

MONOLITE HOLDINGS INC.

FOUR STOREY DEVELOPMENT, 181 BURTON AVENUE, BARRIE, ON GEOTECHNICAL INVESTIGATION

SEPTEMBER 01, 2023





FOUR STOREY
DEVELOPMENT, 181
BURTON AVENUE,
BARRIE, ON
GEOTECHNICAL
INVESTIGATION

MONOLITE HOLDINGS INC.

PROJECT NO.: 201-09517-00

DATE: SEPTEMBER 01, 2023

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September 01, 2023

MONOLITE HOLDINGS INC.
343 Sugar Maple Lane
Richmond Hill, ON
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Dear Ms. Rozentsvayg,

Subject: Four Storey Development, 181 Burton Avenue, Barrie - Geotechnical Investigation

WSP Canada Inc. (WSP) is pleased to submit the attached Geotechnical Investigation report to support the design and construction of the proposed development of a four-storey building at 181 Burton Avenue, Barrie, Ontario.

The purpose of the geotechnical investigation is to identify the subsurface conditions at the borehole locations and to provide design recommendations toward the design of the proposed development, as well as identify potential geotechnical related constraints which may be encountered during construction.

We trust that this report satisfies your requirements. Please contact me if you have any questions.

Kind regards,

A handwritten signature in black ink, appearing to read 'NLP' followed by a stylized flourish.

Nick La Posta, P.Eng.
Team Lead – Ground Engineering West

NLP/kj
Encl.
WSP ref.: 201-09517-00

SIGNATURES

PREPARED BY



Nick La Posta, P.Eng.
Team Lead – Ground Engineering West

September 1, 2023

Date

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1 INTRODUCTION

WSP Canada Inc. (WSP) was retained by Ms. Maria John Rozentsvayg on behalf of Monolite Holdings Inc. (Client) to provide a geotechnical investigation for a proposed four storey development at 181 Burton Avenue, in the City of Barrie, Ontario. The location of the site is shown on the attached **Site Location Plan - Figure 1**.

The scope of this geotechnical investigation was to obtain information about the subsurface conditions through the advancement of five (5) boreholes and based upon the findings of the boreholes ultimately provide recommendations herein pertaining to the following:

- Excavation and backfill;
- Soil parameters for excavations;
- Groundwater levels;
- Soil bearing capacity;
- Reuse of existing soil;
- Site preparation and grading; and
- Preliminary pavement design.

This report deals with geotechnical issues only. It does not include any chemical or hydrogeological testing and does not provide any commentary regarding the site's environmental or hydrogeological status.

This report is provided based on the terms of reference presented above and, on the assumption, that the design will be in accordance with the applicable codes and standards. If there are any changes in the design features relevant to the geotechnical analyses, or if any questions arise concerning the geotechnical aspects of the codes and standards, this office should be contacted to review the design.

The site investigation and recommendations follow generally accepted practice for Geotechnical Consultants in Ontario. The format and contents are guided by client specific needs and economics and do not conform to generalized standards for services. Laboratory testing follows ASTM or CSA Standards or modifications of these standards that have become standard practice.

This report has been prepared for the Client. Third party use of this report without WSP consent is prohibited.

2 SITE AND PROJECT DESCRIPTION

The Site is located on the north side of Burton Avenue, south of Cumberland Street, west of Melinda Crescent and is currently vacant. Evidence of previously placed fill was apparent on site. The existing ground elevation at the borehole locations differed by approximately 0.7m from south to north.

Based on information provided to our office, it is understood that the development comprises the construction of a 24 unit, four storey residential sab on grade structure and infiltration area at the site. The preliminary ISM Architects Site Plan, Floor Plans and Elevations are attached to this report in **Appendix A**. The Pearson Engineering municipal engineering servicing and drainage drawings are attached to this report in **Appendix B**.

3 INVESTIGATION METHODOLOGY

3.1 FIELD INVESTIGATION

Prior to the field investigation, public utility companies were consulted to identify where existing public utilities entered the Site boundaries. In addition, given that the Site is a private property, WSP retained the services of a private utility locator, All Clear Locates Inc., to clear the specific borehole locations from potential interference with private utilities.

The field investigation consisted of drilling five (5) boreholes (BH21-1 through BH21-5), at the Site on January 21, 2021. The boreholes were advanced and installed at the approximate locations shown on **Borehole Plan - Figure 2**.

The boreholes were advanced to depths of approximately 5.0 meters below the existing ground surface (mbgs) (BH21-1, BH21-2, BH21-4 and BH21-5) and 6.6 mbgs (BH21-3) utilizing a track-mounted drill rig and a truck-mounted drill rig. The Track-Mounted CME 75 drill rig used 110 mm outside diameter continuous flight solid stem augers. WSP field personnel supervised the drilling operations and recorded the conditions encountered in the boreholes at the time of the investigation. Soil samples were recovered from the boreholes at approximately 0.75 m to 1.5 m intervals using a 51 mm outside diameter split spoon sampler, driven in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). It should be noted that the split spoon samplers used limit the particle size of the retrieved samples to less than 50 mm. As such any particles greater than that are not retrieved or represented within the laboratory particle size distributions analyses. The results of SPTs in terms of N values are referred to in this report to define the Compactness Condition for the non-cohesive (non-plastic) materials encountered at the Site or Consistency of the cohesive materials encountered.

The boreholes were checked for groundwater seepage and general stability prior to backfilling. All soil samples recovered from the boreholes were placed in moisture proof bags and transported to our laboratory for detailed classification and testing as required.

The ground surface elevations at the respective borehole locations were obtained by WSP. The ground surface elevations at the borehole locations are presented on **Figure 2** and Borehole Logs. The borehole elevations are for geotechnical engineering analytical purposes only and must be verified prior to finalizing any design or construction parameters upon which they are based.

3.2 GROUNDWATER MONITORING WELLS

Water level observations were made during the drilling and in the open boreholes upon the completion of drilling operations.

Groundwater monitoring wells, designated as BH21-1 and BH21-4, were installed as part of the investigation. The monitoring wells have been used to measure the depth of the groundwater table and can be utilized to evaluate further parameters such as hydraulic conductivity and seepage potential of the site subsoils. The monitors were installed to meet Provincial Standards (O. Reg. 903 as amended) and extend to depths of approximately 5.0 mbgs. The monitors were constructed of 50 mm inside diameter Schedule 40 PVC machine-slotted screen and riser pipe. Screened intervals were backfilled with filter sand and the annuli of riser sections were sealed with bentonite hole plugs. Monitors were protected by pedestal mount casing at each installed location. Additional details are provided on the **Borehole Logs – Enclosures 1 and 4**. WSP presumes that the Client will be responsible for decommissioning the two (2) groundwater monitoring wells either prior to or at the time of construction, as required by O.Reg. 903 (as amended).

WSP returned to the site on February 10, 2021 to obtain water levels at the site.

3.3 LABORATORY TESTING

The field investigation was supplemented with a laboratory testing program. Upon completion of drilling, all recovered soil samples were transported to our geotechnical laboratory for detailed visual examination and classification. Moisture Content Tests were conducted on all samples and Particle Size Distribution analyses were conducted on four (4) representative soil samples. The laboratory test results are summarized on the **Borehole Logs – Enclosures 1 to 5** and the individual Particle Size Distribution curves are included in **Particle Size Analysis – Enclosures 6 to 9**.

Unless requested in advance, the soil samples from this investigation will be stored in our laboratory facility for a period of up to three (3) months after the final issuance of this report.

4 SITE AND SUBSURFACE CONDITIONS

Details of the subsurface conditions encountered are presented on the Borehole Logs and summarized in the following sections. It is noted that subsurface conditions can change between boreholes and the details provided below refer to soil conditions that were encountered at the borehole locations only.

4.1 SOIL PROFILE

Based on the information obtained from the boreholes, the Site included a surface cover of topsoil over a moderately thick fill with underlying soil profile that included both organic soil and a predominantly non-cohesive (non-plastic) stratified deposit at all boreholes. Individual soil units encountered in the boreholes are described as follows.

4.1.1 TOPSOIL

Topsoil was encountered at the ground surface at all borehole locations and measured approximately 0.1 m in thickness. A second layer of topsoil was encountered in BH21-1 and BH21-5 beneath a layer of fill. The lower topsoil measured approximately 0.1 m to 0.5 m in thickness. The topsoil was dark brown and moist.

The natural moisture content of the topsoil as determined by the laboratory tests ranged from 8% to 26%.

It should be noted that topsoil thickness should not be calculated from the borehole information, as large variations in depth may exist between and beyond the boreholes. A detailed topsoil layer thickness and composition survey is required to determine an accurate evaluation of quantity and quality.

4.1.2 FILL

Fill predominantly comprised of moist non-cohesive sandy silt to sand was encountered underlying the topsoil at all borehole locations. The fill was brown and moist with some gravel and extended to depths ranging from 1.9 mbgs (BH21-1) to 3.2 mbgs (BH21-5).

The natural moisture content of the non-cohesive fill as determined by the laboratory tests ranged from 8% to 23%.

The measured SPT 'N' values in the fill deposit ranged from 1 blow per 0.3 m of penetration to 12 blows per 0.3 m of penetration, indicating that the fill was generally very loose to loose.

It should be noted that topsoil and fill quantities should not be calculated from the borehole information, as large variations in depth may exist between boreholes. A detailed topsoil/fill thickness survey is recommended to determine an accurate evaluation of quantity. The topsoil was generally dark brown in colour and moist but frozen.

A summary of the fill encountered at the site is provided in the following table.

BOREHOLE	DEPTH OF TOP OF FILL (MBGS)	DEPTH OF BOTTOM OF FILL (MBGS)	FILL THICKNESS (M)	FILL COMPOSITION
BH21-1	0.1	1.9	1.8	Silty Sand
BH21-2	0.1	2.4	2.3	Sand
BH21-3	0.1	3.0	2.9	Silt and Sand
BH21-4	0.1	3.1	3.0	Sand
BH21-5	0.1	3.2	3.1	Sand

Grain size analyses of samples of fill was completed and the gradation curve is presented in **Enclosures 6 and 8 - Laboratory Analyses**. A review of the grain size analyses indicates the following ranges of clay, silt, sand and gravel percentages:

- Gravel: 0%
- Sand: 57% - 70%
- Fines (Silt & Clay): 30% - 43%

4.1.3 NON-COHESIVE DEPOSITS

A stratified deposit predominantly comprised of sand and silt and sand was encountered in all boreholes. The non-cohesive soil extended beyond the final depth investigated at all boreholes. The sand and silt layers were brown and moist to saturated.

The measured SPT 'N' values in the non-cohesive deposit ranged from 10 blows per 0.3 m of penetration to 70 blows per 0.3 m of penetration, indicating that the deposit varied from compact to very dense, generally being compact to dense.

The natural moisture content of non-cohesive soils ranged between 8% and 19%.

Grain size analyses of two (2) samples of the deposit were completed and the gradation curves are presented in **Enclosures 7 and 9 - Laboratory Analyses**. A review of the grain size analyses indicates the following ranges of clay, silt, sand and gravel percentages:

- Gravel: 3% - 4%
- Sand: 53% - 73%
- Fines (Silt & Clay): 23% - 44%

4.2 GROUNDWATER

A summary of the groundwater levels observed at the site, both upon completion of the drilling of the boreholes, as well as in the standpipes installed in two (2) of the boreholes, is tabulated below.

BOREHOLE	DATE	GROUNDWATER DEPTH (MBGS)	MEASUREMENT SOURCE
BH21-1	January 11, 2021	4.7	Borehole open to 5.0 mbgs
	February 10, 2021	4.1	Well
BH21-2	January 11, 2021	4.9	Borehole open to 5.0 mbgs
BH21-3	January 11, 2021	4.6	Borehole caved to 5.5 mbgs
BH21-4	January 11, 2021	4.5	Borehole open to 5.0 mbgs
	February 10, 2021	4.0	Well
BH21-5	January 11, 2021	--	Borehole open & dry to 5.0 mbgs

It should be noted that the groundwater levels can vary and are subject to seasonal fluctuations in response to major weather events. Future monitoring of the groundwater levels is recommended to determine the seasonal high level.

5 DISCUSSIONS/RECOMMENDATIONS

5.1 GENERAL

The following recommendations for the proposed site development are based on the information obtained from the borehole investigation and laboratory testing, which we believe fairly represents the subsurface conditions of the site. These recommendations are intended for the guidance of the design engineer to establish constructability and should not be construed as instructions to contractors. If significant differences in the subsurface conditions described above are found, we request to be contacted immediately to review and revise our findings and recommendations, if necessary.

The construction methods described in this report must not be considered as being specifications or recommendations to the prospective contractors, or as being the only suitable methods. Prospective contractors should evaluate all the information, obtain additional subsurface information as they might deem necessary and should select their construction methods, sequencing and equipment based on their own experience in similar ground conditions. The readers of this report are also reminded that the conditions are known only at the borehole locations and in view of the generally wide spacing of the boreholes, conditions may vary significantly between boreholes.

5.2 SITE BACKGROUND

As indicated above the proposed development comprises the construction of a four storey, 24 unit residential, slab on grade structure, an at grade exterior parking area and infiltration area.

The results of the geotechnical investigation indicate that the subsurface conditions at the site comprise a moderately deep layer of fill with underlying topsoil overlying compact to very dense silty sand and sand. Groundwater was encountered during the drilling at boreholes BH21-1 to BH21-4 between 4.5 mbgs and 4.9 mbgs. The water level was also measured within the two (2) wells on February 10, 2021 as high as 4.0 mbgs.

5.3 SITE PREPARATION AND GRADING

Removal of all fill and any organic matter will be required to facilitate the proposed development on the site. Regarding the reuse of the site fill, fill may be reused as engineered fill or compacted subgrade parking area fill applications if not organically included. WSP should be contacted to review all proposed fill reuse on site. The existing fill in the proposed parking areas must be assessed during construction operations by confirmation sampling and proof roll to ascertain whether it should be removed and replaced in the proposed parking area.

After the completion of the required stripping and removal of unsuitable materials (fill and organic matter), the subgrade should be proof-rolled and inspected by experienced WSP geotechnical engineering personnel. The proof-rolling and compaction of the exposed sub-grade is recommended to be conducted using a vibratory compactor with a minimum static weight of 10 tonnes. The proof-rolling program should consist of a minimum of six (6) passes per unit area and be tested to assure that the sub-grade is compacted to a minimum of 98% of the exposed material's Standard Proctor Maximum Dry Density (SPMDD). Any loose/soft or wet areas identified at the time of proof-rolling that cannot be uniformly compacted are recommended to be sub-excavated and backfilled with approved fill.

Compacted fill may be required to develop the design grades and elevations or for use in other backfilling excavations created through the removal of unsuitable materials or soils as described above. The excavated native on-site non-cohesive materials may be re-used, subject that these are free of organic and other unsuitable materials and have adequate moisture content. Boulders or cobbles greater than 200 mm in size should be removed from the fill. Alternatively, Ontario Provincial Standard Specification (OPSS) Granular B – Type I, OPSS Select Subgrade Material (SSM), or an approved equal may be used at the site for engineered fill purposes.

All fill materials imported to the site must meet all applicable municipal, provincial and federal guidelines and requirements associated with environmental characterization of the materials.

