December 08, 2025

Project No. CA0062172.9424-Rev0

Andrew Kent, Senior Director, Developments
Killam
3700 Kempt Rd, Suite 100
Halifax NS, B3K 4X8

GEOTECHNICAL DESKTOP AND PAVEMENT DESIGN RECOMMENDATIONS FOR THE PROPOSED NEW SURFACE PARKING AT 2280 CITY PARK DRIVE, OTTAWA, ONTARIO.

Dear Mr. Kent

WSP Canada Inc. (hereinafter referred to as “WSP”), was requested by Killam (the “Client”) to conduct a desktop geotechnical review of previous geotechnical studies completed for the City Park Residential development by Golder Associates Ltd (hereinafter referred to as “Golder”) and to verify that the previous pavement design recommendations are still valid for the proposed surface parking west of Phase 2 Tower (the “Project”), located at 2280 City Park Drive, Ottawa, Ontario (the “Site”).

The ‘Important Information and Limitations of This Report’ which follows should be referenced and forms an integral part of our methodology and findings. This report has been prepared at the request and for the sole use of Killam. The use of this report by any third party, as well as any decisions based upon this report, are under that party’s sole responsibility. WSP shall not be held financially or legally accountable for any possible claims or damages resulting from third-party decisions based on this report.

1 PROJECT UNDERSTANDING

It is understood that the Client is proposing a new surface parking area to accommodate 64 parking spots at the west side of the property to the west and northwest of Phase 2 Tower. The proposed new parking lot location is currently occupied by a grassed area and is not partially or fully underlain by underground parking/basement levels.

It is understood that the City of Ottawa (the “City”) has requested a geotechnical letter to support the proposed parking development application. The letter was requested to confirm whether the pavement design recommendations provided in the previous geotechnical report completed for this Site are valid or need to be revised.

2 SCOPE OF WORK

The scope of geotechnical services addressed in this letter included the following:

- Review of available geotechnical reports, addendums, historical borehole records for the Site including:
  - Geotechnical Report No. 1522569 (10001) “Detailed Design Geotechnical Investigation Proposed Gloucester Silver City Residential Intensification, City Park Drive, Ottawa, Ontario” dated November 2015, by Golder.
  - Geotechnical Addendum No. 1668958 “Addendum No. 1 – Geotechnical Investigation City Park Residential – Phase 2, Ottawa, Ontario” dated July 2018, by Golder.
- Review the Site Plan provided by the Client “Phase 2.1 Site Plan – Additional Parking” dated July 16, 2019 by Hobin Architecture, and
- Preparation of this summary report.

3 DESKTOP REVIEW

A detailed geotechnical investigation report dated November 2015 was completed by Golder for the Site to investigate the subsurface conditions at the site and provide design recommendations for the development including pavement design. A grade raise of about 2 m was proposed at the north entrance side of the Phase 2 tower.

A geotechnical addendum (Addendum No.1) dated July 2018 by Golder was completed to discuss the implications of constructing 3 underground levels within the footprint of Phase 2 tower located at 2280 City Park Dr. Due to the limited space on the north side of the Phase 2 tower and due to the 2 m grade raise at the north entrance, a temporary retaining structure was proposed at that area to support the front main drive isle during construction excavation. It is understood that the grade raise and the temporary retaining structure were limited to the main drive isle and was not extended further towards the west end of the Site which where the new parking is proposed.

3.1 Subsurface Conditions

Based on the original Site Plan included in the geotechnical report, BH15-01 was drilled within proximity of the proposed new parking area. Also, a few historical borehole records including BH9, BH10, E-080 and E-081 were available and presented in the geotechnical report within proximity of the proposed parking area. The general soil stratigraphy at the time of investigation at the Site consists of a flexible pavement underlain by fill, underlain by glacial till over shallow shale bedrock.

3.2 Proposed Pavement Design

The proposed parking pavement structure is designed for properly prepared, proof rolled and approved subgrade. All topsoil, disturbed, or otherwise deleterious materials should be removed from the paved areas. All existing fill at this site should be removed from below paved areas.

Any grade raise to bring the subgrade to the sub-base of the pavement level, acceptable (compactable and inorganic) earth borrow or OPSS Select Subgrade Material meeting the requirements of OPSS.MUNI 1010.

These materials should be placed in loose uniform lifts not thicker than 300 mm and compacted to at least 95% of the standard Proctor maximum dry density (SPMDD) using suitable vibratory compaction equipment.

The surface of the pavement subgrade should be crowned to promote drainage of the pavement granular structure into longitudinal sub drains or sub drain leads that extend at least 3 m from the catch basins.

A heavy-duty pavement structure was proposed for emergency access roadways, and a light-duty pavement structure was given for parking areas. The proposed pavement structure is presented in Table 1 below:

Table 1: Pavement Structure

Material		**Thickness (mm)**	
		Light-Duty for Parking Areas	**Heavy-Duty for Emergency Access Roads**
Surface Course	Superpave 12.5 mm, PG 58-34	50	40
Base Course	Superpave 19 mm, PG 58-34	--	50
Base	OPSS 1010 Granular A	150	150
Subbase	OPSS 1010 Granular B Type II	300	300

The above pavement design is based on the assumption that the pavement subgrade has been acceptably prepared (i.e., where the trench backfill and grade raise fill have been adequately proof-rolled, compacted to the required density and the subgrade surface not disturbed by construction operations or precipitation), and reviewed and approved by a geotechnical engineer.

The base and subbase materials, i.e., Granular A for base and Granular B Type II for subbase, shall conform to OPSS 1010. Both base and sub-base should be placed in maximum 300 mm thick lifts and compacted to 100% SPMDD in accordance with OPSS.MUNI 501 Method A. Asphalt layers should be compacted to comply with OPSS 310. Asphalt layers shall be compacted to minimum 92% and maximum 97% density, 4% air void is ideal. The recommended Superpave 12.5 and 19 can be replaced with HL-3 and HL-8, respectively if desired.

4 DISCUSSION

The pavement design recommendations provided in Golder’s geotechnical report No. 1522569 (10001) is valid for the proposed new parking lot assuming that the new surface parking will not be constructed partially or fully on top of any underground levels.

The recommendations presented in this report are based on the assumption that an adequate level of construction monitoring by qualified geotechnical personnel during construction will be provided. The subgrade should be inspected and approved by experienced geotechnical personnel prior to placing the pavement structure.

In addition, an adequate level of construction monitoring should include laboratory and field tests during construction. This includes compaction testing the grade raise fill, subbase, base and asphalt with laboratory testing for the proposed fill soils for this Site.

5 CLOSURE

Should you have any questions regarding the contents of this report or require additional information, please do not hesitate to contact this office.

WSP Canada Inc.

LICENSED PROFESSIONAL ENGINEER
M. A. T. AL-KHAZAALI
100512672
Dec08,2025
PROVINCE OF ONTARIO

Mohammed Al-Khazaali, Ph.D., P.Eng.
Senior Geotechnical Engineer

MAK/yj

Attachments: Important Information and Limitations on this Report
APPENDIX A – Phase 2.1 Site Plan – Additional Parking
APPENDIX B – Golder's Geotechnical Report No. 1522569 (10001)
APPENDIX C – Previous Golder's Addendum No. 1668958

https://wsponlinecan.sharepoint.com/sites/ca-ca0062172.9424/shared documents/06. deliverables/ca0062172.9424-l-rev0-final geotechnical desktop review_2280 city park drive_2025'12'08.docx

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

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

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

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

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

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

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

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

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

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

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

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

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

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

APPENDIX A

Phase 2.1 Site Plan – Additional Parking

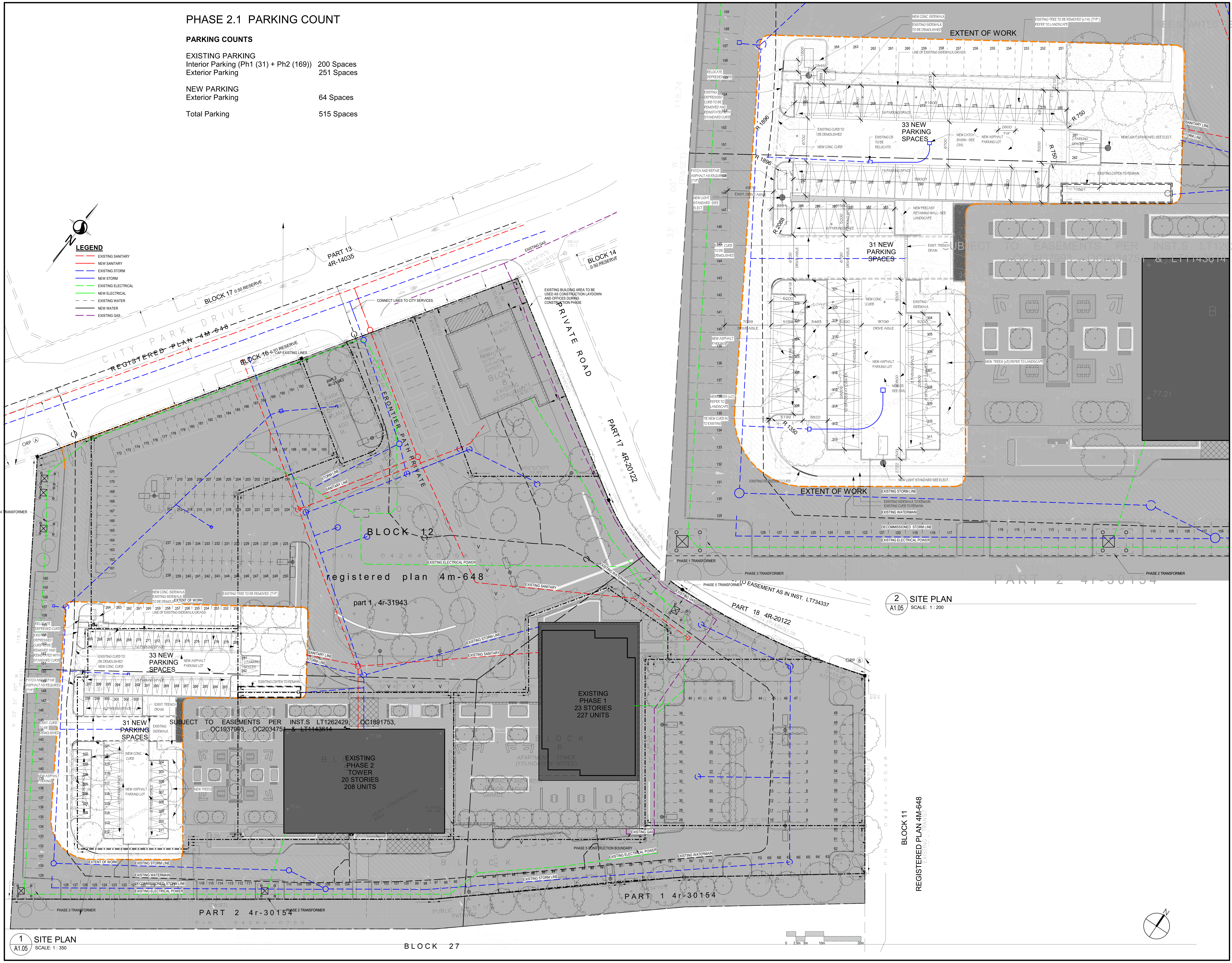
PHASE 2.1 PARKING COUNT

PARKING COUNTS

EXISTING PARKING	
Interior Parking (Ph1 (31) + Ph2 (169))	200 Spaces
Exterior Parking	251 Spaces
NEW PARKING	
Exterior Parking	64 Spaces
Total Parking	515 Spaces

LEGEND

🟥 – –	EXISTING SANITARY
🟥 ——	NEW SANITARY
🟦 – –	EXISTING STORM
🟦 ——	NEW STORM
🟩 – –	EXISTING ELECTRICAL
🟩 ——	NEW ELECTRICAL
– – –	EXISTING WATER
——	NEW WATER
🟪 – –	EXISTING GAS



1 SITE PLAN
A1.05 SCALE: 1 : 350

2 SITE PLAN
A1.05 SCALE: 1 : 200

SURVEY INFORMATION TAKEN FROM STANTEC GEOMATICS DRAWING

PLAN OF SUBDIVISION OF PART OF BLOCK 12
REGISTERED PLAN 4M-648
CITY OF OTTAWA
COMPRISING OF PART OF PIN 04264-0756
BLOCKS 1 TO 8 INCLUSIVE ARE SUBJECT TO EASEMENTS PER INST. LT1262429, OC1891753, OC1937993, OC2034751, LT1143614.

AREA OF SITE 26, 622 SQUARE METERS

No.	Date	Revision
C	25.10.20	Reissued for SPRA
B	25.08.20	Issued for SPRA
A	25.08.05	Issued for 75% review

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PROJECT
RIOCAN - CITY PARK
PHASE 2
200 FRONTIER PATH PVT.
(FORMALLY 2280 City Park Dr.)

DRAWING TITLE
PHASE 2.1 SITE PLAN - ADDITIONAL PARKING

DRAWN	DATE	SCALE
DH&WIK	07/16/19	As indicated

ONTARIO ASSOCIATION OF ARCHITECTS
BARRY J. HOBIN
LICENCE
3049

PROJECT 1759

DRAWING NO. A1.05

REVISION NO. C

A1.05-RC-PHASE 2.1 SITE PLAN - ADDITIONAL PARKING

APPENDIX B

Golder’s Geotechnical Report No. 1522569 (10001)

November 2015

REPORT ON

Detailed Design Geotechnical Investigation
Proposed Gloucester SilverCity Residential Intensification
City Park Drive,
Ottawa, Ontario

Submitted to:
RioCan Management Inc.
RioCan Yonge Eglinton Centre,
2300 Yonge Street, Suite 500
P.O. Box 2386
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REPORT

Report Number: 1522569 (10001)

Distribution:

1 e-copy - RioCan
1 e-copy - Golder Associates Ltd.

Golder Associates

Table of Contents

FIGURES

APPENDICES

1.0 INTRODUCTION

This report presents the results of a geotechnical investigation for the detailed design phase of proposed redevelopment of the existing commercial property located at 2280 and 2401 City Park Drive in Ottawa, Ontario (see Key Map inset on Site Plan, Figure 1).

The purpose of this subsurface investigation was to determine the general soil, bedrock and groundwater conditions across the site by means of advancing a limited number of boreholes at the site and a limited number of laboratory tests. Based on an interpretation of the factual information obtained, supplemented with existing subsurface information available for this site, a general description of the subsurface conditions is presented. These interpreted subsurface conditions, in conjunction with available project details, were used to provide engineering input on the geotechnical design aspects of the project, including construction considerations which could influence design decisions.

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

2.0 DESCRIPTION OF THE PROJECT AND SITE

The topography of the site of the proposed re-development is relatively flat. Drainage is provided by surface runoff directly to a stormwater sewer system. The site is bordered by City Park Drive to the north, the Gloucester Centre mall to the east, the new Light Rail Transit (LRT) Confederation Line to the south, and a low-rise residential development to the west.

Based on the latest information provided by RioCan, the proposed residential intensification at the City Park – Silver City Gloucester site will consist of three high-rise residential towers on the southeast half of the site, and three low-rise residential and commercial buildings on the northwest half of the site. A multi-level parking garage (2.5 levels) is being proposed at the west end of the site. The preliminary plans provided indicate that the project will be built in three phases: Phase I will include a thirty storey tower at the southeast corner of the site; Phases II and III will include two twenty storey towers in the south center (Phase II) and southwest corner (Phase III) of the site. The high-rise towers will only have one level of basement which will have a slab-on-grade at about elevation 72.7 metres with a drive-out / walk-out parking level on the south side. A grade raise of about 2 metres is currently proposed at the north entrance side of the three new towers. Entrance to the basement level will be provided from the south side of the building with a ramp down to the basement elevation. The three towers will all be connected by one to two podium levels above the basement with the ground floor at about elevation 77.3 metres.

The low-rise buildings proposed for the north side of the site might be included as part of the Phase I development. These buildings will be one to three stories in height. It is understood that the low-rise buildings will be of slab on grade construction.

At grade exterior parking areas will be provided around the new buildings. Once all phases of the proposed development are completed, additional parking will be provided in a multi-level parking garage located along the west side of the site.

The following previous studies at the site were carried out by Golder:

- Golder Report No. 10-1121-0222 titled: “Geotechnical Data Report, Geotechnical and Hydrogeological Investigation, Ottawa Light Rail Transit (OLRT), East At-Grade (Segments 3, 4 & 5), Ottawa, Ontario”, and dated October 2011;
- Golder Report No. 871-2120-1 titled: “Geotechnical Investigation, Subsurface Conditions, East Transitway Station 12+680 to 15+150, Regional Municipality of Ottawa Carleton”, and dated January 1988; and,
- Golder Report No. 841-2062 titled: “Preliminary Geotechnical Evaluation, Eastgate Property, Gloucester, Ontario”, and dated April 1984.

From these previous studies, the site is indicated to be underlain by up to about 2 metres of overburden over bedrock. Based on the previous studies, and published geology maps available from the Geologic Survey of Canada (GSC) for this area, the bedrock beneath this site consists of black shale of the Billings formation. This formation of shale is known to swell when exposed to air, and special design considerations are required if the foundations or basement levels of buildings are to be placed within this rock unit.

3.0 PROCEDURE

The field work for this investigation was carried out on October 19 and 20, 2015 during which time seven boreholes (numbered 15-01 to 15-07, inclusive) were put down at the approximate locations shown on the Site Plan, Figure 1. The boreholes were advanced using a truck-mounted, hollow-stem auger drill rig supplied and operated by Downing Estate Drilling of Hawkesbury, Ontario.

The boreholes were advanced to depths ranging from about 3.1 to 16.6 metres below the existing ground surface. Within the boreholes, standard penetration tests (SPTs) (ASTM D1586) were carried out at regular intervals of depth and soil samples were recovered using split spoon sampling equipment.

Upon reaching the bedrock surface in boreholes 15-01, 15-03, 15-04 and 15-7, these boreholes were advanced further into the bedrock for lengths of between about 1.8 and 6.1 metres, using rotary diamond drilling techniques while retrieving NQ sized bedrock core. Borehole 15-02 was also extended into the bedrock for a length of about 13.6 metres while retrieving HQ sized bedrock core required for seismic geophysical testing.

Due to the weathered/fractured nature of the bedrock, the boreholes were advanced into the bedrock using hollow stem augers for lengths of between 0.2 and 4.4 metres.

To allow for subsequent measurement of the groundwater level, a standpipe piezometer was installed in borehole 15-03. A water level measurement was taken in the monitoring well on October 28, 2015. To facilitate the seismic geophysical testing, a PVC casing was grouted into borehole 15-02.

The field work was supervised by an experienced technician from our geotechnical staff who located the boreholes, monitored the drilling operations, logged the subsurface conditions encountered in the boreholes, directed the in situ testing and took custody of samples.

On completion of drilling, soil and bedrock samples were transported to our laboratory for examination by the project engineer and for laboratory testing. Index and classification tests, including water content determinations and two grain size distribution tests were carried out on select soil samples. Uniaxial Compressive Strength (UCS) tests were carried out on selected bedrock samples.

The borehole locations were selected, marked in the field, and subsequently surveyed by Golder personnel. The locations and elevations of the boreholes, except for borehole 15-02 (due to interference from the proximity of the existing building), were surveyed using a GPS R8-Trimble unit. Borehole 15-02 was referenced to existing site features, and its elevation was surveyed to the other boreholes. All current borehole elevations provided herein are referenced to Geodetic datum, and their locations are referenced to the UTM NAD83 coordinate system.

4.0 SUBSURFACE CONDITIONS

4.1 General

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

- The results of the boreholes from the current investigation are provided on the Record of Boreholes in Appendix A.
- Relevant borehole records from previous investigations are provided in Appendix B.
- The results of Golder laboratory testing on samples of soil are provided in Appendix C.
- The results of UCS laboratory testing on samples of bedrock are provided in Appendix D.
- The results of geophysical testing in borehole 15-02 are provided in Appendix E.

In general, the subsurface conditions at the site consist of a flexible pavement underlain by fill over shale bedrock. In two of the seven boreholes, a discontinuous and relatively thin (i.e., less than 1 metre) layer of glacial till was encountered. From the Geological Survey of Canada published bedrock geology maps, the bedrock in this area is indicated to be shale from the Billings formation. Detailed descriptions of the subsurface soil, bedrock and groundwater conditions are provided on the individual sheets provided in Appendix A.

The following sections present a more detailed overview of the subsurface conditions encountered in the boreholes advanced during the current investigation.

4.2 Pavement Structure / Fill

Asphaltic concrete was encountered at all boreholes for the current investigation; the thickness of the asphaltic concrete is provided in the table below.

Borehole No.	Asphalt Thickness (mm)
15-01	80
15-02	100
15-03	100
15-04	80
15-05	100
15-06	80
15-07	80

The granular fill used for the pavement base subbase and general grade generally consists of grey, silty sand sand with varying amounts of gravel. The results of two grain size distribution tests on select samples of the granular fill indicate that this layer may be described as gravelly silty sand. The depth of the fill at each of the borehole locations is provided in the table below.

Borehole No.	Depth of Granular Pavement Structure below existing grades (m)
15-01	1.5
15-02	1.2
15-03	1.3
15-04	1.7
15-05	2.1
15-06	1.8
15-07	2.0

SPT “N”-values measured within the fill ranged from 6 to 18 blows per 0.3 m of penetration. The SPT “N” values suggest that the state of packing of the granular fill is loose to compact.

4.3 Glacial Till

A discontinuous deposit of glacial till was encountered below the fill in boreholes 15-01 and 15-07. The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sandy clayey silt. The glacial till was fully penetrated and has a thickness of 0.8 metres at borehole 15-01 and 0.2 metres at borehole 15-07.

