Stormwater Management Report & Servicing Brief

The Georgian Apartments
290, 294, 298 & 302 Georgian Drive
City of Barrie
County of Simcoe

PEARSON ENGINEERING LTD.
PEARSONENG.COM

(Revised September 2021)
April 2019
18037

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Stormwater Management Report
& Servicing Brief
290, 294, 298 & 302 Georgian Drive, Barrie

1. Introduction

PEARSON Engineering Ltd. has been retained by TMD-Atria Corporation (Client) to prepare a Stormwater Management Report & Servicing Brief in support of the proposed Georgian Apartments (Project) located on the north side of Georgian Drive in the City of Barrie (City). The subject lands are located west of Johnson Street and east of Gallie Court and can be seen on Figure 1.

The subject property is approximately 1.68 ha in size, consisting of four residential lots with a common driveway into the adjacent townhouse development with a forested area on the northern portion of the site. The Project site slopes from south to north at an average gradient of 6% towards a steep slope on the northern side of the side with an average gradient of 25%. The Project proposes the development of 0.76 ha of the total area through the construction of a 375 unit apartment building with 350 m^2 of commercial space.

This Report assesses the existing municipal infrastructure in the vicinity of the Project and the onsite Stormwater Management (SWM) facilities and internal services required to service the proposed Project. The report also includes design calculations and an outline of the proposed internal services, as well as comments regarding the ability of the various secondary utilities to service the site.

2. Design Population

The proposed development is to consist of an apartment building with a 350 m^2 of commercial space. The apartment building will have one, two, and three bedroom apartments. The townhouses are to be 3 bedroom units. Based on these figures, a design population of 1272 persons is estimated for the project and 350.0 m^2 of commercial space.

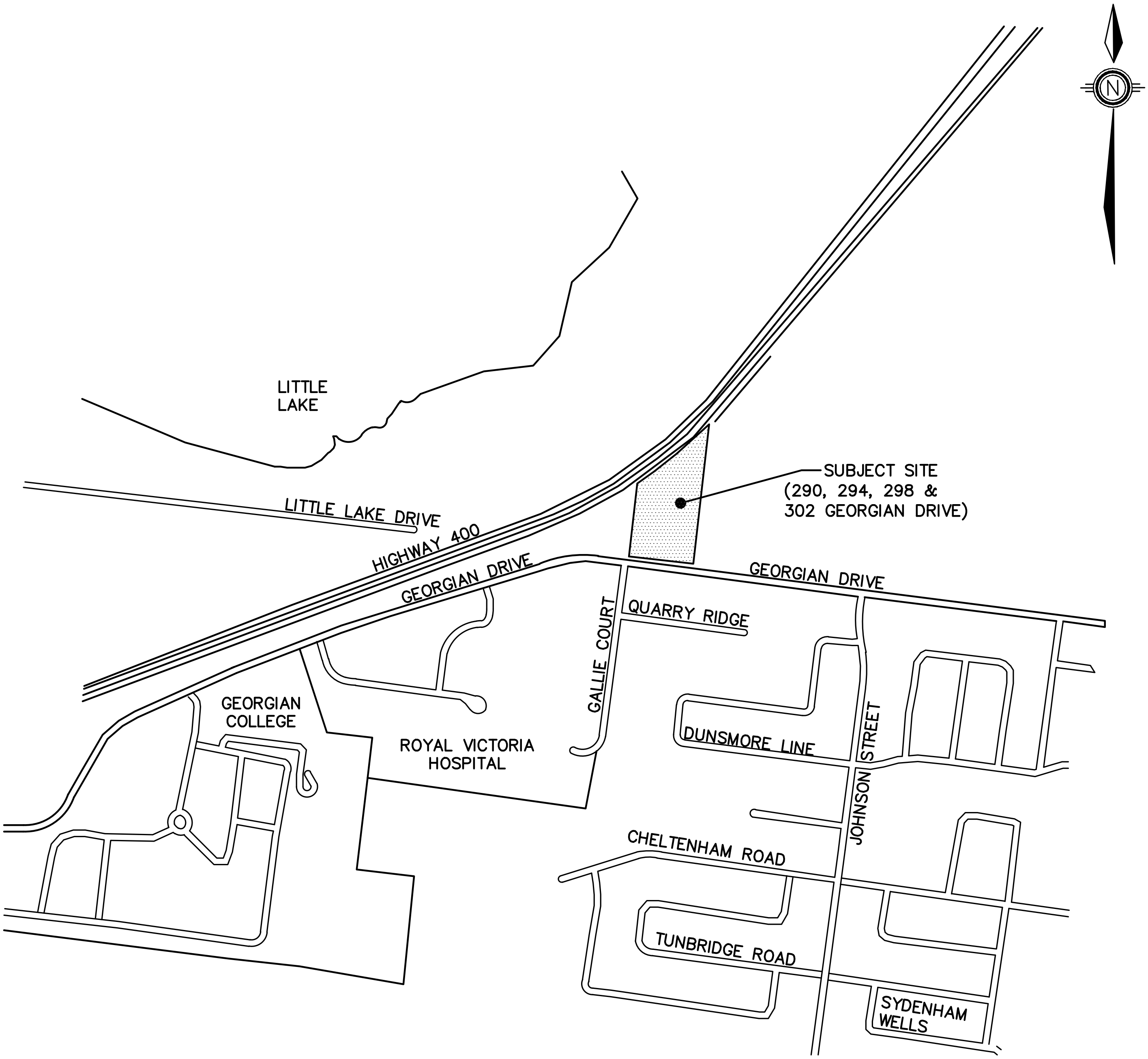
3. Water Supply and Distribution

3.1. Water Servicing Design Criteria

The site is to have a total population of 1,272 persons plus 350.0 m^2 of commercial area. Utilizing the City of Barrie Guidelines for domestic water use of 225 L/capita/day and 5 L/m^2 of floor space for commercial space, the Average Day Demand (ADD) that is required is 3.33 L/sec. The Peak Rate factor of 3.75 is used in calculating the Peak Hour Demand of 12.50 L/sec for the development. Calculations for the domestic water requirements for the site can be found in Appendix A.

3.2. Internal Water Distribution System

To service the project, a 100 mm diameter domestic water service will be extended into the project to the apartment building which will service both the apartment building, commercial space and the townhouse units. The internal water distribution will connect into the existing 300 mm diameter municipal watermain on the south side of Georgian Drive. Refer to DWG SS-1 for the domestic water service layout.



CASCADE RENTAL APARTMENTS
290,294,298,&302 GEORGIAN DRIVE
BARRIE, ONTARIO

SITE LOCATION PLAN

PEARSON ENGINEERING LTD.
PEARSONENG.COM PH. 705.719.4785

DESIGNED BY	PCO	HORIZ SCALE	NTS	PROJECT #	18037
DRAWN BY	MKRW	VERT SCALE		DRAWING #	FIG-1
CHECKED BY	GMP	DATE	SEPTEMBER 2018	REVISION #	0

3.3. Fire Fighting Requirements

A 200 mm diameter fire fighting water service is proposed to extend into the apartment building to provide water supply for internal fire suppression. The proposed building is expected to be sprinklered. A proposed fire hydrant located on the southeast corner of the site will provide adequate firefighting coverage for the proposed development as per City Standards. Refer to DWG SS-1 for the fire hydrant locations.

A fire protection water demand was completed by Vipond Inc., dated September 2021, on the proposed buildings for the project development. The fire flow demands were calculated based on the Ontario Building Code (OBC) and National Fire Protection Association (NFPA) codes and standards. A combined sprinkler standpipe demand of approximately 625 GPM (39 L/s) was calculated for the proposed building. Detailed demand calculations can be found in Appendix B.

A hydrant fire flow test was completed in front of the site in August 2021 indicating that a static pressure of 74 psi was available. This flow test also resulted in a fire flow that can be supplied to the Project site with a flow of approximately 1,390 GPM (88 L/s) at a residual pressure of 60 psi from the existing hydrant. As the required fire flow for the development is 625 GPM (39 L/s), the proposed water infrastructure can supply the flow as per the City of Barrie requirements. Provided fire flow information can be found in Appendix A.

4. Sanitary Servicing

4.1. Sanitary Design Criteria

The site is to have a total population of 1272 persons plus 350.0 m^2 of commercial area. Utilizing the City of Barrie flow value for domestic sewer use of 225 L/capita/day, an Average Daily Flow (ADF) of 3.33 L/s was calculated. Using a Peaking Factor of 3.73 for this project, a peak flow of 12.43 L/sec was calculated for the entire development. The proposed 250 mm diameter sanitary sewer has a capacity of 60 L/sec at 1.0% and is sufficient to convey the sanitary design flows. Sanitary design flow calculations can be found in Appendix B.

4.2. Sanitary Sewer System

The sanitary sewer for the proposed apartment building and townhouse units will converge internally and exit the proposed apartment building on the south side of the building. The proposed 200 mm diameter sanitary sewer from the site will connect into the existing sanitary manhole SAL08205 on Georgian Drive. The proposed sanitary sewer system for the site can be seen on DWG SS-1.

4.3. External Sanitary Sewer Analysis

The sewage disposal system that currently services Georgian Drive consists of a 250 mm diameter gravity collection system at 0.36% with a capacity of 0.04 m^3/s. The existing sewer drains easterly down Georgian Drive, then southerly down Johnson Street, eventually draining to the existing Grove Street Pump Station. The proposed Project will increase sanitary flow by 12.43 L/s which will be added to manhole SAL08205, located in front of the site on Georgian Drive. Based on the sanitary sewer design sheet, the maximum percent full in the existing 250 mm sewer is 45.6%. As this is less than the maximum of 50% as per City of Barrie guidelines for a sanitary sewer less than 375 mm in diameter, the existing sewer on Georgian Drive has sufficient capacity to convey flows from the site.

As per discussions with the City of Barrie, the Sanitary Sewer capacity calculations of the downstream sewer have been completed using the hydraulic design approach. The pipe capacity of the downstream sanitary sewer currently has sufficient capacity to convey flows down Johnson Street. Since a new sanitary capacity model is being completed by the City of Barrie, we suggest that the proposed flows from the site are included in the updated analytical model to determine the impacts, if any, on the sanitary sewer in the future.

5. Stormwater Management

A key component of the development is the need to address environmental and related SWM issues. These are examined in a framework aimed at meeting the City, Nottawasaga Valley Conservation Authority (NVCA), and Ministry of the Environment, Conservation and Parks (MECP) requirements. SWM parameters have evolved from an understanding of the location and sensitivity of the site's natural systems. This Report focuses on the necessary measures to satisfy the MECP's SWM requirements.

It is understood the objectives of the SWM plan are to:

- Protect life and property from flooding and erosion
- Maintain water quality for ecological integrity, recreational opportunities etc.
- Protect and maintain groundwater flow regime(s).
- Protect aquatic and fishery communities and habitats.
- Maintain and protect significant natural features.

5.1. Analysis Methodology

The design of the SWM Facilities for this site has been conducted in accordance with:

- The Ministry of the Environment Stormwater Management Planning and Design Manual, March 2003
- City of Barrie, Storm Drainage and Stormwater Management Policies and Design Guidelines – December 2017
- Nottawasaga Valley Conversation Authority Stormwater Technical Guide – December 2013

5.2. Existing Conditions

The existing Project site is comprised of four residential lots with a forested area to the north. On the east side of the Project site, a joint driveway entrance with the adjacent townhouse development has been constructed. Topographic survey of the site identifies that the majority of the site's stormwater flows from the south to the north towards Highway 400 via overland flow. Details of existing drainage conditions are shown on DWG STM-1.

According to the drawing "Georgian Drive and External Storm Drainage Areas" completed by Jones Consulting Group Ltd. November 2000, an area of 0.04 ha on the southeast corner of the site with a runoff coefficient of 0.45 and an area of 0.01 ha in the southwest corner of the site with a runoff coefficient of 0.79 has been allocated to contribute flow into the storm sewer on Georgian Drive.

Golder Associates Ltd. completed a Geotechnical and Hydrogeological Investigation on the site which identifies surficial layers of fine sandy loam across most of the site.

Allowable peak flows for the site were calculated using the site's conditions prior to the construction of the shared driveway entrance between the adjacent townhouse development and can be seen in Table 1 below. Detailed calculations for the existing drainage conditions can be found in Appendix C.

Table 1: Pre-Development Peak Flows

	2 Year (m^3/s)	5 Year (m^3/s)	10 Year (m^3/s)	25 Year (m^3/s)	50 Year (m^3/s)	100 Year (m^3/s)
Flows to North towards Highway 400 (m^3/s)	0.06	0.08	0.09	0.11	0.14	0.16
Allowable Flows to Georgian Drive (m^3/s)	0.01	0.01	0.01	0.01	0.01	0.02

5.3. Proposed Storm Drainage System

The post development drainage for the development will generally follow pre-development conditions. The area on the south side of the proposed building will drain via overland flow towards Georgian Drive and into the existing storm sewer system. The small area of driveway leading towards the underground parking will drain into a catch basin and flow via gravity into the building's internal storm sewer. Rooftop runoff from the apartment building and the proposed townhouse units will drain internally and outlet to a concrete storage tank located within the building. Two orifice tubes in the storage tank will control flows and outlet to a dispersion basin located on the slope north of the building which will create sheet flow through a 40 m wide overflow weir. Post development storm drainage patterns are shown on drawing STM-2.

5.4. Stormwater Quantity Control

Quantity control on site will be provided using a concrete storage tank located within the apartment building. All stormwater flows from the proposed building and proposed townhouse units will converge internally, and outlet directly into the proposed storage tank. The storage tank will provide 72 m^3 of storage with a depth of 2.0 m. A 150 mm orifice tube and a 200 mm orifice tube will be used to reduce the post development peak flow from the storage area to pre-development levels. The storage tank will outlet to a 40 m long dispersion basin which will convey the peak flow as sheet flow towards Highway 400.

Uncontrolled flows from the south side of the building will be nominally greater than the allowable flow to Georgian Drive. The proposed 5-year flow from the site towards Georgian Drive is 0.02 m^3/s while the allowable is 0.01 m^3/s. The existing 375 mm storm pipe in front of the site has a flow capacity of 0.11 m^3, thus the contributing 5-year flow from the site will have a minimal impact on the pipe's capacity. As such, no further quantity controls are proposed for the flows towards Georgian Drive.

In the event of a storm greater than the 100 year storm, a 300 mm diameter overflow pipe complete with rodent grate is proposed at the top of the storage tank allowing flows to spill directly into the dispersion basin which will direct emergency flows as sheet flow down the forested slope. Flows from the small driveway area will surcharge through the catch basin and flow overland north down the forested slope.

Table 2 below summarizes post-development peak flows and demonstrates that the post-development flows for all storm events are acceptable compared to the pre-development peak flows in Table 1.

Table 2: Post Development Peak Flows

	2 Year (m^3/s)	5 Year (m^3/s)	10 Year (m^3/s)	25 Year (m^3/s)	50 Year (m^3/s)	100 Year (m^3/s)
Uncontrolled Flows to Georgian Drive (m^3/s)	**0.02**	**0.02**	**0.02**	**0.03**	**0.03**	**0.04**
Uncontrolled Flows to North (m^3/s)	0.01	0.01	0.01	0.01	0.01	0.01
Controlled Flows to North (m^3/s)	0.05	0.06	0.07	0.09	0.12	0.13
Total Flow North (m^3/s)	**0.06**	**0.07**	**0.08**	**0.10**	**0.13**	**0.14**

5.5. Stormwater Quality Control

The Ministry of the Environment, Conservation and Parks (MECP) in March 2003 issued a "Stormwater Management Planning and Design Manual". This manual has been adopted by a variety of agencies including the City of Barrie. The objective of our Stormwater Quality Control will be to ensure Enhanced Protection quality control as stated in the MECP manual. To achieve enhanced protection, permanent and temporary control of erosion and sediment transport are proposed and are discussed in the following sections.

5.5.1. Permanent Quality Control

The development's parking facilities pose a risk to stormwater quality through the collection of grit, sand and oils on the paved surface. The majority of the site consists of rooftop area which is considered generally clean. However, stormwater runoff from the small driveway area will drain to a catchbasin which will be conveyed to the internal concrete storage tank. As the internal tank includes a 300 mm deep sump which will be utilized for irrigation purposes, an oil/grit separator unit is proposed to treat the small paved area.

A CDS PMSU20_15_4m treatment unit is the proposed OGS to treat the storm water released from this site to the MECP's Enhanced Level Protection standard. This MECP standard stipulates a Total Suspended Solids (TSS) removal of at least 80%. The OGS unit will treat the post development flows to the required MECP quality standard, with a TSS removal rate of approximately 92%. The TSS removal rate calculations and OGS information can be seen in Appendix F.

5.5.2. During Construction Activities

During construction, earth grading, and excavation will create the potential for soil erosion and sedimentation. It is imperative that effective environmental and sedimentation controls are in place and maintained throughout the duration of construction activities to ensure stormwater runoff's quality.

Therefore, the following recommendations shall be implemented and maintained during construction to achieve acceptable stormwater runoff quality:

- Installation of silt fence along the entire perimeter of the site to reduce sediment migration onto surrounding properties.
- Installation of a construction entrance mat to minimize transportation of sediment onto roadways.
- Restoration of exposed surfaces with vegetative and non-vegetative material as soon as construction schedules permit, the duration of which is not to exceed 30 days.
- Reduce stormwater drainage velocities where possible.
- Minimize the amount of existing vegetation removed.

5.6. Flow Spreader

Following the internal quantity control storage tank, stormwater runoff is conveyed to a dispersion basin located immediately outside of the building which will reduce the energy and velocity of stormwater at the outlet of the storm sewer. The dispersion basin acts as a wide weir which promotes sheet flow, low velocity, and reduces the potential for erosion of the downstream bank.

The dispersion basin has a length of 40 m and will reduce the turbulence of pipe flow and flow velocity before spilling over the weir and towards Highway 400.

To prevent erosion and channelization, rip rap will be installed within the dispersion basin along the southern slope and at the inlet of the controlled flows. Additionally, the overflow weir will be equipped with CCG2 cable mats. The cable mats will be infilled and seeded to establish growth as well as reduce erosion.

5.7. Water Balance

Since the post development state will increase the imperviousness of the site, considerations were taken in regard to groundwater recharge. Golder and Associates completed a preliminary Water Balance Assessment for the site in November 2018. It was determined that the pre-development infiltration volume over the site is 4,280 m^3 and the post development unmitigated infiltration volume is 3,150 m^3. The Golder assessment can be seen in Appendix D.

Typically, in order to obtain water balance for the site, infiltration measures would be implemented such as infiltrating stormwater over the rooftop area in an underground infiltration gallery. However, the geotechnical engineer had slope stability concerns with infiltrating at the top of the hill. Therefore, the only suitable location for infiltration would be at the bottom of the bank which would have required significant tree removal for the installation of approximately 80 m of storm sewer.

