REVISED

Geotechnical Investigation – Proposed Commercial/Residential Development

320 McRae Avenue, 1976 Scott Street, and 311 & 315 Tweedsmuir Avenue
Ottawa, Ontario

Prepared for:

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1.0 INTRODUCTION

Pinchin Ltd. (Pinchin) was retained by The Great-West Life Assurance Company (Client) to conduct a Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed commercial/residential development to be located at 320 McRae Avenue, 1976 Scott Street, and 311 & 315 Tweedsmuir Avenue Ottawa, Ontario (Site). The Site location is shown on Figure 1.

The Client provided proposed future development plans for the Site, which include a 25-storey mixed-use commercial and residential building with a two level underground parking garage (UPG) located on the north portion of the Site, and a four-storey commercial and residential building which will be complete with a single level UPG located on the south portion of the Site. At the time of this report the depth to the underside of the footings for the UPGs is unknown; as such, for the purpose of this report, Pinchin has assumed an approximate depth of 4.0 metres below the existing ground surface (mbgs) per level of UPG. Therefore, the depths to the underside of the footings for the two level UPG and the single level UPG are approximately 8 mbgs and 4 mbgs, respectively.

Pinchin's geotechnical comments and recommendations are based on the results of the Geotechnical Investigation and our understanding of the project scope.

The purpose of the Geotechnical Investigation was to delineate the subsurface conditions and soil engineering characteristics by advancing a total of nine sampled boreholes (Boreholes BH4 to BH12) at the Site. The information gathered from the Geotechnical Investigation will allow Pinchin to provide geotechnical design recommendations for the proposed development. It is noted that Pinchin completed a Phase II Environmental Site Assessment (ESA) in conjunction with the geotechnical field investigation; as such, the information obtained from the Phase II ESA was also used to aid in providing geotechnical design recommendations. A copy of the Phase II ESA monitoring well logs are included in Appendix II.

Based on a desk top review and the results of the Geotechnical Investigation, the following geotechnical data and engineering design recommendations are provided herein:

- A review of relevant area geology and Site background information;
- A detailed description of the observed soil, bedrock and groundwater conditions;
- Site preparation;
- Site service trench design;
- Open cut excavations;
- Anticipated groundwater management;

- Foundation design recommendations including bedrock bearing resistances at Ultimate Limit States (ULS) design;
- Potential total and differential settlements;
- Foundation frost protection and engineered fill specifications and installation;
- Seismic Site classification for seismic Site response;
- Underground parking garage design recommendations;
- Interior concrete floor slab-on-grade (including modulus of subgrade reaction); and
- Asphaltic concrete pavement structure design for parking areas and access roadways.

Abbreviations terminology and principle symbols commonly used throughout the report are enclosed in Appendix I.

2.0 SITE DESCRIPTION AND GEOLOGICAL SETTING

The Site is an L-shaped property which is bounded by Scott Street to the north, McRae Avenue to the east, Tweedsmuir Avenue to the northwest, single family residential dwellings to the southwest, and an asphalt surfaced parking area to the south. The Site is currently developed with a combination of single family residential dwellings, and a single storey multi-tenant commercial building. The Site is also complete with a combination of gravel and asphalt surfaced parking areas as well as areas of soft landscaping (i.e. grassed areas with trees).

Data obtained from the Ontario Geological Survey Maps, as published by the Ontario Ministry of Natural Resources, indicates that the Site is located on sandy silt to silty sand textured till on Paleozoic terrain. The underlying bedrock at this Site is of the Shadow Lake Formation consisting of limestone, dolostone, shale, arkose, and sandstone (Ontario Geological Survey Map 1972, published 1978).

3.0 GEOTECHNICAL FIELD INVESTIGATION AND METHODOLOGY

Pinchin completed a field investigation at the Site on November 2, 9, and 12, 2018 by advancing a total of nine sampled boreholes (Boreholes BH4 to BH12) throughout the Site. The boreholes were advanced to sampled depths ranging from approximately 0.8 to 3.2 mbgs, where refusal was encountered on bedrock. In addition, a 3.0 m and a 19.8 m long bedrock core with NQ sized diamond bit core barrel were advanced at the base of Boreholes BH4 and BH12, respectively, to confirm the presence of bedrock and to evaluate the Rock Quality Designation (RQD). The approximate spatial locations of the boreholes advanced at the Site are shown on Figure 2.

The boreholes were advanced with the use of a Geoprobe 7822 DT direct push drill rig which was equipped with standard soil sampling equipment. Soil samples were collected at 0.76 m intervals using a 51 mm outside diameter (OD) split spoon barrel in conjunction with Standard Penetration Tests (SPT) “N” values (ASTM D1586). The SPT “N” values were used to assess the compactness condition of the non-cohesive soil.

Groundwater observations and measurements were obtained from the open boreholes during and upon completion of drilling. The groundwater observations and measurements recorded are included on the appended borehole logs.

The bedrock cores were advanced in accordance with ASTM D2113. The bedrock types and RQD’s were evaluated immediately upon core retrieval.

The field investigation was monitored by experienced Pinchin personnel. Pinchin logged the drilling operations and identified the soil samples and rock cores as they were retrieved. The recovered soil samples were sealed into plastic bags and carefully transported to an independent and accredited materials testing laboratory for detailed analysis and testing. All soil samples were classified according to visual and index properties by the project engineer.

At the request of the Client, Pinchin retained the services of Geophysics GPR International Inc. (Geophysics GPR) to complete one shear wave velocity sounding at the Site in January 2020. The purpose of the shear wave velocity sounding was to determine Seismic Site Classification for the Site.

The field logging of the soil and groundwater conditions was performed to collect geotechnical engineering design information. The borehole logs include textural descriptions of the subsoil in accordance with a modified Unified Soil Classification System (USCS) and indicate the soil boundaries inferred from non-continuous sampling and observations made during the borehole advancement. These boundaries reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. The modified USCS classification is explained in further detail in Appendix I. Details of the soil and groundwater conditions encountered within the boreholes are included on the Borehole Logs within Appendix II.

Select soil samples collected from the boreholes were submitted to a material testing laboratory to determine the grain size distribution of the soil, the results of which are provided in Appendix III. In addition, the collected samples were compared against previous geotechnical information from the area, for consistency and calibration of results.

4.0 SUBSURFACE CONDITIONS

4.1 Borehole Soil Stratigraphy and Bedrock Lithology

In general, the soil stratigraphy at the Site consists of either surficial organics, surficial asphalt, or granular fill material overlying bedrock to the maximum borehole refusal depth of approximately 3.2 mbgs. The surficial organic material is typically located on the northwest portion of the Site and was measured to be approximately 200 mm thick. The surficial asphalt is located in the various parking areas and driveways and was measured to range in thickness from 50 to 100 mm.

The granular fill material was encountered within all boreholes either at the surface or underlying the surficial organics and surficial asphalt materials. The granular fill material extended to the underlying bedrock surface at each location and was noted to range in soil matrix from gravelly sand containing trace to some silt to gravelly, silty sand. It is noted that trace brick pieces, trace glass, and bedrock fragments were encountered within the fill. The granular fill material was observed to typically range in thickness from approximately 0.8 to 1.7 m with the exception of the Borehole BH9 which was measured to be approximately 3.2 m thick. Based on uncorrected SPT “N” values of between 1 and 50 blows per 300 mm penetration of a split spoon sampler, the granular fill material had a variable very loose to dense relative density; however, with the exception of isolated pockets within select boreholes, the granular fill generally had a compact to dense relative density. The results of three particle size distribution analyses performed on samples of the fill material indicate that the samples contain 23 to 34% gravel, 40 to 58% sand, and 19 to 26% silt sized particles.

The bedrock cores recovered consisted of limestone rock, which was slightly weathered in the upper layers and transitioned to fresh in the deeper rock core. The bedrock was grey with black and white banding, fine to medium grained, and contained few natural fractures with little to no oxidation. The bedrock at the fracture locations was mostly sharp and angular, which indicates minor water migration. Natural fractures were closely to moderately spaced, and were generally found to occur in sets oriented at approximately 45 to 90º to the core axis. An approximate 20% wash return within the rock cores was observed. The wash return was grey to milky white in colour. The rock core recovery ranged from 60 to 100%, with an average RQD of 33% in the upper 3.0 m, and an average RQD of 84% below approximately 3.0 m. Based on the RQDs obtained, the bedrock is considered to be weathered and poor quality in the upper 3.0 m and unweathered and good quality below the upper 3.0 m.

4.2 Groundwater Conditions

Groundwater observations and measurements were obtained in the open boreholes at the completion of drilling and are summarized on the appended borehole logs. Groundwater was not observed within the boreholes advanced at the Site; however, groundwater measurements were obtained from the groundwater monitoring wells which were installed as part of Pinchin's Phase II ESA. Groundwater was measured on November 13, 2018 at depths ranging from 4.4 to 6.1 mbgs. Seasonal variations in the water table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during dry weather conditions.

5.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

5.1 General Information

The recommendations presented in the following sections of this report are based on the information available regarding the proposed construction, the results obtained from the geotechnical investigation, and Pinchin's experience with similar projects. Since the investigation only represents a portion of the subsurface conditions, it is possible that conditions may be encountered during construction that are substantially different than those encountered during the investigation. If these situations are encountered, adjustments to the design may be necessary. A qualified geotechnical engineer should be on-Site during the foundation preparation to ensure the subsurface conditions are the same/similar to what was observed during the investigation.

It is Pinchin's understanding that the proposed development is to consist of a 25-storey mixed-use building complete with a two level underground parking garage (UPG) located on the north portion of the Site, and a four-storey mixed-use building complete with a single level UPG located on the south portion of the Site. At this time the depth to the underside of the footings for the UPGs is unknown; as such, for the purpose of this report, Pinchin has assumed an approximate depth of 4.0 mbgs per level of UPG. Therefore, the depths to the underside of the footings for the two level UPG and the single level UPG are approximately 8 mbgs and 4 mbgs, respectively.

5.2 Site Preparation

Prior to Site preparation activities commencing, the existing building structures will need to be demolished and removed from the Site, including all foundations and service pipes.

Preparation of the Site for the proposed development will consist of removing all trees, vegetation, surficial and overburden materials down to the underlying bedrock surface. The existing inorganic granular fill material may be used to raise grades below soft landscaping areas only.

Prior to placing any fill material at the Site, the bedrock and/or subgrade soil should be inspected by a qualified geotechnical engineer, and loosened/soft pockets should be sub excavated. All fill material is to be installed in maximum 200 mm thick loose lifts, compacted to 98% of its Standard Proctor Maximum Dry Density (SPMDD), within plus 2 to minus 4 of the optimum moisture content.

A qualified geotechnical engineering technician should be on site to observe fill placement operations and perform field density tests at random locations throughout each lift, to indicate the specified compaction is being achieved.

5.3 Open Cut Excavations

It is anticipated that the excavations for the building foundations will extend to depths of approximately 4.0 and 8.0 mbgs in order to accommodate the proposed levels of underground parking. As such, a portion of the bedrock will need to be removed to accommodate the underground levels.

Based on the subsurface information obtained from within the boreholes it is anticipated that the excavated material will consist of a combination of asphalt, organics, granular fill, bedrock fragments, and bedrock. Groundwater was measured to be located at depths ranging from approximately 4.4 to 6.1 mbgs.

Where workers must enter trench excavations deeper than 1.2 m, the trench excavations should be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act (OHSA), Ontario Regulation 213/91, Construction Projects, July 1, 2011, Part III - Excavations, Section 226. Alternatively, the excavation walls may be supported by either closed shoring, bracing, or trench boxes complying with sections 235 to 239 and 241 under O. Reg. 231/91, s. 234(1). Steel sheet piles are not possible due to the shallow bedrock. The shoring system may be designed as full cantilevers or the lateral loads can be taken up to the installation of internal bracing of rakers or tie back soil anchors. The temporary shoring design must include appropriate factors of safety, and any possible surcharge loading must be taken into account.

Based on the OHSA, the in-situ soil may be classified as Type 3 soil above the groundwater table. Temporary excavations in these soils must be cut at an inclination of 1 horizontal to 1 vertical (H to V) or less from the base of the excavation.

The upper approximate 3.0 m of bedrock in this area is typically weathered and can usually be removed with mechanical equipment, such as a large excavator and hydraulic hammer (hoe ram) and where required, with line drilling on close centres. Often a hydraulic hammer can be utilized to create an initial opening for the excavator bucket to gain access of the layered rock. The bedrock is known to contain vertical joints and near horizontal bedding planes. Therefore, some vertical and horizontal over break of the bedrock should be expected.

Depending on the ability of the mechanical equipment to advance through the bedrock, drilling and blasting may be required. It is often difficult to blast “neat” lines using conventional drilling and blasting procedures, as such, problems with “over break” are common. This may affect quantities claimed by the contractor for rock excavations, as well as the potential for off-site disposal of the blasted rock, if necessary. Allowances should be made for over break conditions. Due consideration should also be given to controlled blasting procedures in order to prevent potential damage to the surrounding environment.

In addition, we recommend that a pre-blast survey of all neighbouring properties be undertaken prior to conducting drilling and blasting activities. The preconstruction survey will serve to protect the Client from claims unrelated to the construction activities in the development of this property.

Pinchin notes that, local contractors are familiar with excavating the local bedrock and have specialized knowledge and techniques for its removal. Depending on the block size and degree of weathering of the rock they may have a different approach than what is presented in the preceding paragraphs.

