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Geotechnical Investigation

Proposed High-Rise Development
1200 Maritime Way
Ottawa, Ontario

Prepared For

Claridge Homes (Timberwalk) LP

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Report PG5281-1
Revision 1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Claridge Homes (Timberwalk) LP to conduct a geotechnical investigation for the proposed High-Rise Development to be located at 1200 Maritime Way in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

☐ Determine the subsoil and groundwater conditions at this site by means of boreholes.

☐ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

2.0 Proposed Development

Although drawings were not available during the preparation of this report, it is understood that the proposed development will consist of a high-rise structure with 2 levels of underground parking. It is also anticipated that the proposed building will be surrounded by asphalt-paved access lanes and parking areas with landscaped margins.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The initial field program for the investigation was conducted on March 26 and 27, 2020, and consisted of 4 boreholes advanced to a maximum depth of 9.8 m below the existing ground surface. A supplemental geotechnical investigation was conducted on July 2, 2020, and consisted of 22 probeholes advanced to a maximum depth of 16.2 m. The borehole and probehole locations were distributed in a manner to provide general coverage of the subject site. The approximate locations of the boreholes are shown on Drawing PG5281-1 - Test Hole Location Plan included in Appendix 2.

All boreholes were advanced using a track-mounted auger drill rig, which was operated by a two-person crew. The drilling procedure consisted of augering to the required depths at the selected locations, and sampling and testing the overburden. All probeholes were advanced using a track-mounted, pneumatic drill rig which was operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. All samples were visually inspected and initially classified on site and subsequently placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the auger and split spoon samples were recovered from the boreholes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

A Standard Penetration Test (SPT) was conducted at each borehole in conjunction with the recovery of the split spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at select borehole locations. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

The subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the subject site taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson with respect to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5281-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

3.4 Analytical Testing

One soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential for sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is generally vacant with the exception of an access road which runs parallel with Maritime Way through the northern portion of the site. Numerous fill piles approximately 1 to 2 m in height are also present across the southern portion of the site. The site is bordered by Maritime Way to the north, undeveloped land to the east and south, Kanata Avenue to the southwest and a multi-storey development to the northwest. The ground surface across the site is relatively level at approximate geodetic elevation of 95 to 96 m.

Based on available historical photographs , a pond was formerly located on the northern portion of the site and has since been filled in. Reference should be made to the aerial photographs in Figure 2 - Aerial Photograph - 1976 and Figure 3 - Aerial Photograph - 2017 which illustrate the former and present site conditions, respectively.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of an approximate 1.7 to 5.6 m thickness of fill underlying the existing ground surface, with the exception of BH 3 in which fill was not encountered. The thickness of the fill generally increased towards the northern portion of the subject site. The fill was observed to consist of a brown silty clay to silty sand with numerous cobbles and boulders. Within BH 1A, auger refusal was encountered in the fill at an approximate depth of 4 m.

A silty clay deposit was encountered underlying either a 100 to 200 mm thick topsoil layer or fill material. The silty clay deposit was observed to consist of a stiff to firm, brown silty clay, becoming a firm to stiff, grey silty clay at approximate depths ranging from 2.3 to 5.6 m below the existing ground surface.

Underlying the silty clay, a glacial till deposit was encountered at approximate depths of 9.1 to 11.5 m below the existing ground surface. The glacial till deposit was generally observed to consist of a compact to very dense, brown to grey silty sand with gravel, cobbles and boulders.

Based on the probeholes and practical refusal to augering/DCPTs in the boreholes, the inferred bedrock surface was encountered at depths ranging from approximately 3.7 m at the west end of the site, descending to depths of approximately 16.2 m on the east end of the site. It should also be noted that in PH-1 and PH-2, the upper 1 to 2 m of the bedrock was inferred to be weathered. Inferred bedrock surface contours on Drawing PG5281-2 - Bedrock Contour Plan.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of soil profile encountered at each test hole location.

Bedrock

Based on available geological mapping, the bedrock at the subject site consists of paragneiss or quartzite interlayered with paragneiss with a drift thickness of 3 to 10 m.

4.3 Groundwater

Groundwater levels were measured on April 3, 2020 in the standpipes installed in the completed boreholes. The observed groundwater levels are summarized in Table 1.

Table 1 - Summary of Groundwater Level Readings

Test Hole	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH 1	95.06	1.41	93.65	April 3, 2020
BH 2	95.52	0.72	94.80	April 3, 2020
BH 3	96.15	2.64	93.51	April 3, 2020
BH 4	95.50	1.18	94.32	April 3, 2020

Note: The ground surface elevations are referenced to a geodetic datum.

It should be noted that the groundwater levels could be influenced by surface water infiltrating the backfilled boreholes. Long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 4 to 5 m below ground surface. The recorded groundwater levels are provided on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

It should also be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed high-rise building. It is recommended that the proposed building generally be founded on conventional shallow footings bearing on the clean, surface sounded bedrock, or on lean concrete trenches which extend to the clean, surface sounded bedrock.

However, where the bedrock is located more than approximately 6 m below the anticipated bottom of excavation, such as on the eastern end of the site, lean concrete trenches would likely not be feasible. In these areas, a deep foundation system is recommended, such as end-bearing piles, which extend to the bedrock surface.

Conventional shallow footings bearing on the undisturbed, stiff to firm silty clay could be used to support portions of the underground parking structure which extend beyond the high-rise building footprint, provided that the bearing capacities provided below are sufficient for design requirements.

Due to the presence of a silty clay deposit, the proposed development will be subjected to grade raise restrictions.

The above and other considerations are further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas and other settlement sensitive structures. Existing construction debris should be entirely removed from within the perimeter of all buildings.

Fill Placement

Fill used for grading beneath the proposed building should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building should be compacted to at least 98% of the material’s standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

Compacted Granular Fill Working Platform (Pile Foundation)

Should a portion of the proposed building be supported on a driven pile foundation, the use of heavy equipment would be required to install the piles (i.e. pile driving crane). It is conventional practice to install a compacted granular fill layer, at a convenient elevation, to allow the equipment to access the site without getting stuck and causing significant disturbance.

A typical working platform could consist of 0.6 m of OPSS Granular B, Type II crushed stone which is placed and compacted to a minimum of 98% of its standard Proctor maximum dry density (SPMDD) in lifts not exceeding 300 mm in thickness.

Once the piles have been driven and cut off, the working platform can be regraded, and soil tracked in, or soil pumping up from the pile installation locations, can be bladed off and the surface can be topped up, if necessary, and recompacted to act as the substrate for further fill placement for the basement slab.

Lean Concrete Filled Trenches

Where the proposed footings are to be founded on bedrock which is located below the underside of footing elevation, zero-entry vertical trenches should be excavated to the clean, surface sounded bedrock, and backfilled with lean concrete to the founding elevation (minimum **17 MPa** 28-day compressive strength). Typically, the excavation side walls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing (strip and pad footings) at the base of the excavation. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock. Once the trench excavation is approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on clean, surface sounded bedrock, or on lean concrete trenches placed directly over clean, surface sounded bedrock, can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,500 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistence at ULS.

Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Pad footings, up to 5 m wide, and strip footings up to 3 m wide, placed on an undisturbed, stiff to firm silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

Footings placed on a soil bearing surface and designed using the bearing resistance values at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm respectively.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to silty clay above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil. Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending horizontally and vertically from the footing perimeter at a minimum of 1H:6V (or shallower) passes through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete.

Pile Foundation

A deep foundation system driven to refusal in the bedrock may be required for portions of the proposed building where lean concrete trenches are not feasible due to the bedrock depth. For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance values at SLS and ULS are given in Table 2. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 2 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	925	1110	6	27
245	11	1050	1260	6	31
245	13	1200	1440	6	35

The minimum centre-to-centre pile spacing is 2.5 times the pile diameter. The closer the piles are spaced, however, the more potential that the driving of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously driven piles, are checked as part of the field review of the pile driving operations.

