FINAL REPORT

GEOTECHNICAL INVESTIGATION

PROPOSED RESIDENTIAL MIXED-USE BUILIDNG, 770-774 BRONSON AVENUE, OTTAWA, ONTARIO

Submitted to:

KTS Properties

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Submitted by:

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Distribution List

1 e-copy: KTS Properties

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1 INTRODUCTION

WSP Canada Inc. (WSP) was retained by KTS Properties (the "Client") to carry out a geotechnical desktop review of the previous geotechnical investigations completed at 770-774 Bronson Avenue in Ottawa, Ontario (hereinafter referred as the "Site") and provide a revised and updated geotechnical report combining all pertinent technical information.

This geotechnical report summarizes the factual results presented in the previous geotechnical investigations and associated laboratory testing, previous geotechnical letters and addendums, presents an interpretation of the available factual information, and provides geotechnical recommendations related to geotechnical design aspect of the project and construction considerations for the proposed development.

The geotechnical desktop review and the revision of the geotechnical investigation report were carried out in general accordance with the detailed scope of work outlined in WSP's proposal (Reference No. 2025CA462203), dated August 25, 2025, which was accepted by the Client by means of a purchase order number 10-270825 dated August 28, 2025.

1.1 Objective and Limitations

The purpose of this geotechnical report is to conduct a geotechnical desktop review of the previous geotechnical investigations, assess the implications of a proposed additional third underground level within the Phase 1 building footprint on the geotechnical design recommendations, and to compile the previous Geotechnical Report dated Jun 02, 2021, letter and addendum into one document and issue a revised and updated geotechnical report.

The reader is referred to the "*Important Information and Limitations of This Report*" in the Appendices, which forms an integral part of this document. Specifically, this report is intended for the Client and Designers of the facility. Third parties and Contractors referencing the report shall not rely on the report without the express written consent of WSP and the authors. This report shall not be represented or considered as a Geotechnical Baseline Report for bidders or contract administrators for construction.

1.2 Project Understanding and Scope of Work

It is understood that two geotechnical investigations and a Phase II Environmental Site Assessment (ESA) were carried out in the past for different parts of the site. Since then, there have been a number of design changes that yielded the proposed development plan, including the building height and footprint as well as the number of basement levels. An updated geotechnical report was therefore produced on June 02, 2021 in support of the revised design requirements and for a construction permit for the new proposed building.

Originally, the Client planned to construct a mixed-use residential building in two phases which included:

- Phase 1 of the development located on the eastern half of the site. Phase 1 was planned to be a student residence building that varies from 9 to 26 storeys in height.
- Phase 2 of the development located on the western half of the site consisting of a 9-sotrey residential building.
- The proposed development was planned to include two levels of underground parking across the entire building footprint.

- The ground floor for the proposed building will be at Elevation 75.38 m. The finished floor for the lower parking garage was planned to be at approximately Elevation 68.9 m.
- The development will also consist of outdoor surface parking and landscaping areas.

It is WSP's understanding that the Client is considering adding an additional third underground level to the Phase 1 building. The final floor slab level for the proposed new third parking level will be approximately 9.6 m depth (i.e., 65.78 m elevation). Therefore, an updated geotechnical assessment has been prepared for the proposed additional parking level in order to address requirements for excavations, temporary and permanent dewatering, and foundation design.

WSP understanding of the project is based on the email request dated July 10, 2025 and the following documents:

- "Geotechnical Investigation - Proposed Residential Mixed-Use Building, 770-774 Bronson Avenue, Ottawa, Ontario", dated June 02, 2021 (Project No. 211-05706-00) by WSP Canada Inc.
- "Geotechnical Letter – City of Ottawa Comments – Geotechnical and Hydrogeological Investigation – New Proposed 22 Storey Building Located at 770-774 Bronson Avenue, Ottawa, Ontario", dated July 21, 2023 (Project No. CA0007991.6340) by WSP Canada Inc.
- "Addendum 1 – Geotechnical Investigation - Proposed Residential Mixed-Use Building, 770-774 Bronson Avenue, Ottawa, ON", dated July 21, 2023 (Project No. CA0007991.6340) by WSP Canada Inc.
- Site Plan number A105 for the proposed development, dated October 09, 2020, by LRL Engineering.
- Cross-section Plan numbers A.200 to A.203 for the proposed development, dated February 01, 2021, by LRL Engineering.
- Building Plan numbers A.118, A.119, A.120, A.121, A.122, A.125, A.128, A.130, A.131, and A.143 for the proposed development, dated February 03, 2022, by LRL Engineering.

WSP's scope of work included reviewing the available geotechnical information and the supplied plans for the proposed development and revising the existing Geotechnical Report to address the proposed third underground parking level and incorporate the information contained in the previous geotechnical letter and addendum into one report.

It must be noted that due to the proposed excavation depth to accommodate the three underground parking levels, intense construction dewatering may be encountered. A hydrogeological assessment is recommended to address the need for Permit to Take Water (PTTW). The current WSP's scope of work does not include a provision for any hydrogeological assessment. WSP can conduct a hydrogeological assessment for the Site at the Client request.

1.3 Site Geology

Based on the physiography and surficial geology maps of Ontario published by Ontario Geological Survey (OGS), the Site is located within Ottawa Valley Clay plains. Surficial geology maps indicate that the site is located within both exposed Paleozoic bedrock and till deposits comprising of stone-poor, sandy silt to silty sand-textured till overlying Paleozoic bedrock.

The bedrock at the project site is expected to consist of limestone of the Shallow Lake Formation.

2 DESKTOP REVIEW

WSP carried out a desktop review of the previous geotechnical and environmental investigations completed within the project area. The results of those previous investigations are summarized in the following reports:

- “Phase I and Limited Phase II Environmental Site Assessment, Existing Office/Commercial/Residential Buildings 551, 553, 555, 557 Cambridge Street South, 774, 780, 782, 784 Bronson Avenue, Ottawa Ontario”, dated May 18, 1999 (Ref. E1738-1) by John D. Patterson and Associates.
- “Geotechnical Investigation, Proposed Residential Development, 770 Bronson Avenue, Ottawa, Ontario” dated August 2015 (Project No. 1525987-02) by Golder Associates Ltd.
- “Updated Geotechnical Study, Projected New Building at 774 Bronson Ave., Ottawa, ON” Draft Report dated November 2015 (Project No. 151-12490-00) by WSP Canada Inc.
- “Updated Geotechnical Study, Projected New Building at 774 Bronson Ave., Ottawa, ON” Final Report dated February 2016 (Project No. 151-12490-00) by WSP Canada Inc.
- “Phase Two Environmental Site Assessment, 774 Bronson Avenue and 557 Cambridge Street South, Ottawa, Ontario”, dated March 29, 2016 (Project No. 151-13503-00) by WSP Canada Inc.
- “Geotechnical Investigation, Proposed Residential Development, 770 Bronson Avenue, Ottawa, Ontario” dated August 2015 (Project No. 1525987-02) by Golder Associates Ltd.
- “Geotechnical Investigation - Proposed Residential Mixed-Use Building, 770-774 Bronson Avenue, Ottawa, Ontario”, dated June 02, 2021 (Project No. 211-05706-00) by WSP Canada Inc.
- “Geotechnical Letter – City of Ottawa Comments – Geotechnical and Hydrogeological Investigation – New Proposed 22 Storey Building Located at 770-774 Bronson Avenue, Ottawa, Ontario”, dated July 21, 2023 (Project No. CA0007991.6340) by WSP Canada Inc.
- “Addendum 1 – Geotechnical Investigation - Proposed Residential Mixed-Use Building, 770-774 Bronson Avenue, Ottawa, ON”, dated July 21, 2023 (Project No. CA0007991.6340) by WSP Canada Inc.
- Site Plan number A105 for the proposed development, dated October 09, 2020, by LRL Engineering.
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Based on a review of the above-noted previous investigations, the proposed excavation for the underground parking levels will extend to a water bearing zone in the upper bedrock. Analysis of the reported bedrock quality shows the upper bedrock zone to be more fractured beneath the 774 Bronson Avenue property and more intact beneath the 770 Bronson Avenue property. Based on the available information from the previous reports, it is WSP’s opinion that a desktop study is sufficient, at this stage, to submit an updated geotechnical report in view of the new proposed construction. The details of the previous geotechnical investigations completed at this site are summarized in the sections below.

It should be noted that the 2015 Golder report provided preliminary estimates of expected short and long-term water infiltration into the future foundation excavation (which was based on up to 11 m below ground surface) using assumed hydraulic parameters; however, no in-situ hydraulic conductivity was conducted nor was there any water quality analysis performed on the raw water contained in the water bearing zone for comparative analysis to the City of Ottawa Sewer Use Bylaw (Bylaw No. 2003-514). A hydrogeological study will therefore be required to address the gaps in the groundwater quantity and quality data. Such work will be necessary to assist the design of foundations and drainage system, as well as to evaluate the requirements and impacts of construction dewatering and the long-term groundwater management.

3 PREVIOUS INVESTIGATIONS

3.1 Previous Geotechnical Investigation

Previous subsurface investigations were carried out across the site. The locations of the previous boreholes are shown in Drawing 2. The borehole logs from those previous investigations are presented in Appendix A.

In December, 2011, WSP carried out a geotechnical investigation on the southern portion of the site, which included the drilling of five boreholes (FE-1-2011, FE-2-2011 and FG-1-2011 thru FG-3-2011). The boreholes were advanced using a truck-mounted CME-55 drill-rig, equipped with hollow stem auger and split spoon sampling equipment, supplied and operated by Forage André Roy Inc. of Saint-Isidore, Québec.

- FE-1-2011 and FE-2-2011 were advanced to depths of approximately 2.2 and 0.9 m below the ground surface (mbgs) (El. 72.8 to 74.1 m), respectively.
- FG-1-2011 thru FG-3-2011 were advanced to auger refusal, which ranged from depths of 0.8 mbgs (El. 73.8 m) to 1.1 mbgs (El. 74.1 m). Upon encountering auger refusal, the boreholes were extended into the bedrock using NQ sized coring equipment to final depths ranging from 4.1 mbgs (El. 70.5 m) to 4.7 mbgs (El. 70.8 m).
- Monitoring wells were installed in FG-1-2011 through FG-3-2011 to permit subsequent groundwater level measurement.

In the March and June 2015 Golder geotechnical investigation, five boreholes (15-1 to 15-5) were advanced at the northeastern portion of the site. The boreholes were advanced using a truck-mounted drill rig, equipped with hollow stem augers, supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario.

- Boreholes 15-1 through 15-5 were advanced to refusal at depths ranging from 2.4 mbgs (El. 73.4 m) to 3.1 mbgs (El. 72.6 m). Upon encountering auger refusal, the boreholes were extended into the bedrock to final depths ranging from about 5.6 mbgs (El. 70.2 m) to 15.3 mbgs (El. 60.2 m) using rotary diamond drilling equipment while retrieving NQ or HQ sized bedrock cores.
- Monitoring wells were installed in all of the boreholes to permit subsequent groundwater level measurement.

In the January 2016 WSP environmental investigation, a total of seven boreholes were advanced in the southern portion of the site. The additional boreholes were identified as BH15-1 to BH15-6, BH15-3A and BH15-3B. The boreholes were advanced using a track-mounted CME-55 drill-rig supplied and operated by Downing Estate Drilling Ltd. of Grenville-sur-la Rouge, Quebec.

- BH15-1 to BH15-6, BH15-3A and BH15-3B were advanced to depths of 0.8 mbgs (El. 74.8 m) to 2.2 mbgs (El. 73.3 m).

- Upon encountering auger refusal, four of the boreholes (BH15-2, BH15-3B, BH15-4 and BH15-6) were advanced into the underlying bedrock using HQ sized coring equipment to final depths ranging from 7.4 mbgs (El. 67.1 m) to 8.0 mbgs (El. 65.8 m).
- Eight monitoring wells were installed at four borehole locations, with the shallow wells identified as BH15 2A, BH15-3A, BH15-4A and BH15-6A and the deeper wells identified as BH15-2B, BH15-3B, BH15-4B, and BH15-6B.

In the 2021 WSP geotechnical investigation report, WSP carried out a desktop review of the previous geotechnical and environmental investigations completed within the project area. The report compiled the results of the above-listed investigations and provided design recommendations for the proposed development.

A geotechnical letter and an addendum were issued in 2023 by WSP to address City of Ottawa comments related to the 2021 geotechnical report and to provide additional recommendations for a proposed cistern raft foundation.

The ground surface elevation and location of each borehole was surveyed and referenced to geodetic datum, except for FE-1-2011 and FE-2-2011 where the elevations were approximated based on Ontario Topographic Map by Ministry of Natural Resources and other boreholes within the Site. The ground surface elevation and depth of the various boreholes advanced during the previous investigations are summarized in Table 1 below.

Table 1: Ground Surface Elevation and Depth of Boreholes from Previous Investigations

Borehole Number	Location	Ground Surface El. (m)	Borehole Depth (mbgs)	Bottom of Borehole El. (m)
FE-1-2011	Southern Portion of Site	75.0$^{(1)}$	2.2	72.8
FE-2-2011	South (Outside) of Site Limit	75.0$^{(1)}$	0.9	74.1
FG-1-2011	Southern Portion of Site	75.1	4.2	70.9
FG-2-2011	Southern Portion of Site	75.5	4.7	70.8
FG-3-2011	Southern Portion of Site	74.6	4.1	70.5
15-1	Northeastern Portion of Site	75.9	5.6	70.2
15-2	Northeastern Portion of Site	75.7	5.9	69.8
15-3	Northeastern Portion of Site	75.8	5.9	69.9
15-4	Northeastern Portion of Site	75.6	6.0	69.6
15-5	Northeastern Portion of Site	75.5	15.3	60.2
BH15-1	Southern Portion of Site	75.0	2.1	72.9
BH15-2	Southern Portion of Site	75.6	7.8	67.8
BH15-3A	Southern Portion of Site	75.5	2.6	72.9
BH15-3B	Southern Portion of Site	75.5	7.9	67.6
BH15-4	Southern Portion of Site	74.5	7.4	67.1
BH15-5	Southern Portion of Site	74.7	1.4	73.3
BH15-6	Southern Portion of Site	73.7	8.0	65.7

Note: $^{(1)}$ Ground surface elevation was approximated.

3.1.1 Subsurface Conditions

The following provides a general description of the major soil and bedrock types reported in the various previous geotechnical and environmental investigations. It should be noted that the following discussion includes some simplifications for the purposes of discussing broadly similar soil strata. Any discrepancy between the generalized (and simplified) soil description presented below and on the borehole logs, the factual descriptions on the borehole logs shall take precedence.

The stratigraphic boundaries shown on the borehole records are inferred from observations of drilling progress and non-continuous soil sampling the very location of these holes. Therefore, they represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

A detailed description of the soil and bedrock stratigraphy encountered at each borehole location is shown on the borehole logs provided in Appendix A. The soil and bedrock stratigraphy are shown on the profiles on Drawings 3A and 3B.

3.1.1.1 Topsoil

A surficial topsoil layer was reported in three of the boreholes (BH15-3A, BH15-5B and BH15-5) advanced during the WSP's 2016 environmental investigation. At the borehole locations, the topsoil was approximately 120 to 150 mm thick.

3.1.1.2 Pavement Structure and Fill

Pavement structure was reported at boreholes FE-1-2011, FE-2-2011, FG-1-2011, FG 3 2011 from WSP's 2011 investigation, boreholes 15-1 through 15-5 from Golder's 2015 investigation, and boreholes BH15-1 and BH15-2 from WSP's 2015 investigation.

Golder's 2015 boreholes were drilled within the existing parking lot at the northeastern portion of the site. At the borehole locations, the pavement structure consisted of 100 mm of asphaltic concrete, overlying 150 to 210 mm of gravelly sand granular base (at boreholes 15-4 and 15-5) while the granular base was not identified in the remaining boreholes. The pavement structure was in turn underlain by a layer of sand and gravel fill, containing cobbles and organic matter. The fill extended to depths of about 2.4 mbgs (El. 73.4 m) to 3.1 mbgs (El. 72.6 m).

WSP's 2011 and 2016 investigations were advanced at the southern portion of the site. At the borehole locations (FE-1-2011, FE-1-2011, FG-1-2011, FG-3-2011, BH15-1 and BH15-2), the pavement structure, where encountered, consisted of 20 to 50 mm of asphaltic concrete, overlying 100 to 350 mm of sand and gravel granular base (except at FG-1-2011 where no granular base was identified). At FE-1-2011, the granular base was underlain by 820 mm of sand granular subbase, which was not encountered at the remaining boreholes.

Fill was encountered at all of WSP's boreholes either beneath topsoil, pavement structure or at the ground surface. The fill consisted of a heterogenous mixture ranging from sandy silt, silt and sand, silty sand, sand, to sand and gravel, with varying amounts of gravel, organic matter and construction debris (e.g. pieces of brick, asphalt, wood, black carbon ashes). The fill extended to depths of about 0.5 mbgs (El. 74.6 m) to 2.2 mbgs (El. 73.3 m).

Standard penetration tests (SPTs) carried out within the pavement structure and fill measured 'N' values ranging widely from 3 to greater than 50 blows per 0.3 m of penetration, indicating a very loose to very dense state of packing. Some of the higher blow counts towards the lower portion of the overburden likely reflect the bedrock surface rather than the state of packing of the soil matrix.

It should be noted that the thickness of pavement structure and fill was based on the results of the previous investigations and may have been altered as a result of site activities since the investigations were completed.

3.1.1.3 Native Sandy and Gravelly Soils

A thin deposit of sandy and gravelly silt was reported beneath the fill at borehole FE-1-2011. The deposit was approximately 0.4 m thick and extended to a depth of 2.2 mbgs (El. 72.8 m) prior to encountering sampler refusal. The deposit was described as a probable compact glacial till.

In Golder's borehole 15-5, a deposit of silty sand was reported below the fill. The silty sand deposit was approximately 0.3 m thick and contained a trace of gravel as well as organic matter, extending to a depth of 2.6 mbgs (El. 72.9 m). One SPT 'N' value of greater than 50 blows per 0.3 m of penetration was measured within the silty sand. However, this high blow count likely reflects the presence of the bedrock surface rather than the state of packing of the soil matrix.

At boreholes BH15-1 and BH15-5 (WSP's 2016 investigation), a gravel layer was encountered beneath the fill. The gravel layer was approximately 0.2 m thick, containing sand and shale fragments, and extended to depths of 1.4 mbgs (El. 73.6 m) and 0.9 mbgs (El. 73.8 m), respectively.

3.1.1.4 Bedrock

3.1.1.4.1 Weathered Bedrock

Weathered limestone was encountered below the fill or gravel layer at approximately 0.5 to 2.2 m below the ground surface (El. 73.3 to 74.8 m) in WSP's boreholes FG-1-2011, BH15-1, BH15-2, BH15-3A, BH15-4 and BH15-5. Hollow stem augers were able to penetrate past this upper portion of bedrock. The weathered zone is estimated to be approximately 0.3 to 0.7 m in thickness prior to encountering refusal to augering.

No weathered bedrock was identified within any of Golder's 2015 boreholes advanced at the northeastern portion of the site.

The depths and elevations of the weathered bedrock surface are summarized in Table 2.

Table 2: Weathered Bedrock Surface Depths and Elevations

Borehole Number	Ground Surface El. (m)	Weathered Bedrock Surface Depth (mbgs)	Weathered Bedrock Surface El. (m)	Weathered Bedrock Thickness
FG-1-2011	75.1	0.5	74.6	0.6
BH15-1	75.0	1.4	73.6	0.7
BH 15-2	75.6	0.8	74.8	0.3
BH15-3A	75.5	2.2	73.3	0.4
BH15-4	74.5	1.0	73.5	0.3
BH15-5	74.7	0.9	73.8	0.4

3.1.1.4.2 Unweathered Limestone Bedrock

Unweathered limestone bedrock was encountered at boreholes FG-1-2011 to FG-3-2011 from WSP's 2011 investigation, boreholes 15-1 through 15-5 from Golder's 2015 investigation, and boreholes BH15-2, BH15-3B, BH15 4 and BH15-6 from WSP's 2016 investigation. The bedrock was described in Golder's geotechnical report (2015) as fresh, thinly to medium bedded, grey, fine grained, non-porous limestone, with black shale partings.

The bedrock was confirmed by diamond drilling techniques while retrieving NQ or HQ sized bedrock cores. Table 3 summarizes the depths and elevations of the unweathered bedrock surface.

Table 3: Unweathered Bedrock Surface Depths and Elevations

Borehole Number	Location	Ground Surface El. (m)	Unweathered Bedrock Surface Depth (mbgs)	Unweathered Bedrock Surface El. (m)
15-1	Northeastern Portion of Site	75.9	2.4	73.4
15-2	Northeastern Portion of Site	75.7	2.7	73.0
15-3	Northeastern Portion of Site	75.8	2.8	73.0
15-4	Northeastern Portion of Site	75.6	3.1	72.6
15-5	Northeastern Portion of Site	75.5	2.6	72.9
FG-1-2011	Southern Portion of site	75.1	1.1	74.0
FG-2-2011	Southern Portion of site	75.4	1.0	74.4
FG-3-2011	Southern Portion of site	74.6	0.8	73.8
BH15-2	Southern Portion of site	75.6	1.1	74.5
BH15-3B	Southern Portion of site	75.5	2.2	73.3
BH15-4	Southern Portion of site	74.5	1.3	73.2
BH15-6	Southern Portion of site	73.7	1.5	72.2

The RQD values measured in Golder's 2015 boreholes, which were advanced at the northeastern portion of the site, ranged from 81% to 100%, indicating that the rock quality of the limestone bedrock is good to excellent throughout the entire core lengths.

Based on the RQD values in WSP's boreholes, which were advanced at the southern portion of the site, in general, the rock quality of the upper 1.2 m of the limestone bedrock is poor to very poor, becomes fair between depths of about 1.2 m and 2.8 m, and below which the rock quality is good to excellent. The measured RQD values from the Golder's 2015, WSP's 2011 and 2016 investigations are presented in Table 4 below.

Table 4: Limestone Rock Quality Based on RQD as a Function of Depth

Borehole Number	Quality Zones within the Bedrock Based on RQD			
	Very Poor to Poor (Range of RQD)	Fair (Range of RQD)	Good (Range of RQD)	Excellent (Range of RQD)
15-1	-	-	4.0 – 5.6 (85%)	2.4 – 4.0 (98%)
15-2	-	-	4.3 – 5.9 (85%)	2.7 – 4.3 (100%)
15-3	-	-	2.8 – 4.4 (80%)	4.4 – 5.9 (90%)
15-4	-	-	-	3.1 – 6.0 (95% - 96%)
15-5	-	-	14.1 – 15.3 (85%)	2.6 – 14.1 (90% - 100%)
FG-1-2011	1.1 – 1.4 (0%)	1.4 – 2.8 (56%)	-	2.8 – 4.2 (90%)
FG-2-2011	1.0 – 3.1 (0% – 49%)	-	3.1 – 4.7 (80%)	-
FG-3-2011	0.8 – 1.2 (0%)	1.2 – 2.8 (70%)	2.8 – 4.1 (75%)	-
BH15-2	0.9 – 1.2 and 2.7 – 4.2 (36% and 49%)	1.2 – 2.7 and 7.2 – 7.8 (67% and 54%)	4.2 – 7.2 (87%)	-

Borehole Number	Quality Zones within the Bedrock Based on RQD			
	Very Poor to Poor (Range of RQD)	Fair (Range of RQD)	Good (Range of RQD)	Excellent (Range of RQD)
BH15-3B	-	1.8 – 2.7 (55%)	-	2.7 – 7.9 (92% – 98%)
BH15-4	-	-	4.4 – 5.9 (78%)	1.3 – 4.4 and 5.9 – 7.4 (92% – 96%)
BH15-6	-	1.5 – 2.8 (63%)	4.3 – 8.0 (75% – 90%)	2.8 – 4.3 (100%)

The RQD values measured from the recovered bedrock are plotted against the elevation of each sample, as shown on Drawing 4.

Unconfined Compressive Strength (UCS) testing was carried out on three selected bedrock core samples from WSP's 2011 investigation. The laboratory test results are provided in Appendix B and are summarized in the Table 5 below. Based on the results of the UCS testing, the limestone bedrock at this site is strong to very strong.

Table 5: Results of Unconfined Compressive Strength

Borehole Number	Sample Number	Core Depth (mbgs)	UCS (MPa)
FG 1 2011	DC-5	2.5 to 2.7	109
FG 2 2011	DC-6	3.9 to 4.2	74
FG 3 2011	DC-6	3.3 to 3.7	128

3.1.2 Groundwater Conditions

Groundwater levels in the monitoring wells were measured on December 12, 2011 (at boreholes FG 1 2011 to FG 3 2011), on March 27, 2015 (at boreholes BH15-1 thru BH15-4), and on January 19, 2016 (at boreholes BH15-2, BH15-3A, BH15-3B, BH15-4 and BH15-6). Table 6 presents the results of the groundwater level measurements.

It should be noted that the groundwater levels are only representative of the period during which the readings were taken. Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as springs, or following heavy rainfall events.

Table 6: Groundwater Depth and Elevations from Previous Investigations

Borehole Number	Geological Unit	Ground Surface El. (m)	Groundwater Depth (mbgs)	Groundwater El. (m)	Date of Measurement
FG-1-2011	Weathered/Unweathered Bedrock	75.1	2.1	73.0	Dec 12, 2011
FG-2-2011	Fill/Bedrock	75.4	2.3	73.1	Dec 12, 2011
FG-3-2011	Fill/Bedrock	74.6	1.9	72.7	Dec 12, 2011
1525987 15-1	Bedrock	75.9	2.5	73.4	Mar 27, 2015
1525987 15-2	Bedrock	75.7	2.9	72.8	Mar 27, 2015
1525987 15-3	Bedrock	75.8	3.4	72.4	Mar 27, 2015
1525987 15-4	Bedrock	75.6	2.7	72.9	Mar 27, 2015
BH15-2	Bedrock	75.6	1.5(1) 2.0(2)	74.1(1) 73.6(2)	Jan 19, 2016
BH15-3A	Fill/Bedrock	75.5	1.3	74.2	Jan 19, 2016

Borehole Number	Geological Unit	Ground Surface El. (m)	Groundwater Depth (mbgs)	Groundwater El. (m)	Date of Measurement
BH15-3B	Bedrock	75.5	5.1	70.4	Jan 19, 2016
BH15-4	Bedrock	74.5	2.4[(1)] 5.8[(2)]	72.1[(1)] 68.7[(2)]	Jan 19, 2016
BH15-6	Bedrock	73.7	2.0[(1)] 6.6[(2)]	71.7[(1)] 67.1[(2)]	Jan 19, 2016

Notes: [(1)] Shallow monitoring well screen
[(2)] Deeper monitoring well screen

3.2 Geophysical Testing

During the Golder 2015 investigation, a 50 mm inside diameter PVC pipe was installed in borehole 15-5, with the outside of the pipe above the well screen backfilled with a bentonite-cement grout, to allow for subsequent geophysical testing.

The geophysical testing was carried out on June 24, 2015 and consisted of Vertical Seismic Profiling (VSP) through the overburden soils and the underlying bedrock. A detailed description of the procedure used for the VSP testing is provided in Appendix D.

4 DISCUSSIONS AND RECOMMENDATIONS

4.1 General

This section of the report provides engineering guidance related to the geotechnical design aspects of the project based on our interpretation of the available information described herein and the project requirements. The geotechnical recommendations herein are provided in consideration of guidelines in the Canadian Foundation Engineering Manual (CFEM 2023) and excerpts of the Ontario Building Code (OBC 2024) where relevant.

The information in this portion of the report is provided for planning and design purposes for the guidance of the design engineers. 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 may 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, costs, sequences, schedules, equipment and other resource requirements, and safety. Please refer to the appended statement of limitations and important information pertaining to this report.

4.2 Overview

In general, the subsurface conditions on this site consist of about 0.5 to 3.1 m of pavement structure and fill, overlying thin deposits of native sandy and gravelly soils, above limestone bedrock. The surface of the limestone bedrock varies from about 0.8 to 3.1 mbgs (El. 72.2 to 74.8 m). The upper portion of the limestone bedrock is generally weathered.

The proposed development will consist of a mixed-use of residential building, which will be 9- to 26- storeys in heights and contain 3 underground parking levels within the Phase 1 building, and 2 underground parking levels within the Phase 2 building.

