Terraprobe
Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

GEOTECHNICAL INVESTIGATION
6158 ALLENDALE AVENUE &
5592 & 5566 ROBINSON STREET
NIAGARA FALLS, ONTARIO

Prepared For: **La Pue International Inc.**
6158 Allendale Avenue
Niagara Falls, Ontario
L2G 0A5

Attention: Mr. Pawel Fugiel

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TABLE OF CONTENTS

1.0 INTRODUCTION

Terraprobe Inc. was retained by La Pue International Inc. to carry out a geotechnical investigation at the Property located at 6158 Allendale Avenue & 5592 & 5566 Robinson Street in Niagara Falls, Ontario, hereafter referred to as the 'the Property'. The location of the site is shown on the Site Location Plan, Figure 1. A proposal and cost estimate to carry out the work were outlined in our letter of October 5, 2021. Authorization to proceed with the investigation was provided by Pawel Fugiel on October 5, 2021.

The purpose of the work was to investigate and report on the subsurface soil, rock, and ground water conditions in a series of boreholes drilled at the site. Based on this information, advice is provided with respect to the geotechnical aspects of the proposed development. The anticipated construction conditions pertaining to excavation, shoring support, backfill and temporary ground water control are also discussed, but only with regard to how they might influence the proposed design.

Phase One & Two Environmental Site Assessments (ESA) and a hydrogeological assessment were also carried out concurrently with the geotechnical investigation and are being reported under separate cover.

2.0 SITE AND PROJECT DESCRIPTION

2.1 Existing Site Conditions

The Property consists of a 0.43 hectare (1.1 acre) irregular shaped parcel of land situated at the southeast corner of the intersection of Allendale Avenue and Robinson Street. The Property is bound to the north by Robinson Street, to the south by a public parking lot, to the west by Allendale Avenue and to the east by a multi-tenant commercial building and associated parking lot. The Property is currently occupied by a vacant commercial/industrial building at 6158 Allendale Avenue, a commercial/residential building at 5592 Robinson Avenue, and a residential dwelling at 5566 Robinson Avenue. A gravel covered parking area and driveway is located east and south of the commercial/residential building at 5592 Robinson Avenue. An asphalt paved parking area is located south and west of the vacant commercial/industrial building at 6158 Allendale Avenue.

The site topography has mildly sloped to the east. There is a fall of about 3.5 m in elevation across the 82 m traverse of the site from the southwest to the southeast corners of the property. The high ground was at about elevation 199.9 m and the lowest at about 196.5 metres.

2.2 Site Geology

Based on published geological information for the general area, near surface soil at and in the vicinity of the subject property generally consists of glaciolacustrine deposits comprised of sand, gravelly sand and gravel.[1] Beneath the overburden deposits is bedrock of the Guelph Formation consisting of sandstone, shale, dolostone and siltstone.[2] The surface of the bedrock was encountered in five (5) boreholes at depths in the range of about 21.0 to 24.7 metres below the existing ground surface (m BGS), or between elevation 174.3 and 175.8 m.

2.3 Proposed Development

The proposed development features are shown on Figure 3, as derived from drawing A001, dated June 18 2021, prepared by Chamberlain Architect Services Limited (Project No. 121034). It is understood that the proposed redevelopment of the Property would include the demolition of the existing buildings and construction of 77-storey residential high-rise building. The structure will rest on a 6-storey podium, including five (5) above grade parking levels. The entire Site will contain five (5) levels of underground parking. The lowest finished floor elevation (FFE) for P5 was not know at the time of report preparation but is anticipated to be approximately 16 m below grade, near elevation 182 ± masl. The excavation will extend to the property limits, and as such the overburden soils will have to be supported during the excavation.

3.0 PROCEDURE

The field work for this investigation was carried out on November 15, 16, 19, 22 and 23, 2021, during which time five (5) boreholes (BH1 to BH5) were drilled to depths of about 21.0 to 29.3 m BGS. Terraprobe also advanced one (1) borehole (BH6) to a depth of about 1.6 m BGS as part of a concurrent environmental investigation at the site. The locations of the boreholes are shown on the Borehole Location Plan, Figure 2. The results of the boreholes are shown on the Log of Borehole sheets and Rock Core Logs presented in Appendix A.

The boreholes were drilled using truck-mounted B-57 drill rig supplied and operated by a specialist drilling contractor. The boreholes were advanced using conventional interval augering and sampling techniques. Soil samples were recovered by split barrel sampling in accordance with ASTM D1586. In all of the boreholes, the overburden drilling and sampling were terminated upon practical refusal to conventionally auger which was assumed to coincide with the surface of the underlying bedrock.

The bedrock was explored in Boreholes 1 and 2 by rotary core drilling with HQ sized rock coring equipment in core runs of 1.5 m lengths, and to depths of about 26.1 and 29.3 m BGS, or to elevations in the range of about 170.1 to 170.4 m. Our field staff logged and photographed the rock cores recovered from the boreholes. The Total Core Recovery (TCR), Solid Core Recovery (SCR) and Rock Quality

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1 Quaternary Geology, Niagara-Welland, Southern Ontario; Ministry of Natural Resources; Ontario Geological Survey; Map No. 2496; 1984.

2 Paleozoic Geology, Niagara, Southern Ontario; Ontario Division of Mines; Map No. 2344; 1976.

Designation (RQD) values were recorded in accordance with the conventions used by the International Society for Rock Mechanics (ISRM).

Ground water observations were made in each borehole during and upon completion of drilling and sampling. In addition, monitoring wells were installed and sealed in Boreholes 1, 3, and 4. The monitoring wells extended to depths of about 6.1 to 19.8 m BGS and were constructed of 50 mm diameter schedule 40 PVC screen and riser with a silica sand pack, and bentonite seal. The screened section of the wells were 3.0 m long. The remainder of the monitoring well sections were sealed with bentonite to the existing ground surface. A conventional 50 mm diameter J-plug was used to seal the top of the wells and a flush-mount steel well casing was installed at the ground surface and sealed with concrete. Details of the construction of the monitoring wells are presented on the attached corresponding borehole logs in Appendix A. The water levels were measured in the monitoring wells on December 3, 2021 by a member of our field staff, with subsequent readings taken on January 7, 24, 29, and February 2, 2022.

Boreholes that were not equipped with a monitoring well were decommissioned and sealed with bentonite pellets in accordance with Ontario Regulation 903 and sealed with nominally compacted commercial grade cold-mix asphalt patch at the pavement surface.

The field work was observed throughout by a member of our engineering staff who located the boreholes, arranged for the underground utility clearances at the borehole locations and cared for the samples obtained during the investigation. The borehole locations were located in the field in advance of drilling. The elevations of the boreholes on the Property were surveyed by Terraprobe using a Trimble R10 Global Navigation Satellite System (GNSS). The Trimble R10 system is a differential global positioning system (GPS) which involves the cooperation of two receivers, one that is stationary and another that is roving making position measurements.

All of the samples recovered in the course of the investigation were brought to our Stoney Creek laboratory for further examination and laboratory testing. The results of laboratory testing of the recovered samples are shown on the Log of Borehole sheets in Appendix A. Uniaxial compressive strengths were determined on four (4) sections of the cored bedrock, the results of which are shown on the corresponding Rock Core Logs in Appendix A.

4.0 SUBSURFACE CONDITIONS

The subsurface soil, rock, and ground water conditions encountered in the boreholes are presented on the attached Log of Borehole and Rock Core Log sheets in Appendix A. The stratigraphic boundaries indicated on the borehole logs are inferred from non-continuous samples and observations of drilling resistance and typically represent a transition from one soil type to another. These boundaries should not be interpreted to represent exact planes of geological change. The subsurface conditions have been confirmed in six (6) generally evenly spaced boreholes and may vary between and beyond the borehole locations. The discussion has been simplified in terms of the major soil strata for the purposes of geotechnical design. For more specific subsurface details, refer to the enclosed Log of Borehole and Rock Core Log sheets in Appendix A.

4.1 Stratigraphy

The following stratigraphy is based on the results of the borehole findings, as well as the geotechnical laboratory testing conducted on selected representative soil samples. In general, three main stratigraphic units were encountered which included earth fill overlying strata of undisturbed silts and sands, and dolostone bedrock of the Guelph Formation.

4.1.1 Existing Pavements

Boreholes 2 to 6 encountered approximately 50 to 100 mm of asphaltic concrete at the existing ground surface. The asphalt was underlain by about 100 to 300 mm of granular fill.

4.1.2 Fill

Underlying the surficial materials at each borehole location, all boreholes encountered a layer of earth fill extending to depths ranging from 1.5 to 2.3 m below existing grades (Elev. 195.0 to 197.9 masl). The earth fill was variable but generally consisted of sand and gravel to sandy silt with varying amounts of clay. Borehole BH6 was terminated within the earth fill on an inferred obstruction. Standard Penetration Testing carried out within the earth fill determined N values generally in the range of 4 to greater than 50 blows per 0.3 m, indicating a very loose to very dense state of packing. The higher N values were likely due to obstructions encountered while driving the split spoon. The in-situ water content of the samples of fill recovered from the boreholes ranged from about 7 to 42 percent.

4.1.3 Silt Till

Underlying the fill materials, boreholes BH2 and BH4 encountered a stratum of silt till with some clay, trace sand, and gravel, extending to depths ranging from 7.6 to 9.1 m below existing grade (Elev. 188.9 to 190.8 masl). The N values determined in the silt till stratum ranged from 16 to greater than 50 blows per 0.3 m, indicating a compact to dense relative density. The in-situ water content of the samples of silt till recovered from the boreholes ranged from about 9 to 24 percent.

4.1.4 Silts and Sands

All boreholes encountered a stratum of silts and sands, with varying amounts of clay to depths ranging from 18.9 to 24.2 m below existing grade (Elev. 175.0 to 181.0 masl). Standard Penetration Testing Carried out within the silts and sands determined N values generally in the range of 10 to greater than 50 blows per 0.3 m, with an average N value of 49 blows per 0.3 m, indicating a dense relative density. The in-situ water content of the samples of silts and sands recovered from the boreholes ranged from about 5 to 25 percent.

4.1.5 Sand and Gravel

Boreholes BH1 to BH3 encountered a stratum of sand and gravel below the silts and sands, extending to depths ranging from 21.6 to 24.7 m below existing grade (Elev. 174.7 to 175.0 masl). Standard Penetration Testing Carried out within the sand and gravel determined N values generally in the range of greater than 50 blows per 0.3 m, indicating a very dense relative density. The in-situ water content of the samples of sand and gravel recovered from the boreholes ranged from about 8 to 15 percent.

4.1.6 Bedrock (Guelph Formation)

The augering and interval sampling method that was used for subsurface exploration on this project is conventionally accepted investigative practice. However, this method does not define the bedrock surface with precision, particularly in this instance where the surficial zone of the bedrock may be weathered. As best could be practically determined, the surface of the bedrock was encountered in boreholes 1 to 5 at depths of about 21.0 to 24.7 metres below the existing ground surface (m BGS), or between elevation 174.3 and 175.8 m. The following table shows those boreholes that were advanced to the bedrock or auger refusal interpreted as the top of the bedrock surface.

Top of Bedrock Depth/Elevation					
Borehole #	**1**	**2**	**3**	**4**	**5**
Depth (m)	24.7	21.6	21.8	25.6	21.0
Elevation (m)	174.7	174.9	175.0	174.3	175.8

The bedrock was continuously cored in boreholes BH1 and BH2, and to depths of about 26.1 to 29.3 m BGS, or to elevations of 170.1 to 170.4 m. The bedrock consisted of light brown to grey dolostone of the Guelph Formation. The cored rock was observed to be medium to thickly bedded, vuggy with calcite and/or gypsum, and with fine crystalline. The TCR (total core recovery) ranged from 83 to 125 percent. The SCR (solid core recovery) ranged from 67 to 118 percent. The RQD (rock quality designation) value for the bedrock ranged from 48 to 86 percent, generally indicating a poor to good quality. The discontinuities observed in the rock cores were typically horizontal and generally associated with the bedding planes. The joints were filled with layers of bituminous chert and the joint spacing was generally close. Uniaxial compressive strengths in the range of 87 to 131 MPa (average strength 105 MPa) were determined on four (4) specimens of the rock cores.

4.2 Ground Water

Unstabilized ground water level observations were made in the open boreholes during and after drilling, as noted on the borehole logs. Monitoring wells were installed in three of the boreholes. The water levels measured within the installed wells are summarized below and are shown on the corresponding log of borehole sheet in Appendix A.

BH	Elevation of Well Screen (m)	Stratum Captured by Well Screen	Depth (m) / Elevation of Water Level in Well (m)					
			During Drilling	Dec 3, 2021	Jan 7, 2022	Jan 24, 2022	Jan 29, 2022	Feb 2, 2022
1	182.7-179.6	Sand & Silt	Dry	Dry	Dry	Dry	*	*
3	193.8-190.7	Sandy Silt	Dry	Dry	Dry	Dry	*	*
4	190.2-187.1	Silty Sand	Dry	11.4/188.5	11.3/188.6	12.5/187.4	Dry	Dry

*-Monitoring well not accessible due to significant snow cover

Note:
BH4 was purged dry on January 24th, 2022, to allow for fresh formation water to flow into the well. As of February 2nd, the well had not recovered and was dry. Terraprobe will continue to monitor BH4 in order to complete additional ground water sampling events for the Phase II ESA.

These conditions may not necessarily represent stabilized conditions. Fluctuation in the ground water levels will also occur due to seasonal variations and precipitation conditions. Longer term ground water monito ring should be undertaken in order to accurately determine the elevation of the stabilized water table at the site.

5.0 GEOTECHNICAL DESIGN

The following discussion is based on our interpretation of the factual data obtained during this investigation and is intended for the use of the design engineer only. Comments made regarding the construction aspects are provided only in as much as they may impact on design considerations. Contractors bidding on or undertaking any work at the site should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, equipment capabilities, costs, sequencing, and the like.

This report is based on the assumption that the design features relevant to the geotechnical analyses will be in accordance with applicable codes, standards and guidelines of practice. If there are any changes to the site development features, or there is any additional information relevant to the interpretations made of the subsurface information with respect to the geotechnical analyses or other recommendations, then Terraprobe should be retained to review the implications of these changes with respect to the contents of this report.

The proposed development features are shown on Figure 3, as derived from drawing A001, prepared by Chamberlain Architect Services Limited. It is understood that the proposed redevelopment of the Property would include demolition of the existing buildings and construction of a 77-storey residential condominium building. The structure will rest on a six (6) storey podium, including five (5) above grade parking levels, and the entire site will contain five (5) levels of underground parking. The excavation will extend to the property limits, and as such the overburden soils will have to be supported during the excavation.

5.1 Site Preparation

The site is underlain by shallow fill deposits that are not considered competent to support foundations, floor slabs on grade, or any settlement sensitive structures or underground plant. The nature of the site pre-grading will depend on the foundation design alternative selected as well as the scope of any work that may be required to address environmental issues.