Prior to the commencement of production pile driving, a limited number of indicator piles should be installed across the site. It is recommended that each indicator pile be dynamically load tested to evaluate pile stresses, hammer efficiency, pile load transfer, and end-of-driving criteria for end-bearing in the bedrock.

Buildings founded on piles driven to refusal in the bedrock will have negligible post-construction settlement.

Foundation Lateral Load Resistance

Lateral loads on the foundations can be resisted using passive resistance on the sides of the foundations. For Limit States Design, the resistance factor to be applied to the ultimate lateral resistance, including passive pressure, is 0.50. The total lateral resistance will be comprised of the individual contributions from up to several material layers, as follows.

Geotechnical parameters for the native silty clay and for typical backfill materials compacted to 95% of MPMDD in 300 mm lift thicknesses are provided in Table 3, below, along with the associated earth pressure coefficients for horizontal resistance calculated for footings under lateral loads or deadman anchors. Friction factors between concrete and the various subgrade materials are also provided in Table 3, where normal loads allow them to be used.

Where granular soils and/or granular backfill materials are present, the passive pressure can be calculated using a triangular distribution equal to $K_P \cdot \gamma \cdot H$ where:

K_P = factored passive earth pressure coefficient of the applicable retained soil, 1.5
γ = unit weight of the fill of the applicable retained soil (kN/m^3)
H = height of the equivalent wall or footing side (m)

Note that for cases where the depth to the top of the structure pushing against the soil does not exceed 50% of the depth to the base of the structure, the effective value of H in the above noted relationship will be the overall depth to the base of the structure. There will also be “edge effects” where the effective width of soil providing the resistance can be increased by 50% of the effective depth on each side of the pushing structural component.

Note that where the foundation extends below the groundwater level, the effective unit weight should be utilized for the saturated portion of the soil or fill.

Should additional passive resistance be require, the horizontal component of the axial resistance of battered piles (up to 1H:3V inclination), or anchors can be used in the building foundation design.

Foundation Uplift Resistance

Uplift forces on the proposed foundations can be resisted using the dead weight of the concrete foundations, the weight of the materials overlying the foundations, and the submerged weight of the piles, where utilized. Unit weights of materials are provided in Table 3.

For soil above the groundwater level, calculate using the “drained” unit weight and below groundwater level use the “effective” unit weight. Backfilled excavations in low permeability soils can be expected to fill with water and the use of the effective unit weights would be prudent if drainage of the anchor footings is not provided.

As noted above, piles would generally be located below the groundwater level, so the submerged, or effective, weight of the pile will be available to contribute to the uplift resistance, if required. Considering that this is a reliable uplift resistance, and is really counteracting a dead load, it is our opinion that a resistance factor of 0.9 is applicable for the ULS weight component.

Should the pile uplift resistance capacities be insufficient for the foundation uplift loads, rock anchors should be utilized. This is discussed further in Section 5.8.

A sieve analysis and standard Proctor test should be completed on each of the fill materials proposed to obtain an accurate soil density to be expected, so the applicable unit weights can be estimated.

Table 3 - Geotechnical Parameters for Uplift and Lateral Resistance Design

Material Description	Unit Weight (kN/m^3)		Internal Friction Angle (°) φ'	Friction Factor, tan δ	Earth Pressure Coefficients		
	Drained γ_{dr}	Effective γ'			Active K_A	At-Rest K_O	Passive K_P
OPSS Granular A (Crushed Stone)	22.0	13.7	38	0.60	0.22	0.36	8.8
OPSS Granular B, Type II (Well-Graded Sand-Gravel)	21.5	13.4	36	0.55	0.26	0.41	7.5
In Situ Silty Clay	17.0	10.0	33	0.40	0.30	0.45	3.4

Notes:
❑ Properties for fill materials are for condition of 98% of standard Proctor maximum dry density.
❑ The earth pressure coefficients provided are for horizontal backfill profile.
❑ Passive pressure coefficients incorporate wall friction of 0.5 φ'.

Permissible Grade Raise Recommendations

Due to the presence of the silty clay deposit, a permissible grade raise restriction of **2 m** is recommended for grading at the subject site.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

A seismic shear wave velocity test was completed at the subject site to accurately determine the applicable seismic site classification for the proposed building based on Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012. The shear wave velocity test was completed by Paterson personnel. Two seismic shear wave velocity profiles from the on site testing are presented in Appendix 2.

Field Program

The seismic array testing location was placed directly on the northern end of the site in an east-west direction as presented on Drawing PG5281-1 - Test Hole Location Plan in Appendix 2. Paterson field personnel placed 24 horizontal 4.5 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 3 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12 pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (8) times at each shot location to improve signal to noise ratio. The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations are located at 25, 4.5 and 3 m away from the first geophone, 3, 4.5, and 20 m away from the last geophone, and at the centre of the seismic array.

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves. The interpretation is repeated at each shot location to provide an average shear wave velocity, Vs_{30}, of the upper 30 m profile, immediately below the building's foundation. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Site Class for Footings supported on Lean Concrete which extends to Bedrock

The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the OBC 2012, as presented b

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\left(\frac{(Depth_{Layer1}(m)}{Vs_{Layer1}(m/s)} + \frac{Depth_{Layer2}(m)}{Vs_{Layer2}(m/s)}\right)}$$

$$V_{s30} = \frac{30m}{\left(\frac{30m}{2{,}266m/s}\right)}$$

$$V_{s30} = 2{,}266m/s$$

Based on the results of the seismic testing, the average shear wave velocity, Vs_{30}, for footings supported on lean concrete trenches which extend to the bedrock surface is **2,266 m/s**. Therefore, a **Site Class A** is applicable for design of the proposed building in this case, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

Site Class for End-Bearing Piles

The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the OBC 2012, as presented below.

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\left(\frac{(Depth_{Layer1}(m)}{Vs_{Layer1}(m/s)} + \frac{Depth_{Layer2}(m)}{Vs_{Layer2}(m/s)}\right)}$$

$$V_{s30} = \frac{30m}{\left(\frac{9m}{172m/s} + \frac{21m}{2{,}266m/s}\right)}$$

$$V_{s30} = 487m/s$$

Based on the results of the seismic testing, should the proposed building foundation include end-bearing piles which extend to the bedrock surface, the average shear wave velocity, Vs_{30}, would be **487 m/s**. Therefore, a **Site Class C** would be applicable for design of the proposed building in this case, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Floor Slab

With the removal of all topsoil and deleterious fill from within the footprint of the proposed building, the native soil surface or existing fill will be considered an acceptable subgrade on which to commence backfilling for floor slab construction.

Where the subgrade consists of existing fill, a vibratory drum roller should complete several passes over the subgrade surface as a proof-rolling program. Any poor performing areas should be removed and reinstated with an engineered fill, such as Granular B Type II.

It is recommended that the upper 200 mm of sub-floor fill consists of 19 mm clear crushed stone. Further, a sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided underlying the basement slab. This is discussed further in Subsection 6.1.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the proposed structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

Where undrained conditions are anticipated (i.e. below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight for undrained conditions.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall.

The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45 - a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Pavement Structure

Car only parking areas, access lanes and heavy truck parking areas are anticipated at this site. The proposed pavement structures are shown in Tables 4 and 5.

Table 4 - Recommended Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
50	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
300	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 5 - Recommended Pavement Structure - Access Lanes and Ramp

Thickness (mm)	Material Description
40	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
50	Binder Course - HL-8 or Superpave 19.0 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
450	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or fill	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment.

5.8 Rock Anchor Design

Overview of Anchor Features

The geotechnical design of grouted rock anchors is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or a 60 to 90 degree pullout of rock cone with the apex of the cone near the middle of the bonded length of the anchor. Interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each individual anchor.

A third failure mode of shear failure along the grout/steel interface should be reviewed by the structural engineer to ensure all typical failure modes have been reviewed.