Fill is to be placed in maximum 200 mm thick loose lifts under full time supervision of qualified WSP geotechnical personnel. Each lift is to be uniformly compacted to achieve the required degree of compaction.

5.4 FOUNDATION RECOMMENDATIONS

The proposed structure is situated in an area with the fill and an underlying layer of topsoil. Due to the presence of these unsuitable soils in combination with some underlying loose native soil, conventional shallow strip and spread foundations are not suitable for the structure without soil removal and improvement or deep foundations extended to the compact to dense non-cohesive soil deposits revealed below a depth of approximately 2.5 mbgs to 3.3 mbgs.

Although the initial investigation scope did not include deep boreholes to assess subsurface conditions for supporting a structure on a system of deep foundations very dense soil was encountered at the bottom of the proposed boreholes. Boreholes BH21-3, Bh21-4 and Bh21-5 were extended to either very dense soil or through the fill encountered. As such, alternatives for deep foundations maybe considered; however, confirmation of the very dense underlying strata with deeper boreholes is recommended.

5.4.1.1 FOUNDATION ON ENGINEERED FILL

Consideration could be given to removing the unsuitable soils and replacing approved material as engineered fill to raise the grade back up to the proposed final grades. Building foundations could then be founded within the engineered fill materials. Detailed information related to the placement and compaction of engineered fill can be found in **Appendix C**. A summary of the required excavation depths at the borehole locations for engineered fill, where building will be constructed, is provided below.

Table 5-1 Summary of Suitable Stripped Soil Levels

BOREHOLE NO.	EXISTING GROUND SURFACE ELEVATION (M)	APPROXIMATE DEPTH TO SUITABLE FOUNDING SOIL (M)	APPROXIMATE ELEVATION TO SUITABLE FOUNDING SOIL (M)
BH21-1	232.2	2.8	229.4
BH21-2	232.2	2.5	229.7
BH21-3	231.9	3.1	228.8

For preliminary design assessment purposes, a factored geotechnical resistance at Ultimate Limit States (ULS) of 225 kPa and a geotechnical resistance at Serviceability Limit States (SLS) of 150 kPa (assuming 25 mm of settlement) may be used in the design of shallow foundations founded within engineered fill materials.

Engineered fill materials should not be placed during winter/periods of freezing weather; **as such it is recommended that the engineered fill operations be completed in either the spring, summer, or fall seasons.**

As discussed later in this report, the removal of the existing loose soil materials and subsequent replacement with engineered fill materials is recommended to limit potential settlements of the floor slab of the proposed building.

5.4.1.2 HELICAL PILES

Helical piles can be considered to support the proposed structure following the confirmation of the subsurface conditions with deep boreholes. The helical pile comprises one (1) or multi helices on the end of a small diameter solid steel shaft. The steel helices are screwed into the ground to the depth of competent bearing or refusal. We do advise that the soil and groundwater conditions observed in the boreholes, SPT completed at the site indicate bearing of the helical pile systems would be at an approximate depth below 6 mbgs. It must be noted that removal and replacement of the topsoil is recommended unless the building is designed and constructed with a passive methane collection system and the helical piles are designed to accommodate reduced lateral support.

Numerous options are available for helical piles. The design and installation of the helical piles should be done by a specialist contractor and in accordance with the Canadian Foundation Engineering Manual. Furthermore, the most economical and effective option would best be determined with a design specialist who could establish the most appropriate product available for the site-specific conditions encountered. As an example, for preliminary design, a grouted shaft SS225 multi helix screw piles extended into the very dense sand deposit could provide an allowable load of 680 kN SLS and 915 kN ULS. We do stress that the pile capacity must be verified in the field through a load test.

5.4.1.3 CFA PILES

Continuous Flight Auger (CFA) piles can be considered to support the proposed structure following the confirmation of the subsurface conditions with deep boreholes. CFA piles are cast-in-place concrete piles which are formed by drilling using hollow augers to the target depth then pumping concrete through the augers as they are retracted. A conventional rebar cage is pushed into the concrete after forming the pile. The most important consideration in CFA pile construction is matching the rate of concrete pumping with the rate of withdrawal of the augers such that ‘necking’ of the pile does not occur. The potential CFA contractors must review the borehole information and ensure that CFA piles are feasible for the soil and groundwater conditions explored at the site. In addition, the quality and reliability of CFA piles essentially depend on the experience of the contractor and the operators installing the piles.

The bearing resistance of the piles will highly depend on the quality and procedure of the pile installation, as well as on the diameter and depth of the piles. The scope of the investigation did not include boreholes of sufficient depth to ascertain the length or capacity of CFA piles. Although dense to very dense soils were encountered at the termination depths of BH21-3, additional deeper boreholes would be required to verify the founding depths of the CFA piles. Further, the design bearing resistance and CFA pile length must be confirmed by field load tests of test

piles prior to the installation of the production piles. The test piles must be loaded to at least two (2) times the ULS bearing resistance. Depending on the load test results, deeper/longer piles may be required to achieve the design bearing resistances or higher capacities determined if piles are loaded to failure.

In order to ignore the group effect on the pile capacity (compression), the centre-to-centre spacing of the CFA piles should be at least 3 times its diameter.

As mentioned above, the bearing resistances of CFA piles will be highly dependent on the contractor's experience, the quality and procedure of the pile installation, and the skills of the installation operator(s). The CFA contractor must review the borehole information and evaluate bearing capacity of the piles based on their experience. The quality and the design bearing resistance of the piles must be ensured by the CFA contractor. A specialty contractor should be retained to design and install the CFA piles based on the performance specification and design bearing resistances.

Prior to the pile construction, the contractor should submit the details of the installation plan, load test program, installation procedure, automated monitoring system and control parameters, grout/concrete mix design, and reinforcement installation etc. for the review by the structural engineer and the geotechnical engineer.

All pile installation must be inspected by this office. In addition to the pre-production pile load tests, post-installation integrity tests and/or verification load tests are also required. Piles that have installation records out of specification or that otherwise appear abnormal can be selected for integrity tests (i.e. sonic echo tests) or verification load tests to determine if they should be accepted or rejected.

5.4.2 GENERAL FOUNDATION COMMENTS

All footings exposed to seasonal freezing conditions should be provided with at least 1.5 m of earth cover or equivalent thermal insulation against frost. It is recommended to keep footings as high as possible to avoid or minimize penetration below groundwater levels while considering the minimum frost cover requirement.

Variations in the soil conditions are expected in between the borehole locations, and during construction, the geotechnical resistances should be confirmed by experienced WSP site personnel.

Where it is necessary to place footings at different levels, the upper footing must be founded below an imaginary 10 horizontal to 7 vertical line drawn up from the base of the lower footing. The lower footing must be installed first to help minimize the risk of undermining the upper foundations.

The silt soils at the base of footings can be easily disturbed by construction machinery and foot traffic or lose their strength in contact with surface water. We recommend that an allowance be made for placing a 50-mm thick skim coat of low-strength concrete on the founding subgrade immediately after its approval, to prevent its disturbance by construction activities and from ground or surface water, where necessary.

During winter construction, foundations and slab on grades must not be poured on frozen soil. Foundations must be adequately protected always from cold weather and freezing conditions.

Near the existing buried utilities, all footings must be lowered to undisturbed native soils, or alternatively the services must be structurally bridged.

It should be noted that the recommended geotechnical resistances have been calculated by WSP from the borehole information for the preliminary design stage only. Additional input may be required as new design information becomes available and is refined. For example, more specific information is available with respect to conditions between boreholes when construction is underway. In this regard, the interpretation between boreholes and the recommendations of this report must therefore be checked through field inspections provided by WSP to validate the information for use during the construction stage.

5.5 FLOOR SLAB CONSTRUCTION AND DRAINAGE

Any existing organic soil, fill and disturbed native soils below the floor slab must be sub-excavated and replaced in the building area. Following the removal of all unsuitable soil, the native soil in its undisturbed state is considered suitable for supporting a floor slab or compacted approved fill. The backfill required to raise the grade must consist of approved inorganic soil, placed in shallow lifts and compacted to at least 98 percent of Standard Proctor Maximum Dry Density (SPMDD).

Based on the borehole information, the proposed floor slab of the building must either be supported on replaced compacted fill or be constructed as a structurally supported slab.

A perimeter drainage system would be required around the exterior underground walls for a floor slab placed below grade.

For bedding and moisture barrier purposes, a 200-mm thick layer of 19 mm clear crushed stone is recommended under the floor slab. Where wet and/or fine-grained soil conditions exist, underfloor sub drains and moisture barrier should be installed and should be separated from the subgrade by a geotextile fabric to avoid loss of soil/fines and settlement problems.

5.6 EARTHQUAKE CONSIDERATIONS

The parameters for determination of Site Classification for Seismic Site Response are set out in Table 4.1.8.4A of the Ontario Building Code (2012). The classification is based on the determination of the average shear wave velocity in the top 30 meters of the site stratigraphy, where shear wave velocity measurements have been taken or alternatively estimated based on rational analysis of un-drained shear strength or penetration resistance.

At this site, the subsurface conditions include fill overlying compact to very dense non-cohesive sand and silt soil. We note that the boreholes only extended to a maximum depth of 6.6 mbgs. The site designation for seismic analysis is estimated as Class D (OBC 4.1.8.4 Table 4.1.8.4.A.) for seismic design purposes.

5.7 TEMPORARY EXCAVATIONS AND GROUNDWATER CONTROL

The details for the proposed services installations are not available at the time of preparing this report. The recommendations provided below assume that conventional depths for services will be carried out (approximately three (3) to four (4) mbgs).

Based upon the subsurface conditions at the borehole locations, excavations can be carried out with a heavy hydraulic backhoe. It is recommended that provision be carried in the contract for the excavation and disposal of obstructions on site, including cobbles and boulders.

All temporary excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). In accordance with OHSA, the deposits (assuming they are above the groundwater table or properly dewatered) would be classified as a Type 3 soil. If space limitations exist due to adjacent structures or facilities, consideration could be given to the construction of a temporary support system to provide protection to the structures and/or facilities. All excavated spoil should be placed at least the depth of the trench away from the edge of the trench for safety reasons.

As noted above, water was measured as high 4.0 mbgs (BH21-4) in the wells installed at the site. We further stress that the groundwater will likely rise during the spring and fall seasons. As such and depending upon the season of construction, it is possible that dewatering will be required at the site for any excavation greater than 3.5 mbgs and an Environmental Activity and Sector Registry (EASR) or a Permit to Take Water (PTTW) would be required for the service excavations.

Daily water takings of 50 m3/day require registration of the MECP EASR database, and daily water takings of 400 m3/day require a PTTW. Both the EASR and the PTTW require a hydrogeological assessment report to support the specific application. In addition, a permit to discharge the collected water to the sewer system/water body will be required from the applicable agency. A PTTW application requires a minimum of 90 days for the MOECC to process; in this regard, appropriate lead time should be factored into the overall project schedule to accommodate the PTTW process, if required.

In any areas requiring dewatering, the groundwater table must be lowered a minimum of one meter below the lowest excavation level. A specialized dewatering contractor should be retained to design and install the dewatering system.

5.8 LATERAL EARTH PRESSURES

The lateral earth pressure for the design of retaining walls, shoring, or trench boxes can be estimated from the following expressions; this equation assumes that the backfill is effectively drained to eliminate hydrostatic pressure on the wall.

$$p = K(\gamma h + q)$$

Where:

p	=	Lateral earth pressure in kPa acting at depth h
K	=	Earth pressure coefficient, assuming vertical walls and horizontal backfill for permanent construction
γ	=	Unit weight of backfill
h	=	Depth to point of interest in metres
q	=	Equivalent value of surcharge on the ground surface in kPa

The above expression assumes that the perimeter drainage system prevents the buildup of any hydrostatic pressure behind the wall.

The suggested soil parameters (unfactored) for the retaining wall design and/or ground support systems are provided in Table 5.2.

Table 5-2 Soil Parameters

Soil Type	Unit Weight γ (kN/m ³)	Effective angle of internal friction (Φ')	Coefficient of Earth Pressure		
			Active Ka	At rest Ko	Passive Kp
Granular A	22	35	0.27	0.43	3.69
Granular B	21	32	0.31	0.47	3.25
Non-Cohesive Deposits	19	30	0.33	0.50	3.00

It is essential that imported free-draining OPSS Granular 'B' type fill be used as backfill against foundation walls. Backfilling of the footing wall excavations is recommended to be placed in 200 mm thick lifts, uniformly compacted to 100% SPMDD to proposed sub-grade elevations.

5.9 PIPE BEDDING AND COVER

The native soil above the groundwater level or properly dewatered soil, where excavations are extended below the groundwater level, will provide adequate support for the sewer pipes and allow the use of normal Class B type bedding. The recommended minimum thickness of granular bedding below the invert of the pipes is 150 mm. The thickness of the bedding may, however, must be increased depending on the pipe diameter or in accordance with local standards or if wet or weak subgrade conditions are encountered, especially when the soil at the trench base level consists of wet, dilatant silt.

The bedding material should consist of well graded granular material such as Granular 'A' or equivalent. The bedding material should be compacted to at least 95 percent of its SPMDD. After installing the pipe on the bedding, a granular surround of approved bedding material, which extends at least 300 mm above the obvert of the pipe, or as set out by the local authority or municipality, should be placed. It is recommended that WSP be on site during excavations to assess the suitability of the subgrade materials to support the pipes.

If localized wet trench conditions are encountered, a uniformly graded clear stone may be used provided a suitable, approved filter fabric (geotextile) is placed in conjunction with the clear stone. The geotextile must extend underneath the clear stone, along the sides of the trench, and wrapped on top of the clear stone such that **the clear stone is fully wrapped by the geotextile**. A minimum geotextile overlap of 1 m is required; alternatively stitching of the geotextile could be considered. **WSP should be on site on a full-time basis if this method is being considered.**

Localized, wet and unstable soils encountered within generally stable soil zones can be generally stabilized by 'punching' a 50 mm well graded crusher run limestone pad into the soft subgrade prior to bedding placement. The thickness of the 'pad' will depend on field conditions and should be examined by WSP personnel during the construction operations.

5.10 TRENCH BACKFILL

The excavated native soils can be used as construction backfill provided their moisture content at the time of placement is within 2% of the optimum moisture content. Boulders or cobbles greater than 200 mm in size should be removed from the trench backfill. Portions of the fill / reworked soils contained organic materials; any soils with organics should not be used as trench backfill.

For the non-cohesive soils, smooth drum type vibratory rollers are recommended. Cohesive soils, if encountered or imported to the site for fill, should be compacted with sheepfoot type vibratory compactors. The trench backfill should be placed in maximum 0.3 m lift thickness and compacted to at least 98 percent of its SPMDD. Trench backfilling operations should be avoided during freezing weather.

It is preferable that the native soils be re-used from approximately the position at which they are excavated so that frost response characteristics of the soils after construction remain essentially similar. If required, consideration may also be given to backfilling trenches with a well graded, compacted granular soil such as Granular 'B' material.

It should be noted that the excavated soils are subject to moisture content increase during wet weather which would make these materials too wet for the compaction requirements noted above. Stockpiles should therefore be covered with tarpaulins to help minimize moisture increases.

