# Hydrogeological Assessment

# 16054 & 16060 Airport Road

# Town of Caledon, Ontario

Prepared for:
Azure Group

Prepared by:
Azimuth Environmental
Consulting, Inc.

September 2019

AEC 19-216

AZIMUTH ENVIRONMENTAL CONSULTING, INC.

Environmental Assessments & Approvals

September 30th 2019 AEC 19-216

Azure Group
6751 Professional Court, Suite 201
Mississauga, ON
L4V 1Y3

Attention: Ahmed Al-Temimi
President

Re: **Hydrogeological Assessment**
16054 & 16060 Airport Road, Town of Caledon, Municipality of Peel

Dear Mr. Al-Temimi:

Azimuth Environmental Consulting, Inc. (Azimuth) is pleased to provide our Hydrogeological Assessment for the property located at 16054 & 16060 Airport Road within the Town of Caledon, Municipality of Peel, ON (the "Site"). This evaluation focused on the existing soil and ground water regime underlying the Site and the potential for the proposed development to impact the existing conditions.

Should you have any questions or wish to discuss the report in greater detail, please do not hesitate to contact the undersigned.

Yours truly,

AZIMUTH ENVIRONMENTAL CONSULTING, INC.

PROFESSIONAL GEOSCIENTIST
JENNIFER MILLINGTON
PRACTISING MEMBER
2643
ONTARIO

Jennifer Millington, M.A.Sc., P.Geo.
Hydrogeologist

642 Welham Road, Barrie, Ontario L4N 9A1
telephone: (705) 721-8451 • fax: (705) 721-8926 • info@azimuthenvironmental.com • www.azimuthenvironmental.com

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# 1.0 INTRODUCTION

Azimuth Environmental Consulting Inc. (Azimuth) has been retained by Azure Group to conduct a Hydrogeological Assessment for the property located at 16054 & 16060 Airport Road within the Town of Caledon and the Municipality of Peel, Ontario (the "Site")(Figure 1). The Site is rectangular in shape and is approximately 0.2 hectares (ha) in size. The Site is located on the south west side of Airport Road, south east of the Walker Road intersection (Figure 2). The Site is currently composed of two residential homes each with a gravel driveway and adjacent landscaped space.

It is our understanding that the Site will be developed into restaurant with parking and drive-thru use (Site Plan - Appendix B). The parking lot will be accessible off of Airport Road in the north corner of the parcel. The purpose of this assessment is to characterize the existing hydrogeological conditions at the Site and the potential for the proposed development to impact the existing environmental conditions.

# 2.0 ENVIRONMENTAL SETTING

## 2.1 Soil

The Soil Map of Peel County (Report No. 18) defines the surficial soil for the Site as Brighton sandy loam which is a dark grey-brown loam which is stone free and neutral to slightly alkaline. Brighton sandy loam is classified within Soil Group AB. Group A soils represent material with low runoff potential and high infiltration rates. Group B soils represent material with moderate infiltration rates when thoroughly wetted.

## 2.2 Physiography

The Site is located within the Oak Ridges Moraine Physiographic region. This region extends from the Niagara Escarpment to the Trent River and forms the height of land dividing the streams of the Lake Ontario drainage basin from those flowing into Georgian Bay and the Trent River. The surface of this region is hilly with a knob and basin relief typical of end moraine. For the most part, these hills are composed of sandy or gravelly material; however some till is also apparent.

## 2.3 Topography and Drainage

The Site is found at an elevation between approximately 291 - 293 masl, sloping from south west to north east toward Airport Road. The Site is located within the Humber River Watershed. There is currently no storm water infrastructure at the Site, so existing runoff is expected to follow the local topography and flow via sheet flow. The closest surface water feature is an unnamed tributary about 90 m south west of the Site.

## 2.4 Bedrock Geology

The Ontario Geologic Survey Earth Database shows that the uppermost bedrock unit at the Site consists of shale, siltstone, and minor limestone sand sandstone of the Queenston Formation (OGS, 2016). The Queenston Formation is Upper Ordovican in age.

## 2.5 Quaternary Geology

The Quaternary Soil Map of Ontario (Barnett *et al*., 1991) defines the local surficial soils in the vicinity of the Site as glaciofluvial outwash deposits consisting of river deposits composed of gravel and sand.

## 2.6 Hydrogeology

The Ontario Ministry of Environment, Conservation, and Parks (MECP) Water Well Records were referenced for any recorded well information within the vicinity of the Site (approximately 500 m)(GIN, 2018). The Site will be serviced with water and sewer utilities prior to development; however well records can be used to gain subsurface information which can provide insight into shallow geological formation within the area. The well records found in the vicinity of the Site are summarized in Table 1 and are shown on Figure 3:

**Table 1: MECP Water Well Database Summary (GIN, 2019)[1]**

| Well No. | Distance from the Site (m) | Date Drilled | Elevation (masl) | Depth (mbgs) | Static Water Level (mbgs) | Depth to Bedrock (mbgs) | Water Use/Status |
|---|---|---|---|---|---|---|---|
| 4908767 | 115 | SE | - | - | - | - | Abandoned |
| 4900676 | 165 | S | 294 | 38.4 | 0.6 | 23.8 | Test Hole |
| 4909686 | 200 | SE | - | 7.3 | - | 7.3+ | Observation |
| 7043256 | 285 | SE | - | 4.9 | - | 4.9+ | Observation |
| 4900038 | 290 | SE | 291 | 35.7 | 0.9 | 35.7+ | Municipal |
| 4909685 | 315 | SE | - | 7.6 | - | 7.6+ | Observation |
| 4900675 | 345 | SE | 293 | 48.8 | 22.0 | 48.8+ | Domestic |
| 4900032 | 410 | SE | 290 | 37.5 | 1.2 | 36.3 | Test Hole |
| 4905724 | 440 | NW | 305 | 26.2 | 3.7 | 26.2+ | Livestock |
| 4910121 | 450 | SE | - | 8.0 | 0.8 | 8.0+ | Abandoned |
| 4904257 | 475 | SE | 290 | 31.4 | 10.1 | 31.4+ | Municipal |
| 4905698 | 480 | NW | 308 | 18.3 | 6.1 | 18.3+ | Domestic |
| 4907104 | 485 | NW | - | 12.8 | 5.5 | 12.8+ | Domestic |

Notes: [1] - values rounded for presentation purposes

The surrounding wells in the MECP well record were drilled for observation, municipal, livestock, or domestic use, while two records were listed as abandoned. The wells were drilled to depths between 4.9 and 49 m. The static water level ranged from 0.6 to 22.0 mbgs, where recorded.

In general, the wells were drilled into surficial sand overlying clay and deeper sand deposits. Shale bedrock was encountered in two records, at a depth of 23.8 and 36.3 mbgs. The well records included in Table 1 are appended (Appendix C).

The Site is not considered a Significant Ground Water Recharge Area (SGRA); however it is considered a Highly Vulnerable Aquifer (HVA). It is located within Wellhead Protection Area C (WHPA-C) but is not in a WHPA-Q1 or WHPA-Q2. Two municipal wells are located within 500m of the Site, to the south east.

The Oak Ridges Moraine Groundwater Program (ORMGP, 2019) includes a digital representation of the water table. This layer is meant to represent an average water table since the values used in its creation were collected from various seasons throughout the year. ORMGP (2019) suggests that the actual water table at any given time of the year may be up to 2-3m lower or higher than the values indicated on the water table layer. According to ORMGP (2019) the water table in the vicinity of the Site is 292 - 293 masl, flowing south east. These values should be used as a general description of the regional ground water flow patterns that can be supplemented with Site-specific data.

# 3.0 MONITORING

## 3.1 Geotechnical

Azure Group Inc. completed a geotechnical program at the Site on July 24th, 2019 (Appendix D). The program included advancing five boreholes to depths up to 6.6 mbgs. Three of the holes (MW-1, MW-3, and MW-4) were completed as monitoring wells.

The subsurface material encountered includes a combination of topsoil over sand or silty sand. Soil cave-in was noted in each borehole, at depths ranging between 1.8 – 5.5 mbgs. The complete borehole logs and a location map can be found in Appendix D.

## 3.2 Ground Water Monitoring

The water level at each of the monitoring wells was measured on July 31st, 2019. The ground water table levels are given in the below Table 2:

**Table 2: Summary of Ground Water Measurements (July 2019)**

| Location ID | Water Level (mbgs) | Water Level (masl) |
|---|---|---|
| MW-1 | 3.15 | 288.58 |
| MW-3 | 4.09 | 288.96 |
| MW-4 | 4.23 | 288.97 |

The measured water levels at the Site ranged from 3.15 to 4.23 mbgs, representing 288.6 to 289.0 in July of 2019 and are shown on Figure 4. These levels are expected to fluctuate seasonally, with the highest readings expected between March and June.

### 3.3 Hydraulic Conductivity Testing

In order to understand the hydraulic characteristics of the underlying overburden, a transient slug test can be performed within monitoring wells to determine the average hydraulic conductivity of the screened interval. A slug test involves the instantaneous injection or withdrawal of a volume or slug of water or solid cylinder of known volume. This is accomplished by adding or displacing a known volume to/from a well and measuring water level response time to return to equilibrium.

Hydraulic conductivity testing was completed at the Site by Azimuth staff within the on-Site monitoring wells in July of 2019. Water level measurements were recorded both manually and with automatic dataloggers, which were programmed to record the pressure of water above the data logger every second. Data was analyzed using the Hvorslev Method (1951) for unconfined aquifers, which assumes a homogeneous, isotropic medium in which soil and water are incompressible. Hydraulic testing results are summarized in Table 3, and within Appendix E.

**Table 3: Hydraulic Testing Results**

| Monitoring Well | Screen Depth (mbgs) | Hydraulic Conductivity (m/s) | Soil Description |
|---|---|---|---|
| MW-1 | 2.0 – 5.0 | $1.1 \times 10^{-6}$ | Silty sand |
| MW-3 | 2.2 – 5.3 | $1.2 \times 10^{-6}$ | Silty sand |
| MW-4 | 2.4 – 5.4 | $1.3 \times 10^{-6}$ | Silty fine sand |

Slug test data indicates that the hydraulic conductivity of the deposits is very similar across the Site and range between $1.1 \times 10^{-6}$ to $1.3 \times 10^{-6}$ m/s. The measured hydraulic conductivity is within the published range for a silty sand material (Freeze & Cherry, 1979).

## 4.0 WATER BALANCE

In order to determine the potential changes to the natural ground water recharge conditions, a pre- and post-development water balance assessment has been completed using the Thornthwaite and Mather method (1957). This method evaluates evapotranspiration based on precipitation and temperature. Residual soil saturation is a function of topography and soil type. Monthly data are tabulated from daily average temperature and precipitation, and the water budget is a continuous calculation over the period of record. To clarify, the method and the approach used by many individuals in examining infiltration resets annual conditions (moisture deficit, snow storage, etc) over the winter months because of the general lack of infiltration during the frost period. However, we maintain those records and carry them forward from month to month during the entire period of record.

Values were determined on a monthly basis, compiled from daily Environment Canada meteorological data station located in Orangeville, Ontario between 1969 and 2015. The calculations are based on the average conditions during this period; the average precipitation was 896 mm, rainfall was 613 mm, evapotranspiration was 502 mm and the surplus was 394 mm.

## 4.1 Land Use

### 4.1.1 Pre-Development

The pre-development Site area was classified according to land use/vegetation type. Land within the pre-development area is provided in Table 4.

**Table 4: Pre Development Area Classification**

| Land Use | Land Area ($m^2$) |
|---|---|
| Gravel (driveway / walkway) | 190 |
| Structure | 355 |
| Forest | 475 |
| Landscaped | 970 |
| **TOTAL** | **1,990** |

Land within the pre-development scenario is considered 27% impervious. Gravel is considered impervious for the purpose of this assessment. The pre-development areas are shown on Figure 2.

### 4.1.2 Post-Development

The land classification in the post-development scenario was classified based on the Site Plan (Appendix B). Land within the post-development Site is summarized in the below Table 5:

**Table 5: Post Development Area Classification**

| Land Use | Land Area ($m^2$) |
|---|---|
| Structure | 232 |
| Other Impervious | 1,088 |
| Landscaped | 670 |
| **TOTAL** | **1,990** |

Land within the post-development scenario is considered 66% impervious.

## 4.2 Infiltration

Infiltration is generated one of two ways: (1) directly from rainfall impact or snowmelt on pervious surfaces; and (2) indirectly when runoff from impervious surfaces is diverted into adjacent naturalized areas.

Infiltration factors for the Site were estimated based on the underlying soil, local topography, and ground cover as per Table 2 of the Ministry of Environment and Energy (MOEE) Hydrogeological Technical Information Requirements for Land Development Applications (1995).

The soil variable factor was determined by taking into account information obtained from the regional geologic mapping and the field work programs completed for the Site. This information suggests that the surficial material at the Site is primarily composed of silty sand. The infiltration factors utilized in the water balance assessment are summarized in Table 6 below.

**Table 6: Summary of Pervious Land Infiltration Factor**

| Scenario | Land Use | Infiltration Factor | Assumption |
|---|---|---|---|
| Pre-Development | Landscaped | 0.60 | Rolling land, silt/sand soil, lawn |
| Pre-Development | Forest | 0.70 | Rolling land, silt/sand soil, woodland |
| Post-Development | Landscaped | 0.60 | Rolling land, silt/sand soil, lawn |

### 4.2.1 Pre-Development Infiltration

Pre-development direct infiltration was determined by multiplying the annual average surplus amount, the area of each land use, and the infiltration factor for each land use. The pre-development annual infiltration is therefore 360 $m^3$/year (Appendix F).

It is also assumed that additional infiltration was gained when rainfall runoff from rooftop downspouts are directed onto adjacent grassy areas. It is assumed that half of the rooftop downspouts were directed to the grass. The volume of infiltration gained from rooftop runoff is therefore determined by multiplying the rooftop area by the total rainfall volume, by 80% (to account for evaporation) and by the infiltration factor of the receiving land use. Infiltration gained from rooftop runoff is therefore 52 $m^3$/year, producing a total pre-development infiltration of 413 $m^3$/year.

#### 4.2.2 Post-Development Infiltration

Post-development infiltration (without mitigation) was determined by multiplying the annual average surplus amount, the area of each land use, and the infiltration factor for each land use. The post-development annual direct infiltration is therefore 158 $m^3$/year. There is therefore a decrease in infiltration of 254 $m^3$/year from pre- to post-development without mitigation measures employed.

The post-development drainage plan includes low impact development (LID) to promote infiltration. An infiltration trench will be located south west of the structure and will collect rooftop runoff. The trench will be 10.5m long, 8m wide, and 0.3m deep with a retention volume of 10.1 $m^3$. The LID is designed to drain within a period of 24 hours. Based on the size of the rooftop (232.26 $m^2$) the trench will infiltrate up to the 43 mm rainfall event.

In order to correlate event based rainfall data, for which the LID's are designed (i.e. 25 mm rainfall event), to annual averages, as is what is utilized in water balances, an event based assessment has been completed for the Orangeville Climate station. Rainfall events over the past 6 years of complete data (2010 – 2015) were broken down by event size, such that total volumes for each of these events could be calculated. These totals were then related to the total volume over the same period to obtain a percentage. This percentage is then multiplied by the annual average value (675 mm) utilized in the overall water balance to obtain an annual average amount / depth for the various intervals. It was determined that an event rainfall depth of 43mm represented an average annual rainfall total of 662 mm or 98% of the rainfall events over this period of record. It is noted that these represent cumulative amounts for all rainfall amounts less than the stated event size.

In order to quantify the annual infiltration volumes for each LID, the annual rainfall depth (event equivalent) discussed above is multiplied by the catchment area for that specific LID, while a 20% evaporation loss factor was employed for runoff collected on all impervious surfaces. It is noted that this factor is a common assumption in water balance assessments and is based on standards presented in *Conservation Guidelines for Hydrogeological Assessments* (Cuddy & Chan, 2013). The total infiltration gained through the infiltration LID is 123 $m^3$.

It is noted that added conservancy is reflected in these numbers through discounting of snow melt. Although difficult to quantify due to seasonal storage and movement (i.e. snow banks, snow dumps), it can provide a potential meaningful contribution as it represents ~17% of total precipitation.

### 4.3 Water Balance Summary

Using the climate model data and calculations mentioned above, the water balance was completed for pre-development, post-development, and post-development with mitigation (Appendix F).

The pre-development infiltration volume is 413 $m^3$/year. This assumes the Site is composed of the existing driveways/walkways, residential structures, forest, and landscaped grass. The post-development without mitigation infiltration volume is 158 $m^3$/year, which is a deficit of 254 $m^3$/year. This assumes the Site is composed of a commercial restaurant, drive-through, parking, and landscaped grass. An additional 123 $m^3$/year of infiltration can be obtained by diverting the runoff from the structure rooftop directly to the buried infiltration trench. The post-development with mitigation volume is therefore 281 $m^3$/year which represents 68% of the pre-development volume or a loss of approximately 131 $m^3$ per year.

Although the post-development infiltration volume does not match the pre-development infiltration volume, a best efforts approach has been taken. The infiltration trench has been designed to capture 97% of the rooftop runoff. This is considered clean runoff and is ideal for onsite infiltration. Since the Site is located within a Highly Vulnerable Aquifer, infiltration from parking lots within commercial land use is not idea due to the potential for increased contaminants. The Site development has therefore infiltrated almost all of the clean runoff from impervious areas. Although a small proportion of runoff is also available from pervious land within the Site, this land is spread out around the perimeter of the development on the down gradient side of a retaining wall. The location of this runoff is therefore not ideal for water collection, retention, and transport to the proposed storm water conveyance system.

## 5.0 DEWATERING ASSESSMENT

The proposed development will include the construction of one commercial restaurant with the necessary underground servicing (water, sewer, storm water). The service connections will be off Airport Road to the north east. Due to the presence of shallow ground water at the Site, dewatering may be required to maintain dry conditions during construction.

Shallow excavations into the ground water table can typically be controlled by conventional sump pump technologies and will likely be appropriate for use at the Site. If the required drawdown is greater than 1.5m, the use of shallow well points or eductor systems may be required. The exact dewatering methodology will depend on Site specific conditions and will be determined by a specialist contractor.

## 5.1 Drawdown Conditions

The finished floor elevation of the proposed structure is ~291.65 masl, which is 0.65 to 1.35 m below the existing ground elevation of the structure footprint (292.3 to 293.0 masl). It is assumed that the structure will be slab-on-grade construction, with approximately 0.5m for foundation footings. It is then assumed that a further 0.5m will be required for foundation installation. Based on the above assumptions, the proposed excavation base is therefore 290.65 masl.

Based on the information provided in Section 3.2, the ground water elevation in the vicinity of the structure footprint is between 288.65 and 288.85 masl. The proposed foundation excavation is 1.8m above the measured ground water elevation for the Site and is therefore not expected to overlap with the ground water table. Based on this data, ground water lowering will not be required for foundation construction. In addition, the ground water elevations are expected to fluctuate seasonally, with the highest readings expected between March and June. Given a lack of ground water table elevation measurements during the March to June window, the above assessment should therefore be confirmed once the high ground water has been measured and detailed structural foundation depths are known.

The structure will be serviced with water, sanitary, and storm water services. Information on the service corridors was obtained from the Site Grading, Servicing and Stormwater Management Plan (Drawing G1) provided by A.M. Candaras Associates Inc. and is provided in Appendix G. To estimate the dewatering conditions at the Site, the minimum depth and maximum (estimate) ground water elevation was assigned to segments of each service corridor to determine the drawdown required for service installation. It was assumed that dewatering 0.5m below the minimum depth was necessary to maintain dry conditions during installation. The data assigned to each corridor can be found in Appendix G.

Based on the information provided in Appendix G, none of the storm water corridors will require dewatering for installation. The water line will require a drawdown of 0.15m, while the sanitary lines will require a maximum drawdown of 1.68m. These corridors are located between the proposed structure and the existing lines along Airport Road. The required drawdown can be confirmed once servicing depths and high ground water levels are finalized.

## 5.2 Approximate Dewatering Volumes

Calculations for the dewatering rate / volume were completed using the steady state method from Powers et al. (2007) for estimating flow from a line source from an unconfined aquifer in addition to the radial flow from each end of the excavation.

The following equation was used:

$$Q = \{[(\pi * K)*(H^2-h^2)] / [\ln(R_o/r)] + 2*[a*K*(H^2-h^2)/(2R_o)]\}* 86400*1000$$

(Ref: Powers *et al.* (2007)

The calculation was completed for an estimated trench area using a 22.8m length, 2m width, and the maximum drawdown of 1.68m (which includes the 0.5m contingency). Based on the hydraulic (slug) test completed at the Site monitoring well (Section 3.3), the measured hydraulic conductivity used was 1.30 x $10^{-6}$ m/s.

**Table 7: Estimated Trench Dewatering Conditions**

| Variable | Sanitary Line (Worst Case) |
|---|---|
| Hydraulic Conductivity **[K]** (m/s) | 1.30 x $10^{-6}$ |
| Length of Excavation **[a]** (m) | 22.8 |
| Width of Excavation [b] | 2 |
| Maximum Required Drawdown **[H-h]** (m) | 1.68 |
| Saturated Thickness Before Pumping **[H]** (m) | 2.68 |
| Depth of Water During Pumping **[h]** (m) | 1.0 |
| Radius of Influence **[Ro]** (m) [2] | 10 |
| Estimate of Equivalent Radius **[r]** (m) | 8 |
| Discharge **[Q]** (L/day) | 5,100 |
| **Discharge [Q] (L/day) 3 X Safety Factor Applied** | **15,300** |

[1] $r_e = (a+b) / \pi$ - assuming a/b >1.5, (Driscoll, 1986)

[2] $Ro = r_e + 3000 * (H-h) * \sqrt{k}$ - Sichardts Formula, (Cashman and Preene, 2001)

Based on the information provided in Table 7, the dewatering volume for a trench 22.8m wide, 2m deep, and with a drawdown of 1.68 m is anticipated to be 5,100 L/day, while applying a 3X safety factor would create an estimated daily volume of 15,300 L/day. These values are based on the groundwater level of 288.65 masl and the estimated excavation depth of 286.97 masl.

A Permit To Take Water (PTTW) is required from the Ontario Ministry of the Environment, Conservation, and Parks (MECP) if the volume of pumped water is greater than 400,000 L/day. If the volume of water pumped is greater than 50,000 L/day but less than 400,000L/day, then registration under the Environmental Activity and Sector Registry (EASR) is recommended. Based on Table 7, a PTTW is likely not required for the Site, but registration under EASR may be required if the higher volumes are encountered. It should be noted that higher volumes may be encountered if higher ground water levels are encountered in the spring. The above dewatering analysis should be confirmed once high ground water levels have been measured, and when the foundation footing elevation of the structure can be confirmed.

### 5.3 Impact Assessment

Based on the information provided in Table 7, the largest zone of influence is 10m from the dewatering zone; however, this influence represents the extent of any measurable influence, whereas the degree of influence at the outer extents would be expected to be limited (i.e. <0.1 m).

In Section 2.6, the well record review identified the closest private domestic well to be 345m from the Site. This well is located far outside of the zone of influence and therefore no impacts are anticipated. It is also noted that given the municipal servicing in the area, it is unlikely that this well is still in use.

The dewatering zone of influence will not overlap with any natural heritage features. Any water pumped from trench excavations inside the Site will likely be discharged to the road side catch basin located in front of the Site within the Airport Road right-of-way.

## 6.0 SUMMARY AND CONCLUSIONS

Azimuth was retained by Azure Group to conduct a Hydrogeological Assessment for the property located at 16054 & 16060 Airport Road within the Town of Caledon and the Municipality of Peel, Ontario. The Site is rectangular in shape and is approximately 0.2 hectares (ha) in size. The Site is located on the south west side of Airport Road, south east of the Walker Road intersection. The Site is currently composed of two residential homes each with a gravel driveway and adjacent landscaped space. The Site will be developed into restaurant with parking and drive-thru use. The parking lot will be accessible off of Airport Road in the north corner of the parcel. The purpose of this assessment is to characterize the existing hydrogeological conditions at the Site and the potential for the proposed development to impact the existing environmental conditions.

The Site is found at an elevation between 291 - 293 masl, sloping from south west to north east toward Airport Road. The Site is located within the Humber River Watershed. There is currently no storm water infrastructure at the Site, so existing runoff is expected to follow the local topography and flow via sheet flow. The closest surface water feature is an unnamed tributary about 90 m south west of the Site. Boreholes logs from the Site show the subsurface is composed of topsoil overlying sand or silty sand material.