Based on the current proposed design, preparation of the site will generally consist of the removal of existing asphalt pavements as well as any unsuitable fill and cutting to achieve the design subgrade elevations. While specifically not encountered in the boreholes, it is possible that building foundations from former structures at the site could still exist within the overburden soils.

Some improvement of the condition of the near surface soil for the support of underground services and flexible pavements may be feasible. Such work would typically consist of the removal of any existing foundations, abandoned underground plant, as well as a portion of the fill, compacting the remaining fill to the extent possible from surface and restoring the site with engineered fill. The extent of fill that should be removed in these areas should be addressed by the geotechnical engineer at the time of construction.

5.2 Foundation Design Parameters

A building with five (5) levels of underground parking will penetrate the overburden silty sand stratum. The following discussion is provided with the understanding that any and all buildings proposed for the site will be designed in conformance to the current Ontario Building Code (OBC) or other regulatory bodies within the jurisdiction. This section addresses the feasibility of constructing conventional spread and/or strip footings, as well as drilled piers (caissons) at the site.

5.2.1 Conventional Spread Footings

The proposed five (5) levels of below grade structure for this project will result in the lowest basement floor at a level 16 m BGS or at about elevation 182 m. It is expected the foundations for the structure would be established on the dense to very dense silty sand.

Spread footing foundations made on the undisturbed dense to very dense silty sand can be designed using a factored bearing resistance at Ultimate Limit States (ULS) of 900 kPa and a bearing reaction of 600 kPa at Serviceability Limit States (SLS), for an estimated total settlement of 25 mm. These recommendations are applicable to footings that are minimum 1.2 m wide (or 1.0 m wide for strip footings). These minimum dimensions apply regardless of loading considerations.

5.2.2 Drilled Piers (Caissons)

Depending on the final design grade, consideration may be given to the use of end-bearing caissons (drilled piers) for the structures. Drilled piers established to bear in the dense to very dense sandy silt to silty sand stratum at or below about elevation 182 m could be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 1200 kPa and a bearing resistance at Serviceability Limit States (SLS) of 800 kPa, for up to 25 mm of settlement, provided the length to diameter ratio is at least 3. Drilled shafts with a length to diameter ratio of less than 3 should be regarded as drilled footings, the design bearing resistance reduced in accordance with the parameters presented in Section 5.2.1 of this report.

It is likely that load testing (ASTM D1143 - Standard Test Methods for Deep Foundations Under Static Axial Compressive Load) will improve the bearing capacity. The factored geotechnical resistance at ULS provided above uses a factor of 0.4 as per the Canadian Foundation Engineering Manual and the Ontario Building Code. When the static load test is performed the reduction factor will be 0.6, which will most likely provide a higher factored geotechnical resistance at ULS. The load test will also provide settlement parameters. It is anticipated that the geotechnical reaction at SLS will be similar to the ULS value. If a load test is not conducted, we recommend that additional boreholes be advanced within the tower footprint for caisson design.

Caissons could also be established to bear in the bedrock and could be designed using a factored bearing resistance at Ultimate Limit States (ULS) of 8 MPa. In Limit States Design, the factored bearing resistance at ULS will govern the design since bedrock is non-yielding and the loading to produce 25 mm of axial deformation is greater than the factored resistance at ULS.

The caissons are to be end bearing units and will require base inspection and hand cleaning of the base prior to concrete placement. End-bearing caissons should have a minimum diameter of 915 mm regardless of loading considerations to facilitate foundation subgrade inspection and hand cleaning of the base. The current minimum caisson diameter requirements for entry are 760 mm. However, in our experience, deep foundation contractors request a minimum 915 mm caisson diameter to permit entry. Caisson foundations at different elevations must be designed such that the higher caissons are set below a line drawn up at 10 horizontal to 7 vertical from the closest edge of the lower caisson.

There are boulders and cobbles within the deposits above the bedrock which will be encountered during the construction of drilled piers. There could be delays relating to these obstructions and therefore the contract documents for this work must address the allocation of risk between the owner and contractor for such delays. It is suggested that unit prices for time lost to obstructions be established in conjunction with contract development and Terraprobe can provide an independent assessment of delays, in conjunction with the foundation installation inspection activities.

Excavation and installation of the caissons must conform to all applicable sections of the Occupational Health and Safety Act. The caisson contract must stipulate that the contractor will be responsible for the provision of all necessary equipment (including steel liner of adequate strength) and monitoring devices (as needed) for a safe access of the inspection and base cleaning personnel into the caissons, in accordance with the Occupational Health and Safety Act requirements.

5.2.2.1 End-Bearing Reduced Capacity Caissons

In the event that the contractor or design team wishes to avoid base inspection of end-bearing caissons, the following construction methodology may be utilized (bearing in mind that reduced bearing capacities apply, see below):

1. All caisson excavations are to be inspected on a full-time basis by Terraprobe as per the OBC;
2. Caissons are to be initially advanced to the top of bearing stratum as identified in the geotechnical boreholes and as confirmed by Terraprobe through observation of the auger cuttings at each location;
3. Visual inspection of the augered hole is to take place to ensure that auger cleaning has been carried out as thoroughly as practically possible;
4. Place 2 cu-m minimum of 25 MPa concrete in the base of the hole to be stirred with the auger without advancing the auger any further; and
5. Complete placing of concrete to cut off elevation.

With the above methodology in place, caissons established to bear in the dense to very dense sandy silt to silty sand stratum can be designed without base inspection and without hand cleaning using a maximum factored geotechnical resistance at ULS of 900 kPa and a net geotechnical reaction at SLS of 600 kPa for up to 25 mm of total settlement. Similarly, caissons established to bear on the bedrock without base inspection and without hand cleaning can be designed using a maximum factored geotechnical resistance at ULS of 6 MPa.

5.2.2.2 Lateral Resistance of Caisson Foundations

The design of lateral resistance of caisson foundations can be carried out in accordance with the following documents and papers.

- Canadian Highway Bridge Design Code and Commentary (2000). CAN/CSA-S6-00 and S6.1-00.
- Ministry of Transportation, Ontario (2007) "Sign Support Manual", Bridge Office, Engineering Standards Branch.
- Broms, B.B.: Design of Laterally Loaded Piles, Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 91. Paper No. SM3, May 1965.

The recommended soil parameters for the lateral resistance design of augered caisson foundation units are given in the table below.

Type of Soil	Consistency or Compactness Condition	c_u (kPa)	ϕ (deg.)	Soil Density Coefficient, n_h (kN/m^3)	γ (kN/m^3)	Ground Water Elevation
Interbedded Sands and Silts	Dense to Very Dense (cohesionless)	-	32	4,400	20	182 m

The notations used in the table are defined as follows:

c_u = undrained shear strength (kPa) for cohesive soils (estimated based on judgement and correlations with SPT "N" values),
ϕ = apparent angle of friction for cohesionless soils (degrees),
n_h = coefficient related to soil density (kN/m^3), and
γ = bulk unit weight of soil in (kN/m^3).

The coefficient of horizontal subgrade reaction in cohesionless soils can be estimated as follows:

$$k_s = n_h z/d$$

where
k_s = coefficient of horizontal sub-grade reaction (kN/m^3)
n_h = coefficient related to soil density (kN/m^3)
z = depth (m)
d = pile diameter (m)

In order to take into account frost action and surficial disturbance, the ultimate lateral passive resistance in front of a caisson and caisson sidewall adhesion within the upper 1.2 m below final grade should be neglected in the foundation design. It is also recommended that all surficial weak or variable soils, as well as existing earth fill be neglected in determining lateral resistance. The required depth of the drilled shaft will be governed by lateral loads, including wind loads. Appropriate load and resistance factors should be applied for caisson design.

5.3 Site Classification for Seismic Site Response

Under Ontario Regulation 88/19, the ministry amended Ontario's Building Code (O. Reg 332/12) to further harmonize Ontario's Building Code with the 2015 National Codes. These changes are intended to help reduce red tape for businesses and remove barriers to interprovincial trade throughout the country. The amendments are based on code change proposals the ministry consulted in 2016 and 2017. The majority of the amendments came into effect on January 1, 2020, which includes structural sufficiency of buildings to withstand external forces and improve resilience.

Seismic hazard is defined in the 2012 Ontario Building Code (OBC 2012) by uniform hazard spectra (UHS) at spectral coordinates of 0.2 s, 0.5 s, 1.0 s and 2.0 s and a probability of exceedance of 2% in 50 years. The OBC method uses a site classification system defined by the average soil/bedrock properties (e.g., shear wave velocity (vs), Standard Penetration Test (SPT) resistance, and undrained shear strength (su)) in the top 30 meters of the site stratigraphy below the foundation level, as set out in Table 4.1.8.4A of the Ontario Building Code (2012). There are 6 site classes from A to F, decreasing in ground stiffness from A, hard rock, to E, soft soil; with site class F used to denote problematic soils (e.g., sites underlain by thick peat deposits and/or liquefiable soils). The site class is then used to obtain peak ground acceleration (PGA), peak ground velocity (PGV) site coefficients Fa and Fv, respectively, used to modify the UHS to account for the effects of site-specific soil conditions.

Based on the above noted information, it is recommended that the site designation for seismic analysis be 'Site Class C', as per Table 4.1.8.4.A of the Ontario Building Code (2012). Consideration may be given to conducting a site-specific Multichannel Analysis of Surface Waves (MASW) at this site to determine the average shear wave velocity in the top 30 metres of the site stratigraphy. An improved seismic site designation (Site Class B) may be possible for a structure founded on caissons supported entirely on bedrock.

The values of the site coefficient for design spectral acceleration at period T, F(T), and of similar coefficients F(PGA) and F(PGV) shall conform to Tables 4.1.8.4.B. to 4.1.8.4.I of the OBC 2012, as amended January 1, 2020, using linear interpolation for intermediate values of PGA.

5.4 Earth Pressure Design Parameters

The parameters used in the determination of earth pressures acting on retaining walls are defined below.

Parameter	Definition	Units
φ	internal angle of friction	degrees
γ	bulk unit weight of soil	kN / m^3
K_a	active earth pressure coefficient (Rankin)	dimensionless
K_o	at-rest earth pressure coefficient (Rankin)	dimensionless
K_p	passive earth pressure coefficient (Rankin)	dimensionless

The appropriate values for use in the design of structures subject to unbalanced earth pressures at this site are tabulated as follows:

Stratum/Parameter	ϕ	γ	Ka	Ko	Kp
Compact Granular Fill Granular 'B' (OPSS 1010)	32	21.0	0.31	0.47	3.25
Sandy Silt to Silty Sand or Similar Fill	30	19.0	0.33	0.50	3.00

Walls subject to unbalanced earth pressures must be designed to resist a pressure that can be calculated based on the following equation:

$$\mathbf{P = K\,[\gamma\,(h-h_w) + \gamma' h_w + q] + \gamma_w h_w}$$

where,

$\mathbf{P}$ = the horizontal pressure at depth, $\mathbf{h}$ (m)
$\mathbf{K}$ = the earth pressure coefficient,
$\mathbf{h_w}$ = the depth below the ground water level (m)
$\boldsymbol{\gamma}$ = the bulk unit weight of soil, (kN/m^3)
$\boldsymbol{\gamma'}$ = the submerged unit weight of the exterior soil, (γ - 9.8 kN/m^3)
$\mathbf{q}$ = the complete surcharge loading (kPa)

Where the wall backfill can be drained effectively to eliminate hydrostatic pressures on the wall, acting in conjunction with the earth pressure, this equation can be simplified to:

$$\mathbf{P = K[\gamma h + q]}$$

The factored geotechnical resistance to sliding of earth retaining structures is developed by friction between the base of the footing and the soil. This friction **(R)** depends on the normal load on the soil contact (**N**) and the frictional resistance of the soil **(tan ϕ)** expressed as: $\mathbf{R = N \tan \phi}$. This is an unfactored resistance. The factored resistance at ULS is $\mathbf{R_f = 0.8\, N \tan \phi}$. The K value to be used for the design will depend on the rigidity of the wall.

5.5 Slab on Grade Design Parameters

It is expected that the elevation of the finished floors of the underground parking structure will be within the dense to very dense silty sand stratum. The modulus of subgrade reaction appropriate for slab on grade design in the sandy silt to silty sand is 40,000 kPa/m.

It is understood that the underground levels will be used primarily for parking, storage and for mechanical/electrical plant. On this basis it is anticipated that moisture sensitive floor coverings are not proposed for this level, and it may not be necessary to incorporate a vapour barrier into the design of the floor slabs on grade. If moisture sensitive floor finishes are proposed, a capillary moisture barrier and drainage layer will be required beneath the slab. If a polyethylene barrier is selected for this application, care must be taken to ensure that the barrier is not damaged during concrete placement. The polyethylene barrier should be not less than 10 mill and must be supported by a layer of well-graded granular material to provide uniform support for the slab. It should be recognized that the use of a polyethylene barrier could have a negative impact on the quality of slab finish (i.e., curling).

Any buildings with below grade space must have both perimeter and subfloor drainage. The subfloor drainage system is made by placing the slab on a minimum 200 mm layer of 19 mm stone (OPSS 1004) compacted by vibration to a dense state. The nominal spacing of subdrains, in the order of 8 to 10 metres is expected to provide an adequate avenue for the removal of water beneath the slab. Basement drainage is required as discussed in the following Section 5.6.

All slabs on grade should be structurally separate from foundation walls and columns. Saw cut control joints should be incorporated into the slabs along column lines and at regular intervals. Interior load bearing walls should not be founded on the slab but on spread footings as outlined above.

5.6 Basement Drainage

Foundation walls must be damp-proofed in conformance to Section 5.8.2 of the Ontario Building Code (2012). Prefabricated drainage composites, such as Miradrain 2000 (Mirafi) or Terradrain 200 (Terrafix), should be incorporated between the shoring wall and the cast-in-place concrete foundation wall to make a drained cavity. Drainage from the cavity must be collected at the base of the wall in non-perforated pipes and conveyed directly to the sumps. The flow to the building storm water sump from the subsurface drainage will be governed largely by the building perimeter drainage collection during rainfall and runoff events. Typical shored excavation drainage, and sub-floor drainage details are provided in Appendix B.

The drainage system is a critical structural element since it keeps water pressure from acting on the floor slabs on grade or on the foundation walls. As such, the sump that ensures the performance of this system must have a duplexed pump arrangement for 100% pumping redundancy and these pumps must be on emergency power. The size of the pump should be adequate to accommodate the anticipated ground water and storm event flows. It is expected that the seepage can be controlled with typical widely available, commercial sump pumps.

To assist in maintaining floor slabs and preventing seepage, it is recommended that exterior grades around the building be sloped away at a 2 percent gradient or more, for a distance of at least 1.2 m. Provision of nominal subfloor drainage is required in conjunction with the perimeter drainage of the structure, to collect and remove the water that infiltrates at the building perimeter and under the floor. Perimeter and subfloor drainage are recommended for this development.

It is recommended that the subfloor drainage system consists of minimum 100 mm diameter perforated pipes spaced at a maximum of 9 metres on centre. The pipes must be surrounded on all sides by a minimum of 100 mm of 19 mm clear stone (OPSS 1004), and the pipe inverts should be a minimum 300 mm below the base of the slab. The elevator pits can be drained separately with an independent lower pumping sump or can be designed as waterproof structures which are below the drainage level.