Pearson attended a site visit with NVCA staff to discuss the infiltration concerns as well as determine if there were any suitable areas along the bank for a flow spreader. An area at the top of the bank with a gentler slope was found and therefore a flow spreader was designed in that location. However, it was agreed that for this development it would be better environmentally to protect the existing trees and not provide any additional infiltration mitigation measures.

5.8. Phosphorus

Local conservation authorities have determined the importance of reducing phosphorus levels in water courses in this area. Best efforts are to be employed in order to reduce phosphorus levels being contributed from the site.

The existing site is comprised of residential and undeveloped land with approximately 630 m^2 being asphalt driveways, 1,130 m^2 of building and concrete, and 5,830 m^2 of grassed or forested area. The proposed development will also be categorized as residential. The building and concrete walkway footprint will increase to 4,000 m^2 and the grassed and forested areas (including the green roof) will decrease to 3,590 m^2, however the asphalt driveway will be decreased to 200 m^2. The reduction of driveway area will improve the water quality as rooftop runoff is generally considered clean. The storage tank has 18 m^3 which is to be used for irrigation purposes which will further reduce phosphorous levels. Stormwater will then be conveyed over the dispersion basin and sheet flow through the forested area which will act as a vegetated buffer.

As the post development land use does not change, there is no increase in phosphorous leaving the site. While no removal efficiencies were calculated, the rooftop irrigation and vegetated slope are expected to provide inherent phosphorus removal for the site. The following Table 3 details the anticipated phosphorus loadings for the pre and post-development conditions.

Table 3: Phosphorus Loadings

	Total P (kg)
Pre-Development	1.00
Post Development	1.00

Detailed calculations can be found in Appendix E.

6. Conclusions

The proposed development will require the connection of sanitary and watermain services to the existing municipal services on Georgian Drive. Storm services for the development will be conveyed to a dispersion basin, ultimately outletting to the forested area north of the development.

The SWM design for this site considers the existing conditions and is contained within the site's boundaries. A concrete storage tank has been proposed for quantity control and an OGS unit has been implemented to treat the small, paved area to improve stormwater quality.

No infiltration for the site is proposed for water balance due to site constraints. Post development phosphorous loading for the site does not increase and will be inherently reduced by various stormwater management features on site.

All of which is respectfully submitted,
Pearson Engineering Ltd.

Taylor Arkell, P. Eng Senior Project Engineer	Gary Pearson, P.Eng. Principal

LICENSED PROFESSIONAL ENGINEER
G. M. PEARSON
100061986
09/08/21
PROVINCE OF ONTARIO

Appendix A

Water Servicing and Fire Flow Calculations

PEARSON ENGINEERING LTD.

290, 294, 298, & 302 Georgian Drive
Water Flow Calculations

Design Criteria

Demand per capita (Q):	225 L/cap/day	
Peak Rate Factor (Max. Hour	4.13	(Table 3-1: Peaking Factors, MOE Design Guidelines for Drinking-Water Systems)
Max. Day Factor	2.75	(Table 3-1: Peaking Factors, MOE Design Guidelines for Drinking-Water Systems)

Site Data

Description	Density		Units		Flow Rate		Peaking Factors*	
1 Bedroom/Studi	2.0	person/unit	179	Units	225	L/cap/d	MAX DAY FACTOR	2.75
2 Bedroom	4.0	person/unit	146	Units	225	L/cap/d	PEAK RATE FACTOR	4.13
3 Bedroom	6.0	person/unit	50	Units	225	L/cap/d	*From MOE Manual Table 3.1	
Commercial	350.0	m^2	1	Units	5	L/m^2 of floor space		
Town Houses	6.0	person/unit	5	Units	225	L/person		

Calculate Population

Pop.	=	2.0	x	179	+	4.0	x	146	+	6.0	x	50	+	6.0	x	5
Pop.	=	1272	people													

Calculate Average Day Demand

ADD	=	225	x	1272	+	350.0	x	5
ADD	=	287950	L/day					
ADD	=	3.33	L/s					

Calculate Max Day Flow

MDF	=	3.33	x	2.75
MDF	=	9.17	L/s	

Calculate Peak Hour Demand

PHD	=	3.33	x	3.75
PHD	=	12.50	L/s	

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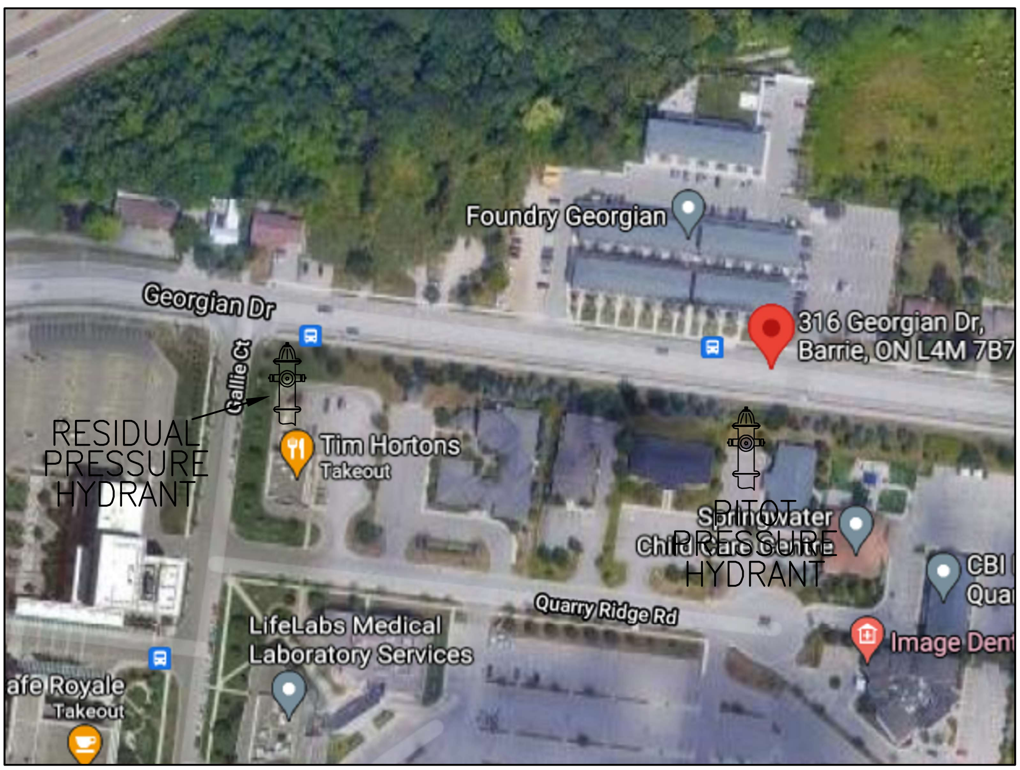
VIPOND INC
est. 1945

FLOW TEST RESULTS

DATE : AUG 17, 2021 TIME : 11:00 AM

LOCATION : 316 GEORGIAN DR, EAST
BARRIE
ONTARIO

TEST BY : VIPOND FIRE PROTECTION AND LOCAL PUC



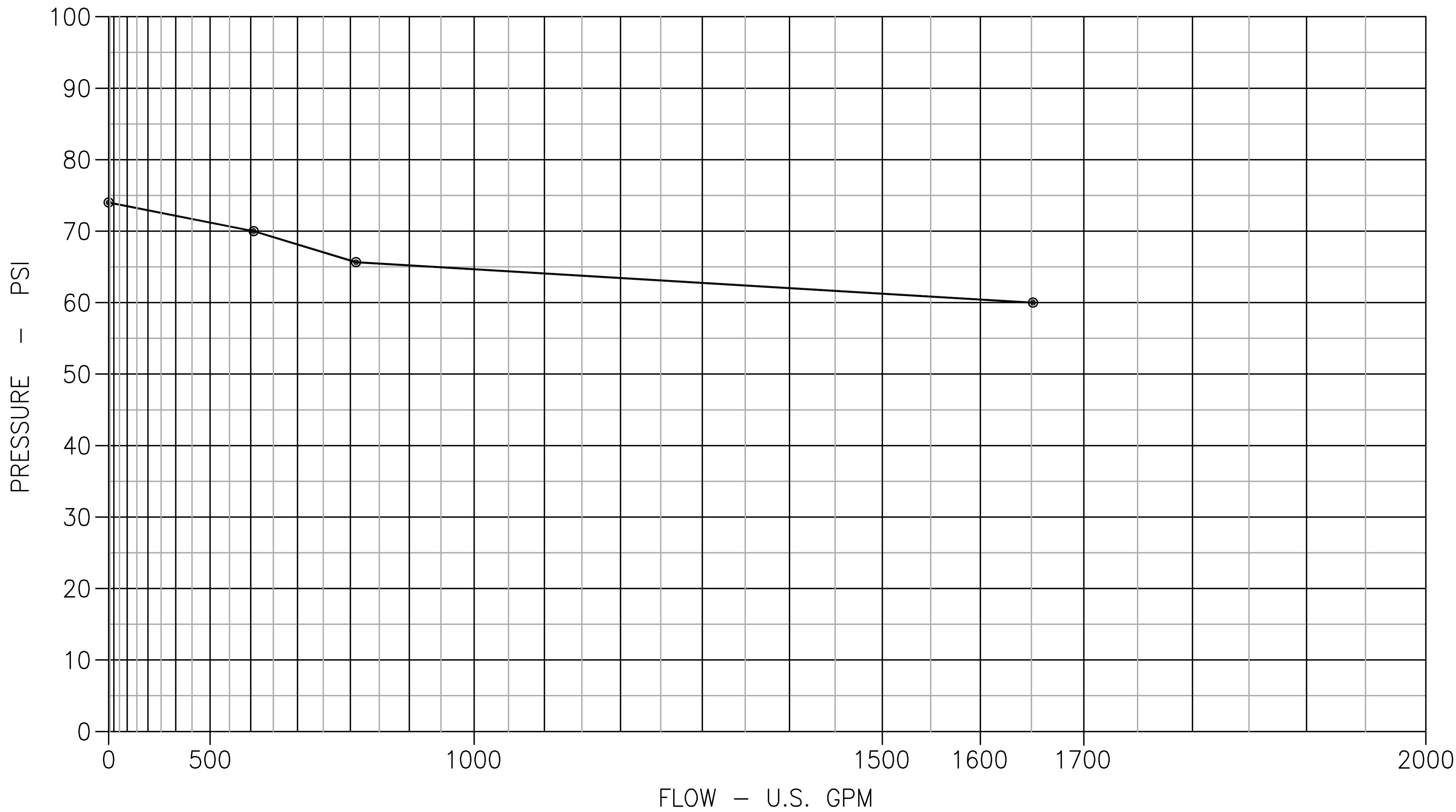
N

STATIC PRESSURE : 74 PSI

TEST NO.	NO. OF NOZZLES	NOZZLE DIAMETER (INCHES)	DISCHARGE CO-EFFICIENT	RESIDUAL PRESSURE (PSI)	PITOT PRESSURE (PSI)	DISCHARGE (U.S.GPM)
1	1	1-3/4	0.995	70	54	654
2	1	2-1/2	0.9	66	32	954
3	2	2-1/2	0.9	60	24	1390

316 GEORGIAN DR E	BY : LEN K./KRYSTIAN.K
BARRIE	OFFICE : BARRIE
ONTARIO	TEST BY : VIPOND & PUC
	DATE : APRIL 17, 2021

STATIC:		RESIDUAL:		FLOW:
74 PSI	TEST#1	70 PSI	@	617 GPM
	TEST#2	66 PSI	@	826 GPM
	TEST#3	60 PSI	@	1652 GPM



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BUILD SAFE AND SECURE COMMUNITIES

TMD-Atria Corp.
6 Carlaw Avenue, Suite 200
Toronto, ON M4M 2R5

Vipond Project # 3F5118
September 14, 2021

Attention: Hitesh Gajiwal

Project: The Georgian Apartments
290, 294, 298 & 302 Georgian Drive
Barrie, Ontario

Re: Fire Protection Water Demand

Single Building/Structure Site

19 Storey Residential Tower (15 residential levels, 4 parking levels above grade, two levels below grade with mechanical penthouse level)
3.2.2.42 – Group C Any Height, Any Area, Sprinklered.
High Building
Standpipe Required.
Closest hydrant – Proposed private hydrant shown on South east corner of property entrance from Georgian Drive. Existing Municipal hydrant across the street On Georgian Drive.

Fully sprinklered buildings define their required water supply minimum flow rates & durations (and therefore, minimum storage volumes) per OBC B 3.2.5.7 (1 &2) & 3.2.5.13 (1) as the combined demand of the sprinkler system and hose flow for the required durations as per NFPA 13.

Sprinkler system calculations for the applicable design criteria as per anticipated occupancies:

A - NFPA 13, Light Hazard residential dwelling units
B - NFPA 13, Light Hazard residential areas outside of dwelling units
C - NFPA 13, Ordinary Hazard Group 1 - Mechanical spaces, Parking Garage
D - NFPA 13, Ordinary Hazard Group 2 – Mercantile (Commercial) Tenant Spaces

A - Light Hazard residential dwelling units = 0.10 usgpm / 4 most demanding residential sprinklers
4 sprinklers heads at approximately 20 usgpm each = 80 usgpm
80 + Hose Allowance per NFPA 13.
NFPA 13 Hose allowance for “Light Hazard” Occupancy = 100 usgpm combined inside and outside.
80 + 100 = **180 usgpm Total system demand**

B - Light Hazard residential non dwelling units = 0.10 usgpm / 1500
Therefore: 0.10 usgpm over an area of 1500 sq ft +/- = 1500 usgpm
Plus, up to 25% for system overage/design variance = 150 + 25% = 188 usgpm
188 + Hose Allowance per NFPA 13.
NFPA 13 Hose allowance for “Light Hazard” Occupancy = 100 usgpm combined inside and outside.
188 + 100 = **288 usgpm Total system demand**

C - Ordinary Hazard Group 1 = 0.15 usgpm / 1500 sq ft
∴ 0.15 usgpm over an area of 1500 sq ft +/- = 225 usgpm
Plus, up to 25% for system overage/design variance = 225 + 25% = 281 usgpm
281 + Hose Allowance per NFPA 13.
NFPA 13 Hose allowance for “Ordinary Hazard” = 250 combined inside and outside.
281 250 = **531 usgpm Total system demand**

C- Alternate – If Parking level are protected with Dry Sprinkler system the design area increases 30% without revising the density.

Ordinary Hazard Group 1 = 0.15 usgpm / 1500 sq ft +30% for Dry system
∴ 0.15 usgpm over an area of 1950 sq ft +/- = 292.5 usgpm
Plus, up to 25% for system overage/design variance = 292.5 + 25% = 365 usgpm
365 + 250 usgpm Hose Allowance per NFPA 13.= **615 usgpm Total Dry system demand**

D - Ordinary Hazard Group 2 = 0.20 usgpm / 1500 sq ft
∴ 0.20 usgpm over an area of 1500 sq ft +/- = 300 usgpm
Plus, up to 25% for system overage/design variance = 300 + 25% = 375 usgpm
375 + Hose Allowance per NFPA 13.
NFPA 13 Hose allowance for “Ordinary Hazard” = 250 combined inside and outside.
375 + 250 = **625 usgpm Total system demand**

BLDG	Sprinkler or Standpipe	Area description for demand Caluclation	Combined Sprinkler Standpipe Demand or Standpipe Only Demand or OBC A.3.2.5.7		Highest Demand This Bldg.
1	Sprinkler	Light Hazard residential dwelling unit	180 usgpm	680 liters/min	
1	Sprinkler	Light Hazard areas other than dwelling unit	288 usgpm	1089 liters/min	
1	Sprinkler	Ordinary Hazard mechanical / parking wet system	531 usgpm	2007 liters/min	
1	Sprinkler	Ordinary Hazard mechanical / parking dry system	615 usgpm	2328 liters/min	
1	Sprinkler	Ordinary Hazard Commercial/Tanants	625 usgpm	2363 liters/min	<
1	Standpipe	Class III Standpipe	500 usgpm	1890 liters/min	

Note that for this building, no on-site water storage is required because the property is served by a reliable municipal water connection and therefore the only requirement of note is the minimum available flow rate.

Please note that these calculations may be superseded by the yet to be completed engineer reviewed sprinkler plans and associated hydraulic calculations that would be submitted formally, but I would not anticipate a significant variance.

Yours truly,
VIPOND FIRE PROTECTION, DIV. OF VIPOND INC.

Mike Powell, Service Department Manager

Cc – Russ Kerr, *GSC*, General Manager - Barrie

Appendix B

Sanitary Servicing Calculations

PEARSON ENGINEERING LTD.