The inferred ground water flow direction is north east toward Airport Road. This follows the local topographic surface. Ground water at the Site was measured at between 288.6 and 289 masl. Hydraulic conductivity testing was completed within the three monitoring wells at the Site by Azimuth staff. Slug test data indicates that the hydraulic conductivity of the deposits is very similar across the Site and range between $1.1x10^{-6}$ to $1.3x10^{-6}$ m/s.

The pre-development infiltration volume is 413 $m^3$/year. This assumes the Site is composed of the existing driveways/walkways, residential structures, forest, and landscaped grass. The post-development without mitigation infiltration volume is 158 $m^3$/year, which is a deficit of 254 $m^3$/year. This assumes the Site is composed of a commercial restaurant, drive-through, parking, and landscaped grass. An additional 123 $m^3$/year of infiltration can be obtained by diverting the runoff from the structure rooftop directly to the buried infiltration trench. The post-development with mitigation volume is therefore 281 $m^3$/year which represents 68% of the pre-development volume or a loss of approximately 131 $m^3$ per year.

Although the post-development infiltration volume does not match the pre-development infiltration volume, a best efforts approach has been taken. The infiltration trench has been designed to capture 97% of the rooftop runoff. This is considered clean runoff and is ideal for onsite infiltration. Since the Site is located within a Highly Vulnerable Aquifer, infiltration from parking lots within commercial land use is not idea due to the potential for increased contaminants. The Site development has therefore infiltrated almost all of the clean runoff from impervious areas. Although a small proportion of runoff is also available from pervious land within the Site, this land is spread out around the perimeter of the development on the down gradient side of a retaining wall. The location of this runoff is therefore not ideal for water collection, retention, and transport to the proposed storm water conveyance system.

The proposed development will include the construction of one commercial restaurant with the necessary underground servicing (water, sewer, storm water). The service connections will be off Airport Road to the north east. Due to the presence of shallow ground water at the Site, dewatering may be required to maintain dry conditions during construction.

Based on the ground water measurements and the proposed finished floor elevation, ground water lowering will not be required for foundation construction. Ground water lowering will also not be required for installation of the storm water infrastructure. Lowering will be required however for the installation of the water and sewer lines connecting the proposed structure to the existing lines along Airport Road. Based on the information provided in Table 7, the dewatering volume for a trench 22.8m wide, 2m deep, and with a drawdown of 1.68 m is anticipated to be 5,100 L/day, while applying a 3X safety factor would create an estimated daily volume of 15,300 L/day. This value is below the requirement for an EASR or PTTW.

It should be noted that higher volumes may be encountered if higher ground water levels are encountered in the spring. The above dewatering analysis should be confirmed once high ground water levels have been measured, and when the foundation footing elevation of the structure can be confirmed.

## 7.0 REFERENCES

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## APPENDICES

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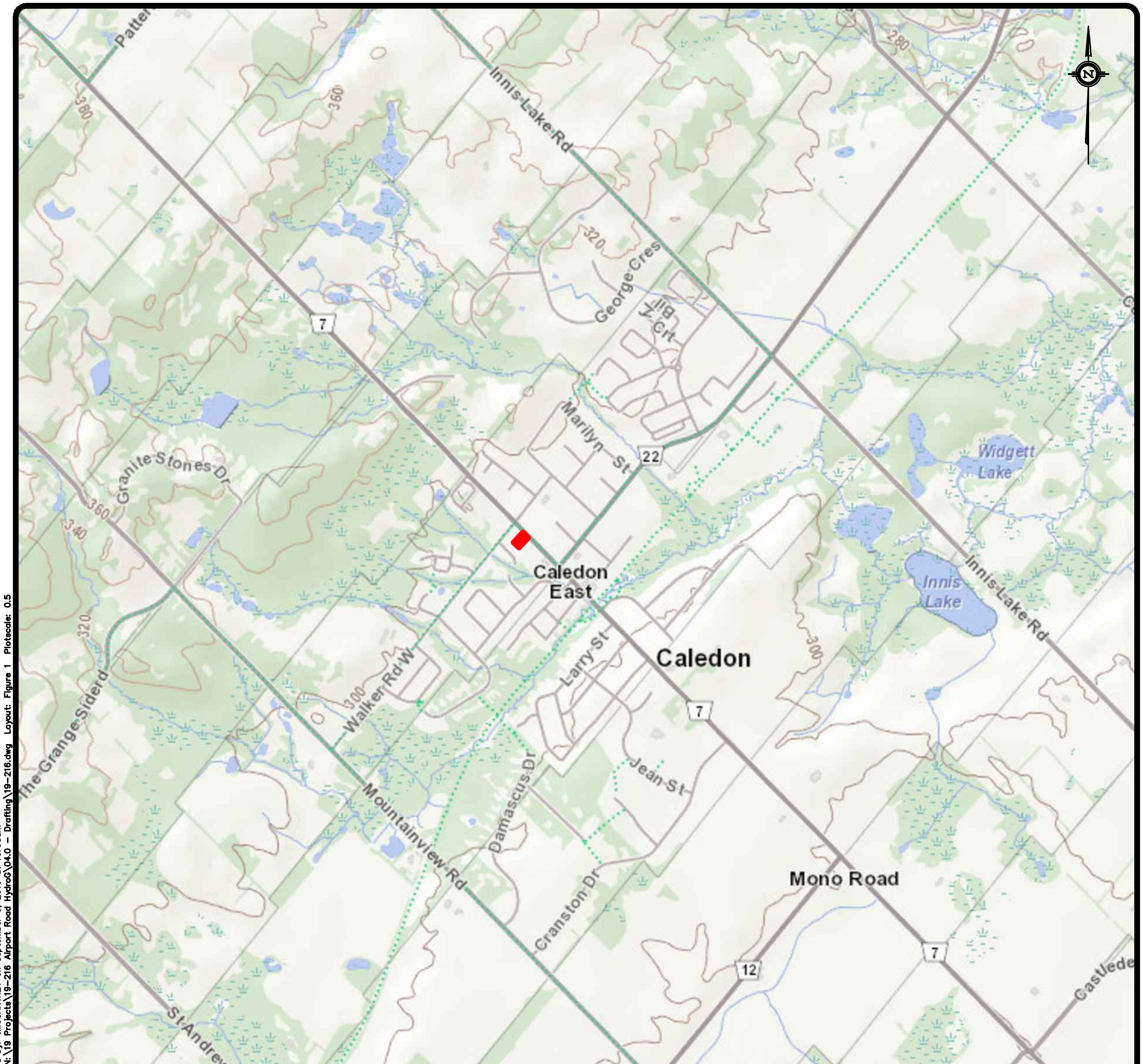
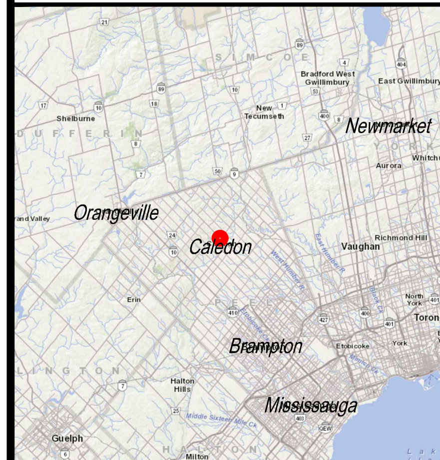
## APPENDIX A

## Figures




LEGEND:

Approx. Property Boundary




REG MAP

250m 0 750m

HORIZONTAL SCALE 1: 25,000

AZIMUTH ENVIRONMENTAL CONSULTING, INC.

Study Area Location

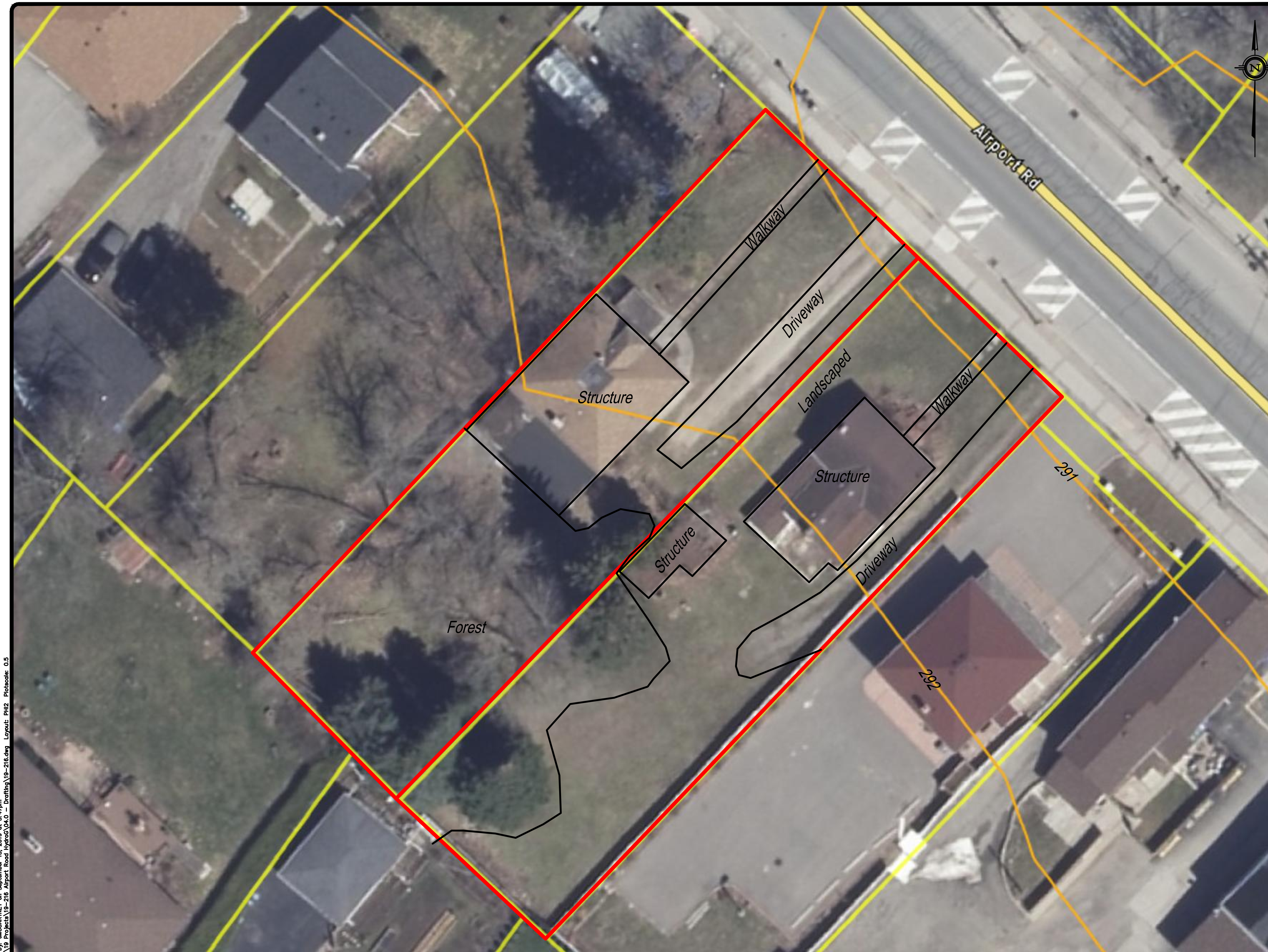
16054 & 16060 Airport Rd.,
Caledon, ON

DATE ISSUED: *September 2019*
CREATED BY: *JLM*
PROJECT NO.: *19-216*
REFERENCE: *MNRF*

Figure No.
1

DAYSTAMP: N:\19 Projects\19-216 Airport Road HydroG\04.0 - Drafting\19-216.dwg

Plotted by: JMCCARTNEY on September 5, 2019 at 10:38am Layout: Figure 1 Plotscale: 0.5
File: N:\19 Projects\19-216 Airport Road HydroG\04.0 – Drafting\19-216.dwg




LEGEND:
- Approx. Property Boundary
- 1m FBS Contours

5m 0 10m
HORIZONTAL SCALE 1:300

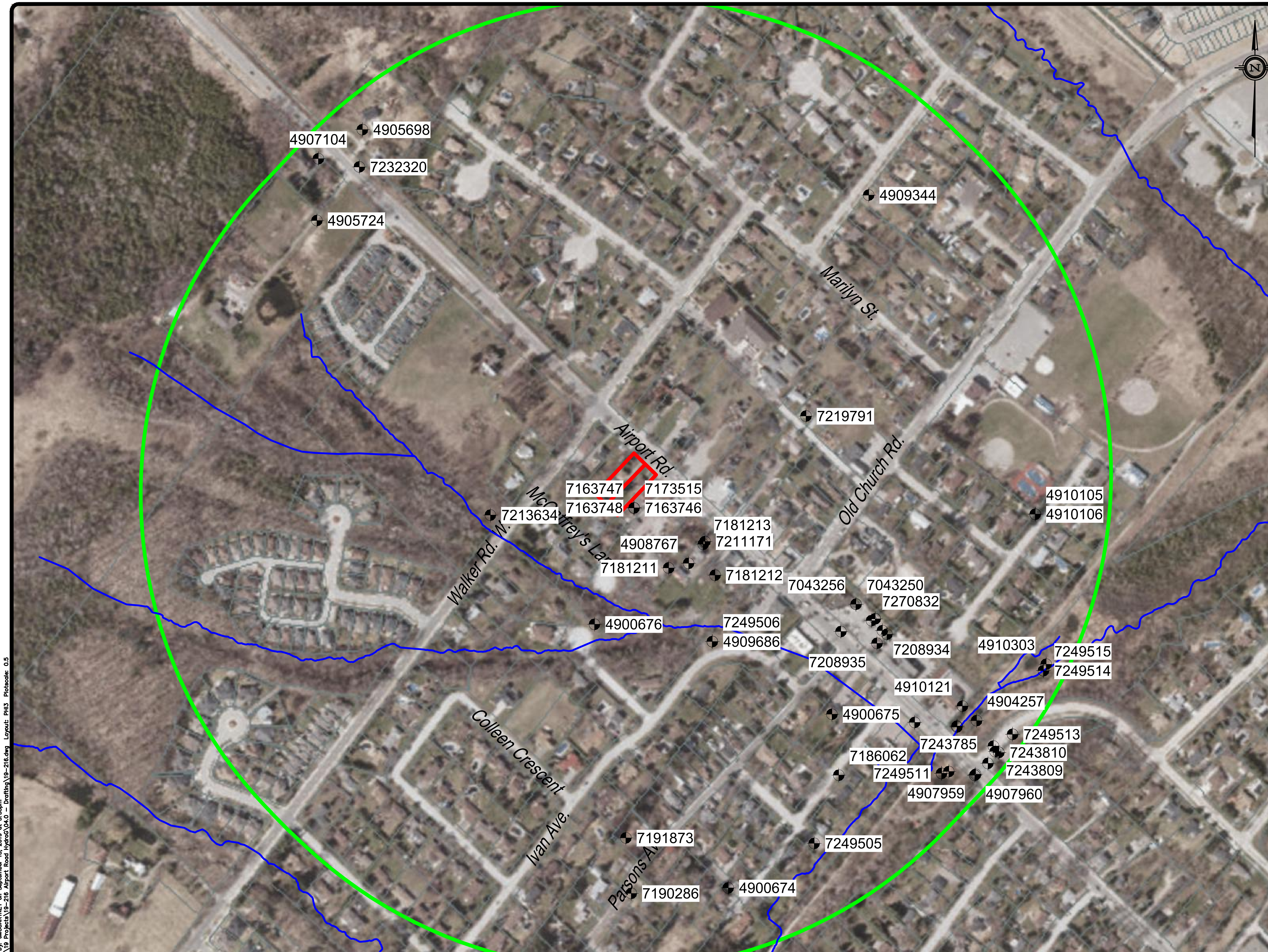
AZIMUTH ENVIRONMENTAL CONSULTING, INC.

Site Layout

16054 & 16060 Airport Rd.,
Caledon, ON

| | | |
|---|---|---|
| DATE ISSUED: | September 2019 | Figure No. |
| CREATED BY: | JLM | 2 |
| PROJECT NO.: | 19-216 | |
| REFERENCE: | MNRF | |

DAYSTAMP: N:\19 Projects\19-216 Airport Road HydroG\04.0 - Drafting\19-216.dwg




DAYSTAMP: N:\19 Projects\19-216 Airport Road HydroG\04.0 - Drafting\19-216.dwg

Plotted by: JMCCARTNEY on September 13, 2019 at 3:55pm Layout: PHI3 Plotscale: 0.5
File: N:\19 Projects\19-216 Airport Road HydroG\04.0 - Drafting\19-216.dwg

LEGEND:

- *Approx. Property Boundary*
- *500m Site Boundary*
- *Watercourse*
- *Water Well Locations*

*Note:*
*Well locations are based on MECP well records and/ or GIN (2019) mapping.*

60m 0 120m

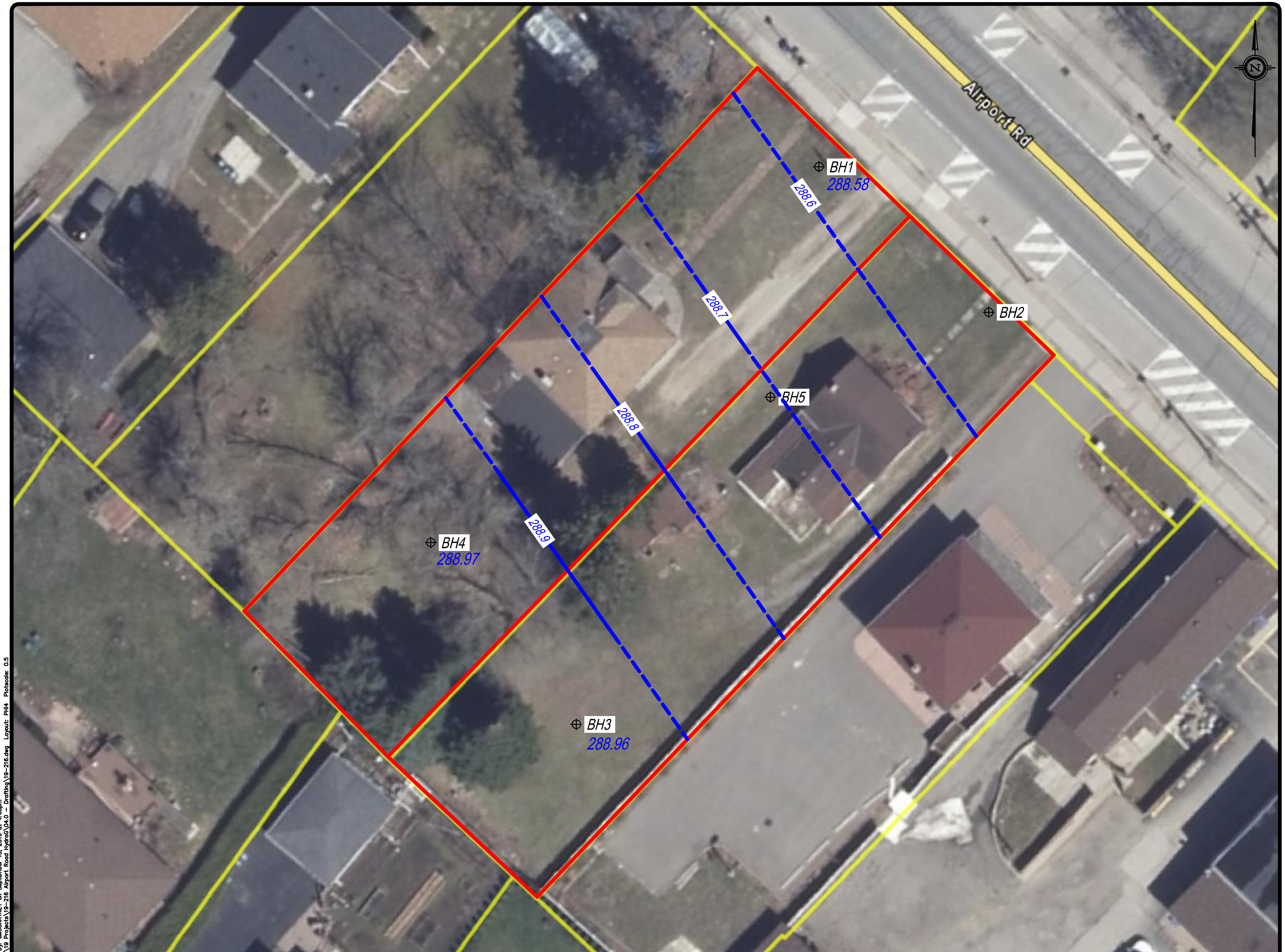
HORIZONTAL SCALE 1: 4,000

AZIMUTH ENVIRONMENTAL CONSULTING, INC.

Surrounding Well Records

16054 & 16060 Airport Rd.,
Caledon, ON

| | | |
|---|---|---|
| DATE ISSUED: | September 2019 | Figure No. |
| CREATED BY: | JLM | 3 |
| PROJECT NO.: | 19-216 | |
| REFERENCE: | MNRF | |




LEGEND:

- Approx. Property Boundary
- Borehole Locations
- 0.1m Ground Water Contours
- Inferred 0.1m Ground Water Contours
- 288.58 Ground Water Elevations (July 31, 2019)

5m 0 10m

HORIZONTAL SCALE 1: 300

AZIMUTH ENVIRONMENTAL CONSULTING, INC.

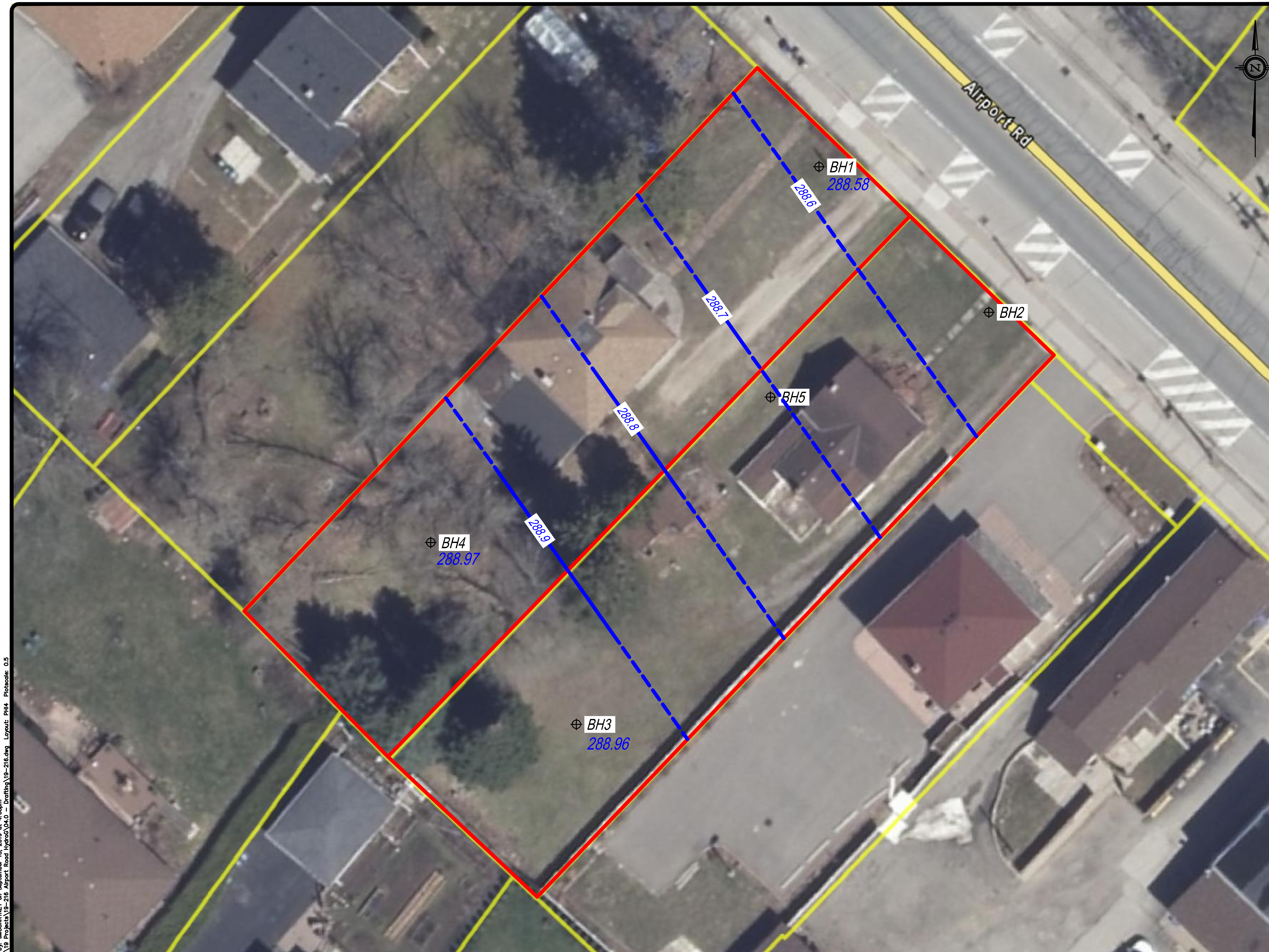
Ground Water Information

16054 & 16060 Airport Rd.,
Caledon, ON

| | | |
|---|---|---|
| DATE ISSUED: | September 2019 | Figure No. |
| CREATED BY: | JLM | 4 |
| PROJECT NO.: | 19-216 | |
| REFERENCE: | MNRF | |

DAYSTAMP: N:\19 Projects\19-216 Airport Road HydroG\04.0 - Drafting\19-216.dwg

Plotted by: JMCCARTNEY on September 13, 2019 at 4:08pm
File: N:\19 Projects\19-216 Airport Road HydroG\04.0 - Drafting\19-216.dwg Layout: PHI4 Plotscale: 0.5




LEGEND:

- Approx. Property Boundary
- Borehole Locations
- 0.1m Ground Water Contours
- Inferred 0.1m Ground Water Contours
- 288.58 Ground Water Elevations (July 31, 2019)

5m 0 10m

HORIZONTAL SCALE 1:300

AZIMUTH ENVIRONMENTAL CONSULTING, INC.