Centre-to-centre spacing between anchors should be at least four times the anchor hole diameter and greater than 1/5 of the total anchor length (minimum of 1.2 m) to lower the group influence effects. Anchors in close proximity to each other are recommended to be grouted at the same time to ensure any fractures or voids are completely in-filled and grout does not flow from one hole to an adjacent empty one.

Each anchor should be provided with a bonded length at the base of the anchor which will provide the anchor capacity, as well an unbonded length between the rock surface and the top of the bonded length.

Permanent anchors should be provided with corrosion protection. As a minimum, the entire drill hole should be filled with cementious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break, with the sleeve filled with grout or a corrosion inhibiting mastic.

Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems International or Williams Form Engineering Corp. Recognizing the importance of the anchors for the long term performance of the foundation of the proposed building, the rock anchors for this project are recommended to be provided with double corrosion protection.

Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength(UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.4, can be calculated. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing bedrock information, a **Rock Mass Rating (RMR) of 65** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.821 and 0.00293**, respectively.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 6 below.

Table 6 - Parameters used in Rock Anchor Review

Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Paragneiss bedrock Hoek and Brown parameters	65 m=0.821 and s=0.00293
Unconfined compressive strength - Paragneiss	50 MPa
Unit weight - Submerged Bedrock	15.2 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm and 125 mm diameter hole are provided in Table 7. The factored tensile resistance values given in Table 7 are based on a single anchor with no group influence effects. A detailed analysis, including potential group influence effects, could be provided once loading for the proposed building is determined.

Table 7 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	2.0	0.8	2.8	450
	2.6	1.0	3.6	600
	3.2	1.2	4.4	750
	4.5	2.0	6.5	1000
125	1.6	0.6	2.2	600
	2.0	1.0	3.0	750
	2.6	1.4	4.0	1000
	3.2	1.8	5.0	1250

Other considerations

The anchor drill holes should be within 1.5 to 2 times the rock anchor tendon diameter, inspected by geotechnical personnel and should be flushed clean prior to grouting. A tremie tube is recommended to place grout from the bottom of the anchor holes. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day that grout is prepared.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is expected that the foundation walls will be blind poured against a drainage system and waterproofing system fastened to the shoring system. Waterproofing of the foundation walls is recommended and the membrane is to be installed from 4 m below finished grade down the foundation walls to the bottom of foundation.

It is also recommended that a composite drainage system, such as Delta Drain 6000 or equivalent, be installed between the waterproofing membrane and the foundation wall, and extend from the exterior finished grade to the founding elevation. The purpose of the composite drainage system is to direct any water infiltration resulting from a breach of the waterproofing membrane to the building sump pit. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the foundation wall at the footing interface to allow the infiltration of water to flow to an interior perimeter sub-slab drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

It is also recommended that a perimeter foundation drainage system be provided at a depth of 2 m for the proposed structure to control any surficial groundwater. The perimeter drainage pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Sub-slab Drainage

Sub-slab drainage will be required to control water infiltration. For preliminary design purposes, we recommend that 100 or 150 mm diameter perforated pipes be placed at approximate 6 m centres. The spacing of the sub-slab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

Where sufficient space is available, backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose. A waterproofing system should be provided for any elevator pits (pit bottom and walls).

6.2 Protection of Footings Against Frost Action

Perimeter foundations of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m thick soil cover, or a minimum of 0.6 m of soil cover in conjunction with adequate foundation insulation, should be provided.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the heated structure and require additional protection, such as soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation.

However, the foundations are expected to have sufficient frost protection due to the founding depth. Unheaded structures such as the access ramp may require insulation against the deleterious effects of frost action.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be excavated at acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

It is anticipated that temporary shoring will be required to support the overburden soils. The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes. Furthermore, the design of the temporary shoring system should take into consideration a full hydrostatic condition which can occur during significant precipitation events.

The temporary shoring system may consist of a soldier pile and lagging system or steel sheet piles which could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist to failure, if required, by means of rock bolts or extending the piles into the bedrock through pre-augured holes, if a soldier pile and lagging system is the preferred method.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. The temporary shoring system design should also consider that trenches excavated to the bedrock for lean concrete placement may be excavated in close proximity to the temporary shoring system. The earth pressures acting on the shoring system may be calculated using the following parameters.

Table 8 - Soil Parameters

Parameters	Values
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	21
Submerged Unit Weight (γ), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

6.4 Pipe Bedding and Backfill

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 98% of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the moist (not wet) brown silty clay above the cover material if the operations are carried out in dry weather conditions.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.5 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material standard Proctor maximum dry density.

6.5 Groundwater Control

Due to the relatively impervious nature of the silty clay, it is anticipated that groundwater infiltration into the excavations should be low and controllable using open sumps. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

A temporary Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater which breaches the building's perimeter groundwater infiltration control system will be directed to the proposed building's sump pit. Provided the proposed groundwater infiltration control system and the tanked system are properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be very low to negligible (less than 2,000 L/day).

Impacts on Neighbouring Properties

As the proposed multi-storey building will be founded below the long term groundwater level, a groundwater infiltration control system has been recommended to mitigate the effects of groundwater infiltration. Any long term dewatering of the site will be minimal and should have no adverse effects to the surrounding buildings or structures. The short term dewatering during the excavation program will be managed by the excavation contractor, as discussed above.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The subsoil conditions at this site mostly consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters, tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches.

6.7 Corrosion Potential and Sulphate

One (1) soil sample was submitted for analytical testing. The analytical test results of the soil sample indicate that the sulphate content is less than 0.01%. These results along with the chloride and pH value are indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a moderate to slightly aggressive corrosive environment.

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

☐ Complete a supplemental investigation to delineate bedrock surface for detailed foundation design purposes.

☐ Review of the grading plan from a geotechnical perspective.

☐ Review of the Contractor's design of the temporary shoring system (if required)

☐ Observation of all bearing surfaces prior to the placement of concrete.

☐ Inspection of the foundation waterproofing and all foundation drainage systems.

☐ Sampling and testing of the concrete and fill materials used.

☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

☐ Observation of all subgrades prior to backfilling.

☐ Field density tests to determine the level of compaction achieved.

☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, we request immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine its suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Claridge Homes (Timberwalk) LP or their agents is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Scott Dennis

Scott S. Dennis, P.Eng.

LICENSED PROFESSIONAL ENGINEER
July 16, 2020
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PROVINCE OF ONTARIO

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Report Distribution

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

TABLE 9 - PROBEHOLE SUMMARY

ANALYTICAL TESTING RESULTS

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Development - 1200 Maritime Way
Ottawa, Ottawa

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE March 26, 2020

FILE NO. PG5281

HOLE NO. BH 1A

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20, 40, 60, 80)	Piezometer Construction
GROUND SURFACE						0	95.06		
FILL: Brown silty clay with gravel and cobbles, trace sand		AU	1						
		SS	2	33	55	1	94.06		
		SS	3	8	8	2	93.06		
		SS	4	17	22				
		SS	5	4	27	3	92.06		
3.81									
FILL: Very dense, brown silty sand with gravel, cobbles, boulders, trace clay 3.96		SS	6		50+				
End of Borehole Practical refusal to augering at 3.96m depth									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Development - 1200 Maritime Way
Ottawa, Ottawa

