MORRISON HERSHFIELD

REPORT

665 Albert Street LeBreton Flats Library Parcel Confederation Line Proximity Study

Presented to:

Andrew McCreight
Development Review Project Manager, Buildings and Parks

Project: 202093300 April 22, 2022

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1. Introduction

Morrison Hershfield Limited (MHL) has been retained by Dream Unlimited to prepare this Confederation Line Proximity Study (CLPS) for the proposed development of the 'LeBreton Library Parcel' at 665 Albert Street ("the development").

The purpose of this CLPS report is to demonstrate that potential impacts of the development on the Confederation Line assets and operations have been identified and addressed, and vice versa.

The City of Ottawa (City) has requested that a CLPS for the development because the site lays partially within the Confederation Line's Development Zone of Influence.

The required scope of a CLPS varies depending on the type of impacts that a development is expected to cause to the Confederation Line assets and operations. In general the scope addresses temporary impacts (during construction) and permanent impacts, and considers issues such as structural/geotechnical interaction (potential settlement, or loading on tunnels and retaining walls), changes to maintenance access routes, changes to station access (including wayfinding and accessibility), and encroachment on the Confederation Line during construction.

This CLPS has been completed in accordance with the City's guidelines (Confederation Line Proximity Study Guidelines, October 2013). The scope of the CLPS was determined based on our understanding of the development (described in **Section 2.1**) and is described in **Section 3**. **Section 4** includes a detailed examination of the potential Confederation Line impacts, and explains how the development will address these.

2. Proposed Development

2.1 Description of Proposed Development

It is understood that the proposed development will include two residential towers with retail uses on the ground floor of each building, and a daycare centre on the second floor of the east building. The two towers (31-storey East Tower and 36-storey West Tower) will be located on top of a common two-level underground parking garage, to be constructed on the 665 Albert Street property which is located just south of the Confederation Line Light Rail Transit (LRT) corridor. **Figure 1** indicates the project location in relation to the LRT corridor and Confederation Line's Development Zone of Influence.

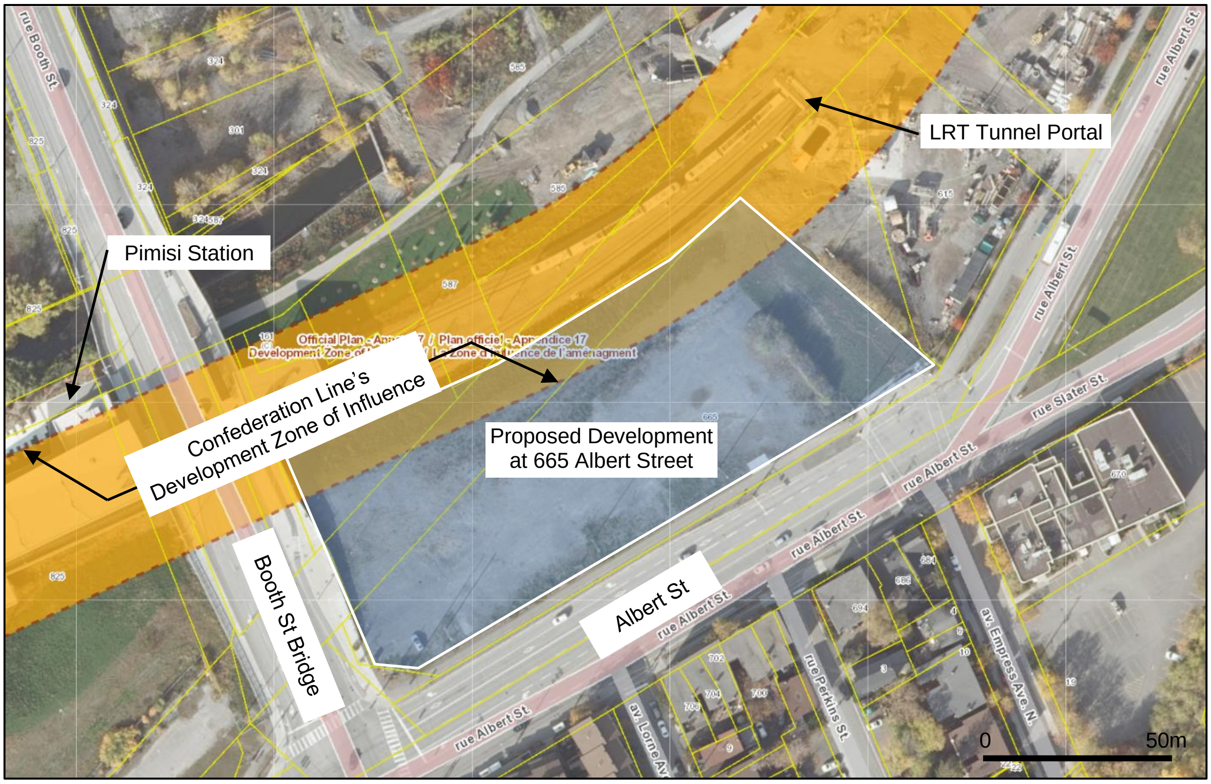


Figure 1 – Project Location

As indicated by the topographic survey (**Appendix A**) the site is at a higher elevation than the LRT tracks, with a slope down to the tracks along the north edge of the site. Adjacent to the site the LRT profile is relatively flat. A slope down towards the tunnel portal begins at approximately the north-east corner of the site.

There are two separate existing retaining walls adjacent to the site:

- From Booth St bridge, continuing approximately 20m east, a small reinforced concrete retaining wall allows for the proposed multi-use pathway (MUP) to be installed at a lower elevation than the tracks.
- From mid-way along the site continuing eastwards, a Mechanically Stabilized Earth (MSE) wall supports the approach to the tunnel portal.

An existing chain-link fence separates the LRT tracks from the development property.

The proposed Site Plan (**Appendix B**) includes the proposed MUP adjacent to the LRT. The MUP will be located within the development property parcel.

A retaining wall (or series of terraced walls) is proposed to replace the existing slope between the MUP alignment and the finished grade of the site.

Within the site, the area above the parking garage and a triangular area between the building and north / west property lines will be landscaped with plantings, paved areas and seating. Because of the elevation difference, along the north side of the site the landscaped areas will form a terrace overlooking the LRT tracks.

The site plan allows for a future pedestrian bridge across the LRT from this terrace, potentially connecting to the MUP on the north side of the LRT. The pedestrian bridge is currently a concept only. It does not form part of the proposed development.

An emergency generator is proposed to be located in the north-west corner of the site, at the foot of the new retaining wall, adjacent to the Booth Street bridge and MUP.

There are existing sewers within the former Wellington Street right-of-way which crossed the north-west edge of the site. These include a 2400mm diameter combined sewer from the Booth/Wellington Regulator to the first shaft of the Combined Sewer Storage Tunnel (CSST) the 1800mm diameter Interceptor Outfall Sewer (IOS), and a 3000mm diameter storm sewer. The proposed site plan indicates that the building (including parking garage) will be set back from these sewers.

2.2 Basis of Study

This report is based on MHL's understanding of existing conditions and the proposed development, obtained from review of the following documents.

Existing conditions:

- Topographic Survey of Lots 10, 12, 14, 16, 18, 20 and Part of Lots 7, 8, 9, 11, 13, 15, 17, 19 and 21, Block 'M', Registered Plan 2, City of Ottawa, Stantec Geomatics Ltd., March 24, 2022, Revision 1
- Confederation Line As-built Drawings, HATCH, OLRT Constructors, July 5th, 2019

Proposed development:

- Architectural Plans, 665 Albert Street, KPMB / Perkins & Will, August 26, 2021, Revision 1
- Geotechnical Investigation Report, 665 Albert Street, Report Number 22511882, Golder Associates Ltd., April 2022

3. Proximity Study Requirements

The guidelines identify three levels of CLPS, from the Level 1 (most basic) to Level 3 (most stringent). The CLPS level is selected based on the anticipated degree of impact to the Confederation Line.

For 665 Albert Street a Level 2 review is considered appropriate. While the building itself is set back from the LRT and unlikely to cause substantial impact, replacement of the existing slope from the site down to the Confederation Line will require retaining wall construction in close proximity to the tracks, warranting a Level 2 review.

Table 1 summarizes the three review levels and relevant characteristics of the development.

Table 1 – CLPS Review Level

Level of Review	Type of Review	Relevant Characteristics of Development
1	Development within Development Zone of Influence, minimal impact on Confederation Line structures anticipated	• Proposed 30 and 35 storey towers at 665 Albert Street
2	Development within Development Zone of Influence, substantial impact on Confederation Line structures anticipated	• Proposed MUP and associated grading • Proposed retaining wall(s)
3	On top of or within approximately 1 m of a Confederation Line structure	• Future pedestrian bridge (by others)

4. Proximity Requirement Responses

This section lists the CLPS requirements and describes how each requirement will be met for the development.

Table 2 - Level 1 CLPS Requirements and Responses

Requirement (from CLPS Guidelines)	**Response**
Required for all developments:	
A site plan of the development with the centreline or reference line of the Confederation Line structure and/or right-of-way located and the relevant distances between the Confederation Line and developer's structure shown clearly;	Presented in **Appendix B**
Plan and cross-sections of the development locating the Confederation Line structure/right-of-way and founding elevations relative to the development, including any underground storage tanks and associated piping;	Presented in **Appendix B**. Note that site service connections (water, sewer) and stormwater detention tanks are proposed to be on the south side of the site, between the building and Albert Street, outside the Confederation Line's Development Zone of Influence.
A geotechnical investigation report showing up-to-date geotechnical conditions at the site of the development. The geotechnical investigation shall be prepared in accordance with the Geotechnical Investigation and Reporting Guidelines for Development Applications in the City;	Presented in **Appendix C**
Structural, foundation, excavation, and shoring drawings; including all associated plans, sections and details, schedules, loads	<u>Foundations (Building)</u> Structural and foundation design drawings were not available at the time of the

Requirement (from CLPS Guidelines)	**Response**
on foundation, shoring design criteria, and description of excavation and shoring method.	preparation of this study. However, considering that the lowest level of the parking garage (P2 level) is indicated to be at approximately 53.6m elevation (significantly lower than the LRT tracks which are at approximately 56m elevation), it is clear that the building will be founded deeper than the LRT tracks. The foundations are therefore not expected to impact the LRT tracks or associated structures. Support-of-Excavation (Building) Detailed Support-of-Excavation plans were not available at the time of preparation of this study. It is anticipated that shoring will be installed along the south (Albert Street) and west (Booth Street) elevations of the parking garage wall. A tied-back soldier pile and lagging system is expected. Sloped excavations are anticipated along the north (LRT) and east elevations of the parking garage, where sufficient space is available between the property line and planned parking garage wall. Preliminary review suggests that sloped excavation will be feasible where the set-back of the proposed parking garage from the property line is approximately 17m or more, considering the Confederation Line Top-of-Rail elevation of approximately 56.5m, base of excavation of approximately 51m, and requirement for a 3H:1V excavation slope below the water table (expected at or around the LRT rail elevation). The set-back at the east end of the site reaches a minimum of 10m, which may necessitate a shoring system Retaining Walls The retaining walls along the north edge of the site will be constructed in relatively close proximity to the LRT (although set back by at least the 3m width of the MUP). A gravity wall system is currently proposed. Excavation may be required for installation of a granular base and/or footing. It is not anticipated that shoring will be required, but this will be confirmed as design progresses.

Requirement (from CLPS Guidelines)	**Response**
Acknowledgement that the potential for noise, vibration, electro-magnetic interference and stray current from Confederation Line operations have been considered in the design of the project, and appropriate mitigation measures applied.	A noise and vibration study is in progress. Once complete, any applicable recommendations will be implemented in the design.
Required depending on type of development:	
Architectural, mechanical, electrical and utility drawings.	Refer to **Appendix D** for Architectural plans**.** As described above, all main utilities and servicing is planned to be from Albert Street. New services between the building and north property line are expected to be limited to local drainage, lighting, and service connections to the proposed emergency generator. No impact to the Confederation Line is expected.
A National Fire Prevention Association (NFPA) 130 Standard review to ensure design requirements in relation to Confederation Line infrastructure are met.	Not applicable due to the limited impacts of the development on the Confederation Line infrastructure.
Crane locations, loadings.	Crane locations and loadings will be determined by the construction contractor at a later date.
Up-to-date surveys, signed and sealed by an Ontario Land Surveyor, as follows: o A property survey of existing and proposed property lines prepared to Strata Reference Plan Standards; o A topographic survey of existing surface items, such as buildings, contours, roads, tracks; o A utility survey of existing building gridlines, including those of Confederation Line structures; o A preliminary gridline layout survey of proposed building gridlines on architectural and structural drawings.	An up-to-date topographic and legal survey is provided in **Appendix A**. The Site Plan (**Appendix B**) indicates the location of the Confederation Line and associated structures. These locations were determined from As-Built drawings obtained from the City (**Appendix E**). The use of As-Built drawings is considered sufficiently accurate for the this project, and avoids the complexity and disruption of completing topographic survey within the Confederation Line right-of-way.
Staging of operations.	It is currently anticipated that construction will not require any work within the Confederation Line right-of-way, or disruption of Confederation Line operations.
Traffic management plan, which shall included site access provisions during and	No direct impacts on roads servicing Confederation Line facilities are anticipated.

Requirement (from CLPS Guidelines)	**Response**
after construction (ultimate), lane closures and staging of traffic management plan.	

Table 3 - Level 2 CLPS Requirements and Responses

Requirement (from CLPS Guidelines)	**Response**
A structural analysis or calculations of the effects of loadings, including construction loading, on the Confederation Line structure, and demonstrating that the Confederation Line structure will not be adversely affected by the development, including solutions to mitigate any impact on the Confederation Line structure. The documentation must include identification of the "affected" Confederation Line structural units	As described in **Table 2**, the proposed building is not expected to impose any loads on the Confederation Line or related structures due to it being founded at a lower depth. Formal confirmation of this can be provided once structural design is complete, if required.
Documentation showing that the excavation support system and permanent structure adjacent to the Confederation Line property are designated for at-rest earth pressures. Unless otherwise proven through mutually accepted geotechnical analysis, At-rest pressures shall be determined using a pressure coefficient of 0.5 (K0 = 0.5)	Lateral earth pressures for design are recommended within the Geotechnical Report (**Appendix C**). All temporary support-of-excavation systems, and permanent retaining walls adjacent to the Confederation Line should be designed in accordance with the recommendations of the Geotechnical Report. Design of temporary support-of-excavation systems will be the responsibility of the Construction Contactor.
Structural drawings, including caisson/foundation plans, sections and details, floor plans, column and wall schedules and loads on foundation for the development. The relationship of the development to the Confederation Line structure should be depicted in both plan and section	As described in **Table 2**, due to the distance between the building and the Confederation Line, and the depth of the foundations, the building is not expected to impact the Confederation Line. Design drawings of the proposed retaining wall (in closer proximity to the Confederation Line) were not available at the time of report preparation.
Shoring design criteria and description of excavation and shoring method	The Geotechnical Report (**Appendix C**) provides shoring design criteria. As described in **Table 2**, shoring is currently not anticipated adjacent to the Confederation Line.
Ground water control plan, including the determination of the short-term (during construction) and long-term effects of dewatering on the Confederation Line structure, and provision of assurances that	The Geotechnical Report (**Appendix C**) indicates that both temporary and permanent groundwater control will be required.

Requirement (from CLPS Guidelines)	**Response**
the influences of dewatering will have no impact on the Confederation Line structure	A groundwater control plan should be prepared prior to construction, and will be required to address the potential for any impacts to the Confederation Line structures.
Proposal to replace/repair waterproofing system of the affected Confederation Line structure, including the Confederation Line expansion joint;	Not applicable.
Identification of utility installations proposed through or adjacent to Confederation Line property. Where known, show Confederation Line utility connections where associated municipal connections are to be modified	No utility installations are expected within the Confederation Line property as a result of the development.
Identification of the exhaust air quality and relationship of air in-take/discharge to the Confederation Line at-grade vent shaft openings and station entrance openings. (Confederation Line shaft openings would typically be located a minimum of 12 meters from entrances or exits because vent shaft openings are used as emergency ventilation in-take or exhaust vents for high temperature smoke in the event of a fire.)	The building is located more than 12 meters from the tunnel portal. There are no other Confederation Line tunnel ventilation facilities in the vicinity.
Proposal for a pre-construction condition survey of the Confederation Line structure, including a survey to confirm locations of existing walls and foundations	The contractor will be required to undertake a pre-construction survey of Confederation Line structures within 30m of the site prior to the start of construction. Pre-construction surveys will be in accordance with City of Ottawa Special Provision F-1011.
Monitoring Plan for movement of the shoring and Confederation Line structure prior to and during construction of the development, including an Action Protocol	A monitoring plan will be developed if shoring is confirmed to be required in the vicinity of the Confederation Line (i.e. along the north property line of the site).

5. Conclusions

Due to the layout of the site, which includes a set-back between the Confederation Line right-of-way and the proposed building, the impacts of the proposed development on Confederation Line structures are expected to be limited.

No construction work is currently proposed within the Confederation Line right-of-way.

The existing slope from the site to the Confederation Line will be replaced by a retaining wall or walls. Subject to review of the detailed design of this wall (not available at the time of report preparation) the wall is not expected to have any impact on the Confederation Line. Regrading immediately south of the existing fence will be required for construction of the MUP. Sections of retaining wall may be required between the MUP and fence to accommodate grade differentials. It is anticipated that all grading and retaining walls will be design to enable construction without impact to the Confederation Line.

The excavation for the proposed building is currently planned to be sloped rather than shored wherever possible along the north elevation (closest to the Confederation Line). A shoring system may be required towards the east end of the site, where the set-back appears insufficient for an unsupported sloped excavation. Shoring plans will be the responsibility of the construction contractor.

Dewatering will be required. The potential for any impacts to the Confederation Line structures should be reviewed through preparation of a Groundwater Control Plan.

No impacts to Confederation Line operations are expected.

6. Closure

We trust that this report is sufficient for your current requirements. Please contact the undersigned with any questions or clarifications.

Sincerely,

Morrison Hershfield Limited

LICENSED PROFESSIONAL ENGINEER
J.G. FOOKES
April 22, 2022
PROVINCE OF ONTARIO

James Fookes, P.Eng.	Dillon O'Neil, B.Eng.
Senior Municipal Engineer	Municipal Designer

APPENDIX A: Topographic and Legal Survey

Stantec

Stantec Geomatics Ltd.
300-1331 Clyde Avenue
Ottawa ON
Tel. 613.722.4420
www.stantec.com

TOPOGRAPHIC SURVEY OF

LOTS 10, 12, 14, 16, 18, 20 AND PART OF LOTS 7, 8, 9, 11, 13, 15, 17, 19 AND 21 BLOCK 'M' REGISTERED PLAN 2

CITY OF OTTAWA

Scale 1:250

5 0 5 10 15 METRES

Stantec Geomatics Ltd.
ONTARIO LAND SURVEYORS

METRIC CONVERSION

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

BEARING NOTE

BEARINGS ARE GRID, DERIVED FROM THE CAN-NET VRS NETWORK OBSERVATIONS ON NCC HORIZONTAL CONTROL MONUMENTS 19773035 AND 19680191, CENTRAL MERIDIAN, 76°30' WEST LONGITUDE MTM ZONE 9. NAD83 (ORIGINAL).

19773035 N:5006060.42 E:324888.04
19680191 N:5033564.26 E:388064.94

ELEVATION NOTE

ELEVATIONS SHOWN HEREON ARE GEODETIC (CGVD-1928:1978) AND ARE DERIVED FROM THE CAN-NET VRS NETWORK MONUMENT: OTTAWA ELEVATION=95.230.

LEGEND

■	DENOTES	FOUND MONUMENTS
□	"	SET MONUMENTS
IB	"	IRON BAR
IBø	"	ROUND IRON BAR
SIB	"	STANDARD IRON BAR
SSIB	"	SHORT STANDARD IRON BAR
CC	"	CUT CROSS
RB	"	ROCK BAR
CP	"	CONCRETE PIN
WIT	"	WITNESS
PIN	"	PROPERTY IDENTIFICATION NUMBER
M/MEAS	"	MEASURED
PROP	"	PROPORTIONED
OU	"	ORIGIN UNKNOWN
STANTEC	"	STANTEC GEOMATICS LTD.
P1	"	PLAN 4R-34419
P2	"	PLAN 4R-32006
AN	"	ANCHOR
BKR	"	BIKE RACK
BENCH	"	BENCH
BOL	"	BOLLARD
CB	"	CATCH BASIN
CBSI	"	SIDE INLET CB
HYD	"	FIRE HYDRANT
LS	"	LIGHT STANDARD
MH	"	MAINTENANCE HOLE UNIDENTIFIED
MHSA	"	MAINTENANCE HOLE SANITARY
MHST	"	MAINTENANCE HOLE STORM
MW	"	MONITORING WELL
SN	"	SIGN
	"	TREE CONIFEROUS (D.B.H. SHOWN)
	"	TREE DECIDUOUS (D.B.H. SHOWN)

SURVEYOR'S CERTIFICATE

I CERTIFY THAT :

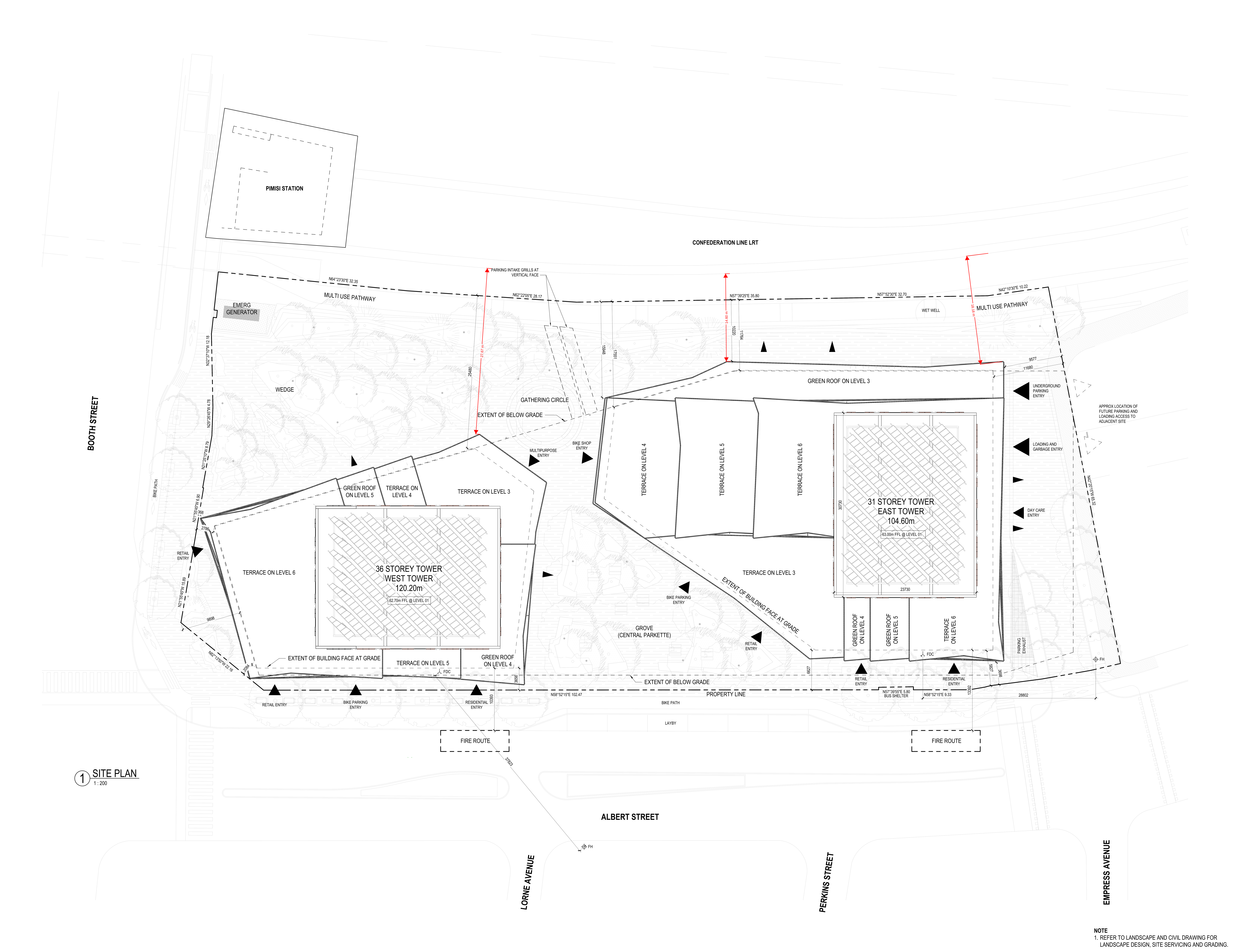
1. THIS SURVEY AND PLAN ARE CORRECT AND IN ACCORDANCE WITH THE SURVEYS ACT, THE SURVEYORS ACT AND THE LAND TITLES ACT AND THE REGULATIONS MADE UNDER THEM.
2. THE SURVEY WAS COMPLETED ON THE 24th DAY OF MARCH, 2022.

DATE

R. G. BENNETT
ONTARIO LAND SURVEYOR

DRAWN: ZF CHECKED: CK PM: CT FIELD: ES PROJECT No.: 161614531-111

APPENDIX B: Site Plan



NOTE
1. REFER TO LANDSCAPE AND CIVIL DRAWING FOR LANDSCAPE DESIGN, SITE SERVICING AND GRADING.

ISSUED FOR OPA, ZBA & SPA - NOT FOR CONSTRUCTION 2022 - 04 - 22

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CONTRACTOR
EllisDon
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PROJECT

LEBRETON LIBRARY PARCEL
665 Albert St
Ottawa, ON
Canada

DREAM
30 Adelaide St. E., Suite 301,
Toronto, ON M5C 3H1

KEYPLAN

ISSUE CHART

1	ISSUED FOR OPA / ZBA / SPA	2022-04-22

Job Number

TITLE

SITE PLAN

SHEET NUMBER

A01-03

APPENDIX C: Geotechnical Investigation Report

REPORT

Geotechnical Investigation

LeBreton Library Parcel
665 Albert Street
Ottawa, Ontario

Submitted to:

Dream Impact Master LP
30 Adelaide St. East
Toronto, ON M5C 3H1

Submitted by:

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April 2022

Distribution List

1 e-copy - City of Ottawa

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FIGURES

APPENDICES

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) was retained by Dream Impact Master LP (Dream) to conduct a geotechnical investigation at the property located at 665 Albert Street. The site is located north of Albert Street, east of Booth Street, south of the Fleet Street Aqueduct (open aqueduct), and west of the site of the new Ottawa Public Library (currently under construction). A Site Location Plan is attached as Figure 1.

The purpose of this investigation was to assess the general subsurface and groundwater conditions within the study area by means of a limited number of boreholes and associated laboratory testing. Based on an interpretation of the factual information obtained during the current investigation, along with the existing subsurface information available for the site from previous investigations, a general description of the soil and groundwater conditions is presented. These interpreted subsurface conditions and available project details were used to prepare engineering guidelines on the geotechnical design aspects of the project, including construction considerations which could influence design decisions. A Phase Two Environmental Site Assessment was completed concurrently with the geotechnical investigation, the results of which are presented under separate cover.

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

2.0 DESCRIPTION OF PROJECT AND SITE

It is understood that the proposed new development is an irregularly shaped structure which will consist of a four-storey podium filling the entire site, above which will be two 30 to 35 storey residential towers, covering a portion of the site. The development will include 2 levels of underground parking (below the entire footprint of the podium). The lowest level of parking is indicated to have a finished floor elevation of 53.6 m. The main ground floor of the development is indicated to be at an elevation of 62.0 m.

The site is currently vacant and forms part of the larger LeBreton Flats area which included a variety of historical industrial uses (past uses of the property are discussed in detail in the Phase One and Two Environmental Site Assessments. The site is unsurfaced and is relatively flat with existing ground elevations ranging from 60.5 m to 62.9 m (based on spot elevations at borehole locations).

Based on the results of previous investigations and the published geology maps available from the Geologic Survey of Canada (GSC) for this area, the subsurface conditions at this site are expected to consist of a surficial layer of fill, overlying a thick deposit of glacial till. The glacial till is underlain by interbedded limestone and shale bedrock of the Verulam formation.

3.0 PROCEDURE

3.1 Desktop Study

A previous geotechnical investigation was completed at the site by Golder Associates in 2011. This investigation included six boreholes located within the subject site. The boreholes (BH11-33, BH11-35 and BH11-37 to BH11-40) have been used to supplement the current investigation. The locations of these previous boreholes are shown on Figure 1. Copies of the previous borehole logs are included in Appendix A.

Based on the results of previous investigations and the published geology maps available from the Geologic Survey of Canada (GSC) for this area, the subsurface conditions at this site are expected to consist of a surficial layer of fill, overlying a thick deposit of glacial till. The glacial till is underlain by limestone and shale bedrock of the Verulam formation.

3.2 Field Investigation

The fieldwork for this current investigation was carried out between February 14th and 24th, 2022. During that time, a total of five boreholes (BH22-01 to BH22-05) were advanced at the approximate locations shown on Figure 1.

The boreholes were advanced using a track-mounted CME-55 hollow-stem auger drill rig with diamond coring capabilities supplied and operated by Downing Drilling of Hawkesbury, Ontario. The boreholes were advanced to depths ranging from 12.2 m to 16.5 m below the existing ground surface using a combination of auger drilling and diamond coring using NQ sized core barrels. Standard Penetration Tests (SPTs) were carried out within the overburden at regular intervals of depth. Samples of the soils encountered were recovered using 35 mm diameter split-spoon sampling equipment.

The fieldwork was supervised by technicians from our staff who located the boreholes, directed the drilling and in-situ testing operations, logged the boreholes and samples, and took custody of the soil and bedrock samples retrieved. On completion of the drilling operations, the soil samples were transported to our laboratory for further examination and laboratory testing. Laboratory testing was carried out on selected soil samples, including natural water content and grain size distribution tests. Basic chemical analysis related to potential sulphate attack on buried concrete elements and potential corrosion of buried ferrous elements was also completed on selected soil samples. Selected rock core samples were tested to determine the Uniaxial Compressive Strength (UCS) of the Rock.

Laboratory testing is in progress at the time of this draft report and the results will be included in the final report.

The borehole locations were selected in consultation with the City of Ottawa, marked in the field, and subsequently surveyed by City of Ottawa personnel. The geodetic reference system used for the survey is the North American Datum of 1983 (NAD83). The borehole coordinates are based on the Modified Transverse Mercator (MTM Zone 9) coordinate system. The elevations are referenced to Geodetic datum (CGVD28).

4.0 SUBSURFACE CONDITIONS

4.1 General

Information on the subsurface conditions is presented as follows:

- Borehole records are provided in Appendix A.
- Results of water content testing, grain size testing, UCS testing and basic chemical analyses will be included in the final report.

The Record of Borehole sheets describe the subsurface conditions at the borehole locations only.
The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling in some cases, observations of drilling progress as well as results of SPTs and, therefore, represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface soil, bedrock and groundwater conditions will vary between and beyond the borehole locations.

Unless otherwise noted, the following sections present an overview of the subsurface conditions encountered in the boreholes advanced during the current investigation. It should be noted that the shallow subsurface conditions noted on the borehole logs from the previous investigations may have changed since the boreholes were drilled, as such only auger refusal/bedrock depths and hydraulic response tests from previous drilling are discussed herein.

4.2 Overview of Subsurface Conditions

In general, the subsurface stratigraphy within the area of the investigation consists of surficial fill materials overlying glacial till, which in turn overlies limestone and shale bedrock.

4.3 Fill Material

Fill material was encountered in each of the boreholes from ground surface. The fill is heterogeneous in nature predominantly ranging from silty sand to sand. The fill also contains gravel, brick fragments, concrete and mortar fragments, glass, wood and layers of organic material and clay. Cobbles and boulders were also encountered during drilling. Fill material is, by its nature a heterogeneous material and other debris or obstructions could also be encountered with the fill.

SPT “N” values measured within the fill ranged from 6 to greater than 50 blows per 0.3 m of penetration during the two investigations (in 2011 and the current 2022 investigation). The SPT “N” values suggest that the fill has a highly variable very loose to very dense state of packing.

The fill material was fully penetrated in all of the boreholes at depths of between 2.1 and 3.7 m below the existing ground surface.