Construction slopes in intact bedrock should stand near vertical provided the “loose” rock is properly scaled off the face. Once the blasting is completed, if there are any permanent bedrock shear walls, they will have to be reviewed by a Rock Mechanics Specialist to determine if it is stable or if it needs reinforcing, such as rock bolting.

In addition to compliance with the OHSA, the excavation procedures must also be in compliance to any potential other regulatory authorities, such as federal and municipal safety standards.

5.4 Anticipated Groundwater Management

Groundwater measurements were obtained from the groundwater monitoring wells which were installed as part of Pinchin’s Phase II ESA. Groundwater was measured on November 13, 2018 at depths ranging from 4.4 to 6.1 mbgs and is located within the bedrock.

Moderate groundwater inflow through the overburden soil and bedrock face is expected where the excavations extend less than 0.50 m below the groundwater table. It is believed that this groundwater inflow can be controlled using a gravity dewatering system with perimeter interceptor ditches and high capacity pumps. For excavations extending more than 0.5 m below the stabilized groundwater table, a dewatering system installed by a specialist dewatering contractor may be required to either lower the groundwater level prior to excavation, or to maintain the groundwater level during construction. The design of the dewatering system should be left to the contractor’s discretion, and the system should meet a performance specification to maintain and control the groundwater at least 0.50 m below the excavation base.

Seasonal variations in the water table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during dry weather conditions. If construction commences during wet periods (typically spring or fall), there is a greater potential that the groundwater elevation could be higher and/or perched groundwater may be present. Any potential precipitation of perched groundwater should be able to be controlled from pumping from filtered sumps, and should be pumped away immediately (not allowed to pond).

Prior to commencing excavations, it is critical that all existing surface water and potential surface water is controlled and diverted away from the Site to prevent infiltration and subgrade softening. At no time should excavations be left open for a period of time that will expose them to precipitation and cause subgrade softening.

All collected water is to discharge a sufficient distance away from the excavation to prevent re-entry. Sediment control measures, such as a silt fence should be installed at the discharge point of the dewatering system. The utmost care should be taken to avoid any potential impacts on the environment.

It is the responsibility of the contractor to propose a suitable dewatering system based on the groundwater elevation at the time of construction. The method used should not adversely impact any nearby structures. A Permit to Take Water or a submission to the Environmental Activity and Sector Registry (EASR) would be required if the daily water takings exceed 50,000 L/day. It is the responsibility of the contractor to make this application if required.

5.5 Site Servicing

5.5.1 Pipe Bedding and Cover Materials for Flexible and Rigid Pipes

The subgrade conditions beneath the Site services will consist of bedrock. No support problems are anticipated for flexible or rigid pipes founded on the bedrock. Service pipes require an adequate base to ensure proper pipe connection and positive flow is maintained post construction. As such, pipe bedding should be placed to be of uniform thickness and compactness. The pipe bedding and cover material should conform to OPSD 802.010 and 802.013 specifications for flexible pipes and to OPSD 802.031 to 802.033 with Class ‘B’ bedding for rigid pipes.

For pipes installed within bedrock trenches, the following is recommended:

- Install 300 mm of 19 mm clear stone gravel (OPSS 1004) or Granular ‘A’ (OPSS 1010) below the pipe extending up the sides to the spring line;
- If clear stone is used as bedding material, than a non-woven geotextile (Terrafix 360R or equivalent) is to be placed over the clear stone and pipe extending up vertically along the side walls of the bedrock and pipe a minimum distance of 500 mm;
- The pipe cover material should consist of either a Granular ‘B’ Type I (OPSS 1010) with a maximum particle diameter size of 26.5 mm or bedding sand and should extend to a minimum of 300 mm above the top of the pipe; and

- If rock shatter is present a non-woven geotextile (Terrafix 360R or equivalent) may be required to prevent the migration of fines from the bedding material into the rock shatter. Where blasting is required for site services, over blast of at least 600 mm of rock shatter should be performed. Over blast material may stay in the trench.

All granular fill material is to be placed in maximum 200 mm thick loose lifts compacted to a minimum of 98% SPMDD.

If constant groundwater infiltration becomes an issue, than an approximate 150 mm granular pad consisting of 19 mm clear stone gravel (OPSS 1004) wrapped in a non-woven geotextile (Terrafix 270R or equivalent) should be considered. The clear stone should contain a minimum of 50% crushed particles. Water collected within the stone should be controlled through sumps and filtered pumps.

5.5.2 Trench Backfill

Where the adjacent material consists of bedrock, the trench can be backfilled with well graded blast rock fill, with a gradation similar to OPSS 1010 Granular ‘B’ Type I. The soil should be placed to the underside of the granular subbase of the pavement structure, and be compacted in maximum 300 mm thick lifts to 98% SPMDD within 4% of the optimum moisture content. This is recommended to provide soil compatibility and help minimize potential abrupt differential frost heave between surrounding natural materials similar in composition.

All stockpiled material should be protected from deleterious materials, additional moisture and be kept from freezing.

Quality control will be the utmost importance when selecting the material. The selection of the material should be done as early in the contract as possible to allow sufficient time for gradation and proctor testing on representative samples to ensure it meets the projects specifications.

It is anticipated that imported material will be required to backfill the trenches due to minimal amount of natural soil observed at the Site. Imported material should consist of a Granular ‘A’, Granular ‘B’ Type I, or Select Subgrade Material (OPSS 1010). Heavy construction equipment and truck traffic should not cross any pipe until at least 1 m of compacted soil is placed above the top of the pipe.

Post compaction settlement of finer grained soil can be expected, even when placed to compaction specifications. As such, fill materials should be installed as far in advance as possible before finishing the roadway in order to mitigate post compaction settlements.

5.5.3 Frost Protection

The frost penetration depth in Ottawa, Ontario is estimated to extend to approximately 2.1 mbgs in open roadways cleared of snow. As such, it is recommended to place water services at a minimum depth of 300 mm below this elevation with the top of the pipe located at 2.4 mbgs or lower as dictated by municipal service requirements. If a minimum of 2.4 m of soil cover cannot be provided, then the pipe should be insulated with a rigid polystyrene insulation (DOW Styrofoam HI40, or equivalent) or a pre-insulated pipe be utilized.

The insulation design configuration may either consist of placing horizontal insulation to a specified design distance beyond the outside edge of the pipe or an inverted “U” surrounding the top and sides of the pipe. Any method chosen requires suitable design and installation in accordance with the manufactures recommendations. To accommodate the placement of horizontal insulation a wider excavation trench may be required.

5.6 Foundation Design

5.6.1 Discussion

Bedrock was encountered within the boreholes at depths ranging from approximately 0.8 to 3.2 mbgs. As such, based on the anticipated depths to the underside of footings of 4.0 mbgs and 8.0 mbgs, Pinchin recommends to construct the building on conventional shallow strip and spread footings founded on the limestone bedrock.

5.6.2 Shallow Foundations Bearing on Bedrock

For conventional shallow strip and spread footings established directly on the weathered bedrock surface encountered approximately 4.0 mbgs, a factored bearing resistance of 750 kPa may be used at Ultimate Limit States (ULS) design. For conventional shallow strip and spread footings established on unweathered competent bedrock, a factored bearing resistance of 2,000 kPa at ULS may be used. Prior to installing foundation formwork, the bedrock is to be reviewed by a geotechnical engineer. SLS does not apply to foundations bearing directly on bedrock, since the loads required for unacceptable settlements to occur would be much larger than the factored ULS and would be limited to the elastic compression of the bedrock and concrete.

The above bearing resistances assume the bedrock is cleaned of all overburden material and any loose rock pieces. In addition, it is assumed that the bedrock is free of soil filled seams. Therefore, the bedrock should be cleaned with air or water pressure exposing clean sound bedrock, and 1.5 m long probe holes should be advanced at selected locations to check for bedrock defects and soil filled seams. In the event soil filled seams are encountered, bedrock may need to be removed to the soil seam in order to achieve the recommended bearing resistances.

If construction proceeds during freezing weather conditions water should not be allowed to pool and freeze in bedrock depressions. All concrete should be installed and maintained above freezing temperatures as required by the concrete supplier.

The bedrock is to be relatively level with slopes not exceeding 10 degrees from the horizontal. Pinchin notes that it may be beneficial to install an approximate 150 mm thick layer of 19 mm clear stone gravel overlying the bedrock surface, to provide the forming contractor with a level working surface. Where the bedrock slope exceeds 10 degrees from the horizontal and does not exceed 25 degrees from the horizontal, shear dowels can be incorporated into the design to resist sliding. Where rock slopes are steeper, the bedrock is to be levelled and stepped as required. The change in vertical height will be a function of the rock quality at the proposed foundation location and will need to be determined at the time of construction.

As an alternative to levelling the bedrock, where the bedrock surface is irregular and jagged, it may be more practical to provide a level benching over these areas by pouring lean mix concrete (minimum 10 MPa) prior to constructing the foundations. This decision is made on Site, since each situation will depend on the Site specific bedrock conditions.

5.6.3 Foundation Transition Zones

Where strip footings are founded at different elevations, the bedrock is to have a maximum slope of 2 H to 1 V, with the concrete footing having a maximum rise of 600 mm and a minimum run of 600 mm between each step, as detailed in the latest edition of the Ontario Building Code (OBC). The lower footing should be installed first to mitigate the risk of undermining the upper footing.

Individual spread footings are to be spaced a minimum distance of one and a half times the largest footing width apart from each other to avoid stress bulb interaction between footings. This assumes the footings are at the same elevation.

5.6.4 Estimated Settlement

All individual spread footings should be founded on bedrock, reviewed and approved by a licensed geotechnical engineer.

Foundations installed in accordance with the recommendations outlined in the preceding sections are not expected to exceed total settlements of 25 mm and differential settlements of 19 mm.

All foundations are to be designed and constructed to the minimum widths as detailed in the latest edition of the OBC.

5.6.5 Building Drainage

To assist in maintaining the building dry from surface water seepage, it is recommended that exterior grades around the buildings be sloped away at a 2% gradient or more, for a distance of at least 2.0 m. Roof drains should discharge a minimum of 1.5 m away from the structure to a drainage swale or appropriate storm drainage system.

It is recommended that exterior perimeter foundation drains be installed where subsurface walls are exposed to the interior (basement walls).

The foundation drains should consist of a minimum 150 mm diameter fabric wrapped perforated drainage tile surrounded by 19 mm diameter clear stone (OPSS 1004) with a minimum cover of 150 mm on top and sides and 50 mm below the drainage tile. The clear stone gravel should be wrapped in a non-woven geotextile (Terrafix 270R or equivalent). The water collected from the weeping tile should be directed away from the building to appropriate drainage areas; either through gravity flow or interior sump pump systems. All subsurface walls should be damp proofed.

5.6.6 Shallow Foundation Frost Protection & Foundation Backfill

In the Ottawa, Ontario area, exterior perimeter foundations for heated buildings require a minimum of 1.8 m of soil cover above the underside of the footing to provide soil cover for frost protection.

It is noted that for foundations established on well-draining bedrock (i.e. no ponding adjacent to the foundation), frost protection is not required. This decision is typically made on Site, since each situation will depend on Site specific bedrock conditions.

Where the foundations for heated buildings do not have the minimum 1.8 m of soil cover frost protection, they should be protected from frost with a combination of soil cover and rigid polystyrene insulation, such as Dow Styrofoam or equivalent product. If required, Pinchin can provide appropriate foundation frost protection recommendations as part of the design review.

To minimize potential frost movements from soil frost adhesion, the perimeter foundation backfill should consist of a free draining granular material, such as a Granular 'B' Type I (OPSS 1010) or an approved sand fill, extending a minimum lateral distance of 600 mm beyond the foundation. The backfill material used against the foundation must be placed so that the allowable lateral capacity is achieved. All granular material is to be placed in maximum 300 mm thick lifts compacted to a minimum of 100% SPMDD in hard landscaping areas and 95% SPMDD in soft landscaping areas. It is recommended that inspection and testing be carried out during construction to confirm backfill quality, thickness and to ensure compaction requirements are achieved.

5.6.7 Site Classification for Seismic Site Response and Soil Behaviour

The following information has been provided to assist the building designer from a geotechnical perspective only. These geotechnical seismic design parameters should be reviewed in detail by the structural engineer and be incorporated into the design as required.

The seismic site classification has been based on the 2012 OBC. The parameters for determination of Site Classification for Seismic Site Response are set out in Table 4.1.8.4.A of the OBC. The site classification is based on the average shear wave velocity in the top 30 m of the site stratigraphy.

Geophysics GPR completed one shear wave velocity sounding at the Site (see Appendix VI). Based on the results of this shear wave velocity sounding, this Site has been classified as Class B; however, a re-calculated Site Class A has been provided for foundations founded directly on competent bedrock. Pinchin notes that as the final foundation design has not been completed, it is recommended that should a Site Class A be used for design purposes, it is clearly stated that the foundations must be founded on competent, unweathered bedrock.

5.7 Underground Parking Garage Design

At this time the final grades for the underside of the underground parking garage footings is unknown. As such, depending on the proposed final grades, there is a potential for the building to have to be designed to either resist hydrostatic uplift or to be provided with underfloor and foundation wall drainage systems connected to a suitable frost-free outlet.

The magnitude of the hydrostatic uplift may be calculated using the following formula:

$$P = \gamma \times d$$

Where:

P = hydrostatic uplift pressure acting on the base of the structure (kPa)

γ = unit weight of water (9.8 kN/m^3)

d = depth of base of structure below the design high water level (m)

Due to the close proximity of the Ottawa River, it is recommended that the 100-year flood level be assumed as the high water level.

The resistance of gross uplift of the structure can be increased by simply increasing the mass of the structure, incorporating oversize footings into the structure or by installing soil/rock anchors.