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE March 26, 2020

FILE NO. **PG5281**

HOLE NO. **BH 1B**

SOIL DESCRIPTION

GROUND SURFACE

FILL: Brown silty sand with gravel, cobbles and boulders, trace clay

- cored through boulder from 2.3 to 3.7m depth

- cored through boulder from 4.6 to 5.2m depth

5.64

Very stiff to stiff, grey **SILTY CLAY**

9.14

GLACIAL TILL: Compact, grey silty sand with gravel, cobbles and boulders

9.83

End of Borehole

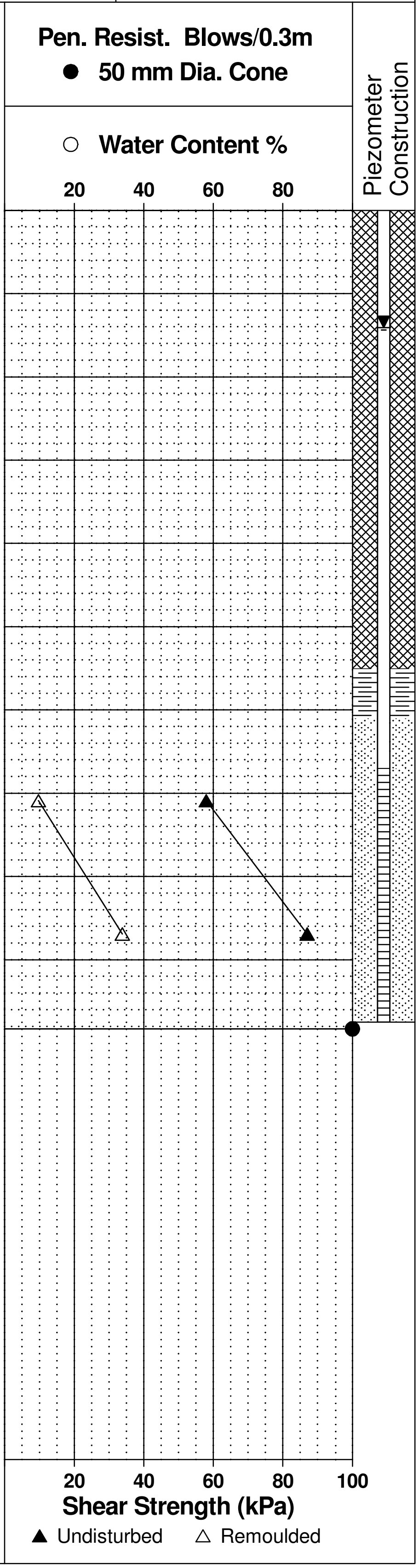
Dynamic Cone Penetration Test commenced at 9.75m depth. Practical DCPT refusal at 9.83m depth.

(GWL @ 1.41m - April 3, 2020)

SAMPLE

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	33	19
SS	3	25	14
RC	1	100	
RC	2	33	
SS	4	33	13
SS	5	100	50+
RC	3	42	
SS	6	100	50
SS	7		17

DEPTH (m)	ELEV. (m)
0	95.06
1	94.06
2	93.06
3	92.06
4	91.06
5	90.06
6	89.06
7	88.06
8	87.06
9	86.06



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Development - 1200 Maritime Way
Ottawa, Ottawa

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

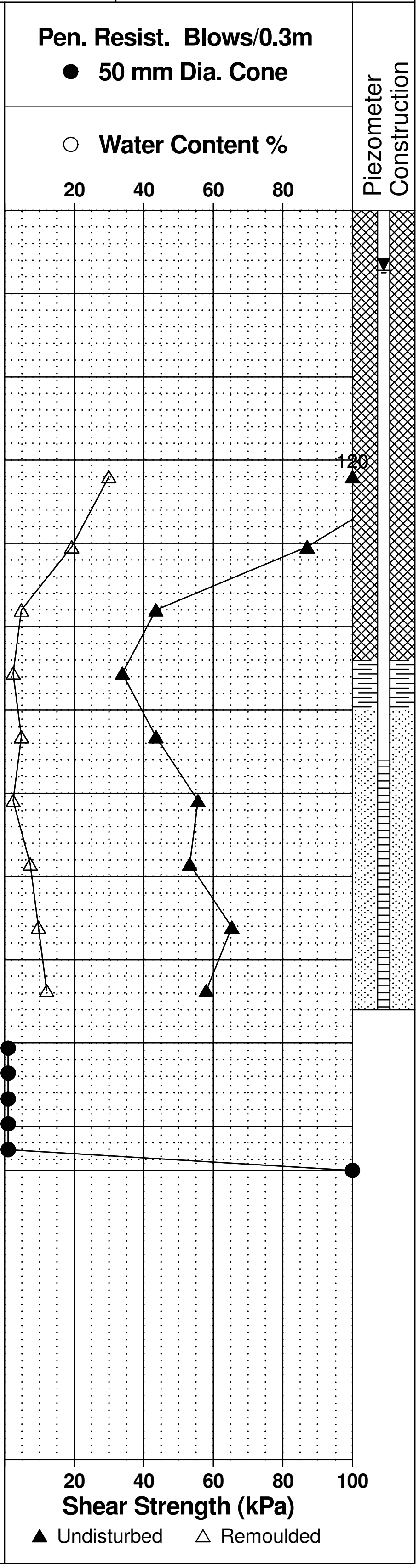
DATE March 26, 2020

FILE NO. PG5281

HOLE NO. BH 2

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty clay, trace sand, gravel, cobbles and boulders	AU	1		
	SS	2	42	9
1.68				
Very stiff to stiff, brown **SILTY CLAY** - firm to stiff and grey by 4.6m depth	SS	3	100	3
	SS	4	25	10
9.60				
Dynamic Cone Penetration Test commenced at 9.60m depth				
11.53				
End of Borehole Practical DCPT refusal at 11.53m depth. (GWL @ 0.72m - April 3, 2020)				

DEPTH (m)	ELEV. (m)
0	95.52
1	94.52
2	93.52
3	92.52
4	91.52
5	90.52
6	89.52
7	88.52
8	87.52
9	86.52
10	85.52
11	84.52



patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Development - 1200 Maritime Way
Ottawa, Ottawa

DATUM Geodetic

FILE NO. **PG5281**

REMARKS

HOLE NO. **BH 3**

BORINGS BY Track-Mount Power Auger

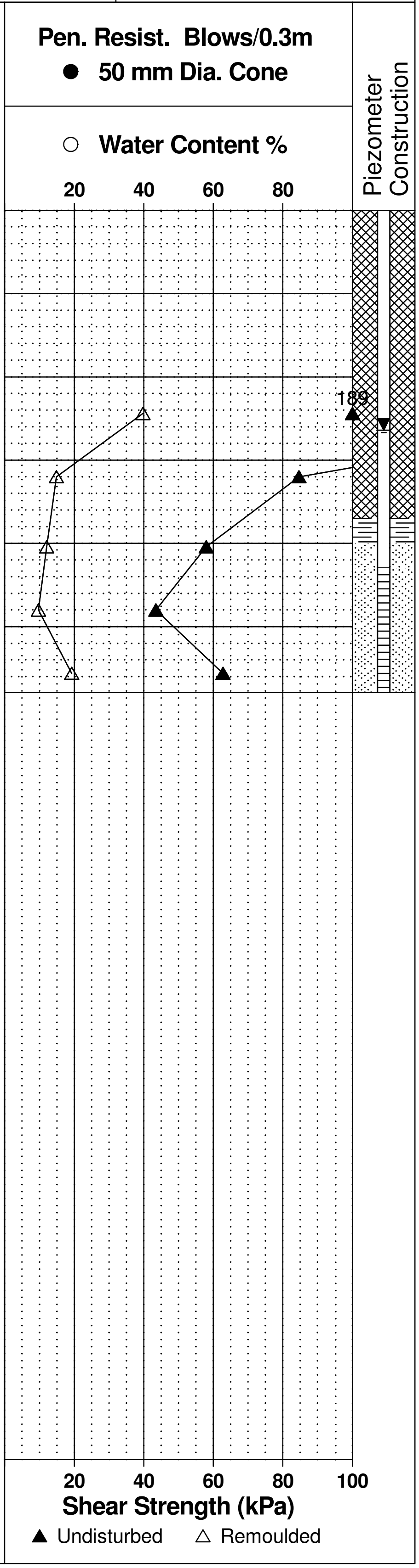
DATE March 26, 2020

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	96.15
TOPSOIL 0.18		AU	1				
		SS	2	100	4	1	95.15
		SS	3	100	5	2	94.15
Firm to stiff, brown **SILTY CLAY**						3	93.15
- grey by 3.8m depth						4	92.15
						5	91.15
5.79							

End of Borehole

Practical refusal to augering at 5.79m depth.

