July 5, 2018 Project No. 1668958

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ADDENDUM NO. 1 – GEOTECHNICAL INVESTIGATION
CITY PARK RESIDENTIAL – PHASE 2
OTTAWA (FORMER GLOUCESTER), ONTARIO

Dear Mr. Calvé

This letter serves as an addendum to, and provides additional information and clarifications to, Golder Associates Ltd.'s (Golder's) geotechnical report numbered 1522569, titled "*Detailed Design Geotechnical Investigation, Proposed Gloucester SilverCity, Residential Intensification, City Park Drive, Ottawa, Ontario*", dated November 2015. In this regard, this letter should be read in conjunction with the contents of the original geotechnical report including the "Important Information and Limitations" document included as part of that report.

The recommendations presented in the original geotechnical report were based on the understanding that the proposed residential high-rise towers would only have one level of basement. Following the submittal of our geotechnical report, it is understood that the Phase 2 Tower will have three levels of basements, which is two levels deeper than initially assumed. In addition, a temporary retaining structure will be required to support the front main drive isle during excavation and construction. This letter provides additional geotechnical recommendations to reflect the design changes and construction requirements.

Project Description

Based on the most recent architectural drawings provided to Golder by Hobin Architecture Inc. (Hobin), which are attached to this letter, the building footprint of the Phase 2 Tower will measure about 74 metres by 35 metres in plan area. The tower will be about 23 stories in height, and will have three levels of basement with a slab-on-grade at about elevation 68.4 metres. The base of the excavation will extend a further 1.2 metres to about elevation 67.2 metres to accommodate the basement floor slab, granular base, under-slab services and foundations.

As shown on the attached drawing (SKA-001), on the east side of the Phase 2 Tower, the foundation wall of the structure will abut the foundation wall of the existing Phase 1 Tower (which was constructed in 2017) to about elevation 71.8 metres. Below that elevation, the Phase 2 Tower foundation wall will extend to the base of excavation (67.2 metres) and will abut the excavated bedrock face.

Also shown on the attached drawing (SKA-002), due to the limited space on the north side of the Phase 2 Tower, a temporary retaining structure will be required in this area to support the front main drive isle during construction excavation.

A grade raise of about 2 metres is currently proposed at the north entrance side of the new tower, with the finished ground floor and exterior grade at about elevation 77.3 metres. On the south side of the building, the final exterior grades will be at about elevation 74.4 metres. At grade exterior parking areas will be provided on the north and south sides of the new building.

The geotechnical recommendations and guidance provided in the November 2015 Golder geotechnical report are still valid for the Phase 2 tower. Due to the deeper basement level proposed for the Phase 2 tower, the following additional recommendations are provided to those provided in the original report.

Excavations

The bedrock surface was encountered at about elevation 72.5 metres in borehole 15-01, about elevation 72.6 metres in borehole 15-02, and about elevation 73.2 metres in borehole 15-03, in the area of the Phase 2 tower. Considering that the base of excavation will extend to about elevation 67.2 metres, the bulk excavation will extend through the fill and glacial till, where present, and to about 5 to 6 metres below the expected bedrock surface.

No unusual problems are anticipated with excavating in the overburden using conventional hydraulic excavating equipment, recognizing that construction debris from previous foundations may be encountered and that boulders should be expected within glacial till, if encountered.

The Occupational Health and Safety Act (OHSA) of Ontario indicates that side slopes in the overburden above the water table could be sloped at a minimum of 1 horizontal to 1 vertical (i.e., Type 3 soils). However, given the construction boundary from the proposed structure, it is expected that shoring of the overburden will be necessary, at least on the north side of excavation. Additional guidelines on temporary shoring or retaining structure are provided in the following section.

Bedrock removal will be required for foundation construction. For shallow depths of excavation, it may be possible to remove the upper weathered and fractured portion of the bedrock, to about 2 to 3 metres depth (at least locally), using large hydraulic excavating equipment. Further bedrock removal within the less fractured bedrock could be accomplished using mechanical methods (such as hoe ramming), although this method may be slow. Such excavations could be carried out by hoe remaining in conjunction with closely spaced line drilling, particularly in areas where the foundation walls will abut the bedrock face. Closely spaced line drilling should also be used in areas where space restrictions onsite require a near vertical bedrock excavation.