The following list summarizes some key geotechnical issues associated with this project:

- Excavation for the construction of the basement and building foundations and basement levels will extend to through the surficial fill, sandy and gravelly soils, and into the underlying limestone bedrock. Excavation into the unweathered bedrock needs to be carried out using techniques with minimum disturbance to the adjacent structures and services. Vibration monitoring will be required during excavation activities.
- Given the constraints imposed by adjacent properties and roadways, it is expected that temporary shoring systems will be necessary to support the overburden. Design of a shoring system is beyond the scope of this report. However, along the perimeter where no adjacent structures exists (north, east, as well as a portion of the south wall), it is anticipated typical system may consist of steel soldier piles and timber lagging. Along the perimeter where adjacent structure exists (west wall and remaining portion of south wall), a shoring system consisting of interlocking steel sheet piles or diaphragm walls that controls movement to within tolerable limits may required (this will depend significantly on whether or not the adjacent structures are founded on bedrock). The use of ground anchors may also be required.
- Underpinning of the adjacent structures located adjacent to the western and southern portions of the property may be necessary, particularly if they are not founded on bedrock.
- Shallow foundations, such as spread footings and raft, founded on or within unweathered limestone bedrock can be designed using an Ultimate Limit States (ULS) factored bearing resistance of 7.4 MPa in accordance with the Canadian Foundation Manual (CFEM). For seismic design, this site can be assigned a Site Class of A in accordance with the Ontario Building Code (OBC 2024) regulations.
- The groundwater levels on this site were measured at depths of about 1.3 to 6.6 m below the ground surface (Elevation 67.1 to 74.2 m). A hydrogeological study will be required to evaluate the requirements and impacts of construction dewatering and long-term groundwater management.

4.3 Excavation

The proposed mixed-use residential development is planned to be constructed in two phases: Phase 1 and Phase 2 as follow:

- Phase 1 of the development will be located on the eastern half of the site and will include between 9 and 26 storeys above grade and 3 underground parking levels. The final floor slab level for the proposed third parking level will be approximately 9.6 mbgs (i.e., 65.8 m elevation). The excavation is expected to extend for 1.0 to 1.5 m below the final floor slab level to accommodate foundation and elevator pits. The final depth of excavation is expected to be 10.6 to 11.1 mbgs (El. 64.8 to 64.3 m).
- Phase 2 of the development will be located on the western half of the site and will consist of a 9-sotrey residential building with two underground parking levels. The finished floor of the lowest basement level at approximately Elevation 68.9 m, which is approximately 5 to 7 mbgs. Considering that the excavation will likely extend a further 1.0 to 1.5 m below the lowest basement floor level to accommodate the foundations and possible elevator pits, it is expected that the excavation will extend to about 6.5 to 8.5 mbgs (El. 67.4 to 67.9 m).

Based on the above, the excavation for basement and foundation construction will extend through the existing fill, native sandy and gravelly soils, and into the underlying limestone bedrock.

The structures that are at risk of being impacted by ground movements around the excavation are the low-rise buildings located immediately west and southeast of site. These structures may be founded on the bedrock surface and, if that is the case, the excavation will likely have little impact on the structure. Otherwise, if the foundations of the adjacent structures are founded on overburden and are within the close proximity of the excavation, then underpinning may be required.

As general guideline for excavation, a minimum distance of 1 m should be maintained between adjacent footings and the boundaries of excavation. To avoid undermining of the rock and/or disturbance of the rock (which could jeopardize the support for the structure), careful line drilling of the excavation limits in this area must be undertaken.

Geotechnical recommendations on excavations of overburden and bedrock are discussed further below.

4.3.1 Overburden Excavation

All excavations should be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). Temporary excavation details and requirements are given in Part III of Ontario Regulation 213/91.

The soils at the sites include granular fill and native sandy to gravelly soils, which above the groundwater level can be classified as Type 3 Soils. Excavation side slopes for Type 3 soils could be sloped at a minimum of 1H:1V. However, if the excavation is not first dewatered, the overburden below the groundwater level would be classified as a Type 4 soil; side slopes of at least 3H:1V would be required in accordance with OSHA.

It is understood that the proposed building will encompass essentially the full limits of the property. Given the constraints imposed by the adjacent properties and roadways, it is expected that temporary support (shoring) systems will be required to support the excavation faces within the overburden along the property boundaries. Additional guidance on temporary shoring is provided in Section 4.3.4 below.

All excavated surfaces should be kept free of frost, water, etc. during construction operations, and all excavated surfaces should be inspected by qualified geotechnical personnel, to confirm the consistency of the findings presented in this investigation report and the design and construction of similar structures.

Stockpiling of soil beside the excavations should be avoided; the weight of the stockpiled soil could lead to basal instability of braced excavations, or slope instability of unsupported excavations.

It is recommended heavy vehicles not be parked close to excavation edges, or within the projected 2H:1V distance from the bottom of the excavation. If excavation material is to be temporarily stored on site, it must be placed at a minimum distance from the crest of the slope, equivalent to twice the depth of the excavation. This must be respected at all times unless specific geotechnical analyses to confirm otherwise.

The excavated soils should be disposed of in a proper manner, depending on its environmental quality and following the recommendations from the Phase II ESA report completed for this site and in accordance with all applicable environmental legislation (i.e., O. reg 406/19).

4.3.2 Bedrock Excavation

The bedrock surface varies from El. 72.2 to 74.8 m. Bedrock removal will be required for excavation that extends below these elevations.

The Contractor is responsible for proposing suitable means and methods for rock excavation. The proposed excavation method should enable control of the extent of excavation and should mitigate the potential risk of overbreak, unexpected over-excavation, and other rock disturbances or damage due to excavation.

Excavations of the upper weathered bedrock could be accomplished using mechanical methods (such as by hydraulic shovel and hoe ramming). Deeper excavations into the unweathered bedrock could be carried out using drill and blast procedures, subject to the bylaws and blasting restrictions that are in effect for the area. If blasting is required, these operations should be conducted carefully and in accordance to Ontario Provincial Standard Specification OPSS.MUNI 120 (November 2019) “General Specification for the use of Explosives”.

Near vertical walls in the limestone bedrock is considered feasible during the construction period.

No major open fractures were detected in the bedrock during the previous geotechnical investigations. However, it is possible that such fractures be encountered during the bedrock excavation. If present, such fractures may have an effect on the behaviour of the rock mass during excavation due to undesirable rock movements and the foundations integrity of the existing adjacent buildings may be compromised. In such case, as would be pointed out by the geotechnical inspection during excavation, rock bolting may be required to support the excavated rock faces. Inspection during excavation is recommended at the early stages to evaluate the potential need for rock reinforcement.

Blast induced damage to the bedrock must be avoided; otherwise rock reinforcement would be required. It should be planned to either line drill the bedrock along the perimeter of the excavation at a close spacing in advance of blasting so that a clean bedrock face is formed, or to carry out perimeter drilling and pre-shearing of the excavation limits using controlled blasting.

Significant caution should be exercised in carrying out blasting due to the near proximity of existing buildings and services. The blasting should be controlled to limit the peak particle velocities at all adjacent structures or services such that the blast induced damage will be avoided. A blast design specialist in this field will be required.

A pre-construction survey of all of the surrounding structures and utilities within at least 75 m radius distance from the Site shall be carried out by the Contractor. Selected existing interior and exterior cracks in the structures identified during the pre-construction survey should be monitored for lateral or shear movements by means of pins, glass plate tell-tales, and/or movement tell-tales. As per the City of Ottawa requirements, the Contractor shall notify the dwellings within 150 m radius about the planned work before commencing the excavation.

4.3.3 Vibration Monitoring

The Contractor should be required to submit a detailed blasting design and vibration monitoring plan proposal prepared by a blasting/vibration specialist prior to commencing work. The excavation plan should provide detailed information on the proposed excavation methods, vibration monitoring equipment, monitoring locations, frequency of readings, vibration limit criteria for reporting, suitable mitigation actions, and other relevant information. The plan should be reviewed and accepted by the project owner and geotechnical consultant. The Contractor should also provide to the project owner copies of the pre-construction inspection, vibration monitoring reports, and post-construction inspection reviews among any other relevant deliverables.

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 and should be verified by the specialist:

Table 7: Vibration Limits

Frequency (Hz)	Maximum Peak Particle Velocity, PPV (mm/s)
<10	5
10 to 40	5 to 50 (sliding scale)
>40	20

If practical, blasting should be carried out at the furthest points from the closest structure or service to assess the ground vibration attenuation characteristics, to confirm the anticipated ground vibration levels, and to adjust the Contractor's blasting methods as needed.

4.3.4 Temporary Support (Shoring)

4.3.4.1 Shoring Options

The excavation will essentially encompass the full limits of the property and therefore near vertical excavation walls will be required.

Design of the shoring system is beyond the scope of this report. Detailed design and performance of the temporary shoring systems needs to be established prior to start of the excavation. The shoring system design should take into considerations the impact of movement of the installed shoring system on the adjacent structures (both buildings and infrastructure), and feasibility of construction. A detailed investigation of the adjacent structures is required to incorporate in the shoring design. Below are typical shoring systems that are suitable for the soil conditions at the site.

- It is envisioned that steel soldier piles and timber lagging shoring would be feasible along the northern (Carling Avenue), eastern (Bronson Avenue), and southwestern (undeveloped) limits of the site, where the excavation will be adjacent to the existing roadways or undeveloped land. Soldier piles and lagging systems are suitable where the objective is to maintain an essentially vertical excavation wall and where the movements above and behind the wall need only be sufficiently limited that relatively flexible features (such as roadways) will not be adversely affected.

- For excavations where existing buildings are present adjacent to the excavation (such as at the western and southeastern limits of the site), interlocking driven steel sheet pile system with pre-stressed tiebacks may be needed. The sheet piling systems with pre-stressed tie could greatly limit the shoring deflections. However, the need for these stiffer shoring systems will depend significantly on the foundation conditions of the nearby structures (i.e., if the structures are founded on rock, the risk of settlement during excavation is much lower).

- Continuous concrete shoring (such as a secant piles or diaphragm walls) could also be a feasible alternative for the sides of the excavation adjacent to the existing structures. Diaphragm walls are appropriate where difficulties may be encountered installing sheet piles, where heavily loaded foundations exist adjacent to the shoring, or where groundwater inflow needs to be controlled. Such systems could greatly mitigate the potential for foundation movements but would also be much more expensive.

- At locations where structures sensitive to movement exists, such as adjacent to western and southeastern sides of the site, underpinning may also be used to control displacement. That is, if the resulting displacement due to the movements of the applied shoring system is unacceptable and/or if the loads on the adjacent foundations are large. Further details on the foundations of the existing structures will be required for a full assessment of the required shoring to be implemented.

For all of the above systems, some form of lateral support to the shoring system is typically required for excavation depths in soil greater than about 3 or 4 m. Lateral restraint could be provided by means of tiebacks consisting of grouted bedrock anchors. However, the use of rock anchor tiebacks would require the permission of the adjacent property owners (including the City of Ottawa, who owns the adjacent roadways), since the anchors would be installed beneath their properties. The presence of utilities beneath the adjacent streets, which could interfere with the tiebacks, should also be considered. Alternatively, interior struts can be considered, connected either to the opposite side of the excavation (if not too distant) or design of raker piles and/or footings within the excavation. However, internal struts could interfere with the construction of the foundations and superstructure.

The shoring should also be designed to account for lateral earth pressures resulting from the weight of the retained earth and other dead and surcharge loads. The earth pressure distribution used for shoring design is dependent upon the specific wall design and on the nature of the lateral support provided. The potential for the loads from the adjacent foundations to apply additional lateral pressure to the shoring system should be considered. The selection of that design lateral earth pressure should therefore be the responsibility of the contractor who will be responsible for the shoring design.

4.3.4.2 Ground Movements

Some unavoidable inward horizontal deformation and vertical settlement of the adjacent ground will occur as a result of excavation, installation of shoring system, deflection of the ground support system (including bending of the walls, compression of the struts and/or extension of the tiebacks), as well as deformation of the soil/rock in which the toes of the shoring walls are embedded. The ground movements could affect the performance of buildings, surface structures, and underground utilities adjacent to the excavation.

Settlements behind soldier pile and lagging shoring systems are expected within the overburden only (i.e., settlement within bedrock is negligible). As a preliminary guideline, typical settlements are expected to be less than about 0.3 % of the excavation depth within the overburden, provided good construction practices are used, voids are not left behind the lagging, and also provided that large foundation loads from existing buildings are not applied behind the shoring. This guideline would suggest that about 3 to 10 mm of ground settlement would occur for shoring systems installed through the overburden to about 1 to 3 m depth.

Movements behind a properly constructed steel sheet pile or continuous caisson wall would be less than what would be expected for a soldier pile and lagging wall. However, this is only a preliminary guideline and is provided only to assist the owner’s designers in carrying out an initial assessment of the expected settlements and the potential impacts of these settlements. A more detailed assessment of the expected settlements should be undertaken by the contractor and must consider the effects of adjacent foundation loads.

Should the preliminary assessment carried out using this estimated settlement indicate unacceptably large settlements to adjacent structures, roadways, or utilities, then a more detailed assessment should be carried out during future design stages (but prior to tender) to better assess the shoring requirements, or a more rigid form of shoring should be selected.

The Contractor shall conduct a pre-construction survey of all adjacent structures within a minimum of 75 m radius distance from the Site prior to the commencement of the excavation. A notice of planned work shall be given in advance to dwellings within 150 m radius distance from the Site as per the City of Ottawa requirements.

Underground utilities should be considered during the shoring design in terms of possible conflicts with tieback installation and/or possible restrictions on the acceptable ground/shoring movements. Therefore, an inventory of these utilities should be made at an early stage in the design.

4.4 Groundwater Management

The groundwater levels at this site were measured at elevations varying from 67.1 m to 74.2 m, which is above the anticipated base of the excavation. Groundwater inflows into the rock excavation is therefore expected.

The actual rate of groundwater inflow will depend on many factors including the contractor's schedule and rate of excavation, the size of the excavation, the number of working areas being excavated at one time, and the time of year at which the excavation is made. Also, there may be instances where volumes of precipitation, surface runoff and/or groundwater collects in an open excavation must be pumped out. The contractor shall provide a pumping system to remove all water to the bottom of the excavation. Excavation shall be kept dry at all times.

A Permit-to-Take Water (PTTW) is required from the Ministry of the Environment and Climate Change (MOECC) if a volume of water pumped from the excavations under normal operation will be greater than 400,000 L/day. However, if the volume of water to be pumped will be less than 400,000 L/day but more than 50,000 L/day, the water taking will not require a PTTW, but will instead need to be registered in the Environmental Activity Sector Registry (EASR) as a prescribed activity.

A hydrogeological study will be required to determine the groundwater pumping requirements for this site and support an application for a PTTW or registration of EASR.

The planned temporary (during construction) and permanent dewatering (long-term due to the foundation drainage system, if one is provided) would directly impact ground settlements. Consequently, adjacent structures founded on sensitive and compressible clay would be affected if within the zone influenced by the lowering of ground water table. The results of this investigation as well as the published geologic mapping do not reveal the presence of such soils, at least within the immediate vicinity of the site. Regardless, a hydrogeological study will be required to confirm/evaluate the potential impacts of the proposed development on the adjacent structures.

Design of a dewatering system is beyond the scope of this report. Typical design may be provided by WSP under separate mandate and/or can be reviewed if proposed by a specialty contractor. An outlet (or outlets) should be identified which the contractor can use to dispose of the pumped groundwater and incident precipitation. In order for pumped groundwater to be discharged to a City of Ottawa's sewer, the groundwater quality needs to meet the City of Ottawa's Sewer Use By law criteria and a separate sewer discharge permit must be obtained. Additional ongoing chemical testing should be carried out at the time of construction to monitor the groundwater quality so that disposal requirements can be confirmed throughout the duration of construction.

4.5 Foundation

4.5.1 Subgrade Preparation

The excavation for the proposed development is generally expected to extend down to unweathered bedrock to El. 64.8 to 64.3 m within Phase 1 building and El. 67.4 to 67.9 m within Phase 2 building.

Subgrade preparation for footings founded on rock will involve the removal of all soils and weathered bedrock to expose unweathered bedrock. Any pieces of rock that can be manipulated by conventional excavation equipment should be removed, and as directed by the Geotechnical Engineer. Final subgrade surfaces should be brushed and cleaned. The exposed bedrock surface should be examined and approved by the Geotechnical Engineer to confirm the competency to support the design bearing pressures. Lean mixed concrete should be used for levelling the unweathered bedrock. The lean mix concrete shall have a minimum compressive strength of 30 MPa. If lean mixed concrete is used below footings at the bedrock surface (i.e., not confined within bedrock), it must extend a minimum of 0.3 m beyond the edge of the footing and then downward at a 1H:1V.

Confirmation of bedrock quality during construction will require the contractor to perform probing of the bedrock using 50 mm diameter drill holes drilled to a depth of 1.5 m within the footprint of the building. These holes will need to be reviewed by the geotechnical engineer to confirm that no significant mud seams or voids exist at the proposed footing locations. If mud seams are found, localized areas may need to be lowered below the mud seam. The locations of these probe holes should be selected under the direction of the geotechnical engineer during construction. Contractors should plan for one probe per pad footing and a minimum or 1 probe every 6 m in strip footings.

4.5.2 Geotechnical Bearing Resistance

As previously noted, the proposed building will consist of three underground parking levels within Phase 1 with final floor slab level at approximately El. 65.8 and two underground parking levels within Phase 2 with the final floor slab level at approximately El. 68.9 m. Based on the assumed depth of excavations, it is anticipated that the underside of foundation will be founded within the unweathered limestone bedrock.

The results of UCS testing from the WSP's 2011 investigation indicated that the limestone bedrock at this site is strong to very strong. Based on the results of the UCS testing, for spread footings or raft foundations constructed within the unweathered limestone bedrock surface may be designed using an Ultimate Limit States (ULS) factored geotechnical resistance of 7.4 MPa in accordance with CFEM. The factored ULS bearing resistance includes a geotechnical resistance factor of $\Phi = 0.5$. The upper portion of the bedrock was noted to be weathered in some of the boreholes. However, based on the founding depths of the interior foundations, the underside of the footings is expected to be below the depth of weathered zone.

Provided the bedrock surface is properly cleaned of soil and weathered material at the time of construction, settlement under the ULS condition is expected to be negligible. Therefore, there is no corresponding design bearing pressure recommended under Serviceability Limit State (SLS) conditions for bedrock.

4.5.3 Sliding Resistance

The factored ultimate resistance of the footings to lateral loading 'shear resistance for sliding' across the interface between the footing, and the bedrock may be calculated using Mohr-Coulomb criterion below with load and resistance factors given in Table 8.

$$\tau = f_c c' + (\sigma - f_u U) f_\phi tan\phi'$$

where c' is cohesion, ϕ' is shearing angle, U is water pressure, and σ is the normal stress on the sliding surface.

Table 8: Minimum Lateral Load and Resistance Factors after Meyerhof (1984) (Wyllie 2009)

Category	Item	Load Factor	Resistance Factor
Loads	Dead Loads, (f_{DL})	1.25 **(0.8*)**	--
	Live Loads, Wind, Earthquake, (f_{LL})	1.5	--
	Water Pressure (f_U)	1.25 **(0.8*)**	--
Shear Strength	Cohesion (c') – Stability, Earth Pressure, (f_c)		0.65
	Cohesion (c') – Foundation, (f_c)		0.5
	Friction Angle (ϕ'), (f_ϕ)		0.8

Note: * The values given in the parenthesis apply to beneficial loading conditions such as dead loads resist overturning or up lift

It is prudent to ignore the cohesion component when estimating the shear resistance against sliding. This is because the cohesive bond may be lost when separation takes place between concrete and rock foundation upon relative movement. The shearing angle ϕ' may be taken as 35 deg.

To increase the lateral resistance against sliding, the footings can be supplied with a shear key and/or anchored to the bedrock by means of rock anchors. The design of both, the shear key (i.e., width and impediment), and the rock anchor system (i.e., the number and interval of the anchors, and the embedment length of anchors in concrete and rock) shall be provided by a structural engineer.

4.6 Rock Anchors

Rock anchors may be required to resist overturning and/or uplift forces. The anchors could consist of either grouted or mechanical anchors. In designing grouted rock anchors, consideration should be given to four possible anchor failure modes.

- Failure of the steel tendon or top anchorage.
- Failure of the grout/tendon bond.
- Failure of the rock/grout bond.
- 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. If the response of the anchor under SLS conditions needs to be evaluated, for a preliminary assessment it may conservatively be taken as the elastic elongation of the unbonded portion of the anchor under the design loading.

For potential failure mode iv), the resistance should be calculated based on the buoyant weight of the potential mass of rock which could be mobilized by the anchor. 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:

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

Where:

Q_r = Factored uplift resistance of the anchor, kilonewtons;

Φ = Resistance factor, 0.3;

γ' = Effective unit weight of rock, use 27 kN/m^3 above groundwater level, 17 kN/m^3 below the groundwater level;

D = Anchor length, meters; and,

θ = Half of the apex angle of the rock failure cone, use 30 degrees.

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 in cubic metres;

d = Depth of anchor group in metres;

a = Width of anchor group in metres; and,

b = Length of the anchor group in metres.

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:

$$Q_r = \Phi \, \gamma' V$$

It is recommended that pull-out tests be carried out on anchors to confirm their pull-out capacity (as required by Canadian Foundation Engineering Manual (CFEM) 2023 for the use of a resistance factor of 0.6). For preliminary evaluation purposes, the testing procedures should be in accordance with the Post-Tensioning Institute’s Recommendations for Prestressed Rock and Soil Anchors and testing procedures outlined in the OPSS. A more detailed testing program should be developed once further details on the rock anchors (e.g., required loads, total number of anchors, anchor spacing etc.) are known.

Rock anchors intended as permanent structural elements should be provided with double corrosion protection and tested in accordance with OPSS 942.

The installation and testing of the anchors should be supervised by qualified geotechnical personnel. Care must be taken during grouting to ensure that the grouting pressure is sufficient to bond the entire length of the grout area with a minimum of voids. It is also suggested that the anchor holes be thoroughly flushed with water to remove all debris and rock flour. It is essential that rock flour be completely removed from the anchor holes to be grouted to ensure an adequate bond between the grout and the rock.

Prestressing of the anchors prior to loading would minimize anchor movement due to service loads.

Further guidance can be provided for assessing the anchor resistance once the final anchor layout and loads have been established, if requested.

4.7 Seismic Design

The OBC 2024 specifies that the structure should be designed to withstand forces due to earthquakes. For the purpose of earthquake design, the information relevant to the geotechnical conditions at this site is the ‘Site Class’. The seismic design provisions of the OBC depend, in part, on the shear wave velocity of the upper 30 m of soil and/or rock below founding level.

Site specific shear wave velocity profiling using the VSP testing (a down-hole geophysical method) was carried out within Golder's borehole 15-5. The results of that testing are provided in Appendix D.

The results of the VSP testing indicate that the bedrock below about 8 m depth has an average shear wave velocity of greater than 1,750 m/s. In accordance with OBC 2024, the proposed building can be designed using a Site Class A designation.

4.8 Frost Protection

All perimeter and exterior foundation elements or interior foundation elements in heated areas should be provided with a minimum of 1.5 m of earth cover for frost protection purposes. Isolated, unheated exterior footings adjacent to surfaces which are cleared of snow cover during winter months should be provided with a minimum of 1.8 m of earth cover.

With the anticipated founding level to accommodate the below-grade parking, and assuming that the parking garage will not be allowed to freeze, there will be sufficient earth cover to protect against frost. In addition, the foundations are expected to be placed directly on unweathered limestone bedrock (following blasting and cleaning), which is considered as non-frost susceptible in the absence of soil filled joints and/or seams.

However, the buildings immediately west and southeast of the site are likely supported on shallow foundations, which may be founded on frost susceptible soils. The excavation shoring will be constructed within very close proximity to the foundations of the buildings and, if construction is carried out during the winter months, the existing foundations may be adversely affected due to frost movement. Therefore, if construction is anticipated during sub-zero temperatures, provision should be made to protect the soils behind the shoring from frost movement.

4.9 Cistern Raft Foundation

It is understood that cisterns are projected to be built with thee building area. Detailed structural design have not been provided. A raft foundation placed on an engineered fill base might is being considered to support the cistern element.

The geotechnical resistance of the raft itself at SLS will depend on the settlement characteristics of the soil below the slab, as well as magnitude and geometry of loading. The geotechnical parameter typically used for analysis of settlement below a raft or slab is the modulus of subgrade reaction. The modulus of subgrade reaction is defined as:

$$K_{B\times B} = q/\delta$$

where:

K_{BxB} = modulus of subgrade reaction in megapascals per meter;

q = applied bearing or contact pressure on footing in kilopascal; and,

d = settlement of footing under the applied pressure in millimetres.

The modulus of subgrade reaction is not a fundamental soil property but is dependent upon the size and shape of the loaded area, soil type, relative stiffness of the raft and soil, duration of loading, etc. as a result, the modulus for a 300 mm square footing is typically used as a standard basis.

A value of 50 MPa/m may be used for the modulus of subgrade reaction for placed on a minimum of 200 mm engineered fill constructed with OPSS1010 Granular A material compacted to 100% of Standard Proctor Maximum Dry Density (SPMDD).

For loaded areas greater than 300 mm square, the modulus of subgrade reaction should be calculated as follows:

$$K_{B\times B} = K_{0.3}\left(\frac{B+0.3}{2B}\right)^2$$

where:

$K_{0.3}$ = modulus of subgrade reaction for a 0.3 m x 0.3 m steel plate. Use 50 MPa/m for engineered fil; and,

B = width of loaded area, meters.

4.10 Basement Floor Slab

In preparation for the construction of the basement floor slab, all loose, wet, and disturbed material should be removed from beneath the floor slab.

It is not known if the basement levels will be designed to be of drained or water-tight construction. If a "drained" structure will be considered, provision should be made for at least 300 mm of free draining granular material, such as 19-mm diameter clear crushed stone, to form the base of the floor slab. To prevent hydrostatic pressure, build up beneath the floor slab, the granular base for the floor slab should be drained. This should be achieved by installing rigid 100 mm diameter perforated pipes in the floor slab bedding at 6 m centres. The perforated pipes should discharge to a positive outlet such as a sump from which the water is pumped.

If or where an asphalt surface will be provided for the basement level, at least 150 mm of Granular A (City of Ottawa SP F-3147) base should be provided above the clear stone and compacted to at least 100 % of SPMDD.

If water-tight construction is required for this structure, then the basement floor slab will have to be of concrete slab construction, rather than asphalt, and would have to be designed to be integral with the foundation walls (i.e., to form a raft slab). The basement floor and foundation walls will have to be designed to resist hydro-static uplift pressures. Rock anchors may be required to resist the hydro-static uplift pressures (buoyant forces). Recommendations for rock anchors are provided in Section 4.6 above.

4.11 Basement Wall

The backfill and drainage requirements for basement walls, as well as the lateral earth pressures, will depend on the type of excavation that is made to construct the basement levels and the forming methods. The following sections assume that water-tight construction will not be required.

If water-tight construction is needed, additional design guidance will need to be provided.

4.11.1 Basement Walls Against Soils

The soils at this site are potentially frost susceptible and should not be used as backfill against exterior walls or foundation elements.

Free draining backfill materials shall be used for backfill for the foundation walls to prevent hydrostatic pressure build-up (and potential leakage). The foundation and basement walls therefore should be backfilled with non-frost susceptible sand or sand and gravel conforming to the requirements for Granular B Type I. To avoid ground settlements around the foundations, which could affect site grading and drainage, all of the backfill materials should be placed in 0.3 m thick lifts and compacted to at least 95% of SPMDD.

The basement wall backfill (for the full height of the wall) should be drained by means of a perforated pipe subdrain in a surround of 19-mm diameter clear stone, fully wrapped in a geotextile, which leads by positive drainage to a storm sewer or to a sump from which the water is pumped.

4.11.2 Basement Walls Against Bedrock

Where basement walls will be poured against bedrock, vertical drainage such as Miradrain or equivalent product must be installed on the face of the bedrock to provide the necessary drainage. The top edge of the Miradrain should be sealed or covered with a geotextile to prevent the loss of soil into the void between the sheet and geotextile of the Miradrain.

Where the basement walls will be constructed using formwork, it will be necessary to backfill a narrow gallery between the shoring or bedrock face and the outside of the walls. The backfill should consist of 6 mm clear stone 'chip', placed by a stone slinger or chute.

In no case should the clear stone chip be placed in direct contact with other soils. For example, surface landscaping or backfill soils placed near the top of the clear stone backfill should be separated from the clear stone with a geotextile.

Both the drain pipes for the wall backfill and/or the Miradrain should be connected to a perimeter drain at the base of the excavation which is connected to a sump pump.

4.11.3 Lateral Earth Pressure

It is considered that three design conditions exist with regards to the lateral earth pressures that will be exerted on the basement walls:

1) Walls cast directly against the bedrock face.
2) Walls cast against formwork with a narrowly backfilled gallery provided between the basement wall and the adjacent excavation bedrock face.
3) Walls cast against formwork with a wide backfilled gallery provided between the basement wall and the adjacent excavation face (including the upper portions of the walls, above the bedrock surface).

For the first case (walls cast against the bedrock with Miradrain), there will be no effective lateral earth pressures on the basement wall.

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 basement wall. The magnitude of the lateral earth pressure can be calculated as:

$$\sigma_{h(z)} = \frac{\gamma B}{2tan\delta}\left(1 - e^{-2k\frac{z}{b}tan\delta}\right) + kq$$

where:

$\sigma_{h(z)}$ = Lateral earth pressure on the basement wall at depth z, kilopascals;

k = Earth pressure coefficient, use 0.6;

γ = Unit weight of retained soil, use 22 kN/m^3;

B = Width of backfill between basement wall and bedrock face, meters;

δ = Average interface friction angle at backfill-basement wall and backfill-rock face interfaces, use 15 degrees;

z = Depth below top of shoring, meters; and,

q = Uniform surcharge at ground surface to account for traffic, equipment, or stockpiled materials, use 15 kPa. Additional/higher surcharge loads associated with existing building foundations should also be accounted for where existing buildings are located adjacent to the basement walls.