290, 294, 298, & 302 Georgian Drive
Sanitary Flow Calculations

Design Criteria

Flow per capita (Q):	225 L/cap/day
Peak Flow	Qp = P * Q * M / 86400
Peaking Factor (Harmon Formula)	M = 1 + (14 / (4 + (P / 1(2 <= "M" <= 4

Site Data

Description	Density		Units		Flow Rate	
1 Bedroom/Studio	2.0	person/unit	179	Units	225	L/cap/d
2 Bedroom	4.0	person/unit	146	Units	225	L/cap/d
3 Bedroom	6.0	person/unit	50	Units	225	L/cap/d
Town Houses	6.0	person/unit	5	Units	255	L/person
Commercial	350.0	m^2	1	Units	5	L/m^2 of floor space

Equivilant Population for Commercial = 350.0 m^2 x 5 L/m^2 of floor space

$$= \frac{1750.0 \text{ L/day}}{225.0 \text{ L/cap/d}}$$

= 8 cap

<u>Calculate Population</u>

Pop. = 2.0 x 179 + 4.0 x 146 + 6.0 x 50 + 6.0 x 5

Pop. = 1272 people

Equivalent Population Including Commerc = 1280 people

<u>Calculate Average Daily Flows</u>

$$ADF = \frac{225 \times 1272 + 350.0 \times 5}{86400}$$

ADF = 3.33 L/s

<u>Calculate Peaking Factor</u>

$$M = 1 + \frac{14}{4 + \left(\frac{1272}{1,000}\right)^{0.5}}$$

M = 3.73

<u>Calculate Peak Flow</u>

Qp = 3.33 x 3.73

= 12.43 L/s

Sanitary Pipe Capacity - 200mm @ 1.0% = 33 L/s n=0.013

M=1+(14/(4+(P/1000)^0.5))   (2<="M"<=4)

Qp=P*Q*M/86400   (Q=225 l/day/person)

Qi=L/1000*I*DIAMETER/86400   (I=180 l/day/mm(dia.)/km)

(includes peaking factor)

Qtot=Qp+Qi

CITY OF BARRIE

SANITARY SEWER DESIGN

DATE: 3-Oct-19

FILE: 18037

CONTRACT/PROJECT: DUNSMORE

STREETS	MANHOLE FROM	MANHOLE TO	DWELLING UNITS	DENSITY P.P.U	POP. (P)	POP. (ACC.)	M	Qp (l/s)	LENGTH (m)	LENGTH (ACC.) (m)	Qi (l/s)	TOTAL Q (l/s)	D (mm)	S (%)	Q FULL (l/s)	V FULL (m/s)	d/D < 0.5 for D < 375 mm / < 0.7 for D > 375 mm	PERCENT FULL (%)
	SAL08203	SAL08204	2	3.50	7.00	7.00	4.00	0.07	40.50	40.50	0.02	0.09	250	1.31	68.08	1.39	YES	0.14
GEORGIAN DRIVE	SAL08204	SAL08205	4	3.50	14.00	21.00	4.00	0.22	101.40	141.90	0.07	0.29	250	0.37	36.18	0.74	YES	0.81
	SAL08205	SAL08206	1280	1.00	1280.00	1301.00	3.72	12.61	110.10	252.00	0.13	12.75	250	0.36	35.69	0.73	YES	35.72
	SAL08206	SAL08207	374	1.00	374.00	1675.00	3.64	15.90	109.10	361.10	0.19	16.08	250	0.39	37.14	0.76	YES	43.30
	SAL08207	SAL08208	7	3.50	24.50	1699.50	3.64	16.11	109.30	470.40	0.25	16.35	250	0.48	41.21	0.84	YES	39.69
			ADD 2.0Ha EXTERNAL AREA @ 84cu.m/day/Ha = 1.94 L/s															
JOHNSON STREET	SAL08208	SAL08209	4	3.50	14.00	1713.50	3.64	16.23	50.10	520.50	0.27	18.44	250	2.42	92.53	1.88	YES	19.93
	SAL08209	SAL08210	2	3.50	7.00	1720.50	3.64	16.29	38.80	559.30	0.29	18.52	250	2.55	94.98	1.93	YES	19.50
DUNSMORE LANE	SAL08219	SAL08220	6	3.50	21.00	21.00	4.00	0.22	53.00	53.00	0.03	0.25	250	0.98	58.88	1.20	YES	0.42
	SAL08220	SAL08221	13	3.50	45.50	66.50	4.00	0.69	67.10	120.10	0.06	0.76	250	0.97	58.58	1.19	YES	1.29
	SAL08221	SAL08222	8	3.50	28.00	94.50	4.00	0.98	68.30	188.40	0.10	1.08	250	0.81	53.53	1.09	YES	2.02
	SAL08222	SAL08223	9	3.50	31.50	126.00	4.00	1.31	53.70	242.10	0.13	1.44	250	0.65	47.95	0.98	YES	3.00
	SAL08223	SAL08210	0	3.50	0.00	126.00	4.00	1.31	24.00	266.10	0.14	1.45	250	0.63	47.21	0.96	YES	3.07
JOHNSON STREET	SAL08210	SAL08211	5	3.50	17.50	1864.00	3.61	17.52	48.00	873.40	0.45	19.92	250	0.54	43.71	0.89	YES	45.56
	SAL08211	SAL08212	12	3.50	42.00	1906.00	3.60	17.88	93.80	967.20	0.50	20.32	250	0.77	52.19	1.06	YES	38.94
	SAL08212	SAL08153	6	3.50	21.00	1927.00	3.60	18.06	90.30	1057.50	0.55	20.55	250	0.60	46.07	0.94	YES	44.60
DUNSMORE LANE	SAL08219	SAL08217	22	3.20	70.40	70.40	4.00	0.73	104.00	104.00	0.05	0.79	250	1.05	60.95	1.24	YES	1.29
			ADD 3.43 L/S FROM BUSINESS PARK (10A TO 14A)															
DUNSMORE LANE	SAL08217	SAL08218	19	3.20	60.80	131.20	4.00	1.37	83.40	187.40	0.10	4.89	250	0.70	49.76	1.01	YES	9.84
	SAL08218	SAL08156	58	3.20	185.60	316.80	4.00	3.30	84.30	271.70	0.14	6.87	250	0.70	49.76	1.01	YES	13.81
			ADD 30.0 L/S FROM HOSPITAL (SAL081157 TO SAL08156)															
DUNSMORE LANE	SAL08156	SAL08155	37	3.20	118.40	435.20	4.00	4.53	109.80	381.50	0.30	38.26	375	0.23	84.10	0.76	YES	45.50
	SAL08155	SAL08154	21	3.20	67.20	502.40	3.97	5.20	105.30	486.80	0.38	39.01	375	0.23	84.10	0.76	YES	46.38
	SAL08154	SAL08153	17	3.50	59.50	561.90	3.95	5.78	117.30	604.10	0.47	39.68	375	0.25	87.68	0.79	YES	45.25
AMBLER BAY	SAL08246	SAL08254	12	3.50	42.00	42.00	4.00	0.44	91.60	91.60	0.05	0.49	250	2.69	97.55	1.99	YES	0.50
	SAL08254	SAL08255	12	3.50	42.00	84.00	4.00	0.88	90.60	182.20	0.09	0.97	250	2.81	99.70	2.03	YES	0.97
	SAL08255	SAL08257	0	3.50	0.00	84.00	4.00	0.88	22.50	204.70	0.11	0.98	250	0.67	48.69	0.99	YES	2.02
DUNSMORE LANE	SAL08253	SAL08256	10	3.20	32.00	32.00	4.00	0.33	43.00	43.00	0.02	0.36	250	0.70	49.76	1.01	YES	0.71
	SAL08256	SAL08257	10	3.20	32.00	64.00	4.00	0.67	40.10	83.10	0.04	0.71	250	0.65	47.95	0.98	YES	1.48
DUNSMORE LANE	SAL08257	SAL08258	9	3.50	31.50	179.50	4.00	1.87	69.10	356.90	0.19	2.06	250	0.48	41.21	0.84	YES	4.99
	SAL08258	SAL08153	2	3.50	7.00	186.50	4.00	1.94	27.90	384.80	0.20	2.14	250	0.29	32.03	0.65	YES	6.69
JOHNSON STREET	SAL08153	SAL08152	4	3.50	14.00	2689.40	3.48	24.39	101.60	2148.00	1.68	61.44	375	0.22	82.25	0.74	YES	74.70

NOTE:

1. Sanitary Design Sheet information from City of Barrie for the Dunsmore Subdivision, updated to include flows from the adjacent Podium site and flows from the proposed development.
2. Highlighted rows indicate flow path of sanitary flows from the proposed development.

Appendix C

Stormwater Management Calculations

PEARSON
ENGINEERING LTD.

290, 294, 298, & 302 Georgian Drive
Calculation of Runoff Coefficients

Runoff Coefficient	=	0.15	0.10	0.95	0.95	0.52	0.60	0.95	Weighted
Surface Cover	=	Grass	Forest	Asphalt	Building	*Green Roof	Gravel	Concrete	Runoff Coefficient
PRE DEVELOPMENT	Total Area (m^2)	Area (m^2)	Area (m^2)	Area (m^2)	Area (m^2)	Area (m^2)	Area (m^2)	Area (m^2)	
1	7590	4520	1306	629	912	0	0	222	0.33
Pre Total	7590	4520	1306	629	912	0	0	222	0.33
POST DEVELOPMENT	Total Area (m^2)	Area (m^2)	Area (m^2)	Area (m^2)	Area (m^2)	Area (m^2)	Area (m^2)	Area (m^2)	
1	4278	50	0	199	2993	895	0	142	0.85
2	2196	1596	600	0	0	0	0	0	0.14
3	1116	451	0	0	0	0	0	665	0.63
Post Total	7590	2097	600	199	2993	895	0	807	0.61

*Assuming Green Roof would reduce runoff by approximately 45% (TRCA and CVC, 2010)

PEARSON ENGINEERING LTD.

290, 294, 298, & 302 Georgian Drive
Pre-Development Peak Flows

Storm (yrs)	City of Barrie Coeff A	Coeff B	Coeff C
2	678.085	4.699	0.781
5	853.608	4.699	0.766
10	975.865	4.699	0.760
25	1146.275	4.922	0.757
50	1236.152	4.699	0.751
100	1426.408	5.273	0.759

Modified Rational Method
$Q = C_i CIA / 360$

Where:
- Q - Flow Rate (m^3/s)
- Ci - Peaking Coefficient
- C - Rational Method Runoff Coefficient
- I - Storm Intensity (mm/hr)
- A - Area (ha.)

Area Number	Flow to North (Area 1)		Allowable to Georgian Drive	
Area	0.76	ha	0.05	ha
Runoff Coefficient	0.33		0.51	
Time of Concentration	10	min	10	min
Return Rate	2	year	2	year
Peaking Coefficient (C_i)	1.0		1.0	
Rainfall Intensity	83.1	mm/hr	83.1	mm/hr
Pre-Development Peak Flow	0.06	m^3/s	0.01	m^3/s
Return Rate	5	year	5	year
Peaking Coefficient (C_i)	1.0		1.0	
Rainfall Intensity	108.9	mm/hr	108.9	mm/hr
Pre-Development Peak Flow	0.08	m^3/s	0.01	m^3/s
Return Rate	10	year	10	year
Peaking Coefficient (C_i)	1.0		1.0	
Rainfall Intensity	126.5	mm/hr	126.5	mm/hr
Pre-Development Peak Flow	0.09	m^3/s	0.01	m^3/s
Return Rate	25	year	25	year
Peaking Coefficient (C_i)	1.1		1.1	
Rainfall Intensity	148.2	mm/hr	148.2	mm/hr
Pre-Development Peak Flow	0.11	m^3/s	0.01	m^3/s
Return Rate	50	year	50	year
Peaking Coefficient (C_i)	1.2		1.2	
Rainfall Intensity	164.2	mm/hr	164.2	mm/hr
Pre-Development Peak Flow	0.14	m^3/s	0.01	m^3/s
Return Rate	100	year	100	year
Peaking Coefficient (C_i)	1.25		1.25	
Rainfall Intensity	180.2	mm/hr	180.2	mm/hr
Pre-Development Peak Flow	0.16	m^3/s	0.02	m^3/s

290, 294, 298, & 302 Georgian Drive
Post-Development Peak Flows

Storm (yrs)	City of Barrie Coeff A	Coeff B	Coeff C
	678.085	4.699	0.781
5	853.608	4.699	0.766
10	975.865	4.699	0.760
25	1146.275	4.922	0.757
50	1236.152	4.699	0.751
100	1426.408	5.273	0.759

Modified Rational Method
$Q = C_iCIA / 360$

Where:

Q -	Flow Rate (m^3/s)
Ci -	Peaking Coefficient
C -	Rational Method Runoff Coefficient
I -	Storm Intensity (mm/hr)
A -	Area (ha.)

	Controlled Area (Building)		Uncontrolled Area (To North)		Uncontrolled Area (To Georgian Drive)	
Area Number	1		2		3	
Area	0.43	ha	0.22	ha	0.11	ha
Runoff Coefficient	0.85		0.14		0.63	
Time of Concentration	10	min	10	min	10	min
Return Rate	2	year	2	year	2	year
Peaking Coefficient (C_i)	1.00		1.00		1.00	
Rainfall Intensity	83.1		83.1		83.1	
Post-Development Peak Flow	0.08	m^3/s	0.01	m^3/s	0.02	m^3/s
Return Rate	5	year	5	year	5	year
Peaking Coefficient (C_i)	1.00		1.00		1.00	
Rainfall Intensity	108.9		108.9		108.9	
Post-Development Peak Flow	0.11	m^3/s	0.01	m^3/s	0.02	m^3/s
Return Rate	10	year	10	year	10	year
Peaking Coefficient (C_i)	1.00		1.00		1.00	
Rainfall Intensity	126.5		126.5		126.5	
Post-Development Peak Flow	0.13	m^3/s	0.01	m^3/s	0.02	m^3/s
Return Rate	25	year	25	year	25	year
Peaking Coefficient (C_i)	1.10		1.10		1.10	
Rainfall Intensity	148.2		148.2		148.2	
Post-Development Peak Flow	0.15	m^3/s	0.01	m^3/s	0.03	m^3/s
Return Rate	50	year	50	year	50	year
Peaking Coefficient (C_i)	1.20		1.20		1.20	
Rainfall Intensity	164.2		164.2		164.2	
Post-Development Peak Flow	0.17	m^3/s	0.01	m^3/s	0.03	m^3/s
Return Rate	100	year	100	year	100	year
Peaking Coefficient (C_i)	1.25		1.25		1.25	
Rainfall Intensity	180.2		180.2		180.2	
Post-Development Peak Flow	0.18	m^3/s	0.01	m^3/s	0.04	m^3/s

PEARSON ENGINEERING LTD.

290, 294, 298, & 302 Georgian Drive
Stage-Storage-Discharge Table

Elevation (m)	Area (m^2)	Volume (m^3)	Cum. Vol. (m^3)	Orifice Tube 1 Head (m)	Orifice Tube 1 Flow (m^3/s)	Orifice Tube 2 Head (m)	Orifice Tube 2 Flow (m^3/s)	Weir Head (m)	Weir Flow (m^3/s)	Total Flow (m^3/s)
256.70	36	0	0	0.00	0.000	0.00	0.000	0.00	0.000	0.000
256.80	36	4	4	0.03	0.010	0.00	0.000	0.00	0.000	0.010
256.90	36	4	7	0.13	0.022	0.00	0.000	0.00	0.000	0.022
257.00	36	4	11	0.23	0.030	0.00	0.000	0.00	0.000	0.030
257.10	36	4	14	0.33	0.036	0.00	0.000	0.00	0.000	0.036
257.20	36	4	18	0.43	0.041	0.00	0.000	0.00	0.000	0.041
257.30	36	4	22	0.53	0.045	0.00	0.000	0.03	0.000	0.045
257.40	36	4	25	0.63	0.050	0.00	0.000	0.13	0.000	0.050
257.50	36	4	29	0.73	0.053	0.00	0.000	0.23	0.000	0.053
257.60	36	4	32	0.83	0.057	0.00	0.000	0.33	0.000	0.057
257.70	36	4	36	0.93	0.060	0.00	0.000	0.43	0.000	0.060
257.80	36	4	40	1.03	0.063	0.00	0.000	0.53	0.000	0.063
257.90	36	4	43	1.13	0.066	0.00	0.000	0.63	0.000	0.066
258.00	36	4	47	1.23	0.069	0.00	0.000	0.73	0.000	0.069
258.10	36	4	50	1.33	0.072	0.10	0.035	0.83	0.000	0.107
258.20	36	4	54	1.43	0.075	0.20	0.050	0.93	0.000	0.125
258.30	36	4	58	1.52	0.077	0.30	0.061	1.03	0.000	0.138
258.40	36	4	61	1.62	0.080	0.40	0.070	1.13	0.000	0.150
258.50	36	4	65	1.72	0.082	0.50	0.079	1.23	0.000	0.161
258.60	36	4	68	1.82	0.085	0.60	0.086	1.33	0.000	0.171
258.70	36	4	72	1.92	0.087	0.70	0.093	1.43	0.000	0.180

Orifice Tube 1	
Diameter	150 mm
Invert Elevation	256.70
Orifice Constant	0.80
Orifice Centroid	256.78
Orifice Flow Formula	$0.80\pi(D/2000)^2 x(2x9.81xH)^{0.5}$

Orifice Tube 2	
Diameter	200 mm
Invert Elevation	257.90
Orifice Constant	0.80
Orifice Centroid	258.00
Orifice Flow Formula	$0.80\pi(D/2000)^2 x(2x9.81xH)^{0.5}$

PEARSON ENGINEERING LTD.

290, 294, 298, & 302 Georgian Drive
Quantity Control Volume Calculations

DATE:	3-Oct-19
FILE:	18037
CONTRACT/PROJECT:	Georgian Drive-Atria
COMPLETED BY:	MKRW

Modified Rational Method Parameters

Pre Development Area (ha)	Post Development Area (ha)	Time of Concentration (min)	Time Increments (min)	Pre Development Runoff Coefficient	Post Development Runoff Coefficient
0.76	0.43	10	1	0.33	0.85

Note: Refer to page Calculation of Runoff Coefficients for detailed calculations of Modified Rational Method parameters.

