FUNCTIONAL SERVICING AND STORMWATER MANAGEMENT REPORT

RESIDENTIAL DEVELOPMENT

4 STOREY RESIDENTIAL BUILDING

15, 21, 27 SHORE STREET
TOWN OF CALEDON

APPLICATION FILE NUMBERS:
TOWN: PRE-2023-0109
REGION: PRE-23-109B

Project No.: 24-0002CA **September 2025**

Revision	*Date*	*Description*
0	*May 2024*	*First Submission*
1	*September 2025*	*Second Submission*
2		

TOWN OF CALEDON
PLANNING
RECEIVED

September 4th, 2025

Urbanworks Engineering Corporation
1945 Dundas St E | Mississauga, Ontario | L4X 2T8
Unit 200 | Email: general@urbanworkseng.com

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1.0 INTRODUCTION

1.1 Study Objectives and Location

This Functional Servicing and Stormwater Management Report has been prepared in support of a 4-storey apartment building with a basement on 15, 21, 27 Shore Street, located west of Queen Street South (Highway #50) and north of Elwood Drive West. The site can be legally described as being located within Registered Plan BOL-7, with the following parts:

- Municipal No. 27 – PIN 14322-0319(LT)
- Municipal No. 21 – PIN 14322-0320(LT)
- Municipal No. 15 – PIN 14322-0321 (LT)

The subject property is bounded by Shore Street to the South, Oak Street to the West, Queen Street South to the East, and William Street to the North. A site location plan is provided in Figure 1-1.

The following report provided information regarding site servicing and stormwater management for the subject development while ensuring compatibility with surrounding lands. The report will also address concerns and comments raised by Regulatory Agencies (i.e Town of Caledon (Town), Region of Peel (Region), Toronto & Region Conservation Authority (TRCA), Ministry of Environment (MOE)

1.2 Existing Conditions

The site is roughly a rectangular shape covering a total area of 0.21 ha. Currently the site is occupied by 3 single-family residential homes, one for each municipal address mentioned above. The current properties consist of asphalt driveways, landscaping features, sheds, grassed area, and minor retaining walls internal to the property.

In general, vehicle access to the site is provided by existing driveway entrances fronting Shore Street. The subject property only has fronting on Shore Street and Shore Street is bounded by Oak Street to the West and Highway 50 to the east.

The subject property is bounded by residential lands to the north adjacent to the property, residential lands to the west across Oak Street, commercial lands to the east adjacent to the property, and commercial lands to the south across Shore Street.

Shore Street is currently an un-urbanized 6.7 m wide road within an approximately 15.0 m right-of-way. Shore Street has a roadside ditch and driveway culverts on the north side and no roadside ditch on the south side. No sidewalk exists on Shore Street on either side. The nearest sidewalk is where Shore Street intersects Highway 50.

The existing property is currently split drainage with the front half of the lots draining to the roadside ditches uncontrolled and the back yards drainage towards the north property line and being conveyed from there by existing means.

1.3 Proposed Development

The proposed development will consist of development of the subject property (Internal Works) and urbanization of Shore Street (External Works).

1.3.1 Internal Works

The internal works will involve removing all internal aboveground structures, underground structures, pavement structures, retaining walls internal to the property, fencing, underground servicing, landscaping, accessory structures, and any other underground utilities as required.

Once all removed, the proposed development will consist of construction a 4-storey residential apartment building (19 residential units) with a basement, a new surface parking lot, retaining walls, storm sewer infrastructure, sanitary servicing, and watermain servicing, and landscaping features.

The main vehicle access to the proposed development will be provided by two new 6.0 m wide driveways fronting Shore Street. Pedestrian access will be provided by new sidewalk construction as part of external works. Please refer to Site Plan prepared by FC Architects in **Appendix A** for more details.

Pedestrian access to the proposed development will be provided by the new 1.5 m concrete sidewalk in the Shore Street right-of-way that is part of the External works described in Section 1.3.2 below.

1.3.2 External Works

The external works will consist of urbanizing Shore Street in its entirety to be a curb and guttered road with its own minor storm sewer system of catch basins and storm sewer mains. The new road will be as per Town Standard Drawing 202. The new road will have an edge of pavement width of 7.9 m and come with a new 1.5 m wide concrete sidewalk on the north side along the subject property line connecting to the existing sidewalk on Highway 50.

A new storm sewer main will be installed to convey the storm run-off from the new right-of-way and to convey the controlled storm run-off from the subject property.

It is to be noted that the external works will involve a 1.5 m widening of the right-of-way along the north property line. As such the ultimate property line is to be 1.5 m away from the existing property line.

The limit of the urbanization work will be from Oak Street to the existing curb and gutter on Highway 50. As shown in the Site Grading Plan in **Appendix F**

1.4 Proposed Design Populations

Based on the site plan, the site is to remain entirely residential. The proposed development design population as per Region of Peel criteria is provided below in Table 1-1. The total proposed design population considered for 15, 21, 27 Shore Street is 52 persons.

Table 1-1: Population Estimate

Land Use	Site Area**	Population Density (Large)	Population Density (Small)	Large Unit	Small Unit	Design Population*
Residential	0.21	3.1 / large unit	1.7 / small unit	14	5	52
Total Design Population	**0.21**	3.1 / large unit	1.7 / small unit	**14**	**5**	**52**

*Population and unit estimates are based on the population densities set out by the Region of Peel - Linear Waste Water Standards (2023) - Section 2.1.2. The resulting populations have been used for engineering design capacity purposes only and therefore may not be consistent with proposed planning populations for the development.
**Unit Counts based on the latest Site Plan.

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LEGEND

SITE BOUNDARY

FIGURE 1-1
SITE LOCATION PLAN

4-STOREY RESIDENTIAL BUILDING
15, 21, 27 SHORE STREET
CALEDON, ON

24-0002CA	Sept. 2025	N.T.S.

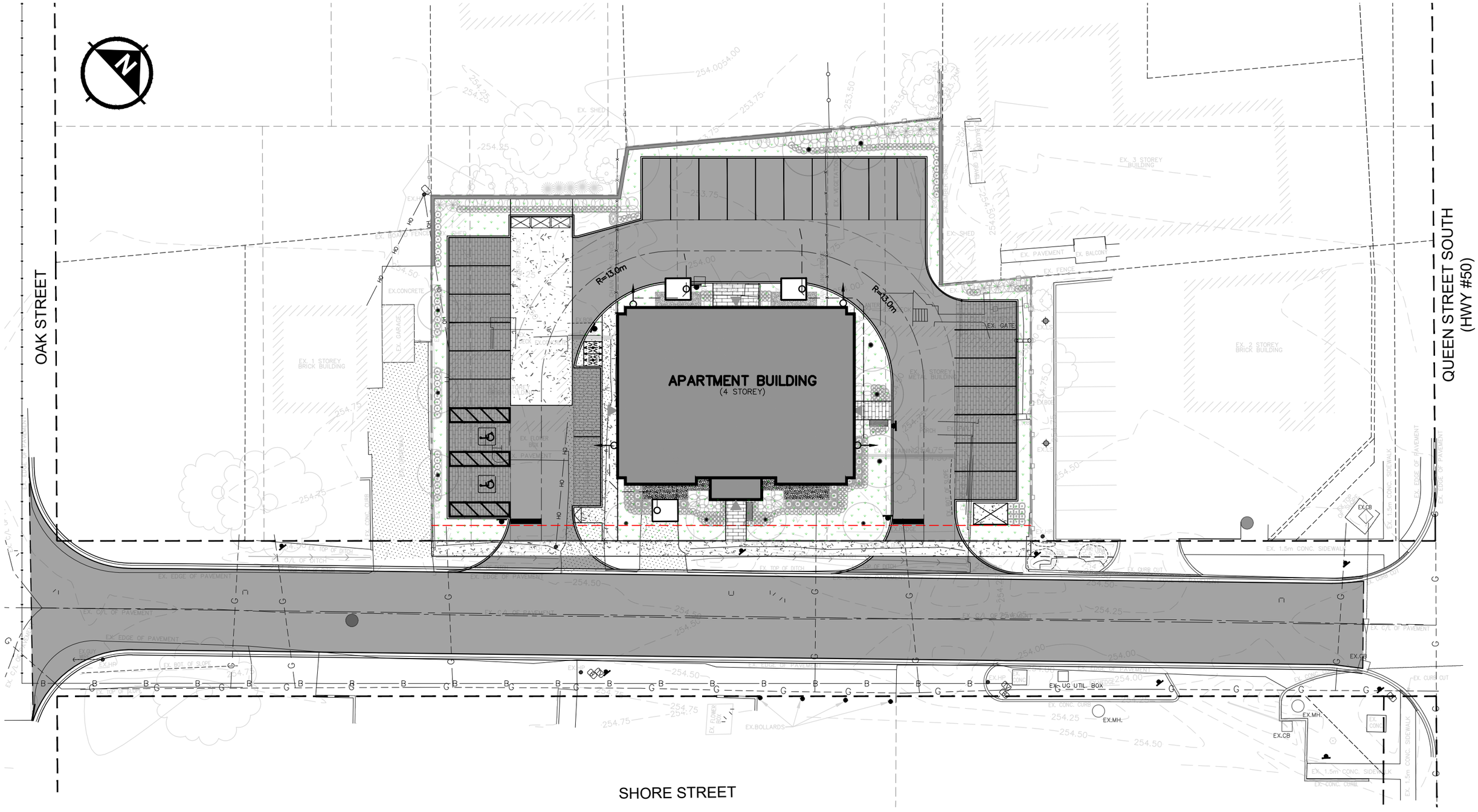

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FIGURE 1-2
SITE DEVELOPMENT PLAN

4-STOREY RESIDENTIAL BUILDING
15, 21, 27 SHORE STREET
CALEDON, ON

24-0001BR	Sept. 2025	SCALE 1:350

1.5 Background References

The following material has been reviewed during the preparation of this report:

- Guido Papa Surveying, *Topographic Survey,* dated August 11, 2023
- Fausto Cortese Architects., Site Plan dated August 2024
- **Toronto and Region Conservation Authority**, *Stormwater Management Criteria*, August 2012.
- **Toronto and Region Conservation Authority/Credit Valley Conservation**, *Low Impact Development Stormwater Management Planning and Design Guide*, 2010.
- Ministry of the Environment, *Stormwater Management Planning and Design Manual*, dated March 2003.
- **Region of Peel**, *Linear Waste Water Standards,* dated March 2024
- **Region of Peel,** *Linear Infrastructure – Watermain Design Criteria,* dated June 2010
- **Town of Caledon –** Development Standards Manual, Version 5.0, dated 2019.

2.0 Stormwater Management

2.1 Existing Conditions

2.1.1 Internal Works Topography and Drainage

In general, the existing site grading can be described as having split drainage. The front half of the existing lots drain via overland sheet flow into the roadside ditch along the north side of Shore Street. Meanwhile the backyards drain via overland sheet flow towards the north property line and continue to be conveyed northward away from the subject property limits. All overland flow is ultimately captured by storm inlets at the intersection of Shore Street and Highway 50, as shown in Engineering Drawings provided in Appendix F. The Shore Street major overland flow route drains east towards Highway 50 and subsequently southwards along the highway.

The site is relatively flat with an average slope of 2.4% for the rear of existing lots, and the northeast corner of the property. An average slope of 1.6% exists from the front of the existing houses towards Shore Street.

Based on available topographic information, the existing site drainage area has been delineated and presented in Figure 2-1.

2.1.2 Existing Storm Drainage Infrastructure

As-builts drawings were obtained through Region of Peel and were used to identify the following stormwater infrastructure in proximity of the site.

- There is no existing storm sewer infrastructure within the Shore Street right-of-way
- A 375 mm diameter concrete storm sewer that extends from Highway 50 towards Shore Street and terminates at CBMH RR050-347

2.1.3 External Works Topography and Drainage

Shore Street is a relatively flat road that is graded at approximately 0.75% to the east towards Highway 50. There is no existing storm sewer infrastructure within Shore Street, and all storm run-off drains via overland sheet flow into the CBMH RR050-347 and CB RR050-247 at the northwest corner of Shore Street and Highway 50 intersection.

2.1.4 Soil Conditions

A Geotechnical Report was prepared by Soil Engineers Ltd, dated July 2024. As part of the report

/ investigation a series of boreholes and groundwater monitoring wells were installed on-site. Based on the results of the monitoring period, groundwater elevations were found to range between 1.1 metres below ground surface (mbgs) to 5.5 mbgs consistent with elevations 253.0 m to 248.6 m depending on the monitoring well location. Furthermore, hydraulic in-situ values and percolation times were calculated for the soils screened. The coefficient of permeability was calculated to be $1x10^{-7}$ cm / s. The percolation time for the soils within the subject property limit was 100 min / cm which translate to a drawdown rate of 6 mm / hour.

Based on field investigations, soil types on site ranged from earth fill near the surface to primarily silty clay / silty clay till across the entire site. Please refer to the geotechnical report in **Appendix B** for more details.

2.2 Stormwater Management Design Criteria

The stormwater management design criteria applicable to the proposed development was established through a review of regulatory agency design standards. The relevant stormwater management design criteria and regulatory bodies are summarized in this section.

Town of Caledon Criteria

- Rainfall intensities shall be derived from the IDF curves as outlined in Town Standard No. 103 with information as presented in the following table:

Table 2-1: Town of Caledon IDF Curve Parameters

RETURN PERIOD	A	B	C
2-Year	1070	0.8759	7.85
5-Year	1593	0.8789	11
10-Year	2221	0.9080	12
25-Year	3158	0.9335	15
50-Year	3886	0.9495	16
100-Year	4688	0.9624	17

**Values as per Town Std No 103.*

The average rainfall intensity shall be calculated using the equation:

$$(i = A * B / (t + c))$$

Where:

i = rainfall intensity (mm/hr)
t = Time of concentration (hours)

- Quantity Control - Post-development peak flow rates at the proposed discharge location shall be limited to the 5-year pre-development peak flow, up to and including the 100-year return event.

- Quality Control – 80% (*enhanced)* TSS removal required for water quality protection to be provided in accordance with MECP, and Town requirements.

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LEGEND

- DEVELOPMENT BOUNDARY
- DRAINAGE BOUNDARY
- EXISTING STM. SEWER
- EMERGENCY OVERLAND FLOW ROUTE
- PERVIOUS AREA
- IMPERVIOUS AREA

DRAINAGE AREA
AREA I.D.
100 | 0.50
1.00ha
RUNOFF COEFF.
DRAINAGE AREA

FIGURE 2-1
EX. SITE STORM DRAINAGE PLAN

4-STOREY RESIDENTIAL BUILDING
15, 21, 27 SHORE STREET
CALEDON, ON

24-0001BR	Sept. 2025	SCALE 1:350

2.3 Proposed Site Grading and Drainage

2.3.1 Internal Works

The predevelopment drainage conditions are to be altered to have the site be self contained and reduce the run-off being directed to the north property line.

To achieve a self-contained site the north property line grades will be raised via the use of retaining walls. All storm runoffs will be directed to the minor storm conveyance system within the site. Major overland flow will drain towards Shore Street via the easternmost driveway entrance in the case of an emergency event, or storm larger than the 100-year event. The site promotes a major flow route with a maximum ponding depth of 0.30 m.

The controlled drainage areas on site will have runoff collected via catch basins or catch basin manholes to a manhole with an orifice plate for quantity control. Quality treatment will then be provided by an on-site jellyfish filtration unit draining to the site control manhole before ultimately releasing site flows to the proposed storm sewer system within Shore Street, provided as part of proposed external works.

The roof runoff from proposed building will drain underground into rainwater harvesting cisterns (Graf Platin 3000 L x 3) and in the event the cisterns are full, the roof run-off will drain into the control manhole via an overflow pipe. All downspouts will come equipped with a leaf filter, an overflow pipe, and aboveground splash pad as a safeguard.

2.3.2 External Works

As part of the urbanization of Shore Street a minor storm sewer system is being proposed with-in the Shore Street right-of-way to convey minor flows. The storm sewers are proposed to be sized to convey the maximum of the 5-year storm event as per Town Standards, as well as the controlled flows from subject property. The proposed storm sewers will connect to the existing CBMH RR050-347 and shall not exceed the size of the existing downstream sewer system.

The Shore Street major overland flow route for is to maintain its existing drainage route towards Highway 50. The proposed grading for the urbanization work will provide a max ponding depth of 0.30 m within proposed ponding areas.

Please refer to Figure 2-3 for the proposed drainage area plan for the proposed storm sewer within the Shore Street right-of-way

2.4 Proposed Stormwater Management Plan

The proposed stormwater management system will be designed in accordance with Town of

Caledon and TRCA guidelines. The intent of the design is to maintain existing drainage conditions to the extent possible, while adhering to the quantity control criteria set out by both the aforementioned criteria.

During the design process, low impact development (LID) measures were evaluated as part of the storm water management strategy; the TRCA LID SWM planning and Design Guide was used as a reference. The following LID measure is being utilized:

- Surface Storage: All required quantity storage to attenuate the 100-year post-development to the 5-year pre-development flow rate will be provided via surface storage. The surface storage will have a maximum ponding depth of 0.30 m, and the limits of the surface storage are shown on the Site Grading Plan provided in **Appendix F.**

- Rainwater Harvesting: To provide the site erosion control volume (i.e. 5 mm retention on site) rainwater harvesting is proposed. All roof-runoff will drain into the rainwater harvesting cisterns before being re-used on site.

Other LID options such as, green roofs, permeable pavement, and swales were not utilized given the development design and spatial constraints.

2.5 Internal Works Water Quantity Control

In accordance with the Town design criteria, it is proposed to control site post development peak flows to match pre-development levels for all storm events up to and including the 100-year event.

The existing site peak flows were estimated for each return period from the 2-year to 100-year. These calculations were performed using the Modified Rational Method in conjunction with the Town IDF curves as per Town Std No. 103. This was done in order to establish the site's maximum permissible post-development rate into the proposed storm sewer system.

2.5.1 Pre-development Flows

The existing drainage pattern within the proposed development limits is characterized by three (3) drainage area: Area 101, 102, and 103

Area 101 consists of 3 detached single-family homes, accessory structures, surface asphalt parking, and landscaped area. Area 101 represents the area that will continue to be self-contained in the proposed condition.

Area 102 consists of asphalt parking, driveway and grassed/planted landscaped area. This area

will continue to drain towards the Shore Street right-of-way in post-development conditions.

Area 103 consists of approximately 0.091 ha of entirely landscaped backyard area which drains via sheet flow towards the northern property line. In the post-development condition, the area draining towards the north property line will be reduced in order to promote self-contained drainage to the extent possible.

Please see Figure 2-1 which depicts the pre-development drainage areas. A summary of the pre-development land cover is provided below in Table 2-2.

Table 2-2: Pre-development land use Summary

Surface	Area 101 (m^2)	Area 102 (m^2)	Area 103 (m^2)	Total Coverage
Asphalt	167.82	19.30	0	9.1%
Roof	369.74	0	0	17.9%
Hardscape	108.18	0	0	5.2%
Landscape	1306.01	37.0	58.83	67.8%
% Impervious	33%	34%	0%	32%
Runoff Coefficient	0.47	0.47	0.25	0.46

The total imperviousness of the existing site was calculated to be 32% and the corresponding runoff coefficient was calculated to be 0.46 based on Town of Caledon Engineering Standards. For more detailed calculations please see **Appendix E**.