(GWL @ 2.64m - April 3, 2020)



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. High-Rise Development - 1200 Maritime Way
Ottawa, Ottawa

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE March 27, 2020

FILE NO. PG5281

HOLE NO. BH 4

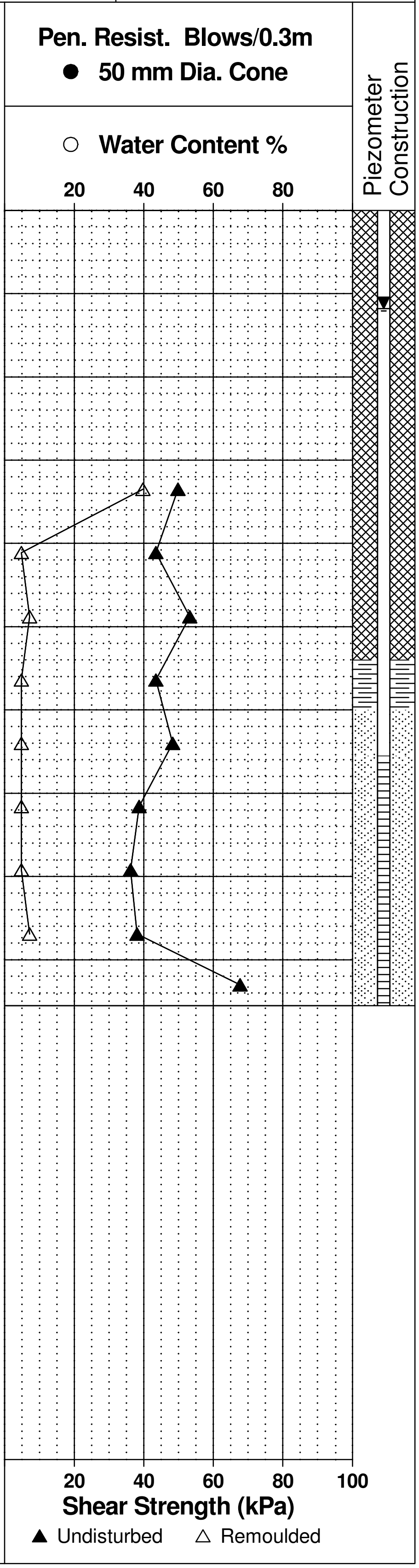
SOIL DESCRIPTION	TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.10	AU	1		
FILL: Grey silty clay with gravel, cobbles and boulders - cored through boulder from 0.7 to 2.3m depth 2.30	RC	1		
Firm to stiff, grey **SILTY CLAY** 9.45	SS	2	100	2
GLACIAL TILL: Very dense, grey silty sand with gravel, cobbles and boulders 9.55	SS	3	100	50+

End of Borehole

Practical refusal to augering at 9.55m depth.

(GWL @ 1.18m - April 3, 2020)

DEPTH (m)	ELEV. (m)
0	95.50
1	94.50
2	93.50
3	92.50
4	91.50
5	90.50
6	89.50
7	88.50
8	87.50
9	86.50



SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

Relative Density	‘N’ Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

Consistency	Undrained Shear Strength (kPa)	‘N’ Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NXL size core. However, it can be used on smaller core sizes, such as BX, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic limit, % (water content above which soil behaves plastically)
PI - Plasticity index, % (difference between LL and PL)

Dxx - Grain size which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay
(more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

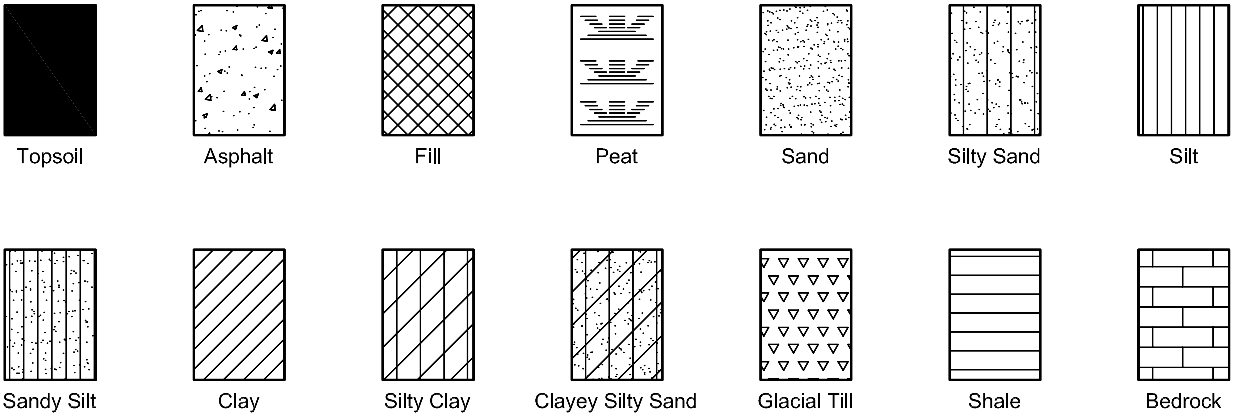
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

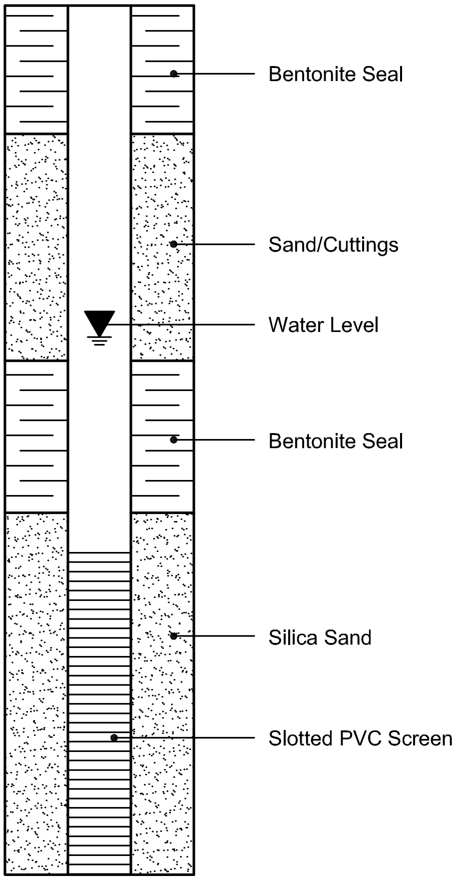
SYMBOLS AND TERMS (continued)