4.4 Glacial Till

A deposit of glacial till was encountered beneath the fill material at all of the boreholes. The glacial till typically consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sand and silt with a trace to some clay. Cobbles and boulders were encountered throughout the till during drilling and should be expected during construction.

The 2011 boreholes were terminated at auger and/or sampler refusal within the glacial till, and therefore did not fully penetrate the till layer. The five boreholes drilled during the 2022 investigation were all extended through the till and into the underlying bedrock, confirming the till extended to depth of 11.2 to 14.7 m.

SPT “N” values within the glacial till layer gave ‘N’ values ranging from 8 blows to greater than 100 blows per 0.3 m of penetration, and are typically greater than 50 blows per 0.3 m of penetration suggesting the majority of the till has a dense to very dense state of packing. Very high blow counts, however, could be indicative of boulders and cobbles in the till rather than the state of packing.

Borehole 22-04 encountered a layer of “till-like” silty sand and sandy silt which was less dense and not as coarse as the till at lower depths (and at similar depths in the surrounding boreholes) between approximately 2.1 and 6.1 m depth.

4.5 Bedrock

The 2011 boreholes were terminated at refusal in the glacial till layer at depths of 4.2 m to 10.0 m below the existing ground surface. Based on the current 2022 boreholes, it is unlikely that the majority of these refusals were the result of encountering the bedrock surface and were more likely due to cobbles and boulders within the till.

The current 2022 boreholes were extended through the glacial till deposit into the underlying bedrock using rotary diamond drilling techniques, while retrieving NQ core. The depths and elevations to bedrock surface in the current investigation are summarized below:

Borehole No.	Ground Surface Elevation (masl)	Depth to Bedrock (m)	Elevation of Bedrock (masl)
22-01	62.9	14.7	48.2
22-02	62.5	14.2	48.3
22-03	61.7	11.2	50.5
22-04	60.5	11.2	48.3
22-05	62.3	11.2	48.3

The bedrock consists of limestone with shale interbeds of the Verulam formation. Additional description of the bedrock is provided on the Borehole records provided in Appendix A.

4.6 Groundwater Conditions

Monitoring wells were installed in boreholes 22-01 to 22-05 during the current investigation. The groundwater levels observed in the monitoring wells have been summarized in the following table:

Well ID	Geologic Unit of Screened Interval	Groundwater Level		Date of Measurement	Date Well Installed
		Depth (mbgs)	Elevation (masl)		
22-01	Glacial Till	7.8	55.1	February 25, 2022	February 15, 2022
22-02	Glacial Till	7.9	54.6	February 25, 2022	February 16, 2022
22-03	Bedrock	13.0	48.7	February 25, 2022	February 22, 2022
22-04	Glacial Till	10.7	49.8	February 25, 2022	February 22, 2022
22-05	Glacial Till	8.2	54.1	February 25, 2022	February 24, 2022

It should be noted that groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as spring.

4.7 Corrosion Testing

To be included in final report.

5.0

5.0 DISCUSSION AND GEOTECHNICAL RECOMMENDATIONS

This section of the report provides engineering information related to the geotechnical design aspects of the project based on our interpretation of the available subsurface information and on our understanding of the project requirements. The discussion below focuses on the development of the proposed structure.

The information in this portion of the report is provided for detailed design purposes in support of the design by the engineers and architects. Where comments are made on construction, they are provided only in order to highlight aspects of construction which could affect the design of the project. Contractors bidding on or undertaking any work at the site should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, equipment capabilities, costs, sequencing and the like.

This report addresses only the geotechnical aspects of the subsurface conditions at this site.
The geo-environmental (chemical) aspects, including the consequences 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 report. The results of concurrent Environmental Site Assessment(s) for this project are provided under separate cover(s).

5.1 Site Grading

It is understood that a grade raise of up to 2.4 m is proposed at the site to match the proposed grade raise of Albert Street. The proposed grade raise is within acceptable limits for the soils at this site. A proposed grading plan was not available for review at the time of writing this report. The currently proposed ground floor level (Level 0) is indicated to have a finished floor elevation of 62.0 m. Based on elevations of the existing boreholes the current site grades are slightly above this (between 62 and 63 m elevation), with localized areas being lower (for example, BH22-02 and BH22-04 at 61.7 m and 60.5 m). The majority of the developed site will be excavated to accommodate the two floors of underground parking.

Based on the underlying soil conditions, there are no significant concerns with settlement due to the relatively minor grade raises required to develop the site.

5.2 Foundation Design

Based on the preliminary drawings provided, the entire footprint of the proposed development includes two floors of underground parking. The finished floor elevation of the lower (P2) parking level is indicated to be 53.6 m elevation. This compares with existing grades of approximately 62 to 63 m elevation based on the borehole elevations, and a bedrock surface at approximately 48 m elevation at the majority of borehole locations.

There are a number of options, from a foundation perspective:

- Assuming the lowest level of parking remains at 53.6 m elevation as shown on the drawings it will be within the glacial till. It is unlikely that the large 30 to 35 storey towers can be founded on conventional spread footings on the glacial till. Deep foundations (piles or caissons) would be appropriate for the high-rise towers. Deep foundations are discussed in Section X.X below.
- It may be feasible to found the lower podium structure (which is only 4 storeys) on shallow foundations (spread footings). Shallow foundations are discussed in Section X.X below.

- It would also likely be feasible to found the podium structure on a raft or mat foundation within the glacial till. Raft/mat foundations are discussed in Section X.X below.
- If the foundations (for the high-rise towers) can be lowered to bedrock (approximately 5 m lower than the current P2 level) it would be feasible to found the large towers on shallow foundations on bedrock).
- It may be feasible to found the entire development (podium and towers) on a single large raft within the till. A raft foundation suitable for the high-rise towers, however, would likely have a significant thickness (potentially several metres) to provide the rigidity required. Given, however, that there is only approximately 5 m of soil between the bedrock surface and the P2 level it is likely that it would be more cost-effective to simply found the building on rock than to construct a very thick continuous raft below the entire development.

5.2.1 Deep Foundations

Assuming foundation level cannot be lowered to bedrock, it is likely that at least the large high-rise towers would need to be founded on deep foundations. Typically, driven steel piles or cast-in-place concrete piles (with rock sockets) would be used.

Driven steel piles are typically more cost-effective for moderate vertical loads, but because of the short length (less than 5 m) they will provide almost no uplift or lateral resistance. Driven steel piles typically require larger groups of piles, with associated pile caps to resist larger loads. Cast-in-place concrete piles tend to be more expensive for resisting purely vertical loading but can provide very large lateral and uplift resistances. Cast-in-place concrete piles can also generate very high compressive resistances and therefore a single pile (or small group) can be used in place of a larger group of driven piles.

5.2.1.1 Driven Steel Piles

The proposed hospital structure may be supported on driven steel piles. Steel H-piles and closed-ended steel pipe piles are both commonly used in the area.

In general, the subsurface conditions in the vicinity of the proposed hospital building consist of variable deposits of fill with some localized areas of silty clay overlying a deposit of glacial till, overlying localized deposits of interlayered sands which in turn overlies shaley limestone bedrock. A piled foundation system could be used to transfer the foundation loads through the overburden soils to the underlying bedrock.

Axial Resistance

Piles driven to sound rock generate high ultimate geotechnical capacities, generally equal to or in excess of the structural capacity of the steel section (i.e., with increased loading or driving stresses, the steel section will become damaged and fail before the bedrock yields). For the purposes of design, the ultimate geotechnical resistance of the rock may be assumed to be equal to the ultimate resistance of the steel section.

A resistance factor of 0.4 should be applied to this value to obtain the factored resistance of a pile driven to sound rock. The resistance factor may be increased to 0.5 if a program of dynamic (PDA) testing is implemented, or 0.6 if static load testing is performed.

As an example, an HP310x79 has an ultimate resistance of 3,493 kN (based on the cross-sectional area, assuming 350 MPa yield stress and ignoring buckling, bending, lateral loads, sacrificial thicknesses or other more complex conditions which may reduce the structural capacity). The factored geotechnical resistance of an HP310x79 driven to sound rock may therefore be assumed to be 1,397 kN (3,493 kN x 0.4). A similar methodology may be used to estimate the geotechnical resistance of other pile sections.

Settlements for piles driven to sound rock are generally negligible, and the geotechnical resistance mobilized at 25 mm of settlement (a typical SLS condition) would be expected to exceed the factored axial resistance at ULS. Geotechnical SLS considerations therefore do not generally govern the design of pile driven to sound rock.

Piles spacings should not be less than three pile diameters (centre-to-centre) to prevent group effects. If closer pile spacings are required they can be accommodated, but the individual pile capacity may need to be reduced to account for the closer spacing. This can be reviewed in detailed design if required.

Uplift Resistance

The uplift resistance of a driven pile is a result of skin friction acting along the surface area of the embedded pile. The unfactored shaft resistance may be assumed to be equal to:

$$q_s = \beta\sigma_v'$$

Where:

q_s = the unfactored shaft resistance (in kPa);

β = a shaft resistance factor based on soil type and strength (use 0.8);

σ_v' = the vertical effective stress at the adjacent to the pile at depth z, equal to $z\gamma'$;

γ'= the effective unit weight of the soil which may be assumed to be 9 kN/m^3

A resistance factor of 0.3 should be applied to this value, to obtain the factored geotechnical uplift resistance. The dead weight of the pile itself, with an appropriate resistance factor for dead weight, may also be added to the geotechnical resistance in calculating the total uplift resistance.

The total uplift resistance of a pile group is the lesser of the sum of the individual pile resistances as described above, or the resistance of a single “block” of soil with a perimeter equal to the perimeter of the pile group (the mass of the soil inside the “block” may be included in the calculation; use a soil weight of 9 kN/m^3).

It should be noted that the uplift resistance of piles is highly dependent upon the installation of the piles as well as the layout of the pile groups. If the piles are relied upon to resist significant uplift loads, and uplift governs the design, consideration may be given to carrying out a tension test to confirm the uplift capacity.

Negative Friction

The raising of the grade or lowering of the groundwater table at or around the site may cause settlement of the existing soils. Localized settlement could also potentially be caused during a seismic event. In any of these cases, the potential will exist to develop negative skin friction (or downdrag) along the piles, and this should be considered in the design.

The magnitude of negative friction depends on the pile loading, pile dimensions and the final configuration of the site as well as the details of the below-grade portions of the building. The location of negative friction forces is also dependent on the location of the neutral axis of the pile which can only be determined once all of the pile details are known. For preliminary design, however, the negative friction can be assumed to be equal to the shaft friction calculated as described above for uplift resistance (the resistance factor of 0.3 should not be applied).

Negative friction is typically only considered in conjunction with dead and sustained live loads (not transient loads such as wind, earthquake and transient live loads) in evaluating the structural capacity of the pile. Negative friction does not impact the geotechnical resistance of the piles.

Lateral Resistance

The lateral resistance of a slender pile is typically governed by limiting the deflection which will occur under loading to some acceptable level. The geotechnical parameter most commonly used to determine lateral deflection of piles is the coefficient of horizontal subgrade reaction (k_h). For this site, k_h may be assumed to be:

$$k_h = \eta_h z$$

Where:

k_h = the modulus of subgrade reaction (kN/m^3);

η_h = a coefficient based on soil type (use 4.4 MPa/m); and,

z = the depth under consideration

The value above is for a single pile group. Group interaction must be considered when piles are spaced closely together. Group effects may be accounted for by reducing the coefficient of horizontal reaction (k_h) by an appropriate factor as follows:

Table 1: Coefficient of Horizontal Subgrade Rection Reduction Factors

Pile Spacing in Direction of Loading (d = Pile Diameter)	Reduction Factor
6d	1.0
3d	0.25

Values for other spacings may be interpolated from the values above. No reduction is required for the first row of piles (i.e., the row which bears against undisturbed soil with no piles in front).

It should be noted that the method of applying a linear "spring" to represent the soil reaction to loading is a significant simplification of the soil/pile behaviour. If lateral load resistance governs the pile design, more rigorous, non-linear methods of analysing resistance exist, one common one being the method of p-y curves. These methods, however, require knowledge of the pile size, location, loading, pile cap construction, etc. and are therefore typically more suited to the detailed design phase when these items are known. Golder can provide additional assistance during detailed design, if required.

Construction Considerations

The piles will be driven to bedrock through a layer of glacial till which is known to contain cobbles and boulders. Piles can deflect or become damaged if they encounter boulders in the glacial till. Piles (both H-piles and pipe piles) should be equipped with pile points (e.g., Titus Standard H Point, or similar) to provide additional protection to the pile tips against damage from boulders during driving. Even with this measure, it should be expected that damage may occur to some piles and replacement piles will be required. For piles driven to refusal on bedrock, and as described in OPSS 903, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and then gradually increase the energy over a series of blows to seat the pile.

Provision should be made for restriking all piles at least once to confirm the design set and/or the permanence of the set and to check for upward displacement due to driving adjacent piles. Piles that do not meet the design set criteria on the first restrike should receive additional restriking until the design set is met. All restriking should be performed a minimum of 48 hours after the previous set.

Pile driving criteria depend not only on the details of the pile (size, length, load, etc.) but also on the equipment used for installation. Preliminary pile driving criteria should be established prior to construction using wave equation analysis (WEAP or similar) or other approved means and confirmed through a program of dynamic (PDA) testing carried out at an early stage in the piling program. Additional PDA testing should be used to confirm the pile capacities at regular intervals as the project progresses. As a preliminary guideline, the specification should require that at least 10% of the piles be included in the dynamic testing program. CASE method estimates of the capacities should be provided for all piles tested. These estimates should be provided by means of a field report on the day of testing. As well, CAPWAP analyses should be carried out for at least one half of the piles tested, with the results provided no later than three days following testing. The final report should be stamped by an engineer licensed in the province of Ontario. The PDA testing program will justify an increase in the geotechnical resistance factor to 0.5 as discussed above.

It should be noted that the driving energies required confirm the full ultimate resistance of the pile (typically the testing is intended to prove a load of twice the design load) may be higher than the energy required to install the pile. Insufficient energy is a common problem in demonstrating the true ultimate capacity of piles during PDA testing, and larger pile driving hammers may be required for the testing where piles are driven to rock in order to generate high axial capacities.

The piling specifications should be reviewed by Golder prior to tender, as should the contractor's submission (shop drawings, equipment, procedures, preliminary set criteria, etc.) prior to construction. Piling operations should be inspected on a full-time basis by geotechnical personnel to monitor the pile locations and plumbness, initial sets, penetrations on restrike, and to check the integrity of the piles following installation.

5.2.1.2 Drilled Cast-in-Place Piles

If drilled piles are used, they should be socketed into the limestone bedrock. The use of a casing will be required to advance the caisson through the glacial till material into the underlying bedrock. The casing should be extended so that it is "seated" a minimum of 500 mm into the bedrock.

5.2.1.3 Axial Geotechnical Resistance

Due to the difficulty in socketing liners into the limestone bedrock to completely cut off the water infiltrations, it may not be feasible to dewater and clean the base of the piles, or to inspect the base prior to concreting. As such, end-bearing support may not be fully developed and should be neglected in the design. The axial geotechnical resistance for rock-socketed caissons is therefore recommended to be based on the side-wall (shaft) resistance of the rock socket rather than end-bearing.

Rock-socketed cast-in-place piles should be designed based on the sidewall (shaft) resistance of the rock socket and a factored geotechnical resistance at ULS of 1.1 MPa, provided that the caisson socket is within competent bedrock (i.e., RQD greater than 50 percent). For preliminary design this condition can be assumed to be 1 m below the bedrock surface. This value assumes that the side wall of the socket will be cleaned of any cuttings or smeared material.

Settlements for rock-socket piles are typically small, and the factored ULS axial resistance will be reached before the pile has experienced 25 mm of settlement (a typical SLS condition). Geotechnical SLS considerations therefore do not generally govern the design of rock-socketed cast-in-place piles.

SLS resistances do not apply to caissons founded within the limestone bedrock, because the SLS resistance for 25 mm of settlement is greater than the factored axial

Pile spacings should not be less than three pile diameters (centre-to-centre) to prevent group effects. If closer pile spacings are required they can be accommodated, but the individual pile capacity may need to be reduced to account for the closer spacing. This can be reviewed in detailed design if required.

5.2.1.4 Lateral Geotechnical Resistance

To provide full fixity, the drilled cast-in-place piles should be provided with a minimum socket length equal to the greater of 2 times the caisson diameter below the depth of any broken or highly weathered surficial bedrock (which may be assumed to be 1 m). The structural engineer should confirm that the shear strength of the concrete is adequate to support these loads. In this condition, the rock sockets may be assumed to be "fixed" at the rock socket for preliminary design. This assumption should be confirmed during detailed design based and the actual pile dimensions, and depths.

The SLS geotechnical response of the soil in front of the caissons under lateral loading may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h, is based on the equation given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (3rd Edition). It may be assumed that this resistance (from the soil in front of the piles) will be nearly the same for vertical and inclined piles.

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

Where: n_h is the constant of horizontal subgrade reaction, Use 4.4 MN/m^3;

z is the depth (m); and,

B is the pile diameter/width (m)?

The discussion provided in Section 5.2.1.1.4 regarding the use of a "spring constant" to represent the relatively complex behaviour of the soil/rock/pile applies to drilled piles as well. Golder can undertake additional analysis during detailed design if lateral loading is a significant issue.

5.2.2 Shallow Spread Footings

Although not likely suitable for the high-rise towers, it may be feasible to support more lightly loaded parts of the structure on shallow spread footings on the dense glacial till. If lowering the foundations is a feasible option, then shallow foundations on bedrock are also suitable (both for the podium and the towers).

5.2.2.1 Footings on Glacial Till

Spread footings founded on the dense glacial till below the currently proposed P2 level may be a feasible option for lighter parts of the structure. An SLS net bearing resistance of 250 kPa and a factored ULS bearing resistance of 400 kPa can be used for design of pad footings up to 5.0 m in width and for strip footings up to 2.0 m in width placed on native and undisturbed glacial till below this elevation. The SLS values provided correspond to calculated total and differential settlement values of 25 and 19 mm, respectively.

It should be noted that because the expected settlements of any piled foundations are very small, differential settlements of up to about 25 mm may occur between the spread footings placed on glacial till and any parts of the development supported on piles. The design of the new structure will have to consider these differential settlements. Structural separation may be required between the foundations supported on piles, and those supported on glacial till.

For ULS sliding resistance of a cast-in-place footing placed on glacial till, an unfactored friction coefficient of 0.45 can be used. In accordance with OBC 2012 requirements, a resistance factor of 0.8 should be applied to the sliding resistance between the footings and the underlying glacial till.

5.2.2.2 Footings on Bedrock

For spread footings placed on sound bedrock, a factored Ultimate Limit States (ULS) bearing resistance of 4,000 kPa may be used for preliminary design. Serviceability Limit States (SLS) net bearing resistances do not generally apply to the design of foundations on the bedrock, provided the bedrock surface is properly cleaned of soil and loose highly weathered/fractured bedrock at the time of construction. As discussed above, differential settlements of up to 25 mm should be anticipated between areas which are founded on rock (which would be expected to experience negligible settlement) and areas which are founded on the glacial till.

For ULS sliding resistance of a cast-in-place footing placed on bedrock, an unfactored sliding friction coefficient of 0.70 can be used. In accordance with OBC 2012 requirements, a resistance factor of 0.8 should be applied to the sliding resistance between the footings and the underlying bedrock.

5.2.3 Raft or Mat Foundations

It may be feasible to support the structures (or portions of the structures) on a raft or mat foundation on the dense to very dense. A raft or mat foundation would need to be sufficiently rigid to ensure that the loading is uniformly distributed over the entire footprint of the raft, and to minimise the potential for differential settlement between heavily and lightly loaded areas.

Supporting the four-storey podium, plus two levels of parking on a raft foundation would be relatively straight-forward. Supporting the entire structure on a large raft would be more complex and because of the thickness of the heavily reinforced raft which would be required and the relatively thin layer of soil below the building it may be simpler to just lower the foundation level to the bedrock.

The design of a large, rigid raft foundation is not typically governed by an overall bearing capacity of the soil, but rather by the need to limit the differential settlement between different parts of the raft to some acceptable value. A raft foundation in soil typically experiences relatively large total settlement, but due to its stiffness limits differential settlement.

The geotechnical parameter most commonly used in this assessment is the vertical modulus of subgrade reaction (k_{v1}). For the dense glacial till, the vertical modulus of subgrade reaction may be assumed to be 65 MPa/m. This value is for a 300 mm by 300 mm loaded area which has been adopted as a standard for comparison.

The modulus of subgrade reaction is not a fundamental soil property and its value depends, in part, on the size and shape of the loaded area. The design modulus should be adjusted based on the loaded area as outlined in Section 7.7.1 of the CFEM (4th Edition, 2006). For a rectangular loaded area of width b and length mb:

$$k_{vb} = \left(\frac{kv1}{3.28b}\right) * \frac{m+0.5}{1.5m}$$

where

k_{vb} = the modulus for the actual loaded area; and

b = the width of the loaded area

The modulus of subgrade reaction is a significant simplification of actual soil behaviour. The presence of rock at relatively shallow depth as well as the likely variety of differently loaded areas also complicate the analysis. For detailed design a more rigorous design method such as a three-dimensional settlement analysis or finite element model would be more appropriate for a project of this scale. These analyses, however, cannot be undertaken without knowledge of the proposed foundation loading.

For the analysis of the contact stress distribution beneath a slab on grade foundation, the modulus of subgrade reaction value obviously depends on the size of the areas over which increased/concentrated contact stresses are anticipated and the stiffness of the raft itself (analogous to equivalent footings beneath the columns); the size of these areas is in turn related to the value of the modulus of subgrade reaction, i.e., they are inter-related. The design of a raft foundation is therefore typically an iterative process requiring both geotechnical and structural analysis of the settlement, load distribution and stiffness of the structure.

If the preliminary values provided above suggest that a raft foundation may be possible, Golder can assist with additional analysis during detailed design using this iterative approach.

5.3 Rock Anchors

The use of rock anchors to resist uplift forces on the foundations could be considered where additional uplift resistance is required.

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

i) Failure of the steel tendon or top anchorage

ii) Failure of the grout/tendon bond

iii) Failure of the rock/grout bond, and

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

Potential failure modes i) and ii) are structural and are best addressed by a structural engineer.

For potential failure mode iii), the *factored* bond stress at the grout/rock interface may be taken as 1,000 kPa (or 1/30 of the compressive strength of the grout) for ULS design purposes. This value should be used in calculating the resistance under ULS conditions. If the response of the anchor under SLS conditions needs to be evaluated, 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 is calculated based on the weight of the potential mass of rock and soil 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:

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

Where: Q_r = Factored uplift resistance of the anchor (kN);

φ = Geotechnical resistance factor (use 0.4);

γ′ = Effective unit weight of rock and soil (use 10 kN/m^3 below the groundwater level);

D = Anchor length in metres; and,

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

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:

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

Where: V = Volume of the truncated trapezoid failure zone (m^3);

D = Depth of anchor group (m);

a = Width of anchor group (m);

b = Length of the anchor group (m); and,

φ = ½ of the apex angle of the rock failure cone, use 30°.

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

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

Where: Qr = Factored uplift resistance of the anchor (KN);

φ = Geotechnical resistance factor, use 0.4;

γ′ = Effective unit weight of rock and soil, use 10 kN/m^3 below the water table; and,

V = Volume of truncated trapezoid (m^3).

It is recommended that proof load tests be carried out on any new anchors to confirm their resistance. The proof load tests should be carried out in accordance with the Post Tensioning Institute (PTI) Recommendations for Prestressed Rock and Soil Anchors (2004).

A geotechnical engineer should be present during the installation and testing of the anchors. Care must be taken during grouting to ensure that the grouting pressure is sufficient to bond the entire length of the grouted area with minimum voids.

Confirmation of sufficient embedment into the rock beneath the foundations should be carried out during construction to make sure that the anchors are being installed in rock of adequate quality. The anchor holes must be thoroughly flushed with water to remove all debris and rock flour. It is essential that rock flour be completely removed from the holes to be grouted to promote an adequate bond between the grout and the rock. Prestressing of the anchors prior to loading will minimize anchor movement due to service loads.

5.4 Frost Protection

All perimeter and exterior foundation elements or interior foundation elements (i.e., footings, pile caps, grade beams, etc.) in unheated areas should be provided with a minimum of 1.5 m of earth cover for frost protection purposes. Isolated, unheated exterior foundation elements 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.

As an alternative to earth cover, consideration could be provided to the use of an insulation detail. Additional guidance on insulation details can be provided if required.

5.5 Seismic Design Considerations

5.5.1 Seismic Liquefaction

There is no significant risk of liquefaction at the site during a seismic event.

5.5.2 Seismic Site Class

The OBC 2012 contains seismic analysis and design methodology. The seismic Site Class value, as defined in Section 4.1.8.4 of the OBC 2012, depends on the average shear wave velocity of the upper 30 m of soil and/or rock below founding level.

Based on the in-situ testing data, this site can be assigned a Site Class of C for seismic design purposes according to the 2012 OBC.

A higher site Class (Site Class A or B) can be assigned for “rock” sites (where the foundations are on, or very close to rock). The lowest level of the currently proposed development is indicated to be at an elevation of 53.6 m. This compares with a rock elevation of approximately 48 m over the majority of the site. If the final design is such that the underside of the foundations is within 3 m of the bedrock (i.e., at or below approximately 51 m elevation) a higher site class (i.e. a Site Class A or B) would apply. This would need to be confirmed with site specific shear wave velocity testing.

5.6 Excavations and Shoring

Based on the preliminary site plan provided, the lowest finished floor elevation is at 52.6 m. The main excavation will be lower than this by at least the thickness of the lower-level slab-on-grade, granular base, drainage, etc. Localized excavations would also be required for pile caps, footings, etc. as well as services. Based on borehole elevations this will require excavations on the order of 9 to 10 m deep over the entire site, with deeper localized excavations for foundations and services.

Excavations for the construction of the foundations and basement levels will be through the existing fill, and into the underlying glacial till. No unusual problems are anticipated with excavating the overburden using conventional hydraulic excavating equipment. Cobbles and boulders should be expected in the fill, glacial till and sand and gravel deposits. Debris (e.g., organics, brick, metal, wood, stone, concrete, etc.) should also be expected in the fill.

It is likely that significant portions of the site will require shoring (due to insufficient space to complete open cut excavations; for example, along Albert St., Booth St., adjacent to the OLRT ROW, etc.). There may be other areas where sufficient space exists for open excavation. Both are discussed below.

5.6.1 Open Cut Excavations

Above the groundwater level and within the fill, silty sand, native silty clay and glacial till side slopes should be stable in the short term at 1 horizontal to 1 vertical; these soils would be classified as Type 3 soils in accordance with the Occupational Health and Safety Act of Ontario (OHSA). This would also apply to areas where the groundwater table was drawn down and maintained below the final excavation depth in advance of excavation (in which case the soils are effectively above the water table at the time of excavation).

Excavations within the silty and sandy soils (both fill and till) *below the water table* would be classified as a Type 4 soil; these excavations would therefore require side slopes at a minimum slope of 3H:1V (i.e., flatter than 3H:1V).

It is expected that open-cut methods will generally be feasible (from a technical perspective) provided sufficient space exists to accommodate the excavations, though given the height they may require benching, access ramps, etc. to be incorporated into the design. It should be noted that the height of the excavations (10 m) exceeds the height for prescriptive design under the OHSA. Deeper portions of the excavation (even if open cut) will require an engineered design to comply with the relevant regulations.

Temporary excavations for foundations or site services (if required) will be through similar soils as discussed above. These excavations can also likely be made with sloping excavations where space permits. Where space does not exist, localized excavations for foundations or temporary services could be carried out with vertical sides and fully braced, steel trench boxes or shoring systems.

5.6.2 Shored Excavations

Where sufficient space does not exist (or if it is preferable to limit the size and impact of the excavation as well as associated excavation and backfilling) the temporary excavations could be carried out using a shoring system to ensure support for the soil and provide for worker safety. This section of the report provides some general guidelines on possible concepts for the shoring to be used by the designers for assessing the possible impacts of the shoring design and site works as well as to evaluate, at the design stage, the potential for impacts of this shoring on the adjacent properties and infrastructure. Temporary shoring can be used in combination with open cuts above the top of shoring, however, the earth pressure distribution must take into account the effects of the soil pressures from the upper sloped section.

This type of shoring system is typically designed and constructed by a specialist contractor who is fully responsible for the detailed design and performance of the temporary shoring systems. In addition to supporting the soils surrounding the excavation, the design of temporary support systems will need to consider the support requirements of adjacent structures, roads, utilities, etc.

The shoring method(s) chosen (and in particular the selection of the appropriate design earth pressures; higher design earth pressures are required if it is necessary to limit the deflection of the shoring) to support the excavation sides must take into account the soil and bedrock stratigraphy, the permissible movement of the shoring, the groundwater conditions, the methods adopted to manage the groundwater and construct the shoring systems, the potential ground movements associated with the excavation and construction of the shoring system, and their impact on adjacent structures and utilities.

The City of Ottawa rights-of-way for Albert Street and Booth Street, which contain below grade services (as well as bridge structures in the case of Booth St.) are located adjacent to the south and west sides, respectively, of the proposed excavation for the building. As such, any services located in close proximity to and/or within the zone of influence of the shoring system could be affected by ground movements behind the shoring. Details on the utilities in these areas should be confirmed during the detailed design studies to better tailor the shoring guidelines provided herein. Additionally, the right-of-way for the OLRT, as well as Pimisi Station is located adjacent to the north side of the proposed development and, if in close proximity to and/or within the zone of influence of the shoring system, could be affected by ground movements behind the shoring.

Shoring for this type of project would typically include tied back sheet pile walls or soldier pile and lagging systems (if a soldier pile and lagging system is employed the potential for flowing sands below the water table must be considered and addressed as part of the shoring/dewatering design). Due to the presence of very dense till with boulders at shallow depth on the site, soldier piles may require predrilling to provide sufficient embedment for toe fixity. Depending on the final design it may also be possible/necessary to socket the toe of the piles into rock. The shoring system must be provided with appropriate lateral support. Steel sheet piles cannot be pre-drilled and may have difficulty penetrating cobbles and boulders within the till (and certainly cannot be extended into rock for additional toe support).

Where foundations or settlement sensitive infrastructure, such as buried utilities, are present within the zone of influence of the shoring system and deflections need to be greatly limited a secant pile wall with pre-stressed tie backs may also be considered. Soldier pile and lagging walls are considered suitable for the sides of the excavations (provided that settlement-sensitive structures or utilities are not present in the zone of influence of the walls) where the objective is to maintain an essentially vertical excavation wall and the movements above and behind the wall need only be sufficiently limited so that relatively flexible features (such as roadways or sidewalks) will not be adversely affected.

Some form of lateral support to the wall is typically required for excavation depths greater than about 3 to 4 m. Lateral restraint could be provided by means of tie-backs consisting of grouted rock anchors. The use of rock anchor tie-backs would require the permission of the adjacent property owners since the anchors would be installed beneath their properties. The presence of utilities beneath the adjacent streets, which could interfere with the tie-backs, should also be considered, though this is typically manageable provided the first row of anchors is below the typical burial depth of municipal services. Alternatively, interior struts can be considered, connected either to the opposite side of the excavation (if not too distant) or to raker piles and/or footings within the excavation.

5.6.3 Ground Movements

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

Based on previous experience with nearby projects, the OLRT right-of-way may require additional analysis and review of the shoring design than is normally the case.

5.7 Groundwater Control

During the current investigation groundwater was encountered within the glacial till as high as 55.1 m. Lower groundwater levels were encountered in some boreholes, but these measurements were taken relatively quickly after drilling and may not represent fully stabilized groundwater levels.

It should also be noted that these represent the groundwater level on a single date (February 2022). These levels may not represent typical groundwater levels (because they were measured in winter) and certainly do not represent the maximum levels which could be encountered. As a comparison, Golder has experience with an adjacent site which encountered groundwater in the large excavation at 57 m elevation.

Based on this it is evident that the proposed development will extend below the groundwater level at the site and temporary and permanent groundwater control will be required.

5.7.1 Temporary Groundwater Control

Given the anticipated size and depth of the excavation, as well as the likely groundwater conditions at the site dewatering of the site will be required during construction to maintain a safe, dry working area and to prevent disturbance of the soil subgrade.

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

Calculation of anticipated groundwater flows have not been completed as part of this current phase, however, based on previous experience it is recommended that it be assumed a PTTW will be required. Once the final excavation footprint and depth are confirmed a hydrogeological study will be required to support the permit application.