IDF parameters from the Town Standard No. 103 were used to calculate the existing site's peak runoff. These IDF curves are a function of the time of concentration using the Town minimum time of concentration of 10 minutes for the development area.

The pre-development peak runoff flows determined for Catchment 101 based on the above model parameters are summarized in Table 2-3.

Table 2-3: Pre-development Peak Runoff Rate

Return Period	Intensity (mm/hr)	Area 101 – Runoff Aea = 0.195ha (C=0.46) (L/s)	Area 102 – Runoff Aea = 0.006 ha (C=0.47) (L/s)	Area 103 – Runoff Aea = 0.006 ha (C=0.25) (L/s)
2-yr	52.5	13.1	0.4	0.2

5-yr	66.7	16.6	0.5	0.3
10-yr	91.7	22.8	0.7	0.4
25-yr	117.9	29.3	0.9	0.5
50-yr	141.9	35.3	1.1	0.6
100-yr	167.1	**41.6**	**1.2**	**0.7**

2.5.2 Post Development Flows

The proposed post-development drainage pattern within the proposed development is characterized by three (3) drainage areas. Area 201, 202, and 203.

Area 201 consists of the controlled 4-storey residential building, surface parking, landscaped areas, concrete walkways, and retaining walls.

Area 202 consists of a small amount of gravel area and a majority of landscaped planting area. Due to grading constraints along the frontage of the site, it is proposed to drain uncontrolled to the Shore Street right-of-way.

Area 203 consists of landscaped planting area. This area currently drains uncontrolled to the north property line. It is proposed to continue to drain uncontrolled to the north property line due to grading constraints, particularly as a way to mitigate impact on adjacent trees along the site boundary.

The proposed development will be serviced by a minor storm system that has been sized to the capture and control site runoff to the 5-year pre-development level before draining into the proposed storm sewer within Shore Street.

A summary of the site's post-development land cover is provided below in Table 2-4.

Table 2-4: Post-development land use summary

Surface	Area 201 (m^2)	Area 202 (m^2)	Area 203 (m^2)	Total Coverage
Asphalt	1055.04	0	0	51.0%
Roof	423.43	0	0	20.5%
Hardscape	114.8	8.0	0	5.9%
Landscape	358.74	48.60	58.83	22.6%
% Impervious	82%	14%	0%	77%
Runoff Coefficient	0.78	0.34	0.25	0.75

The total imperviousness of the proposed development was estimated to be 77% with a corresponding runoff coefficient of 0.75 based on Town of Caledon Engineering Standards. For more detailed calculations please see **Appendix E**. Final site land cover estimates, alongside their impact on the proposed SWM design, will be confirmed at the detailed design stage.

IDF parameters from the Town Standard No. 103 were used to calculate the existing site's peak runoff. These IDF curves are a function of the time of concentration; the minimum time of concentration of 10 minutes was used for the development area.

The post-development peak runoff flows determined for all drainage areas based on the above parameters are summarized in **Table 2-5.**

Table 2-5: Post-development Peak Runoff Rate

Return Period	Intensity	Area 201 – Runoff Aea = 0.195ha (C=0.78)	Area 202 – Runoff Aea = 0.006 ha (C=0.34)	Area 201 – Runoff Aea = 0.006 ha (C=0.25)
	(mm/hr)	(L/s)	(L/s)	(L/s)
2-yr	52.5	13.1	0.3	0.2
5-yr	66.7	16.6	0.4	0.3
10-yr	91.7	22.8	0.5	0.4
25-yr	117.9	29.3	0.6	0.5
50-yr	141.9	35.3	0.8	0.6
100-yr	167.1	**41.6**	**0.9**	**0.7**

As shown in **Error! Reference source not found.**5, the post-development peak flows are a nticipated to exceed the target 5-year pre-development peak flow, as presented in Table 2-3. As a result, surface storage paired with a control orifice is proposed to attenuate all storm events up to and including the 100-year post development level to the target 5-year pre-development peak flow.

2.5.3 Water Quantity Control

Based on the pre-development flows outlined in Table 2-3, all storm events up to and including the 100-year post development flow rate will be controlled to the 5-year predevelopment flow rate, as summarised in Table 2-6 below.

Table 2-6: Allowable Release Rate

Target Pre-Development Peak Flow (5-year) (L/s)	**100-year Post-Development Uncontrolled Flows (L/s)**	**Allowable Post Development Release Rate (L/s)**
16.6	1.6 (Areas 202 + 203)	15.0

In order to achieve the aforementioned quantity control objectives, Catchment 201 will need to have runoff flows from the 100-year event controlled to a release rate of 15.0 L/s. This will ensure that the total of the site controlled and uncontrolled flows will remain within the 5-year pre-development peak flow rate to the proposed sewer along Shore Street.

In order to attenuate the 100-year post development flow to the required control rate, an orifice plate will be utilized and placed upstream of the site control manhole. An orifice diameter of 78 mm is estimated to be required and will require an associated stormwater storage volume of approximately 48.2 cubic metres. This volume is proposed to be provided via aboveground surface storage within the proposed parking areas.

Table 2-7 summarizes the site's key proposed quantity control design features. Please see **Appendix E** for detailed calculations. The final required orifice sizing, storage requirements and design will be confirmed at the detailed design stage.

Table 2-7: Orifice Control and Required Storage

Post Development Flow (100-year) (L/s)	Allowable Post Development Release Rate (L/s)	**Orifice Diameter (mm)**	**Actual Provided Post-development flow (L/s)**	**Required Water Quantity Storage (m3)**
71.6	15.0	78	14.9	46.4

Please see Table 2-8 below for the provided preliminary water quantity storage volumes summary. The final provided storage volumes will be confirmed at the detailed design stage. Supporting calculations for the site's quantity control design are provided in **Appendix E**. Please refer to the Functional Site Servicing Plan in **Appendix F** for more details.

Table 2-8: Preliminary Water Quantity Storage Summary

Ponding Location	Top of Grate Elev.	Max Ponding Elev.	Total Surface Storage Area(m^2)	Provided Water Quantity Storage (m^3)	Required Water Quantity Storage (m^3)
CB1	254.15	254.23	194.50	5.2	-
CBMH 1	254.00	254.23	295.04	22.6	-
CBMH 2 (w/ Orifice)	253.95	254.23	243.83	22.8	-
Total	-	254.23	733.37	50.56	46.4

2.5.4 Extended Detention and Erosion

As per TRCA criteria, urban developments are to provide a minimum extended detention of the 5mm rainfall event over the entire site.

Given the total post development impervious site area of 0.16 ha an extended detention volume of 8.0 cubic metres will be required for the 5 mm rainfall event.

Extended detention will be provided by the underground rainwater harvesting cisterns that will only collect roof-runoff. Table 2-9 summarises the extended detention volume within the cisterns. Please refer to **Appendix E** for detailed calculations.

Table 2-9: Summary of Extended Detention

Drainage Area	Required Detention Storage (m^3)	Provided Extended Detention Storage (m^3)
0.16 ha	8.0	9.0

The water stored within the rainwater cistern will be re-used on site in one of the two ways.

1. Landscaping / irrigation
2. Grey-water re-use

The details of the water re-use application will be determined during detailed design.

As the Geotechnical report identified silty clay soils with a high ground water table and a draw down rate of 6 mm / hour. As a result of the low infiltration rates exhibited on-site, infiltration measures were not considered possible as part of the subject development.

2.6 External Works Water Quantity Plan

As the existing storm run-off is conveyed uncontrolled to the inlets at CBMH RR050-347 and CB RR050-247 and the major overland flow drains towards the east to the Highway 50 right-of-way. No quantity controls are proposed within the right-of-way storm sewer and the major overland flow route will continue to drain towards Highway 50. Please refer to Figure 2-3 for the proposed Shore Street drainage area plan.

The proposed Shore Street storm sewer has been sized to convey the 5-year storm event.

A storm sewer design sheet for the proposed Shore Street sewer has been provided in **Appendix E** for reference.

2.7 Internal Works Water Quality Control

As per the Town of Caledon design criteria, site runoff must be treated to meet an enhanced level of quality control (removal of 80% TSS). The proposed site will consist of a combination of roof, driveable paved surfaces, pervious landscaped areas, and concrete walkways.

The proposed treatment to obtain the required water quality control consists of a Jellyfish (JF4-1-1) filtration unit sized to provide the 80% TSS removal for the entire site before being discharged into the proposed storm sewer with Shore Street right-of-way.

2.8 External Works Water Quality Control

As the existing storm run-off is conveyed untreated to the inlets at CBMH RR050-347 and CB RR050-247 and the major overland flow drains towards the east to the Highway 50 right-of-way. No quality controlled are proposed the right-of-way storm sewer and the major overland flow route will continue to drain towards highway 50. Please refer to Figure 2-3 for the Shore Street drainage area plan.

OAK STREET

QUEEN STREET SOUTH (HWY #50)

203 | 0.25
0.006ha

UNDERGROUND RAINWATER HARVESTING CISTERNS

APARTMENT BUILDING
(4 STOREY)

201 | 0.78
0.20ha

JELLYFISH OGS

202 | 0.34
0.006ha

R=13.0m

R=13.0m

EX. 1 STOREY BRICK BUILDING

EX. GARAGE

EX. SHED

EX. 3 STOREY BUILDING

EX. 2 STOREY BRICK BUILDING

EX. PAVEMENT

EX. BALCONY

EX. FENCE

EX. BOARD FENCE

EX.CONCRETE

EX. CURB STOP (TYP.)

EX. CULVERT

EX. VB

EX. C/L OF PAVEMENT

EX. EDGE OF PAVEMENT

EX. TOP OF DITCH

EX. BOT. OF SLOPE

EX. 1.5m CONC. SIDEWALK

EX. CURB CUT

EX.CB

EX. UG UTIL. BOX

EX.MH.

EX.BOLLARDS

EX. FLOWER BOX

EX.GUY

EX.HP

SHORE STREET

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LEGEND

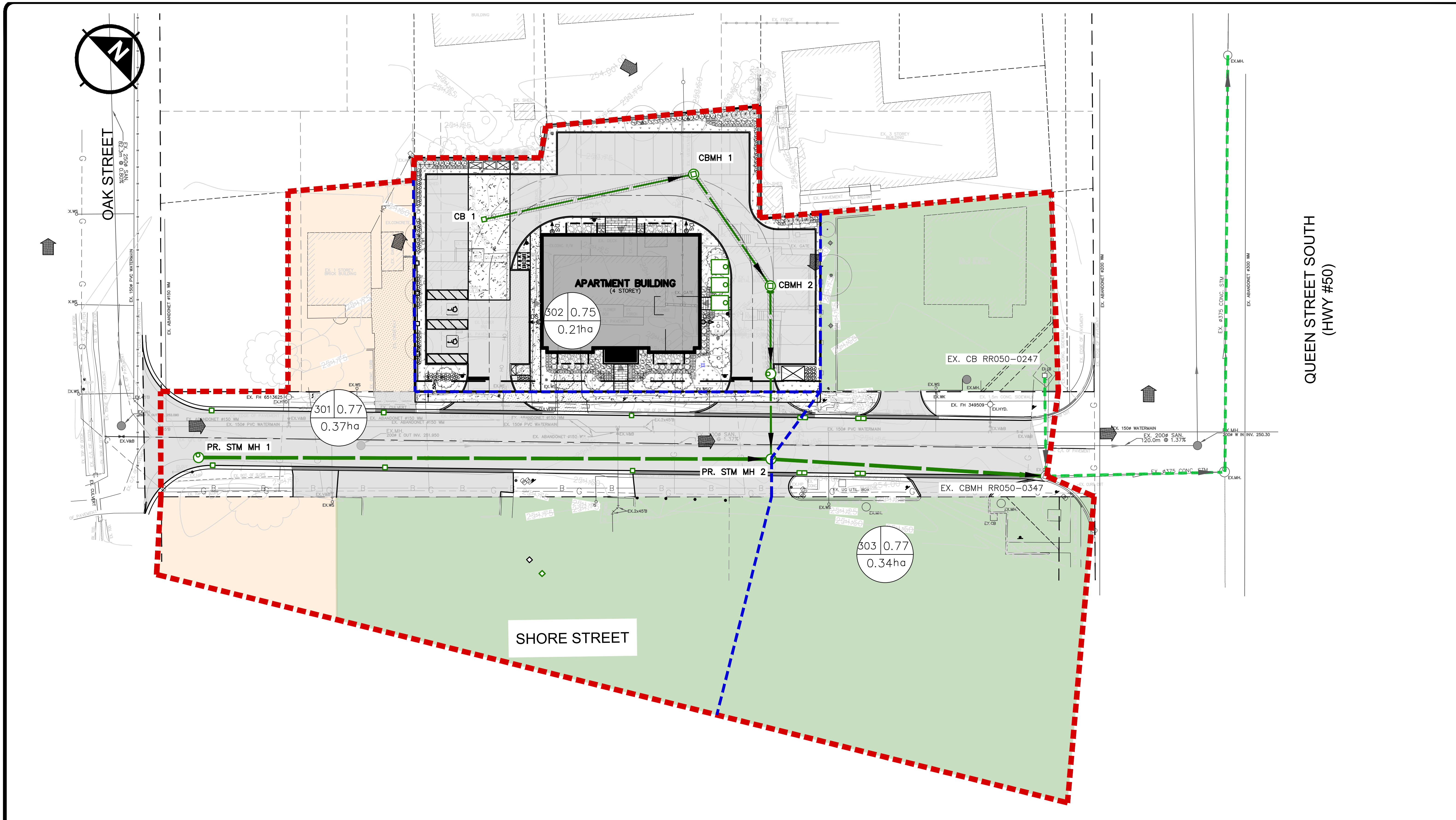
- DEVELOPMENT BOUNDARY
- DRAINAGE BOUNDARY
- EXISTING STM. SEWER
- PROPOSED STM. SEWER
- EMERGENCY OVERLAND FLOW ROUTE
- PERVIOUS AREA
- IMPERVIOUS AREA

DRAINAGE AREA
AREA I.D.
100 | 0.50
1.00ha
RUNOFF COEFF.
DRAINAGE AREA

FIGURE 2-2
PR. SITE STORM DRAINAGE PLAN

4-STOREY RESIDENTIAL BUILDING
15, 21, 27 SHORE STREET
CALEDON, ON

24-0002BR	Sept. 2025	SCALE 1:350



LEGEND

- DEVELOPMENT BOUNDARY
- DRAINAGE BOUNDARY
- EXISTING STM. SEWER
- PROPOSED STM. SEWER
- EMERGENCY OVERLAND FLOW ROUTE
- RESIDENTIAL AREA
- COMMERCIAL AREA

DRAINAGE AREA
AREA I.D. — 100
RUNOFF COEFF. — 0.50
DRAINAGE AREA — 1.00ha

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1945 Dundas Street E., Unit 200, Mississauga, ON, L4X 2T8
general@urbanworkseng.com | www.urbanworkseng.com | 905-901-0224

FIGURE 2-3
SHORE STREET STORM DRAINAGE PLAN

4-STOREY RESIDENTIAL BUILDING
15, 21, 27 SHORE STREET
CALEDON, ON

24-0001BR	Sept. 2025	SCALE 1:500

3.0 Sanitary Servicing

3.1 Existing Sanitary Sewers

Region records were used to identify any existing Region wastewater infrastructure adjacent to the proposed development.

- A 200 mm diameter Sanitary Sewer within the Shore Street right-of-way. As shown in Region As-built Drawing #22534-D
- The 200 mm diameter Sanitary Sewer in Shore Street connects to a 300 mm diameter Sanitary Sewer within the Highway 50 right-of-way.

Region As-built Drawing #22534-D has been provided in **Appendix A** for reference.

3.2 Proposed Sanitary Servicing

Sanitary servicing for the proposed development will be provided by connection a new 150 mm diameter sanitary service lateral to the existing 200 mm diameter sanitary sewer within the Shore Street right-of-way. The new sanitary service will come with sanitary sampling manhole at the ultimate property line as per Region Standard Drawing 2-5-3. The proposed 150 mm diameter sanitary service will utilize a minimum slope of 2%

Refer to **Drawing SS-01** for the Site Servicing Plan in **Appendix F** for the proposed sanitary servicing design.

3.3 Design Criteria

The sanitary design parameters, as outlined in Table 3-1, for the proposed development are based on the municipal design criteria as outlined in *Region of Peel, Linear Wastewater Standards*, dated March 2023:

Table 3-1: Proposed Sanitary Design Parameters

Parameter	Value
Population Density (Large Apartment)	**3.1 person / large unit**
Population Density (Small Apartment)	**1.7 person / small unit**
Large Unit Count	**14**
Small Unit Count	**5**
Site Area	**0.21 ha**
Total Population	52
Unit Institutional Flow Rates	270 L/cap/day
Infiltration Rate	0.26 L/s/ha
Extraneous Flow	0.28 L/s/mh
Minimum Actual Flow Velocity	0.75 m/s
Maximum Full Flow Velocity	3.0 m/s
Minimum Sewer Lateral Grade	2.00%
Minimum Sewer Depth of Cover	2.3m

3.4 Sanitary Development Demands

The Sanitary demands for existing and proposed conditions have been calculated using *Region of Peel, Linear Wastewater Standards*, dated March 2023, and the results are summarized below in Table 3-2 and Table 3-3 A capacity analysis on the existing 200 mm diameter sanitary sewer within the Shore Street right-of-way was completed under pre-development conditions to assess available flow capacity.

In existing conditions, the existing 200 mm diameter sanitary sewer experience a total design flow

of 1.67 L/s. Detailed Sanitary calculations can be found in the Sanitary Sewer Design Sheet – Existing Conditions provided in **Appendix D**

Table 3-2: Shore Street Sanitary Sewer – Existing Conditions

Land Use	Area (ha)	Ex. Pop. *	Average Flow (L/s)	Peak Factor	Peak Flow (L/s)	Infiltration+ Extran. Flows (L/s)	Total Sanitary Peak Flow (L/s)	% Full
Mixed	0.93	42	0.14	4.00	0.54	0.52	**1.07**	**2.7**

*Estimate considers 5 existing detached units (4.1 persons/detached unit) with 0.42 ha of existing commercial area (50 persons/ha commercial area), as per Town of Caledon design criteria.
**Existing Pipe Capacity is based on existing 200 mm PVC sewer at 1.40% on Shore Street.

Table 3-3: Proposed Site Sanitary Flow Estimate

Land Use	Area (ha)	Proposed Population	Average Flow (L/s)	Peak Factor	Peak Flow (L/s)	Infiltration +Extran. Flows (L/s)	Total Sanitary Peak Flow (L/s)
Residential	0.21	52	0.18	4.00	0.70	0.05	**1.03**

*Residential flow rate of 290 L/cap/day, as per Town of Caledon Design Criteria.