5.11 PRELIMINARY PAVEMENT DESIGN

The investigation has shown that the predominant subgrade soils encountered at the site, after stripping any fill, will be non-cohesive sand and silt or possibly compacted fill.

Prior to the placement of granular materials as part of the pavement structure, the subgrade should be prepared and heavily proof-rolled under the supervision of WSP. Any poorly performing areas should be sub-excavated and

replaced with either granular earth fill approved by WSP or imported Granular B, Type I material conforming to the requirements of OPSS.

Based on the above, the following minimum pavement thickness is recommended:

PAVEMENT LAYER	COMPACTION REQUIREMENTS	DRIVEWAY	PARKING
Asphaltic Concrete	92.0 to 96.5% Maximum Relative Density (MRD)	40 mm HL 3	
		50 mm HL 4 / HL 8	50 mm HL 4
OPSS Granular A Base	100% SPMDD	150 mm	150 mm
OPSS Granular B	100% SPMDD	300 mm	300 mm

We note that the pavement design noted above should be considered preliminary only. If required, a more refined pavement structure design can be performed based on specific traffic data and design life requirements and will involve specific laboratory tests to determine frost susceptibility and strength characteristics of the subgrade soils, as well as specific data input from the client.

5.12 INFILTRATION CHARACTERISTICS

Graphical depiction of the laboratory grain size analyses performed on a sample recovered from the borehole advanced near the proposed infiltration area are provided on **Enclosures 8 and 9 – Laboratory Analyses**. Based on the gradation results, the material encountered is tabulated below.

MATERIAL	BOREHOLE SAMPLE	PERCOLATION TIME PERMEABILITY (min/cm)
Silt and Sand Fill	BH21-3, Sample 3	20 to 40
Silt and Sand	BH21-5, Sample 6	20 to 40

We note that the Percolation Time (“T” time) or Permeability of the subsoil sampled was estimated. The materials, as defined in the Ministry of the Environment Manual of Policy, Procedures and Guidelines for Onsite Sewage Systems, in the appendices 6.3.1 and 6.3.2, mostly resemble sand and gravel with Medium Permeability or sand silt with Medium to Low Permeability.

We must state that these values are strictly for an unsaturated sample.

The value is solely based on the grain size distribution analysis shown in appendices 6.3.1 and 6.3.2 in the Ministry of the Environment Manual of Policy, Procedures and Guidelines for Onsite Sewage Systems. Furthermore, the estimates provided is indicative of the sample in a disturbed state only. We must emphasize that factors between boreholes such as, but not limited to, structure, consistency, density, organic content and degree of saturation influence the estimates.

An accurate analysis of soil infiltration characteristic must be determined with on-site permeameter testing at the location and level of the proposed infiltration condition.

5.13 DESIGN REVIEW, TESTING AND INSPECTIONS

WSP requests to be afforded the opportunity to complete a final review of the proposed development discussed in this report to verify that geotechnical recommendations are appropriate. If not given this opportunity, we cannot assume liability for omissions, misinterpretations or deficiencies in our recommendations.

WSP should be contacted to provide geotechnical testing and inspections during construction operations. Exposed subgrade soils for all structures are to be inspected to confirm the material is stable and competent. Inspections of seepage and groundwater conditions during construction are also required, as discussed in this report. Testing and inspections for general QA/QC are to include sampling and laboratory testing of fill materials and asphalt, compaction testing for the placement of fill materials and asphalt, and field and laboratory testing of concrete (including mix design reviews).

6 GENERAL COMMENTS AND LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment considering the information available to WSP Canada Inc. at the time of preparation. Unless otherwise agreed in writing by WSP Canada Inc., it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on the information determined at the test hole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of test holes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. WSP Canada Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.

FIGURES

FIGURE 1 : SITE LOCATION PLAN

FIGURE 2 : BOREHOLE LOCATION PLAN





NOTE: BASE IMAGE OBTAINED FROM COUNTY OF SIMCOE MAPS, 2018.

 561 BRYNE DRIVE, UNITS C & D BARRIE, ONTARIO CANADA L4N 9Y3 TEL.: 705-735-9771 FAX: 705-735-6450 WWW.WSP.COM	PROJECT:	GEOTECHNICAL STUDY, 181 BURTON AVE., BARRIE, ON.	SCALE:	1:2000
	CLIENT:	MONOLITE HOLDINGS INC.	DATE:	FEB / 2021
	TITLE:	SITE LOCATION PLAN	PROJECT NO:	201-09517-00
			FIGURE NO:	1
			REV. #	



 561 BRYNE DRIVE, UNITS C & D BARRIE, ONTARIO CANADA L4N 9Y3 TEL.: 705-735-9771 FAX: 705-735-6450 WWW.WSP.COM	PROJECT:		SCALE:	
	GEOTECHNICAL STUDY 181 BURTON AVENUE, BARRIE, ONTARIO		1:600	
			DRAWN BY:	
			KMT	
	TITLE:		PROJECT NO:	
			201-09517-00	
			DATE:	
FEB / 2021				
		FIGURE NO:	REV. #.	
		2	-	

ENCLOSURES

ENCLOSURES 1 – 5: BOREHOLE LOGS

ENCLOSURES 6 – 9: LABORATORY ANALYSES



PROJECT: 181 Burton Avenue
CLIENT: Monolite Holdings Inc.
PROJECT LOCATION: Barrie, ON
DATUM: Geodetic
BH LOCATION: See Figure 2

Method: Solid Stem Auger
Diameter: 100 mm
Date: Jan-11-2021

REF. NO.: 201-09517-00
ENCL NO.: 1

[illegible]

GROUNDWATER ELEVATIONS

	1st	2nd	3rd	4th
Measurement				

GRAPH
NOTES

+ 3, × 3: Numbers refer to Sensitivity

○ $\epsilon = 3\%$ Strain at Failure

PROJECT: 181 Burton Avenue
CLIENT: Monolite Holdings Inc.
PROJECT LOCATION: Barrie, ON
DATUM: Geodetic
BH LOCATION: See Figure 2

Method: Solid Stem Auger
Diameter: 100 mm
Date: Jan-11-2021

REF. NO.: 201-09517-00
ENCL NO.: 2

[illegible]

GROUNDWATER ELEVATIONS

	1st	2nd	3rd	4th
Measurement				

GRAPH
NOTES

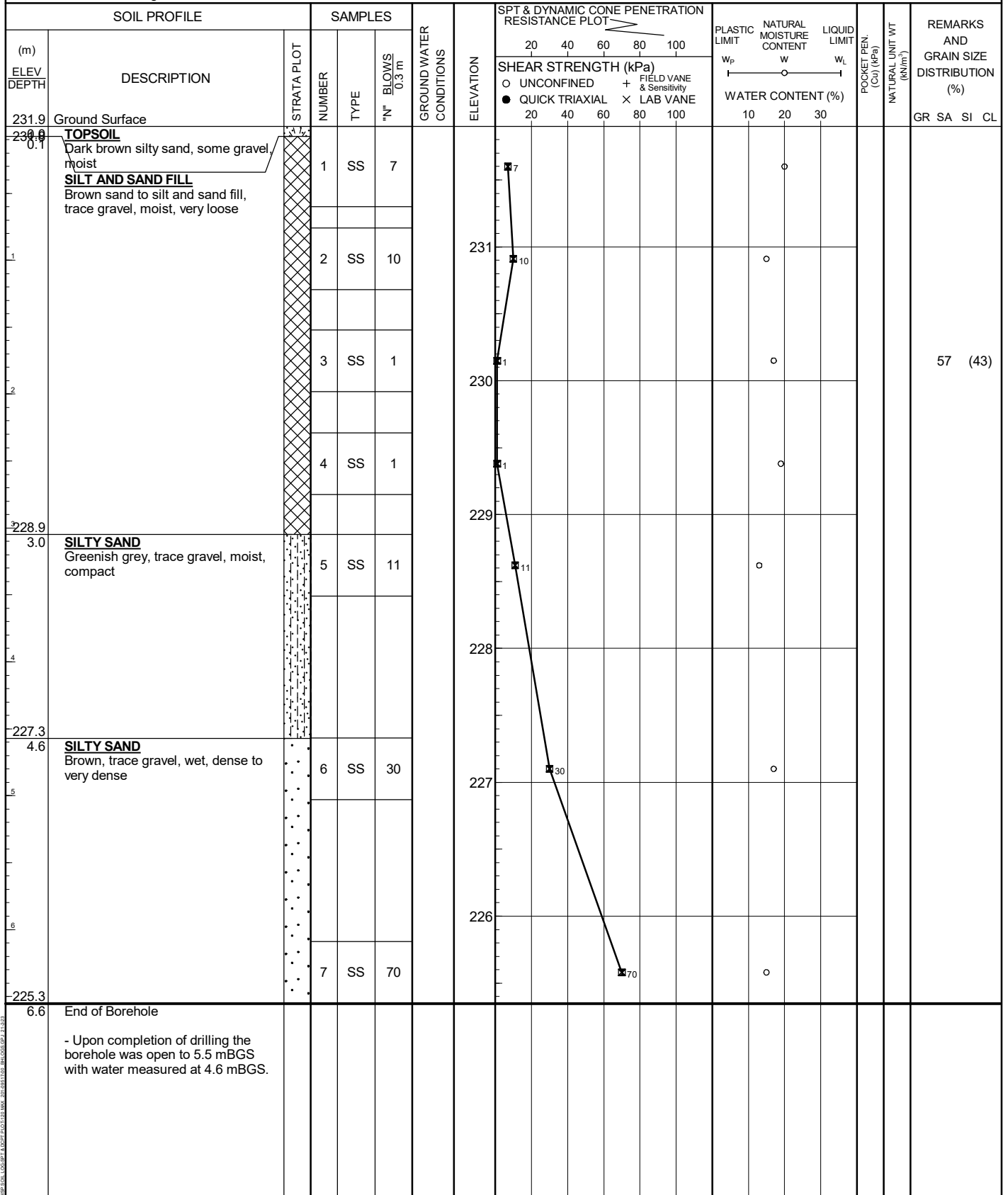
+ 3, × 3: Numbers refer to Sensitivity

○ $\epsilon = 3\%$ Strain at Failure

PROJECT: 181 Burton Avenue
 CLIENT: Monolite Holdings Inc.
 PROJECT LOCATION: Barrie, ON
 DATUM: Geodetic
 BH LOCATION: See Figure 2

Method: Solid Stem Auger
 Diameter: 100 mm
 Date: Jan-11-2021

REF. NO.: 201-09517-00
 ENCL NO.: 3



GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th

GRAPH NOTES

+ 3, × 3: Numbers refer to Sensitivity

○ = 3% Strain at Failure

PROJECT: 181 Burton Avenue
CLIENT: Monolite Holdings Inc.
PROJECT LOCATION: Barrie, ON
DATUM: Geodetic
BH LOCATION: See Figure 2

Method: Solid Stem Auger
Diameter: 100 mm
Date: Jan-11-2021

REF. NO.: 201-09517-00
ENCL NO.: 4

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION	SPT & DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	POCKET PEN. (C _u) (kPa)	NATURAL UNIT WT (kN/m ³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m			SHEAR STRENGTH (kPa)							
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE & Sensitivity × LAB VANE						
231.6	Ground Surface														
230.9	TOPSOIL Dark brown silty sand, some gravel, moist		1	SS	5										
	SAND FILL Brown, some silt, trace gravel, moist, loose to compact		2	SS	5										
			3	SS	13										
			4	SS	bounced										
228.5	SILTY SAND Brown, trace gravel, trace clay, moist to wet, compact		5	SS	17										
			6	SS	28										
	- Brownish grey, wet														
226.5	End of Borehole														

SPT & DYNAMIC CONE PENETRATION RESISTANCE PLOT
 20 40 60 80 100
 SHEAR STRENGTH (kPa)
 ○ UNCONFINED + FIELD VANE & Sensitivity
 ● QUICK TRIAXIAL × LAB VANE
 WATER CONTENT (%)
 10 20 30
 W_P W W_L

W. L. 227.5 m
 Water level measured on February 10, 2021 at 4.03 mBGS

Water level
measured on
February 10,
2021 at 4.03
mBGS

GROUNDWATER ELEVATIONS

	1st	2nd	3rd	4th
Measurement				

GRAPH
NOTES

+ 3, × 3: Numbers refer to Sensitivity

○ **ε**=3% Strain at Failure

PROJECT: 181 Burton Avenue
CLIENT: Monolite Holdings Inc.
PROJECT LOCATION: Barrie, ON
DATUM: Geodetic
BH LOCATION: See Figure 2

Method: Solid Stem Auger
Diameter: 100 mm
Date: Jan-11-2021

REF. NO.: 201-09517-00
ENCL NO.: 5

[illegible]

GROUNDWATER ELEVATIONS

	1st	2nd	3rd	4th
Measurement				

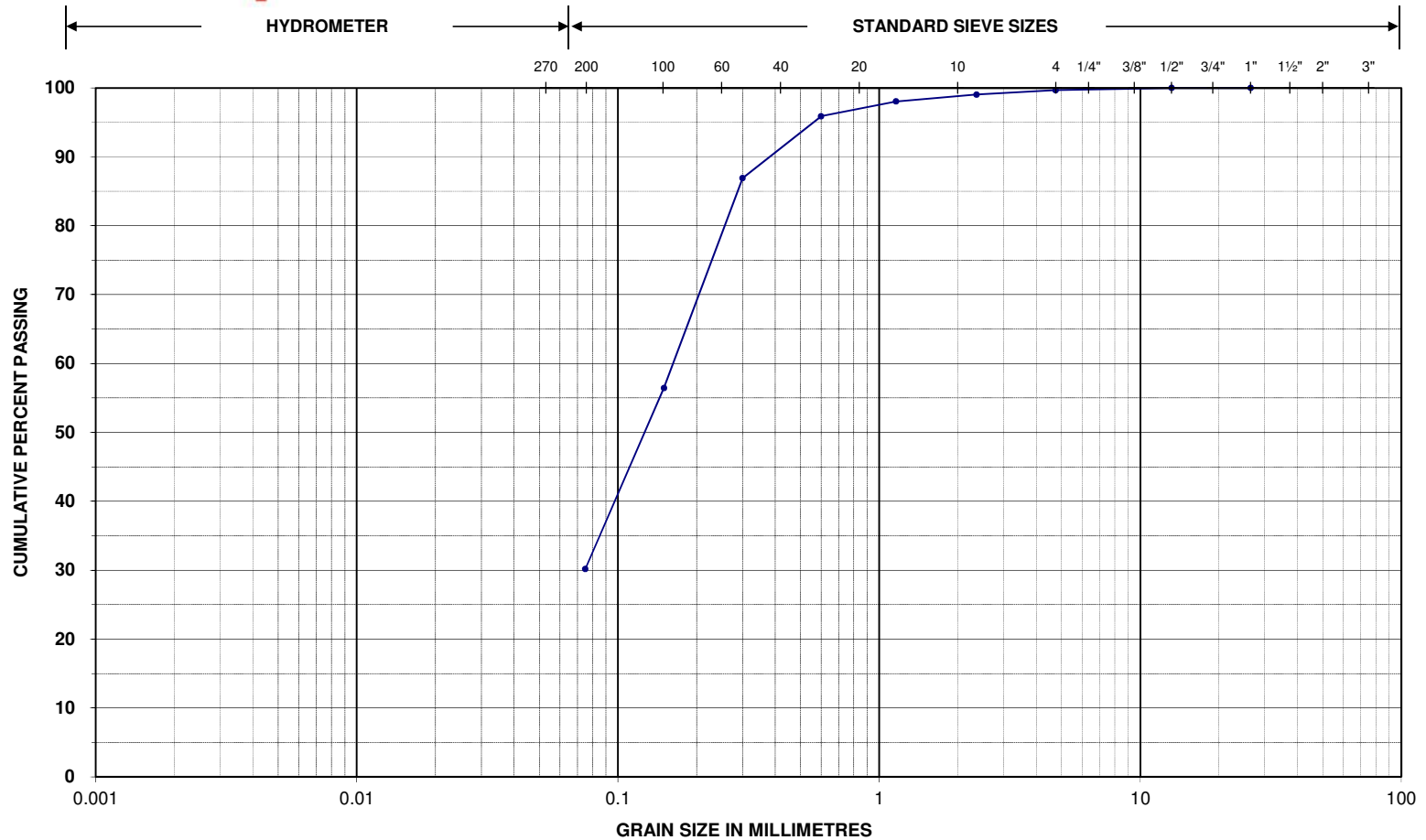
GRAPH
NOTES

$+^3, \times^3$: Numbers refer to Sensitivity

○ **ε**=3% Strain at Failure



PARTICLE SIZE DISTRIBUTION



Project Name: 181 Burton Ave.