If the municipality does not allow for long-term discharge of ground water to the municipal storm sewer system, the underground parking levels will have to be constructed as a waterproofed structure.

5.7 Site Servicing

It is expected that site services will consist of storm and sanitary sewers and watermains, with relatively shallow inverts (less than 3 m). The invert elevation is expected to be within the silts and sands, and silt till. Excavations for underground services should be made as outlined in Section 6.1 of this report. The locations and depths of any building foundations which would potentially be affected by the proposed utilities should be identified prior to commencing the excavation.

5.7.1 Bedding

Bedding for the pipes should consist of well graded free draining granular material such as Granular A, which is compatible with the size and type of pipe. All bedding material should be uniformly compacted to at least 95 percent of standard Proctor maximum dry density.

It is possible that the excavation for some services would terminate in the fill. In this event, it will be necessary to sub-excavate all fill and replace it with engineered fill to ensure the service is properly supported and to minimize the potential of settlement. Engineered fill should consist of OPSS 1010 Granular A or Granular B Type II material placed and uniformly compacted to 98 percent of standard Proctor maximum dry density. Consideration could also be given to the use of lean concrete to restore the grade to the proposed invert elevation.

5.7.2 Backfill

Based on the results of the boreholes, it is assumed that the majority of excavated soil at the site from the construction of service trenches will consist of existing fill. Any fill containing topsoil and/or soil containing high amounts of organic material or debris should not be used or re-used for service trench backfill.

Service trench backfill should consist of clean earth, free of excessively wet or frozen soil and should be placed in lifts of 300 mm thickness or less and uniformly compacted to at least 95 percent of standard Proctor maximum dry density at placement water contents within 2 percent of the corresponding laboratory optimum water content for compaction. The upper 1 m of the backfill forming any pavement subgrade should be uniformly compacted to 98 percent of standard Proctor maximum dry density.

It may be difficult to consistently achieve the degree of compaction specified above using existing fills on site or the native excavated soil as trench backfill, particularly in narrow trenches. For this reason, consideration could be given to using free draining granular material, such as Granular A or Granular B Type I (OPSS 1010) as trench backfill to allow for adequate, uniform compaction.

5.8 Pavement Design

5.8.1 Subgrade Preparation

Topsoil and deleterious fill should be stripped from all areas to be developed for new pavements. It is recommended that the subgrade be cut as cleanly as possible to minimize disturbance and be proof rolled with a static roller to identify any loose or disturbed areas. The preparation of the subgrade and the compaction of all fills should be monitored by the geotechnical engineer at the time of construction.

If fill is required to raise the grade, there may by some select on-site fill which could be used, provided it is free of topsoil and other deleterious material and is at suitable placement water contents. The fill should be placed in large areas where it can be uniformly compacted in 300 mm thick lifts with each lift uniformly compacted to at least 95 percent of standard Proctor maximum dry density. The upper 1 m of backfill beneath areas to be developed as pavements should be compacted to 98 percent of standard Proctor maximum dry density.

Control of surface water is a significant factor in achieving good pavement life. Grading of adjacent pavement areas must be designed so that water is not allowed to pond adjacent to the outside edges of the pavement or curb. The existing earth fill and native soils have anywhere from a slight to severe frost susceptibility to frost heave, and pavement on these materials must be designed accordingly. The subgrade must be free of depressions and sloped (preferably at a minimum grade of two percent) to provide effective drainage toward subgrade drains.

Continuous pavement subdrains should be provided along both sides of the driveway/access routes and drained into catch-basins to facilitate drainage of the subgrade and the granular materials. The subdrain invert should be maintained at least 0.3 metres below subgrade level. Subdrains should also be provided at all catch-basins within the parking areas..

5.8.2 Asphaltic Concrete Pavement Design

The industry pavement design methods are based on a design life of 15 to 20 years for typical weather conditions and for the design traffic loadings. On this basis, the following pavement component thicknesses are recommended for flexible pavements which will be subjected to “heavy duty” use (i.e., main site accesses and service accesses) and “light duty” use (i.e., car parking) constructed on a properly prepared sandy silt or silty sand subgrade.

Minimum Asphaltic Concrete Pavement Structure

Pavement Layer	Compaction Requirements	Light Duty Minimum Component Thickness	Heavy Duty Minimum Component Thickness
Surface Course Asphaltic Concrete HL3 (OPSS 1150)	92% MRD	40 mm	40 mm

Pavement Layer	Compaction Requirements	Light Duty Minimum Component Thickness	Heavy Duty Minimum Component Thickness
Base Course Asphaltic Concrete HL8 (OPSS 1150)	92% MRD	50 mm	60 mm
Base Course: Granular A (OPSS 1010) or 19mm Crusher Run Limestone	98% standard Proctor Maximum Dry Density (ASTM-D1557)	150 mm	150 mm
Subbase Course: Granular B Type II (OPSS 1010) or 50mm Crusher Run Limestone	98% standard Proctor Maximum Dry Density (ASTM-D1557)	300 mm	400 mm

Some adjustment to the thickness of the granular subbase material may be required depending on the condition of the subgrade at the time of the pavement construction. The need for such adjustments can be best assessed by the geotechnical engineer during construction.

Consideration should be given to delaying the placement of the final wearing surface for at least one year after construction of the binder course in order to minimize the effects of post construction settlement. Prior to placing the wearing surface, the binder course should be evaluated by the geotechnical engineer and remedial work carried out as required in preparation for final construction..

5.8.3 Drainage

Control of surface water is a significant factor in achieving good pavement life. Grading adjacent to pavement areas must be designed so that water is not allowed to pond adjacent to the outside edges of the pavement or curb. The subgrade must be free of depressions and sloped (preferably at a minimum grade of two percent) to provide effective drainage toward subgrade drains or swales and/or ditches.

Continuous perimeter subdrains should be provided in paved areas and short perforated sub drains should be provided at all catch basins locations. The subdrain invert elevations should be maintained at least 0.3 metres below subgrade level.

It should be noted that in addition to a strict adherence to the above pavement design recommendations, a close control on the pavement construction process will be required in order to obtain the desired pavement life. It is therefore recommended that regular inspection and testing should be conducted during the construction to confirm material quality, thickness, drainage, and to ensure adequate compaction.

6.0 DESIGN CONSIDERATIONS FOR CONSTRUCTABILITY

6.1 Excavations

Excavations must be carried out in accordance with the Occupational Health and Safety Act, Ontario Regulation 213/91 (as amended), Construction Projects, Part III – Excavations, Sections 222 through 242. These regulations designate four (4) broad classifications of soils to stipulate appropriate measures for excavation safety. For practical purposes at this site, the existing fill, and cohesionless soils are considered a Type 3 Soil, provided that effective ground water control is achieved where required and surface water is directed away from open excavations.

Where workers must enter excavations advanced deeper than 1.2 m, the trench walls should be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. The regulation stipulates maximum slopes of excavation by soil type as follows:

Soil Type	Base of Slope	Steepest Slope Inclination
1	within 1.2 metres of bottom of trench	1 horizontal to 1 vertical
2	within 1.2 metres of bottom of trench	1 horizontal to 1 vertical
3	from bottom of trench	1 horizontal to 1 vertical
4	from bottom of trench	3 horizontal to 1 vertical

Minimum support system requirements for steeper excavations are stipulated in the Occupational Health and Safety Act and Regulations for Construction Projects, and include provisions for timbering, shoring and moveable trench boxes.

The need for shoring to support adjacent property will depend on the proximity of the building footprint to the property line. For preliminary consideration temporary unsupported excavations should be cut to an overall inclination of 1 horizontal to 1 vertical or flatter and a buffer of 2 to 3m should be provided between the top of the excavation and the property line. If this minimum geometry cannot be achieved, then consideration will need to be given to the use of shoring. The requirement for shoring will need to be examined when the actual building footprints and the number of basement levels have been finalized.

It is possible that large particles (concrete rubble) may be found in the earth fill material. Similarly, larger size particles (cobbles and boulders) that are not specifically identified in the boreholes may be present in the underlying native soils. The size and distribution of such obstructions cannot be predicted with boreholes, as the sampler size is insufficient to secure representative samples of particles of this size. Provision must be made in excavation contracts to allocate risks associated with the time spent and equipment utilized to remove or penetrate such obstructions when encountered.

All boreholes were generally observed to by dry upon the completion of drilling. There can be seams or lenses of sand encountered in these deposits which will yield seepage if exposed. Flow from these permeable zones is, however, limited by the storage volume of the sand pockets. There is also a potential for encountering localized perched ground water conditions within the fill deposits.

If caisson foundations are selected, the caissons must be lined with a temporary steel casing to facilitate hand cleaning and inspection of the rock surface prior to placing concrete. The soil immediately above the rock is a saturated sand. The contractors bidding on these works should be made aware that they are likely to have to tamp the temporary liners, into the top of rock, and/or use a bentonite slurry to affect an adequate seal of the liner at the top of rock.

6.2 Shoring Design Considerations

The Property is bound to the north by Robinson Street, to the south by a public parking lot, to the west by Allendale Avenue and to the east by a multi-tenant commercial building and associated parking lot. No excavation shall extend below the foundations of existing adjacent structures without adequate alternative support being provided. Underpinning guidelines are provided as Appendix C.

Where excavations cannot be sloped, they can be supported using conventional soldier pile and lagging walls. If required by the underpinning guidelines provided, the north walls of the excavation may need to be constructed as a rigid shoring system to preserve the integrity and support of the soil beneath existing foundations of the adjacent buildings, in a state approximating the at-rest condition. This is to be achieved using continuous interlocking caisson wall shoring.

6.2.1 Earth Pressure Distribution

Where multiple supports are used to support the excavation, research has shown that a distributed pressure diagram more realistically approximates the earth pressure on a shoring system of this type, when restrained by pre-tensioned anchors. The multi-level supported shoring can be designed based on an earth pressure distribution consisting of a trapezoidal pressure distribution with a maximum pressure defined by:

$$\boldsymbol{P = 0.65\,K[\gamma H + q]}$$

where,
- **P** = the horizontal pressure at depth, **h** (kPa)
- **K** = the earth pressure coefficient
- **H** = the total depth of the excavation (m)
- **γ** = the bulk unit weight of soil, (kN/m^3)
- **q** = the complete surcharge loading (kPa)

6.2.2 Soldier Pile Toe Design

Soldier pile toes will be made in dense sandy silt to silty sand. The horizontal resistance of the soldier pile toes will be developed by embedment below the base of excavation, where resistance is developed from passive earth pressure.

There are zones of material in the subsurface soils that are sufficiently wet, cohesionless, and permeable that augered holes for soldier piles could experience dry and wet cave. It will be necessary to advance temporarily cased holes to maintain sidewall support and to prevent the ingress of water during soldier pile installation. Contractors must adjust their means and methods to ensure that the augered borehole base remains stable.

6.2.3 Shoring Support

If anchor support is necessary and determined to be feasible, the shoring system should be supported by pre-stressed soil anchors extending beneath the adjacent lands. Pre-stressed anchors are installed and stressed in advance of excavation and this limits movement of the shoring system as much as is practically possible. The use of anchors on adjacent properties requires the consent of the adjacent landowners, expressed in encroachment agreements.

Raker footings established on dense sandy silt to silty sand soils at an inclination of 45 degrees can be designed for a maximum factored geotechnical resistance at ULS of 120 kPa.

Post-grouted wash bored anchors can be made. The design adhesion for earth anchors is controlled as much by the installation technique as the soil and therefore a proto-type anchor must be made in each anchor level executed to demonstrate the anchor capacity and validate the design assumptions. It is expected that post-grouted anchors can be made such that an anchor will safely carry about 60 kN/m of adhered anchor length (at a nominal diameter of 150 mm) within the sandy silt to silty sand stratum.

6.3 Depth of Frost Penetration

The design earth cover for frost protection of foundations exposed to ambient environmental temperatures is 1.2 metres in the Greater Toronto area. Experience suggests that the temperature in “unheated” underground parking levels two or more levels below grade with normal ventilation provisions is not as severe as the ambient open-air condition. The earth cover required to prevent frost effects on foundations in the lower parking levels need not be any greater than 1.2 metres, and experience in a number of structures has shown that perimeter foundations provided with 600 mm of cover perform adequately as do interior isolated foundations with 900 mm of cover. At locations adjacent to ventilation shafts, it is normal practice to provide insulation to ensure that foundations are not affected by the cold air flow.

For buried utility lines, variations from the above noted depth of frost penetration might be considered, depending on various factors such as the type of backfilling materials or the temperature and moisture exposure of the area (prevailing winds, drifting snow, etc.). However, these variations do not generally represent a concern unless special equipment and/or buried utilities have specific requirements regarding the subsurface temperature and moisture regime (i.e., water lines or sensitive electrical utilities etc.). In such special situations further tests and analysis should be conducted on a case-by-case basis.

The depth of frost penetration is also defined as the zone of active weathering where sizeable variations in the moisture content accompany the yearly temperature fluctuations. Therefore, the foundation grades should be established at or below this depth. For the light poles and other light structures that are to be installed on a single footing, if some frost heave (25 mm to 50 mm) cannot be tolerated, the foundation elements should also be provided with the above noted minimum depth of soil cover or equivalent exterior-grade insulation.

The soil at this site is susceptible to frost effects which would have the potential to deform hard landscaping adjacent to the building. At locations where buildings are expected to have flush entrances, care must be taken in detailing the exterior slabs / sidewalks, providing insulation / drainage / non-frost susceptible backfill to maintain the flush threshold during freezing weather conditions.

6.4 Site Work

The soil at this site is fine-grained and will become weakened when subjected to traffic when wet. If there is site work carried out during periods of wet weather, then it can be expected that the subgrade will be disturbed unless an adequate granular working surface is provided to protect the integrity of the subgrade soils from construction traffic. Subgrade preparation works cannot be adequately accomplished during wet weather and the project must be scheduled accordingly. The disturbance caused by the traffic can result in the removal of disturbed soil and use of fill material for site restoration or underfloor fill that is not intrinsic to the project requirements. Attempting to build slabs and pavements at this site during wet weather could significantly increase earthworks and pavement costs.

The most severe loading conditions on the subgrade may occur during construction. Consequently, special provisions such as end dumping and forward spreading of earth and aggregate fills, restricted construction lanes, and half-loads during paving and other work are required, especially if construction is carried out during unfavourable weather.

If construction proceeds during freezing weather conditions, adequate temporary frost protection for the founding subgrade and concrete must be provided. The soil at this site is highly susceptible to frost damage. Consideration must be given to frost effects, such as heave or softening, on exposed soil surfaces in the context of this particular project development.

6.5 Quality Control

6.5.1 Shoring

The City of Niagara Falls will require that the shoring installations be monitored during the period of construction to demonstrate that the shoring is performing adequately. Terraprobe has considerable experience in the provision of shoring instrumentation and monitoring services for a number of similar sites.