The upper 2 to 3 metres of the bedrock is very fractured and weathered (note: the fracturing and weathering reduces with depth), and will not likely stand vertically; it should therefore be planned to slope back this zone of bedrock and/or to stabilize the rock face with shotcrete and rock anchors. Near vertical bedrock walls in the moderately fractured and slightly weathered to fresh shale bedrock will be feasible for the construction period provided the bedrock is protected from drying with shotcrete, and that no potentially unstable pillars or slabs of rock are present. Experienced geotechnical personnel should review the bedrock faces during the excavations and prior to the application of shotcrete to identify any areas of potentially unstable rock that would require rock face stabilization measures, such as rock bolts.

Excavations for the foundations will result in exposure of the shale bedrock to the air. The Billing formation shale bedrock at this site has the potential to swell causing heave when exposed to air. The presence of heat can also accelerate the swelling process. To prevent expansion of the shale and/or reaction with the concrete, the shale must be protected from exposure to oxygen, by covering it with a layer of sulfate resistant concrete or shotcrete (Type HS or HSb cement) as soon as possible but within about 24 hours after exposure. When shale is exposed on the sides of the excavation where the placement of concrete is difficult, the use of shotcrete placed within about 24 hours after exposure could be considered. The use of structural shotcrete (i.e., steel fibre reinforced) will also be required to maintain steep excavation walls within the fractured shale, as discussed above. The risk of the basement floor slab heaving due to swelling of the underlying shale bedrock would be reduced if the time between exposure and placement of the concrete cover is very short (i.e., only a few hours but no longer than 24 hours).

Wherever the foundation walls will be in contact with the bedrock, the excavated rock face should be covered with at least 50 millimetres thick of compressible rigid insulation, along with waterproofing and a drainage system, and the concrete for the foundation walls should also contain sulfate resistant concrete (Type HS or HSb cement). In order to keep the shale bedrock wet and minimize the levels of air exposure, the waterproofing membrane should be placed onto the insulation layer first, followed by the drainage system and then the foundation walls. The waterproofing membrane should be extended along the entire height of the bedrock face, and along the floor of the excavation for a distance of at least 3 metres from the sides of the excavation.

Where the Phase 2 tower excavation will abut the Phase 1 tower foundations, the crest of the near vertical bedrock excavation should be set back at least 3.5 metres from the edge of the footings at the higher elevation level. Although the remaining ledge of bedrock will be protected with a mud slab and shotcrete, it is possible that some air exposure near the crest of the bedrock ledge will occur as a result of the limited groundwater drawdown in proximity to the Phase 2 excavation. Therefore, any link between the Phase 1 tower basement and the Phase 2 tower basement should consist of a structural slab placed on a void form at least 300 millimetres in thickness to mitigate the risk of floor slab heaving as a result of the swelling shale (as is currently shown on the attached drawing SKA-001).

Rock Stabilization

In areas where the bedrock will be excavated at a slope steeper than about 1 horizontal to 1.5 vertical and where significant loading will be imposed near the crest of the rock slope, rock stabilization measures will be required to stabilize the upper weathered zone and to prevent a wedge failure within the rock. It is understood that these areas will include the north side of the excavation where the main drive isle will be built near the rock slope, as wells as on the south side of the excavation where heavy construction traffic will be located in proximity to the steep excavation rock slope. To reduce the amount of additional bedrock fracturing caused by the excavation process, the steep rock slopes should be excavated in combination with closely spaced line drilling.

We recommend the following rock stabilization measures for areas of heavy loading near the crest of the rock slopes on the perimeter of the Phase 2 excavation:

- Installation of 3 metres long 25 M grade 75 dowels at 0.5 meters back from the excavated rock face at a spacing of 1 metre along the rock face. Dowels should be angled at 10 degrees from vertical away from the vertical rock face.
- The toe of the retaining structure should be placed at least 0.75 to 1.0 metre back from the rock face.
- The rock face should be covered with a minimum 75 millimetres thick steel mesh or steel fibre reinforced shotcrete over the upper fractured zone which is about 2 to 3 metres in thickness.
- Installation of 4 metres long 25 M grade 75 threaded rebar rock bolts at 1.5 metres below the crest of the excavated rock face at a horizontal spacing of 2 metres offset midway between the vertical dowels. Rock bolts should be inclined at 15 degrees down from horizontal and should have a domed plate and spherical washer so that the rock bolts can be tensioned to 100 kilonewtons.

The above rock stabilization measures should be carried out prior to the construction of the retaining structure on the north side, and prior to allowing any construction traffic within 4 metres from the crest. Where heavier than standard construction loading will be imposed near the crest of the rock slopes, such as large cranes, an additional set back from the crest of at least 2.5 metres is required in addition to the stabilization measured listed above.