Therefore, it is anticipated that the proposed development will produce a total sanitary flow demand of 1.03 L/s in the proposed condition. Detailed Sanitary calculations can be found in the Sanitary Sewer Design Sheet titled “Proposed Conditions” provided in **Appendix D.**

In order to the assess the impact of the proposed development on the existing Shore Street sewers, a post-development design sheet analysis of Shore Street was conducted. With the construction of the proposed development and the removal of 3 single-family homes, the existing 200 mm diameter sanitary sewer on Shore Street will experience a total peak flow of 1.59 L/s versus the 1.07 L/s in pre-development conditions. This will increase its percent full value from 2.7% in existing conditions to 4.1% in proposed conditions. Based on these calculations, no upgrades to the existing sanitary sewer along Shore Street is anticipated to be required.

Table 3-4: Shore Street Sanitary Sewer - Proposed Conditions

Land Use	Total Area (ha)	Total Pop. *	Average Flow (L/s)	Peak Factor	Peak Flow (L/s)	Infiltration + Extran. Flows (L/s)	Total Sanitary Peak Flow (L/s)	% Full
Mixed	0.93	81	0.27	4.00	1.07	0.52	**1.59**	**4.1**

*Considers a total of 14 Large apartment, and 5 small apartment units, with 2 existing single detached units. A total 0.42 ha of ex. commercial area also remains.

4.0 Water Supply Servicing

4.1 Existing Water Supply Infrastructure

Region records were used to identify any existing Region Water infrastructure adjacent to the proposed development. Based on the as-built information, the existing municipal watermain infrastructure adjacent to the proposed development is as follows:

- A 150mm diameter watermain within the Shore Street right-of-way
- Existing 20 mm diameter water service laterals for each of the 3 lots within the Subject property limits.

Existing fire hydrant location near the subject property is as follows:

- Fire hydrant (#6513625) located in the north boulevard of Shore Street and located approximately 18 m west of the southwest property corner. This existing fire hydrant is connected to the existing 150 mm diameter watermain within Shore Street.
- Fire hydrant (#309549) located in the north boulevard of Shore Street and located approximately 25 m west of the southeast property corner. This existing fire hydrant is connected to the existing 150 mm diameter watermain within Shore Street.

4.2 Proposed Water Supply Servicing

Proposed domestic and fire water servicing for the subject property will be provided by connecting a new fire water service and domestic water service for to the existing 150 mm diameter watermain within Shore Street. The existing water service laterals are to be abandoned as per region requirements.

The fire water service will be 150 mm diameter and be installed as per Region Standard Drawing 1-8-3 and connect to the existing 150 mm diameter watermain within Shore Street with a cut-in-place tee. The proposed fire water service design will also consist of a detector check valve in chamber, a private fire hydrant, and valves at the ultimate property line. The fire water service will be capped 1.0 m away from the building foundation for continuation by others.

Domestic water supply will be provided by a 100 mm diameter domestic water service that tees off the 150 mm fire water service lateral as per Region Standard Drawing 1-8-3. The domestic

water service will be capped 1.0 m away from the building foundation for continuation by others.

A fire hydrant is proposed within the subject property to provide the required 90m fire hydrant coverage for all building perimeters. The proposed hydrant will connect to the internal 150mm fire water service with a 150mm fire hydrant lead, be privately owned, and be privately maintained. The proposed fire hydrant will be a maximum of 8.8 m away from the furthest Siamese connection.

Please refer to **Drawing SS-01** in **Appendix F** for the proposed water servicing layout.

4.3 Design Criteria

The water main design parameters, as outlined in Table 4-1, for the proposed development are based on the municipal design criteria as outlined in **Region of Peel,** *Linear Infrastructure – Watermain Design Criteria,* dated June 2010

Table 4-1: Proposed Watermain Design Parameters

Parameter	Value
Population Density (Large Apartment)	**3.1 persons / unit**
Population Density (Small Apartment)	**1.7 persons / unit**
Large Apartments	**14**
Small Apartment	**5**
Site Area	**0.21 ha**
Total Residential Population	52
Unit Institutional Demand Rates	280 L/cap/day
Maximum Day Factor	2.0
Peak Hour Factor	3.0

4.4 Domestic Water Demands

The design criteria outlined above was used to determine to total domestic water demand for the subject property in proposed conditions.

Average day demand (ADD), Maximum day demand (MDD), and Peak Hour Demand (PHD) factors were calculated using demand peaking factors and population values as outlined above.

The estimated domestic water demand for the proposed development of the subject property is summarised below in **Table 4-2**.

Table 4-2: Proposed Domestic Water Demand

Parameter	Value
Population Density (Large Aparment)	**3.1 persons / unit**
Population Density (Small Apartment)	**1.7 persons / unit**
Large Apartments	**14**
Small Apartments	**5**
Site Area	**0.21 ha**
Total Institutional Population	52
Unit Institutional Demand Rates	280 L/cap/day
Average Daily Demand	10.09 L/min (0.17 L/s)
Maximum Day Demand	20.18 L/min (0.34 L/s)
Peak Hour Demand	30.28 L/min (0.50 L/s)

A detailed breakdown of the calculated demand can be found in **Appendix C**

4.5 Fire Flow Demands

Fire flow demands have been calculated using Water Supply for Public Fire Protection (2020) prepared by the Fire Underwriters Survey (FUS).

Fire flow demands were calculated for the proposed 4-storey apartment building. As part of the FUS calculations, the following construction factors were assumed for the proposed 4-storey apartment building.

- Type III Ordinary Construction – C value of 1.0

- Limited Combustible Occupancy – 15% reduction to the base fire flow
- Sprinkler system with standard water supply – 40% reduction to the base fire flow
- Calculated exposure factor of 14%

Detailed fire flow calculations are provided in **Appendix C** and summarized below in **Table 4-3**.

Table 4-3: Proposed Fire Flow Demand

Building	Fire Flow Demand [5] (L/min)	Fire Flow Demand [5] (L/s)
Proposed 4 storey apartment building	5000	83.33

From the fire flow calculations, it was determined that the recommended fire flow of 5000 L/min 83.33 L/s) is required for the proposed development.

A hydrant flow test was conducted by BA Fire Safety on June 21, 2024, at 1:30 PM on existing fire hydrant #6513625. The results indicate an available maximum flow of 4163 GPM (262.6 L/s) at a residual pressure of 20 psi. This is higher than the calculated required fire flow demand of 83.33 L/s.

Therefore, sufficient fire flow is available for the proposed development. The results of the hydrant flow test can be found in **Appendix C.**

4.6 Total Water Demand

Based on the total water demand and the fire flow requirements, the proposed demand will be the Maximum Day Demand plus the Fire Flow demand. **Table 4-3** below outlines the total water demand for the proposed development, detailed calculations can be found in **Appendix C**.

Table 4-4: Total Water Supply Demands

Land Use	Expected Population [1]	Average Day Demand [2] (L/s)	Peak Hour Demand [3] (L/s)	Max. Day Demand [4] (L/s)	Fire Flow Demand (L/s)	Max Day + Fire (L/s)
Residential	52	0.17	0.50	0.37	83.33	83.67

Note:

[1] Expected population based on population estimate in Section 1.4
[2] Based on average day consumption rate as 280 l/cap/day
[3] Based on peak hour factor as 3.0
[4] Based on maximum day factor as 2.0

Based on the assessment of the calculated total water demand (83.67 L/s) and the hydrant flow test, it is confirmed that the existing 150mm diameter watermain on Shore Street has sufficient pressure and flow to service the proposed development.

5.0 Erosion & Sediment Control Measures

A preliminary Erosion and Sediment Control (ESC) Plan for the proposed development will be prepared in accordance with TRCA and City requirements as part of the detailed design phase.

The proposed erosion and sediment control works during construction will consider of the following:

- temporary silt fences;
- individual catch basin silt sacks, within the subject property and on Guru Nanak Street:
- mud mats at the construction access point;
- one (1) topsoil stockpile, equipped with silt fencing;
- one (1) equipment staging and refuelling area, equipped with silt fencing
- Siltsoxx check dams

6.0 SUMMARY

This report outlines the desired stormwater management and servicing scheme for the proposed development at 15, 21, 27 Shore Street, Caledon, ON. The following summarizes the conclusions and recommendations of this report:

Stormwater Management

- Post to pre-development peak flow controls such that all storm events up to and including the 100-year storm event are controlled to the 5-year pre-development level. This is proposed in accordance with Town SWM criteria.
- Detention storage is required to achieve this requirement and will be provided using surface storage.
- Retention of 5 mm of rainfall for erosion control through rainwater harvesting and re-use.
- Quality control will be provided by a centralized Jellyfish Filter Unit.

Sanitary Servicing

- Sanitary servicing to be provided by a 150mm sanitary service lateral connecting to the existing 200 mm sanitary sewer within Shore Street. A new sanitary sampling manhole is provided for the proposed building at the ultimate property line.

Water Supply Servicing

- Domestic water service will be provided by a new 100m m domestic water service for the proposed 4 storey apartment building. The domestic water service will tee off the 150 mm diameter fire water service as per Region Standard drawing 1-8-3.
- Fire water service will be provided by a 150mm fire water service that will connect to the existing 150 mm diameter watermain within Shore Street with a cut-in-place tee and valve.
- A private hydrant is to be installed within the subject property limits and is approximately 8.80 m away from the Siamese Connection and provides 90 m hydrant coverage to all building perimeters within the subject property.

We believe that the above study and materials are satisfactory to address the concerns of reviewing agencies for the proposed development. Should there be any comments regarding the materials of this report, please contact the undersigned.

Respectfully Submitted,

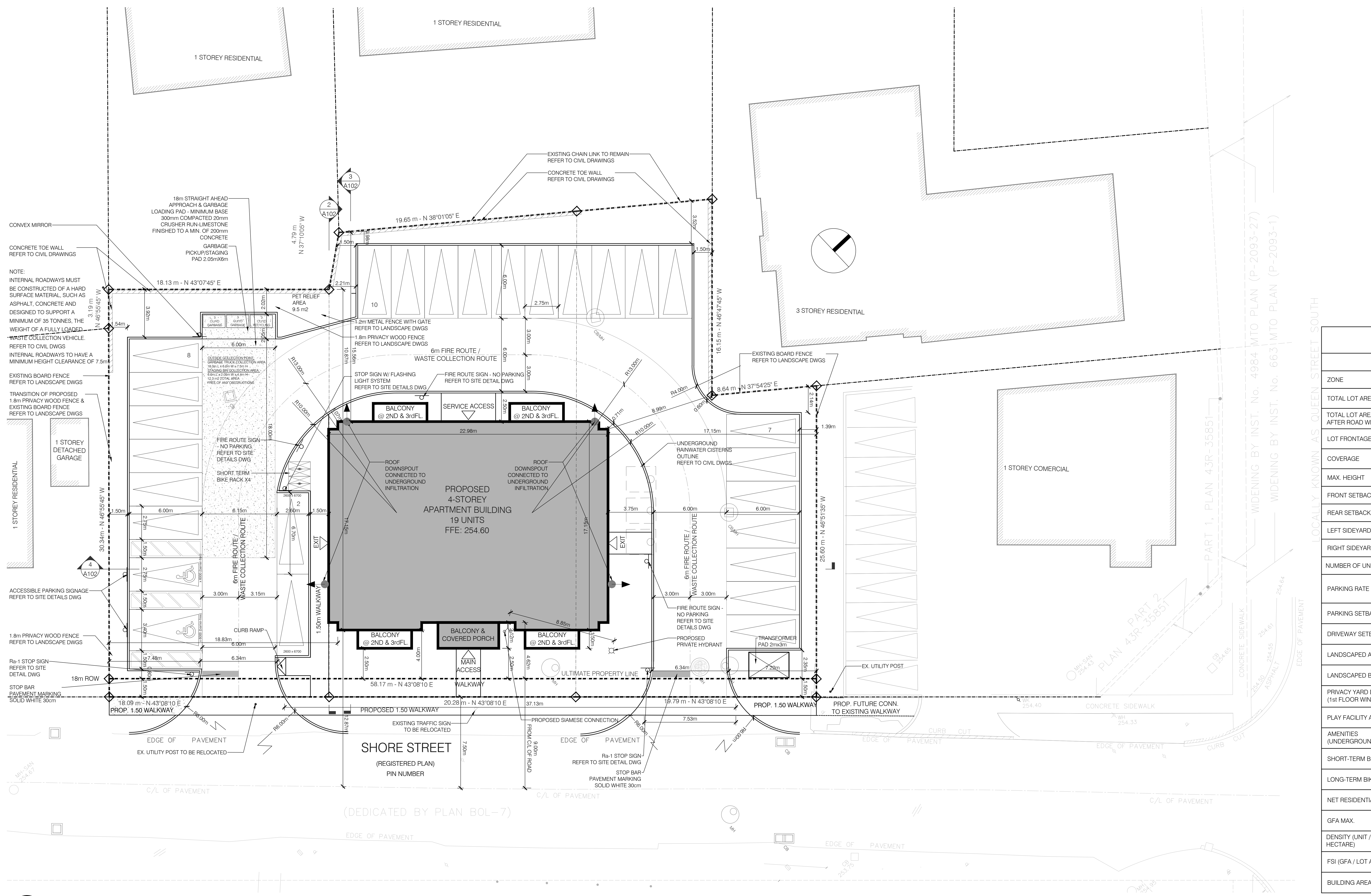
Urbanworks Engineering Corporation

Giancarlo Volpe, P.Eng., M.Eng.
Project Engineer

APPENDIX A

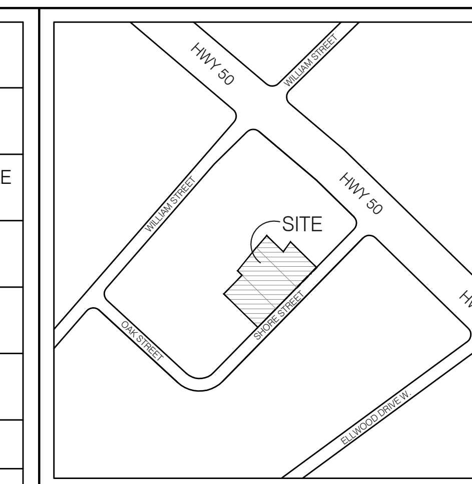
SITE PLAN, TOPOGRAPHIC SURVEY, AND REGION AS-BUILT

SITE PLAN



1 SITE PLAN
A100 SCALE: 1:250

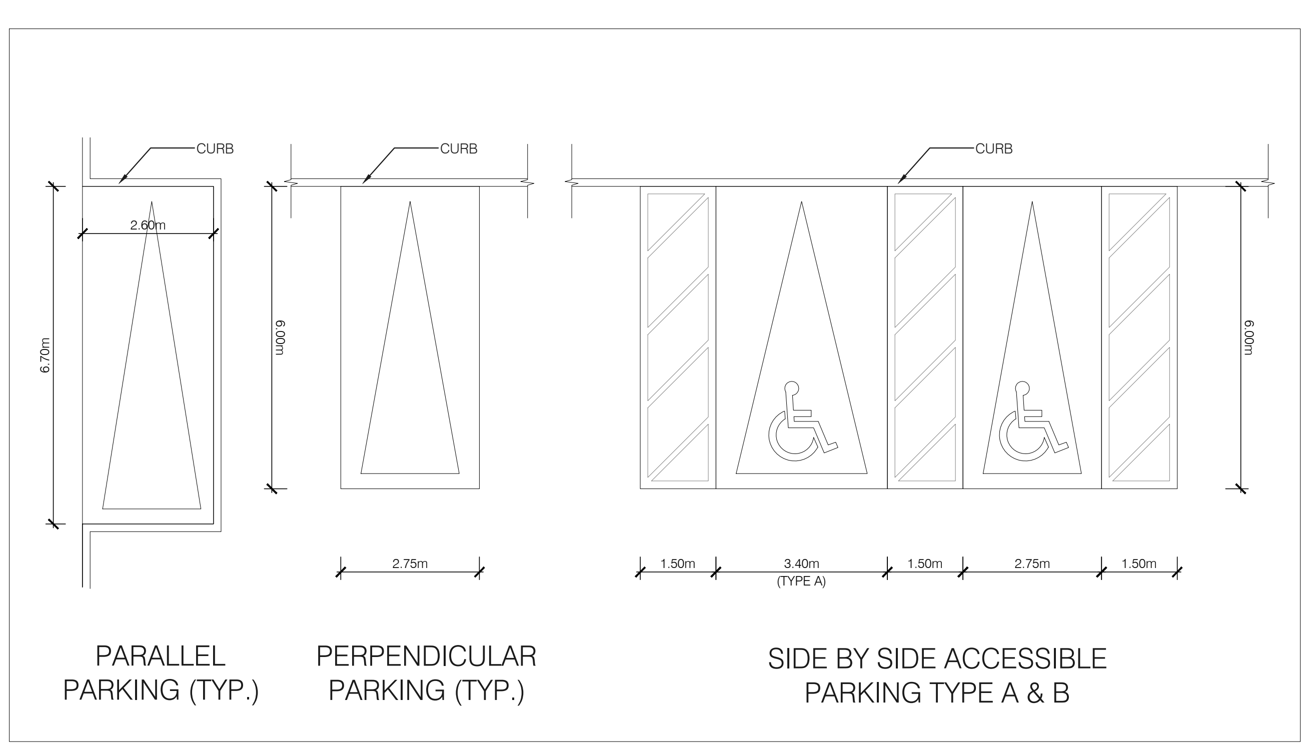
	SITE PLAN LEGEND
CB	DENOTES CATCHBASIN REFER TO CIVIL DWGS
CB/MH	DENOTES CATCHBASIN/MANHOLE REFER TO CIVIL DWGS
MH	DENOTES MANHOLE REFER TO CIVIL DWGS
	DENOTES BIKE STORAGE
	DENOTES TREES/PLANTS REFER TO LANDSCAPE DWGS
	DENOTES WASTE RECEPTACLE
	DENOTES UNIT PAVING REFER TO LANDSCAPE DWGS
	DENOTES CONCRETE PAVING REFER TO LANDSCAPE DWGS
	DENOTES CONCRETE TOE WALL REFER TO LANDSCAPE DWGS