The rate of groundwater inflow to the excavation will depend on many factors including the contractor’s schedule and rate of excavation, the size of the excavation, the material, incident precipitation, and the time of year at which the excavation is made (e.g., fluctuation in seasonal groundwater elevation). Moderate flows into the main excavation could potentially be managed using properly filtered sumps in closely space trenches or pits. Groundwater inflow for service trenches or smaller localized excavations for foundations, elevator pits, etc., should also be possible to control by pumping from within the excavations.

If higher flows are encountered, then a more active dewatering system (wells or well points) could also be considered to maintain the groundwater level below the base of the excavation. This requirement is particularly critical if shallow foundations (either footings or a raft/mat foundation) are considered as the uncontrolled seepage into the floor of the excavation (even if collected and pumped out in sumps) is likely to cause disturbance and piping of the subgrade resulting in a need to over-excavated and replace soils to maintain a suitable bearing surface.

The contractor should be fully responsible for design of the groundwater control system.

The glacial till soils that will form the floor of the foundation excavations are expected to be sensitive to disturbance. Consideration should therefore be given to protecting the subgrade in foundation areas with a mud slab of lean concrete or a layer of compacted granular fill materials (particularly if the areas will remain open for extended periods of time such as if a raft is used). The thickness of the mud slab and/or compacted granular fill working mat will depend on the size and weight of the equipment to be used at the bottom of the excavation. Any disturbed soil will need to be removed prior to placing the protective layer. That mud slab/granular fill materials should be placed immediately following inspection and approval of the subgrade. The period of time between exposure of the subgrade and covering with the protective layer should be limited to as brief as possible and, in the interim, no construction traffic should be permitted on the subgrade.

5.7.2 Permanent Groundwater Control

The measured groundwater depth at the site is variable, but it is above the lowest level of the proposed underground parking. To manage the long-term groundwater levels a drainage system diverting collected groundwater inflow to the sewer system is recommended. It is recommended that a hydrogeological assessment be completed to provide input toward the volumes of water anticipated to be diverted to the municipal sewer system (this can be done in conjunction with the study for the PTTW discussed above).

The subfloor drainage system (i.e., below the lowest garage level) should consist of a network of robust sub-drain pipes conveying collected groundwater to a sump or sumps from which the groundwater can be pumped to a municipal sewer. The drainage system would consist of interconnected perforated drain pipes (bedded and backfilled with free draining granular soils) installed around the perimeter and within the building footprint.
The capacity of the subfloor drainage system should be initially based on the hydrogeology assessment and then modified during construction if required.

Drainage, such as a composite synthetic drainage system or equivalent, should be provided to the exterior walls. The composite drain must withstand the design horizontal earth pressures used for basement wall design and should be connected to the basement level underslab drainage system. The drainage system collector pipes should drain to a sump for collection and discharge to a sewer.

5.8 Garage Floor Slab

In preparation for the construction of the lowest floor slab, all loose, wet, and disturbed material should be removed from beneath the floor slab down to the undisturbed native soil. Provision should be made for at least 250 mm of OPSS Granular A to form the base of the floor slab. Any bulk fill required to raise the grade up to the underside of the Granular A (as well as any areas where over-excavation and replacement are required) should consist of OPSS Granular B Type II. The under-slab fill should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% of the Standard Proctor Maximum Dry Density (SPMDD) using suitable vibratory compaction equipment.

Provision should be made for drainage underneath the floor slab consisting of perforated pipe subdrains in a surround of 19 mm clear stone, fully wrapped in geotextile, which leads by gravity drainage to an adjacent storm sewer or sump pit from which the water is pumped. For preliminary design purposes, these drains should be placed at approximately 6 m centres.

5.9 Foundation Wall Backfill

Foundation/basement walls should be backfilled with free draining non-frost susceptible granular fill meeting the requirements of OPSS Granular B Type I or II materials. Basement walls should be covered with drainage board such as Miridrain (or equivalent).

Backfill should be compacted to 95% of the material's SPMDD using suitable compaction equipment. To reduce compaction induced stresses, only light compaction rollers or plate tampers should be used within 1 m of the wall.

Beneath hard surfacing (e.g., pavements or sidewalks/walkways), the granular backfill for the foundation wall should be placed to form a frost taper at 3 horizontal to 1 vertical from a depth of 1.8 m (i.e., the frost depth) to the underside of the granular base for the hard surfacing. The purpose of this frost taper is to limit the severity of differential heaving that could occur between areas backfilled with non-frost susceptible engineered fill and the adjacent areas underlain by the existing frost susceptible soils.

5.10 Lateral Earth Pressures for Design

The lateral earth pressures acting on the basement walls and retaining walls will depend on the existing soil conditions, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

Where the wall support and structure allow lateral yielding, (e.g., for unrestrained retaining walls), active earth pressures may be used in the design of the wall. Where the support does not allow lateral yielding, (i.e., for typical basement walls) at-rest earth pressures should be assumed. The following parameters (unfactored) may be used for design where there is limited granular material between the basement and the native soil (for example where the site is shored):

Soil	Unit Weight (kN/m^3)	Coefficients of static lateral earth pressure	
		Active, Ka	At rest, Ko
Granular Backfill or Glacial Till	21	0.33	0.50
Glacial Till	22	0.31	0.47

If the garage/foundation wall is backfilled with granular free draining fill either in a zone with width equal to at least 50 percent of the height of the wall or within the wedge-shaped zone defined by a line drawn at 1 horizontal to 1 vertical (1H:1V) extending up and back from the rear face of the footing/pile cap/grade beam, the following parameters (unfactored) may be used:

Material	Unit Weight (kN/m^3)	Coefficients of static lateral earth pressure	
		Active, Ka	At rest, Ko
Granular A or Granular B Type II	22	0.27	0.43
Granular B Type I	22	0.31	0.47

For the purposes of shoring design, the designer (who is entirely responsible for the design including selection of design parameters) should carefully review the subsurface information and determine appropriate earth pressure parameters for use in their design. In particular, higher values than indicated in the tables above may need to be assumed in order to limit deflection of the shoring near existing structures.

Seismic loading will result in increased lateral earth pressures acting on the walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure.

The horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient is taken as 1.0 times the design PGA (i.e., k_h = 0.32). For structures which allow lateral yielding, k_h is taken as 0.5 times the design PGA (i.e., k_h = 0.16).

The following seismic active pressure coefficients (K_{AE}) may be used in design; these coefficients reflect the K_{AE} obtained using the k_h values described above and assumed no vertical acceleration and wall to soil friction. These seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

Wall Type	Site PGA (2475-year Earthquake)	K_{AE}	
		Granular A/Granular B Type II	Granular B Type I
Yielding Wall	0.32g	0.39	0.43
Non-Yielding Wall		0.53	0.59

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).

A minimum surcharge pressure of 12 kPa due to traffic and compaction induced pressure should be included in the total lateral earth pressures for the structural design of the wall.

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

$$\sigma_h(d) = K_o \gamma d + (K_{AE} - K_a) \gamma (H-d) + q$$

Where:

$\sigma_h(d)$	=	Lateral earth pressure at depth, d, (kPa)
K_o	=	Coefficient of static earth pressure
γ	=	Unit weight of the backfill soil (kN/m^3); as given previously
d	=	Depth below the top of the wall (m)
K_{AE}	=	Seismic active earth pressure coefficient
q	=	Surcharge to account for traffic and compaction pressure, where applicable
H	=	Total height of the wall (m)

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

5.11 Site Servicing

At least 150 mm of OPSS Granular A should be used as pipe bedding for sewer and water pipes. Where unavoidable disturbance to the subgrade surface occurs during construction, it may be necessary to place a sub-bedding layer consisting of 300 mm of compacted OPSS Granular B Type II beneath the Granular A. The bedding material should, in all cases, extend to the spring line of the pipe and should be compacted to at least 95% of the material's SPMDD. The use of clear crushed stone as a bedding layer should not be permitted anywhere on this project since fine particles from the sandy backfill materials and native soils could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

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

It should generally be possible to re-use the existing inorganic fill and glacial till as trench backfill provided it is properly moisture conditioned. Where trenches will be covered with hard surfaced areas, the type of material placed in the frost zone (between subgrade level and 1.8 mm 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 SPMDD using suitable vibratory compaction equipment.

Seepage barriers should be constructed at periodic intervals along the trench and at the connection points to off-site infrastructure to reduce the potential for groundwater level lowering in the surrounding area due to the "french drain" effect on the granular bedding and surround. Groundwater level lowering could lead to long-term settlement of nearby structures that are supported on the sensitive silty clay soil underlying the site.

It is important that these barriers extend from trench wall to trench wall and that they fully penetrate the granular surround materials to the trench bottom. The seepage barriers should be at least 1.5 metres long. In addition to providing a drainage cut-off, these cut-offs also serve as impenetrable cut-offs to stop the potential migration of contaminants along the relatively permeable backfill in the trenches.

Construction of the seepage barriers should also be in accordance with the City of Ottawa's Standard Drawing No. S8 of the Department of Public Works and Services, Infrastructure Services branch.

5.12 Pavement Design

In preparation for pavement construction, all topsoil, unsuitable fill, disturbed, or otherwise deleterious materials (i.e., those materials containing organic material) should be removed from the pavement areas. Some of the existing fill could remain provided that it is free of organic matter, and that the subgrade be subjected to a proof roll with a loaded tandem truck to reveal weak or soft areas prior to the construction of the new pavement structure. Soft or weak areas should be removed and repaired with acceptable earth borrow or OPSS Select Subgrade Material (SSM) or Granular B.

Pavement areas requiring grade raising to proposed subgrade level should be brought to grade using acceptable (compactable and inorganic) earth borrow, OPSS SSM or Granular B. These materials should be placed in maximum 300 mm thick lifts and should be compacted to at least 95% of the material's SPMDD using suitable compaction equipment.

The surface of the pavement subgrade should be crowned or sloped to promote drainage of the pavement granular structure towards perimeter swales or subdrains placed at the subgrade level

No traffic or paving details are available at the current stage. The following pavement designs are recommended for preliminary purposes based on experience with similar projects and developments. These designs should be confirmed during detailed design based on actual traffic requirements.

Material		Light Duty Pavement Thickness of Pavement Elements (mm)	Heavy Duty Pavement Thickness of Pavement Elements (mm)	Loading Dock Thickness of Pavement Elements (mm)
Bituminous Concrete OPSS 1150	Superpave 12.5 mm	60	40	-
	Superpave 19.0 mm	-	50	-
Portland Cement Concrete	Portland Cement Concrete	-	-	200
Granular Material OPSS 1010	Granular A Base	150	150	150
	Granular B, Type II Subbase	300	450	450
	Prepared and Approved Subgrade			

The granular base and subbase materials should be uniformly compacted as per OPSS 310, Method A. The asphaltic concrete should be compacted in accordance with the procedures outlined in OPSS 310.

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

The Portland cement concrete should meet the requirements of CSA A 23.1 Class C2 exposure. Concrete joint specifications and spacing should be in accordance with OPSD 552.020 and 551.010.

The above pavement designs are based on the assumption that the pavement subgrade has been acceptably prepared (i.e., grade raise fill has 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.

Where the new pavements will connect to existing pavements, the new pavement structures should be continued at least to the limits of construction, with any longitudinal transitions and/or tapers occurring thereafter. At these locations, the longitudinal transitions should be constructed by cutting the existing pavement structure vertically to the bottom of the existing subbase. The new granular layers should then be tapered up or down, as required, at a slope of 5 horizontal to 1 vertical to match the existing pavement structure. The asphaltic concrete does not need to be tapered between the new construction and the existing pavement. However, the asphaltic concrete of the existing pavement should be milled back an additional 300 mm to a depth of about 60 mm or 40 mm in areas where its thickness is greater than 100 mm, matching the proposed surface course of the new asphaltic concrete. A tack coat should be provided and the new surface course asphaltic concrete placed over the milled surface to form the new pavement joint. Where the existing pavement is less than 100 mm, then a butt joint on a vertical saw cut surface is acceptable. A tack coat should be placed on the vertical saw cut surface. The tack coat should be in accordance with the City SP F-3107.

5.13 Corrosion and Cement Type

To be included in final report.

6.0 ADDITIONAL CONSIDERATIONS

At the time of writing this report, only conceptual details related to the proposed building as well as adjacent significant structures such as the CSST and OLRT were available. Golder Associates should review the final drawings and specifications for this project prior to tendering to confirm that the guidelines in this report have been adequately interpreted.

The construction activities could impact the existing adjacent structures and buildings. Appropriate damage assessments (pre and post condition surveys for example) should be carried out as necessary.

During construction, sufficient foundation inspections, subgrade inspections, in-situ density tests, materials testing, pile and rock anchor installation monitoring should be carried out to confirm that the conditions exposed are consistent with those encountered in the boreholes, and to monitor conformance to the pertinent project specifications. Concrete testing should be carried out in a CCIL certified laboratory.

The soils at this site are sensitive to disturbance from ponded water, construction traffic and frost. All bearing surfaces must be inspected prior to filling or concreting to ensure that strata having adequate bearing capacity have been reached and that the bearing surfaces have been properly prepared.

7.0 CLOSURE

We trust that this report provides sufficient geotechnical engineering information to facilitate the design of this project. If you have any questions regarding the contents of this report or require additional information, please do not hesitate to contact this office.

Golder Associates Ltd.

Chris Hendry, P.Eng.
Sr. Principal Geotechnical Engineer

Sarah MacDonald, P.Eng.
Senior Geotechnical Engineer

CH/SM/hdw

https://golderassociates.sharepoint.com/sites/158117/project files/6 deliverables/geotechnical/22511882-rev0-dream lebreton library lands geotechnical report-1806_20.docx

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

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

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

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

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

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

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

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

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

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

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

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

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

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

LEGEND

- APPROXIMATE BOREHOLE/MONITORING WELL LOCATION (GOLDER 2022)
- APPROXIMATE BOREHOLE LOCATION (GOLDER 2011)
- ROADWAY
- FORMER ROADWAY (WELLINGSTON STREET)
- FORMER RAILWAY
- SITE BOUNDARY

NOTE(S)

1. ALL LOCATIONS ARE APPROXIMATE

REFERENCE(S)

1. LAND INFORMATION ONTARIO (LIO) DATA PRODUCED BY GOLDER ASSOCIATES LTD. UNDER LICENCE FROM ONTARIO MINISTRY OF NATURAL RESOURCES, © QUEENS PRINTER 2020
2. PROJECTION: TRANSVERSE MERCATOR, DATUM: NAD 83, COORDINATE SYSTEM: MTM ZONE 9, VERTICAL DATUM: CGVD28

DRAFT

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1:1,500 METRES

CLIENT
DREAM

PROJECT
GEOTECHNICAL SITE INVESTIGATION
LEBRETON LIBRARY PARCEL, OTTAWA, ONTARIO

TITLE
BOREHOLE AND MONITORING WELL LOCATION PLAN

CONSULTANT
WSP GOLDER

YYYY-MM-DD	2022-03-25
DESIGNED	----
PREPARED	JEM/MG
REVIEWED	----
APPROVED	----

PROJECT NO.	CONTROL	REV.	FIGURE
22511882	0003	A	1

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APPENDIX A

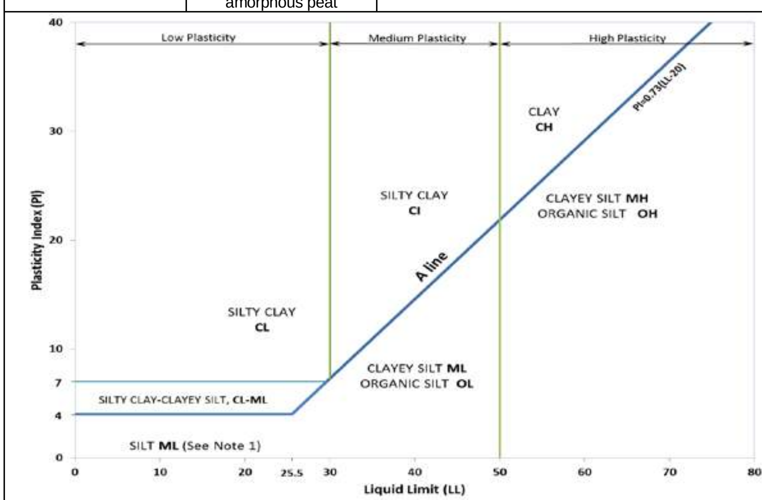
Borehole Logs – Current Investigation

METHOD OF SOIL CLASSIFICATION

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

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

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



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

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

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

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

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

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

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

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

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

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

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

PENETRATION RESISTANCE

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

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

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

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

SAMPLES

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

SOIL TESTS

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

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

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

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

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

Field Moisture Condition

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

COHESIVE SOILS

Consistency

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

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

Water Content

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

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes: 1 $\tau = c' + \sigma' \tan\phi'$
2 shear strength = (compressive strength)/2

PROJECT: 22511882

RECORD OF BOREHOLE: 22-01

LOCATION: N 5030733.9 ;E 366525.1

BORING DATE: February 14-15, 2022

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ *ND = Not Detected* (20, 40, 60, 80)

HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □ *ND = Not Detected* (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)

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.30m	HEADSPACE
	GROUND SURFACE	62.92				
0	FILL - SILTY SAND, trace gravel; brown; compact to dense	0.00	1	AS	-	ND
1			2	SS	20	
2			3	SS	33	ND
2.6			4	SS	58	ND
3			5	SS	43	
4			6	SS	72	
4.88	SILTY SAND to sandy SILT, trace clay and gravel; grey, contains cobbles and boulders (GLACIAL TILL); very dense	58.04 / 4.88	7	SS	50/0.10	
5			8	SS	50/0.13	
6			9	SS	77/0.15	
7			10	SS	50/0.05	
7.7			11	SS	64/0.15	
8			12	SS	66/0.13	
9			13	SS	50/0.05	
10			14	SS		

PIEZOMETER OR STANDPIPE INSTALLATION: Bentonite Seal; Silica Sand; Screen

ADDITIONAL LAB. TESTING

CONTINUED NEXT PAGE

DEPTH SCALE 1 : 50

WSP GOLDER

LOGGED: ALB

CHECKED:

PROJECT: 22511882

RECORD OF BOREHOLE: 22-01

LOCATION: N 5030733.9 ;E 366525.1

BORING DATE: February 14-15, 2022

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/0.30m); HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ *ND = Not Detected* (20, 40, 60, 80); HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □ *ND = Not Detected* (20, 40, 60, 80); HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT Wp ⊢—O^W—⊣ Wl (20, 40, 60, 80); ADDITIONAL LAB. TESTING; PIEZOMETER OR STANDPIPE INSTALLATION

--- CONTINUED FROM PREVIOUS PAGE ---

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.30m	PIEZOMETER OR STANDPIPE INSTALLATION
10	Power Auger 200 mm Diam. (Hollow Stem)	SILTY SAND to sandy SILT, trace clay and gravel; grey, contains cobbles and boulders (GLACIAL TILL); very dense		14	SS	50/ 0.08	Screen
11				15	SS	50/ 0.10	
				16	SS	50/ 0.10	
12				17	SS	50/ 0.13	
13				18	SS	50/ 0.08	
14				19	SS	50/ 0.05	
			48.21 14.71				
15		Borehole continued on RECORD OF DRILLHOLE 22-01					
16							
17							
18							
19							
20							

DEPTH SCALE

1 : 50

WSP GOLDER

LOGGED: ALB

CHECKED:

LOCATION: N 5030733.9 ;E 366525.1

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 14-15, 2022

DRILL RIG: CME 55

DRILLING CONTRACTOR: Downing Drilling

DATUM: Geodetic

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.
		BEDROCK SURFACE		48.21	
15–16	Rotary Drill, NQ Core	Weathered, thin to medium bedded, grey black LIMESTONE and SHALE		14.71	1
				46.46	
17		End of Drillhole Note(s): 1. Water level in screen measured at a depth of 7.78 m (Elev. 55.14 m) on February 25, 2022		16.46	

Other column headings: COLOUR % RETURN; FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 0.25 m; DISCONTINUITY DATA (DIP w.r.t. CORE AXIS, TYPE AND SURFACE DESCRIPTION, Jcon, Jr, Ja); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); Diametral Point Load Index (MPa); RMC -Q' AVG.

Depth scale: 15, 16, 17, 18, 19, 20, 21, 22, 23, 24

PROJECT: 22511882

RECORD OF BOREHOLE: 22-02

LOCATION: N 5030713.1 ;E 366476.0

BORING DATE: February 16, 2022

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	HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ (ND = Not Detected) 20 40 60 80 / HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □ (ND = Not Detected) 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} / WATER CONTENT PERCENT Wp —O^W— Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		62.47							
0	Power Auger, 200 mm Diam. (Hollow Stem)	FILL - SILTY SAND, trace gravel; brown		0.00	1	AS	-	⊕ ND			Bentonite Seal
1					2	SS	28				
2					3	SS	16				
					4	SS	20	⊕ ND			
3		SILTY SAND to sandy SILT, trace gravel, trace clay; grey, contains cobbles and boulders (GLACIAL TILL); very dense		59.42 3.05	5	SS	50/0.15				
4					6	SS	50/0.05				
5					7	SS	50/0.08				
					8	SS	50/0.13				
6					9	SS	50/0.10				
7					10	SS	50/0.13				Silica Sand
8		- Auger Refusal on boulder at 7.44 m depth									Screen
9					11	SS	50/0.10				
10											

CONTINUED NEXT PAGE

DRAFT

DEPTH SCALE

1 : 50

WSP GOLDER

LOGGED: ALB

CHECKED:

RECORD OF BOREHOLE: 22-02

LOCATION: N 5030713.1 ;E 366476.0

BORING DATE: February 16, 2022

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	SAMPLES: TYPE	SAMPLES: BLOWS/0.30m	HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ ND = Not Detected (20 40 60 80); HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □ ND = Not Detected (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
		--- CONTINUED FROM PREVIOUS PAGE ---									
10	Power Auger; 200 mm Diam. (Hollow Stem)	SILTY SAND to sandy SILT, trace gravel, trace clay; grey, contains cobbles and boulders (GLACIAL TILL); very dense									Screen
					12	SS	50/0.03				
11											
12					13	SS	50/0.03				
13											
14				48.27 14.2							
		Borehole continued on RECORD OF DRILLHOLE 22-02									
15											
16											
17											
18											
19											
20											

DEPTH SCALE
1 : 50

WSP GOLDER

LOGGED: ALB
CHECKED:

RECORD OF DRILLHOLE: 22-02

PROJECT: 22511882
LOCATION: N 5030713.1 ;E 366476.0
INCLINATION: -90° AZIMUTH: ---
DRILLING DATE: February 16, 2022
DRILL RIG: CME 55
DRILLING CONTRACTOR: Downing Drilling
DATUM: Geodetic

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	RECOVERY TOTAL CORE % / SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 m	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.
		BEDROCK SURFACE		48.27													
15	Rotary Drill, NQ Core	Grey, thin to medium bedded LIMESTONE and SHALE		48.27 / 14.20	1	FLUSH											
				46.62													
16		End of Drillhole		15.85													

Note(s):

1. Water level in screen measured at a depth of 7.88 m (Elev. 54.59 m) on February 25, 2022

Depth scale: 17, 18, 19, 20, 21, 22, 23, 24

DRAFT

MIS-RCK 004 22511882.GPJ GAL-MISS.GDT 4/4/22 ZS

DEPTH SCALE
1 : 50

WSP GOLDER

LOGGED: ALB
CHECKED:

PROJECT: 22511882

RECORD OF BOREHOLE: 22-03

LOCATION: N 5030756.8 ;E 366500.4

BORING DATE: February 22, 2022

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.30m	HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ ND = Not Detected / HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □ ND = Not Detected	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT PERCENT Wp W Wl	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		61.65							
0	Power Auger 200 mm Diam. (Hollow Stem)	FILL - SILTY SAND, trace gravel and brick; brown		0.00	1	AS	-	ND			
1					2	SS	8				
2					3	SS	11	ND			
		SILTY SAND to sandy SILT, trace gravel; grey, contains cobbles and boulders (GLACIAL TILL); very dense		59.36 2.29	4	SS	43	ND			
3					5	SS	58				
4					6	SS	50/ 0.10	ND			
					7	SS	50/ 0.10				
5					8	SS	50/ 0.08				Bentonite Seal
6					9	SS	50/ 0.08				
7											
8					10	SS	50/ 0.05				
9					11	SS	50/ 0.08	ND			
10											

Graph scales: headspace 20, 40, 60, 80; hydraulic conductivity 10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}; water content 20, 40, 60, 80.

CONTINUED NEXT PAGE

DEPTH SCALE 1 : 50

WSP GOLDER

LOGGED: ALB

CHECKED:

RECORD OF BOREHOLE: 22-03

PROJECT: 22511882

LOCATION: N 5030756.8 ;E 366500.4

BORING DATE: February 22, 2022

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	SAMPLES: TYPE	SAMPLES: BLOWS/0.30m	HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ ND = Not Detected (20, 40, 60, 80) / HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □ ND = Not Detected (20, 40, 60, 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}) / WATER CONTENT PERCENT Wp —O W— Wl (20, 40, 60, 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---									
10	Power Auger 200 mm Diam. (Hollow Stem)	SILTY SAND to sandy SILT, trace gravel; grey, contains cobbles and boulders (GLACIAL TILL); very dense									Bentonite Seal
					12	SS	61/ 0.15				
11				50.48 11.17							
		Borehole continued on RECORD OF DRILLHOLE 22-03									
12											
13											
14											
15											
16											
17											
18											
19											
20											

MIS-BHS 001 22511882.GPJ GAL-MIS.GDT 4/4/22 ZS

DEPTH SCALE

1 : 50

LOGGED: ALB

CHECKED:

RECORD OF DRILLHOLE: 22-03

PROJECT: 22511882

LOCATION: N 5030756.8 ;E 366500.4

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 22, 2022

DRILL RIG: CME 55

DRILLING CONTRACTOR: Downing 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.	INSTALLATION
		BEDROCK SURFACE	50.48		
	Rotary Drill, NQ Core	Slightly weathered to fresh, thin to medium bedded, grey black LIMESTONE and SHALE	11.17	1	Bentonite Seal; Silica Sand
12			49.38		
		Fresh, thin to medium bedded, grey to black LIMESTONE and SHALE	12.27	2	
13					Screen (water level ▽)
14				3	
			47.10		
		End of Drillhole Note(s): 1. Water level in screen measured at a depth of 13.00 m (Elev. 48.65 m) on February 25, 2022	14.55		
15					
16					
17					
18					
19					
20					
21					

Other column headings: SYMBOLIC LOG; COLOUR % RETURN; FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 0.25 m; DISCONTINUITY DATA (DIP w.r.t. CORE AXIS; TYPE AND SURFACE DESCRIPTION; Jcon; Jr; Ja); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); Diametral Point Load Index (MPa); RMC -Q' AVG.

DRAFT

DEPTH SCALE 1 : 50

WSP GOLDER

LOGGED: ALB

CHECKED:

PROJECT: 22511882

RECORD OF BOREHOLE: 22-04

LOCATION: N 5030713.2 ;E 366411.4

BORING DATE: February 23, 2022

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.30m	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	60.47				
0	Power Auger, 200 mm Diam. (Hollow Stem)	FILL - SILTY SAND, trace gravel; brown	0.00	1	AS	-	
1				2	SS	10	
2				3	SS	6	
		SILTY SAND to sandy SILT, trace gravel, trace clay; (possibly till); loose to compact	58.34 / 2.13	4	SS	8	
3				5	SS	8	Bentonite Seal
4				6	SS	16	
5				7	SS	7	
				8	SS	14	
6		SILTY SAND to sandy SILT, trace gravel, trace clay; grey, contains cobbles and boulders (GLACIAL TILL); dense to very dense	54.37 / 6.10	9	SS	27	
7							Silica Sand
8				10	SS	78	
9				11	SS	47	Screen
10							

Column headings also include: 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; STRATA PLOT.

DRAFT

CONTINUED NEXT PAGE

DEPTH SCALE 1 : 50

WSP GOLDER

LOGGED: ALB

CHECKED:

PROJECT: 22511882

RECORD OF BOREHOLE: 22-04

SHEET 2 OF 3

LOCATION: N 5030713.2 ;E 366411.4

BORING DATE: February 23, 2022

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
		--- CONTINUED FROM PREVIOUS PAGE ---									
10	Power Auger 200 mm Diam. (Hollow Stem)	SILTY SAND to sandy SILT, trace gravel, trace clay; grey, contains cobbles and boulders (GLACIAL TILL); dense to very dense			12	SS	50/ 0.05				Screen
11				49.29 11.18							Silica Sand
		Borehole continued on RECORD OF DRILLHOLE 22-04									
12											
13											
14											
15											
16											
17											
18											
19											
20											

DRAFT

DEPTH SCALE

1 : 50

WSP GOLDER

LOGGED: ALB

CHECKED:

PROJECT: 22511882

RECORD OF DRILLHOLE: 22-04

LOCATION: N 5030713.2 ;E 366411.4

DRILLING DATE: February 23, 2022

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Downing 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	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.
		BEDROCK SURFACE		49.29														
12	Rotary Drill NQ Core	Fresh, thin to medium bedded, grey black LIMESTONE and SHALE		11.18 / 48.28	1													
		End of Drillhole		12.19														

Note(s):

1. Water level in screen measured at a depth of 10.70 m (Elev. 49.77 m) on February 25, 2022

Depth scale: 12, 13, 14, 15, 16, 17, 18, 19, 20, 21

DEPTH SCALE
1 : 50

WSP GOLDER

LOGGED: ALB
CHECKED:

RECORD OF BOREHOLE: 22-05

PROJECT: 22511882
LOCATION: N 5030679.9 ;E 366442.7
SAMPLER HAMMER, 64kg; DROP, 760mm

BORING DATE: February 24, 2022

DATUM: Geodetic
PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.30m	HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] (ND = Not Detected)	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	62.34					
0	Power Auger, 200 mm Diam. (Hollow Stem)	FILL - SILTY SAND, trace gravel; brown	0.00					
				1	AS	-	ND	
1				2	SS	33		
2				3	SS	31		
				4	SS	84		
3		SILTY SAND to sandy SILT, trace gravel, trace clay; grey, contains cobbles and boulders (GLACIAL TILL); very dense	59.29 / 3.05					Bentonite Seal
				5	SS	94		
4				6	SS	50	ND	
				7	SS	50/0.05		
5				8	SS	50/0.03		
6				9	SS	50/0.05		Silica Sand
7								Screen
8				10	SS	50/0.10	ND	∇
9				11	SS	50/0.13		
10								

Additional column headings: HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] (ND = Not Detected); 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; STRATA PLOT.