Groundwater Control

The groundwater level was measured at about elevation 72.6 metres in borehole 15-3 during the previous 2015 geotechnical study (measurement taken on November 11, 2015), and at 74.0 metres in November 2016. The bulk excavation on this site will therefore extend about 5 to 7 metres below the measured water table. Any dry structure that extends more than about 0.5 metres below the groundwater table will require special consideration, including shotcrete/concrete cover of the shale within 24 hours of exposure, and the use of waterproofing membranes on the perimeter of the excavation to minimize the level of groundwater drawdown in the area.

Some groundwater inflow from the overburden into the excavation should be expected particularly during wet periods. The very fractured rock, loss of flush water and the total core recoveries less than 100 percent within the upper 2 to 3 metres of bedrock suggests that there are water bearing seams within this zone. The initial groundwater inflow could therefore be significant.

An Environment Activity and Sector Registry (EASR) registration is required for construction dewatering between 50,000 and 400,000 L/day, and a PTTW is required for dewatering greater than 400,000 L/day. Based on available site information and our previous experience during the Phase 1 construction, groundwater inflows to excavations that extend into the bedrock can be handled by pumping from sumps within the floor of the excavation. It is expected that construction dewatering pumping rates for this project will likely be less than 400,000 L/day (i.e., an EASR registration will be required). Due to the deeper excavation for Phase 2, an additional hydrogeological assessment is currently being carried out to evaluate the potential impacts of the temporary and permanent groundwater level lowering as well as the estimated inflow rates. This additional study will also be required to support an EASR registration and the results of the study will be provided in a separate hydrogeological report.

It is important to note that the excavation into the bedrock could extend up to about 5 to 7 metres below the water table, and significant hydrostatic water pressure buildup will occur behind the shotcrete without any groundwater control measures. To prevent the shotcrete from being pushed off the rock faces along the sides of the excavation, some temporary depressurization measures (i.e., wells around the perimeter of the excavation, or horizontal wells drilled through the shotcrete, etc.) will be required to remove the groundwater pressure from behind the shotcrete. The temporary depressurization measures are considered temporary works and remain the contractor's responsibility, but they should be removed and adequately sealed once the foundation walls are in place to allow the surrounding groundwater levels to recover.

Temporary Retaining Structure

As previously noted, a temporary retaining structure will be required to support the front main drive isle (north side of the tower) during construction excavation. The lateral earth pressures acting on the retaining structure will depend on the type and method of placement of the backfill, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

The following recommendations are provided regarding the lateral earth pressures for static (i.e., not earthquake) loading conditions, as it is understood that the structure will be temporary. These lateral earth pressures are based on the assumption of using engineered granular backfill behind the temporary retaining structure and that the slope on top of the structure will be flat. If the inclination of the slope changes, then new lateral earth pressures will need to be calculated.

The retaining structure should be designed to resist lateral earth pressures calculated using a triangular distribution of the stress, which may be determined as follows:

$$\sigma_h(z) = K\,(\gamma\, z + q)$$

Where:	$\sigma_h(z)$	=	Lateral earth pressure at depth 'z' (kPa);
	K	=	Active or at rest earth pressure coefficient (K_a or K_o), see below;
	γ	=	Unit weight of backfill soil (kN/m^3), see below;
	z	=	Depth below top of wall (m);
	h	=	Total height of wall (m); and,
	q	=	Surcharge due to live loads on ground surface above wall (kPa).

If the retaining structure allows lateral yielding (which is expected to be the case for the temporary structure), active earth pressures may be used in the geotechnical design of the structure. The movement to allow active pressures to develop within the compacted granular backfill, and thereby assume an unrestrained structure, may be taken as:

- Rotation of approximately 0.002 about the base of a vertical wall;
- Horizontal translation of 0.001 times the height of the wall; or,
- A combination of both.

The value of the surcharge due to live loading (q) should consider the potential traffic loading above the structure and also the potential construction loads from equipment or materials. A value of no less than 15 kPa would be reasonable.

The following parameters are recommended when using engineered granular backfill behind the retaining structure, and are unfactored:

Material	Granular 'A'	Granular 'B' Type II
Soil Unit Weight:	**21.5 kN/m³**	**22 kN/m³**
Coefficients of static lateral earth pressure: Active, K_a At rest, K_o	 0.30 0.50	 0.30 0.50

It is also recommended that drainage be provided at the toe, behind the retaining structure to prevent hydrostatic pressure from building up behind the temporary retaining structure.