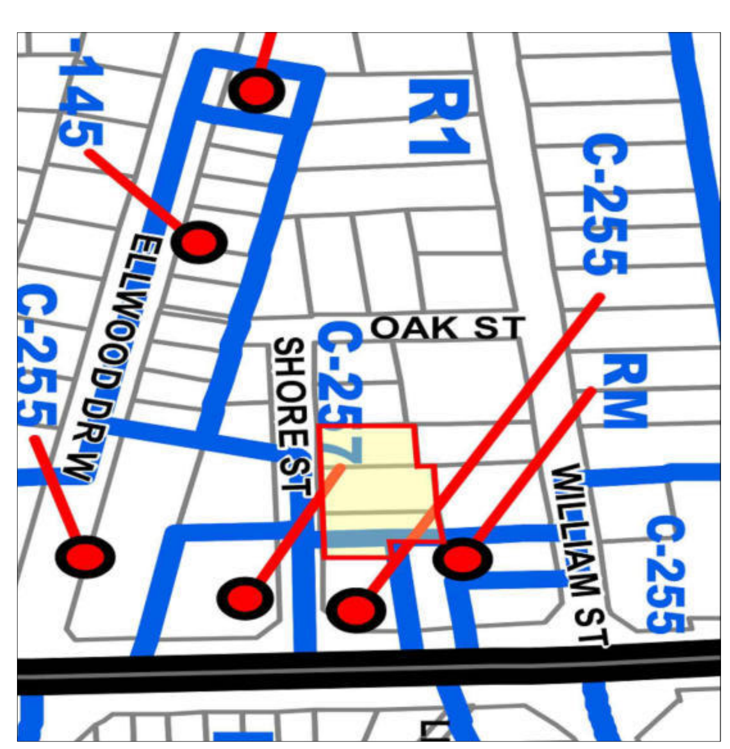
KEY PLAN

SITE DEVELOPMENT

	EXISTING	REQUIRED	REQUIRED	PROPOSED
ZONE	C255 & R1	RM	RM-XXXX	RM-XXXX
TOTAL LOT AREA		925.00 m2	-	2067.32 m2
TOTAL LOT AREA - AFTER ROAD WIDENING		925.00 m2	-	1980.00 m2
LOT FRONTAGE		30.00 m	30.00 m	58.17 m
COVERAGE		20%	20%	19.64% (388.83 m2)
MAX. HEIGHT		12.20 m	14.50 m	14.18 m
FRONT SETBACK		(3.00 m MIN.)	3.00 m MIN.	4.00 m
REAR SETBACK		(7.50 m MIN.)	7.50 m MIN.	10.87 m
LEFT SIDEYARD		(7.50 m MIN.)	7.50 m MIN.	18.08 m
RIGHT SIDEYARD		(7.50 m MIN.)	7.50 m MIN.	8.82 m
NUMBER OF UNITS		-	-	19
PARKING RATE		34 (1.75 PER UNIT)	25 (1.0 PER UNIT + 0.29 PER UNIT VISITORS)	27 (1.42 PER UNIT)
PARKING SETBACK		6m	0.5m INT. SIDE YARD, 0.8m ANY LOT LINE	0.74m
DRIVEWAY SETBACK		0.5m	0.4m	0.6m
LANDSCAPED AREA		45%	20% (396 m2)	21.2% (420 m2)
LANDSCAPED BUFFER		3m	0.4m	VARIES (SEE SITE PLAN)
PRIVACY YARD DEPTH (1st FLOOR WINDOWS)		5m	0m	VARIES (SEE SITE PLAN)
PLAY FACILITY AREA		4% (82.67 m2)	0 m2	0 m2
AMENITIES (UNDERGROUND)		-	-	126 m2
SHORT-TERM BIKE ST.		-	4	4
LONG-TERM BIKE ST.		-	10 (0.5/Unit)	10 (0.5/Unit)
NET RESIDENTIAL AREA		-	-	1,197.46 m2 12,889 SF
GFA MAX.		-	1,600 m2	1,557.40 m2 16,764 SF
DENSITY (UNIT / HECTARE)		-	-	96
FSI (GFA / LOT AREA) MAX.		-	0.8	0.78
BUILDING AREA MAX.		-	20% (396 m2)	19.7% (389.35 m2)



2 PARKING STANDARDS
A100 SCALE: 1:150



3 ZONING MAP
A100 SCALE: NTS

4 LOT FABRIC
A100 SCALE: 1:500

No.	DESCRIPTION	DATE
10	ISSUED FOR SPA	2024-10-01
9	ISSUED FOR COORDINATION	2024-07-10
8	ISSUED FOR COORDINATION	2024-06-24
7	ISSUED FOR REVIEW	2024-02-20
6	ISSUED FOR PAC	2023-11-13
5	ISSUED FOR CLIENT REVIEW	2023-11-08
4	ISSUED FOR CLIENT REVIEW	2023-10-31
3	ISSUED FOR CLIENT REVIEW	2023-09-21
2	ISSUED FOR CLIENT REVIEW	2023-08-24
1	ISSUED FOR PAC	2023-05-11
#	REV. DESCRIPTION	YYYY-MM-DD

REVISIONS

ISSUED FOR CONSTRUCTION
ISSUED FOR BID
ISSUED FOR BUILDING PERMIT
ISSUED FOR SITE PLAN APPROVAL

SUBMITTALS

CONTRACTORS MUST CHECK AND VERIFY ALL DIMENSIONS AND CONDITIONS ON THE PROJECT AND MUST REPORT ANY DISCREPANCIES TO THE DESIGNER BEFORE PROCEEDING WITH CONSTRUCTION.

THIS DRAWING MUST NOT BE USED FOR CONSTRUCTION PURPOSES UNTIL SEALED AND SIGNED BY THE DESIGNER.

DO NOT SCALE DRAWINGS.

FCA
FAUSTO CORTESE
ARCHITECTS
3590 RUTHERFORD RD. UNIT 7
VAUGHAN, ONTARIO, L4H 3T8
416-806-7000
FCORTESE@FCARCHITECTS.CA

DRAWING:
PROPOSED
MULTI-UNIT RESIDENTIAL
ON
15, 21, 27 SHORE ST.
BOLTON, ON
CALEDON

DRAWING:
SITE PLAN

PLOTTED: N/A

DATE: 21/08/2025	PROJECT No: 2023-04
SCALE: AS NOTED	DRAWING No: A100
DRAWN BY: S.H. / REVIEWED BY: F.C.	

FILE NAME: SHORE STREET - SITE PLAN - 2025-08-
PLOT SCALE: 1=

TOPOGRAPHIC SURVEY

PART 2 - SURVEY REPORT PIN 14322-0321(LT)

- **DESCRIPTION**
  MUNICIPALLY KNOWN AS No. 15 SHORE STREET
  PART OF LOTS 12 AND 13, ON BLOCK 6
  REGISTERED PLAN BOL-7
  TOWN OF CALEDON
  REGIONAL MUNICIPALITY OF PEEL
- **REGISTERED EASEMENTS AND/OR RIGHTS-OF-WAY**
  -NONE
- **ZONING COMPLIANCE**
  NO INVESTIGATION WITH RESPECT TO MUNICIPAL ZONING REQUIREMENT HAS BEEN MADE IN CONNECTION WITH THIS REPORT.
- **ADDITIONAL REMARKS**
  - THE FENCES ARE AS SHOWN ON THE SURVEY PLAN.
  - MONUMENTS SHOWN AS "WIT" ARE WITNESS MONUMENTS AND ARE NOT AT THE PROPERTY CORNER.

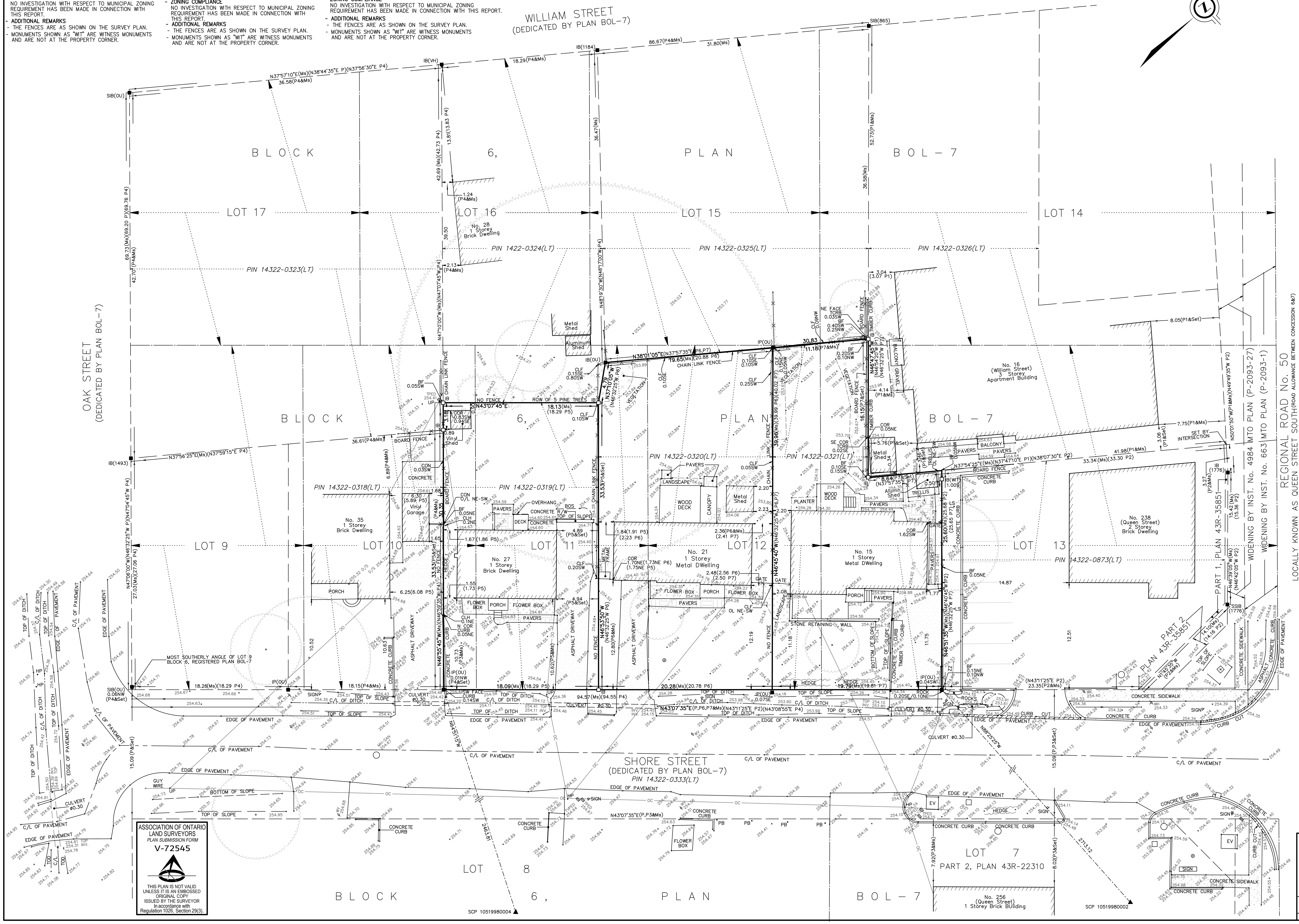
PART 2 - SURVEY REPORT PIN 14322-0320(LT)

- **DESCRIPTION**
  MUNICIPALLY KNOWN AS No. 21 SHORE STREET
  PART OF LOTS 11 AND 12, ON BLOCK 6
  REGISTERED PLAN BOL-7
  TOWN OF CALEDON
  REGIONAL MUNICIPALITY OF PEEL
- **REGISTERED EASEMENTS AND/OR RIGHTS-OF-WAY**
  -NONE
- **ZONING COMPLIANCE**
  NO INVESTIGATION WITH RESPECT TO MUNICIPAL ZONING REQUIREMENT HAS BEEN MADE IN CONNECTION WITH THIS REPORT.
- **ADDITIONAL REMARKS**
  - THE FENCES ARE AS SHOWN ON THE SURVEY PLAN.
  - MONUMENTS SHOWN AS "WIT" ARE WITNESS MONUMENTS AND ARE NOT AT THE PROPERTY CORNER.

PART 2 - SURVEY REPORT PIN 14322-0319(LT)

- **DESCRIPTION**
  MUNICIPALLY KNOWN AS No. 27 SHORE STREET
  PART OF LOTS 10 AND 11, ON BLOCK 6
  REGISTERED PLAN BOL-7
  TOWN OF CALEDON
  REGIONAL MUNICIPALITY OF PEEL
- **REGISTERED EASEMENTS AND/OR RIGHTS-OF-WAY**
  -NONE
- **ZONING COMPLIANCE**
  NO INVESTIGATION WITH RESPECT TO MUNICIPAL ZONING REQUIREMENT HAS BEEN MADE IN CONNECTION WITH THIS REPORT.
- **ADDITIONAL REMARKS**
  - THE FENCES ARE AS SHOWN ON THE SURVEY PLAN.
  - MONUMENTS SHOWN AS "WIT" ARE WITNESS MONUMENTS AND ARE NOT AT THE PROPERTY CORNER.

METRIC DISTANCES AND/OR COORDINATES SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048.



SURVEYOR'S REAL PROPERTY REPORT
PART 1 - PLAN OF SURVEY OF
PART OF LOTS 10, 11, 12 AND 13 BLOCK 6 REGISTERED PLAN BOL-7

GEOGRAPHIC TOWNSHIP OF BOLTON
NOW IN THE
TOWN OF CALEDON
REGIONAL MUNICIPALITY OF PEEL
SCALE 1 : 200
5 0 5 10 metres

GUIDO PAPA SURVEYING
A DIVISION OF J.D. BARNES LTD

© COPYRIGHT 2023

GEODETIC

ELEVATIONS SHOWN ON THIS PLAN ARE RELATED TO GEODETIC DATUM AND ARE DERIVED FROM THE TOWN OF CALEDON BENCH MARK No 758057 HAVING A PUBLISHED ELEVATION OF 251.929 METRES.

LEGEND:

■	DENOTES	SURVEY MONUMENT FOUND
□	DENOTES	SURVEY MONUMENT SET
SIB	DENOTES	STANDARD IRON BAR
SSIB	DENOTES	SHORT STANDARD IRON BAR
IB	DENOTES	IRON BAR
IP	DENOTES	IRON PIPE
OU	DENOTES	ORIGIN UNKOWN
WIT	DENOTES	WITNESS
Ms	DENOTES	MEASURED
PL	DENOTES	REGISTERED PLAN BOL-7
P1	DENOTES	PLAN OF SURVEY BY ROBERT STEPHENSON, O.L.S. DATED DECEMBER 12, 1967
P2	DENOTES	PLAN 43R-35851
P3	DENOTES	PLAN 43R-22310
P4	DENOTES	SRPR BY VAN HARTEN SURVEYING INC. O.L.S. DATED DECEMBER 13, 2021 (PROJECT No. 30324-21)
P5	DENOTES	PLAN OF SURVEY BY BROWNE, CAVELL AND ASSOCIATES O.L.S. DATED NOVEMBER 28, 1956 (REFERENCE: 20-213)
P6	DENOTES	PLAN OF SURVEY BY JOHN C. MOORE, O.L.S. DATED JULY 29, 1980
P7	DENOTES	PLAN OF SURVEY BY BROWNE, CAVELL AND ASSOCIATES O.L.S. DATED JANUARY 16, 1956
1184	DENOTES	E. BIASON, O.L.S.
865	DENOTES	DONALD P. McLEAN, O.L.S.
1493	DENOTES	YOUNG & YOUNG SURVEYING INC., O.L.S.
1776	DENOTES	BELA KAPOSI O.L.S.
VH	DENOTES	DAVID J. PESCE O.L.S.
SRPR	DENOTES	SURVEYOR'S REAL PROPERTY REPORT
COR	DENOTES	CORNER
BF	DENOTES	BOARD FENCE
CLF	DENOTES	CHAIN LINK FENCE
TCRB	DENOTES	TIMBER CURB
CRB	DENOTES	CURB
CLH	DENOTES	CENTRELINE OF HEDGE
D/S	DENOTES	DOOR SILL
G/S	DENOTES	GARAGE SILL
WW	DENOTES	WINDOW WELL
EV	DENOTES	ELECTRIC VAULT
OL	DENOTES	ON LINE
-OC-	DENOTES	OVERHEAD WIRES AND CABLES
N,S,E,W	DENOTES	NORTH, EAST, SOUTH, WEST
CB	DENOTES	CATCHBASIN
MH	DENOTES	MANHOLE
MH-SAN	DENOTES	SANITARY MANHOLE
PB	DENOTES	BOLLARD
HP	DENOTES	HYDRO POLE
LS	DENOTES	LIGHT STANDARD
TPED	DENOTES	TELEPHONE PEDESTAL
CPED	DENOTES	CABLE TV PEDESTAL
WH	DENOTES	FIRE HYDRANT
WK	DENOTES	WATER KEY
WV	DENOTES	WATER VALVE
ZZ	DENOTES	GUY WIRE
	DENOTES	TREE CONIFEROUS
	DENOTES	TREE DECIDUOUS
⊗	DENOTES	SHRUB

NOTES

BEARINGS ARE UTM GRID, DERIVED FROM SPECIFIED CONTROL POINTS (SCPs) 10519980004 AND 10519980002, UTM ZONE 17, NAD83 (CSRS)(2010).

DISTANCES ARE GROUND AND CAN BE CONVERTED TO GRID BY MULTIPLYING BY THE COMBINED SCALE FACTOR OF 0.999694.

FOR BEARING COMPARISONS, A ROTATION OF 1°01'50" COUNTER-CLOCKWISE WAS APPLIED TO BEARINGS ON P1.

FOR BEARING COMPARISONS, A ROTATION OF 1°07'25" COUNTER-CLOCKWISE WAS APPLIED TO BEARINGS ON P,P6,P7.

FOR BEARING COMPARISONS, A ROTATION OF 0°00'35" CLOCKWISE WAS APPLIED TO BEARINGS ON P3.

INTEGRATION DATA

SPECIFIED CONTROL POINTS (SCPs): UTM ZONE 17, NAD83 (ORIGINAL).

COORDINATES TO A RURAL ACCURACY PER SECTION 14 (2) OF O.REG 216/10.

POINT ID	EASTING	NORTHING
SCP 10519980002	604 592.72	4 858 634.89
SCP 10519980004	604 071.30	4 857 620.44

COORDINATES CANNOT, IN THEMSELVES, BE USED TO RE-ESTABLISH CORNERS OR BOUNDARIES SHOWN ON THIS PLAN.

THE RESULTANT TIE BETWEEN SCP 10519980004 AND SCP 10519980002 IS 1140.96 N27°12'10"E

NOTES

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SURVEYOR'S CERTIFICATE

I CERTIFY THAT:

1. THIS SURVEY AND PLAN ARE CORRECT AND IN ACCORDANCE WITH THE SURVEYS ACT, THE SURVEYORS ACT AND THE REGULATIONS MADE UNDER THEM.
2. THE SURVEY WAS COMPLETED ON AUGUST 4, 2023.