Ground Water Information

16054 & 16060 Airport Rd.,
Caledon, ON

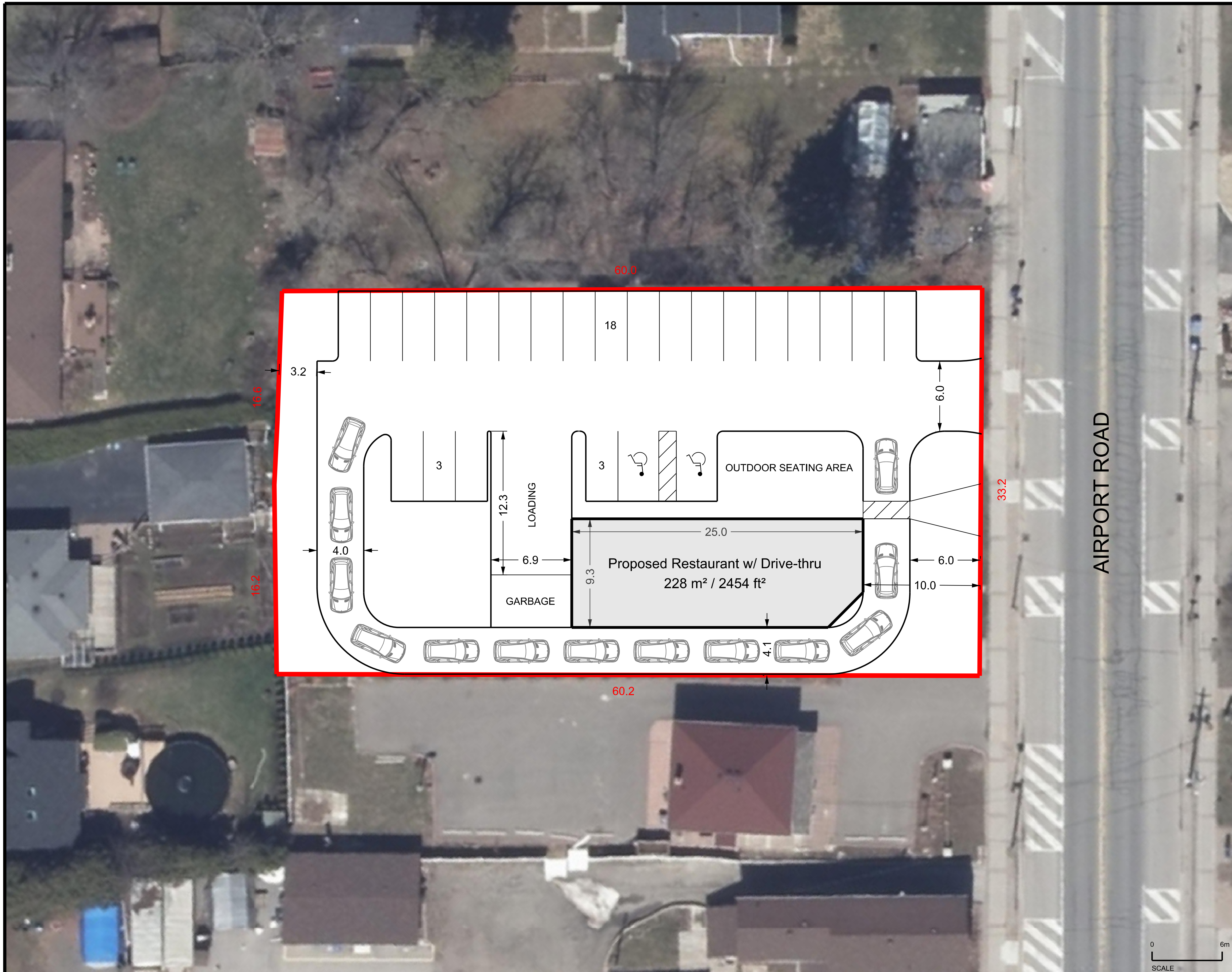
| | | |
|---|---|---|
| DATE ISSUED: | September 2019 | Figure No. |
| CREATED BY: | JLM | 4 |
| PROJECT NO.: | 19-216 | |
| REFERENCE: | MNRF | |

DAYSTAMP: N:\19 Projects\19-216 Airport Road HydroG\04.0 - Drafting\19-216.dwg

Plotted by: JMCCARTNEY on September 13, 2019 at 4:08pm
File: N:\19 Projects\19-216 Airport Road HydroG\04.0 – Drafting\19-216.dwg Layout: PH4 Plotscale: 0.5

## APPENDIX B

## Site Plan




# DEVELOPMENT CONCEPT

16054 & 16060 AIRPORT ROAD
TOWN OF CALEDON
REGIONAL MUNICIPALITY OF PEEL

WESTON CONSULTING
planning + urban design

**DRAFT**
FOR DISCUSSION PURPOSES ONLY

DEVELOPMENT STATISTICS:

| | |
|---|---|
| Site Area: | 1,990 m² |
| Building Footprint: | 228 m² |
| Coverage: | 11.5% |
| FSI: | 0.12 |
| Total GFA: | 228 m² |
| Proposed Restaurant w/ Drive-thru (1-sty) | 228 m² |
| Required Parking: *By-law 2006-50* | 15.2 sp |
| Restaurant @ 1 sp/15m² | 15.2 sp |
| Parking Provided: | 24 sp |

Notes:
- Property Boundaries digitized from available mapping, subject to confirmation by survey.
- Areas and dimensions are approximate and subject to confirmation by survey.
- Air photo from First Base Solutions Inc., 2018 image.


WESTON CONSULTING
planning + urban design
1-800.363.3558 westonconsulting.com

Vaughan:
201 Millway Ave. Suite 19
Vaughan, Ontario L4K 5K8
T. 905.738.8080 F. 905.738.6637

Toronto:
268 Berkeley St.
Toronto, Ontario M5A 2X5
T. 416.640.9917 F. 905.738.6637


DRAWN / REVISED

| | |
|---|---|
| 15 JAN 2019 | First Draft |

| | |
|---|---|
| File Number: | 9368 |
| Drawn By: | SD |
| Planner: | RG |
| Scale: | see scale bar |
| CAD: | 9368/concepts/C2.dgn |

Drawing Number:
C2

## APPENDIX C

## MECP Well Records

**Water Well**

**Identity :** ca.on.waterWell.4908767
**External identity :** ca.on.waterWell.4908767
**Source :** Ontario Ministry of Environment
**Online resource :** http://www.ene.gov.on.ca/environment/en/subject/wells/index.htm
**Elevation :** NaNm
**Well status :** Abandoned-Other
**Well type :** Unknown
**Sealing components :** From 0.00 to 0.91m; From 1.22 to 4.57m; From 0.91 to 1.22m.

## Water Well

**Identity :** ca.on.waterWell.4900676
**External identity :** ca.on.waterWell.4900676
**Source :** Ontario Ministry of Environment
**Online resource :** http://www.ene.gov.on.ca/environment/en/subject/wells/index.htm
**Length :** 38.40m
**Elevation :** 294.13m
**Water level :** 0.61m
**Water yield :** 40.91lpm
**Water use :** Not Used
**Well status :** Test Hole
**Well type :** Unknown

## Well Log

| Depth from (m) | Depth to (m) | GIN Lithology | Original Lithology | Porosity* | Hydraulic Conductivity* |
|---|---|---|---|---|---|
| 0.00 | 0.30 | **Soil** | TOPSOIL | | |
| 0.30 | 5.49 | **Clay**<br>**Sand**<br>**Gravel** | CLAY FINE SAND STONES | [34,57]%<br>[26,53]%<br>[24,44]% | [1E-11,4.7E-9]m.s-1<br>[2E-7,6E-3]m.s-1<br>[3E-4,3E-2]m.s-1 |
| 5.49 | 10.67 | **Sand** | FINE SAND | [26,53]% | [2E-7,6E-3]m.s-1 |
| 10.67 | 12.50 | **Sand**<br>**Clay** | FINE SAND CLAY | [26,53]%<br>[34,57]% | [2E-7,6E-3]m.s-1<br>[1E-11,4.7E-9]m.s-1 |
| 12.50 | 20.12 | **Sand** | QUICKSAND | [26,53]% | [2E-7,6E-3]m.s-1 |
| 20.12 | 23.77 | **Clay** | CLAY | [34,57]% | [1E-11,4.7E-9]m.s-1 |
| 23.77 | 24.69 | **Diamicton**<br>**Shale** | HARDPAN SHALE | [1,10]% | [1E-13,2E-9]m.s-1 |
| 24.69 | 38.40 | **Shale**<br>**Sand** | SHALE MEDIUM SAND | [1,10]%<br>[26,53]% | [1E-13,2E-9]m.s-1<br>[2E-7,6E-3]m.s-1 |

*Note: Porosity and hydraulic conductivity values are NOT measured but are derived from tables showing statistical averages for lithologies

# Groundwater Information Network
# Réseau d'Information sur les Eaux Souterraines

## Water Well

**Identity :** ca.on.waterWell.4909686
**External identity :** ca.on.waterWell.4909686
**Source :** Ontario Ministry of Environment
**Online resource :** http://www.ene.gov.on.ca/environment/en/subject/wells/index.htm
**Length :** 7.30m
**Elevation :** NaNm
**Well status :** Observation Wells
**Well type :** Unknown
**Well casings :** From 0.00 to 4.20m.
**Sealing components :** From 3.70 to 3.70m; From 0.00 to 0.00m; From 0.30 to 0.30m.
**Screen components :** From 4.2 to 7.30m.

## Well Log

| Depth from (m) | Depth to (m) | GIN Lithology | Original Lithology | Porosity* | Hydraulic Conductivity* |
|---|---|---|---|---|---|
| 0.00 | 0.40 | **Soil** | TOPSOIL | | |
| 0.40 | 1.90 | **Silt**<br>**Sand**<br>**Gravel** | SILT SAND GRAVEL | [34,61]%<br>[26,53]%<br>[24,44]% | [1E-9,2E-5]m.s-1<br>[2E-7,6E-3]m.s-1<br>[3E-4,3E-2]m.s-1 |
| 1.90 | 2.80 | **Sand** | COARSE SAND | [26,53]% | [2E-7,6E-3]m.s-1 |
| 2.80 | 2.90 | **Peat** | PEAT | | |
| 2.90 | 3.70 | **Silt** | SILT | [34,61]% | [1E-9,2E-5]m.s-1 |
| 3.70 | 4.80 | **Sand** | FINE SAND | [26,53]% | [2E-7,6E-3]m.s-1 |
| 4.80 | 7.30 | **Silt**<br>**Sand** | SILT FINE SAND | [34,61]%<br>[26,53]% | [1E-9,2E-5]m.s-1<br>[2E-7,6E-3]m.s-1 |

*Note: Porosity and hydraulic conductivity values are NOT measured but are derived from tables showing statistical averages for lithologies

# Groundwater Information Network
# Réseau d'Information sur les Eaux Souterraines

## Water Well

**Identity :** ca.on.waterWell.7043256
**External identity :** ca.on.waterWell.7043256
**Source :** Ontario Ministry of Environment
**Online resource :** http://www.ene.gov.on.ca/environment/en/subject/wells/index.htm
**Length :** 4.88m
**Elevation :** NaNm
**Well status :** Observation Wells
**Well type :** Unknown
**Well casings :** From 0.00 to 1.83m.
**Sealing components :** From 0.00 to 0.00m; From 1.50 to 1.50m; From 0.30 to 0.30m.
**Screen components :** From 1.83 to 4.88m.

## Well Log

| Depth from (m) | Depth to (m) | GIN Lithology | Original Lithology | Porosity* | Hydraulic Conductivity* |
|---|---|---|---|---|---|
| 0.00 | 0.30 | **Anthropogenic material** | FILL | | |
| 0.30 | 3.66 | **Sand**<br>**Silt** | SAND SILT | [26,53]%<br>[34,61]% | [2E-7,6E-3]m.s-1<br>[1E-9,2E-5]m.s-1 |
| 3.66 | 4.88 | **Silt**<br>**Sand** | SILT SAND | [34,61]%<br>[26,53]% | [1E-9,2E-5]m.s-1<br>[2E-7,6E-3]m.s-1 |

*Note: Porosity and hydraulic conductivity values are NOT measured but are derived from tables showing statistical averages for lithologies

## Water Well

**Identity :** ca.on.waterWell.4900038
**External identity :** ca.on.waterWell.4900038
**Source :** Ontario Ministry of Environment
**Online resource :** http://www.ene.gov.on.ca/environment/en/subject/wells/index.htm
**Length :** 35.66m
**Elevation :** 291.08m
**Water level :** 0.91m
**Water use :** Municipal
**Well status :** Water Supply
**Well type :** Unknown
**Screen components :** From 19.812 to 22.86m.

## Well Log

| Depth from (m) | Depth to (m) | GIN Lithology | Original Lithology | Porosity* | Hydraulic Conductivity* |
|---|---|---|---|---|---|
| 0.00 | 0.61 | **Soil** | TOPSOIL | | |
| 0.61 | 6.10 | **Sand**<br>**Silt** | MEDIUM SAND SILT | [26,53]%<br>[34,61]% | [2E-7,6E-3]m.s-1<br>[1E-9,2E-5]m.s-1 |
| 6.10 | 11.89 | **Clay** | CLAY | [34,57]% | [1E-11,4.7E-9]m.s-1 |
| 11.89 | 14.02 | **Gravel**<br>**Sand**<br>**Clay** | GRAVEL MEDIUM SAND CLAY | [24,44]%<br>[26,53]%<br>[34,57]% | [3E-4,3E-2]m.s-1<br>[2E-7,6E-3]m.s-1<br>[1E-11,4.7E-9]m.s-1 |
| 14.02 | 14.94 | **Sand**<br>**Silt** | MEDIUM SAND SILT | [26,53]%<br>[34,61]% | [2E-7,6E-3]m.s-1<br>[1E-9,2E-5]m.s-1 |
| 14.94 | 17.98 | **Sand** | FINE SAND | [26,53]% | [2E-7,6E-3]m.s-1 |
| 17.98 | 20.42 | **Sand** | MEDIUM SAND | [26,53]% | [2E-7,6E-3]m.s-1 |
| 20.42 | 35.66 | **Sand**<br>**Silt** | FINE SAND SILT | [26,53]%<br>[34,61]% | [2E-7,6E-3]m.s-1<br>[1E-9,2E-5]m.s-1 |

*Note: Porosity and hydraulic conductivity values are NOT measured but are derived from tables showing statistical averages for lithologies

## Water Well

**Identity :** ca.on.waterWell.4909685
**External identity :** ca.on.waterWell.4909685
**Source :** Ontario Ministry of Environment
**Online resource :** http://www.ene.gov.on.ca/environment/en/subject/wells/index.htm
**Length :** 7.60m
**Elevation :** NaNm
**Well status :** Observation Wells
**Well type :** Unknown
**Well casings :** From 0.00 to 4.30m.
**Sealing components :** From 0.30 to 0.30m; From 3.50 to 3.50m; From 0.00 to 0.00m.
**Screen components :** From 4.3 to 7.30m.

## Well Log

| Depth from (m) | Depth to (m) | GIN Lithology | Original Lithology | Porosity* | Hydraulic Conductivity* |
|---|---|---|---|---|---|
| 0.00 | 0.40 | **Soil** | TOPSOIL | | |
| 0.40 | 1.50 | **Silt**<br>**Clay** | SILT CLAYEY | [34,61]%<br>[34,57]% | [1E-9,2E-5]m.s-1<br>[1E-11,4.7E-9]m.s-1 |
| 1.50 | 3.00 | **Silt**<br>**Sand** | SILT FINE SAND | [34,61]%<br>[26,53]% | [1E-9,2E-5]m.s-1<br>[2E-7,6E-3]m.s-1 |
| 3.00 | 7.60 | **Silt**<br>**Sand**<br>**Clay** | SILT SAND CLAY | [34,61]%<br>[26,53]%<br>[34,57]% | [1E-9,2E-5]m.s-1<br>[2E-7,6E-3]m.s-1<br>[1E-11,4.7E-9]m.s-1 |

*Note: Porosity and hydraulic conductivity values are NOT measured but are derived from tables showing statistical averages for lithologies

## Water Well

**Identity :** ca.on.waterWell.4900675
**External identity :** ca.on.waterWell.4900675
**Source :** Ontario Ministry of Environment
**Online resource :** http://www.ene.gov.on.ca/environment/en/subject/wells/index.htm
**Length :** 48.77m
**Elevation :** 292.61m
**Water level :** 21.95m
**Water use :** Domestic
**Well status :** Water Supply
**Well type :** Unknown

## Well Log

| Depth from (m) | Depth to (m) | GIN Lithology | Original Lithology | Porosity* | Hydraulic Conductivity* |
|---|---|---|---|---|---|
| 0.00 | 1.83 | **Clay** | CLAY | [34,57]% | [1E-11,4.7E-9]m.s-1 |
| 1.83 | 7.01 | **Sand** | MEDIUM SAND | [26,53]% | [2E-7,6E-3]m.s-1 |
| 7.01 | 24.38 | **Clay** | CLAY | [34,57]% | [1E-11,4.7E-9]m.s-1 |
| 24.38 | 36.88 | **Sand** | QUICKSAND | [26,53]% | [2E-7,6E-3]m.s-1 |
| 36.88 | 46.33 | **Clay** | CLAY | [34,57]% | [1E-11,4.7E-9]m.s-1 |
| 46.33 | 48.77 | **Sand** | FINE SAND | [26,53]% | [2E-7,6E-3]m.s-1 |

*Note: Porosity and hydraulic conductivity values are NOT measured but are derived from tables showing statistical averages for lithologies

Groundwater Information Network
Réseau d'Information sur les Eaux Souterraines

## Water Well

**Identity :** ca.on.waterWell.4900032
**External identity :** ca.on.waterWell.4900032
**Source :** Ontario Ministry of Environment
**Online resource :** http://www.ene.gov.on.ca/environment/en/subject/wells/index.htm
**Length :** 37.49m
**Elevation :** 289.56m
**Water level :** 1.22m
**Water use :** Not Used
**Well status :** Test Hole
**Well type :** Unknown

## Well Log

| Depth from (m) | Depth to (m) | GIN Lithology | Original Lithology | Porosity* | Hydraulic Conductivity* |
|---|---|---|---|---|---|
| 0.00 | 0.61 | **Sand** | MEDIUM SAND | [26,53]% | [2E-7,6E-3]m.s-1 |
| 0.61 | 4.88 | **Unconsolidated material**<br>**Sand** | MUCK MEDIUM SAND | [26,53]% | [2E-7,6E-3]m.s-1 |
| 4.88 | 5.49 | **Clay** | CLAY | [34,57]% | [1E-11,4.7E-9]m.s-1 |
| 5.49 | 14.02 | **Sand** | QUICKSAND | [26,53]% | [2E-7,6E-3]m.s-1 |
| 14.02 | 14.63 | **Sand**<br>**Gravel** | QUICKSAND GRAVEL | [26,53]%<br>[24,44]% | [2E-7,6E-3]m.s-1<br>[3E-4,3E-2]m.s-1 |
| 14.63 | 18.90 | **Clay**<br>**Sand** | CLAY FINE SAND | [34,57]%<br>[26,53]% | [1E-11,4.7E-9]m.s-1<br>[2E-7,6E-3]m.s-1 |
| 18.90 | 30.78 | **Sand** | QUICKSAND | [26,53]% | [2E-7,6E-3]m.s-1 |
| 30.78 | 31.09 | **Sand**<br>**Gravel** | QUICKSAND GRAVEL | [26,53]%<br>[24,44]% | [2E-7,6E-3]m.s-1<br>[3E-4,3E-2]m.s-1 |
| 31.09 | 36.27 | **Clay**<br>**Sand** | CLAY MEDIUM SAND | [34,57]%<br>[26,53]% | [1E-11,4.7E-9]m.s-1<br>[2E-7,6E-3]m.s-1 |
| 36.27 | 37.49 | **Diamicton**<br>**Shale** | HARDPAN SHALE | [1,10]% | [1E-13,2E-9]m.s-1 |

*Note: Porosity and hydraulic conductivity values are NOT measured but are derived from tables showing statistical averages for lithologies

# Groundwater Information Network
# Réseau d'Information sur les Eaux Souterraines

## Water Well

**Identity :** ca.on.waterWell.4905724
**External identity :** ca.on.waterWell.4905724
**Source :** Ontario Ministry of Environment
**Online resource :** http://www.ene.gov.on.ca/environment/en/subject/wells/index.htm
**Length :** 26.21m
**Elevation :** 304.80m
**Water level :** 3.66m
**Water yield :** 27.28lpm
**Water use :** Livestock
**Well status :** Water Supply
**Well type :** Unknown
**Screen components :** From 25.2984 to 26.21m.

## Well Log

| Depth from (m) | Depth to (m) | GIN Lithology | Original Lithology | Porosity* | Hydraulic Conductivity* |
|---|---|---|---|---|---|
| 0.00 | 8.53 | **Sand**<br>**Clay**<br>**Silt** | SAND CLAY SILTY | [26,53]%<br>[34,57]%<br>[34,61]% | [2E-7,6E-3]m.s-1<br>[1E-11,4.7E-9]m.s-1<br>[1E-9,2E-5]m.s-1 |
| 8.53 | 9.75 | **Clay**<br>**Gravel** | CLAY BOULDERS | [34,57]%<br>[24,44]% | [1E-11,4.7E-9]m.s-1<br>[3E-4,3E-2]m.s-1 |
| 9.75 | 12.19 | **Clay** | CLAY | [34,57]% | [1E-11,4.7E-9]m.s-1 |
| 12.19 | 25.30 | **Clay**<br>**Gravel** | CLAY STONES | [34,57]%<br>[24,44]% | [1E-11,4.7E-9]m.s-1<br>[3E-4,3E-2]m.s-1 |
| 25.30 | 26.21 | **Sand**<br>**Gravel** | SAND STONES | [26,53]%<br>[24,44]% | [2E-7,6E-3]m.s-1<br>[3E-4,3E-2]m.s-1 |

*Note: Porosity and hydraulic conductivity values are NOT measured but are derived from tables showing statistical averages for lithologies

## Water Well

**Identity :** ca.on.waterWell.4910121
**External identity :** ca.on.waterWell.4910121
**Source :** Ontario Ministry of Environment
**Online resource :** http://www.ene.gov.on.ca/environment/en/subject/wells/index.htm
**Length :** 8.00m
**Elevation :** NaNm
**Water use :** Not Used
**Well status :** Abandoned-Other
**Well type :** Unknown
**Well casings :** From 0.00 to 4.50m.
**Sealing components :** From 0.00 to 0.00m; From 6.00 to 6.00m.
**Screen components :** From 4.5 to 8.00m.