Closure

We trust that this report is sufficient for your present requirements. If you have any questions concerning this report, please feel free to contact the undersigned.

Yours truly,

Golder Associates Ltd.

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CK/NRL/CC/mvrd
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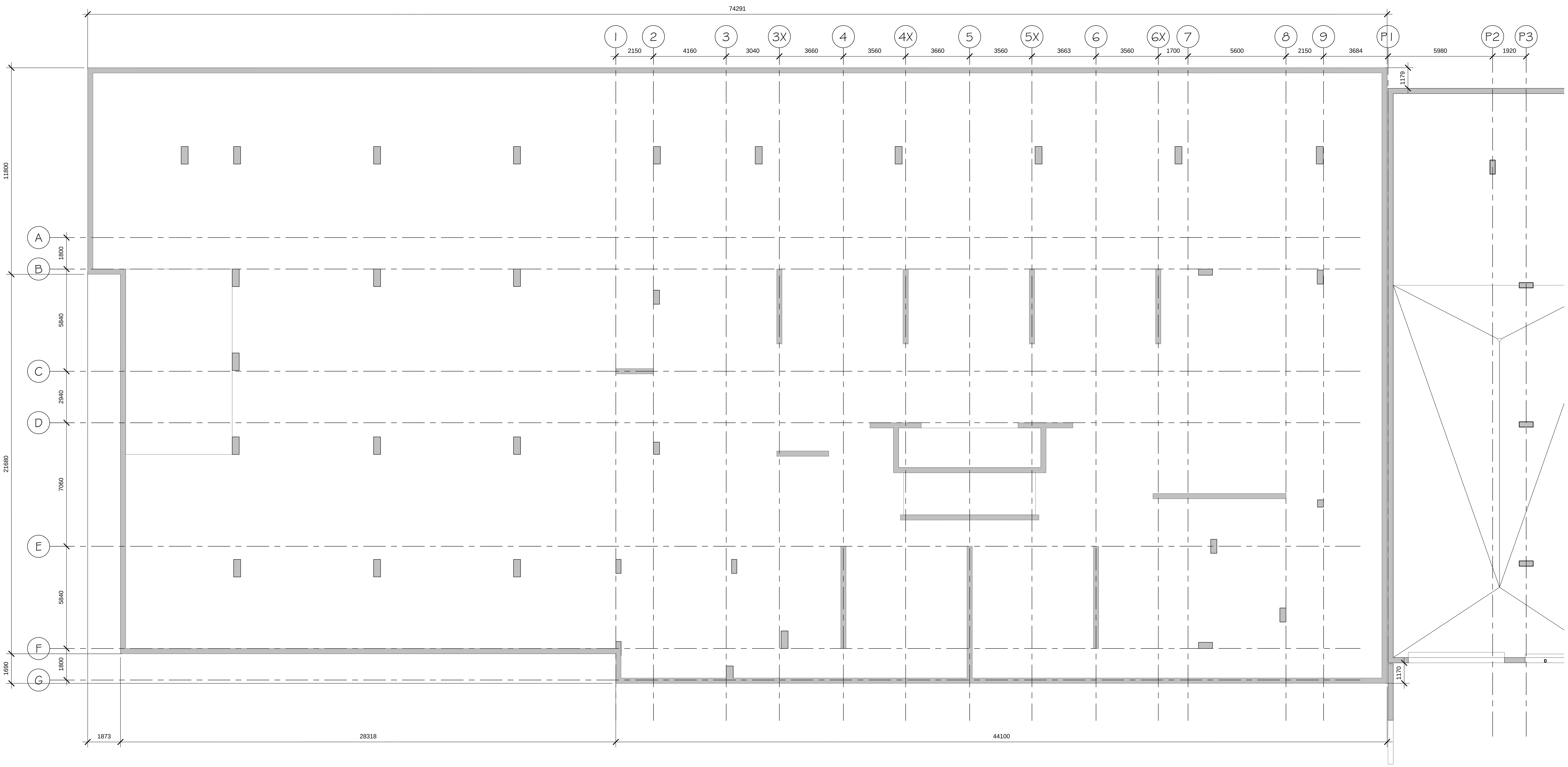
Attachments: *Level P1 to Level P3 Parking* (Drawings Nos. A2.01 to A2.03) by Hobin Architecture dated October 9, 2014

Section Through East Wall of Existing and New Parking Garage (Drawing No. SKA-001) by Hobin Architecture dated June 25, 2018

Section Through North Wall of Parking Garage (Drawing No. SKA-002) by Hobin Architecture dated June 25, 2018



1 BASEMENT LEVEL 1
A2.01 SCALE: 1 : 100

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no.	date (yymmdd)	revision

It is the responsibility of the appropriate contractor to check and verify all dimensions on site and report all errors and/ or omissions to the engineer.