Pre-Development Runoff Rate

	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
C	0.33	0.33	0.33	0.33	0.33	0.33
C_i	1.00	1.00	1.00	1.10	1.20	1.25
I	83.11	108.92	126.55	148.15	164.22	180.15
A	0.76	0.76	0.76	0.76	0.76	0.76
Q	0.06	0.08	0.09	0.11	0.14	0.16

Note: Q= 0.00278CC,IA

Rainfall Station	Barrie

SWM Pond Design Input

Storm (yrs)	Chicago Storm Coefficient A	Chicago Storm Coefficient B	Chicago Storm Coefficient C	Allowable Outflow (m3/s)	Post Development Runoff Coefficient
2	678.085	4.699	0.781	0.05	0.85
5	853.608	4.699	0.766	0.06	0.85
10	975.865	4.699	0.760	0.07	0.85
25	1146.275	4.922	0.757	0.09	0.94
50	1236.152	4.699	0.751	0.12	1
100	1426.408	5.273	0.759	0.13	1

Results

Storm Event	Storage m^3	Time min
2	24	17
5	35	19
10	44	20
25	49	18
50	53	17
100	56	17

Note: Storage volume calculated as per Hydrology Handbook, Second Edition, American Society of Civil Engineers, 1996

Time (min)	2 Year Intensity mm/hr	2 Year Inflow m^3/s	2 Year Outflow m^3/s	2 Year Storage m^3	Difference	5 Year Intensity mm/hr	5 Year Inflow m^3/s	5 Year Outflow m^3/s	5 Year Storage m^3	Difference
1	174.18	0.18	0.05	-5	7	225.07	0.23	0.06	-6	9
2	153.52	0.16	0.05	1	5	198.85	0.20	0.06	3	7
3	137.71	0.14	0.05	6	4	178.75	0.18	0.06	9	5
4	125.19	0.13	0.05	10	3	162.79	0.16	0.06	14	4
5	114.99	0.12	0.05	13	2	149.77	0.15	0.06	19	3
6	106.50	0.11	0.05	16	2	138.93	0.14	0.06	22	3
7	99.33	0.10	0.05	18	2	129.73	0.13	0.06	25	2
8	93.16	0.09	0.05	19	1	121.83	0.12	0.06	27	2
9	87.81	0.09	0.05	20	1	114.96	0.12	0.06	29	2
10	83.11	0.08	0.05	21	1	108.92	0.11	0.06	30	1
11	78.94	0.08	0.05	22	1	103.57	0.10	0.06	32	1
12	75.23	0.08	0.05	23	0	98.78	0.10	0.06	33	1
13	71.88	0.07	0.05	23	0	94.48	0.10	0.06	33	1
14	68.86	0.07	0.05	24	0	90.58	0.09	0.06	34	0
15	66.12	0.07	0.05	24	0	87.04	0.09	0.06	35	0
16	63.61	0.06	0.05	24	0	83.80	0.08	0.06	35	0
17	61.31	0.06	0.05	24	0	80.82	0.08	0.06	35	0
18	59.19	0.06	0.05	24	0	78.08	0.08	0.06	35	0
19	57.23	0.06	0.05	24	0	75.55	0.08	0.06	35	0
20	55.41	0.06	0.05	24	0	73.19	0.07	0.06	35	0
21	53.72	0.05	0.05	24	0	71.00	0.07	0.06	35	0
22	52.14	0.05	0.05	23	0	68.95	0.07	0.06	35	0
23	50.67	0.05	0.05	23	0	67.04	0.07	0.06	35	0
24	49.28	0.05	0.05	23	0	65.24	0.07	0.06	34	0
25	47.98	0.05	0.05	22	-22	63.55	0.06	0.06	34	0
26	46.76	0.05	0.00	0	0	61.96	0.06	0.06	33	0
27	45.60	0.05	0.00	0	0	60.46	0.06	0.06	33	-1
28	44.51	0.05	0.00	0	0	59.04	0.06	0.06	32	-32
29	43.47	0.04	0.00	0	0	57.69	0.06	0.00	0	0
30	42.49	0.04	0.00	0	0	56.41	0.06	0.00	0	0
31	41.56	0.04	0.00	0	0	55.20	0.06	0.00	0	0
32	40.67	0.04	0.00	0	0	54.04	0.05	0.00	0	0
33	39.83	0.04	0.00	0	0	52.94	0.05	0.00	0	0
34	39.02	0.04	0.00	0	0	51.89	0.05	0.00	0	0
35	38.25	0.04	0.00	0	0	50.89	0.05	0.00	0	0
36	37.51	0.04	0.00	0	0	49.92	0.05	0.00	0	0
37	36.81	0.04	0.00	0	0	49.01	0.05	0.00	0	0

Time (min)	10 Year Intensity mm/hr	10 Year Inflow m^3/s	10 Year Outflow m^3/s	10 Year Storage m^3	Difference	25 Year Intensity mm/hr	25 Year Inflow m^3/s	25 Year Outflow m^3/s	25 Year Storage m^3	Difference
1	260.01	0.26	0.07	-6	10	298.22	0.33	0.09	-11	13
2	229.94	0.23	0.07	4	8	264.99	0.29	0.09	1	10
3	206.87	0.21	0.07	12	6	239.26	0.27	0.09	11	8
4	188.54	0.19	0.07	18	5	218.67	0.24	0.09	19	6
5	173.57	0.18	0.07	23	4	201.77	0.22	0.09	25	5
6	161.10	0.16	0.07	27	3	187.63	0.21	0.09	30	4
7	150.52	0.15	0.07	30	3	175.59	0.20	0.09	34	3
8	141.42	0.14	0.07	33	2	165.20	0.18	0.09	37	3
9	133.51	0.13	0.07	35	2	156.14	0.17	0.09	40	2
10	126.55	0.13	0.07	37	2	148.15	0.16	0.09	42	2
11	120.37	0.12	0.07	38	1	141.05	0.16	0.09	44	1
12	114.85	0.12	0.07	40	1	134.69	0.15	0.09	45	1
13	109.89	0.11	0.07	41	1	128.97	0.14	0.09	47	1
14	105.39	0.11	0.07	42	1	123.77	0.14	0.09	48	1
15	101.30	0.10	0.07	42	1	119.04	0.13	0.09	48	0
16	97.56	0.10	0.07	43	0	114.71	0.13	0.09	49	0
17	94.12	0.10	0.07	43	0	110.72	0.12	0.09	49	0
18	90.95	0.09	0.07	43	0	107.05	0.12	0.09	49	0
19	88.02	0.09	0.07	43	0	103.64	0.12	0.09	49	0
20	85.30	0.09	0.07	44	0	100.48	0.11	0.09	49	0
21	82.77	0.08	0.07	43	0	97.53	0.11	0.09	49	0
22	80.40	0.08	0.07	43	0	94.78	0.11	0.09	48	0
23	78.18	0.08	0.07	43	0	92.19	0.10	0.09	48	-1
24	76.10	0.08	0.07	43	0	89.77	0.10	0.09	47	-1
25	74.15	0.07	0.07	42	0	87.49	0.10	0.09	47	-1
26	72.31	0.07	0.07	42	0	85.34	0.09	0.09	46	-46
27	70.57	0.07	0.07	42	-1	83.31	0.09	0.00	0	0
28	68.92	0.07	0.07	41	-1	81.39	0.09	0.00	0	0
29	67.36	0.07	0.07	41	-41	79.56	0.09	0.00	0	0
30	65.88	0.07	0.00	0	0	77.83	0.09	0.00	0	0
31	64.47	0.07	0.00	0	0	76.19	0.08	0.00	0	0
32	63.13	0.06	0.00	0	0	74.62	0.08	0.00	0	0
33	61.86	0.06	0.00	0	0	73.13	0.08	0.00	0	0
34	60.64	0.06	0.00	0	0	71.70	0.08	0.00	0	0
35	59.47	0.06	0.00	0	0	70.33	0.08	0.00	0	0
36	58.36	0.06	0.00	0	0	69.03	0.08	0.00	0	0
37	57.29	0.06	0.00	0	0	67.78	0.08	0.00	0	0

Time (min)	50 Year Intensity mm/hr	50 Year Inflow m^3/s	50 Year Outflow m^3/s	50 Year Storage m^3	Difference	100 Year Intensity mm/hr	100 Year Inflow m^3/s	100 Year Outflow m^3/s	100 Year Storage m^3	Difference
1	334.55	0.40	0.12	-15	15	353.96	0.42	0.13	-19	16
2	296.30	0.35	0.12	0	11	316.38	0.38	0.13	-3	12
3	266.91	0.32	0.12	11	9	286.90	0.34	0.13	9	10
4	243.52	0.29	0.12	20	7	263.10	0.31	0.13	19	8
5	224.41	0.27	0.12	27	6	243.43	0.29	0.13	27	6
6	208.47	0.25	0.12	32	5	226.85	0.27	0.13	33	5
7	194.94	0.23	0.12	37	4	212.68	0.25	0.13	38	4
8	183.29	0.22	0.12	41	3	200.41	0.24	0.13	42	3
9	173.15	0.21	0.12	44	2	189.66	0.23	0.13	46	3
10	164.22	0.20	0.12	46	2	180.15	0.21	0.13	49	2
11	156.30	0.19	0.12	48	2	171.69	0.20	0.13	51	2
12	149.22	0.18	0.12	50	1	164.09	0.20	0.13	53	1
13	142.84	0.17	0.12	51	1	157.23	0.19	0.13	54	1
14	137.07	0.16	0.12	52	1	151.00	0.18	0.13	55	1
15	131.81	0.16	0.12	52	0	145.31	0.17	0.13	56	0
16	127.00	0.15	0.12	53	0	140.09	0.17	0.13	56	0
17	122.58	0.15	0.12	53	0	135.29	0.16	0.13	56	0
18	118.50	0.14	0.12	53	0	130.86	0.16	0.13	56	0
19	114.72	0.14	0.12	52	0	126.75	0.15	0.13	56	0
20	111.22	0.13	0.12	52	-1	122.92	0.15	0.13	56	-1
21	107.95	0.13	0.12	52	-1	119.35	0.14	0.13	55	-1
22	104.90	0.12	0.12	51	-1	116.02	0.14	0.13	54	-1
23	102.04	0.12	0.12	50	-50	112.89	0.13	0.13	53	-53
24	99.36	0.12	0.00	0	0	109.95	0.13	0.00	0	0
25	96.84	0.12	0.00	0	0	107.18	0.13	0.00	0	0
26	94.46	0.11	0.00	0	0	104.57	0.12	0.00	0	0
27	92.21	0.11	0.00	0	0	102.10	0.12	0.00	0	0
28	90.09	0.11	0.00	0	0	99.76	0.12	0.00	0	0
29	88.07	0.10	0.00	0	0	97.55	0.12	0.00	0	0
30	86.16	0.10	0.00	0	0	95.44	0.11	0.00	0	0
31	84.34	0.10	0.00	0	0	93.44	0.11	0.00	0	0
32	82.61	0.10	0.00	0	0	91.53	0.11	0.00	0	0
33	80.96	0.10	0.00	0	0	89.71	0.11	0.00	0	0
34	79.38	0.09	0.00	0	0	87.97	0.10	0.00	0	0
35	77.87	0.09	0.00	0	0	86.31	0.10	0.00	0	0
36	76.43	0.09	0.00	0	0	84.71	0.10	0.00	0	0
37	75.05	0.09	0.00	0	0	83.19	0.10	0.00	0	0

Maximum Storage Volume

Appendix D

Water Balance Calculations

TECHNICAL MEMORANDUM

DATE March 29, 2019 **Project No.** 18108181

TO Mr. Hitesh Gajiwala
TMD – Atria Corp.

CC Rafael Abdulla, PEng

FROM David Hinton, PEng
Gerard Van Arkel, PEng

EMAIL: David_Hinton@Golder.com

WATER BALANCE ASSESSMENT FOR PROPOSED DEVELOPMENT AT 290, 294, 298, 302 GEORGIAN DRIVE, BARRIE, ONTARIO

1.0 INTRODUCTION

TMD – Atria Corp. retained Golder Associates Ltd. (Golder) to provide a conceptual water balance for the proposed development at 290, 294, 298, 302 Georgian Drive, Barrie, Ontario (the Site).

As shown in the attached site plan, the Site has an area of 17,280 m^2 and is proposed to be developed with a 19-storey apartment building and above ground parking garage. The Site is bordered by Georgian Drive to the south, low density residential properties to the west, Highway 400 to the north, and three-storey townhouses to the east.

A water balance analysis was carried out for both existing and proposed conditions. The existing and proposed conditions were based on the proposed Site Plan provided by TMD – Atria Corp. (Attached), including the existing features. The purpose of the water balance assessment was to:

- Compare average monthly and average annual surplus (runoff + infiltration), under existing and proposed conditions, to estimate the difference in surplus;
- Recommend suitable Low Impact Development (LID) features which could assist in maintaining existing infiltration conditions under post-development conditions, based on the water balance analysis and data review; and,
- Complete a mitigated water balance analysis including the proposed LID features to estimate the surplus with mitigation options, which is compared to the existing conditions and the target annual infiltration.

2.0 METHODOLOGY

The water balance assessment was based on meteorological data, land use data and soil types observed during recent geotechnical investigations. Meteorological data was obtained from Environment and Climate Change Canada (ECCC) Shanty Bay Station, which is approximately 6 km east of the Site (Climate ID 6117684). The data for this composite was collected for the time period between for 1973 to 2015.

Water balance calculations were based on the following equation:

$$P = S + ET + R + I$$

Where: P = precipitation;

S = change in soil water storage;

ET = evapotranspiration;

R = surface runoff; and

I = infiltration (groundwater recharge).

Precipitation data obtained from ECCC for the Shanty Bay station indicates a mean annual precipitation (P) of 974 mm/yr.

Short-term or seasonal changes in soil water storage (S) occur as demonstrated by the dry conditions in the summer months and the wet or flooded conditions in the winter and spring. Long-term changes (e.g., year to year) in soil water storage are considered to be negligible.

Evapotranspiration (ET) refers to water lost to the atmosphere from vegetated surfaces. The term combines evaporation (i.e., water lost from soil or water surfaces) and transpiration (i.e., water lost from plants and trees) because of the difficulties in measuring these two processes separately. Potential ET refers to the calculated maximum potential loss of water from a vegetated surface to the atmosphere under conditions of an unlimited water supply. In this analysis, that potential ET is calculated following the Thornthwaite methodology using daily temperatures. The actual ET is a measure of ET that actually occurs, and is typically less than the potential ET rate as dry conditions (e.g., during the summer months when there is a moisture deficit) means that the potential ET for some months sometimes exceeds the amount of water available in the soil to evaporate. The mean annual potential ET for Shanty Bay is 607 mm/yr based on data provided by ECCC.

Annual water surplus is the difference between P and the actual ET. The water surplus represents the total amount of water available for either surface runoff (R) or groundwater infiltration (I) on an annual basis. On a monthly basis, surplus water remains after actual evapotranspiration has been removed from the sum of rainfall and snow melt, and maximum soil or snow pack storage is exceeded. Maximum soil storage is quantified using a water holding capacity (WHC) specific to the soil type and land use.

Infiltration rates and WHC were estimated using the MOE Stormwater Management Planning and Design (SWM) Manual (2003). There are three main factors that determine the percent infiltration of the total surplus: topography, soil type and ground cover. The sum of the fractions representing the three characteristics establishes the approximate annual percentage of surplus which can be infiltrated in an area with a sufficient downward groundwater gradient.

2.1 Site Water Balance Areas

Land use within the Site under existing conditions was identified using the provided Site Plan (Attachment 1) and Google Earth images. Existing land use includes three detached dwellings, asphalt driveways and open spaces with urban lawns. The northern portion of the property is a wooded area.

Land use within the study area under post-development conditions was identified from the Site Plan (Attachment 1). The Site Plan provides the extents of the proposed apartment building, as well as the total area of the proposed parking, pathway, and open space/urban lawn area. The northern portion of the property was assumed to remain wooded north of the tree protection line.

3.0 WATER BALANCE PARAMETERS

Golder's Geotechnical and Hydrogeological Investigations identified surficial layers of fine sandy loam across the majority of the Site. Hydrologic Cycle Component Values from Table 3.1 of the MOE SWM Manual (MOE 2003)) were used to determine the WHC and infiltration factors assuming a fine sandy loam soil type over the described land uses. For impervious areas, it was assumed that 90% of the sum of rainfall plus snowmelt resulted in runoff after evaporation was removed as per the Hydrogeological Assessment Submissions Conservation Authority Guidelines to Support Development Applications (Conservation Ontario, 2013).

Infiltration factors were calculated using a sum of site-specific topography, surficial soil type and vegetative cover factors for each land use, as shown in table 3.1 of the MOE SWM manual (MOE 2003). Runoff was calculated as the difference between surplus and infiltration. Annual evapotranspiration values were based on the water holding capacity of each land use area. Surplus values were calculated as the annual precipitation minus annual actual evapotranspiration.

3.1 Water Holding Capacities, Infiltration Factors, and evapotranspiration rates

The following pre- and post-development water holding capacities and infiltration factors were assigned to the Site.

Pre-development Water Holding Capacity and Infiltration Factors

- For existing grassed and open areas, a WHC of 75 mm was used, representing urban lawns in fine sandy loam soils. An infiltration factor of 0.6 was used, representing hilly land with an average slope of between 28 m/km and 47 m/km (factor of 0.1), open sandy loam (factor of 0.4), and urban lawns (factor of 0.1).
- For existing wooded areas, a WHC of 300 mm was used, representing mature forests in fine sandy loam soils. An infiltration factor of 0.7 was used, representing hilly land with an average slope of between 28 m/km and 47 m/km (factor of 0.1), open sandy loam (factor of 0.4), and woodland (factor of 0.2).
- For existing houses and paved areas, an infiltration factor of zero was applied, indicating no infiltration occurring on these surfaces. The surplus value for these impervious areas was calculated as the annual precipitation minus annual potential evapotranspiration. As stated above, it was assumed that 90% of rainfall plus snowmelt resulted in runoff after evaporation was removed.

Post-Development Water Holding Capacity and Infiltration Factors

- For post-development grassed and open areas, a WHC of 75 mm was used, representing urban lawns in fine sandy loam soils. An infiltration factor of 0.6 was used, representing hilly land with an average slope of between 28 m/km and 47 m/km (factor of 0.1), open sandy loam (factor of 0.4), and urban lawns (factor of 0.1).
- For post-development wooded areas, a WHC of 300 mm was used, representing mature forests in fine sandy loam soils. An infiltration factor of 0.7 was used, representing hilly land with an average slope of between 28 m/km and 47 m/km (factor of 0.1), open sandy loam (factor of 0.4), and woodland (factor of 0.2).
- For the post-development impervious areas, including walkways, pavement surfaces and the building roof, an infiltration factor of zero was applied, indicating no infiltration occurring on these surfaces. The surplus value for these impervious areas was calculated as the annual precipitation minus annual potential evapotranspiration. As stated above, it was assumed that 90% of rainfall plus snowmelt resulted in runoff after evaporation was removed (10% evapotranspiration).

4.0 PRELIMINARY WATER BALANCE RESULTS

Water balance results for the 17,280 m^2 Site catchment area are summarized herein.

4.1 Pre-Development Condition

Table 1 presents the results of the water balance for the Site area under existing conditions on an average annual basis. Under existing conditions, the estimated average annual runoff from the Site is approximately 2,750 m^3, and the estimated average annual infiltration from the Site is approximately 4,280 m^3.

4.2 Unmitigated Post-Development Condition

Detailed land use areas associated with the proposed development were assumed as described above. Approximately 28% of the post-development site is proposed to be impervious, including the building area, parking area, and walkways.

Table 1 presents the results of the water balance within the study area under post-development conditions on an average annual basis with no mitigation (i.e., no LID) measures applied. The estimated average annual runoff from the un-mitigated Site is approximately 5,720 m^3 and the estimated average annual infiltration is approximately 3,210 m^3. Runoff contributions from the un-mitigated Site are estimated to increase by approximately 2,970 m^3/yr or 108%, while the infiltration is estimated to decrease by approximately 1,070 m^3/yr or 25%, relative to the pre-development conditions.

Table 1: Pre-Development and Unmitigated Post-Development Annual Water Balance Results

Parameter	Existing Conditions		Unmitigated Post-Development Conditions	
	mm/yr	m^3/yr	mm/yr	m^3/yr
Precipitation	974	16,830	974	16,830
Evapotranspiration	567	9,800	457	7,900
Surplus	407	7,030	527	8,930
Infiltration	248	4,280	183	3,210
Runoff	159	2,750	344	5,720

4.3 Mitigated Post-Development Condition

The attached site plan identified an 897 m^2 section of the fourth floor terrace to be used as a green roof. Although the design is preliminary, it is assumed that the green roof would reduce runoff from that area of the roof by approximately 45% through increased evapotranspiration (TRCA and CVC, 2010). This is a typical estimate for the purpose of initial screening of LID practices, and should be confirmed during detailed design.

In order to further reduce runoff and increase infiltration at the site, additional LID measures could be considered. Since groundwater levels observed at the Site generally ranged between 8 mbgs and 14 mbgs, it is expected that subsurface LID measures (e.g., soakaways, infiltration trenches and chambers) could be designed to maintain at least 1 m vertical separation from the seasonally high groundwater level (TRCA and CVC, 2010).