It should be noted that the resulting pressure distribution for the first case is not triangular and that the lateral earth pressures above the gallery (i.e., bedrock surface) should be calculated as noted below for the second case.

For the third case, the basement walls should be designed to resist lateral earth pressures calculated as:

$$\sigma_{h(z)} = k_0 + (\gamma z + q)$$

Where:

k_o = at-rest earth pressure coefficient, use 0.5.

For all cases, hydrostatic groundwater and different lateral earth pressures (e.g., effective unit weights of the soils would apply to the above equations) would also need to be considered if the structure is designed to be water-tight. Additional guidance will therefore need to be provided if water-tight construction is considered.

Conventional damp proofing of the basement walls is appropriate with the above design approach. For concrete walls poured against shoring or bedrock (i.e., without a drainage layer), damp proofing using a crystalline barrier such as Crystal Lok or Xypex could be used. The use of a concrete additive that provides reduced permeability should also be considered.

These lateral earth pressures would increase under seismic loading conditions. The earthquake-induced dynamic pressure distribution, which is to be added to the static earth 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). The combined pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_{h(z)} = k_0 \gamma z + (k_{AE} - k_0)\, \gamma\, (H - z)$$

where:

k_{AE} = The seismic earth pressure coefficient, use 0.6; and,

H = The total depth to the bottom of the foundation wall, meters.

Hydrodynamic groundwater pressures would also need to be considered if the structure is designed to be water-tight. However, if this option is selected, more sophisticated analyses would need to be carried before guidance could be provided.

All of the lateral earth pressure equations are given in an unfactored format and will need to be factored for Limit States Design (LSD) purposes.

It has been assumed that the underground parking levels will be maintained at minimum temperatures but will not be permitted to freeze. If these areas are to be unheated, additional guidance for the design of the basement walls and foundations will need to be provided.

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 wall may have to be placed to form a frost taper, depending on the composition of the existing fill. The frost taper should be brought up to pavement subgrade level from 1.5 m below the finished exterior grade at a slope of 3H:1V, 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 standard Proctor maximum dry density using suitable vibratory compaction equipment.

4.12 Site Servicing

Recommendations on excavations for site servicing (e.g., storm, sanitary sewers etc.) are provided in Section 4.3.

At least 150 mm of Granular A should be used as pipe bedding. Depending on the condition of the subgrade, it may be necessary to place a sub-bedding layer consisting of 300 mm of Granular B Type II beneath the Granular A, or the Granular A layer could be thickened. The bedding material should, in all cases, extend to the spring line of the pipe and should be compacted to at least 98% of the material’s standard Proctor maximum dry density.

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 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 Granular A or Granular B Type II. The cover material should be compacted to at least 98% of the material’s standard Proctor maximum dry density.

All trench backfill should conform to City of Ottawa specification SP F-2120. The trench backfill should consist of Granular A, Granular B Type I or II (City of Ottawa SP F-3147). The granular fill should be placed in maximum 200 mm thick lifts and should be compacted to at least 95% of the material’s standard Proctor maximum dry density using suitable vibratory compaction equipment.

Where the trench will be covered with hard surfaced areas, the type of native material placed in the frost zone (between subgrade level and 1.8 m depth) should match the soil exposed on the trench walls for frost heave compatibility. Trench backfill should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% of the material’s standard Proctor maximum dry density using suitable compaction equipment.

Clay dykes are not required for this site.

4.13 Pavement Design

In preparation for pavement construction, all loose and deleterious should be subexcavated from all pavement areas (which is expected to be completed as part of the foundation excavations).

Sections requiring grade raising to proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow or Select Subgrade Material (SSM) meeting the requirements of OPSS.MUNI 212 and SP F-3147, respectively. These materials should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% of the materials' standard Proctor maximum dry density using suitable compaction equipment.

The surface of the subgrade or fill should be crowned to promote drainage of the pavement granular structure. Perforated pipe subdrains should be provided at subgrade level extending from the catchbasins for a distance of at least 3 m in four orthogonal directions, or longitudinally where parallel to a curb.

The pavement structure for parking lots and heavy traffic lanes are provided in Table 5 1 and Table 5 2 below:

Table 9: Pavement Design

Layer	Material	Thickness (mm)		Compaction (SPMDD) %
		Light Duty	Heavy Duty	
Asphaltic concrete (Surface course)	Superpave 12.5 mm, PG 58-34	40	40	See Note 1
Asphaltic concrete (Base course)	Superpave 19.0 mm, PG 58-34	50	50	See Note 1
Granular Base	OPSS1010 Granular A	200	200	100%
Granular Subbase	OPSS1010 Granular B Type II	300	400	100%

Note: 1Asphaltic concrete should be compacted in accordance with Table 10 of OPSS 310.

The granular base and subbase should consist of Granular A and Granular B Type II (City of Ottawa SP F-3147), respectively, and should be uniformly compacted to at least 100% of SPMDD using suitable vibratory compaction equipment.

The asphaltic concrete should meet the requirements of City of Ottawa specification F 3106. The Performance Graded Asphalt Cement (PGAC) should consist of PG 58-34. The asphaltic concrete should be compacted in accordance with Table 10 of OPSS 310.

The above pavement designs are 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.

4.14 Corrosion and Cement Type

One sample of groundwater from Golder's previous borehole was submitted to Paracel for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements. The results of the testing are provided in Appendix C.

Table 10: Results of Chemical Analysis

Borehole Number	Well Screen Depth (m)	Chloride (mg/l)	Sulphate (mg/l)	pH	Conductivity (µS/cm)
1525987 15-3	4.4 – 5.9	1,800	141	7.2	5,730

The results indicate an elevated potential for corrosion of exposed ferrous metal, which should be considered in the design of exposed steel (such as rock anchors). The results also indicate that concrete made with Type GU Portland cement should be acceptable for substructures.

5 CONSTRUCTION CONSIDERATIONS

5.1 QUALITY CONTROL/ASSURANCE

The successful execution of the project will depend upon excellent workmanship and quality control/assurance during construction.

All foundation and subgrade areas should be inspected by experienced geotechnical engineer prior to filling or concreting to ensure that competent bedrock has been reached and that the bearing surfaces have been properly prepared.

The installation and testing of the rock anchors, if required, should be supervised by qualified geotechnical personnel. The placement and compaction of any engineered fill should be inspected to ensure that the materials used conform to the project specifications.

WSP can provide these services upon request. WSP should also be retained to review the detailed drawings and specifications for this project prior to tendering to ensure that the recommendations provided in this report have been adequately interpreted.

In addition, the proposed blasting design and monitoring plans proposed by the contractor should be reviewed prior to commencement of the work.

The monitoring wells installed during the previous investigations at this site will need to be abandoned in accordance with Ontario Regulation 903. However, these devices may be useful and more economically removed during construction. It is therefore proposed that decommissioning of these devices be made part of the construction contract.

5.2 EXCESS SOIL MANAGEMENT

For projects that will generate excess soil, management of such soil both on-site and off-site will need to be carried out in compliance with the Excess Soil Regulation (Ontario Regulation 406/19). The Excess Soil Management and Reuse Planning Requirements (as noted below) have come into effect on January 2023.

i) Assessment of Past Uses (similar to a Phase One Environmental Site Assessment)

ii) Sampling and Analysis Plan

iii) Soil Characterization Report

iv) Excess Soil Destination Assessment Report

v) Filing of Notice on Registry

vi) Soil Tracking

For most earthworks projects, these planning requirements would be required for sites that generate greater than 2,000 m^3 of excess soil within a settlement area (e.g. built-up areas such as towns and cities) and/or sites that have potentially contaminating activities (PCAs) associated with them.

WSP can provide more guidance on the Excess Soil Regulation to ensure full compliance with the regulation, if requested.

Signature Page

WSP Canada Inc.

LICENSED PROFESSIONAL ENGINEER
M.A. T. AL-KHAZAALI
100512672
Dec03,2025
PROVINCE OF ONTARIO

Mohammed Al-Khazaali, Ph.D., P.Eng.	Chris Hendry, P.Eng.
Senior Geotechnical Engineer	*Lead Principal Geotechnical Engineer*

MAK/CH/yj

https://wsponlinecan.sharepoint.com/sites/ca-2025ca462203/shared documents/06. deliverables/revised reprot sept. xx, 2025/ca0059449.2203-r-rev0-770-774 bronson-final geotech report-2025'12'03.docx

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: WSP Canada Inc. (WSP) 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 practising 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 WSP by the Client. 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. WSP cannot be responsible for use of this report, or portions thereof, unless WSP 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 WSP's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, WSP may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Any other use of this report by others is prohibited and is without responsibility to WSP. The report, all plans, data, drawings and other documents as well as all electronic media prepared by WSP are considered its professional work product and shall remain the copyright property of WSP, 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 WSP. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client can not rely upon the electronic media versions of WSP'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 WSP by the Client, communications between WSP and the Client, and to any other reports prepared by WSP 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. WSP can not 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 Ground Water 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, WSP does not warrant or guarantee the exactness of the descriptions.

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 WSP 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: WSP 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 WSP's report. WSP 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 WSP's report.

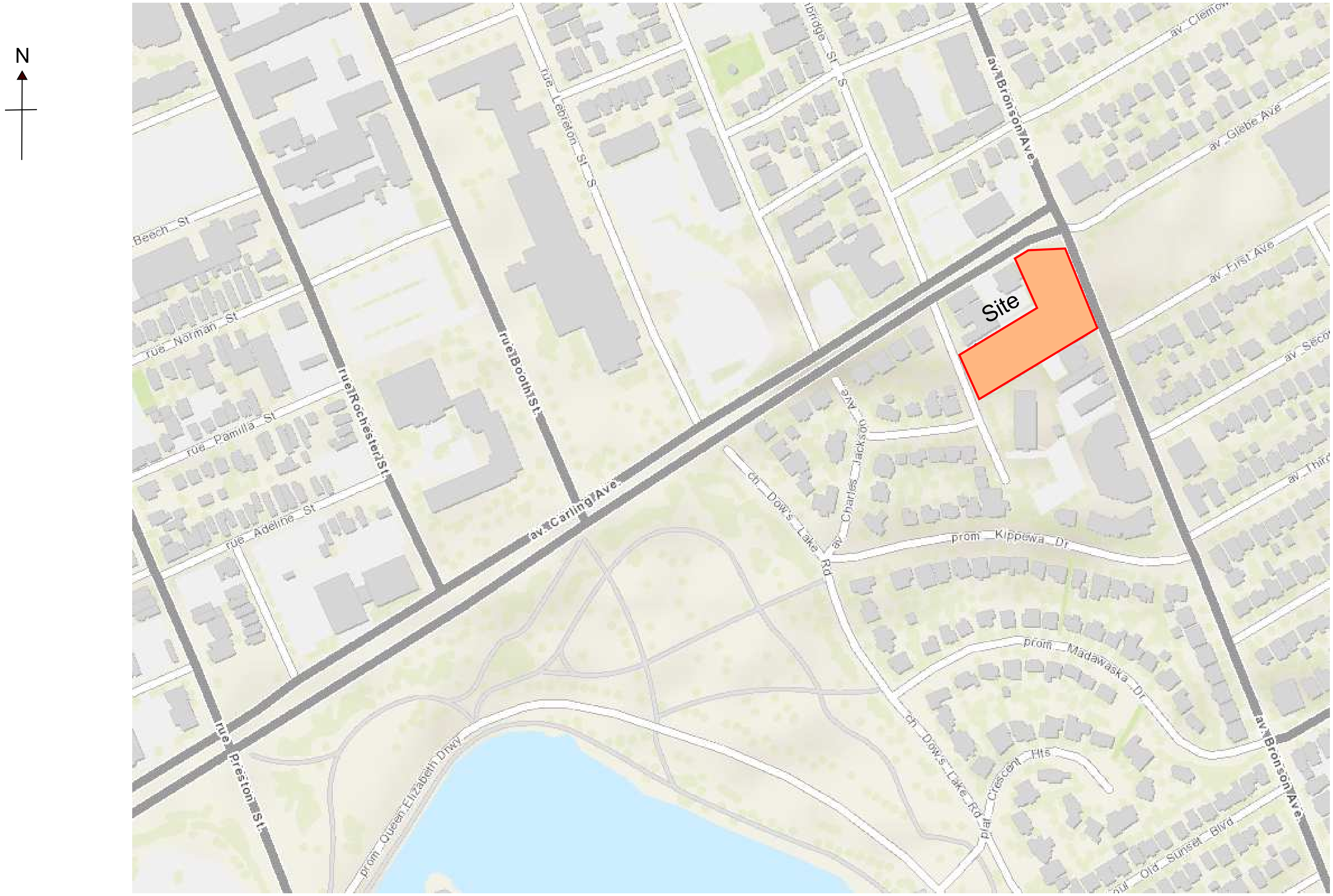
During construction, WSP should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of WSP's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in WSP's report. Adequate field review, observation and testing during construction are necessary for WSP 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, WSP'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 WSP 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 WSP be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

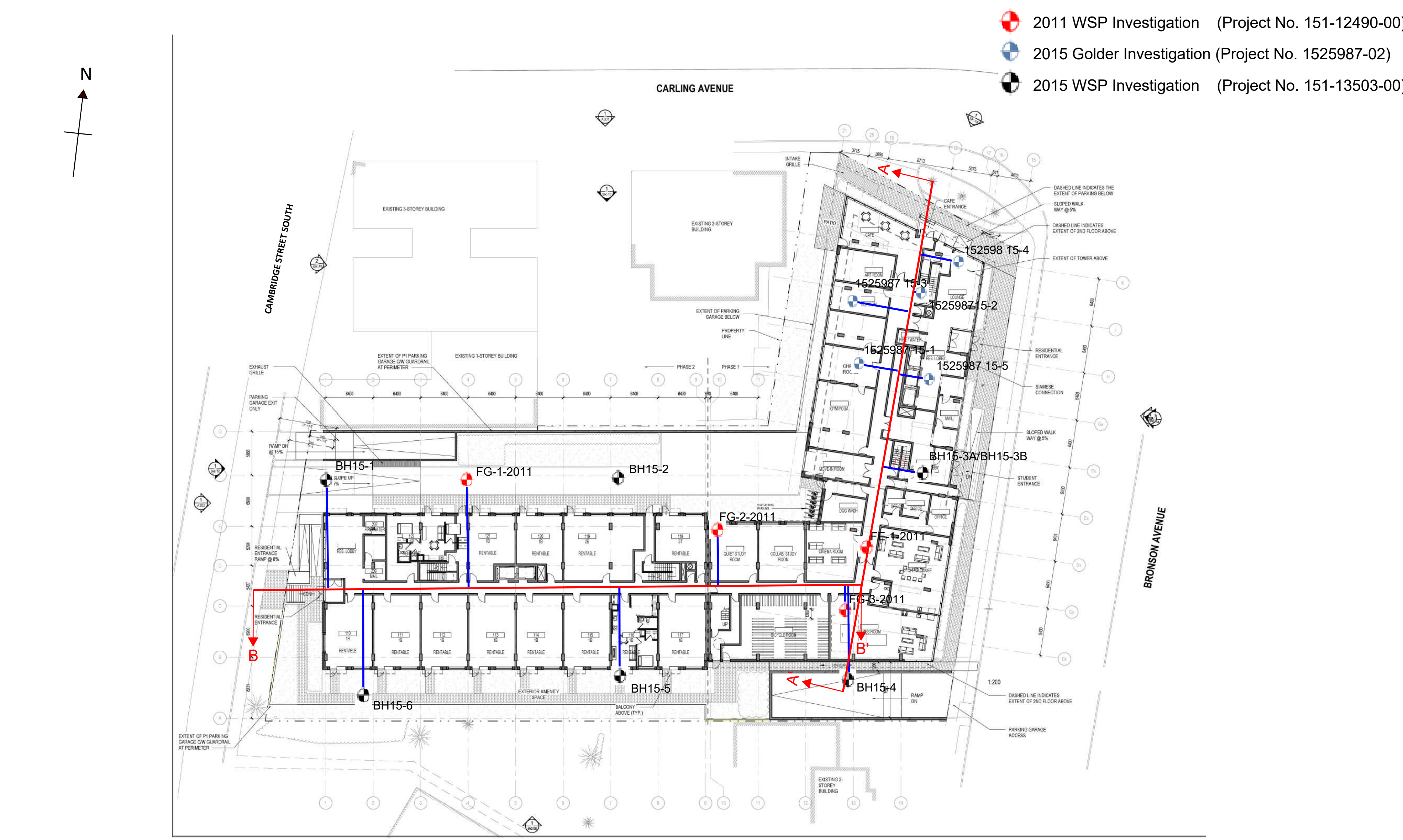
Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. WSP takes no responsibility for the effects of drainage unless specifically involved in the detailed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

DRAWINGS

Drawing 1 to 4



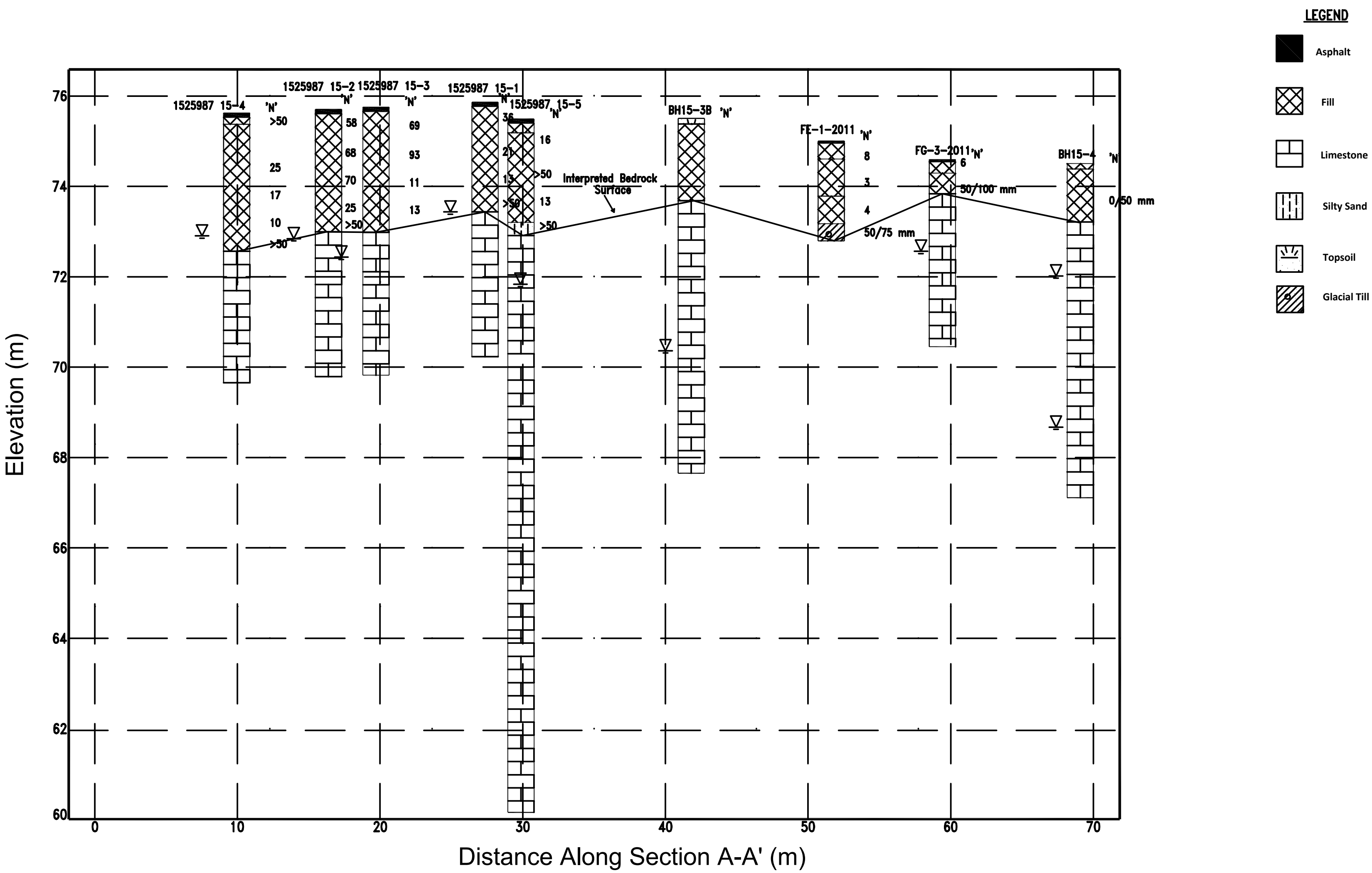
Client:	Katasa Groupe + Développement			Title:	Site Location Plan
Project#:	211-05706-00	DWG #:	1	Project:	Geotechnical Investigation Proposed Residential Mixed-Use Building, 770-774 Bronson Ave, Ottawa, Ontario
Drawn:	DW	Approved:	DW		
Date:	May 2021	Scale:	N. T. S.		wsp
Size:	Letter	Rev:	0		



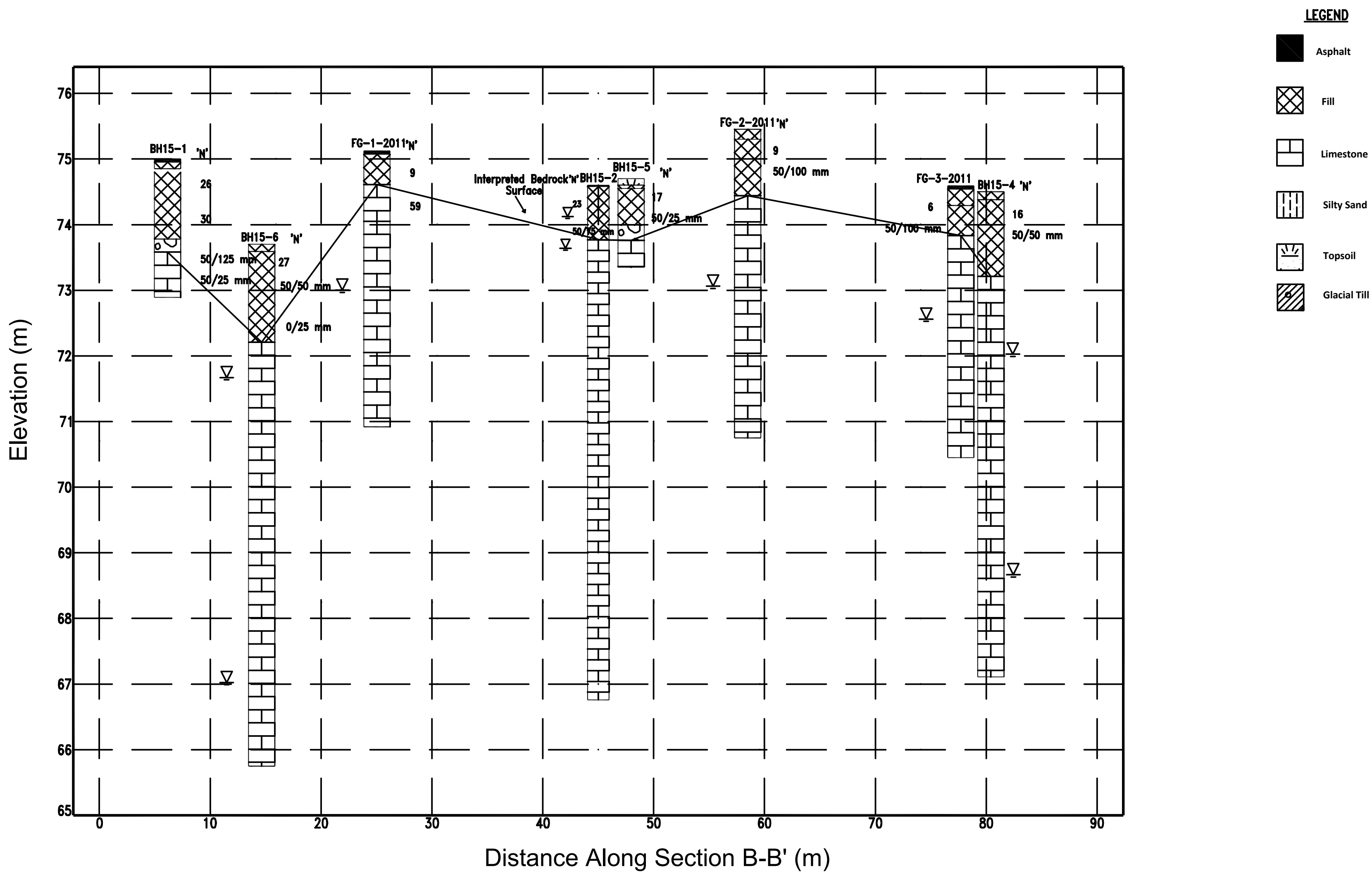
Client:	Katasa Groupe + Développement		
Project#:	211-05706-00	DWG #:	2
Drawn:	DW	Approved:	DW
Date:	May 2021	Scale:	N. T. S.
Size:	Letter	Rev:	0

Title: Borehole Location Plan

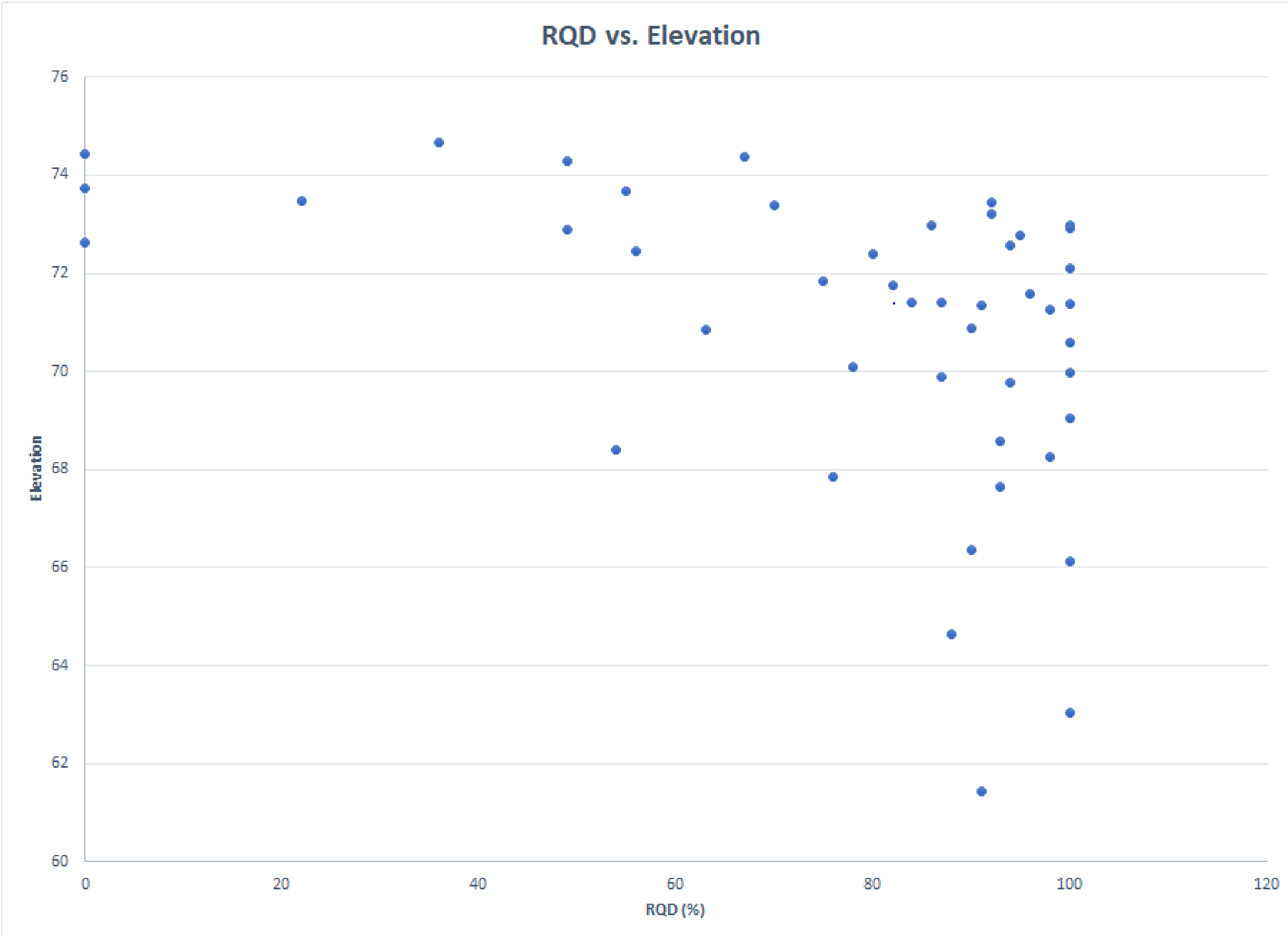
Project: Geotechnical Investigation
Proposed Residential Mixed-Use Building, 770-774 Bronson Ave, Ottawa, Ontario



Client:	Katasa Groupe + Développement			Title:	Profile Along Section A-A'
Project#:	211-05706-00	DWG #:	3A	Project:	Geotechnical Investigation Proposed Residential Mixed-Use Building, 770-774 Bronson Ave, Ottawa, Ontario
Drawn:	DW	Approved:	DW		
Date:	May 2021	Scale:	N. T. S.		
Size:	Letter	Rev:	0		



Client:	Katasa Groupe + Développement	Title:	Profile Along Section B-B'
Project#: 211-05706-00	DWG #: 3B	Project:	Geotechnical Investigation Proposed Residential Mixed-Use Building, 770-774 Bronson Ave, Ottawa, Ontario
Drawn: DW	Approved: DW		
Date: May 2021	Scale: N. T. S.		
Size: Letter	Rev: 0		



Client:	Katasa Groupe + Développement		
Project#:	211-05706-00	DWG #:	4
Drawn:	DW	Approved:	DW
Date:	May 2021	Scale:	N. T. S.
Size:	Letter	Rev:	0

Title:	RQD vs. Elevation
Project:	Geotechnical Investigation Proposed Residential Mixed-Use Building, 770-774 Bronson Ave, Ottawa, Ontario

APPENDIX A

Borehole Logs – Previous Investigations

BOREHOLE DRILLING RECORD : **FE-1-2011**

WSP

Prepared by: **David Feghali, ing.** Date (Start): **2011-12-09**
Reviewed by: **Pierre Jean** Date (End): **2011-12-09**

Project Name:	**Geotechnical Investigation**	Project Number:	**111-26060-00**
Site:	**Projected building between Bronson Ave and Cambridge S. St.**	Geographic Coordinates:	X = 445239 W Y = 5027674 N
Sector:	**Eastern Site Entrance**	Surface Elevation:	75 m *(Approximatif)*
Client:	**Samcon Inc.**	Plunge / Azimuth:	-90 deg / 0 deg

Drilling Company:	Forage André Roy Inc.
Drilling Equipment:	CME 55
Drilling Method:	Auger
Borehole Diameter:	200 mm
Drilling Fluid:	None

WELL DETAILS
COPING Elevation :
SCREEN Bottom Depth :
Length :
Opening :
WATER Elevation:
WATER Date:
▽ Water Level ▼ Free Phase

SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
▨ Undisturbed
⊠ Remoulded
■ Lost
◧ Cored

GEOTECHNICAL: R □ Shear (kPa) 30 60 90 120 I ■; SPT=N Value ▲; PENTEST △; RQD (%) ⊙; PLASTIC LIMIT w (%) ● LIQUID; 20 40 60 80

DEPTH / *ELEVATION* (m)	STRATIGRAPHY	DESCRIPTION	NUMBER	LABORATORY TESTING	DUPLICATE	TYPE & NO.	STATE	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)	GEOTECHNICAL	WELL DIAGRAM
		Ground surface.									
0.05 / *74.95*	■	Bituminous concrete.				MA-1	■				
0.40 / *74.60*		Fill: brown gravel and sand, trace to some silt. Loose compactness.				SS-2	⊠	64	7 5 3 3 (8)	▲	
1.22 / *73.78*		Fill : brown fine sand, some silt, trace gravel. Very loose compactness.				SS-3	⊠	83	2 2 1 1 (3)	▲	
1.83 / *73.17*		Glacial till (probable) : dark brown sandy silt, trace organic material, trace gravel. Very loose compactness.				SS-4	⊠	33	1 2 2 3 (4)	▲	
2.21 / *72.79*		Glacial till (probable): brown sandy and gravelly silt. Compact compactness. Split spoon refusal at 2.21 m.				SS-5	⊠	92	7 13 50/8 cm (R)	>>▲	
		End of borehole at 2.21 m.									