All contractors must comply with all

Do not scale drawings.

This drawing may not be used for construction until signed.

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PROJECT
RIOCAN - CITY PARK PHASE 2
2280 City Park Drive

DRAWING TITLE
LEVEL P1 - PARKING

DRAWN	DATE	SCALE
Author	10/09/14	1 : 100

PROJECT 0001

DRAWING NO. A2.01

REVISION NO.

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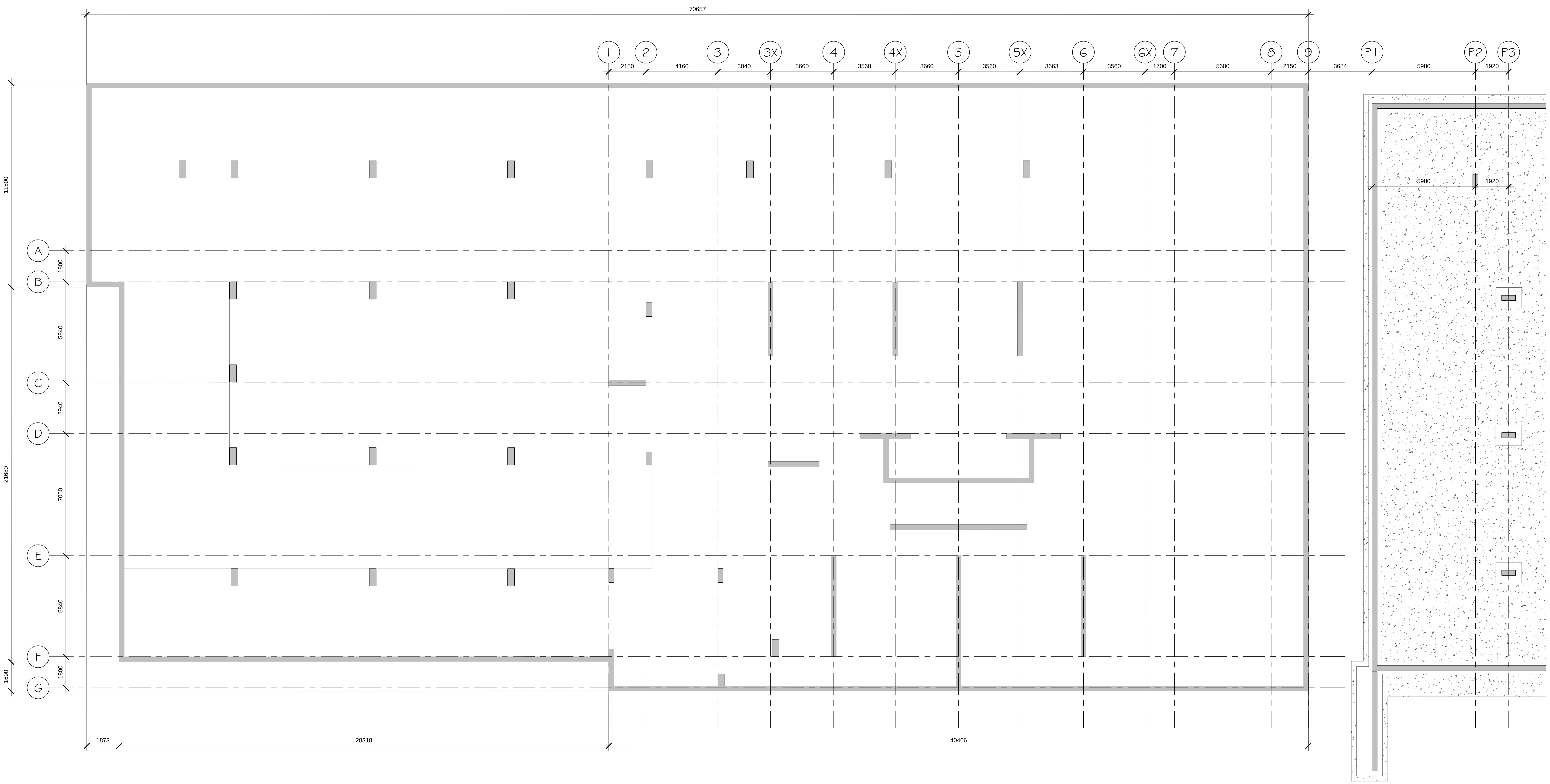