Project No.: 201-09517-00

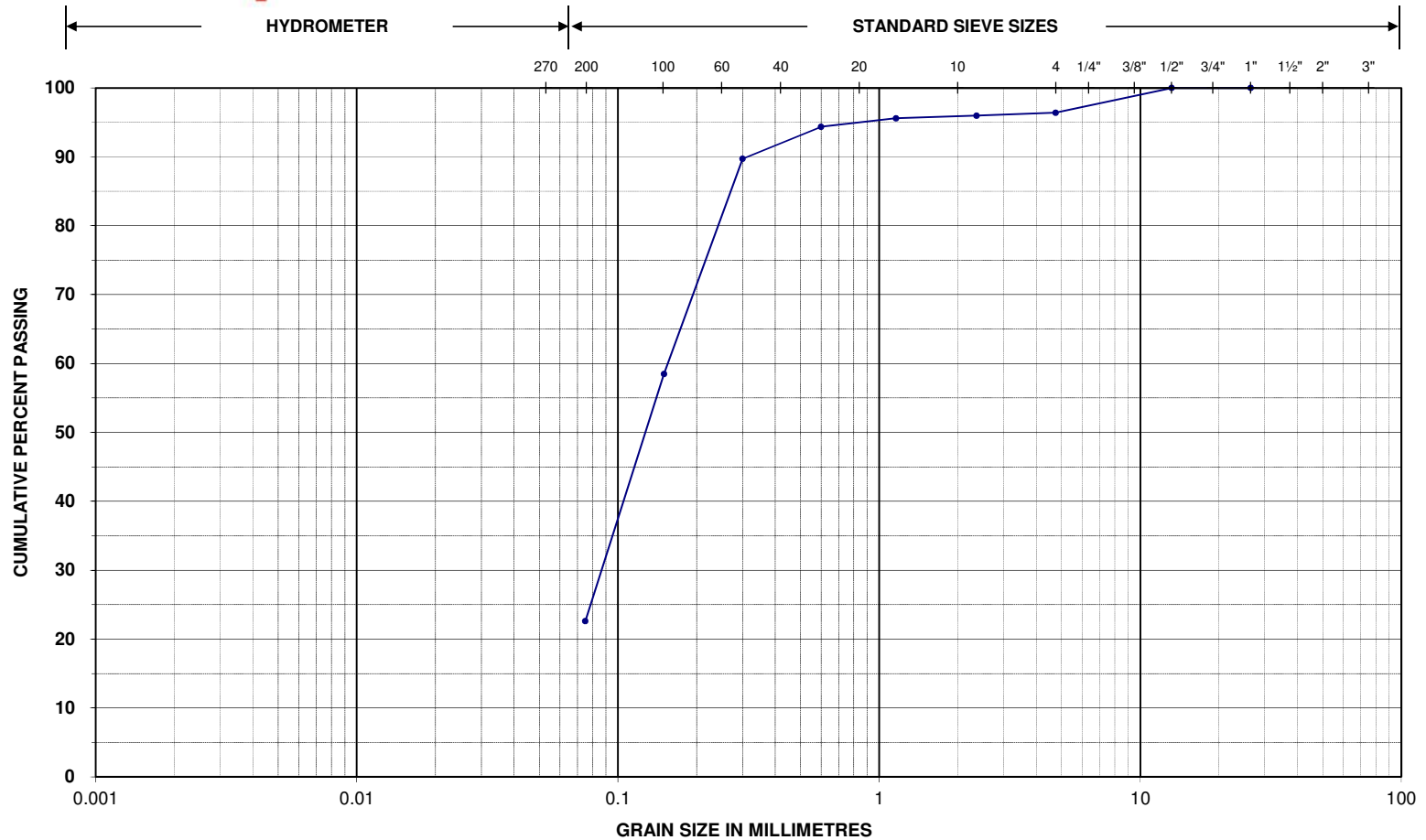
Location ID.: BH21-1

Sample No./Depth: SS2 / 0.8-1.2m

Sieve Size	% Passing Coarse	Sieve Size	% Passing Fine
37.5 mm	100.0	1.16 mm	98.0
26.5 mm	100.0	0.60 mm	95.9
13.2 mm	100.0	0.30 mm	86.9
4.75 mm	99.7	0.15 mm	56.5
2.36 mm	99.1	0.075 mm	30.2



PARTICLE SIZE DISTRIBUTION



Unified Classification System

SILT AND CLAY	SAND	GRAVEL
---------------	------	--------

Project Name: 181 Burton Ave.

Project No.: 201-09517-00

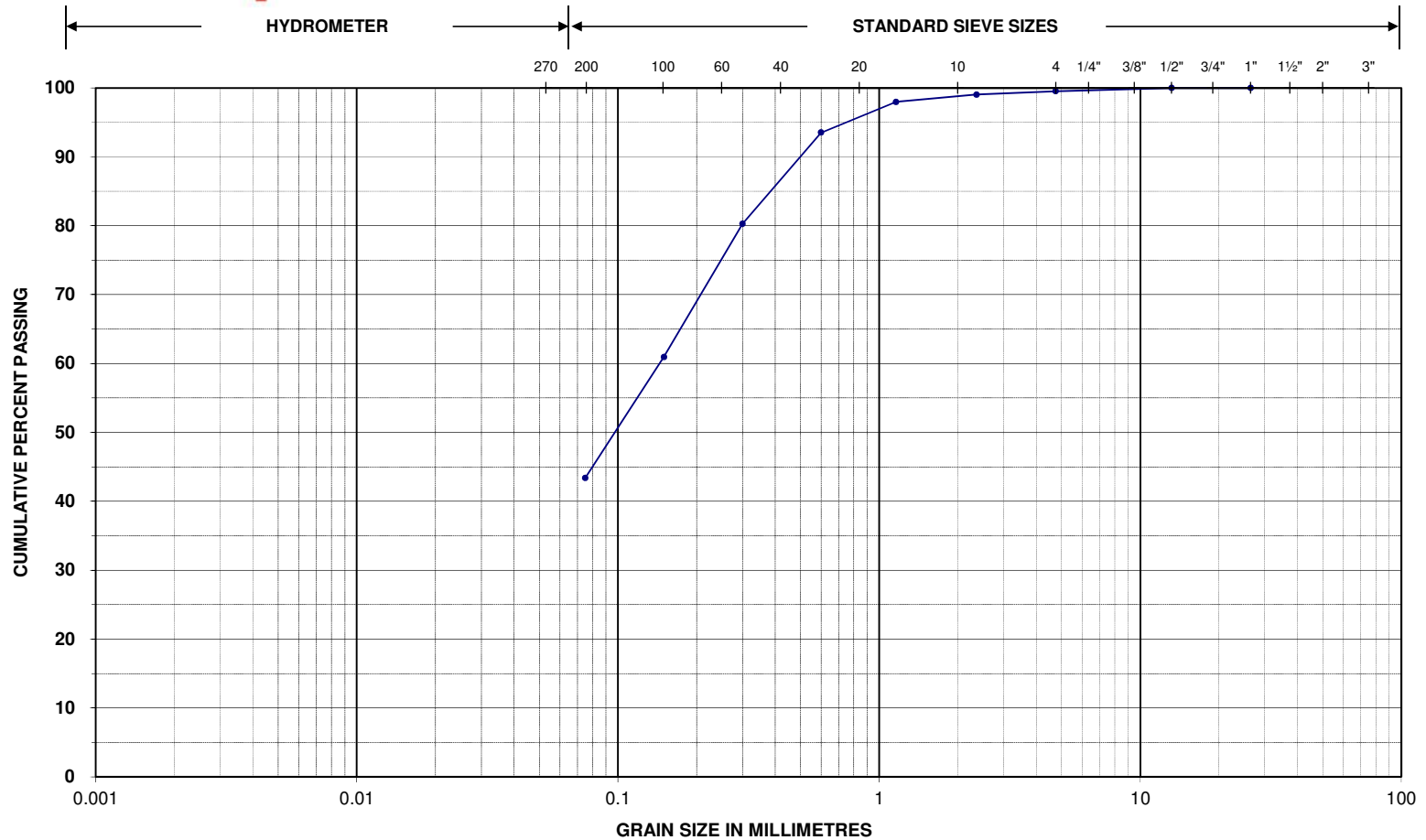
Location ID.: BH21-2

Sample No./Depth: SS6 / 4.6-5.0m

Sieve Size	% Passing Coarse	Sieve Size	% Passing Fine
37.5 mm	100.0	1.16 mm	95.6
26.5 mm	100.0	0.60 mm	94.4
13.2 mm	100.0	0.30 mm	89.7
4.75 mm	96.4	0.15 mm	58.5
2.36 mm	96.0	0.075 mm	22.6



PARTICLE SIZE DISTRIBUTION



Unified Classification System

SILT AND CLAY	SAND	GRAVEL
---------------	------	--------

Project Name: 181 Burton Ave.

Project No.: 201-09517-00

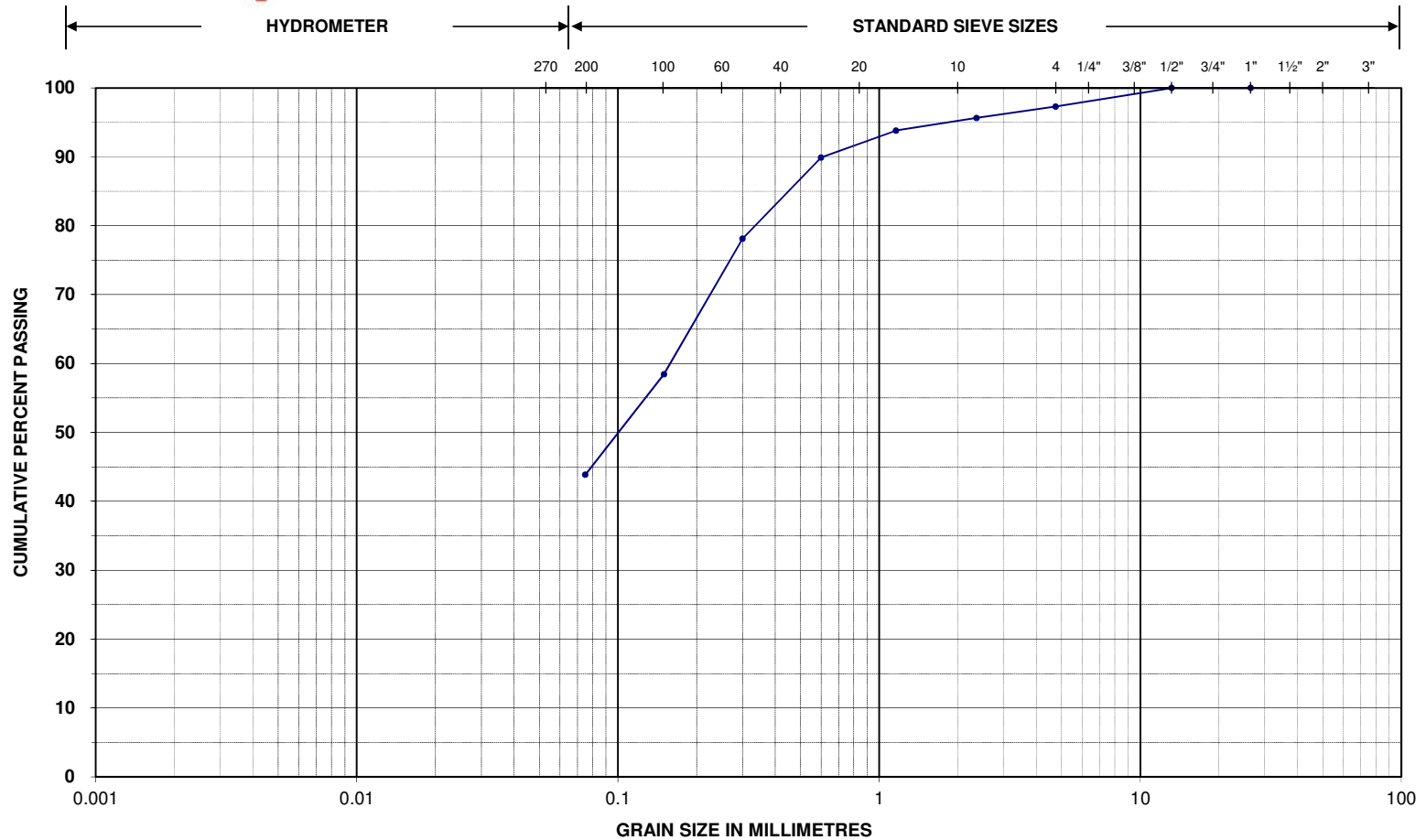
Location ID.: BH21-3

Sample No./Depth: SS3 / 1.5-2.0m

Sieve Size	% Passing Coarse	Sieve Size	% Passing Fine
37.5 mm	100.0	1.16 mm	98.0
26.5 mm	100.0	0.60 mm	93.5
13.2 mm	100.0	0.30 mm	80.3
4.75 mm	99.5	0.15 mm	60.9
2.36 mm	99.0	0.075 mm	43.4



PARTICLE SIZE DISTRIBUTION



Unified Classification System

SILT AND CLAY	SAND	GRAVEL
---------------	------	--------

Project Name: 181 Burton Ave.