Depth scale (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0

BOREHOLE DRILLING RECORD : **FE-2-2011**

Prepared by: **David Feghali, ing.**
Reviewed by: **Pierre Jean**
Date (Start): **2011-12-09**
Date (End): **2011-12-09**

Project Name: **Geotechnical Investigation**
Site: **Projected building between Bronson Ave and Cambridge S. St.**
Sector: **Southwest of Site**
Client: **Samcon Inc.**

Project Number: **111-26060-00**
Geographic Coordinates: X = 445236 W
Y = 5027625 N
Surface Elevation: 75 m *(Approximatif)*
Plunge / Azimuth: -90 deg / 0 deg

Drilling Company: Forage André Roy Inc.
Drilling Equipment: CME 55
Drilling Method: Auger
Borehole Diameter: 200 mm
Drilling Fluid: None

WELL DETAILS
COPING Elevation :
SCREEN Bottom Depth :
Length :
Opening :
WATER Elevation:
WATER Date:
Water Level / Free Phase

SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

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PENTEST - Blow Counts/300mm
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Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
Undisturbed
Remoulded
Lost
Cored

DEPTH / ELEVATION (m)	DESCRIPTION	NUMBER	LABORATORY TESTING	DUPLICATE	TYPE & NO.	STATE	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)
	Ground surface.							
0.05 / 74.95	Bituminous concrete.				MA-1		70	
0.20	Fill: brown sand and gravel, trace to some silt. Compact compactness.				SS-2			5 7 7 12 (14)
0.30 / 74.70	Red brick debris.							
0.71 / 74.29	Fill: brown sand and gravel, trace red brick debris. Compact compactness.				SS-3		81	9 50/12 cm
0.88 / 74.12	Probable fill: dark brown sandy silt, trace organic material. Compact compactness. Split spoon refusal at 0.88 m.							
	End of borehole at 0.88 m.							

GEOTECHNICAL: R □ Shear (kPa) I ■ 30 60 90 120; SPT=N Value; PENTEST; RQD (%); PLASTIC LIMIT w (%) LIQUID 20 40 60 80

WELL DIAGRAM

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0

Projet : ENGLISH LOG FORAGE SAMCON.GPJ Type rapport : WSP_EN_WELL-GEOTECHNICAL ONLY Data Template : WSP_TEMPLATE_GEOTECH.GDT 2015-11-12

BOREHOLE DRILLING RECORD : **FG-1-2011**

WSP

Prepared by: **David Feghali, ing.** Date (Start): **2011-12-08**
Reviewed by: **Pierre Jean** Date (End): **2011-12-09**

Project Name:	**Geotechnical Investigation**	Project Number:	**111-26060-00**
Site:	**Projected building between Bronson Ave and Cambridge S. St.**	Geographic Coordinates:	X = 445187 W Y = 5027657 N
Sector:	**West of Site**	Surface Elevation:	75.12 m *(Geodetic)*
Client:	**Samcon Inc.**	Plunge / Azimuth:	-90 deg / 0 deg

Drilling Company:	Forage André Roy Inc.
Drilling Equipment:	CME 55
Drilling Method:	Auger / HQ Casing
Borehole Diameter:	200 mm / 96 mm
Drilling Fluid:	Water

WELL DETAILS

COPING Elevation :	75.08 m
SCREEN Bottom Depth :	4.2 m
Length :	3.66 m
Opening :	0.51 mm
WATER Elevation:	72.96 m
WATER Date:	2011-12-12

Water Level; Free Phase

SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
Undisturbed
Remoulded
Lost
Cored

GEOTECHNICAL: R (30) Shear (kPa) 60 90 I (120); SPT=N Value; PENTEST; RQD (%); PLASTIC LIMIT w (%) LIQUID (20 40 60 80)

DEPTH (m)	ELEVATION (m)	STRATIGRAPHY	DESCRIPTION	NUMBER	LABORATORY TESTING	DUPLICATE	TYPE & NO.	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)	GEOTECHNICAL (RQD %)	WELL DIAGRAM
			Ground surface.								
0.05	75.07		Bituminous concrete.				MA-1				
0.51	74.61		Probable fill: dark brown silt and sand, trace gravel, trace organic material. Loose compactness.				SS-2	83	7 4 5 23 (9)		
1.07	74.05		Weathered bedrock : limestone gravel, soil-like behavior. Very dense compactness.				SS-3	43	12 29 30 50/1 cm (59)		
4.20	70.92		Bedrock: grey fossiliferous limestone, from the Trenton Group. Rock quality: poor to fair, then excellent after 2.82 m of depth.		UCS		DC-4 DC-5 DC-6	83 (0) 99 (56) 100 (90)		0 56 90	
			End of borehole at 4.20 m.								

Projet : ENGLISH LOG FORAGE SAMCON.GPJ Type rapport : WSP_EN_WELL-GEOTECHNICAL ONLY Data Template : WSP_TEMPLATE_GEOTECH.GDT 2015-11-12

WSP

BOREHOLE DRILLING RECORD : **FG-2-2011**

Prepared by: **David Feghali, ing.**
Reviewed by: **Pierre Jean**

Date (Start): **2011-12-08**
Date (End): **2011-12-08**

Project Name:	**Geotechnical Investigation**	Project Number:	**111-26060-00**
Site:	**Projected building between Bronson Ave and Cambridge S. St.**	Geographic Coordinates:	X = 445219 W Y = 5027667 N
Sector:	**Site Centre**	Surface Elevation:	75.45 m *(Geodetic)*
Client:	**Samcon Inc.**	Plunge / Azimuth:	-90 deg / 0 deg

Drilling Company:	Forage André Roy Inc.
Drilling Equipment:	CME 55
Drilling Method:	Auger / HQ Casing
Borehole Diameter:	200 mm / 96 mm
Drilling Fluid:	Water

WELL DETAILS

COPING Elevation :	75.51 m
SCREEN Bottom Depth :	4.7 m
Length :	3.96 m
Opening :	0.51 mm
WATER Elevation:	73.12 m
WATER Date:	2011-12-12

▽ Water Level ▼ Free Phase

SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
Undisturbed
Remoulded
Lost
Cored

DEPTH / ELEVATION (m)	DESCRIPTION	NUMBER	LABORATORY TESTING	DUPLICATE	TYPE & NO.	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)
	Ground surface.						
75.45 0.15 75.30	Fill: brown sand and gravel, trace to some silt. Loose compactness.				SS-1	82	9 5 4 3 (9)
	Probable fill: dark brown sandy silt, trace gravel, trace organic material. Very loose to loose compactness.				SS-2	87	3 2 50/10 cm (R)
1.01 74.44	Bedrock: grey fossiliferous limestone, from the Trenton Group. Rock quality: poor to very poor, then good after 3.07 m of depth.				DC-3	81 (0)	
					DC-4	81 (49)	
					DC-5	73 (22)	
					DC-6	96 (80)	
			UCS				
4.70 70.75	End of borehole at 4.70 m.						

GEOTECHNICAL: R □ Shear (kPa) 30 60 90 120 I ■; SPT=N Value ▲; PENTEST △; RQD (%) ⊙; PLASTIC LIMIT w (%) LIQUID; 20 40 60 80

RQD plotted: 0, 49, 22, 80

Depth scale (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0

WELL DIAGRAM

BOREHOLE DRILLING RECORD : **FG-3-2011**

WSP

Prepared by: **David Feghali, ing.**
Reviewed by: **Pierre Jean**
Date (Start): **2011-12-09**
Date (End): **2011-12-09**

Project Name: **Geotechnical Investigation**
Site: **Projected building between Bronson Ave and Cambridge S. St.**
Sector: **East of Site**
Client: **Samcon Inc.**

Project Number: **111-26060-00**
Geographic Coordinates: X = 445240 W, Y = 5027666 N
Surface Elevation: 74.59 m *(Geodetic)*
Plunge / Azimuth: -90 deg / 0 deg

Drilling Company: Forage André Roy Inc.
Drilling Equipment: CME 55
Drilling Method: Auger / HQ Casing
Borehole Diameter: 200 mm / 96 mm
Drilling Fluid: Water

WELL DETAILS
COPING Elevation : 74.69 m
SCREEN Bottom Depth : 4.14 m
Length : 3.66 m
Opening : 0.51 mm
WATER Elevation: 72.66 m
WATER Date: 2011-12-12
▽ Water Level ▼ Free Phase

SAMPLE TYPE
DC - Diamond Core
SS - Split Spoon
PS - Piston Sample
TC - Hollow Tube
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TT - DT-32 Liner

ANALYSIS
AL - Atterberg Limits
GSA - Grain Size Analysis
PENTEST - Blow Counts/300mm
PL - Point Load Test
Sg - Specific Gravity
SPT - N Value (Blow Counts/300mm)
UCS - Uniaxial Compressive Strength
w - Moisture Content
wL - Liquidity Limit
wP - Plasticity Limit

SAMPLE STATE
Undisturbed
Remoulded
Lost
Cored

DEPTH / *ELEVATION* (m)	DESCRIPTION	LABORATORY TESTING	TYPE & NO.	% RECOVERY (RQD)	Blows Counts/6" (N Value = SPT)	GEOTECHNICAL
	Ground surface.					
0.05 / *74.54*	Bituminous concrete.		MA-1	62		
0.30 / *74.29*	Fill: brown sand and gravel, trace to some silt. Loose compactness.		SS-2		5 4 2 3 (6)	SPT ▲ 6
0.76 / *73.83*	Probable fill: dark brown sandy silt, trace to some gravel, trace organic material. Very loose compactness.		SS-3	80	2 50 /10 cm	
	Bedrock: grey fossiliferous limestone, from the Trenton Group. Rock quality: very poor, then good after 1.22 m of depth.		DC-4	56 (0)		RQD 0
			DC-5	100 (70)		RQD 70
		UCS	DC-6	100 (75)		RQD 75
4.14 / *70.45*	End of borehole at 4.14 m.					

Shear (kPa): R □ I ■ 30 60 90 120
SPT=N Value ▲ PENTEST △ RQD (%) ⊙
PLASTIC LIMIT w (%) LIQUID 20 40 60 80

Depth scale (m): 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5 5.0

Projet : ENGLISH LOG FORAGE SAMCON.GPJ Type rapport : WSP_EN_WELL-GEOTECHNICAL ONLY Data Template : WSP_TEMPLATE_GEOTECH.GDT 2015-11-12

PROJECT: 1525987

RECORD OF BOREHOLE: 15-1

LOCATION: N 5027695.2 ;E 445226.2

BORING DATE: March 25, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	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		75.86							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE		0.00							Flush Mount Casing
		FILL - (GW/SW) SAND and GRAVEL; grey brown; non-cohesive, moist, compact to very dense		0.10	1	SS	36				
		- Black staining from 0.25 m to 0.46 m									
1					2	SS	21				
					3	SS	13				
2					4	SS	>50				Bentonite Seal
		Borehole continued on RECORD OF DRILLHOLE 15-1		73.43 2.43							
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: JD
CHECKED: TMS

PROJECT: 1525987

RECORD OF DRILLHOLE: 15-1

SHEET 2 OF 2

LOCATION: N 5027695.2 ;E 445226.2

DRILLING DATE: March 25, 2015

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG:

DRILLING CONTRACTOR: Marathon Drilling

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	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 m	DISCONTINUITY DATA B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.	
		BEDROCK SURFACE		73.43																
3	Rotary Drill NQ Core	Fresh, thinly to medium bedded, grey, fine grained, non-porous LIMESTONE BEDROCK, with partings to thin interbeds of black shale		2.43	1															Bentonite Seal
4																				Silica Sand
5				70.23	2															38 mm Diam. PVC #10 Slot Screen
6		End of Drillhole		5.63																W.L. in Screen at Elev. 73.36 m on March 27, 2015
7																				
8																				
9																				
10																				
11																				
12																				

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: JD

CHECKED: TMS

PROJECT: 1525987

RECORD OF BOREHOLE: 15-2

SHEET 1 OF 2

LOCATION: N 5027707.7 ;E 445228.3

BORING DATE: March 24, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	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		75.70							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE		0.00							Flush Mount Casing
		FILL - (GW/SW) SAND and GRAVEL; grey brown; non-cohesive, moist, compact to very dense		0.10	1	SS	58				
1					2	SS	68				
					3	SS	70				
2					4	SS	25				Bentonite Seal
				72.99	5	SS	>50				
		Borehole continued on RECORD OF DRILLHOLE 15-2		2.71							
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: JD

CHECKED: TMS

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RECORD OF DRILLHOLE: 15-2

PROJECT: 1525987
SHEET 2 OF 2

LOCATION: N 5027707.7 ;E 445228.3
DRILLING DATE: March 24, 2015
DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---
DRILL RIG:
DRILLING CONTRACTOR: Marathon Drilling

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.	FLUSH % RETURN	INSTALLATION NOTES
		BEDROCK SURFACE	72.99			
3–5	Rotary Drill, NQ Core	Fresh, thinly to medium bedded, grey, fine grained, non-porous LIMESTONE BEDROCK, with partings to thin interbeds of black shale	2.71	1	100	Bentonite Seal; Silica Sand
				2	85	38 mm Diam. PVC #10 Slot Screen
6		End of Drillhole	69.79 / 5.91			W.L. in Screen at Elev. 72.79 m on March 27, 2015

Column headings: RECOVERY (TOTAL CORE %, SOLID CORE %), R.Q.D. %, FRACT. INDEX PER 0.25 m, DISCONTINUITY DATA (B Angle, DIP w.r.t. CORE AXIS, TYPE AND SURFACE DESCRIPTION, Jcon, Jr, Ja), HYDRAULIC CONDUCTIVITY K, cm/sec, Diametral Point Load Index (MPa), RMC -Q' AVG.

Depth scale: 3, 4, 5, 6, 7, 8, 9, 10, 11, 12

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: JD
CHECKED: TMS

MIS-RCK 004 1525987.GPJ GAL-MISS.GDT 08/21/15 JM

PROJECT: 1525987

RECORD OF BOREHOLE: 15-3

SHEET 1 OF 2

LOCATION: N 5027716.1 ;E 445220.9

BORING DATE: March 24, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE - DESCRIPTION	STRATA PLOT	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		75.75							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE		0.00							Flush Mount Casing
		FILL - (GW/SW) SAND and GRAVEL; grey brown; non-cohesive, moist, compact to very dense		0.10	1	SS	69				
1					2	SS	93				
					3	SS	11				
2					4	SS	13				Bentonite Seal
				72.98	5	SS	>50				
		Borehole continued on RECORD OF DRILLHOLE 15-3		2.77							
3											
4											
5											
6											
7											
8											
9											
10											

MIS-BHS 001 1525987.GPJ GAL-MIS.GDT 08/21/15 JM

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: JD

CHECKED: TMS

PROJECT: 1525987

RECORD OF DRILLHOLE: 15-3

SHEET 2 OF 2

LOCATION: N 5027716.1 ;E 445220.9

DRILLING DATE: March 24, 2015

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG:

DRILLING CONTRACTOR: Marathon Drilling

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	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 m	B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.	
		BEDROCK SURFACE		72.98																
3	Rotary Drill, NQ Core	Fresh, thinly to medium bedded, grey, fine grained, non-porous LIMESTONE BEDROCK, with partings to thin interbeds of black shale		2.77	1	85														Bentonite Seal
4																				Silica Sand
5					2															38 mm Diam. PVC #10 Slot Screen
				69.82																
6		End of Drillhole		5.93																W.L. in Screen at Elev. 72.38 m on March 27, 2015
7																				
8																				
9																				
10																				
11																				
12																				

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: JD
CHECKED: TMS

MIS-RCK 004 1525987.GPJ GAL-MISS.GDT 08/21/15 JM

PROJECT: 1525987

RECORD OF BOREHOLE: 15-4

LOCATION: N 5027715.5 ;E 445230.4

BORING DATE: March 24, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

Column headings: DEPTH SCALE METRES; BORING METHOD; SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m)); SAMPLES (NUMBER, TYPE, BLOWS/0.3m); 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

Boring method (0 to 3.06 m): Power Auger, 200 mm Diam. (Hollow Stem)

Description	Elev. / Depth (m)
GROUND SURFACE	75.62
ASPHALTIC CONCRETE	0.00
FILL - (SW) gravelly SAND; grey with dark grey staining; non-cohesive, moist, very dense	0.10
FILL - (GW/SW) SAND and GRAVEL, trace silt; grey brown; non-cohesive, moist, compact to very dense	0.25
Borehole continued on RECORD OF DRILLHOLE 15-4	72.56 / 3.06

Number	Type	Blows/0.3m
1	SS	>50
2	SS	25
3	SS	17
4	SS	10
5	SS	>50

Piezometer or standpipe installation: Flush Mount Casing; Bentonite Seal

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: JD

CHECKED: TMS

PROJECT: 1525987

RECORD OF DRILLHOLE: 15-4

LOCATION: N 5027715.5 ;E 445230.4

DRILLING DATE: March 24, 2015

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG:

DRILLING CONTRACTOR: Marathon Drilling

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	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 m	B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.	
		BEDROCK SURFACE		72.56																
	Rotary Drill NQ Core	Fresh, thinly to medium bedded, grey, fine grained, non-porous LIMESTONE BEDROCK, with partings and thin interbeds of black shale		3.06	1	95														Bentonite Seal
4																				Silica Sand
5					2	80														38 mm Diam. PVC #10 Slot Screen
6		End of Drillhole		69.65 5.97																W.L. in Screen at Elev. 72.87 m on March 27, 2015
7																				
8																				
9																				
10																				
11																				
12																				
13																				

MIS-RCK 004 1525987.GPJ GAL-MISS.GDT 08/21/15 JM

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: JD

CHECKED: TMS

RECORD OF BOREHOLE: 15-5

PROJECT: 1525987

SHEET 1 OF 3

LOCATION: N 5027698.4 ;E 445234.4

BORING DATE: June 19, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	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		75.49							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE		0.00							Cement Grout
		FILL - (SW) gravelly SAND, angular; grey (PAVEMENT STRUCTURE)		0.10 / 75.18							
		FILL - (SW) gravelly SAND; brown, contains cobbles; non-cohesive, moist, dense to very dense		0.31	1	SS	61				
1					2	SS	>50				Peltonite and Cement Grout
2					3	SS	32				
		(SM) SILTY SAND, trace gravel; brown, contains organic matter; non-cohesive, wet, very dense		73.20 / 2.29	4	SS	>50				
		Borehole continued on RECORD OF DRILLHOLE 15-5		72.91 / 2.58							
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: HEC

CHECKED: TMS

RECORD OF DRILLHOLE: 15-5

PROJECT: 1525987

LOCATION: N 5027698.4 ;E 445234.4

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: June 19, 2015

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon 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.	FLUSH COLOUR % RETURN	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION
		BEDROCK SURFACE	72.91			
	Rotary Drill NQ Core	Fresh, thinly to medium bedded, grey, fine grained, non-porous LIMESTONE BEDROCK, with partings to thin interbeds of black shale	2.58			
3				1	100	
						,BD,PL,RO
4				2	100	
						,BD,PL,RO
5						,BD,PL,RO
						,BD,PL,RO
				3	100	
6						
7				4	100	
						,BD,CU,RO
8						,BD,IR,RO
						,BD,PL,RO
				5	100	,BD,PL,RO
						,BD,PL,RO
9						,BD,PL,SM
10				6	100	
						,BD,PL,SM
						,BD,PL,RO
11		- Broken core from 10.85 m to 10.90 m				
		- Broken core from 11.35 m to 11.38 m				,BD,PL,RO
				7	100	
12						
				8		,BD,IR,RO

Peltonite and Cement Grout

CONTINUED NEXT PAGE

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: HEC

CHECKED: TMS

PROJECT: 1525987

RECORD OF DRILLHOLE: 15-5

SHEET 3 OF 3

LOCATION: N 5027698.4 ;E 445234.4

DRILLING DATE: June 19, 2015

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

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.	FLUSH % RETURN	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---					
13	Rotary Drill NQ Core	Fresh, thinly to medium bedded, grey, fine grained, non-porous LIMESTONE BEDROCK, with partings to thin interbeds of black shale		8	100		Silica Sand
14–15			60.15 / 15.34	9	100	,BD,PL,RO ,BD,PL,RO ,BD,PL,RO ,BD,PL,RO ,BD,PL,RO	50 mm Diam. PVC #10 Slot Screen
		End of Drillhole					W.L. in Screen at Elev. 71.83 m on August 20, 2015

Depth scale: 13, 14, 15, 16, 17, 18, 19, 20, 21, 22

MIS-RCK 004 1525987.GPJ GAL-MISS.GDT 08/21/15 JM

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: HEC

CHECKED: TMS

BOREHOLE DRILLING RECORD : BH15-1

WSP

Prepared by: **Kathryn Maton** Date (Start): **1/11/2016**
Reviewed by: **Phil Romeril** Date (End): **1/11/2016**

Project Name: **Phase Two Environmental Site Assessment**
Site: **774 Bronson Avenue and 557 Cambridge Street South, Ottawa, Ontario**
Sector:
Client: **Textbook Student Suites**

Project Number: **151-13503-00**
Geographic Coordinates: X = 445171 mE, Y = 5027646 mN
Surface Elevation: 75 m *(Approximate)*
Top of PVC Elevation:

Drilling Company: Downing Estate Drilling Ltd.
Drilling Equipment: CME 55
Drilling Method: Auger
Borehole Diameter: 200 mm
Drilling Fluid: None
Sampling Method: Split Spoon

ODOUR: F - Light; M - Medium; P - Persistent
VISUAL: D - Disseminated Product; S - Saturated with Product

SAMPLE TYPE: DC - Diamond Corer; SS - Split Spoon; MA - Manual Auger; TR - Trowel; ST - Shelby Tube; TU - DT32 Liner

Water Level / Free Phase

CHEMICAL ANALYSIS
PCB Poly-Chlorinated Biphenyls
BTEX Benzene, Toluene, Ethylbenzene, Xylene
Inorg. C. Inorganic Compounds
Phenol. C. Phenolic Compounds
VOC Volatil Organic Compounds (MAH & CAH)
Diox. & Fur. Dioxins & Furans
CAH Chlorinated Aliphatic Hydrocarbons
MAH Monocyclic Aromatic Hydrocarbons
PAH Polycyclic Aromatic Hydrocarbons
PH C_{10}-C_{50} Petroleum Hydrocarbons C_{10}-C_{50}
PH F1-F4 Petroleum Hydrocarbons F1-F4 (C_{10}-C_{50})
Metals Arsenic, Barium, Cadmium, Chromium, Cobalt, Copper, Lead, Manganese, Molybdenum, Nickel, Silver, Tin, Zinc.
HWR Leacheate Tests (Haz. Waste Reg.)

DEPTH / ELEVATION (m)	DESCRIPTION	VAPOR CONC. (ppm OR % LIE)	ODOUR (F M P)	VISUAL (D S)	SAMPLE TYPE	% RECUPERATION	N (Blow/6")	NUMBER	ANALYSIS	DUPLICATE	MONITORING WELL DIAGRAM	MONITORING WELL DESCRIPTION	REMARKS
	Ground surface.												
0.05 / 74.95	ASPHALT	1.6						BH15-1 1					
0.15 / 74.85	FILL, crushed limestone gravel and sand												
	FILL, sand and crushed limestone gravel, with some asphalt and pieces of brick, compact, dry, grey	1.2			SS	40	12 14 7	BH15-1 2	PHCs F1-F4 BTEX PAH VOC				
		1.3			SS	50	5 5 25 30	BH15-1 3					
1.22 / 73.78	GRAVEL, shale fragments	2.1			SS	37	39 50 50-5"	BH15-1 4					
1.42 / 1.44 / 73.56	BEDROCK, shale	6.1						BH15-1 5					
	BEDROCK, limestone with black shale partings				SS	0	50-1"						
2.11 / 72.89	*Auger Refusal at 2.11 mbgs*												
	End of borehole at 2.11 m.												

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0

Projet : PHASE II ESA - 774 BRONSON AVE.GPJ Type rapport : WSP_EN_WELL-ENVIRONMENTAL Data Template : WSP_TEMPLATE_GEOTECH.GDT 2/9/2016

BOREHOLE DRILLING RECORD : **BH15-2**

Page 1 of 1

Prepared by: **Kathryn Maton** Date (Start): **1/11/2016**
Reviewed by: **Phil Romeril** Date (End): **1/13/2016**

Project Name: **Phase Two Environmental Site Assessment**
Site: **774 Bronson Avenue and 557 Cambridge Street South, Ottawa, Ontario**
Sector:
Client: **Textbook Student Suites**

Project Number: **151-13503-00**
Geographic Coordinates: X = 445204 mE, Y = 5027668 mN
Surface Elevation: 75.6 m *(Approximate)*
Top of PVC Elevation: 76.62 m *(Approximate)*

Drilling Company: Downing Estate Drilling Ltd.
Drilling Equipment: CME 55
Drilling Method: Auger / HQ Casing
Borehole Diameter: 200 mm / 96 mm
Drilling Fluid: Municipal Water
Sampling Method: Split Spoon

ODOUR: F - Light; M - Medium; P - Persistent
VISUAL: D - Disseminated Product; S - Saturated with Product
SAMPLE TYPE: DC - Diamond Corer; SS - Split Spoon; MA - Manual Auger; TR - Trowel; ST - Shelby Tube; TU - DT32 Liner
Water Level / Free Phase

CHEMICAL ANALYSIS
PCB Poly-Chlorinated Biphenyls
BTEX Benzene, Toluene, Ethylbenzene, Xylene
Inorg. C. Inorganic Compounds
Phenol. C. Phenolic Compounds
VOC Volatil Organic Compounds (MAH & CAH)
Diox. & Fur. Dioxins & Furans
CAH Chlorinated Aliphatic Hydrocarbons
MAH Monocyclic Aromatic Hydrocarbons
PAH Polycyclic Aromatic Hydrocarbons
PH C_{10}-C_{50} Petroleum Hydrocarbons C_{10}-C_{50}
PH F1-F4 Petroleum Hydrocarbons F1-F4 (C_{10}-C_{50})
Metals Arsenic, Barium, Cadmium, Chromium, Cobalt, Copper, Lead, Manganese, Molybdenum, Nickel, Silver, Tin, Zinc.
HWR Leacheate Tests (Haz. Waste Reg.)