1
A2.02
BASEMENT LEVEL 2
SCALE: 1 : 100

no.	date (yymmdd)	revision

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PROJECT
RIOCAN - CITY PARK PHASE 2
2280 City Park Drive

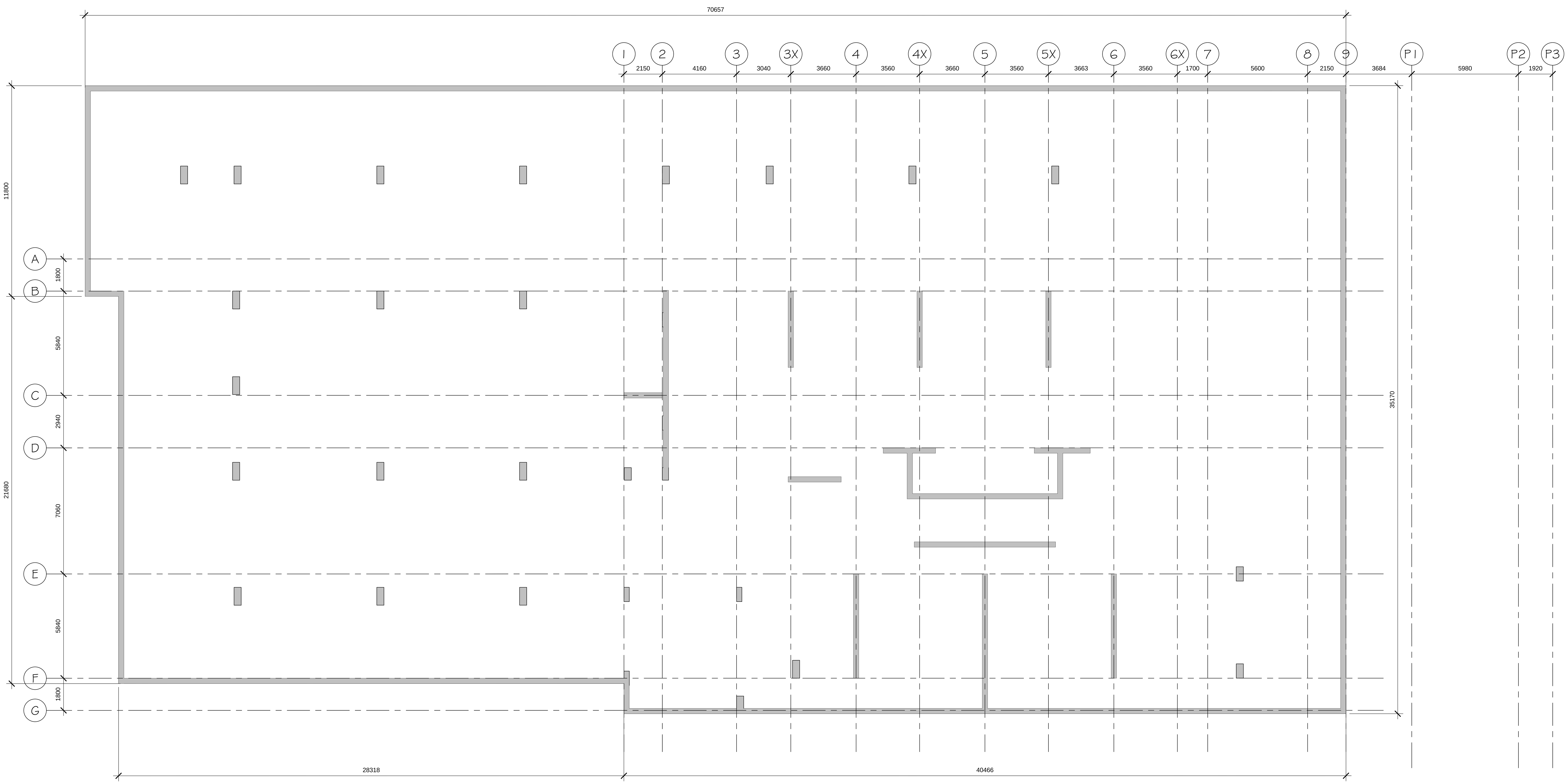
DRAWING TITLE
LEVEL P2 - PARKING

DRAWN	DATE	SCALE
Author	10/09/14	1 : 100

PROJECT
0001

DRAWING NO.
A2.02

REVISION NO.