4.4 Bedrock

Black shale bedrock of the Billings formation was encountered in all of the boreholes at depths of about 1.2 to 2.3 metres below the ground surface, between elevations 72.5 and 73.2 metres.

The approximate depths and elevations of the bedrock surface, as well as the ground surface elevations at the boreholes are shown in the following table.

Borehole Number	Ground Surface Elevation in Borehole (m)	Bedrock Depth (m)	Bedrock Surface Elevation (m), Geodetic
15-01	74.74	2.29	72.45
15-02	73.85	1.22	72.63
15-03	74.54	1.30	73.24
15-04	74.83	1.68	73.15
15-05	74.70	2.06	72.64
15-06	74.63	1.75	72.88
15-07	74.79	cati	72.50

The bedrock encountered in the boreholes generally consists of moderately weathered to fresh, laminated to thinly bedded, black and very fine grained shale bedrock with thin laminates of limestone.

The upper portion of the bedrock is moderately weathered and very fractured. The very fractured shale bedrock generally extends to about 4 metres in depth below ground surface in boreholes 15-1, 15-2 and 15-3 where the very fractured shale bedrock layer was completely penetrated.

The Rock Quality Designation (RQD) values ranged from 0 to 100 percent indicating very poor to excellent quality rock. In general, the RQD values increase with depth.

A total of twelve Uniaxial Compressive Strength (UCS) tests were carried out on selected samples of the bedrock core retrieved in the boreholes. The detailed results of this testing is provided in Appendix D, and summarized in the table below.

Borehole Number	Sample Number	Sample Depth (m)	UCS (MPa)	Young's Modulus, E (GPa)
15-01	1	5.54 – 5.70	26.9	4.7
15-01	2	6.38 – 6.80	51.3	14.7
15-01	3a	7.11 – 7.36	30.1	7.0
15-01	3b	7.11 – 7.36	63.6	10.0
15-02	--	4.99 – 5.15	31.3	--
15-02	--	13.09 – 13.22	35.3	--
15-02	1	15.03 – 15.22	37.2	7.0
15-03	--	3.59 – 3.69	33.7	--
15-03	1	4.10 – 4.30	45.2	7.8
15-03	2a	5.30 – 5.55	30.6	5.0
15-03	2b	5.30 – 5.55	39.2	5.8
15-03	3	6.07 – 6.20	45.6	9.1

The results of the testing indicate that the shale rock UCS varies between about 26.9 and 63.9 megapascals with an average of 39.2 megapascals, and a standard deviation of 10.2 megapascals. Young's Modulus (E) varies between 4.7 and 14.7 gigapascals with an average of 7.9 gigapascals and a standard deviation of 2.9 gigapascals.

4.5 Groundwater

The groundwater level was measured on October 14, 2015 and on November 11, 2015 in a standpipe sealed into the bedrock in borehole 15-03. The measured groundwater levels are summarized in the table below.

Borehole Number	Ground Surface Elevation (m)	GWL Depth (m)	GWL Elevation (m)	Date of Reading
BH 15-03	74.54	1.87	72.67	October 14, 2015
BH 15-03	74.54	1.95	72.59	November 11, 2015

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as spring.

5.0 DISCUSSION

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the boreholes and the project requirements.

The foundation engineering guidelines presented in this section of the report have been developed in a manner consistent with Part 4 of the 2012 Ontario Building Code (OBC) for Limit States Design.

Reference should be made to the “Important Information and Limitations of This Report” which follows the text but forms an integral part of this document.

The interpretation and geotechnical design input provided in this report are intended to provide the designers with information for design and to assess feasible construction approaches and constraints to construction that may be related to the ground conditions. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design of the project, and for which special provisions or operational constraints may be required in the Contract Documents. Those planning and undertaking specific aspects of construction should make their own interpretation of the factual information provided, as it may affect equipment selection, proposed construction methods, scheduling and the like.

5.2 Excavations

The currently available plans and information during discussion indicate that the three high rise towers on the south side of the site will have a basement level at about elevation 72.7 metres with a main level floor slab elevation of about 77.3 metres. The three low rise buildings on the north side of the site and the parking garage on the west side of the site will have slab-on-grade of the lowest level at or near the existing site grades. The three low rise buildings will have a finished floor slab elevation of about 75.1 metres.

Considering that the bulk excavation will extend to about elevation 72.2 metres to accommodate the basement floor slab, granular base and under-slab services, it is expected that the bulk excavation will extend into the bedrock. The excavation will likely extend a further 1.2 metres below the bulk excavation level to accommodate the foundations and elevator pits and will extend into the bedrock to about elevation 71.0 metres for all proposed structures. Where the bedrock is very fractured and weathered, additional excavation into the bedrock could also be required. The excavations will therefore extend through the fill and glacial till, where present.

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). Steeper side slopes would require shoring to meet the requirements of the OHSA. Given the distance adjacent roadways and structures from the proposed structures, it is expected that shoring of the overburden will not be necessary. Additional guidelines on temporary shoring can be provided if required.

The groundwater level was measured at about 1.9 metres depth in borehole 15-3. On this basis, some minor 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. Based on previous experience with excavations within the Billings shale, groundwater inflows to excavations that extend into the bedrock can be handled by pumping from within the excavation. A Permit-To-Take-Water (PTTW) from the Ministry of the Environment and Climate Change will likely need to be obtained for handling of groundwater inflow into the excavation. A PTTW is required if the daily groundwater pumping would exceed 50,000 Litres. A hydrogeological assessment of the potential impacts of the temporary and permanent groundwater level lowering will need to be carried out; this study will also be required to support a PTTW application.

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 (note: refusal to auger advancement was not observed in any of the boreholes, including borehole 15-06 where it was possible to advance a 200 millimetre diameter hollow stem augers to about 6.1 metres depth). Further bedrock removal below 4 metres depth from ground surface 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.

The upper 1.5 to 2.5 metres of the bedrock is weathered and fractured, and will not likely stand vertically; it should therefore be planned to slope back this zone of bedrock, or to stabilize the rock face with shotcrete. 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.

Excavations for the foundations will result in exposure of the shale bedrock to the air. The shale bedrock at this site has the potential to swell causing heave. The Billings formation shale is known to contain small quantities of pyrite and heaving is caused by pyrite oxidation. The mechanisms causing the expansion (heaving) of the shale are complex and involve both chemical and bio-chemical (bacterial) reactions. Factors contributing to pyrite oxidation include the amount of dissolved atmospheric oxygen, the surface area of the pyrite crystals, humidity, temperature and the presence of clay minerals.

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 (Type HS or HSb cement) within about 24 hours after exposure. Where shale is exposed on the sides of the excavation, the exposed shale should be shotcreted so that concrete covers the shale. Shotcreting will also be required to maintain vertical excavation walls within the shale. 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).

Based on the preliminary drawings and information provided during discussions, the lower basement level and any required sumps for all proposed structures on this site will extend about 0.4 to 0.5 metre below the measured water table. A 0.4 to 0.5 metre drawdown of the water table is not anticipated to cause significant swelling of the shale bedrock around the new structures. In addition, the impacted area should be limited to the area immediately surrounding the new excavation.

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 a ‘water tight’ construction to maintain the level of the water table in the area. Further guidance on this issue can be provided if required. Because the process of expansion of the shale is both chemical and biological, it is recommended that all bedrock surfaces be protected from air once exposed. The swelling is a time dependent phenomenon and occurs over several years.

Even with the precautions listed above, some structural elements, such as grade beams, may get impacted over time by the shale swelling process. As such, the use of void forms should be considered at locations where the shale swelling process may impact these structural elements.

5.3 Site Servicing

Excavation for the installation of site services will be through surficial fill and possibly through the bedrock depending on the design inverts.

The existing fill at this site is considered suitable for the support of site services, provided the excavation and bottom of trenches are first inspected by the geotechnical engineer. Where fill is encountered below invert level, the surface of the fill should be compacted and if unsuitable for support should be subexcavated and replaced with engineered fill consisting of OPSS Granular B Type I or II. The engineered fill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material’s standard Proctor maximum dry density using suitable vibratory compaction equipment.

At least 150 millimetres of OPSS Granular A should be used as pipe bedding for sewer and water pipes. The bedding material should in all cases extend to the springline of the pipe and should be compacted to at least 95 percent of the material’s standard Proctor maximum dry density. The use of clear crushed stone as a bedding layer should not be permitted anywhere on this project since fine particles from the sandy/silty backfill could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

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

The existing fill could be reused as trench backfill. Alternatively, an approved imported sandy material which meets the requirements for OPSS Select Subgrade Material (SSM) could be used. Backfilling operations during cold weather should avoid inclusions of frozen lumps of material, snow and ice. Trench backfill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material’s standard Proctor maximum dry density using suitable vibratory compaction equipment.

Impervious dykes or cut-offs should be constructed in the service trenches near the street connection just inside the property. These dykes will prevent the migration of contaminated surface or ground water within the bedding from migrating along these linear pathways. It is important that these barriers extend from trench wall to trench wall and that they fully penetrate the granular bedding to the trench bottom. The dykes should be at least 1.5 metres wide and could be constructed using relatively dry (i.e., compactable) grey brown silty clay from the weathered zone or the native glacial till which overlies the bedrock in the vicinity of this site. If compactable silty clay or native glacial till is not available such material will need to be imported.

5.4 Foundations

In general, the subsurface conditions at this site consist of up to about 2.3 metres of fill and glacial till, overlying shale bedrock. The fill is not considered suitable founding soil for predictable performance of structures, because of the variable composition and state of packing. In addition, the glacial till is relatively thin and more compressible than the underlying bedrock. The foundations should therefore be founded within the more competent bedrock. The upper part of the shale bedrock is highly fractured and moderately weathered to a depth of up to about 4 metres below the existing ground surface (i.e., at about elevation 70.2 metres).

The available information indicates that the first floor of the podium, which will join the three towers on the south side of the site, will have a finished floor elevation of about 77.3 metres. The basement level will have a basement slab elevation of about 72.7 metres. Approximately 2 metres of fill will be placed on the north side of the new podium to have the main drive aisle at the same level as the podium (i.e., about elevation 77.3 metres). Based on the subsurface conditions at this site (i.e., shallow bedrock), there is no grade raise restriction for foundations on or within the bedrock.

From the boreholes, the bedrock is generally very fractured and moderately weathered to an elevation of about 70.2 metres at the location of the three towers. Higher bearing resistance can be achieved if the foundations are extended to reach the less fractured and more competent shale bedrock at about elevation 70.2 metres. For foundations at this site, four possible options have been considered:

- **Option 1 – Spread footing foundations placed on glacial till or engineered fill on bedrock**: For compatibility, all footings for the same structure should be placed on the same medium (i.e., glacial till or compacted engineered fill). If required below the foundations, engineered fill should be placed on undisturbed glacial till or bedrock. The engineered fill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment. Pad footing sizes up to 3 metres square and strip footings up to 1.5 metres width could be sized using the, the following bearing resistances values:
  - Serviceability Limit States (SLS): 250 kilopascals
  - Factored Ultimate Limit States (ULS): 500 kilopascals
- **Option 2 – Spread footing foundations placed on or within very fractured bedrock above elevation 71.0 metres:** For compatibility, all footings for the same structure should be placed on the same medium (i.e., very fractured bedrock). All footings should be inspected by a qualified geotechnical engineer prior to placing concrete and all highly weathered or loose fractured rock and deleterious material should be removed. Pad footing sizes up to 3 metres square and strip footings up to 1.5 metres width could be sized using the following bearing resistances values:
  - Factored Ultimate Limit States (ULS): 1,500 kilopascals
- **Option 3 – Spread footing foundations placed on or within very fractured bedrock between elevation 71.0 metres and 70.2 metres:** For compatibility, all footings for the same structure should be placed on the same medium (i.e., very fractured bedrock). All footings should be inspected by a qualified geotechnical engineer prior to placing concrete and all highly weathered or loose fractured rock and deleterious material should be removed. Pad footing sizes up to 3 metres square and strip footings up to 1.5 metres width could be sized using the following bearing resistances values:
  - Factored Ultimate Limit States (ULS): 3,000 kilopascals

- **Option 4 – Spread footing or end bearing caisson foundations placed below the fractured bedrock at or below elevation 70.2 metres:** For compatibility, all footings or caissons for the same structure should be placed on the same medium (i.e. competent bedrock below elevation 70.2 metres). All footings or end bearing caissons should be inspected by a qualified geotechnical engineer prior to placing concrete and all moderately to highly weathered or loose fractured rock and deleterious material should be removed. Pad footings up to 3 metres square, caissons up to 3 metres diameter, and strip footings up to 1.5 width could be sized using the following bearing resistances values:
    - Factored Ultimate Limit States (ULS): 5,000 kilopascals

For footings bearing on or within bedrock, Serviceability Limit States (SLS) generally do not govern the design since the stresses required to induce 25 millimetres of movement (the typical SLS criteria) exceed those at ULS. Accordingly, the post construction settlement of structural elements which derive their support from footings bearing on bedrock are anticipated to be less than 25 millimetres. At the time of the preparation of this report, the information on footing sizes, founding elevations and foundation loads was preliminary. Once more detailed information is available (i.e., elevator shaft shear walls), SLS and ULS bearing values should be assessed.

Where the excavation for Options 2, 3 and 4 requires temporarily drawing down of the water table, the exposed shale should be protected from air exposure by placing a 50 mm thick layer of concrete/shotcrete on the rock surface within 24 hours of being exposed. In areas where the new foundations will be bearing against the concrete/shotcrete (i.e., on the floor of the excavation, or the sides for lateral resistance), then the concrete/shotcrete should be made with Type HS or HSb cement.

The above values were calculated based on vertical concentric loads only, and using a resistance factor of 0.5 for vertical bearing resistance from semi-empirical analysis using laboratory data.

The sliding resistance between the shale bedrock and concrete footings can be computed based on an unfactored friction coefficient of 0.36.

5.5 Rock Anchors

It is expected that the foundations may be required to resist uplift forces related to unbalanced lateral loads (i.e., resulting from seismic forces on the building) on foundations or to increase the sliding resistance of the foundations. These uplift forces could be resisted using grouted anchors in the bedrock. The presence of fractured rock conditions and groundwater should be considered carefully by the specialty contractor and may require post-grouting to ensure adequate anchor resistance is obtained.

The anchors should consist of grouted rock anchors.

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 the structural engineer. Adequate corrosion protection of the steel components should be provided to prevent potential premature failure due to steel corrosion.

For potential failure mode iii), the unfactored ULS bond strength at the concrete/rock interface may be taken as 2,000 kilopascals. Using a resistance factor of 0.6, based on static test in tension during construction (as per OBC 2012), the factored ULS bond strength is 1,200 kPa. However, all drill holes must be drilled with equipment that will create a rough texture along the socket (i.e. tri-cone or air track drill).

For potential failure mode iv), the resistance should be calculated based on the buoyant weight of the potential mass of rock which could be mobilised by the anchor. This is typically considered as the mass of rock and surface shear resistance 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 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.

Further guidelines by the geotechnical engineer can and must be provided for assessing the anchor resistance once the final anchor layout and loads have been established.

It is recommended that proof load tests be carried out on anchors to confirm their design (required by OBC 2012 for the use of a resistance factor of 0.6). For permanent anchors, the proof load tests should be carried out in accordance with Ontario Provincial Standard Specification (OPSS) 942 which specifies a testing load of 1.5 times the anchor service loads, and at least 10 percent of the anchors should be tested in this manner. It is also recommended to carry out one pre-production performance test in accordance with OPSS 942 for each anchor type used on the project.

Given the high potential for corrosion to buried steel elements (see Section 5.11), rock anchors intended as permanent structural elements should be provided with double corrosion protection (in accordance with OPSS 942).

The installation and testing of the anchors should be observed by the geotechnical engineer. Care must be taken during grouting to ensure that the grouting is injected from the bottom of the anchor hole to bond the entire grouted length with a minimum of voids. It is also suggested that the anchor holes be thoroughly flushed with water to remove all debris, sludge, and rock flour prior to grouting. It is essential that sludge and rock flour be completely removed from the holes to be grouted to ensure an adequate bond between the grout and the rock.

Prestressing of the anchors prior to loading will reduce anchor movement due to service loads.

5.6 Lower Level Floor Slab

In preparation for the construction of the basement floor slab, all loose, wet, and disturbed material should be removed from beneath the floor slab. The existing fill could remain below the floor slab provided that it is free of organic matter and it is proof rolled to reveal any weak areas. Provision should be made for at least 300 millimetres of granular base consisting of OPSS Granular A or O or clear crushed stone to form the base of the basement floor slab. Any bulk fill required to raise the grade up to the underside of the granular base should consist of OPSS Granular B Type II. The underslab fill should be placed in maximum 300-millimetre thick lifts and should be compacted to at least 95 percent of the material’s standard Proctor maximum dry density using suitable vibratory compaction equipment.

5.7 Foundation Seismic Design

The Ontario Building Code 2012 (OBC 2012) requires the use of time-averaged (harmonic) shear wave velocity (V_s) in the upper 30 metres for determining the appropriate site class. The measured shear waver velocities are to be averaged over 30 metres immediately below the bottom of the basement or spread footing foundation.

Accordingly, shear wave velocities were measured in borehole 15-02 and a technical memorandum giving details of the study is included in Appendix E of this report. Table 1 of the technical memorandum shows a tabular presentation of 1 metre interval shear and compression wave velocities over the depth of exploration together with calculated Poisson's ratios, shear, Young's and bulk moduli using typical soil densities for each layer. The harmonic mean shear wave velocity of the subsurface soil and bedrock in the upper 30 metres depth was calculated by the following equation:

$$V_s = \text{total thickness of all layers} / \sum (\text{each layer thickness/each layer shear wave velocity})$$

For this proposed development, the bearing stratum for the three towers, and 30 metres below the bearing level, will be shale bedrock. The harmonic mean shear wave velocity in the upper 30 metres below the foundation level of 71.0 metres was calculated at 1,194 m/s. On this basis, the site is classified as "Site Class B" as per Table 4.1.8.4.A given in Part 4 of OBC 2012.

For the low rise structures on the north side of the site, the harmonic mean shear wave velocity in the upper 30 metres below ground surface was calculated to be 925 m/s. Therefore, the low rise structures can also be designed using a "Site Class B", provided that there's less than 3 metres of soil between the underside of the foundations and the bedrock surface.

5.8 Foundation Wall Backfill

The soils at this site are potentially frost susceptible and should not be used as backfill against exterior or unheated foundation elements (e.g., footing, foundation walls, pile caps, etc.). To avoid problems with frost adhesion and heaving, these foundation elements should be backfilled with non-frost-susceptible sand which meets that gradation requirements for OPSS Granular B Type I, or crushed rock fill meeting the gradation requirements of OPSS Granular A or Granular B Type II.

In areas where pavement or other hard surfacing will abut the building, differential frost heaving could occur between the granular fill (if sand or crushed stone is used) and other areas. To reduce this differential heaving, the backfill adjacent to the wall should be placed to form a frost taper. The frost taper should be brought up to pavement subgrade level from 1.5 metres below finished exterior grade or the top of bedrock (whichever is higher) at a slope of 3 horizontal to 1 vertical, or flatter, away from the wall. The fill should be placed in maximum 300-millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

The pavement could be expected to perform better in the long term if the granular backfill against the foundation walls is drained by means of a perforated pipe subdrain in a surround of 19 millimetre clear stone, fully wrapped in geotextile, which leads by gravity drainage to a positive outlet.

The passive resistance offered by the foundation wall backfill soils could also be considered in evaluating the lateral resistance applied to the foundations. The magnitude of that lateral resistance will depend on the backfill materials and backfill conditions adjacent to the foundation walls. If the backfill materials consist of compacted sand or sand and gravel (OPSS Granular B Type I) as discussed herein, then the passive resistance acting on the foundation wall may be taken as:

$$\sigma_h(z) = K_p \gamma z$$

where:

$\sigma_h(z)$ = lateral earth resistance applied to the foundation wall at depth z, kilopascals

K_p = passive earth pressure coefficient, use 3.0

γ = unit weight of retained soil, use 20 kilonewtons per cubic metre

z = depth below top of wall, metres

This resistance is provided in unfactored format. Factoring of the calculated resistance value will be required if the design is being carried out using Limit States Design.

Movement of the backfill and wall is required to mobilize the passive resistance. As a preliminary guideline, about 75 millimetres of movement would be required to fully mobile the passive resistance. If the foundation wall is considered non-yielding, then the at rest earth pressure of $K_o = 0.5$ should be used instead of the passive earth pressure ($K_p = 3.0$).

Where the granular backfill is below the water table, then the buoyant unit weight of the granular backfill should be used (i.e., $\gamma = 10$ kilonewtons per cubic metre)

5.8.1 Lateral Earth Pressures

The magnitude of the lateral earth pressures will depend on the backfill materials and backfill conditions adjacent to the foundation walls. If the backfill consists of compacted granular soil (OPSS Granular ‘A’, Granular ‘B’ Type I or II), then the lateral earth pressures may be taken as:

$$\sigma_h(z) = K_o (\gamma z + q)$$

Where: $\sigma_h(z)$ = Lateral earth pressure on the wall at depth z, kilopascals;

K_o = At-rest earth pressure coefficient, use 0.5;

γ = Unit weight of retained soil, use 22 kilonewtons per cubic metre;

z = Depth below top of wall, metres; and,

q = Uniform surcharge at ground surface to account for traffic and equipment (not less than 15 kilopascals), plus any surcharge due to adjacent foundation loads.

If a water-tight structure may be required, then the water pressures will need to be considered for that portion below the groundwater level. Further input would need to be provided.