Alternatively, exterior perimeter foundation drains should be installed where subsurface walls are exposed to the interior. The foundation drains should consist of a minimum 150 mm diameter fabric wrapped perforated drainage tile surrounded by 19 mm diameter clear stone (OPSS 1004) with a minimum cover of 150 mm on top and sides and 50 mm below the drainage tile. Since the natural soil contains a significant amount of silt sized particles, the clear stone gravel should be wrapped in a non-woven geotextile (Terrafix 270R or equivalent). The water collected from the weeping tile should be directed away from the building to appropriate drainage areas; either through gravity flow or interior sump pump systems. All subsurface walls should be water proofed.

If the proposed basement floor level is constructed close to the stabilized groundwater level, an underfloor drainage system should be installed beneath the slab, in addition to the installation of perimeter weeping tiles at the footing level. The floor slab sub drains should be constructed in a similar fashion to the foundation drains and be connected to a suitable frost free outlet or sump.

If the building is constructed below the groundwater table and utilities sub drains and pumps are used to remove the groundwater from around the building footprint, there is the potential that a Permit to Take Water from the Ministry of the Environment, Conservation and Parks (MECP) will be required for the long term dewatering of the Site.

The walls must also be designed to resist lateral earth pressure. Depending on the design of the building the earth pressure computations must take into account the groundwater level at the Site. For calculating the lateral earth pressure, the coefficient of at-rest earth pressure (K_0) may be assumed at 0.5 for non-cohesive sandy soil. The bulk unit weight of the retained backfill may be taken as 20 kN/m^3 for well compacted soil. An appropriate factor of safety should be applied.

5.7.1 Lower Level Parking Garage Concrete Slab-on-Grade

Prior to the installation of the engineered fill material, all organics and deleterious materials should be removed to the underlying bedrock surface. The underlying bedrock encountered within the boreholes is considered adequate for the support of a concrete slab-on-grade provided it is inspected and approved by an experienced geotechnical engineering consultant.

Based on the in-situ conditions, it is recommended to establish a concrete floor slab-on-grade on a minimum 200 mm thick layer of Granular ’A’ (OPSS 1010). The purpose of the Granular ’A’ is mainly to provide a level surfaced for the concrete formwork. Alternatively, consideration may also be given to using a 200 mm thick layer of uniformly compacted 19 mm clear stone. Any required up fill should consist of a Granular ‘B’ Type I or Type II (OPSS 1010).

The installation of a vapour barrier may be required under the floor slab. If required, the vapour barrier should conform to the flooring manufacturer’s and designer’s requirements. Consideration may be given to carrying out moisture emission and/or relative humidity testing of the slab to determine the concrete condition prior to flooring installation. To minimize the potential for excess moisture in the floor slab, a concrete mixture with a low water-to-cement ratio (i.e. 0.5 to 0.55) should be used.

The following table provides the unfactored modulus of subgrade reaction values:

Material Type	Modulus of Subgrade Reaction (kN/m^3)
Granular A (OPSS 1010)	85,000
Granular ‘B’ Type I (OPSS 1010)	75,000
Granular ‘B’ Type II (OPSS 1010)	85,000

5.8 Asphaltic Concrete Pavement Structure Design

5.8.1 Discussion

Parking areas and access driveways will be constructed adjacent to the proposed buildings. Pinchin presumes that all overburden material will be removed during the construction of the buildings. As such, it is believe that any surficial asphalt pavement structure will be on foundation wall backfill. In areas where the existing fill is not removed due to construction activities, the fill could remain below the pavement structure subject to proof rolling, inspection by a geotechnical engineering and any future settlements accepted by the owner.

At this time Pinchin is unaware of the proposed final grades for the parking lot and access roadways. As such, the following pavement structure is recommended based on the pavement structure overlying granular backfill or the existing fill material.

5.8.2 Pavement Structure

The following table presents the minimum specifications for a flexible asphaltic concrete pavement structure:

Pavement Layer	Compaction Requirements	Light Duty Traffic and Parking Areas	Heavy Duty Traffic Areas and Access Laneways
Surface Course Asphaltic Concrete HL-4 (OPSS 1150)	92% MRD as per OPSS 310	50 mm	50 mm
Binder Course Asphaltic Concrete HL-8 (OPSS 1150)	92% MRD as per OPSS 310	N/A	70 mm

Pavement Layer	Compaction Requirements	Light Duty Traffic and Parking Areas	Heavy Duty Traffic Areas and Access Laneways
Base Course: Granular “A” (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm	150 mm
Subbase Course: Granular ‘B’ Type I (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM D698)	300 mm	400 mm

Notes:

i) Any required up fill material below the asphalt concrete pavement structure is to consist of a Granular B Type I (OPSS 1010) installed in maximum 300 mm thick lifts and compacted to 100% SPMDD.

ii) The recommended pavement structure may have to be adjusted according to the City of Ottawa standards.

iii) Performance grade PG 58-34 asphaltic concrete should be specified for Marshall mixes.

5.8.3 Pavement Structure Subgrade Preparation and Granular Up Fill

The proper placement of base and subbase fill materials becomes very important in addressing the proper load distribution to provide a durable pavement structure.

The subgrade should be inspected and approved by a qualified geotechnical engineering consultant prior to placement of the Granular ‘B’ up fill and/or subbase course.

Samples of both the Granular ‘A’ and Granular ‘B’ Type I aggregates should be tested for conformance to OPSS 1010 prior to utilization on Site and during construction. All stockpiled material should be protected from deleterious materials, additional moisture and be kept from freezing.

Post compaction settlement of fine grained soil can be expected, even when placed to compaction specifications. As such, fill material should be installed as far in advance as possible before finishing the parking lot and access roadways for best grade integrity.

Where the subgrade material types differ below the underside of the pavement structure, the transition between the materials should be sloped as per frost heave taper OPSD 205.60.

5.8.4 Drainage

Control of surface water is a critical factor in achieving good pavement structure life. The pavement thickness designs are based on a drained pavement subgrade via sub-drains or ditches. It is recommended that sub drains be installed in the low areas of the on grade parking and be connected to the catch basins.

The surface of the roadways should be free of depressions and be sloped at a minimum grade of 1% in order to drain to appropriate drainage areas. Subgrade soil should slope a minimum of 3% toward stormwater collection points. Positive slopes are very important for the proper performance of the drainage system. The granular base and subbase materials should extend horizontally to any potential ditches or swales.

In addition, routine maintenance of the drainage systems will assist with the longevity of the pavement structure. Ditches, culverts, sewers and catch basins should be regularly cleared of debris and vegetation.

7.0 SITE SUPERVISION & QUALITY CONTROL

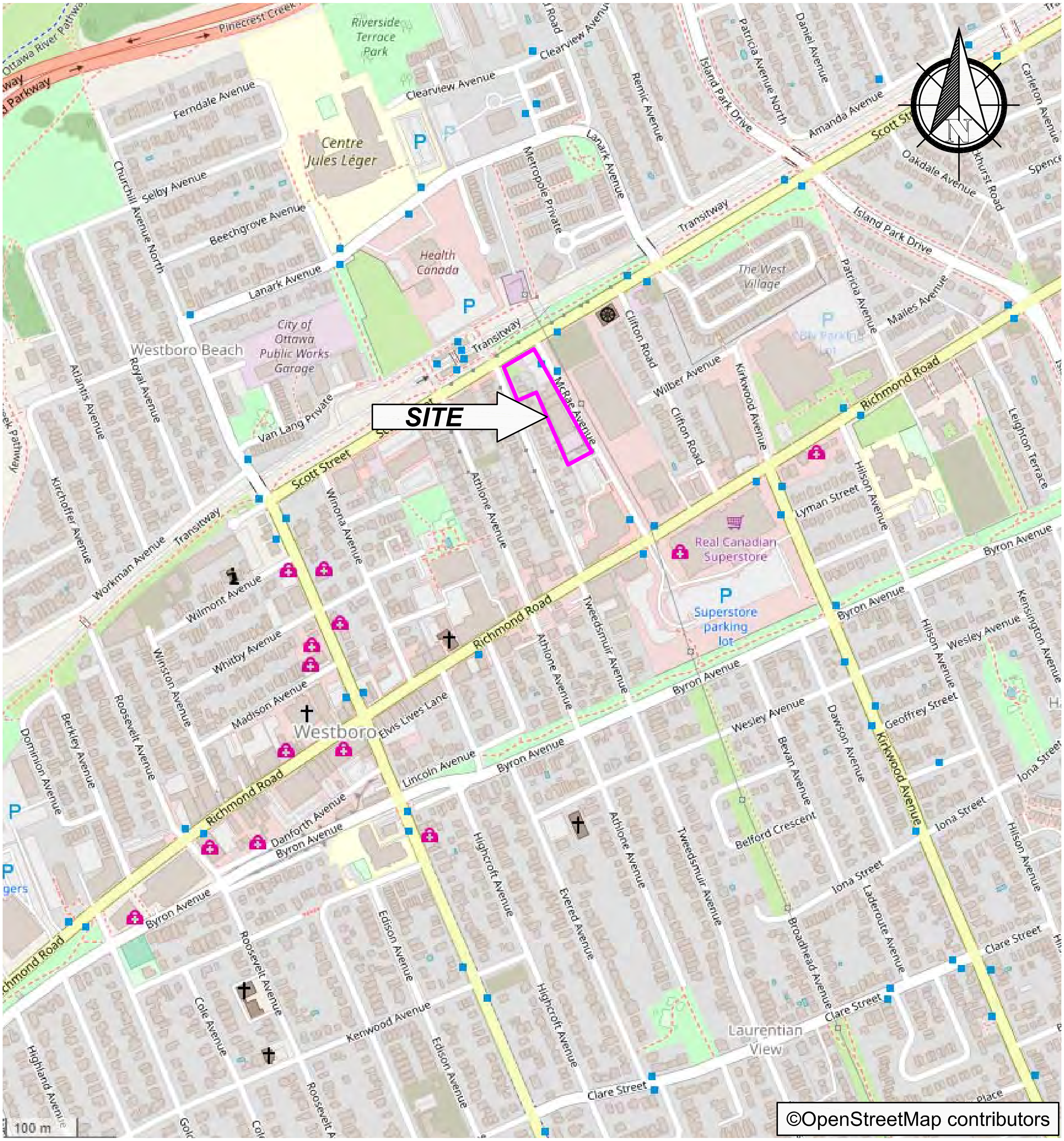
It is recommended that all geotechnical aspects of the project be reviewed and confirmed under the appropriate geotechnical supervision, to routinely check such items. This includes but is not limited to inspection and confirmation of the granular fill and bedrock prior to pouring any foundations or footings, backfilling, or engineered fill installation to ensure that the actual conditions are not markedly different than what was observed at the borehole locations and geotechnical components are constructed as per Pinchin’s recommendations. Compaction quality control of engineered fill material (full-time monitoring) is recommended as standard practice, as well as regular sampling and testing of aggregates and concrete, to ensure that physical characteristics of materials for compliance during installation and satisfies all specifications presented within this report.

8.0 DISCLAIMER

This report was prepared pursuant to and in accordance with the master services agreement (the “MSA”) dated July 16, 2007 (amended May 1, 2017) between The Pinchin Group of Companies (“Consultant”) and the other parties listed thereto, and the project specific agreement dated May 5, 2014 between Consultant and The Great West Life Assurance Company. The report was prepared by Consultant for the use of Owner and Manager (as those terms are defined under the MSA). In addition to the use of and reliance on this report by Owner and Manager, any person who has received a reliance letter for this report may use and rely on this report as if it was prepared for such persons. Any use of or reliance on this report by any other person (i.e., a person other than any Owner, Manager or otherwise permitted person) is the sole and exclusive responsibility of such other person. Consultant accepts no responsibility for damages, if any, suffered by such other person as a result of the use of or reliance on this report.