AUGUST 11, 2023
DATE

VALERIO G. PAPA
ONTARIO LAND SURVEYOR

GUIDO PAPA SURVEYING
A Division of J. D. Barnes Limited
SURVEYING MAPPING GIS
9135 KEELE ST., UNIT B7, VAUGHAN, ON L4K 0J4
T: (289) 553-5961 F: (289) 553-5986 www.jdbarnes.com

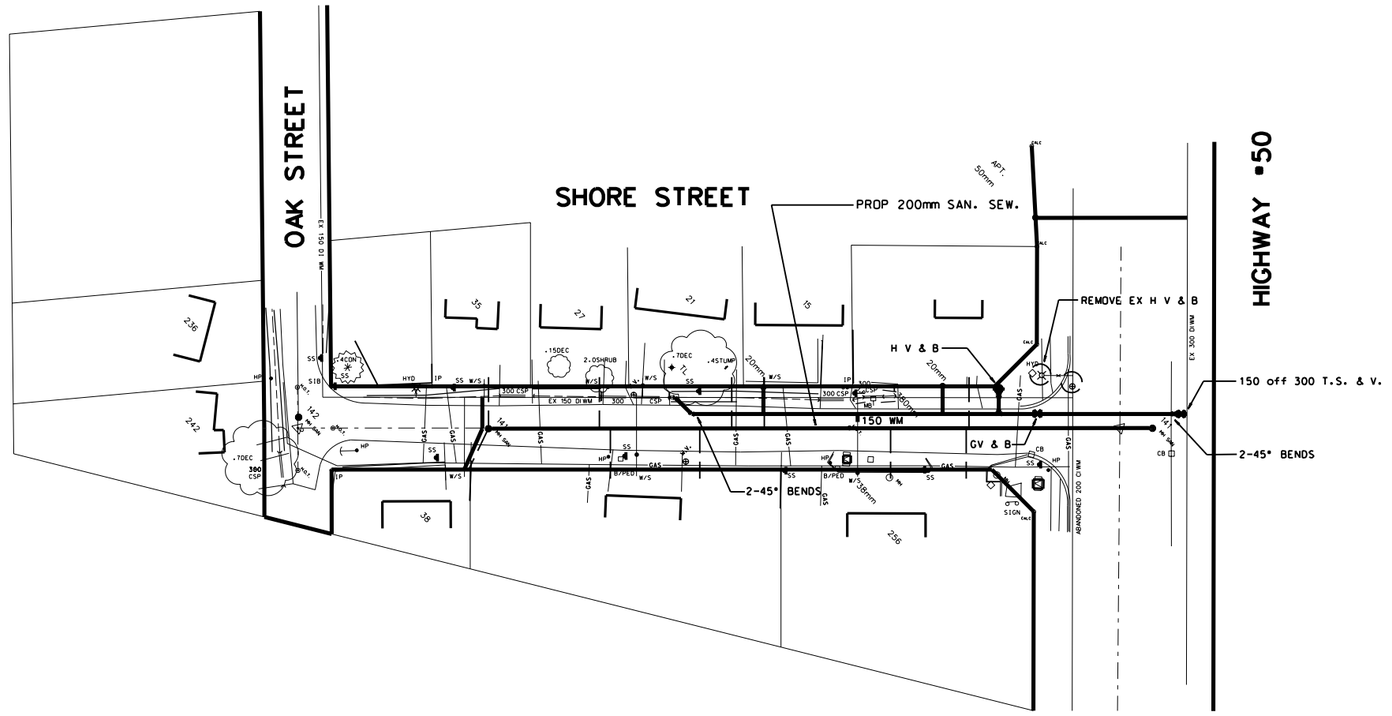
DRAWN BY:	CHECKED BY:	REFERENCE NO.:
H.S.	V.G.P.	23-18-550-00

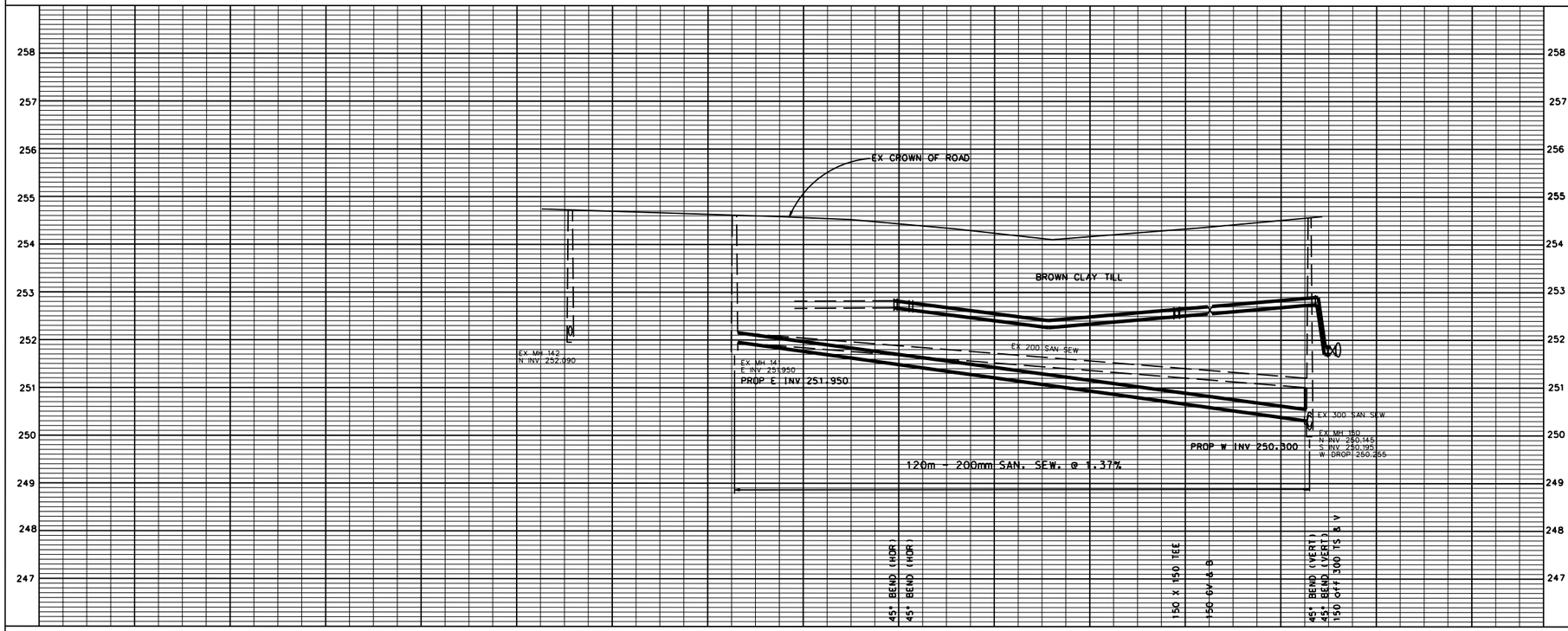
FILE: G:\Jobs\2023\23-18-550-00 Fausto Cortese - Shore Street\Drawing\23-18-550-00.dgn

REGION AS-BUILT

SERVICE DATA					
SERVICE	DATE	INIT.	SERVICE	DATE	INIT.
SAN. SEWERS			GAS MAINS		
STORM SEWERS			BELL U/G CABLE		
WATERMAINS			HYDRO U/G CABLE		
TRANSIT			ONT. HYDRO		
PARKS & REC.			CTV		
ONT. CLEAN. WATER					

REVISIONS		
DATE	DETAILS	INIT.
SEPT 97	AS CONSTRUCTED	EWK





General Notes

- ALL DRIVEWAYS ASPHALT UNLESS OTHERWISE NOTED.
- ALL SERVICE LOCATIONS ARE APPROXIMATE AND MUST BE LOCATED ACCURATELY IN THE FIELD
- DENOTES BUILDING - NOT LOCATED
- DENOTES BUILDING LOCATED TYPE 'B' BEDDING UNLESS OTHERWISE NOTED (SAN)

B.M. NO. ELEV.

THE CONTRACTOR IS RESPONSIBLE FOR LOCATING AND PROTECTING ALL EXISTING UTILITIES PRIOR TO AND DURING CONSTRUCTION LOCATION OF EXISTING UTILITIES APPROXIMATE ONLY. TO BE VERIFIED IN FIELD BY CONTRACTOR.

DESIGNED BY _____ APPROVED BY _____

CHKD. _____

NOTICE TO CONTRACTOR

48 HOURS PRIOR TO COMMMENCING WORK NOTIFY THE FOLLOWING

THE REGIONAL MUNICIPALITY OF PEEL
CITY OF MISSISSAUGA WORKS DEPT.
CITY OF BRAMPTON WORKS DEPT.
TOWN OF CALEDON WORKS DEPT.
BELL TELEPHONE COMPANY
CONSUMERS GAS COMPANY
MINISTRY OF TRANSPORTATION
ONTARIO CLEAN WATER AGENCY
HYDRO ELECTRIC POWER COMM. OF ONTARIO
HYDRO ELECTRIC COMM. CITY OF MISSISSAUGA
HYDRO ELECTRIC COMM. CITY OF BRAMPTON
CABLE TELEVISION

10m 0 10 20 30m HORIZONTAL SCALE

1m 0 1 2 3m VERTICAL SCALE

Region of Peel
Public Works

SHORE STREET
PROP. SANITARY SEWER

Sta. 0+000 **To Sta.** 0+000

LOTS	AREA C-12	PROJECT NO. 97-2250
CHECKED BY	DRAWN BY EWK	PLAN NO. 22534-D
DATE DEC 96	SHEET 1 OF 1	

22534_D.DGN 12/05/04 10:46:56 AM

APPENDIX B

GEOTECHNICAL REPORT

GEOTECHNICAL REPORT

Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL • ENVIRONMENTAL • HYDROGEOLOGICAL • BUILDING SCIENCE
90 WEST BEAVER CREEK ROAD, SUITE 100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL: (416) 754-8515 · FAX: (905) 881-8335

BARRIE	MISSISSAUGA	OSHAWA	NEWMARKET	MUSKOKA	HAMILTON
TEL: (705) 721-7863	TEL: (905) 542-7605	TEL: (905) 440-2040	TEL: (905) 853-0647	TEL: (705) 721-7863	TEL: (905) 777-7956
FAX: (705) 721-7864	FAX: (905) 542-2769	FAX: (905) 725-1315	FAX: (905) 881-8335	FAX: (705) 721-7864	FAX: (905) 542-2769

A REPORT TO
BOLTON SHORE HOLDINGS LTD.

A GEOTECHNICAL INVESTIGATION FOR PROPOSED 4-STOREY BUILDING WITH BASEMENT

15, 21 AND 27 SHORE STREET

TOWN OF CALEDON

REFERENCE NO. 2404-S107

JULY 2024

DISTRIBUTION

Digital copy - Bolton Shore Holdings Ltd.
Digital copy - Blackthorn Development Corp.

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TABLES

ENCLOSURES

1.0 INTRODUCTION

In accordance with a written authorization dated April 19, 2024, from Mr. Mark Cancian of Bolton Shore Holdings Ltd., a geotechnical investigation was carried out at 15, 21 and 27 Shore Street in the Town of Caledon.

The purpose of the investigation was to reveal the subsurface conditions and determine the engineering properties of the disclosed soils for the construction of a 4-storey apartment building with a basement. The geotechnical findings and recommendations for the proposed development are presented in this report.

2.0 SITE AND PROJECT DESCRIPTION

The site is situated on Halton till plain, where the drift dominates the soil stratigraphy. In places, lacustrine sand, silt, clay and drift which has been reworked by the water action of Peel Ponding (glacial lake) have modified the drift stratigraphy.

The investigation was carried out within the properties of 15, 21 and 27 Shore Street in the Town of Caledon. The combined property consists of 3 residential dwellings with associated driveway and landscape areas. The grading within the study area is relatively flat.

A review of the Site Plan prepared by Fausto Cortese Architects dated March 8, 2024, indicates that the existing dwellings will be demolished to make way for a new 4-storey building with basement. It will be provided with on-grade parking, access driveway and landscape areas.

3.0 FIELD WORK

The field work, consisted of 4 boreholes extending to depths of 8.1 and 8.5 m below grade, was completed on May 29 and 30, 2024, at the locations shown on the Borehole and Monitoring Well Location Plan, Drawing No. 1.

The boreholes were advanced at intervals to the sampling depths by a compact track-mounted drill rig equipped with solid stem augers and split spoons for soil sampling. Standard Penetration Tests, using the procedures described on the enclosed “List of Abbreviations and Terms”, were performed at the sampling depths. The results are recorded as the Standard Penetration Resistance (or ‘N’ values) of the subsoil. The relative density of

the non-cohesive strata and the consistency of the cohesive strata are inferred from the 'N' values. Split-spoon samples were recovered for soil classification and laboratory testing.

Upon completion of borehole drilling, monitoring wells were installed in 3 of the 4 boreholes to facilitate a hydrogeological assessment. Details of the monitoring wells are presented in the borehole logs.

The field work was supervised and the findings were recorded by a Geotechnical Technician. The ground elevation at each of the borehole location was obtained using the Global Navigation Satellite System (GNSS).

4.0 SUBSURFACE CONDITIONS

Detailed descriptions of the subsurface conditions are presented on the Borehole Logs, comprising Figures 1 to 4 inclusive. The revealed stratigraphy is plotted in the Subsurface Profiles, Drawing No. 2. The engineering properties of the disclosed soils are discussed herein.

The investigation has disclosed that beneath the topsoil and a layer of earth fill, the site is generally underlain by a stratum of silty clay, with a localized deposit of silty clay till.

4.1 Topsoil

The revealed topsoil is approximately 8 to 10 cm in thickness. Thicker topsoil layers may be contacted beyond the borehole locations. The topsoil is void of engineering value and must be stripped for site development.

4.2 Earth Fill

A layer of earth fill, extending to a depth of 0.8 m below the prevailing ground surface, was encountered in all boreholes. It is dark brown in colour and consists of silty clay, with a variable amount of topsoil and rootlets.

The natural water content of the samples was determined and the results are plotted on the Borehole Logs; the values range from 23% to 33%, with a median of 28%. The high water content value indicates the presence of topsoil.

The obtained 'N' values range from 5 to 10, with a median of 8 blows per 30 cm of penetration, indicating that the fill was likely placed with non-uniform compaction. Due to its unknown history and non-uniform density, the earth fill is not suitable to support any structures sensitive to settlement. It must be subexcavated, sorted free of deleterious material and organics and properly recompacted in layers.

4.3 Silty Clay/Silty Clay Till

The silty clay is the predominant soil in the revealed stratigraphy. It contains traces of sand and gravel, with occasional silt seams. The silty clay till was encountered beneath the topsoil and earth fill, overlying the silty clay in Borehole 3. It consists of a random mixture of particle sizes ranging from clay to gravel, with the silt and clay being the dominant fraction. A grain size analysis was performed on a representative sample each of the silty clay and silty clay till. The results are plotted on Figures 5 and 6, respectively.

The obtained 'N' values range from 8 to 45 blows per 30 cm of penetration, with a median of 23 blows per 30 cm of penetration, indicating the clay/clay till is stiff to hard, being generally very stiff in consistency. The 'N' values of 8 and 9 were encountered near the ground surface where the soil has been weathered.

Atterberg Limits were determined on a sample each of the clay till and clay sample. The till sample has liquid limit of 34% and plastic limit of 18%, while the clay has a liquid limit of 41% and plastic limit of 21%, indicating the clay till is low in plasticity while the clay is medium in plasticity.

The natural water content of the soil samples ranges between 16% and 29%, at a median of 22%, showing a moist to very moist, generally moist condition.

The engineering properties of the clay till deposit are given below:

- High frost susceptibility and low water erodibility.
- The clay till will be stable in relatively steep slopes; however, prolonged exposure will allow the sand seams to slough, which may lead to local sliding.

5.0 GROUNDWATER CONDITION

All boreholes remained dry upon completion of the field work.

Monitoring wells were installed at Boreholes 1, 2 and 3. Two rounds of stabilized groundwater levels were recorded in the monitoring wells and are summarized in Table 1.

Table 1 - Groundwater Levels in Monitoring Wells

Borehole No.	Ground Elevation (m)	Well Depth (m)	Measured Groundwater Level					
			Jun 11, 2024		Jun 24, 2024		July 9, 2024	
			Depth (m)	El. (m)	Depth (m)	El. (m)	Depth (m)	El. (m)
1	254.1	7.6	5.5	248.6	4.6	249.5	2.6	251.5
2	254.1	7.6	1.3	252.8	1.1	253.0	1.4	252.7
3	254.5	7.6	1.8	252.7	1.8	252.7	1.9	252.6

The groundwater level recorded in the monitoring wells ranges from 1.1 to 5.5 m below the ground surface, or El. 248.6 to 253.0 m.

Additional groundwater assessment will be presented in the hydrogeological assessment report, to be presented under separate cover.

6.0 DISCUSSION AND RECOMMENDATIONS

The investigation has disclosed that beneath the topsoil and a layer of earth fill extending to a depth of 0.8 m below the prevailing ground surface, the site is underlain by a stratum of silty clay, stiff to hard consistency. A localized deposit of very stiff silty clay till was also encountered overlying the silty clay at one borehole.

All boreholes remained dry on completion of the field work. Three rounds of groundwater levels in the monitoring wells were recorded and the results range from El. 248.6 to 253.0 m. Further groundwater assessment will be presented in the hydrogeological assessment report, to be provided under separate cover.

The proposed development will consist of a 4-storey apartment building with a conventional basement. The geotechnical recommendations appropriate for the design and construction of the development are presented herein. One must be aware that the subsurface conditions may vary between boreholes. Should subsurface variances become apparent during construction, a geotechnical engineer must be consulted.

A soil cover having a thickness at least equal to the diameter of the pipe should be in place at all times after pipe installation, to prevent pipe floatation when the trench is deluged with water derived from precipitation.

6.5 Backfilling in Trenches and Excavated Areas

The backfill in service trenches should be compacted to at least 98% SPDD, particularly below concrete floor subgrade and in the zone within 1.0 m below the pavement. The material should be compacted with the water content at 2% to 3% drier than the optimum. The lifts must be limited to 20 cm or less (before compaction).

As a general guide, the typical water content values of the revealed soils for Standard Proctor compaction are presented in Table 2.

Table 2 - Estimated Water Content for Compaction of On-Site Material

Soil Type	Determined Natural Water Content (%)	Water Content (%) for Standard Proctor Compaction	
		100% (optimum)	Range for 95% or +
Earth Fill	23 to 33 (median 28)	15	13 to 20
Silty Clay Till/Silty Clay	16 to 29 (median 22)	15 to 21	13 to 25

The in-situ clay and clay till are generally suitable for use as trench backfill. Where they are too wet, they will require aeration by spreading them thinly in dry, warm weather. The till should be sorted free of oversized boulders (over 15 cm in size) before use as backfill. The existing fill must be sorted free of topsoil inclusions and deleterious materials, if any, prior to its use as structural backfill.

In normal construction practice, the problem areas of pavement settlement largely occur adjacent to manholes, catch basins, services crossings, foundation walls and columns, it is recommended that a sand backfill should be used.

The narrow trenches for services crossings should be cut at 1 vertical:2 horizontal so that the backfill in the trenches can be effectively compacted. Otherwise, soil arching in the trenches will prevent achievement of the proper compaction. In confined areas where the desired slope cannot be achieved or the operation of a proper kneading-type roller cannot be facilitated, imported sand fill, which can be appropriately compacted by using a smaller vibratory compactor, must be used.

6.6 Pavement Design

The pavement design for the parking area and fire route is presented in Table 3.

Table 3 - Pavement Design

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL3
Asphalt Binder	65	HL8
Granular Base	150	Granular 'A'
Granular Sub-base Light-Duty/Parking Heavy-Duty/Fire Route	 300 450	Granular 'B'

In preparation of pavement subgrade, all compressible material should be removed. The final subgrade must be proof-rolled. Any soft spot identified must be rectified by subexcavation and replacing with selected dry inorganic material. The subgrade within 1.0 m below the underside of the granular sub-base must be compacted to at least 98% SPDD, with the water content at 2% to 3% drier than its optimum.

All the granular bases should be compacted in 150 to 200 mm lifts to 100% SPDD.

An intercept subdrain system should be installed along the perimeter of the parking area where surface runoff may drain onto the pavement. In paved areas, catch basins with stub drains in all four directions should be provided. The stub drains and subdrains should drain into the catch basin through filter-sleeved weepers. The invert of the subdrains should be at least 0.4 m beneath the underside of the granular sub-base and should be backfilled with free-draining granular material.