These lateral earth pressures would increase under seismic loading conditions. The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). For preliminary design, the total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(z) = K_o \, \gamma \, z + (K_{AE} – K_o) \, \gamma \, (H\text{-}z)$$

Where: $\sigma_h(d)$ = Lateral earth pressure at depth z, kilopascals;

K_o = At-rest earth pressure coefficient, use 0.5;

K_{AE} = Seismic earth pressure coefficient, use 0.8;

γ = Unit weight of backfill soil, use 22 kilonewtons per cubic metre;

z = Depth below the top of the wall, metres; and,

H = Total height of the wall, metres.

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

The above lateral earth pressure equations assume that the foundation walls will be drained. If the walls are design to be water-tight, the walls will have to be designed to resist the additional hydro-static pressure.

5.9 Frost Protection

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

5.10 Pavement Design

5.10.1 Hot Mix Asphaltic Concrete

Superpave 12.5 (Level B) surface course and Superpave 19.0 (Level B) base course asphaltic concrete should be used on this project. The hot mix asphaltic concrete should meet the requirements of OPSS 301.

5.10.2 Asphalt Cement

The asphaltic concrete used on this project should be made with PG 58-28 asphalt cement on all lifts.

5.10.3 Granular Base and Subbase

The granular base and subbase for new construction should consist of Granular A and Granular B Type II, respectively. The granular materials used on site should meet the requirements of OPSS.MUNI 1010.

5.10.4 Compaction

Compaction of the granular base, subbase and grade raise fill should be carried out in accordance with OPSS.MUNI 501 Method A. The asphaltic concrete should be compacted as per Table 10 of OPSS 310.

5.10.5 Pavement Structure

In preparation for pavement construction, all topsoil, disturbed, or otherwise deleterious materials should be removed from the roadway areas. All existing fill at this site should be removed from below paved areas.

Pavement areas requiring grade raising to proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow or OPSS Select Subgrade Material meeting the requirements of OPSS.MUNI 1010. These materials should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density using suitable vibratory compaction equipment.

The surface of the pavement subgrade should be crowned to promote drainage of the pavement granular structure into longitudinal sub drains or sub drain leads that extend at least 3 metres from the catch basins.

The pavement structure for the emergency access roadways should consist of:

Pavement Component	Thickness (millimetres)
Asphaltic Concrete	90
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	300

The pavement structure for parking lots should consist of:

Pavement Component	Thickness (millimetres)
Asphaltic Concrete	50
OPSS Granular A Base	150
OPSS Granular B Type II Subbase	300

The composition of the asphaltic concrete pavement should be as follows:

Roadways:

- Superpave 12.5 – 40 millimetres
- Superpave 19.0 – 50 millimetres

Parking Lot:

- Superpave 12.5 – 50 millimetres

The above pavement designs are based on the assumption that the pavement subgrade has been acceptably prepared (i.e., where the trench backfill and grade raise fill have been adequately compacted to the required density and the subgrade surface not disturbed by construction operations or precipitation).

5.11 Corrosion and Cement Type

Due to the known aggressive nature of the shale bedrock in this area, no site specific chemical testing was carried out on a groundwater or soil samples from this site.

The shale bedrock is known to require sulphate resistant concrete cover upon exposure to the atmosphere to prevent future heave. As such, Type HS or HSb should be used for all concrete in contact with the bedrock. In addition, any concrete that will be located below the bedrock surface should also contain Type HS or HSb cement.

The pyrite contained within the Billings Shale formation is known to breakdown to sulfides when exposed to air. The sulfides can corrode unprotected buried steel elements, such as rock anchors. As such, any buried steel elements, such as rock anchors, should be provided with adequate corrosion protection.

Elsewhere, all foundations and site services will be above the water level and backfilled with inert imported granular backfill. This backfill does not pose an issue with respect to corrosion on buried steel or concrete elements.

5.12 Impacts on Adjacent Confederation Line

As part of the site plan approval process for developments located in proximity to the new Confederation LRT Line, a proximity study is required. The goal of this study is to identify any potential risks of the proposed development onto the new LRT line.

Based on the known details of the proposed project, the redevelopment of this site will include the following:

- Three 20 to 30 storey tower sitting on a one to two storey mezzanine with one level of basement on the south side of the site;
- A parking structure with 2.5 levels of parking at the southwest corner of the site; and,
- Three low-rise buildings on the north portion of the site.

Once completed, the proposed new development should not impact the operations of the Confederation LRT Line from a geotechnical perspective.

However, there is a low risk that the LRT line could be affected by the construction of the new development if:

- Blasting is used for rock removal; and,
- The groundwater is drawn down to significant depth over an extended period of time, causing the Billings shale bedrock to swell below the LRT line.

Therefore, the following recommendations are provided to mitigate the above noted risks to the LRT line during construction of the project:

- Due to the fractured and thinly bedded nature of the shale, rock removal should be completed using mechanical means only (i.e., hoe ramming), and blasting should not be permitted (likely not required anyway); and,
- The temporary lowering of the groundwater table during construction should not extend below elevation 70 metres, and should not exceed a period of 1 week.

6.0 ADDITIONAL CONSIDERATIONS

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.

Golder Associates should review the design drawings and specifications to make sure that the intent of this geotechnical report has been met.

During construction, sufficient foundation inspections, subgrade inspections, in situ density tests, materials testing 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 by Golder 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 is sufficient for your present requirements. If you have any questions concerning this report, please feel free to contact the undersigned.

GOLDER ASSOCIATES LTD.

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IMPORTANT INFORMATION AND LIMITATIONS
OF THIS REPORT

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

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

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

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

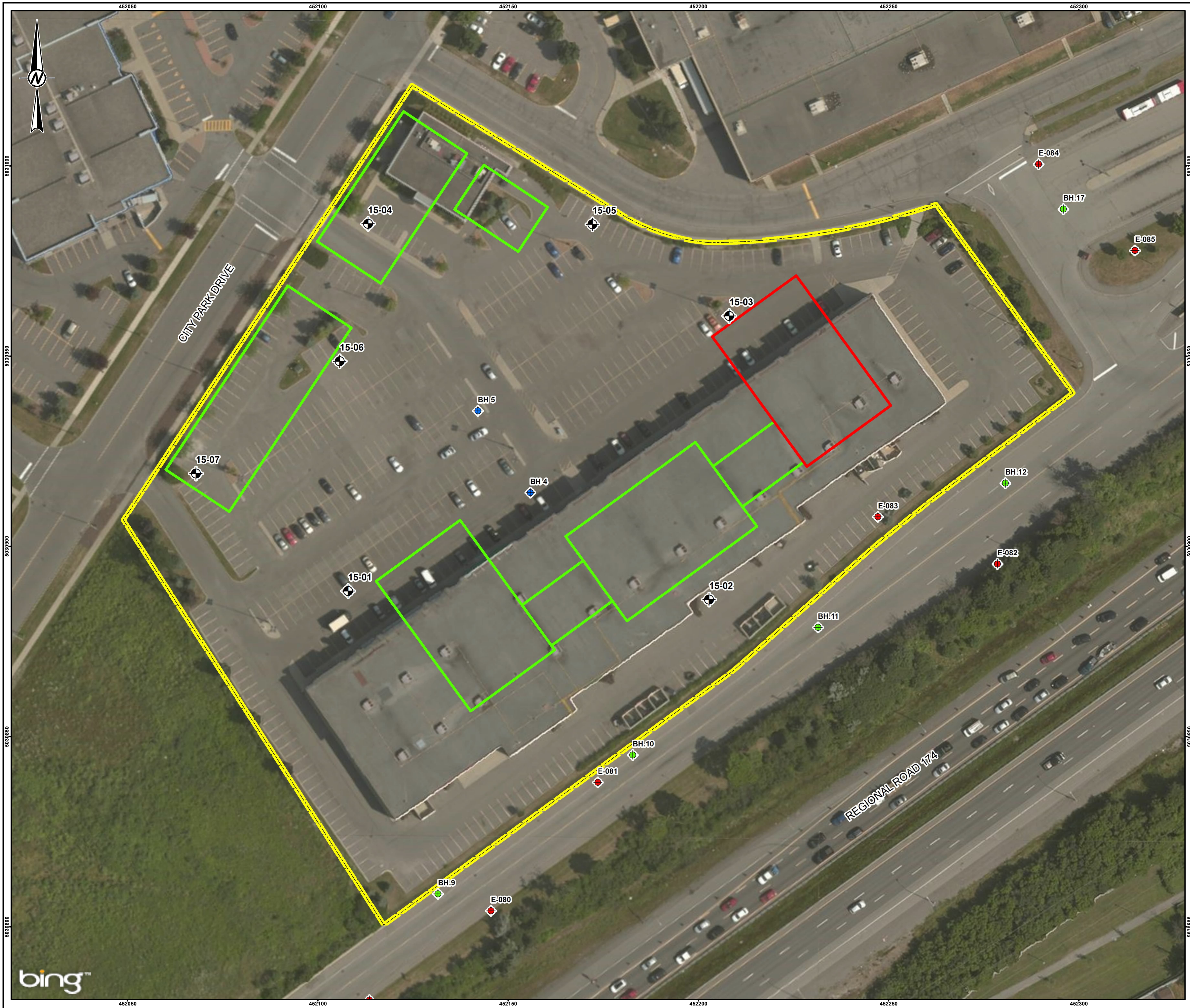
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater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

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

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

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

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



LEGEND

⌖	APPROXIMATE BOREHOLE/MONITORING WELL LOCATION
⊕	APPROXIMATE BOREHOLE LOCATION, PREVIOUS INVESTIGATION (10-1121-0222)
⊕	APPROXIMATE BOREHOLE LOCATION, PREVIOUS INVESTIGATION (8412062)
⊕	APPROXIMATE BOREHOLE LOCATION, PREVIOUS INVESTIGATION (8712120)
🟥	PROPOSED PHASE I BUILDING FOOTPRINT
🟩	PROPOSED BUILDING FOOTPRINT
🟨	APPROXIMATE SITE BOUNDARY

KEY MAP

0 25 50

1:1,000 METRES

REFERENCE(S)

BASE DATA - ATLAS OF CANADA, NATURAL RESOURCES CANADA, 2011. MNR LIO, OBTAINED 2015.
PRODUCED BY GOLDER ASSOCIATES LTD UNDER LICENCE FROM ONTARIO MINISTRY OF NATURAL RESOURCES, © QUEENS PRINTER 2015
BASE IMAGERY - MICROSOFT BING ©2015 MICROSOFT CORPORATION AND ITS DATA SUPPLIERS.
PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83 COORDINATE SYSTEM: UTM ZONE 18N

CLIENT
RIOCAN MANAGEMENT INC.

PROJECT
DETAILED DESIGN GEOTECHNICAL INVESTIGATION
2280 & 2401 CITY PARK DRIVE, OTTAWA, ONTARIO

TITLE
SITE PLAN

CONSULTANT
Golder Associates

YYYY-MM-DD	2015-10-15
DESIGNED	JT
PREPARED	JT/JEM
REVIEWED	NRL
APPROVED	TJN

PROJECT NO.	PHASE	REV.	FIGURE
1522569	10000	0	1

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IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: 25mm 0

APPENDIX A

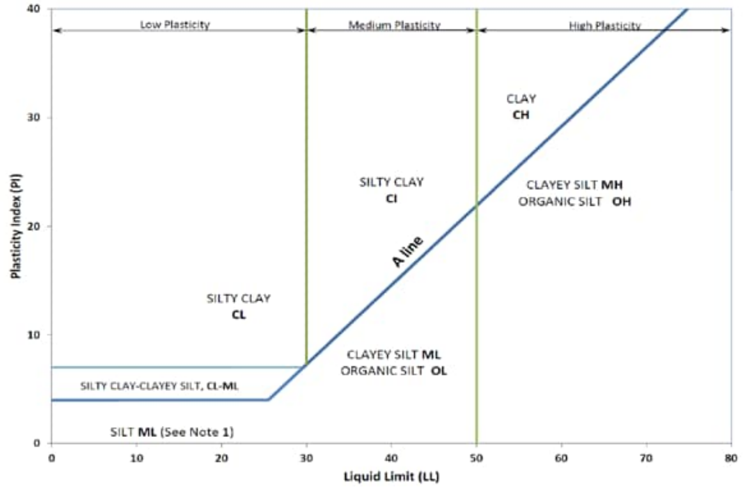
Method of Soil Classification
Abbreviations and Terms Used On Record of Boreholes and Test Pits
List of Symbols
Lithological and Geotechnical Rock Description Terminology
Record of Borehole Sheets – Current Investigation

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



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

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

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

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

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

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

A borderline symbol should be used to indicate that the soil has been identified as having properties that are on the transition between similar materials. In addition, a borderline symbol may be used to ~~or~~ indicate~~s~~ 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, SAND and CLAY)
> 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.).

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
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size
TP	Thin-walled, piston – note size
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 which are 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 - 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 overburden pressure effects.
2. Definition of compactness descriptions based on SPT 'N' ranges from Terzaghi and Peck (1967) and correspond to typical average N_{60} values.

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] (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.

Water Content

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

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$ - u)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

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

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN	Joint	PL	Planar
FLT	Fault	CU	Curved
SH	Shear	UN	Undulating
VN	Vein	IR	Irregular
FR	Fracture	K	Slickensided
SY	Stylolite	PO	Polished
BD	Bedding	SM	Smooth
FO	Foliation	SR	Slightly Rough
CO	Contact	RO	Rough
AXJ	Axial Joint	VR	Very Rough
KV	Karstic Void		
MB	Mechanical Break		

PROJECT: 1522569-10000

RECORD OF BOREHOLE: 15-01

SHEET 1 OF 2

LOCATION: N 5030888.3 ;E 452107.9

BORING DATE: October 20, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

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

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

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

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

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	74.74					
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE						
		FILL - (SM) gravelly SILTY SAND; grey brown, (PAVEMENT STRUCTURE); non-cohesive, moist, loose	0.08					
1				1	SS	8	M	
		(ML) sandy CLAYEY SILT, trace gravel; brown, (GLACIAL TILL); non-cohesive, moist, loose	73.22 1.52					
2				2	SS	7		
		Slightly to moderately weathered SHALE BEDROCK	72.45 2.29	3	SS	>50		
3		Borehole continued on RECORD OF DRILLHOLE 15-01	71.72 3.02					
4								
5								
6								
7								
8								
9								
10								
11								
12								
13								
14								
15								

MIS-BHS 001 1522569-10000.GPJ GAL-MIS.GDT 11/16/15 JEM

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: JD

CHECKED: NRL

PROJECT: 1522569-10000

RECORD OF DRILLHOLE: 15-01

SHEET 2 OF 2

LOCATION: N 5030888.3 ;E 452107.9

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: October 20, 2015

DRILL RIG: Truck Mount

DRILLING CONTRACTOR: Downing Drilling

DATUM: Geodetic

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.
		BEDROCK SURFACE		72.45	
	Power Auger 200 mm	See RECORD OF BOREHOLE 15-01		2.29	
3				71.72	
	Rotary Drill NQ Core	- broken rock from 3.02 to 3.48 m depth Slightly weathered to fresh, laminated to thinly bedded, black, very fine grained, weak to strong SHALE BEDROCK, thin laminates of limestone - broken rock from 3.52 to 3.54 m depth			C1
4					
5					C2
		UCS = 26.9 MPa			
6					
		UCS = 51.3 MPa			C3
7		UCS = 30.1 MPa			
		UCS = 63.6 MPa		67.22	
		End of Drillhole		7.52	
8					
9					
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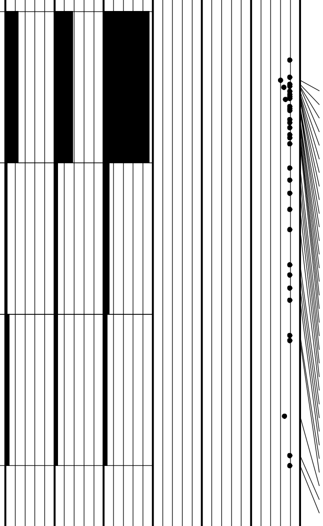
15					
16					
17					

COLOUR / % RETURN

FLUSH



TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja
,BD,PL,SM	16		
,BD,PL,SM	16		
,JN,CU,SM	20		
,BD,PL,SM	16		
,BD,PL,SM	16		
,JN,CU,SM	20		
,BD,PL,SM	16		
,BD,PL,SM	16		
,BD,PL,SM	16		
,BD,PL,SM	16		
,JN,CU,SM	20		
,BD,PL,SM	16		
,BD,PL,SM	16		
,BD,PL,SM	16		
,BD,PL,SM	16		
,BD,PL,SM	16		
,BD,PL,SM	16		
,BD,PL,SM	16		
,BD,PL,SM	16		
,BD,PL,SM	16		
,BD,PL,SM	16		
,BD,PL,SM	16		
,BD,PL,SM	16		
,BD,PL,SM	16		
,BD,PL,SM	16		
,BD,PL,SM	16		
,BD,PL,SM	16		
,BD,PL,SM	16		
,BD,PL,SM	16		
,BD,PL,SM	16		
,BD,PL,SM	16		
,JN,CU,SM	20		
,BD,PL,SM	16		
,BD,PL,SM	16		

HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.
10^{-6} 10^{-5} 10^{-4} 10^{-3}	2 4 6	

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: JD

CHECKED: NRL

PROJECT: 1522569-10000

RECORD OF BOREHOLE: 15-02

SHEET 1 OF 3

LOCATION: N 5030886.0 ;E 452202.8

BORING DATE: October 19, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		73.85							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE		0.10							Flush Mount Protective Casing
		FILL - (SW) gravelly SAND, trace fines; grey brown, (PAVEMENT STRUCTURE); non-cohesive, moist, compact		72.63 1.22							50 mm Diam. PVC Pipe
1					1	SS	12				
		Moderately to slightly weathered SHALE BEDROCK			2	SS	>50				
2					3	SS	>50				
3				70.85 3							
		Borehole continued on RECORD OF DRILLHOLE 15-02									
4											
5											
6											
7											
8											50 mm Diam. PVC Pipe
9											
10											
11											
12											
13											
14											
15											

MIS-BHS 001 1522569-10000.GPJ GAL-MIS.GDT 11/16/15 JEM

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: JD

CHECKED: NRL

PROJECT: 1522569-10000

RECORD OF DRILLHOLE: 15-02

SHEET 2 OF 3

LOCATION: N 5030886.0 ;E 452202.8

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: October 19, 2015

DRILL RIG: Truck Mount

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

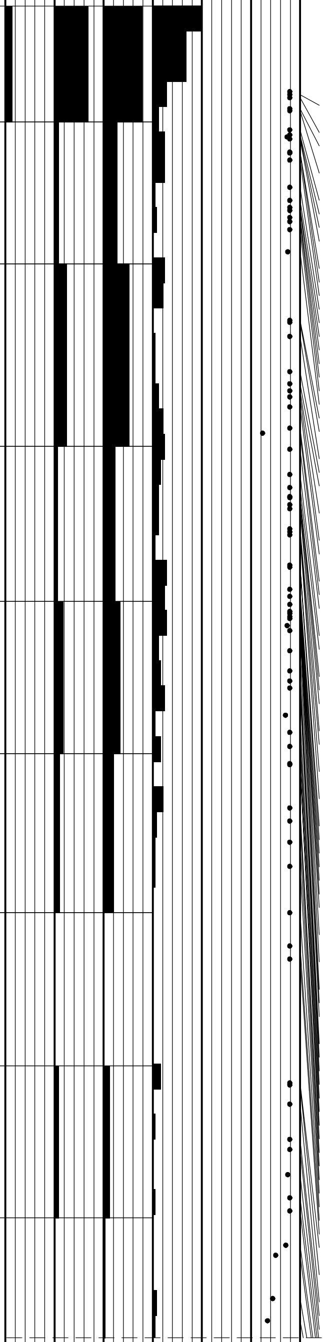
PO- Polished
K - Slickensided
SM- Smooth
Ro - Rough
MB- Mechanical Break

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		72.63	
	Power Auger 200 mm Diam.	See RECORD OF BOREHOLE 15-02		1.22	
2					
3	Rotary Drill HQ Core	Slightly weathered to fresh, laminated to thinly bedded, dark grey to black, porous, weak to strong SHALE BEDROCK, with occasional thin laminates of limestone		70.85 3.00	C1
4		- broken rock from 3.00 to 3.82 m depth - broken core from 3 to 3.82 m depth			
		- broken rock from 4.56 to 4.72 m depth			C2
5		- lost core from 4.83 to 4.85 m depth UCS = 31.3 MPa			
6		- becoming slightly weathered - becoming slightly weathered - broken rock from 5.56 to 6.00 m depth			C3
7		- broken rock from 7.1 to 7.15 m depth			
		- broken rock from 7.56 to 7.59 m depth - broken rock from 7.71 to 7.73 m depth			C4
8					
		- broken rock from 8.63 to 8.68 m depth			
9		- becoming fresh - becoming fresh - broken rock from 9.1 to 9.13 m depth - broken rock from 9.46 to 9.53 m depth			C5
10		- broken rock from 9.92 to 9.96 m depth			
11		- broken rock from 10.76 to 10.79 m depth			C6
12					
13		- broken rock from 12.95 to 12.98 m depth UCS = 35.3 MPa			C7
14		- broken rock from 13.59 to 13.66 m depth			C8
15		UCS = 37.2 MPa			C9
16		- broken rock from 15.7 to 15.77 m depth			

FLUSH | COLOUR % RETURN | RECOVERY: TOTAL CORE %, SOLID CORE % (80 60 40 20) | R.Q.D. % (80 60 40 20) | FRACT. INDEX PER 0.25 m (5 10 15 20) | DISCONTINUITY DATA: B Angle (0 90 180 270), DIP w.r.t. CORE AXIS (0 30 60 90), 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) (2 4 6) | RMC -Q' AVG.