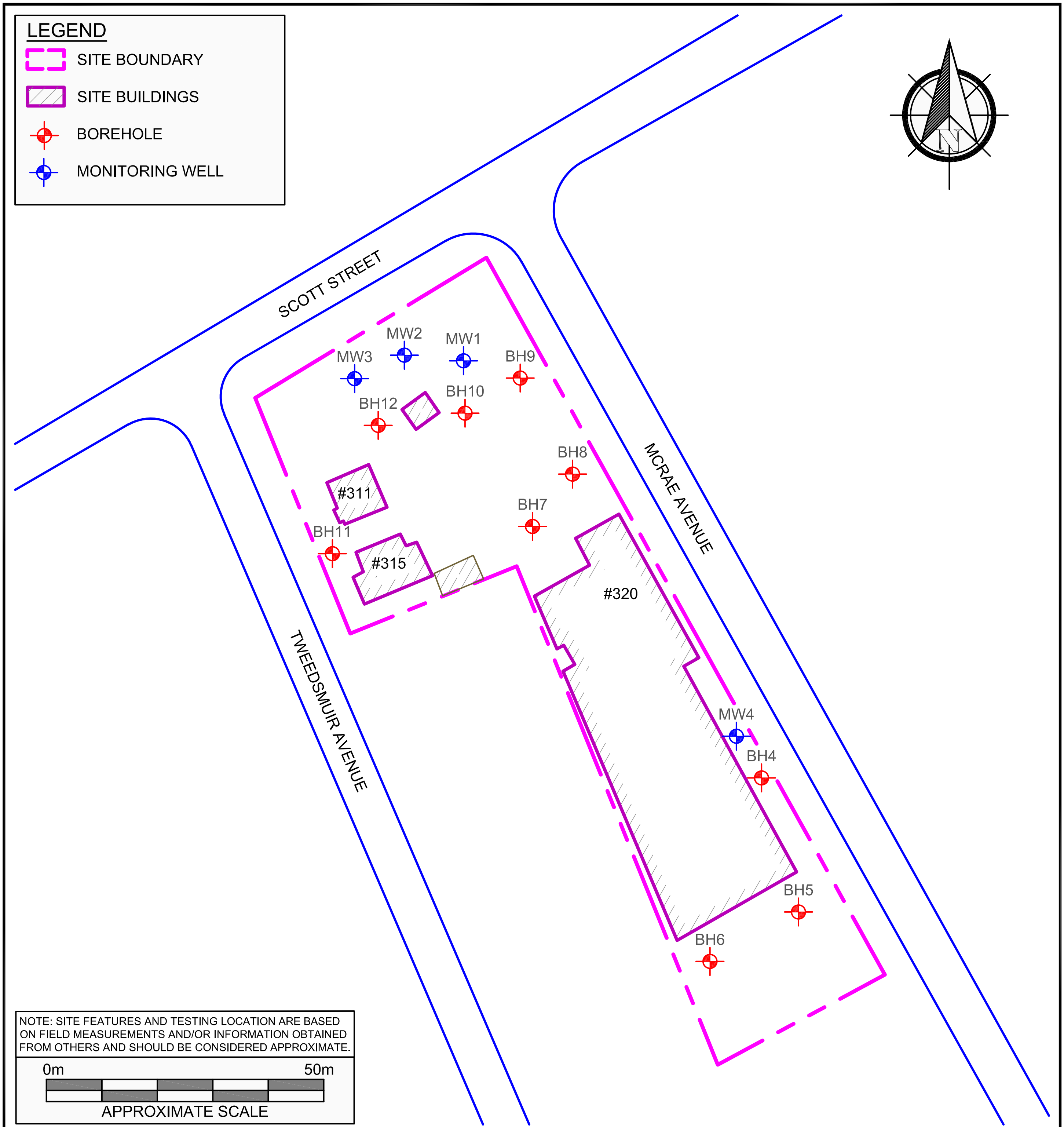
This report is based on the best information available to Consultant at the time of preparing this report after Consultant has used best industry practices, in the circumstances, to obtain information. To the extent that Consultant was required to rely on information from other persons, Consultant has verified such information to the extent reasonably possible in the circumstances. The material provided in this report reflects best industry judgment in light of the information available at the time of preparation of this report.

FIGURES



PINCHIN

PROJECT NAME	GEOTECHNICAL INVESTIGATION
CLIENT NAME	THE GREAT-WEST LIFE ASSURANCE COMPANY
PROJECT LOCATION	320 MCRAE AVENUE, 1976 SCOTT STREET, 311 AND 315 TWEEDSMUIR AVENUE, OTTAWA, ONTARIO
FIGURE NAME	KEY MAP
FIGURE NO.	1
APPROXIMATE SCALE	AS SHOWN
PROJECT NO.	230236.004
DATE	NOVEMBER 2018



PINCHIN

PROJECT NAME: GEOTECHNCIAL INVESTIGATION			
CLIENT NAME: THE GREAT-WEST LIFE ASSURANCE COMPANY			
PROJECT LOCATION: 320 MCRAE AVENUE, 1976 SCOTT STREET, 311 AND 315 TWEEDSMUIR AVENUE, OTTAWA, ONTARIO			
FIGURE NAME: BOREHOLE LOCATION PLAN			FIGURE NO. 2
APPROXIMATE SCALE: AS SHOWN	PROJECT NO.: 230236.004	DATE: NOVEMBER 2018	

APPENDIX I

Abbreviations, Terminology and Principle Symbols used in Report and Borehole Logs

ABBREVIATIONS, TERMINOLOGY & PRINCIPAL SYMBOLS USED

Sampling Method

AS	Auger Sample	**w**	Washed Sample
SS	Split Spoon Sample	**HQ**	Rock Core (63.5 mm diam.)
ST	Thin Walled Shelby Tube	**NQ**	Rock Core (47.5 mm diam.)
BS	Block Sample	**BQ**	Rock Core (36.5 mm diam.)

In-Situ Soil Testing

Standard Penetration Test (SPT), "N" value is the number of blows required to drive a 51 mm outside diameter spilt barrel sampler into the soil a distance of 300 mm with a 63.5 kg weight free falling a distance of 760 mm after an initial penetration of 150 mm has been achieved. The SPT, "N" value is a qualitative term used to interpret the compactness condition of cohesionless soils and is used only as a very approximation to estimate the consistency and undrained shear strength of cohesive soils.

Dynamic Cone Penetration Test (DCPT) is the number of blows required to drive a cone with a 60 degree apex attached to "A" size drill rods continuously into the soil for each 300 mm penetration with a 63.5 kg weight free falling a distance of 760 mm.

Cone Penetration Test (CPT) is an electronic cone point with a 10 cm2 base area with a 60 degree apex pushed through the soil at a penetration rate of 2 cm/s.

Field Vane Test (FVT) consists of a vane blade, a set of rods and torque measuring apparatus used to determine the undrained shear strength of cohesive soils.

Soil Descriptions

The soil descriptions and classifications are based on an expanded Unified Soil Classification System (USCS). The USCS classifies soils on the basis of engineering properties. The system divides soils into three major categories; coarse grained, fine grained and highly organic soils. The soil is then subdivided based on either gradation or plasticity characteristics. The classification excludes particles larger than 75 mm. To aid in quantifying material amounts by weight within the respective grain size fractions the following terms have been included to expand the USCS:

Soil Classification		Terminology	Proportion
Clay	< 0.002 mm		
Silt	0.002 to 0.06 mm	“trace”, trace sand, etc.	1 to 10%
Sand	0.075 to 4.75 mm	“some”, some sand, etc.	10 to 20%
Gravel	4.75 to 75 mm	Adjective, sandy, gravelly, etc.	20 to 35%
Cobbles	75 to 200 mm	And, and gravel, and silt, etc.	>35%
Boulders	>200 mm	Noun, Sand, Gravel, Silt, etc.	>35% and main fraction

Notes:

- Soil properties, such as strength, gradation, plasticity, structure, etcetera, dictate the soils engineering behaviour over grain size fractions; and
- With the exception of soil samples tested for grain size distribution or plasticity, all soil samples have been classified based on visual and tactile observations. The accuracy of visual and tactile observation is not sufficient to differentiate between changes in soil classification or precise grain size and is therefore an approximate description.

The following table outlines the qualitative terms used to describe the compactness condition of cohesionless soil:

Cohesionless Soil	
Compactness Condition	**SPT N-Index (blows per 300 mm)**
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

The following table outlines the qualitative terms used to describe the consistency of cohesive soils related to undrained shear strength and SPT, N-Index:

Cohesive Soil		
Consistency	**Undrained Shear Strength (kPa)**	**SPT N-Index (blows per 300 mm)**
Very Soft	<12	<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

Note: Utilizing the SPT, N-Index value to correlate the consistency and undrained shear strength of cohesive soils is only very approximate and needs to be used with caution.

Soil & Rock Physical Properties

General

W Natural water content or moisture content within soil sample

γ Unit weight

γ' Effective unit weight

γ_d Dry unit weight

γ_{sat} Saturated unit weight

ρ Density

ρ_s Density of solid particles

ρ_w Density of Water

ρ_d Dry density

ρ_{sat} Saturated density e Void ratio

n Porosity

S_r Degree of saturation

E_{50} Strain at 50% maximum stress (cohesive soil)

Consistency

W_L Liquid limit

W_P Plastic Limit

I_P Plasticity Index

W_S Shrinkage Limit

I_L Liquidity Index

I_C Consistency Index

e_{max} Void ratio in loosest state

e_{min} Void ratio in densest state

I_D Density Index (formerly relative density)

Shear Strength

C_u, S_u Undrained shear strength parameter (total stress)

C'_d Drained shear strength parameter (effective stress)

r Remolded shear strength

τ_p Peak residual shear strength

τ_r Residual shear strength

ø' Angle of interface friction, coefficient of friction = tan ø'

Consolidation (One Dimensional)

C_C Compression index (normally consolidated range)

C_r Recompression index (over consolidated range)

C_S Swelling index

m_V Coefficient of volume change

c_V Coefficient of consolidation

T_V Time factor (vertical direction)

U Degree of consolidation

σ'_o Overburden pressure

σ'_p Preconsolidation pressure (most probable)

OCR Overconsolidation ratio

Permeability

The following table outlines the terms used to describe the degree of permeability of soil and common soil types associated with the permeability rates**:**

Permeability (k cm/s)	Degree of Permeability	Common Associated Soil Type
> 10^{-1}	Very High	Clean gravel
10^{-1} to 10^{-3}	High	Clean sand, Clean sand and gravel
10^{-3} to 10^{-5}	Medium	Fine sand to silty sand
10^{-5} to 10^{-7}	Low	Silt and clayey silt (low plasticity)
>10^{-7}	Practically Impermeable	Silty clay (medium to high plasticity)

Rock Coring

Rock Quality Designation (RQD) is an indirect measure of the number of fractures within a rock mass, Deere et al. (1967). It is the sum of sound pieces of rock core equal to or greater than 100 mm recovered from the core run, divided by the total length of the core run, expressed as a percentage. If the core section is broken due to mechanical or handling, the pieces are fitted together and if 100 mm or greater included in the total sum**.**

RQD is calculated as follows:

$$\text{RQD (\%)} = \frac{\Sigma \text{ Length of core pieces} > 100 \text{ mm} \times 100}{\text{Total length of core run}}$$

The following is the Classification of Rock with Respect to RQD Value:

RQD Classification	RQD Value (%)
Very poor quality	**<25**
Poor quality	25 to 50
Fair quality	50 to 75
Good quality	75 to 90
Excellent quality	90 to 100

APPENDIX II

Pinchin's Borehole Logs

Log of Borehole: MW-1

Project #: 230236.002 **Logged By:** MK

Project: Phase II Environmental Site Assessment

Client: The Great-West Life Assurance Company

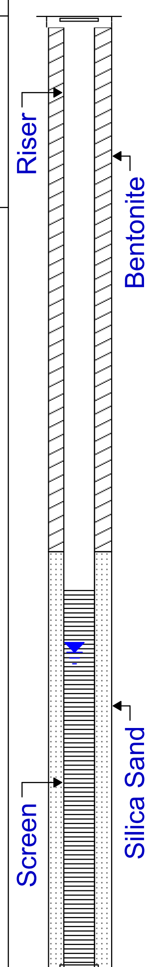
Location: 320 McRae Avenue, 1976 Scott Street, 311 and 316 Tweedsmuir Avenue, Ottawa, Ontario

Drill Date: November 1, 2018

SUBSURFACE PROFILE / SAMPLE

Depth (ft / m)	Description	Measured Depth (m)	Recovery (%)	Sample ID	Soil Vapour Concentration (ppm)(HEX/IBL)	Laboratory Analysis
0 / 0	Ground Surface	0.00				
0–5 ft / 0–1.52 m	***Asphalt***					
	Sand and Gravel - Fill Brown, damp.		30	SS1	0/0	
		1.52	30	SS2	0/1	Metals, PHCs, PAHs, VOCs, pH
5–25 ft / 1.52–7.62 m	***Limestone***	7.62				
	End of Borehole					

Monitoring Well Details:



Water level measured at 5.07 mbgs on November 13, 2018.

Contractor: Strata Drilling Group

Drilling Method: Direct Push / Air Rotary

Well Casing Size: 5.08 cm

Note:
Soil vapour concentrations measured using a RKI Eagle 2 equipped with a photoionization detector (PID) and a combustible gas indicator (CGI).

Grade Elevation: 100.29

Top of Casing Elevation: 100.19

Sheet: 1 of 1

Log of Borehole: MW-2

Project #: 230236.002 **Logged By:** MK

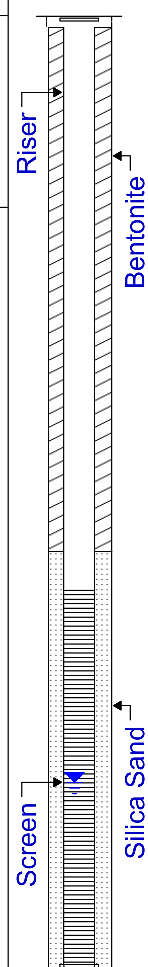
Project: Phase II Environmental Site Assessment

Client: The Great-West Life Assurance Company

Location: 320 McRae Avenue, 1976 Scott Street, 311 and 316 Tweedsmuir Avenue, Ottawa, Ontario

Drill Date: November 1, 2018

SUBSURFACE PROFILE					SAMPLE			
Depth (ft / m)	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Recovery (%)	Sample ID	Soil Vapour Concentration (ppm)(HEX/IBL)	Laboratory Analysis
0 ft / 0 m		Ground Surface	0.00					
0–5 ft / 0–1.52 m		***Sand and Gravel*** Grey/brown, damp.			30	SS1	0/1	
			1.52		30	SS2	0/1	Metals, PHCs, PAHs, VOCs
5–25 ft / 1.52–7.62 m		***Limestone***	7.62					
		End of Borehole		Water level measured at 6.13 mbgs on November 13, 2018.				



Contractor: Strata Drilling Group

Drilling Method: Direct Push / Air Rotary

Well Casing Size: 5.08 cm

Note:
Soil vapour concentrations measured using a RKI Eagle 2 equipped with a photoionization detector (PID) and a combustible gas indicator (CGI).