The provisions of the Ontario Building Code require that the construction of the earth retaining structures be monitored on a continuous basis. The shoring system constitutes an earth retaining structure as provided in Section 4.2.2.3 of the Ontario Building Code 2012. Terraprobe should be retained to provide this review as the shoring installations are made. It is an integral part of the geotechnical design function as it relates to shoring design considerations.

Assuming soil anchors will be used to support the shoring system on this site, a minimum of one anchor as each target anchorage level must be performance tested to verify the design adhesion used for the anchorages. This performance test anchor shall be consistent dimension in anchor and free stressing zones with the proposed production anchors and be provided with adequate tendon steel capacity to test the anchor to twice the design working load. The performance tests shall be monitored and evaluated by the geotechnical engineer. Production anchorages should not be installed until the performance test at each level has adequately demonstrated the design adhesion value. All production anchorages shall be monitored during stressing and evaluated by a geotechnical engineer.

6.5.2 Foundations

The proposed structures will be founded on conventional spread footings. All foundation installations must be reviewed in the field by Terraprobe, the geotechnical engineer, as they are constructed. The on-site review of the condition of the foundation soil as the foundations are constructed is an integral part of the geotechnical engineering design function and is required by Section 4.2.2.2 of the Ontario Building Code 2012. If Terraprobe is not retained to carry out foundation engineering field review during construction, then Terraprobe accepts no responsibility for the performance or non-performance of the foundations, even if they are ostensibly constructed in accordance with the conceptual design advice contained in this report.

6.5.3 Slabs on Grade

The long-term performance of the slab on grade is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure that uniform subgrade moisture and density conditions are achieved as much as practically possible. The design advice in this report is based on an assessment of the subgrade support capabilities as indicated by the boreholes.

These conditions may vary across the site depending on the final design grades and therefore, the preparation of the subgrade and the compaction of all fill should be monitored by Terraprobe at the time of construction to confirm material quality, thickness, and to ensure adequate compaction.

6.5.4 General

The requirements for fill placement on this project have been stipulated relative to Standard Proctor Maximum Dry Density (SPMDD). In situ determinations of density during fill and asphaltic pavement placement on site are required to demonstrate that the specified placement density is achieved. Terraprobe is a CNSC certified operator of appropriate nuclear density gauges for this work and can provide sampling and testing services for the project as necessary, with our qualified technical staff.

Concrete will be specified in accordance with the requirements of CAN3 - CSA A23.1/2. Terraprobe maintains a CSA certified concrete laboratory and can provide concrete sampling and testing services for the project as necessary.

Terraprobe staff can also provide quality control services for Building Envelope, Roofing and Structural Steel, as necessary, for the Structural and Architectural quality control requirements of the project. Terraprobe is certified by the Canadian Welding Bureau under W178.1.

7.0 LIMITATIONS AND USE OF REPORT

7.1 Procedures

This investigation has been carried out using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by Terraprobe and other engineering practitioners, working under similar conditions and subject to the time, financial and physical constraints applicable to this project. The discussions and recommendations that have been presented are based on the factual data obtained from this investigation.

The drilling work was carried out by a drilling contractor and was observed and recorded by Terraprobe on a full-time basis. The boreholes were made by a continuous flight power auger machine using solid stem augers and HQ rock coring. The Terraprobe technician logged the boreholes and examined the samples as they were obtained. The samples obtained were sealed in clean, air-tight containers or rock core boxes and transferred to the Terraprobe laboratory, where they were reviewed for consistency of description by a geotechnical engineer. Ground water monitoring wells were installed in select boreholes to measure long-term ground water levels.

The samples of the strata penetrated were obtained using the Split-Barrel Method technique (ASTM D1586). The samples were taken at intervals. The conventional interval sampling procedure used for this investigation does not recover continuous samples of soil at any borehole location. There is consequently some interpolation of the borehole layering between samples and indications of changes in stratigraphy as shown on the borehole logs are approximate.

It must be recognized that there are special risks whenever engineering or related disciplines are applied to identify subsurface conditions. A comprehensive sampling and testing programme implemented in accordance with the most stringent level of care may fail to detect certain conditions. Terraprobe has assumed for the purposes of providing design parameters and advice, that the conditions that exist between sampling points are similar to those found at the sample locations.

It may not be possible to drill a sufficient number of boreholes, or sample and report them in a way that would provide all the subsurface information and geotechnical advice to completely identify all aspects of the site and works that could affect construction costs, techniques, equipment and scheduling. Contractors bidding on or undertaking work on the project must be directed to draw their own conclusions as to how the subsurface conditions may affect them, based on their own investigations and their own interpretations of the factual investigation results, and their approach to the construction works, cognizant of the risks implicit in the subsurface investigation activities.

7.2 Changes in Site and Scope

It must be recognized that the passage of time, natural occurrences, and direct or indirect human intervention at or near the site have the potential to alter subsurface conditions. In particular, caution should be exercised in the consideration of contractual responsibilities as they relate to control of seepage, disturbance of soils, and frost protection.

The design parameters provided, and the engineering advice offered in this report are based on the factual data obtained from this investigation made at the site by Terraprobe and are intended for use by the owner and its retained design consultants in the design phase of the project. If there are changes to the project scope and development features, the interpretations made of the subsurface information, the geotechnical design parameters, advice, and comments relating to constructability issues and quality control may not be relevant or complete for the project. Terraprobe should be retained to review the implications of such changes with respect to the contents of this report.

7.3 Use of Report

This report is prepared for the express use of La Pue International Inc. and their retained design consultants. It is not for use by others. This report is copyright of Terraprobe Inc., and no part of this report may be reproduced by any means, in any form, without the prior written permission of Terraprobe.

It is recognized that the City of Niagara Falls, in their capacity as the planning and building authority under Provincial statues, will make use of and rely upon this report, cognizant of the limitations thereof, both as are expressed and implied.

Terraprobe Inc.

LICENSED PROFESSIONAL ENGINEER
A. A. FELICE
100213553
Feb10/22
PROVINCE OF ONTARIO

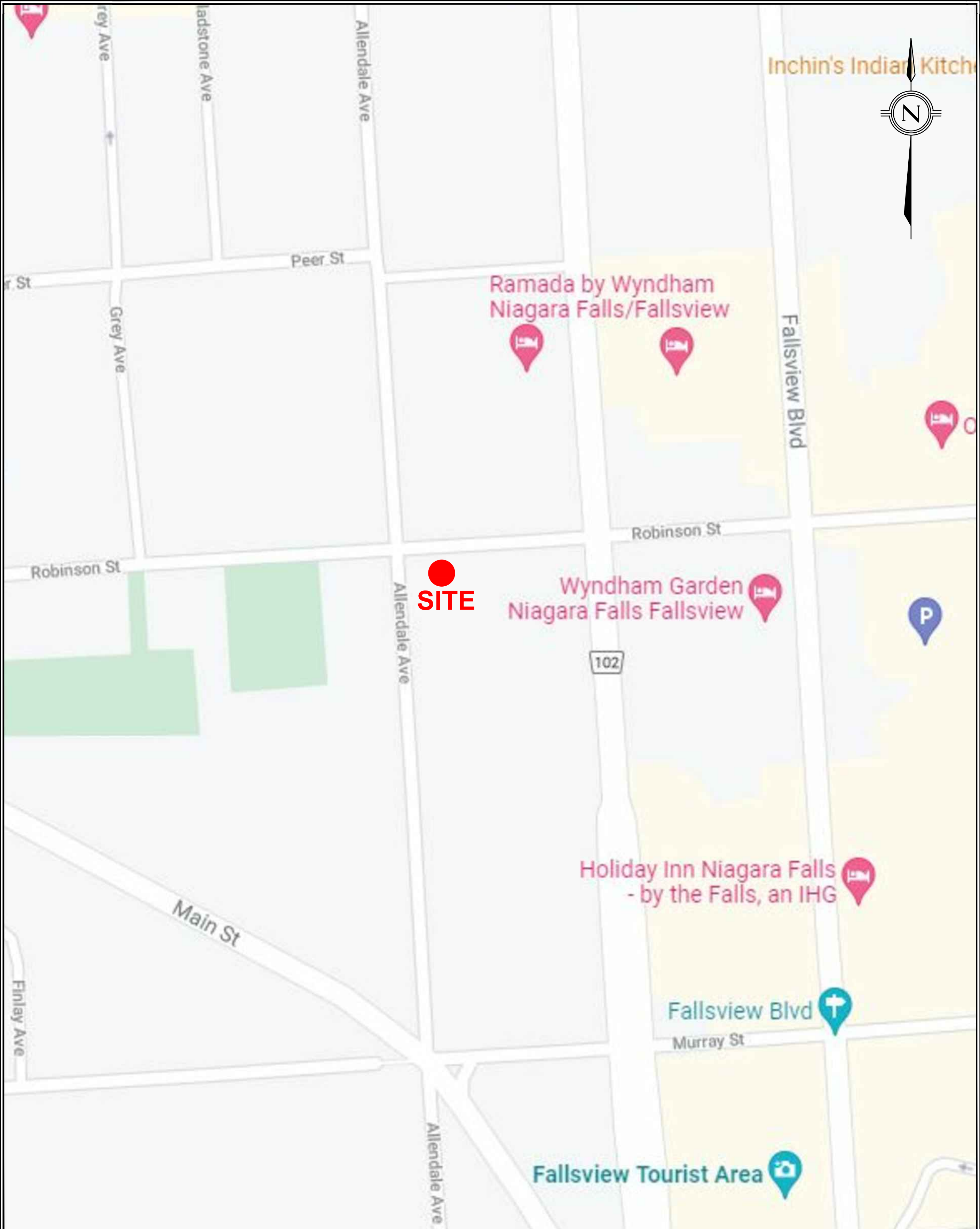
LICENSED PROFESSIONAL ENGINEER
P. T. CANNON
Feb10/22
PROVINCE OF ONTARIO

Anthony Felice, P. Eng.,
Project Manager, Geotechnical

Patrick Cannon, P. Eng.,
Principal, Branch Manager

FIGURES

Terraprobe Inc.



Terraprobe

903 Barton Street - Unit 22, Stoney Creek, Ontario, L8E 5R
Tel: (905) 643-7560, Fax: (905) 643-7559

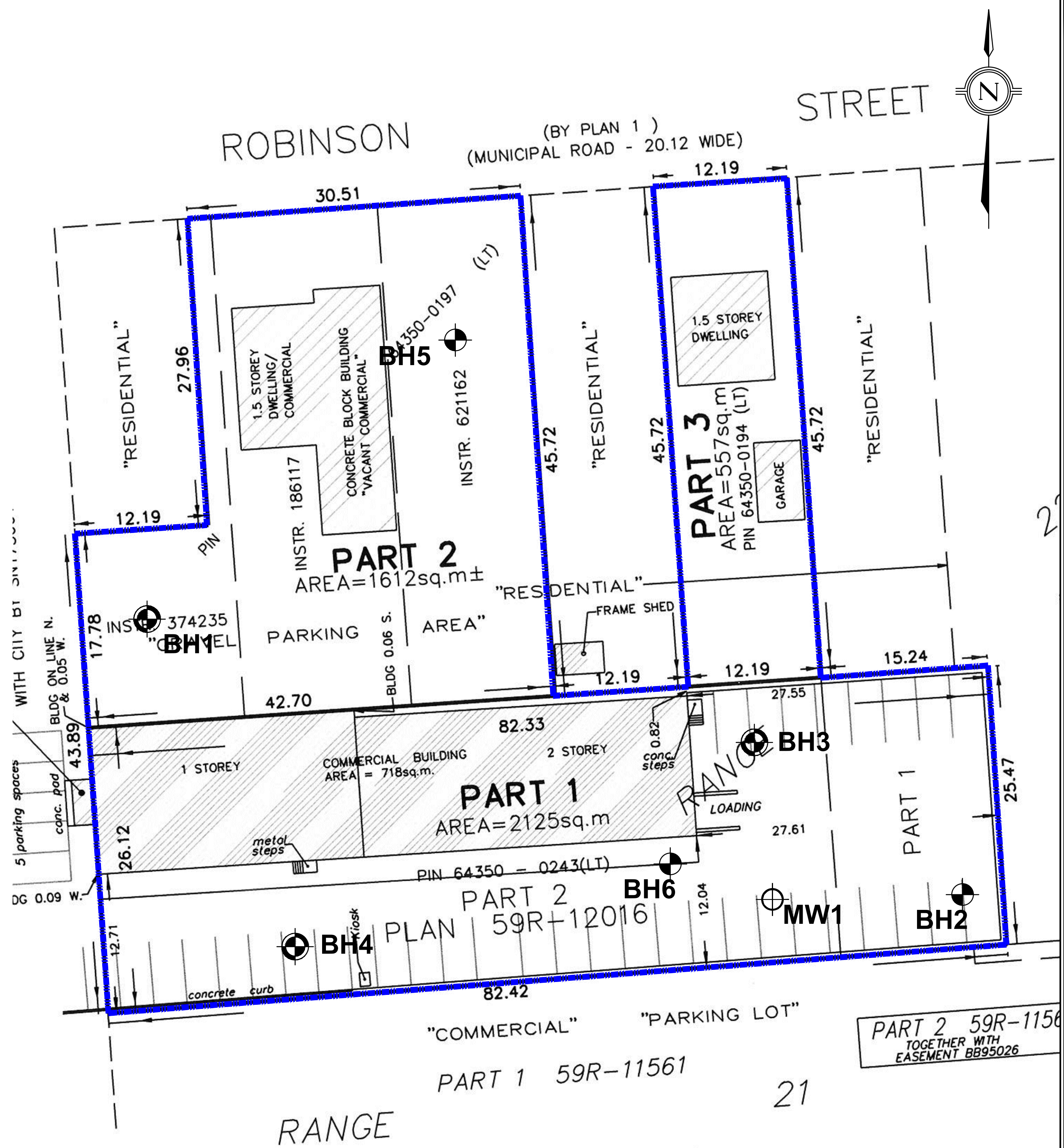
Title:

SITE LOCATION PLAN

6158 Allendale Ave & 5592, 5566 Robinson Street, Niagara Falls, Ontario

File No. 7-21-0103-01

FIGURE : 1



LEGEND

BH Borehole Location

Terraprobe

903 Barton Street - Unit 22, Stoney Creek, Ontario, L8E 5R7
Tel: (905) 643-7560, Fax: (905) 643-7559

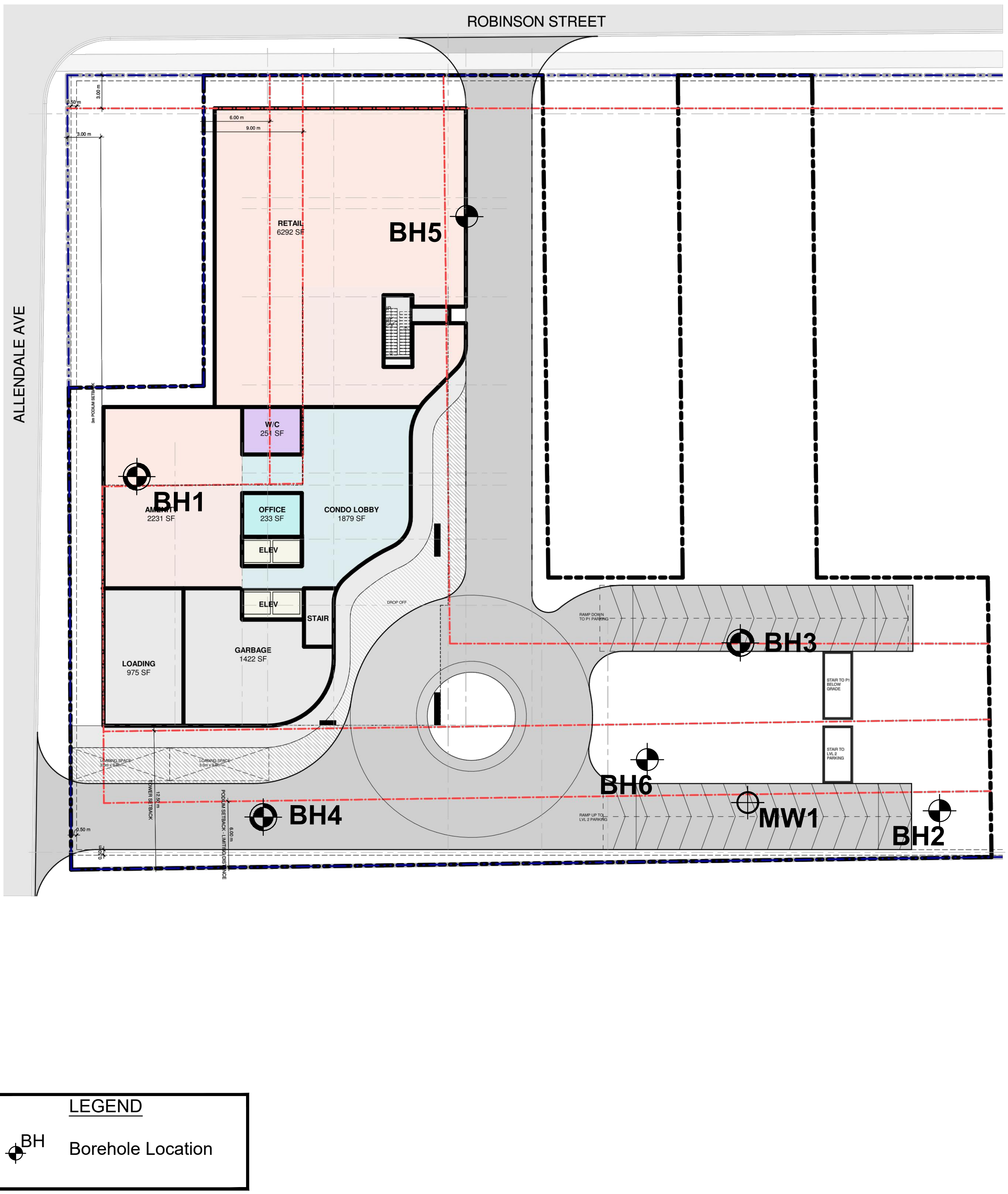
Title:

BOREHOLE LOCATION PLAN

6158 Allendale Ave & 5592, 5566 Robinson Street, Niagara Falls, Ontario

File No. 7-21-0103-01

FIGURE : 2



Terraprobe
903 Barton Street - Unit 22, Stoney Creek, Ontario, L8E 5R
Tel: (905) 643-7560, Fax: (905) 643-7559

Title: PROPOSED SITE DEVELOPMENT PLAN
6158 Allendale Ave & 5592, 5566 Robinson Street, Niagara Falls, Ontario

File No. 7-21-0103-01

FIGURE : 3

LOGS OF BOREHOLES

APPENDIX A

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SAMPLING METHODS

AS	auger sample
CORE	cored sample
DP	direct push
FV	field vane
GS	grab sample
SS	split spoon
ST	shelby tube
WS	wash sample

PENETRATION RESISTANCE

Standard Penetration Test (SPT) resistance ('N' values) is defined as the number of blows by a hammer weighing 63.6 kg (140 lb.) falling freely for a distance of 0.76 m (30 in.) required to advance a standard 50 mm (2 in.) diameter split spoon sampler for a distance of 0.3 m (12 in.).

Dynamic Cone Test (DCT) resistance is defined as the number of blows by a hammer weighing 63.6 kg (140 lb.) falling freely for a distance of 0.76 m (30 in.) required to advance a conical steel point of 50 mm (2 in.) diameter and with 60° sides on 'A' size drill rods for a distance of 0.3 m (12 in.)."

COHESIONLESS SOILS

Compactness	'N' value
very loose	< 4
loose	4 – 10
compact	10 – 30
dense	30 – 50
very dense	> 50

COHESIVE SOILS

Consistency	'N' value	Undrained Shear Strength (kPa)
very soft	< 2	< 12
soft	2 – 4	12 – 25
firm	4 – 8	25 – 50
stiff	8 – 15	50 – 100
very stiff	15 – 30	100 – 200
hard	> 30	> 200

COMPOSITION

Term (e.g)	% by weight
trace silt	< 10
some silt	10 – 20
silty	20 – 35
sand *and* silt	> 35

TESTS AND SYMBOLS

MH	mechanical sieve and hydrometer analysis
w, w_c	water content
w_L, LL	liquid limit
w_P, PL	plastic limit
I_P, PI	plasticity index
k	coefficient of permeability
γ	soil unit weight, bulk
φ'	internal friction angle
c'	effective cohesion
c_u	undrained shear strength

∇	Unstabilized water level
▼	1st water level measurement
▼	2nd water level measurement
▼	Most recent water level measurement
3.0 +	Undrained shear strength from field vane (with sensitivity)
C_c	compression index
c_v	coefficient of consolidation
m_v	coefficient of compressibility
e	void ratio

FIELD MOISTURE DESCRIPTIONS

Damp	refers to a soil sample that does not exhibit any observable pore water from field/hand inspection.
Moist	refers to a soil sample that exhibits evidence of existing pore water (e.g. sample feels cool, cohesive soil is at plastic limit) but does not have visible pore water
Wet	refers to a soil sample that has visible pore water

Terraprobe Inc.

Greater Toronto
11 Indell Lane
Brampton, Ontario L6T 3Y3
(905) 796-2650 Fax: 796-2250

Hamilton – Niagara
903 Barton Street, Unit 22
Stoney Creek, Ontario L8E 5P5
(905) 643-7560 Fax: 643-7559

Central Ontario
220 Bayview Drive, Unit 25
Barrie, Ontario L4N 4Y8
(705) 739-8355 Fax: 739-8369

Northern Ontario
1012 Kelly Lake Rd., Unit 1
Sudbury, Ontario P3E 5P4
(705) 670-0460 Fax: 670-0558

www.terraprobe.ca

RECOVERY

TCR Total Core Recovery is the total length of core pieces, irrespective of their individual lengths obtained in a core run, and expressed as a percentage of the length of that core run.

SCR Solid Core Recovery is the total length of sound full-diameter core pieces obtained in a core run, expressed as a percentage of the length of that core run.

RQD Rock Quality Designation pertains to the sum of those pieces of sound core which are 10 cm or greater in length obtained in a core run, expressed as a percentage of the length of that core run.

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
QUALITY	**very poor**	**poor**	**fair**	**good**	**excellent**

JOINT CHARACTERISTICS

Orientation

horizontal / flat = 0 - 20°
dipping = 20 - 50°
vertical = 50 - 90°

Joint Aperture

Classification	Aperture
closed	< 0.5 mm
gapped	0.5 to 10 mm
open	> 10 mm

Joint Spacing (ISRM, 1981)

Classification	Spacing (m)
extremely close	< 0.02
very close	0.02 to 0.06
close	0.06 to 0.2
moderately close	0.2 to 0.6
wide	0.6 to 2
very wide	2 to 6
extremely wide	> 6

Joint Filling

Description	Approx φ
tight, hard, non-softening	25 - 35
oxidation, surface staining only	25 - 30
slightly altered, clay-free	25 - 30
sandy particles, clay-free	20 - 25
sandy and silty, minor clay	16 - 24
non-softening clays	6 - 12
swelling clay fillings	n/a

Degree of Weathering (after MTO, RR229 Evaluation of Shales for Construction Projects)

Zone	Degree	Description
Z1	unweathered	shale, regular jointing
Z2	partially weathered	angular blocks of unweathered shale, no matrix, with chemically weathered but intact shale
Z3	partially weathered	soil-like matrix with frequent angular shale fragments < 25mm diameter
Z4a	partially weathered	soil-like matrix with occasional shale fragments < 3mm diameter
Z4b	fully weathered	soil-like matrix only

Strength classification (after Marinos and Hoek, 2001)

Grade	Term	UCS (MPa)	Field Estimate (Description)
R6	extremely strong	> 250	can only be chipped by geological hammer
R5	very strong	100 - 250	requires many blows from geological hammer
R4	strong	50 - 100	requires more than one blow from geological hammer
R3	medium strong	25 - 50	can't be scraped, breaks under one blow from geological hammer
R2	weak	5 - 25	can be peeled / scraped with knife with difficulty
R1	very weak	1 - 5	easily scraped / peeled, crumbles under firm blow of geo. hammer
R0	extremely weak	< 1	indented by thumbnail

LOG OF BOREHOLE 1

Project No. : 7-21-0103-01
Date started : November 16, 2021
Sheet No. : 1 of 1

Client : La Pue International Inc
Project : Allendale Avenue and Robinson Street
Location : Niagara Falls, Ontario

Originated by : KG
Compiled by : KG
Checked by : TW

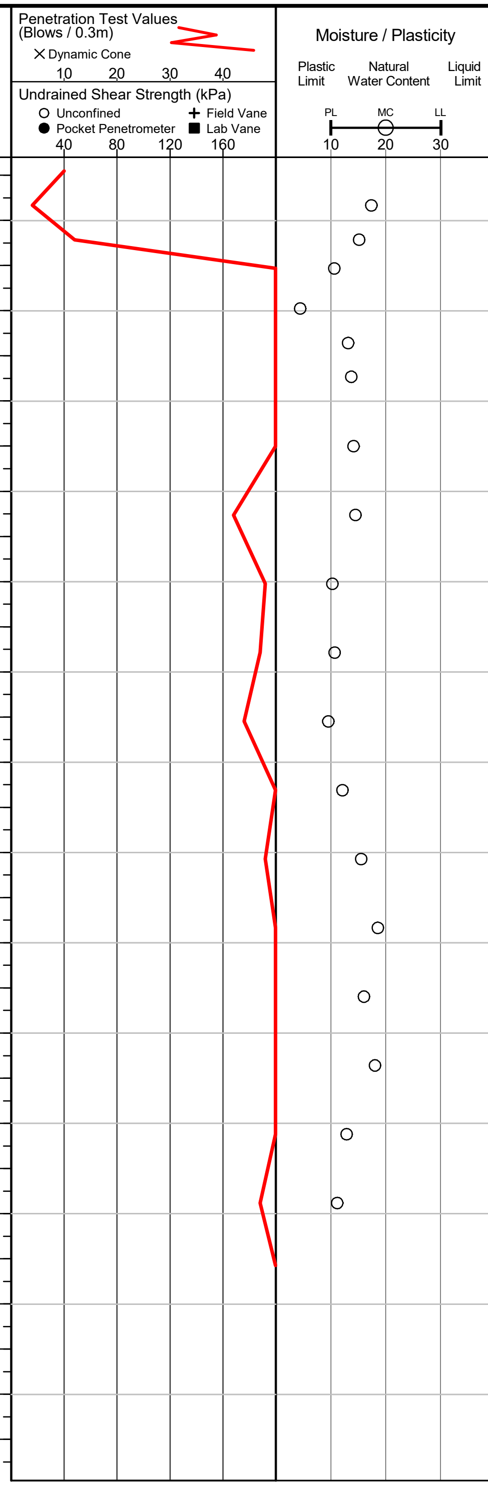
Position : E: 655794, N: 4772128 (UTM 17T)
Rig type : Truck-mounted
Elevation Datum : Geodetic
Drilling Method : Solid stem augers, HQ rock coring
Core Diameter : **HQ**, OD=96mm, ID=64mm

SOIL PROFILE

Elev / Depth (m)	Description
199.4	**GROUND SURFACE**
198.3 / 1.1	**FILL**, sand and gravel, trace brick fragments, concrete, compact, grey
197.9 / 1.5	**FILL**, sandy silt, trace clay, loose, brown
	SANDY SILT to SILTY SAND, trace clay, dense to very dense, brown
	...trace gravel at 19.8m
175.0 / 24.4	**SAND AND GRAVEL**, very dense, brown
174.7 / 24.7	**GUELPH FORMATION**
170.1 / 29.3	**END OF BOREHOLE**

SAMPLES

Number	Type	SPT 'N' Value	Headspace Vapour (ppm)
1	SS	10	PID: 5
2	SS	4	PID: 5
3	SS	12	PID: 0
4	SS	90 / 200mm	PID: 5
5	SS	62	PID: 10
6	SS	82	PID: 10
7	SS	85	PID: 5
8	SS	52	PID: 5
9	SS	42	PID: 0
10	SS	48	PID: 5
11	SS	47	PID: 5
12	SS	44	PID: 0
13	SS	51	PID: 0
14	SS	48	PID: 10
15	SS	56	PID: 0
16	SS	55	PID: 0
17	SS	55	PID: 0
18	SS	69	PID: 0
19	SS	47	PID: 0
20	SS	70 / 175mm	PID: 0
	RUN		
	RUN		
	RUN		



Lab Data and Comments

GRAIN SIZE DISTRIBUTION (%) (MIT)

SS1 Analysis: M&I, PAH, PCB

SS2 Analysis: VOC, PHC

SS3 Analysis: M&I, PAH

SS14 Analysis: VOC, PHC

GR	SA	SI	CL
0	56	37	7

END OF BOREHOLE
Auger refusal

Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Dec 3, 2021	dry	n/a
Jan 7, 2022	dry	n/a
Jan 24, 2022	dry	n/a

file: 7-21-0103-01 - allendale ave.gpj

Terraprobe

ROCK CORE LOG 1

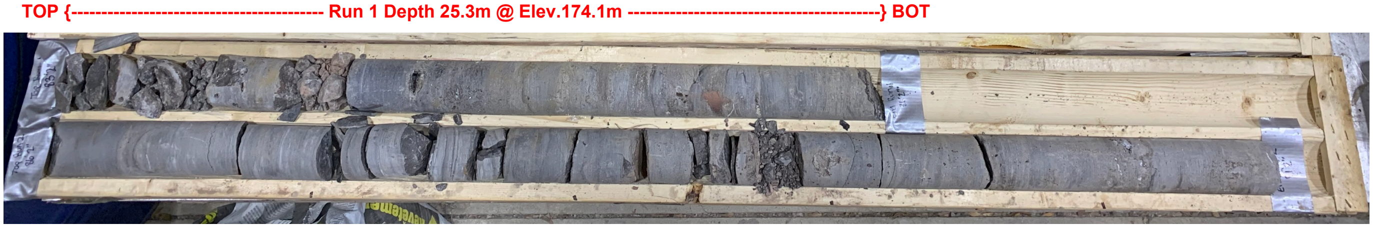
Project No. : 7-21-0103-01	Client : La Pue International Inc	Originated by : KG
Date started : November 16, 2021	Project : Allendale Avenue and Robinson Street	Compiled by : KG
Sheet No. : 1 of 1	Location : Niagara Falls, Ontario	Checked by : TW