STRATA PLOT



MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION

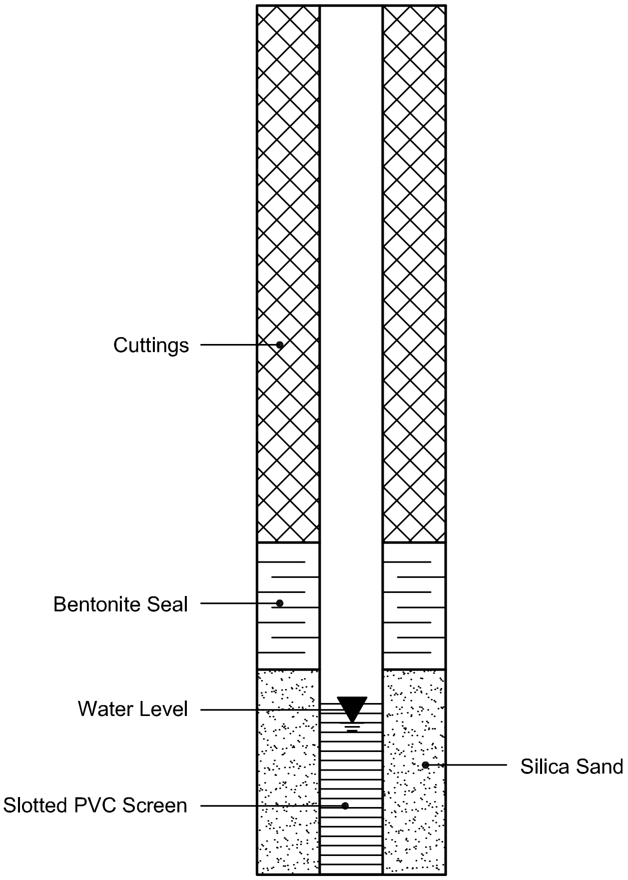


TABLE 9
Probehole Summary

Project No. PG5281
Location: 1200 Maritime Way, Ottawa, ON
Date: July 2, 2020

Probehole #	Surface Elevation (m)	Overburden Thickness (m)	Inferred Bedrock Elevation (m)	Bottom of PH Elevation (m)	Comments
PH-1	95.48	9.75	85.73	82.35	Bedrock was weathered to appx. EL. 83.90 m
PH-2	95.06	12.80	82.26	80.10	Bedrock was weathered to appx. EL. 81.34 m
PH-3	95.24	14.17	81.07	79.82	
PH-4	95.01	10.67	84.34	82.98	
PH-5	95.32	8.53	86.78	85.25	
PH-6	95.23	9.53	85.69	84.10	
PH-7	95.03	9.75	85.27	83.86	
PH-8	94.78	14.33	80.45	78.95	
PH-9	94.76	3.66	91.10	88.41	
PH-10	94.92	6.40	88.52	87.10	
PH-11	95.20	10.67	84.53	83.17	
PH-12	95.66	9.75	85.90	84.38	
PH-13	94.83	11.58	83.24	81.82	
PH-14	94.83	16.15	78.67	77.19	
PH-15	96.72	8.84	87.88	86.21	
PH-16	95.45	8.84	86.61	85.20	
PH-17	95.13	14.94	80.19	78.74	
PH-18	96.87	8.23	88.64	87.14	
PH-19	97.18	9.14	88.03	86.68	
PH-20	95.57	9.45	86.12	84.54	
PH-21	95.65	9.75	85.90	84.61	
PH-22	96.00	11.58	84.42	82.76	

	Client ID:	BH4-SS3	-	-	-
	Sample Date:	27-Mar-20 15:30	-	-	-
	Sample ID:	2019127-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	71.9	-	-	-
General Inorganics					
pH	0.05 pH Units	7.71 [1]	-	-	-
Resistivity	0.10 Ohm.m	19.5	-	-	-
Anions					
Chloride	5 ug/g dry	57 [1]	-	-	-
Sulphate	5 ug/g dry	318 [1]	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURE 2 - AERIAL PHOTOGRAPH - 1976

FIGURE 3 - AERIAL PHOTOGRAPH - 2017

FIGURES 4 & 5 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG5281-1 - TEST HOLE LOCATION PLAN