CONTINUED NEXT PAGE

DRAFT

DEPTH SCALE
1 : 50

WSP GOLDER

LOGGED: ALB
CHECKED:

PROJECT: 22511882

RECORD OF BOREHOLE: 22-05

LOCATION: N 5030679.9 ;E 366442.7

BORING DATE: February 24, 2022

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.30m	HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ ND = Not Detected (20 40 60 80) / HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □ ND = Not Detected (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
		--- CONTINUED FROM PREVIOUS PAGE ---									
10	Power Auger 200 mm Diam. (Hollow Stem)	SILTY SAND to sandy SILT, trace gravel, trace clay; grey, contains cobbles and boulders (GLACIAL TILL); very dense									
					12	SS	50/ 0.13				
11											
12					13	SS	86				
13											
				48.50	14	SS	50/ 0.13				
14		Borehole continued on RECORD OF DRILLHOLE 22-05		13.84							
15											
16											
17											
18											
19											
20											

DRAFT

DEPTH SCALE

1 : 50

WSP GOLDER

LOGGED: ALB

CHECKED:

MIS-BHS 001 22511882.GPJ GAL-MIS.GDT 4/4/22 ZS

RECORD OF DRILLHOLE: 22-05

PROJECT: 22511882

LOCATION: N 5030679.9 ;E 366442.7

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: February 24, 2022

DRILL RIG: CME 55

DRILLING CONTRACTOR: Downing Drilling

DATUM: Geodetic

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 m	DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.
		BEDROCK SURFACE		48.50														
14–15	Rotary Drill NQ Core	Slightly weathered to fresh, thin to medium bedded, grey black LIMESTONE and SHALE		13.84	1													
				47.10 15.24														
		End of Drillhole																

Note(s):

1. Water level in screen measured at a depth of 8.22 m (Elev. 54.12 m) on February 25, 2022

DEPTH SCALE 16, 17, 18, 19, 20, 21, 22, 23

DEPTH SCALE 1 : 50

WSP GOLDER

LOGGED: ALB

CHECKED:

RECORD OF BOREHOLE: 11-33

PROJECT: 11-1122-0199

LOCATION: See Site Plan

BORING DATE: December 8, 2011

SHEET 1 OF 1

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

Boring method: Power Auger, 200 mm Diam. (Hollow Stem)

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.30m
0	GROUND SURFACE	62.22			
	Dense dark grey crushed stone (Gravel lot BASE)	0.08	1	50 DO	46
	Dense brown fine to medium sand, some coarse sand, some gravel, trace silt (Gravel lot SUBBASE)	61.69 / 0.53			
1	Loose to very dense dark brown silty sand, trace to some gravel, brick, wood, organics, concrete, occasional grey silty clay layer (FILL)		2	50 DO	9
			3	50 DO	60
2			4	50 DO	12
			5	50 DO	56
3	Compact to very dense brown to grey brown SILTY SAND to SANDY SILT, trace to some gravel (GLACIAL TILL)	59.32 / 2.90	6	50 DO	23
			7	50 DO	48
4			8	50 DO	74
5			9	50 DO	49
			10	50 DO	55
6			11	50 DO	>89
			12	50 DO	>100
7			13	50 DO	>100
			14	50 DO	>100
	Very dense grey brown SILTY SAND, trace to some gravel, occasional grey silt seam, occasional fine to medium sand seam (GLACIAL TILL)	54.60 / 7.62	15	50 DO	>111
8			16	50 DO	>105
			17	50 DO	>50
9			18	50 DO	>100
			19	50 DO	>50
			20	50 DO	>110
10	End of Borehole Split Spoon Refusal	52.26 / 9.96			
11					

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

DEPTH SCALE 1 : 55

Golder Associates

LOGGED: RI

CHECKED: GDC

RECORD OF BOREHOLE: 11-35

PROJECT: 11-1122-0199

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: December 12, 2011

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		62.56							
0	Power Auger, 200 mm Diam. (Hollow Stem)	Dense grey sand and gravel (Gravel lot BASE)		0.00							
		Compact brown medium to fine sand, trace gravel (Gravel lot SUBBASE)		62.25 0.31	1	50 DO	52				
1		Compact dark brown to black silty sand, trace gravel, ash, wood, brick, mortar (FILL)		61.65 0.91	2	50 DO	17				
					3	50 DO	19				
2		Compact brown fine to medium sand, trace gravel (FILL)		60.88 1.68	4	50 DO	24				
		Dense to very dense light brown to brown SILTY SAND, occasional gravel and medium sand layers, trace gravel (GLACIAL TILL)		60.43 2.13	5	50 DO	45				
					6	50 DO	65				
3					7	50 DO	176				
4					8	50 DO	>50				
		End of Borehole Auger Refusal		58.16 4.40							
5											
6											
7											
8											
9											
10											

MIS-BHS 001 1111220199.GPJ GAL-MIS.GDT 02/24/15 JEM

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: BM

CHECKED: GDC

RECORD OF BOREHOLE: 11-37

PROJECT: 11-1122-0199

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: December 12, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

DEPTH SCALE METRES	SOIL PROFILE – DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.30m
	GROUND SURFACE	62.76			
0	Compact sand and gravel (Gravel lot BASE)	0.00			
	Compact brown medium to fine sand, trace gravel (Gravel lot SUBBASE)	62.46 0.30	1	50 DO	29
1	Loose dark brown to black silty sand, trace gravel, occasional layers of ash, gravel, sandy mortar, glass, construction debris (FILL)	61.85 0.91	2	50 DO	20
			3	50 DO	6
2	Compact brown medium to fine sand, trace gravel (FILL)	60.63 2.13	4	50 DO	34
3	Dense to very dense grey brown SILTY SAND, some gravel, trace cobbles (GLACIAL TILL)	60.32 2.44	5	50 DO	73
4			6	50 DO	>75
			7	50 DO	>65
5			8	50 DO	>75
			9	50 DO	40
6			10	50 DO	>50
	End of Borehole Auger Refusal	56.23 6.53			
7					
8					
9					
10					

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

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

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

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

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: BM

CHECKED: GDC

PROJECT: 11-1122-0199

RECORD OF BOREHOLE: 11-38

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: December 19, 2011

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		62.11							
0	Power Auger, 200 mm Diam. (Hollow Stem)	Compact to dense brown sand and gravel (Gravel lot BASE)		0.00							
		Loose to compact brown medium to fine sand, some gravel (Gravel lot SUBBASE)		0.10	1	50 DO	35				
1					2	50 DO	8				
		Compact to very dense grey brown sand, some gravel, trace silt (FILL)		60.89 1.22	3	50 DO	15				
2					4	50 DO	52				
		Very dense grey brown SILTY SAND, some gravel, medium brown sand seams (GLACIAL TILL)		59.67 2.44	5	50 DO	61				
3					6	50 DO	112				
4					7	50 DO	148				
		End of Borehole Auger Refusal		57.94 4.17							
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: JDR

CHECKED: GDC

MIS-BHS 001 1111220199.GPJ GAL-MIS.GDT 02/24/15 JEM

RECORD OF BOREHOLE: 11-39

PROJECT: 11-1122-0199

LOCATION: See Site Plan

BORING DATE: December 15, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

Boring method: Power Auger, 200 mm Diam. (Hollow Stem)

Description	Elev. (m) / Depth (m)	Sample Number	Type	Blows/0.30m
GROUND SURFACE	62.81			
Compact sand and gravel (Gravel lot BASE)	0.00			
Compact brown to red sandy silt, trace gravel (FILL)	0.15	1	50 DO	15
Compact to dense light brown fine to medium sand, trace gravel, silt, and mortar (FILL)	61.90 / 0.91	2	50 DO	20
		3	50 DO	40
Dense sandy gravel to brown fine to medium sand and gravel (FILL)	60.68 / 2.13	4	50 DO	120
		5	50 DO	67
		6	50 DO	99
Compact to very dense grey SILTY SAND, some gravel (GLACIAL TILL)	59.15 / 3.66	7	50 DO	34
		8	50 DO	27
		9	50 DO	33
		10	50 DO	>50
		11	50 DO	>100
		12	50 DO	>100
End of Borehole Auger Refusal	56.46 / 6.35			

Column headings: 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

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

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: BM/JD

CHECKED: GDC

RECORD OF BOREHOLE: 11-40

PROJECT: 11-1122-0199

LOCATION: See Site Plan

BORING DATE: December 16, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

Soil Profile Description	Elev. / Depth (m)	Sample Number	Type	Blows/0.30m
GROUND SURFACE	62.77			
Compact red to fine brown sand, some gravel (Gravel lot BASE)	0.00	1	50 DO	13
Compact fine to medium brown sand, some gravel, red brick (FILL)	62.39 / 0.38	2	50 DO	19
Compact light brown fine to medium sand, trace gravel, silt, red brick (FILL)	61.55 / 1.22	3	50 DO	15
		4	50 DO	25
		5	50 DO	51
Very dense grey brown SAND, some gravel, trace silt (GLACIAL TILL)	59.78 / 2.99	6	50 DO	59
Very dense grey brown SILTY SAND, some gravel (GLACIAL TILL)	59.11 / 3.66	7	50 DO	100
		8	50 DO	>50
		9	50 DO	>100
		10	50 DO	187
		11	50 DO	>50
End of Borehole Auger Refusal	56.52 / 6.25			

Columns also present (no data): DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa (nat V. + Q - ●, rem V. ⊕ U - ○); 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.

DEPTH SCALE 1 : 50

Golder Associates

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CHECKED: GDC

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APPENDIX D: Architectural Plans

Perkins&Will KPMB

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SHEET LIST

SHEET NUMBER	SHEET NAME	FOR OPA/ZBA/SPA | 2022-04-22
A00-00	COVER AND SHEET LIST	•
A01-00	SURVEY	•
A01-01	STATISTICS AND CONTEXT PLAN	•
A01-03	SITE PLAN	•
A10-0P1	LEVEL P1 FLOOR PLAN	•
A10-0P2	LEVEL P2 FLOOR PLAN	•
A10-01	LEVEL 01 FLOOR PLAN	•
A10-02	LEVEL 02 FLOOR PLAN	•
A10-03	LEVEL 03 FLOOR PLAN	•
A10-04	LEVEL 04 FLOOR PLAN	•
A10-05	LEVEL 05 FLOOR PLAN	•
A10-06	LEVEL 06 FLOOR PLAN	•
A10-07	LEVEL 07-17 FLOOR PLAN	•
A10-08	LEVEL 18-31 FLOOR PLAN	•
A10-09	LEVEL 32-36 FLOOR PLAN	•
A10-10	ROOF PLAN	•
A20-01	OVERALL NORTH ELEVATION	•
A20-02	OVERALL SOUTH ELEVATION	•
A20-03	EAST BUILDING ELEVATIONS	•
A20-04	WEST BUILDING ELEVATIONS	•
A21-01	OVERALL BUILDING SECTION	•
A21-02	PARTIAL BUILDING SECTIONS	•

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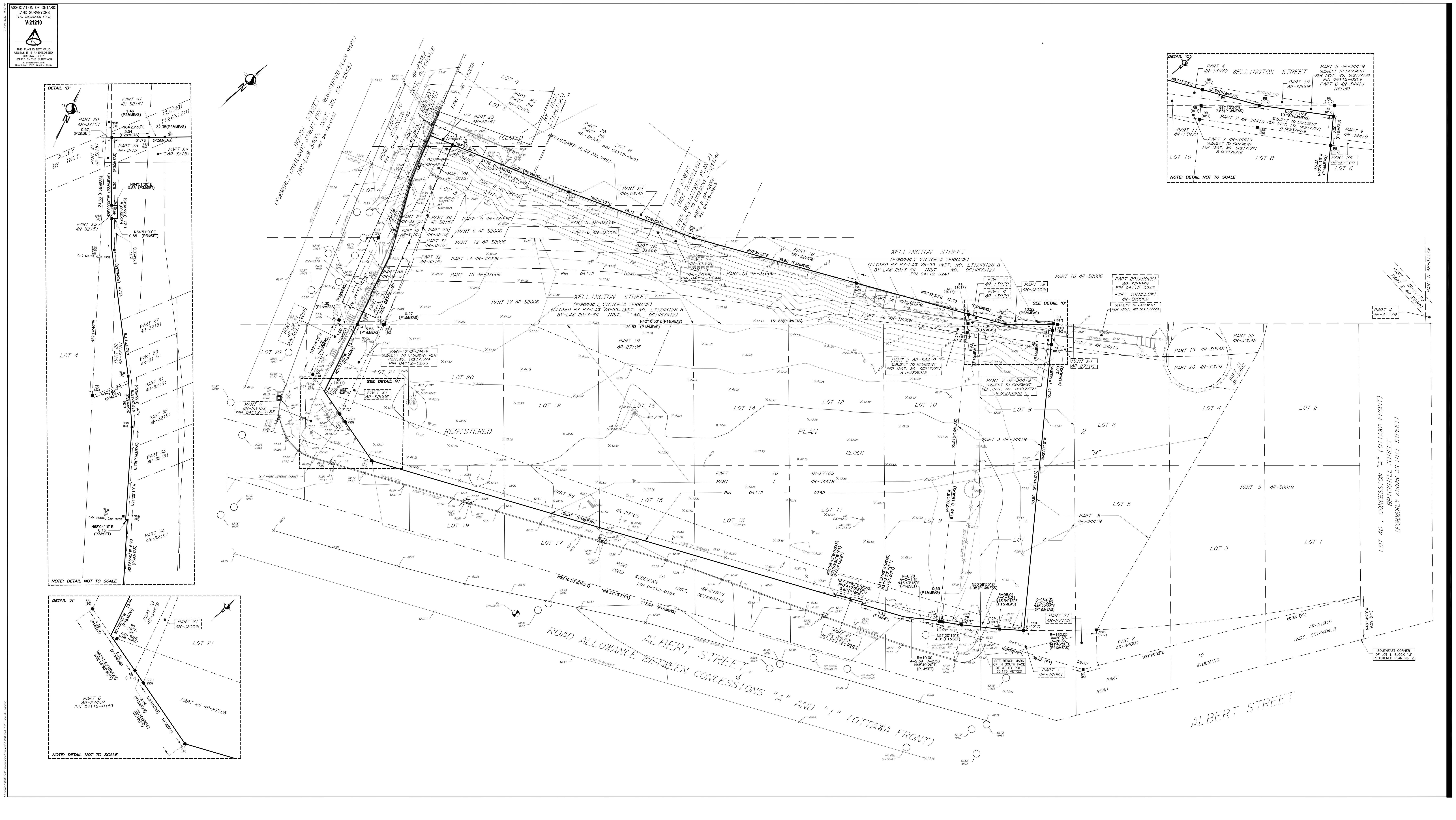

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2680 Queensview Dr., Ottawa, ON K2B 8J9

Stantec

Stantec Geomatics Ltd.
300-1331 Clyde Avenue
Ottawa ON
Tel. 613.722.4420
www.stantec.com

TOPOGRAPHIC SURVEY OF
LOTS 10, 12, 14, 16, 18 & 20 AND PART OF LOTS 9, 11, 13, 15, 17, 19 AND 21
BLOCK 'M'
PART OF WELLINGTON STREET (FORMERLY VICTORIA TERRACE) (CLOSED BY BY-LAW 73-99, INST. NO. LT1243128 & BY-LAW 2013-64, INST. NO. OC1457912)
REGISTERED PLAN 2
PART OF LOTS 1, 2, 3, 4 & 8
PART OF THE ALLEY (CLOSED BY INST. NO. LT1243120) AND PART OF LLOYD STREET (NOT TRAVELLED)
REGISTERED PLAN 9481
CITY OF OTTAWA
Stantec Geomatics Ltd.
ONTARIO LAND SURVEYORS
Scale 1:250
5 0 5 10 15 METRES

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

BEARING NOTE
BEARINGS ARE GRID, DERIVED FROM THE CAN-NET VRS NETWORK OBSERVATIONS ON NCC HORIZONTAL CONTROL MONUMENTS 19773035 AND 19680191, CENTRAL MERIDIAN, 76°30' WEST LONGITUDE MTM ZONE 9. NAD83 (ORIGINAL).

19773035 N:5006060.42 E:324888.04
19680191 N:5033564.26 E:388064.94

ELEVATION NOTE
ELEVATIONS SHOWN HEREON ARE GEODETIC (CGVD-1928:1978) AND ARE DERIVED FROM THE CAN-NET VRS NETWORK MONUMENT: OTTAWA ELEVATION=95.230.

LEGEND

■	DENOTES	FOUND MONUMENTS
□	"	SET MONUMENTS
IB	"	IRON BAR
IBø	"	ROUND IRON BAR
SIB	"	STANDARD IRON BAR
SSIB	"	SHORT STANDARD IRON BAR
CC	"	CUT CROSS
RB	"	ROCK BAR
CP	"	CONCRETE PIN
WIT	"	WITNESS
PIN	"	PROPERTY IDENTIFICATION NUMBER
M/MEAS	"	MEASURED
PROP	"	PROPORTIONED
OU	"	ORIGIN UNKNOWN
STANTEC	"	STANTEC GEOMATICS LTD.
P1	"	PLAN 4R-34419
P2	"	PLAN 4R-32006
P3	"	PLAN 4R-32151
1017	"	CALLON DIETZ INC.
AN	"	ANCHOR
BKR	"	BIKE RACK
BENCH	"	BENCH
BOL	"	BOLLARD
CB	"	CATCH BASIN
CBSI	"	SIDE INLET CB
HYD	"	FIRE HYDRANT
LS	"	LIGHT STANDARD
MH	"	MAINTENANCE HOLE UNIDENTIFIED
MHSA	"	MAINTENANCE HOLE SANITARY
MHST	"	MAINTENANCE HOLE STORM
MW	"	MONITORING WELL
SN	"	SIGN
	"	TREE CONIFEROUS (D.B.H. SHOWN)
	"	TREE DECIDUOUS (D.B.H. SHOWN)

SURVEYOR'S CERTIFICATE
I CERTIFY THAT :
1. THIS SURVEY AND PLAN ARE CORRECT AND IN ACCORDANCE WITH THE SURVEYS ACT, THE SURVEYORS ACT AND THE LAND TITLES ACT AND THE REGULATIONS MADE UNDER THEM.
2. THE SURVEY WAS COMPLETED ON THE 24th DAY OF MARCH, 2022.

April 7, 2022
DATE

R. G. BENNETT
ONTARIO LAND SURVEYOR

DRAWN: ZF CHECKED: CK PM: CT FIELD: ES PROJECT No.: 161614531-111

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ONTARIO ASSOCIATION OF ARCHITECTS
MATTHEW JOHNSTON
LICENCE
6101

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Toronto, ON M5C 3H1

KEYPLAN

TRUE NORTH

ISSUE CHART

1	ISSUED FOR OPA / ZBA / SPA	2022-04-22
MARK	ISSUE	DATE

Job Number

TITLE

SURVEY

SHEET NUMBER

A01-00

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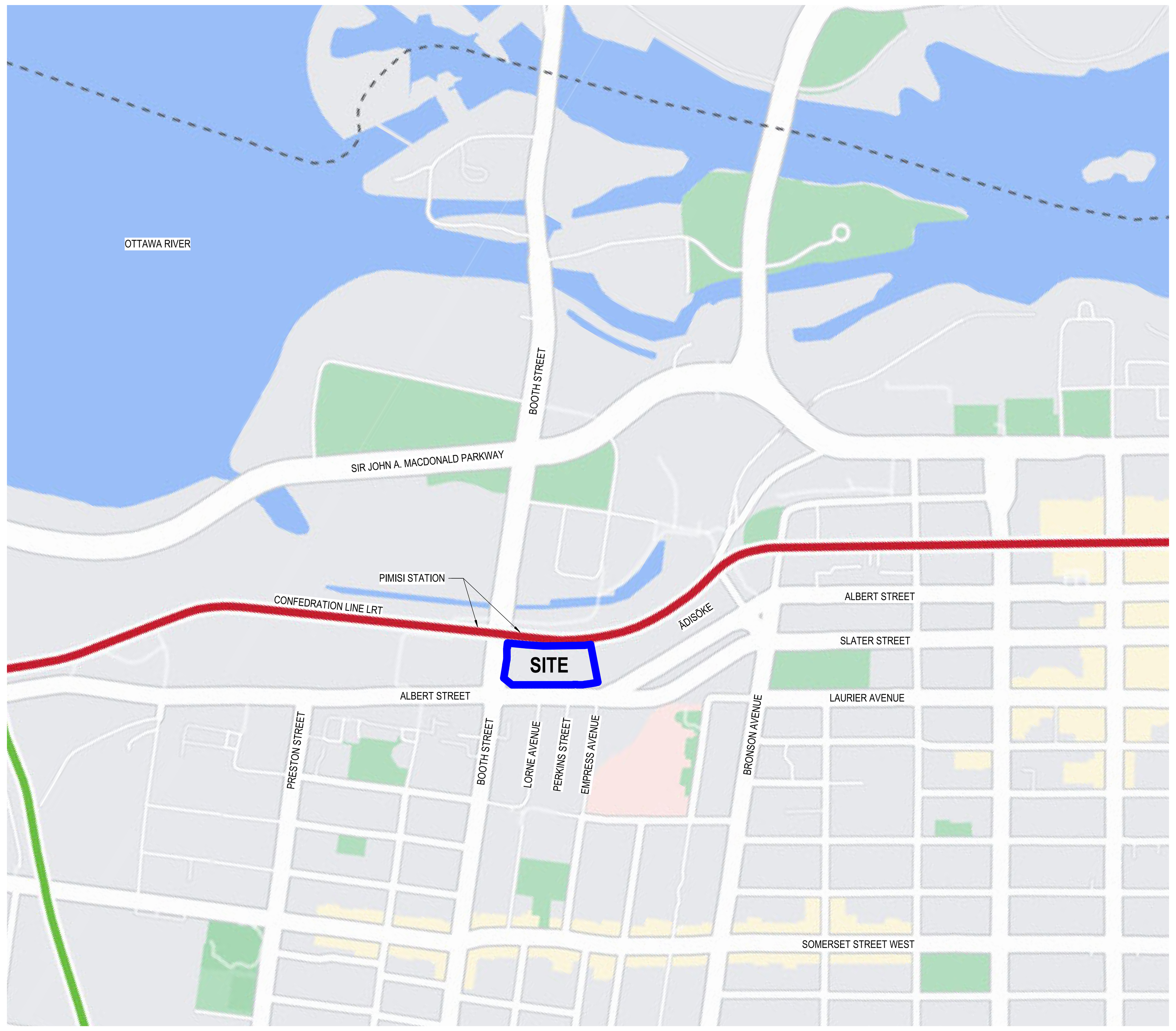

1 CONTEXT PLAN
1 : 5000

OVERALL BUILDING STATISTICS

Level	Gross Construction Area (see Note 1)	GFA Deductions (see Note 2)	Gross Floor Area (see Note 3)
LEVEL P2	5870.92 m²	5870.92 m²	0.00 m²
LEVEL P1	5293.13 m²	5262.65 m²	30.48 m²
LEVEL 01	3797.22 m²	2742.02 m²	1055.20 m²
LEVEL 02	1211.81 m²	1211.81 m²	0.00 m²
LEVEL 03	2830.02 m²	684.68 m²	2145.34 m²
LEVEL 04	2394.55 m²	805.39 m²	1589.16 m²
LEVEL 05	1999.45 m²	826.44 m²	1173.02 m²
LEVEL 06	1101.18 m²	1101.18 m²	0.00 m²
LEVEL 07	1401.29 m²	264.94 m²	1136.35 m²
LEVEL 08	1401.29 m²	264.94 m²	1136.35 m²
LEVEL 09	1401.29 m²	264.94 m²	1136.35 m²
LEVEL 10	1401.29 m²	264.94 m²	1136.35 m²
LEVEL 11	1401.29 m²	264.94 m²	1136.35 m²
LEVEL 12	1401.29 m²	264.94 m²	1136.35 m²
LEVEL 13	1401.29 m²	264.94 m²	1136.35 m²
LEVEL 14	1401.29 m²	264.94 m²	1136.35 m²
LEVEL 15	1401.29 m²	264.94 m²	1136.35 m²
LEVEL 16	1401.29 m²	264.94 m²	1136.35 m²
LEVEL 17	1401.29 m²	264.94 m²	1136.35 m²
LEVEL 18	1401.28 m²	268.91 m²	1132.37 m²
LEVEL 19	1401.28 m²	268.91 m²	1132.37 m²
LEVEL 20	1401.28 m²	268.91 m²	1132.37 m²
LEVEL 21	1401.28 m²	268.91 m²	1132.37 m²
LEVEL 22	1401.28 m²	268.91 m²	1132.37 m²
LEVEL 23	1401.28 m²	268.91 m²	1132.37 m²
LEVEL 24	1401.28 m²	268.91 m²	1132.37 m²
LEVEL 25	1401.28 m²	268.91 m²	1132.37 m²
LEVEL 26	1401.28 m²	268.91 m²	1132.37 m²
LEVEL 27	1401.28 m²	268.91 m²	1132.37 m²
LEVEL 28	1401.28 m²	268.91 m²	1132.37 m²
LEVEL 29	1401.28 m²	268.91 m²	1132.37 m²
LEVEL 30	1401.28 m²	268.91 m²	1132.37 m²
LEVEL 31	1401.28 m²	268.91 m²	1132.37 m²
LEVEL 32	700.64 m²	132.46 m²	568.17 m²
LEVEL 33	700.64 m²	132.46 m²	568.17 m²
LEVEL 34	700.64 m²	132.46 m²	568.17 m²
LEVEL 35	700.64 m²	132.46 m²	568.17 m²
LEVEL 36	700.64 m²	132.46 m²	568.17 m²
GRAND TOTAL	63033.52 m²	25846.46 m²	37187.06 m²

NOTES
1. Gross Construction Area (GCA) defined as total area of each floor area, below and above grade, measured from the interiors of outside walls, including floor area occupied by interior walls, with no deductions per City of Ottawa Zoning By-law No. 2008-250
2. Deductions as per City of Ottawa Zoning By-law No. 2008-250 definition of Gross Floor Area.
3. Gross Floor Area (GFA) is GCA minus GFA Deductions.

AMENITY AREA

Amenity Location	Amenity Function	Area
INDOOR AMENITY	COMMUNAL AMENITY	2105.94 m²
OUTDOOR AMENITY	COMMUNAL AMENITY	2411.61 m²
OUTDOOR AMENITY	PRIVATE AMENITY	804.61 m²
Total Amenity Space		5322.15 m²

REQUIRED AMENITY AREA = 601 UNITS x 6sm = 3606 m^2
50% IS REQUIRED TO BE COMMUNAL = 1803 m^2

UNIT COUNT BY TYPE

Unit Type	Reference Number	Count
AFFORDABLE		
1 BEDROOM	A1	45
2 BEDROOM	A2	67
3 BEDROOM	A3	5
		117
MARKET		
1 BEDROOM	M1	121
1 BEDROOM + DEN	M1D	102
2 BEDROOM	M2	103
2 BEDROOM + DEN	M2D	14
STUDIO	MS	14
		354
MULTI FAITH HOUSING		
1 BEDROOM	MHI A1	67
2 BEDROOM	MHI A2	52
3 BEDROOM	MHI A3	11
		130
GRAND TOTAL		601

UNIT COUNTS BY LOCATION

Location	Count
EAST PODIUM	20
EAST TOWER	250
WEST PODIUM	31
WEST TOWER	300
GRAND TOTAL	601

AREAS BY LEVEL

Function of Space	Area
LEVEL P2	
CIRCULATION	174.44 m²
CORE	224.83 m²
MECHANICAL	763.06 m²
PARKING	3883.63 m²
STORAGE	824.96 m²
	5870.92 m²
LEVEL P1	
BUILDING SERVICES	442.48 m²
CIRCULATION	194.43 m²
CORE	249.78 m²
DAY CARE	40.89 m²
MECHANICAL	487.36 m²
PARKING	3671.13 m²
STORAGE	207.06 m²
	5293.13 m²
LEVEL 01	
BIKE PARKING	240.50 m²
BUILDING SERVICES	193.01 m²
CIRCULATION	293.79 m²
COMMUNAL AMENITY	629.50 m²
CORE	267.52 m²
DAY CARE	59.62 m²
MECHANICAL	205.27 m²
OFFICE	21.68 m²
PARKING	741.67 m²
RETAIL	968.07 m²
STORAGE	39.74 m²
WC	136.84 m²
	3797.22 m²
LEVEL 02	
BIKE PARKING	566.05 m²
BUILDING SERVICES	38.85 m²
CIRCULATION	154.71 m²
CORE	231.12 m²
MECHANICAL	69.38 m²
STORAGE	151.70 m²
	1211.81 m²
LEVEL 03	
CIRCULATION	225.85 m²
COMMUNAL AMENITY	172.95 m²
CORE	201.31 m²
DAY CARE	1036.26 m²
RESIDENTIAL	1109.07 m²
STORAGE	84.58 m²
	2830.02 m²
LEVEL 04	
CIRCULATION	284.71 m²
COMMUNAL AMENITY	193.34 m²
CORE	196.86 m²
RESIDENTIAL	1589.16 m²
STORAGE	130.47 m²
	2394.55 m²

Function of Space	Area
LEVEL 05	
CIRCULATION	259.13 m²
COMMUNAL AMENITY	256.48 m²
CORE	178.00 m²
RESIDENTIAL	1173.02 m²
STORAGE	132.83 m²
	1999.45 m²
LEVEL 06	
CIRCULATION	42.49 m²
COMMUNAL AMENITY	853.67 m²
CORE	138.69 m²
MECHANICAL	19.54 m²
WC	46.79 m²
	1101.18 m²
LEVEL 07	
CIRCULATION	130.42 m²
CORE	134.52 m²
RESIDENTIAL	1136.35 m²
	1401.29 m²
LEVEL 08	
CIRCULATION	130.42 m²
CORE	134.52 m²
RESIDENTIAL	1136.35 m²
	1401.29 m²
LEVEL 09	
CIRCULATION	130.42 m²
CORE	134.52 m²
RESIDENTIAL	1136.35 m²
	1401.29 m²
LEVEL 10	
CIRCULATION	130.42 m²
CORE	134.52 m²
RESIDENTIAL	1136.35 m²
	1401.29 m²
LEVEL 11	
CIRCULATION	130.42 m²
CORE	134.52 m²
RESIDENTIAL	1136.35 m²
	1401.29 m²
LEVEL 12	
CIRCULATION	130.42 m²
CORE	134.52 m²
RESIDENTIAL	1136.35 m²
	1401.29 m²
LEVEL 13	
CIRCULATION	130.42 m²
CORE	134.52 m²
RESIDENTIAL	1136.35 m²
	1401.29 m²
LEVEL 14	
CIRCULATION	130.42 m²
CORE	134.52 m²
RESIDENTIAL	1136.35 m²
	1401.29 m²

Function of Space	Area
LEVEL 15	
CIRCULATION	130.42 m²
CORE	134.52 m²
RESIDENTIAL	1136.35 m²
	1401.29 m²
LEVEL 16	
CIRCULATION	130.42 m²
CORE	134.52 m²
RESIDENTIAL	1136.35 m²
	1401.29 m²
LEVEL 17	
CIRCULATION	130.42 m²
CORE	134.52 m²
RESIDENTIAL	1136.35 m²
	1401.29 m²
LEVEL 18	
CIRCULATION	130.40 m²
CORE	138.51 m²
RESIDENTIAL	1132.37 m²
	1401.28 m²
LEVEL 19	
CIRCULATION	130.40 m²
CORE	138.51 m²
RESIDENTIAL	1132.37 m²
	1401.28 m²
LEVEL 20	
CIRCULATION	130.40 m²
CORE	138.51 m²
RESIDENTIAL	1132.37 m²
	1401.28 m²
LEVEL 21	
CIRCULATION	130.40 m²
CORE	138.51 m²
RESIDENTIAL	1132.37 m²
	1401.28 m²
LEVEL 22	
CIRCULATION	130.40 m²
CORE	138.51 m²
RESIDENTIAL	1132.37 m²
	1401.28 m²
LEVEL 23	
CIRCULATION	130.40 m²
CORE	138.51 m²
RESIDENTIAL	1132.37 m²
	1401.28 m²
LEVEL 24	
CIRCULATION	130.40 m²
CORE	138.51 m²
RESIDENTIAL	1132.37 m²
	1401.28 m²
LEVEL 25	
CIRCULATION	130.40 m²
CORE	138.51 m²
RESIDENTIAL	1132.37 m²
	1401.28 m²

Function of Space	Area
LEVEL 26	
CIRCULATION	130.40 m²
CORE	138.51 m²
RESIDENTIAL	1132.37 m²
	1401.28 m²
LEVEL 27	
CIRCULATION	130.40 m²
CORE	138.51 m²
RESIDENTIAL	1132.37 m²
	1401.28 m²
LEVEL 28	
CIRCULATION	130.40 m²
CORE	138.51 m²
RESIDENTIAL	1132.37 m²
	1401.28 m²
LEVEL 29	
CIRCULATION	130.40 m²
CORE	138.51 m²
RESIDENTIAL	1132.37 m²
	1401.28 m²
LEVEL 30	
CIRCULATION	130.40 m²
CORE	138.51 m²
RESIDENTIAL	1132.37 m²
	1401.28 m²
LEVEL 31	
CIRCULATION	130.40 m²
CORE	138.51 m²
RESIDENTIAL	1132.37 m²
	1401.28 m²
LEVEL 32	
CIRCULATION	65.62 m²
CORE	66.84 m²
RESIDENTIAL	568.17 m²
	700.64 m²
LEVEL 33	
CIRCULATION	65.62 m²
CORE	66.84 m²
RESIDENTIAL	568.17 m²
	700.64 m²
LEVEL 34	
CIRCULATION	65.62 m²
CORE	66.84 m²
RESIDENTIAL	568.17 m²
	700.64 m²
LEVEL 35	
CIRCULATION	65.62 m²
CORE	66.84 m²
RESIDENTIAL	568.17 m²
	700.64 m²
LEVEL 36	
CIRCULATION	65.62 m²
CORE	66.84 m²
RESIDENTIAL	568.17 m²
	700.64 m²
	63033.52 m²

SITE DATA

TOTAL LOT AREA	**9,629 m^2**
GROSS FLOOR AREA	**37,187.06 m^2**
FLOOR SPACE INDEX	**3.89**
LOT FRONTAGE ON ALBERT STREET	**144.0 m**
LOT FRONTAGE ON BOOTH STREET	**77.9 m**
LOT FRONTAGE ON LRT	**138.6 m**
LOT FRONTAGE ON EAST PROPERTY LINE	**63.8 m**
PROPOSED BUILDING LENGTH AT GRADE ON ALBERT STREET	
EAST TOWER (PARALLEL)	29.5 m
EAST TOWER (ANGLED)	43.3 m
WEST TOWER (PARALLEL)	42.7 m
WEST TOWER (ANGLED)	30.2 m
PROPOSED BUILDING LENGTH AT GRADE ON BOOTH STREET	
WEST TOWER	24.7 m
PROPOSED BUILDING LENGTH AT GRADE ON LRT	
EAST TOWER (PARALLEL)	42.7 m
EAST TOWER (ANGLED)	21.2 m
EAST TOWER (ANGLED)	15.2 m
WEST TOWER (PARALLEL)	46.2 m
WEST TOWER (ANGLED)	11.4 m
PROPOSED BUILDING LENGTH AT GRADE ON EAST PROPERTY LINE	
EAST TOWER	46.8 m
FRONT YARD SETBACK	**1.8-4.9 m**
CORNER SIDE YARD SETBACK (BOOTH)	**0-9.9 m**
INTERIOR SIDE YARD SETBACK (EAST)	**9.8 m**
REAR YARD SETBACK (NORTH)	**10 m**
EAST TOWER AVG GRADE	**63.000m**
MIDPOINT OF EAST TOWER ROOF	**167.548m**
EAST TOWER BUILDING HEIGHT	**104.55m**
WEST TOWER AVG GRADE	**62.800m**
MIDPOINT OF WEST TOWER ROOF	**183.403m**
WEST TOWER BUILDING HEIGHT	**120.605m**

PARKING SPACES

TYPE	REQUIRED RATE	REQUIRED	PROPOSED
RESIDENTIAL	N/A, MAX 1.5/UNIT	0 MIN, 902 MAX	128
RESIDENTIAL BARRIER-FREE	N/A	0	3
TOTAL RESIDENTIAL		0	131
VISITOR RESIDENTIAL	0.1/unit	61	60 (+1 B-F)
VISITOR RETAIL	N/A, MAX 10	0	6
VISITOR DAY CARE	N/A	0	4
VISITOR BARRIER-FREE	20-99 spaces = 1	1 (of 61)	1
TOTAL VISITOR		61	71
GRAND TOTAL		61	202

NOTES
Refer to A10-0P1 for vehicular further information on parking space sizes and waste statistics.

