

May 31, 2021 Reference No. 11227097-A1

Mr. Russell Beach Senior Development Manager Medusa LP Kirkland, Quebec H9H 4M7 By Email russell.beach@broccolini.com

Re: Geotechnical Investigation – Preliminary Summary of Design Recommendations Proposed Sortation Facility Intersection of Leikin Drive and Merivale Road, Nepean, Ontario

Dear Sir,

This letter presents the preliminary results of the geotechnical investigation carried out for a proposed sortation facility development at the property located just east of the Leikin Drive and Merivale Road intersection in Nepean, Ontario, hereinafter referred to as the "Site" or "Property".

A Site location Plan is provided as Drawing No. 11227097-A1-1 attached to this letter. The purpose of this letter is to provide a summary of design recommendations for building foundations and floor slabs for the proposed development.

It is understood that the proposed structure will be a single-storey building, approximately 270,000 ft<sup>2</sup>, surrounded by asphaltic concrete and rigid pavements. Currently, a final finished floor elevation of 91.8 meters (m) has been established by the client for the development.

Based on stratigraphic information provided within geotechnical investigative report previously completed by Fondex in 1991, for a site directly adjacent to the current investigated property, the anticipated subsoil conditions consist in a stiff to firm silty clay/clayey silt deposit overlying and a loose to compact silty sand to sandy silt deposit and ultimately bedrock.

Based on a review of the available Fondex report, as well as the details with respect to the proposed development, a scope of work for the investigation was developed and is presented below.

# 1. Scope of Work

Based on the available geotechnical information in relation to the current development plan, the proposed site grading, and considering the potential foundation and the slab design options, the following scope of work was established for the current geotechnical investigation:





- Advancement of nine boreholes across the investigated site. Six of these boreholes were advanced within the proposed building footprints (identified as BH-01 to BH-6). The three remaining boreholes were advanced within surrounding pavement and stormwater pond areas (identified as BH-11 to BH-13). The majority of the boreholes (BH-01, BH-04 to BH-06 and BH-11 to BH-13) were sampled to depths varying between 6.1 m and 10.5 m BGS and completed with a dynamic penetration test to penetration refusal. Borehole Nos. BH-02, BH-03, both located within the proposed building footprint, were drilled within bedrock.
- Completion of cone penetration test (CPT) at 11 locations (identified as CPT-01 to CPT-08, CPT-14, CPT-16 and CPT-18) to a maximum depth of 22 m BGS to assess subsurface soil conditions at each location. The CPT is an in-situ instrumentation/tool to assess soil stratigraphy and soil strength parameters in a continuous fashion.
- Collection of soil samples for the geotechnical laboratory testing and carrying out in situ testing
  including Standard Penetration Test (SPT) and vane tests to estimate initial undrained shear strength
  of the underlying cohesive deposit.
- Excavation of 32 test pits across the Site to depths varying between 1.5 and 3.4 m BGS to assess topsoil layer thickness, reworked native thickness and condition of the existing fill materials to determine stripping requirements.

The borehole, CPT and test pit location plan is provided as Drawing No. 11227097-A1-1A in the Attachment A. The corresponding borehole and test pit reports are also provided in Attachment B of this letter.

# 2. Summary of Field Work

Prior to initiating the subsurface investigation activities, all applicable utility companies (gas, bell, network cables, pipeline and municipal sewers, etc.) were contacted through Ontario One-Call. In addition, a private utility locator (USL-1) was retained to demarcate the location of the privately owned utilities within the area of the boreholes, CPT's and test pits to ensure that the private utilities are not damaged during the investigation work.

# 2.1 Site Staking

The proposed boreholes, CPT's and test pit soundings were staked and surveyed by GHD's technical representative using a portable Leica GPS unit with a 3 cm precision. The MTM-08 NAD 83 coordinates are presented on each separate borehole and test pit report. The elevations are also illustrated on Drawing N° 11227097-A1-1A presented in Attachment A.