Position : E: 655794, N: 4772128 (UTM 17T)	Elevation Datum : Geodetic	Core Diameter : **HQ**, OD=96mm, ID=64mm
Rig type : Truck-mounted	Drilling Method : Solid stem augers, HQ rock coring	

Depth (m)	Graphic Log	GENERAL DESCRIPTION	Run / Elev Depth (m)	Recovery	Elevation (m)	Shale Weathering Zones (Z1 Z2 Z3 Z4)	UCS (MPa) ● 5 25 50 100 250 / Estimated Strength (R1 R2 R3 R4 R5 R6)	Natural Fractures: Frequency	Natural Fractures: Spacing	Laboratory Testing	Comments
		Rock coring started at 25.3m below grade	174.1								
25.3–26.3		**GUELPH FORMATION** light brown to grey, medium bedded to thickly bedded, strong to very strong, finely crystalline (vuggy with calcite); joints are horizontal, closely spaced, bituminous joints	25.3 R1 173.1	TCR = **100%** SCR = **81%** RQD = **69%**	174			5, 1, 0	close		**25.3-25.5m:** *Rubblized / Weathered Zone*; **25.7-25.8m:** *Vuggy with clacite deposit*
26.3–27.8			26.3 R2 171.6	TCR = **100%** SCR = **79%** RQD = **52%**	173, 172			1, 6, 4, 2, 0	close	**26.3m to 26.4m:** UCS = 101.2 MPa; **27.5m to 27.6m:** UCS = 102.2 MPa	**26.7-26.8m:** *Sub-vertical Fracture*
27.8–29.3			27.8 R3 170.1	TCR = **100%** SCR = **88%** RQD = **52%**	171			3, 5, 6, 2, 2	close		**28.7-28.7m:** *Calcite deposit*
			29.3m								

END OF COREHOLE

TOP {-- Run 1 Depth 25.3m @ Elev.174.1m --} BOT



TOP {-- Run 2 Depth 26.3m @ Elev.173.1m --} BOT

TOP {-- Run 3 Depth 27.8m @ Elev.171.6m --} BOT

LOG OF BOREHOLE 2

Terraprobe

Project No.	: 7-21-0103-01	Client	: La Pue International Inc	Originated by	: KG
Date started	: November 15, 2021	Project	: Allendale Avenue and Robinson Street	Compiled by	: KG
Sheet No.	: 1 of 1	Location	: Niagara Falls, Ontario	Checked by	: TW

Position : E: 655866, N: 4772105 (UTM 17T)
Elevation Datum : Geodetic
Core Diameter : **HQ**, OD=96mm, ID=64mm
Rig type : Truck-mounted
Drilling Method : Solid stem augers, HQ rock coring

SOIL PROFILE

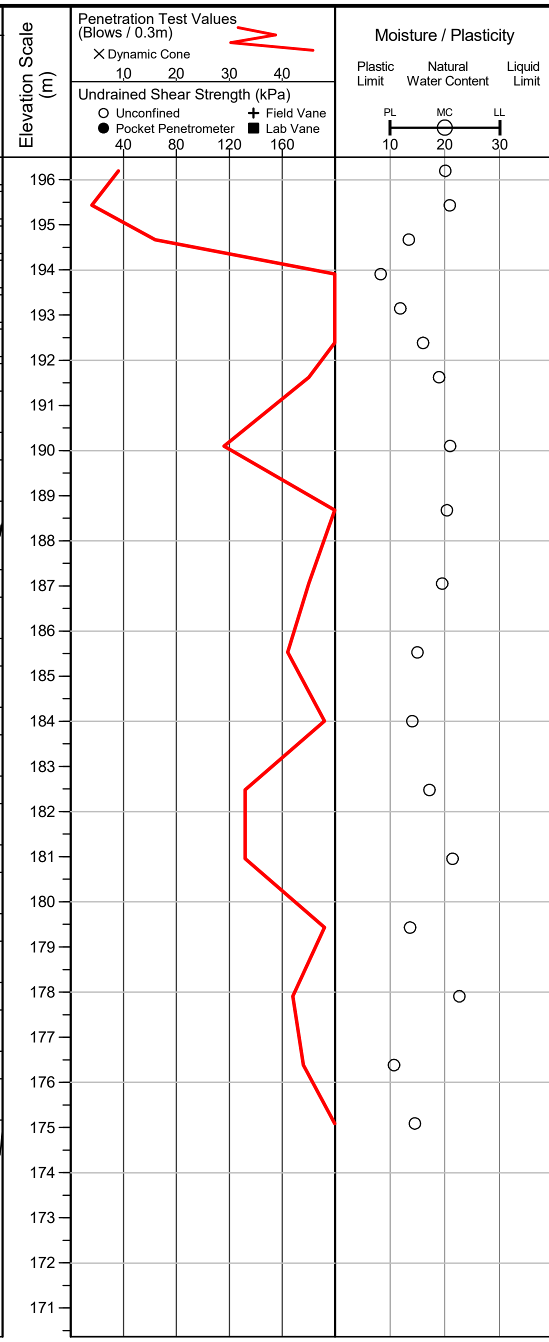
Elev (m)	Depth (m)	Description
196.5		**GROUND SURFACE**
196.2	0.3	100mm **ASPHALTIC CONCRETE**
		FILL, granular, greyish black
195.0	1.5	**FILL**, clayey silt, trace sand, gravel, loose, black to brown
		SILT, some clay, with sand, gravel, compact to very dense, reddish brown (GLACIAL TILL)
188.9	7.6	**SILTY SAND**, dense to very dense, brown
176.7	19.8	**COARSE SAND and GRAVEL**, dense to very dense, brown
174.9	21.6	**GUELPH FORMATION**
170.4	26.1	

END OF BOREHOLE
Auger refusal

Borehole was dry and open upon completion of drilling.

SAMPLES

Number	Type	SPT 'N' Value
1	SS	9
2	SS	4
3	SS	16
4	SS	65
5	SS	54
6	SS	94
7	SS	45
8	SS	29
9	SS	95 / 250mm
10	SS	45
11	SS	41
12	SS	48
13	SS	33
14	SS	33
15	SS	48
16	SS	42
17	SS	44
18	SS	50 / 0mm
	RUN	
	RUN	
	RUN	



Headspace Vapour (ppm)

Instrument Details

Unstabilized Water Level

Lab Data and Comments

GRAIN SIZE DISTRIBUTION (%) (MIT)

GR SA SI CL

SS2 Analysis:
M&I, PAH, VOC, PHC

SS3 Analysis:
M&I, VOC, PHC
11 14 60 15

Terraprobe

ROCK CORE LOG 2

Project No. : 7-21-0103-01	Client : La Pue International Inc	Originated by : KG
Date started : November 15, 2021	Project : Allendale Avenue and Robinson Street	Compiled by : KG
Sheet No. : 1 of 1	Location : Niagara Falls, Ontario	Checked by : TW

Position : E: 655866, N: 4772105 (UTM 17T)
Elevation Datum : Geodetic
Core Diameter : **HQ**, OD=96mm, ID=64mm
Rig type : Truck-mounted
Drilling Method : Solid stem augers, HQ rock coring

Rock coring started at 21.9m below grade

Depth (m)	GENERAL DESCRIPTION	Run	Elev Depth (m)	Recovery	Natural Fractures Frequency	Spacing	Laboratory Testing	Comments
21.8–23.1	**GUELPH FORMATION** light brown to grey, medium bedded to thickly bedded, very strong, finely crystalline (vuggy with calcite); joints are horizontal, closely spaced, bituminous joints	R1	174.6 / 21.8 – 173.4	TCR = **92%** SCR = **72%** RQD = **55%**	1, 2, 3, 4	close	**22.4m to 22.5m:** UCS = 87.6 MPa	
23.1–24.9		R2	173.4 / 23.1 – 171.6	TCR = **83%** SCR = **67%** RQD = **48%**	3, 2, 2, 4, 5, 3	close		**23.1-23.3m:** *Vuggy with calcite deposits*; **24.0-24.2m:** *Weathered Zone*
24.9–26.1		R3	171.6 / 24.9 – 170.4	TCR = **125%** SCR = **118%** RQD = **86%**	2, 2, 3, 2	close	**25.7m to 25.8m:** UCS = 131.8 MPa	**25.1-25.5m:** *Vuggy with calcite deposits*

Depth scale (m): 22, 23, 24, 25, 26. Elevation (m): 174, 173, 172, 171.
Shale Weathering Zones: Z1, Z2, Z3, Z4. UCS (MPa): 5, 25, 50, 100, 250. Estimated Strength: R1, R2, R3, R4, R5, R6.

26.1m

END OF COREHOLE

TOP {-- Run 1 Depth 21.8m @ Elev.174.6m ---} BOT

TOP {-- Run 2 Depth 23.1m @ Elev.173.4m --} BOT

TOP {-- Run 3 Depth 24.9m @ Elev.171.6m -->>

Cont -------} BOT

file: 7-21-0103-01 - allendale ave.gpj

Terraprobe

LOG OF BOREHOLE 3

Project No. : 7-21-0103-01	Client : La Pue International Inc	Originated by : KG
Date started : November 19, 2021	Project : Allendale Avenue and Robinson Street	Compiled by : KG
Sheet No. : 1 of 1	Location : Niagara Falls, Ontario	Checked by : TW

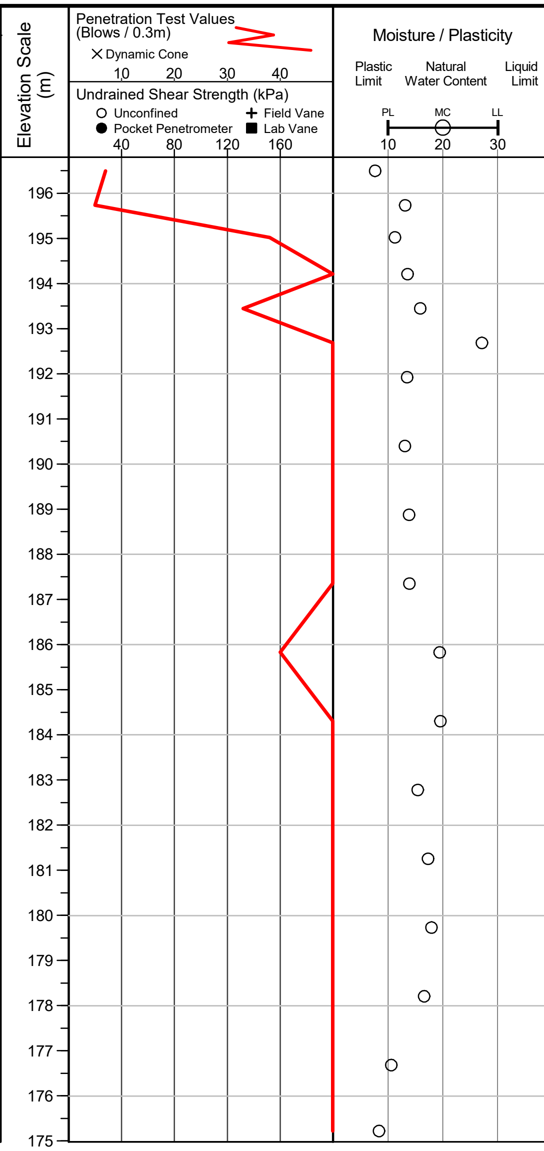
Position : E: 655851, N: 4772113 (UTM 17T)
Elevation Datum : Geodetic
Rig type : Truck-mounted
Drilling Method : Solid stem augers

SOIL PROFILE

Elev Depth (m)	Description
196.8	GROUND SURFACE
	100mm **ASPHALTIC CONCRETE**
	FILL, granular base
195.3 / 1.5	**FILL**, sandy silt, trace clay, rootlets, loose, brown
	SANDY SILT, trace clay, dense to very dense, brown
192.2 / 4.6	**SILTY SAND**, dense to very dense, brown
177.0 / 19.8	**COARSE SAND and GRAVEL**, very dense, brown
175.0 / 21.8	

SAMPLES

Number	Type	SPT 'N' Value
1	SS	7
2	SS	5
3	SS	38
4	SS	68
5	SS	33
6	SS	60
7	SS	73
8	SS	58
9	SS	59
10	SS	60
11	SS	40
12	SS	59
13	SS	70
14	SS	88
15	SS	74
16	SS	87
17	SS	95
18	SS	60



Headspace Vapour (ppm)

Instrument Details

Unstabilized Water Level

Lab Data and Comments

GRAIN SIZE DISTRIBUTION (%) (MIT)
GR SA SI CL

SS1 Analysis:
M&I, VOC, PHC

SS2 Analysis:
PAH

SS3 Analysis:
M&I

SS7 Analysis:
VOC, PHC

END OF BOREHOLE
Refusal on inferred bedrock

Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Dec 3, 2021	dry	n/a
Jan 7, 2022	dry	n/a
Jan 24, 2022	dry	n/a

LOG OF BOREHOLE 4

Terraprobe

Project No.	: 7-21-0103-01	Client	: La Pue International Inc	Originated by	: KG
Date started	: November 22, 2021	Project	: Allendale Avenue and Robinson Street	Compiled by	: KG
Sheet No.	: 1 of 1	Location	: Niagara Falls, Ontario	Checked by	: TW