LOADING SPACES

TYPE	REQUIRED	PROPOSED
RETAIL	0	1
OTHER (DAY CARE)	1	1
RESIDENTIAL	0	SHARED W/DAY CARE
TOTAL TYP LOADING SPACES	1	2

1 TYPE G LOADING SPACE IS PROVIDED FOR WASTE COLLECTION

BICYCLE PARKING SPACES

TYPE	REQUIRED RATE	REQUIRED	PROPOSED
REQUIRED SPACES			
RESIDENTIAL	0.5/unit	301	
RETAIL	1/250 m^2	4	
DAY CARE	1/250 m^2	5	
TOTAL REQUIRED		**310**	
PROVIDED SPACES			
INTERIOR SECURE SPACES	MIN 25%	78	604
EXTERIOR SPACES	MAX 50%	N/A	138
TOTAL PROVIDED			**742**
AT GROUND LEVEL HORIZONTAL	50% of REQD	155	222

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665 Albert St
Ottawa, ON
Canada

dream

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30 Adelaide St. E., Suite 301,
Toronto, ON M5C 3H1

KEYPLAN

TRUE NORTH

ISSUE CHART

NUMBER	ISSUE	DATE
1	ISSUED FOR OPA / ZBA / SPA	2022-04-22

Job Number

TITLE

STATISTICS AND CONTEXT PLAN

SHEET NUMBER

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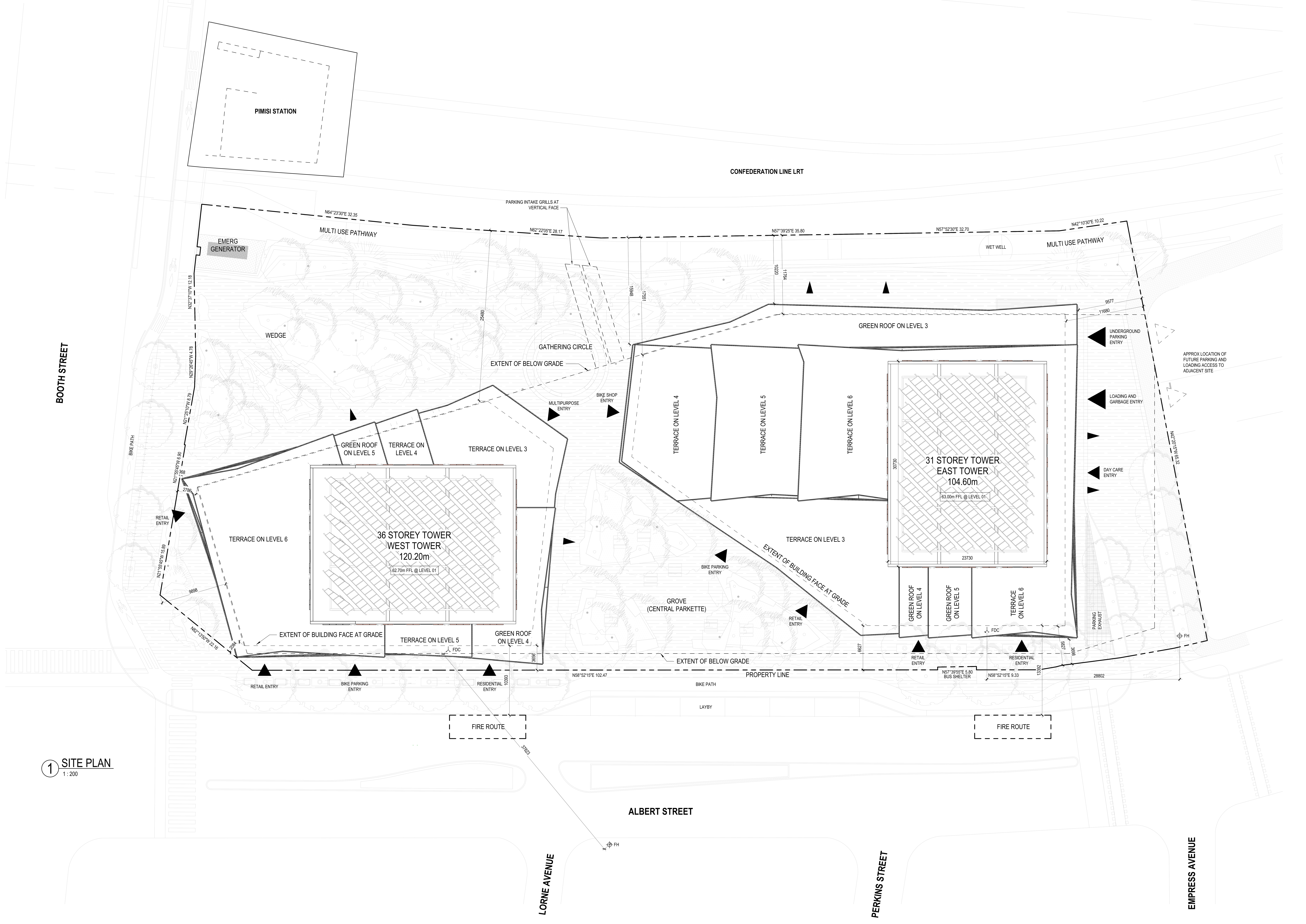

1 SITE PLAN
1 : 200

NOTE

1. REFER TO LANDSCAPE AND CIVIL DRAWING FOR LANDSCAPE DESIGN, SITE SERVICING AND GRADING.

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PROJECT

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665 Albert St
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DREAM
30 Adelaide St. E., Suite 301, Toronto, ON M5C 3H1

KEYPLAN

TRUE NORTH

ISSUE CHART

MARK	ISSUE	DATE
1	ISSUED FOR OPA / ZBA / SPA	2022-04-22

Job Number

TITLE

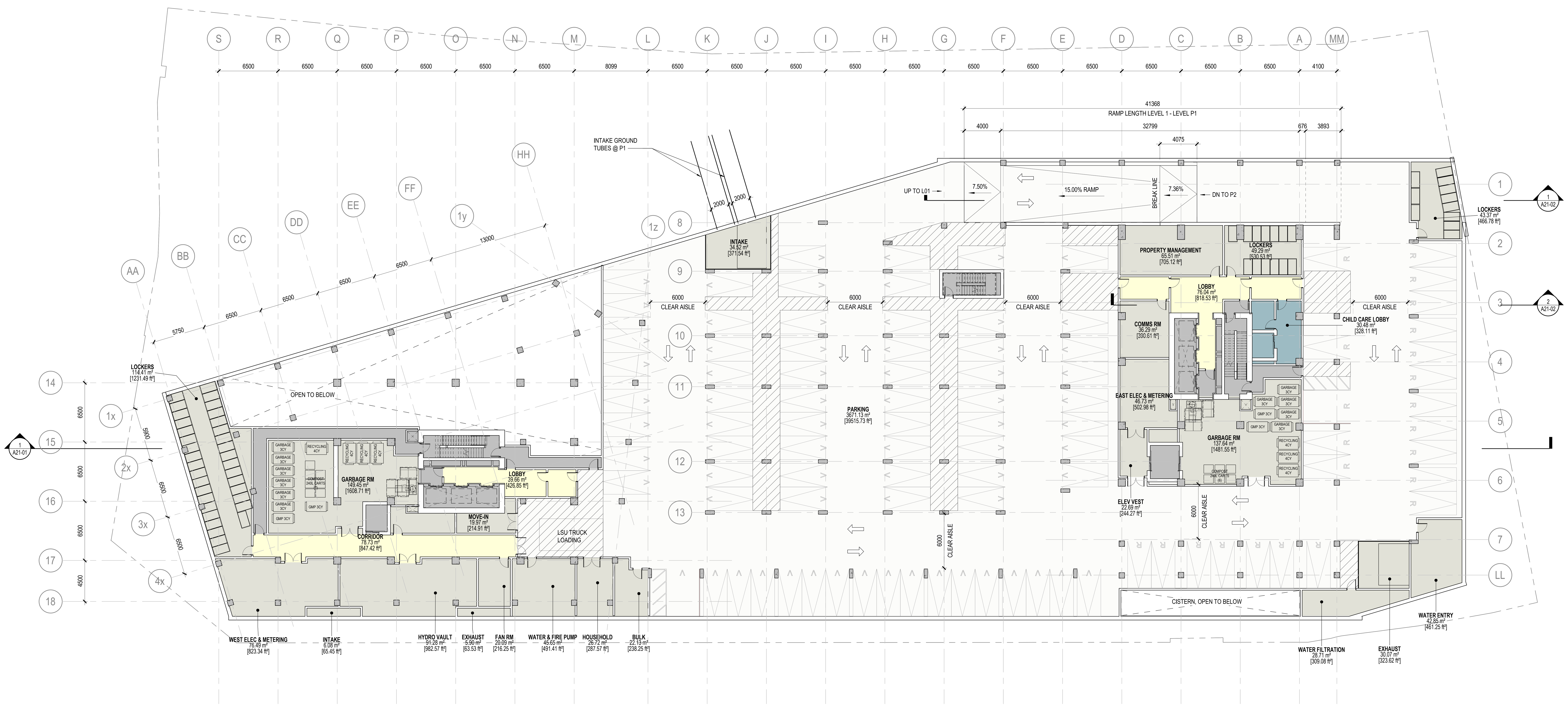
SITE PLAN

SHEET NUMBER

A01-03

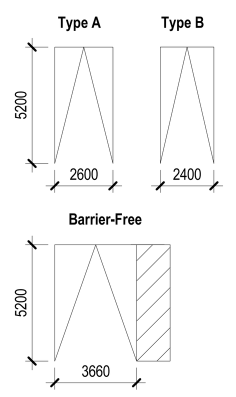
SPACE TYPE

- A1
- A2
- A3
- BUILDING OPS
- CIRCULATION
- CORE
- DAY CARE
- INDOOR AMENITY
- M1
- M1D
- M2
- M2D
- MHI A1
- MHI A2
- MHI A3
- MS
- PARKING
- RETAIL
- STORAGE



1 LEVEL P1
1 : 200

PARKING LEGEND



Parking Space Types:

Type A - Space abutting columns/walls
5.2m (D) x 2.6m (W)

Type B - Space with no obstructions
5.2m (D) x 2.4m (W)

"V" denotes visitor parking.
"R" denotes residents parking.
"SC" denotes small car parking associated with reduced length Type B spaces.

Barrier-Free
5.2m (D) x 3.66m (W) + 1.5m access aisle

Parking space types determined per City of Ottawa By-Law 2008-250

PARKING CALCULATIONS

Visitor Parking
0.1 x 601 units = 61 spaces required

Barrier-Free Parking
Public Parking Spaces = Spaces to be Reserved for Persons with Disabilities
20-99 = 1 visitor parking space is required to be barrier-free

Rates and quantities determined per City of Ottawa By-Law 2008-250

PARKING SCHEDULE

cvt	Type	Count
LEVEL P1	Barrier Free Resident	1
LEVEL P1	Barrier Free Visitor	1
LEVEL P1	Residents (2.6m)	22
LEVEL P1	Visitor (2.6m)	70
LEVEL P2	Barrier Free Resident	2
LEVEL P2	Residents (2.4m)	3
LEVEL P2	Residents (2.6m)	102
LEVEL P2	Residents Small Car (2.6mX4.6M)	1

GRAND TOTAL: 202

WASTE CALCULATIONS

RESIDENTIAL WASTE COLLECTION:

WEST TOWER

Garbage
331 units x 0.053 yards
= 17.5 yards/3CY = 6 bins

Recycling
331 units x 0.038 yards
= 12.5 yards/4CY = 4 bins

GMP
331 units x 0.018 yards
= 5.9 yards/3CY = 2 bins

Compost
331 units / 50
= 7 x 240L bins

EAST TOWER

Garbage
270 units x 0.053 yards
= 14.31 yards/3CY = 5 bins

Recycling
270 units x 0.038 yards
= 10.26 yards/4CY = 3 bins

GMP
270 units x 0.018 yards
= 4.86 yards/3CY = 2 bins

Compost
270 units / 50
= 6 x 240L bins

LOCKER SCHEDULE

Level	Count
LEVEL 02	40
LEVEL 03	23
LEVEL 04	39
LEVEL 05	39
LEVEL P1	60
LEVEL P2	247

GRAND TOTAL: 448

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KEYPLAN

TRUE NORTH

ISSUE CHART

MARK	ISSUE	DATE
1	ISSUED FOR OPA / ZBA / SPA	2022-04-22

Job Number

TITLE

LEVEL P1 FLOOR PLAN

SHEET NUMBER

A10-0P1

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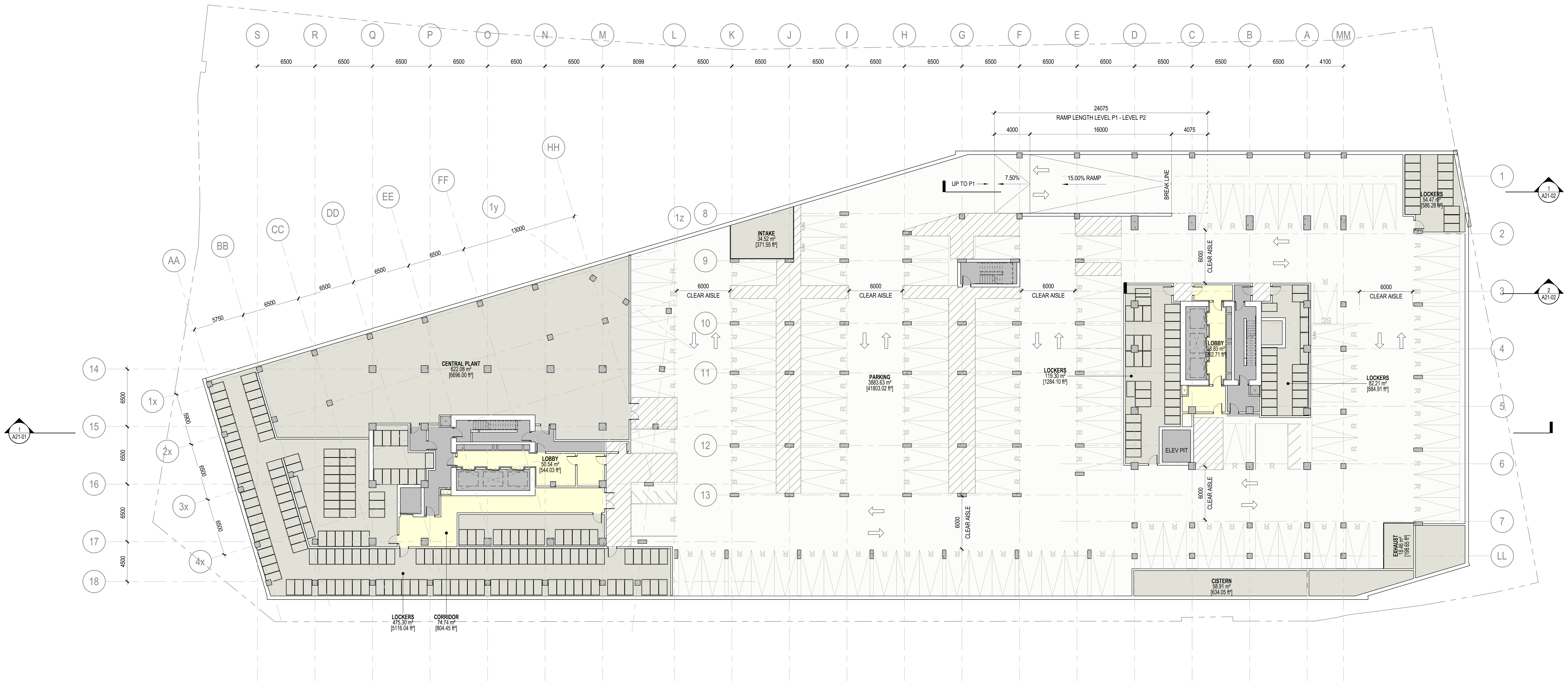
SPACE TYPE

A1
A2
A3
BUILDING OPS
CIRCULATION
CORE
DAY CARE
INDOOR AMENITY
M1
M1D
M2
M2D
MHI A1
MHI A2
MHI A3
MS
PARKING
RETAIL
STORAGE



1 LEVEL P2
1 : 200

PARKING LEGEND

Type A
Type B
Barrier-Free

Parking Space Types:

Type A - Space abutting columns/walls
5.2m (D) x 2.6m (W)

Type B - Space with no obstructions
5.2m (D) x 2.4m (W)

"V" denotes visitor parking.
"R" denotes residents parking.
"SC" denotes small car parking associated with reduced length Type B spaces.

Barrier-Free
5.2m (D) x 3.66m (W) + 1.5m access aisle

Parking space types determined per City of Ottawa By-Law 2008-250

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ISSUE CHART

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MARK	ISSUE	DATE

Job Number

TITLE

LEVEL P2 FLOOR PLAN

SHEET NUMBER

A10-0P2

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KEYPLAN

ISSUE CHART

1	ISSUED FOR OPA / ZBA / SPA	2022-04-22

Job Number

TITLE

LEVEL 01 FLOOR PLAN

SHEET NUMBER

A10-01

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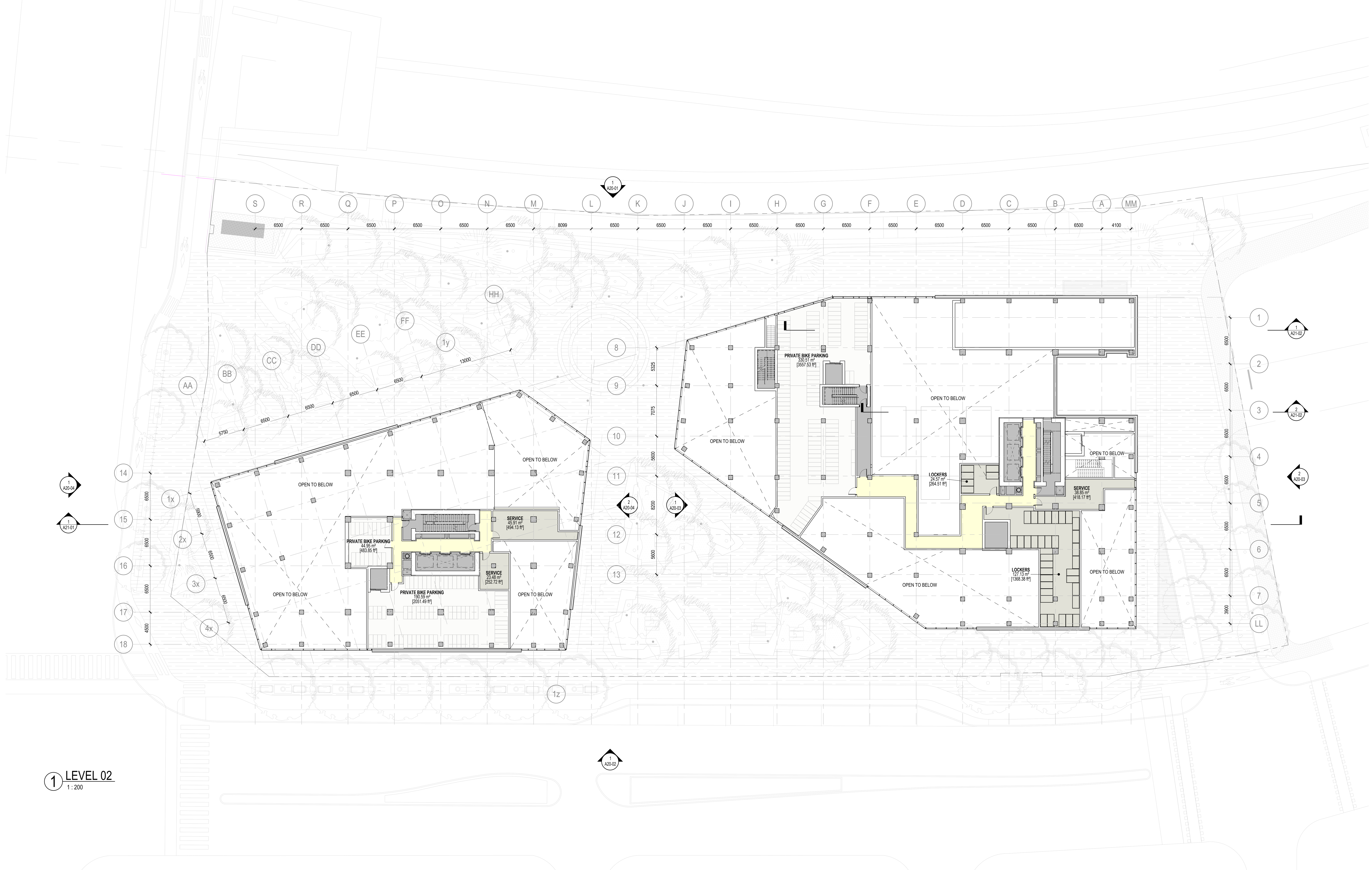
SPACE TYPE

A1	M1	PARKING
A2	M1D	RETAIL
A3	M2	STORAGE
BUILDING OPS	M2D	
CIRCULATION	MHI A1	
CORE	MHI A2	
DAY CARE	MHI A3	
INDOOR AMENITY	MS	



1 LEVEL 02
1 : 200

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Job Number

TITLE

LEVEL 02 FLOOR PLAN

SHEET NUMBER

A10-02

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SPACE TYPE

A1	M1	PARKING
A2	M1D	RETAIL
A3	M2	STORAGE
BUILDING OPS	M2D	
CIRCULATION	MHI A1	
CORE	MHI A2	
DAY CARE	MHI A3	
INDOOR AMENITY	MS	

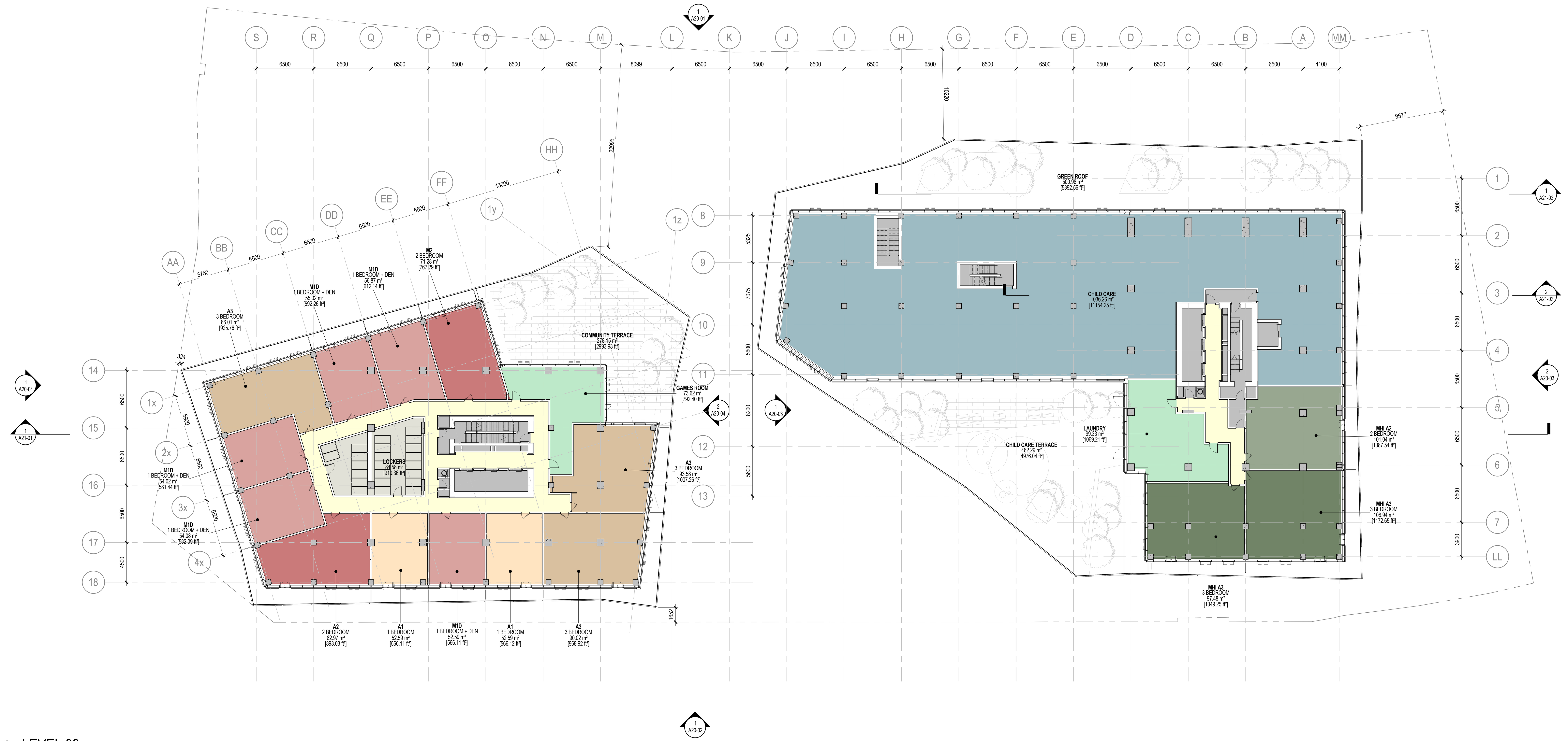
UNIT TYPE LEGEND

DREAM UNITS:

A1	Affordable 1 Bedroom
A2	Affordable 2 Bedroom
A3	Affordable 3 Bedroom
MS	Market Studio
M1	Market 1 Bedroom
M1D	Market 1 Bedroom plus Den
M2	Market 2 Bedroom
M2D	Market 2 Bedroom plus Den

MULTIFAITH HOUSING INITIAIVE UNITS:

MHI A1	MHI Affordable 1 Bedroom
MHI A2	MHI Affordable 2 Bedroom
MHI A3	MHI Affordable 3 Bedroom



1 LEVEL 03
1 : 200

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TITLE

LEVEL 03 FLOOR PLAN

SHEET NUMBER

A10-03

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SPACE TYPE

A1	M1	PARKING
A2	M1D	RETAIL
A3	M2	STORAGE
BUILDING OPS	M2D	
CIRCULATION	MHI A1	
CORE	MHI A2	
DAY CARE	MHI A3	
INDOOR AMENITY	MS	

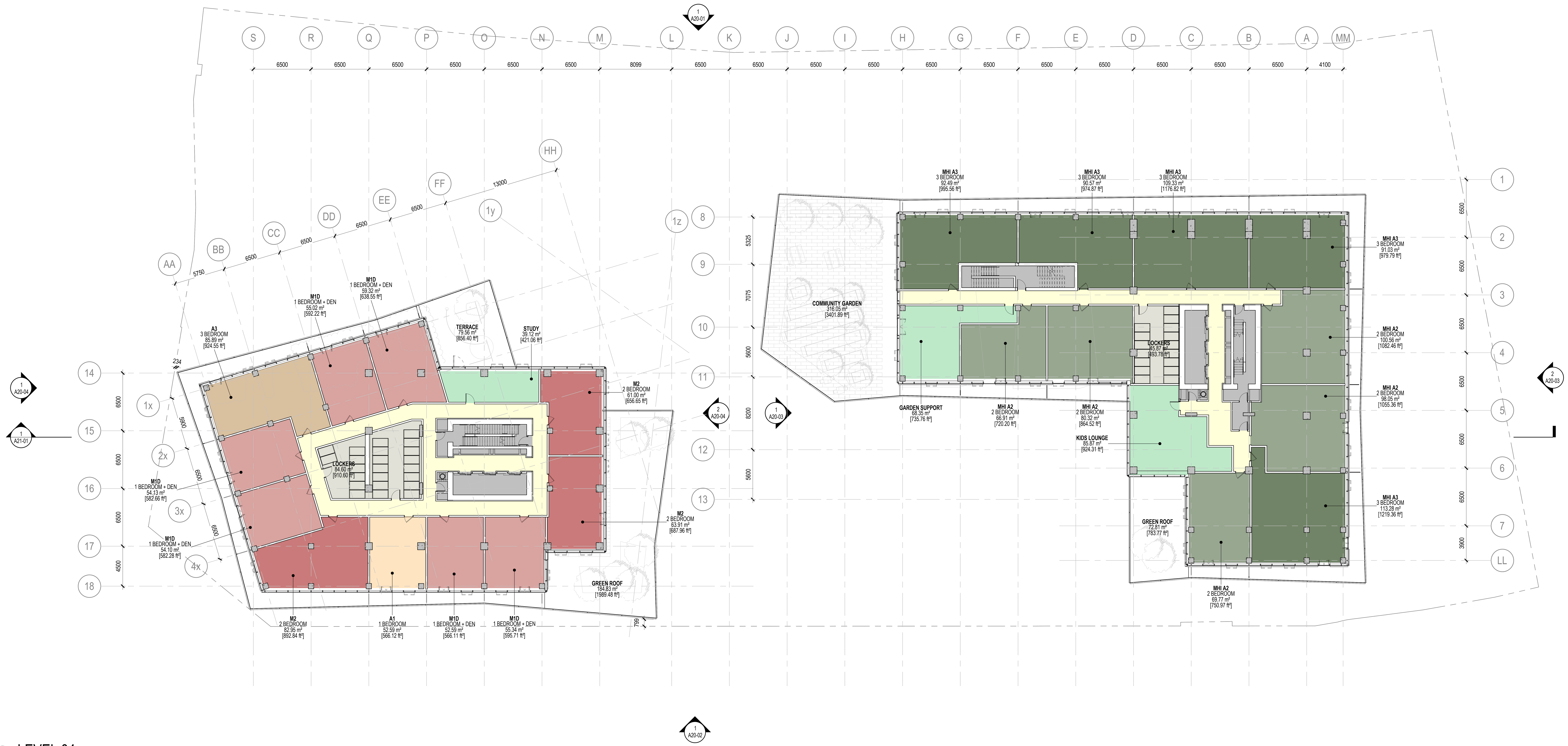
UNIT TYPE LEGEND

DREAM UNITS:

A1	Affordable 1 Bedroom
A2	Affordable 2 Bedroom
A3	Affordable 3 Bedroom
MS	Market Studio
M1	Market 1 Bedroom
M1D	Market 1 Bedroom plus Den
M2	Market 2 Bedroom
M2D	Market 2 Bedroom plus Den

MULTIFAITH HOUSING INITIAIVE UNITS:

MHI A1	MHI Affordable 1 Bedroom
MHI A2	MHI Affordable 2 Bedroom
MHI A3	MHI Affordable 3 Bedroom



1 LEVEL 04
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Job Number

TITLE

LEVEL 04 FLOOR PLAN

SHEET NUMBER

A10-04

SPACE TYPE

A1	M1	PARKING
A2	M1D	RETAIL
A3	M2	STORAGE
BUILDING OPS	M2D	
CIRCULATION	MHI A1	
CORE	MHI A2	
DAY CARE	MHI A3	
INDOOR AMENITY	MS	

UNIT TYPE LEGEND

DREAM UNITS:

A1	Affordable 1 Bedroom
A2	Affordable 2 Bedroom
A3	Affordable 3 Bedroom
MS	Market Studio
M1	Market 1 Bedroom
M1D	Market 1 Bedroom plus Den
M2	Market 2 Bedroom
M2D	Market 2 Bedroom plus Den

MULTIFAITH HOUSING INITIAIVE UNITS:

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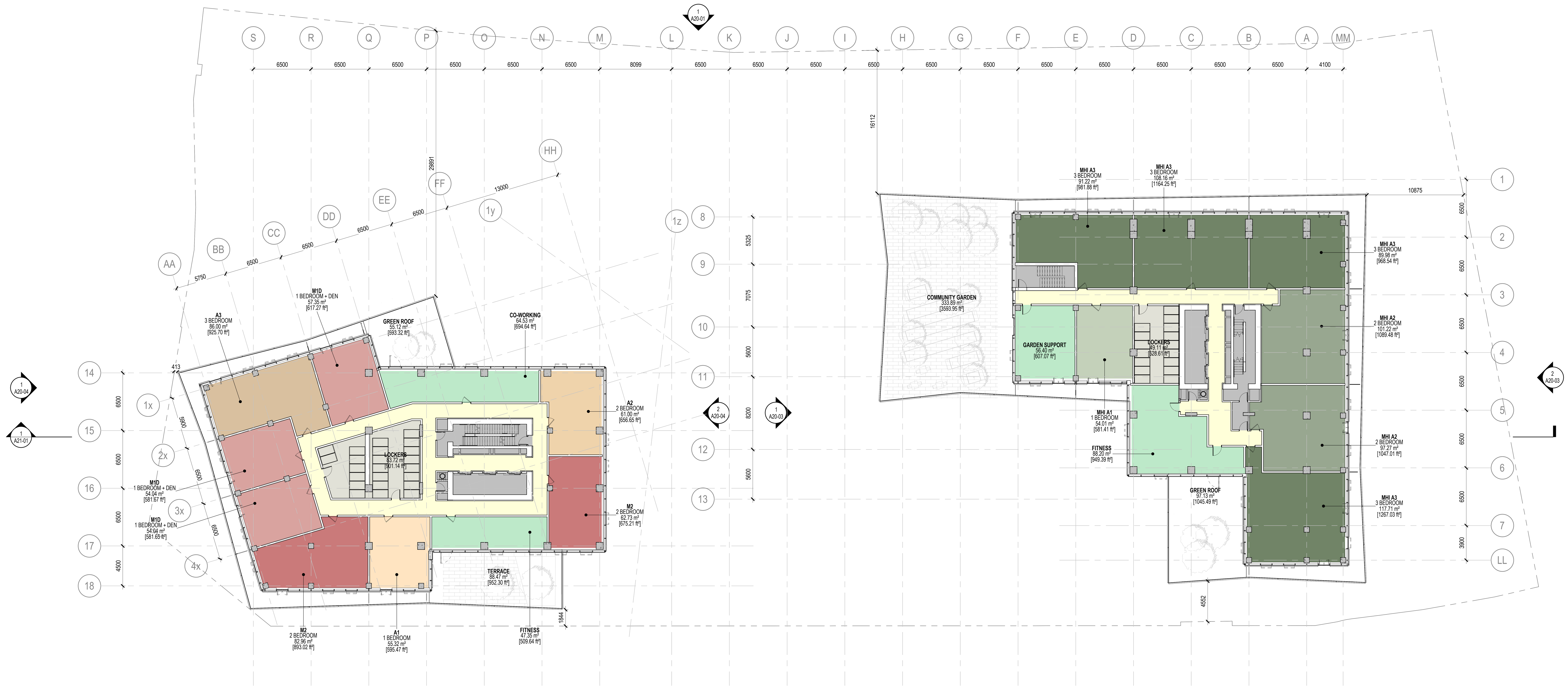
MHI A3	MHI Affordable 3 Bedroom



1 LEVEL 05
1 : 200

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KEYPLAN

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ISSUE CHART

MARK	ISSUE	DATE
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Job Number

TITLE

LEVEL 05 FLOOR PLAN

SHEET NUMBER

A10-05

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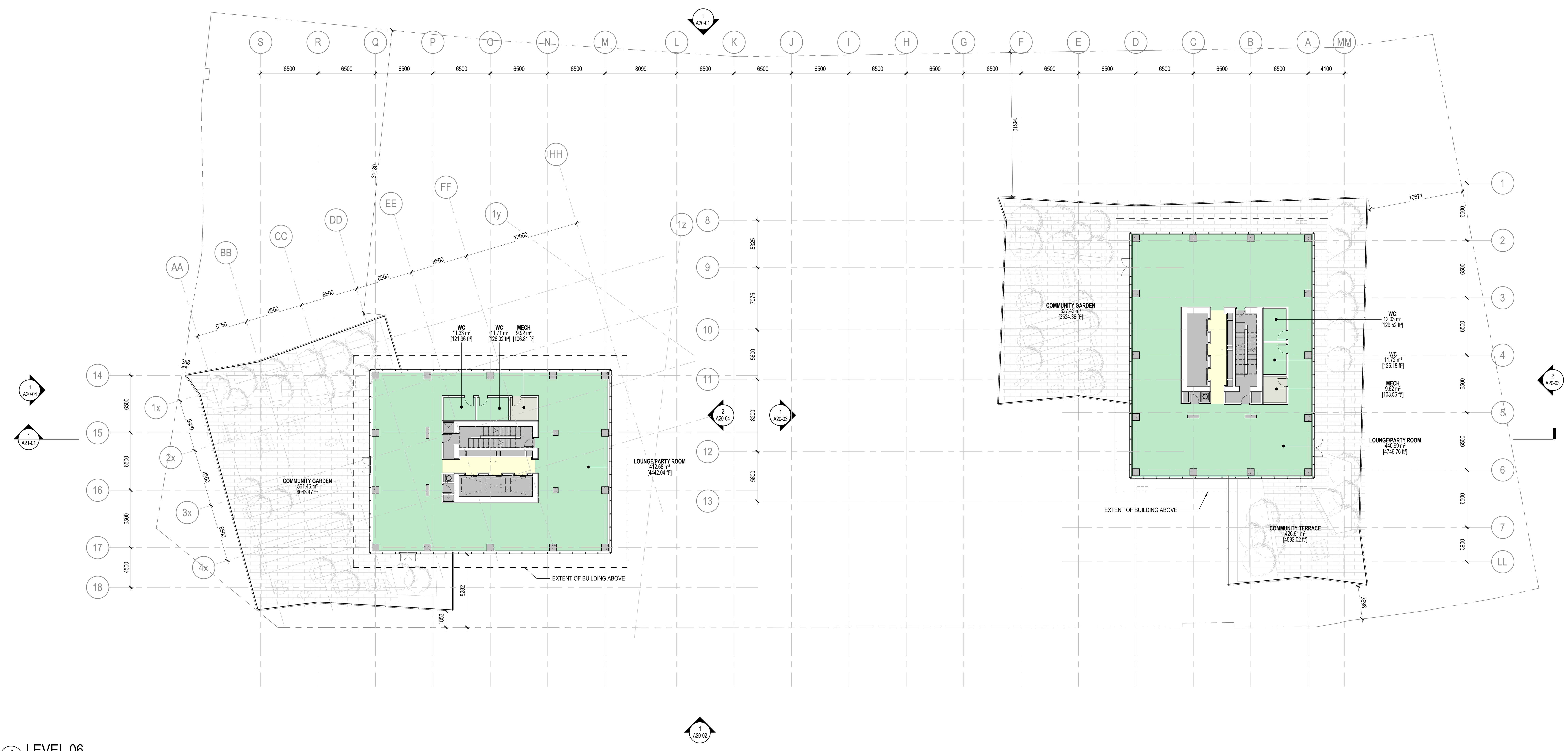

1 LEVEL 06
1 : 200

SPACE TYPE

A1	M1	PARKING
A2	M1D	RETAIL
A3	M2	STORAGE
BUILDING OPS	M2D	
CIRCULATION	MHI A1	
CORE	MHI A2	
DAY CARE	MHI A3	
INDOOR AMENITY	MS	

UNIT TYPE LEGEND

DREAM UNITS:	
A1	Affordable 1 Bedroom
A2	Affordable 2 Bedroom
A3	Affordable 3 Bedroom
MS	Market Studio
M1	Market 1 Bedroom
M1D	Market 1 Bedroom plus Den
M2	Market 2 Bedroom
M2D	Market 2 Bedroom plus Den
MULTIFAITH HOUSING INITIAIVE UNITS:	
MHI A1	MHI Affordable 1 Bedroom
MHI A2	MHI Affordable 2 Bedroom
MHI A3	MHI Affordable 3 Bedroom

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TITLE

LEVEL 06 FLOOR PLAN

SHEET NUMBER

A10-06

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SPACE TYPE

A1	M1	PARKING
A2	M1D	RETAIL
A3	M2	STORAGE
BUILDING OPS	M2D	
CIRCULATION	MHI A1	
CORE	MHI A2	
DAY CARE	MHI A3	
INDOOR AMENITY	MS	

UNIT TYPE LEGEND

DREAM UNITS:	
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A3	Affordable 3 Bedroom
MS	Market Studio
M1	Market 1 Bedroom
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M2D	Market 2 Bedroom plus Den
MULTIFAITH HOUSING INITIAIVE UNITS:	
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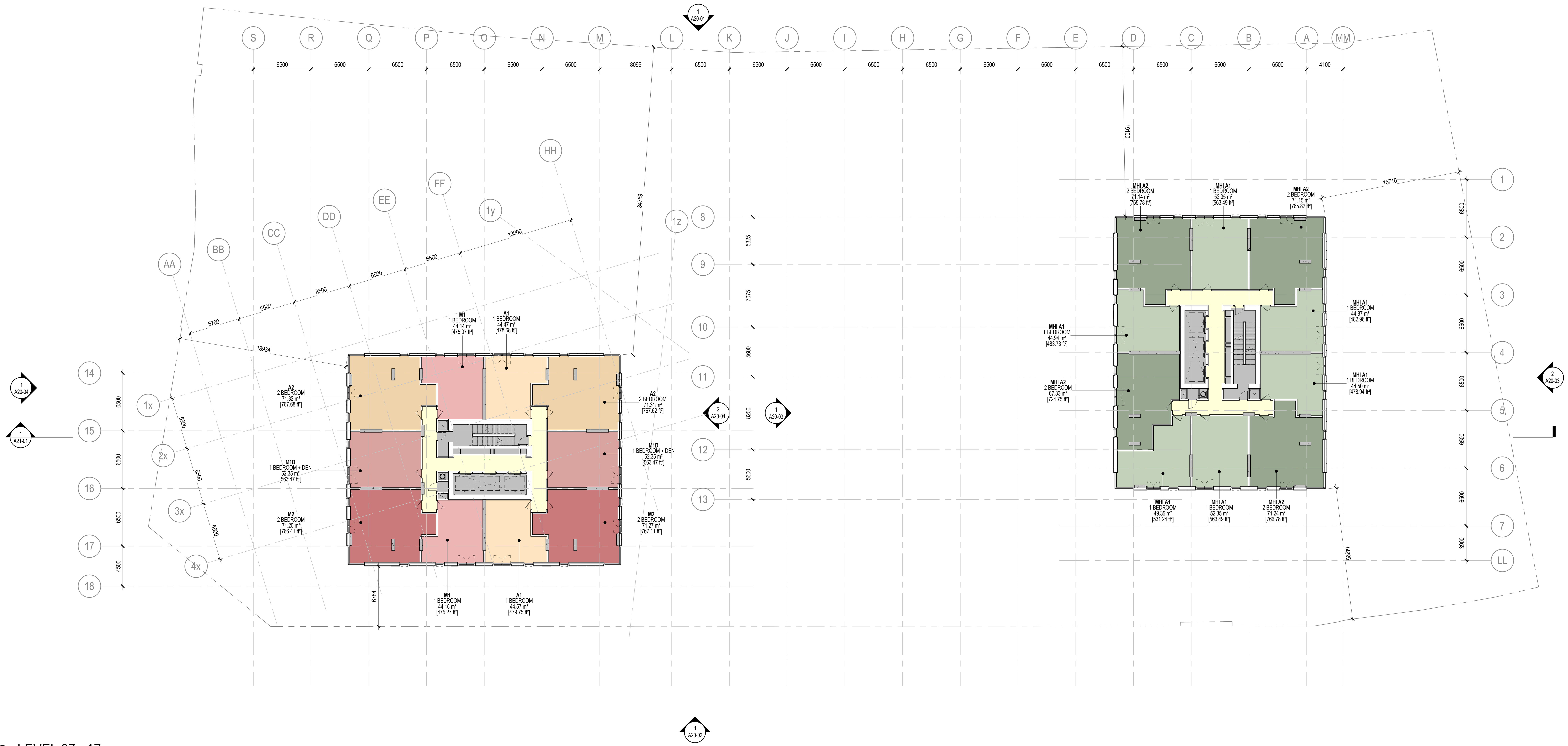
MHI A3	MHI Affordable 3 Bedroom



1 LEVEL 07 - 17
1 : 200

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ISSUE CHART

MARK	ISSUE	DATE
1	ISSUED FOR OPA / ZBA / SPA	2022-04-22

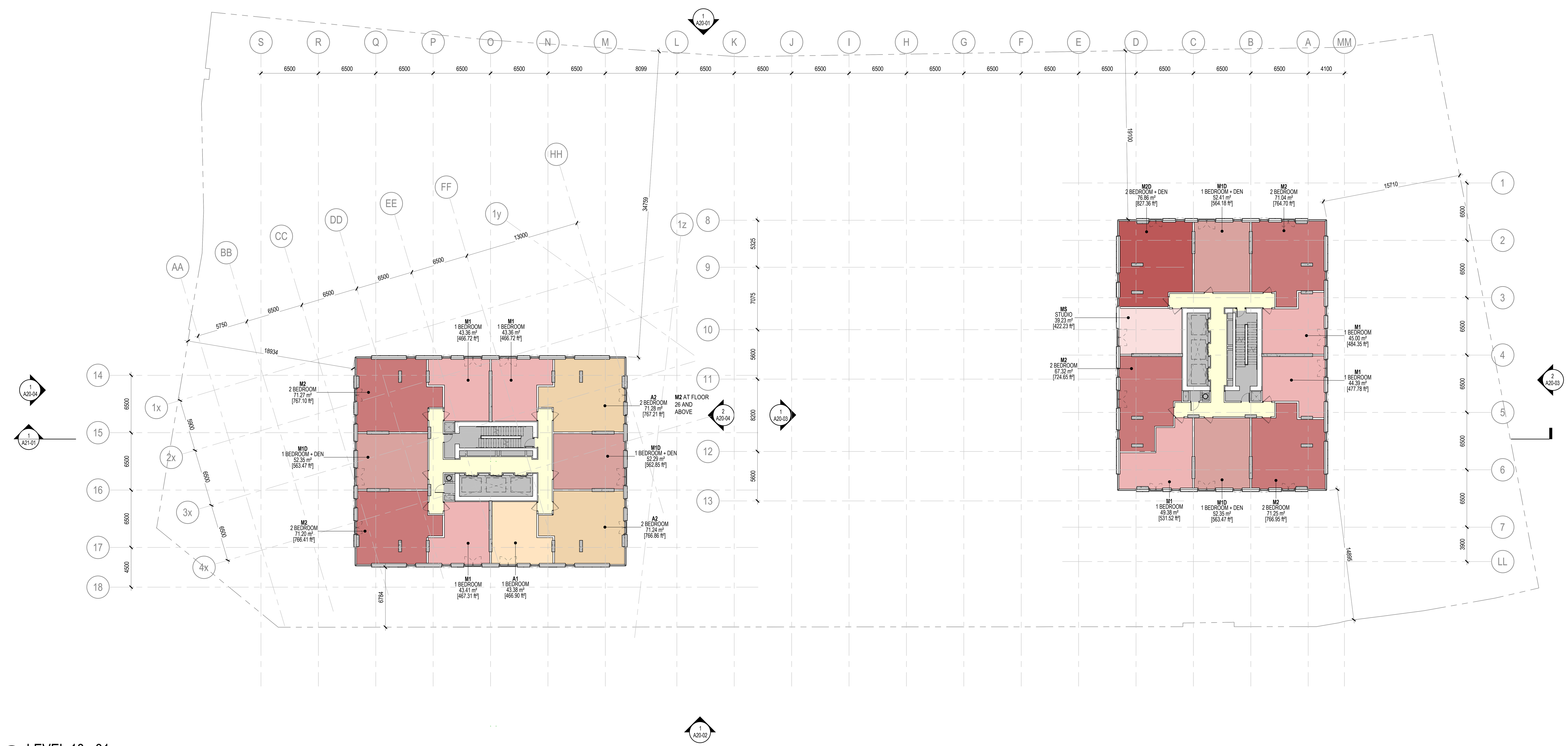
Job Number

TITLE

LEVEL 07-17 FLOOR PLAN

SHEET NUMBER

A10-07



1 LEVEL 18 - 31
1 : 200

SPACE TYPE

A1	M1	PARKING
A2	M1D	RETAIL
A3	M2	STORAGE
BUILDING OPS	M2D	
CIRCULATION	MHI A1	
CORE	MHI A2	
DAY CARE	MHI A3	
INDOOR AMENITY	MS	

UNIT TYPE LEGEND

DREAM UNITS:	
A1	Affordable 1 Bedroom
A2	Affordable 2 Bedroom
A3	Affordable 3 Bedroom
MS	Market Studio
M1	Market 1 Bedroom
M1D	Market 1 Bedroom plus Den
M2	Market 2 Bedroom
M2D	Market 2 Bedroom plus Den
MULTIFAITH HOUSING INITIAIVE UNITS:	
MHI A1	MHI Affordable 1 Bedroom
MHI A2	MHI Affordable 2 Bedroom
MHI A3	MHI Affordable 3 Bedroom

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ONTARIO ASSOCIATION OF ARCHITECTS
MATTHEW JOHNSTON
LICENCE 6101

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dream

DREAM
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KEYPLAN

TRUE NORTH

ISSUE CHART

MARK	ISSUE	DATE
1	ISSUED FOR OPA / ZBA / SPA	2022-04-22

Job Number

TITLE

LEVEL 18-31 FLOOR PLAN

SHEET NUMBER

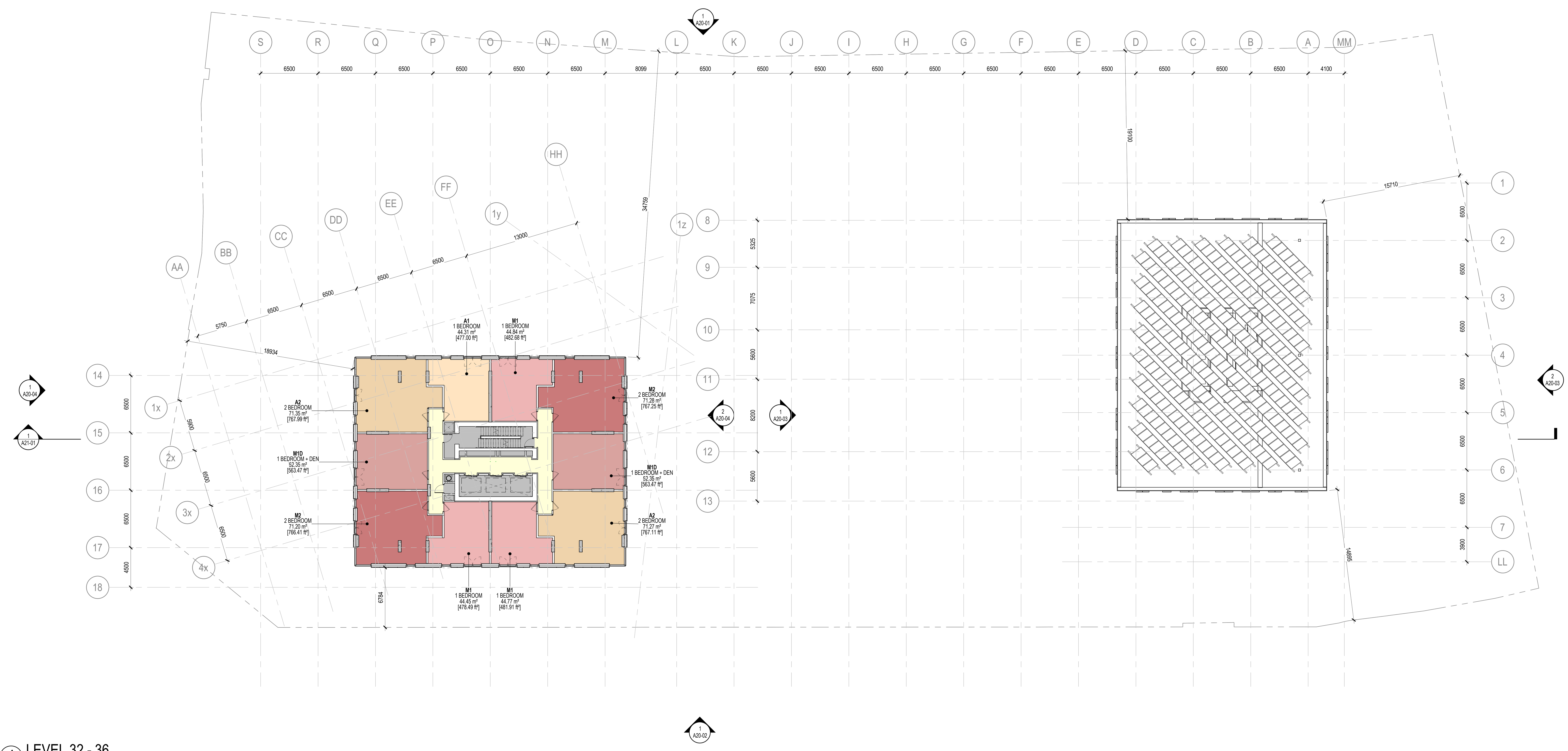
A10-08

SPACE TYPE

A1	M1	PARKING
A2	M1D	RETAIL
A3	M2	STORAGE
BUILDING OPS	M2D	
CIRCULATION	MHI A1	
CORE	MHI A2	
DAY CARE	MHI A3	
INDOOR AMENITY	MS	

UNIT TYPE LEGEND

DREAM UNITS:	
A1	Affordable 1 Bedroom
A2	Affordable 2 Bedroom
A3	Affordable 3 Bedroom
MS	Market Studio
M1	Market 1 Bedroom
M1D	Market 1 Bedroom plus Den
M2	Market 2 Bedroom
M2D	Market 2 Bedroom plus Den
MULTIFAITH HOUSING INITIAIVE UNITS:	
MHI A1	MHI Affordable 1 Bedroom
MHI A2	MHI Affordable 2 Bedroom
MHI A3	MHI Affordable 3 Bedroom



1 LEVEL 32 - 36
1 : 200

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LICENCE
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ISSUE CHART

MARK	ISSUE	DATE
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Job Number

TITLE

LEVEL 32-36 FLOOR PLAN

SHEET NUMBER

A10-09

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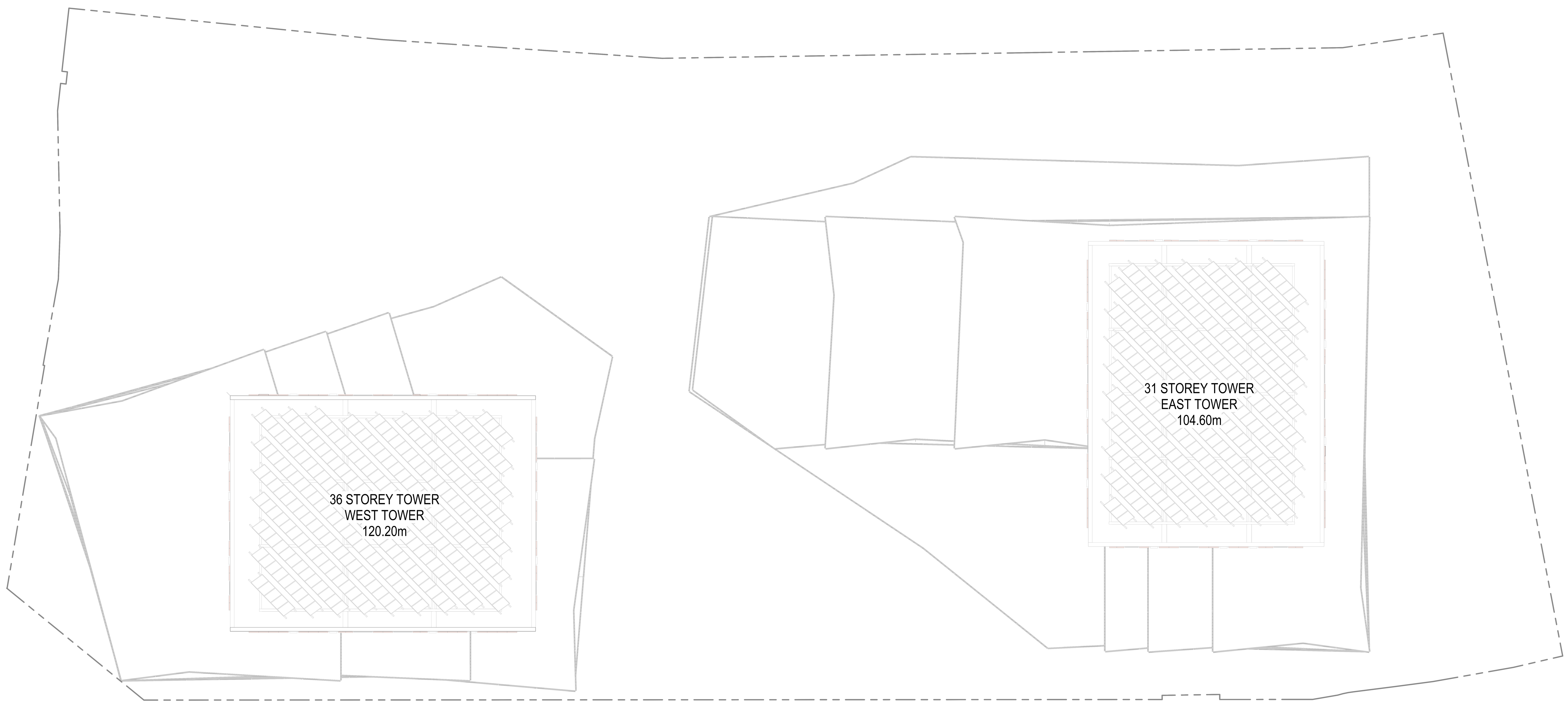

1 ROOF PLAN
1 : 200

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CONSULTANT
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CONTRACTOR
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ISSUE CHART

1	ISSUED FOR OPA / ZBA / SPA	2022-04-22
MARK	ISSUE	DATE

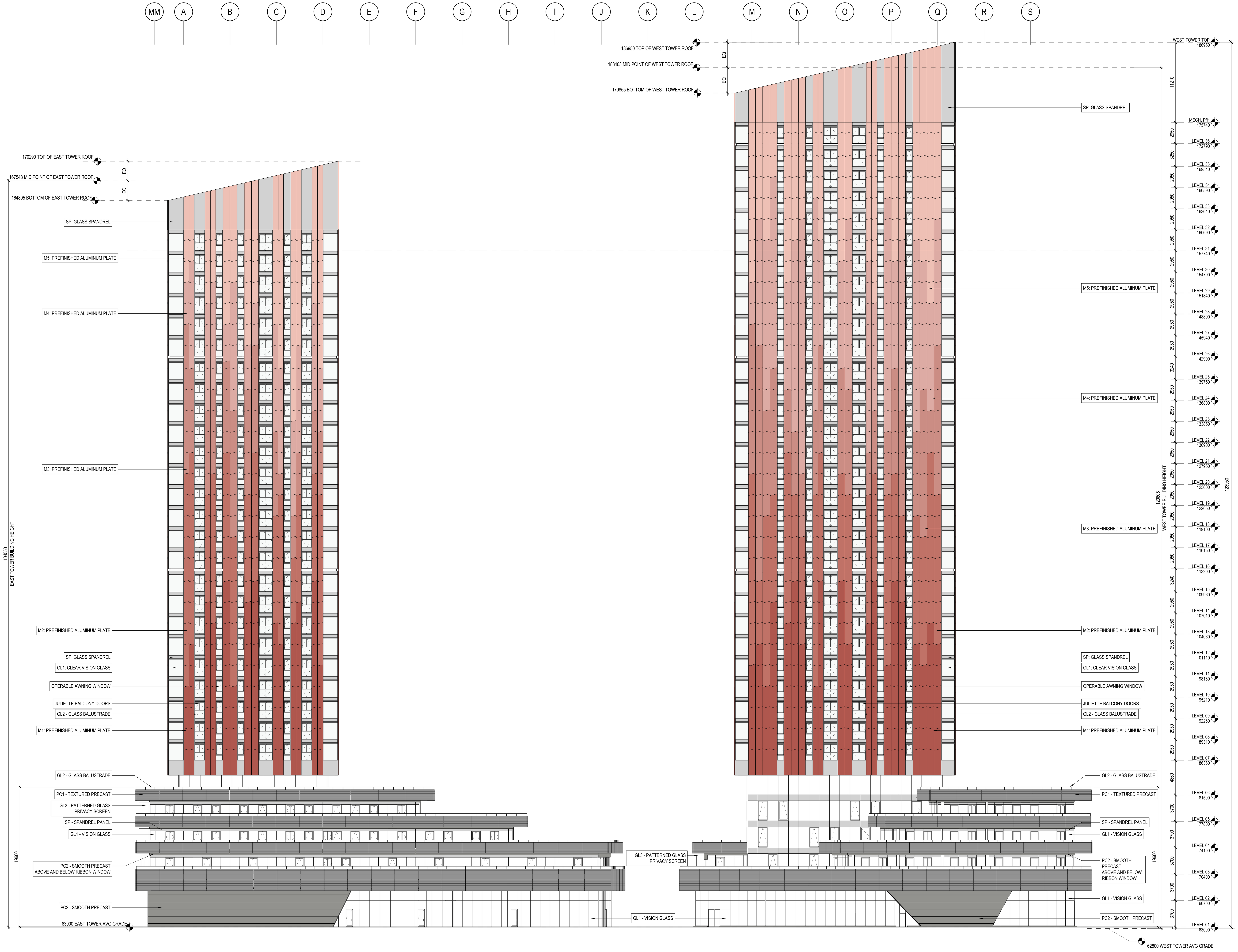
Job Number

TITLE

ROOF PLAN

SHEET NUMBER

A10-10



1 NORTH ELEVATION
1 : 200

ELEVATIONS LEGEND

M1	PREFINISHED ALUMINUM PLATE (COLOUR 1)
M2	PREFINISHED ALUMINUM PLATE (COLOUR 2)
M3	PREFINISHED ALUMINUM PLATE (COLOUR 3)
M4	PREFINISHED ALUMINUM PLATE (COLOUR 4)
M5	PREFINISHED ALUMINUM PLATE (COLOUR 5)
PC1	TEXTURED PRECAST DARK LIMESTONE COLOUR
PC2	SMOOTH PRECAST DARK LIMESTONE COLOUR
SP	SPANDREL PANEL (REFER TO NOTE 1 FOR EXTENT OF BIRD-FRIENDLY FRIT)
GL1	VISION GLAZING (REFER TO NOTE 1 FOR EXTENT OF BIRD-FRIENDLY FRIT)
GL2	GLASS BALUSTRADE
GL3	PATTERNED GLASS PRIVACY SCREEN
LV	PREFINISHED STORM-RESISTANT LOUVER

NOTE

1. BIRD-FRIENDLY VISUAL MARKERS (5MM DIAM. DARK GREY DOTS SPACED 50MM APART) TO BE ETCHED ON FACE 1 OF AT LEAST 90% OF ALL EXTERIOR TRANSPARENT GLASS PANELS TO HEIGHT OF 16M ABOVE GRADE AND / OR 4M ABOVE GREEN ROOFS AND TERRACES, AND AT ALL GLASS CORNERS.