1 A2.03 BASEMENT LEVEL 3
SCALE: 1 : 100

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no.	date (yymmdd)	revision

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PROJECT
RIOCAN - CITY PARK PHASE 2
2280 City Park Drive

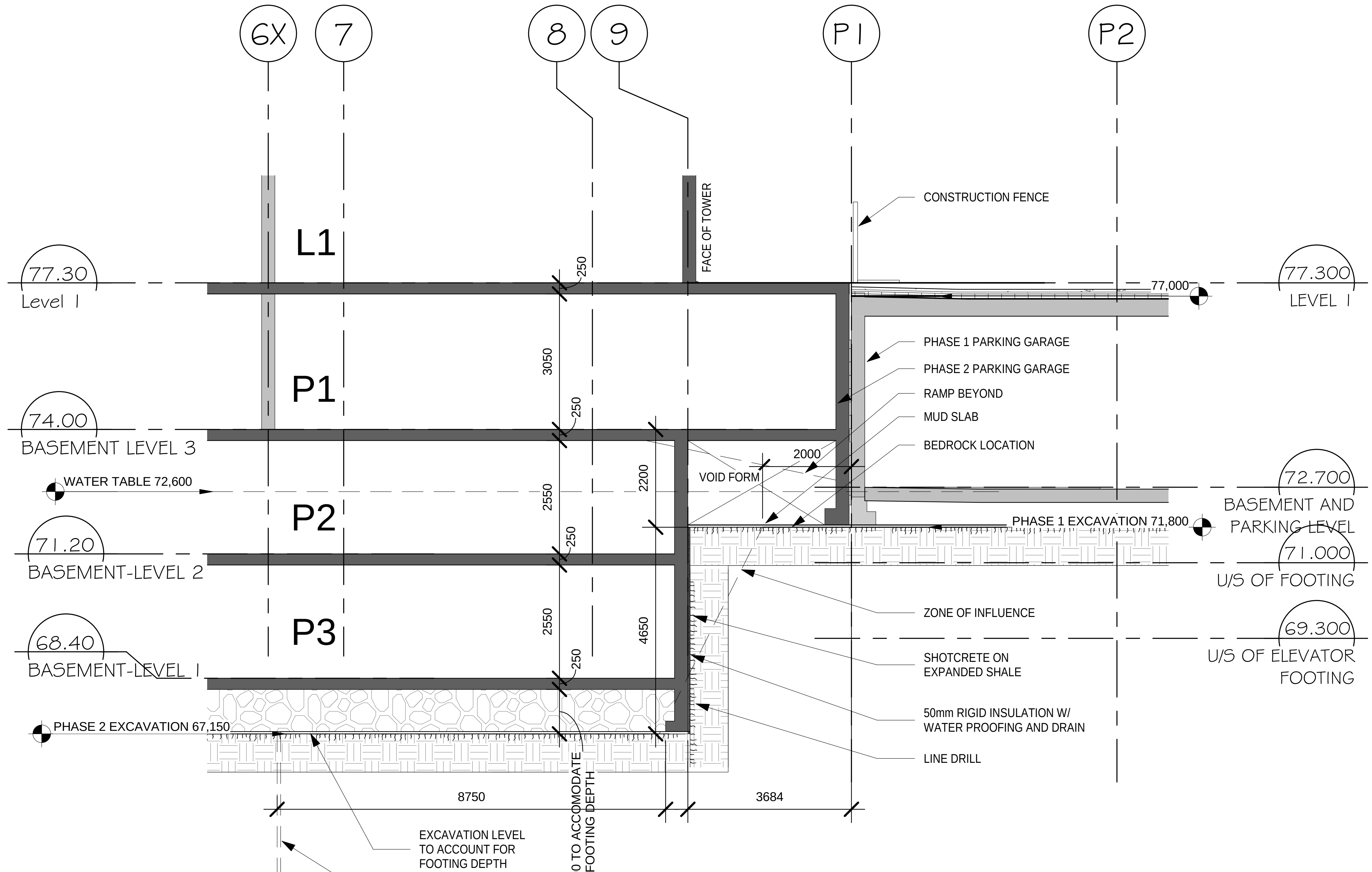
DRAWING TITLE
LEVEL P3 - PARKING

DRAWN	DATE	SCALE
Author	10/09/14	1 : 100

PROJECT
0001

DRAWING NO.
A2.03

REVISION NO.