TYPE AND SURFACE DESCRIPTION:

,BD,PL,SM
,BD,PL,SM Br
<1mm
,BD,PL,SM
,BD,PL,SM Br
<1mm
,BD,PL,SM Br
<1mm
,BD,PL,SM
,BD,PL,SM
,JN,CU,SM
,BD,PL,SM Cl
<1mm
,BD,PL,SM
,BD,PL,SM
,BD,PL,SM
,BD,PL,SM
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,BD,PL,SM
,BD,PL,SM
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,BD,PL,SM
,BD,PL,SM
,JN,PL,SM
,BD,PL,SM
,BD,PL,SM Br
<1mm
,BD,PL,SM
,BD,PL,SM
,BD,PL,SM Cl
<1mm
,BD,PL,SM Cl
1-2mm
,BD,PL,SM
,BD,PL,SM Br
<1mm
,BD,PL,SM
,JN,PL,SM
,BD,PL,SM Br
<1mm
,BD,PL,SM
,BD,PL,SM Br
<1mm
,BD,PL,SM
,BD,PL,SM
,BD,PL,SM Cl
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,BD,PL,SM
,BD,PL,SM Cl
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,BD,PL,SM Br
<1mm
,BD,PL,SM Cl
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,BD,PL,SM
,BD,PL,SM
,BD,PL,SM
,BD,PL,SM
,BD,PL,SM
,BD,PL,SM
,BD,PL,SM Br
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,BD,PL,SM Br
<1mm
,BD,PL,SM Br
2mm
,JN,CU,SM
,BD,PL,SM
,BD,PL,SM Br
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,BD,PL,SM
,BD,PL,SM
,JN,CU,SM
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,BD,PL,SM Cl
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,BD,PL,SM

Jcon:

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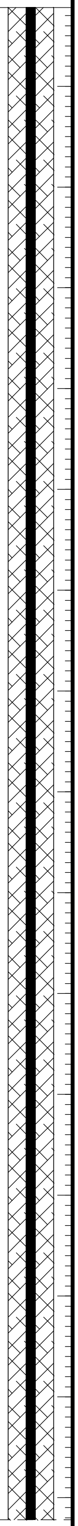
50 mm Diam. PVC Pipe



CONTINUED NEXT PAGE

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: JD

CHECKED: NRL

PROJECT: 1522569-10000

RECORD OF DRILLHOLE: 15-02

SHEET 3 OF 3

LOCATION: N 5030886.0 ;E 452202.8

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: October 19, 2015

DRILL RIG: Truck Mount

DRILLING CONTRACTOR: Downing Drilling

DATUM: Geodetic

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	COLOUR / % RETURN / FLUSH	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 m 5 10 15 20	DISCONTINUITY DATA B Angle 0 90 180 270	DIP w.r.t. CORE AXIS 0 30 60 90	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) 2 4 6	RMC -Q' AVG.	
		--- CONTINUED FROM PREVIOUS PAGE ---																		
				57.30	C9								,JN,CU,SM ,BD,PL,SM ,BD,PL,SM ,JN,CU,SM ,JN,CU,SM ,JN,CU,SM ,JN,CU,SM							50 mm Diam. PVC Pipe
		End of Drillhole		16.55																
17																				
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DEPTH SCALE

1 : 75

Golder Associates

LOGGED: JD

CHECKED: NRL

PROJECT: 1522569-10000

RECORD OF BOREHOLE: 15-03

SHEET 1 OF 2

LOCATION: N 5030960.6 ;E 452208.1

BORING DATE: October 19, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 / SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} / WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		74.54							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE		0.10							Flush Mount Protective Casing
		FILL - (SM) gravelly SILTY SAND; grey brown; non-cohesive, moist, loose		73.24 1.30							Bentonite Seal
1					1	SS	9				
		Moderately to slightly weathered SHALE BEDROCK		1.52							
2		Borehole continued on RECORD OF DRILLHOLE 15-03									

3 4 5 6 7 8 9 10 11 12 13 14 15

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: JD

CHECKED: NRL

MIS-BHS 001 1522569-10000.GPJ GAL-MIS.GDT 11/16/15 JEM

PROJECT: 1522569-10000

RECORD OF DRILLHOLE: 15-03

SHEET 2 OF 2

LOCATION: N 5030960.6 ;E 452208.1

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: October 19, 2015

DRILL RIG: Truck Mount

DRILLING CONTRACTOR: Downing Drilling

DATUM: Geodetic

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

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.
		BEDROCK SURFACE	73.24	
	Power Auger 200 mm Diam.	See RECORD OF BOREHOLE 15-03	1.30	
		- broken rock from 1.50 to 1.64 m depth	1.52	
2		Moderately weathered to fresh, laminated to thinly bedded, grey, porous, weak to strong SHALE BEDROCK, with occasional thin laminates of limestone		C1
3		- broken rock from 2.82 to 2.93 m depth		
		- broken rock from 3.59 to 3.66 m depth		C2
4	Rotary Drill NQ Core	- becoming less weathered UCS = 33.7 MPa UCS = 45.2 MPa		
5		UCS = 30.6 MPa UCS = 39.2 MPa		C3
6		UCS = 45.6 MPa		
7			66.97	C4
		End of Drillhole	7.57	
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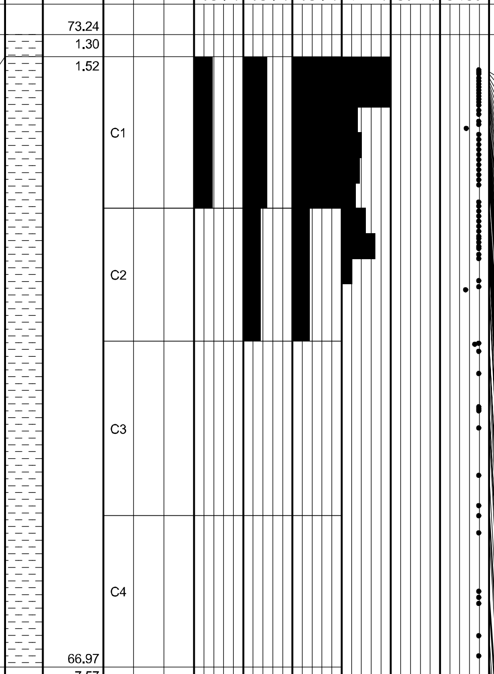

TYPE AND SURFACE DESCRIPTION

,BD,PI,Sm
,BD,PI,Sm
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,BD,PI,Sm
,BD,PI,Sm
,JN,Cu,Sm
,BD,PI,Sm
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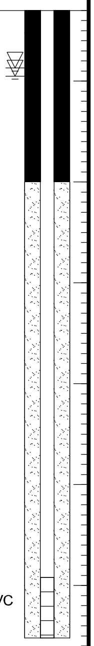
HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}

Diametral Point Load Index (MPa) 2 4 6

RMC -Q' AVG.



Bentonite Seal

Bentonite Seal

Silica Sand

19 mm Diam. PVC Slot Screen

W.L. in Screen at Elev. 72.67 m on October 14, 2015

W.L. in Screen at Elev. 72.59 m on November 11, 2015

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: JD

CHECKED: NRL

PROJECT: 1522569-10000

RECORD OF BOREHOLE: 15-04

SHEET 1 OF 2

LOCATION: N 5030984.9 ;E 452113.2

BORING DATE: October 20, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 / SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} / WATER CONTENT PERCENT Wp ├──○W──┤ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	74.83							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE	0.08							
		FILL - (SM) gravelly SILTY SAND; grey brown; non-cohesive, moist, compact								
1				1	SS	18				
		Moderately to slightly weathered SHALE BEDROCK	73.15 1.68	2	SS	>50				
2										
			72.39 2.44	3	SS	>50				
		Borehole continued on RECORD OF DRILLHOLE 15-04								
3										
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11										
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13										
14										
15										

DEPTH SCALE

1 : 75

LOGGED: JD

CHECKED: NRL

PROJECT: 1522569-10000

RECORD OF DRILLHOLE: 15-04

SHEET 2 OF 2

LOCATION: N 5030984.9 ;E 452113.2

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: October 20, 2015

DRILL RIG: Truck Mount

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 FLUSH	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25 m 5 10 15 20	DISCONTINUITY DATA B Angle 0 90 180 270	DIP w.r.t. CORE AXIS 0 30 60 90	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) 2 4 6	RMC -Q' AVG.
		BEDROCK SURFACE		73.15															
2	Power Auger 200 mm Diam.	See RECORD OF BOREHOLE 15-04		1.68 72.39															
3 4	Rotary Drill NQ Core	Moderately weathered, laminated to thinly bedded, dark grey, weak SHALE BEDROCK		2.44 70.62	C1														
5 6 7 8 9 10 11 12 13 14 15 16		End of Drillhole		4.21															

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: JD

CHECKED: NRL

PROJECT: 1522569-10000

RECORD OF BOREHOLE: 15-05

SHEET 1 OF 1

LOCATION: N 5030984.7 ;E 452172.1

BORING DATE: October 20, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 / SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} / WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		74.70							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE	■								
		FILL - (GW) gravelly SAND, trace fines; grey brown; non-cohesive, moist, loose	▩	0.10							
1					1	SS	8				
		FILL - (SM) SILTY SAND, some gravel; grey; non-cohesive, moist, compact	▩	73.18 1.52	2	SS	17				
2		Moderately to slightly weathered, laminated to thinly bedded, dark grey, weak SHALE BEDROCK	☰	72.64 2.06							
					3	SS	33				
3					4	SS	>50				
		End of Borehole		71.60 3.10							
4											
5											
6											
7											
8											
9											
10											
11											
12											
13											
14											
15											

MIS-BHS 001 1522569-10000.GPJ GAL-MIS.GDT 11/16/15 JEM

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: JD

CHECKED: NRL

PROJECT: 1522569-10000

RECORD OF BOREHOLE: 15-06

SHEET 1 OF 1

LOCATION: N 5030948.7 ;E 452105.8

BORING DATE: October 20, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80; SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3}; WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		74.63							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE	■	0.08							
		FILL - (SM) gravelly SILTY SAND; grey brown; non-cohesive, moist, compact	▨	72.88 1.75							
1					1	SS	11		○	M	
2		Moderately to slightly weathered, laminated to thinly bedded, dark grey, weak SHALE BEDROCK		68.53 6.10	2	SS	>50				
					3	SS	>50				
3					4	SS	>50				
4					5	SS	>50				
5					6	SS	>50				
					7	SS	>50				
6		End of Borehole									
7											
8											
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11											
12											
13											
14											
15											

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: JD

CHECKED: NRL

PROJECT: 1522569-10000

RECORD OF BOREHOLE: 15-07

SHEET 1 OF 2

LOCATION: N 5030919.2 ;E 452068.0

BORING DATE: October 20, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		74.79							
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE	■	0.08							
1		FILL - (SM) gravelly SILTY SAND, some gravel; grey brown; non-cohesive, moist, loose	▨	72.80 1.99	1 2	SS SS	9 6				
2		(ML) sandy CLAYEY SILT, trace gravel; grey to black, contains organics, (GLACIAL TILL); non-cohesive; moist		72.50 2.29							
		Moderately to slightly weathered SHALE BEDROCK		71.74 3.05	3	SS	>50				
3		Borehole continued on RECORD OF DRILLHOLE 15-07									
4											
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7											
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11											
12											
13											
14											
15											

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: JD

CHECKED: NRL

PROJECT: 1522569-10000

RECORD OF DRILLHOLE: 15-07

SHEET 2 OF 2

LOCATION: N 5030919.2 ;E 452068.0

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: October 20, 2015

DRILL RIG: Truck Mount

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 / FLUSH	RECOVERY TOTAL CORE % (80 60 40 20)	RECOVERY SOLID CORE % (80 60 40 20)	R.Q.D. % (80 60 40 20)	FRACT. INDEX PER 0.25 m (5 10 15 20)	DISCONTINUITY DATA B Angle (0 90 180 270)	DIP w.r.t. CORE AXIS (0 30 60 90)	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) (2 4 6)	RMC -Q' AVG.
		BEDROCK SURFACE																	
	Power Auger 200 mm Diam.	See RECORD OF BOREHOLE 15-07		72.50 2.29															
3	Rotary Drill NQ Core	Moderately to slightly weathered, laminated to thinly bedded, dark grey, weak SHALE BEDROCK		71.74 3.05															
4					C1														
5		End of Drillhole		69.97 4.82															
6																			
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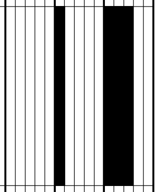
17																			



DEPTH SCALE

1 : 75

Golder Associates

LOGGED: JD

CHECKED: NRL

APPENDIX B

Record of Boreholes – Previous Investigations

10-1121-0222	Boreholes E-080 to E-085
87-12120-1	Boreholes 9 to 17
84-12062	Boreholes 4 and 5

PROJECT: 10-1121-0222

RECORD OF BOREHOLE: E-080

SHEET 1 OF 1

LOCATION: N 5032515.84 ;E 374303.42

BORING DATE: December 10, 2010

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			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^{-8} 10^{-6} 10^{-4} 10^{-2} WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m			
		GROUND SURFACE		72.69						
0	Power Auger 200mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE	■	0.00						
		Dense grey to dark grey fine to coarse sand and gravel, trace silt (Crushed Stone FILL)	▨	0.08	1	50 DO	43			
				71.70	2	50 DO	>50			
1		Borehole continued on RECORD OF DRILLHOLE E-080								
2										
3										
4										
5										
6										
7										
8										
9										
10										

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: CHM

CHECKED: HD

PROJECT: 10-1121-0222

RECORD OF DRILLHOLE: E-080

SHEET 1 OF 1

LOCATION: N 5032515.84 ;E 374303.42

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: December 10, 2010

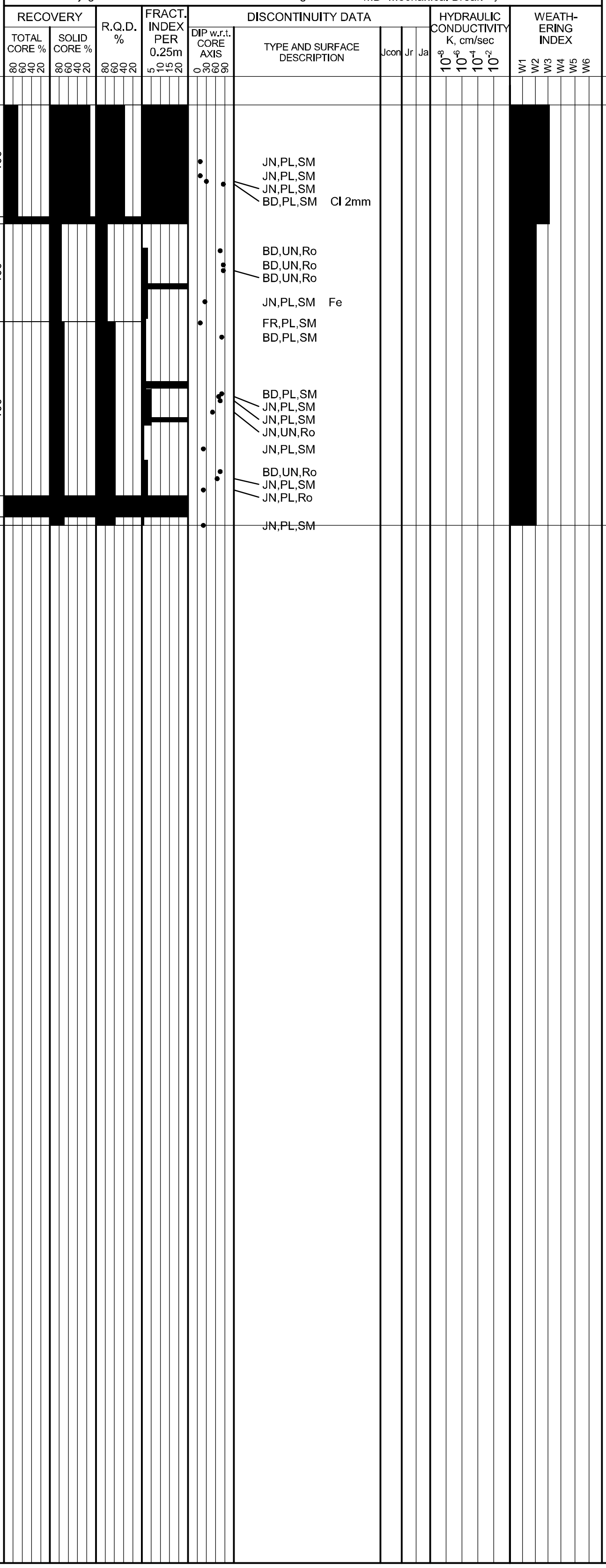
DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	NOTES
		BEDROCK SURFACE		71.70			
1	Rotary Drill NQ Core	Moderately weathered, laminated, black to dark grey, weak SHALE BEDROCK BILLINGS FORMATION - Broken core from 0.99m to 1.68m - No recovery from 1.78m to 1.83m		0.99	1	100	
2		Slightly weathered, laminated, black to dark grey, weak SHALE BEDROCK, interbedded with grey calcareous siltstone laminations - Broken core from 2.25m to 2.29m		70.86 1.83	2	100	
3		- Broken core from 2.94m to 2.99m - Broken core from 3.20m to 3.23m - No recover from 3.75m to 3.90m			3	100	
4		End of Drillhole		68.73 3.96	3		
5							
6							
7							
8							
9							
10							



JN,PL,SM
JN,PL,SM
JN,PL,SM
BD,PL,SM Cl 2mm
BD,UN,Ro
BD,UN,Ro
BD,UN,Ro
JN,PL,SM Fe
FR,PL,SM
BD,PL,SM
BD,PL,SM
JN,PL,SM
JN,PL,SM
JN,UN,Ro
JN,PL,SM
BD,UN,Ro
JN,PL,SM
JN,PL,Ro
JN,PL,SM

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: CHM

CHECKED: HD

PROJECT: 10-1121-0222

RECORD OF BOREHOLE: E-081

SHEET 1 OF 1

LOCATION: N 5032550.12 ;E 374330.90

BORING DATE: December 17, 2010

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/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-8} 10^{-6} 10^{-4} 10^{-2}); WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING
		GROUND SURFACE		72.90						
0	Power Auger 200mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE	■	0.00						
		Dense grey silty fine to coarse sand, some gravel (FILL)	▨	0.10	1	50 DO	74			
1		Very loose dark grey sandy gravel, trace silt (FILL)	▨	72.14 0.76	2	50 DO	4		○	M
2		Highly weathered, dark grey, very weak SHALE BEDROCK	▤	71.20 1.70	3	50 DO	47			
			▤		4	50 DO	39			
3		Borehole continued on RECORD OF DRILLHOLE E-081		69.85						
4										
5										
6										
7										
8										
9										
10										

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: CHM
CHECKED: HD

PROJECT: 10-1121-0222

RECORD OF DRILLHOLE: E-081

SHEET 1 OF 1

LOCATION: N 5032550.12 ;E 374330.90

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: December 17, 2010

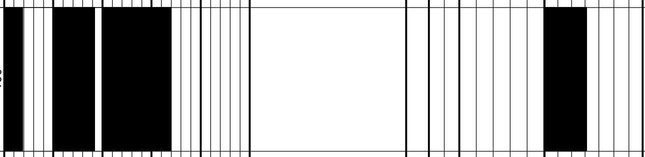
DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

JN - Joint FLT - Fault SH - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Clean	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 RETURN	RECOVERY TOTAL CORE % 80 60 40 20	RECOVERY SOLID CORE % 80 60 40 20	R.Q.D. % 80 60 40 20	FRACT. INDEX PER 0.25m 5 10 15 20	DISCONTINUITY DATA DIP w.r.t. CORE AXIS 0 30 60 90	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-8} 10^{-6} 10^{-4} 10^{-2}	WEATH-ERING INDEX W1 W2 W3 W4 W5 W6	NOTES
		BEDROCK SURFACE at 1.70m		69.85														
	Rotary Drill NQ Core	Moderately weathered, laminated, black to dark grey, weak SHALE BEDROCK BILLINGS FORMATION - Core recovered as broken core		3.05 68.90	1	100												
4		End of Drillhole		4.00														
5																		
6																		
7																		
8																		
9																		
10																		
11																		
12																		
13																		



OLRT-ROCK 1011210222-1300.GPJ GAL-MISS.GDT 06/13/12 JEM/JM

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: CHM

CHECKED: HD

PROJECT: 10-1121-0222

RECORD OF BOREHOLE: E-082

SHEET 1 OF 1

LOCATION: N 5032609.50 ;E 374434.84

BORING DATE: July 14, 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/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-8} 10^{-6} 10^{-4} 10^{-2}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	
		GROUND SURFACE		73.34							
0	Power Auger 200mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE	■	0.00							
		Loose brown to dark brown silty clay, trace gravel (FILL)	▨	0.10	1	50 DO	4		⊢○⊣		
		Highly weathered, dark grey mottled brown, very weak SHALE BEDROCK		72.73 0.61	2	50 DO	>50				
1											
				71.74	3	50 DO	>50				
		End of Borehole Sampler Refusal		1.60							
2											Borehole dry upon completion of drilling
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: JRM

CHECKED: HD

PROJECT: 10-1121-0222

RECORD OF BOREHOLE: E-083

SHEET 1 OF 1

LOCATION: N 5032621.33 ;E 374403.21

BORING DATE: December 3-6, 2010

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/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 / SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-8} 10^{-6} 10^{-4} 10^{-2} / WATER CONTENT PERCENT Wp ├──○W──┤ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	MON. WELL
		GROUND SURFACE		73.70							
0	Power Auger 200mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE	■	0.00							Flush Mount Protective Casing set in Bentonite
		Compact dark brown to dark grey gravel, some sand and silt (Crushed Stone FILL)	▨	0.08							Bentonite Seal
					1	50 DO	20		○	M	Native Backfill
1					2	50 DO	23				
		Highly weathered, dark grey, weak SHALE BEDROCK	▤	72.18 1.52	3	50 DO	>50				W.L. in Screen at Elev. 69.4m on Aug. 5, 2011
				71.97							
2		Borehole continued on RECORD OF DRILLHOLE E-083									
											Bentonite Seal
3											
4											
5											
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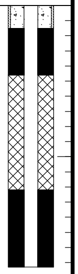
7											
8											
9											
10											



DEPTH SCALE

1 : 50

Golder Associates

LOGGED: CHM

CHECKED: HD

PROJECT: 10-1121-0222

RECORD OF DRILLHOLE: E-083

SHEET 1 OF 1

LOCATION: N 5032621.33 ;E 374403.21

INCLINATION: -90° AZIMUTH: ---

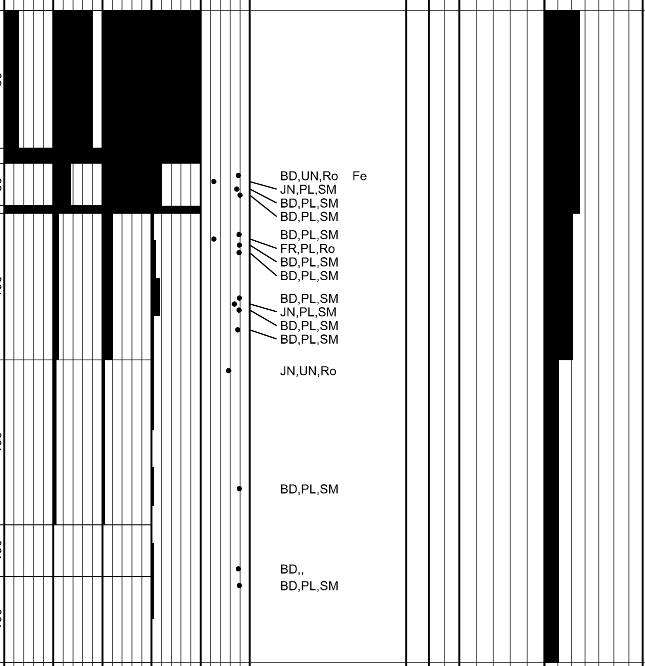
DRILLING DATE: December 3-6, 2010
DRILL RIG: CME 55
DRILLING CONTRACTOR: Marathon Drilling

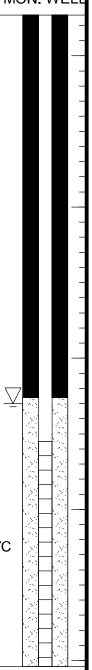
DATUM: Geodetic

JN - Joint
FLT - Fault
SH - Shear
VN - Vein
CJ - Conjugate
BD - Bedding
FO - Foliation
CO - Contact
OR - Orthogonal
CL - Clean
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.

DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES
	BEDROCK SURFACE at 1.52m					MON. WELL
Rotary Drill NQ Core	Moderately to slightly weathered, laminated, dark grey to black, weak SHALE BEDROCK, interbedded with grey calcareous siltstone laminations BILLINGS FORMATION - Broken rock from 1.73m to 2.64m - No recovery from 2.64m to 2.74m - No recovery from 3.02m to 3.07m	71.97 1.73	1 2	80 80	BD,UN,Ro Fe JN,PL,SM BD,PL,SM BD,PL,SM	Bentonite Seal
	Slightly weathered to fresh, laminated, dark grey to black, weak SHALE BEDROCK, interbedded with grey calcareous siltstone laminations	70.63 3.07	3	100	BD,PL,SM FR,PL,Ro BD,PL,SM BD,PL,SM BD,PL,SM JN,PL,SM BD,PL,SM BD,PL,SM	
	Fresh, laminated, dark grey to black, weak SHALE BEDROCK, interbedded with grey calcareous siltstone laminations	69.66 4.04	4 5 6	100 100 100	JN,UN,Ro BD,PL,SM BD,, BD,PL,SM	Silica Sand 32mm Diam. PVC #10 Slot Screen
	End of Drillhole	67.66 6.04				W.L. in Screen at Elev. 69.4m on Aug. 5, 2011





OLRT-ROCK 1011210222-1300.GPJ GAL-MISS.GDT 06/13/12 JEM/JM

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: CHM
CHECKED: HD

PROJECT: 10-1121-0222

RECORD OF BOREHOLE: E-084

SHEET 1 OF 1

LOCATION: N 5032714.84 ;E 374443.69

BORING DATE: July 14, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	SOIL PROFILE: STRATA PLOT	SOIL PROFILE: ELEV. / DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-8} 10^{-6} 10^{-4} 10^{-2}) / WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	
		GROUND SURFACE		73.84							
0	Power Auger 200mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE	■	0.00							
		Dense grey sand and gravel, trace silt (Crushed Stone FILL)	▨	73.66 0.18	1	50 DO	59		○	M	
					2	50 DO	>50				
1		Highly weathered, dark grey, very weak SHALE BEDROCK	▤	72.93 0.91							
					3	50 DO	>50				
		End of Borehole		72.19 1.65							
2											Borehole dry upon completion of drilling
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: JRM

CHECKED: HD

PROJECT: 10-1121-0222

RECORD OF BOREHOLE: E-085

SHEET 1 OF 1

LOCATION: N 5032692.59 ;E 374469.43

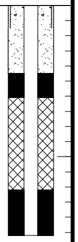
BORING DATE: December 2-3, 2010

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/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-8} 10^{-6} 10^{-4} 10^{-2}) WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	MON. WELL
		GROUND SURFACE		73.63							
0	Power Auger 200mm Diam. (Hollow Stem)	TOPSOIL		0.00							
		Compact brown to grey gravelly fine to coarse sand, some silt (FILL)		0.08 73.02	1	50 DO	16		○	M	Flush Mount Protective Casing set in Bentonite Silica Sand Bentonite Seal
1		Compact grey to dark grey gravel and sand (FILL)		0.61 72.41	2	50 DO	19				Native Backfill
		Highly weathered, dark grey, weak SHALE BEDROCK		1.22 72.11							W.L. in Screen at Elev. 71.1m on Aug. 5, 2011
		Borehole continued on RECORD OF DRILLHOLE E-085									
2											
											Bentonite Seal
3											
4											
5											
6											
7											
8											
9											
10											



DEPTH SCALE
1 : 50

Golder Associates

LOGGED: CHM
CHECKED: HD

PROJECT: 10-1121-0222

RECORD OF DRILLHOLE: E-085

SHEET 1 OF 1

LOCATION: N 5032692.59 ;E 374469.43

DRILLING DATE: December 2-3, 2010

DATUM: Geodetic

DRILL RIG: CME 55

INCLINATION: -90° AZIMUTH: —

DRILLING CONTRACTOR: Marathon Drilling

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

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

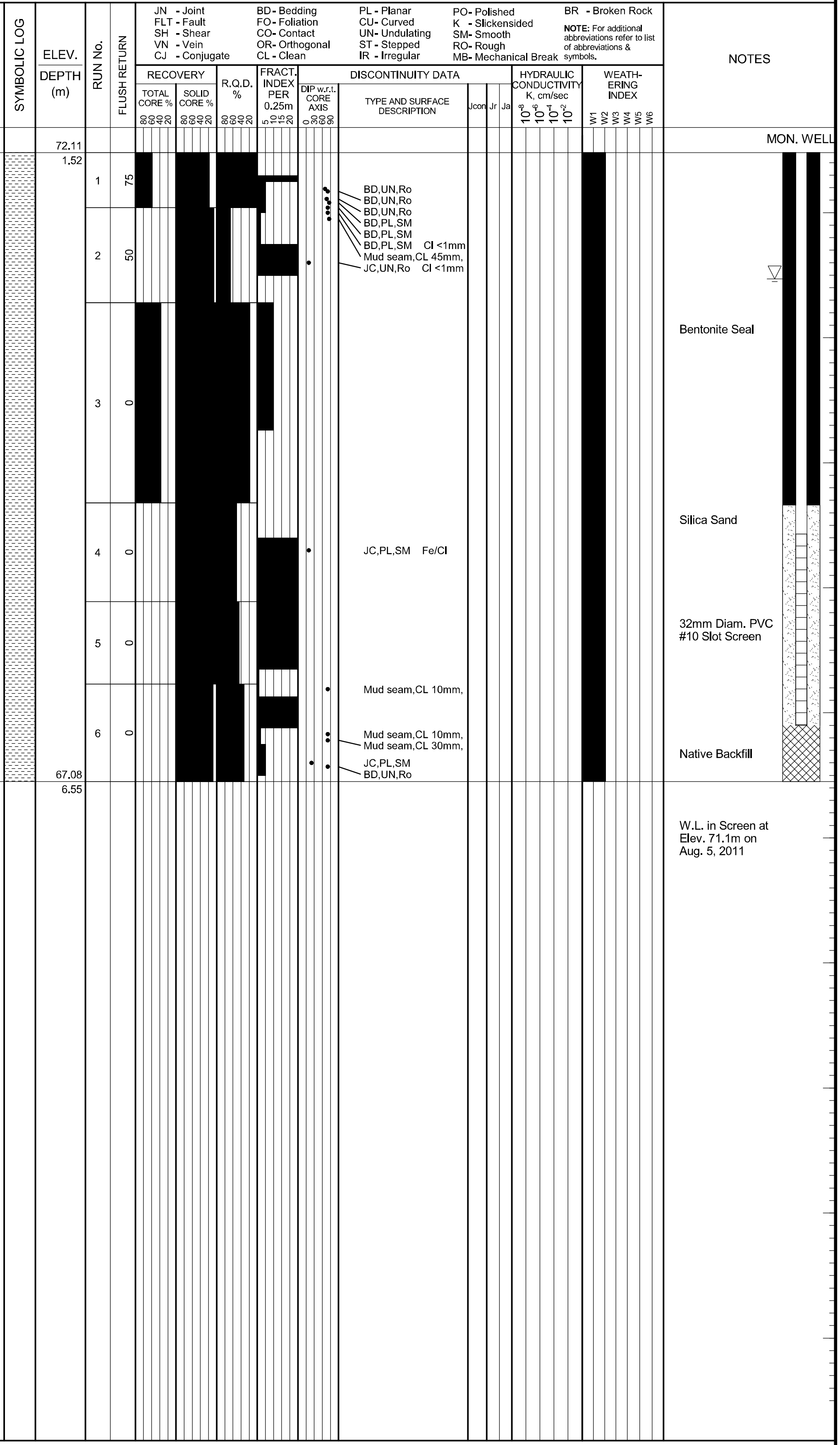
PL - Planar
CU - Curved
UN - Undulating
ST - Stepped
IR - Irregular

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

BR - Broken Rock

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN
		BEDROCK SURFACE at 1.22m	72.11		
	Rotary Drill NQ Core	Slightly weathered, laminated, dark grey, weak SHALE BEDROCK, interbedded with grey calcareous siltstone laminations BILLINGS FORMATION - Broken core from 1.70m to 1.80m - Broken core from 2.25m to 2.47m - Mechanical breaks along vertical/sub-vertical joints, rock recovered as broken in some sections from 2.72m to 3.74m - Broken core from 4.60m to 5.65m - Clay band from 5.87m to 6.12m	1.52	1	75
2				2	50
3				3	0
4				4	0
5				5	0
6			67.08 6.55	6	0
		End of Drillhole			
7					
8					
9					
10					
11					



BD,UN,Ro
BD,UN,Ro
BD,UN,Ro
BD,PL,SM
BD,PL,SM
BD,PL,SM Cl <1mm
Mud seam,CL 45mm,
JC,UN,Ro Cl <1mm
JC,PL,SM Fe/Cl
Mud seam,CL 10mm,
Mud seam,CL 10mm,
Mud seam,CL 30mm,
JC,PL,SM
BD,UN,Ro

Bentonite Seal

Silica Sand

32mm Diam. PVC
#10 Slot Screen

Native Backfill

W.L. in Screen at
Elev. 71.1m on
Aug. 5, 2011

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: CHM
CHECKED: HD

8712120-1

PRC 871-2

RECORD OF BOREHOLE 9

SHEET 1 of 1

LOCATION See Figure 2

BORING DATE April.22,1987

DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

STA. 13+159, 7.0m Lt. of CL.

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (M)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.3M	CORE RECOVERY (%)	R.Q.D. (%)	S.C.R. (%)	PIEZOMETER OR STANDPIPE INSTALLATION
0		Ground Surface	73.62							
	POWER AUGER 200mm DIAM. (HOLLOW STEM)	FILL	0.00 73.47							Bentonite Seal
		TOPSOIL	0.15 73.35							Backfill
		Compact grey brown CLAYEY SILT, trace organic matter	0.27 73.01	1	50 DO	12				
		Dense sandy silt (GLACIAL TILL	0.61 72.86							
1		Brown to grey brown highly weathered SHALE BEDROCK	0.76 72.28	2	50 DO	70				
2	ROTARY DRILLING BXL CORE	Fresh to faintly weathered laminated dark grey SHALE BEDROCK. Occasional near vertical joint. Bedding near horizontal. Upper 0.3m of bedrock is fractured (BILLINGS FORMATION)	1.34	3	BX RC	--	92	19	48	
3				4	BX RC	--	98	39	89	
4			69.51							
		End of Hole	4.11							
5										W.L in Standpipe at Elev.71.94 May.5,1987

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa nat.V.- + Q.- ● rem.V.- ⊕ U.- ○

HYDRAULIC CONDUCTIVITY, k, CM/SEC

WATER CONTENT, PERCENT Wp W Wl 20 40 60 80

ADDITIONAL LAB. TESTING

0
15 -⊕- 5 PERCENT AXIAL STRAIN AT FAILURE
10

DEPTH SCALE
1 : 25

Golder Associates

LOGGED S.L

CHECKED AC

8712120-1

PRO 871-2

RECORD OF BOREHOLE 10

SHEET 1 of 1

LOCATION See Figure 2

BORING DATE April.22,1987

DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa nat.V.- + Q.- ● rem.V.- ⊕ U.- ○

HYDRAULIC CONDUCTIVITY, k, CM/SEC

WATER CONTENT, PERCENT Wp W Wl 20 40 60 80

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES NUMBER	TYPE	BLOWS/0.3M	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		Ground Surface		73.87					
	Power Auger 200mm Diam (Hollow Stem)	Dark brown to black organic clayey silt (FILL)		0.00 73.26	1	AS	-		
		TOPSOIL		0.61 73.11	2	AS	-		
1		Very stiff grey brown SILTY CLAY and CLAYEY SILT		0.76 72.74	3	50 DO	17		
		Brown to grey brown highly weathered SHALE BEDROCK		1.13 72.35					▽
		Moderately to slightly weathered dark grey SHALE BEDROCK		1.52 71.89	4	50 DO	40/150 mm		
2		End of Hole Auger Refusal		1.98					
3									
4									
5									

STA. 13+221, 7.3m Lt of CL.

W.L. in Open Hole at Elev.72.50 May.5,1987

0
15 -⊕- 5 PERCENT AXIAL STRAIN AT FAILURE
10

DEPTH SCALE
1 : 25

Golder Associates

LOGGED S.L

CHECKED

8712120-1

PROJECT 871-2120

RECORD OF BOREHOLE 11

SHEET 1 of 1

LOCATION See Figure 2

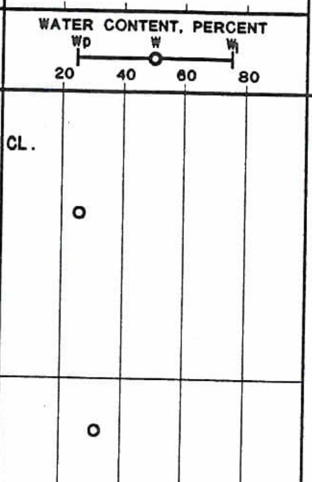
BORING DATE April 22,1987

DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (M)	SAMPLES: NUMBER	TYPE	BLOWS/0.3M	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa nat.V.- + Q.- ● rem.V.- ⊕ U.- ○	HYDRAULIC CONDUCTIVITY, k, CM/SEC; WATER CONTENT, PERCENT Wp W Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		Ground Surface		73.86							
	Power Auger 200mm Diam (Hollow Stem)	Very stiff grey brown silty clay, some shale fragments, trace organic matter (FILL)	▨	0.00	1	50 DO	5	STA. 13+280, 4.8m Lt. of CL.	○		
		TOPSOIL		73.24 0.61							
1		Stiff grey brown SILTY CLAY and CLAYEY SILT, trace organic matter		73.06 0.79							
		Grey brown highly weathered SHALE BEDROCK		72.81 1.04	2	50 DO	46		○		
		End of Hole Auger Refusal		72.33 1.52							
2											Open Hole dry May 5, 1987
3											
4											
5											



DEPTH SCALE
1 : 25

0
15 ⊕ 5 PERCENT AXIAL STRAIN AT FAILURE
10

Golder Associates

LOGGED S.L
CHECKED

8712120-1

RECORD OF BOREHOLE 12

LOCATION See Figure 2
SAMPLER HAMMER, 63.5kg, DROP, 760mm
BORING DATE April.22,1987
SHEET 1 of 1
DATUM GEODETIC
PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

PROJECT 871-2

STA. 13+342, 4.2m Lt. of CL.

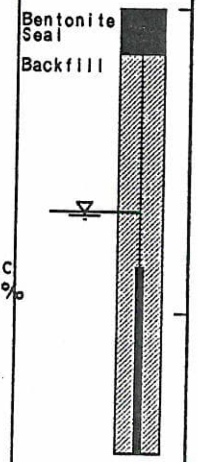
DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (M)	SAMPLES NUMBER	TYPE	BLOWS/0.3M	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200mm Diam (Hollow Stem)	Ground Surface	73.27					
		Grey brown and black silty clay and clayey silt, some organic matter (FILL)	0.00	1	50 DO	1		Bentonite Seal Backfill
		Black PEAT	72.60 0.67					
1		Highly becoming moderately weathered dark grey SHALE BEDROCK	72.27 1.00	2	50 DO	46	Org.C 19.5%	
		End of Hole Auger Refusal	71.81 1.46					
2								W.L. in Standpipe at Elev.72.60 May.5,1987
3								
4								
5								

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa nat.V.- + Q.- ● rem.V.- ⊕ U.- ○

HYDRAULIC CONDUCTIVITY, k, CM/SEC

WATER CONTENT, PERCENT Wp W Wl 20 40 60 80



15 ⊕ 5 PERCENT AXIAL STRAIN AT FAILURE 0 10

DEPTH SCALE
1 : 25

Golder Associates

LOGGED S.L
CHECKED AC

8712120-1

PRO. 871-21.

RECORD OF BOREHOLE 17

SHEET 1 of 1

LOCATION See Figure 2

BORING DATE April 22, 1987

DATUM GEODETIC

SAMPLER HAMMER, 63.5kg, DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg, DROP, 760mm

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa nat.V.- + Q.- ● rem.V.- ⊕ U.- ○

HYDRAULIC CONDUCTIVITY, k, CM/SEC

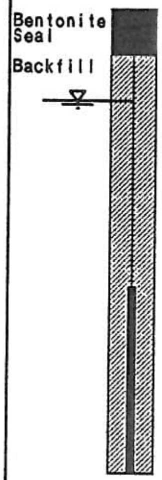
WATER CONTENT, PERCENT Wp W Wl 20 40 60 80

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. DEPTH (M)	NUMBER	TYPE	BLOWS/0.3M
0		Ground Surface	73.75			
	Power Auger 200mm Diam. (Hollow Stem)	TOPSOIL	0.00 73.57			
		Very stiff grey brown SILTY CLAY AND CLAYEY SILT	0.18 73.35	1	50 DO	5
		Grey brown sandy silt, some shale fragments, trace clay (GLACIAL TILL)	0.40 73.14			
1		Moderately to slightly weathered dark grey SHALE BEDROCK	0.61 72.23	2	AS	--
		End of Hole Auger Refusal	1.52			
2						
3						
4						
5						

STA. 13+397, 53.5m Lt. of CL.

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



W.L. in Standpipe at Elev. 73.45 May 5, 1987

0
16-⊕-5 PERCENT AXIAL STRAIN AT FAILURE
10

DEPTH SCALE
1 : 25

Golder Associates

LOGGED J.C.
CHECKED AC

RECORD OF BOREHOLES 3 ≑ 4

84-12062

LOCATION See Figure 2

BORING DATE MARCH 27, 1984

DATUM GEODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm

PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa NAT. V. - + Q. - ● REM. V. - ⊕ U. - O

HYDRAULIC CONDUCTIVITY, k, cm/sec. 1x10 1x10 1x10 1x10

WATER CONTENT, PERCENT Wp W WL

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATIO

BH. 3

BORING METHOD	ELEV'N. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/0.3m	ELEVATION SCALE
POWER AUGER 200mm DIAM. (HOLLOW STEM)	73.5±	GROUND SURFACE				74
	0.00	TOPSOIL				
	0.21 72.89	LOOSE BROWN SILTY SAND	1	C.S.		73
	0.61 71.52	DENSE TO VERY DENSE BROWN TO DARK GREY SANDY SILT, SOME CLAY, GRAVEL AND SHALE FRAGMENTS (SANDY SILT TILL)	2 3	50 mm D.O. "	31 81	72
	1.98 70.60	WEATHERED BLACK SHALE	4	"	>100	71
	2.90	END OF HOLE AUGER REFUSAL SHALE BEDROCK				70

GROUND SURFACE

NATIVE BACKFILL

PLASTIC TUBING

STANDPIPE

W.L. IN STANDPI
AT ELEV. 73.
APRIL 12, 19

BH. 4

BORING METHOD	ELEV'N. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/0.3m	ELEVATION SCALE
POWER AUGER 200mm DIAM. (HOLLOW STEM)	73.8±	GROUND SURFACE				74
	0.00	TOPSOIL				
	0.15 73.19	BROWN CLAYEY SILT, SOME SAND AND SHALE FRAGMENTS	1	A.S.	–	
	0.61 72.28	WEATHERED BLACK SHALE	2	50 mm D.O.	>100	73
	1.52	END OF HOLE				72

W.L. IN OPEN HO
AT ELEV. 73.
APRIL 12, 19

15 0 5 10 Percent axial strain at failure

VERTICAL SCALE 1:50

Golder Associates

DRAWN J.C.