Project No.: 201-09517-00

Location ID.: BH21-5

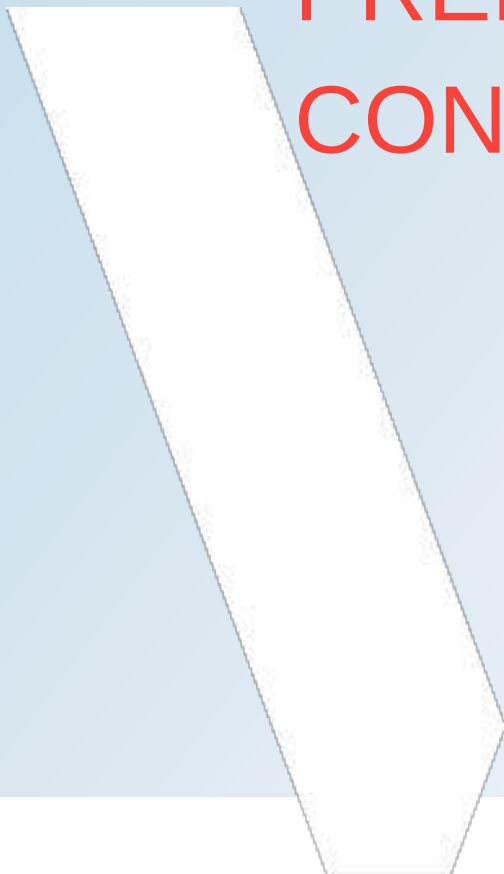
Sample No./Depth: SS6 / 4.6-5.0m

Sieve Size	% Passing Coarse	Sieve Size	% Passing Fine
37.5 mm	100.0	1.16 mm	93.8
26.5 mm	100.0	0.60 mm	89.9
13.2 mm	100.0	0.30 mm	78.1
4.75 mm	97.3	0.15 mm	58.4
2.36 mm	95.6	0.075 mm	43.8

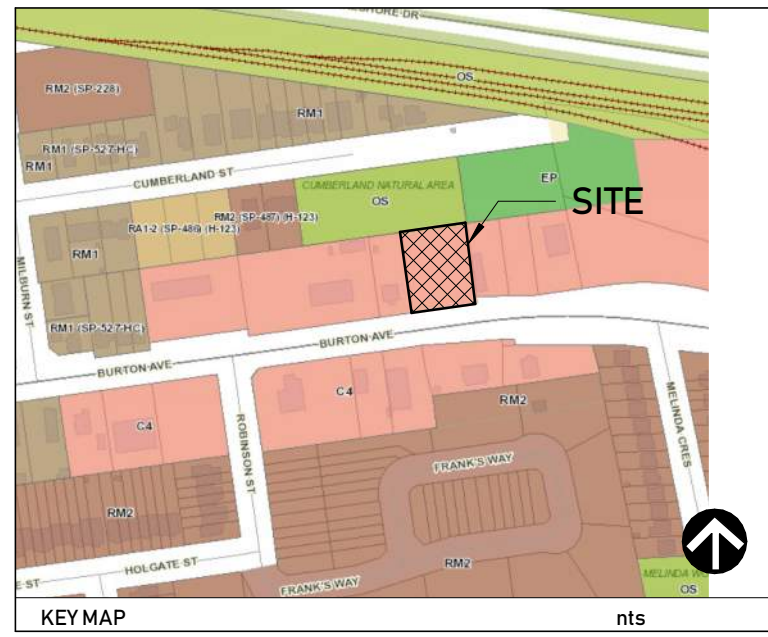
APPENDIX

A

ISM ARCHITECTS
PRELIMINARY
CONCEPT PLANS



SITE PLAN
SCALE: 1:100



ISM ARCHITECTS Inc.

97 Toronto Street
Barrie, ON, L4N 1V1
Tel: 705-726-2342
Email: ian@ismarchitects.ca
Website: www.ismarchitects.ca

[illegible]

Stamp

Orientation



The diagram shows a circle with a vertical line passing through its center. A shaded area is located in the upper right quadrant, bounded by the vertical line and a curve. The label 'N' is placed above the circle, and 'N' is also placed near the top of the vertical line.

Client Information
MARIA ROZENTS

Project
181 BURTON -
RESIDENTIAL

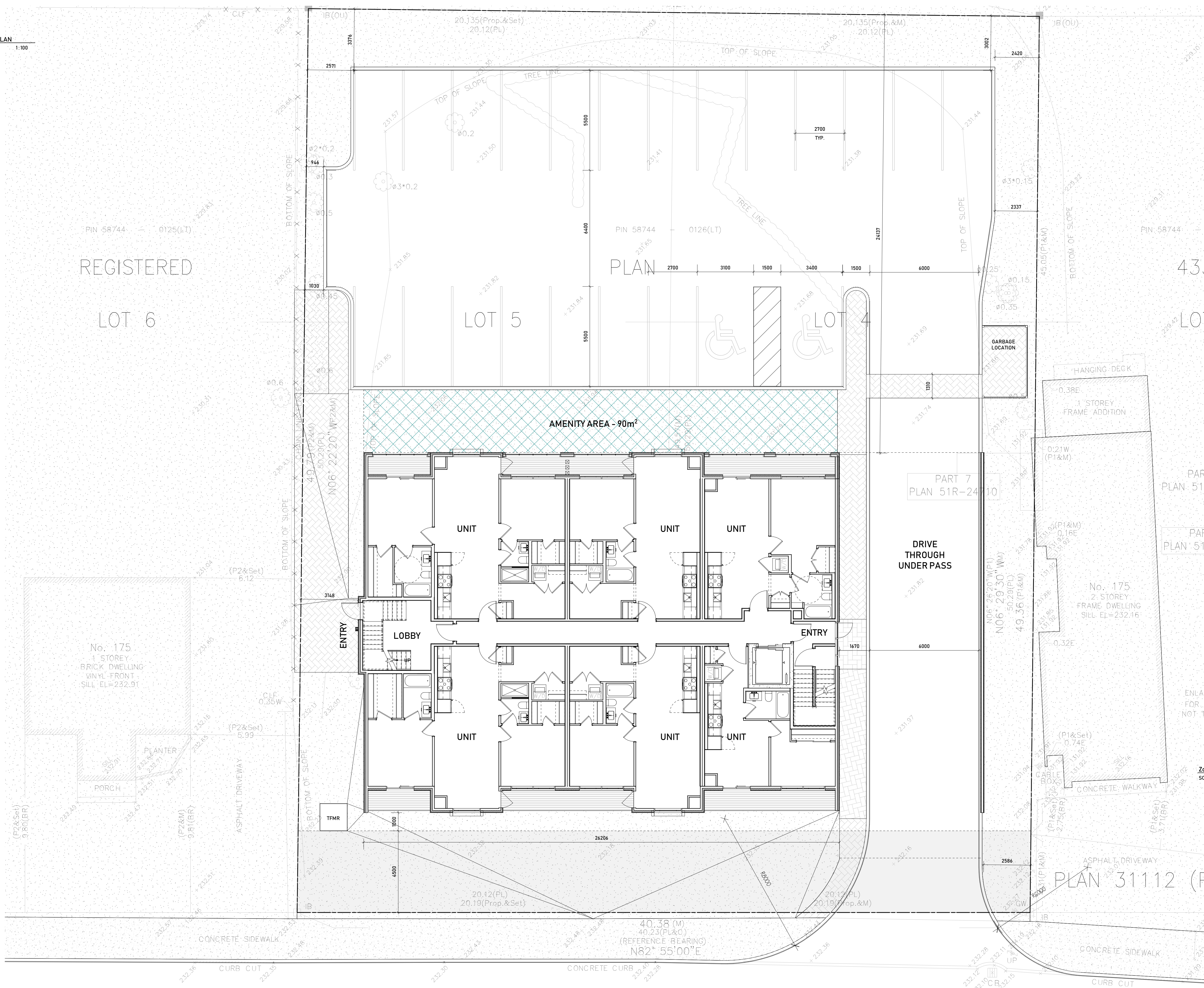
Project Information	
Project No.:	194060
Drawn by:	JB
Checked by:	ISM
Date:	APRIL 16, 2019
Scale:	As indicated

Drawing

SITE PLAN

Drawing No.	

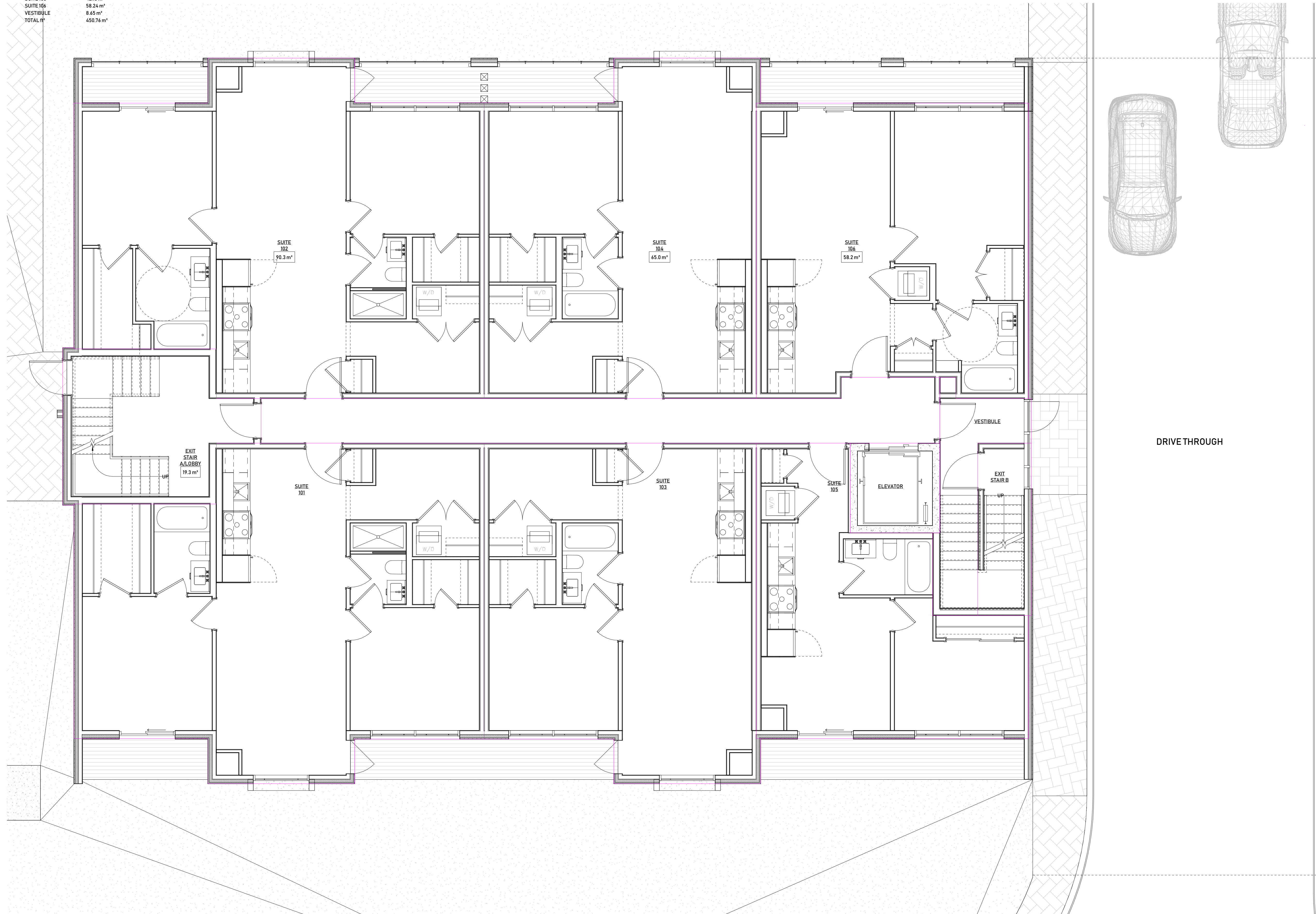
A100



BURTON AVENUE
(BURTON STREET BY REGISTERED PLAN 433)

SCALE: 1:50

AREAS	
ELEVATOR	6.01 m²
EXIT STAIR A	19.32 m²
EXIT STAIR B	6.92 m²
SUITE 101	89.08 m²
SUITE 102	90.26 m²
SUITE 103	65.05 m²
SUITE 104	65.05 m²
SUITE 105	42.18 m²
SUITE 106	58.24 m²
VESTIBULE	8.65 m²
TOTAL ft²	450.76 m²



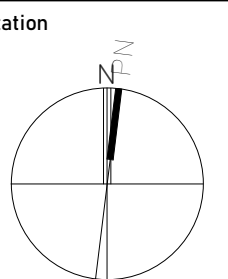
DRIVE THROUGH

ISM ARCHITECTS Inc.

97 Toronto Street
Barrie, ON, L4N 1V1
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Email: ian@ismarchitects.ca
Website: www.ismarchitects.ca

[illegible]

Stamp



Client Information
MARIA ROZENTS

Project
181 BURTON -
RESIDENTIAL

Project Information	
Project No.:	194060
Drawn by:	JB
Checked by:	ISM
Date:	APRIL 16, 2019
Scale:	1:50

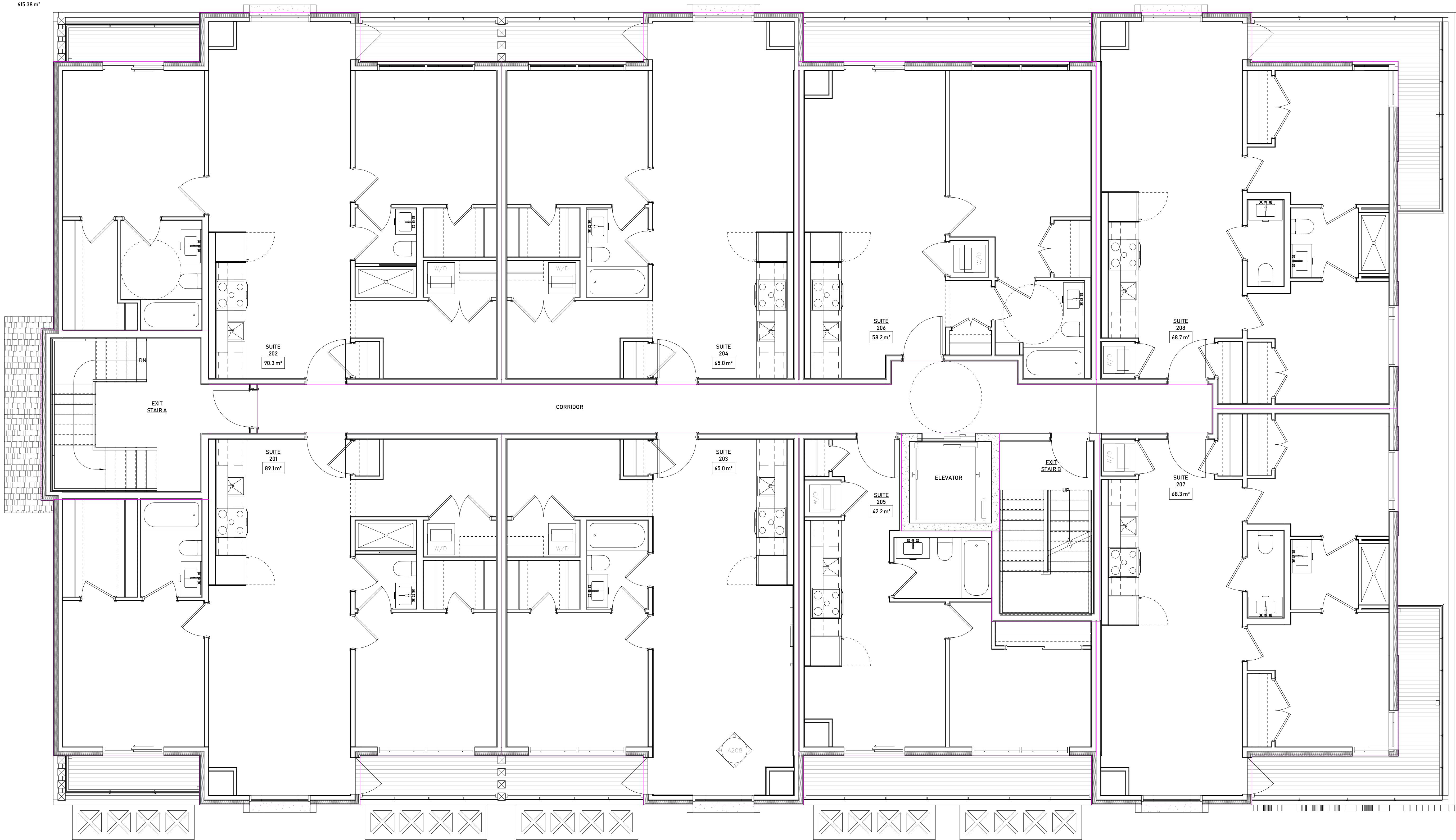
Drawing

**MAIN FLOOR
PLAN**

Drawing No.