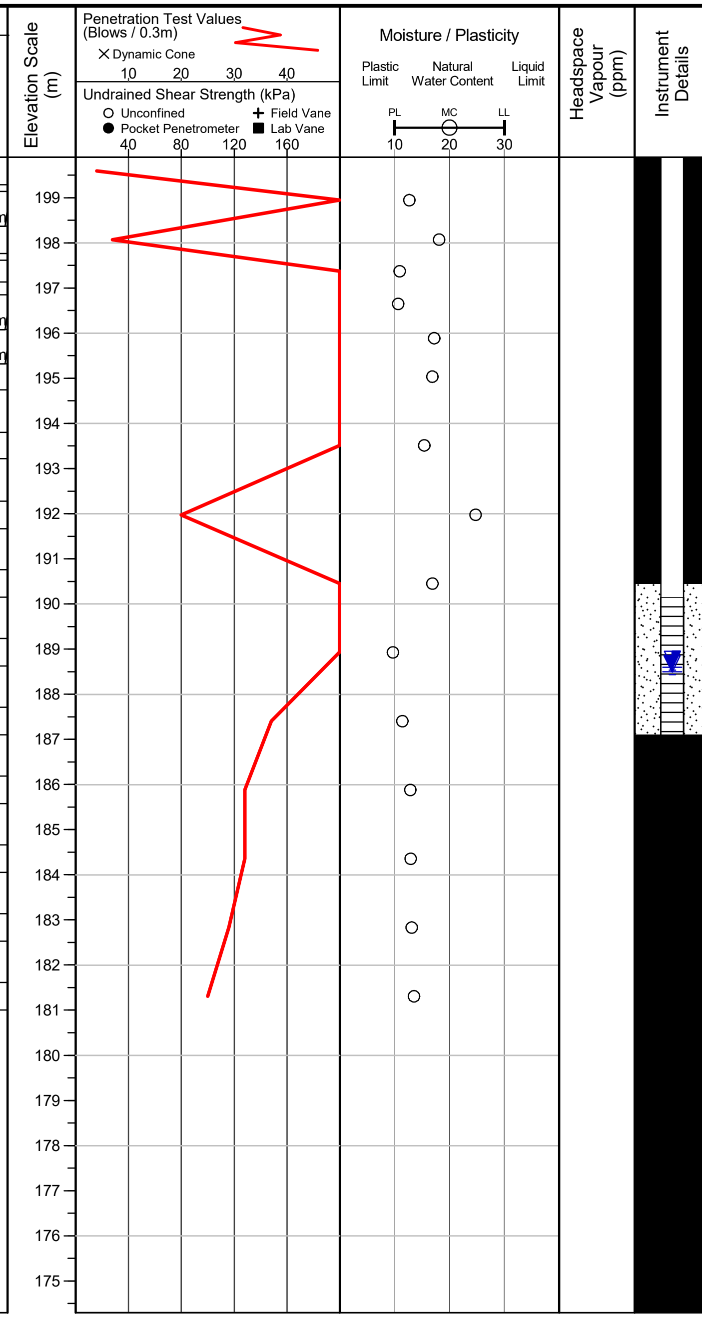
Position : E: 655810, N: 4772095 (UTM 17T)
Elevation Datum : Geodetic
Rig type : Truck-mounted
Drilling Method : Solid stem augers

SOIL PROFILE

Elev Depth (m)	Description
199.9	**GROUND SURFACE**
	100mm **ASPHALTIC CONCRETE**
198.4 / 1.5	**FILL**, granular base
197.6 / 2.3	**FILL**, silty sand and gravel, trace asphalt, rootlets, loose to very dense, brown
	FILL, clayey silt, trace sand, loose, brown
190.8 / 9.1	**SILT**, with sand, gravel, compact to very dense, brown (GLACIAL TILL)
181.0 / 18.9	**SILTY SAND**, trace clay, compact to very dense, brown
174.3 / 25.6	

SAMPLES

Number	Type	SPT 'N' Value
1	SS	4
2	SS	55 / 225mm
3	SS	7
4	SS	60
5	SS	76 / 250mm
6	SS	90 / 250mm
7	SS	96
8	SS	79
9	SS	20
10	SS	64
11	SS	57
12	SS	37
13	SS	32
14	SS	32
15	SS	29
16	SS	25



Lab Data and Comments

GRAIN SIZE DISTRIBUTION (%) (MIT)

GR SA SI CL

SS2 Analysis: PAH, PCB, VOC, PHC

SS3 Analysis: M&I

1 72 20 7

SS10 Analysis: VOC, PHC

END OF BOREHOLE
Refusal on inferred bedrock

Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Dec 3, 2021	11.4	188.5
Jan 7, 2022	11.3	188.6
Jan 24, 2022	12.5	187.4
Jan 29, 2022	dry	n/a
Feb 2, 2022	dry	n/a

file: 7-21-0103-01 - allendale ave.gpj

Terraprobe

LOG OF BOREHOLE 5

Project No. : 7-21-0103-01
Client : La Pue International Inc
Originated by : KG

Date started : November 23, 2021
Project : Allendale Avenue and Robinson Street
Compiled by : KG

Sheet No. : 1 of 1
Location : Niagara Falls, Ontario
Checked by : TW

Position : E: 655824, N: 4772154 (UTM 17T)
Elevation Datum : Geodetic

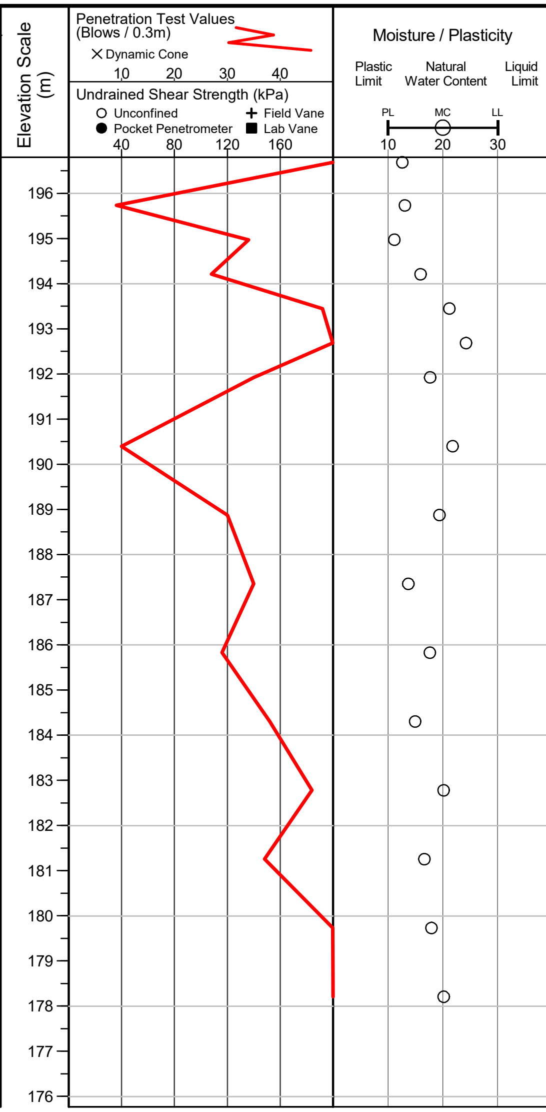
Rig type : Truck-mounted
Drilling Method : Solid stem augers

SOIL PROFILE

Elev / Depth (m)	Description
196.8	GROUND SURFACE
	FILL, gravel
	50mm **ASPHALTIC CONCRETE**
195.3 / 1.5	**FILL**, silt, trace sand, clay
	SANDY SILT, trace clay, compact to dense, brown ...spoon refusal on interpreted concrete slab at 2.7m depth
193.8 / 3.0	**SILTY SAND**, compact to very dense, brown ...trace gravel at 15.7m
177.9 / 18.9	
175.8 / 21.0	

SAMPLES

Number	Type	SPT 'N' Value
1	SS	50 / 75mm
2	SS	9
3	SS	34
4	SS	27
5	SS	48
6	SS	68
7	SS	35
8	SS	10
9	SS	30
10	SS	35
11	SS	29
12	SS	38
13	SS	46
14	SS	37
15	SS	55
16	SS	50



Headspace Vapour (ppm) | Instrument Details | Unstabilized Water Level

Lab Data and Comments (GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL)

SS2 Analysis: M&I, PAH, VOC, PHC

SS3 Analysis: M&I

END OF BOREHOLE
Refusal on inferred bedrock

Borehole was dry and open upon completion of drilling.

file: 7-21-0103-01 - allendale ave.gpj

Terraprobe

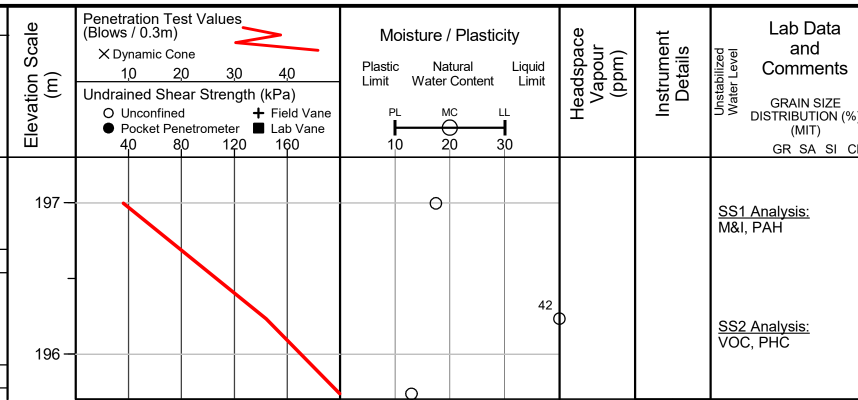
LOG OF BOREHOLE 6

Project No. : 7-21-0103-01	Client : La Pue International Inc	Originated by : KG
Date started : November 23, 2021	Project : Allendale Avenue and Robinson Street	Compiled by : KG
Sheet No. : 1 of 1	Location : Niagara Falls, Ontario	Checked by : TW

Position : E: 655843, N: 4772103 (UTM 17T)	Elevation Datum : Geodetic
Rig type : Truck-mounted	Drilling Method : Solid stem augers

SOIL PROFILE / SAMPLES

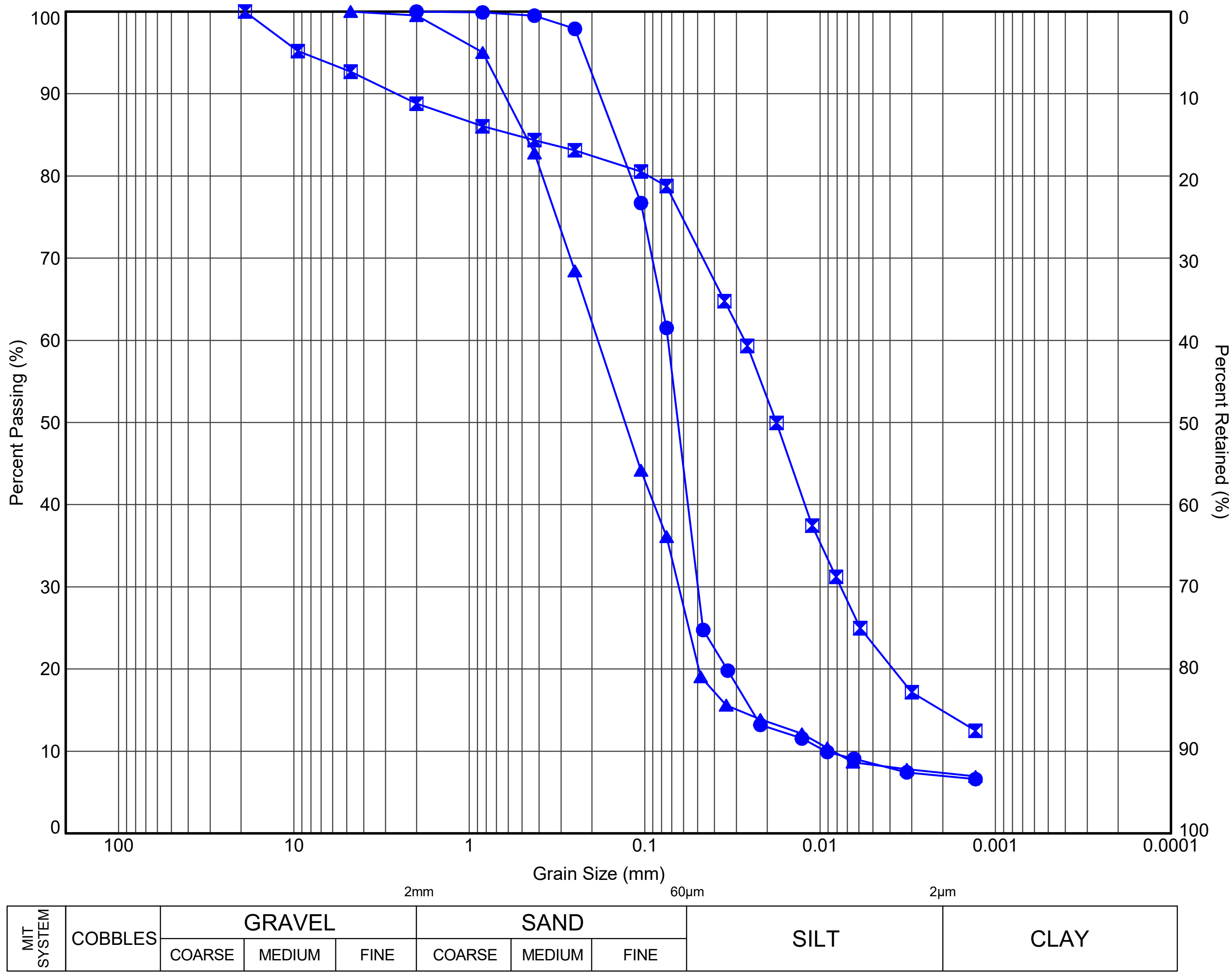
Elev Depth (m)	Description	Sample Number	Type	SPT 'N' Value	Lab Data and Comments
197.3	GROUND SURFACE				
	100mm **ASPHALTIC CONCRETE**				
197.0 / 0.3	**FILL**, granular base				
	FILL, clayey silt, trace sand, gravel, loose to very dense, dark brown to brown	1	SS	9	SS1 Analysis: M&I, PAH
		2	SS	36	SS2 Analysis: VOC, PHC
195.7 / 1.6		3	SS	50 / 75mm	



END OF BOREHOLE
Refusal (obstruction in the hole)

Borehole was dry and open upon completion of drilling.

file: 7-21-0103-01 - allendale ave.gpj



MIT SYSTEM

	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	*(Fines, %)*
●	**1**	SS15	17.1	182.3	0	56	37	7	
⊠	**2**	SS4	2.6	193.9	11	14	60	15	
▲	**4**	SS10	9.4	190.5	1	72	20	7	

Terraprobe

903 Barton Street, Unit 22, Stoney Creek ON L8E 5P5
(905) 643-7560

Title:

GRAIN SIZE DISTRIBUTION

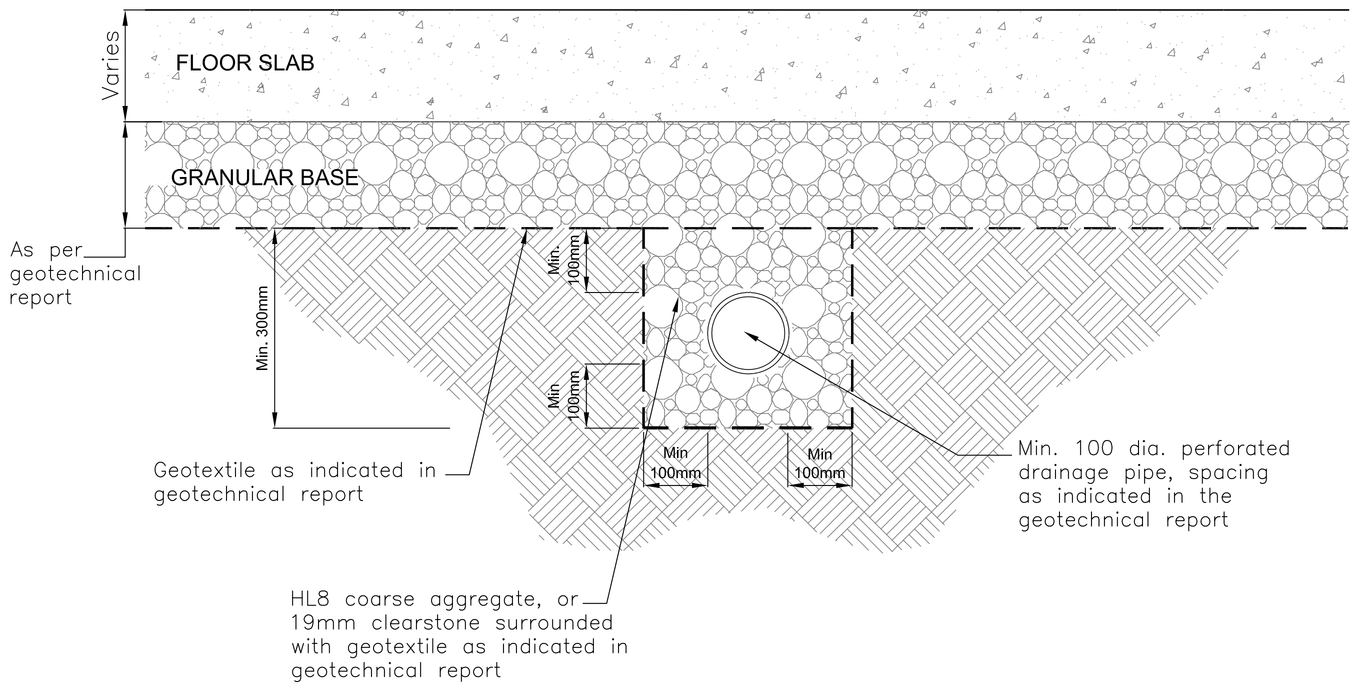
File No.:

7-21-0103-01

BASEMENT DRAINAGE DETAILS

APPENDIX B

Terraprobe Inc.

