 18T**	**Drill Rig**	**: Truckmount Drill Rig**	**Job Number**	**: 25012**
Latitude	**: 45.34036**	**Driller Supplier**	**: Downing Drilling**	**Client**	**: Pritec**
Longitude	**: -75.71458**	**Logged By**	**: Jeremy Milsom**	**Project**	**: 299 Hunt Club**
Ground Elevation	**: 86.9 (ft)**	**Reviewed By**	**: Greg Davidson**	**Location**	**: 299 West Hunt Club Road, Nepean, ON, Canada**
Total Depth	**: 8m BGL**	**Date**	**: 25/02/2025**	**Loc Comment**	**:**

Samples: SPT Sample	Samples: Grab Sample	Blow Counts	Graphic Log	Elevation / Depth (m)	Material Description	Vane Data
				86.75 / 0.15	Pavement ASPHALT	
	GS1				Fill material - medium grained silty sand, moist	
SS1		18,12,11,5 (N=23) R = 100				
SS2		2,3,4,6 (N=7) R = 100				
SS3		1,2,3,4 (N=5) R = 100				
SS4		1,1,1,2 (N=2) R = 100				
SS5		1,1,1,2 (N=2) R = 100		82.15 / 4.6		
SS6		WH,WH,WH,WH (N=0) R = 100			Grey, very moist, firm, silty clay	
				76.05 / 6.1		SV 90 -
SS7		14,22,33,30 (N=>50) R = 60			Silty Sand (SM): wet, grey/brown, dense,	
SS8		7,13,11,11 (N=24) R = 60				

BH2-25 Terminated at 8m (Terminated)

AllRock Consulting

Phone:

Geotechnical Log - Borehole

BH3-25

UTM	**: 18T**	**Drill Rig**	**: Truckmount Drill Rig**	**Job Number**	**: 25012**
Latitude	**: 45.34035**	**Driller Supplier**	**: Downing Drilling**	**Client**	**: Pritec**
Longitude	**: -75.71484**	**Logged By**	**: Jeremy Milsom**	**Project**	**: 299 Hunt Club**
Ground Elevation	**: 87.1 (ft)**	**Reviewed By**	**: Greg Davidson**	**Location**	**: 299 West Hunt Club Road, Nepean, ON, Canada**
Total Depth	**: 8 m BGL**	**Date**	**: 25/02/2025**	**Loc Comment**	**:**

Elevation / Depth (m)	Material Description
86.95 / 0.15	Pavement ASPHALT
	Fill material - medium grained silty sand, moist
82.35 / 4.6	Grey, very moist, firm, silty clay
75.65 / 6.1	Silty Sand (SM): grey/brown, dense,

SPT Sample	Blow Counts
SS1	7,5,5,5 (N=10) R = 100
SS2	3,3,5,6 (N=8) R = 100
SS3	2,2,3,5 (N=5) R = 100
SS4	3,2,3,5 (N=5) R = 100
SS5	1,1,1,1 (N=2) R = 100
SS6	2,1,3,5 (N=4) R = 100
SS7	8,15,20,25 (N=35) R = 60
SS8	9,8,8,8 (N=16) R = 60
SS9	9,9,15,18 (N=24) R = 60

Graphic Log | Well Diagram | Water

Water: Standing

BH3-25 Terminated at 8m (Terminated)

APPENDIX C

Proximity Assessment

Prepared by Paterson Group

Report No. PG7560-LET.01 dated May 29, 2025

Consulting Engineers
9 Auriga Drive
Ottawa, Ontario
K2E 7T9
Tel: (613) 226-7381

Geotechnical Engineering
Environmental Engineering
Hydrogeology
Materials Testing
Building Science
Rural Development Design
Temporary Shoring Design
Retaining Wall Design
Noise and Vibration Studies

patersongroup.ca

May 29, 2025
PG7560-LET.01

Pritec Management
2755 Carp Road,
Carp, ON
K0A 1L0

Attention: **Taylor White**

Subject: **Proximity Assessment**
Proposed Development
299 West Hunt Club Road - Ottawa, ON

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 development with respect to the subject alignment of the rail line located approximately 15 m north of the site boundary.

1.0 Background Information

Based on current plans, it is understood that the proposed development will consist of a multi-storey parking structure with one basement level and an approximate footprint of 1015 m^2. The subject site is currently occupied by a single-storey commercial building with an associated driveway, parking and landscaped areas. The ground surface at the subject property is relatively flat and generally at grade with West Hunt Club Road. The property is surrounded by the Rail Line to the north, commercial properties to the east and west, and West Hunt Club Road to the south.

The site is situated at an elevation of approximately 87 m, with the Rail Line located about 15 m to the north of subject site.

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

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

2.0 Subsurface Conditions

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

- The overburden soils consist of an approximate 4.5 m thickness of silty sand fill, underlain by silty clay. A dense silty sand was observed underlying the silty clay at a depth of about 6 m.

- Based on available geological mapping, bedrock is present at approximate depths of 15 to 25 m.

Rail Line Location

Available information indicates that the rail line is located approximately 12 m from the north property line of the subject site. The ground surface at the Rail Line alignment is located at approximate geodetic elevation 88 m, while the foundation elevation of the proposed commercial building will below the rail alignment at an approximate geodetic elevation of 83.50 m. Therefore, a vertical differential of approximately 4.5 m is present between the founding levels of the two structures.

3.0 Construction Precautions and Recommendations

Influence of Proposed Development on Rail Line

From existing soil information and building design details, the proposed building will be founded on spread footings bearing on the undisturbed, silty clay or on a pad of compacted engineered fill above the silty clay layer, as recommeded by others. Consequently, lateral loads due to the building footings will be transferred directly to the clay within a conservative 1.5H:1V zone of influence from the outside face of the footing.

Further, based on GeoOttawa maps, the proposed building is located at least 15 m from the rail line railway. At this distance, no additional loads from the proposed building will be transferred to the Rail Line, ensuring the stability and integrity of the Rail Line.

Excavation and Temporary Shoring

The overburden along the northern perimeter of the proposed building footprint is expected to be temporarily shored with a solder pile and lagging system, as sufficient setback is likely not available for a sloped excavation on this side.

The overburden along the remaining perimeter of the proposed building footprint will need to be sloped or shored in order to complete the construction of the underground level.

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

Pre-Construction Survey

A pre-construction survey will be required for the rail line corridor structure. Any existing structures in the immediate area of the proposed building will also undergo a pre-construction survey as per standard construction practices, where temporary shoring installation is required.

Groundwater Control

Groundwater observations during the geotechnical investigation indicated groundwater levels at approximately 6 m below the existing ground surface. Further, due to the proposed underside footing bearing above the ground water table, dewatering is expected to be negligible and will not impact the Rail Line.

4.0 Conclusions and Recommendations

Based on the currently available information for the subject alignment of the proposed building and the existing subsurface information, the proposed building will not negatively impact the Rail 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, and field monitoring program as described in the application conditions.

We trust that this information satisfies your immediate request.

Best Regards,

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
May 29, 2025
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Deepak K Rajendran, E.I.T.

Scott S. Dennis, P.Eng.