A201

SECOND FLOOR	
SCALE:	1:50
AREAS	
CORRIDOR	30.94 m²
ELEVATOR	6.01 m²
EXIT STAIR A	19.32 m²
EXIT STAIR B	12.26 m²
SUITE 201	89.08 m²
SUITE 202	90.26 m²
SUITE 203	65.05 m²
SUITE 204	65.05 m²
SUITE 205	42.18 m²
SUITE 206	58.24 m²
SUITE 207	68.3 m²
SUITE 208	68.69 m²
TOTAL 1 st	615.38 m²

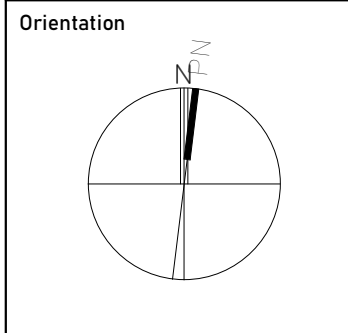


ISM ARCHITECTS Inc.

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Email: ian@ismarchitects.ca
Website: www.ismarchitects.ca

#	Description	Date
1	CLIENT REVIEW	APR 23/19
2	DESIGN REVIEW	MAY 16/19
3	PERMIT REVIEW	MAY 16/19
4	CONTRACT REVIEW	MAY 16/19
5	CONTRACT REVIEW	MAY 16/19
6	CONTRACT REVIEW	MAY 16/19
7	CONTRACT REVIEW	MAY 16/19
8	CONTRACT REVIEW	MAY 16/19
9	CONTRACT REVIEW	MAY 16/19
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97	CONTRACT REVIEW	MAY 16/19
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99	CONTRACT REVIEW	MAY 16/19
100	CONTRACT REVIEW	MAY 16/19

Stamp



Client Information
MARIA ROZENTS

Project
181 BURTON -
RESIDENTIAL

Project Information
Project No.: 194060
Drawn by: JB
Checked by: ISM
Date: APRIL 16, 2019
Scale: 1:50

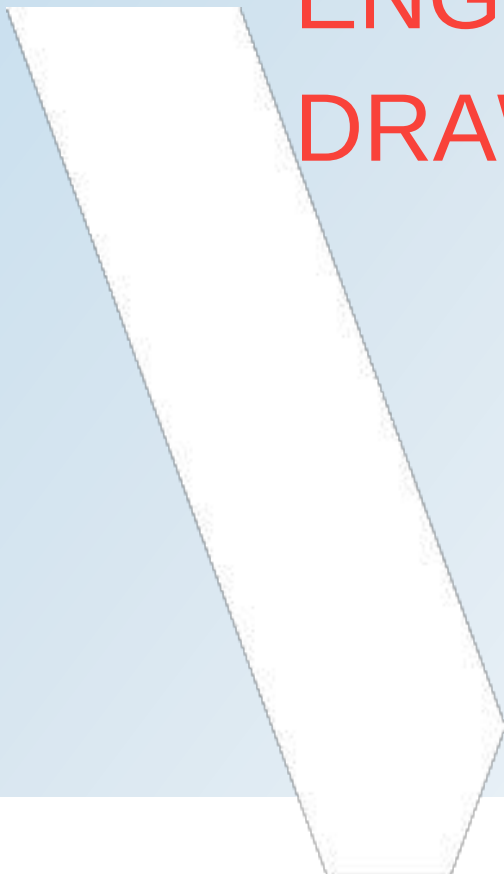
Drawing
SECOND &
THIRD FLOOR
PLAN

Drawing No.
A202

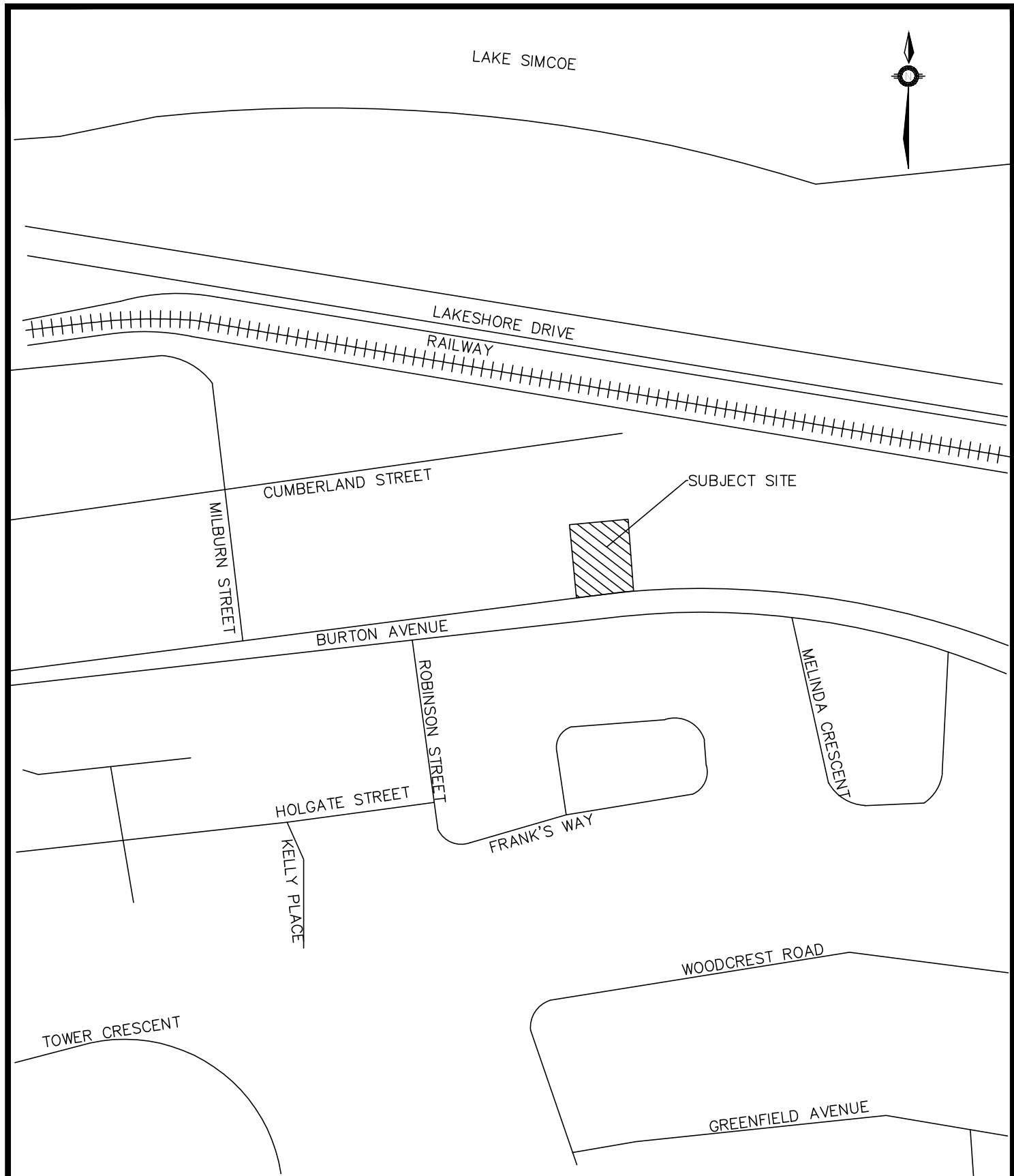
APPENDIX

B

PEARSON
ENGINEERING
DRAWINGS



C:\Users\ocleaves\AppData\Local\Temp\AcPublish_15960\19100-BASE.dwg Layout:FIG-1 Plotted Jul 30, 2020 @ 8:21am by ocleaves @ PEARSON ENGINEERING LTD.



APARTMENT DEVELOPMENT
181 BURTON AVENUE
BARRIE, ON

SITE LOCATION PLAN

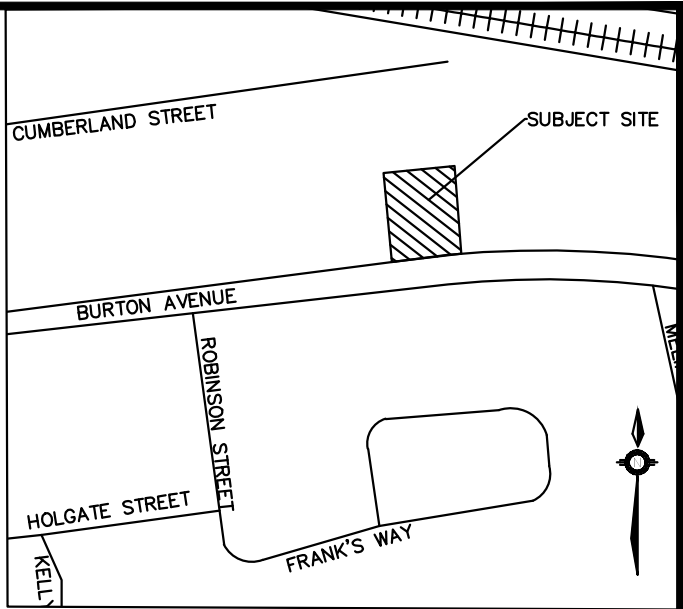
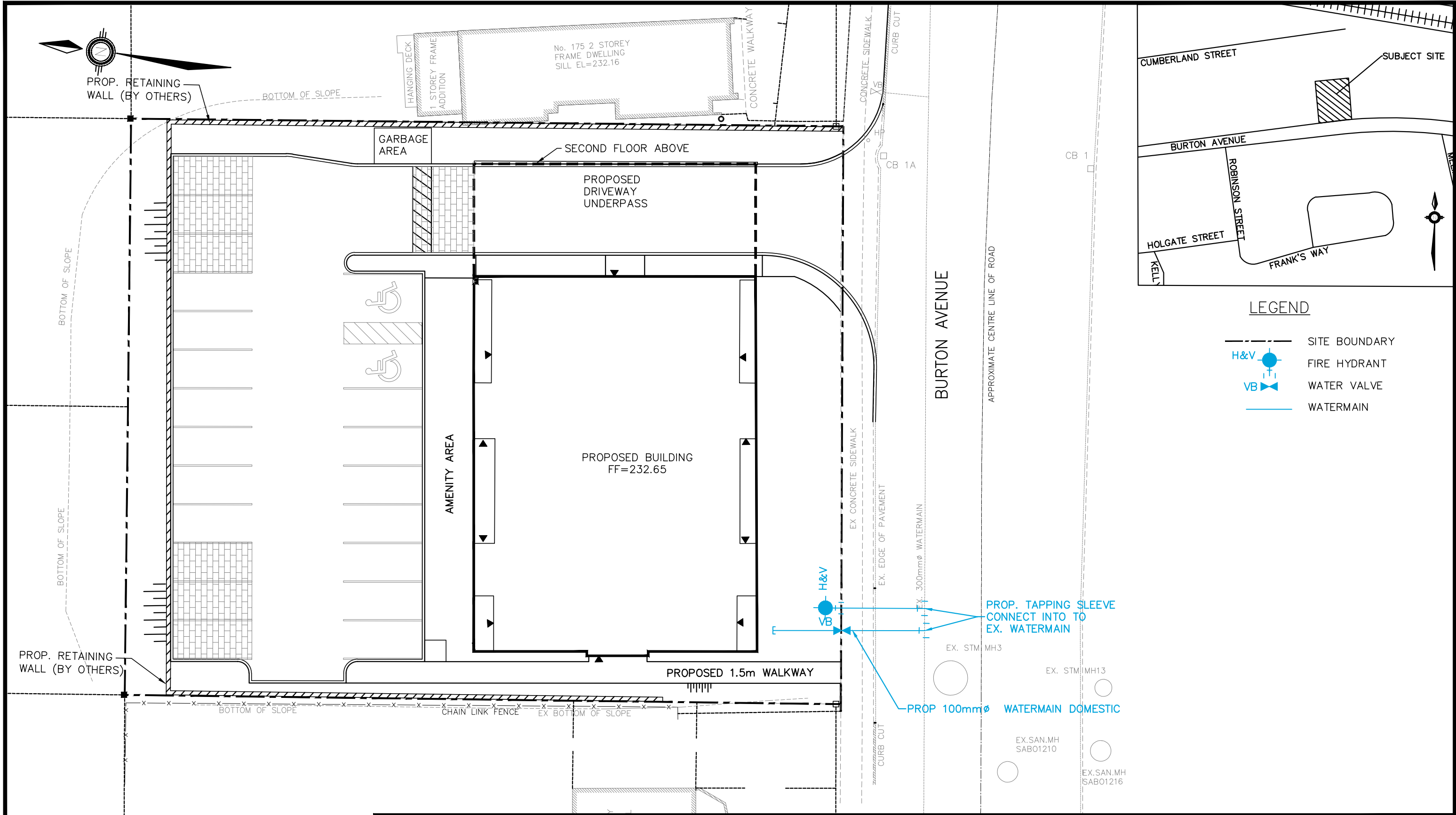


PEARSON
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PEARSONENG.COM PH. 705.719.4785

DESIGNED BY	AMC	HORIZ SCALE	PROJECT #	19100
DRAWN BY	ZZ	VERT SCALE	DRAWING #	FIG-1
CHECKED BY	MWD	DATE	REVISION #	0
		NOVEMBER 2019		

C:\Users\acleaves\AppData\Local\Temp\AcPublish_15960\19100-BASE.dwg Layout:FIG-2 WATER Plotted Jul 30, 2020 @ 8:21am by acleaves @ PEARSON ENGINEERING LTD.