6.7 Soil Parameters

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

Table 4 - Soil Parameters

Unit Weight and Bulk Factor	**Bulk Unit Weight γ (kN/m^3)**	**Estimated Bulk Factor**	
		Loose	**Compacted**
Silty Clay Till	22.0	1.30	1.05
Earth Fill/Silty Clay	20.5	1.25	1.00
Lateral Earth Pressure Coefficients	**Active K_a**	**At Rest K_o**	**Passive K_p**
Silty Clay	0.39	0.56	2.56
Silty Clay Till	0.33	0.50	3.00
Effective Shear Strength Parameters	**Cohesion c' (kPa)**	**Angle of Internal Friction, ϕ'**	
Silty Clay	5	26°	
Silty Clay Till	5	30°	
Coefficient of Permeability (K) and Percolation Time (T)			
	K (cm/sec)	**T (min/cm)**	
Silty Clay/Silty Clay Till	10^{-7}	100	
Coefficients of Friction			
Between Concrete and Granular Base		0.50	
Between Concrete and Sound Native Soils		0.35	

6.8 Excavation

Excavation should be carried out in accordance with Ontario Regulation 213/91. The types of soils are classified in Table 4.

Table 5 - Classification of Soils for Excavation

Material	**Type**
Sound Silty Clay/Silty Clay Till	2
Earth Fill and Weathered Soils	3

In the silty clay and clay till, any perched groundwater yield can be collected and removed by conventional pumping from sumps. The yield is expected to be small and limited.

Excavation into the till with boulders will require extra effort and the use of properly equipped heavy-duty excavator.

Prospective contractors may be asked to asses the subsurface conditions by digging test pits to the intended depth of excavation. These test pits should be allowed to remain open for a few hours to asses the trenching conditions and the dewatering requirement for excavation.

7.0 LIMITATIONS OF REPORT

This report was prepared by Soil Engineers Ltd. for the account of Bolton Shore Holdings Ltd. and for review by the designated consultants, financial institutions, government agencies and contractors. The material in the report reflects the judgment of Kelvin Hung, P.Eng., and Bernard Lee, P.Eng., in light of the information available to it at the time of preparation.

Use of the report is subject to the conditions and limitations of the contractual agreement. Any use which a Third Party makes of this report, and/or any reliance on decisions to be made based on it is the responsibility of such Third Parties. Soil Engineers Ltd. accepts no responsibility for damages, if any, suffered by any Third Party as a result of decisions made or actions based on this report.

SOIL ENGINEERS LTD.

Kelvin Hung, P.Eng.

Bernard Lee, P.Eng.
KH/BL

LIST OF ABBREVIATIONS AND DESCRIPTION OF TERMS

The abbreviations and terms commonly employed on the borehole logs and figures, and in the text of the report, are as follows:

SAMPLE TYPES

AS Auger sample
CS Chunk sample
DO Drive open (split spoon)
DS Denison type sample
FS Foil sample
RC Rock core (with size and percentage recovery)
ST Slotted tube
TO Thin-walled, open
TP Thin-walled, piston
WS Wash sample

PENETRATION RESISTANCE

Standard Penetration Resistance or 'N' Value:

The number of blows of a 63.5 kg hammer falling from a height of 76 cm required to advance a 51 mm outer diameter drive open sampler 30 cm into undisturbed soil, after an initial penetration of 15 cm.

Plotted as '○'

Dynamic Cone Penetration Resistance:

A continuous profile showing the number of blows per each 30 cm of penetration of a 51 mm diameter, 90° point cone driven by a 63.5 kg hammer falling from a height of 76 cm.

Plotted as '—●—'

WH Sampler advanced by static weight
PH Sampler advanced by hydraulic pressure
PM Sampler advanced by manual pressure
NP No penetration

SOIL DESCRIPTION

Cohesionless Soils:

'N' (blows/30 cm)	Relative Density
0 to 4	very loose
4 to 10	loose
10 to 30	compact
30 to 50	dense
>50	very dense

Cohesive Soils:

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

Method of Determination of Undrained Shear Strength of Cohesive Soils:

x 0.0 Field vane test in borehole; the number denotes the sensitivity to remoulding
△ Laboratory vane test

METRIC CONVERSION FACTORS

1 ft = 0.3048 m
1 inch = 25.4 mm
1 lb = 0.454 kg
1 ksf = 47.88 kPa

JOB NO.: 2404-S107

LOG OF BOREHOLE: 1

FIGURE NO.: 1

PROJECT DESCRIPTION: Proposed 4-Storey Apartment with Basement

METHOD OF BORING: Solid-Stem Augers

PROJECT LOCATION: 15, 21 and 27 Shore Street, Town of Caledon

DRILLING DATE: May 30, 2024

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90 / Shear Strength (kN/m²) 50 100 150 200 / Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL / Moisture Content (%) 10 20 30 40	WATER LEVEL
254.1	Ground Surface							
0.0	7.6 cm TOPSOIL				0			
	Dark brown EARTH FILL silty clay occ. rootlets and topsoil inclusions	1	DO	7			24	
253.3								
0.8	Stiff to hard	2	DO	8	1		23	
		3	DO	31	2		20	
		4	DO	39			21	
	SILTY CLAY	5	DO	45	3		21	
	brown / grey	6	DO	20	4		21	
	traces of sand and gravel occ. silt seams	7	DO	20	5		25	
		8	DO	28	6			
		9	DO	25	7		19	
245.6					8			
8.5	END OF BOREHOLE							
	Installed 50 mm Ø monitoring well to 7.6 m completed with 3.1 m screen Sand backfill from 4.0 m to 7.6 m Bentonite seal from 0.0 m to 4.0 m Provided with a steel monument casing				9			Dry on completion
					10			

Soil Engineers Ltd.

JOB NO.: 2404-S107

LOG OF BOREHOLE: 2

FIGURE NO.: 2

PROJECT DESCRIPTION: Proposed 4-Storey Apartment with Basement

METHOD OF BORING: Solid-Stem Augers

PROJECT LOCATION: 15, 21 and 27 Shore Street, Town of Caledon

DRILLING DATE: May 29, 2024

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90 / Shear Strength (kN/m²) 50 100 150 200 / Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL / Moisture Content (%) 10 20 30 40	WATER LEVEL
254.1	Ground Surface							
0.0	7.6 cm TOPSOIL				0			
	Dark brown EARTH FILL silty clay occ. rootlets and topsoil inclusions	1	DO	10			29	
253.3								
0.8	Stiff to hard	2	DO	9	1		25	
		3	DO	24	2		20	
		4	DO	26			23	
	SILTY CLAY	5	DO	37	3		21	
					4			
	traces of sand and gravel occ. silt seams	6	DO	22	5		25	
	brown / grey	7	DO	14	6		29	
					7			
		8	DO	21	8		20	
246.0								
8.1	END OF BOREHOLE							
	Installed 50 mm Ø monitoring well to 7.6 m completed with 3.1 m screen Sand backfill from 4.0 m to 7.6 m Bentonite seal from 0.0 m to 4.0 m Provided with a steel monument casing				9			Dry on completion
					10			

Soil Engineers Ltd.

JOB NO.: 2404-S107

LOG OF BOREHOLE: 3

FIGURE NO.: 3

PROJECT DESCRIPTION: Proposed 4-Storey Apartment with Basement

METHOD OF BORING: Solid-Stem Augers

PROJECT LOCATION: 15, 21 and 27 Shore Street, Town of Caledon

DRILLING DATE: May 29, 2024

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90 / Shear Strength (kN/m²) 50 100 150 200 / Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL / Moisture Content (%) 10 20 30 40	WATER LEVEL
254.5	Ground Surface							
0.0	10 cm TOPSOIL	1A			0			
	Dark brown EARTH FILL silty clay	1B	DO	5			23	
253.7	occ. rootlets and topsoil inclusions							
0.8	Brown, very stiff SILTY CLAY TILL some sand, a trace of gravel	2	DO	21	1		16	
253.0								
0.8	Very stiff to hard	3	DO	28	2		22	
		4	DO	42			21	
		5	DO	35	3		22	
	SILTY CLAY				4			
	brown / grey	6	DO	23	5		24	
	traces of sand and gravel occ. silt seams				6			
		7	DO	23			22	
					7			
		8	DO	22	8		28	
246.4								
8.1	END OF BOREHOLE							
	Installed 50 mm Ø monitoring well to 7.6 m completed with 3.1 m screen Sand backfill from 4.0 m to 7.6 m Bentonite seal from 0.0 m to 4.0 m Provided with a flush mounted cover				9			Dry on completion
					10			

Soil Engineers Ltd.

JOB NO.: 2404-S107

LOG OF BOREHOLE: 4

FIGURE NO.: 4

PROJECT DESCRIPTION: Proposed 4-Storey Apartment with Basement

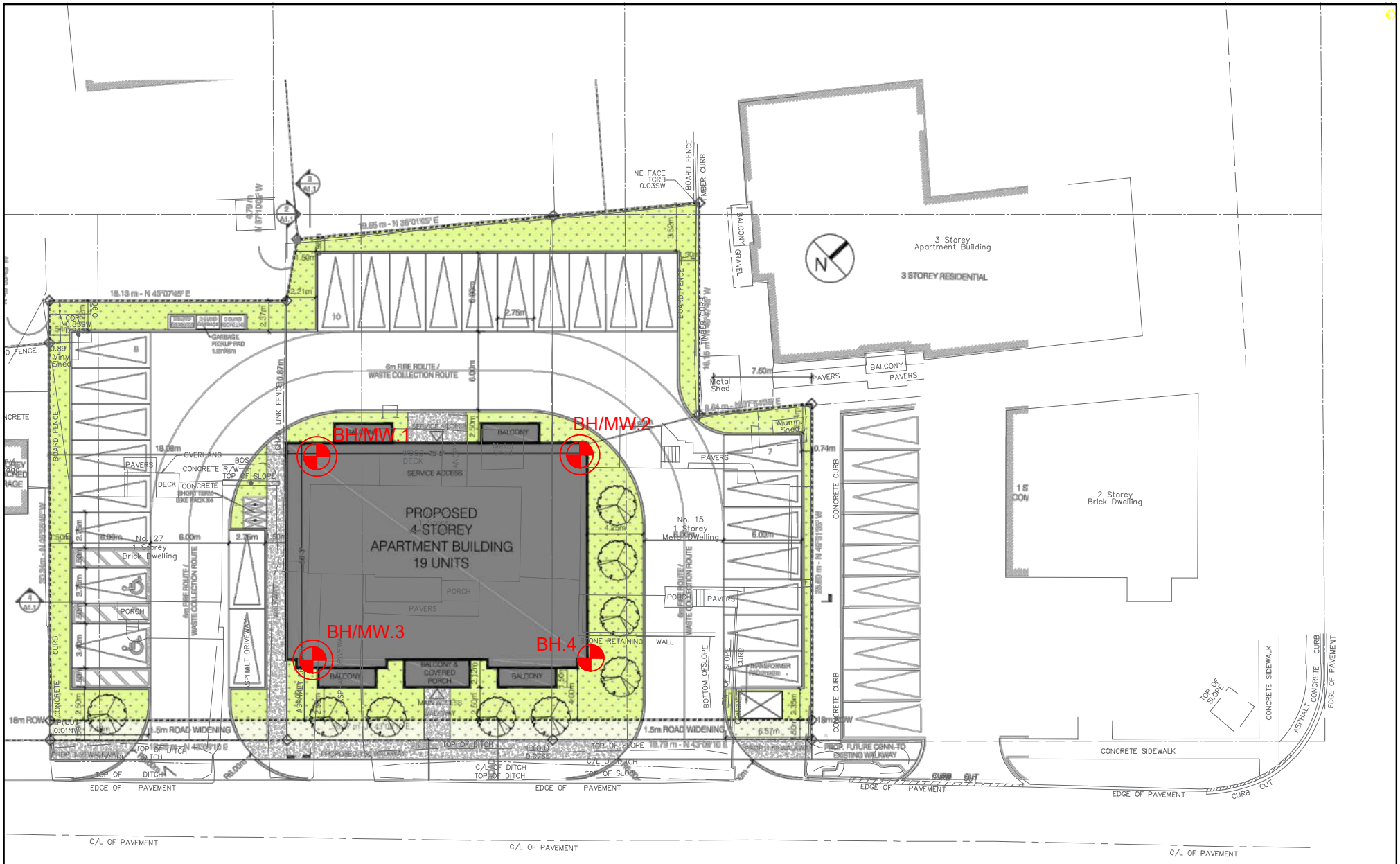
METHOD OF BORING: Solid-Stem Augers

PROJECT LOCATION: 15, 21 and 27 Shore Street, Town of Caledon

DRILLING DATE: May 29, 2024

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90 / Shear Strength (kN/m²) 50 100 150 200 / Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL / Moisture Content (%) 10 20 30 40	WATER LEVEL
254.3	Ground Surface							
0.0	7.6 cm TOPSOIL				0			
	Dark brown EARTH FILL silty clay occ. rootlets and topsoil inclusions	1	DO	8			33	
253.5 / 0.8	Very stiff to hard	2	DO	29	1		20	
		3	DO	31	2		22	
		4	DO	26			23	
	SILTY CLAY	5	DO	22	3		25	
		6	DO	23	4		25	
	traces of sand and gravel occ. silt seams				5			
	brown / grey	7	DO	20	6		26	
		8	DO	23	7		21	
246.2 / 8.1	END OF BOREHOLE				8			
					9			
					10			

Dry on completion



LEGEND

- Borehole with monitoring well
- Borehole

Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL | ENVIRONMENTAL | HYDROGEOLOGICAL | BUILDING SCIENCE
90 WEST BEAVER CREEK ROAD, SUITE #100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL: (416) 754-8515 · FAX: (905) 881-8335

BOREHOLE AND MONITORING WELL LOCATION PLAN

SITE: 15, 21 and 27 Shore Street, Town of Caledon

DESIGNED BY: -	**CHECKED BY:** -	**DWG NO.:** 1	
SCALE: 1:400	**REF. NO.:** 2404-S107	**DATE:** June 2024	**REV** -

APPENDIX C

WATER DEMAND CALCULATIONS

WATER SUPPLY CALCULATION

PROJECT: 15,21,27 Shore Street, Caledon
FILE No.: 24-0002CA
DATE: 2024-06-11
PREPARED BY: DV

Fire Flow (residential)	83.33	L/s
Demand (residential)	280	L/cap/day *
Max Day Factor	2	*
Peak Hour Factor	3	*
Units (Large)	14	
Units (Small)	5	
Density (Large)	3.1	cap / unit *
Density (Small	1.7	cap / unit *
Population	52	

Calculated as per FUS 2020

* Per Region of Peel Water Main Design Criteria (June 2010) - Table 1

AVERAGE-DAY DEMAND

Land Use	Units (Large)	Units (small)	Density (large p/unit)	Density (small p/unit)	Population	Avg. Day Demand (L/s)
Residential	14	5	3.1 p/unit	1.7 p/unit	52	0.17

MAXIMUM-DAY DEMAND + FIRE FLOW

Land Use	Avg. Day Demand (L/s)	Peak Hour Demand Factor*	Peak Hour Demand (L/s)	Max. Day Demand Factor*	Max. Day Demand (L/s)	Max. Day Demand + Fire Flow (L/s)
Residential	0.17	3.0	0.50	2.0	0.34	**83.67**

* Per Region of Peel Water Main Design Criteria (June 2010) - Table 1

FIRE FLOW CALCULATION

PROJECT: 15,21,27 Shore Street, Caledon
FILE No.: 24-0002CA
DATE: 2024-06-11
PREPARED BY: DV

urbanworks ENGINEERING CORPORATION

Calculation of required fire flow is based on the Fire Underwriters Survey (FUS), Water Supply for Fire Protection publication, 2020

$$F = 220C\sqrt{A}$$

Where: F = Requied fire flow (L/min.)

C = Coefficient related to the type of construction

1.5	**Type V**	Wood Frame Construction
0.8	**Type IV-A**	Mass Timber Construction
0.9	**Type IV-B**	Mass Timber Construction
1.0	**Type IV-C**	Mass Timber Construction
1.5	**Type IV-D**	Mass Timber Construction
1.0	**Type III**	Ordinary Construction
0.8	**Type II**	Noncombustible Construction
0.6	**Type I**	Fire Resistive Construction

A = Total floor area (m^2)

Includes all storeys, but excluding basements at least 50% below grade.

* For fire-resistive buildings, consider the 2 largest adjoining floors plus 50% of each of any floors immediately above up to 8, when vertical openings are inadequately protected.

If the vertical openings and exterior vertical communications are properly protected (one hour rating), consider only the area of the largest floor plus 25% of eachh of the 2 immediately adjoining floors.

Adjustments to the calculated fire flow can be made based on occupancy, sprinkler protection and exposure to other structures. The table below summarizes the adjustments made to the basic fire flow demand.

			1		2			3		4		Final Adjusted		
Building	Area "A"	C	Base Fire Flow		Occupancy			Sprinkler		Exposure		Fire Flow		
	(m2)		(L/min)	(L/s)	%	Fire Flow Adjustment (L/min)	Adjusted Flow (L/min)	%	Fire Flow Adjustment (L/min)	%	Fire Flow Adjustment (L/min)	Final Fire Flow (L/min)	Rounded Fire Flow (L/min)	(L/s)
Site	1557.4	1	8682	144.7	-15	-1302	7380	-40	-2952	14%	1033	5461	5000	83.33

Note - Total GFA = 1557.4 square metres

GFA estimated based on site plan prepared by FC Architects Inc., dated February 20, 2024

(2) Occupancy

Non-Combustible	-25%
Limited Combustible	**-15%**
Combustible	no change
Free Burning	15%
Rapid Burning	25%

(3) Spinkler

It is assumed that the building will have a

Sprinkler system (-30%)

Additional credit for standard water supply (-10%)

(4) Exposure - Type III - Unprotected

Face	Distance	Length Height Factor	Exposure Factor
North	30+m	92	0%
East	10.1 to 20 m	69	6%
South	20.1 to 30 m	92	5%
West	20.1 to 30 m	69	3%
Total Exposure (max 75%)			14%

Calculate for all sides and all buildings.