## Well Log

| Depth from (m) | Depth to (m) | GIN Lithology | Original Lithology | Porosity* | Hydraulic Conductivity* |
|---|---|---|---|---|---|
| 0.00 | 1.80 | **Anthropogenic material**<br>**Sand** | FILL SANDY | [26,53]% | [2E-7,6E-3]m.s-1 |
| 1.80 | 3.10 | **Peat** | PEAT | | |
| 3.10 | 7.50 | **Silt** | SILT | [34,61]% | [1E-9,2E-5]m.s-1 |
| 7.50 | 8.00 | **Clay**<br>**Silt** | CLAY SILTY | [34,57]%<br>[34,61]% | [1E-11,4.7E-9]m.s-1<br>[1E-9,2E-5]m.s-1 |

*Note: Porosity and hydraulic conductivity values are NOT measured but are derived from tables showing statistical averages for lithologies

# Groundwater Information Network
# Réseau d'Information sur les Eaux Souterraines

## Water Well

**Identity :** ca.on.waterWell.4904257
**External identity :** ca.on.waterWell.4904257
**Source :** Ontario Ministry of Environment
**Online resource :** http://www.ene.gov.on.ca/environment/en/subject/wells/index.htm
**Length :** 31.39m
**Elevation :** 289.56m
**Water level :** 10.06m
**Water yield :** 1022.87lpm
**Water use :** Municipal
**Well status :** Water Supply
**Well type :** Unknown
**Screen components :** From 24.0792 to 30.18m.

## Well Log

| Depth from (m) | Depth to (m) | GIN Lithology | Original Lithology | Porosity* | Hydraulic Conductivity* |
|---|---|---|---|---|---|
| 0.00 | 6.40 | **Clay**<br>**Silt**<br>**Sand** | CLAY SILT SAND | [34,57]%<br>[34,61]%<br>[26,53]% | [1E-11,4.7E-9]m.s-1<br>[1E-9,2E-5]m.s-1<br>[2E-7,6E-3]m.s-1 |
| 6.40 | 10.67 | **Clay**<br>**Silt** | CLAY SILT | [34,57]%<br>[34,61]% | [1E-11,4.7E-9]m.s-1<br>[1E-9,2E-5]m.s-1 |
| 10.67 | 12.80 | **Clay**<br>**Silt** | CLAY SILT | [34,57]%<br>[34,61]% | [1E-11,4.7E-9]m.s-1<br>[1E-9,2E-5]m.s-1 |
| 12.80 | 13.41 | **Diamicton** | HARDPAN | | |
| 13.41 | 20.73 | **Sand**<br>**Clay** | SAND CLAY | [26,53]%<br>[34,57]% | [2E-7,6E-3]m.s-1<br>[1E-11,4.7E-9]m.s-1 |
| 20.73 | 21.64 | **Diamicton** | HARDPAN | | |
| 21.64 | 22.86 | **Sand** | FINE SAND | [26,53]% | [2E-7,6E-3]m.s-1 |
| 22.86 | 31.39 | **Sand** | FINE SAND | [26,53]% | [2E-7,6E-3]m.s-1 |

*Note: Porosity and hydraulic conductivity values are NOT measured but are derived from tables showing statistical averages for lithologies

Groundwater Information Network
Réseau d'Information sur les Eaux Souterraines

## Water Well

**Identity :** ca.on.waterWell.4905698
**External identity :** ca.on.waterWell.4905698
**Source :** Ontario Ministry of Environment
**Online resource :** http://www.ene.gov.on.ca/environment/en/subject/wells/index.htm
**Length :** 18.29m
**Elevation :** 307.85m
**Water level :** 6.10m
**Water use :** Domestic
**Well status :** Water Supply
**Well type :** Unknown

## Well Log

| Depth from (m) | Depth to (m) | GIN Lithology | Original Lithology | Porosity* | Hydraulic Conductivity* |
|---|---|---|---|---|---|
| 0.00 | 0.61 | **Soil** | TOPSOIL | | |
| 0.61 | 6.10 | **Sand**<br>**Clay** | SAND CLAY | [26,53]%<br>[34,57]% | [2E-7,6E-3]m.s-1<br>[1E-11,4.7E-9]m.s-1 |
| 6.10 | 15.24 | **Clay**<br>**Gravel**<br>**Sand** | CLAY GRAVEL SAND | [34,57]%<br>[24,44]%<br>[26,53]% | [1E-11,4.7E-9]m.s-1<br>[3E-4,3E-2]m.s-1<br>[2E-7,6E-3]m.s-1 |
| 15.24 | 18.29 | **Diamicton** | HARDPAN | | |

*Note: Porosity and hydraulic conductivity values are NOT measured but are derived from tables showing statistical averages for lithologies

## Water Well

**Identity :** ca.on.waterWell.4907104
**External identity :** ca.on.waterWell.4907104
**Source :** Ontario Ministry of Environment
**Online resource :** http://www.ene.gov.on.ca/environment/en/subject/wells/index.htm
**Length :** 12.80m
**Elevation :** NaNm
**Water level :** 5.49m
**Water yield :** 9.09lpm
**Water use :** Domestic
**Well status :** Water Supply
**Well type :** Unknown

## Well Log

| Depth from (m) | Depth to (m) | GIN Lithology | Original Lithology | Porosity* | Hydraulic Conductivity* |
|---|---|---|---|---|---|
| 0.00 | 0.30 | **Soil**<br>**Unknown material** | TOPSOIL HARD | | |
| 0.30 | 6.10 | **Clay**<br>**Unknown material** | CLAY HARD | [34,57]% | [1E-11,4.7E-9]m.s-1 |
| 6.10 | 12.80 | **Sand**<br>**Unknown material** | SAND LOOSE | [26,53]% | [2E-7,6E-3]m.s-1 |

*Note: Porosity and hydraulic conductivity values are NOT measured but are derived from tables showing statistical averages for lithologies

## APPENDIX D

## Geotechnical Program

Azure Group

# A SOIL INVESTIGATION FOR PROPOSED COMMERCIAL BUILDING

## 16054 AND 16060 AIRPORT ROAD CALEDON (KLEINBURG), ONTARIO

| | |
|---|---|
| **PREPARED FOR** | **2610808 ONTARIO LTD.** |
| **ADDRESS** | **60 LACOSTE BOULEVARD,<br>UNIT 201<br>BRAMPTON, ONTARIO<br>L6P 2K2** |
| **PROJECT NO.** | **1906-003** |
| **DATE** | **SEPTEMBER 13, 2019** |

**Azure Group Inc.**
**6751 Professional Court, Suite 201, Mississauga, ON L4V 1Y3, Canada**
T: 416 779 2694 F: 416 907 2694 E: info@azuregroup.ca www.azuregroup.ca

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## DIAGRAMS

# 1.0 INTRODUCTION

In accordance with the written authorization dated on Monday June 10, 2019, from Mr. Sam Ganni of 2610818 Ontario Ltd., a soil investigation was carried out at 16054-16060 Airport Road, in the Town of Caledon, for a proposed commercial development (Tim Hortons).

The purpose of the investigation was to reveal the subsurface conditions at the locations of the proposed buildings and to determine the engineering properties of the disclosed soils for the design and construction of the proposed building.

The findings and resulting geotechnical recommendations are presented in this Report.

The hydrological study must be consulted when examining the groundwater characteristics for this site.

## 2.0 SITE AND PROJECT DESCRIPTION

The Town of Caledon is situated on the Markham-Peel till plain where drift dominates the soil stratigraphy. In places, lacustrine sands, silts, silty clay and drift reworked by the water action of Peel Ponding (glacial lake) have modified the soil stratigraphy.

The subject site extends into two properties with municipal addresses of 16054 and 16060 airport road in the Town of Caledon. The site is currently occupied by two abandoned dwelling and associated landscaping, green areas along with heavy bushes and mature trees.

The project consists of a single-story commercial development with associated access road, including a drive through, landscaping area and parking lot. It is understood that the proposed project will be provided with municipal services and roadways meeting the municipal standards.

## 3.0 FIELD WORK

The field work, consisting of 5 boreholes to the sampling depths 6.55 m, was performed and concluded on July 24, 2019, at the locations shown on the Borehole Location Plan, Drawing No. 1. Monitoring wells (MW) 1, 3 and 4 were installed in the boreholes for the associated hydrological scope of work.

The holes were advanced at intervals to the sampling depths by a track mounted, continuous-flight power-auger machine equipped for soil sampling. Standard Penetration Tests, using the procedures described on the enclosed "List of Abbreviations and Terms", were performed at the sampling depths. The test results are recorded as the Standard Penetration Resistance (or 'N' values) of the subsoil. The relative density of the granular strata and the consistency of the cohesive strata are inferred from the 'N' values. Split-spoon samples were recovered for soil classification and laboratory testing.

The field work was supervised and the findings recorded by a Technician under the direction of a geotechnical engineer.

The elevation at each of the borehole locations was determined with reference to the catch basin shown on Drawing No. 1 and with the elevation of 290.95 masl.

## 4.0 SUBSURFACE CONDITIONS

Detailed descriptions of the encountered subsurface conditions are presented on the Borehole Logs, comprising Figures 1 to 5, inclusive, and the engineering properties of the disclosed soils are discussed herein.

This investigation has disclosed that beneath a layer of topsoil full, the site is underlain by a complex stratigraphy of silty fine sand and sandy silt, with localized deposits of silty sand till found at various locations and depths throughout the site.

### 4.1 Topsoil and Topsoil Fill (All Boreholes)

The revealed topsoil / topsoil fill ranges in thickness from 13-42 cm. It is dark brown in colour, indicating it contains an appreciable amount of roots and humus. These materials are unstable and compressible under loads; therefore, the topsoil is considered to be void of engineering value. Due to its humus content, it may produce volatile gases and may generate an offensive odour under anaerobic conditions. Therefore, the topsoil fill must not be buried below any structures or deeper than 1.2 m below the finished grade so it will not have an adverse impact on the environmental well-being of the developed area.

Since the topsoil fill is considered void of engineering value, it can only be used for general landscaping and landscape contouring purposes. A fertility analysis should be carried out to determine the suitability of the topsoil fill for general planting material.

## 4.2 Silty Fine Sand (All Boreholes) and Sandy Silt (Borehole 3)

The silty fine sand was encountered in all boreholes at varying depths throughout the site beneath the topsoil fill veneer and. The sandy silt deposits were encountered at in the form of silt seams in different depths throughout the site and it was sampled in borehole 3. The silty fine sand and sandy silt deposits contain a trace to some clay and occasional gravel with occasional wet sand and silt seams and layers. Some topsoil and root inclusions were found in the upper zone within the deposits beneath the topsoil layer; rock fragments were also noted within the deposits at varying depths. The sorted structures indicate that the silty fine sand and sandy silt are glaciolacustrine deposits.

The soils within depths ranging from 1.5± to 2.44± m from the prevailing ground surface have generally been weathered in the majority of the boreholes.

The obtained ‘N’ values range from 3 to 18, with a median of 10 blows per 30 cm of penetration. These values indicate that the relative density of the silty fine sand and sandy silt is very loose to compact, being generally compact. The very loose to loose deposits are restricted to the badly weathered zone.

The natural water content values of the samples were determined, and the results are plotted on the Borehole Logs; the values range from 4.7% to 25.5%, with a median of 15% for the silty fine sand; while the sandy silt value was 19.8% and 21.3%. This indicates that the deposits are in a damp to saturated, generally wet.

Due to its pervious nature, some water drained during sampling; therefore, the determined water content may not represent the true value of the sand and silt.

The condition, showing that the sand and silt, in places, are water bearing. The samples showed dilatancy when wetted and shaken, indicating that the shear strength of the soils, in a wet state, is susceptible to impact disturbance.

Grain size analyses were performed on 2 representative samples each of the silty fine sand and sandy silt; the results are plotted on Figures 6 and 8, respectively.

Based on the above findings, the deduced engineering properties pertaining to the project are given below:

- High frost susceptibility and soil-adfreezing potential.
- High water erodibility; they will migrate through small openings under low to moderate seepage pressure.
- High capillarity and water retention capability.
- Relatively pervious to pervious, with an estimated coefficient of permeability of $10^{-4}$ cm/sec, and runoff coefficients of:

| **Slope** | |
|---|---|
| 0% - 2% | 0.04 to 0.07 |
| 2% - 6% | 0.09 to 0.12 |
| 6% + | 0.13 to 0.18 |

- Frictional soils, their shear strength is primarily derived from internal friction; therefore, their strength is density dependent. Due to their dilatancy, the shear strength of the wet sand and silt is susceptible to impact disturbance; i.e., the disturbance will induce a buildup of pore pressure within the soil mantle, resulting in soil dilation and a reduction of shear strength.

- When excavated, the damp and moist sand and silt will be stable in relatively steep cuts, while the wet sand and silt will slough, run with water seepage and boil under a piezometric head of 0.4 m.
- Fair pavement-supportive materials, with an estimated California Bearing Ratio (CBR) value of 8%.
- Moderately low corrosivity to buried metal, with an estimated electrical resistivity of 5000 ohm·cm.

## 4.3 Silty Sand Till (All Boreholes)

The fine to coarse sand was the only type of soils found in this site, located, in places, beneath the silty fine sand and in places the sandy silt deposits and extended to the maximum depth of investigation. The sand till contains traces of clay and gravel. It contains occasional wet sand and silt seams and layers, cobbles and boulders. The till is amorphous in structure, showing it is a glacial deposit and has been partially reworked by the past glaciation.

The obtained ‘N’ value varies from 9 to 22, with a median of 15 blows per 30 cm of penetration. indicating that the relative density of the till is loose to compact.

The natural water content values of the silty sand till were determined, and the results are plotted on the Borehole Logs; the values range from 12.1% to 17.5%, with a median of 15% indicating that the till is in a wet condition.

Due to its pervious nature, some water drained during sampling; therefore, the determined water content may not represent the true value of the sand.

A grain size analysis was performed on 3 representative sample. The result is plotted on Figures 7, 9 and 10.

Based on the above findings, the following engineering properties are deduced:

- High frost susceptibility and moderately high water erodibilty.
- Pervious, with an estimated coefficient of permeability of $10^{-4}$ cm/sec, and runoff coefficients of:

| SLOPE | |
|---|---|
| 0% - 2% | 0.04 |
| 2% - 6% | 0.09 |
| 6% + | 0.13 |

- A frictional soil, its shear strength is primarily derived from internal friction, and is augmented by cementation; therefore, its strength is density dependent.
- It will be stable in steep cuts; however, under prolonged exposure, localized sheet collapse will likely occur.
- A fair pavement-supportive material, with an estimated CBR value of 8%.
- Moderately low corrosivity to buried metal, with an estimated electrical resistivity of 5000 ohm·cm.

## 4.4 Compaction Characteristics of the Revealed Soils

The obtainable degree of compaction is primarily dependent on the soil moisture and, to a lesser extent, on the type of compactor used and the effort applied. As a general guide, the typical water content values of the revealed soils for Standard Proctor compaction are presented in Table 1.

**Table 1 - Estimated Water Content for Compaction**

| Soil Type | Determined Natural Water Content (%) | Estimated Water Content (%) for Standard Proctor Compaction | |
|---|---|---|---|
| | | 100% (optimum) | Range for 95% or + |
| Silty Fine Sand | 4.7 to 25.5 (median 15) | 11 | 7 to 15 |
| Sandy Silt | 19.8 and 21.3 | 12 | 8 to 16 |
| Silty Sand Till | 12.1 to 17.5 (median 15) | 11 | 7 to 15 |

The above values show that the in-situ soils are generally suitable for a 95% or + Standard Proctor compaction. However, a portion of the silty fine sand are too dry and will require wetting prior to structural compaction. In addition, a portion of the silty fine sand and silty sand till, and a majority of the sandy silt are too wet and will require aeration or mixing with drier soils prior to structural compaction. Aeration can be achieved by spreading the soils thinly on the ground in the dry, warm weather.

The sand can be compacted by a smooth drum roller, with or without vibration, depending on the water content of the soils being compacted. The lifts for compaction should be limited to 20 cm, or to a suitable thickness as assessed by test strips performed by the equipment which will be used at the time of construction.

When compacting the sands on the dry side of the optimum, the compactive energy will frequently bridge over the chunks in the soil and be transmitted laterally into the soil mantle. Therefore, the lifts must be limited to 20 cm or less (before compaction). It is difficult to monitor the lifts of backfill placed in deep trenches; therefore, it is preferable that the compaction of backfill at depths over 1.0 m below the subgrade be carried out on the wet side of the optimum. This would allow a wider latitude of lift thickness.

If the compaction of the soils is carried out with the water content within the range for 95% Standard Proctor dry density but on the wet side of the optimum, the surface of the compacted soil mantle will roll under the dynamic compactive load. This is unsuitable for pavement construction since each component of the pavement structure is to be placed under dynamic conditions which will induce the rolling action of the subgrade surface and cause structural failure of the new pavement. The slab-on-grade, foundations or bedding of the underground services will be placed on a subgrade which will not be subjected to impact loads. Therefore, the structurally compacted soil mantle with the water content on the wet side or dry side of the optimum will provide adequate subgrade strength for the project construction.

The presence of boulders in the soils will prevent transmission of the compactive energy into the underlying material to be compacted. If an appreciable amount of boulders over 15 cm in size is mixed in the material, it must either be sorted or must not be used for structural backfill and/or construction of engineered fill.

# 5.0 GROUNDWATER CONDITIONS

The boreholes were checked for the presence of groundwater and the occurrence of cave-in upon their completion. All boreholes remained dry upon completion of the investigation. The soil colour remains brown to the maximum investigated depth, indicating that the soils have oxidized. The groundwater level will fluctuate with the seasons. Cave-in occurred in all boreholes at depths various from 1.8+/- m, to 5.5+/- m below the prevailing ground surface.

The hydrological study must be consulted when examining the groundwater characteristics for this site.

If encountered, the groundwater yield from the silty sand till will be slight to some. The yield, if encountered, from the sands and silts is expected to be moderate to appreciable and likely persistent.

## 6.0 DISCUSSION AND RECOMMENDATIONS

This investigation has disclosed that beneath a layer of topsoil full, the site is underlain by a complex stratigraphy of silty fine sand and sandy silt, with localized deposits of silty sand till found at various locations and depths throughout the site.

All boreholes remained dry upon completion of the investigation. The soil colour remains brown to the maximum investigated depth, indicating that the soils have oxidized. Cave-in occurred in all boreholes at depths various from 1.8+/- m, to 5.5+/- m below the prevailing ground surface.

The groundwater level will fluctuate with the seasons.

The Hydrological study must be consulted when examining the groundwater characteristics for this site.

If encountered, the groundwater yield from the silty sand till will be slight to some. The yield, if encountered, from the sands and silts is expected to be moderate to appreciable and likely persistent.

The geotechnical findings which warrant special consideration are presented below:

1. The revealed topsoil fill, 13 to 42 cm thick, will generate volatile gases under anaerobic conditions, is unsuitable for engineering applications and must be stripped.

For the environmental as well as the geotechnical well-being of the future development, it should not be buried over 1.2 m below the proposed finished grade or below any structures. A fertility analysis must be performed to determine the suitability of the topsoil for planting and sodding purposes.

2. The sound natural soils below the topsoil fill, loose and weathered soils are suitable for normal spread and strip footing construction. Due to the presence of these materials, the footing subgrade must be inspected by a geotechnical engineer, or a geotechnical technician under the supervision of a geotechnical engineer, to ensure that its condition is compatible with the design of the foundation.

3. Most of the in situ soils are high in soil-adfreezing potential. Special measures must be implemented to minimize the risk of damage to the foundations caused by frost action.

4. Perimeter subdrains and dampproofing of the foundation walls will be required. The subdrains should be shielded by a fabric filter to prevent blockage by silting, and must be connected to a positive outlet. Depending on the design grade, floor subdrains may be required.

5. For slab-on-grade construction, the slab should be constructed on a granular base, 20 cm thick, consisting of 20-mm Crusher-Run Limestone, or equivalent, compacted to its maximum Standard Proctor dry density.

6. Cut and fill may be required for the site grading and it is generally economical to place engineered fill for normal footing, sewer and pavement construction.

7. A Class 'B' bedding, consisting of compacted 20-mm Crusher-Run Limestone, is recommended for the construction of the underground services. Where water-bearing sand and silts are present, the pipe joints should be leak-proof, or wrapped with an appropriate waterproof membrane. Where extensive dewatering is required, a Class 'A' bedding should be considered.

8. Excavation should be carried out in accordance with Ontario Regulation 213/91.

9. The soils contain occasional cobbles and boulders. Boulders over 15 cm in size must not be used for structural backfill and/or construction of engineered fill.

10. Excavation into the soils containing boulders will require extra effort and the use of a heavy-duty backhoe.

The recommendations appropriate for the project described in Section 2.0 are presented herein. One must be aware that the subsurface conditions may vary between boreholes. Should subsurface variances become apparent during construction, a geotechnical engineer must be consulted to determine whether the following recommendations require revision.

## 6.1 Foundations

Based on the borehole findings, normal spread and strip footings for the proposed buildings can be placed below the topsoil fill and weathered soils onto the sound native soils or onto engineered fill. The recommended soil bearing pressures for use in the design of the footings, together with their corresponding suitable founding levels, are presented in Table 2.

**Table 2 - Founding Levels**

| | **Recommended Maximum Allowable Soil Pressure (SLS)/ Factored Ultimate Soil Bearing Pressure (ULS) and Suitable Founding Level** | | | |
|---|---|---|---|---|
| | **100 kPa (SLS) 150 kPa (ULS)** | | **150 kPa (SLS) 225 kPa (ULS)** | |
| **BH No.** | **Depth (m)** | **El. (m)** | **Depth (m)** | **El. (m)** |
| 1 | 1.98 or + | 289.754 or - | - | - |
| 2 | 1.98 or + | 289.279 or - | 5.03 or + | 286.229 or - |
| 3 | 1.98 or + | 291.067 or - | 6.40 or + | 286.647 or - |
| 4 | 1.20 or + | 291.991 or - | - | - |
| 5 | 2.74 or + | 289.734 or - | - | - |

In areas where foundations are to be extended, it may be more cost effective to subexcavate to a size 30% larger than the designed footing width and fill with lean concrete up to the normal footing elevation immediately after the suitable founding soil is exposed. The sides of the excavation should be properly sloped to stabilize the excavation.

In addition, the existing weathered soils can be replaced with and/or upgraded to engineered fill to allow for a Maximum Allowable Soil Bearing Pressure (SLS) of 150 kPa and a Factored Ultimate Soil Bearing Pressure (ULS) of 250 kPa.

The recommended soil pressure (SLS) for normal footings incorporates a safety factor of 3. The total and differential settlements of the footings are estimated to be 25 mm and 15 mm, respectively.

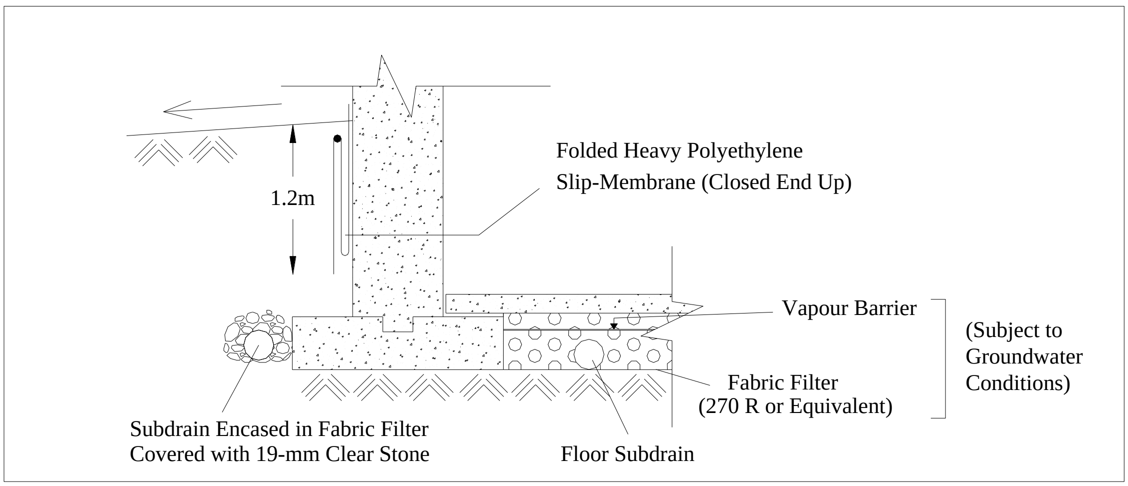
The foundations exposed to weathering, and in unheated areas, should have at least 1.2 m of earth cover for protection against frost action, or must be properly insulated.