LEGEND

- SITE BOUNDARY
- FIRE HYDRANT
- WATER VALVE
- WATERMAIN

NO.	REVISION NOTE	DATE	BY

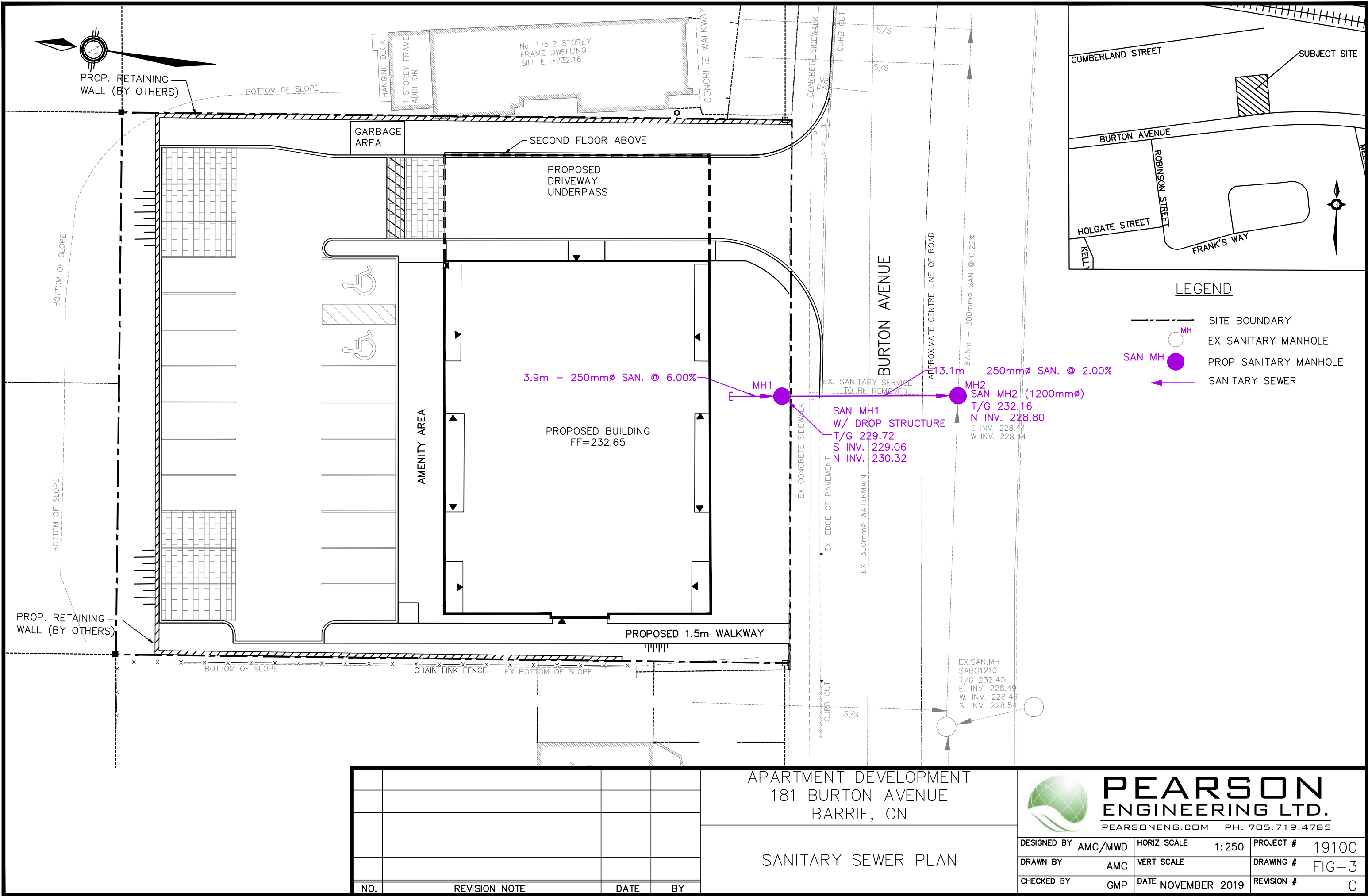
APARTMENT DEVELOPMENT
181 BURTON AVENUE
BARRIE, ON

WATER SERVICING PLAN

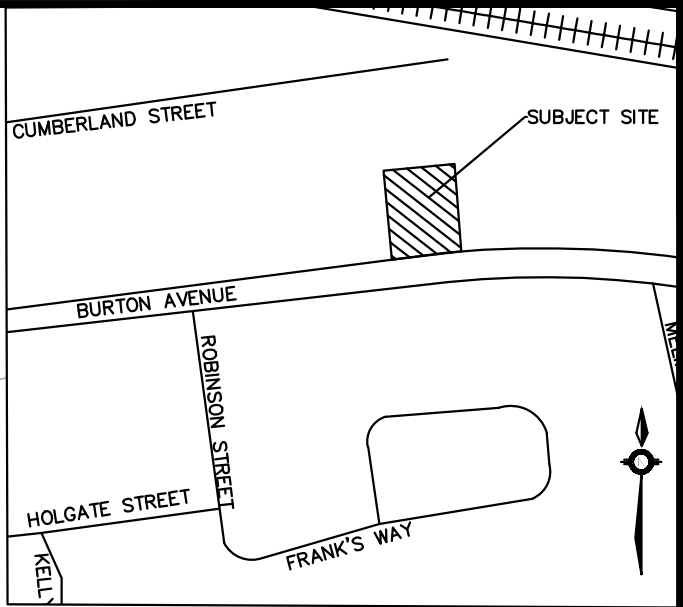
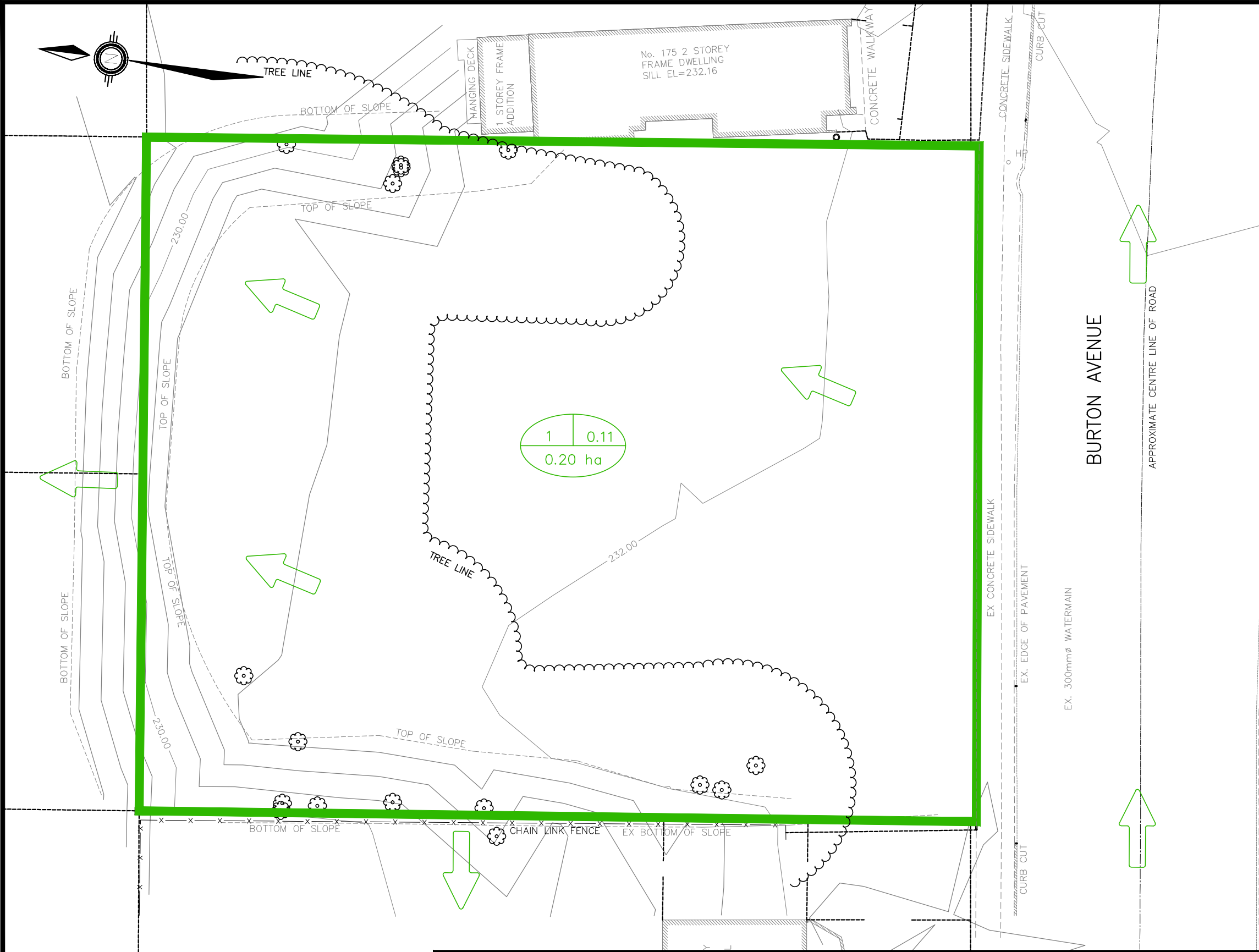


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DESIGNED BY	AMC/MWD	HORIZ SCALE	1: 250	PROJECT #	19100
DRAWN BY	AMC	VERT SCALE		DRAWING #	FIG-2
CHECKED BY	GMP	DATE	NOVEMBER 2019	REVISION #	0



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LEGEND

- SITE BOUNDARY
- ➡ OVERLAND FLOW DIRECTION
- CATCHMENT AREA:

1	0.11
0.20 ha	

 RUNOFF COEFFICIENT:

1	0.75
1.00 ha	

 AREA IN HECTARES
- █ CATCHMENT BOUNDARY

NO.	REVISION NOTE	DATE	BY

APARTMENT DEVELOPMENT
181 BURTON AVENUE
BARRIE, ON

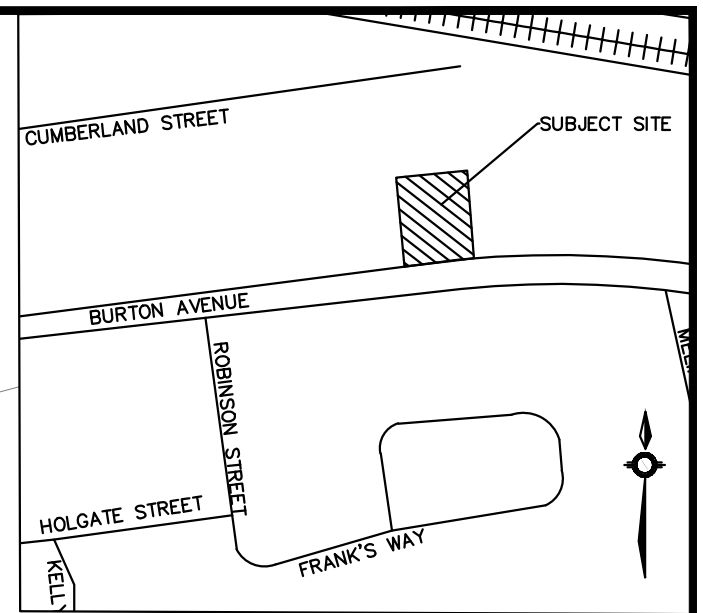
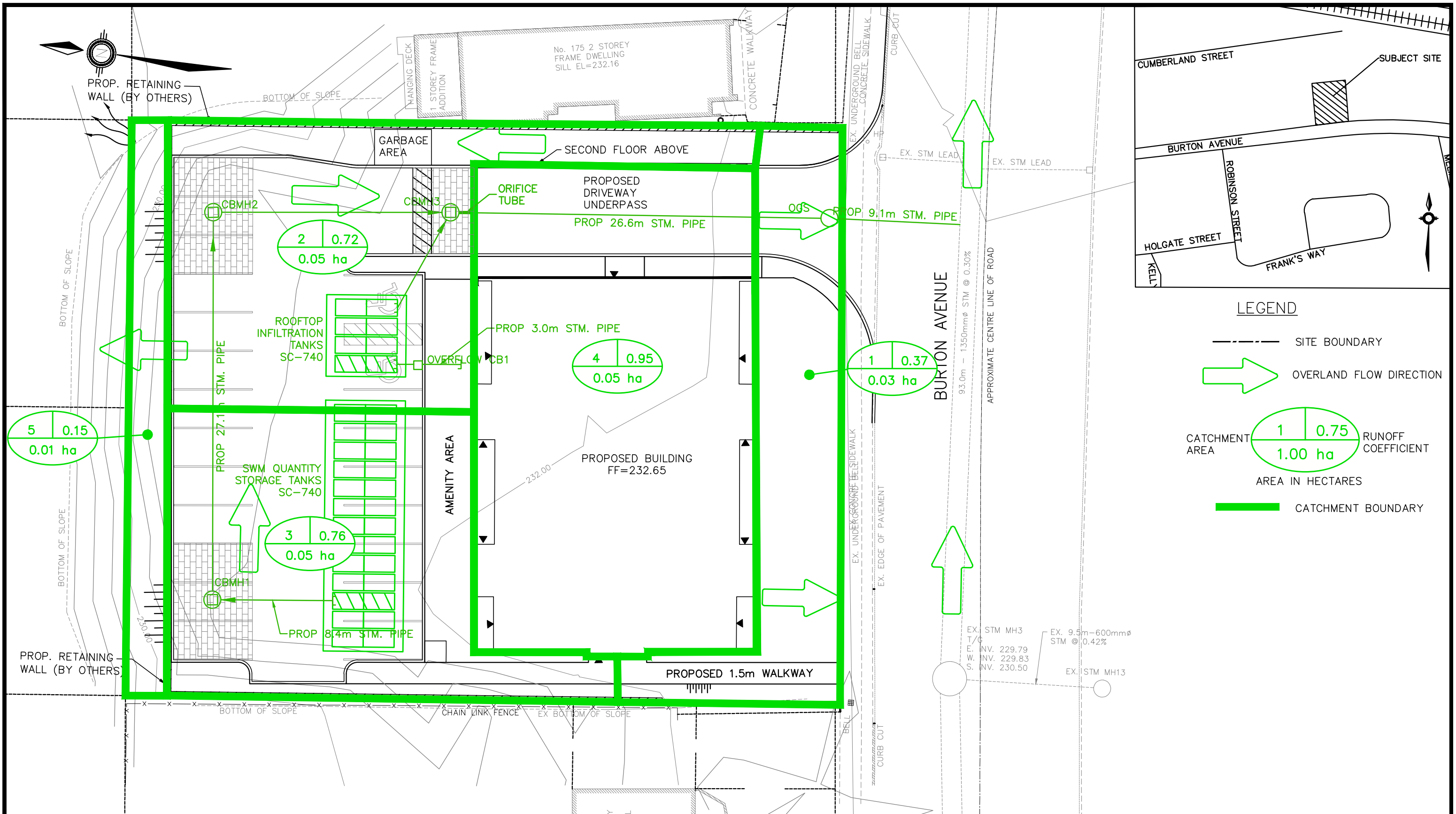
PRE DEVELOPMENT STORM
DRAINAGE PLAN



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DESIGNED BY	AMC/MWD	HORIZ SCALE	1: 250	PROJECT #	19100
DRAWN BY	AMC	VERT SCALE		DRAWING #	FIG 4
CHECKED BY	GMP	DATE	NOVEMBER 2019	REVISION #	0

C:\Users\acleaves\AppData\Local\Temp\AcPublish_15960\19100-BASE.dwg Layout:FIG-5 POST Plotted Jul 30, 2020 @ 8:21am by acleaves @ PEARSON ENGINEERING LTD.