DEPTH / ELEVATION (m)	DESCRIPTION	VAPOR CONC. (ppm OR % LIE)	SAMPLE TYPE	% RECUPERATION	N (Blow/6")	NUMBER	ANALYSIS	REMARKS
	Ground surface.							
0.05	ASPHALT							
0.15 / 75.45	FILL, crushed limestone gravel and sand	8.5	SS	18	7 11 12 14	BH15-2 1		
0.83 / 74.77	FILL, sandy silt and crushed limestone gravel with trace clay, compact , saturated, brown, black crystal material observed	13.8	SS	12	6 50-3"	BH15-2 2	PHCs F1-F4 VOC PAHs BTEX	RQD = 36
	BEDROCK, limestone with black shale partings *Auger Refusal at 1.14 mbgs, HQ Coring begins*		DC	100				RQD = 67
			DC	100				
			DC	100				RQD = 49
			DC	100				RQD = 87
			DC	100				RQD = 87
			DC	100				RQD = 54
7.82 / 67.78	End of borehole at 7.82 m.							

MONITORING WELL

SCREEN
Diam.: 31 mm
Open.: 0.25 mm
Length: 1.52 m

WATER
Depth: m
Elev.: 74.12 m
Date: 1/19/2016

Bentonite

SCREEN
Diam.: 31 mm
Open.: 0.25 mm
Length: 1.52 m

WATER
Depth: m
Elev.: 73.63 m
Date: 1/19/2016

Sand

PVC Slotted Pipe

BOREHOLE DRILLING RECORD : **BH15-3A**

WSP

Prepared by: **Kathryn Maton**
Reviewed by: **Phil Romeril**

Date (Start): **1/11/2016**
Date (End): **1/12/2016**

Project Name: **Phase Two Environmental Site Assessment**
Site: **774 Bronson Avenue and 557 Cambridge Street South, Ottawa, Ontario**
Sector:
Client: **Textbook Student Suites**

Project Number: **151-13503-00**
Geographic Coordinates: X = 445240 mE, Y = 5027685 mN
Surface Elevation: 75.5 m *(Approximate)*
Top of PVC Elevation: 76.53 m *(Approximate)*

Drilling Company: Downing Estate Drilling Ltd.
Drilling Equipment: CME 55
Drilling Method: Auger / HQ Casing
Borehole Diameter: 200 mm
Drilling Fluid: Municipal Water
Sampling Method: Split Spoon

ODOUR
F - Light
M - Medium
P - Persistent

VISUAL
D - Disseminated Product
S - Saturated with Product

SAMPLE TYPE
DC - Diamond Corer
SS - Split Spoon
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TU - DT32 Liner

Water Level ▽ Free Phase ▼

CHEMICAL ANALYSIS

PCB	Poly-Chlorinated Biphenyls	MAH	Monocyclic Aromatic Hydrocarbons
BTEX	Benzene, Toluene, Ethylbenzene, Xylene	PAH	Polycyclic Aromatic Hydrocarbons
Inorg. C.	Inorganic Compounds	PH C_{10}-C_{50}	Petroleum Hydrocarbons C_{10}-C_{50}
Phenol. C.	Phenolic Compounds	PH F1-F4	Petroleum Hydrocarbons F1-F4 (C_{10}-C_{50})
VOC	Volatil Organic Compounds (MAH & CAH)	Metals	Arsenic, Barium, Cadmium, Chromium, Cobalt, Copper, Lead, Manganese, Molybdenum, Nickel, Silver, Tin, Zinc.
Diox. & Fur.	Dioxins & Furans	HWR	Leacheate Tests (Haz. Waste Reg.)
CAH	Chlorinated Aliphatic Hydrocarbons		

DEPTH / ELEVATION (m)	GEOLOGY / LITHOLOGY – DESCRIPTION	VAPOR CONC. (ppm OR % LIE)	ODOUR (F M P)	VISUAL (D S)	SAMPLE TYPE	% RECUPERATION	N (Blow/6")	NUMBER	ANALYSIS	DUPLICATE	MONITORING WELL – DESCRIPTION	REMARKS
	Ground surface.											
75.50	TOPSOIL	15.2			SS	37	11 8 6 5	BH15-3A 1				
0.12 / 0.13 / 75.37	FILL, Black carbon ashes	12						BH15-3A 2			SCREEN Diam.: 51 mm Open.: 0.25 mm Length: 1.52 m	
	FILL, silty sand, dry to moist, compact, brown										WATER Depth: 2.35 m Elev.: 74.18 m Date: 1/19/2016	
0.5												0.5
		12			SS	12	2 14 50-3"	BH15-3A 3			Bentonite	
1.0												1.0
	becoming saturated	13.1			SS	64	7 14 22 31	BH15-3A 4	PHCs F1-F4 BTEX VOC PAH Duplicate			
1.5											Sand	1.5
1.75												
1.82 / 73.68	FILL, crushed limestone gravel and sand										PVC Slotted Pipe	
	FILL, silty sand and crushed limestone gravel, saturated, compact, brown	21.3			SS	29	37 35 50-2"	BH15-3A 5				
2.0												2.0
2.18 / 73.32	BEDROCK, limestone with black shale partings											
2.5												2.5
2.56 / 72.94	*Auger Refusal at 2.56 mbgs*											
	End of borehole at 2.56 m.											
3.0												3.0

Projet : PHASE II ESA - 774 BRONSON AVE.GPJ Type rapport : WSP_EN_WELL-ENVIRONMENTAL Data Template : WSP_TEMPLATE_GEOTECH.GDT 2/9/2016

BOREHOLE DRILLING RECORD : BH15-3B

WSP

Prepared by: **Kathryn Maton**
Reviewed by: **Phil Romeril**
Date (Start): **1/11/2016**
Date (End): **1/12/2016**

Project Name: **Phase Two Environmental Site Assessment**
Site: **774 Bronson Avenue and 557 Cambridge Street South, Ottawa, Ontario**
Sector:
Client: **Textbook Student Suites**

Project Number: **151-13503-00**
Geographic Coordinates: X = 445240 mE
Y = 5027685 mN
Surface Elevation: 75.5 m *(Approximate)*
Top of PVC Elevation: 77.468 m *(Approximate)*

Drilling Company: Downing Estate Drilling Ltd.
Drilling Equipment: CME 55
Drilling Method: Auger / HQ Casing
Borehole Diameter: 200 mm / 96 mm
Drilling Fluid: Municipal Water
Sampling Method: Diamond Corer

ODOUR
F - Light
M - Medium
P - Persistent

VISUAL
D - Disseminated Product
S - Saturated with Product

SAMPLE TYPE
DC - Diamond Corer
SS - Split Spoon
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TU - DT32 Liner

Water Level   Free Phase

CHEMICAL ANALYSIS
PCB Poly-Chlorinated Biphenyls
BTEX Benzene, Toluene, Ethylbenzene, Xylene
Inorg. C. Inorganic Compounds
Phenol. C. Phenolic Compounds
VOC Volatil Organic Compounds (MAH & CAH)
Diox. & Fur. Dioxins & Furans
CAH Chlorinated Aliphatic Hydrocarbons
MAH Monocyclic Aromatic Hydrocarbons
PAH Polycyclic Aromatic Hydrocarbons
PH C_{10}-C_{50} Petroleum Hydrocarbons C_{10}-C_{50}
PH F1-F4 Petroleum Hydrocarbons F1-F4 (C_{10}-C_{50})
Metals Arsenic, Barium, Cadmium, Chromium, Cobalt, Copper, Lead, Manganese, Molybdenum, Nickel, Silver, Tin, Zinc.
HWR Leacheate Tests (Haz. Waste Reg.)

DEPTH / ELEVATION (m)	LITHOLOGY	DESCRIPTION	VAPOR CONC. (ppm OR % LIE)	ODOUR (F M P)	VISUAL (D S)	SAMPLE TYPE	% RECUPERATION	N (Blow/6")	NUMBER	ANALYSIS	DUPLICATE	MONITORING WELL DIAGRAM	MONITORING WELL DESCRIPTION	REMARKS
		Ground surface.												
75.50		FILL, see soil description on BH15-3A												
		Casing refusal at 1.8 mbgs, HQ coring begins				DC	100							RQD = 55
2.18 / 73.32		BEDROCK, limestone with black shale partings												
						DC	100							RQD = 92
						DC	100							RQD = 98
													Bentonite	
													SCREEN Diam.: 51 mm Open.: 0.25 mm Length: 1.52 m	
						DC	100						WATER Depth: 7.11 m Elev.: 70.36 m Date: 1/19/2016	RQD = 94
													Sand	
													PVC Slotted Pipe	
						DC	100							RQD = 98
7.85 / 67.65		End of borehole at 7.85 m.												

Depth scale (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5, 7.0, 7.5, 8.0, 8.5

BOREHOLE DRILLING RECORD : BH15-4

Prepared by: **Kathryn Maton** Date (Start): **1/11/2016**
Reviewed by: **Phil Romeril** Date (End): **1/13/2016**

Project Name: **Phase Two Environmental Site Assessment**
Site: **774 Bronson Avenue and 557 Cambridge Street South, Ottawa, Ontario**
Sector:
Client: **Textbook Student Suites**

Project Number: **151-13503-00**
Geographic Coordinates: X = 445246 mE, Y = 5027658 mN
Surface Elevation: 74.5 m *(Approximate)*
Top of PVC Elevation: 75.53 m *(Approximate)*

Drilling Company: Downing Estate Drilling Ltd.
Drilling Equipment: CME 55
Drilling Method: Auger / HQ Casing
Borehole Diameter: 200 mm / 96 mm
Drilling Fluid: Municipal Water
Sampling Method: Split Spoon

ODOUR
F - Light
M - Medium
P - Persistent

VISUAL
D - Disseminated Product
S - Saturated with Product

SAMPLE TYPE
DC - Diamond Corer
SS - Split Spoon
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TU - DT32 Liner

Water Level / Free Phase

CHEMICAL ANALYSIS

PCB	Poly-Chlorinated Biphenyls
BTEX	Benzene, Toluene, Ethylbenzene, Xylene
Inorg. C.	Inorganic Compounds
Phenol. C.	Phenolic Compounds
VOC	Volatil Organic Compounds (MAH & CAH)
Diox. & Fur.	Dioxins & Furans
CAH	Chlorinated Aliphatic Hydrocarbons
MAH	Monocyclic Aromatic Hydrocarbons
PAH	Polycyclic Aromatic Hydrocarbons
PH C_{10}-C_{50}	Petroleum Hydrocarbons C_{10}-C_{50}
PH F1-F4	Petroleum Hydrocarbons F1-F4 (C_{10}-C_{50})
Metals	Arsenic, Barium, Cadmium, Chromium, Cobalt, Copper, Lead, Manganese, Molybdenum, Nickel, Silver, Tin, Zinc.
HWR	Leacheate Tests (Haz. Waste Reg.)

DEPTH / ELEVATION (m)	DESCRIPTION (GEOLOGY / LITHOLOGY)	VAPOR CONC. (ppm OR % LIE)	ODOUR	VISUAL	SAMPLE TYPE	% RECUPERATION	N (Blow/6")	NUMBER	ANALYSIS	DUPLICATE	MONITORING WELL DESCRIPTION	REMARKS
	Ground surface.											
0.12 / *74.38*	FILL, crushed limestone gravel	11.9			SS	37	8 8 8 6	BH15-4 1	Metals and Inorganics			
	FILL, silty sand and crushed limestone gravel	13			SS	46	4 6 50-2"	BH15-4 2				
0.96 / *73.54*	BEDROCK, limestone with black shale partings *Auger Refusal at 1.29 mbgs, HQ coring begins*				DC	100					SCREEN Diam.: 31 mm Open.: 0.25 mm Length: 1.52 m; WATER Depth: m Elev.: 72.08 m Date: 1/19/2016	RQD = 92
					DC	100						RQD = 96
					DC	100					Bentonite; SCREEN Diam.: 31 mm Open.: 0.25 mm Length: 1.52 m; WATER Depth: m Elev.: 68.67 m Date: 1/19/2016	RQD = 78
					DC	100					Sand; PVC Slotted Pipe	RQD = 93
7.39 / *67.09*	End of borehole at 7.41 m.											

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5, 7.0, 7.5, 8.0, 8.5

BOREHOLE DRILLING RECORD : **BH15-5**

Page 1 of 1

WSP

Prepared by: **Kathryn Maton** Date (Start): **1/11/2016**
Reviewed by: **Phil Romeril** Date (End): **1/11/2016**

Project Name: **Phase Two Environmental Site Assessment**
Site: **774 Bronson Avenue and 557 Cambridge Street South, Ottawa, Ontario**
Sector:
Client: **Textbook Student Suites**

Project Number: **151-13503-00**
Geographic Coordinates: X = 445217 mE, Y = 5027643 mN
Surface Elevation: 74.7 m *(Approximate)*
Top of PVC Elevation:

Drilling Company: Downing Estate Drilling Ltd.
Drilling Equipment: CME 55
Drilling Method: Auger
Borehole Diameter: 200 mm
Drilling Fluid: None
Sampling Method: Split Spoon

ODOUR
F - Light
M - Medium
P - Persistent

VISUAL
D - Disseminated Product
S - Saturated with Product

SAMPLE TYPE
DC - Diamond Corer
SS - Split Spoon
MA - Manual Auger
TR - Trowel
ST - Shelby Tube
TU - DT32 Liner

Water Level   Free Phase

CHEMICAL ANALYSIS
PCB Poly-Chlorinated Biphenyls
BTEX Benzene, Toluene, Ethylbenzene, Xylene
Inorg. C. Inorganic Compounds
Phenol. C. Phenolic Compounds
VOC Volatil Organic Compounds (MAH & CAH)
Diox. & Fur. Dioxins & Furans
CAH Chlorinated Aliphatic Hydrocarbons
MAH Monocyclic Aromatic Hydrocarbons
PAH Polycyclic Aromatic Hydrocarbons
PH C_{10}-C_{50} Petroleum Hydrocarbons C_{10}-C_{50}
PH F1-F4 Petroleum Hydrocarbons F1-F4 (C_{10}-C_{50})
Metals Arsenic, Barium, Cadmium, Chromium, Cobalt, Copper, Lead, Manganese, Molybdenum, Nickel, Silver, Tin, Zinc.
HWR Leacheate Tests (Haz. Waste Reg.)

DEPTH / ELEVATION (m)	LITHOLOGY	DESCRIPTION	VAPOR CONC. (ppm OR % LIE)	ODOUR (F M P)	VISUAL (D S)	SAMPLE TYPE	% RECUPERATION	N (Blow/6")	NUMBER	ANALYSIS	DUPLICATE	MONITORING WELL DIAGRAM	MONITORING WELL DESCRIPTION	REMARKS
		Ground surface.												
74.70		TOP SOIL	8			SS	50	2 10 7 4	BH15-5-1 1					
0.15 0.17 / 74.53		FILL, pieces of asphalt	1.3						BH15-5-2 (0.15-0.17)	Metals and Inorganics				
		FILL, sand and crushed limestone gravel with trace pieces of brick	1.2						BH15-5-3 2					
0.61 / 74.09		FILL, topsoil with some pieces of wood, compact, moist, dark brown	1.3			SS	41	4 32 50-1"	BH15-5-4 3					
0.71 / 73.99		GRAVEL and sand	1						BH15-5-5 4					
0.94 / 73.76		BEDROCK, limestone with black shale partings												
1.35 / 73.35		*Auger Refusal at 1.35 mbgs*												
		End of borehole at 1.35 m.												

Depth scale: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0

Projet : PHASE II ESA - 774 BRONSON AVE.GPJ Type rapport : WSP_EN_WELL-ENVIRONMENTAL Data Template : WSP_TEMPLATE_GEOTECH.GDT 2/9/2016

BOREHOLE DRILLING RECORD : **BH15-6**

Prepared by: **Kathryn Maton** Date (Start): **1/11/2016**
Reviewed by: **Phil Romeril** Date (End): **1/13/2016**

Project Name: **Phase Two Environmental Site Assessment**
Site: **774 Bronson Avenue and 557 Cambridge Street South, Ottawa, Ontario**
Sector:
Client: **Textbook Student Suites**

Project Number: **151-13503-00**
Geographic Coordinates: X = 445189 mE, Y = 5027623 mN
Surface Elevation: 73.7 m *(Approximate)*
Top of PVC Elevation: 74.705 m *(Approximate)*

Drilling Company: Downing Estate Drilling Ltd.
Drilling Equipment: CME 55
Drilling Method: Auger / HQ Casing
Borehole Diameter: 200 mm / 96 mm
Drilling Fluid: Municipal Water
Sampling Method: Split Spoon

ODOUR: F - Light; M - Medium; P - Persistent
VISUAL: D - Disseminated Product; S - Saturated with Product
Water Level; Free Phase

SAMPLE TYPE: DC - Diamond Corer; SS - Split Spoon; MA - Manual Auger; TR - Trowel; ST - Shelby Tube; TU - DT32 Liner

CHEMICAL ANALYSIS:
- PCB: Poly-Chlorinated Biphenyls
- BTEX: Benzene, Toluene, Ethylbenzene, Xylene
- Inorg. C.: Inorganic Compounds
- Phenol. C.: Phenolic Compounds
- VOC: Volatil Organic Compounds (MAH & CAH)
- Diox. & Fur.: Dioxins & Furans
- CAH: Chlorinated Aliphatic Hydrocarbons
- MAH: Monocyclic Aromatic Hydrocarbons
- PAH: Polycyclic Aromatic Hydrocarbons
- PH C_{10}-C_{50}: Petroleum Hydrocarbons C_{10}-C_{50}
- PH F1-F4: Petroleum Hydrocarbons F1-F4 (C_{10}-C_{50})
- Metals: Arsenic, Barium, Cadmium, Chromium, Cobalt, Copper, Lead, Manganese, Molybdenum, Nickel, Silver, Tin, Zinc.
- HWR: Leacheate Tests (Haz. Waste Reg.)

DEPTH / *ELEVATION* (m)	DESCRIPTION (GEOLOGY / LITHOLOGY)	VAPOR CONC. (ppm OR % LIE)	ODOUR	VISUAL	SAMPLE TYPE	% RECUPERATION	N (Blow/6")	NUMBER	ANALYSIS	DUPLICATE	MONITORING WELL DESCRIPTION	REMARKS
	Ground surface.											
0.11 / *73.59*	FILL, top soil, with some pieces of brick, dry, compact, dark brown	11.1			SS	39	3 14 13 8	BH15-6 1	Metals and Inorganics	Duplicate BH15-6 2		
73.09	FILL, crushed limestone gravel and sand, dry, compact, brown-grey; becoming silty with trace pieces of brick, saturated, brown	13.8										
		12.1			SS	25	14 50-2"	BH15-6 3				
					DC	27					SCREEN: Diam.: 31 mm; Open.: 0.25 mm; Length: 1.52 m	
1.49 / *72.21*	*Auger Refusal at 1.52 mbgs, HQ Coring begins*	11.9			SS	16	11 50-1"	BH15-6 4				RQD = 63
					DC	92					WATER: Depth: m; Elev.: 71.67 m; Date: 1/19/2016	
	BEDROCK, limestone and black shale partings				DC	104						RQD = 100
					DC	95					Bentonite	RQD = 76
					DC	105					SCREEN: Diam.: 31 mm; Open.: 0.25 mm; Length: 1.52 m; WATER: Depth: m; Elev.: 67.05 m; Date: 1/19/2016	RQD = 90
					DC	100					Sand; PVC Slotted Pipe	RQD = 75
7.95 / *65.75*	End of borehole at 7.95 m.											

Depth scale (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5, 7.0, 7.5, 8.0, 8.5

APPENDIX B

Laboratory Testing Results – Previous Investigation

Labo S.M. inc.
Une division du Groupe S.M. inc
SMi

■ 740, rue Galt Ouest, 2e étage, Sherbrooke (Qc) J1H 1Z3 Tél: (819) 566-8855 Fax: (819) 566-0224
☐ 1471, boul. Lionel-Boulet, Varennes (Qc) J3X 1P7 Tél: (450) 652-6151 Fax: (450) 652-6451
☐ 75, rue Queen, bureau 5200, Montréal (Qc) H3C 2N6 Tél: (514) 982-6001 Fax: (514) 982-6106
☐ 4540, rue Laval, Lac-Mégantic (Qc) G6B 1C5 Tél: (819) 583-4255 Fax: (819) 583-1997
☐ 2111, boul. Fernand-Lafontaine, Longueuil (Qc) J4G 2J4 Tél: (450) 651-0981 Fax: (450) 651-9542

Rapport n° 1106941

RAPPORT D'ESSAIS
MESURE DE LA RÉSISTANCE EN COMPRESSION SUR CAROTTES DE ROC
ASTM D 7012-07

Numéro de dossier : F115220001
Numéro de laboratoire : 11-10906/11-10908/11-10909
Projet : Étude géotechnique - Reconstruction des conduites d'eau et d'égou
Client : Génivar - Gatineau

Conditionnement : sec
Matériau de coiffe : meule
Température de confinement : 22
Prélevé par : nd ,le
Réalisé par : D. Laroche ,le 11-12-15
Site :
Contrat :

Date rupturée	Forage N°	# échant.	Profondeur d'essais (m)	Diamètre 1	2	3	moyen	Longueur initiale	meulée	Rapport L/D	Charge	Résistance en compression	Temps de rupture
				(mm)				(mm)			(kN)	(MPa)	(sec)
11-12-15	FG-1-2011	11-10906	2,5 à 2,7 m	62,92	62,96	62,85	62,91		142,71	2,27	338,2	**108,8**	370
11-12-15	FG-2-2011	11-10909	3,9 à 4,2 m	62,82	62,76	62,70	62,76		147,02	2,34	229,2	**74,1**	276
11-12-15	FG-3-2011	11-10908	3,3 à 3,7 m	62,96	62,95	62,95	62,95		146,08	2,32	397,2	**127,6**	434

L/D: Rapport Longueur/Diamètre

Remarques:

Préparé par: Sylvie Daigle, tech. Chef Labo Date: 11-12-19 Vérifié par: Éric Ouimet, ing. Date: 11-12-19

Notes : Le résultat s'applique exclusivement à l'échantillon analysé.

Ce rapport ne doit pas être reproduit, sinon en entier, sans l'autorisation écrite de Labo S.M. inc.

APPENDIX C

Chemical Analysis Results – Previous Investigation

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Head Office
300-2319 St. Laurent Blvd.
Ottawa, Ontario K1G 4J8
p: 1-800-749-1947
e: paracel@paracellabs.com
www.paracellabs.com

Certificate of Analysis

Golder Associates Ltd. (Ottawa)
1931 Robertson Rd.
Ottawa, ON K2H 5B7
Attn: Keith Holmes

Phone: (613) 592-9600
Fax: (613) 592-9601

Client PO:
Project: 1525987
Custody: 105457

Report Date: 10-Jul-2015
Order Date: 8-Jul-2015
Order #: 1528298

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
1528298-01	15-3 (2)

Approved By: Mark Foto

Mark Foto, M.Sc. For Dale Robertson, BSc
Laboratory Director

Any use of these results implies your agreement that our total liabilty in connection with this work, however arising shall be limited to the amount paid by you for this work, and that our employees or agents shall not under circumstances be liable to you in connection with this work

Certificate of Analysis

Report Date: 10-Jul-2015
Order Date:8-Jul-2015

Client: **Golder Associates Ltd. (Ottawa)**
Client PO:
Project Description: 1525987

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC	8-Jul-15	8-Jul-15
Conductivity	EPA 9050A- probe @25 °C	9-Jul-15	9-Jul-15
pH	EPA 150.1 - pH probe @25 °C	9-Jul-15	9-Jul-15

PARACEL LABORATORIES LTD.

Order #: 1528298

Certificate of Analysis

Report Date: 10-Jul-2015
Order Date:8-Jul-2015

Client: **Golder Associates Ltd. (Ottawa)**
Client PO:
Project Description: 1525987

	Client ID:	15-3 (2)	-	-	-
	Sample Date:	06-Jul-15	-	-	-
	Sample ID:	1528298-01	-	-	-
	MDL/Units	Water	-	-	-
General Inorganics					
Conductivity	5 uS/cm	5730	-	-	-
pH	0.1 pH Units	7.2	-	-	-
Anions					
Chloride	1 mg/L	1800	-	-	-
Sulphate	1 mg/L	141	-	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	1	mg/L						
Sulphate	ND	1	mg/L						
General Inorganics									
Conductivity	ND	5	uS/cm						

Certificate of Analysis

Report Date: 10-Jul-2015
Order Date:8-Jul-2015

Client: **Golder Associates Ltd. (Ottawa)**
Client PO:
Project Description: 1525987

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	1	mg/L	1.02			0.0	10	
Sulphate	22.5	1	mg/L	22.4			0.4	10	
General Inorganics									
Conductivity	687	5	uS/cm	695			1.1	11	
pH	8.4	0.1	pH Units	8.4			0.2	10	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	10.6	1	mg/L	1.02	96.1	78-112			
Sulphate	31.0	1	mg/L	22.4	86.0	75-111			

Qualifier Notes:

Login Qualifiers :

Sample - Not submitted in the correct container - Submitted in amber glass container, instead of plastic.

Applies to samples: 15-3 (2)

Sample not received in Paracel verified container / media

Applies to samples: 15-3 (2)

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.

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Page 1 of 1

Client Name: Golder	Project Reference: 1525987		TAT: [X] Regular [ ] 3 Day
Contact Name: Keith Holmes	Quote #		[ ] 2 Day [ ] 1 Day
Address:	PO #		
	Email Address:		Date Required:
Telephone: 592-9600			

Criteria: [ ] O. Reg. 153/04 (As Amended) Table __ [ ] RSC Filing [ ] O. Reg. 558/00 [ ] PWQO [ ] CCME [ ] SUB (Storm) [ ] SUB (Sanitary) Municipality: ________ [ ] Other: N/A

Matrix Type: S (Soil/Sed.) GW (Ground Water) SW (Surface Water) SS (Storm/Sanitary Sewer) P (Paint) A (Air) O (Other)

Required Analyses

Paracel Order Number: 1528298

	Sample ID/Location Name	Matrix	Air Volume	# of Containers	Sample Taken Date	Sample Taken Time	PHCs F1-F4+BTEX	VOCs	PAHs	Metals by ICP	Hg	CrVI	B (HWS)	Conductivity	Sulphate	Chloride	pH			
1	15-3 (2)	GW		1	07/07/15									X	X	X	X			
2																				
3																				
4																				
5																				
6																				
7																				
8																				
9																				
10																				

Comments: Proceed with non-Paracel bottle

Method of Delivery: walk-in

Relinquished By (Sign): [signature]	Received by Driver/Depot: Karen Cull	Received at Lab: SUNEEPORN DOKMAI	Verified By: D. Charlebois
Relinquished By (Print): Keith Holmes	Date/Time: July 8/15 11:41	Date/Time: JUL 08, 2015 01:14	Date/Time: July 8 1:39
Date/Time: 8/July/15 11:40	Temperature: 7.8 °C	Temperature: 10.3 °C	pH Verified [X] By: N/A

APPENDIX D

Geophysical Vertical Seismic Profiling Test Results – Previous Investigation

Golder Associates

TECHNICAL MEMORANDUM

DATE July 10, 2015 **PROJECT No.** 152987

TO Troy Skinner, P.Eng.
Golder Associates Ltd.

CC Keith Holmes

FROM Patrick Finlay and
Christopher Phillips

EMAIL pfinlay@golder.com;
cphillips@golder.com

VSP TEST RESULTS
770 BRONSON STREET, OTTAWA, ONTARIO

This memorandum presents the results of the Vertical Seismic Profile (VSP) testing carried out at the proposed residential building located at 770 Bronson Street in Ottawa, Ontario. VSP testing was completed in borehole 15-04, located in the parking lot, on June 24, 2015. Borehole 15-04 was drilled to an approximate depth of 15.3 m below the existing ground and then cased with a PVC pipe grouted in place. The borehole consisted of approximately 2.3 m of fill, followed by 0.3 m of silt over fresh limestone bedrock.

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 (Example 1).

The high resolution results of a VSP survey are often used for earthquake engineering site classification, as per the National Building Code of Canada, 2010.