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KEYPLAN

TRUE NORTH

ISSUE CHART

1	ISSUED FOR OPA / ZBA / SPA	2022-04-22
NUMBER	ISSUE	DATE

Job Number

TITLE

OVERALL NORTH ELEVATION

SHEET NUMBER

A20-01

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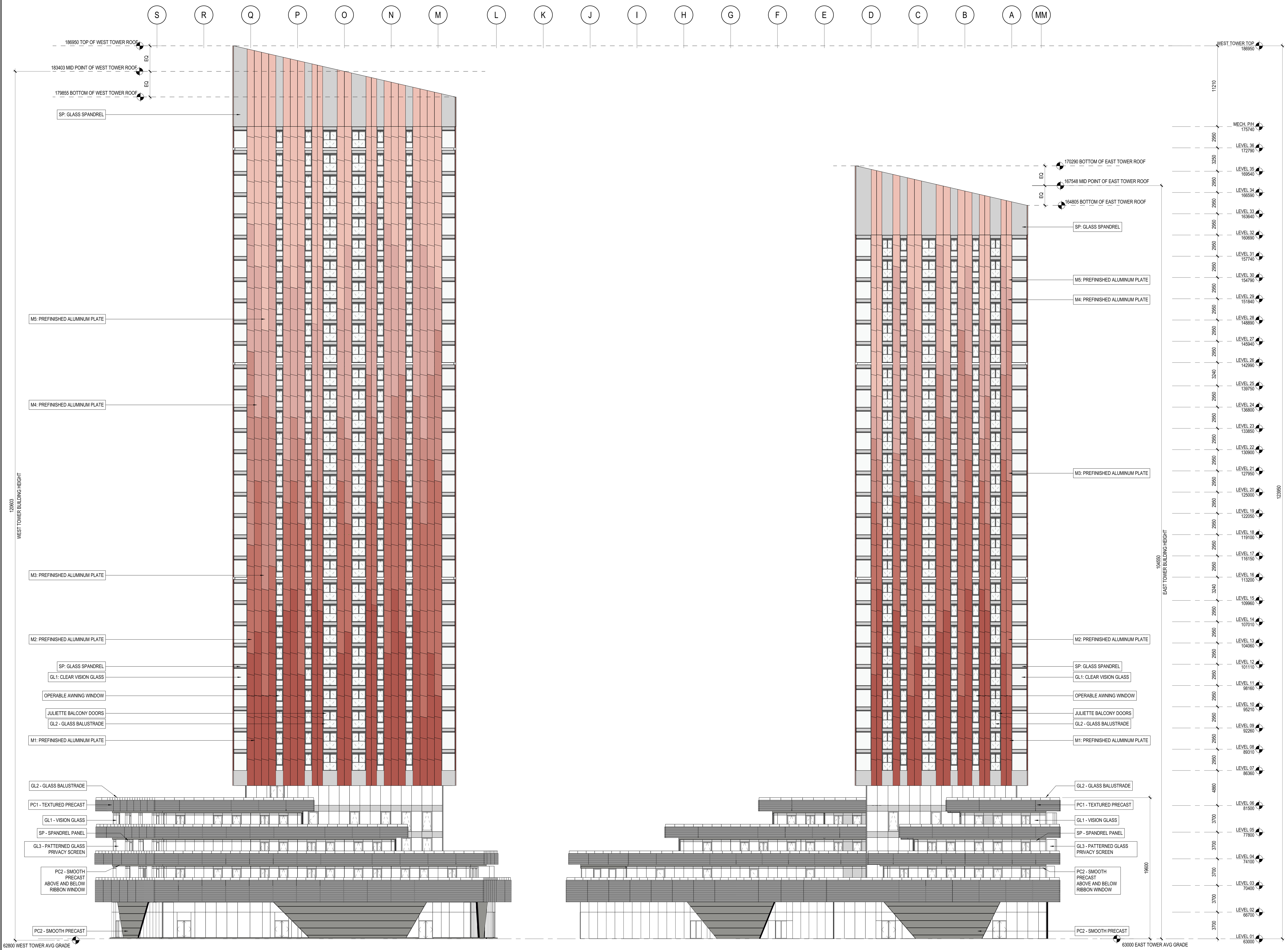

1 SOUTH ELEVATION
1 : 200

ELEVATIONS LEGEND

M1	PREFINISHED ALUMINUM PLATE (COLOUR 1)
M2	PREFINISHED ALUMINUM PLATE (COLOUR 2)
M3	PREFINISHED ALUMINUM PLATE (COLOUR 3)
M4	PREFINISHED ALUMINUM PLATE (COLOUR 4)
M5	PREFINISHED ALUMINUM PLATE (COLOUR 5)
PC1	TEXTURED PRECAST DARK LIMESTONE COLOUR
PC2	SMOOTH PRECAST DARK LIMESTONE COLOUR
SP	SPANDREL PANEL (REFER TO NOTE 1 FOR EXTENT OF BIRD-FRIENDLY FRIT)
GL1	VISION GLAZING (REFER TO NOTE 1 FOR EXTENT OF BIRD-FRIENDLY FRIT)
GL2	GLASS BALUSTRADE
GL3	PATTERNED GLASS PRIVACY SCREEN
LV	PREFINISHED STORM-RESISTANT LOUVER

NOTE

1. BIRD-FRIENDLY VISUAL MARKERS (5MM DIAM. DARK GREY DOTS SPACED 50MM APART) TO BE ETCHED ON FACE 1 OF AT LEAST 90% OF ALL EXTERIOR TRANSPARENT GLASS PANELS TO HEIGHT OF 16M ABOVE GRADE AND / OR 4M ABOVE GREEN ROOFS AND TERRACES, AND AT ALL GLASS CORNERS.

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TRUE NORTH

ISSUE CHART

1	ISSUED FOR OPA / ZBA / SPA	2022-04-22
MARK	ISSUE	DATE

Job Number

TITLE

OVERALL SOUTH ELEVATION

SHEET NUMBER

A20-02

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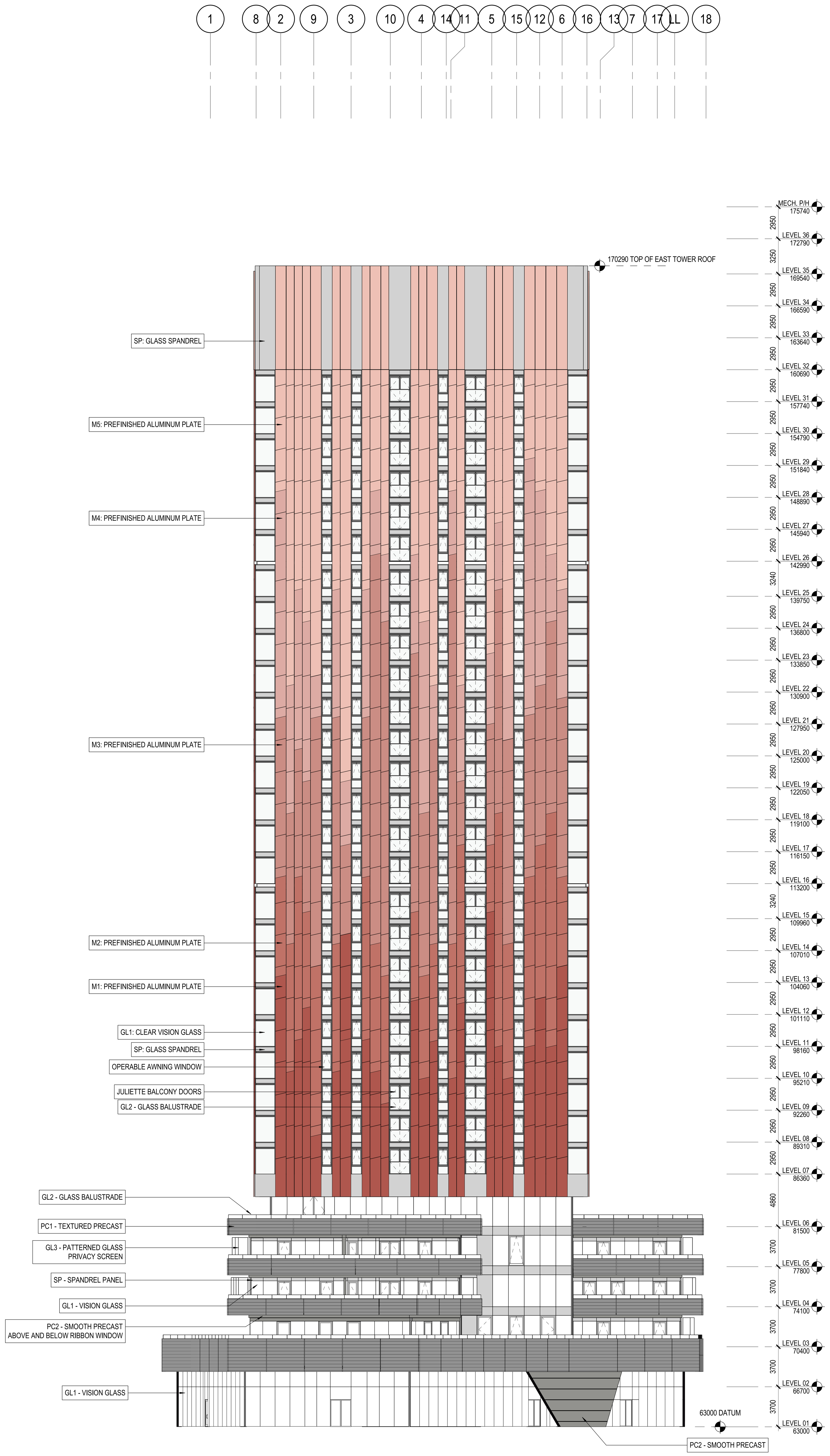
ELEVATIONS LEGEND

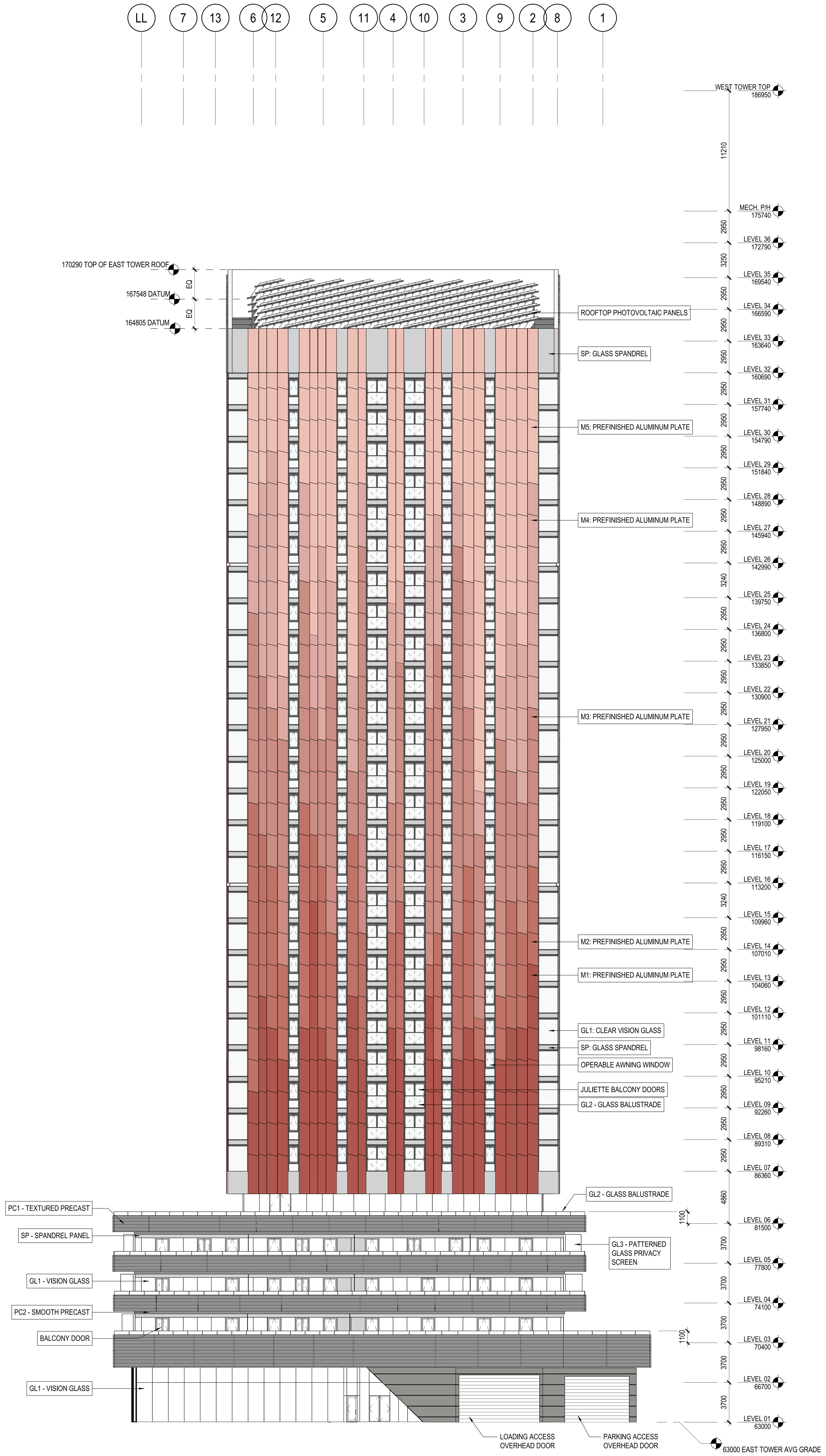
M1	PREFINISHED ALUMINUM PLATE (COLOUR 1)
M2	PREFINISHED ALUMINUM PLATE (COLOUR 2)
M3	PREFINISHED ALUMINUM PLATE (COLOUR 3)
M4	PREFINISHED ALUMINUM PLATE (COLOUR 4)
M5	PREFINISHED ALUMINUM PLATE (COLOUR 5)
PC1	TEXTURED PRECAST DARK LIMESTONE COLOUR
PC2	SMOOTH PRECAST DARK LIMESTONE COLOUR
SP	SPANDREL PANEL (REFER TO NOTE 1 FOR EXTENT OF BIRD-FRIENDLY FRIT)
GL1	VISION GLAZING (REFER TO NOTE 1 FOR EXTENT OF BIRD-FRIENDLY FRIT)
GL2	GLASS BALUSTRADE
GL3	PATTERNED GLASS PRIVACY SCREEN
LV	PREFINISHED STORM-RESISTANT LOUVER

NOTE

1. BIRD-FRIENDLY VISUAL MARKERS (5MM DIAM. DARK GREY DOTS SPACED 50MM APART) TO BE ETCHED ON FACE 1 OF AT LEAST 90% OF ALL EXTERIOR TRANSPARENT GLASS PANELS TO HEIGHT OF 16M ABOVE GRADE AND / OR 4M ABOVE GREEN ROOFS AND TERRACES, AND AT ALL GLASS CORNERS.



① EAST BUILDING - WEST ELEVATION
1 : 200



② EAST BUILDING - EAST ELEVATION
1 : 200

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dream

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KEYPLAN

TRUE NORTH

ISSUE CHART

No.	ISSUE	DATE
1	ISSUED FOR OPA / ZBA / SPA	2022-04-22

Job Number

TITLE

EAST BUILDING ELEVATIONS

SHEET NUMBER

A20-03

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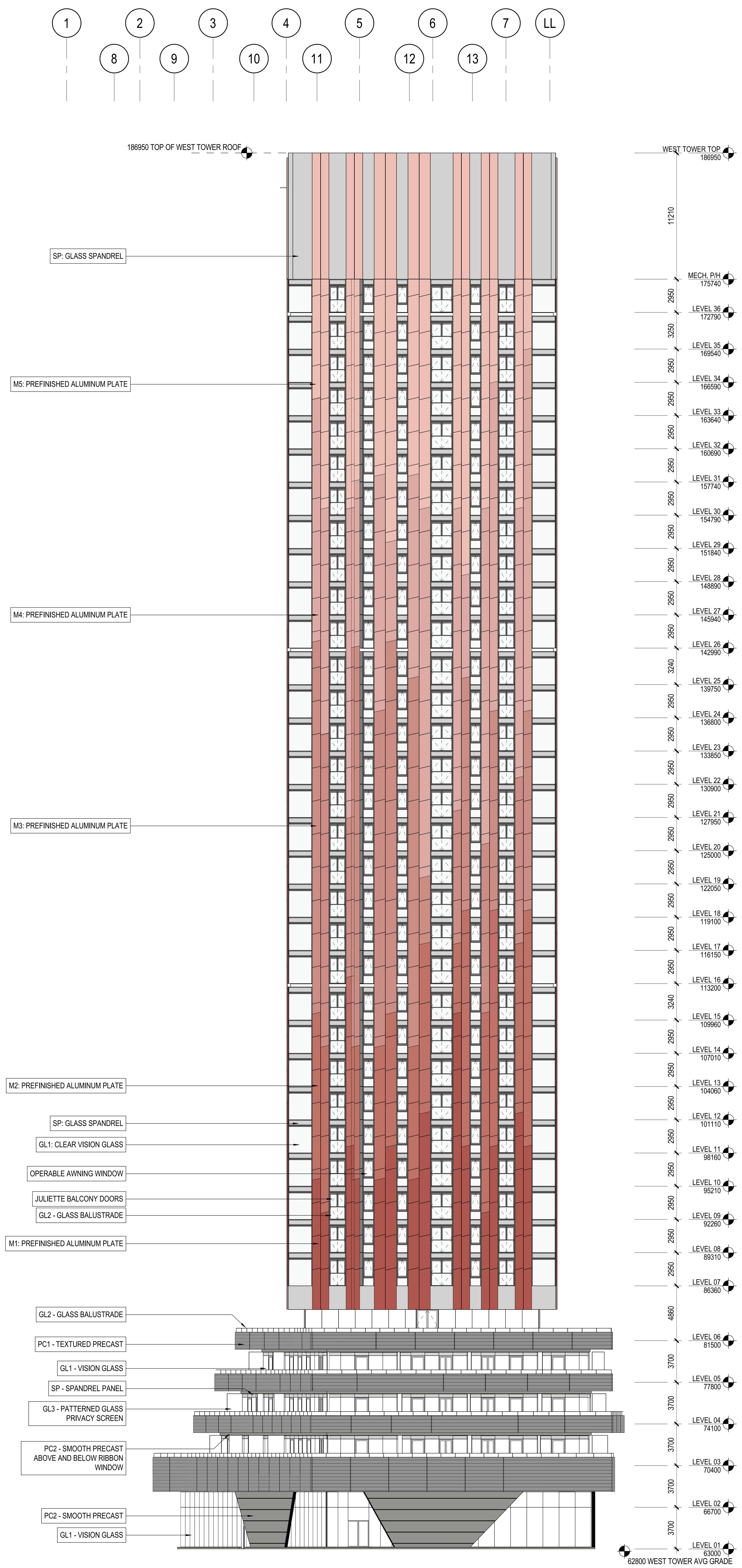
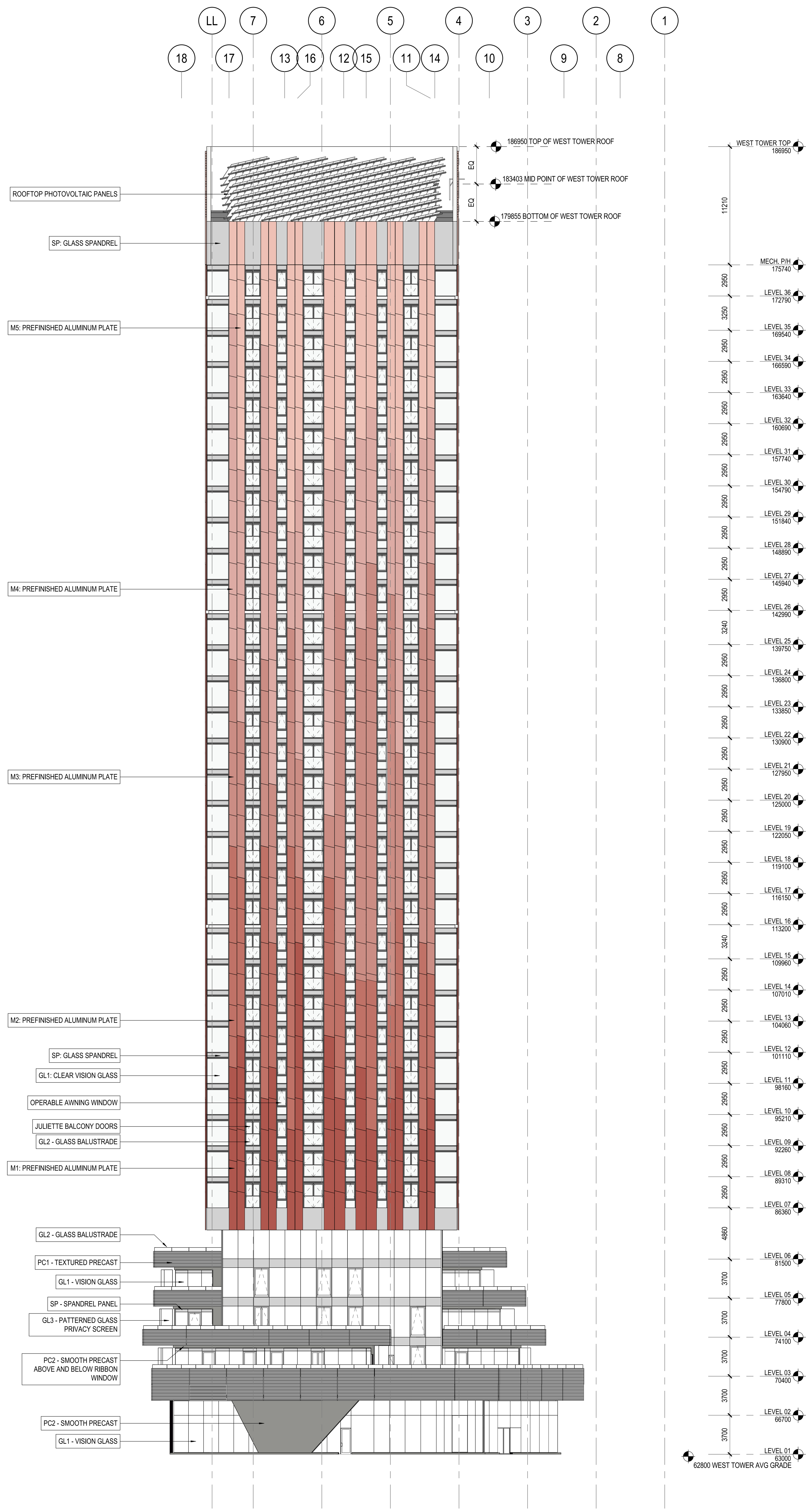


1 WEST BUILDING - WEST ELEVATION
1 : 200



2 WEST BUILDING - EAST ELEVATION
1 : 200

ELEVATIONS LEGEND

M1	PREFINISHED ALUMINUM PLATE (COLOUR 1)
M2	PREFINISHED ALUMINUM PLATE (COLOUR 2)
M3	PREFINISHED ALUMINUM PLATE (COLOUR 3)
M4	PREFINISHED ALUMINUM PLATE (COLOUR 4)
M5	PREFINISHED ALUMINUM PLATE (COLOUR 5)
PC1	TEXTURED PRECAST DARK LIMESTONE COLOUR
PC2	SMOOTH PRECAST DARK LIMESTONE COLOUR
SP	SPANDREL PANEL (REFER TO NOTE 1 FOR EXTENT OF BIRD-FRIENDLY FRIT)
GL1	VISION GLAZING (REFER TO NOTE 1 FOR EXTENT OF BIRD-FRIENDLY FRIT)
GL2	GLASS BALUSTRADE
GL3	PATTERNED GLASS PRIVACY SCREEN
LV	PREFINISHED STORM-RESISTANT LOUVER

NOTE

1. BIRD-FRIENDLY VISUAL MARKERS (5MM DIAM. DARK GREY DOTS SPACED 50MM APART) TO BE ETCHED ON FACE 1 OF AT LEAST 90% OF ALL EXTERIOR TRANSPARENT GLASS PANELS TO HEIGHT OF 16M ABOVE GRADE AND / OR 4M ABOVE GREEN ROOFS AND TERRACES, AND AT ALL GLASS CORNERS.