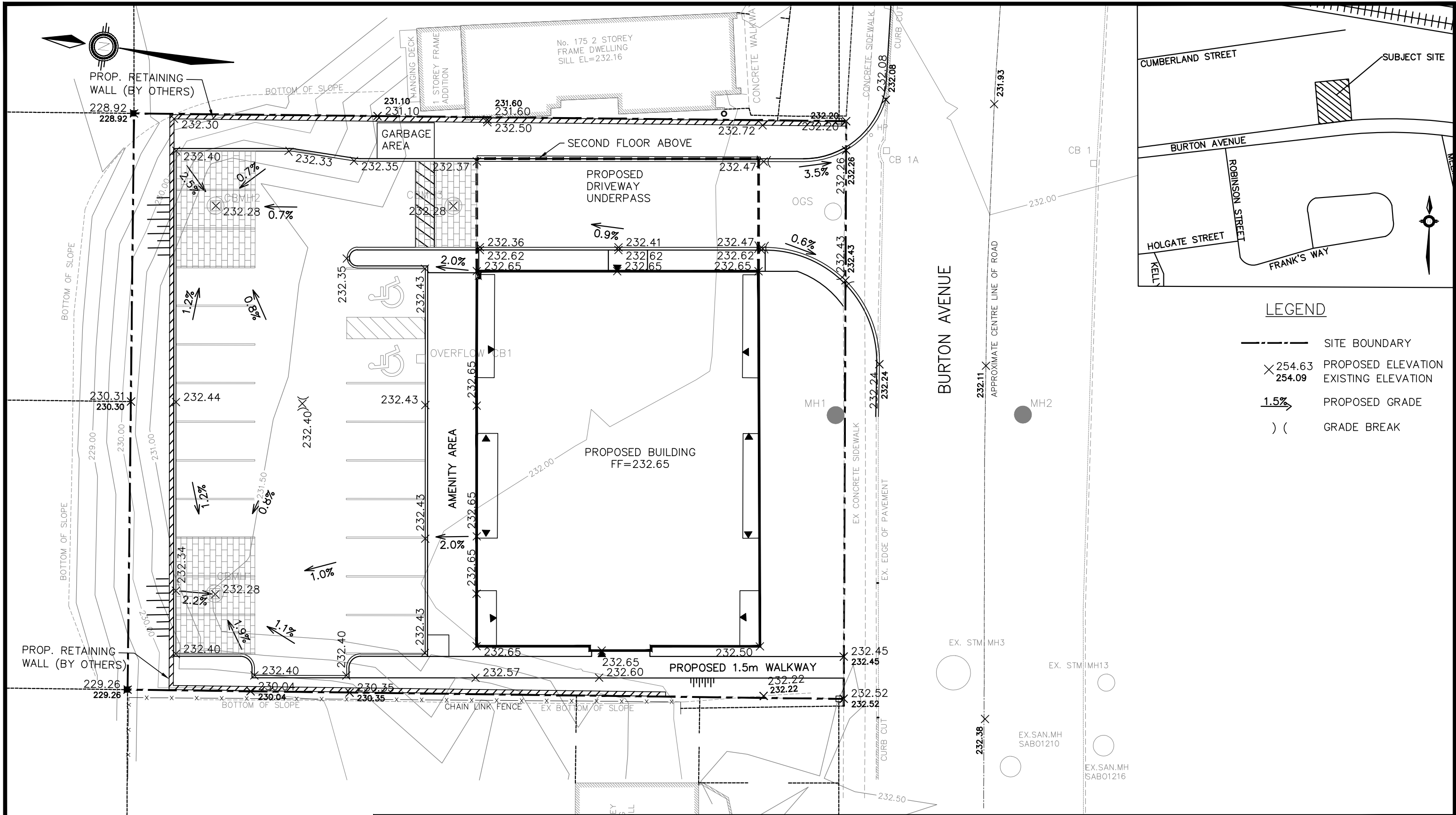


LEGEND

- SITE BOUNDARY
- ➔ OVERLAND FLOW DIRECTION
- CATCHMENT AREA: 1 | 0.75 | 1.00 ha (RUNOFF COEFFICIENT: 0.03, AREA IN HECTARES: 0.05)
- CATCHMENT BOUNDARY

				APARTMENT DEVELOPMENT 181 BURTON AVENUE BARRIE, ON		 PEARSON ENGINEERING LTD. PEARSONENG.COM PH. 705.719.4785			
				POST DEVELOPMENT STORM DRAINAGE PLAN					
NO.	REVISION NOTE	DATE	BY	DESIGNED BY	AMC/MWD	HORIZ SCALE	1:250	PROJECT #	19100
				DRAWN BY	AMC	VERT SCALE		DRAWING #	FIG 5
				CHECKED BY	GMP	DATE	NOVEMBER 2019	REVISION #	0

C:\Users\acleaves\AppData\Local\Temp\AcPublish_15960\19100-BASE.dwg Layout:FIG-6 SG Plotted Jul 30, 2020 @ 8:21am by acleaves @ PEARSON ENGINEERING LTD.



NO.	REVISION NOTE	DATE	BY

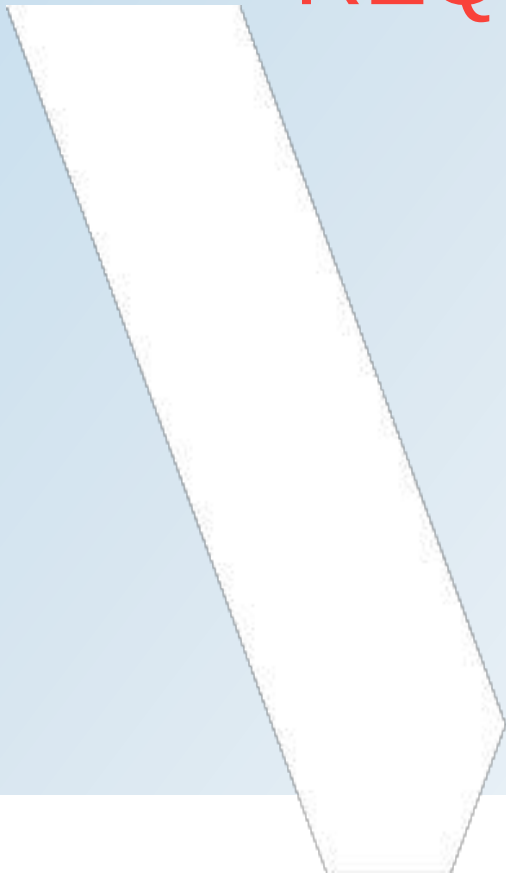


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DESIGNED BY	AMC/MWD	HORIZ SCALE	1: 250	PROJECT #	19100
DRAWN BY	AMC	VERT SCALE		DRAWING #	FIG 6
CHECKED BY	GMP	DATE	NOVEMBER 2019	REVISION #	0

APPENDIX

C ENGINEERED FILL REQUIREMENTS





GENERAL REQUIREMENTS FOR ENGINEERED FILL

Compacted imported soil that meets specific engineering requirements and is free of organics and debris and that has been continually monitored on a full-time basis by a qualified geotechnical representative is classified as engineered fill. Engineered fill that meets these requirements and is bearing on suitable native subsoil can be used for the support of foundations.

Imported soil used as engineered fill can be removed from other portions of a site or can be brought in from other sites. In general, most of Ontario soils are too wet to achieve the 100% Standard Proctor Maximum Dry Density (SPMDD) and will require drying and careful site management if they are to be considered for engineered fill. Imported non-cohesive granular soil is preferred for all engineered fill. For engineered fill, we recommend use of OPSS Granular 'B' sand and gravel fill material.

Adverse weather conditions such as rain make the placement of engineered fill to the required degree of density difficult or impossible; engineered fill cannot be placed during freezing conditions, i.e. normally not between December 15 and April 1 of each year.

The location of the foundations on the engineered fill pad is critical and certification by a qualified surveyor that the foundations are within the stipulated boundaries is mandatory. Since layout stakes are often damaged or removed during fill placement, offset stakes must be installed and maintained by the surveyors during the course of fill placement so that the contractor and engineering staff are continually aware of where the engineered fill limits lie. Excavations within the engineered fill pad must be backfilled with the same conditions and quality control as the original pad.

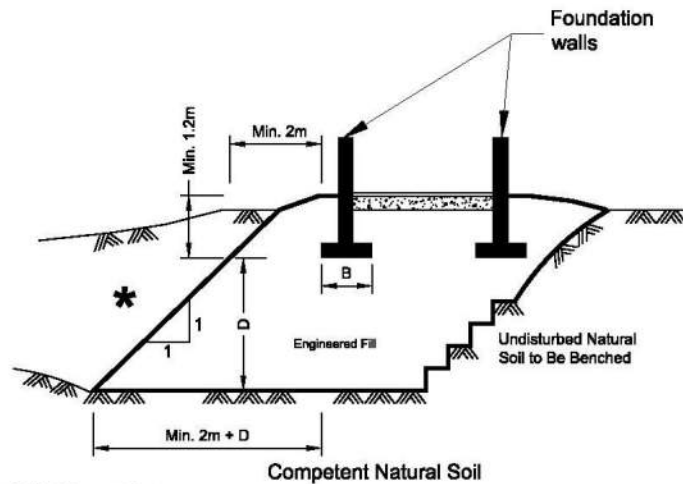
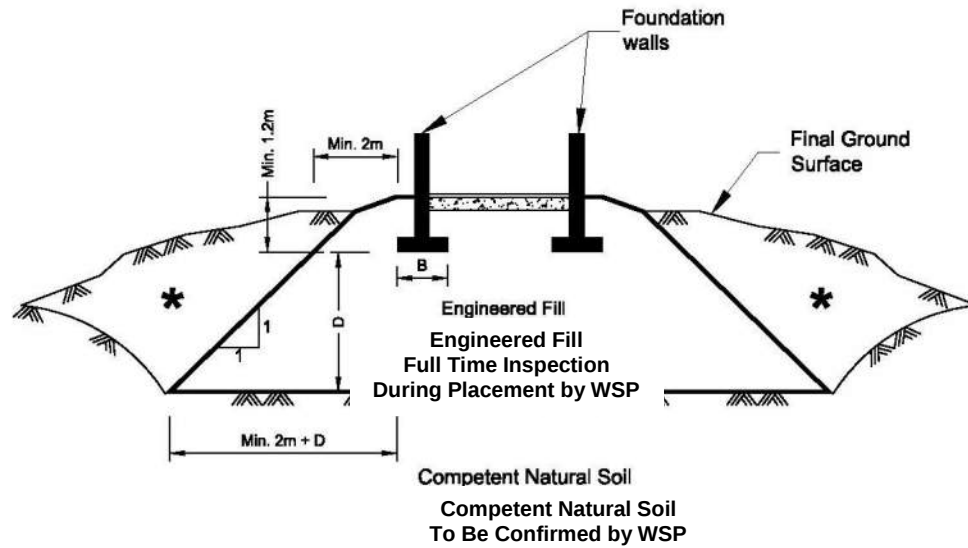
To perform satisfactorily, engineered fill requires the cooperation of the designers, engineers, contractors and all parties must be aware of the requirements. The minimum requirements are as follows, however, the geotechnical report must be reviewed for specific information and requirements.

1. Prior to site work involving engineered fill, a site meeting to discuss all aspects must be convened. The surveyor, contractor, design engineer and geotechnical engineer must attend the meeting. At this meeting, the limits of the engineered fill will be defined. The contractor must make known where all fill material will be obtained from and samples must be provided to the geotechnical engineer for review, and approval before filling begins.
2. Detailed drawings indicating the lower boundaries as well as the upper boundaries of the engineered fill must be available at the site meeting and be approved by the geotechnical engineer.
3. The building footprint and base of the pad, including basements, garages, etc. must be defined by offset stakes that remain in place until the footings and service connections are all constructed. Confirmation that the footings are within the pad, service lines are in place, and that the grade conforms to drawings, must be obtained by the owner in writing from the surveyor and WSP Canada Inc. Without this confirmation no responsibility for the performance of the structure can be accepted by WSP Canada Inc. Survey drawing of the pre and post fill location and elevations will also be required.
4. The area must be stripped of all topsoil and fill materials. Subgrade must be proof-rolled. Soft spots must be dug out. The stripped native subgrade must be examined and approved by a WSP Canada Inc. engineer prior to placement of fill.



5. The approved engineered fill material must be compacted to 100% Standard Proctor Maximum Dry Density throughout. Engineered fill should not be placed during the winter months. Engineered fill compacted to 100% SPMDD will settle under its own weight approximately 0.5% of the fill height and the structural engineer must be aware of this settlement. In addition to the settlement of the fill, additional settlement due to consolidation of the underlying soils from the structural and fill loads will occur and should be evaluated prior to placing the fill.
6. Full-time geotechnical inspection by WSP Canada Inc. during placement of engineered fill is required. Work cannot commence or continue without the presence of the WSP Canada Inc. representative.
7. The fill must be placed such that the specified geometry is achieved. Refer to the attached sketches for minimum requirements. Take careful note that the projection of the compacted pad beyond the footing at footing level is a minimum of 2 m. The base of the compacted pad extends 2 m plus the depth of excavation beyond the edge of the footing.
8. A bearing capacity of 150 kPa at SLS (225 kPa at ULS) can be used provided that all conditions outlined above are adhered to. A minimum footing width of 500 mm (20 inches) is suggested and footings must be provided with nominal steel reinforcement.
9. All excavations must be done in accordance with the Occupational Health and Safety Regulations of Ontario.
10. After completion of the engineered fill pad a second contractor may be selected to install footings. The prepared footing bases must be evaluated by engineering staff from WSP Canada Inc. prior to footing concrete placements. All excavations must be backfilled under full time supervision by WSP Canada Inc. to the same degree as the engineered fill pad. Surface water cannot be allowed to pond in excavations or to be trapped in clear stone backfill. Clear stone backfill can only be used with the approval of WSP Canada Inc.
11. After completion of compaction, the surface of the engineered fill pad must be protected from disturbance from traffic, rain and frost. During the course of fill placement, the engineered fill must be smooth-graded, proof-rolled and sloped/crowned at the end of each day, prior to weekends and any stoppage in work in order to promote rapid runoff of rainwater and to avoid any ponding surface water. Any stockpiles of fill intended for use as engineered fill must also be smooth-bladed to promote runoff and/or protected from excessive moisture take up.
12. If there is a delay in construction, the engineered fill pad must be inspected and accepted by the geotechnical engineer. The location of the structure must be reconfirmed that it remains within the pad.
13. The geometry of the engineered fill as illustrated in these General Requirements is general in nature. Each project will have its own unique requirements. For example, if perimeter sidewalks are to be constructed around the building, then the projection of the engineered fill beyond the foundation wall may need to be greater.

14. These guidelines are to be read in conjunction with WSP Canada Inc. report attached.



* Backfill in this area to be as per WSP report.