DRAWING PG5281-2 - BEDROCK CONTOUR PLAN

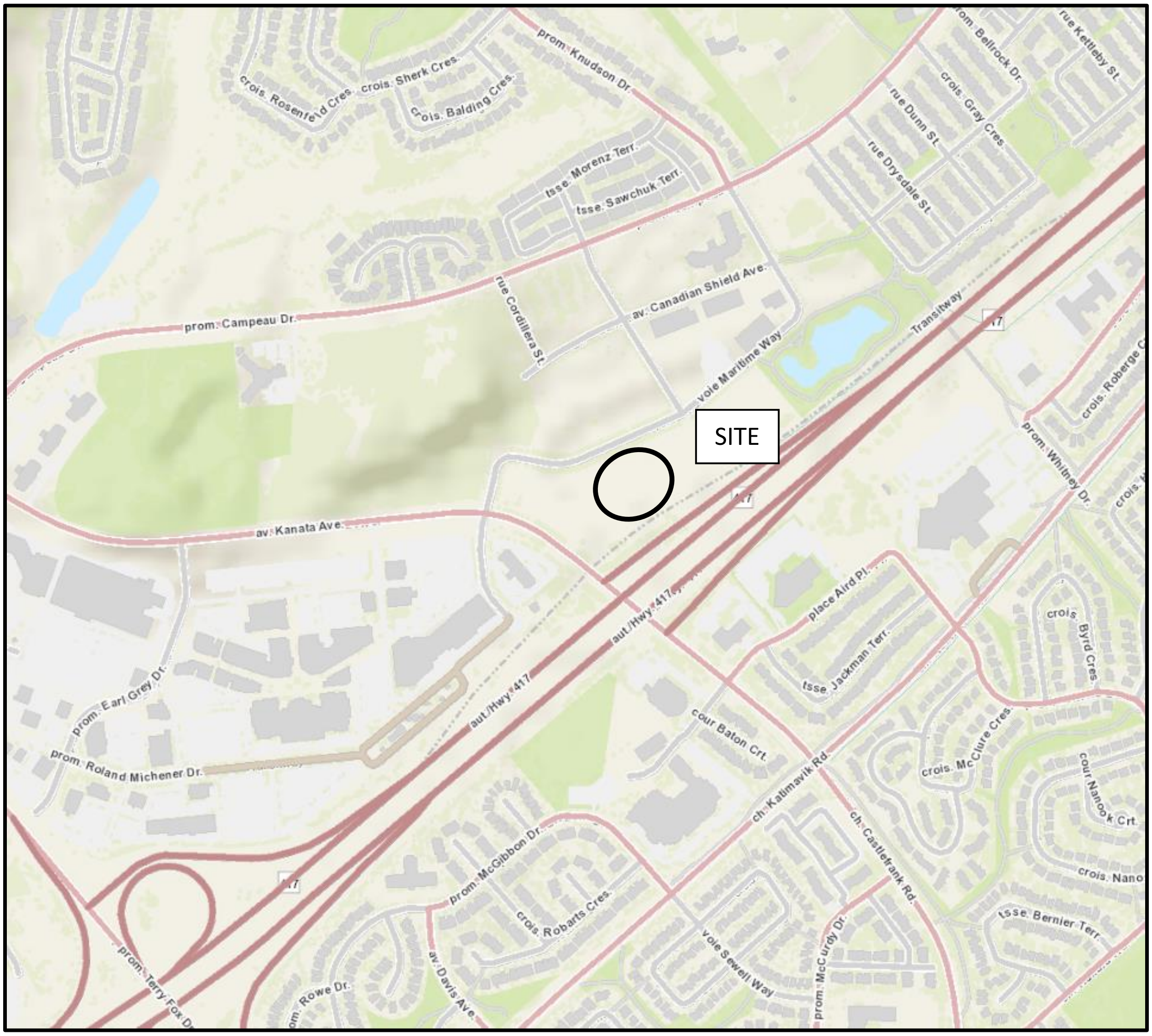


FIGURE 1

KEY PLAN

FIGURE 2

Aerial Photograph - 1976

FIGURE 3

Aerial Photograph - 2017

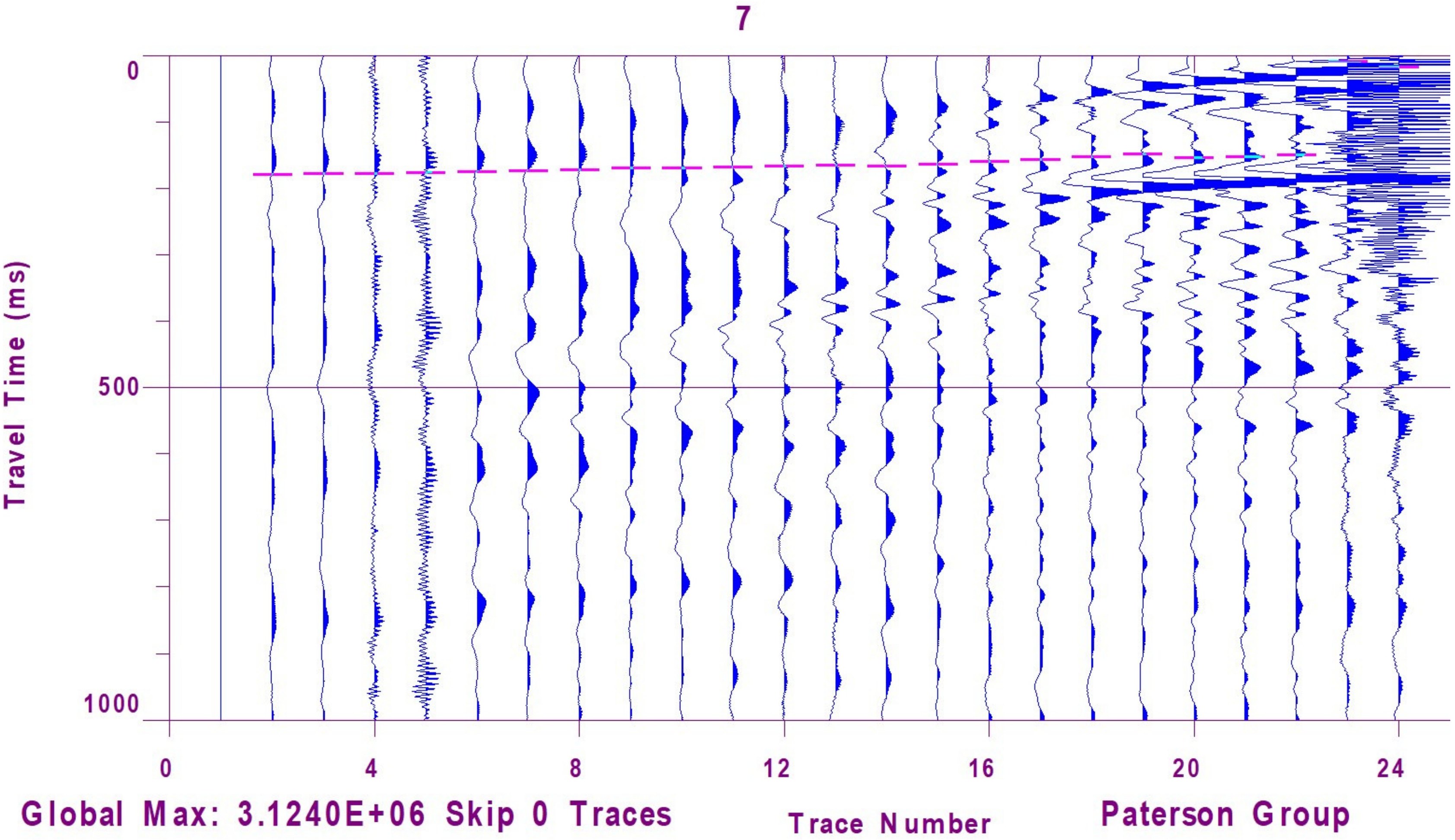


FIGURE 4 – Shear Wave Velocity Profile at Shot Location +72.0 m

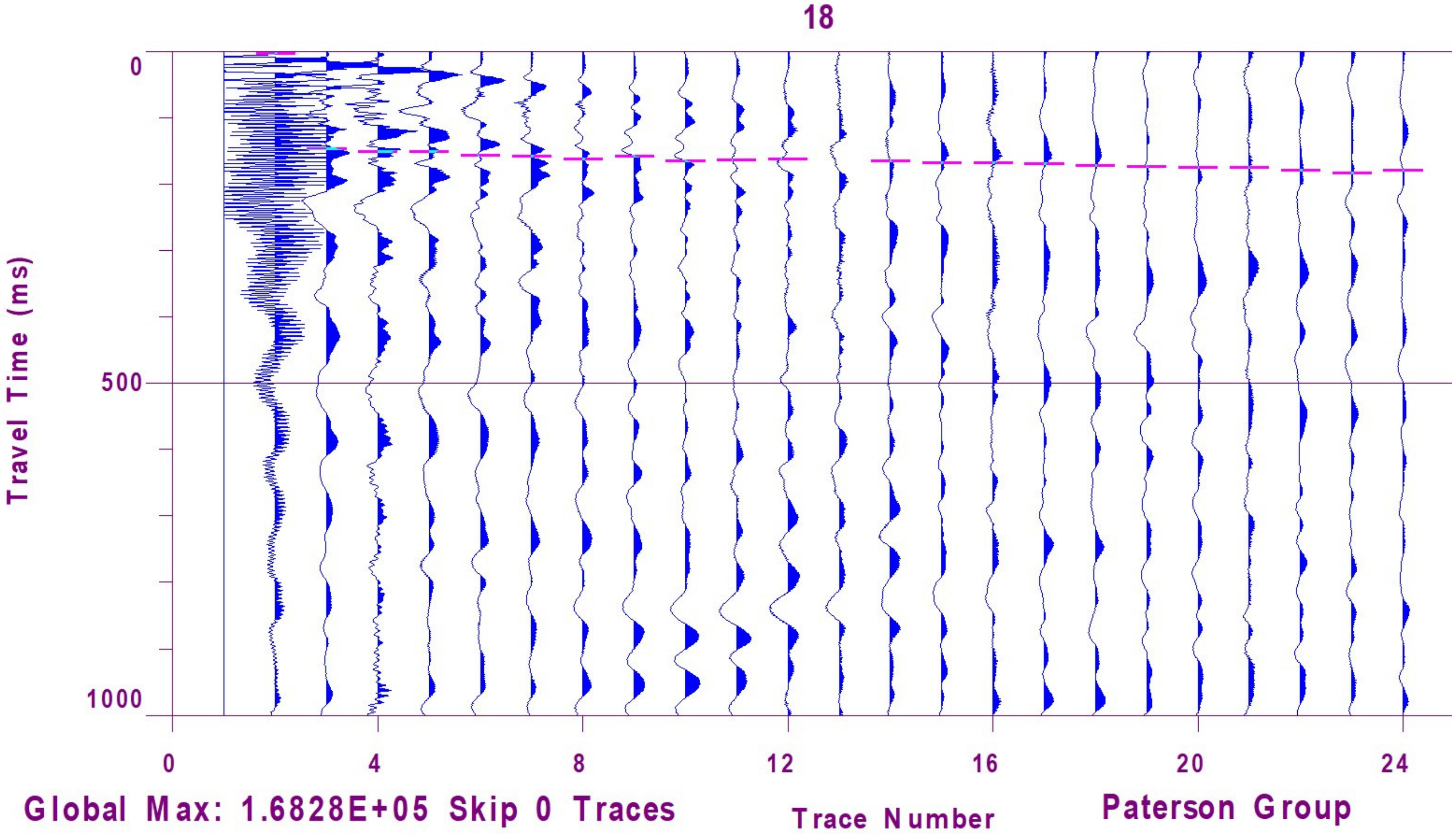
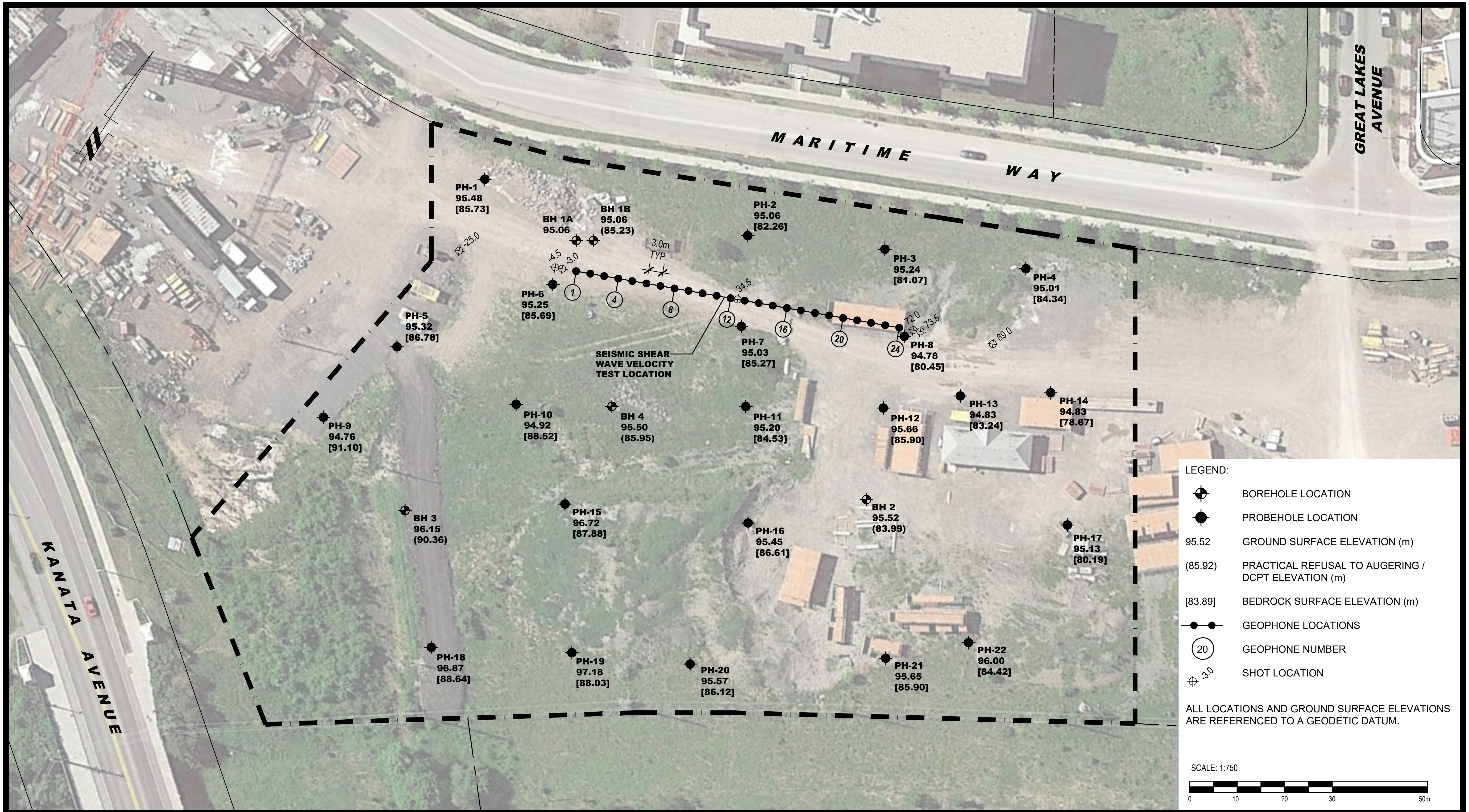