As such, the mitigated post-development water balance for this site incorporated a soakaway pit, located the north of the building. All runoff from the roof would be directed to the soakaway pit directly through pipes. The soakaway pit would consist of an excavation lined with geotextile fabric and filled with a clean granular stone infiltration gallery, sized to detain a design amount of rainfall. The soakaway pit would have an area of at least 190 m^2 in order to achieve an impervious drainage area to treatment facility area ratio of 20:1 (TRCA and CVC, 2010). Placement of the soakaway pit on the north side of the property would be determined as part of detailed design, but should be at least 4 m away from the building foundation (TRCA and CVC, 2010).

Infiltration from the recommended soakaway pit was estimated on an annual basis by analyzing the daily precipitation record at Shanty Bay (1974 to 2017). The soakaway pit was assumed to infiltrate all rainfall depth that was equal to or less than 8 mm in a 24 hour period. This resulted in an infiltration factor of 0.46 (46%) for the entire roof area.

Table 2 presents the results of the post-development site-wide water balance, including the green roof and soakaway mitigations for average annual conditions. The total estimated average annual runoff from the Site is approximately 4,050 m^3/yr and the estimated average annual infiltration is approximately 4,580 m^3/yr for the mitigated post-development condition. Compared to the existing condition, the Site-wide infiltration is expected in increase by approximately 300 m^3/yr or 7%, and the total runoff from the Site is estimated to increase by 1,300 m^3/yr or 47%.

Table 2: Mitigated Post-Development Annual Water Balance Results

Parameter	Existing Conditions		Mitigated Post-Development Conditions	
	mm/yr	m^3/yr	mm/yr	m^3/yr
Precipitation	974	16,830	974	16,830
Evapotranspiration	567	9,800	475	8,200
Surplus	407	7,030	499	8,630
Infiltration	248	4,280	265	4,580
Runoff	159	2,750	234	4,050

5.0 DISCUSSION & CONCLUSION

Based on the results of the preliminary water balance assessment, the average annual runoff from the Site is expected to increase by approximately 47% and the average annual infiltration is expected to increase by approximately 7% under mitigated post-development conditions, compared to pre-development conditions. This assessment included the proposed green roof LID measure as well as a soakaway pit, which would collect runoff from the entire roof of the building. The design of the LID measures will need to be confirmed during detailed design. However, this assessment indicates that, with the proper use of LID measures, the mitigated average annual post-development infiltration rate is considered to approximate pre-development conditions (i.e., it is within +/- 10%).

6.0 REFERENCES

Conservation Ontario, “Conservation Authority Guidelines for Hydrogeological Assessment Submissions”, June, 2013.

Environment and Climate Change Canada. Historical Weather Data. Shanty Bay, Ontario, Climate ID 6117684 http://climate.weather.gc.ca/historical_data/search_historic_data_e.html

Ontario Ministry of the Environment, “Stormwater Management Planning and Design Manual”, 2003.

Toronto and Region Conservation Authority and Credit Valley Conservation Authority (TRCA and CVC), “Low Impact Development Stormwater Management Planning and Design Guide”, 2010.

LICENSED PROFESSIONAL ENGINEER
D. P. HINTON
100228302
19/3/29
PROVINCE OF ONTARIO

David Hinton, PEng
Water Resources Engineer

Gerard Van Arkel, PEng
Associate, Senior Water Resources Engineer

Attachments: Attachment 1: Site Plan

https://golderassociates.sharepoint.com/sites/32312g/5000 water balance/memo/18108181-tm-rev0-georgian dr wb-29mar2019.docx

Appendix E

Phosphorus Calculations

PEARSON ENGINEERING LTD.

290, 294, 298, & 302 Georgian Drive
Phosphorus Budget Tool

	Residential	Commercial	Low Intensity Development	Forest
Phosphorus Export (kg/ha/year)	1.32	1.82	0.07	0.05

Pre-Development Condition

	Residential	Commercial	Low Intensity Development	Forest
Area (ha):	0.76	0.00	0.00	0.00
Total P (kg):	1.00	0.00	0.00	0.00
Total Pre-Development P (kg):		**1.00**		

Post-Development Condition (Uncontrolled)

Total Area	Residential	Commercial	Low Intensity Development	Forest
Area (ha):	0.76	0.00	0.00	0.00
Total P (kg):	1.00	0.00	0.00	0.00
Total Post-Development P (kg):		**1.00**		

Appendix F

Oil/Grit Separator Information

CONTECH ENGINEERED SOLUTIONS

CDS ESTIMATED NET ANNUAL SOLIDS LOAD REDUCTION
BASED ON THE RATIONAL RAINFALL METHOD
BASED ON A FINE PARTICLE SIZE DISTRIBUTION

CDS TECHNOLOGIES

Project Name: 290-302 Georgian Drive
Location: Barrie, ON
OGS #: OGS

Engineer: **Pearson Engineering**
Contact: Taylor Arkell, P.Eng.
Report Date: 3-Oct-19

Area 0.03 ha
Weighted C 0.95
CDS Model 2015-4

Rainfall Station # 203
Particle Size Distribution FINE
CDS Treatment Capacity 20 l/s

Rainfall Intensity[1] (mm/hr)	Percent Rainfall Volume[1]	Cumulative Rainfall Volume	Total Flowrate (l/s)	Treated Flowrate (l/s)	Operating Rate (%)	Removal Efficiency (%)	Incremental Removal (%)
0.5	8.7%	8.7%	0.0	0.0	0.2	98.8	8.6
1.0	10.8%	19.6%	0.1	0.1	0.4	98.7	10.7
1.5	9.5%	29.0%	0.1	0.1	0.6	98.7	9.3
2.0	8.4%	37.4%	0.2	0.2	0.8	98.6	8.3
2.5	6.8%	44.2%	0.2	0.2	1.0	98.6	6.7
3.0	5.6%	49.8%	0.2	0.2	1.2	98.5	5.5
3.5	5.1%	54.9%	0.3	0.3	1.4	98.5	5.0
4.0	4.9%	59.8%	0.3	0.3	1.6	98.4	4.8
4.5	4.1%	63.9%	0.4	0.4	1.8	98.3	4.0
5.0	3.5%	67.4%	0.4	0.4	2.0	98.3	3.4
6.0	4.9%	72.3%	0.5	0.5	2.4	98.2	4.8
7.0	4.0%	76.3%	0.6	0.6	2.8	98.1	3.9
8.0	3.2%	79.5%	0.6	0.6	3.2	97.9	3.2
9.0	2.2%	81.7%	0.7	0.7	3.6	97.8	2.2
10.0	2.0%	83.7%	0.8	0.8	4.0	97.7	1.9
15.0	8.2%	91.9%	1.2	1.2	6.0	97.1	7.9
20.0	3.4%	95.2%	1.6	1.6	8.0	96.6	3.2
25.0	2.5%	97.7%	2.0	2.0	10.0	96.0	2.4
30.0	1.4%	99.1%	2.4	2.4	12.0	95.4	1.4
35.0	0.3%	99.4%	2.8	2.8	14.0	94.8	0.2
40.0	0.6%	100.0%	3.2	3.2	16.0	94.3	0.6
							98.2

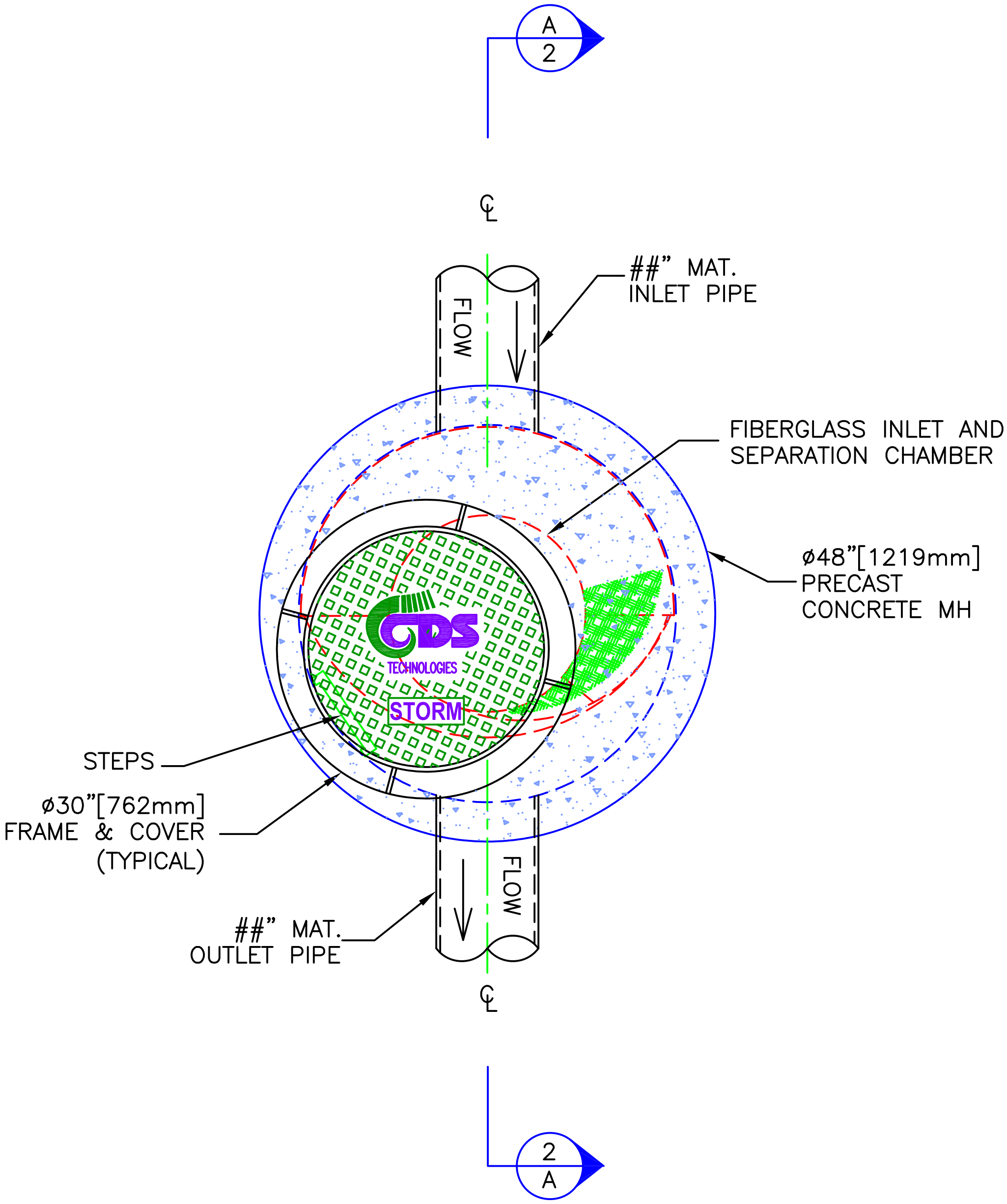
Removal Efficiency Adjustment[2] = 6.5%

Predicted Net Annual Load Removal Efficiency = 91.7%

Predicted % Annual Rainfall Treated = 99.9%

1 - Based on 27 years of hourly rainfall data from Canadian Station 6110557, Barrie ON
2 - Reduction due to use of 60-minute data for a site that has a time of concentration less than 30-minutes.
3 - CDS Efficiency based on testing conducted at the University of Central Florida
4 - CDS design flowrate and scaling based on standard manufacturer model & product specifications

PLAN VIEW

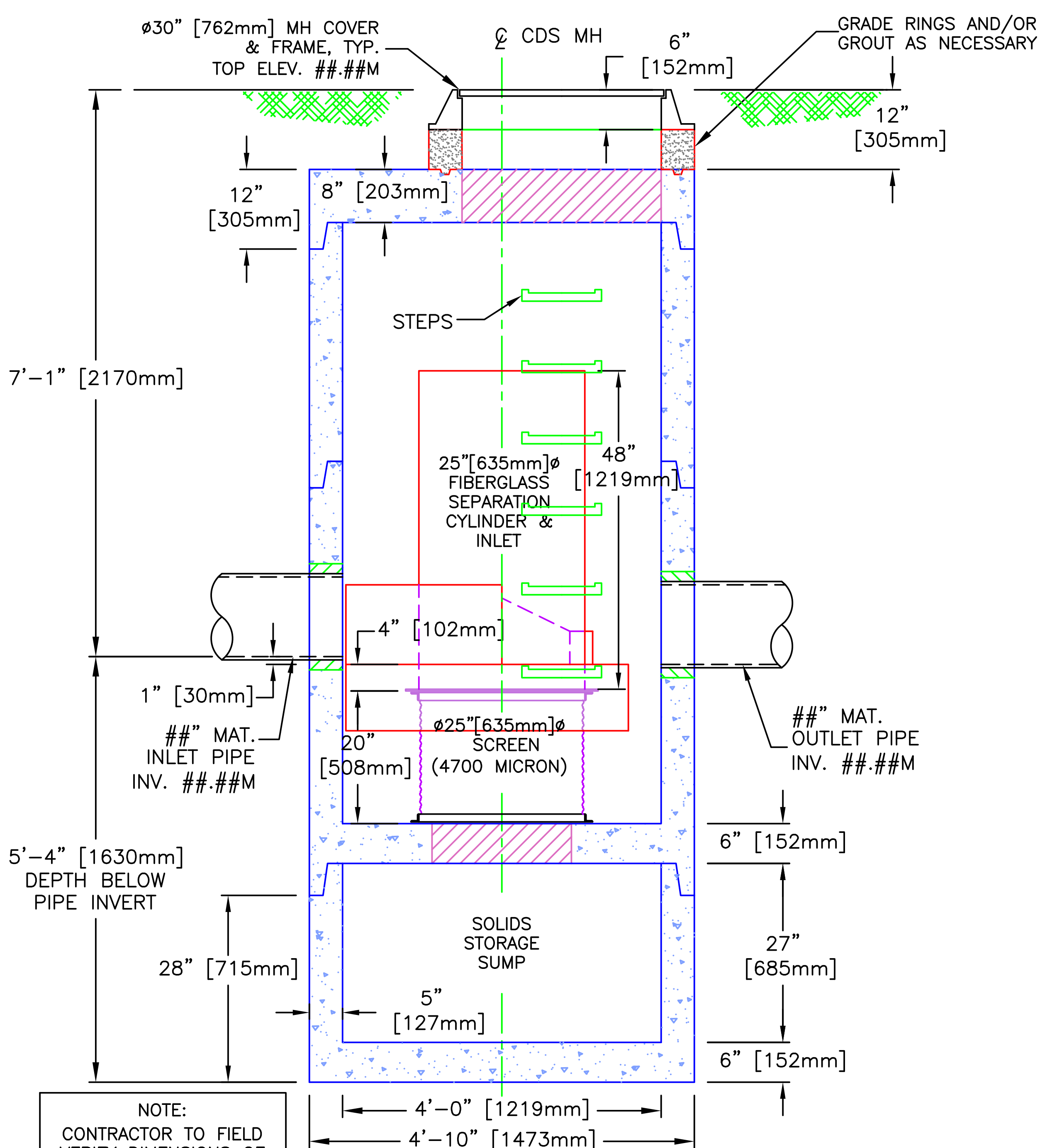


CDS MODEL PMSU20_15_4m
STORMWATER TREATMENT UNIT

CONTECH STORMWATER SOLUTIONS	PROJECT NAME CITY, STATE	JOB# XX-##-###	SCALE 1" = 2'
		DATE ##/##/##	SHEET
		DRAWN INITIALS	1
		APPROV.	

Echelon Environmental 505 Hood Road, Unit 26, Markham, Ontario L3R 5V6 Tel: (905) 948-0000 Fax: (905) 948-0577
CONTECH Stormwater Solutions Inc. 930 Woodcock Road, Suite 101, Orlando, Florida 32803 Tel: (800) 848-9955

SECTION A–A
ELEVATION VIEW



NOTE:
CONTRACTOR TO FIELD
VERIFY DIMENSIONS OF
OR CONCRETE SECTIONS

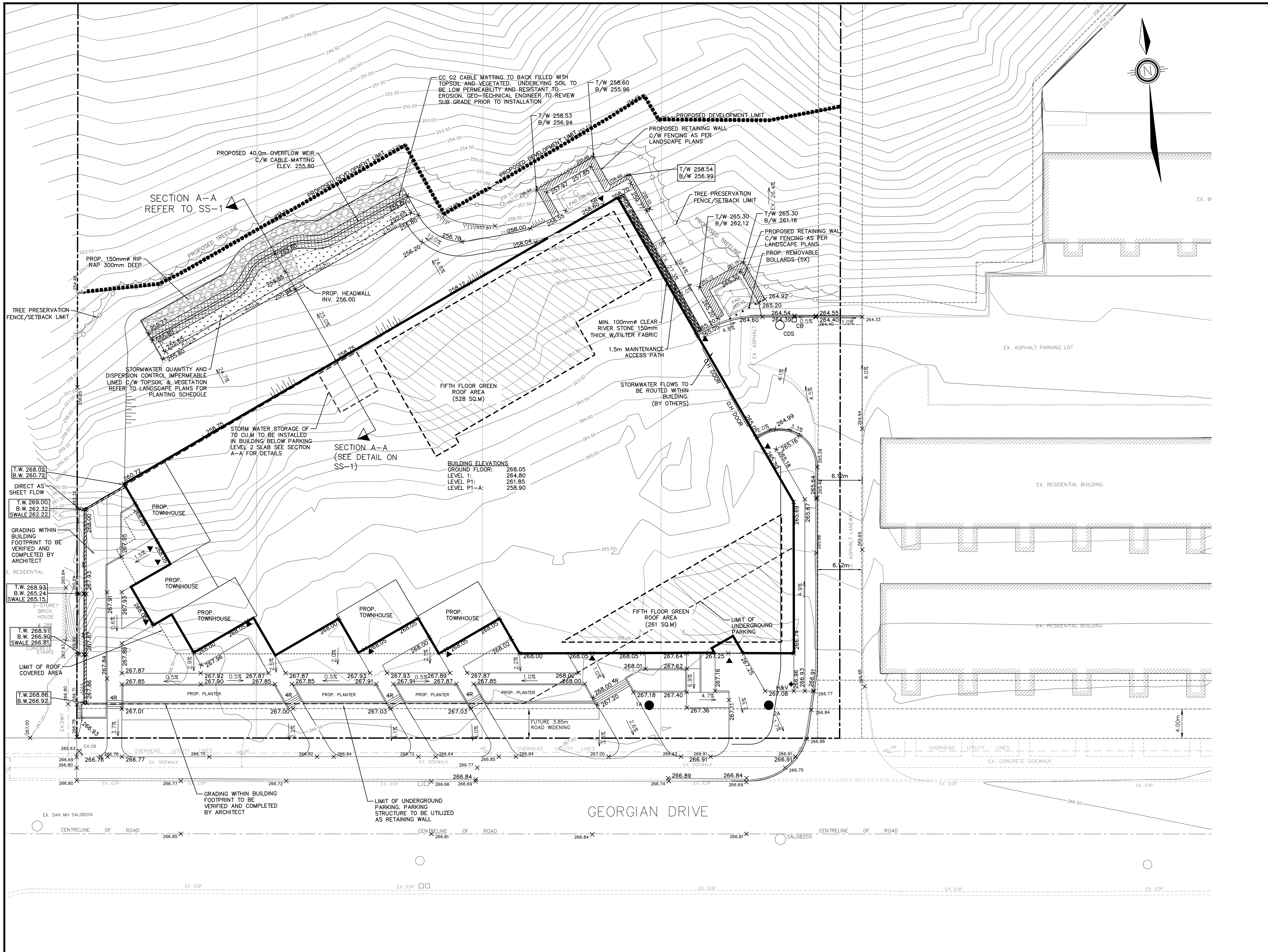
CDS MODEL PMSU20_15_4m
STORMWATER TREATMENT UNIT