# 2.2 Borehole and Test Pit Activity

The drilling program for this geotechnical investigation was carried out between April 19<sup>th</sup> 2021 and April 30<sup>th</sup>, 2021 and consisted of advancing a total of nine boreholes to depths ranging between 5.94 and



23.62 m BGS. Six of the boreholes were located within the proposed building footprint (identified as BH-01 to BH-06). The remaining three boreholes were advanced in proposed pavement areas and the stormwater management pond (identified as BH-11 to BH-13). All nine boreholes were instrumented with 20 mm diameter open standpipes upon completion. In addition, two boreholes were also instrumented with two additional Casagrande piezometers each at specific depth.

The drilling work was carried out using a track-mounted drill rig (CMC-55) under the full-time supervision of GHD technical representative. The boreholes were advanced using hollow stem augers or casing methodology to balance any potential hydraulic uplift pressures. Soil samples were collected every 0.75 m intervals to the confirmation of the firm silty clay deposit and 1.5 m intervals thereafter to the termination depth of the borehole within boreholes drilled to bedrock. All samplings were conducted using a 50 mm outside diameter split-spoon sampler in general accordance with the specifications of the Standard Penetration Test Method (ASTM D1587-8). In addition, at each borehole location the relative density or consistency of the subsurface soil layers were measured using the Standard Penetration Test (SPT) method, by counting the number of blows ('N') required to drive a conventional split-barrel soil sampler 0.3 m depth. Soil samples were retrieved from each borehole location to verify strata boundaries and soil properties.

In situ vane shear testing was carried out where firm to very stiff cohesive soil deposit encountered. Additionally, relatively undisturbed soil samples were collected from these softer deposits using Thin-Walled Shelby Tube (ASTM D1587) for detailed laboratory testing.

In addition to the borehole drilling activities, a total of 32 test pits were excavated between May 3<sup>rd</sup>, 2021 and May 7<sup>th</sup>, 2021 to depths varying between 1.5 and 3.4 m BGS to assess topsoil layer thickness, reworked native thickness, condition of existing fill materials to determine stripping requirements as well as to assess the condition of the underlying native soils. The test pits were advanced by means of a mechanical Shovel.

The GHD technical representative logged the material encountered in the boreholes and test pits and examined the samples as they were obtained. The recovered samples were sealed in clean, airtight bags and transferred to the GHD laboratory, where they were reviewed by a geotechnical engineer. The detailed results of the individual boreholes and test pits are recorded on the accompanying borehole and test pit logs presented in Attachment B.

Groundwater level observations were completed in the boreholes as drilling proceeded and upon completion of drilling. In order to measure the more stabilized ground water table in the area, a 20 mm O.D. open standpipe was installed in all boreholes at an appropriate horizon in consideration of proposed infiltration features and construction excavation depths.

In addition, two Casagrande piezometers were installed within Borehole Nos. BH-02 and BH-03 in order to assess the potential of any hydraulic gradient within the underlying cohesive deposits.

Details of the open standpipe and piezometer construction is presented on the attached borehole logs in Attachment B.



# 2.3 Cone Penetration Testing (CPT's)

In order to assess the engineering properties of the native silty clay/clayey silt and silt/sand deposits, cone penetration tests (CPT's) were conducted at 11 locations to a maximum depth of 22 m BGS). The CPT is considered an effective in-situ instrumentation/tool to assess soil stratigraphy and soil strength parameters. The CPT consists of hydraulically pushing a cone with a cross-sectional area of 1,000 mm² into the ground at a rate of approximately 20 mm/sec. The pressure on the cone tip (Qc), the friction stress on a sleeve located immediately behind the cone tip (fs), and the water pressure in a pore pressure transducer (U2) are continuously recorded electronically during advancement of the CPT to indicate the subsurface conditions and soil strength properties.

#### 2.4 Seismic Site Class Investigation

To determine seismic site class for the Site in accordance with the 2012 Ontario Building Code (2012 OBC), measurements of shear wave velocity was completed within CPT Nos. CPT-03 and CPT-04 to penetration refusal.

The seismic site class is determined by measuring the average shear wave velocity along the CPT's. The shear waves are produced by providing a vibration on the ground surface and then measuring the response at the CPT probe depth.

# 3. Subsurface Conditions

The conditions observed in the boreholes advanced for this investigation at the Site are generally consistent with the previous investigation conducted by Fondex for the adjacent site. Details of the subsurface conditions are provided on the borehole and test pit logs presented in Attachment B. A brief description of each soil stratum is summarized below:

#### 3.1 Topsoil / Reworked Native

A surficial layer of reworked native and topsoil with rootlets and organics was encountered at the ground surface the majority test pits and boreholes advanced at the Site. The thickness of this layer was measured and found to vary between 50 mm and 600 mm during the current investigation.