It should be noted that if groundwater or groundwater seepage is encountered in footing excavations, or where the subgrade of the normal foundations is found to be wet, the subgrade should be protected by a concrete mud-slab immediately after exposure. This will prevent construction disturbance and costly rectification.

A floor subdrain is required if noticeable seepage is encountered during excavation. Also, a vapour barrier should be installed to prevent upfiltration of soil moisture that may wet the floor. All the subdrains should be encased in a fabric filter to protect them against blockage by silting. The necessity to implement the above recommendations should be further assessed by a geotechnical engineer at the time of construction.

As noted, most of the in situ soils are high in frost heave and soil-adfreezing potential. Where these soils are used for foundation backfill, the foundation walls must be properly shielded with a polyethylene slip-membrane extending below the frost depth, or properly insulated. The slip membrane will allow vertical movement of the heaving soil (due to frost) without imposing structural distress on the foundation. The ground must be graded to direct water away from the structure to minimize the frost heave phenomenon generally associated with the disclosed soils. The recommended scheme is presented in ` 1.

**Diagram 1 - Frost Protection Measures (Foundation)**




The design of footings and foundations must meet the requirements specified in the Ontario Building Code 2006. As a guide, the structure should be designed to resist an earthquake force using Site Classification 'D' (stiff soil).

Due to the presence of topsoil fill, loose soils and earth fill, the footing subgrade must be inspected by either a geotechnical engineer, or a geotechnical technician under the supervision of a geotechnical engineer, to assess its suitability for bearing the designed foundations.

## 6.2 Engineered Fill

In areas where earth fill is required to raise the site or extended footings are required, it is generally more economical to place engineered fill for normal footing, underground services and pavement construction. The engineering requirements for a certifiable fill for pavement construction, municipal services, slab-on-grade, and footings designed with a Maximum Allowable Soil Pressures (SLS) of 150 kPa and a Factored Ultimate Soil Bearing Pressure (ULS) of 250 kPa are presented below:

1. All the organics must be removed, and the subgrade must be inspected and proof-rolled prior to any fill placement.
2. Inorganic soils must be used, and they must be uniformly compacted in lifts of 20 cm thick to 98% or + of their maximum Standard Proctor dry density up to the proposed finished grade and/or slab-on-grade subgrade. The soil moisture must be properly controlled on the wet side of the optimum. If the foundations are to be built soon after the fill placement, the densification process for the engineered fill must be increased to 100% of the maximum Standard Proctor compaction.
3. If imported fill is to be used, the hauler is responsible for its environmental quality and must provide a document to certify that the material is free of hazardous contaminants.
4. If the engineered fill is to be left over the winter months, adequate earth cover, or equivalent, must be provided for protection against frost action.
5. The engineered fill must extend over the entire building area with a 3.0 m offset from the building boundary; the engineered fill envelope and finished elevations must be clearly and accurately defined in the field, and they must be precisely documented by qualified surveyors.

6. Foundations founded partially on engineered fill must be reinforced and designed by a structural engineer to properly distribute the stress induced by the abrupt differential settlement (estimated to be 15± mm) between the natural soils and engineered fill.

7. The engineered fill must not be placed during the period from late November to early April, when freezing ambient temperatures occur either persistently or intermittently. This is to ensure that the fill is free of frozen soils, ice and snow.

8. Where the ground is wet due to subsurface water seepage, an appropriate subdrain scheme must be implemented prior to the fill placement, particularly if it is to be carried out on sloping ground or a bank.

9. Where fill is to be placed on a bank steeper than 1 vertical:3 horizontal, the face of the bank must flattened to 3+ so that it is suitable for safe operation of the compactor and the required compaction can be obtained.

10. The fill operation must be inspected on a full-time basis by a technician under the direction of a geotechnical engineer.

11. The footing and underground services subgrade must be inspected by the geotechnical consulting firm that inspected the engineered fill placement. This is to ensure that the foundations are placed within the engineered fill envelope, and the integrity of the fill has not been compromised by interim construction, environmental degradation and/or disturbance by the footing excavation.

12. Any excavation carried out in certified engineered fill must be reported to the geotechnical consultant who inspected the fill placement in order to document the locations of the excavation and/or to inspect reinstatement of the excavated areas to engineered fill status. If construction on the engineered fill does not commence within a period of 2 years from the date of certification, the condition of the engineered fill must be assessed for re-certification.

13. Despite stringent control in the placement of the engineered fill, variations in soil type and density may occur in the engineered fill. Therefore, the foundations must be properly reinforced and designed by the structural engineer for the project. The total and differential settlements of 25 mm and 15 mm, respectively, should be considered in the design of the foundations founded on engineered fill. In sewer construction, the engineered fill is considered to have the same structural proficiency as a natural inorganic soil.

## 6.3 Slab-On-Grade

The subgrade for slab-on grade construction must consist of sound natural soils or properly compacted inorganic earth fill. In preparation of the subgrade, the topsoil fill and any weathered or deleterious material detected must be removed.

The subgrade should be inspected and assessed by proof-rolling prior to slab-on-grade construction. Where weathered soils are detected, they should be subexcavated, aerated and uniformly compacted to 98% or + of their maximum Standard Proctor dry density.

Any new material for raising the grade should consist of organic-free soils compacted to at least 98% of its maximum Standard Proctor dry density.

The slab should be constructed on a granular base 20 cm thick, consisting of 20-mm Crusher-Run Limestone, or equivalent, compacted to its maximum Standard Proctor dry density.

The slab-on-grade in open areas should be designed to tolerate frost heave, and the grading around the slab-on-grade must be such that it directs runoff away from the surface.

A Modulus of Subgrade Reaction of 25 MPa/m can be used for the design of the floor slab founded on engineered fill and sound natural soils.

Where the floor is found to be wet, floor subdrains should be provided and connected to a positive outlet. A vapour barrier should be placed at the crown level of the floor subdrain to prevent up-filtration or moisture that may wet the floor. The necessity of implementing these measures can be assessed during construction.

The slab at the garage entrances should be insulated with 50-mm Styrofoam, or its thermal equivalent, extending 1.2 m internally. This measure is to prevent cold drafts in the winter from inducing frost action in the subgrade and causing damage to the floor slab.

If the subgrade has been loosened due to construction traffic, it must be proof-rolled before placement of the granular base.

## 6.4 Underground Services

The subgrade for the underground services should consist of sound natural soils or properly compacted organic-free earth fill. Where loose soil is encountered, it should be subexcavated and replaced with bedding material compacted to at least 95% or + of its Standard Proctor compaction.

A Class 'B' bedding is recommended for the underground services construction. The bedding material should consist of compacted 20-mm Crusher-Run Limestone, or equivalent. In areas where more extensive dewatering is required, a Class 'A' bedding should be considered.

Where water-bearing sand and silt occurs, the pipe joints must be leak-proof, or the joints must be wrapped with a waterproof membrane. This is to prevent the migration of fines due to leakage, since this would lead to a loss of subgrade support and subsequent sewer collapse.

In order to prevent pipe floatation when the sewer trench is deluged with water, a soil cover at least equal in thickness to the diameter of the pipe should be in place at all times after completion of the pipe installation.

Openings to subdrains and catch basins should be shielded with a fabric filter to prevent blockage by silting.

Since the sand and silt and sand till has moderately low corrosivity to buried metal, all metal fittings for the underground services should be protected against soil corrosion. In determining the mode of protection, an electrical resistivity of 5000 ohm·cm should be used. This, however, should be confirmed by testing the soil along the water main alignment at the time of services construction.

## 6.5 Septic Tile Bed (if applicable)

The limitations for normal in-ground septic tile bed construction are that the bottom of the absorption trenches, or the surface of a filter medium, be located a minimum of 0.9 m above the highest groundwater level and above rock or soils with a percolation time exceeding 50 min/cm. The soil in the treatment zone should possess acceptable effluent absorption properties expressed in a percolation time of between 1 min/cm and 50 min/cm.

The proposed location of the septic tile bed (if applicable) is in the unknown of the site. However, based on the borehole findings, the site is generally underlain by the sand and silt or silty sand till deposits.

The sand and silt or silty sand till is suitable for in-ground septic tile bed construction. The percolation rate ('T') varies from 15 to 26 min/cm.

A detailed design of the septic tile bed system can be obtained from the Ontario Building Code 2006, published by the Ontario Ministry of Municipal Affairs and housing.

The design of the tile bed must conform to the specifications given in the quoted manual.

To prevent effluent mounding over the groundwater regime, the following criteria must be used for the design of a raised bed:

The effluent should be evenly distributed over the entire tile bed area.

The filter medium should have a minimum thickness of 1.1 m.

In order to enhance an efficient bed operation, the following requirements should be incorporated in the septic tile bed construction.

- All topsoil fill and organics should be stripped from the tile bed area.
- Grading of the surrounding areas should be such that it directs surface runoff away from the tile bed area.
- The bed should be located in an unshaded area.
- The fissured pattern of the underling soil should not be disturbed, as this would reduce its capacity for in-ground effluent absorption.
- In the low areas, the septic tile bed should be elevated so that surface runoff will not pond.

The recommendations presented above are subject to the approval of the local regulatory agency.

## 6.6 Backfilling in Trenches and Excavated Areas

The on-site inorganic soils are generally suitable for trench backfill. However, the soils should be sorted free of any organic inclusions and other deleterious materials prior to the backfilling.

The backfill in the trenches should be compacted to at least 95% of its maximum Standard Proctor dry density. In the zone within 1.0 m below the pavement subgrade, the materials should be compacted with the water content 2% to 3% drier than the optimum, and the compaction should be increased to at least 98% of the respective maximum Standard Proctor dry density. This is to provide the required stiffness for pavement construction. In the lower zone, the compaction should be carried out on the wet side of the optimum; this allows a wider latitude of lift thickness.

Backfill below any slab-on-grade which is sensitive to settlement must be compacted to at least 98% of its maximum Standard Proctor dry density.

In normal construction practice, the problem areas of settlement largely occur adjacent to manholes, catch basins, services crossings, foundation walls and columns. In areas which are inaccessible to a heavy compactor, imported sand backfill should be used. Unless compaction of the backfill is carefully performed, the interface of the native soils and the sand backfill will have to be flooded for a period of several days.

The narrow trenches should be cut at 1 vertical:2 or + horizontal so that the backfill can be effectively compacted. Otherwise, soil arching will prevent the achievement of proper compaction. The lift of each backfill layer should either be limited to a thickness of 20 cm, or the thickness should be determined by test strips.

One must be aware of the possible consequences during trench backfilling and exercise caution as described below:

- When construction is carried out in freezing winter weather, allowance should be made for these following conditions. Despite stringent backfill monitoring, frozen soil layers may inadvertently be mixed with the structural trench backfill. Should the in situ soils have a water content on the dry side of the optimum, it would be impossible to wet the soils due to the freezing condition, rendering difficulties in obtaining uniform and proper compaction. Furthermore, the freezing condition will prevent flooding of the backfill when it is required, such as in a narrow vertical trench section, or when the trench box is removed. The above will invariably cause backfill settlement that may become evident within 1 to several years, depending on the depth of the trench which has been backfilled.

- In areas where the construction is carried out during the winter months, prolonged exposure of the trench walls will result in frost heave within the soil mantle of the walls. This may result in some settlement as the frost recedes, and repair costs will be incurred prior to final surfacing of the new pavement and the slab-on-grade construction.

- To backfill a deep trench, one must be aware that future settlement is to be expected, unless the side of the cut is flattened to at least 1 vertical:1.5+ horizontal, and the lifts of the fill and its moisture content are stringently controlled; i.e., lifts should be no more than 20 cm (or less if the backfilling conditions dictate) and uniformly compacted to achieve at least 95% of the maximum Standard Proctor dry density, with the moisture content on the wet side of the optimum.

- It is often difficult to achieve uniform compaction of the backfill in the lower vertical section of a trench which is an open cut or is stabilized by a trench box, particularly in the sector close to the trench walls or the sides of the box. These sectors must be backfilled with sand. In a trench stabilized by a trench box, the void left after the removal of the box will be filled by the backfill. It is necessary to backfill this sector with sand, and the compacted backfill must be flooded for 1 day, prior to the placement of the backfill above this sector, i.e., in the upper sloped trench section. This measure is necessary in order to prevent consolidation of inadvertent voids and loose backfill which will compromise the compaction of the backfill in the upper section. In areas where groundwater movement is expected in the sand fill mantle, anti-seepage collars should be provided.

## 6.7 Driveways, Sidewalks, Interlocking Stone Pavement and Landscaping

Due to the high frost susceptibility of most of the underlying soils, heaving of the pavement is expected to occur during the cold weather.

The driveways at the entrances to the garages must be backfilled with non-frost susceptible granular material, with a frost taper at a slope of 1 vertical:1 horizontal. The garage floor slabs and interior garage foundations walls must be insulated with 50-mm Styrofoam, or equivalent.

Interlocking stone pavement, slab-on-grade and sidewalks in areas which are sensitive to frost-induced ground movement must be constructed on a free-draining, non-frost-susceptible granular material such as Granular 'B'. It must extend to at least 1.2 m below the sidewalk, slab or pavement surface and be provided with positive drainage, such as weeper subdrains connected to manholes or catch basins. Alternatively, the sidewalks, slab-on-grade and interlocking stone pavement should be properly installed with 50-mm Styrofoam, or equivalent, as approved by a geotechnical engineer.

## 6.8 Pylon Sign and Light Standard for Parking Lot

The founding depth for the pylon sign and light standard must be adequate to provide lateral stability and resistance of the pylon sign with respect to the movement induced by lateral wind loads. It should be noted that, due to the effects of yearly freezing and thawing, the lateral resistance of the soils within the frost depth will be weakened. Therefore, the retaining capacity for the lateral load within the frost depth, i.e., about 1.2 m, should be ignored. The recommended earth pressure coefficients for the soils for use in assessing the passive resistance of the foundations are given in Section 6.10.

The footings must meet the requirements specified in the Ontario Building Code 2006.

The passive lateral earth pressure coefficients (Kp) given in Table 4 can be used for the light standard design.

## 6.9 Pavement Design

The recommended pavement design is given in Table 3.

**Table 3 - Pavement Design**

| Course | Thickness (mm) | | OPS Specifications |
|---|---|---|---|
| | Light Duty | Heavy Duty | |
| Asphalt Surface | 40 | 40 | HL-3<br>HL-3F for driveway |
| Asphalt Binder | 75 | 75 | HL-8 |
| Granular Base | 150 | 150 | 20-mm Crusher-Run Limestone or equivalent |
| Granular Sub-base | 300 | 400 | 50-mm Crusher-Run Limestone or equivalent |

Prior to the placement of the granular bases, the subgrade surface should be proof-rolled. Any very loose or loose subgrade, organics and deleterious materials should be subexcavated and replaced by properly compacted, organic-free earth fill or granular materials. Earth fill/engineered fill used to raise the grade for pavement construction should consist of organic-free soil uniformly compacted to 95% or + of its maximum Standard Proctor dry density.

All the granular bases should be compacted to their maximum Standard Proctor dry density.

In the zone within 1.0 m below the pavement subgrade, the backfill should be compacted to at least 98% of its maximum Standard Proctor dry density, with the water content 2% to 3% drier than the optimum. In the lower zone, a 95% or + Standard Proctor compaction is considered adequate.

The pavement subgrade will suffer a strength regression if water is allowed to saturate the mantle. The following measures should, therefore, be incorporated in the construction procedures and pavement design:

- If the pavement construction does not immediately follow the trench backfilling, the subgrade should be properly crowned and smooth-rolled to allow interim precipitation to be properly drained.
- Areas adjacent to the pavement structures should be properly graded to prevent ponding of large amounts of water during the interim construction period.
- Curb subdrains will be required. The subdrains should consist of filter-sleeved weepers to prevent blockage by silting.
- If the pavement is to be constructed during wet seasons and extensively soft subgrade occurs, the granular sub-base should be thickened in order to compensate for the inadequate strength of the subgrade. This can be assessed during construction.

Along the perimeter where surface runoff may drain onto the pavement, a swale or curbside intercept subdrain system should be installed to prevent infiltrating precipitation from seeping into the granular bases (since this may inflict frost damage on the pavement). The subdrains should consist of filter wrapped weepers, and they should be connected to the catch basins and storm manholes in the paved areas. The subdrains should be backfilled with free-draining granular material.

## 6.10 Soil Parameters

The recommended soil parameters for the project design are given in Table 4.

**TABLE 4 - SOIL PARAMETERS**

| **Unit Weight and Bulk Factor** | | | | |
|---|---|---|---|---|
| | **Unit Weight ($kN/m^3$)** | | **Estimated Bulk Factor** | |
| | **Bulk** | **Submerged** | **Loose** | **Compacted** |
| Silty Fine Sand | 20.5 | 10.8 | 1.20 | 1.00 |
| Sandy Silt | 20.5 | 10.5 | 1.20 | 1.00 |
| Silty Sand Till | 22.5 | 12.5 | 1.33 | 1.03 |

| **Lateral Earth Pressure Coefficients** | | | |
|---|---|---|---|
| | **Active $K_a$** | **At Rest $K_0$** | **Passive $K_p$** |
| Sands, Silt and Silty Sand Till | 0.33 | 0.45 | 3.00 |

## 6.11 Excavation

Excavation should be carried out in accordance with Ontario Regulation 213/91.

Excavations in excess of 1.2 m should be sloped at 1 vertical:1 horizontal for stability. The sides of excavation into earth fill and/or where groundwater is encountered may need to be flattened to 1 vertical:1.5 or + horizontal for stability.

For excavation purposes, the types of soils are classified in Table 5.

**Table 5 - Classification of Soils for Excavation**

| Material | Type |
|---|---|
| Weathered Soils and dewatered Sands and Silts | 3 |
| Saturated Sands and Silts | 4 |

Excavation into the soils containing boulders may require extra effort and the use of a heavy-duty backhoe. Boulders larger than 15 cm in size are not suitable for structural backfill and/or construction of engineered fill.

If encountered, the groundwater yield from the silty sand till will be slight to some and controllable by normal pumping from sumps.

If encountered, excavation below groundwater into water-bearing sand and silts will require pumping from closely spaced sump-wells or, if necessary, the use of a dewatering system. This should be assessed by test pumping prior to the project construction when the intended bottom of excavation is determined. In order to provide a stable subgrade for the services or foundation construction, the groundwater should be depressed at least 0.5 m below the subgrade.

Alternatively, sheeting structures can be installed around the excavation. The sheeting structure should be driven to a depth below the bottom of the excavation at least equal to the height of water above the bed of excavation. The sheeting structure must be properly designed to sustain the earth pressure, hydrostatic pressure and applicable surcharge.

Prospective contractors must be asked to assess the in situ subsurface conditions for soil cuts by digging test pits to at least 0.5 m below the intended bottom of excavation. These test pits should be allowed to remain open for a period of at least 4 hours to assess the trenching conditions.

Respectfully Submitted,

**AZURE GROUP**

**Ahmed Al-Temimi**, M.Sc., P.Eng. QP$_{ESA}$

Senior Engineer

aaltemimi@azuregroup.ca

## 7.0 LIMITATIONS OF REPORT

This report was prepared by Azure Group Inc. for the account of 2610818 Ontario Ltd. and for review by its designated agents and financial institutions and government agencies. The material in it reflects the judgement of Mr. Ahmed Al-Temimi, M.Sc., P.Eng. QP(ESA), in light of the information available to it at the time of preparation.

The report may not be relied upon by any other person or entity without the express written consent of Azure Group Inc. and the Client. Any use that a third party makes of this report, or any reliance on decisions made based on it, is the responsibility of such third parties. Azure Group Inc. accepts no responsibility for damages, if any, suffered by any party as a result of decisions made or actions based on this report.

One must understand that the mandate of Azure Group Inc. is to obtain readily available past and present information pertinent to the subject site for a Geotechnical Investigation only. No other warranty or representation, expressed or implied, as to the accuracy of the information is included or intended by this investigation. Azure Group Inc. makes no other representation whatsoever, including those concerning the legal significance of its findings or as to the other legal matters addressed incidentally in this report, including but not limited to, ownership of any property, or the application of any law to the facts set forth herein. With respect to regulatory compliance issues, regulatory statutes are subject to interpretation.

These interpretations may change over time; thus the Client should review such issues with appropriate legal counsel.

It should be noted that the information supplied in this report is not be sufficient to obtain approval for disposal of excess soil or materials generated during construction. The geotechnical site characterization is a limited sampling of a site and it does not include any chemical testing. Some of the information presented in this report was provided through existing documents and by third parties. Although attempts were made, whenever possible, to obtain a minimum of two confirmatory sources of information, Azure Group Inc. in certain instances has been required to assume that this information provided is accurate. Due to the nature of the investigation and the limited data available, Azure Group Inc. cannot warrant against undiscovered geotechnical liabilities. No other warranty or representation, either expressed or implied, is included or intended in this report.