FIGURE 5 –Shear Wave Velocity Profile at Shot Location -4.5 m



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NO.	REVISIONS	DATE	INITIAL
1	PROBEHOLE LOCATION ADDED	07/06/2020	SD

CLARIDGE HOMES (TIMBERWALK) LP
GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE DEVELOPMENT - 1200 MARITIME WAY
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:750
Date: 04/2020
Drawn by: RCG
Report No.: PG5281-1
Checked by: KP
Dwg. No.: PG5281-1
Approved by: SD
Revision No.: 1

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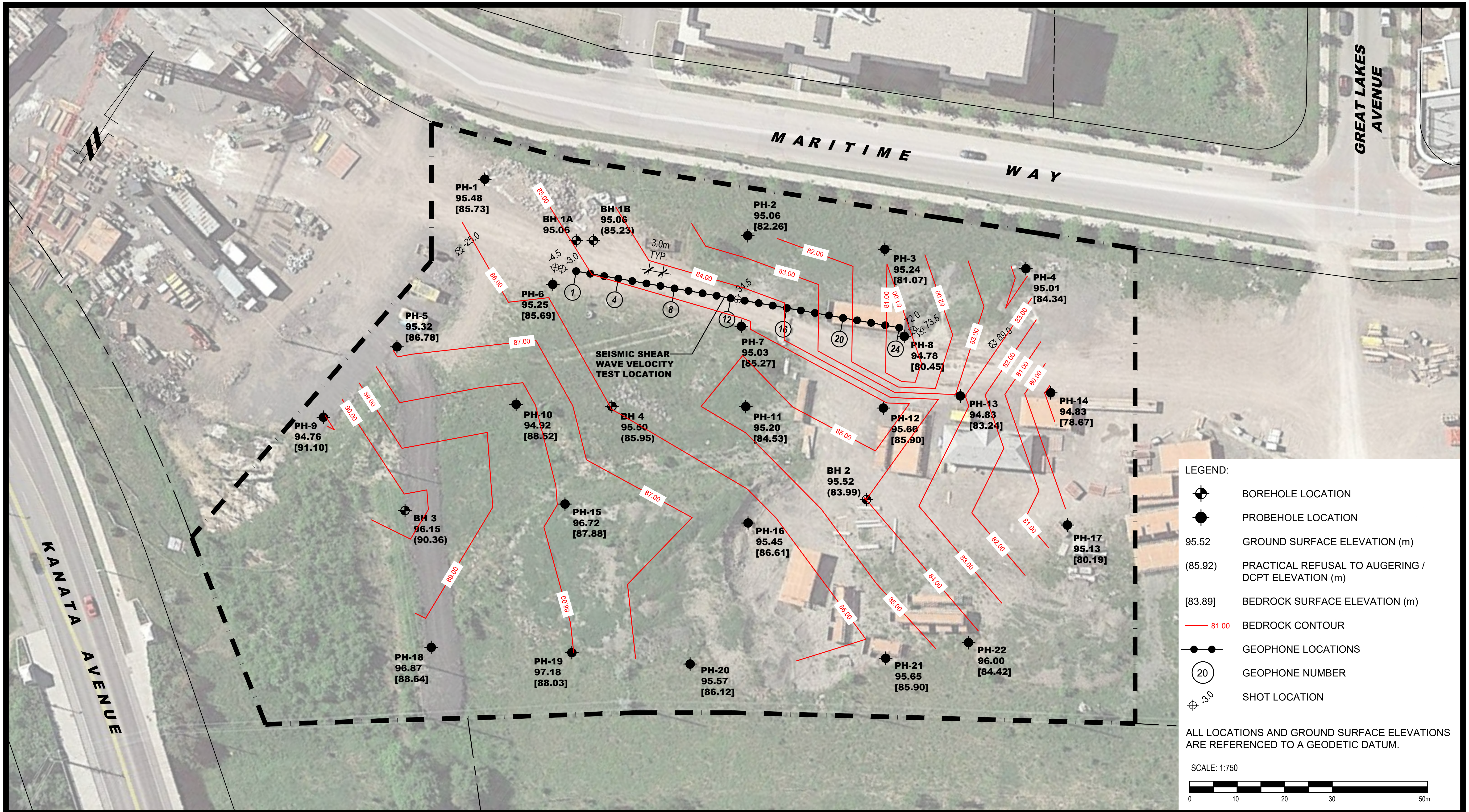

⊕	BOREHOLE LOCATION
●	PROBEHOLE LOCATION
95.52	GROUND SURFACE ELEVATION (m)
(85.92)	PRACTICAL REFUSAL TO AUGERING / DCPT ELEVATION (m)
[83.89]	BEDROCK SURFACE ELEVATION (m)
81.00	BEDROCK CONTOUR
●—●	GEOPHONE LOCATIONS
(20)	GEOPHONE NUMBER
⊕ -3.0	SHOT LOCATION

ALL LOCATIONS AND GROUND SURFACE ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:750

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NO.	REVISIONS	DATE	INITIAL

CLARIDGE HOMES (TIMBERWALK) LP

GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE DEVELOPMENT - 1200 MARITIME WAY

OTTAWA, ONTARIO

Title:

BEDROCK CONTOUR PLAN

Scale: 1:750	Date: 04/2020
Drawn by: RCG	Report No.: PG5281-1
Checked by: KP	Dwg. No.: PG5281-2
Approved by: SD	Revision No.: 0

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