Grade Elevation: 100.17

Top of Casing Elevation: 100.28

Sheet: 1 of 1

Log of Borehole: MW-3

Project #: 230236.002 **Logged By:** MK

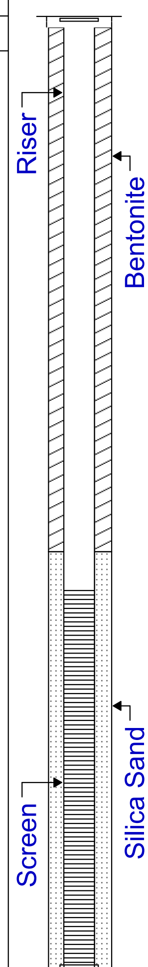
Project: Phase II Environmental Site Assessment

Client: The Great-West Life Assurance Company

Location: 320 McRae Avenue, 1976 Scott Street, 311 and 316 Tweedsmuir Avenue, Ottawa, Ontario

Drill Date: November 1, 2018

SUBSURFACE PROFILE					SAMPLE			
Depth (ft / m)	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Recovery (%)	Sample ID	Soil Vapour Concentration (ppm)(HEX/IBL)	Laboratory Analysis
0 / 0		Ground Surface	0.00					
0–1		***Sand and Gravel*** Grey/brown, damp. Limestone fragments @ 0.75 mbgs	0.28		30	SS1	0/1	PHCs, VOCs, PAHs
1–25 (0.28–7.62 m)		***Limestone***	7.62					
25–29		End of Borehole		Well was submerged and frozen on November 13, 2018.				



Contractor: Strata Drilling Group

Drilling Method: Direct Push / Air Rotary

Well Casing Size: 5.08 cm

Note:
Soil vapour concentrations measured using a RKI Eagle 2 equipped with a photoionization detector (PID) and a combustible gas indicator (CGI).

Grade Elevation: NM

Top of Casing Elevation: NM

Sheet: 1 of 1

Log of Borehole: MW-4

Project #: 230236.002 **Logged By:** MK

Project: Phase II Environmental Site Assessment

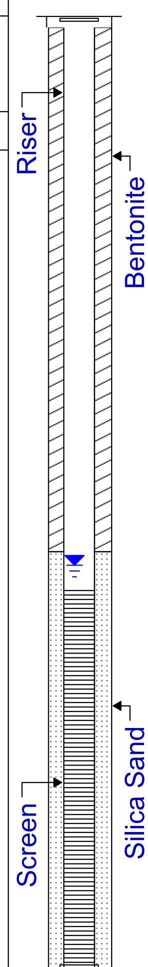
Client: The Great-West Life Assurance Company

Location: 320 McRae Avenue, 1976 Scott Street, 311 and 316 Tweedsmuir Avenue, Ottawa, Ontario

Drill Date: November 2, 2018

SUBSURFACE PROFILE				SAMPLE			
Depth (ft / m)	Symbol	Description	Measured Depth (m)	Recovery (%)	Sample ID	Soil Vapour Concentration (ppm)(HEX/IBL)	Laboratory Analysis
0 ft / 0 m		Ground Surface	0.00				
0–2.5 ft		**Sand and Gravel** With brick fragments, damp.	0.76	50	SS1	0/2	PHCs, VOCs, PAHs, Metals
2.5–3.5 ft		**Fill** Sand, brick and glass.	1.07	20	SS2	0/1	
3.5–25 ft		**Limestone**	7.62				
		End of Borehole					

Monitoring Well Details:



Water level measured at 4.37 mbgs on November 13, 2018.

Contractor: Strata Drilling Group

Drilling Method: Direct Push / Split Spoon / Air Rotary

Well Casing Size: 5.08 cm

Note:
Soil vapour concentrations measured using a RKI Eagle 2 equipped with a photoionization detector (PID) and a combustible gas indicator (CGI).

Grade Elevation: 100.81

Top of Casing Elevation: 100.70

Sheet: 1 of 1

Log of Borehole: BH4

Project #: 230236.004 **Logged By:** WT

Project: Geotechnical Investigation

Client: The Great-West Life Assurance Company

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 12, 2018 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE										
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20 40 60)	Shear Strength kPa (100 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	0.00											
0		Asphalt ~ 50 mm Fill - gravelly silty sand, trace brick, trace glass, trace bedrock fragments, damp, brown, very loose to compact	-1.07		SS	SS1	50	28						
					SS	SS2	5	1						
1		Limestone rock, slightly weathered. Grey with black and white banding, fine to medium grained, and contained few natural fractures with little to no oxidation. Very poor to poor quality	-4.11		NQ	Run 1	60							RQD=7%
					NQ	Run 2	70							RQD=40%
4		End of Borehole												

Contractor: Strata Drilling Group **Grade Elevation:** N/A

Drilling Method: Direct Push/Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 1

Log of Borehole: BH5

Project #: 230236.004 **Logged By:** WT

Project: Geotechnical Investigation

Client: The Great-West Life Assurance Company

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 2, 2018 **Project Manager:** WT

SUBSURFACE PROFILE / SAMPLE

Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20 40 60)	Shear Strength kPa (100 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	0.00											
0		Asphalt ~ 100 mm Fill - gravelly silty sand, trace brick, trace bedrock fragments, damp, brown, very loose to compact			SS	SS1	50	9						
1					SS	SS2	10	44						
			-1.52											
		End of Borehole Due to SPT refusal on bedrock												
2														
3														
4														
5														
6														

Contractor: Strata Drilling Group

Drilling Method: Direct Push/Split Spoon

Well Casing Size: N/A

Grade Elevation: N/A

Top of Casing Elevation: N/A

Sheet: 1 of 1

Log of Borehole: BH6

Project #: 230236.004 **Logged By:** WT

Project: Geotechnical Investigation

Client: The Great-West Life Assurance Company

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 2, 2018 **Project Manager:** WT

SUBSURFACE PROFILE / SAMPLE

Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20 40 60)	Shear Strength kPa (100 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	0.00											
0		Asphalt ~ 100 mm Fill - gravelly silty sand, trace brick, trace bedrock fragments, damp, brown, compact to dense			SS	SS1	50	17						
1					SS	SS2	80	41						
			-1.52											
		End of Borehole Due to SPT refusal on bedrock												
2														
3														
4														
5														
6														

Contractor: Strata Drilling Group

Drilling Method: Direct Push/Split Spoon

Well Casing Size: N/A

Grade Elevation: N/A

Top of Casing Elevation: N/A

Sheet: 1 of 1

Log of Borehole: BH7

Project #: 230236.004 **Logged By:** WT

Project: Geotechnical Investigation

Client: The Great-West Life Assurance Company

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 2, 2018 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20 40 60)	Shear Strength kPa (100 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	0.00											
0		Asphalt ~ 100 mm Fill - gravelly silty sand, trace brick, trace bedrock fragments, damp, brown, compact			SS	SS1	50	10						
1					SS	SS2	80	18						
			-1.52											
		End of Borehole Due to SPT refusal on bedrock												
2														
3														
4														
5														
6														

Contractor: Strata Drilling Group **Grade Elevation:** N/A

Drilling Method: Direct Push/Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 1

Log of Borehole: BH8

Project #: 230236.004 **Logged By:** WT

Project: Geotechnical Investigation

Client: The Great-West Life Assurance Company

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 2, 2018 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE										
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20 40 60)	Shear Strength kPa (100 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	0.00											
0		Asphalt ~ 100 mm Fill - gravelly silty sand, trace brick, trace bedrock fragments, damp, brown, compact	-0.91		SS	SS1	50	13						
1		End of Borehole Due to SPT refusal on bedrock												
2														
3														
4														
5														
6														

Contractor: Strata Drilling Group **Grade Elevation:** N/A

Drilling Method: Direct Push/Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 1

Log of Borehole: BH9

Project #: 230236.004 ***Logged By:*** WT

Project: Geotechnical Investigation

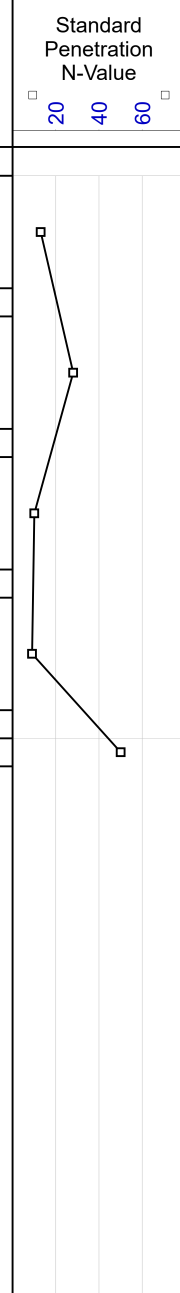
Client: The Great-West Life Assurance Company

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 2, 2018 ***Project Manager:*** WT

SUBSURFACE PROFILE / SAMPLE

Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20 40 60)	Shear Strength kPa (100 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	0.00											
0		Asphalt ~ 100 mm Fill - gravelly sand, trace to some silt, damp, brown, loose to dense			SS	SS1	50	13						
1					SS	SS2	50	28						
2					SS	SS3	60	10						
					SS	SS4	60	9						
3			-3.20		SS	SS5	40	50						
		End of Borehole Due to SPT refusal on bedrock												
4														
5														
6														



Contractor: Strata Drilling Group

Drilling Method: Direct Push/Split Spoon

Well Casing Size: N/A

Grade Elevation: N/A

Top of Casing Elevation: N/A

Sheet: 1 of 1

Log of Borehole: BH10

Project #: 230236.004 **Logged By:** WT

Project: Geotechnical Investigation

Client: The Great-West Life Assurance Company

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 2, 2018 **Project Manager:** WT

Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20 40 60)	Shear Strength kPa (100 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	0.00											
0		Asphalt ~ 100 mm Fill - gravelly silty sand, trace bedrock fragments, damp, brown, loose to dense			SS	SS1	45	9						
1					SS	SS2	80	33						
			-1.68		SS	SS3	40	50						
2		End of Borehole Due to SPT refusal on bedrock												

Contractor: Strata Drilling Group

Drilling Method: Direct Push/Split Spoon

Well Casing Size: N/A

Grade Elevation: N/A

Top of Casing Elevation: N/A

Sheet: 1 of 1

Log of Borehole: BH11

Project #: 230236.004 **Logged By:** WT

Project: Geotechnical Investigation

Client: The Great-West Life Assurance Company

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 2, 2018 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value 20 40 60	Shear Strength kPa 100 200	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
		Ground Surface	0.00											
0		Asphalt ~ 50 mm Fill - gravelly silty sand, trace bedrock fragments, damp, brown, dense	-0.76		SS	SS1	75	35						
1		End of Borehole Due to SPT refusal on bedrock												
2														
3														
4														
5														
6														

Contractor: Strata Drilling Group **Grade Elevation:** N/A

Drilling Method: Direct Push/Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 1

Log of Borehole: BH12

Project #: 230236.004 **Logged By:** WT

Project: Geotechnical Investigation

Client: The Great-West Life Assurance Company

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 9 and 12, 2018 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE										
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20 40 60)	Shear Strength kPa (100 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0		Ground Surface	0.00											
0–1.22		Organics ~ 200 mm Fill - gravelly silty sand, trace brick, trace glass, trace bedrock fragments, damp, brown, compact	-1.22		SS	SS1	60	17						
					SS	SS2	25	12						
1.22–4.27		Limestone rock, slightly weathered in the upper layers and fresh in the deeper layers. Grey with black and white banding, fine to medium grained, and contained few natural fractures with little to no oxidation. Poor Quality	-4.27		NQ	Run 1	72							RQD=33%
					NQ	Run 2	88							RQD=51%
4.27–		Good to excellent quality			NQ	Run 3	92							RQD=82%
					NQ	Run 4	100							RQD=88%
					NQ	Run 5	95							RQD=93%
					NQ	Run 6	98							RQD=93%

Contractor: Strata Drilling Group **Grade Elevation:** N/A

Drilling Method: Direct Push/Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 2

Log of Borehole: BH12

Project #: 230236.004 **Logged By:** WT

Project: Geotechnical Investigation

Client: The Great-West Life Assurance Company

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 9 and 12, 2018 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20 40 60)	Shear Strength kPa (100 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
11					NQ	Run 7	100							RQD=85%
12–13					NQ	Run 8	98							RQD=83%
14					NQ	Run 9	97							RQD=80%
15–16			-16.46		NQ	Run10	100							RQD=77%
17		Very poor quality	-17.37		NQ	Run11	100							RQD=17%
18		Fair quality	-18.90		NQ	Run12	100							RQD=63%
19–20		Excellent quality	-20.42		NQ	Run13	100							RQD=93%
		End of Borehole												

Contractor: Strata Drilling Group **Grade Elevation:** N/A

Drilling Method: Direct Push/Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 2 of 2