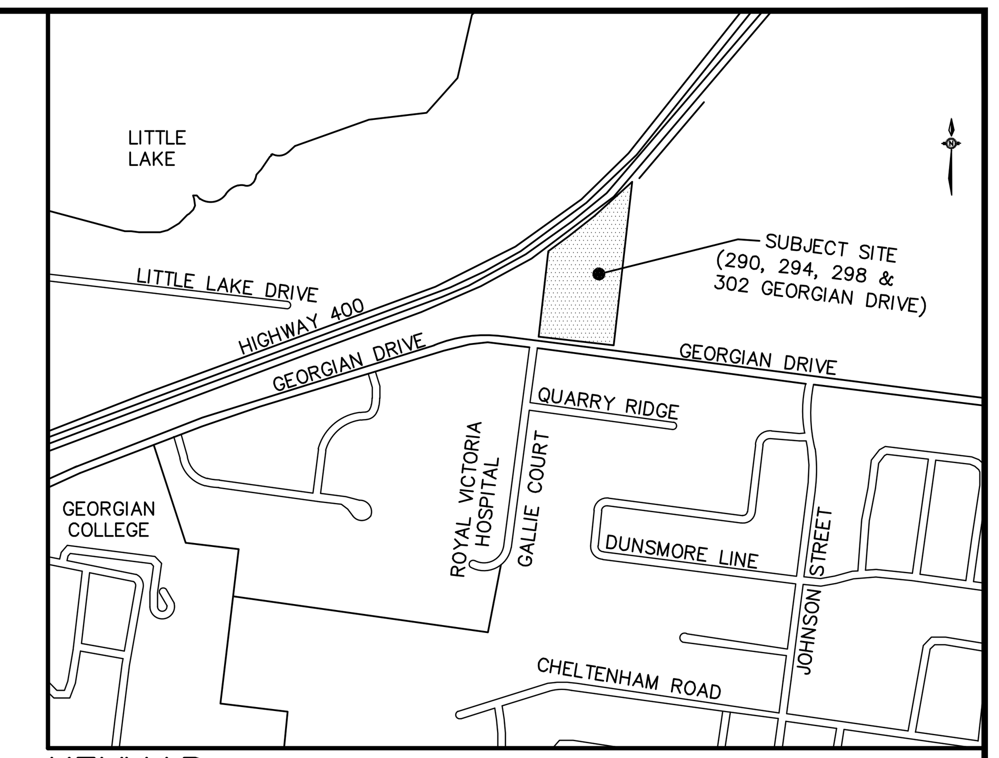
CONTECH STORMWATER SOLUTIONS	PROJECT NAME CITY, STATE	JOB#	XX–##–###	SCALE 1" = 2'
		DATE	##/##/##	SHEET
		DRAWN	INITIALS	2
		APPROV.		

Echelon Environmental 505 Hood Road, Unit 26, Markham, Ontario L3R 5V6 Tel: (905) 948-0000 Fax: (905) 948-0577
CONTECH Stormwater Solutions Inc. 930 Woodcock Road, Suite 101, Orlando, Florida 32803 Tel: (800) 848-9955

Appendix G

Engineering Drawings





KEYMAP

LEGEND

- SAN MAINTENANCE HOLE
- CATCH BASIN
- DOUBLE CATCH BASIN
- DOUBLE CATCH BASIN MAINTENANCE HOLE
- CATCH BASIN MAINTENANCE HOLE
- STORM MAINTENANCE HOLE
- 386.96 PROPOSED ELEVATION
- 386.96 EXISTING ELEVATION
- 2% PROPOSED GRADE
- SERVICE CAP
- FIRE HYDRANT
- WATER VALVE
- CURB STOP
- H.P. HYDRO POLE
- BELL PEDESTAL
- HYDRO TRANSFORMER
- LS LIGHT STANDARD
- WATER METER
- BARRIER CURB
- G GENERATOR PAD
- T TRANSFORMER PAD
- PROP. DEVELOPMENT LIMIT

SITE BENCHMARK:
LOCATED AT TOP OF HYDRANT SPINDLE ON THE SOUTH SIDE OF GEORGIAN DRIVE, SOUTH EAST OF SITE.

ELEV. 267.46m

SURVEY:
TOPOGRAPHIC SURVEY COMPLETED BY RUDY MAK SURVEYING LTD. AND ISSUED ON AUGUST 1, 2014.

NO.	REVISION NOTE	DATE	BY
4.	REVISED AS PER CITY OF BARRIE COMMENTS	09/08/21	JPE
3.	MINOR REVISIONS/RESUBMISSION	04/07/21	PCO
2.	REVISED AS PER COMMENTS	04/21/20	PCO
1.	REVISED AS PER COMMENTS	10/02/19	PCO

CONTROL MONUMENTS:
HORIZONTAL
00119713803 – SOUTHWEST CORNER OF GEORGIAN DR & PENETANGUISHENE RD.
EASTING 607787.028 NORTHING 4918904.790
00119713804 – SOUTH SIDE OF GEORGIAN DR. ~110m WEST OF LARKIN DR.
EASTING 607477.286 NORTHING 4918937.253
VERTICAL
03120030002 – CAP – EASTING 606202 NORTHING 4918802
RVH HELIPORT AT WEST ENTRANCE. ELEV: 260.583
00820038068 – CAP – EASTING 606084, NORTHING 4918831
N.SIDE OF GEORGIAN DR. ~105m EAST OF GOVERNORS DR. ELEV: 258.425

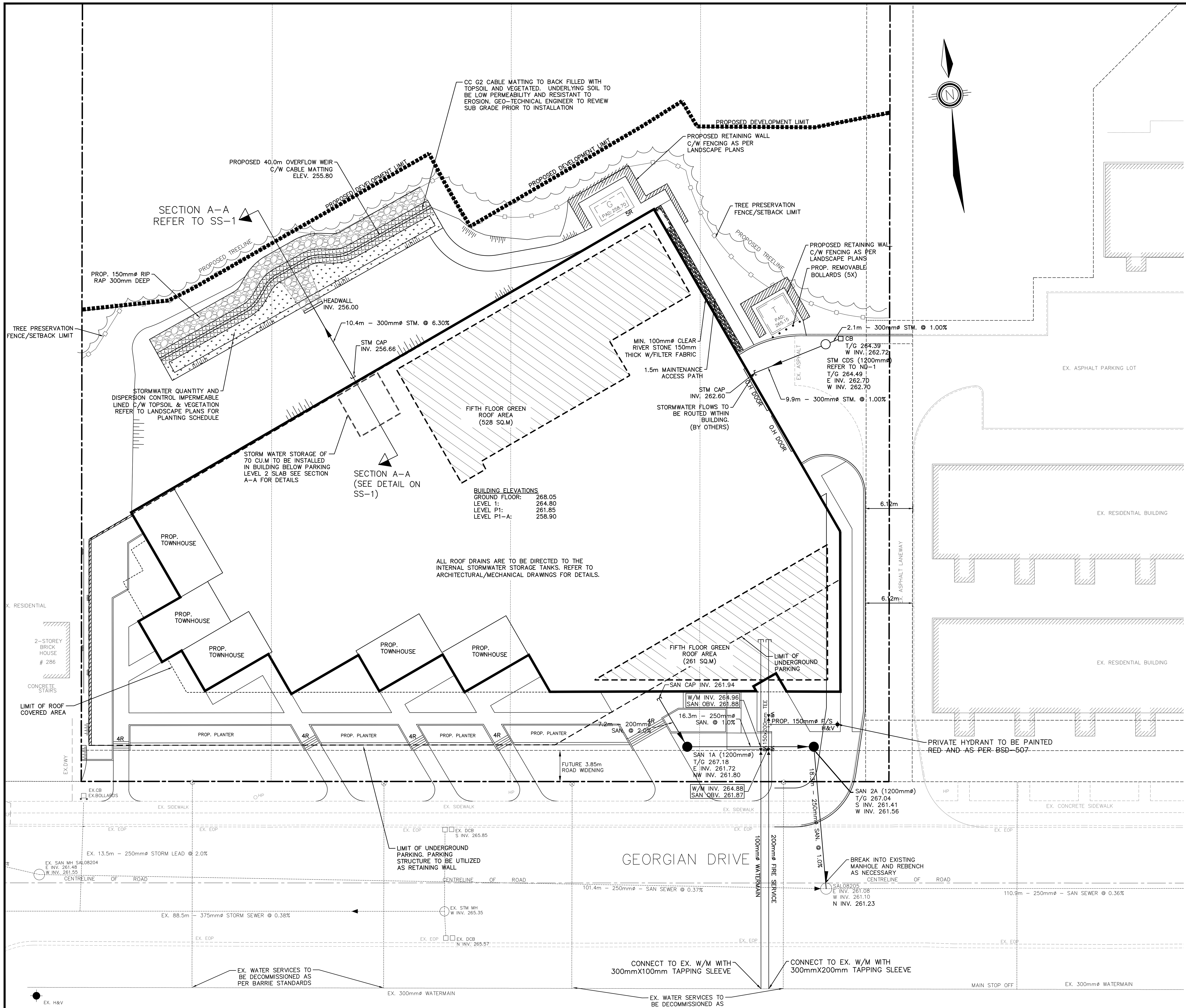
LICENSED PROFESSIONAL ENGINEER
G. M. PEARSON
100061986
PROVINCE OF ONTARIO

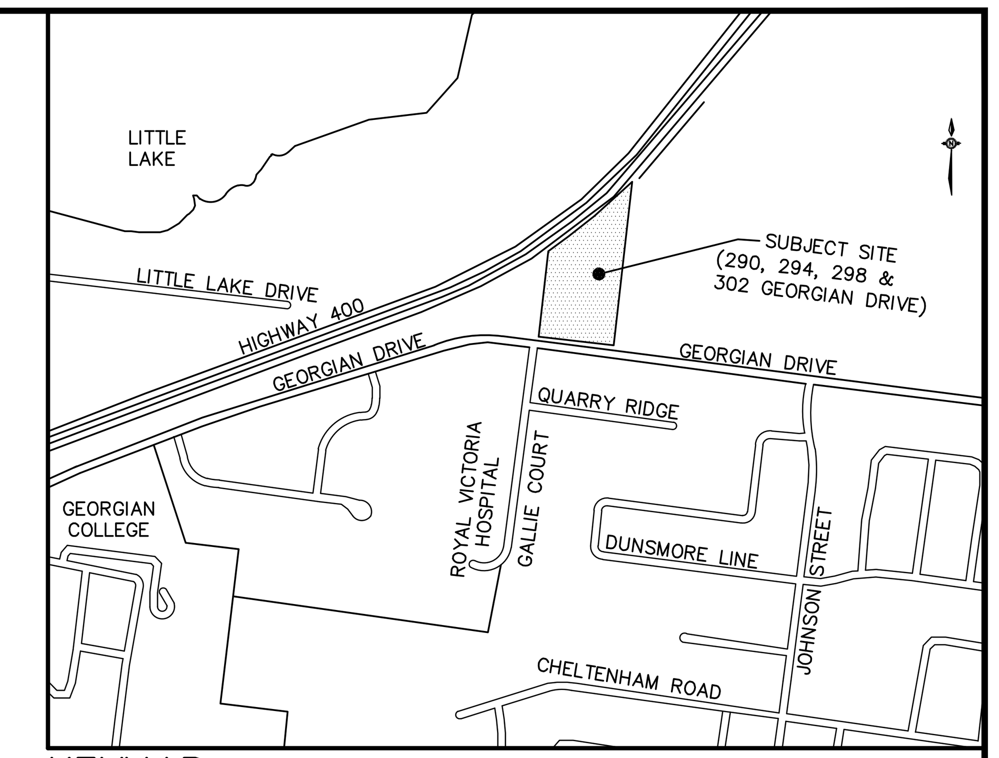
THE GEORGIAN APARTMENTS
290,294,298,&302 GEORGIAN DRIVE
BARRIE, ONTARIO

SITE GRADING PLAN

PEARSON ENGINEERING
PEARSONENG.COM PH. 705.719.4785

DESIGNED BY	PCO	HORIZ SCALE	1:250	PROJECT #	18037
DRAWN BY	MKRW	VERT SCALE		DRAWING #	SG-1
CHECKED BY	GMP	DATE	SEPTEMBER 2018	REVISION #	4





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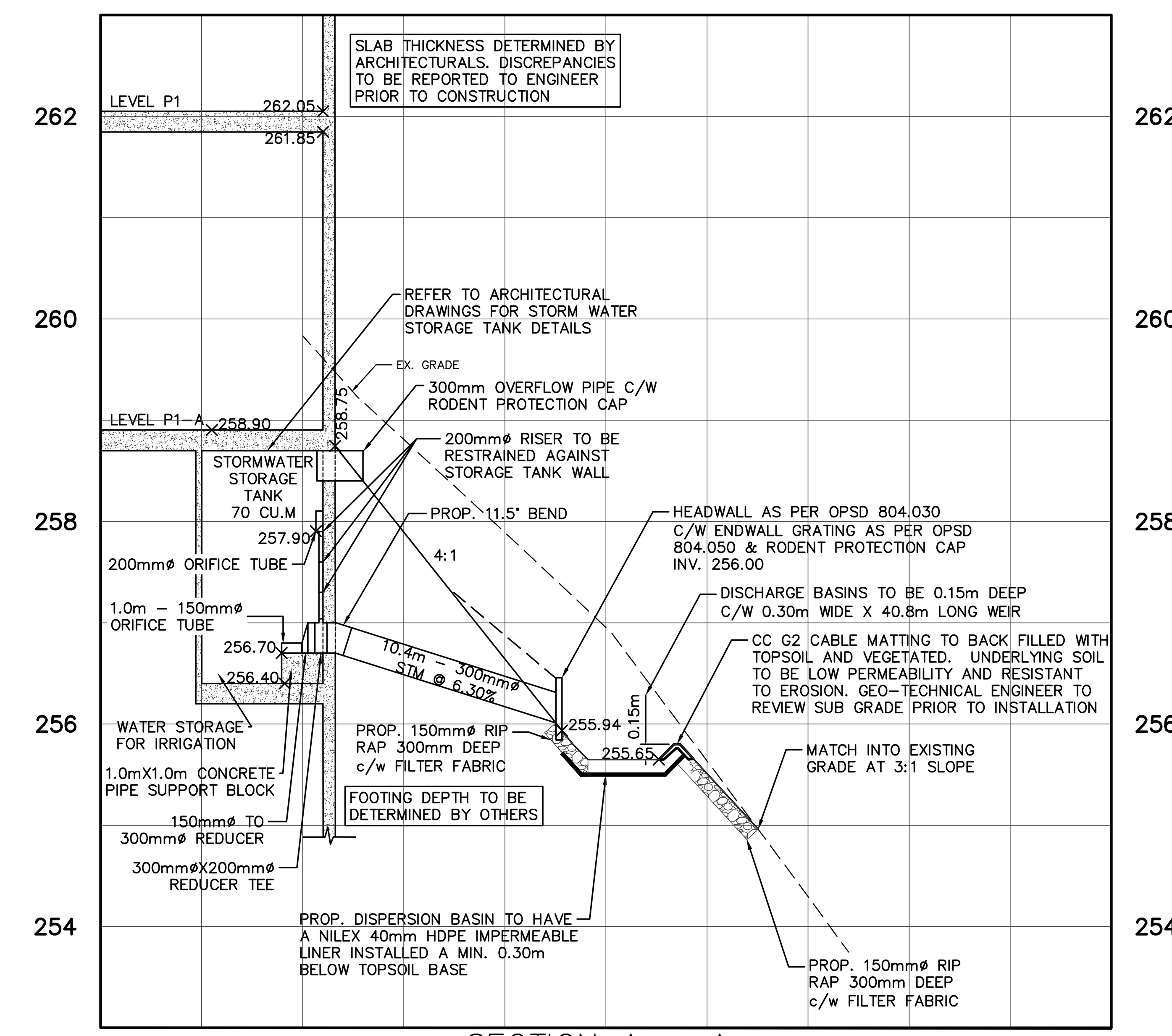

SECTION A – A
HORIZONTAL SCALE: 1:250
VERTICAL SCALE: 1:50

SITE BENCHMARK:
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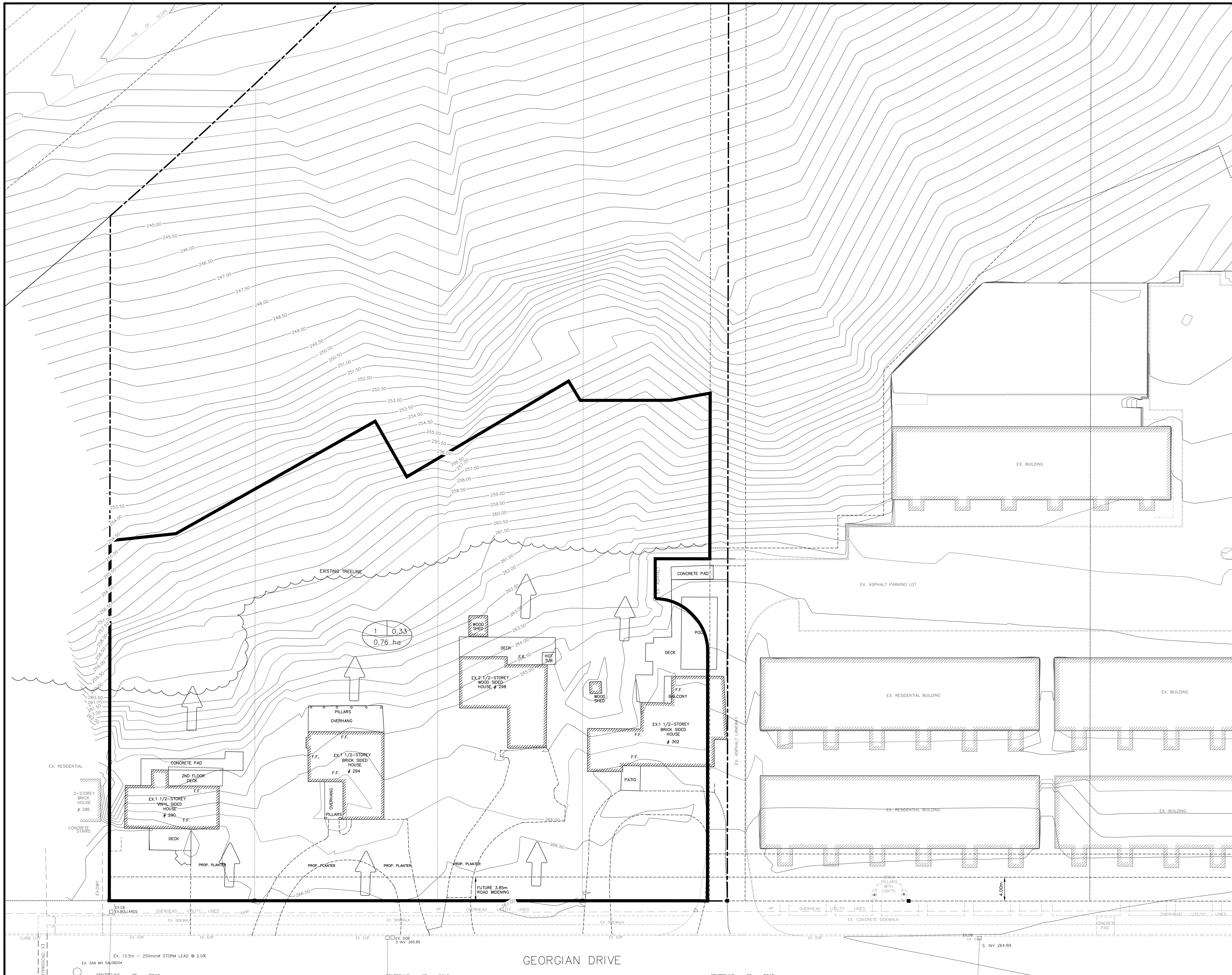
LICENSED PROFESSIONAL ENGINEER G. M. PEARSON 100061986 PROVINCE OF ONTARIO