CHECKED

RECORD OF BOREHOLES 5 & 6

84-12062

LOCATION See Figure 2

BORING DATE MARCH 27, 1984

DATUM GEODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm

PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m

SHEAR STRENGTH Cu, kPa — NAT. V. - + Q. - ● REM. V. - ⊕ U. - ○

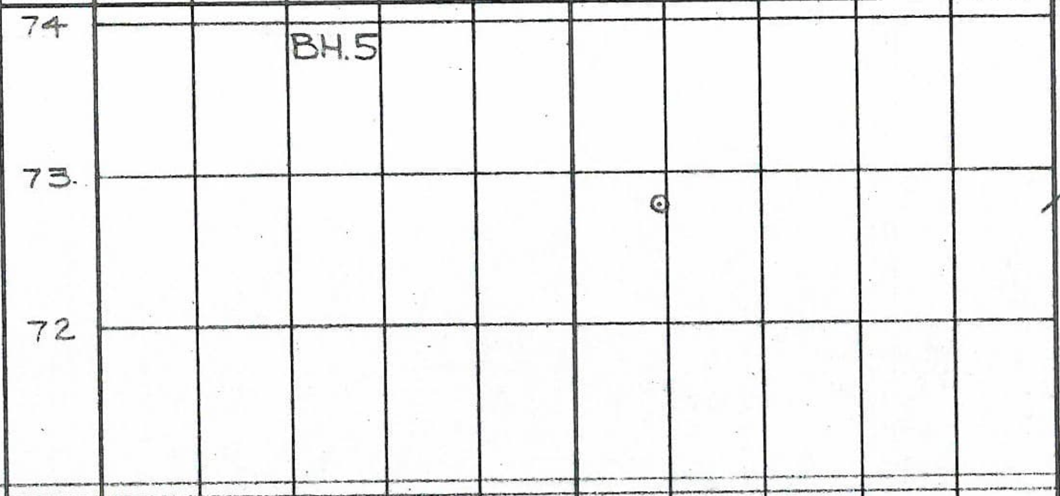
HYDRAULIC CONDUCTIVITY, k, cm/sec. 1x10 1x10 1x10 1x10

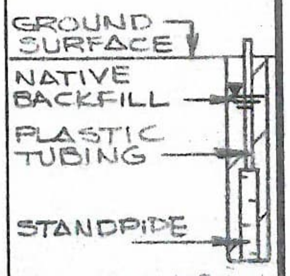
WATER CONTENT, PERCENT Wp W WL 20 40 60 80

BH. 5

200mm DIAM. (HOLLOW STEM)

ELEV'N. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
73.2±	GROUND SURFACE				
0.00	**TOPSOIL**				
0.15	LOOSE BROWN **CLAYEY SILT** WITH SAND AND SHALE FRAGMENTS	1	50 mm O.D.	4	MH
0.40	DARK GREY SANDY SILT, SOME CLAY, GRAVEL AND SHALE **(SANDY SILT TILL)**	2	"	32	
0.62	WEATHERED **BLACK SHALE**				
1.37	END OF HOLE AUGER REFUSAL, SHALE BEDROCK				



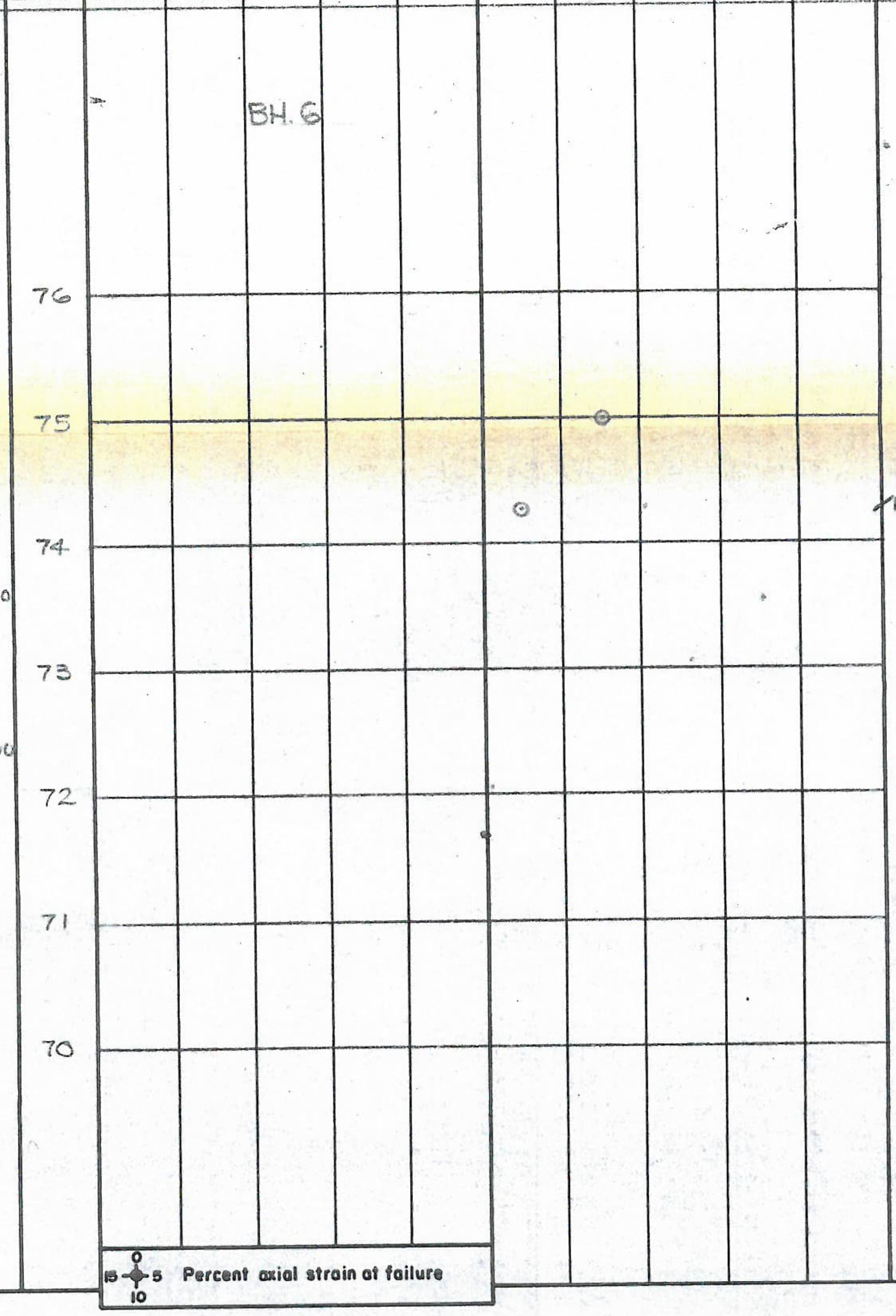


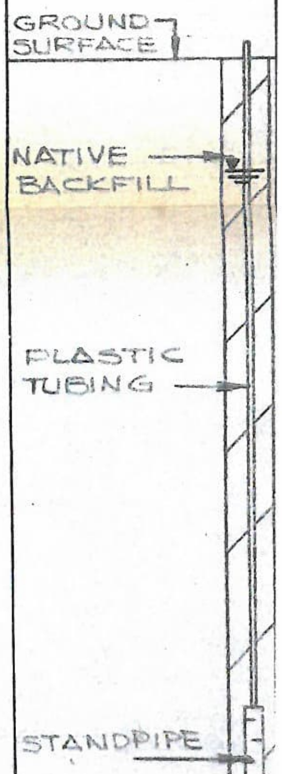
W.L. IN STANDPIPE AT ELEV. 72.93 APRIL 12, 1984

BH. 6

200mm DIAM. (HOLLOW STEM)

ELEV'N. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
75.9±	GROUND SURFACE				
0.00	**TOPSOIL**				
0.24	BROWN **SILTY SAND** SOME GRAVEL	1	50 mm O.D.	9	
74.99 0.91	COMPACT TO VERY DENSE BROWN TO DARK GREY SILTY SAND, SOME CLAY, GRAVEL, SHALE FRAGMENTS, AND SILTY SAND POCKETS **(SILTY SAND TILL)**	2	"	75	MH
		3	"	>100	
		4	"	>100	
71.88 4.02	WEATHERED **BLACK SHALE**				
71.02 4.88	END OF HOLE AUGER REFUSAL SHALE BEDROCK				





WL. IN STANDPIPE AT ELEV. 75.17 APRIL 12, 1984

Percent axial strain at failure

VERTICAL SCALE 1:50

Golder Associates

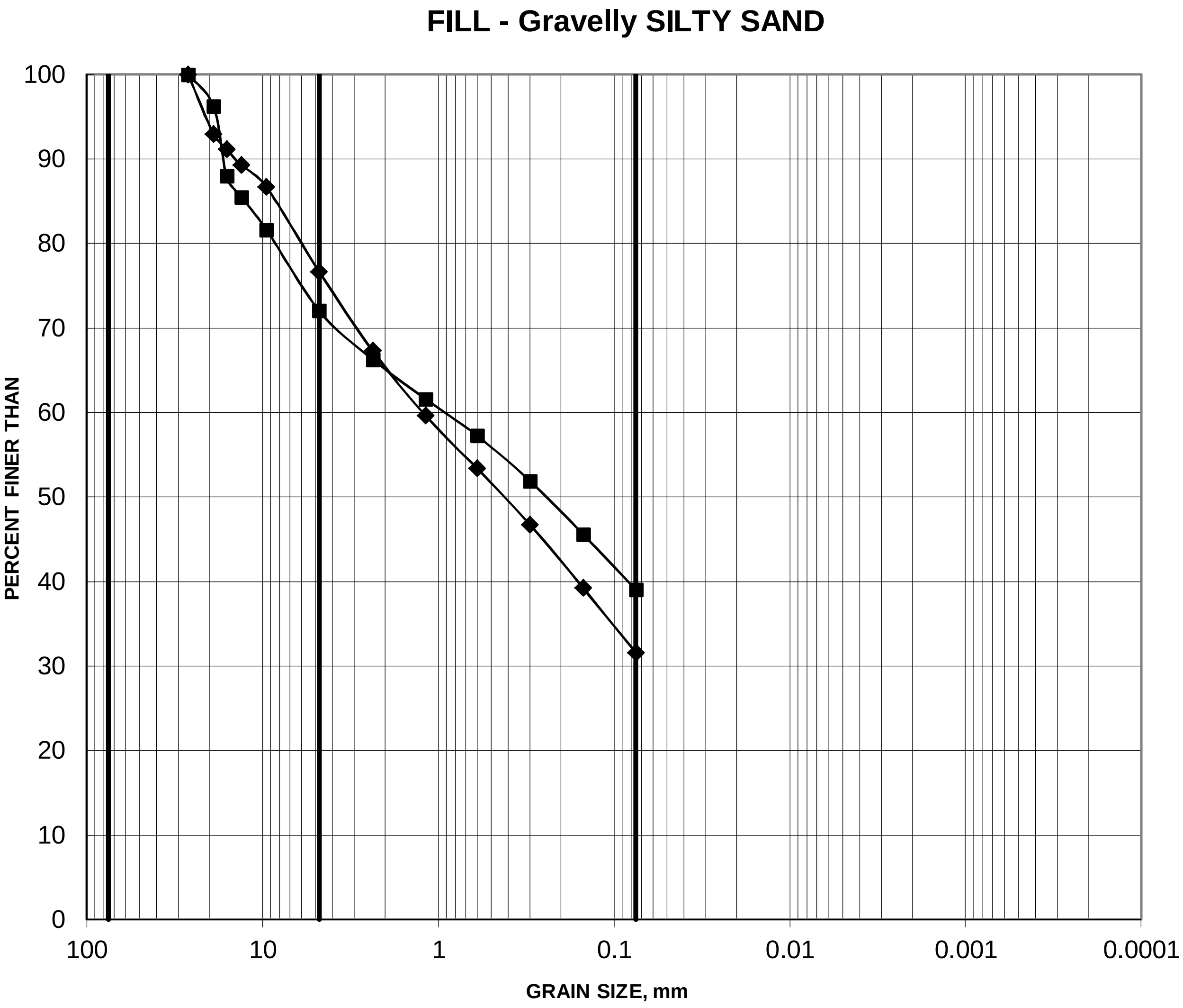
DRAWN J.C.

CHECKED

APPENDIX C

Results of Laboratory Testing

GRAIN SIZE DISTRIBUTION



Cobble	coarse	fine	coarse	medium	fine	SILT AND CLAY
Size	GRAVEL SIZE		SAND SIZE			

Borehole	Sample	Depth (m)
—■— 15-1	1	0.76-1.37
—◆— 15-6	1	0.76-1.37

Created by:	CNM
Checked by:	MI

Project: 1522569/10001

Golder Associates

APPENDIX D

Results of UCS Rock Testing

Golder Associates Ltd.
1931 Robertson Road
Ottawa, Ontario
K2H 5B7

UNCONFINED COMPRESSIVE STRENGTH OF ROCK CORE

Project: RioCan - Various Sites - City Park Site

Project No.: 1522569 /10000

Client: RioCan Management Inc.

Date: November 11, 2015

Location(s): See table below

Bore Hole No.	Depth (m)	Date Tested	Core Size	Diameter (mm)	Density (kg/m³)	Compressive Strength (MPa)	Water Content
15-3	3.59-3.69	Oct 20/15	NQ	47.2	2683	33.7	2.5%
15-2	4.99-5.15	Oct 20/15	HQ	63.2	2638	31.3	2.4%
15-2	13.09-13.22	Oct 20/15	HQ	63.0	2619	35.3	2.9%

REMARKS :
- Rock formation : Billings (swelling shale).
- Cores tested in vertical direction.
- Cores tested in as-received moisture condition, as quickly as possible after unwrapping.
- Specimen ends prepared with sulfur compound, but un-restrained.
- L/D ratio's between 2.2:1 and 2.5:1

TESTING WAS CARRIED OUT IN GENERAL ACCORDANCE WITH ASTM D7012 - Method C

SIGNED:

C.N.Mangione P.Eng.

GEOMECHANICA

Geomechanica Inc.
36 Nesbitt Drive
Toronto Ontario
Canada M4W 2G3

November 6, 2015

Mr. Mark Telesnicki
Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario
Canada L5N 7K2

Re: Billings Shale UCS - Golder Associates Project No. 1522569

Dear Mr. Telesnicki:

On Novemebr 3, 2015 a single shipment of twenty (6) NQ and 3 (HQ) rock core samples was received and identified as being Billings Shale from Golder Associates Project No. 1522569. From these samples, a total of nine (9) test specimens were to be prepared and tested in unconfined compression with the tangent elastic modulus measured. Due to breakage during specimen preparation samples 15-2-2 and 15-2-3 could not be tested. To complete a total of nine (9) UCS tests w/ modulus , two samples were prepared and tested from samples 15-1-3 and 15-3-2

Sample preparation was completed in Geomechanica's laboratory using a diamond core saw and surface grinder. Failure tests were conducted within Geomechanica's rock testing laboratory in in Vaughan Ontario using a 100 ton Enerpac hydraulic testing frame under consistent rates of axial strain controlled by a Lynch 2-speed pressure compensated flow control valve (axial strain rates of approximately 4x 10^{-5} s-^{1}).

The steps of specimen preparation and testing are summarized as follows:

- Diamond cutting of rock cores to obtain cylindrical samples with appropriate length (length:diameter = 2:1) and nearly parallel end faces.
- Abrasive grinding to ensure end faces were flat and parallel within +/- 0.05 mm.
- Axial loading to rupture using a stiff loading frame while recording axial force and axial deformation to measure the UCS and tangent Young's modulus.

The above procedure along with the test results and photographs of each specimen before and after testing has been presented in an accompanying laboratory report.

Sincerely,

Giovanni Grasselli Ph.D., P. Eng.

Geomechanica Inc.
Tel: (647) 478-9767
Email: giovanni.grasselli@geomechanica.com

Rock Laboratory Testing Results

A report submitted to:

Aynsley Neufeld
Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario
Canada L5N 7K2

Prepared by:

Bryan Tatone, PhD
Omid Mahabadi, PhD
Giovanni Grasselli, PhD, PEng

Geomechanica Inc
#300-90 Adelaide St W
Toronto ON
M5H 3V9 Canada
Tel: +1-647-478-9767
info@geomechanica.com

November 6, 2015
Project number: 1522569

Abstract

This document summarizes the results of rock laboratory testing of Billings Shale NQ and HQ core samples under unconfined uniaxial compression. Results including uniaxial compressive strength (UCS) and Young's modulus along with photographs of samples before and after testing are presented herein.

In this document:

1 Overview

This report summarizes the results of rock laboratory testing of Billings Shale NQ and HQ core samples under unconfined uniaxial compression. The tests were performed at Geomechanica's rock testing laboratory in Vaughan Ontario using a stiff loading frame (Figure 1) under axial strain rates of approximately 3.5×10^{-5} s^{-1}. The specimens were prepared and tested according to the following procedure:

1. Diamond cutting of rock cores to obtain cylindrical samples with appropriate length (length:diameter = 2:1) and nearly parallel end faces.
2. Diamond grinding to ensure end faces were flat and parallel within ± 0.05 mm.
3. Axial loading to rupture using a stiff loading frame while recording axial force and axial deformation to determine peak strength (UCS) and (tangent) Young's modulus (E).

Note that prior to cutting and grinding the core samples were wrapped tightly with electrical tape such that the integrity of the core could be maintained prior to testing. With test specimen mounted in the loading frame with a small axial load applied (0.5 kN), the electrical tape was removed and the specimen was subsequently loaded to failure. With this approach it was not possible to obtain the mass of each test specimen prior to testing, thus density measurements could not be obtained.

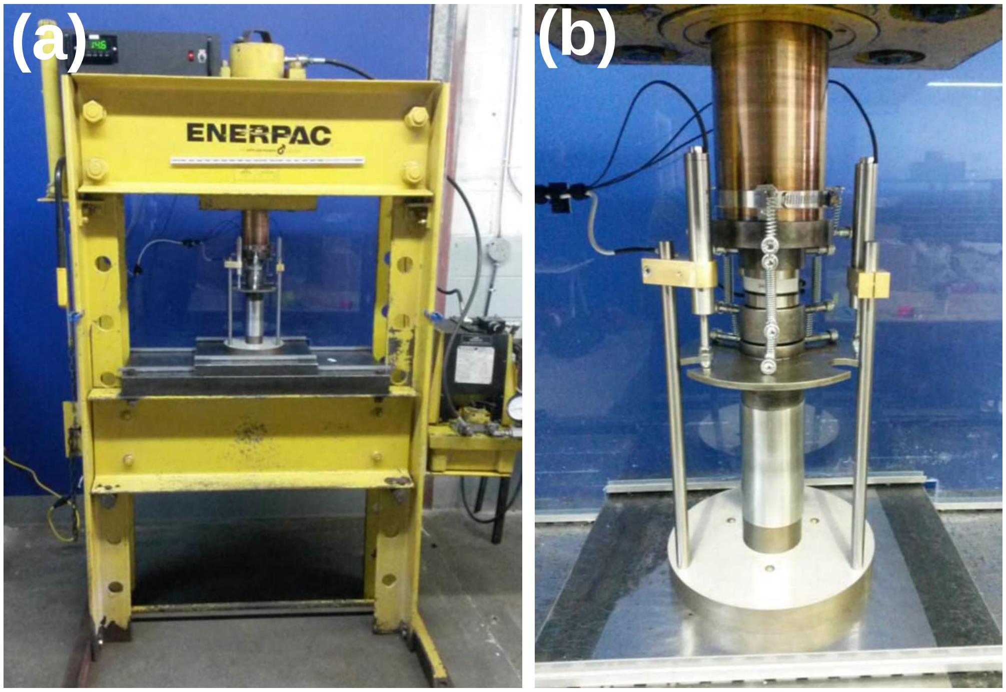


Figure 1: Test setup including (a) the loading frame and (b) a close-up of platens and sensors.

2 Results

The results of the tests are summarized in Table 1. The corresponding stress-strain curves are presented in Figure 2. The Young's modulus is the tangent modulus, calculated as the slope of the best fit line through ±200 data points on either side of the point representing 50% of the peak strength, unless indicated otherwise.

Table 1: Summary of laboratory test results.

Sample	Depth (m)	UCS (MPa)	Young's modulus, E (GPa)	Failure notes
15-1-1	5.54 - 5.70	26.9	4.7	
15-1-2	6.38 - 6.80	51.3	14.7	
15-1-3 (a)	7.11 - 7.36	30.1	7.0	[a, b]
15-1-3 (b)		63.6	10.0	[b]
15-2-1	15.03 - 15.22	37.2	7.0	[e]
15-3-1	4.10 - 4.30	45.2	7.8	[d]
15-3-2 (a)	5.30 - 5.55	30.6	5.0	[d]
15-3-2 (b)		39.2	5.8	
15-3-3	6.07 - 6.20	45.6	9.1	[c]
Mean		41.1	7.9	
Standard Deviation		11.7	3.1	
Min		26.9	4.7	
Max		63.6	14.7	

[a] Used 2 out of 3 displacement transducers to calculate strain due to erroneous readings

[b] Curve not linear at 50 % UCS, tangent modulus measured at 75 %UCS

[c] Pre-peak localized failure

[d] Tangent modulus measured at 50 %UCS as best fit line through -100 to +200 points

[e] HQ-core

2.1 Specimen photographs

Photographs of the specimens prior to and after testing are presented in Figure 3 and Figure 4, respectively.