APPENDIX III

Analytical Laboratory Testing Reports for Soil Samples

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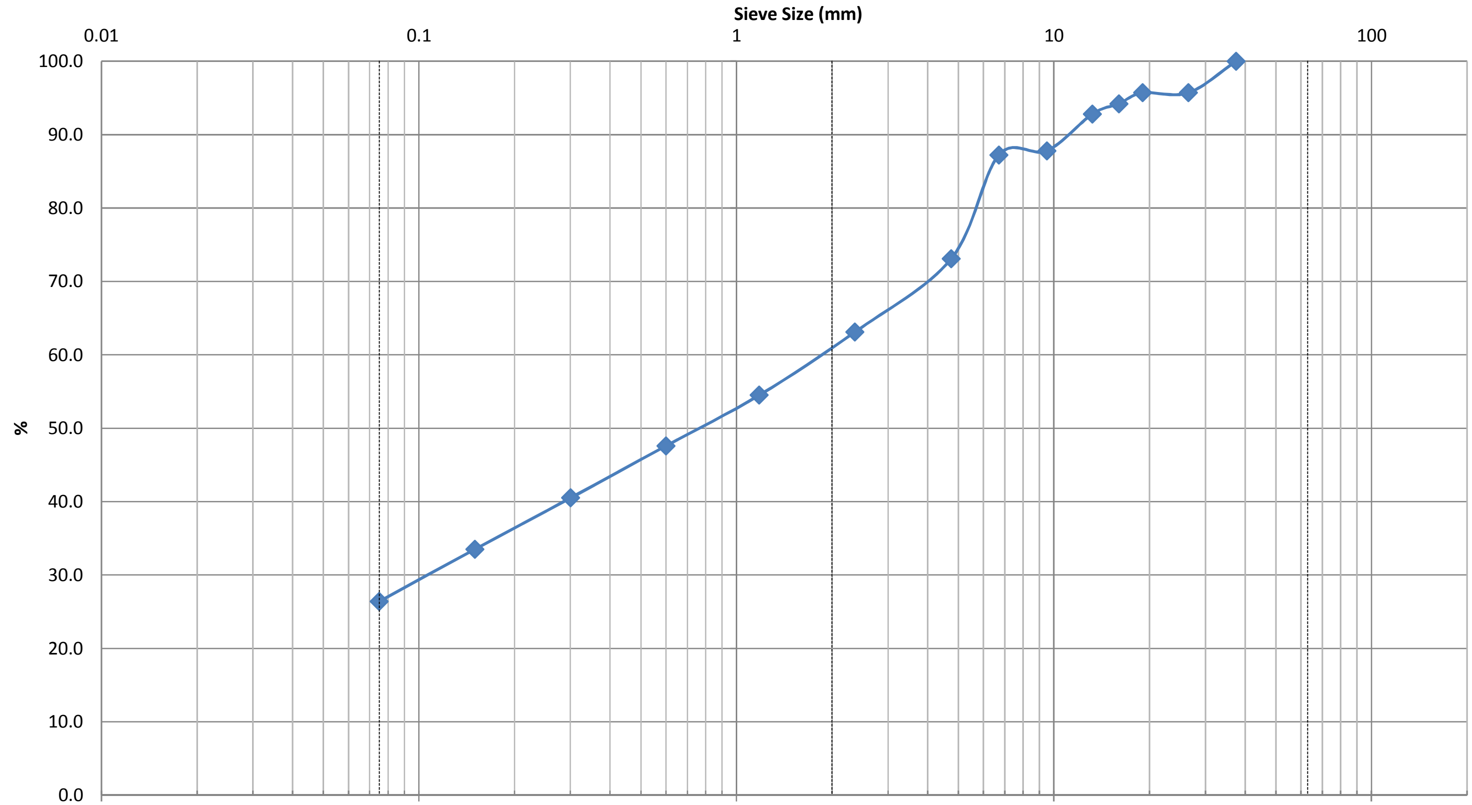
SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin Environmental	DESCRIPTION:	Silty Sand	FILE NO:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	-	LAB NO:	06324
PROJECT:	Laboratory Testing Job # 230236.004	INTENDED USE:	-	DATE RECEIVED:	19-Nov-18
		PIT OR QUARRY:	-	DATE TESTED:	20-Nov-18
DATE SAMPLED:	2-Nov-18	SOURCE LOCATION:	BH6	DATE REPORTED:	22-Nov-18
SAMPLED BY:	Client	SAMPLE LOCATION:	2.5 - 4.5'	TESTED BY:	D.K/D.B



Silt and Clay	Sand			Gravel		Cobble
	Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
									0.33	92.5
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)		Clay (%)	
	38.5	1.85	0.11	0.02	26.9	46.7	26.4			

Comments	

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consulting engineers

SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin Environmental	DESCRIPTION:	Silty Sand	FILE NO.:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	-	LAB NO.:	06324
PROJECT:	Laboratory Testing Job # 230236.004	INTENDED USE:	-	DATE REC'D:	19-Nov-18
		PIT OR QUARRY:	-	DATE TESTED:	20-Nov-18
DATE SAMPLED:	02-Nov-18	SOURCE LOCATION:	BH6	DATE REP'D:	22-Nov-18
SAMPLED BY:	Client	SAMPLE LOCATION:	2.5 - 4.5'	TESTED BY:	D.K/D.B

WEIGHT BEFORE WASH			A+B	934.7
WEIGHT AFTER WASH	A	B	A+B	699.6

SIEVE SIZE (mm)	WEIGHT RETAINED	PERCENT RETAINED	PERCENT PASSING	LOWER SPEC	UPPER SPEC	REMARK
150						
106						
75						
63						
53						
37.5	0.0	0.0	**100.0**			
26.5	**40.6**	4.3	**95.7**			
19	**40.6**	4.3	**95.7**			
16	**54.2**	5.8	**94.2**			
13.2	**67.1**	7.2	**92.8**			
9.5	**114.0**	12.2	**87.8**			
6.7	**119.7**	12.8	**87.2**			
4.75	**251.4**	26.9	**73.1**			
2.36	**345.1**	36.9	**63.1**			
1.18	**425.3**	45.5	**54.5**			
0.6	**489.9**	52.4	**47.6**			
0.3	**555.7**	59.5	**40.5**			
0.15	**621.4**	66.5	**33.5**			
0.075	**687.5**	73.6	**26.4**			
PAN	**699.6**					
SIEVE CHECK FINE		0.00	0.3% max.			**REFERENCE MATERIAL**

OTHER TESTS	**RESULT**	**LAB NO.**	**RESULT**

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

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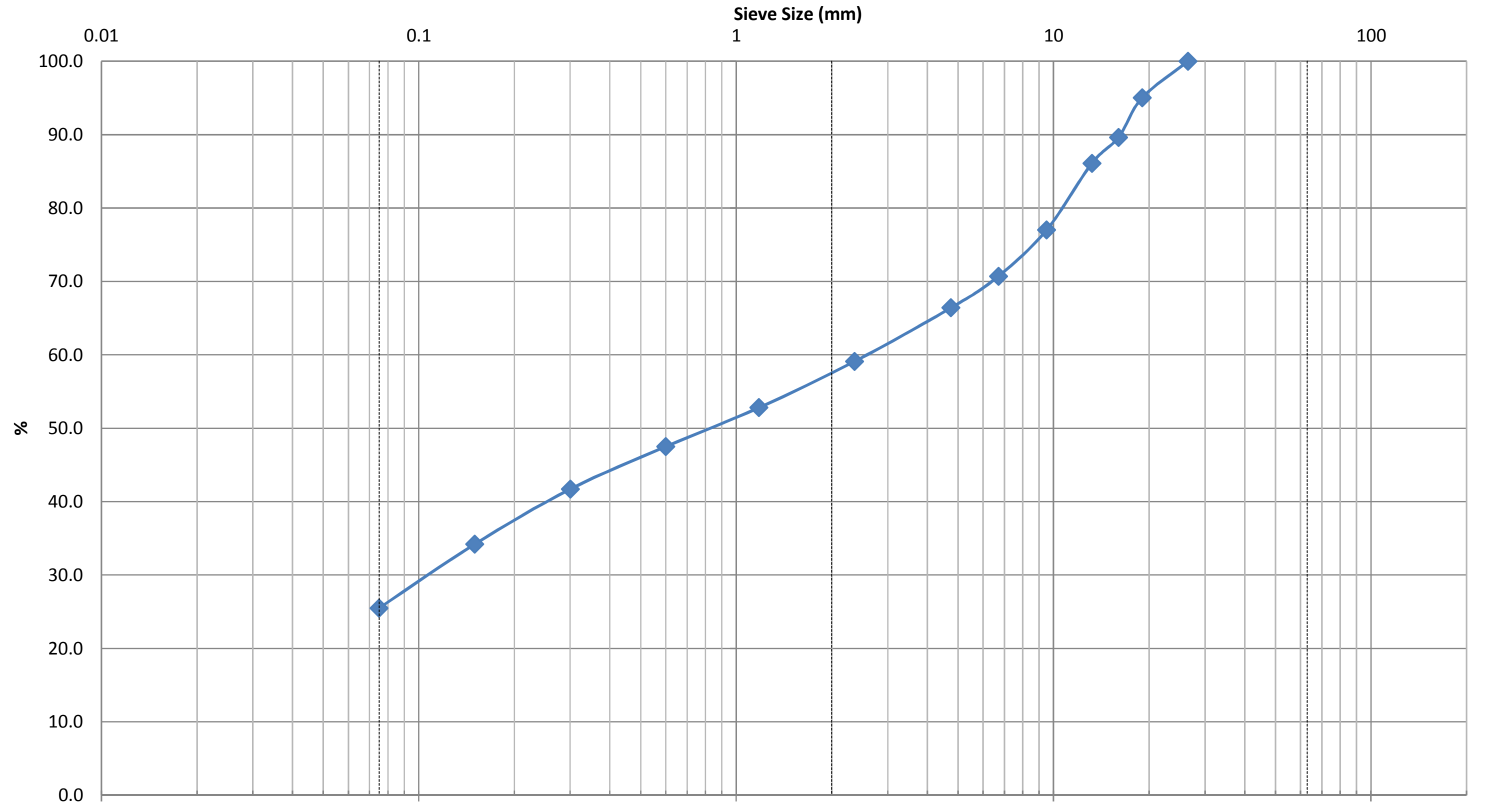
SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin Environmental	DESCRIPTION:	Silty Sand	FILE NO:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	-	LAB NO:	06325
PROJECT:	Laboratory Testing Job # 230236.004	INTENDED USE:	-	DATE RECEIVED:	19-Nov-18
		PIT OR QUARRY:	-	DATE TESTED:	20-Nov-18
DATE SAMPLED:	2-Nov-18	SOURCE LOCATION:	BH10	DATE REPORTED:	22-Nov-18
SAMPLED BY:	Client	SAMPLE LOCATION:	2.5 - 4.5'	TESTED BY:	D.K/D.B



Silt and Clay	Sand			Gravel		Cobble
	Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
										0.19	104.0
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
	26.5	2.6	0.11	0.025	33.6	40.9		25.5			

Comments	

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SIEVE ANALYSIS ASTM C136

CLIENT:	Pinchin Environmental	DESCRIPTION:	Silty Sand	FILE NO.:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	-	LAB NO.:	06325
PROJECT:	Laboratory Testing Job # 230236.004	INTENDED USE:	-	DATE REC'D:	19-Nov-18
		PIT OR QUARRY:	-	DATE TESTED:	20-Nov-18
DATE SAMPLED:	02-Nov-18	SOURCE LOCATION:	BH10	DATE REP'D:	22-Nov-18
SAMPLED BY:	Client	SAMPLE LOCATION:	2.5 - 4.5'	TESTED BY:	D.K/D.B

WEIGHT BEFORE WASH			A+B	983.0
WEIGHT AFTER WASH	A	B	A+B	764.8

SIEVE SIZE (mm)	WEIGHT RETAINED	PERCENT RETAINED	PERCENT PASSING	LOWER SPEC	UPPER SPEC	REMARK
150						
106						
75						
63						
53						
37.5						
26.5	**0.0**	0.0	**100.0**			
19	**49.3**	5.0	**95.0**			
16	**102.0**	10.4	**89.6**			
13.2	**137.1**	13.9	**86.1**			
9.5	**225.6**	23.0	**77.0**			
6.7	**287.9**	29.3	**70.7**			
4.75	**329.8**	33.6	**66.4**			
2.36	**402.2**	40.9	**59.1**			
1.18	**464.1**	47.2	**52.8**			
0.6	**516.1**	52.5	**47.5**			
0.3	**572.8**	58.3	**41.7**			
0.15	**646.6**	65.8	**34.2**			
0.075	**732.7**	74.5	**25.5**			
PAN	**764.8**					
SIEVE CHECK FINE		0.00	0.3% max.			**REFERENCE MATERIAL**