Approximate Location of the Subject Site

Approximate Location Borehole

Approximate Location of Borehole & Monitoring Well

Source: Peel Region Interactive Maps
© 2018 Peel Maps

| Title | Project | Project No. | Scale | Date | Drawing No. |
|---|---|---|---|---|---|
| Site Location Plan | 16054 - 16060 Airport Road, Caledon, Ontario | 1906-003 | As drawn | September 12, 2019 | 1 |

# APPENDIX A

# BOREHOLE LOGS

BORING FIGURE NUMBER 1
PAGE 1 OF 1

CLIENT: ***2610818 Ontario Ltd.*** PROJECT NAME: *Proposed Commercial Property*
PROJECT NUMBER: *1904-006* PROJECT LOCATION: ***16054-16060 Airport Rd, Caledon, ON.***

*DATE STARTER:* ***July/24/2019*** *DATE COMP.:* ***July/24/2019*** *GROUND ELEVATION:* ***291.734 m*** *HOLE SIZE :* ***50 mm***
*WATER LEVEL: Dry on completion / Cave in at 9'*
*DRILLING METHOD:* ***CONTINUOUS AUGER***
*LOGGED BY:* ***A.R.*** *CHECKED BY:* ***A.R.***
*NOTES:* mointoring well screen found at 6-16'

| Depth (ft) | ELEV (291.734 m) | **Lithology**<br>**Soil Group Name:** modifier, color, moisture, density/consistency, grain size, other descriptors<br>**Rock Description:** modifierm color, hardness/degree of concentration, bedding and joint characteristics, solutions, void conditions. | TYPE | N VALUE | % WATER CONTENT | % RECOVERY | "N" VALUE ___20___40___60 SHEAR STRENGTH | WATER CONTENT % _10__20__30 PL___O ____LL | REMARKS |
|---|---|---|---|---|---|---|---|---|---|
| 1 | | TOPSOIL Fill- 240 mm thick | DO 1 | 7 | 4.7 | 100% | | | |
| 2 | | Silty Fine Sand, trace of clay<br>brown, damp to saturated very loose to compact | | | | | | | |
| 3 | | | DO 2 | 3 | 20.1 | 85% | | | |
| 4 | | | | | | | | | |
| 5 | | weathered | | | | | | | |
| 6 | | | DO 3 | 17 | 21.5 | 100% | | | |
| 7 | | | | | | | | | |
| 8 | | | DO 4 | 18 | 13.2 | 100% | | | GSA SS4<br>gravel: 0%<br>sand: 71%<br>silt: > 25%<br>clay:<4% |
| 9 | | cave in | | | | | | | |
| 10 | | | | | | | | | |
| 11 | | | DO 5 | 17 | 17.9 | 100% | | | |
| 12 | | | | | | | | | |
| 13 | | | | | | | | | |
| 14 | | | | | | | | | |
| 15 | | | | | | | | | |
| 16 | | Silty Sand Till, traces of clay and gravel<br>brown, wet, loose to compact | DO 6 | 17 | 15.1 | 100% | | | GSA SS6<br>gravel: 9%<br>sand: 68%<br>silt: > 18%<br>clay:<5% |
| 17 | | | | | | | | | |
| 18 | | | | | | | | | |
| 19 | | | | | | | | | |
| 20 | | | | | | | | | |
| 21 | | | DO 7 | 9 | 13.9 | 100% | | | |
| | Borehole end at 21'- mointoring well installed | | | | | | | | |

BORING FIGURE NUMBER 2
PAGE 1 OF 1

CLIENT: ***2610818 Ontario Ltd.*** PROJECT NAME: *Proposed Commercial Property*
PROJECT NUMBER: *1904-006* PROJECT LOCATION: ***16054-16060 Airport Rd, Caledon, ON.***

*DATE STARTER:* ***July/24/2019*** *DATE COMP.:* ***July/24/2019*** *GROUND ELEVATION:* ***291.259 m*** *HOLE SIZE :* ***50 mm***
*WATER LEVEL: Dry on completion / Cave in at 6'*
*DRILLING METHOD:* ***CONTINUOUS AUGER***
*LOGGED BY:* ***A.R.*** *CHECKED BY:* ***A.R.***
*NOTES:*

| Depth (ft) | ELEV (291.259 m) | **Lithology**<br>**Soil Group Name:** modifier, color, moisture, density/consistency, grain size, other descriptors<br>**Rock Description:** modifierm color, hardness/degree of concentration, bedding and joint characteristics, solutions, void conditions. | TYPE | N VALUE | % WATER CONTENT | % RECOVERY | "N" VALUE ___20___40___60 SHEAR STRENGTH | WATER CONTENT % _10__20__30 PL___O ____LL | REMARKS |
|---|---|---|---|---|---|---|---|---|---|
| 1 | | TOPSOIL Fill- 420 mm thick | DO 1 | 2 | 19 | 100% | | | |
| 2 | | Silty Fine Sand, trace of clay<br>brown, wet to saturated, loose to compact | | | | | | | |
| 3 | | | DO 2 | 4 | 25.5 | 76% | | | |
| 4 | | | | | | | | | |
| 5 | | | | | | | | | |
| 6 | | cave in / weathered | DO 3 | 14 | 18.6 | 100% | | | |
| 7 | | | | | | | | | |
| 8 | | | DO 4 | 12 | 17.9 | 100% | | | |
| 9 | | | | | | | | | |
| 10 | | | | | | | | | |
| 11 | | | DO 5 | 10 | 15.6 | 11% | | | |
| 12 | | | | | | | | | |
| 13 | | | | | | | | | |
| 14 | | | | | | | | | |
| 15 | | | | | | | | | |
| 16 | | Silty Sand Till, traces of clay and gravel<br>brown, wet, compact | DO 6 | 14 | 15.6 | 100% | | | |
| 17 | | | | | | | | | |
| 18 | | | | | | | | | |
| 19 | | | | | | | | | |
| 20 | | | | | | | | | |
| 21 | | | DO 7 | 22 | 12.1 | 100% | | | |

Borehole end at 21'

BORING FIGURE NUMBER 3



CLIENT: ***2610818 Ontario Ltd.*** *PROJECT NAME:* *Proposed Commercial Property*

PROJECT NUMBER: *1904-006* *PROJECT LOCATION:* ***16054-16060 Airport Rd, Caledon, ON.***

*DATE STARTER:* ***July/24/2019*** *DATE COMP.:* ***July/24/2019*** *GROUND ELEVATION:* ***293.047 m*** *HOLE SIZE* ***: 50 mm***

*WATER LEVEL: Dry on completion / Cave in at 10'*

*DRILLING METHOD:* ***CONTINUOUS AUGER***

*LOGGED BY:* ***A.R.*** *CHECKED BY:* ***A.R.***

*NOTES:* mointoring well screen found at 8-18'

| Depth (ft) | ELEV (293.047 m) | **Lithology**<br>**Soil Group Name:** modifier, color, moisture, density/consistency, grain size, other descriptors<br>**Rock Description:** modifierm color, hardness/degree of concentration, bedding and joint characteristics, solutions, void conditions. | TYPE | N VALUE | % WATER CONTENT | % RECOVERY | "N" VALUE<br>___20___40___60<br>SHEAR STRENGTH | WATER CONTENT %<br>_10__20__30 PL___O ____LL | REMARKS |
|---|---|---|---|---|---|---|---|---|---|
| | | TOPSOIL Fill- 130 mm thick | | | | | | | |
| 1 | | Silty Fine Sand and Silty Sand, trace of clay | DO 1 | 3 | 8 | 63% | | | |
| 2 | | brown, damp to saturated very loose to compact | | | | | | | |
| 3 | | | DO 2 | 4 | 6.6 | 70% | | | |
| 4 | | | | | | | | | |
| 5 | | | | | | | | | |
| 6 | | weathered | DO 3 | 12 | 8.8 | 100% | | | |
| 7 | | | | | | | | | |
| 8 | | wet silt seam | DO 4 | 9 | 21.3 | 100% | | | |
| 9 | | | | | | | | | |
| 10 | | | | | | | | | |
| 11 | | cave in | DO 5 | 10 | 19.8 | 100% | | | GSA SS5<br>gravel: 0%<br>sand: 40%<br>silt: > 55%<br>clay:<5% |
| 12 | | | | | | | | | |
| 13 | | | | | | | | | |
| 14 | | | | | | | | | |
| 15 | | | | | | | | | |
| 16 | | | DO 6 | 13 | 21.2 | 33% | | | |
| 17 | | | | | | | | | |
| 18 | | | | | | | | | |
| 19 | | | | | | | | | |
| 20 | | | | | | | | | |
| 21 | | Silty Sand Till, traces of clay and gravel<br>brown, wet, compact | DO 7 | 22 | 17.5 | 100% | | | |
| | Borehole end at 21'- mointoring well installed | | | | | | | | |

BORING FIGURE NUMBER 4
PAGE 1 OF 1

CLIENT: ***2610818 Ontario Ltd.*** *PROJECT NAME:* *Proposed Commercial Property*
PROJECT NUMBER: *1904-006* *PROJECT LOCATION:* ***16054-16060 Airport Rd, Caledon, ON.***

*DATE STARTER:* ***July/24/2019*** *DATE COMP.:* ***July/24/2019*** *GROUND ELEVATION:* ***293.191 m*** *HOLE SIZE :* ***50 mm***
*WATER LEVEL: Dry on completion / Cave in at 18'*
*DRILLING METHOD:* ***CONTINUOUS AUGER***
*LOGGED BY:* ***A.R.*** *CHECKED BY:* ***A.R.***
*NOTES:* mointoring well screen found at 8-18'

| Depth (ft) | ELEV (293.191 m) | **Lithology**<br>**Soil Group Name:** modifier, color, moisture, density/consistency, grain size, other descriptors<br>**Rock Description:** modifierm color, hardness/degree of concentration, bedding and joint characteristics, solutions, void conditions. | TYPE | N VALUE | % WATER CONTENT | % RECOVERY | "N" VALUE ___20___40___60 SHEAR STRENGTH | WATER CONTENT % _10__20__30 PL___O ____LL | REMARKS |
|---|---|---|---|---|---|---|---|---|---|
| 1 | | TOPSOIL Fill- 200 mm thick | DO 1 | 3 | 5.6 | 100% | • | • | |
| 2 | | Silty Fine Sand, trace of clay | | | | | | | |
| 3 | | brown, damp to wet, very loose to compact | DO 2 | 9 | 4.6 | 65% | • | • | |
| 4 | | | | | | | | | |
| 5 | | weathered | | | | | | | |
| 6 | | | DO 3 | 15 | 8.9 | 100% | • | • | |
| 7 | | | | | | | | | |
| 8 | | | DO 4 | 13 | 19.5 | 100% | • | • | |
| 9 | | | | | | | | | |
| 10 | | | | | | | | | |
| 11 | | | DO 5 | 14 | 16.3 | 100% | • | • | |
| 12 | | | | | | | | | |
| 13 | | | | | | | | | |
| 14 | | | | | | | | | |
| 15 | | | | | | | | | |
| 16 | | Silty Sand Till, traces of clay and gravel<br>brown, wet, loose to compact | DO 6 | 15 | 16.1 | 100% | • | • | |
| 17 | | | | | | | | | |
| 18 | | cave in | | | | | | | |
| 19 | | | | | | | | | GSA SS7 |
| 20 | | | | | | | | | gravel: 2% |
| 21 | | | DO 7 | 9 | 16.4 | 100% | • | • | sand: 78%<br>silt: > 16%<br>clay:<4% |
| | Borehole end at 21'- mointoring well installed | | | | | | | | |

BORING FIGURE NUMBER 5
PAGE 1 OF 1

CLIENT: ***2610818 Ontario Ltd.*** PROJECT NAME: *Proposed Commercial Property*
PROJECT NUMBER: *1904-006* PROJECT LOCATION: ***16054-16060 Airport Rd, Caledon, ON.***

*DATE STARTER:* ***July/24/2019*** *DATE COMP.:* ***July/24/2019*** *GROUND ELEVATION:* ***292.474 m*** *HOLE SIZE :* ***50 mm***
*WATER LEVEL: Dry on completion / Cave in at 13'*
*DRILLING METHOD:* ***CONTINUOUS AUGER***
*LOGGED BY:* ***A.R.*** *CHECKED BY:* ***A.R.***
*NOTES:*

| Depth (ft) | ELEV (292.474 m) | **Lithology**<br>**Soil Group Name:** modifier, color, moisture, density/consistency, grain size, other descriptors<br>**Rock Description:** modifierm color, hardness/degree of concentration, bedding and joint characteristics, solutions, void conditions. | TYPE | N VALUE | % WATER CONTENT | % RECOVERY | "N" VALUE ___20___40___60 SHEAR STRENGTH | WATER CONTENT % _10__20__30 PL___O ____LL | REMARKS |
|---|---|---|---|---|---|---|---|---|---|
| 1 | | TOPSOIL Fill- 280 mm thick | DO 1 | 8 | 7.7 | 100% | | | |
| 2 | | Silty Fine Sand, trace of clay | | | | | | | |
| 3 | | brown, moist to wet, very loose to compact | DO 2 | 7 | 13.5 | 43% | | | |
| 4 | | | | | | | | | |
| 5 | | cave in / weathered | | | | | | | |
| 6 | | gravel layer | DO 3 | 3 | 15.4 | 37% | | | GSA SS3<br>gravel: 7%<br>sand: 42%<br>silt: > 43%<br>clay:<8% |
| 7 | | | | | | | | | |
| 8 | | | DO 4 | 16 | 16.2 | 91% | | | |
| 9 | | | | | | | | | |
| 10 | | | | | | | | | |
| 11 | | | DO 5 | 16 | 18.5 | 72% | | | |
| 12 | | | | | | | | | |
| 13 | | | | | | | | | |
| 14 | | | | | | | | | |
| 15 | | | | | | | | | |
| 16 | | Silty Sand Till, traces of clay and gravel<br>brown, wet, compact | DO 6 | 14 | 14.3 | 100% | | | |
| 17 | | | | | | | | | |
| 18 | | | | | | | | | |
| 19 | | | | | | | | | |
| 20 | | | | | | | | | |
| 21 | | | DO 7 | 11 | 15.5 | 100% | | | |

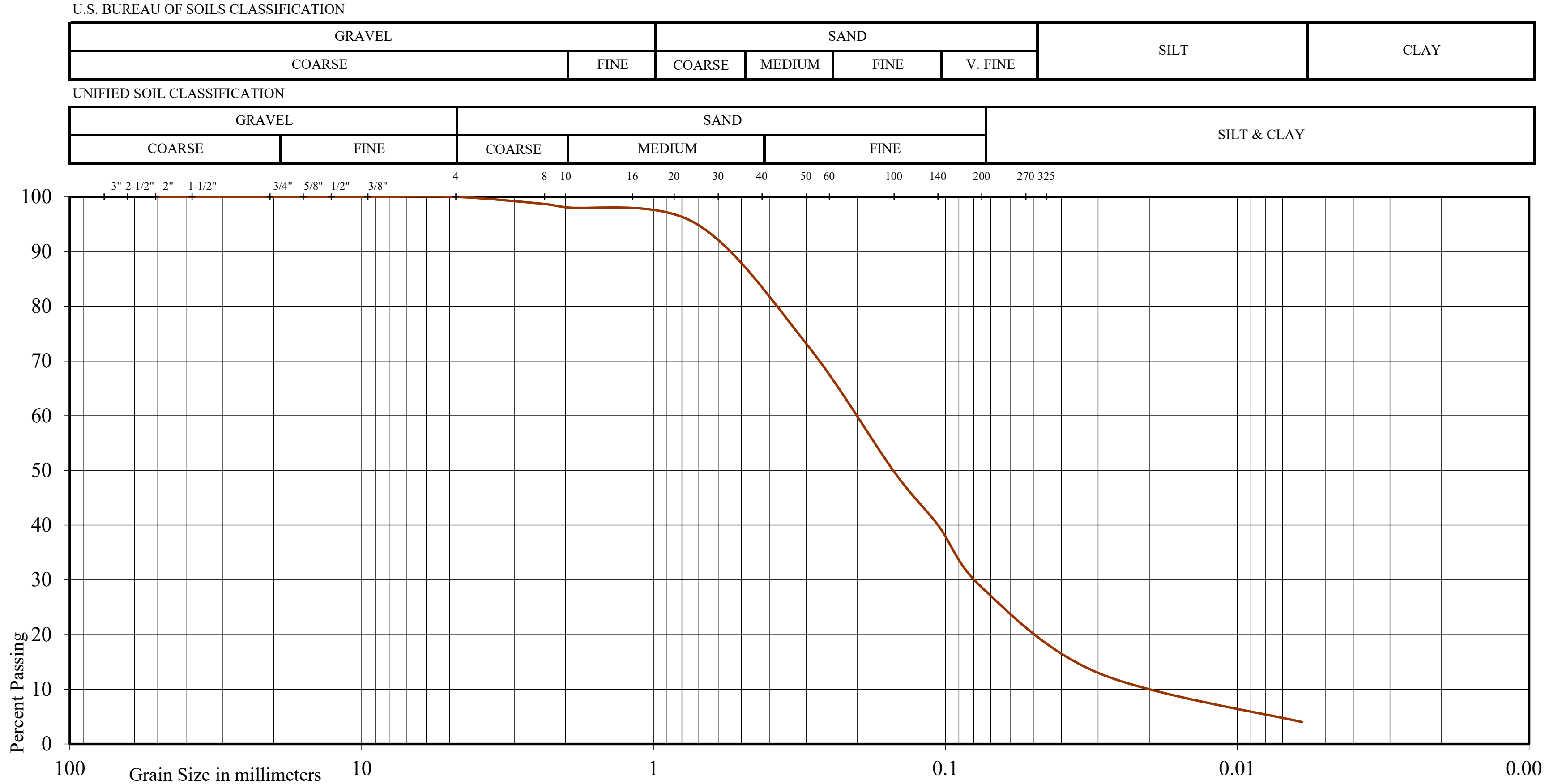
Borehole end at 21'