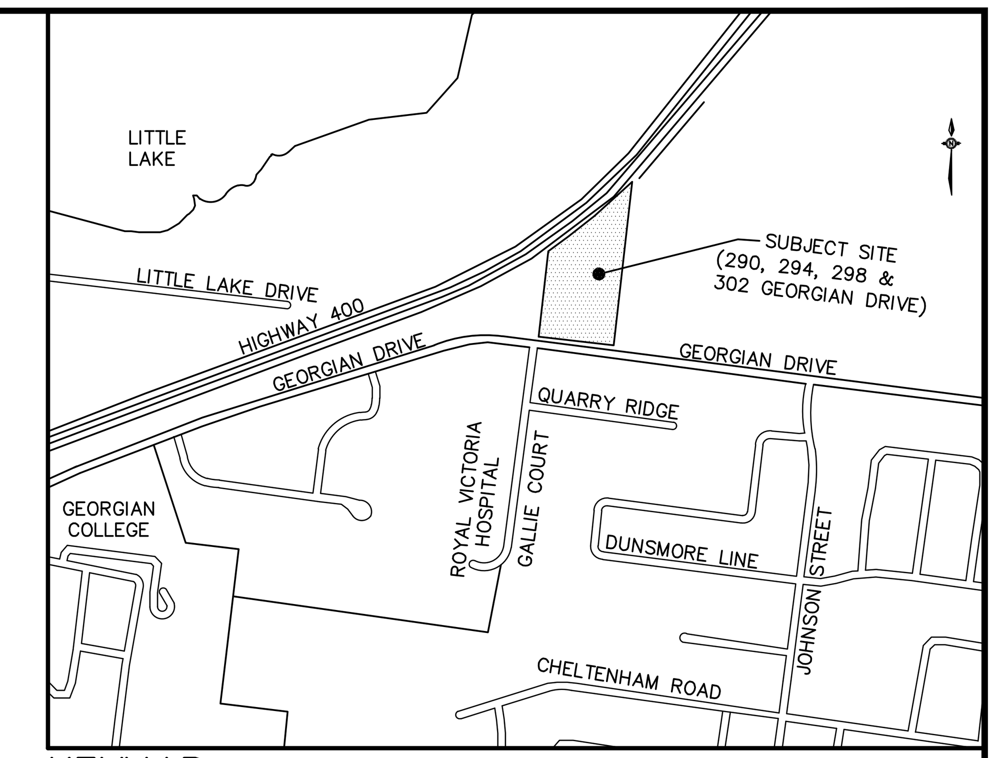
THE GEORGIAN APARTMENTS
290,294,298,&302 GEORGIAN DRIVE
BARRIE, ONTARIO

SITE SERVICING PLAN

PEARSON ENGINEERING
PEARSONENG.COM PH. 705.719.4785

DESIGNED BY	PCO	HORIZ SCALE	1:250	PROJECT #	18037
DRAWN BY	MKRW	VERT SCALE		DRAWING #	SS-1
CHECKED BY	GMP	DATE	SEPTEMBER 2018	REVISION #	4





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- M WATER METER
- BARRIER CURB
- OVERLAND FLOW DIRECTION

CATCHMENT AREA | RUNOFF COEFFICIENT
1 | 0.33
0.76 ha
AREA

- CATCHMENT BOUNDARY

SITE BENCHMARK:

LOCATED AT TOP OF HYDRANT SPINDLE ON THE SOUTH SIDE OF GEORGIAN DRIVE, ACROSS FROM SITE.

ELEV. 267.46m

SURVEY:
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LICENSED PROFESSIONAL ENGINEER G. M. PEARSON 100061986 PROVINCE OF ONTARIO

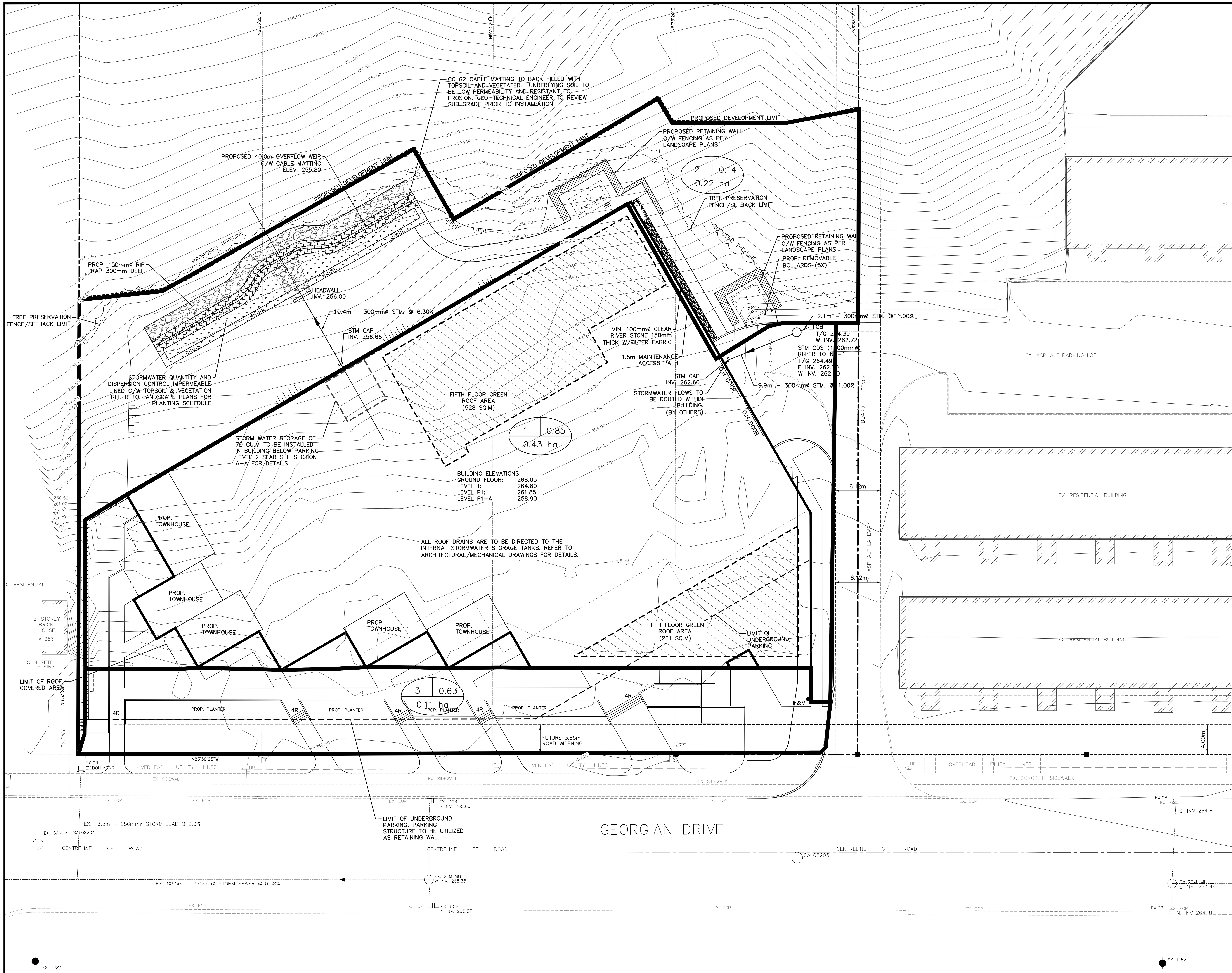
THE GEORGIAN APARTMENTS
290,294,298,&302 GEORGIAN DRIVE
BARRIE, ONTARIO

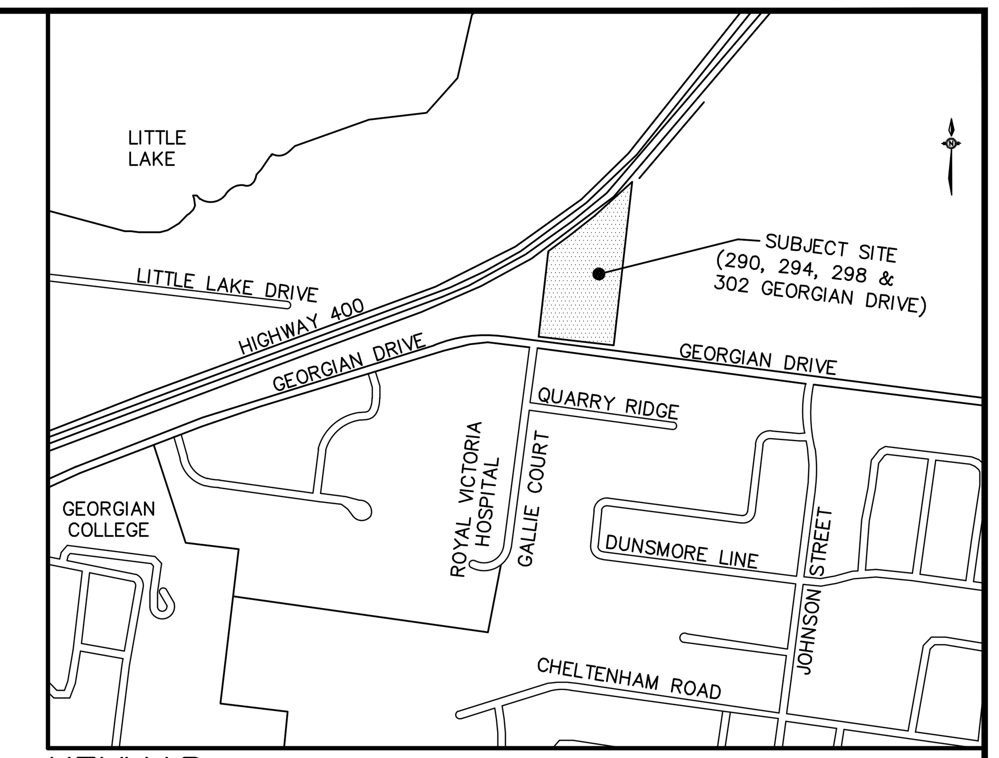
PRE-DEVELOPMENT STORM DRAINAGE PLAN

PEARSON ENGINEERING
PEARSONENG.COM PH. 705.719.4785

DESIGNED BY	PCO	HORIZ SCALE	1:250	PROJECT #	18037
DRAWN BY	MKRW	VERT SCALE		DRAWING #	STM-1
CHECKED BY	GMP	DATE	SEPTEMBER 2018	REVISION #	4

C:\Users\jevans.PENG\AppData\Local\Temp\AcPublish_23680\18037-BASE.dwg Layout:STM-1 Plotted Sep 08, 2021 @ 4:00pm by jevans © PEARSON ENGINEERING LTD.





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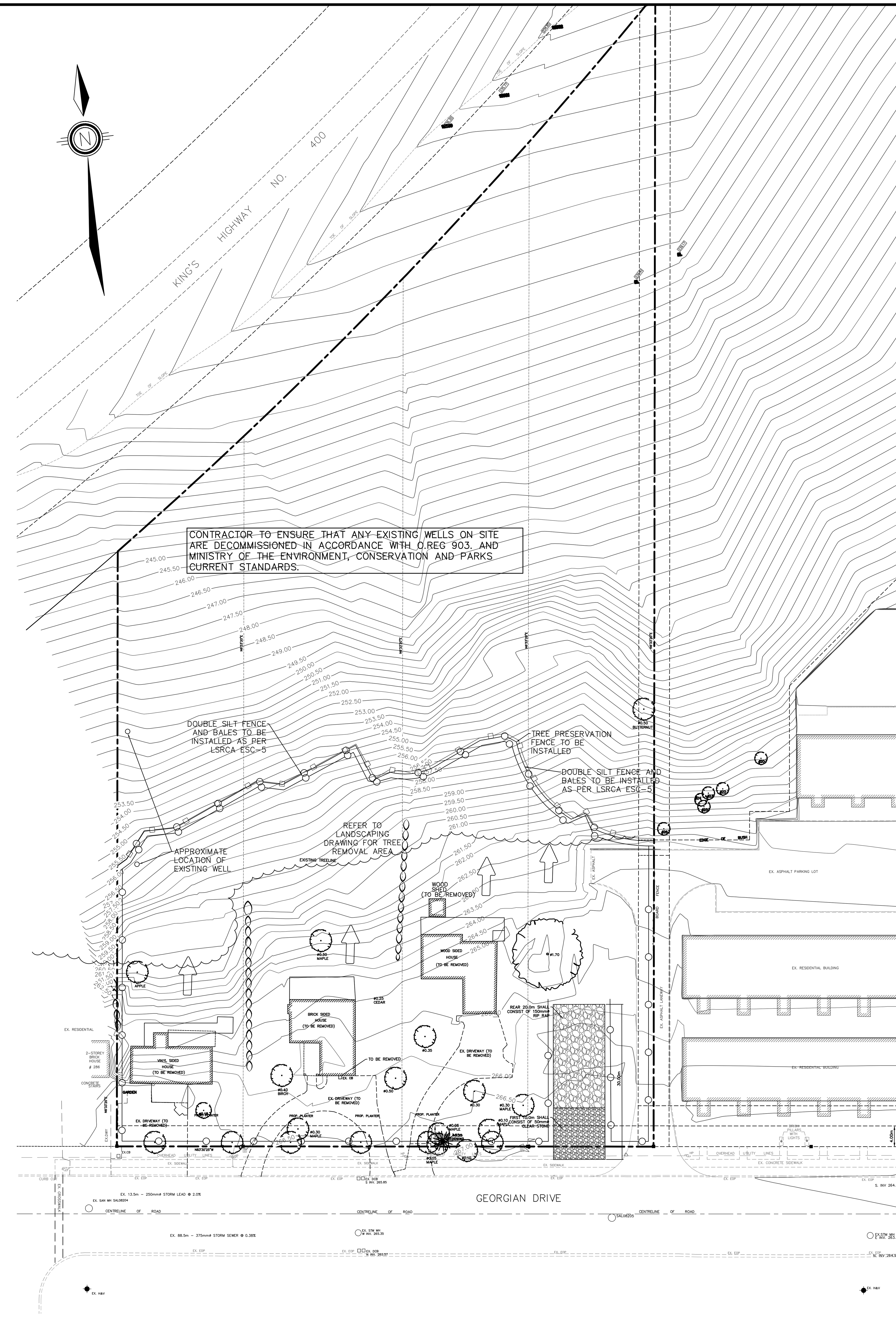
LICENSED PROFESSIONAL ENGINEER
G. M. PEARSON
100061986
PROVINCE OF ONTARIO

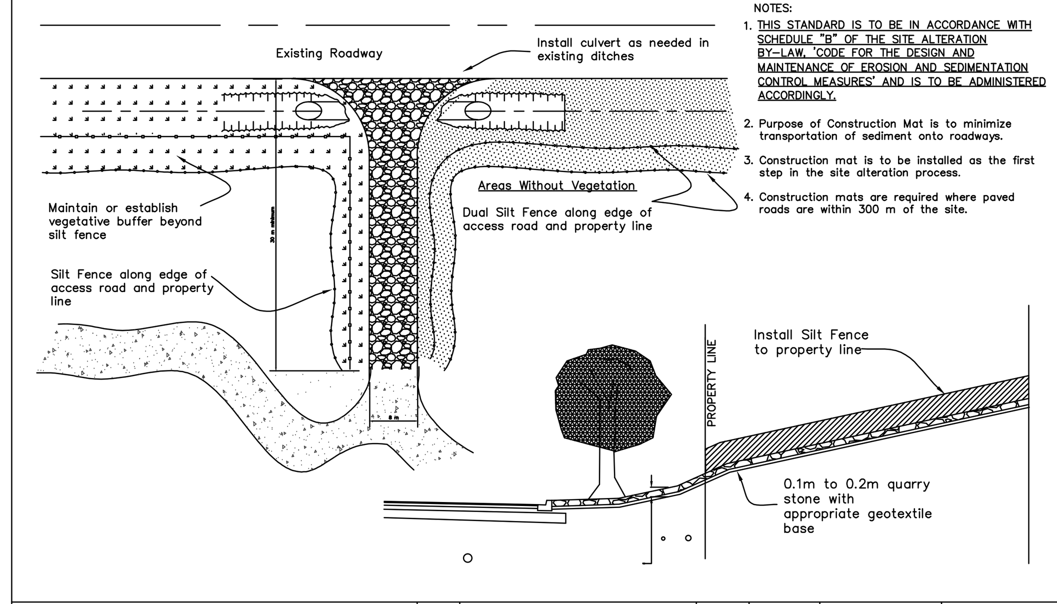
THE GEORGIAN APARTMENTS
290,294,298,&302 GEORGIAN DRIVE
BARRIE, ONTARIO

POST DEVELOPMENT STORM DRAINAGE PLAN

PEARSON ENGINEERING
PEARSONENG.COM PH. 705.719.4785

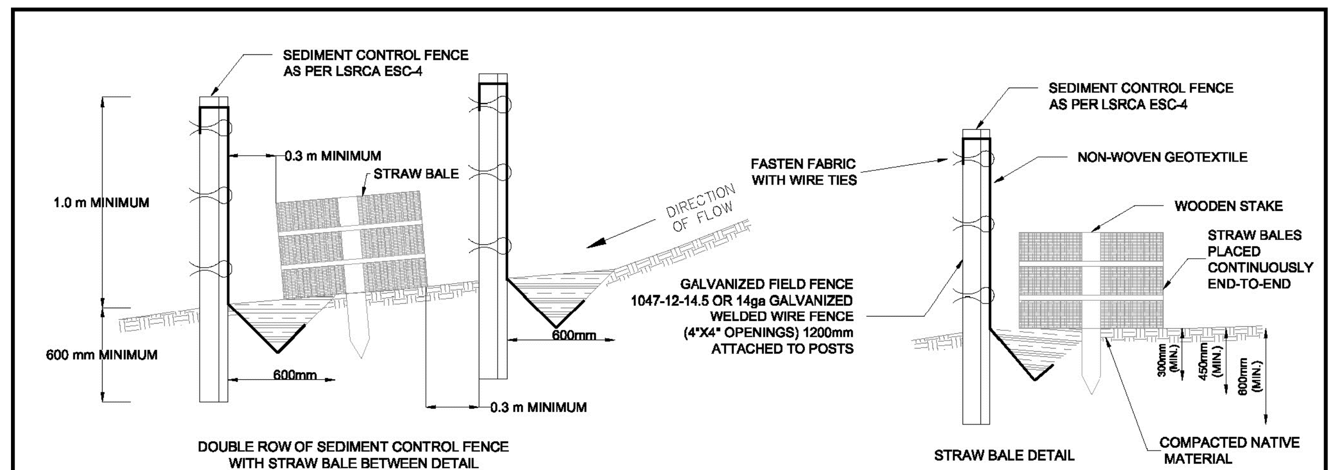
DESIGNED BY	PCO	HORIZ SCALE	1:250	PROJECT #	18037
DRAWN BY	MKRW	VERT SCALE		DRAWING #	STM–2
CHECKED BY	GMP	DATE	SEPTEMBER 2018	REVISION #	4





NOTES:
1. THIS STANDARD IS TO BE IN ACCORDANCE WITH SCHEDULE "B" OF THE SITE ALTERATION BY-LAW, 'CODE FOR THE DESIGN AND MAINTENANCE OF EROSION AND SEDIMENTATION CONTROL MEASURES' AND IS TO BE ADMINISTERED ACCORDINGLY.
2. Purpose of Construction Mat is to minimize transportation of sediment onto roadways.
3. Construction mat is to be installed as the first step in the site alteration process.
4. Construction mats are required where paved roads are within 300 m of the site.