OTHER TESTS	**RESULT**	**LAB NO.**	**RESULT**

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

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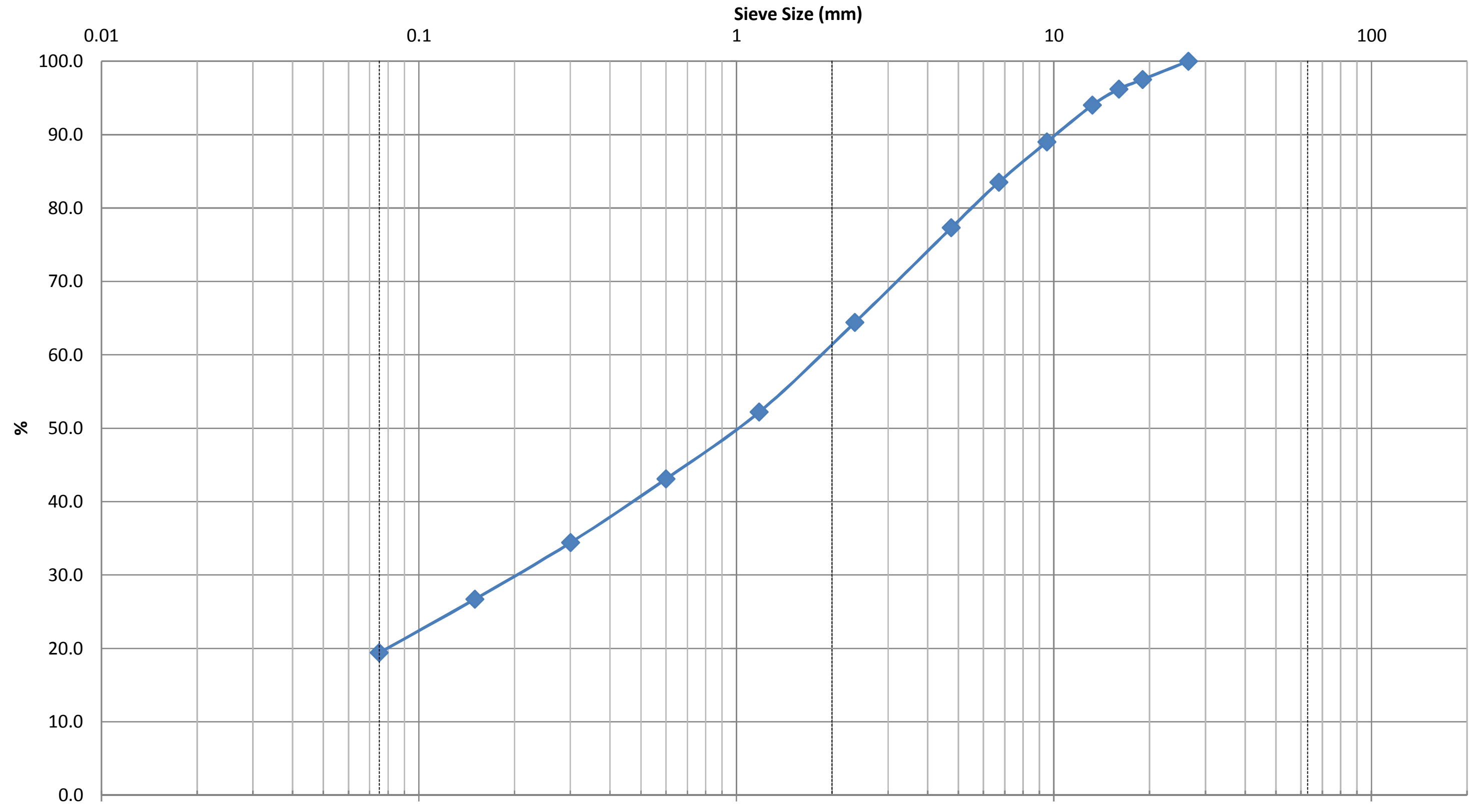
SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin Environmental	DESCRIPTION:	Silty Sand	FILE NO:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	-	LAB NO:	06326
PROJECT:	Laboratory Testing Job # 230236.004	INTENDED USE:	-	DATE RECEIVED:	19-Nov-18
		PIT OR QUARRY:	-	DATE TESTED:	20-Nov-18
DATE SAMPLED:	2-Nov-18	SOURCE LOCATION:	BH11	DATE REPORTED:	22-Nov-18
SAMPLED BY:	Client	SAMPLE LOCATION:	0 - 2'	TESTED BY:	D.K/D.B



Silt and Clay	Sand			Gravel		Cobble
	Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
									1.05	95.0
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)	Clay (%)	
	26.5	1.9	0.2	0.02	22.7	57.9		19.4		

Comments	

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SIEVE ANALYSIS ASTM C136

CLIENT:	Pinchin Environmental	DESCRIPTION:	Silty Sand	FILE NO.:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	-	LAB NO.:	06326
PROJECT:	Laboratory Testing Job # 230236.004	INTENDED USE:	-	DATE REC'D:	19-Nov-18
		PIT OR QUARRY:	-	DATE TESTED:	20-Nov-18
DATE SAMPLED:	02-Nov-18	SOURCE LOCATION:	BH11	DATE REP'D:	22-Nov-18
SAMPLED BY:	Client	SAMPLE LOCATION:	0 - 2'	TESTED BY:	D.K/D.B

WEIGHT BEFORE WASH			A+B	884.6
WEIGHT AFTER WASH	A	B	A+B	728.6

SIEVE SIZE (mm)	WEIGHT RETAINED	PERCENT RETAINED	PERCENT PASSING	LOWER SPEC	UPPER SPEC	REMARK
150						
106						
75						
63						
53						
37.5						
26.5	**0.0**	0.0	**100.0**			
19	**22.3**	2.5	**97.5**			
16	**33.3**	3.8	**96.2**			
13.2	**53.4**	6.0	**94.0**			
9.5	**97.7**	11.0	**89.0**			
6.7	**145.7**	16.5	**83.5**			
4.75	**200.8**	22.7	**77.3**			
2.36	**314.9**	35.6	**64.4**			
1.18	**423.0**	47.8	**52.2**			
0.6	**503.7**	56.9	**43.1**			
0.3	**580.5**	65.6	**34.4**			
0.15	**648.2**	73.3	**26.7**			
0.075	**713.4**	80.6	**19.4**			
PAN	**728.0**					
SIEVE CHECK FINE		0.08	0.3% max.			**REFERENCE MATERIAL**

OTHER TESTS	**RESULT**	**LAB NO.**	**RESULT**

REVIEWED BY:	**Curtis Beadow**	**Joe Forsyth, P. Eng.**

APPENDIX IV

Report Limitations and Guidelines for Use

REPORT LIMITATIONS & GUIDELINES FOR USE

This information has been provided to help manage risks with respect to the use of this report.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES, PERSONS AND PROJECTS

This report was prepared for the exclusive use of the Client and their authorized agents, subject to the conditions and limitations contained within the duly authorized work plan. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of the third parties. If additional parties require reliance on this report, written authorization from Pinchin will be required. Pinchin disclaims responsibility of consequential financial effects on transactions or property values, or requirements for follow-up actions and costs. No other warranties are implied or expressed. Furthermore, this report should not be construed as legal advice.

SUBSURFACE CONDITIONS CAN CHANGE

This geotechnical report is based on the existing conditions at the time the study was performed, and Pinchin's opinion of soil conditions are strictly based on soil samples collected at specific test hole locations. The findings and conclusions of Pinchin's reports may be affected by the passage of time, by manmade events such as construction on or adjacent to the Site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations.

LIMITATIONS TO PROFESSIONAL OPINIONS

Interpretations of subsurface conditions are based on field observations from test holes that were spaced to capture a 'representative' snap shot of subsurface conditions. Site exploration identifies subsurface conditions only at points of sampling. Pinchin reviews field and laboratory data and then applies professional judgment to formulate an opinion of subsurface conditions throughout the Site. Actual subsurface conditions may differ, between sampling locations, from those indicated in this report.

LIMITATIONS OF RECOMMENDATIONS

Subsurface soil conditions should be verified by a qualified geotechnical engineer during construction. Pinchin should be notified if any discrepancies to this report or unusual conditions are found during construction.

Sufficient monitoring, testing and consultation should be provided by Pinchin during construction and/or excavation activities, to confirm that the conditions encountered are consistent with those indicated by the test hole investigation, and to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated. In addition, monitoring, testing and consultation by Pinchin should be completed to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining Pinchin for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions. However, please be advised that any construction/excavation observations by Pinchin is over and above the mandate of this geotechnical evaluation and therefore, additional fees would apply.

MISINTERPRETATION OF GEOTECHNICAL ENGINEERING REPORT

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having Pinchin confer with appropriate members of the design team after submitting the report. Also retain Pinchin to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having Pinchin participate in pre-bid and preconstruction conferences, and by providing construction observation. Please be advised that retaining Pinchin to participation in any 'other' activities associated with this project is over and above the mandate of this geotechnical investigation and therefore, additional fees would apply.

CONTRACTORS RESPONSIBILITY FOR SITE SAFETY

This geotechnical report is not intended to direct the contractor's procedures, methods, schedule or management of the work Site. The contractor is solely responsible for job Site safety and for managing construction operations to minimize risks to on-Site personnel and to adjacent properties. It is ultimately the contractor's responsibility that the Ontario Occupational Health and Safety Act is adhered to, and Site conditions satisfy all 'other' acts, regulations and/or legislation that may be mandated by federal, provincial and/or municipal authorities.

SUBSURFACE SOIL AND/OR GROUNDWATER CONTAMINATION

This report is geotechnical in nature and was not performed in accordance with any environmental guidelines. As such, any environmental comments are very preliminary in nature and based solely on field observations. Accordingly, the scope of services do not include any interpretations, recommendations, findings, or conclusions regarding the, assessment, prevention or abatement of contaminants, and no conclusions or inferences should be drawn regarding contamination, as they may relate to this project. The term "contamination" includes, but is not limited to, molds, fungi, spores, bacteria, viruses, PCBs, petroleum hydrocarbons, inorganics, pesticides/insecticides, volatile organic compounds, polycyclic aromatic hydrocarbons and/or any of their by-products.

Pinchin will not be responsible for any consequential or indirect damages. Pinchin will only be held liable for damages resulting from the negligence of Pinchin. Pinchin will not be liable for any losses or damage if the Client has failed, within a period of two years following the date upon which the claim is discovered within the meaning of the Limitations Act, 2002 (Ontario), to commence legal proceedings against Pinchin to recover such losses or damage.

APPENDIX V

Rock Core Photographs

Photo 1 – Borehole BH4, Rock Core (Runs 1 and 2)

Photo 2 – Borehole BH12, Rock Core (Runs 1 to 3)

Photo 3 – Borehole BH12, Rock Core (Runs 4 to 6)

Photo 4 – Borehole BH12, Rock Core (Runs 7 to 9)

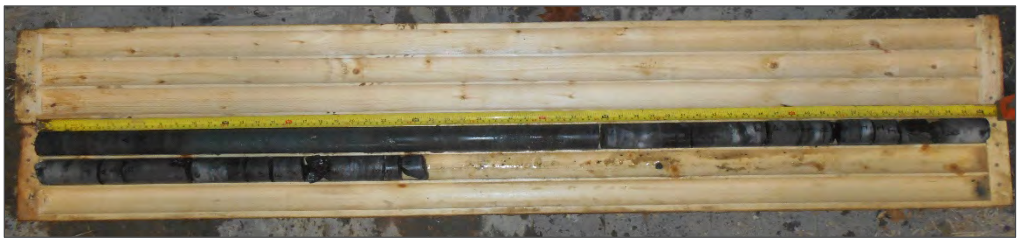

Photo 5 – Borehole BH12, Rock Core (Runs 10 and 11)

Photo 6 – Borehole BH12, Rock Core (Runs 12 and 13)

APPENDIX VI

Geophysics GPR International Inc. Shear-Wave Velocity Sounding

GEOPHYSICS GPR INTERNATIONAL INC.
6741 Columbus Road
Unit 14
Mississauga, Ontario
Canada L5T 2G9
Tel.: (905) 696-0656
Fax: (905) 696-0570
gprtor@gprtor.com
www.geophysicsgpr.com

February 14, 2020 GPR file: T201972

Wesley Tabaczuk, P.Eng.
Project Manager, Geotechnical Services
Pinchin Ltd.
1001 – 555 Legget Drive, Tower A
Kanata, Ontario
K2K 2X3

RE: Shear-wave velocity sounding at 320 McRae Avenue, Ottawa, Ontario

Dear Mr. Tabaczuk:

Geophysics GPR International Inc. has been requested by Pinchin Ltd. to carry out a shear-wave velocity sounding at the above site in Ottawa. Figure 1 shows the location of the test profile.

The survey was performed on February 3rd, 2020.

The investigation included the multi-channel analysis of surface waves (MASW), the micro-tremor array measurements (MAM) and the refraction methods to generate a shear-wave velocity model (Figure 4).

The following paragraphs describe the survey design, the principles of the test method, the methodology for interpreting the data, and provide a culmination of the results in table format.

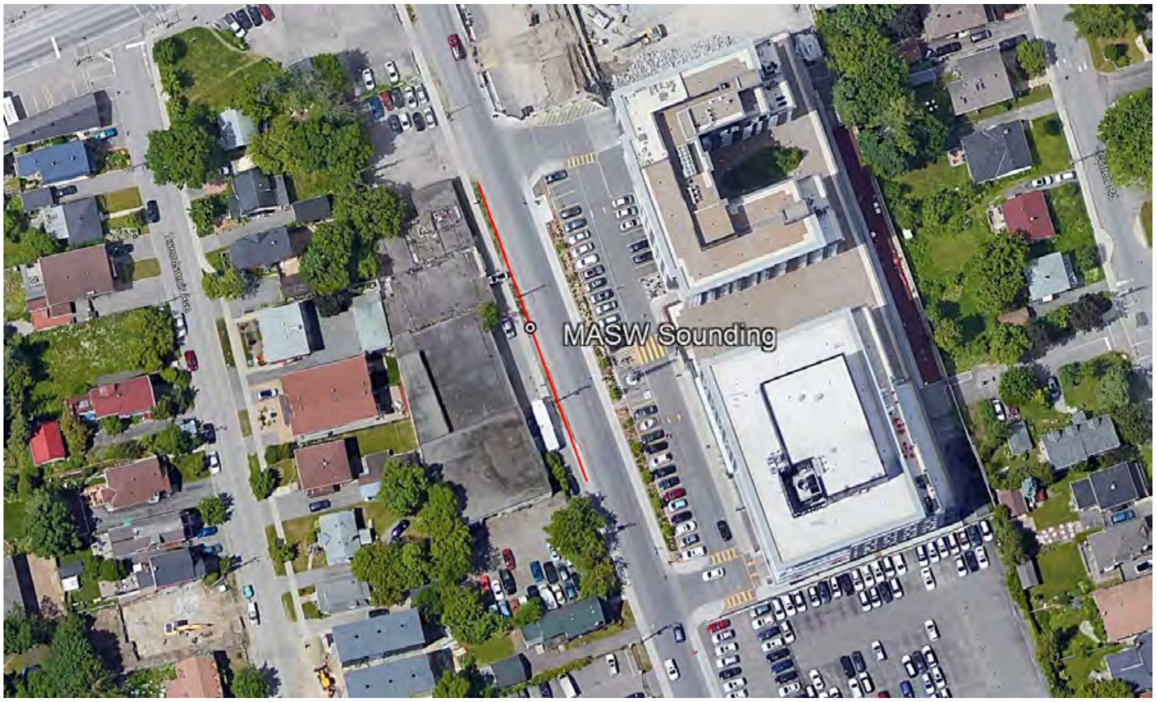

Figure 1: Approximate location of the shear-wave velocity sounding

MASW and MAM Surveys

Basic Theory

The Multi-channel Analysis of Surface Waves (MASW) and the Micro-tremor Array Measurements (MAM) are seismic methods used to evaluate the shear-wave velocities of subsurface materials through the analysis of the dispersion properties of Rayleigh surface waves ("ground roll"). The dispersion properties are measured as a change in phase velocity with frequency. Surface wave energy will decay exponentially with depth. Lower frequency surface waves will travel deeper and thus be more influenced by deeper velocity layering than the shallow higher frequency waves. Inversion of the Rayleigh wave dispersion curve yields a shear-wave (V_s) velocity depth profile (sounding). Figure 2 outlines the basic operating procedure for the MASW method. Figure 3 is an example image of a typical MASW record and resulting 1D V_s model. A more detailed description of the method can be found in the paper *Multi-channel Analysis of Surface Waves,* Park, C.B., Miller, R.D. and Xia, J. Geophysics, Vol. 64, No. 3 (May-June 1999); P. 800–808.