Distance	Max
0 to 3 m	25%
3.1 to 10 m	20%
10.1 to 20 m	15%
20.1 to 30 m	10%
30.1+m	0%

HYDRANT FLOW TEST REPORT

GENERAL INFORMATION:

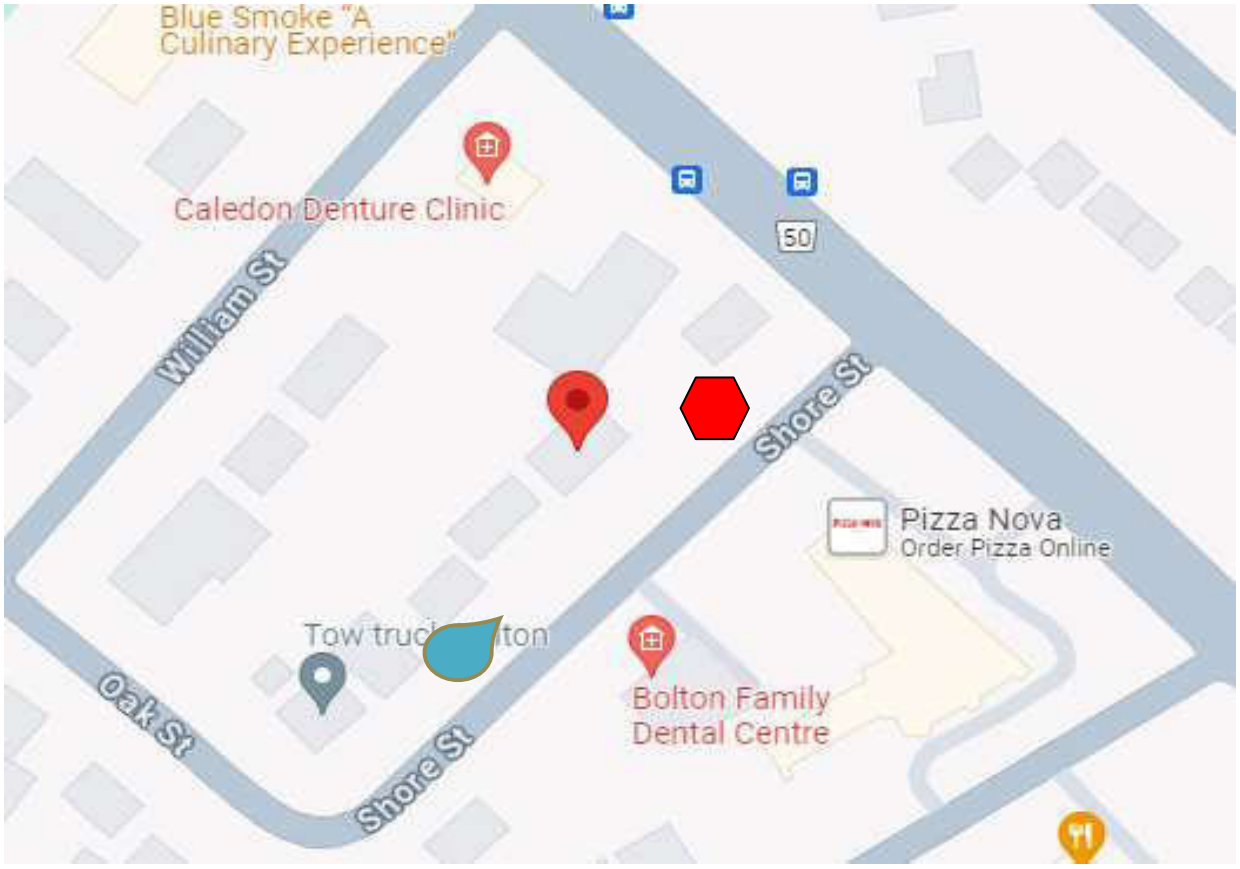
PROJECT ID	1527SS
CLIENT NAME	**Deven Verma**
BUILDING ADDRESS	**15-27 Shore Street** **Caledon, Ontario**

TESTED BY:	AA
DATE	**June 21-24**
TIME	**1:30:00 PM**

WATER MAIN INFORMATION:

MAIN SIZE / MATERIAL	**NA**
CONFIGURATION	**Looped**

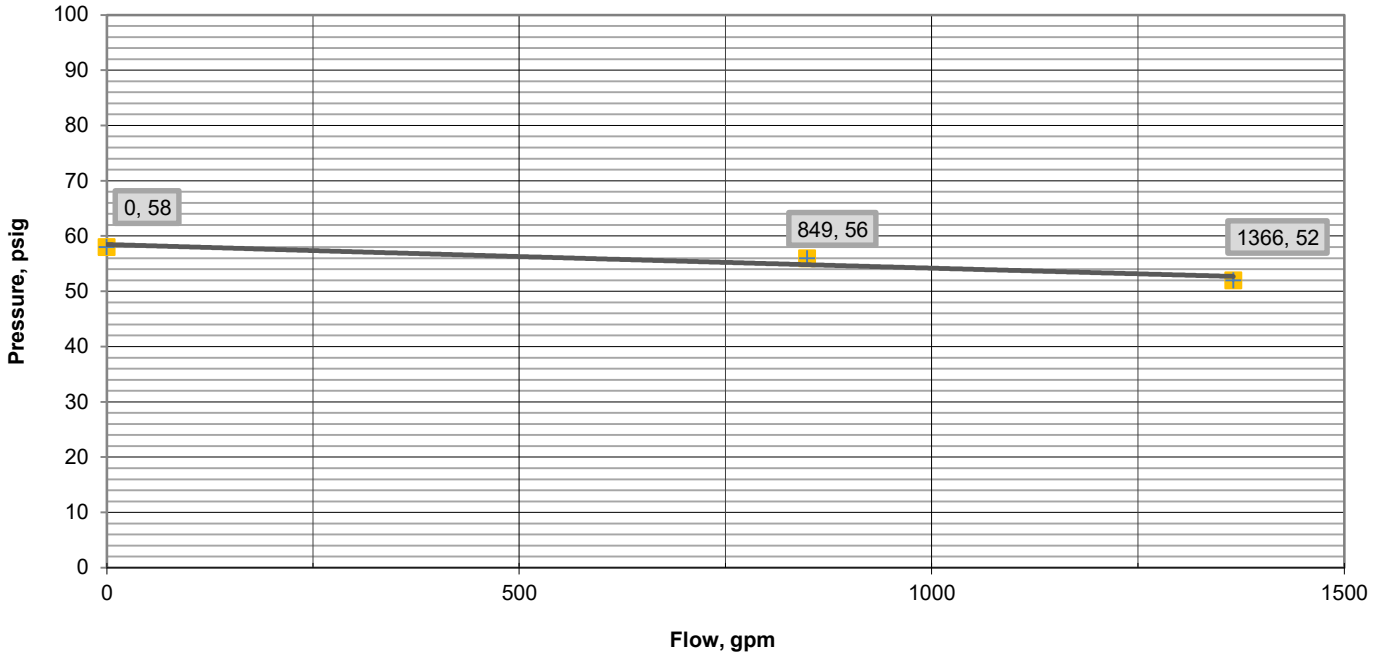
HYDRANT LOCATION:



LEGEND: STATIC HYDRANT RESIDUAL HYDRANT

FINAL RESULTS:

Test #	Number of Outlets	Orifice Size (in)	Pitot Reading (psig)	Equivlnt Flow (usgpm)	Total Flow (usgpm)	Projected flow at 20psi (usgpm)	Gauge Pressure (psig)	Discharg e Coef'nt
Static	N/A	N/A	N/A	N/A	**0**	N/A	**58**	N/A
1	1	2.47	34	849	**849**	4163	**56**	0.8
2	2	2.47	22	683	**1366**	3700	**52**	0.8



Note: Report is in accordance with applicable bylaw standards and NFPA 291 Recommended Practice for Water Flow Testing and Marking of Hydrants

APPENDIX D

SANITARY DEMAND CALCULATIONS AND SINGLE USE DEMAND TABLE

SANITARY DEMAND CALCULATIONS

Region of Peel
SANITARY SEWER DESIGN SHEET - EXISTING CONDITIONS

urbanworks ENGINEERING CORPORATION

Project / Subdivision : 15,21,27 Shore Street, Caledon
Prepared by: Giancarlo Volpe
Last Revised: August 8, 2025

Consulting Engineer : Urbanworks Engineering Corporation
Checked by: Taras Dumyn, P. Eng

Project No.: 24-0002CA

Design Parameters

Manning 'n' =	0.013		Residential Flow Rate	290 L/cap/day (Region)
Infiltration flow =	0.28 L/s/mh	Region of Peel - Linear Waste Water Standards (2023) - Section 2.5.2	Non-Residential Flow Rate	270 L/cap/day (Region)
Extran. Flow (i)=	0.26 L/s/ha	Region of Peel - Linear Waste Water Standards (2023) - Section 2.5.2		
Density (Detached)	4.2 person / unit	Region of Peel - Linear Waste Water Standards (2023) - Table 2-2		
Density (large apartment)	3.1 person / unit	Region of Peel - Linear Waste Water Standards (2023) - Table 2-2		
Density (small apartment)	1.7 person / unit	Region of Peel - Linear Waste Water Standards (2023) - Table 2-2		
Density (Commercial)	50 person / ha	Region of Peel - Linear Waste Water Standards (2023) - Section 2.1.2		

Design Equations

Q(P) = peak population flow (L/s)

Q (P)= (P x q x M) / 86.4 — peak population flow

Q(i)= number of mh * 0.28 l/s/mh — peak infiltration flow

Q(e)= area (ha) * 0.26 l/s/ha — peak extraneous flow

Q(d) = Q(p) + Q(e) + Q(i) — peak design flow

$M = 1 + \frac{14}{4 + (P/1000)^{0.5}}$ min = 2 max = 4

$Qcap = 1/n \times A \times R^{0.67} S^{0.5}$ — Manning's Equation

Notes/Comments:

References: Town of Newmarket, Engineering Design Standards and Criteria, August 2019

Location			Individual Values							Cumulative Values					Flow Data					Sewer Data						
Street	From	To	Gross Area	Commercial area	Detached Units	Large Apartments	Small Aparments	Commercial Population	Residential Population	Commercial Population	Residential Population	Total Population	Area	Peaking Factor	Peak Commercial Flow (L/s)	Peak Residential Flow (L/s)	Peak Infiltration Flow (L/s)	Peak Extraneous Flow (L/s)	Total Design Flow (L/s)	Length	Pipe Size	Type of Pipe	Grade	Full Flow Capacity (Qcap)	Full Flow Velocity	Actual Velocity	%Full
	MH #	MH #	(Ha)	(Ha)	(ea.)	(ea.)	(ea.)	(cap)	(cap)	(ea.)	(cap)	cap	Ha	M	Q(c)	Q(r)	Q (i)	Q(e)	Q(d)	(m)	(mm)		(%)	(L/s)	(m/s)	(m/s)	%
Shore Street	1651195	309324	0.93	0.42	5	-	-	21	21	21	21	42	0.93	4.00	0.26	0.28	0.28	0.24	1.07	120.00	200	PVC	1.40	38.81	1.24	0.53	2.7

Region of Peel
SANITARY SEWER DESIGN SHEET - PROPOSED CONDITIONS

urbanworks ENGINEERING CORPORATION

Project / Subdivision : 15,21,27 Shore Street, Caledon
Prepared by: Giancarlo Volpe
Last Revised: August 8, 2025

Consulting Engineer : Urbanworks Engineering Corporation
Checked by: Taras Dumyn, P. Eng

Project No.: 24-0002CA

Design Parameters

Manning 'n' =	0.013		Residential Flow Rate	290 L/cap/day (Region)
Infiltration flow =	0.28 L/s/mh	Region of Peel - Linear Waste Water Standards (2023) - Section 2.5.2	Non-Residential Flow Rate	270 L/cap/day (Region)
Extran. Flow (i)=	0.26 L/s/ha	Region of Peel - Linear Waste Water Standards (2023) - Section 2.5.2		
Density (Detached)	4.2 person / unit	Region of Peel - Linear Waste Water Standards (2023) - Table 2-2		
Density (large apartment)	3.1 person / unit	Region of Peel - Linear Waste Water Standards (2023) - Table 2-2		
Density (small apartment)	1.7 person / unit	Region of Peel - Linear Waste Water Standards (2023) - Table 2-2		
Density (Commercial)	50 person / ha	Region of Peel - Linear Waste Water Standards (2023) - Section 2.1.2		

Design Equations

Q(P) = peak population flow (L/s)

Q (P)= (P x q x M) / 86.4 peak population flow

Q(i)= number of mh * 0.28 l/s/mh peak infiltration flow

Q(e)= area (ha) * 0.26 l/s/ha peak extraneous flow

Q(d) = Q(p) + Q(e) + Q(i) peak design flow

$M = 1 + \frac{14}{4 + (P/1000)^{0.5}}$ min = 2 max = 4

Qcap = $\frac{1}{n} \times A \times R^{0.67} S^{0.5}$ Manning's Equation

Notes/Comments:

References: Town of Newmarket, Engineering Design Standards and Criteria, August 2019

Location			Individual Values							Cumulative Values					Flow Data					Sewer Data						
Street	From	To	Gross Area	Commercial area	Detached Units	Large Apartments	Small Aparments	Commercial Population	Residential Population	Commercial Population	Residential Population	Total Population	Area	Peaking Factor	Peak Commercial Flow (L/s)	Peak Residential Flow (L/s)	Peak Infiltration Flow (L/s)	Peak Extraneous Flow (L/s)	Total Design Flow (L/s)	Length	Pipe Size	Type of Pipe	Grade	Full Flow Capacity (Qcap)	Full Flow Velocity	Actual Velocity	%Full
	MH #	MH #	(Ha)	(Ha)	(ea.)	(ea.)	(ea.)	(cap)	(cap)	(ea.)	(cap)	cap	Ha	M	Q(c)	Q(r)	Q (i)	Q(e)	Q(d)	(m)	(mm)		(%)	(L/s)	(m/s)	(m/s)	%
Proposed Development	BLDG	Inspection MH	0.21	-	-	14	5	-	52	-	52	52	0.21	4.00	-	0.70	0.28	0.05	1.03	4.00	150	PVC	2.00	21.54	1.22	0.62	4.8
Proposed Development	Inspection MH	Main	-	-	-	-	-	-	-	-	52	52	0.21	4.00	-	0.70	0.28	0.05	1.03	9.00	150	PVC	2.00	21.54	1.22	0.62	4.8
Shore Street	1651195	309324	0.72	0.42	2	-	-	21	8	21	60	81	0.93	4.00	0.26	0.81	0.28	0.24	1.59	120.00	200	PVC	1.40	38.81	1.24	0.60	4.1

=proposed sewer
=existing sewer

SINGLE USE DEMAND TABLE

Water and Wastewater Modelling Demand Table - Site Plan applications

Version - January 2023

		units	persons
Proposed Residential[1)]			
	Singles/Semis	0	0
	townhouses	0	0
	large apartments (>750sqft)	14	44
	small apartments (<=750sqft)	5	8
Total Proposed Residential			52
Proposed Institutional Population[2)]			n/a
Proposed Employment Population[3)]			n/a
Total			52

Proposed GFA (commercial/retail) (sqm)	n/a

WATER CONNECTION

Hydrant flow test		
Hydrant flow test locations [4)]	FH (#6513625) on Shore Street, Caledon	

	Pressure (kPa)	Flow (in l/s)	Time
Minimum water pressure	386.1	53.36	June 21, 2024 1:30 PM
Maximum water pressure	400	0	June 21, 2024 1:30 PM

No.	Water demands				
	Demand type	Demand (in l/s) Use 1 [6)]	Use 2 [6)]	Use 3 [6)]	Total
1	Average day flow	0.17			0.17
2	Maximum day flow	0.34			0.34
3	Peak hour flow	0.50			0.50
4	Fire flow [5)]	83.33			83.33
Analysis					
5	Maximum day plus fire flow	83.67			83.67

WASTEWATER CONNECTION

		Discharge Location[7)]	Flow
6	Wastewater sewer effluent (in l/s)	new connection between EX. MH 1651195 and 309324	1.03
7	Wastewater sewer effluent (in l/s)		
8	Wastewater sewer effluent (in l/s)		
9	Total Wastewater sewer effluent (in l/s)		1.03

[1)] For the design flow calculations, please consider the following PPU's, which are found in the Region of Peel 2020 DC Background Study

☐Singles/Semi – 4.2

☐Multiples (Townhouses) – 3.4
☐Large Apartments (larger than 750 square feet) – 3.0
☐Small Apartments (equal to or less than 750 square feet) – 1.6

[2)] refer to Region of Peel design criteria

[3)] For the commercial and industrial design flow calculations, please use your site specific estimated population or the most current Ontario Building Code Occupant Load determination

[4)] Please include the graphs associated with the hydrant flow test information table

[4)] Hydrant flow tests should be performed within 2 years of submisison to the Region.
The Region will not permit hydrant flow tests during the winter, please check with the Region for scheduling

[5)] Please reference the Fire Underwriters Survey Document

[6)] Please identify the flows for each use type, if applicable

[7)] Please include drainage plan for mutliple discharge locations

The calculations should be based on the development proposal
All required calculations must be submitted with the demand table submission
Table shall include Professional Engineer's signature and stamp
Site servicing concept shall be included

This table will be deemed complete when all the above is submitted and/or included. Modelling will commence with a complete table.

APPENDIX E

STORMWATER MANAGEMENT CALCULATIONS AND JELLYFISH SIZING REPORT

SITE CHARACTERISTICS

PROJECT: 15,21,27 Shore Street, Caledon
FILE No.: 24-0002CA
DATE: 2025-08-08
PREPARED BY: GV

urbanworks ENGINEERING CORPORATION

Pre-Development

Land Use	Impervious Ratio (I)	Area 101 (m^2)	Area 102 (m^2)	Area 103 (m^2)	Total Drainage Area (m2)	% Coverage
Asphalt Driveway	1	171.00	19.30	0.00	190.30	9.2%
Roof	1	372.60	0	17.16	389.76	18.9%
Hardscape	1	36.00	0	79.36	115.36	5.6%
Landscape	0	522.62	37	811.78	1371.40	66.3%
Total		1102.22	56.60	908.30	2067.12	100%
		1102.22				
	Impervious Ratio	0.53	0.34	0.11	0.34	
Runoff coefficient = (0.25*(1- i)+ 0.9i) =		0.59	0.47	0.32	0.47	

Post-Development

Land Use	Impervious Ratio (I)	Area 201 (m^2)	Area 202 (m^2)	Area 203 (m^2)	Total Drainage Area (m2)	% Coverage
Asphalt Driveway	1	1055.04	0	0.00	1055.04	51.0%
Roof	1	423.43	0	0.00	423.43	20.5%
Hardscape	1	114.48	8.00	0.00	122.48	5.9%
Landscape	0	358.74	48.60	58.83	466.17	22.6%
Total		1951.69	56.60	58.83	2067.12	100.0%
	Impervious Ratio	0.82	0.14	0.00	0.77	
Runoff coefficient = (0.25 * (1- i)+ 0.9i) =		0.78	0.34	0.25	0.75	

PEAK FLOW CALCULATION: RATIONAL METHOD

PRE-DEVELOPMENT CONDITION

PROJECT: 15,21,27 Shore Street, Caledon
FILE No.: 24-0002CA
DATE: 2025-08-08
PREPARED BY: GV

IDF DATA SET:	Town Std. No. 103		
STORM	COEFFICIENTS		
EVENT	A	B	C
2 YR.	1070	0.8759	7.85
5 YR.	1593	0.8789	11
10 YR.	2221	0.908	12
25 YR.	3158	0.9335	15
50 YR.	3886	0.9495	16
100 YR.	4688	0.9624	17

Rainfall Intensity: $I = A * B / (t_{c+C})$

Peak Flow: $Q = 0.00278 * (C \cdot I \cdot A)$

Min. Inlet Time: *Tc = 10.0min.*

STORM EVENT	AREA I.D.	AREA (ha)	C	A x C	Tc (min.)	I (mm/hr)	Q (m^3/s)	Q (L/s)
2 YR.	101	0.1102	0.59	0.065	10.0	52.5	0.0095	9.5
5 YR.	101	0.1102	0.59	0.065	10.0	66.7	0.0121	12.1
10 YR.	101	0.1102	0.59	0.065	10.0	91.7	0.0166	16.6
25 YR.	101	0.1102	0.59	0.065	10.0	117.9	0.0214	21.4
50 YR.	101	0.1102	0.59	0.065	10.0	141.9	0.0257	25.7
100 YR.	101	0.1102	0.59	0.065	10.0	167.1	0.0303	30.3