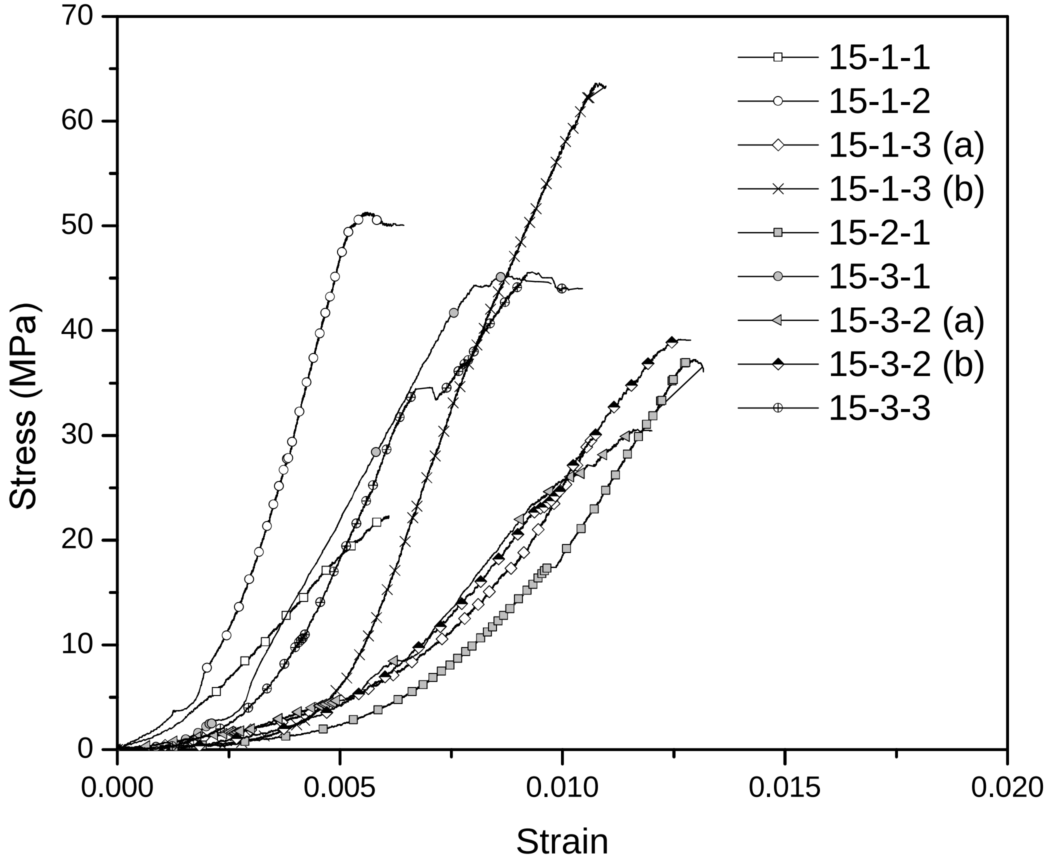


Figure 2: Measured stress-strain curves for the UCS specimens

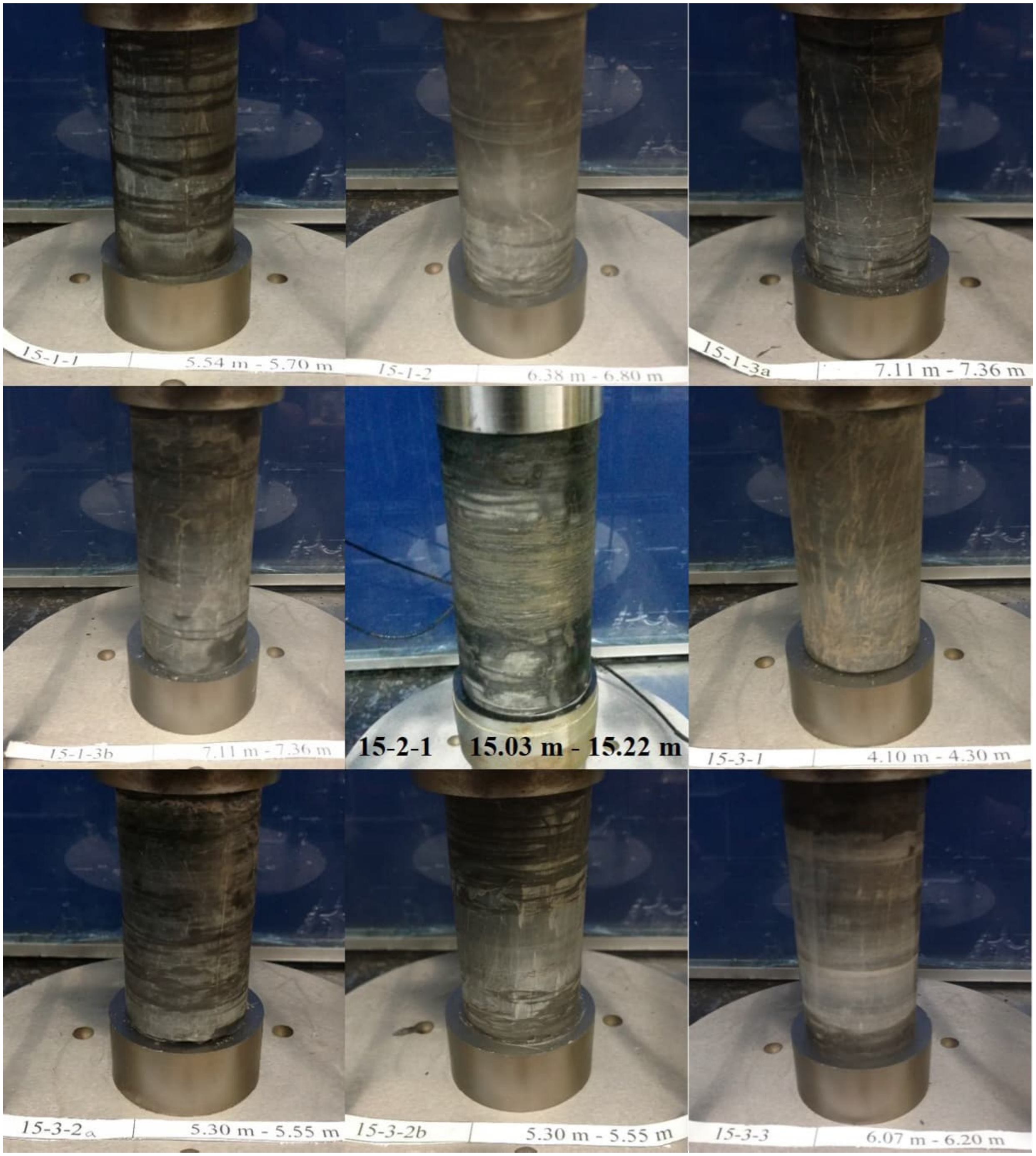


Figure 3: Photographs of UCS specimens prior to testing.

Figure 4: Photographs of UCS specimens after testing.

APPENDIX E

Results of Geophysical Investigation

Golder Associates

TECHNICAL MEMORANDUM

DATE November 25, 2015 **PROJECT No.** 1522569

TO Nicolas Leblanc
Golder Associates

FROM Adam Ramer, Christopher Phillips **EMAIL** aramer@golder.com;
cphillips@golder.com

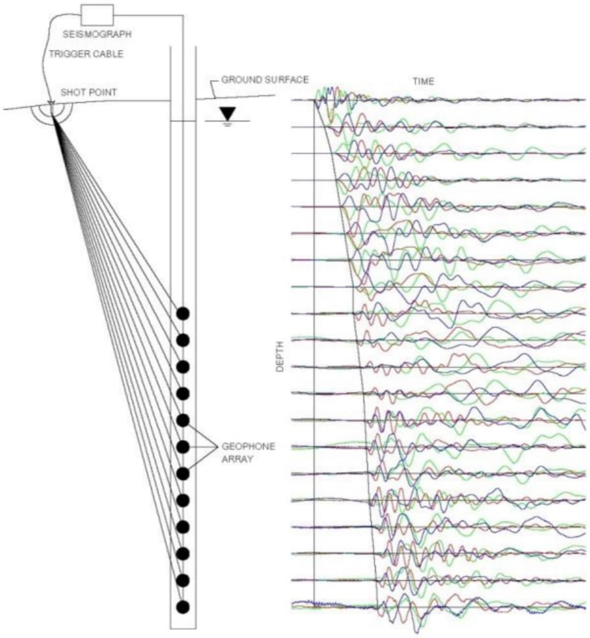
VERTICAL SEISMIC PROFILING TEST RESULTS
SILVERCITY SHOPPING CENTRE, OTTAWA

This memorandum presents the results of the Vertical Seismic Profiling (VSP) testing carried out at the Silvercity Shopping Centre, on City Park Drive in Ottawa, Ontario. VSP testing was carried out in borehole 15-2 on October 28, 2015. Borehole 15-2 was drilled to an approximate depth of 16.55 m below the existing pavement surface and then cased with a PVC pipe grouted in place.

Methodology

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

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



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

Fieldwork

The fieldwork was carried out on October 28, 2015, by personnel from the Golder Mississauga office.

Both compression and shear-wave seismic sources were used and both were located between 1 and 2 m from the boreholes. The seismic source for the compression wave test consisted of a 9.9 kilogram sledge hammer vertically impacted on a metal plate. The plate was located 1.10 m from borehole 15-2. The seismic source for the shear-wave test consisted of a 3 metre long, 150 millimetre by 150 millimetre wooden beam, weighted by a vehicle and horizontally struck with a 9.9 kilogram sledge hammer on alternate ends of the beam to induce polarized shear waves. The centre of the shear source was located 1.10 m from the borehole, and coupled to the ground surface by parking a vehicle on top of it. Test measurements started at ground surface and were recorded in the borehole with a 3-component receiver spaced at 1-metre intervals below the ground surface to a maximum depth of the casing (15.85 m).

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

Data Processing

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

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

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

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

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

Processing of the VSP data was completed using the SeisImager/SW software package (Geometrics Inc.). The seismic records are presented on the following four plots and show the first break picks of the compression wave (Figure 1) and shear wave arrivals (Figure 2) overlaid on the seismic waveform traces recorded at the different geophone depths for borehole 15-2. The arrivals were picked on the vertical component for the compression source and on the two horizontal components for the shear source.

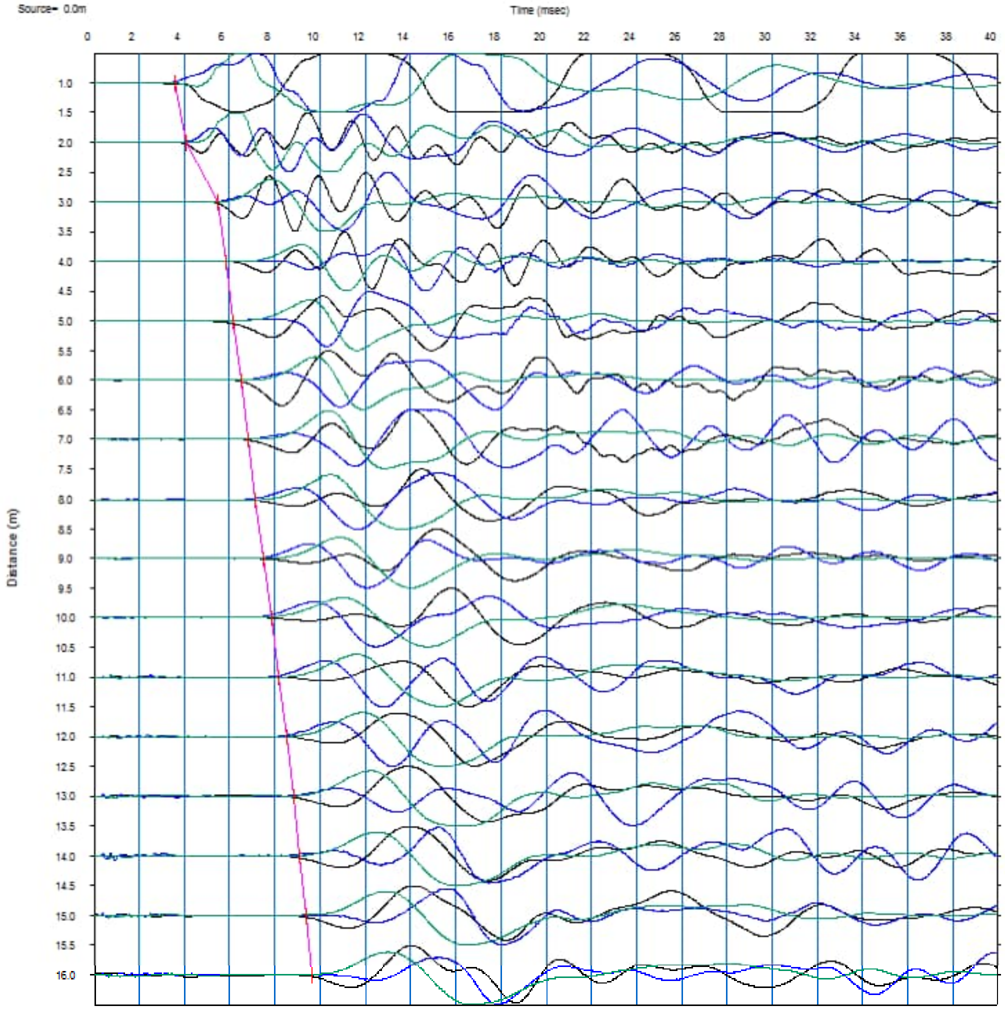


Figure 1: First break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth of borehole 15-2.

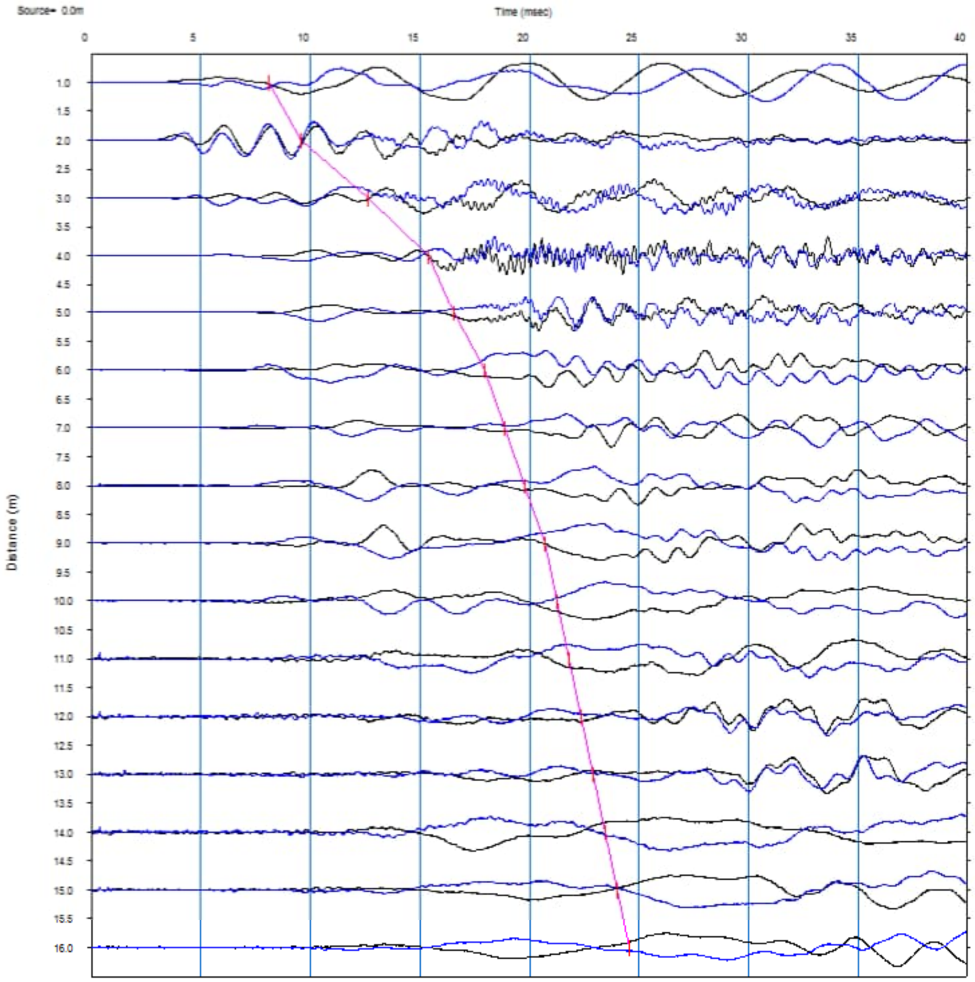


Figure 2: First break picking of shear wave arrivals (red) along the seismic traces recorded at each receiver depth of borehole 15-2.

Results

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

The estimated dynamic engineering moduli, based on the calculated wave velocities, are also presented in Table 1. The engineering moduli were calculated using an estimated bulk density, based on the borehole log. For the top 1 m of fille, the bulk density of 1600 kg/m^3 was used. For the shale bedrock down to the bottom of the borehole at 15.85 metres, a bulk density of 2,650 kilogram per cubic metre was used.

The average shear wave velocity from ground surface to a depth of 30 metres, assuming same velocity of rock from 15.85 m to 30 m below ground surface, was measured to be 925 metres per second.

Survey Limitations

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

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

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

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

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

Closure

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

GOLDER ASSOCIATES LTD.

Adam Ramer

Adam Ramer, P.Geo
Geophysicist

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

AR/CRP/jl
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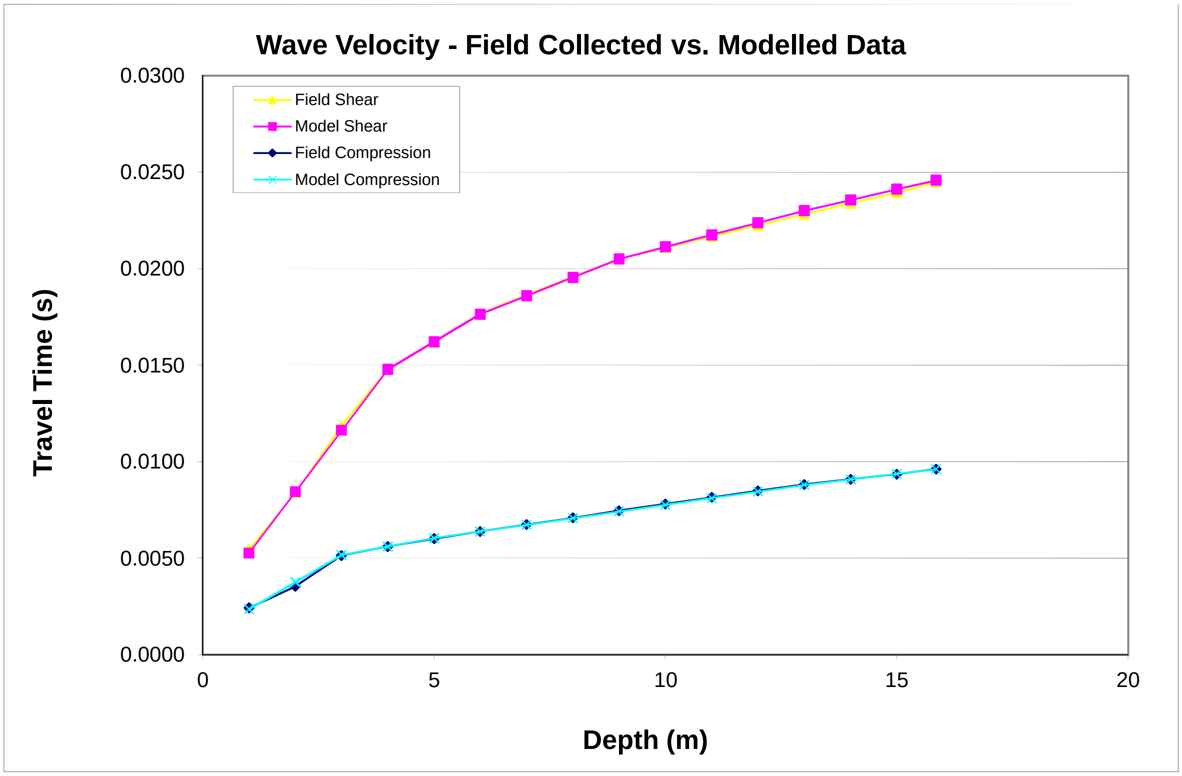
Attachment: Table 1 – Shear Wave Velocity Profile at BH-15-2

TABLE 1
SHEAR WAVE VELOCITY PROFILE AT 15-2

Layer Depth (m)		Wave Velocity (m/s)		Estimated Bulk Density (kg/m^3)	Dynamic Engineering Properties			
Top	Bottom	Compressional	Shear		Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0	1	425	190	1600	0.38	58	159	212
1	2	710	315	2650	0.38	263	724	985
2	3	710	315	2650	0.38	263	724	985
3	4	2300	315	2650	0.49	263	784	13668
4	5	2300	700	2650	0.45	1299	3763	12287
5	6	2900	700	2650	0.47	1299	3815	20555
6	7	2900	1050	2650	0.42	2922	8324	18391
7	8	2900	1050	2650	0.42	2922	8324	18391
8	9	2900	1050	2650	0.42	2922	8324	18391
9	10	2900	1600	2650	0.28	6784	17383	13241
10	11	2900	1600	2650	0.28	6784	17383	13241
11	12	2900	1600	2650	0.28	6784	17383	13241
12	13	2900	1600	2650	0.28	6784	17383	13241
13	14	3500	1800	2650	0.32	8586	22670	21015
14	15	3500	1800	2650	0.32	8586	22670	21015
15	15.85	3500	1800	2650	0.32	8586	22670	21015



Notes

1. Depth Presented relative to ground surface.
2. This Table to be analyzed in conjunction with the accompanying report.

As a global, employee-owned organisation with over 50 years of experience, Golder Associates is driven by our purpose to engineer earth's development while preserving earth's integrity. We deliver solutions that help our clients achieve their sustainable development goals by providing a wide range of independent consulting, design and construction services in our specialist areas of earth, environment and energy.