It should be noted that the thickness of topsoil may vary between borehole and test pit locations. Classification of this material was based solely on visual and textural evidence; testing of organic content or other constituents was not carried out as it was not part of this scope of work.

#### 3.2 Earth Fill / Reworked Native

At the location of apparent soil mounds, a layer of earth fill consisting predominantly of a silt and clay matrix with rootlets and organic inclusions locally was encountered at borehole N<sup>os.</sup> BH-11 and BH-12 as well as test pit N<sup>os.</sup> TP-43, TP-49 and TP-52. The layer was found to extend to a depth of 0.6 to 5.4 m at the sounding locations.



# 3.3 Native Silty Clay / Clayey Silt Deposits

Immediately below the fill, top soil or disturbed materials, the native soil consisting in a cohesive silty clay to clayey silt deposit was encountered. The deposit was initially brown corresponding to a silty clay crust while becoming brownish grey to grey roughly 0.7 to 1.5 m BGS. The upper portion of the silty clay to clayey silt deposit is generally stiff to very stiff with undrained shear strength values more than 50 kPa down to an approximate depth of 2 to 4 m BGS. Beyond this depth the silty clay to clayey silt deposit can be characterized as firm to stiff with undrained shear strength values generally varying between 35 and 60 kPa down to the bottom of the deposit which was encountered 10 and 16 m BGS within both deep boreholes and 9 to 14 m BGS within the 11 CPT's completed across the site.

#### 3.4 Silty Sand/Sandy Silt Deposit

Directly below the aforementioned cohesive deposit, a silty sand to sandy silt deposit was encountered within both deep borehole N<sup>os.</sup> BH-02 and BH-03 at respective depths of 10 and 16 m BGS and within the 11 CPT's at interpreted depths of varying between 9 to 14 m BGS.

Based on SPT values recorded during the borehole drilling operations generally ranging between 1 and 6, the deposit can be characterized as very loose to loose. Beyond depths of 18 and 16 m BGS within these same boreholes, slightly higher SPT values ranging between 9 and 20 were recorded, indicative of a generally compact soil matrix.

The borehole N<sup>os.</sup> BH-01, BH-04 to BH-06, BH-11 to BH-13 were terminated upon dynamic penetration refusal on a hard stratum at depths varying between 18 and 23 m BGS.

# 3.5 Bedrock

A dolomite bedrock was encountered and recovered within borehole  $N^{os.}$  BH-02 and BH-03 at respective depths of 22.1 and 20.0 m BGS.

#### 3.6 Results of CPT Testing

The Cone Penetration Testing was mainly performed to define continuous profiles of geotechnical strength parameters and those of the layers presenting a plastic behaviour (clayey layers). The results of these soundings have confirmed the borehole results and have been used for design analysis.

CPT soundings were terminated upon cone penetration refusal or deviation refusal at depths varying between 15 and 22 m BGS.

### 3.7 Groundwater Level Measurements

Groundwater observations were made in each of the boreholes as they were drilled and upon completion of drilling. The observed conditions are reflected in the logs of the drilled boreholes presented in Attachment B.



A round of water-level measurements were conducted on May 6<sup>th</sup> and May 19<sup>th</sup>, 2021 to obtain a more stabilized groundwater reading and are also presented in the borehole logs. The groundwater levels recorded in the installed open standpipes and piezometers are summarized in the following table.

It should be noted that water levels may vary seasonally or after periods of heavy precipitation.