Survey Design

The geometry of an MASW survey is similar to that of a seismic refraction investigation (i.e. 24 geophones in a linear array). The fundamental principle involves intentionally generating an acoustic wave at the surface and digitally recording the surface waves from the moment of source impact with a linear series of geophones on the surface. This is referred to as an "active source" method. An elastic-wave hammer was used as the primary energy source with traces being recorded at 6 locations: approximately 6 m off both ends, 25 to 30 m off both ends, and in the middle of the spread. Data were collected with geophones spacing of 3m and 1m for a total of 10 shot records per sounding.

Unlike the refraction method, which produces a data point beneath each geophone, the shear-wave depth profile is the average of the bulk area within the middle third of the geophone spread.

The theoretical maximum depth of penetration (34.5m) is half of the maximum seismic array length (69 m), in practice the maximum depth of penetration is often influenced by the geology.

The MAM/passive survey used the same geophone array set up as for the MASW survey. Unlike the MASW survey, the MAM method is considered a "passive source" method in that there is no time break and the motions recorded are from ambient energy generated by cultural noise such as traffic, wind, wave motion, etc. Data collection for the passive method involves recording approximately 10 minutes of background "noise." The records generated by the MAM method contain lower frequency data, thus increasing the data resolution at greater depths of investigation. Typically the MAM results aid in clarifying the MASW results for depths greater than 20 m; however, the direction of noise propagation relative to the spread orientation can influence the results.

Interpretation Method and Accuracy of Results

The main processing sequence involved plotting, picking, and 1-D inversion of the MASW/MAM shot records using the SeisimagerSW™ software package. In theory, all MASW shot records should produce a similar shear-wave velocity profile. In practice, however, differences can arise due to energy dissipation and localized surface variations. The results of the inversion process are inherently non-unique and the final model must be judged to be geologically realistic. The inversion modelling also assumes that all layering is flat/horizontal and laterally uniform.

The results of the MASW/MAM tests are presented in chart format as Figure 4. The chart presents the 1-D shear wave velocity values from the inversion models of the passive and active seismic records.

The V_s30 values for the sounding are presented in Table 1. The V_s30 values are based on the harmonic mean of the shear wave velocities over the upper 30 m. The V_s30 value is calculated by dividing the total depth of interest (e.g. 30 m) by the sum of the time spent in each velocity layer up to that depth. This harmonic mean value reflects the equivalent single layer response.

The estimated error in the average V_s30 value determined through MASW tests is typically +/-10 to 15% for overburden sites. The shear-wave velocities modelled through the MASW method within bedrock have a higher estimated error.

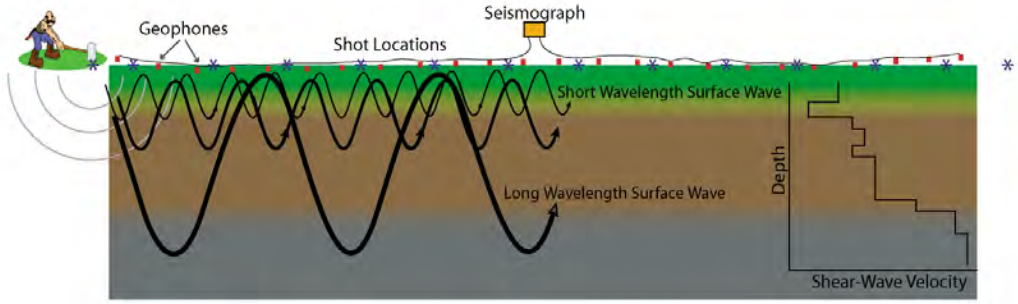

Figure 2: MASW Operating Principle

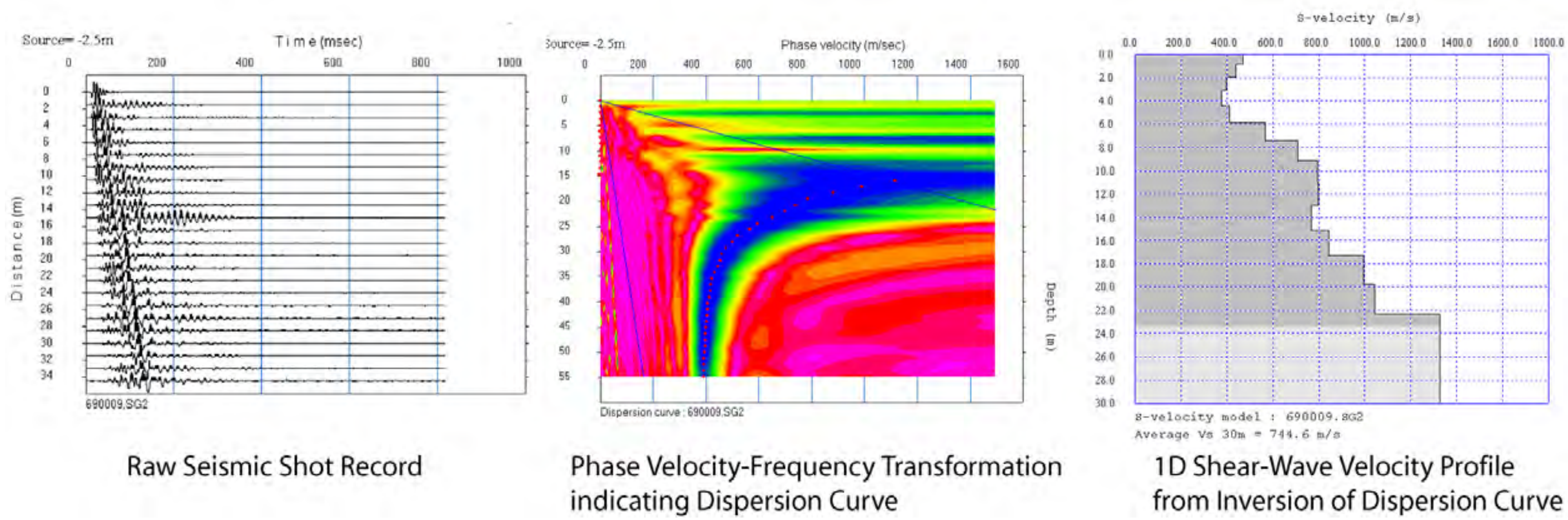

Figure 3: Example of a typical MASW shot record, phase velocity/frequency curve and resulting 1D shear-wave velocity model.

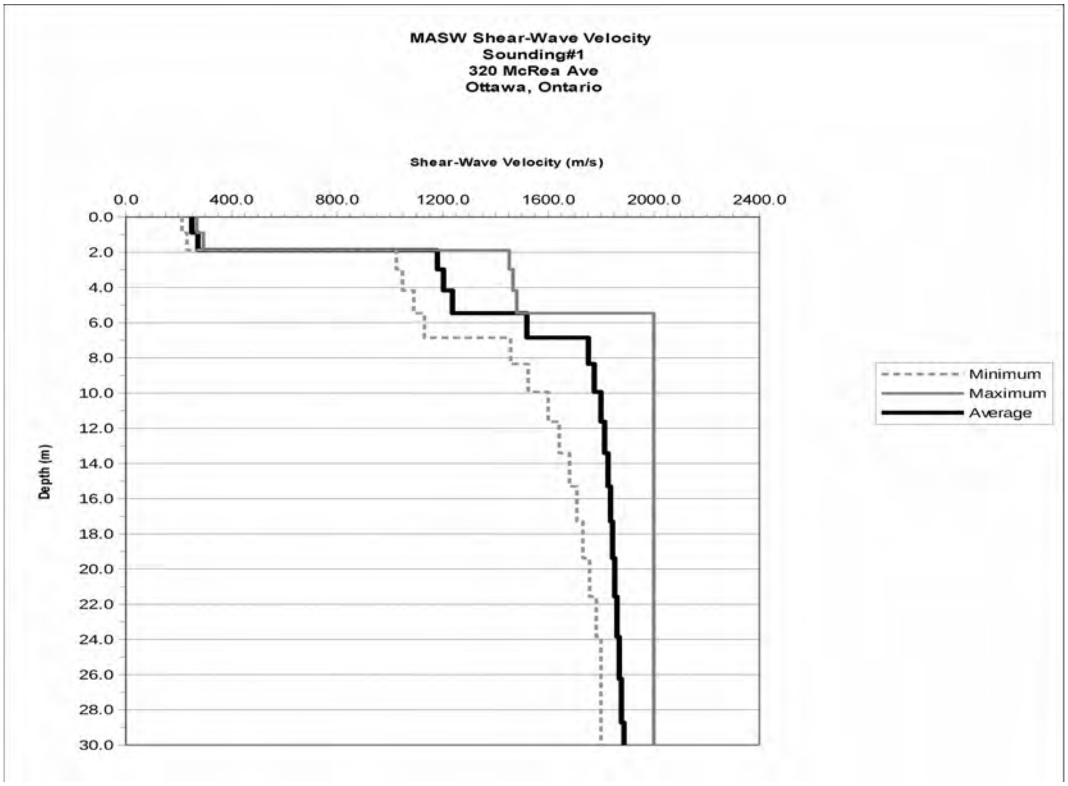

Figure 4: MASW Shear-wave Velocity Sounding

CONCLUSIONS

The approximate location of the shear-wave sounding is indicated in Figure 1.

The MASW shear-wave models are presented in Figure 4. The results are summarized in Table 1. The background seismic noise levels at this site were moderate. The quality of the seismic records and the resulting dispersion were good.

Simple critical distance calculations from refracted P-waves show that bedrock is shallow, in the order of 2m. Refracted P-wave velocities of approximately 4800m/s were measured for the competent bedrock.

The provided boreholes confirmed the general depth of the bedrock in the area.

Table 1: Calculted V_s30 values (m/s) from the MASW data (0 to 30m)

Sounding	Minimum	Average	Maximum	Site Class
1	1121	**1266**	1400	**B***

* NBC 2015 Commentary “J” requirements

The calculated average V_s30 values from the 1D MASW soundings collected was 1266m/s +/-15% to 20%.

The V_s30 values calculated for the minimum and the maximum envelopes ranged from 1121 to 1400m/s.

Based on the average V_s30 values (as determined through the MASW method) and table 4.1.8.4.A of the National Building Code of Canada, 2015 Edition, the investigated area is site class “C” ($360< V_S30 \leq 760$ m/s).

At the request of the client, the V_s30 values have also been re-calculated taking in to consideration of the overburden. The building will be built directly on competent bedrock. The application of these recalculated Vs30* value is discussed below and the validity of these assumptions is at the discretion of the design engineer. The recalculated Vs30* values are presented in Table 2.

Table 2: Re-calculated V_s30 values (m/s) from the MASW data (2 to 32m)

Sounding	Minimum	Average	Maximum	Site Class
1	1627	**1784**	1966	**A***

* NBC 2015 Commentary “J” requirements

Based on the average V_s30* values (as determined through the MASW method), taking into consideration the proposed excavation depth as provided by the client, and table 4.1.8.4.A of the National Building Code of Canada, 2010 Edition, the investigated area is site class “A” ($V_S30 > 1500$ m/s). This assumes that the building will be founded directly on the competent bedrock and that the rock is of consistent or better quality at depth.

The use of site class “A” is conditional on the requirements of Commentary “J” sentence 100, specifically, *“Site Classes A and B, are not to be used if there is more than 3 m of soil between the rock surface and the bottom of the spread footing or mat foundation, even if the computed average shear wave velocity is greater than 760m/s*”.

It must be noted that the site classification provided in this report is based solely on the V_s30 value as derived from the MASW method and that it can be superseded by other geotechnical information. This geotechnical information includes, but is not limited to, the presence of sensitive and/or liquefiable soils, more than 3m of soft clays, high moisture content, etc. The reader is referred to section 4.1.8.4 of the National Building Code of Canada, 2015 Edition for more information on the requirements for site classification.

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