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TRUE NORTH

ISSUE CHART

MARK	ISSUE	DATE
1	ISSUED FOR OPA / ZBA / SPA	2022-04-22

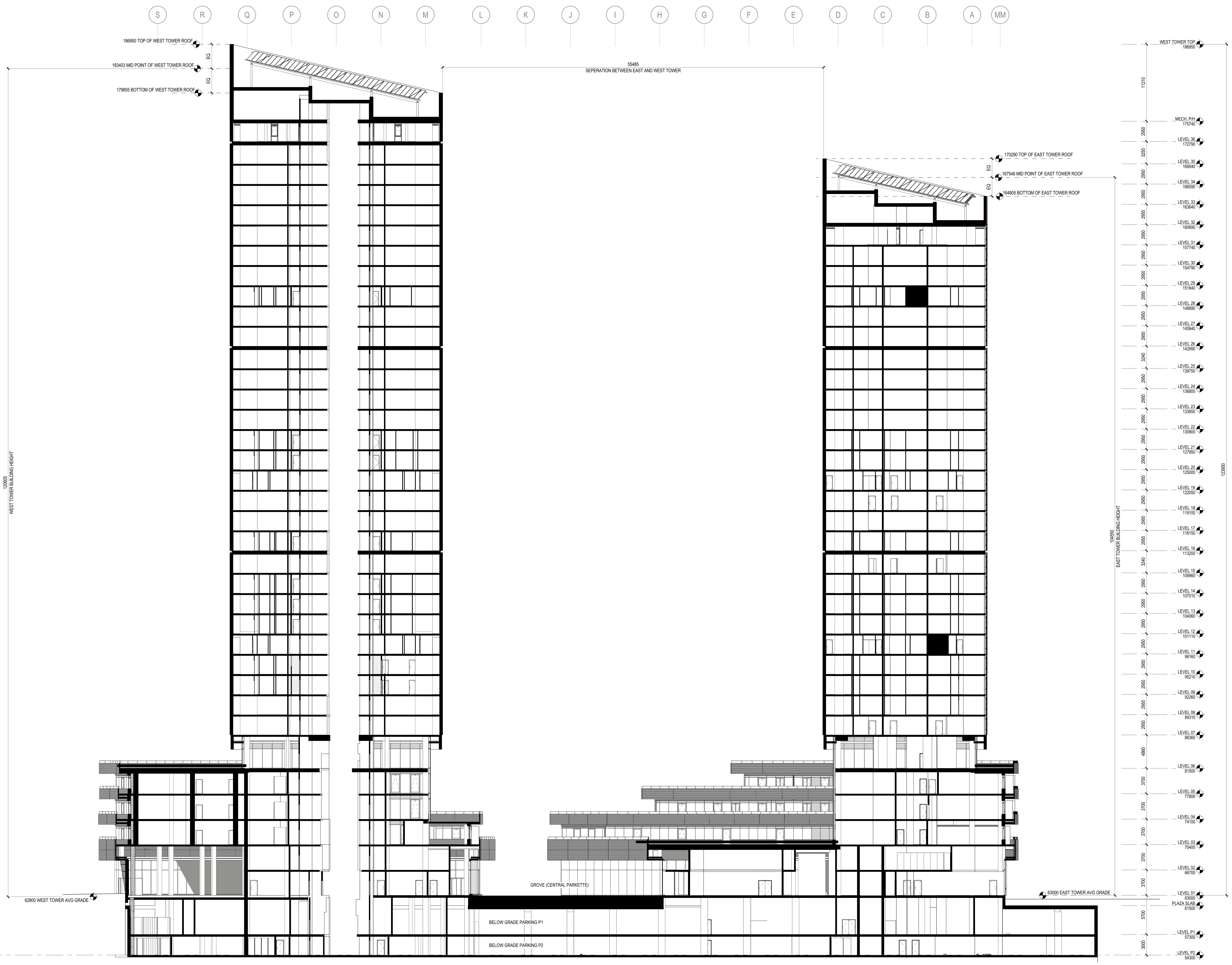
Job Number

TITLE

WEST BUILDING ELEVATIONS

SHEET NUMBER

A20-04



1 OVERALL SECTION
1 : 200

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TRUE NORTH

ISSUE CHART

MARK	ISSUE	DATE
1	ISSUED FOR OPA / ZBA / SPA	2022-04-22

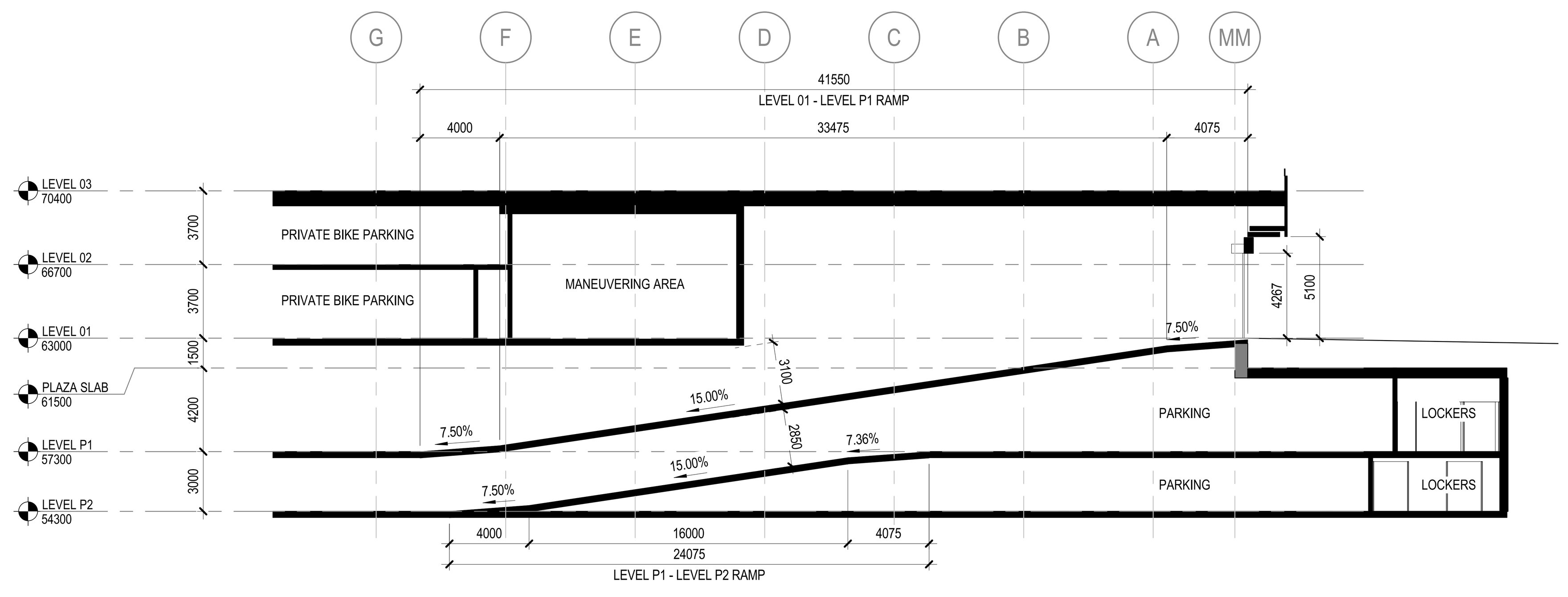
Job Number

TITLE

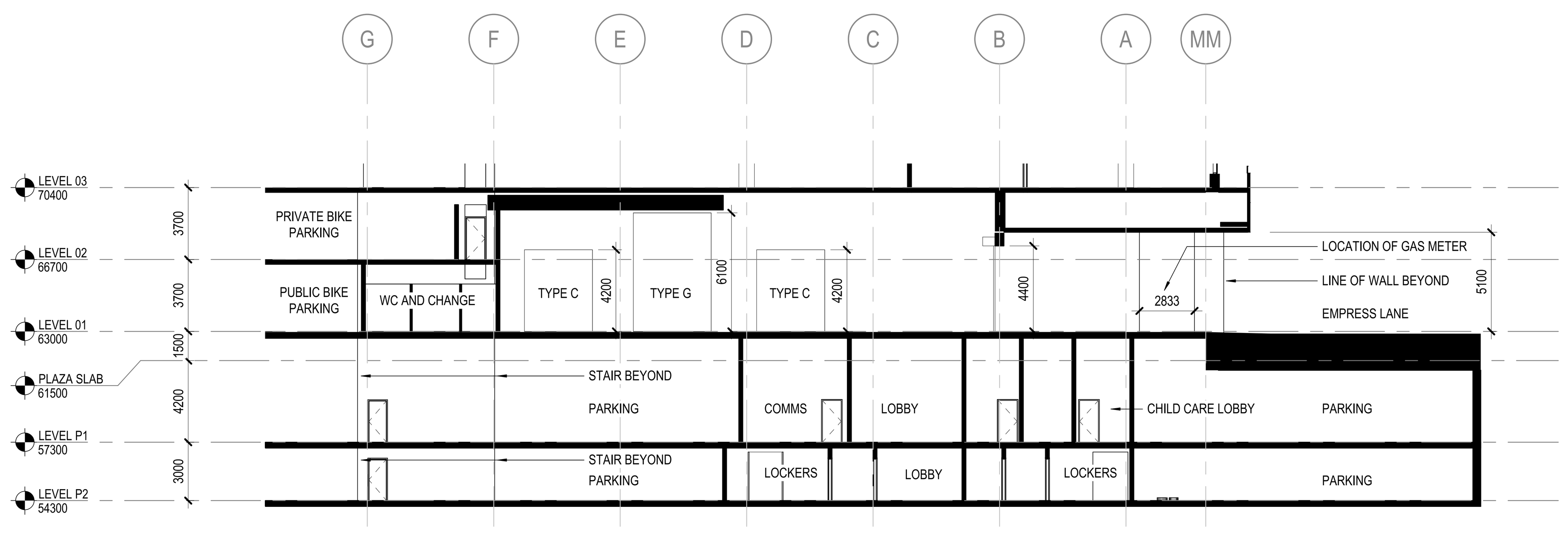
OVERALL BUILDING SECTION

SHEET NUMBER

A21-01



1 PARTIAL SECTION @ PARKING GARAGE RAMP
1 : 200



2 PARTIAL SECTION @ LOADING DOCK ACCESS
1 : 200

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ISSUE CHART

MARK	ISSUE	DATE
1	ISSUED FOR OPA / ZBA / SPA	2022-04-22

Job Number

TITLE

PARTIAL BUILDING SECTIONS

SHEET NUMBER

A21-02

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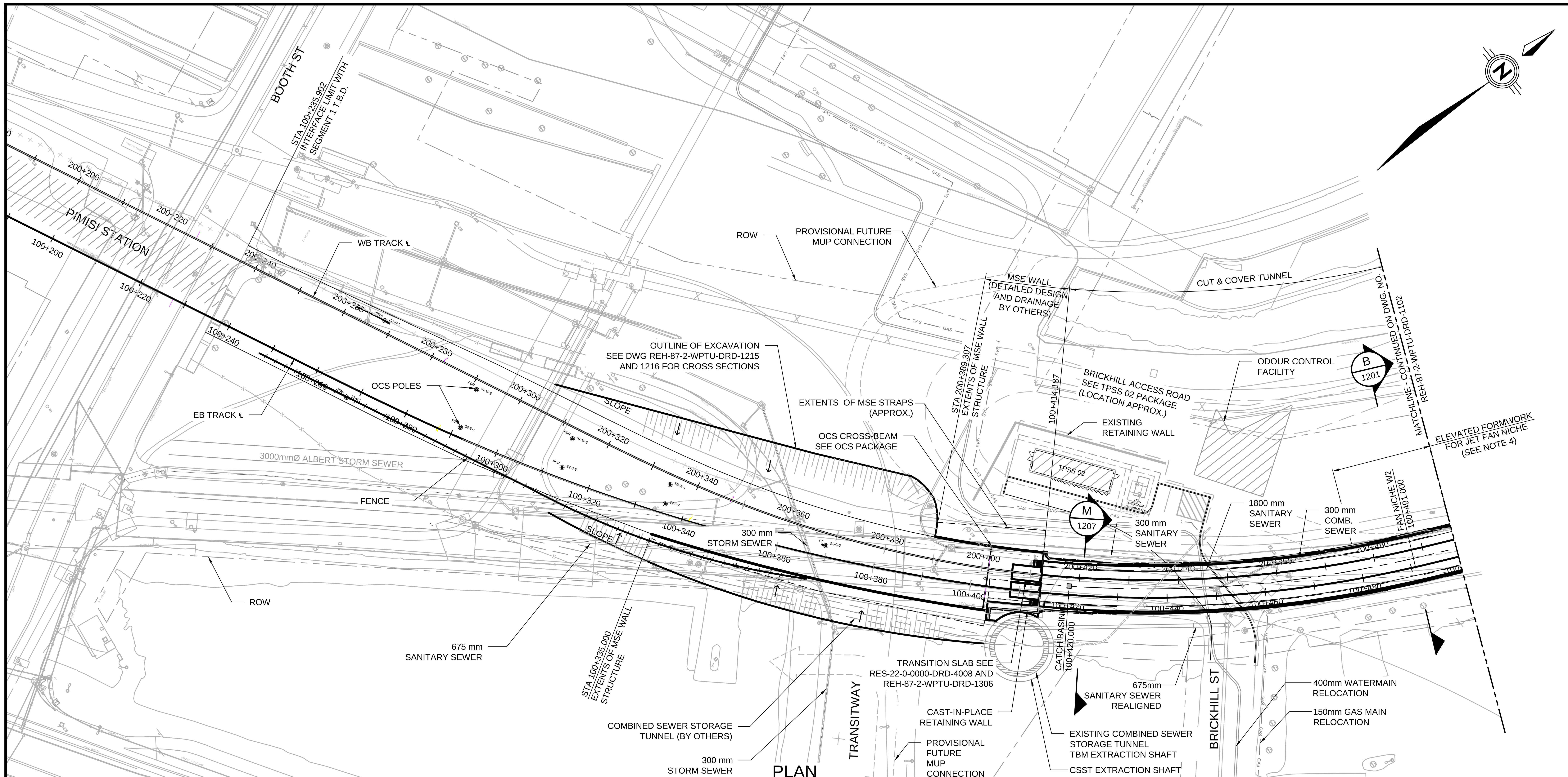
APPENDIX E: As Built Drawings

TITLEBLOCK: 790mm x 534mm RTGE JV 2013

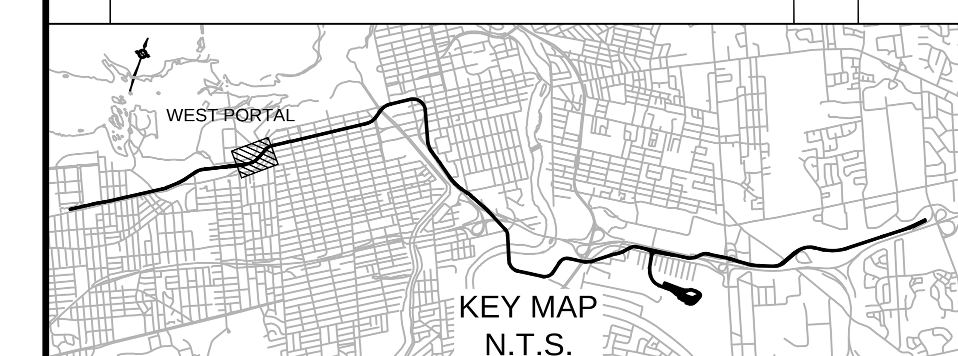


PLAN

SCALE 1:500

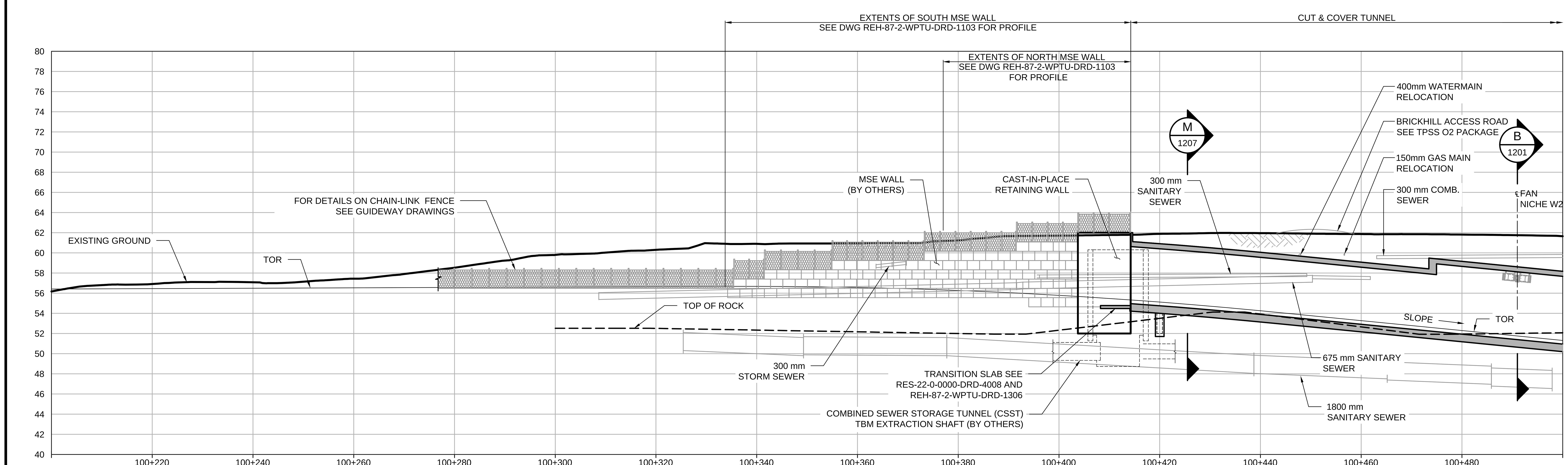
Confederation Line / Ligne de la Confédération	Ottawa
TUNNEL DESIGN WEST PORTAL - STRUCTURAL PLAN AND PROFILE STA 100+235.902 TO 100+500	CONTRACT No. OILC-11-00-P006 DESIGNED: — CHECKED: J. ASPIOTIS DRAWN: V. JUSI SEALED:
DRAWING NUMBER: REH-87-2-WPTU-DRD-1101	PRIMARY SEAL
DESIGN/BUILDER: OLRT CONSTRUCTORS	REFER TO PREVIOUS REVISION FOR THE ORIGINALLY SEALED VERSION
ENGINEERING JV: RTGE Joint Venture	SECONDARY SEAL (IF REQUIRED)
DESIGN FIRM: HATCH	
SCALE: HORIZONTAL 0 5m 10m 20m — 1:500 FULL SIZE, 1:1000 HALF SIZE 40m; VERTICAL 0 2.5m 5m 10m — 1:500 HALF SIZE, 1:250 FULL SIZE 20m	ASSET No. - ASSET GROUP -

REV	DESCRIPTION	BY	DATE
0	SI-0737: ISSUED FOR CONSTRUCTION	JA	2016-05-30
1	SI-0983: ISSUED FOR CONSTRUCTION	JA	2016-09-26
2	AS-BUILT	JA	2019-07-05



KEY MAP
N.T.S.

THIS AS-BUILT DOCUMENT HAS BEEN PREPARED BASED UPON INFORMATION FURNISHED BY OLRT CONSTRUCTORS.



PROFILE - SOUTH WALLS AND EB TRACKS

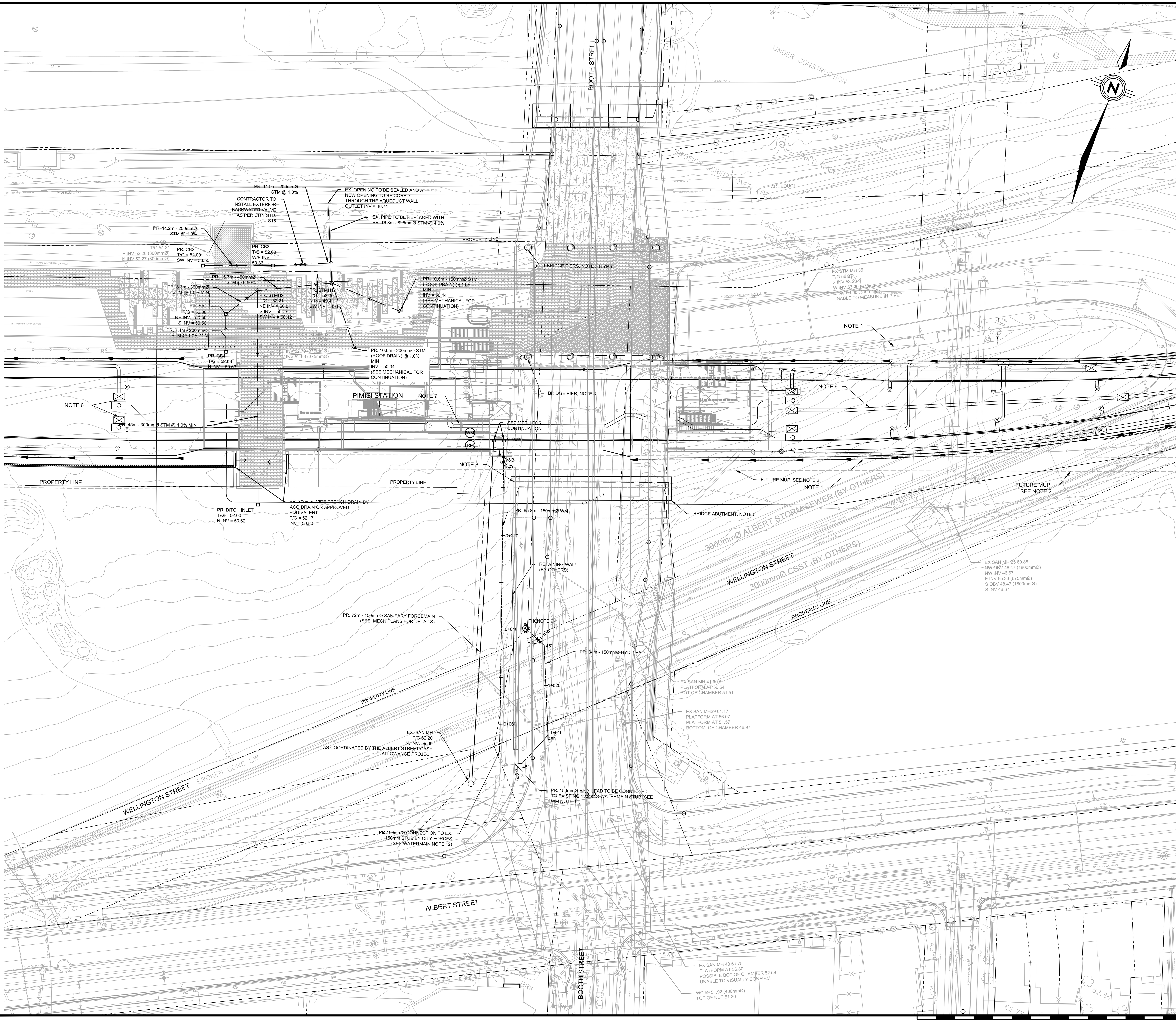
SCALE 1:500 H 1:250 V

NOTES:

1. FOR GENERAL NOTES SEE DRAWING REH-87-2-WPTU-DRD-0030.
2. ONLY MAJOR UTILITIES (300mm AND ABOVE) AND UTILITIES IN THE VICINITY OF THE PORTALS HAVE BEEN SHOWN. FOR ADDITIONAL UTILITY INFORMATION, SEE SEGMENT 1 UTILITIES DRAWINGS.
3. FOR PROPOSED UTILITIES AND UTILITY RELOCATIONS SEE CIVIL DRAWINGS.
4. FOR FAN NICHE PROFILE SEE DRAWING REH-87-2-WPTU-DRD-1202.
5. FOR UTILITY REMOVALS AND RELOCATIONS, SEE REM-30-2-0000-DRD-2731.

BASED ON ALIGNMENT ISSUED 2015-10-19

2019-Jul-15 10:37:52 AM P:\OTTAWA\311921\CAD\DWGS\60_Underground_Works\87_Tunnel_Design\SHEET\REH-87-2-WPTU-DRD-1101.dwg

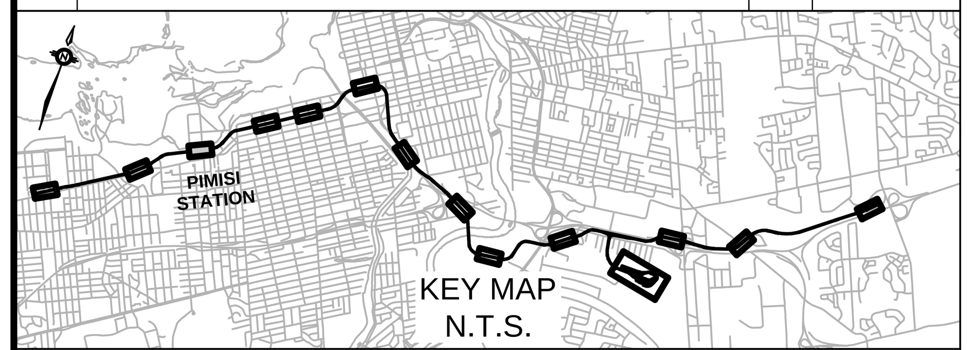


Confederation Line
Ligne de la Confédération

Ottawa

PIMISI STATION CIVIL COMPOSITE UTILITY PLAN	CONTRACT No. OILC-11-00-P006
	DESIGNED: - / CHECKED: -
	DRAWN: REKHA / SEALED: -
DRAWING NUMBER: REM-20-1-LBST-DRD-2800	PRIMARY SEAL
DESIGN/BUILDER: OLRT CONSTRUCTORS CONSTRUCTEURS	Refer to Rev 0 for the originally sealed version
ENGINEERING JV: RTGE Joint Venture	SECONDARY SEAL (IF REQUIRED)
DESIGN FIRM: MMM GROUP	
SCALE: HORIZONTAL 1:800 FULL SIZE, 1:400 HALF SIZE (0 4 8 16 32)	ASSET No. -
	ASSET GROUP -

REV	DESCRIPTION	BY	DATE
0	SI-0463: ISSUED FOR CONSTRUCTION	IJ	2015-12-16
1	AS BUILT	RA	2019-06-14



LAYOUT INFORMATION BASED ON 3 DEGREE MTM ZONE NAD 83 (OLRT) COORDINATE SYSTEM. COMBINED SCALE FACTOR 0.99946

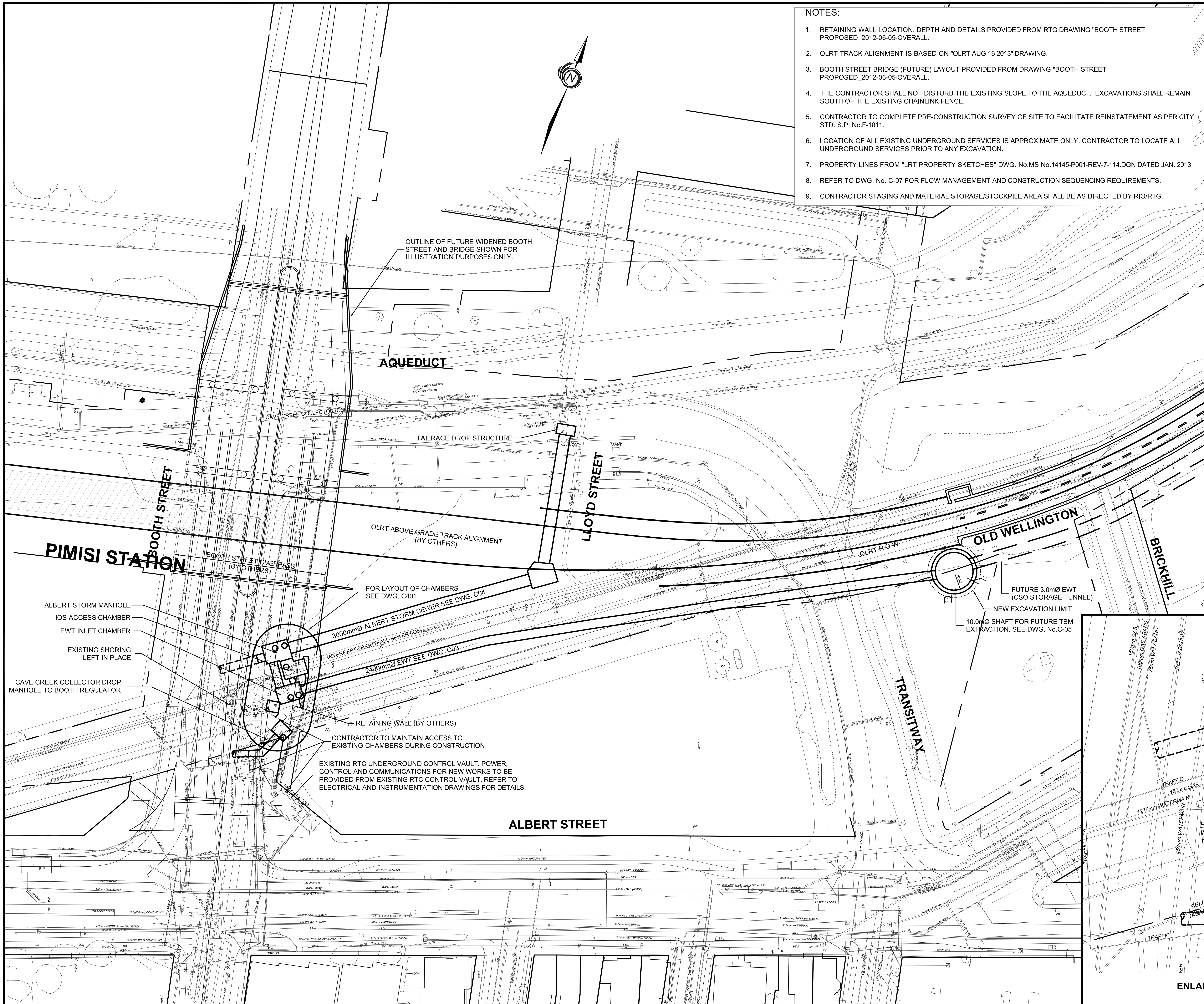
This As-Built Document has been prepared based upon information furnished by OLRT Constructors.

NOTES:

1. REFER TO 'CIVIL - GUIDEWAY DESIGN - SEGMENT 1' FOR UTILITIES RELOCATION, REMOVALS AND NEW CONSTRUCTION.
2. REFER TO THE 'LANDSCAPING - PIMISI STATION' FOR SITE FURNISHING, LANDSCAPING FEATURES AND MUP'S.
3. ROW BASED ON PROPERTY REQUEST PLANS (PRP) PROVIDED BY THE CITY.
4. CONTACTOR TO ENSURE THAT THE EXISTING BRIDGE FOOTING IS NOT UNDERMINED, EXPOSED TO FROST OR OTHERWISE ADVERSELY IMPACTED BY EXCAVATION FOR CONSTRUCTION OF FOOTINGS OR SERVICES. PROVIDE EXCAVATION PLAN IN THE VICINITY OF THE BRIDGE FOOTING FOR REVIEW BY THE BRIDGE ENGINEER
5. REFER TO THE 'POWER SUPPLY AND DISTRIBUTION DESIGN' PACKAGE FOR TPSS AND OCS DETAILS.
6. REFER TO THE 'AT-GRADE DUCTBANK' PACKAGE FOR GUIDEWAY DUCTBANK DETAILS.
7. REFER TO 'ELECTRICAL - PIMISI STATION' FOR ELECTRICAL DESIGN.
8. REFER TO 'STRUCTURES - BOOTH STREET BRIDGE' FOR FOUNDATION LAYOUT AND ABUTMENT FOOTING DETAILS.
9. REFER TO '400mm WATERMAIN RELOCATION BRICKHILL - SEGMENT 1' FOR WATERMAIN RELOCATION DESIGN.
10. THE SEAL ABOVE CERTIFY THAT THE VERTICAL CLEARANCES BETWEEN THE THIRD PARTY UTILITIES AND CIVIL UTILITIES HAVE BEEN REASONABLY COORDINATED. THE SEAL IS NOT INTENDED TO WARRANT THAT THE ENTITIES HAVE BEEN DESIGNED IN ACCORDANCE WITH THE CONTRACT REQUIREMENTS. REFER TO INDIVIDUAL UTILITY DRAWINGS FOR SUCH INFORMATION AND WARRANTS.

TITLEBLOCK: 790mm x 534mm RTGE JV 2013

2019-Jun-17 12:06:00 PM O:\Drawings\WIP As-Built\SUB-01047 Pimisi Station - Civil\Submitted\CAD\2800 to 01\REM-20-1-LBST-DRD-2800 to 01.dwg

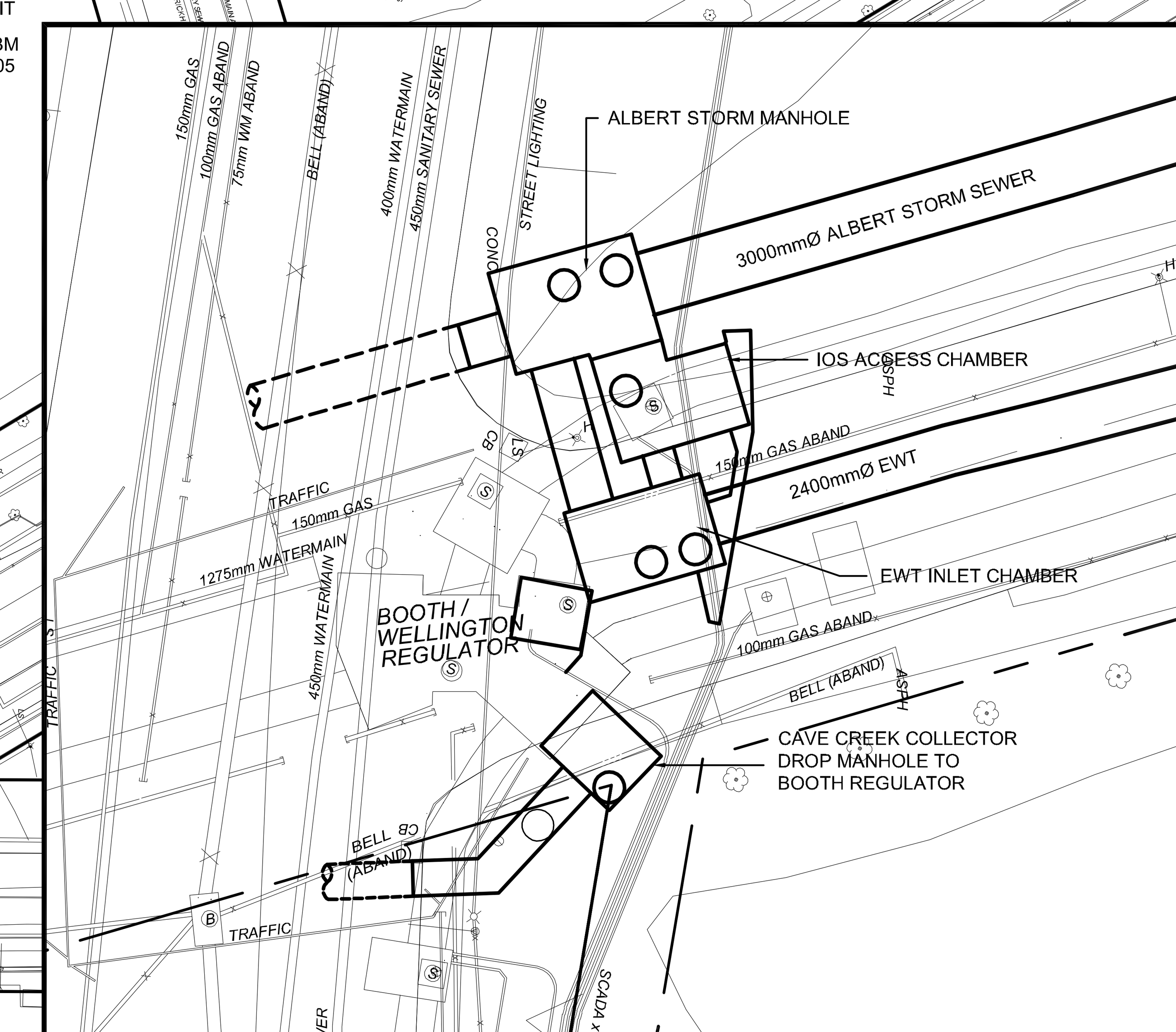


NOTES:

1. RETAINING WALL LOCATION, DEPTH AND DETAILS PROVIDED FROM RTG DRAWING "BOOTH STREET PROPOSED_2012-06-05-OVERALL.
2. OLRT TRACK ALIGNMENT IS BASED ON "OLRT AUG 16 2013" DRAWING.
3. BOOTH STREET BRIDGE (FUTURE) LAYOUT PROVIDED FROM DRAWING "BOOTH STREET PROPOSED_2012-06-05-OVERALL.
4. THE CONTRACTOR SHALL NOT DISTURB THE EXISTING SLOPE TO THE AQUEDUCT. EXCAVATIONS SHALL REMAIN SOUTH OF THE EXISTING CHAINLINK FENCE.
5. CONTRACTOR TO COMPLETE PRE-CONSTRUCTION SURVEY OF SITE TO FACILITATE REINSTATEMENT AS PER CITY STD. S.P. No.F-1011.
6. LOCATION OF ALL EXISTING UNDERGROUND SERVICES IS APPROXIMATE ONLY. CONTRACTOR TO LOCATE ALL UNDERGROUND SERVICES PRIOR TO ANY EXCAVATION.
7. PROPERTY LINES FROM "LRT PROPERTY SKETCHES" DWG. No.MS No.14145-P001-REV-7-114.DGN DATED JAN. 2013
8. REFER TO DWG. No. C-07 FOR FLOW MANAGEMENT AND CONSTRUCTION SEQUENCING REQUIREMENTS.
9. CONTRACTOR STAGING AND MATERIAL STORAGE/STOCKPILE AREA SHALL BE AS DIRECTED BY RIO/RTG.

AS BUILT

THIS DRAWING WAS PREPARED FROM INFORMATION PROVIDED BY THE CONTRACTOR. THE ENGINEER DOES NOT TAKE RESPONSIBILITY FOR INFORMATION ON THIS DRAWING THAT IS IN CONFLICT WITH THE EXISTING CONDITION DUE TO INACCURATE OR MISSING INFORMATION ON SOURCE DOCUMENTS PROVIDED.



ENLARGEMENT SHOWING PROPOSED CHAMBERS

LEBRETON FLATS DIVERSION CHAMBER

Ottawa

Contract No. ISD13-2007 | Dwg. No. C-02

Sheet of

Asset No.

SITE PLAN

W. Newell, P. Eng.
General Manager, Infrastructure Services

R. Dempsey, P. Eng.
Project Manager, Infrastructure Services

Asset Group

Des.	C.G.	Chk'd.	S.D.
Dwn.	E.C.	Chk'd.	C.G.
Utility Circ. No.		Index No.	

Const. Inspector

Scale: HORIZONTAL 500
0m 5 10 20

NOTE:
The location of utilities is approximate only, the exact location should be determined by consulting the municipal authorities and utility companies concerned. The contractor shall prove the location of utilities and shall be responsible for adequate protection from damage.

Stantec
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Ottawa ON Canada
K2C 3G4
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Fax. 613.722.2799
www.stantec.com

REVISIONS

No.	Description	By	Date (dd/mm/yy)
0	ISSUED FOR PRELIMINARY DESIGN	CG/SD	25/10/13
1	ISSUED FOR 90% DESIGN	CG/SD	22/11/13
2	ISSUED FOR TENDER	CG/SD	20/12/13
3	ISSUED FOR CONSTRUCTION	C.G.	23/05/14
4	ISSUED FOR SCM #18	J.R.	04/12/14
5	ISSUED FOR SCM #40	J.R.	04/06/15
6	ISSUED FOR SCM #40R1	J.R.	09/06/15
7	AS BUILT	J.R.	10/19/18