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SCALE: 1 : 100

ASK No. SKA-001

DATE: 06/25/18

PROJECT NO.: 1759

DRAWING NAME: SECTION THROUGH EAST WALL OF EXISTING AND NEW PARKING GARAGE

PROJECT/LOCATION: RIOCAN - CITY PARK PHASE 2
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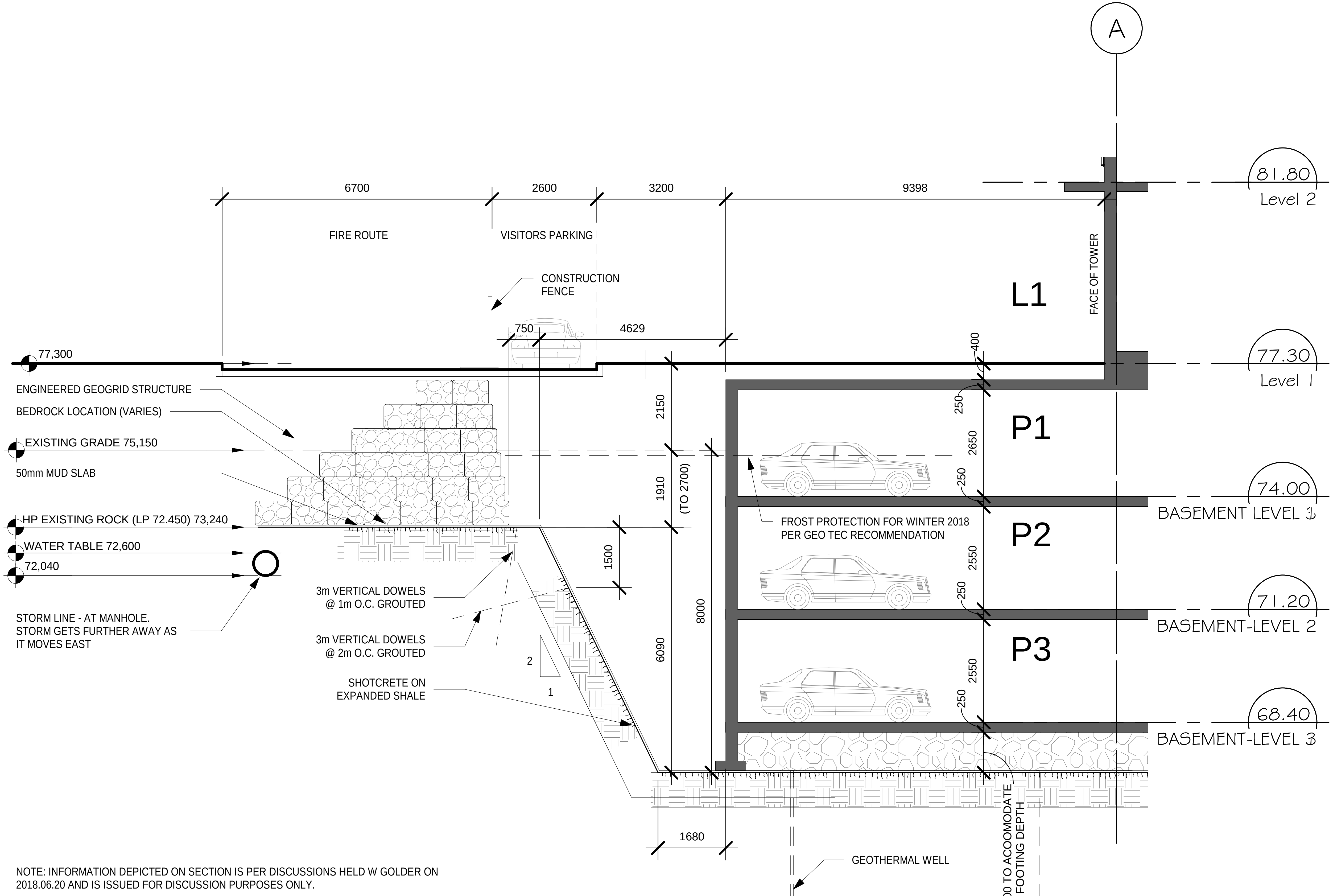
HOBIN ARCHITECTURE



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SCALE: 1 : 100

ASK No. SKA-002

DATE: 06/25/18

PROJECT NO.: 1759

DRAWING NAME: SECTION THROUGH NORTH WALL OF PARKING GARAGE

PROJECT/LOCATION: RIOCAN - CITY PARK PHASE 2
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