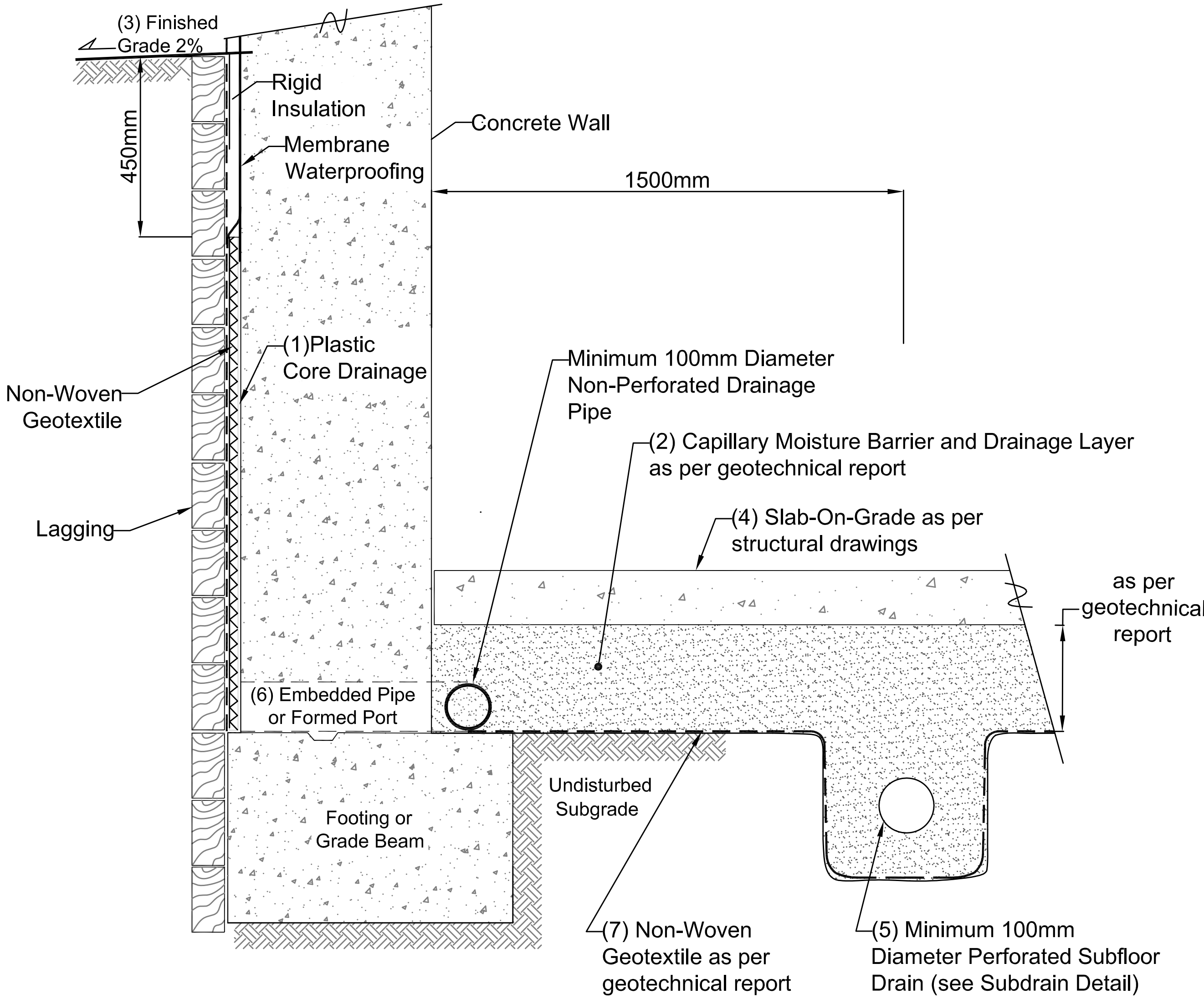


Schematic Only
Not to Scale

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

BASEMENT SUBDRAIN DETAIL



NOTES

1) Prefabricated drainage panels to consist of Terrafix - TERRADRAIN 200, Mirafi - Miradrain 6000, or approved equivalent. Panels should provide continuous cover with a minimum overlap of 300mm.

2) Capillary moisture barrier/drainage layer to consist of a minimum 200mm layer of 19mm clear stone (OPSS 1004), or as indicated in geotechnical report, compacted to a dense state. Upper 50mm can be replaced with Granular "A" (OPSS 1010) compacted to 98% SPMDD where vehicular traffic is required.

3) Exterior finished grade away from wall at a minimum grade of 2% for min. 1.2m.

4) Building floor slab-on-grade shall not be structurally connected to foundation wall or footing.

5) Subfloor drain invert to be a minimum of 300mm below underside of floor slab, to be set in parallel rows, one way, and at the spacing specified in the geotechnical report.

6) Embedded pipes/formed ports to be set a distance of maximum 3m on-centre. Each port to have a minimum cross-sectional area of 1500mm². Perimeter drainage must be collected and conveyed directly to the building sumps in non-perforated pipe.

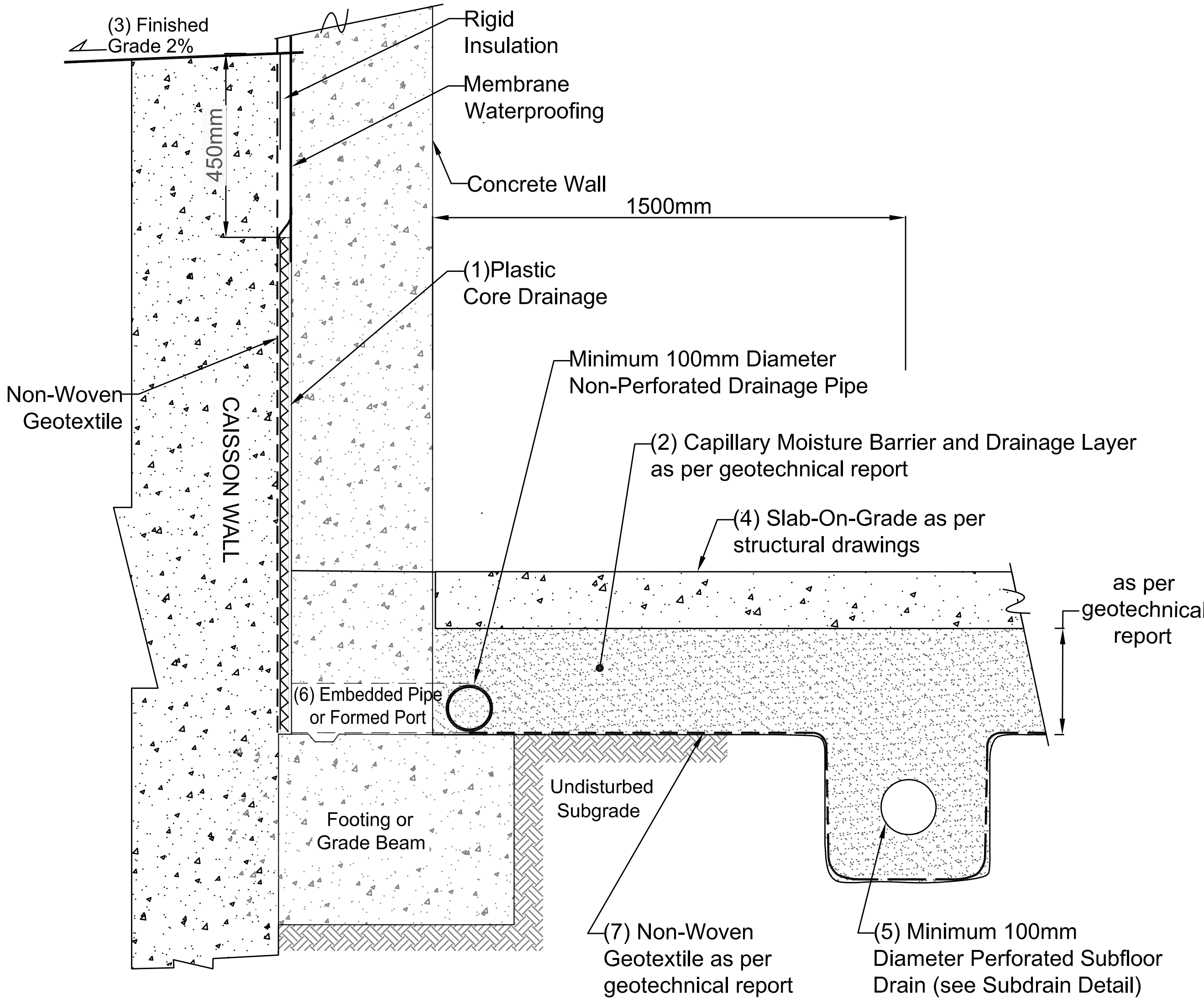
7) When the subgrade consists of a cohesionless soil, the subgrade must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).

N.T.S.

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

SCHEMATIC DRAINAGE DETAIL
SOLDIER PILE & LAGGING SHORING SYSTEM



NOTES

1) Prefabricated drainage panels to consist of Terrafix - TERRADRAIN 200, Mirafi - Miradrain 6000, or approved equivalent. Panels should provide continuous cover with a minimum overlap of 300mm.

2) Capillary moisture barrier/drainage layer to consist of a minimum 200mm layer of 19mm clear stone (OPSS 1004), or as indicated in geotechnical report, compacted to a dense state. Upper 50mm can be replaced with Granular "A" (OPSS 1010) compacted to 98% SPMDD where vehicular traffic is required.

3) Exterior finished grade away from wall at a minimum grade of 2% for min. 1.2m.

4) Building floor slab-on-grade shall not be structurally connected to foundation wall or footing.

5) Subfloor drain invert to be a minimum of 300mm below underside of floor slab, to be set in parallel rows, one way, and at the spacing specified in the geotechnical report.

6) Embedded pipes/formed ports to be set a distance of maximum 3m on-centre. Each port to have a minimum cross-sectional area of 1500mm². Perimeter drainage must be collected and conveyed directly to the building sumps in non-perforated pipe.

7) When the subgrade consists of a cohesionless soil, the subgrade must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).

N.T.S.

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

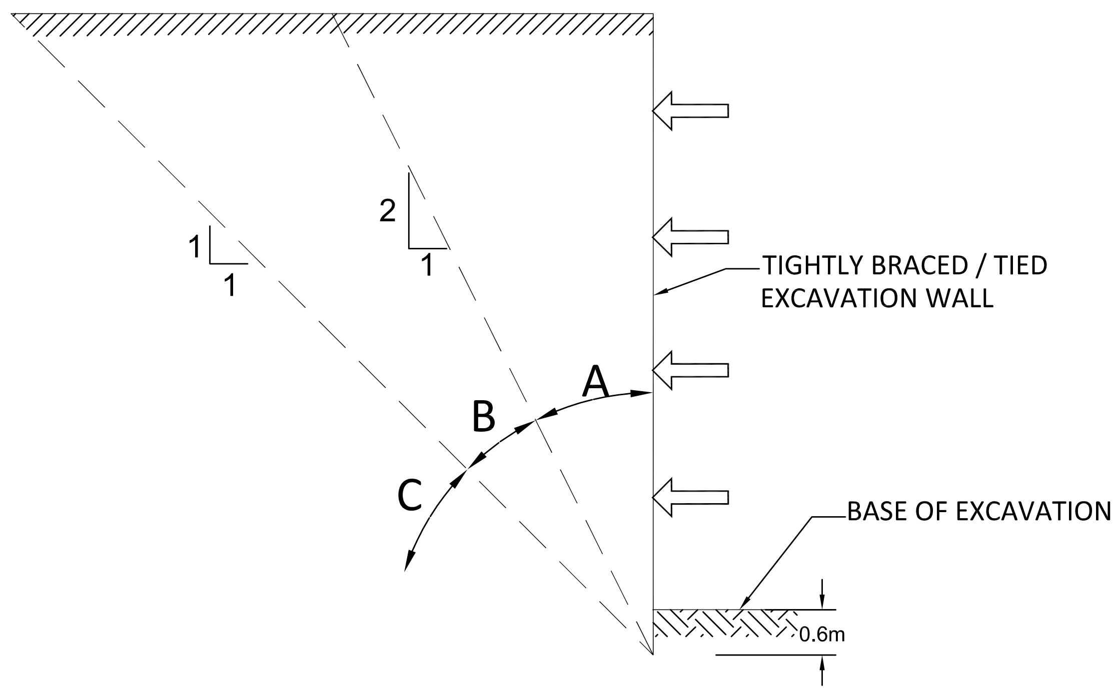
Title:

SCHEMATIC DRAINAGE DETAIL
CAISSON WALL SHORING SYSTEM

UNDERPINNING GUIDELINES

APPENDIX C

Terraprobe Inc.



Zone A: Foundations within this zone often require underpinning. Horizontal and vertical pressures on excavation wall of non-underpinned foundations must be considered.

Zone B: Foundation within this zone often do not require underpinning. Horizontal and vertical pressures on excavation wall of non-underpinned foundations must be considered.

Zone C: Foundations within this zone usually do not require underpinning.

<u>REFERENCE:</u>

User's Guide - NBC 2005 Structural Commentaries
(Part 4 of Division B) - Commentary K

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

GUIDELINES FOR UNDERPINNING SOILS