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APPENDIX C

Previous Golder’s Addendum No. 1668958

July 5, 2018 Project No. 1668958

Marc Calvé
RioCan Real Estate Investment Trust
RioCan Yonge Eglinton Centre
2300 Yonge Street, Suite 500
P.O. Box 2386
Toronto, ON
M4P 1E4

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/m^3**	**22 kN/m^3**
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.

LICENSED PROFESSIONAL ENGINEER
P. F. C. KO
100103970
Jul 5/18
PROVINCE OF ONTARIO

Christine Ko, P.Eng.
Geotechnical Engineer

Nicolas Leblanc, P.Eng.
Senior Geotechnical Engineer

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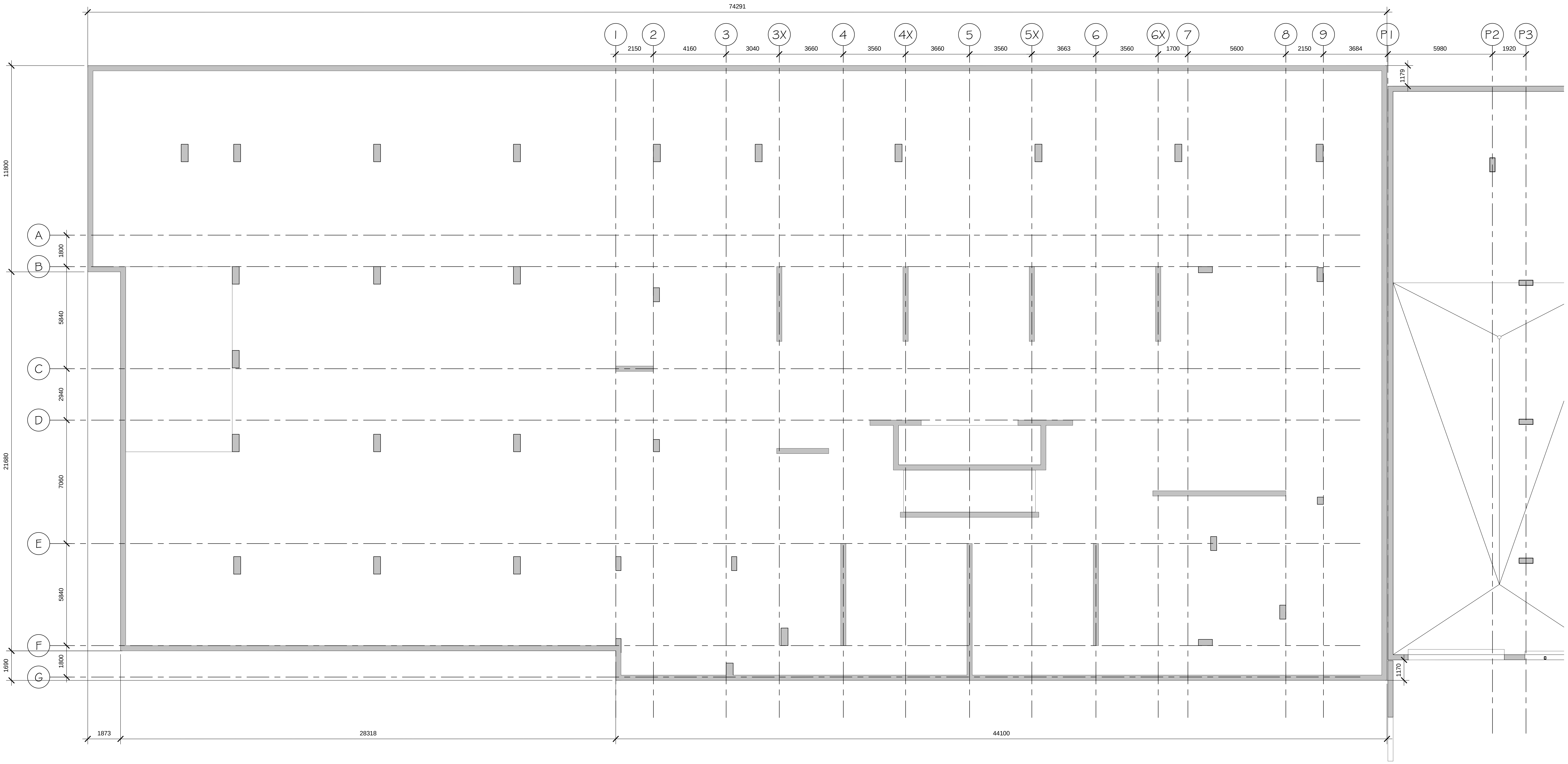
Attachments: *Level P1 to Level P3 Parking* (Drawings Nos. A2.01 to A2.03) by Hobin Architecture dated October 9, 2014

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

RIO CAN
REAL ESTATE INVESTMENT TRUST

Killam
PROPERTIES INC



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

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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HOBIN
ARCHITECTURE

PROJECT
RIOCAN - CITY PARK PHASE 2
2280 City Park Drive

DRAWING TITLE
LEVEL P1 - PARKING

DRAWN
Author

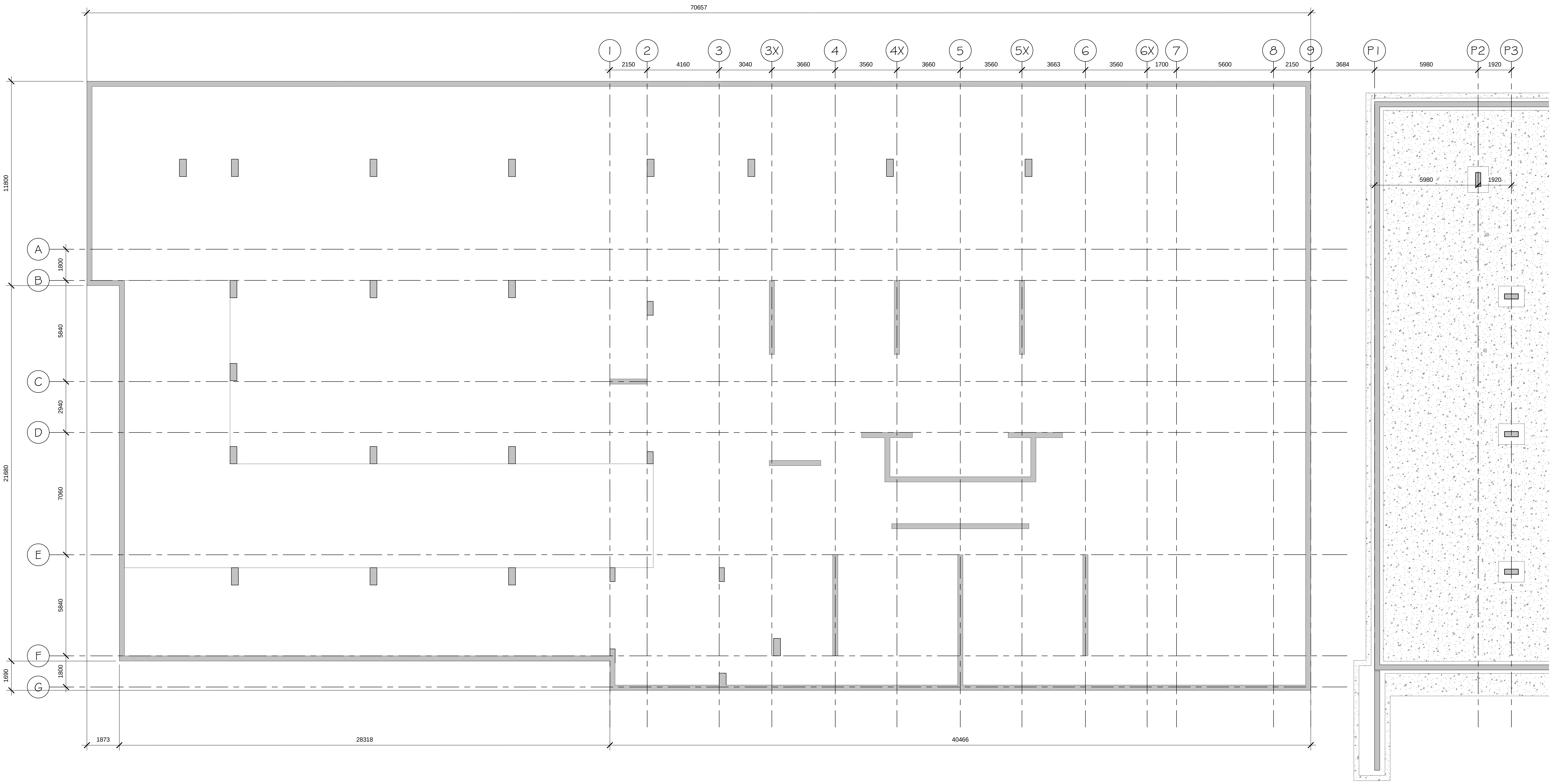
DATE
10/09/14

SCALE
1 : 100

PROJECT
0001

DRAWING NO.
A2.01

REVISION NO.



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

RIO CAN
REAL ESTATE INVESTMENT TRUST

Killam
PROPERTIES INC

no.	date (yymmdd)	revision

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HOBIN
ARCHITECTURE

PROJECT
RIOCAN - CITY PARK PHASE 2
2280 City Park Drive

DRAWING TITLE
LEVEL P2 - PARKING

DRAWN
Author

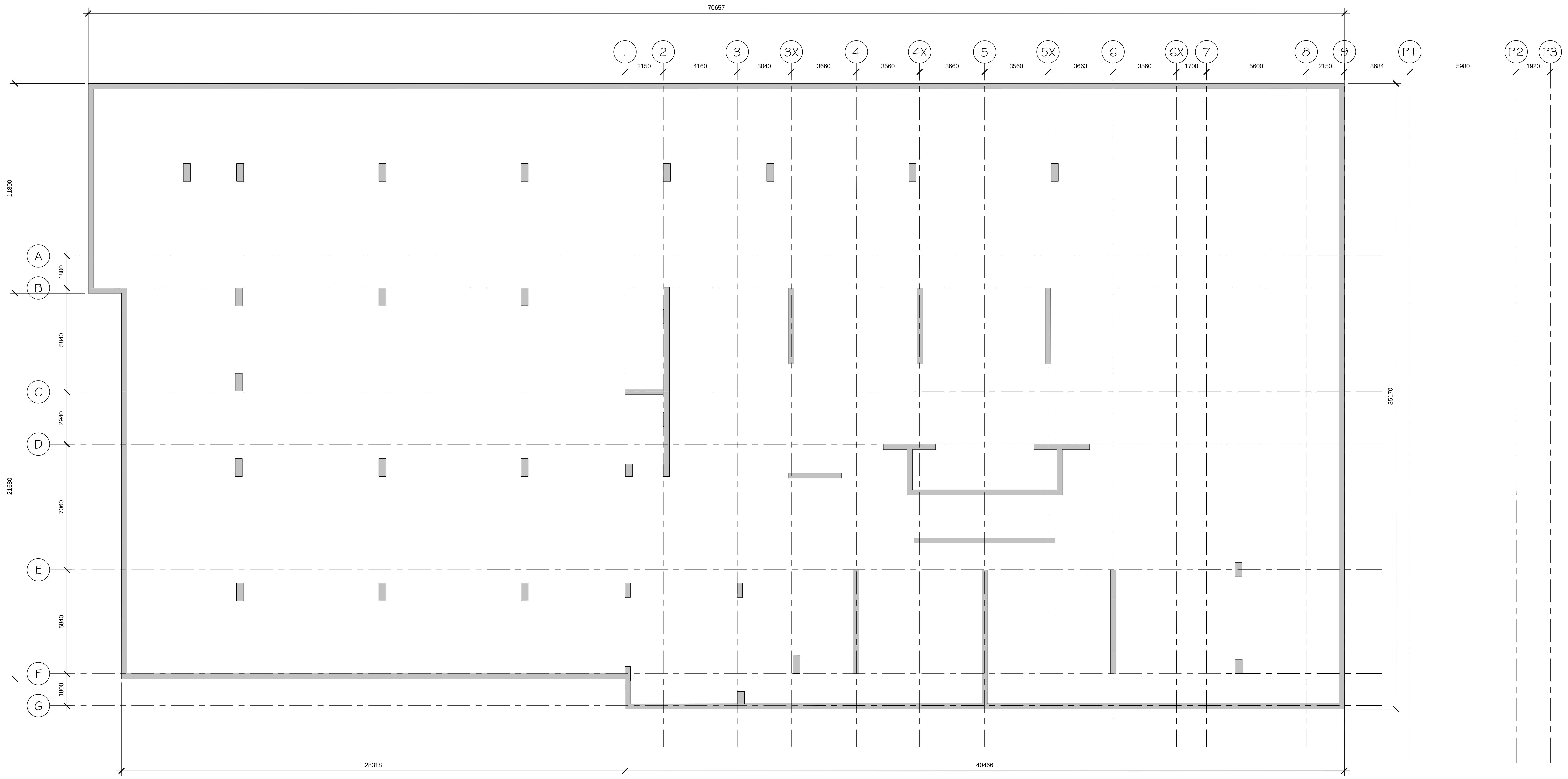
DATE
10/09/14

SCALE
1 : 100

PROJECT
0001

DRAWING NO.
A2.02

REVISION NO.



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

RIO CAN
REAL ESTATE INVESTMENT TRUST

Killam
PROPERTIES INC

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.

Hobin Architecture Incorporated

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Canada K1S 3K7

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HOBIN
ARCHITECTURE

PROJECT
RIOCAN - CITY PARK PHASE 2
2280 City Park Drive

DRAWING TITLE
LEVEL P3 - PARKING

DRAWN
Author

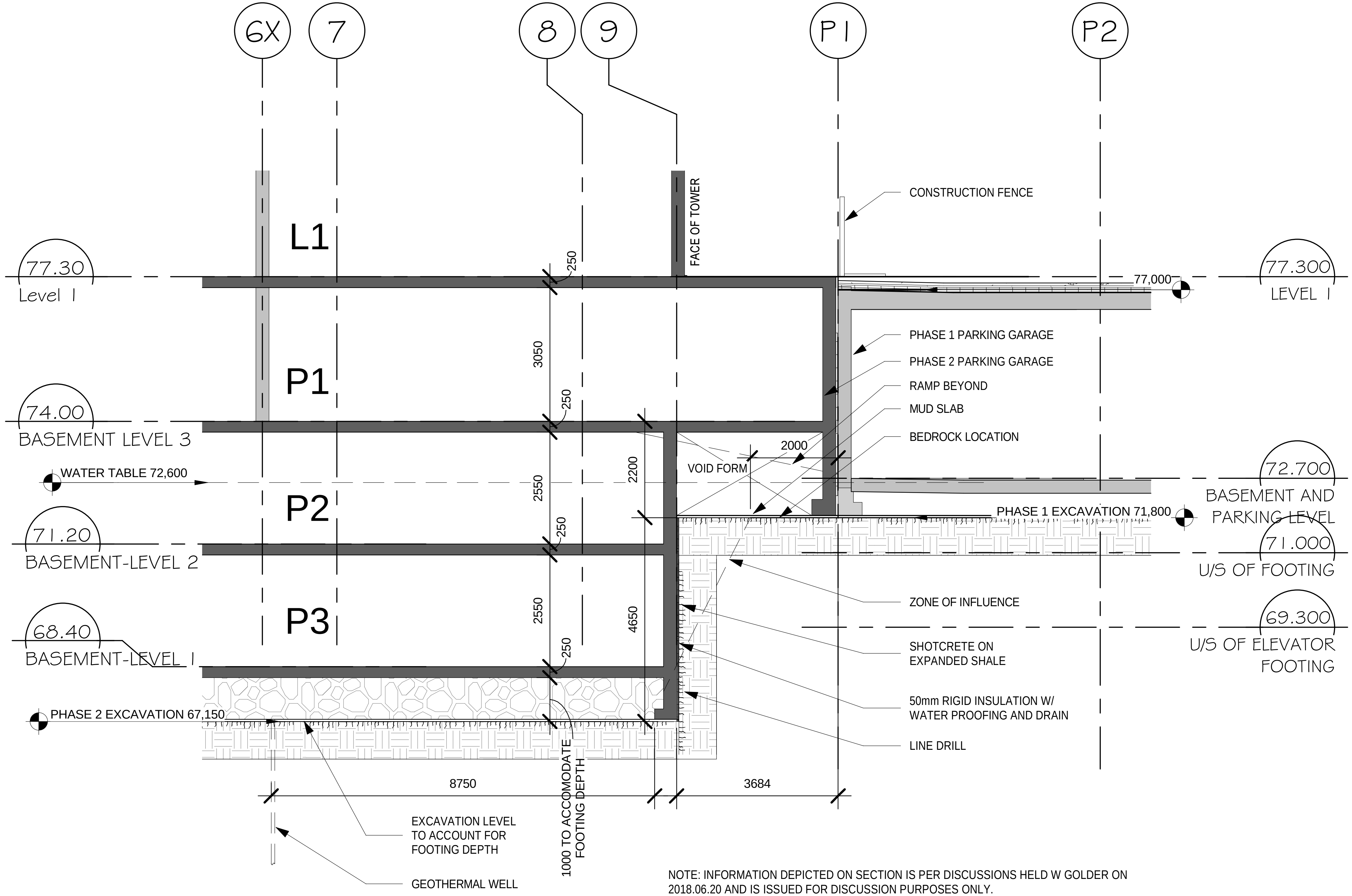
DATE
10/09/14

SCALE
1 : 100

PROJECT
0001

DRAWING NO.
A2.03

REVISION NO.



NOTE: INFORMATION DEPICTED ON SECTION IS PER DISCUSSIONS HELD W GOLDER ON 2018.06.20 AND IS ISSUED FOR DISCUSSION PURPOSES ONLY.

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
2280 City Park Drive

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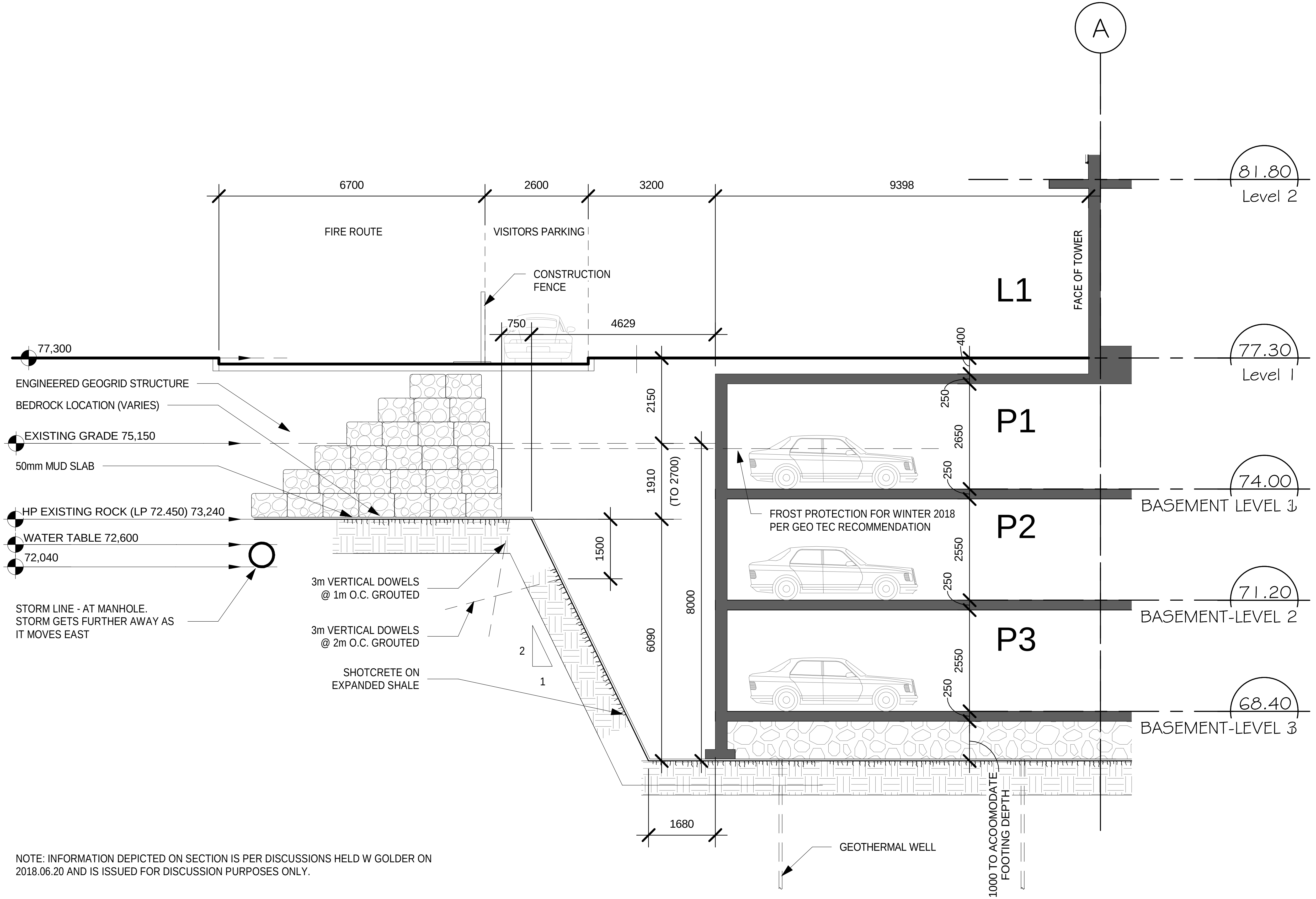
HOBIN ARCHITECTURE



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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:
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2280 City Park Drive

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