**APPENDIX B**

**GRAIN SIZE DISTRIBUTION**

# GRAIN SIZE DISTRIBUTION

Reference No: 1904-006




Project: Proposed Commercial Development

Location: 16054-16060 Airport Road

Caledon ON

Borehole No: BH 1

Sample No: SS 4

Depth (m): 2.29-2.74m

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = 13

Estimated Permeability

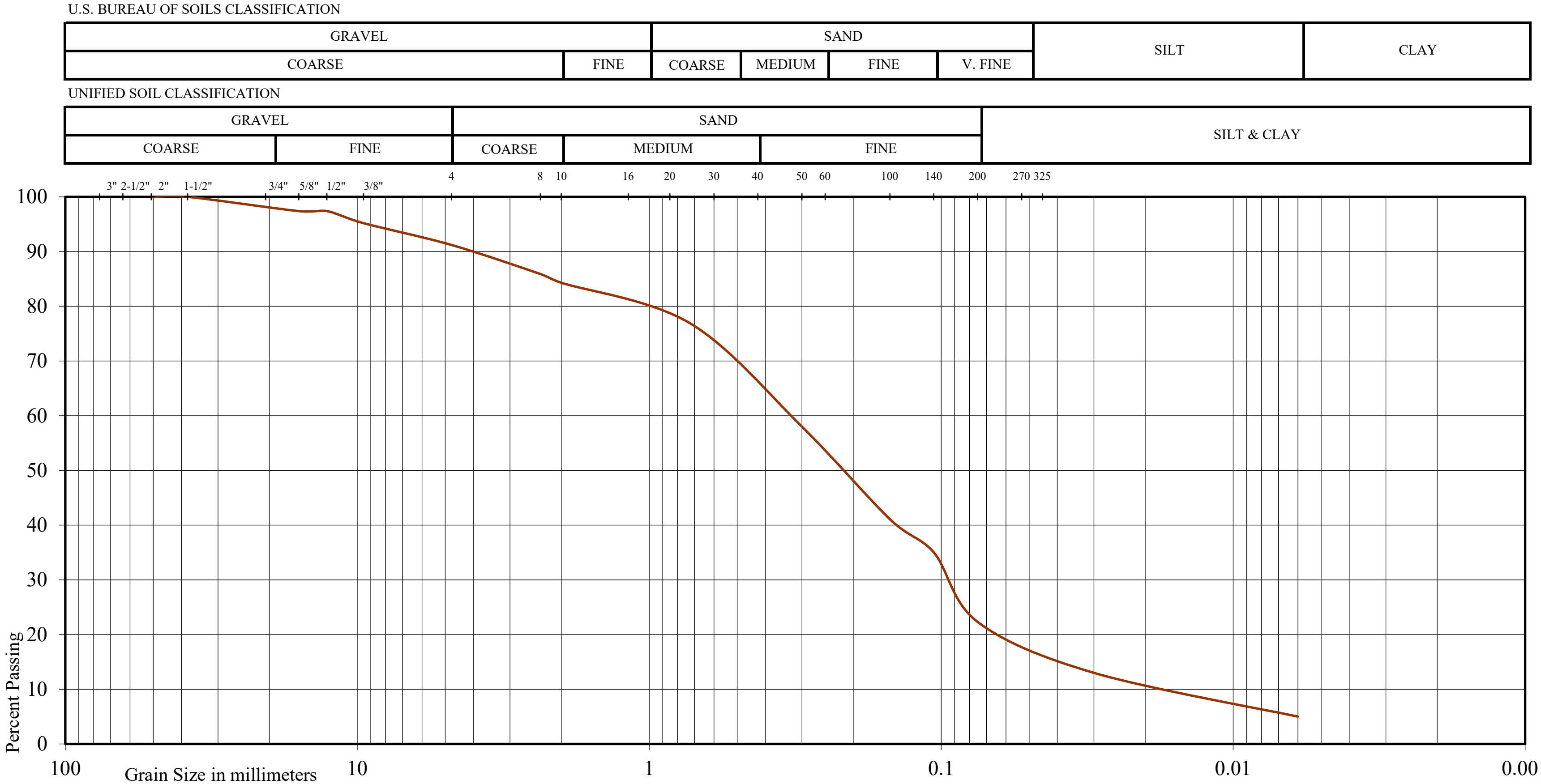
(cm./sec.) = $10^{-4}$

Silty Fine Sand, trace of clay

Figure: 6

# GRAIN SIZE DISTRIBUTION

Reference No: 1904-006




Project: Proposed Commercial Development

Location: 16054-16060 Airport Road

Caledon ON

Borehole No: BH 1

Sample No: SS 6

Depth (m): 4.57-5.03m

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = 15

Estimated Permeability

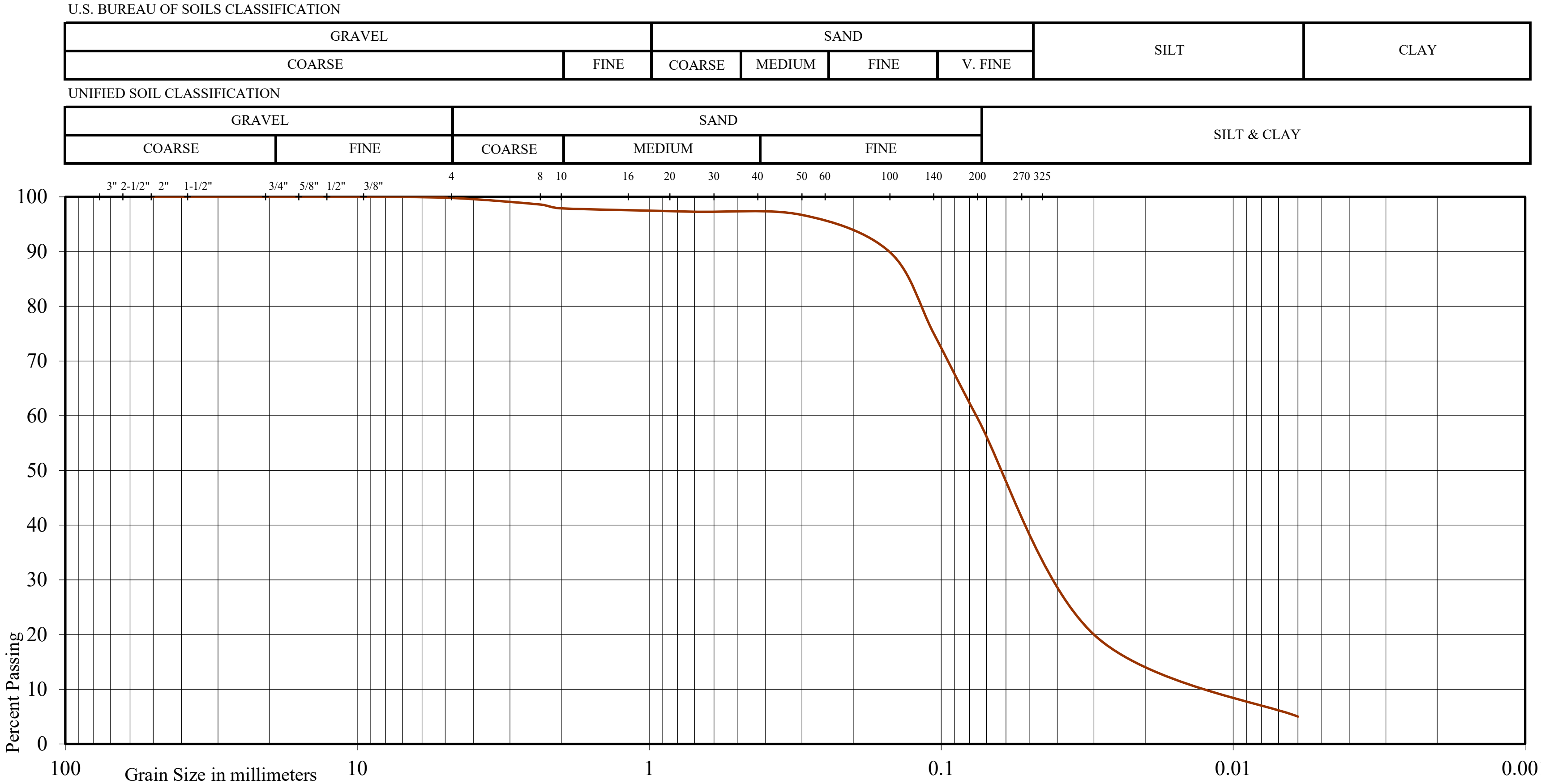
(cm./sec.) = $10^{-4}$

Silty Sand Till, traces of clay and gravel

Figure: 7

# GRAIN SIZE DISTRIBUTION

Reference No: 1904-006




Project: Proposed Commercial Development

Location: 16054-16060 Airport Road

Caledon ON

Borehole No: BH 3

Sample No: SS 5

Depth (m): 3.05-3.51m

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = 20

Estimated Permeability

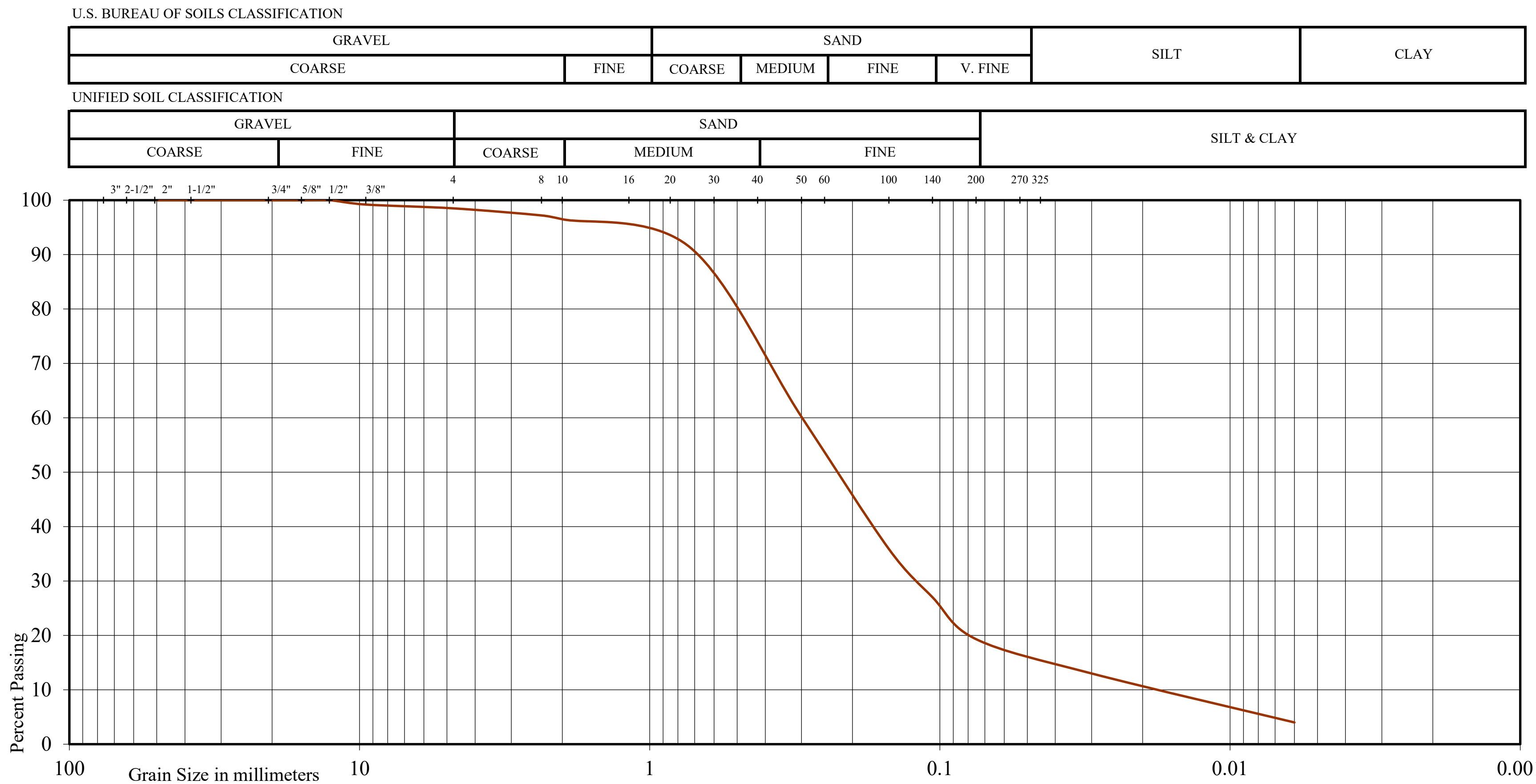
(cm./sec.) = $10^{-4}$

Figure: 8

Sand Silt, trace of clay

# GRAIN SIZE DISTRIBUTION

Reference No: 1904-006




Project: Proposed Commercial Development

Location: 16054-16060 Airport Road

Caledon ON

Borehole No: BH 4

Sample No: SS 7

Depth (m): 6.1-6.55m

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = 16

Estimated Permeability

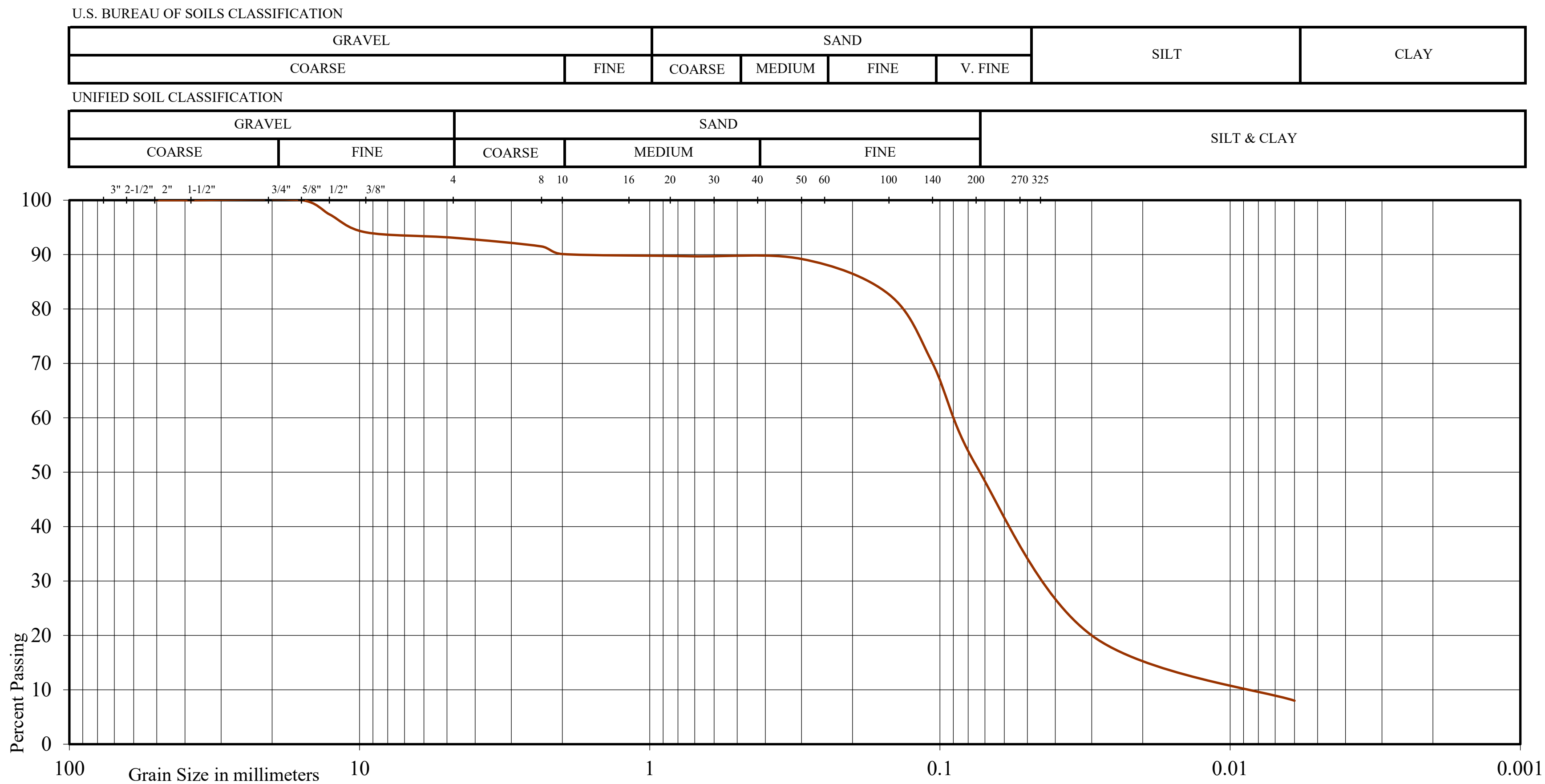
(cm./sec.) = $10^{-4}$

Silty Sand Till, traces of clay and gravel

Figure: 9

# GRAIN SIZE DISTRIBUTION

Reference No: 1904-006




Project: Proposed Commercial Development

Location: 16054-16060 Airport Road

Caledon ON

Borehole No: BH 5

Sample No: SS 3

Depth (m): 1.52-1.98m

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = 15

Estimated Permeability

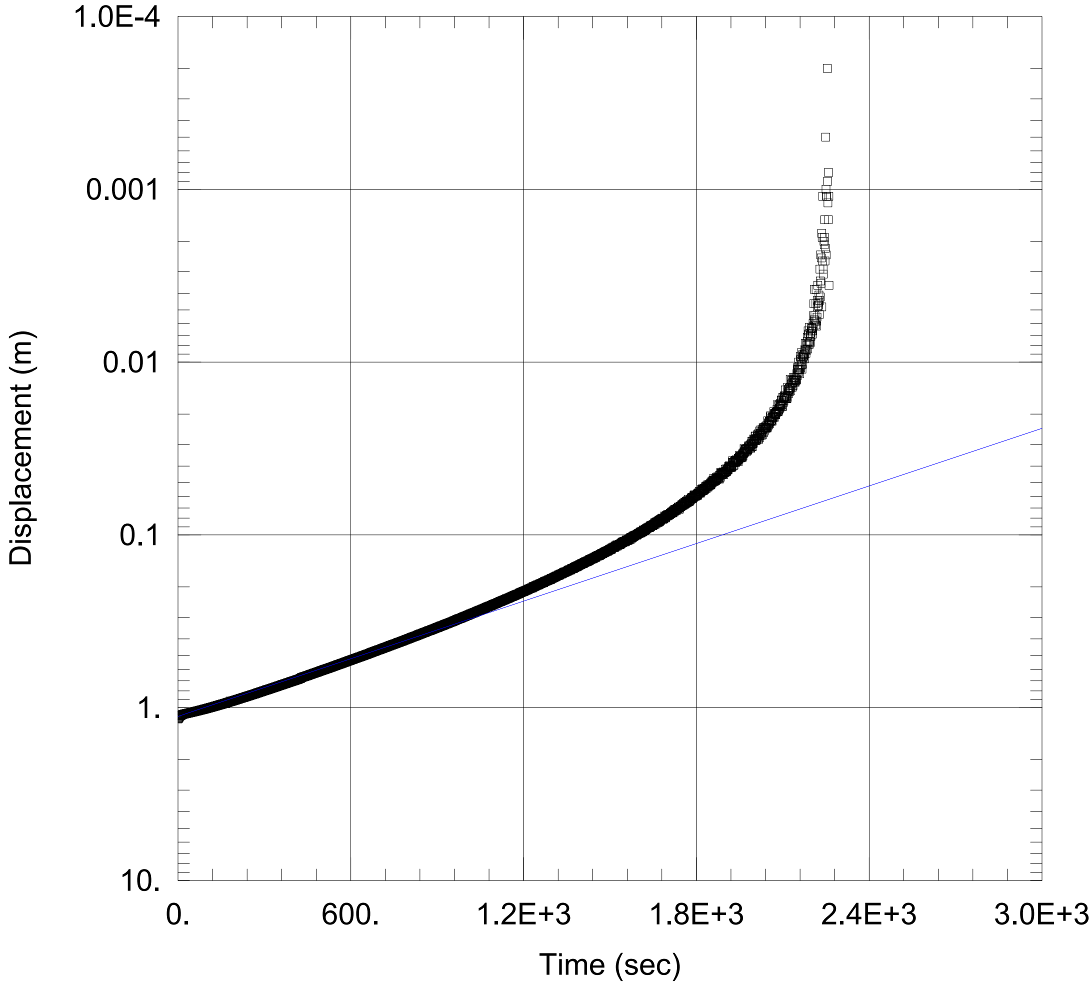
(cm./sec.) = $10^{-4}$

| |
|---|
| Silty Sand Till, traces of clay and gravel |

Figure: 10

## APPENDIX E

## Hydraulic Conductivity Testing




WELL TEST ANALYSIS

Data Set: N:\19 Projects\19-216 Airport Road HydroG\03.0 - Data\03.2 - Field\Slug Tests 0731\BH1.aqt
Date: 09/30/19 Time: 14:01:18

PROJECT INFORMATION

Company: Azimuth Environmental
Project: 19-216
Location: Caledon
Test Well: MW-1
Test Date: July 31

AQUIFER DATA

Saturated Thickness: 1.86 m Anisotropy Ratio (Kz/Kr): 1.

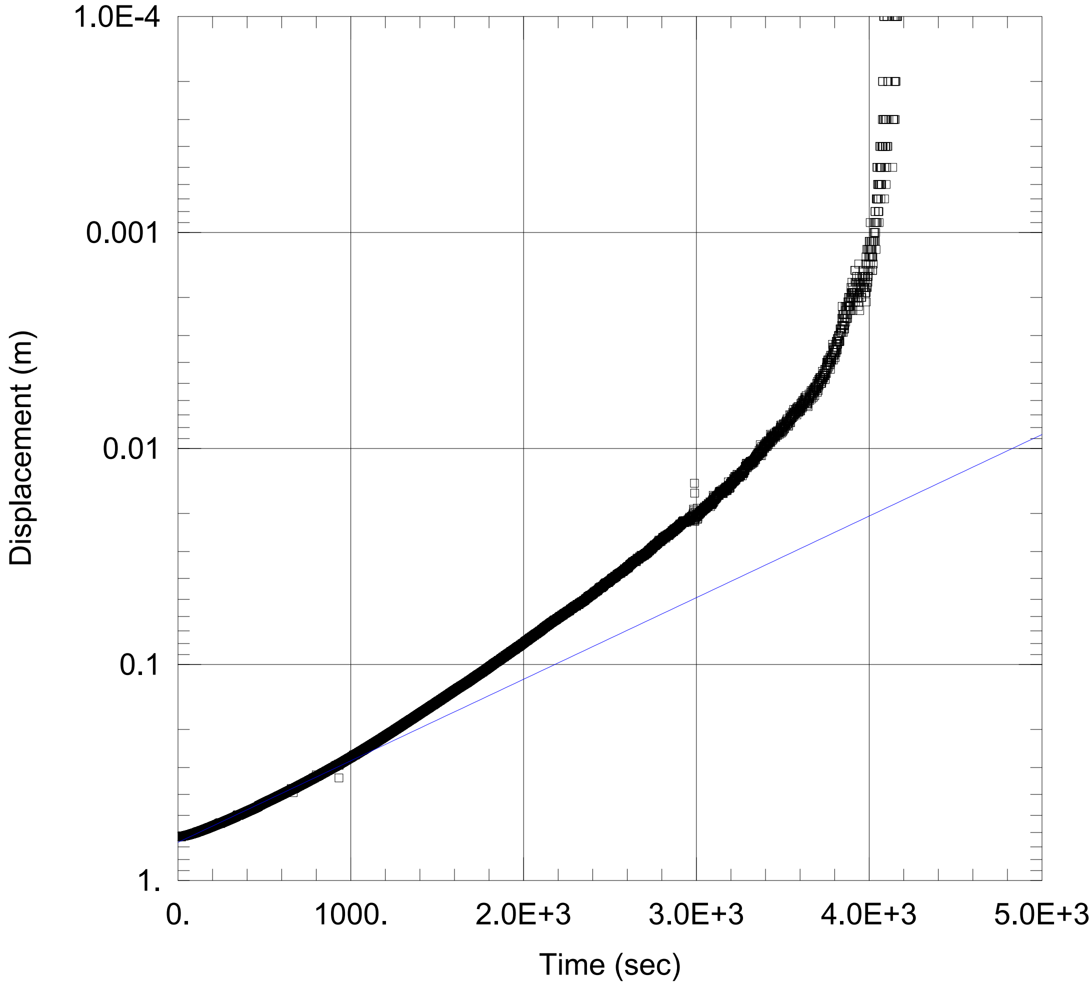
WELL DATA (MW1)

Initial Displacement: 1.162 m Static Water Column Height: 1.86 m
Total Well Penetration Depth: 5.01 m Screen Length: 3.05 m
Casing Radius: 0.025 m Well Radius: 0.025 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Hvorslev

K = 1.139E-6 m/sec y0 = 1.124 m




## WELL TEST ANALYSIS

Data Set: N:\19 Projects\19-216 Airport Road HydroG\03.0 - Data\03.2 - Field\Slug Tests 0731\BH3.aqt
Date: 09/30/19 Time: 14:00:38

## PROJECT INFORMATION

Company: Azimuth Environmental
Project: 19-216
Location: Caledon
Test Well: MW-3
Test Date: July 31

## AQUIFER DATA

Saturated Thickness: 1.17 m Anisotropy Ratio (Kz/Kr): 1.

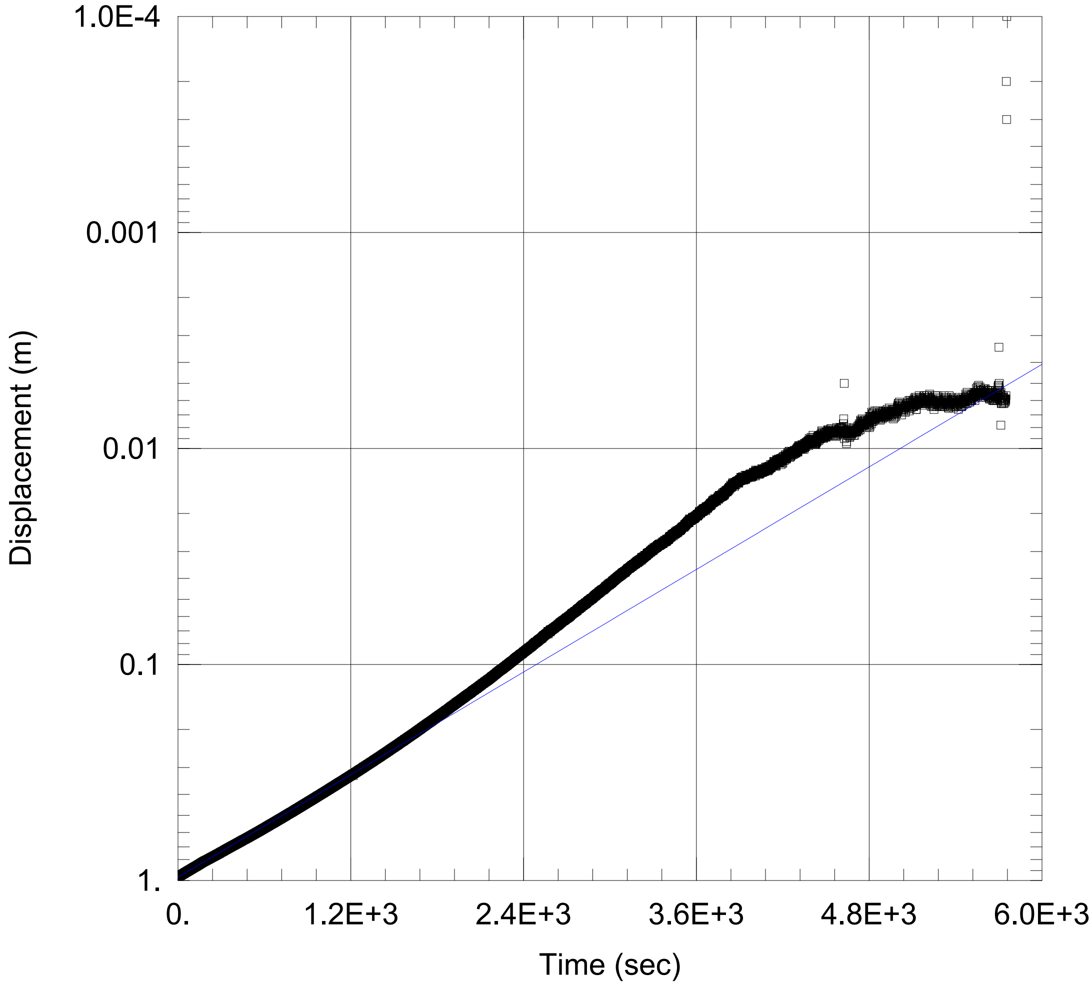
## WELL DATA (MW-3)

Initial Displacement: 0.6295 m Static Water Column Height: 1.17 m
Total Well Penetration Depth: 5.26 m Screen Length: 3.05 m
Casing Radius: 0.025 m Well Radius: 0.025 m

## SOLUTION

Aquifer Model: Unconfined Solution Method: Hvorslev

K = 1.231E-6 m/sec y0 = 0.6667 m




## WELL TEST ANALYSIS

Data Set: N:\19 Projects\19-216 Airport Road HydroG\03.0 - Data\03.2 - Field\Slug Tests 0731\BH4.aqt
Date: 09/30/19 Time: 14:01:35

## PROJECT INFORMATION

Company: Azimuth Environmental
Project: 19-216
Location: Caledon
Test Well: MW-4
Test Date: July 31

## AQUIFER DATA

Saturated Thickness: 1.18 m
Anisotropy Ratio (Kz/Kr): 1.

## WELL DATA (MW4)

Initial Displacement: 0.9605 m
Static Water Column Height: 1.18 m
Total Well Penetration Depth: 5.41 m
Screen Length: 3.05 m
Casing Radius: 0.025 m
Well Radius: 0.025 m

## SOLUTION

Aquifer Model: Unconfined
Solution Method: Hvorslev

K = 1.279E-6 m/sec
y0 = 0.9664 m

**APPENDIX F**

**Water Balance Inforamtion**

## Table A: Pre-Development

| Catchment Designation | Forest | Landscaped Grass | Driveway | Structure | Total |
|---|---|---|---|---|---|
| Area ($m^2$) | 475 | 970 | 190 | 355 | 1,990 |
| Pervious Area ($m^2$) | 475 | 970 | 0 | 0 | 1,445 |
| Impervious Area ($m^2$) | 0 | 0 | 190 | 355 | 545 |
| **Infiltration Factors** | | | | | |
| Topography Infiltration Factor | 0.2 | 0.2 | 0 | 0 | |
| Soil Infiltration Factor | 0.3 | 0.3 | 0 | 0 | |
| Land Cover Infiltration Factor | 0.2 | 0.1 | 0 | 0 | |
| Infiltration Factor | 0.7 | 0.6 | 0 | 0 | |
| Run-Off Coefficient | 0.3 | 0.4 | 1 | 1 | |
| Run-Off From Impervious Surfaces | 0.8 | 0.8 | 0.8 | 0.8 | |
| **Inputs (Per Unit Area)** | | | | | |
| Precipitation (mm/yr) | 896 | 896 | 896 | 896 | 896 |
| Rainfall (mm/yr) | 613 | 613 | 613 | 613 | 613 |
| Run-On (mm/yr) | 0 | 0 | 0 | 0 | 0 |
| Other Inputs (mm/yr) | 0 | 0 | 0 | 0 | 0 |
| **Total Inputs (mm/yr)** | **896** | **896** | **896** | **896** | 896 |
| **Outputs (Per Unit Area)** | | | | | |
| Precipitation Surplus (mm/yr) | 394 | 394 | 717 | 717 | 482 |
| Net Surplus (mm/yr) | 394 | 394 | 717 | 717 | 482 |
| Evapotranspiration (mm/yr) | 502 | 502 | 179 | 179 | 414 |
| Infiltration (mm/yr) | 276 | 236 | 0 | 0 | 181 |
| Surplus Infiltration (mm/yr) | 0 | 0 | 0 | 147 | 26 |
| Total Infiltration (mm/yr) | 276 | 236 | 0 | 147 | 207 |
| Run-Off Pervious Areas (mm/yr) | 118 | 158 | 0 | 0 | 105 |
| Run-Off Impervious Areas (mm/yr) | 0 | 0 | 717 | 570 | 170 |
| Total Run-Off (mm/yr) | 118 | 158 | 717 | 570 | 275 |
| **Total Outputs (mm/yr)** | **896** | **896** | **896** | **896** | **896** |
| **Difference (Inputs - Outputs)** | **0** | **0** | **0** | **0** | **0** |
| **Inputs (Volumes)** | | | | | |
| Precipitation ($m^3$/yr) | 426 | 869 | 170 | 318 | 1,783 |
| Run-On ($m^3$/yr) | 0 | 0 | 0 | 0 | 0 |
| Other Inputs ($m^3$/yr) | 0 | 0 | 0 | 0 | 0 |
| **Total Inputs ($m^3$/yr)** | **426** | **869** | **170** | **318** | **1,783** |
| **Outputs (Volumes)** | | | | | |
| Precipitation Surplus ($m^3$/yr) | 187 | 382 | 136 | 254 | 960 |
| Net Surplus ($m^3$/yr) | 187 | 382 | 136 | 254 | 960 |
| Evapotranspiration ($m^3$/yr) | 238 | 487 | 34 | 64 | 823 |
| Infiltration ($m^3$/yr) | 131 | 229 | 0 | 0 | 360 |
| Surplus Infiltration ($m^3$/yr) | 0 | 0 | 0 | 52 | 52 |
| **Total Infiltration ($m^3$/yr)** | **131** | **229** | **0** | **52** | **413** |
| Run-Off Pervious Areas ($m^3$/yr) | 56 | 153 | 0 | 0 | 209 |
| Run-Off Impervious Areas ($m^3$/yr) | 0 | 0 | 136 | 202 | 338 |
| Total Run-Off ($m^3$/yr) | 56 | 153 | 136 | 202 | 547 |
| **Total Outputs ($m^3$/yr)** | 426 | 869 | 170 | 318 | 1,783 |
| **Difference (Inputs - Outputs)** | **0** | **0** | **0** | **0** | **0** |