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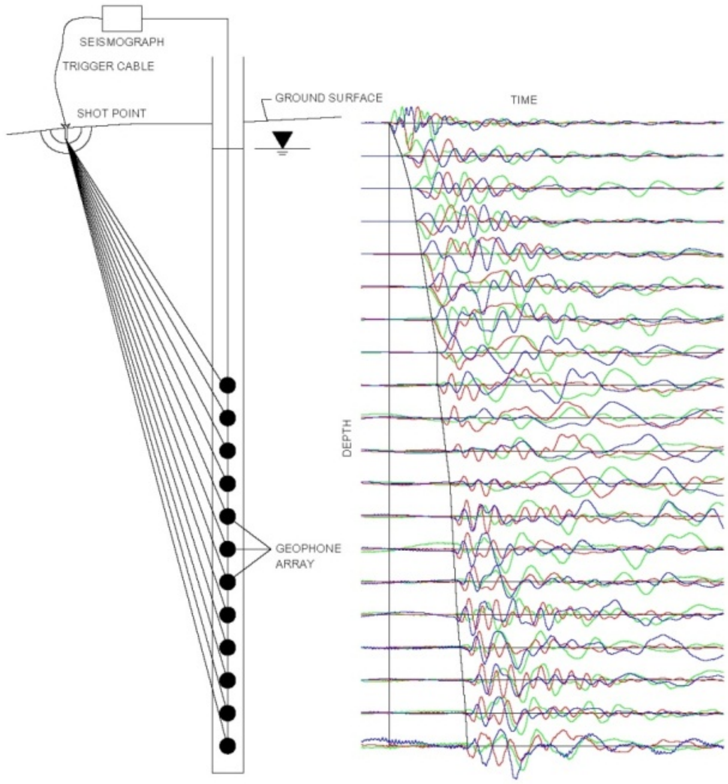


Example 1: Layout and resulting time traces from a VSP survey.

Field Work

The field work was carried out on June 24, 2015, by personnel from the Golder Associates Ltd. (Golder) Ottawa office.

Both compression and shear-wave seismic sources were used and both were located in close vicinity to the borehole. The seismic source for the compression wave test consisted of a 9.9 kilogram sledge hammer vertically impacted on a metal plate. The plate was located 2 metres from the borehole on the ground surface. The seismic source for the shear-wave test consisted of a 3 metre long, 150 millimetres by 150 millimetres wooden beam, weighted by a vehicle and horizontally struck with a 9.9 kilogram sledge hammer on alternate ends of the beam to induce polarized shear waves. The shear source was also located 2 metres from the borehole, and coupled to the ground surface by parking a vehicle on top of it. Test measurements started at 1.0-metre below the ground surface. Data were recorded in the borehole with a 3-component receiver spaced at 1.0-metre intervals in the bedrock and 0.5-metre intervals in the overburden below the ground surface to a maximum depth of the casing (14 metres).

The seismic records collected for each source location were stacked a minimum of ten 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) Combination 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 are presented on the following two plots and show the first break picks of the compression-wave and shear-wave arrivals overlaid on the seismic waveform traces recorded at the different geophone depths (Figures 1 and 2). The arrivals were picked on the vertical component for the compression source and on the two horizontal components for the shear source.

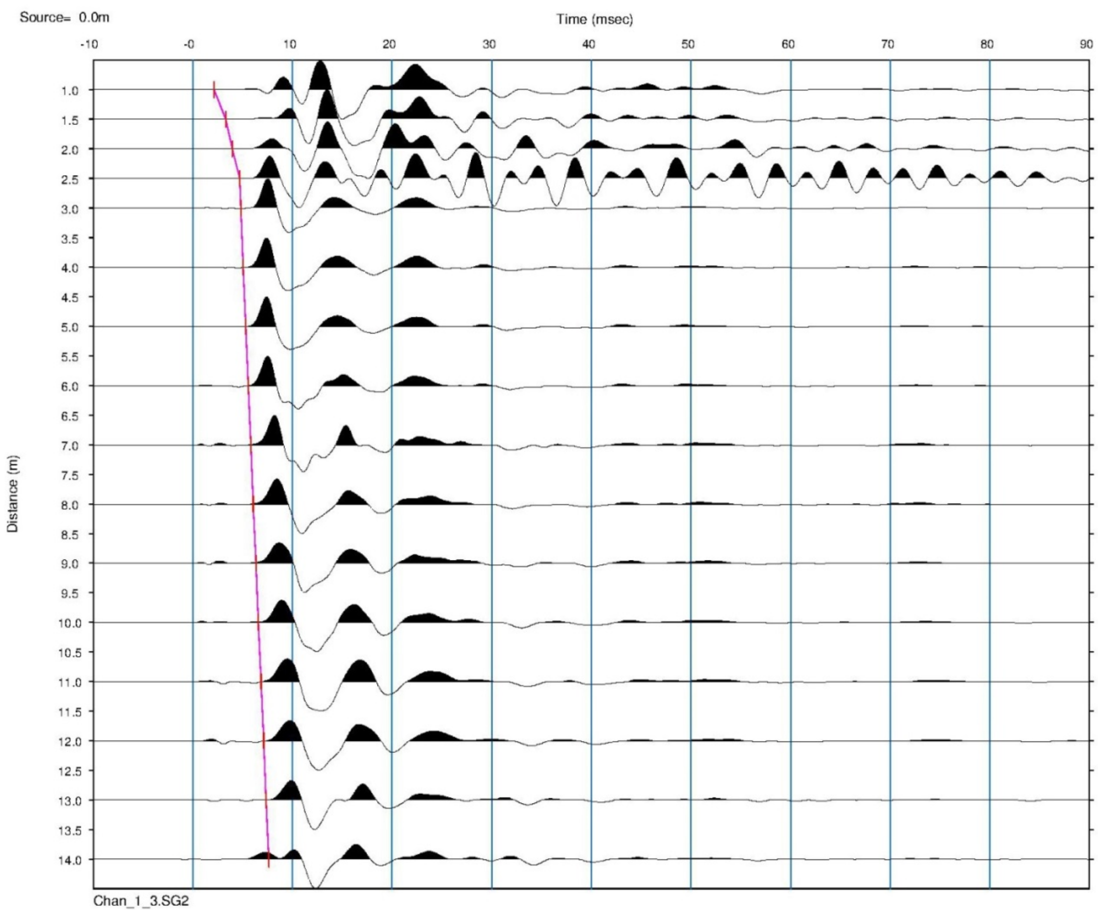


Figure 1: First break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth.

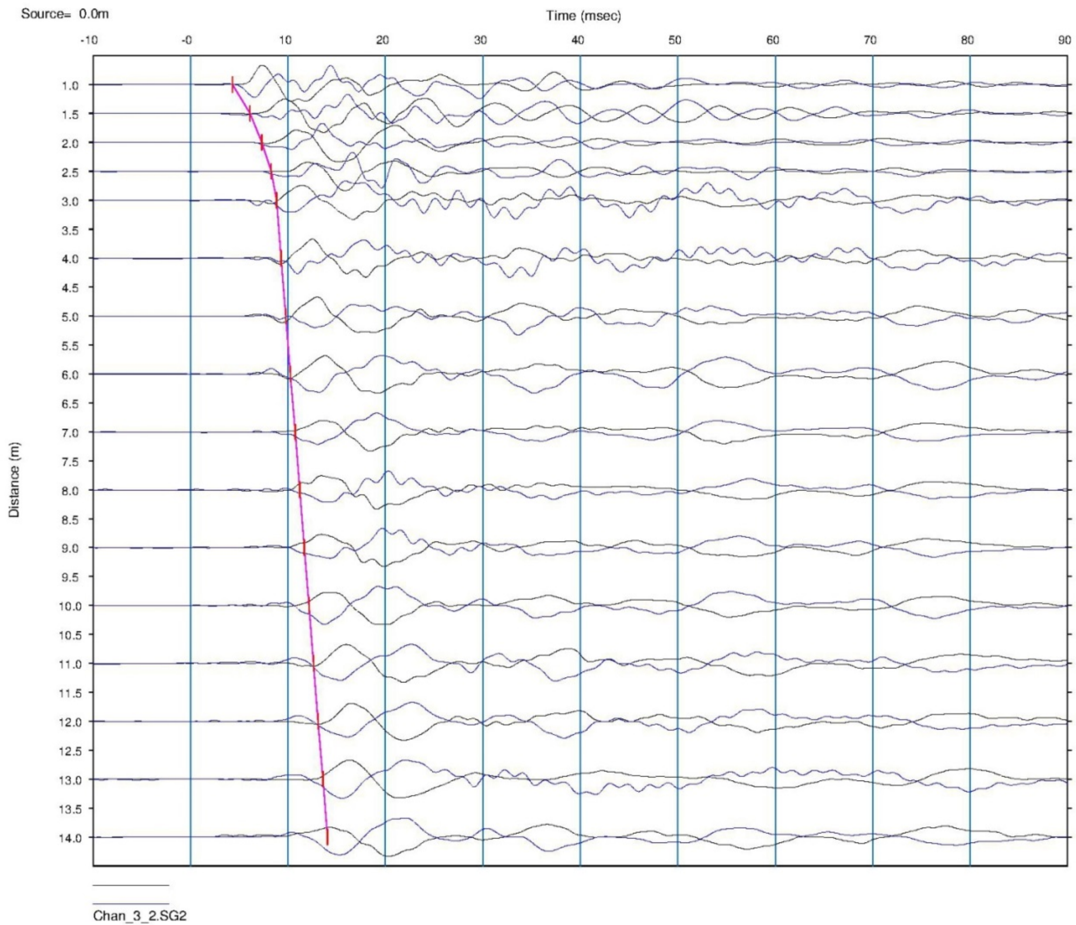


Figure 2: First break picking of shear wave arrivals (red) along the seismic traces recorded at each receiver depth.

Results

The VSP results are summarized in Table 1. The shear-wave and compression-wave layer velocities, at the field collected metre intervals, were calculated by best fitting a theoretical travel time model to the field data collected at half metre intervals. 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 on Table 1. The engineering moduli were calculated using an estimated bulk density, based on the borehole log. A bulk density of 2,200 kg/m^3 was used for fill and silt to a depth of 2.5 m; 2,650 kg/m^3 for used for fresh limestone bedrock from 2.5 m down to 14 m bgs.

The average shear-wave velocity from ground surface to a depth of 30 metres was measured to be 1328 m/s. The average velocity was calculated assuming that the velocity from 14 metres to a depth of 30 metres was constant with an average shear-wave velocity value of 1,650 m/s which is equal to the velocity of the bedrock at the bottom of the borehole.

Assuming the building will be founded on rock at a depth of 2.5 m bgs, the average shear-wave velocity was calculated to be 1,745 m/s, using an assumed velocity of 1,875 m/s from 14 to 32.5 m bgs.

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.

PROFESSIONAL GEOSCIENTIST
PATRICK I. FINLAY
PRACTISING MEMBER
1860
ONTARIO

Patrick Finlay, P.Geo.
Geophysicist

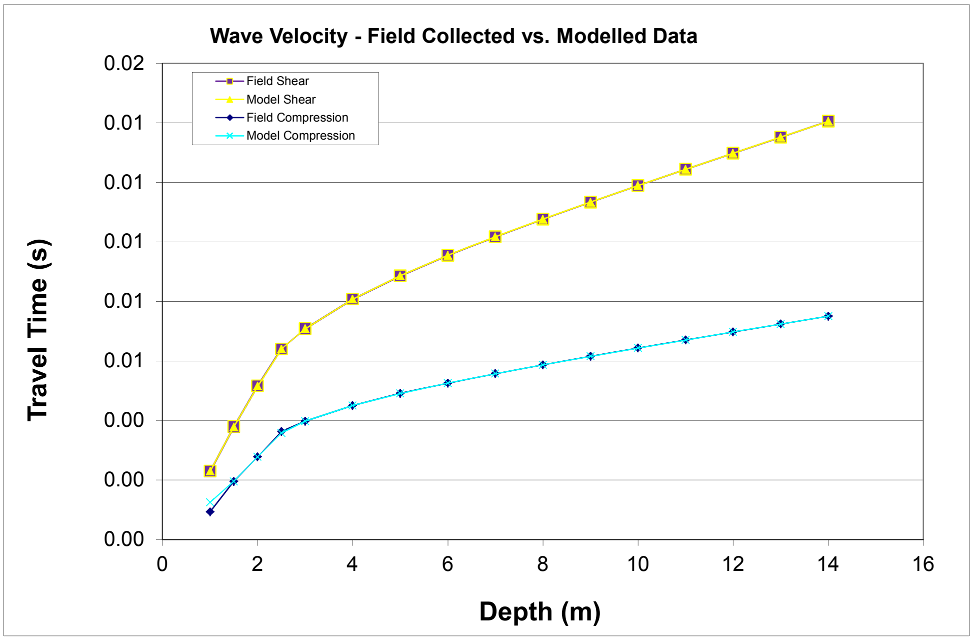
Christopher Phillips, M.Sc., P.Geo.
Senior Geophysicist, Associate

PF/CRP/sg
n:\active\2015\3 proj\1525987 tc united ph ii esa ottawa\geophysics\1525987 1000 770 bronson vsp_10july2015.docx

Attachments: Table 1: Shear Wave Velocity Profile at BH 15-04

TABLE 1
SHEAR WAVE VELOCITY PROFILE AT BH 15-04

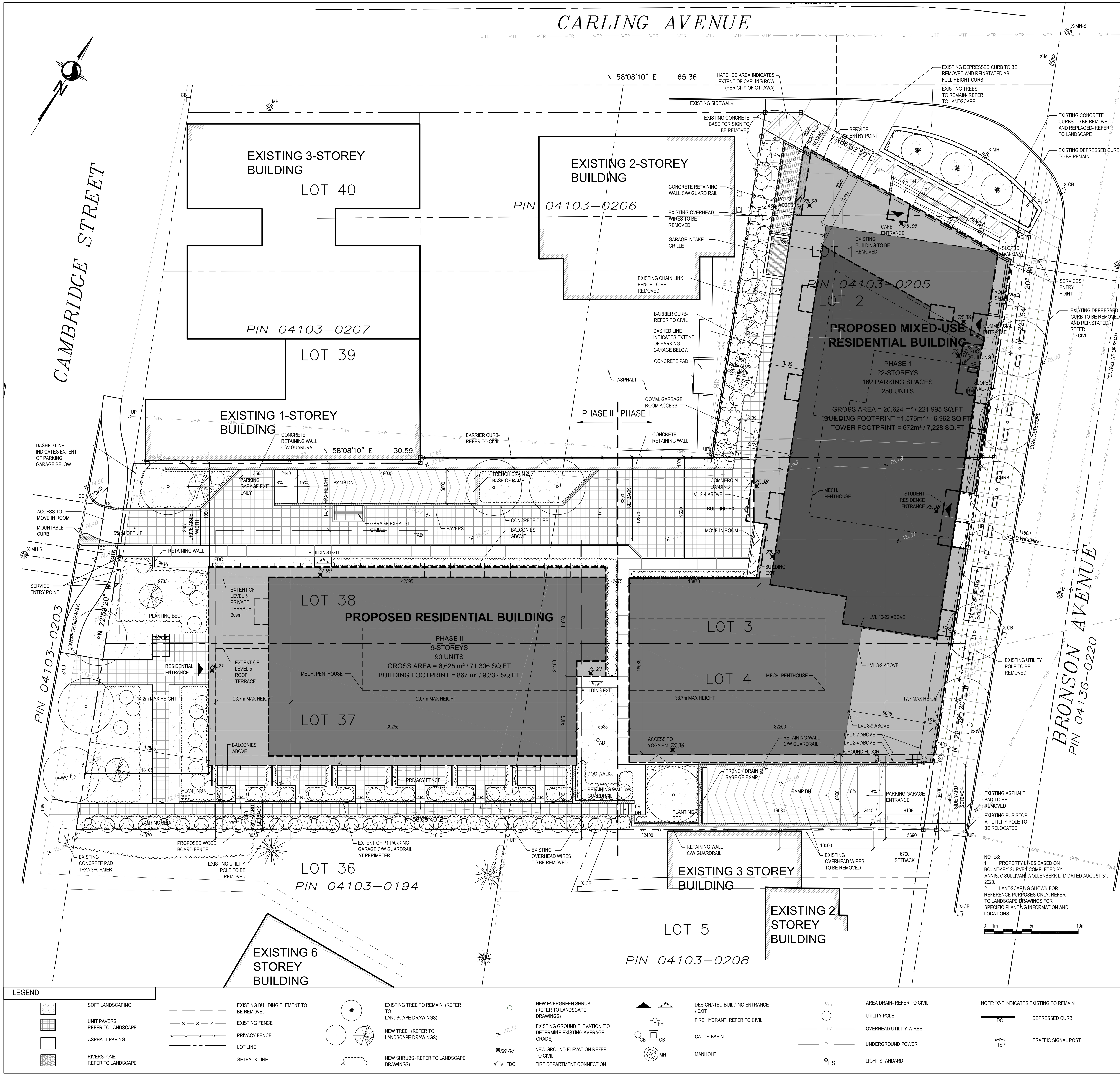
Layer Depth (m)				Estimated Bulk Density	Dynamic Engineering Properties			
Top	Bottom	Compressional Wave (m/s)	Shear Wave (m/s)	(kg/m^3)	Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0.0	1	800	436	2200	0.29	418	1078	850
1.0	1.5	706	335	2200	0.35	247	669	767
1.5	2	606	365	2200	0.22	293	712	417
2.0	2.5	630	402	2200	0.16	356	822	399
2.5	3	1254	733	2650	0.24	1424	3532	2269
3.0	4	1900	1008	2650	0.30	2693	7023	5976
4.0	5	2460	1268	2650	0.32	4261	11241	10356
5.0	6	2870	1464	2650	0.32	5680	15041	14255
6.0	7	3150	1600	2650	0.33	6784	17993	17249
7.0	8	3380	1690	2650	0.33	7569	20183	20183
8.0	9	3500	1750	2650	0.33	8116	21642	21642
9.0	10	3600	1790	2650	0.34	8491	22684	23023
10.0	11	3690	1830	2650	0.34	8875	23729	24250
11.0	12	3700	1850	2650	0.33	9070	24186	24186
12.0	13	3780	1860	2650	0.34	9168	24575	25640
13.0	14	3780	1875	2650	0.34	9316	24909	25442



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

APPENDIX E

Site Plans and Drawings as Provided by the Client



KEY PLAN

PROPERTY DESCRIPTION

22-STOREY MIXED-USE RESIDENTIAL BUILDING	
CITY OF OTTAWA PIN NUMBER	04103 0205, 04103 0125, 04103 0215
MUNICIPAL ADDRESS	770-774 Bronson Avenue & 557 Cambridge Street
PART OF LOTS 1 & 2, ALL OF LOTS 3, 4, 37, & PART OF LOT 38, REGISTERED PLAN 28	

SITE INFORMATION

LOT AREA:	4563 m²
LOT FRONTAGE:	64.02 m
LOT DEPTH:	101.5 m

BUILDING INFORMATION

FLOOR AREA:	2,443 m² (26,296 SF), BELOW GRADE FOOTPRINT = 3,790 m² (40,791 SF)
GROSS AREA:	27,249 m² (293,306 SF)
PROPOSED USE:	MIXED-USE RESIDENTIAL, HIGH-RISE

UNIT BREAKDOWN

	PHASE 1 (250 UNITS TOTAL)	PHASE 2 (90 UNITS TOTAL)
	RESIDENTIAL	RESIDENTIAL
FLOOR 1:	0	0 - STUDIO, 9 - 1BD, 1 - 2BD
FLOORS 2-4:	1 - STUDIO, 12 - 1BD, 2 - 2BD, 2 - 2BD+Den, 2 - 3BD x 3 FLOORS	1 - STUDIO, 9 - 1BD, 2 - 2BD, 0 - 3BD
FLOORS 5:	2 - STUDIO, 8 - 1BD, 4 - 2BD, 2 - 2BD+Den, 0 - 3BD	0 - STUDIO, 6 - 1BD, 2 - 2BD, 0 - 3BD
FLOORS 6-7:	2 - STUDIO, 8 - 1BD, 4 - 2BD, 2 - 2BD+Den, 0 - 3BD x 2 FLOORS	0 - STUDIO, 6 - 1BD, 3 - 2BD, 0 - 3BD
FLOOR 8:	2 - STUDIO, 6 - 1BD, 4 - 2BD, 2 - 2BD+Den, 0 - 3BD	0 - STUDIO, 6 - 1BD, 3 - 2BD
FLOOR 9:	2 - STUDIO, 6 - 1BD, 4 - 2BD, 2 - 2BD+Den, 0 - 3BD	0
FLOORS 10-22:	0 - STUDIO ,6 - 1BD, 3 - 2BD, 0 - 2BD+Den, 0 - 3BD x 13 FLOORS	0
TOTAL	13 - STUDIO,150 - 1BD, 65 - 2BD, 16 - 2BD+Den, 6 - 3BD 250 UNITS	3 - STUDIO, 66 - 1BD, 21 - 21BD 90 UNITS

ZONING TABLE

770 Bronson - AM10 [2373]
774 Bronson & 554 Cambridge - AM1 [2003] S296

CITY OF OTTAWA ZONING BY-LAW No. 2008-250	REQUIRED	PROPOSED
LOT AREA	NO MINIMUM	
LOT WIDTH	NO MINIMUM	
FRONT YARD AND CORNER SETBACK	3m	3m ALONG BRONSON AVE. 3m ALONG CARLING AVE.
MINIMUM INTERIOR SIDE YARD SETBACK	ABUTTING RESIDENTIAL ZONE: 3m (LOT 37 - URBAN EXCEPTION 2003) ALL OTHER CASES: 0m	5.69m ALONG WEST OF PHASE 1 11.7m ALONG NORTH OF PHASE 2
MINIMUM REAR YARD SETBACK	RESIDENTIAL USE BUILDING - 7.5m CASE OF BACK TO BACK LOTS - NO MINIMUM	0m
MAXIMUM BUILDING HEIGHT 770 BRONSON - AM10 [2373] 774 BRONSON & 554 CAMBRIDGE - AM1 [2003], SCHEDULE 296	30m BUT IN NO CASE GREATER THAN 9 STOREYS, OR AS SHOWN ON ZONING MAP (BY-LAW 2015-45) VARIES (SCHEDULE 296)	TOWER (Phase 1) 70.2m LEVEL 2-4 (Phase 1) 14.3m LEVEL 5-7 (Phase 1) 23.7m LEVEL 8-9 (Phase 1) 30.6m LEVEL 1-4 (Phase 2) 12.2m LEVEL 5-9 (Phase 2) 27.6m
GROUND FLOOR HEIGHT/GLAZING	MINIMUM OF 50% OF THE SURFACE AREA OF THE GROUND FLOOR FACADE, MEASURED FROM AVERAGE GRADE TO A HEIGHT OF 4.5m	ALONG BRONSON AVE.: 59% ALONG CARLING AVE.: 54%
MAXIMUM FLOOR SPACE INDEX	NONE (AM10 [2373], 3.0 (AM1 [2003])	7.2
LANDSCAPE PROVISIONS FOR PARKING LOTS	N/A	N/A
VEHICLE PARKING REQUIREMENTS (AREA X, SCHEDULE 1A)	RESIDENTIAL: 0.4 /UNIT AFTER FIRST 12 UNITS (Exception 2003) RESIDENTIAL VISITOR: 0.09/UNIT AFTER FIRST 12 UNITS (Exception 2003) PHASE 1 RESIDENTIAL: 95 SPACES VISITOR- RESIDENTIAL: 21 SPACES PHASE 2 RESIDENTIAL: 31 SPACES VISITOR: 7 SPACES TOTAL PARKING REQUIRED= 154 SPACES	RESIDENTIAL: 134 SPACES VISITOR: 28 SPACES TOTAL PARKING PROVIDED: 162 SPACES
		N/A
AMENITY AREA REQUIREMENTS	6m² PER DWELLING UNIT (MIN. 50% OF THE REQUIRED TOTAL AMENITY AREA TO BE COMMUNAL AND AT LEAST ONE AREA OF MIN. 54m²) 6m² PER UNIT OF EACH DWELLING UNIT: PHASE 1- 6 X 250= [1,500m²] PHASE 2- 6 X 90= [540m²] TOTAL AMENITY REQUIRED= 2,040m² 50% COMMUNAL REQUIRED = 1,020m²	TOTAL AMENITY PROVIDED= 2195 m² TOTAL PHASE 1: [1,500m²] TOTAL PHASE 2: [695.5m²] PHASE 1- BALCONIES/TERRACES LVL 10: 35m² LVL 11-16: 442m² PHASE 2- BALCONIES/TERRACES LVL 1 TERRACES:138m² LVL 2-4: 180m² LVL 5: 31m² LVL 6-7: 75m² LVL 8-9: 75m² PHASE 1- COMMUNAL LVL 1: 700m² LVL 8 TERRACE: 81m² ROOF TERRACE SEC 1: 116m² ROOF TERRACE SEC 2: 126m² PHASE 2- COMMUNAL LVL 5 TERRACE: 64m² LVL 5 AMENITY: 60m² LVL 9 TERRACE: 50m² LVL 1 ENTRY AREA: 22.5m²
BICYCLE PARKING SPACES	0.75/DWELLING UNIT= 340 X 0.75 (Exception 2003) [255 SPACES REQUIRED]	TOTAL PROVIDED= 271 SPACES

No.	Date	Émis pour / Issued FOR
1	2020-10-13	FOR COORD
2	2020-12-10	FOR COORD
3	2021-01-14	FOR COORD
4	2021-02-15	FOR COORD
5	2021-02-18	FOR COORD
6	2021-03-02	FOR COORD
7	2021-03-09	SITE PLAN CONTROL
8	2022-07-25	COORDINATION
9	2022-12-14	SITE PLAN CONTROL RESPONSE
10	2023-06-16	SITE PLAN CONTROL RESPONSE
11	2023-09-18	SITE PLAN CONTROL RESPONSE
12	2023-11-09	SITE PLAN APPROVAL
13	2024-04-29	SITE PLAN APPROVAL (ADDED ROW AREA)
14	2025-04-22	PERMISSION TO PROCEED
15	2025/08/11	SITE PLAN REVISION

Ingénieur / Engineer
(Mécanique & Électricité / Mechanical & Electrical)

Ingénieur / Engineer
(Structure / Structure)

Architecte/ Architect
(paysagiste / Landscape)

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Ingénieur / Engineer
(Civil / Civil)

LRJ

Client / Client

KATASA
GROUPE ✦ DÉVELOPPEMENT

Architecte / Architect
Collectif d'architectes / Architects Collective

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Sceau / Seal

ONTARIO ASSOCIATION OF ARCHITECTS
ROBERTO CAMPOS
LICENCE
7401

Note:

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Projet / Project

22 STOREY APARTMENT BUILDING

770-774 Bronson Avenue & 557 Cambridge Street
Ottawa, ON

Titre / Title

SITE PLAN

Dessiné par / Drawn by: ZK	No. projet / Project number: 2025
Vérifié par / Verified by: RC	No. dessin / Drawing number
Échelle / Scale: AS SHOWN	Révision / Revision: 0
Date de création du dessin / Drawing creation date: 2020/10/09	A105

D07-12-21-0108

#18887

1
A.200
EAST ELEVATION
1 : 200

LEGEND - MATERIALS

- 1a — BRICK VENEER, COLOUR: WARM BROWN
- 1b — BRICK VENEER, COLOUR: CHARCOAL
- 1c — BRICK VENEER, COLOUR: LIGHT GREY
- 2a — METAL PANEL, COLOUR: ORANGE
- 2b — METAL PANEL, COLOUR: CHARCOAL
- 2c — METAL PANEL, COLOUR: MEDIUM GREY
- 3 — METAL PANEL, VERTICAL PATTERN, COLOUR: CHARCOAL
- 4 — METAL PANEL, HORIZONTAL PATTERN, COLOUR: CHARCOAL
- 5 — GLASS RAILING - TYP. FOR ALL BALCONIES, COLOUR: CLEAR
- 6 — METAL RAILING, COLOUR: BLACK
- 7 — CURTAIN WALL GLAZING

No.	Date	Émis pour / Object
7	2021/03/09	SITE PLAN CONTROL
8	2022/12/14	SITE PLAN CONTROL RESPONSE
9	2023/06/16	SITE PLAN CONTROL RESPONSE

Architecte / *Architect*
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Ingénieur / *Engineer*
(Civil / *Civil*)