Borehole N <sup>o.</sup>	Installation	Installation Elevation	Probe Depth	Probe Elevation	Water Depth	Water Elevation	Water Depth	Water Elevation
					6/5/2021		19/5/202	1
BH-01	OS-01	90.59	9.15	81.44	1.01	89.58	0.83	89.76
BH-02	PZ-02a	90.61	7.55	83.06	1.09	89.52	1.10	89.51
	PZ-02b		13.61	77.00	1.40	89.21	1.35	89.26
	OS-02		4.85	85.76	1.02	89.59	0.93	89.68
BH-03	PZ-03a	90.52	7.70	82.82	1.12	89.40	1.10	89.42
	PZ-03b		13.23	77.29	1.09	89.43	1.31	89.21
	OS-03		6.76	83.76	0.89	89.63	0.86	89.66
BH-04	OS-04	90.50	6.10	84.40	0.81	89.69	0.81	89.69
BH-05	OS-05	90.47	6.10	84.37	1.03	89.44	0.92	89.55
BH-06	OS-06	90.53	5.96	84.57	1.08	89.45	1.10	89.43
BH-11	OS-11	95.01	10.67	84.34	8.74	86.27	5.40	89.61
BH-12	OS-12	90.31	5.99	84.32	1.18	89.13	1.12	89.19
BH-13	OS-13	90.38	10.46	79.92	1.08	89.30	0.80	89.58

OS: Open Standpipe

PZ: Piezometer

# 4. Design Recommendations

It is understood the proposed structure will be a single storey sortation facility, approximately 270,000 ft<sup>2</sup> in size and surrounded by asphaltic concrete and rigid pavements to support truck traffic.

Currently, a final finished floor elevation of 91.8 m has been established by the client for the development. Such a finished floor elevation would involve an approximate 1.3 m site grade raise at the building footprint location.

Furthermore, a slab live load of 25 kPa was considered for foundation and slab design comments and recommendations.

Based upon the above-noted comments and the obtained information from the site investigation and assuming them to be representative of the subsurface conditions across the Site, the following summary of comments and recommendations are offered.



# 4.1 Site Preparation, Grading, and Groundwater Control

Based on the conditions encountered in the boreholes test pits, the Site is covered by a surficial topsoil/reworked native layer containing rootlets and organic layer followed by native silty clay to clayey silt and sandy silt to silty sand deposits and ultimately bedrock. The presence of fill material was noted within soil mounds along Bill Leathem and Leikin Drive. With the exception of these soil mounds, the site topography is relatively flat with registered geodetic elevations varying between 90.3 m and 90.7 m across the entire investigated property.

Site preparation for the proposed development will involve stripping operations in order to remove all topsoil/reworked native and fill layers in order to facilitate construction of the proposed building and pavement areas.

The subgrade soils exposed after the removal of the surficial topsoil and unsuitable fill and reworked native soils containing organic will consist of native silty clay / clayey silt. Prior to Site grading activity, the exposed subgrade soils should be visually inspected. Any soft, organic, or unacceptable areas should be removed as directed by the Engineer and replaced with suitable fill materials compacted to a minimum of 98 percent Standard Proctor Maximum Dry Density (SPMDD). Clean earth fill used to raise grades in the proposed building and pavement areas should be placed in thin layers (200 mm thick or less) and compacted by a heavy sheep-foot type roller to 98 percent SPMDD.

The installed piezometers and standpipes indicate that groundwater is globally located at depths ranging between 1 and 2 m below existing grades. The amount of seepage into excavations will depend on the depth of excavation relative to the groundwater level at the time of construction and the hydraulic conductivity of the excavated soils. Some test pits advanced across the Site encountered water seepage near the bottom of certain test pits completed roughly 3.0 m BGS.

Considering the observations made during test pits excavations and the nature of the deposits (silty clay to clayey silt deposit) we are of the opinion that minor to moderate groundwater ingress could occur, which can be readily be handled using the installation of sumps and pumps at strategic locations at the base of excavation.

#### 4.2 Foundations and Slabs

Considering the firm nature of the silty clay/clayey silt deposit beyond its initial stiff layer (initial 4 m) and to significant depth, the proposed site grade raise, proposed slab loading and subsequent foundation loads would render the said deposit to a normally consolidated state. As such, based on the proposed site development requirements, the site, in its current condition, in not suitable to support a normal slab on grade nor conventional foundations as such a construction would lead to uncontrollable settlements.

According to existing site conditions, four foundations and slab options present themselves for this site in order to allow for the proposed development.



#### **Option 1**

The first option would involve the building foundations and slab supported by a deep piled foundation system that could consist of steel (H-pile or tube) piles driven to the refusal upon or within the underlying bedrock. In this particular option, both the building foundations and slab would be structurally supported. As such anticipated settlement of the structure with a properly installed piled foundation system would be negligible and would be a function of the elastic compression of the support members.