## Table B: Post-Development (no mit)

| Catchment Designation | Landscaped Grass | Structure | Other Impervious | Total |
|---|---|---|---|---|
| Area ($m^2$) | 670 | 232 | 1,088 | 1,990 |
| Pervious Area ($m^2$) | 670 | 0 | 0 | 670 |
| Impervious Area ($m^2$) | 0 | 232 | 1,088 | 1,320 |
| **Infiltration Factors** | | | | |
| Topography Infiltration Factor | 0.2 | 0 | 0 | |
| Soil Infiltration Factor | 0.3 | 0 | 0 | |
| Land Cover Infiltration Factor | 0.1 | 0 | 0 | |
| Infiltration Factor | 0.6 | 0 | 0 | |
| Run-Off Coefficient | 0.4 | 1 | 1 | |
| Run-Off From Impervious Surfaces | 0.8 | 0.8 | 0.8 | |
| **Inputs (Per Unit Area)** | | | | |
| Precipitation (mm/yr) | 896 | 896 | 896 | 896 |
| Rainfall (mm/yr) | 613 | 613 | 613 | 613 |
| Run-On (mm/yr) | 0 | 0 | 0 | 0 |
| Other Inputs (mm/yr) | 0 | 0 | 0 | 0 |
| **Total Inputs (mm/yr)** | **896** | **896** | **896** | 896 |
| **Outputs (Per Unit Area)** | | | | |
| Precipitation Surplus (mm/yr) | 394 | 717 | 717 | 608 |
| Net Surplus (mm/yr) | 394 | 717 | 717 | 608 |
| Evapotranspiration (mm/yr) | 502 | 179 | 179 | 288 |
| Infiltration (mm/yr) | 236 | 0 | 0 | 80 |
| Surplus Infiltration (mm/yr) | 0 | 0 | 0 | 0 |
| Total Infiltration (mm/yr) | 236 | 0 | 0 | 80 |
| Run-Off Pervious Areas (mm/yr) | 158 | 0 | 0 | 53 |
| Run-Off Impervious Areas (mm/yr) | 0 | 717 | 717 | 475 |
| Total Run-Off (mm/yr) | 158 | 717 | 717 | 529 |
| **Total Outputs (mm/yr)** | **896** | **896** | **896** | **896** |
| **Difference (Inputs - Outputs)** | **0** | **0** | **0** | **0** |
| **Inputs (Volumes)** | | | | |
| Precipitation ($m^3$/yr) | 600 | 208 | 975 | 1,783 |
| Run-On ($m^3$/yr) | 0 | 0 | 0 | 0 |
| Other Inputs ($m^3$/yr) | 0 | 0 | 0 | 0 |
| **Total Inputs ($m^3$/yr)** | **600** | **208** | **975** | **1,783** |
| **Outputs (Volumes)** | | | | |
| Precipitation Surplus ($m^3$/yr) | 264 | 166 | 780 | 1,210 |
| Net Surplus ($m^3$/yr) | 264 | 166 | 780 | 1,210 |
| Evapotranspiration ($m^3$/yr) | 336 | 42 | 195 | 573 |
| Infiltration ($m^3$/yr) | 158 | 0 | 0 | 158 |
| Surplus Infiltration ($m^3$/yr) | 0 | 0 | 0 | 0 |
| **Total Infiltration ($m^3$/yr)** | **158** | **0** | **0** | **158** |
| Run-Off Pervious Areas ($m^3$/yr) | 106 | 0 | 0 | 106 |
| Run-Off Impervious Areas ($m^3$/yr) | 0 | 166 | 780 | 946 |
| Total Run-Off ($m^3$/yr) | 106 | 166 | 780 | 1,052 |
| **Total Outputs ($m^3$/yr)** | 600 | 208 | 975 | 1,783 |
| **Difference (Inputs - Outputs)** | **0** | **0** | **0** | **0** |

## Table C: Post-Development (with mitigation)

| Catchment Designation | Landscaped Grass | Structure | Other Impervious | Total |
|---|---|---|---|---|
| Area ($m^2$) | 670 | 232 | 1,088 | 1,990 |
| Pervious Area ($m^2$) | 670 | 0 | 0 | 670 |
| Impervious Area ($m^2$) | 0 | 232 | 1,088 | 1,320 |
| **Infiltration Factors** | | | | |
| Topography Infiltration Factor | 0.2 | 0 | 0 | |
| Soil Infiltration Factor | 0.3 | 0 | 0 | |
| Land Cover Infiltration Factor | 0.1 | 0 | 0 | |
| Infiltration Factor | 0.6 | 0 | 0 | |
| Run-Off Coefficient | 0.4 | 1 | 1 | |
| Run-Off From Impervious Surfaces | 0.8 | 0.8 | 0.8 | |
| **Inputs (Per Unit Area)** | | | | |
| Precipitation (mm/yr) | 896 | 896 | 896 | 896 |
| Rainfall (mm/yr) | 613 | 613 | 613 | 613 |
| Run-On (mm/yr) | 0 | 0 | 0 | 0 |
| Other Inputs (mm/yr) | 0 | 0 | 0 | 0 |
| **Total Inputs (mm/yr)** | **896** | **896** | **896** | 896 |
| **Outputs (Per Unit Area)** | | | | |
| Precipitation Surplus (mm/yr) | 394 | 717 | 717 | 608 |
| Net Surplus (mm/yr) | 394 | 717 | 717 | 608 |
| Evapotranspiration (mm/yr) | 502 | 179 | 179 | 288 |
| Infiltration (mm/yr) | 236 | 0 | 0 | 80 |
| Surplus Infiltration (mm/yr) | 0 | 530 | 0 | 62 |
| Total Infiltration (mm/yr) | 236 | 530 | 0 | 141 |
| Run-Off Pervious Areas (mm/yr) | 158 | 0 | 0 | 53 |
| Run-Off Impervious Areas (mm/yr) | 0 | 187 | 717 | 414 |
| Total Run-Off (mm/yr) | 158 | 187 | 717 | 467 |
| **Total Outputs (mm/yr)** | **896** | **896** | **896** | **896** |
| **Difference (Inputs - Outputs)** | **0** | **0** | **0** | **0** |
| **Inputs (Volumes)** | | | | |
| Precipitation ($m^3$/yr) | 600 | 208 | 975 | 1,783 |
| Run-On ($m^3$/yr) | 0 | 0 | 0 | 0 |
| Other Inputs ($m^3$/yr) | 0 | 0 | 0 | 0 |
| **Total Inputs ($m^3$/yr)** | **600** | **208** | **975** | **1,783** |
| **Outputs (Volumes)** | | | | |
| Precipitation Surplus ($m^3$/yr) | 264 | 166 | 780 | 1,210 |
| Net Surplus ($m^3$/yr) | 264 | 166 | 780 | 1,210 |
| Evapotranspiration ($m^3$/yr) | 336 | 42 | 195 | 573 |
| Infiltration ($m^3$/yr) | 158 | 0 | 0 | 158 |
| Surplus Infiltration ($m^3$/yr) | 0 | 123 | 0 | 123 |
| **Total Infiltration ($m^3$/yr)** | **158** | **123** | **0** | **281** |
| Run-Off Pervious Areas ($m^3$/yr) | 106 | 0 | 0 | 106 |
| Run-Off Impervious Areas ($m^3$/yr) | 0 | 43 | 780 | 823 |
| Total Run-Off ($m^3$/yr) | 106 | 43 | 780 | 929 |
| **Total Outputs ($m^3$/yr)** | 600 | 208 | 975 | 1,783 |
| Difference (Inputs - Outputs) | 0 | 0 | 0 | 0 |

**Table D: Water Balance Summary Table**

| Characteristic | Site | | | | | | |
|---|---|---|---|---|---|---|---|
| | Pre-Development | Post-Development | Change (Pre to Post) | | Post-Development with Mitigation | Change (Pre to Post with Mitigation) | |
| **Inputs (Volume)** | | | | | | | |
| Precipitation ($m^3$/yr) | 1,783 | 1,783 | 0 | 0% | 1,783 | 0 | 0% |
| Run-On ($m^3$/yr) | 0 | 0 | 0 | NA | 0 | 0 | NA |
| Other Inputs ($m^3$/yr) | 0 | 0 | 0 | NA | 0 | 0 | NA |
| Total Inputs ($m^3$/yr) | 1,783 | 1,783 | 0 | 0% | 1,783 | 0 | 0% |
| **Outputs (Volume)** | | | | | | | |
| Precipitation Surplus ($m^3$/yr) | 960 | 1,210 | 250 | 26% | 1,210 | 250 | 26% |
| Net Surplus (m3/yr) | 960 | 1,210 | 250 | 26% | 1,210 | 250 | 26% |
| Evapotranspiration ($m^3$/yr) | 823 | 573 | -250 | -30% | 573 | -250 | -30% |
| Infiltration ($m^3$/yr) | 360 | 158 | -202 | -56% | 158 | -202 | -56% |
| Rooftop Infiltration ($m^3$/yr) | 52 | 0 | -52 | -100% | 123 | 71 | 135% |
| **Total Infiltration ($m^3$/yr)** | **413** | **158** | **-254** | **-62%** | **281** | **-131** | **-32%** |
| Run-Off Pervious Areas ($m^3$/yr) | 209 | 106 | -103 | -49% | 106 | -103 | -49% |
| Run-Off Impervious Areas ($m^3$/yr) | 338 | 946 | 608 | 180% | 823 | 485 | 143% |
| Total Run-Off ($m^3$/yr) | 547 | 1,052 | 504 | 92% | 929 | 381 | 70% |
| **Total Outputs ($m^3$/yr)** | **1,783** | **1,783** | **0** | **0%** | **1,783** | **0** | **0%** |

**APPENDIX G**

**Dewatering Details**

**Table A: Dewatering Details**

| Location | Length (m) | Width (m) | Effecitve Radius[1] (m) | Hydraulic Conductivity (m/s) | Hydraulic Conductivity (m/day) | Min Depth (masl) | Max GW Depth[2] (masl) | Max Drawdown (m) | Initial Depth of Water (static head) prior to dewatering (m) | $H^2$ | Depth of Water in the well while pumping (m) | $h^2$ | $H^2$-$h^2$ | Radius of Influence[3] (m) | Ro/r | Discharge Into Ends[4] ($m^3$/day) | Plane Discharge[5] ($m^3$/day) | Total Discharge ($m^3$/day) | Total Discharge (L/day) | Total Discharge x 3 Safety Factor (L/day) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | a | b | re | k | k | | | | H | | h | | | Ro | | Q | Q | Q | Q | Q |
| San 1 | 10.8 | 2 | 4 | 1.30E-06 | 1.12E-01 | 288.28 | 288.65 | 0.87 | 1.87 | 3 | 1 | 1 | 2 | 7 | 1.73 | 2 | 0 | 2 | 2,000 | 6,000 |
| San 2 | 12 | 2 | 4 | 1.30E-06 | 1.12E-01 | 287.47 | 288.65 | 1.68 | 2.68 | 7 | 1 | 1 | 6 | 10 | 2.29 | 3 | 1 | 3 | 3,400 | 10,200 |
| San (entire line) | 22.8 | 2 | 8 | 1.30E-06 | 1.12E-01 | 287.47 | 288.65 | 1.68 | 2.68 | 7 | 2 | 4 | 3 | 10 | 1.29 | 4 | 1 | 5 | 5,100 | 15,300 |
| Wat 1 | 22 | 2 | 8 | 1.30E-06 | 1.12E-01 | 289.00 | 288.65 | 0.15 | 1.15 | 1 | 2 | 4 | -3 | 5 | 0.62 | 2 | -1 | 1 | 600 | 1,800 |
| SWM 1 | 20.4 | 2 | 7 | 1.30E-06 | 1.12E-01 | 289.67 | 288.60 | -0.57 | No dewatering required | | | | | | | | | | | |
| SWM 2 | 19.1 | 2 | 7 | 1.30E-06 | 1.12E-01 | 289.64 | 288.65 | -0.49 | | | | | | | | | | | | |
| SWM 3 | 37.4 | 2 | 13 | 1.30E-06 | 1.12E-01 | 289.64 | 288.90 | -0.24 | | | | | | | | | | | | |
| SWM 4 | 5.8 | 2 | 2 | 1.30E-06 | 1.12E-01 | 289.72 | 288.80 | -0.42 | | | | | | | | | | | | |
| SWM 5 | 17.3 | 2 | 6 | 1.30E-06 | 1.12E-01 | 289.82 | 289.00 | -0.32 | | | | | | | | | | | | |

Notes

[1] *$r_e = (a+b) / \pi$ - assuming a/b >1.5, (Driscoll, 1986)*

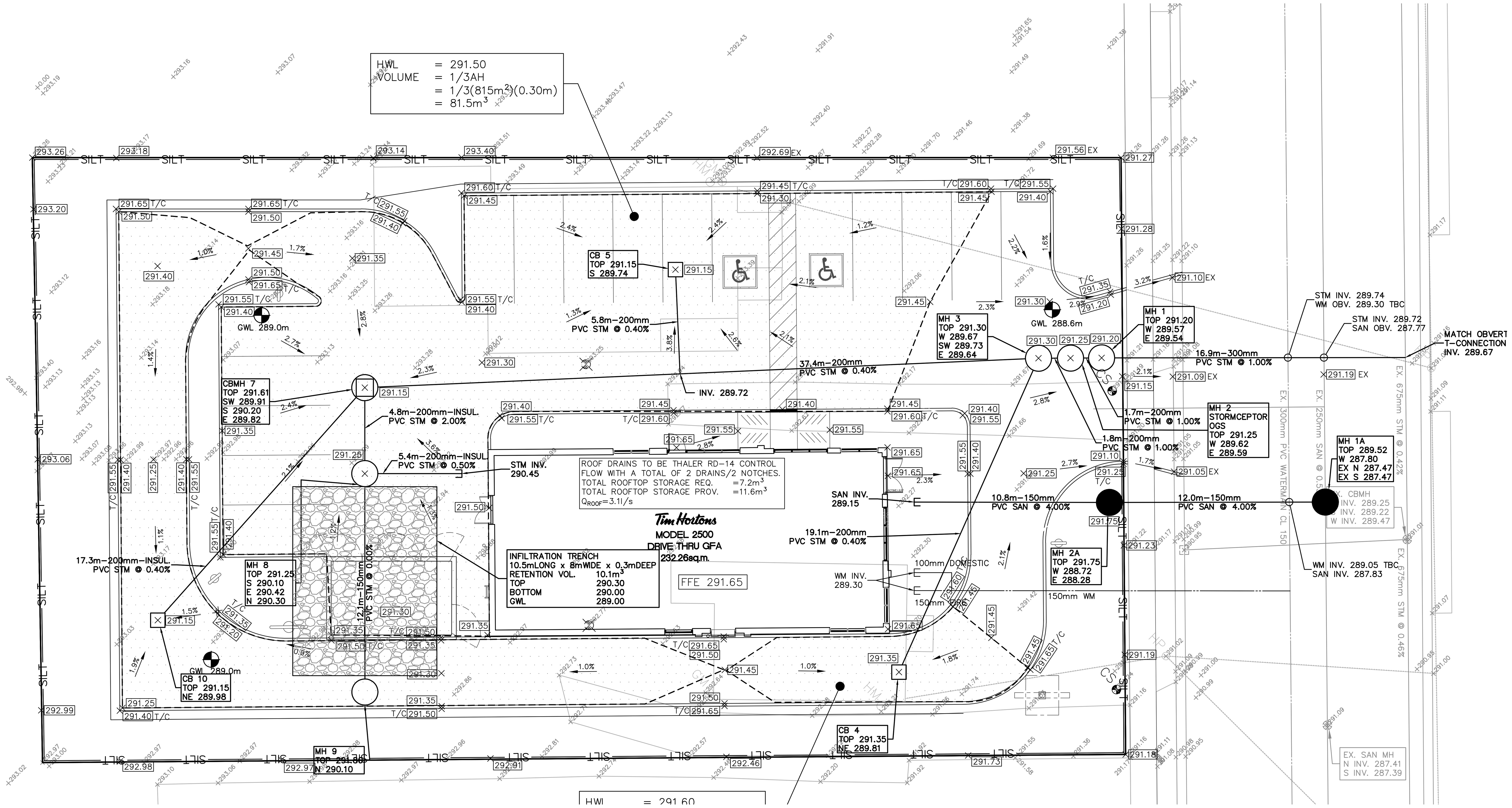
[2] *Ground water levels were collected in July of 2019. These levels should be updated when spring high conditions are known*

[3] *$Ro = r_e + 3000 * (H\text{-}h) * \sqrt{k}$ - Sichardts Formula, (Cashman and Preene, 2001)*

[4] *$Q = [(\pi * K)*(H^2 - h^2)] / [\ln(R_o/r)]$ (Powers et al., 2007)*

[5] *$Q = 2*[a*K*(H^2 - h^2)/(2R_o)]$ (Powers et al., 2007)*

Estimated from Drawing G1



KEY PLAN

## LEGEND

- CATCHBASIN
- DOUBLE CATCHBASIN
- CATCHBASIN MANHOLE
- DOUBLE CATCHBASIN MANHOLE
- STORM MANHOLE
- BOX MANHOLE
- SANITARY MANHOLE
- VALVE AND BOX
- HYDRANT AND VALVE
- STORM
- SANITARY
- WATERMAIN
- +274.00 EXISTING ELEVATION
- 274.00 EX EXISTING ELEVATION
- 274.00 PROPOSED ELEVATION
- 274.00 T/C PROPOSED TOP OF CURB ELEVATION
- 274.00 B/W PROPOSED BOTTOM OF WALL ELEVATION
- 274.00 SW PROPOSED SWALE ELEVATION
- EMERGENCY OVERLAND FLOW ROUTE
- PONDING AREA
- SILT — SILT FENCE
- CONSTRUCTION MUD MAT
- CB FILTER TRAP

SURVEY PROVIDED BY FIDDES CLIPSHAM INC., DATED APRIL 26, 2019
BENCHMARK: CIVIC #16054, HAVING AN ELEVATION OF 291.11m

CATCH

PROPOSED MANHOLE STRUCTURES TO BE 1200mm DIAMETER OPSD 701.010 UNLESS OTHERWISE NOTED

PAVEMENT STRUCTURE DESIGN DETAILS, SPECIFICATIONS AND SUB–DRAINAGE DESIGN, IS NOT UNDER THE CIVIL DESIGN SCOPE. CONTRACTOR IS TO REFER TO GEOTECHNICAL REPORT RELATED SPECIFICATIONS. CONTRACTOR IS TO COORDINATE PAVEMENT MAKEUP AND SUB–DRAINAGE INSPECTION WITH GEOTECHNICAL CONSULTANT.
A.M. CANDARAS ASSOCIATES INC. ASSUMES NO RESPONSIBILITY RELATING TO CONSTRUCTION AND INSPECTION OF THE PAVEMENT STRUCTURE.

CONTRACTOR TO BE RESPONSIBLE FOR VERIFYING THE LOCATIONS OF ALL EXISTING UNDERGROUND AND ABOVE UTILITIES AND SERVICES. THE CONTRACTOR SHALL ADVISE THE ENGINEER OF ANY DISCREPANCIES PRIOR TO PROCEEDING WITH CONSTRUCTION. VARIOUS UTILITIES CONCERNED TO BE GIVEN REQUIRED ADVANCED NOTICE PRIOR TO ANY DIGGING, FOR STAKE OUT.
A.M. CANDARAS ASSOCIATES INC. ASSUMES NO RESPONSIBILITY FOR THE ACCURACY OF THE LOCATION OF EXISTING UTILITIES AS INDICATED ON THIS DRAWING.

## TOWN OF CALEDON GENERAL NOTES (SITE PLANS 2019)

1. CONSTRUCTION FOR THIS PROJECT TO COMPLY WITH THE MOST CURRENT VERSION OF THE DEVELOPMENT STANDARDS, POLICIES AND GUIDELINES, PREPARED BY THE TOWN OF CALEDON AND THE ONTARIO PROVINCIAL STANDARDS AND SPECIFICATIONS.
2. ALL CONSTRUCTION SHALL BE CARRIED OUT IN ACCORDANCE WITH THE REQUIREMENTS OF THE OCCUPATIONAL HEALTH AND SAFETY ACT AND REGULATIONS FOR CONSTRUCTION PROJECTS.
3. A MINIMUM OF FORTY–EIGHT (48) HOURS PRIOR TO COMMENCING CONSTRUCTION WITHIN THE MUNICIPAL RIGHT OF WAY THE CONTRACTOR MUST CONTACT THE FOLLOWING:

   THE TOWN OF CALEDON
   905–584–2272

   | | |
   |---|---|
   | THE REGION OF PEEL | ENBRIDGE CONSUMERS GAS |
   | HYDRO ONE | BELL CANADA |
   | ROGERS CABLE | FIRE AND EMERGENCY SERVICES |

4. A RIGHT OF WAY OCCUPANCY PERMIT MUST BE OBTAINED FROM THE TOWN OF CALEDON A MINIMUM 48 HOURS PRIOR TO COMMENCING ANY WORKS WITHIN THE MUNICIPAL ROAD ALLOWANCE.
5. ALL DRAINAGE TO BE SELF–CONTAINED AND DISCHARGED TO A LOCATION APPROVED BY THE TOWN OF CALEDON AND CONSERVATION AUTHORITY PRIOR TO THE ISSUANCE OF A BUILDING PERMIT.
6. SEDIMENT CONTROL DEVICES ARE TO BE INSTALLED PRIOR TO ANY CONSTRUCTION ON THE SITE AND SHALL BE MAINTAINED THROUGHOUT THE CONSTRUCTION PERIOD TO THE SATISFACTION OF THE TOWN AND THE APPLICABLE CONSERVATION AUTHORITY.
7. ANY CHANGES TO GRADES OR SERVICING FROM THE ORIGINAL APPROVED SITE PLAN MUST BE SUBMITTED BY THE ENGINEER TO THE TOWN FOR APPROVAL PRIOR TO CONSTRUCTION.
8. A MINIMUM OF 1.5M CLEARANCE IS TO BE PROVIDED FROM THE LIMITS OF ALL SIDEWALKS AND DRIVEWAYS TO EXISTING UTILITY STRUCTURES WITHIN THE MUNICIPAL RIGHT OF WAY. IF THIS CLEARANCE IS NOT MAINTAINED, THEY SHALL BE RELOCATED AT THE APPLICANT'S EXPENSE.
9. STREET CURBS ARE TO BE CONTINUOUS THROUGH THE PROPOSED ENTRANCE.
10. MUNICIPAL SIDEWALKS SHALL BE CONTINUOUS THROUGH ALL ENTRANCES TO THE SITE AND THE CURB SHALL BE TAPERED BACK 600MM. SIDEWALKS SHALL BE COMPLETELY REMOVED AND REPLACED WITH A 200MM MINIMUM CONCRETE THICKNESS, 32MPA AND 5% TO 7% AIR ENTRAINMENT AT ALL PROPOSED INDUSTRIAL, COMMERCIAL AND INSTITUTIONAL ENTRANCES.
11. ALL BOULEVARDS TO BE RESTORED WITH 300MM MINIMUM OF TOPSOIL AND SOD TO THE SATISFACTION OF THE TOWN.
12. THE MINIMUM PAVEMENT DESIGN FOR THE ASPHALT DRIVEWAY APRON WITHIN THE MUNICIPAL ROAD ALLOWANCE SHALL BE AS FOLLOWS:

    40mm HL3 ASPHALT
    50mm HL8 ASPHALT
    150mm GRANULAR 'A'
    300mm GRANULAR 'B'

THE CONSULTANT SHOULD REVIEW THE ABOVE WITH RESPECT TO THE EXPECTED USAGE.

13. STRUCTURAL DESIGN OF THE FIRE ROUTE IS REQUIRED TO SUPPORT AN 18 TON VEHICLE.

14. SERVICE CONNECTION BACKFILL TO BE DISCUSSED WITH THE TOWN.

| No. | Date | By | REVISIONS |
|---|---|---|---|


a.m.candaras associates inc.
consulting engineers
8551 Weston rd., suite 203
Woodbridge ont. L4L 9R4
905–850–8020 Fax 905–850–8099
Email: civil@amcai.com


# TIM HORTONS
# 16054 & 16060 AIRPORT ROAD

TOWN OF CALEDON
REGION OF PEEL

## SITE GRADING, SERVICING AND STORMWATER MANAGEMENT PLAN

DRAFT

| | | |
|---|---|---|
| SCALE: 1:150 | DATE: SEPTEMBER 2019 | PROJ No. 1918 |
| DRAWN: Z.S.S. | CHK'D: A.M.C. | PLAN No. |
| DESIGNED: Z.S.S. | SHEET 1 OF 1 | G1 |