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GROUPE + DÉVELOPPEMENT

Architecte / *Architect*

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ROBERTO CAMPOS
LICENCE
7401

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Projet / *Project*

770 BRONSON

770-774 Bronson Avenue & 557 Cambridge Street

Ottawa, ON

Titre / *Title*

EAST ELEVATION

Dessiné par / *Drawn by*
GCG

No. projet / *Project number*
2025

Vérifié par / *Verified by*
RC

No. dessin / *Drawing number*

Révision / Revision
9

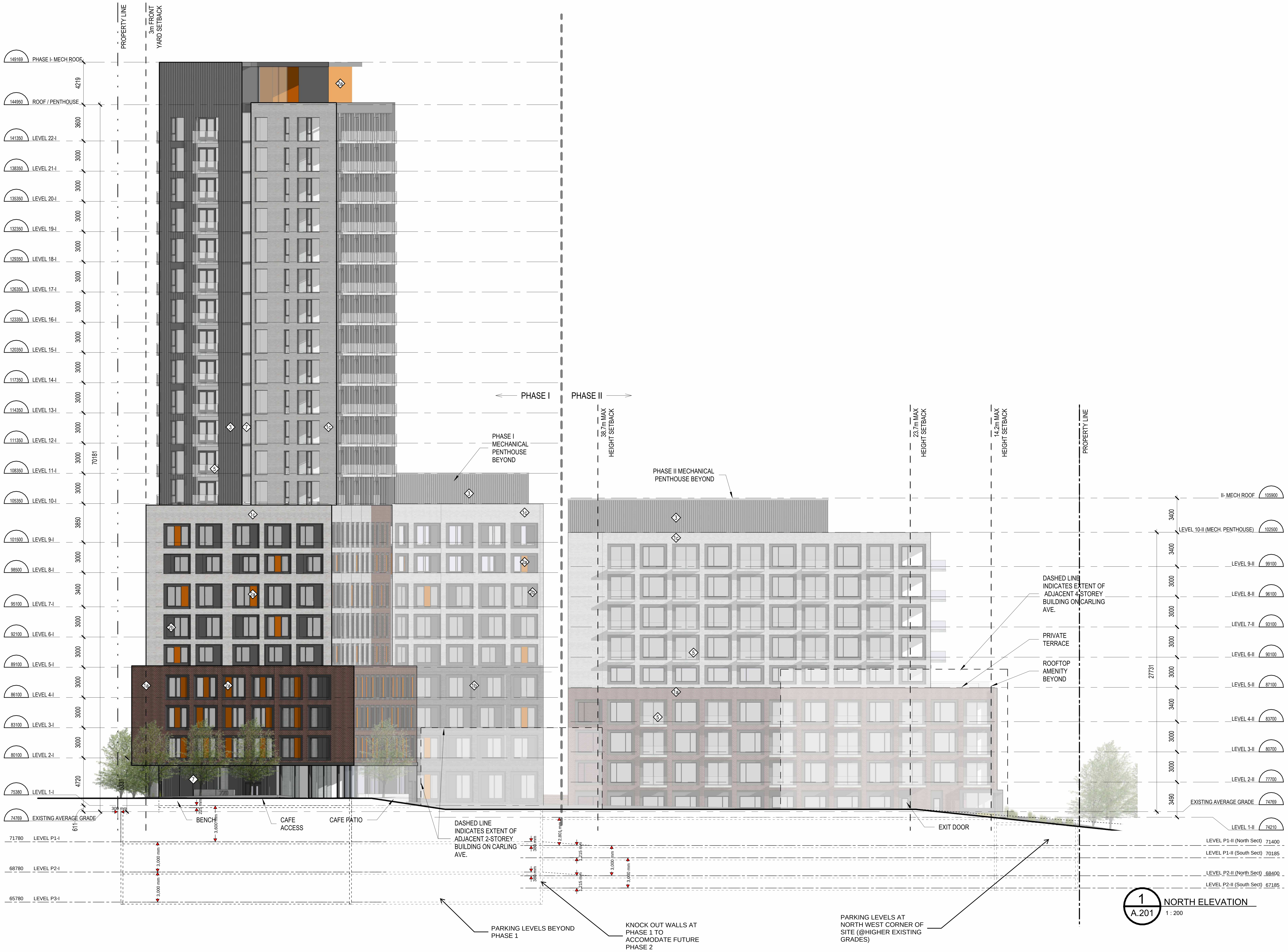
Échelle / Scale
As indicated

Date de création du dessin / *Drawing creation date*
2021/02/01

A.200

D07-12-21-0108

#18887



1 NORTH ELEVATION
A.201 1 : 200

LEGEND - MATERIALS

1a	**BRICK VENEER** COLOUR: WARM BROWN
1b	**BRICK VENEER** COLOUR: CHARCOAL
1c	**BRICK VENEER** COLOUR: LIGHT GREY
2a	**METAL PANEL** COLOUR: ORANGE
2b	**METAL PANEL** COLOUR: CHARCOAL
2c	**METAL PANEL** COLOUR: MEDIUM GREY
3	**METAL PANEL, VERTICAL PATTERN** COLOUR: CHARCOAL
4	**METAL PANEL, HORIZONTAL PATTERN** COLOUR: CHARCOAL
5	**GLASS RAILING - TYP. FOR ALL BALCONIES** COLOUR: CLEAR
6	**METAL RAILING** COLOUR: BLACK
7	**CURTAIN WALL GLAZING**

No.	Date	Émis pour / Object
7	2021/03/09	SITE PLAN CONTROL
8	2022/12/14	SITE PLAN CONTROL RESPONSE
9	2023/06/16	SITE PLAN CONTROL RESPONSE

Architecte / *Architect*
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(Civil / *Civil*)

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Architecte / *Architect*

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fig. 2
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LICENCE
7401

Note:

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Contractor shall verify all information and dimensions on site and immediately report any errors or omissions to the architect.

Projet / *Project*

770 BRONSON

770-774 Bronson Avenue & 557 Cambridge Street

Ottawa, ON

Titre / *Title*

NORTH ELEVATION

Dessiné par / *Drawn by*: GCG

No. projet / *Project number*: 2025

Vérifié par / *Verified by*: RC

No. dessin / *Drawing number*: A.201

Révision / Revision: 9

Échelle / Scale: As indicated

Date de création du dessin / *Drawing creation date*: 2021/02/01

D07-12-21-0108

#18887

1 A.202 WEST ELEVATION 1 : 200

LEGEND - MATERIALS

- 1a BRICK VENEER COLOUR: WARM BROWN
- 1b BRICK VENEER COLOUR: CHARCOAL
- 1c BRICK VENEER COLOUR: LIGHT GREY
- 2a METAL PANEL COLOUR: ORANGE
- 2b METAL PANEL COLOUR: CHARCOAL
- 2c METAL PANEL COLOUR: MEDIUM GREY
- 3 METAL PANEL, VERTICAL PATTERN COLOUR: CHARCOAL
- 4 METAL PANEL, HORIZONTAL PATTERN COLOUR: CHARCOAL
- 5 GLASS RAILING - TYP. FOR ALL BALCONIES COLOUR: CLEAR
- 6 METAL RAILING COLOUR: BLACK
- 7 CURTAIN WALL GLAZING

No.	Date	Émis pour / Object
7	2021/03/09	SITE PLAN CONTROL
8	2022/12/14	SITE PLAN CONTROL RESPONSE
9	2023/06/16	SITE PLAN CONTROL RESPONSE
10	2025/08/11	SITE PLAN REVISION

Architecte / *Architect*
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Client / *Client*

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Architecte / *Architect*

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Titre / *Title*

WEST ELEVATION

Dessiné par / *Drawn by*: GCG

No. projet / *Project number*: 2025

Vérifié par / *Verified by*: RC

No. dessin / *Drawing number*: A.202

Révision / Revision: 9

Échelle / Scale: As indicated

Date de création du dessin / *Drawing creation date*: 2021/02/01

D07-12-21-0108

#18887

1 / A.203 SOUTH ELEVATION
1 : 200

LEGEND - MATERIALS

1a	**BRICK VENEER** COLOUR: WARM BROWN
1b	**BRICK VENEER** COLOUR: CHARCOAL
1c	**BRICK VENEER** COLOUR: LIGHT GREY
2a	**METAL PANEL** COLOUR: ORANGE
2b	**METAL PANEL** COLOUR: CHARCOAL
2c	**METAL PANEL** COLOUR: MEDIUM GREY
3	**METAL PANEL, VERTICAL PATTERN** COLOUR: CHARCOAL
4	**METAL PANEL, HORIZONTAL PATTERN** COLOUR: CHARCOAL
5	**GLASS RAILING - TYP. FOR ALL BALCONIES** COLOUR: CLEAR
6	**METAL RAILING** COLOUR: BLACK
7	**CURTAIN WALL GLAZING**

No.	Date	Émis pour / Object
7	2021/03/09	SITE PLAN CONTROL
8	2022/12/14	SITE PLAN CONTROL RESPONSE
9	2023/06/16	SITE PLAN CONTROL RESPONSE

Architecte / *Architect*
(Paysagiste / *Landscape*)

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Ingénieur / *Engineer*
(Civil / *Civil*)

LRL

Client / *Client*

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Architecte / *Architect*

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LICENCE
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Projet / *Project*

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Ottawa, ON

Titre / *Title*

SOUTH ELEVATION

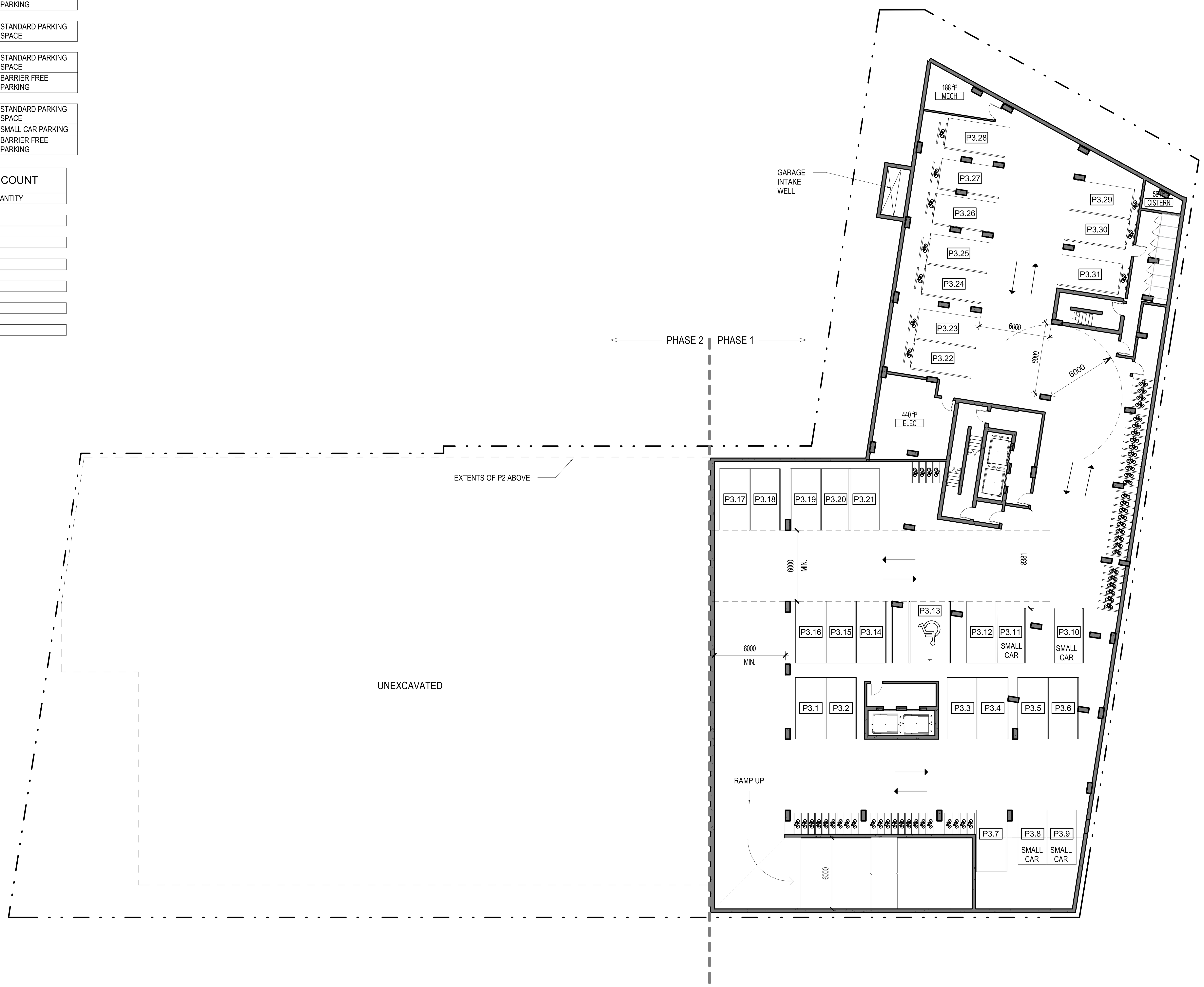
Dessiné par / *Drawn by* GCG	No. projet / *Project number* 2025
Vérifié par / *Verified by* RC	No. dessin / *Drawing number* A.203
Échelle / Scale As indicated	Révision / Revision 9
Date de création du dessin / *Drawing creation date* 2021/02/01	

D07-12-21-0108

#18887

MOTOR VEHICLE PARKING COUNT		
LEVEL	QUANTITY	PARKING TYPE
PARKING P3-I	26	STANDARD PARKING SPACE
PARKING P3-I	4	SMALL CAR PARKING
PARKING P3-I	1	BARRIER FREE PARKING
PARKING P2-II (LOW)	21	STANDARD PARKING SPACE
PARKING P2-II (HIGH)	21	STANDARD PARKING SPACE
PARKING P2-II (HIGH)	1	BARRIER FREE PARKING
PARKING P2-I	25	STANDARD PARKING SPACE
PARKING P2-I	2	SMALL CAR PARKING
PARKING P2-I	1	BARRIER FREE PARKING
PARKING P1-II (LOW)	19	STANDARD PARKING SPACE
PARKING P1-II (HIGH)	13	STANDARD PARKING SPACE
PARKING P1-II (HIGH)	1	BARRIER FREE PARKING
PARKING P1-I	23	STANDARD PARKING SPACE
PARKING P1-I	3	SMALL CAR PARKING
PARKING P1-I	1	BARRIER FREE PARKING
GRAND TOTAL	162	

BICYCLE PARKING COUNT	
LEVEL	QUANTITY
PARKING P1-I	102
PARKING P1-II (HIGH)	18
PARKING P2-I	74
PARKING P2-II (HIGH)	17
PARKING P2-II (LOW)	15
PARKING P3-I	65
GRAND TOTAL: 291	291



No.	Date	Émis pour / Object
1	2020/10/13	FOR COORDINATION
2	2020/11/04	FOR COORD
3	2020/12/07	CLIENT REVIEW
4	2020/12/10	FOR COORDINATION
5	2021/01/26	FOR COORD
6	2021/02/15	FOR COORD
7	2021/03/09	SITE PLAN CONTROL
8	2022/12/14	SITE PLAN CONTROL RESPONSE
9	2023/06/16	SITE PLAN CONTROL RESPONSE

Architecte / *Architect*
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Client / *Client*

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GROUPE + DÉVELOPPEMENT

Architecte / *Architect*

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Projet / *Project*

770 BRONSON

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Ottawa, ON

Titre / *Title*

P3 PARKING PLAN

Dessiné par / *Drawn by*
RD, LK

No. projet / *Project number*
2025

Vérifié par / *Verified by*
MD, RC

No. dessin / *Drawing number*

Révision / Revision
9

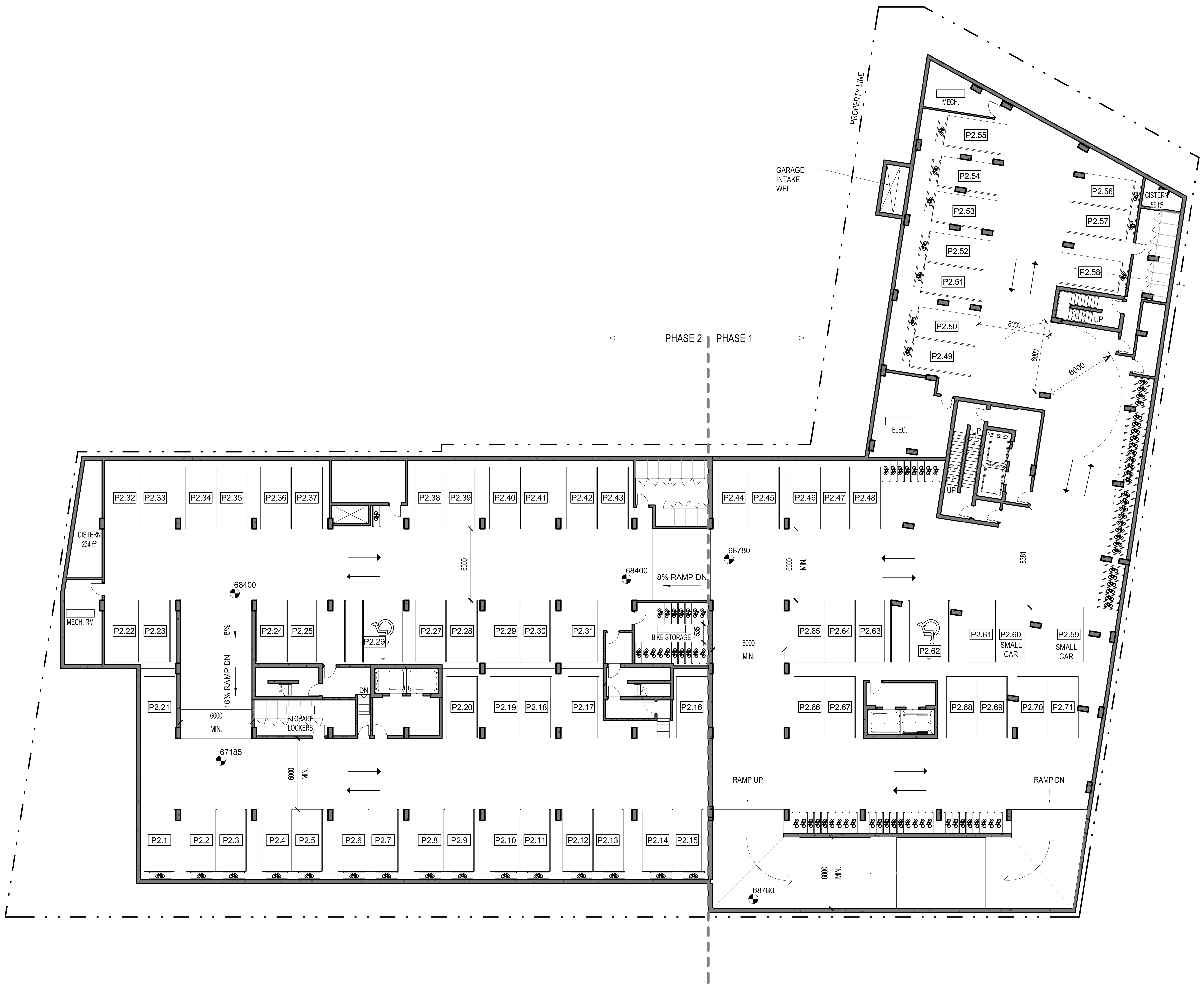
Échelle / Scale
1 : 200

Date de création du dessin / *Drawing creation date*
03/02/22

A.118

D07-12-21-0108

#18887



No.	Date	Émis pour / Object
1	2020/10/13	FOR COORDINATION
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3	2020/12/07	CLIENT REVIEW
4	2020/12/10	FOR COORDINATION
5	2021/01/26	FOR COORD
6	2021/02/15	FOR COORD
7	2021/03/09	SITE PLAN CONTROL
8	2022/12/14	SITE PLAN CONTROL RESPONSE
9	2023/06/16	SITE PLAN CONTROL RESPONSE

Architecte / *Architect*
(Paysagiste / *Landscape)*

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Ingénieur / *Engineer*
(Civil / *Civil*)

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Client / *Client*

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GROUPE + DÉVELOPPEMENT

Architecte / *Architect*

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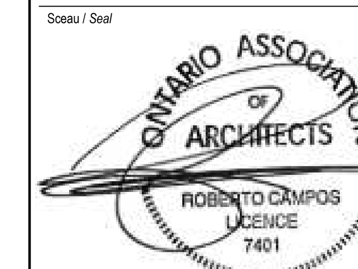

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Projet / *Project*

770 BRONSON

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Ottawa, ON

Titre / *Title*

P2 PARKING PLAN

Dessiné par / *Drawn by*
RD, LK

No. projet / *Project number*
2025

Vérifié par / *Verified by*
MD, RC

No. dessin / *Drawing number*

Révision / Revision
9

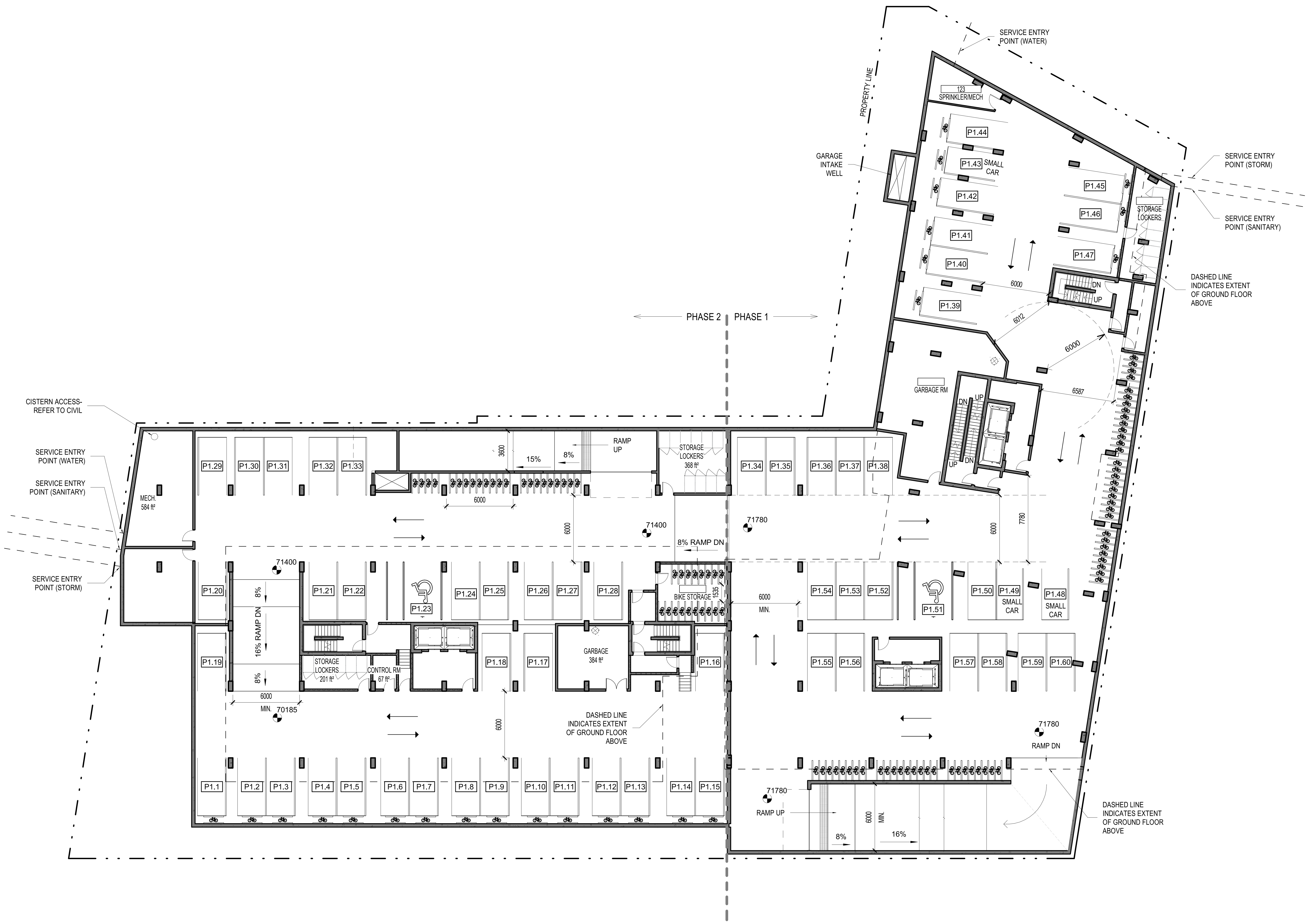
Échelle / Scale
1 : 200

Date de création du dessin / *Drawing creation date*
03/02/22

A.119

D07-12-21-0108

#18887



No.	Date	Émis pour / Object
1	2020/10/13	FOR COORDINATION
2	2020/11/04	FOR COORD
3	2020/12/07	CLIENT REVIEW
4	2020/12/10	FOR COORDINATION
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7	2021/03/09	SITE PLAN CONTROL
8	2022/12/14	SITE PLAN CONTROL RESPONSE
9	2023/06/16	SITE PLAN CONTROL RESPONSE

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(Paysagiste / *Landscape*)

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LRL
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Client / *Client*

KATASA
GROUPE + DÉVELOPPEMENT

Architecte / *Architect*
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LICENCE
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Projet / *Project*

770 BRONSON

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Ottawa, ON

Titre / *Title*

P1 PARKING

Dessiné par / *Drawn by*
RD, LK

No. projet / *Project number*
2025

Vérifié par / *Verified by*
MD, RC

No. dessin / *Drawing number*

Révision / Revision
9

Échelle / Scale
1 : 200

Date de création du dessin / *Drawing creation date*
03/02/22

A.120

D07-12-21-0108

#18887

No.	Date	Émis pour / Object
1	2020/10/13	FOR COORDINATION
2	2020/11/04	FOR COORD
3	2020/12/07	CLIENT REVIEW
4	2020/12/10	FOR COORDINATION
5	2021/01/26	FOR COORD
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7	2021/03/09	SITE PLAN CONTROL
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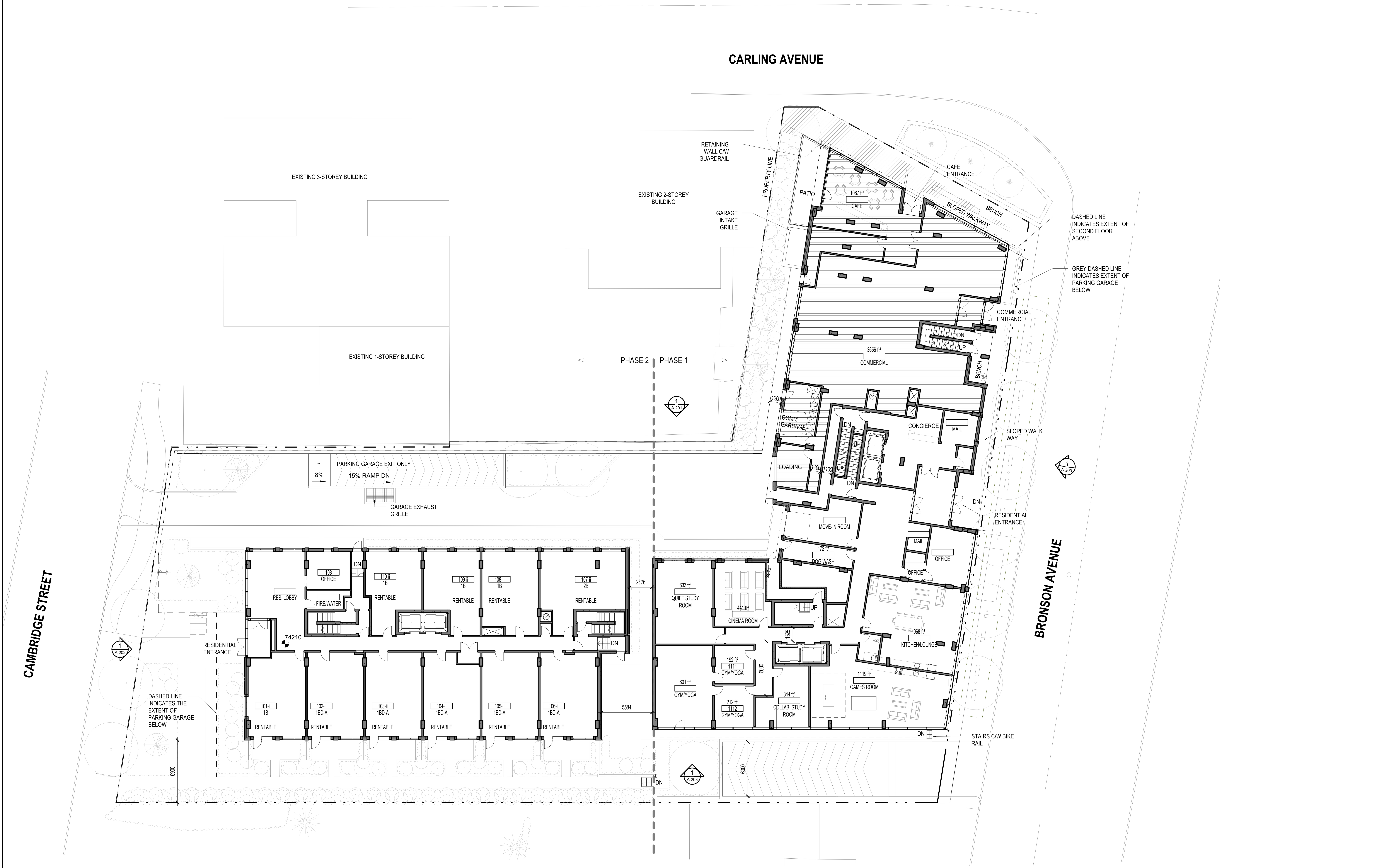
9	2023/06/16	SITE PLAN CONTROL RESPONSE