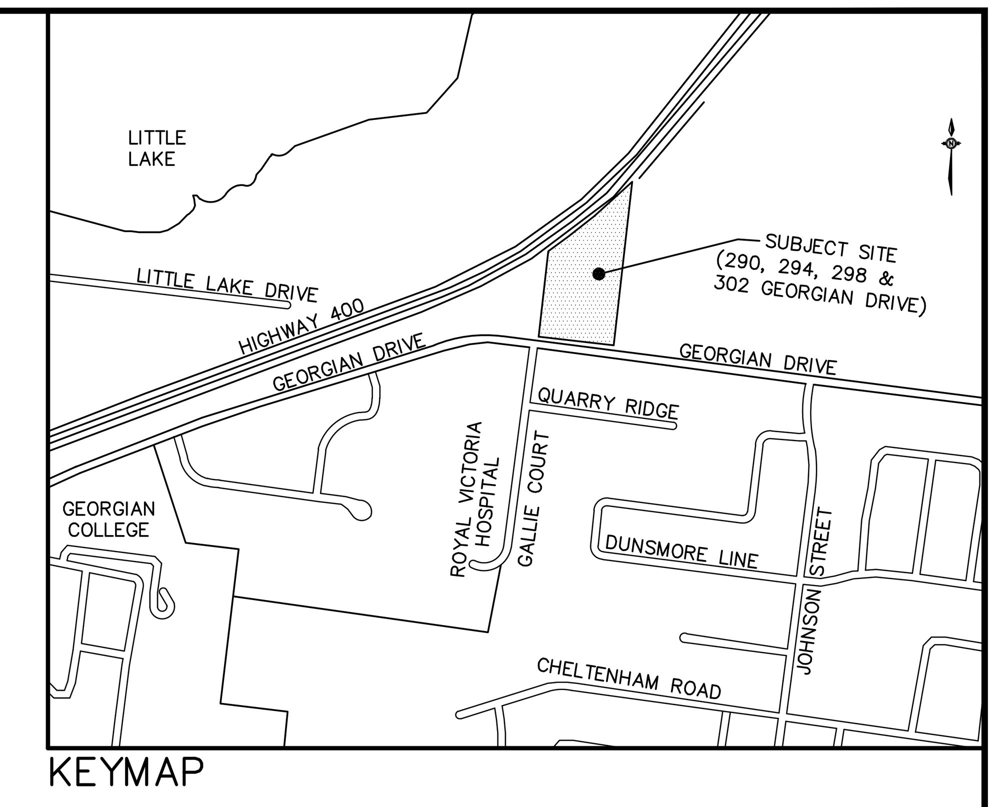
CONSTRUCTION ENTRANCE MAT	1.	Standardized Dimension Text	J.S.	05.10.28	APR'D: R.G.N. DATE: 04.03.16
					DRAWN: A.S.C SCALE: N.T.S
	NO.	REVISION	APR'D	DATE	BSD-23D



NOTES:

1. SEDIMENT CONTROL FENCE SHOULD BE ALIGNED WITH CONTOURS FOR SHEET OVERLAND FLOW.
2. SEDIMENT CONTROL FENCE IS TO BE LOCATED IN AREAS OF LOW SEDIMENT YIELD ON SLOPES THAT CONFORM TO MTO DRAINAGE MANUAL VOLUME 2 CHART F4-3C TOPOGRAPHIC FACTOR LS BASED ON SLOPE LENGTH AND GRADIENT.
3. SEDIMENT CONTROL FENCE SHALL BE INSTALLED WITH FILTER MEDIA FABRIC TIED INTO THE SOIL A MINIMUM OF 300 mm BY EITHER STATIC SLICING OR TRENCH METHODS WITH COMPACTION OF TRENCH MATERIAL MEETING 95% STANDARD PROCTOR MAXIMUM DRY DENSITY.
4. STEEL 'T' BAR POSTS ARE TO BE SPACED A MAXIMUM DISTANCE OF 2000 mm ON CENTER.
5. STRAW BALES TO BE PLACED END-TO-END CONTINUOUSLY BETWEEN SEDIMENT CONTROL FENCES.
6. FROZEN GROUND CONDITIONS REQUIRE FILTER FABRIC TO BE BACKFILLED IN TRENCH WITH CLEAR STONE.
7. GEO–TEXTILE SHALL BE A 270R – NON–WOVEN FILTER FABRIC OR EQUIVALENT A MINIMUM OF 300 mm AND WIRE FASTENED.
8. CONSTRUCTION OPERATIONS SHALL BE CARRIED OUT IN SUCH A MANNER THAT EROSION AND WATER POLLUTION IS MINIMIZED.
9. ALL DIMENSIONS IN MILLIMETRES UNLESS OTHERWISE SHOWN.

1	SWM GUIDELINES UPDATE	06.2016	DOUBLE ROW SEDIMENT CONTROL FENCE	DATE: 06.2016
				SCALE: NTS
NO.	REVISION	DATE		LSRCA ESC-5



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- BARRIER CURB
- SILT FENCE
- OVERLAND FLOW DIRECTION

NOTES FOR SEDIMENT & EROSION CONTROL

1. DISTURBED AREAS THAT HAVE FAILED TO HAVE STABLE GROUND COVER ESTABLISHED SHALL BE PROTECTED WITH A SILTATION CONTROL FENCE OR STRAW MULCH ETC. AND MAINTAINED BY THE CONTRACTOR UNTIL VEGETATION BECOMES ESTABLISHED IN THE SUBSEQUENT GROWING SEASON.
2. SILT FENCE SHALL BE PUT IN PLACE PRIOR TO AND MAINTAINED DURING ALL GRADING. SILT FENCE TO BE INSPECTED PRIOR TO COMMENCEMENT OF EARTH GRADING ACTIVITIES. SILT FENCE TO BE INSPECTED AND REPAIRED OR REPLACED IF DAMAGED AS DIRECTED BY THE SITE ADMINISTRATOR. SILT CONTROLS TO BE INSPECTED ON A REGULAR BASIS AND AFTER EVERY RAIN EVENT. INSTALLATION SHALL BE TO THE MANUFACTURER'S SUGGESTED SPECIFICATIONS.
3. THE CONTRACTOR SHALL BE PREPARED FOR UNEXPECTED CONDITIONS AND ACCORDINGLY HAVE STOCKPILED MATERIALS ON SITE FOR NECESSARY REPAIRS AS A RESULT OF FAILED OR INADEQUATE CONTROL MEASURES. ALL SEDIMENT AND EROSION CONTROL MEASURES SHALL BE INSPECTED AT LEAST ONCE A WEEK, AND AFTER EVERY RAINFALL EVENT.
4. MUD MATS AT ALL LOCATIONS WHERE CONSTRUCTION TRAFFIC ENTERS OR LEAVES THE SITE SHALL BE USED. MUD MATS TO CONSIST OF 300mm MIN.100mm TO 200mm CLEAR STONE HAVING DIMENSIONS 5.0m WIDE X 15.0m LONG.
5. CONTRACTOR SHALL OBTAIN A CURRENT COPY AND BECOME FAMILIAR WITH OPSS 577, CONSTRUCTION SPECIFICATION FOR TEMPORARY EROSION AND SEDIMENT CONTROL MEASURES AS WELL AS ALL APPLICABLE MUNICIPAL STANDARDS.
6. THE CONTRACTOR MAY CONSIDER ALTERNATIVE SEDIMENT AND EROSION CONTROL MEASURES. SUCH MEASURES SHOULD BE PRESENTED IN WRITING FOR APPROVAL OF THE SITE ADMINISTRATOR AND MUST BE APPROVED IN WRITING BY THE MUNICIPALITY AND CONSERVATION AUTHORITY.
7. THE TOPS OF ALL FILTER FABRIC MUST BE A MINIMUM OF 1.0 METRES ABOVE THE GROUND LEVEL AND ATTACHED TO THE FENCE WITH A CONTINUOUS STEEL WIRE. ALTERNATIVELY, THE FILTER FABRIC MUST BE FOLDED OVER THE TOP OF THE FENCE AND ATTACHED TO THE FENCE WITH WIRE LOOPED THROUGH THE FABRIC ON BOTH SIDES OF THE FENCE. FILTER FABRIC IS TO BE TERRAFIX 270R OR EQUIVALENT.
8. ALL DISTURBED GROUND LEFT INACTIVE SHALL BE STABILIZED BY SEEDING, SODDING, MULCHING, OR COVERING OR OTHER EQUIVALENT CONTROL MEASURES. THIS PERIOD OF INACTIVITY SHALL BE AT THE DISCRETION OF THE CITY OF BARRIE MANAGER OF ENGINEERING BUT SHALL NOT EXCEED THIRTY DAYS OR SUCH LONGER PERIOD DEEMED ADVISABLE BY THE CITY OF BARRIE MANAGER OF ENGINEERING.
9. CONTRACTOR SHALL INSTALL AND MAINTAIN CATCHBASIN SEDIMENT BARRIERS THROUGHOUT THE SITE DURING ALL CONSTRUCTION ACTIVITIES IN ORDER TO TRAP SEDIMENT.

SEQUENCE OF CONSTRUCTION

1. ENGINEER TO BE NOTIFIED PRIOR TO INITIATION OF ANY ON SITE WORKS.
2. SILT FENCE AND TO BE INSTALLED PRIOR TO THE COMMENCEMENT OF ANY WORKS ONSITE.
3. VEGETATION REMOVAL MAY COMMENCE AFTER ALL SILT FENCE IS INSTALLED AND APPROVED BY THE ENGINEER.
4. COMMENCE WITH EARTH EXCAVATION AND SITE SERVICING (TO BE REMOVED FROM SITE).
5. EROSION CONTROL MEASURES TO BE MAINTAINED AS DIRECTED BY THE ENGINEER DURING THE CONSTRUCTION PERIOD. ADDITIONAL CONTROL MEASURES MAY BE REQUIRED AT THE DISCRETION OF THE ENGINEER.
6. ALL DISTURBED GROUND LEFT INACTIVE FOR MORE THAN 30 DAYS SHALL BE STABILIZED WITH SEED, SOD, MULCH OR OTHER ADEQUATE COVERING, AS INSTRUCTED BY THE ENGINEER.

I HAVE REVIEWED THE PLANS FOR THE CONSTRUCTION OF THE GEORGIAN APPARTMENTS LOCATED AT 290, 294, 298 & 302 GEORGIAN DRIVE AND HAVE PREPARED THIS PLAN TO INDICATE THE COMPATIBILITY OF THE PROPOSAL TO EXISTING ADJACENT PROPERTIES AND MUNICIPAL SERVICES. IT IS MY BELIEF THAT ADHERENCE TO THE PROPOSED GRADES AS SHOWN WILL PRODUCE ADEQUATE SURFACE DRAINAGE AND PROPER FACILITY OF MUNICIPAL SERVICES WITHOUT ANY DETRIMENTAL EFFECT TO THE EXISTING DRAINAGE PATTERNS OR ADJACENT PROPERTIES.

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LICENSED PROFESSIONAL ENGINEER G. M. PEARSON 100061986 PROVINCE OF ONTARIO

THE GEORGIAN APARTMENTS
290,294,298,&302 GEORGIAN DRIVE
BARRIE, ONTARIO

EROSION PROTECTION AND REMOVALS PLAN

PEARSON ENGINEERING
PEARSONENG.COM PH. 705.719.4785

DESIGNED BY	PCO	HORIZ SCALE	1:250	PROJECT #	18037
DRAWN BY	MKRW	VERT SCALE		DRAWING #	EPR–1
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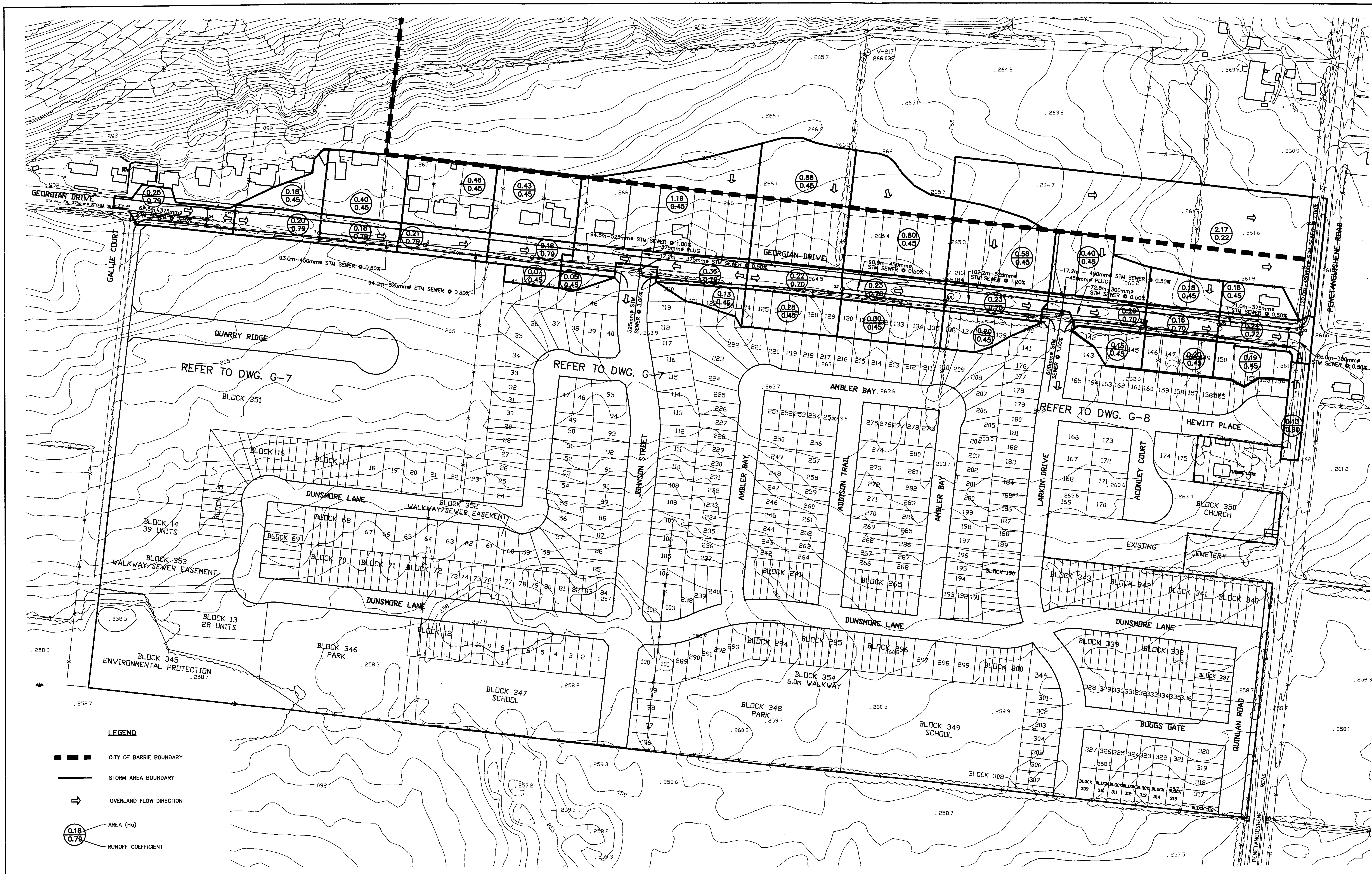

Notes	BENCHMARK:	NO.	REVISIONS	DATE	INITIAL
		2	AS CONSTRUCTED	SEP. 2008	J.U
		1	2ND SUBMISSION COMMENTS	DEC. 2000	D.R.C.

Approved

Approved
CITY OF BARRIE
APPROVED
DATE . . . .
DIRECTOR OF MUNICIPAL WORKS

REGISTERED PROFESSIONAL ENGINEER
R. J. LACKEY
03-10-02
PROVINCE OF ONTARIO

DUNSMORE SUBDIVISION
CRISDAWN CONSTRUCTION INC.

GEORGIAN DRIVE AND EXTERNAL
STORM DRAINAGE AREAS

JONES CONSULTING GROUP LTD
PLANNERS, ENGINEERS & SURVEYORS

DESIGN	M.T.	SCALE: 1:1500	DATE NOVEMBER 2000
DRAWN	M.T.	PRA-20148	DWG. № G-10
CHECKED	M.R.		

2000-086-012