#### Option 2

A second option would involve site improvement operations in order to render the site suitable for conventional foundations and slabs. Based on the site stratigraphic profile, which includes a thick firm silty clay/clayey silt deposit across the majority of the site, such site improvement operations would involve the installation of rigid inclusions in order to transfer the loads induced by a conventional construction (loads from conventional footings and slab) to the competent deeper stratum below the firm silty clay/clayey silt and loose to very loose silty sand/sandy silt deposits, in this particular case, the underlying bedrock.

Essentially, rigid inclusions are composed of concrete columns with diameters generally ranging between 300 to 450 mm and spaced approximately 1 to 2 m center to center depending on soils characteristics and loading requirements. The rigid inclusions are drilled using a hallow stem auger system and filled with concrete upon drilling to practical refusal within the underlying substratum, in this case, the underlying bedrock. To adequately distribute loads from building foundations and slabs to the rigid inclusions and subsequently the competent substratum, a transfer pad usually composed of granular A or Granular B type 2 crushed stone (generally a minimum of 600 mm thick) is placed between the top of the rigid inclusions and the bottom the foundation and slab elements.

The design and methodology for the implementation of the rigid inclusions should be completed by a specialized contractor having experience with this type of soil improvement technique. Considering that the soil improvement design (inclusion size, depth and spacing, transfer pad composition and thickness, permanent long-term water table conditions, required site grade raise, etc.) is integral to the optimum and uniform performance of the building foundation and slab on grade systems within the required soil improvement area, the soil improvement designer/contractor is responsible for their design and subsequent construction of the improved soil area.

Typically, serviceability bearing capacities in the order to 150 kPa to 225 kPa is achievable for such a design.

#### Options 3 and 4

Two additional construction options that may be considered would involve the use of either piles or rigid inclusions to support the building foundations combined with light weight fill in order to render the site suitable for a slab on grade construction. Based on the soil stratigraphy and underlying silty clay/clayey silt strength parameters combined with a 25 kPa slab live load, the site is not permissible for any grade raise.



As such, the lightweight fill would need to account for the entirely of the site grade raise as well as the required 300 mm of crushed stone immediately below the slab. See general comments below.

#### **General Comments**

Regardless of the preferred option the slab foundation should incorporate a granular base layer consisting of at least 300 mm of Granular 'A' material as per Ontario Provincial Standard Specifications (OPSS form 1010), compacted to at least 100% of the material's Standard Proctor maximum dry density (SPMDD). It is also recommended that a vapour barrier be installed to limit vapour emission through the concrete slab.

From a strict engineering point of view Options 1 and 2 would ensure optimal performance and behavior of the slab and foundation elements as both would rest on the same supporting structure. As such, both of these options present minimal risk of differential behaviour between both the building slabs and foundations.

Both remaining options have also been successfully completed on various projects. However, such designs cannot eliminate all risk of differential movement or behavior between both structures as the slab would be considered as a "floating elements" while the building foundations would be supported on rigid elements, in this case, piles or rigid inclusions. That being said, the financial benefits of such a design may outweigh such a risk and could be evaluated by the developer depending on building performance requirements.

#### 4.3 Seismic Site Classification

Based on our calculations, the average shear wave velocity (from 0.0 m bgs to 30.0 m bgs) along the three SCPT's investigation lines is approximately 230 m/s. Therefore, in accordance with Table 4.1.8.4.A of the OBC 2012 (and based on the measured average shear wave velocity, the Site can be classified as Class 'D' for the seismic load calculations subjected to code requirements. This seismic classification is applicable for all foundation and slab designs presented in the section above.

We trust the foregoing information is sufficient for your present requirements. Please do not hesitate to contact this office should any questions arise.