Architecte / *Architect*
(Paysagiste / *Landscape)*

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GROUND FLOOR PLAN

Dessiné par / *Drawn by*: RD, LK	No. projet / *Project number*: 2025
Vérifié par / *Verified by*: MD, RC	No. dessin / *Drawing number*
Échelle / Scale: 1 : 200	Révision / Revision: 9
Date de création du dessin / *Drawing creation date*: 03/02/22	A.121

D07-12-21-0108

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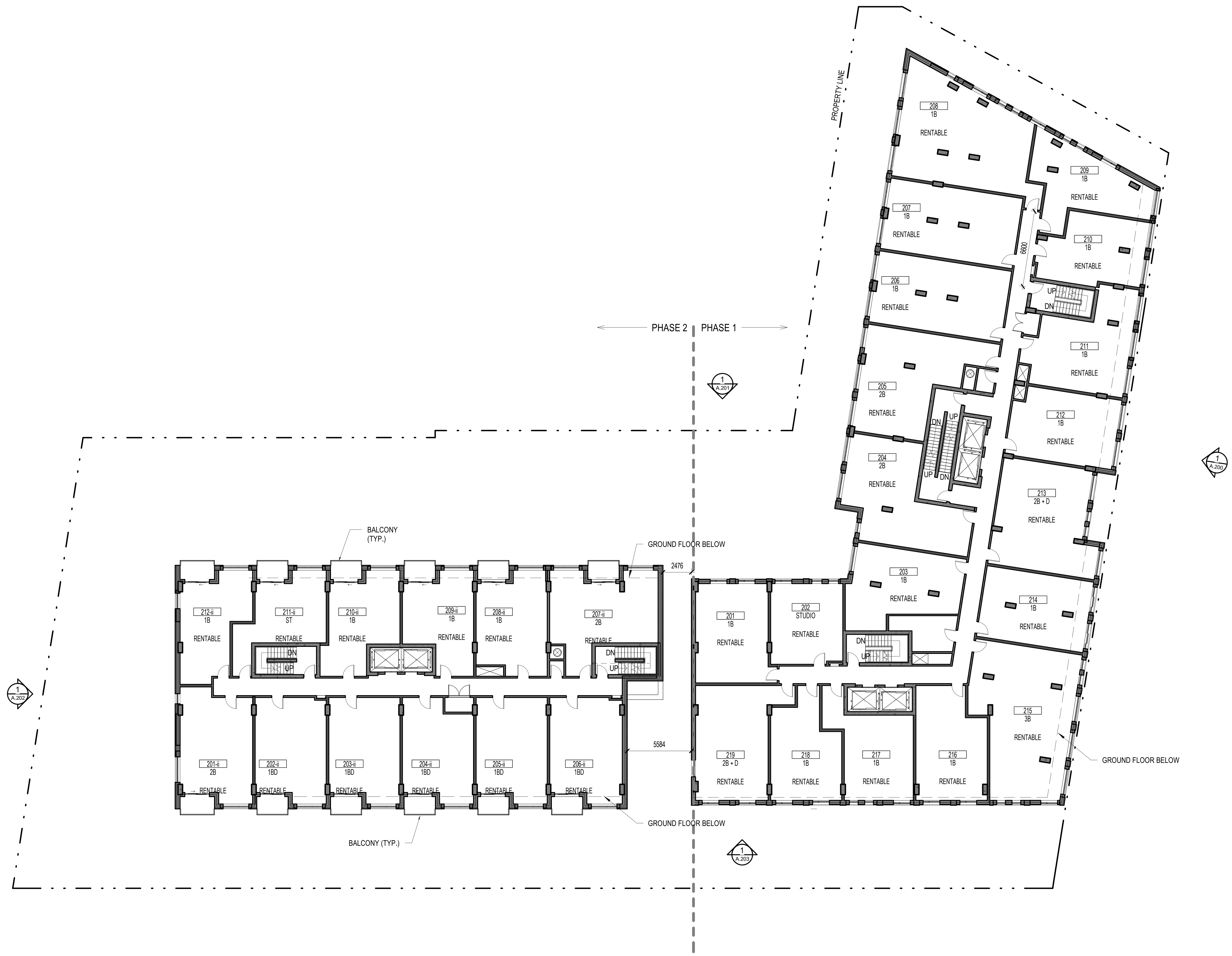

No.	Date	Émis pour / Object
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2	2020/11/04	FOR COORD
3	2020/12/07	CLIENT REVIEW
4	2020/12/10	FOR COORDINATION
5	2021/01/26	FOR COORD
6	2021/02/15	FOR COORD
7	2021/03/09	SITE PLAN CONTROL
8	2022/12/14	SITE PLAN CONTROL RESPONSE
9	2023/06/16	SITE PLAN CONTROL RESPONSE

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FLOOR PLAN - LEVEL 2-4

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A.122

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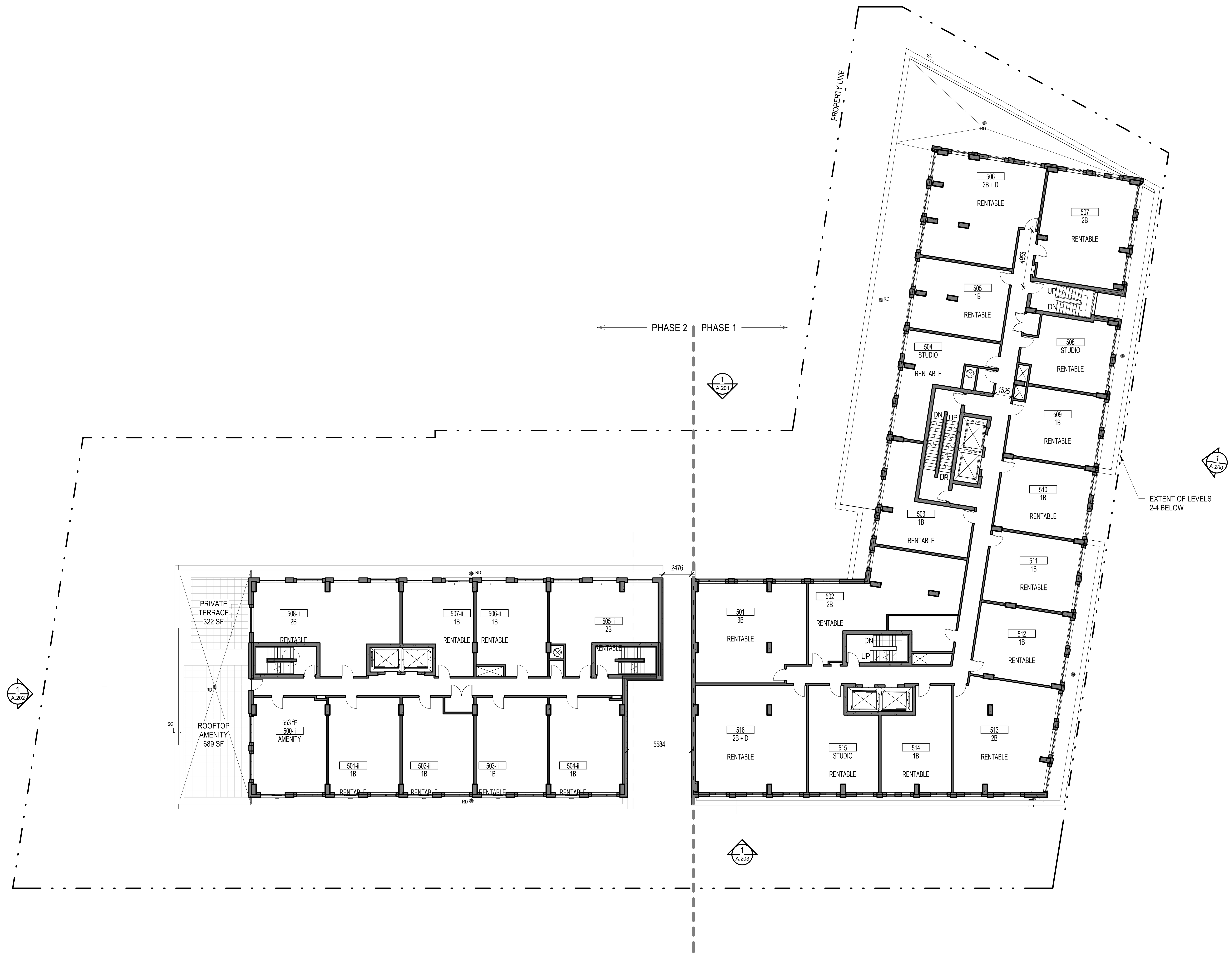

No.	Date	Émis pour / Object
1	2020/10/13	FOR COORDINATION
2	2020/11/04	FOR COORD
3	2020/12/07	CLIENT REVIEW
4	2020/12/10	FOR COORDINATION
5	2021/01/26	FOR COORD
6	2021/02/15	FOR COORD
7	2021/03/09	SITE PLAN CONTROL
8	2022/12/14	SITE PLAN CONTROL RESPONSE
9	2023/06/16	SITE PLAN CONTROL RESPONSE

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FLOOR PLAN - LEVEL 5-7

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A.125

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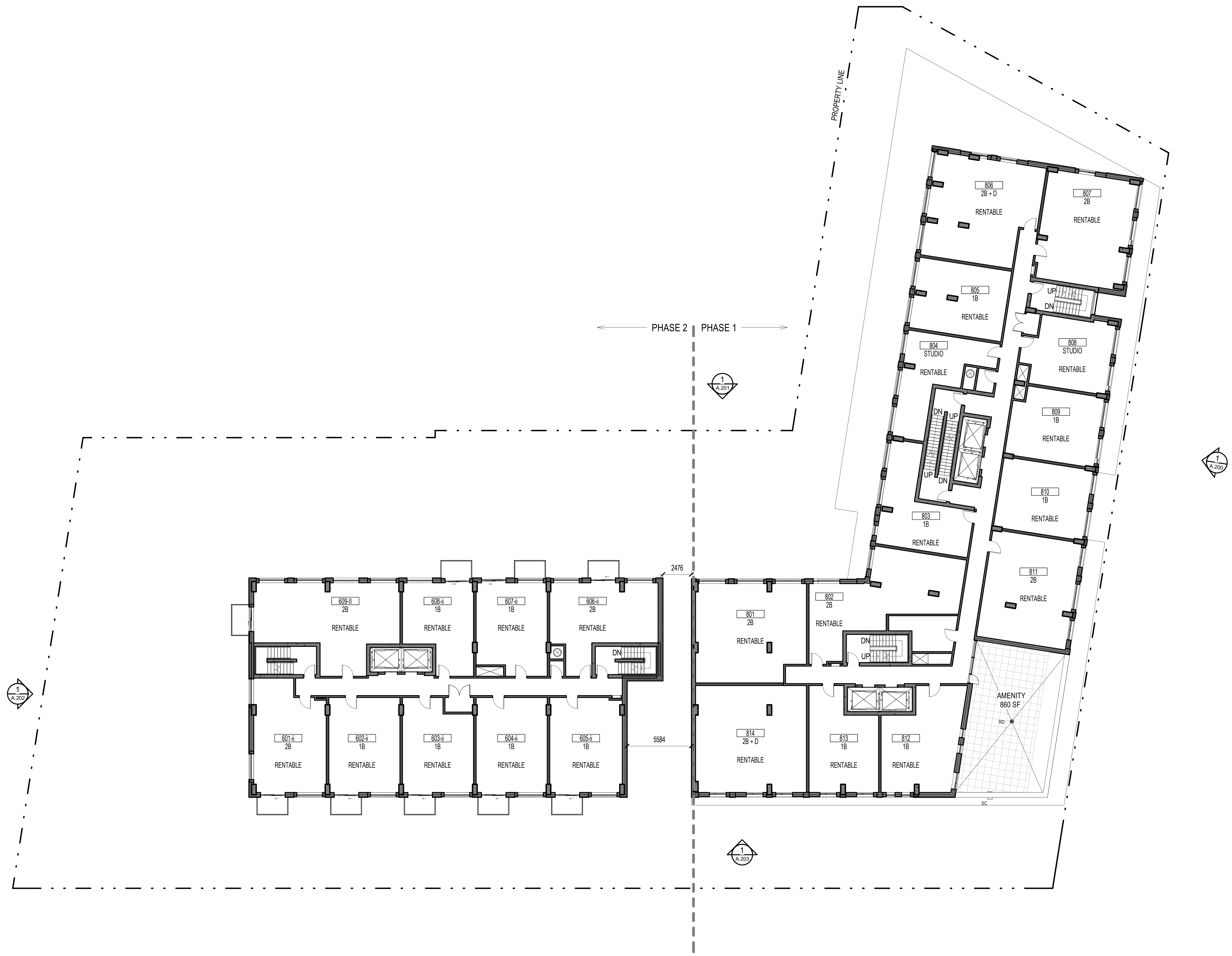

No.	Date	Émis pour / Object
8	2022/12/14	SITE PLAN CONTROL RESPONSE
9	2023/06/16	SITE PLAN CONTROL RESPONSE

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FLOOR PLAN - LEVEL 8-9

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Date de création du dessin / *Drawing creation date*
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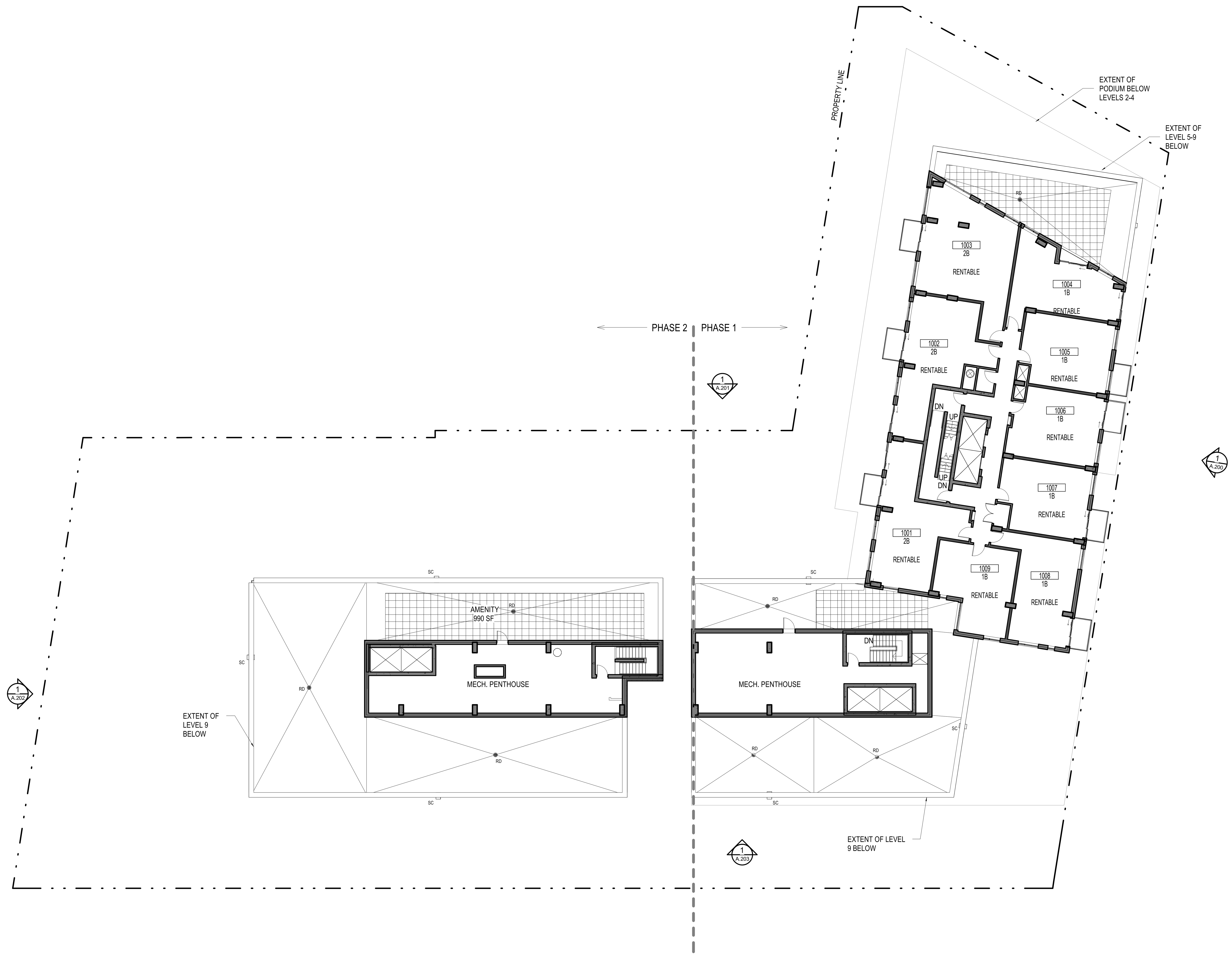
A.128

D07-12-21-0108

#18887

No.	Date	Émis pour / Object
8	2022/12/14	SITE PLAN CONTROL RESPONSE
9	2023/06/16	SITE PLAN CONTROL RESPONSE



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FLOOR PLAN - LEVEL 10

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A.130

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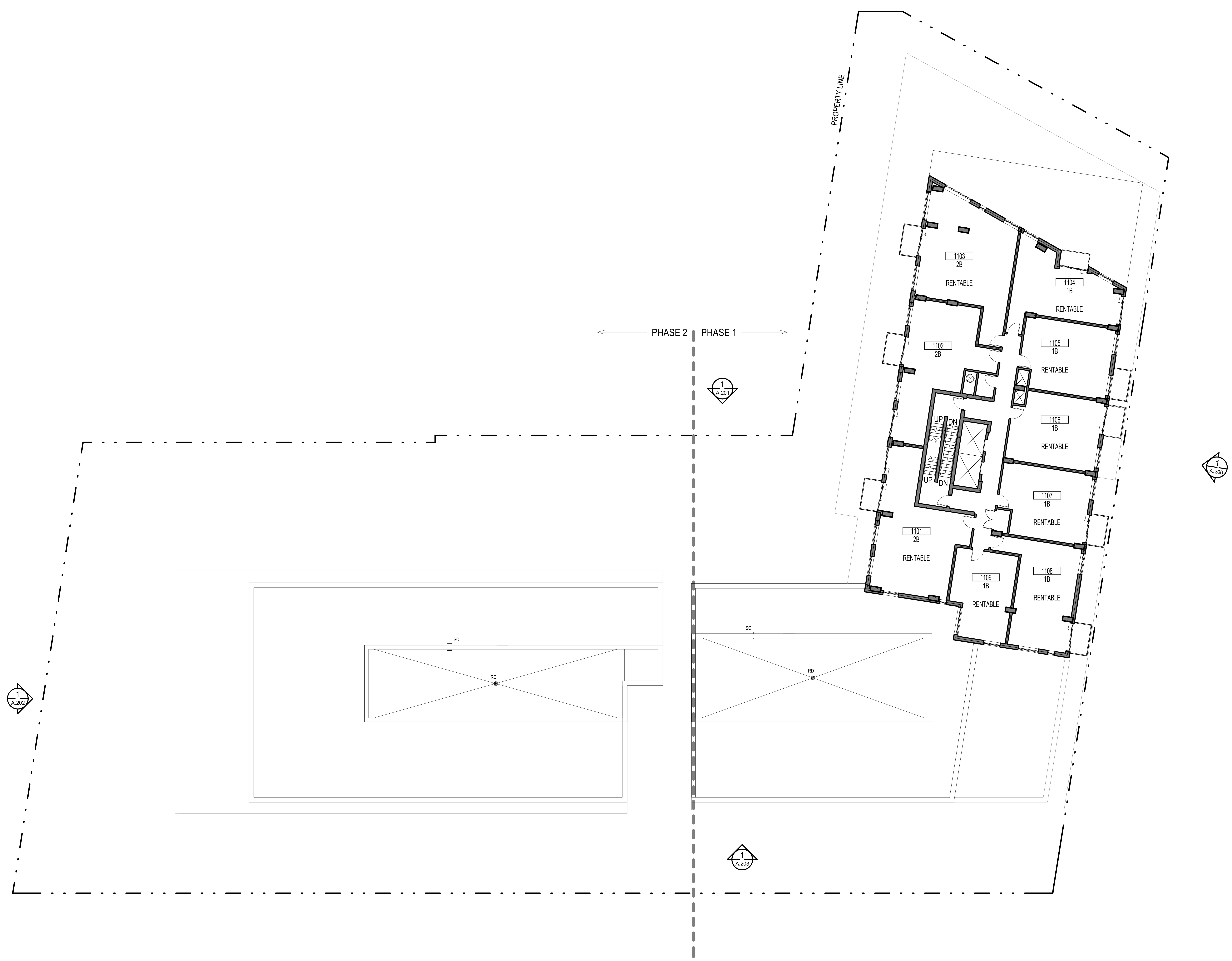

No.	Date	Émis pour / *Object*
8	2022/12/14	SITE PLAN CONTROL RESPONSE
9	2023/06/16	SITE PLAN CONTROL RESPONSE

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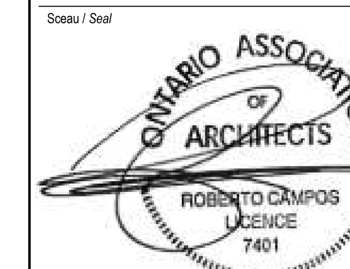

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FLOOR PLAN - LEVEL 11-22

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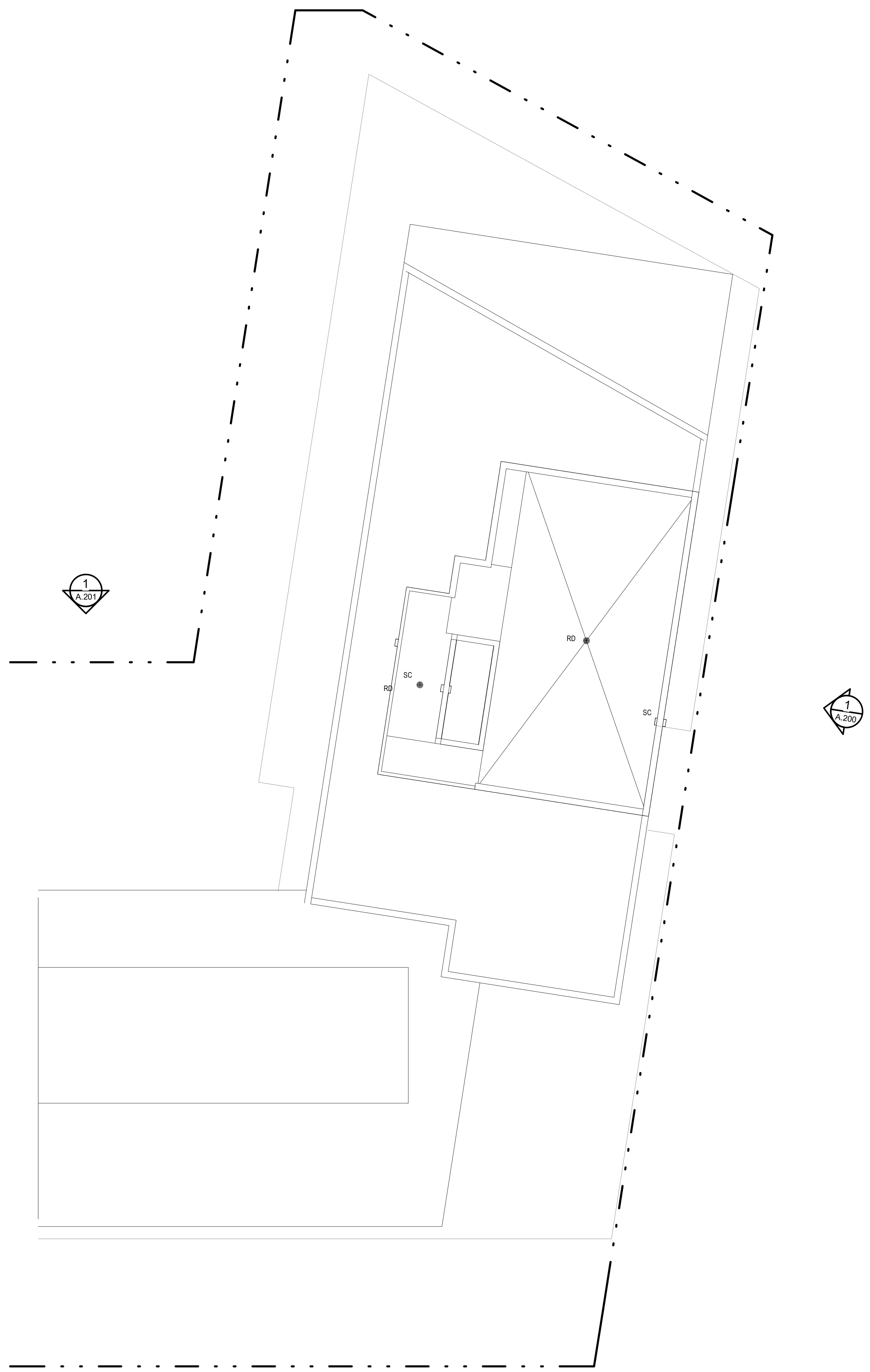
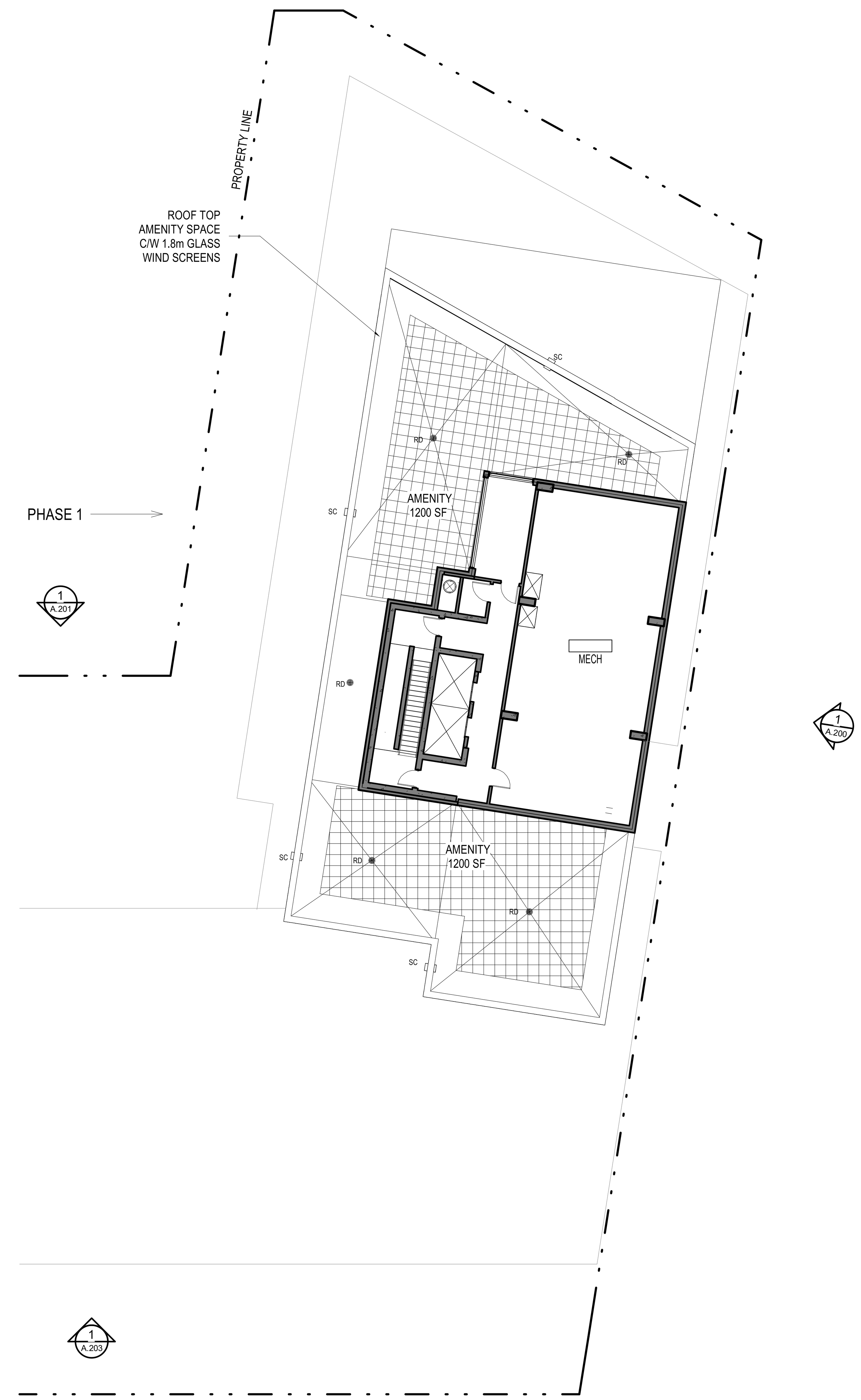


2 A.143 MECHANICAL ROOF PLAN
1 : 200



1 A.143 ROOF PLAN
1 : 200

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ROOF PLAN - TOWER

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