Sincerely,

**GHD** 

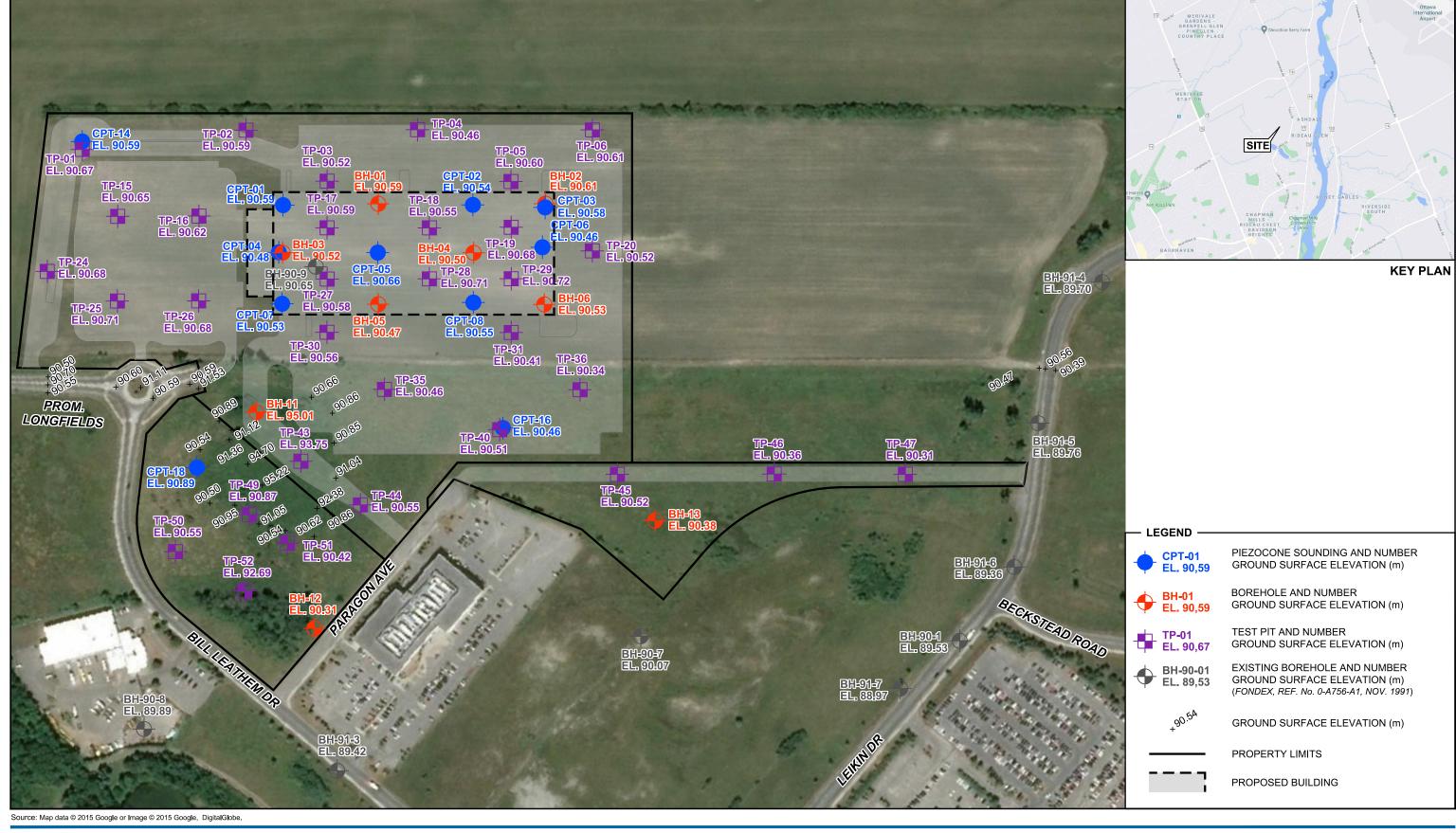
Alex Fiorilli, P.Eng.

AF/KH/sf/1

**Enclosures:** 

**Attachments** 

# Attachment A Drawing



SCALE = 1:3000 0 30 60 90m BENCHMARK: RTCM-REF 3696 (GPS Antenna) EL. 98.527m (Geodetic)

DRAWN BY:

CHECKED BY:

T. NGUYEN / I. CARON

M.-A. RICHARD



MEDUSA LP LEIKIN DRIVE AND MERIVALE ROAD INTERSECTION, NEPEAN, ONTARIO PROPOSED SORTATION FACILITY

GEOTECHNICAL INVESTIGATION SOUNDING LOCATIONS

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# Attachment B Borehole and Test Pit Logs