STORM EVENT	AREA I.D.	AREA (ha)	C	A x C	Tc (min.)	I (mm/hr)	Q (m^3/s)	Q (L/s)
2 YR.	102	0.0057	0.47	0.003	10.0	52.5	0.000	0.4
5 YR.	102	0.0057	0.47	0.003	10.0	66.7	0.000	0.5
10 YR.	102	0.0057	0.47	0.003	10.0	91.7	0.001	0.7
25 YR.	102	0.0057	0.47	0.003	10.0	117.9	0.001	0.9
50 YR.	102	0.0057	0.47	0.003	10.0	141.9	0.001	1.1
100 YR.	102	0.0057	0.47	0.003	10.0	167.1	0.001	1.2

STORM EVENT	AREA I.D.	AREA (ha)	C	A x C	Tc (min.)	I (mm/hr)	Q (m^3/s)	Q (L/s)
2 YR.	103	0.0908	0.32	0.029	10.0	52.5	0.0042	4.2
5 YR.	103	0.0908	0.32	0.029	10.0	66.7	0.0054	5.4
10 YR.	103	0.0908	0.32	0.029	10.0	91.7	0.0074	7.4
25 YR.	103	0.0908	0.32	0.029	10.0	117.9	0.0095	9.5
50 YR.	103	0.0908	0.32	0.029	10.0	141.9	0.0114	11.4
100 YR.	103	0.0908	0.32	0.029	10.0	167.1	0.0135	13.5

PEAK FLOW CALCULATION: RATIONAL METHOD

POST-DEVELOPMENT CONDITION

PROJECT: 15,21,27 Shore Street, Caledon
FILE No.: 24-0002CA
DATE: 2025-08-08
PREPARED BY: GV

IDF DATA SET:	Town Std. No. 103		
STORM	COEFFICIENTS		
EVENT	A	B	C
2 YR.	1070	0.8759	7.85
5 YR.	1593	0.8789	11
10 YR.	2221	0.908	12
25 YR.	3158	0.9335	15
50 YR.	3886	0.9495	16
100 YR.	4688	0.9624	17

Rainfall Intensity: $I = A * B / (t_{c+C})$
Peak Flow: $Q = 0.00278 * (C \cdot I \cdot A)$
Min. Inlet Time: *Tc = 10.0min.*

STORM EVENT	AREA I.D.	AREA (ha)	C	A x C	Tc (min.)	I (mm/hr)	Q (m^3/s)	Q (L/s)
2 YR.	201	0.1952	0.78	0.152	10.0	52.5	0.0222	22.2
5 YR.	201	0.1952	0.78	0.152	10.0	66.7	0.0282	28.2
10 YR.	201	0.1952	0.78	0.152	10.0	91.7	0.0388	38.8
25 YR.	201	0.1952	0.78	0.152	10.0	117.9	0.0499	49.9
50 YR.	201	0.1952	0.78	0.152	10.0	141.9	0.0601	60.1
100 YR.	201	0.1952	0.78	0.152	10.0	167.1	0.0707	70.7

STORM EVENT	AREA I.D.	AREA (ha)	C	A x C	Tc (min.)	I (mm/hr)	Q (m^3/s)	Q (L/s)
2 YR.	202	0.0057	0.34	0.002	10.0	52.5	0.000	0.3
5 YR.	202	0.0057	0.34	0.002	10.0	66.7	0.000	0.4
10 YR.	202	0.0057	0.34	0.002	10.0	91.7	0.000	0.5
25 YR.	202	0.0057	0.34	0.002	10.0	117.9	0.001	0.6
50 YR.	202	0.0057	0.34	0.002	10.0	141.9	0.001	0.8
100 YR.	202	0.0057	0.34	0.002	10.0	167.1	0.001	0.9

STORM EVENT	AREA I.D.	AREA (ha)	C	A x C	Tc (min.)	I (mm/hr)	Q (m^3/s)	Q (L/s)
2 YR.	203	0.0059	0.25	0.001	10.0	52.5	0.0002	0.2
5 YR.	203	0.0059	0.25	0.001	10.0	66.7	0.0003	0.3
10 YR.	203	0.0059	0.25	0.001	10.0	91.7	0.0004	0.4
25 YR.	203	0.0059	0.25	0.001	10.0	117.9	0.0005	0.5
50 YR.	203	0.0059	0.25	0.001	10.0	141.9	0.0006	0.6
100 YR.	203	0.0059	0.25	0.001	10.0	167.1	0.0007	0.7

ALLOWABLEL RELEASE RATE CALCULATION: RATIONAL METHOD

Uncontrolled Flow

PROJECT: 15,21,27 Shore Street, Caledon
FILE No.: 24-0002CA
DATE: 2025-08-08
PREPARED BY: GV

=Total Target Flow rate in proposed conditions

urbanworks ENGINEERING CORPORATION

Existing Conditions - All existing areas are uncontrolled

	To Shore Street	To Shore Street	To North P/L	
Return Period	**Area A101 (L/s)**	**Area A102 (L/s)**	**Area A103 (L/s)**	**Total Flow (L/s)**
2	9.5	0.4	4.2	14.1
5	12.1	0.5	5.4	17.9
10	16.6	0.7	7.4	24.7
25*	21.4	0.9	9.5	31.7
50*	25.7	1.1	11.4	38.2
100*	30.3	1.2	13.5	45.0

Proposed Conditions

	To Shore Street	To Shore Street	To North P/L	
		Uncontrolled	Uncontrolled	
Return Period	**Area A201 (L/s)**	**Area A202 (L/s)**	**Area A203 (L/s)**	**Total Flow (L/s)**
2	22.2	0.3	0.2	22.5
5	28.2	0.4	0.3	28.6
10	38.8	0.5	0.4	39.3
25*	49.9	0.6	0.5	50.5
50*	60.1	0.8	0.6	60.8
100*	70.7	0.9	0.7	71.6

Allowable Release Rate

$Q = Q_{PRE} - Q_U$

Where: Q = Allowable Post-Development Release Rate (100-Year)
Q_{PRE} = Pre-Development Flow (5-Year)
Q_U = Post-Development Uncontrolled Flow (100-Year)

5-YEAR Pre-Development Flow to Shore Street (Areao 101 + 102) Q_{PRE} (L/s)	**100-YEAR Post-Development Uncontrolled Flows to Shore Street (Area 202) Q_U (L/s)**	**Allowable Post-Dev. Release Rate Q (L/s)**
12.6	0.9	11.7

Area 202 and 203 will remain uncontrolled and Area 201 will be controlled to a max of 11.7 L/s

ORIFICE CONTROL SIZING CALCULATION

PROJECT: 15,21,27 Shore Street, Caledon
FILE No.: 24-0002CA
DATE: 2025-08-08
PREPARED BY: GV

urbanworks
ENGINEERING CORPORATION

Orifice Control Equation

$$Q = C \bullet A \bullet (2gh)^{0.5}$$

Where: Q = Flow Rate
C = Discharge Coefficient
C (tube) = 0.80
C (plate) = 0.62
A = Orifice Area
g = Acceleration Due to Gravity = 9.81 m/s^2
h = Head

Orifice Diameter (mm)		78	Orifice to come with debris screen (welded wire cage)
High Water Elev. (m)		254.230	
Orifice Invert Elev. (m)		252.910	
Flow Rate (L/s)	Required	15.0	
	Provided	14.9	

Orifice Center Elev. (m)	*252.95*
Head (m)	*1.281*
C	*0.62*
Orifice Area (m^2)	*0.0048*

STORMWATER STORAGE & RELEASE CALCULATION

100-YEAR STORM

PROJECT: 15,21,27 Shore Street, Caledon
FILE No.: 24-0002CA
DATE: 2025-08-08
PREPARED BY: GV

urbanworks ENGINEERING CORPORATION

DRAINAGE AREA I.D.	201		**IDF DATA SET:**	Town Std. No. 103		
DRAINAGE AREA (ha)	0.195		**STORM**	**COEFFICIENTS**		
RUNOFF COEFF. (C)	0.78		**EVENT**	**A**	**B**	**C**
A x C	0.152		2 YR.	1070	0.8759	7.85
TOTAL A x C	0.152		5 YR.	1593	0.8789	11
TIME OF CONCENTRATION	10.0 min.		10 YR	2221	0.908	12
TIME STEP	1.0 min.		25 YR.	3158	0.9335	15
CONTROLLED RELEASE RATE (Q_C)	0.0149 m^3/s		50 YR.	3886	0.9495	16
MAX. STORAGE REQUIRED	46.4 m^3		100 YR.	4688	0.9624	17

T	*I = A * B / (tc + C)*	*Q_R=(C·I·A)* 0.00278*	*$V_R = Q_R \cdot T \cdot 60$*	*$V_C = Q_C \cdot T \cdot 60$*	*$V = V_R - V_C$*
TIME (min.)	**RAINFALL INTENSITY (mm/hr)**	**RUNOFF (m^3/s)**	**RUNOFF VOL. (m^3)**	**CONTROLLED RELEASE VOL. (m^3)**	**STORAGE VOL. (m^3)**
10	167.1	0.071	42.43	8.91	33.51
11	161.1	0.068	45.00	9.80	35.20
12	155.6	0.066	47.40	10.69	36.71
13	150.4	0.064	49.64	11.58	38.05
14	145.5	0.062	51.73	12.48	39.26
15	141.0	0.060	53.69	13.37	40.33
16	136.7	0.058	55.54	14.26	41.28
17	132.7	0.056	57.27	15.15	42.12
18	128.9	0.055	58.91	16.04	42.87
19	125.3	0.053	60.46	16.93	43.52
20	121.9	0.052	61.92	17.82	44.10
21	118.7	0.050	63.30	18.71	44.59
22	115.7	0.049	64.62	19.61	45.01
23	112.8	0.048	65.87	20.50	45.37
24	110.0	0.047	67.05	21.39	45.67
25	107.4	0.045	68.18	22.28	45.91
26	104.9	0.044	69.26	23.17	46.09
27	102.5	0.043	70.29	24.06	46.23
28	100.3	0.042	71.27	24.95	46.32
29	98.1	0.042	72.22	25.84	46.37
30	96.0	0.041	73.12	26.73	46.38
31	94.0	0.040	73.98	27.63	46.35
32	92.1	0.039	74.81	28.52	46.29
33	90.2	0.038	75.60	29.41	46.19
34	88.5	0.037	76.37	30.30	46.07
35	86.8	0.037	77.10	31.19	45.91
36	85.1	0.036	77.81	32.08	45.73
37	83.6	0.035	78.49	32.97	45.51
38	82.0	0.035	79.14	33.86	45.28
39	80.6	0.034	79.77	34.75	45.02
40	79.2	0.033	80.38	35.65	44.74
41	77.8	0.033	80.97	36.54	44.44
42	76.5	0.032	81.54	37.43	44.11
43	75.2	0.032	82.09	38.32	43.77
44	74.0	0.031	82.63	39.21	43.41
45	72.8	0.031	83.14	40.10	43.04
46	71.6	0.030	83.64	40.99	42.65
47	70.5	0.030	84.12	41.88	42.24
48	69.4	0.029	84.59	42.77	41.81
49	68.4	0.029	85.04	43.67	41.38

PROVIDED UNDERGROUND STORAGE CALCULATION

PROJECT: 15,21,27 Shore Street, Caledon
FILE No.: 24-0002CA
DATE: 2025-08-08
PREPARED BY: DV

urbanworks ENGINEERING CORPORATION

100-YEAR MAX. STORAGE REQUIRED	46.4	m^3	(100 Year)
100-YEAR SURFACE STORAGE PROVIDED	55.5	m^3	(ELEV. 254.25)

Note:
Underground pipe and structure storage not used in surface storage calculations

Structure ID	CB1	CBMH1	CBMH2
Top of Grate	**254.15**	**254.00**	**253.95**
Max Ponding Elev.	**254.23**	**254.23**	**254.23**
Max Ponding Depth	**0.08**	**0.23**	**0.28**
Max Ponding Area (m2)	194.50	295.04	243.83

	CB1	CBMH1	CBMH2	Total Surface	
Flood Elev. (m)	Surface Storage Volume (m^3)	Surface Storage Volume (m^3)	Surface Storage Volume (m^3)	Storage Volume (m^3)	Notes
253.950	0.0	0.0	0.0	0.00	
253.960	0.0	0.0	0.8	0.81	
253.970	0.0	0.0	1.6	1.63	
253.980	0.0	0.0	2.4	2.44	
253.990	0.0	0.0	3.3	3.25	
254.000	0.0	0.0	4.1	4.06	
254.010	0.0	1.0	4.9	5.86	
254.020	0.0	2.0	5.7	7.66	
254.030	0.0	3.0	6.5	9.45	
254.040	0.0	3.9	7.3	11.25	
254.050	0.0	4.9	8.1	13.05	
254.060	0.0	5.9	8.9	14.84	
254.070	0.0	6.9	9.8	16.64	
254.080	0.0	7.9	10.6	18.43	
254.090	0.0	8.9	11.4	20.23	
254.100	0.0	9.8	12.2	22.03	
254.110	0.0	10.8	13.0	23.82	
254.120	0.0	11.8	13.8	25.62	
254.130	0.0	12.8	14.6	27.42	
254.140	0.0	13.8	15.4	29.21	
254.150	0.0	14.8	16.3	31.01	
254.160	0.6	15.7	17.1	33.45	
254.170	1.3	16.7	17.9	35.90	
254.180	1.9	17.7	18.7	38.34	
254.190	2.6	18.7	19.5	40.79	
254.200	3.2	19.7	20.3	43.23	
254.210	3.9	20.7	21.1	45.68	
254.220	4.5	21.6	21.9	48.12	
254.230	5.2	22.6	22.8	50.56	
254.240	5.8	23.6	23.6	53.01	
254.250	6.5	24.6	24.4	55.45	*100-Year Storage Level*

EROSION CONTROL / INFILTRATION CALCULATIONS

PROJECT: 15,21,27 Shore Street, Caledon
FILE No.: 24-0002CA
DATE: 2025-08-08
PREPARED BY: GV

urbanworks ENGINEERING CORPORATION

Erosion Control / Infiltration Target Volume Calculations

Minimum 5mm volume from the new impervious areas is required to be infiltrated throughout the development
Runoff from roof leaders to be directed to infiltration facility

Post-development impervious surfaces = 1593.0 m^2

Erosion Control Volume Required = Total Impervious Drainage Area X 5mm
Erosion Control Volume Required = 8.0 m^3

Total Volume Retained On Site = LID Storage

Storage provided by Undergronud Tank = 9.0 m^3
Total Volume Retained = 9.0 m^3

Storage Facility LID Volume Calculations - Landscaping / Irrigation Use

Drawdown Time

I = infiltration rate* = 6 mm/hr
*As reported by Geotechnical Report by Soil Engineers Ltd, dated July 19, 2024

Infitlration rate is insufficient to support any infiltration measure within the site. Erosion Control to be accomplished with water re-use.

d = height of chambers = 350 mm
Vr = storage void ratio = 0.96

$t = d V_r / i$
t = 56.00 hours

more than the maximum draw down time of 48 hours.

Rain Water Harvesting - Landscaping/Irrigation or Grey Water Re-use

Graf Platin 3000 L (x3) Rainwater Cistern= 9 m^3 Higher than the required Erosion Control Volume of 8.0 m3

STORM SEWER DESIGN SHEET

15, 21, 27 SHORE STREET

TOWN OF CALEDON

PROJECT No.: 24-0002CA
DATE: 2024-09-23
DESIGNED BY: Deven Verma
CHECKED BY: Giancarlo Volpe

		Coeff.	5-Year
Rainfall Intensity:	I = A * B / (tc + C)	a	1593
Design Flow:	Q = (C · I · A)*2.778	b	0.8789
Pipe Roughness:	n = 0.013	c	11.000
Min. Inlet Time:	t = 10.0min.		

System to be designed for 5 year Storm

: Indicates controlled flow after orifice in outlet structure

AREA I.D.	LOCATION	STRUCTURE FROM	STRUCTURE TO	AREA (ha) A	RUNOFF COEFF. C_5	AREA RUNOFF A x C_5	RUNOFF ACCUM. A x C_5	Tc (min.)	RAINFALL INT.(mm/hr) i_5	DESIGN FLOWS 5-YEAR FLOW Q_5 (m^3/s)	DESIGN FLOWS TOTAL Q_{TOTAL} (m^3/s)	PIPE DATA LENGTH (m)	SIZE (mm)	PIPE MAT.	GRADE (%)	Q_{FULL} (m^3/s)	V_{FULL} (m/s)	V_{ACTUAL} (m/s)	TIME SECT. TIME (min.)	TIME TOTAL TIME (min.)	CAPACITY CHECK Q_{DESIGN} / Q_{FULL}	COMMENTS
	SITE	CB1	CBMH1	0.11	0.82	0.089	0.089	10	66.67	0.017	0.017	30.4	200	PVC	0.50	0.023	0.74	0.83	0.69	10.69	71.2%	
		CBMH1	CBMH2 (CONTROL)	0.05	0.74	0.035	0.124	10.69	64.56	0.022	0.022	19.5	250	PVC	0.30	0.033	0.66	0.74	0.49	11.18	68.2%	
		CBMH2 (CONTROL)	DIVERSION MH	0.05	0.64	0.030	0.154	11.18	63.13	0.010	0.010	12.6	250	PVC	0.30	0.033	0.66	0.53	0.32	11.49	31.9%	FLOW CONTROLLED TO 10.4 L/s AFTER ORIFICE PLATE
		DIVERSION MH	PR. STM MH 2				0.154	11.49	62.24	0.010	0.010	12.2	250	PVC	0.30	0.033	0.66	0.53	0.31	11.80	31.9%	

Urbanworks Engineering Corporation
1945 Dundas Str. E., Suite 200, Mississauga, ON L4X 2T8
t. 416.503.4500 | e. general@urbanworkseng.com

STORM SEWER DESIGN SHEET

SHORE STREET

TOWN OF CALEDON

urbanworks ENGINEERING CORPORATION

PROJECT No.: 24-0002CA
DATE: 2024-09-23
DESIGNED BY: Deven Verma
CHECKED BY: Giancarlo Volpe

		Coeff.	5-Year
Rainfall Intensity:	*I = A * B / (tc + C)*	*a*	*1593*
Design Flow:	*Q = (C · I · A)*2.778*	*b*	*0.8789*
Pipe Roughness:	*n = 0.013*	*c*	*11.000*
Min. Inlet Time:	*t = 10.0min.*		

System to be designed for *5 year Storm*

: Indicates controlled flow after orifice in outlet structure

AREA I.D.	LOCATION	STRUCTURE FROM	STRUCTURE TO	AREA (ha) A	RUNOFF COEFF. C_5	AREA RUNOFF A x C_5	RUNOFF ACCUM. A x C_5	Tc (min.)	RAINFALL INT.(mm/hr) i_5	DESIGN FLOWS 5-YEAR FLOW Q_5 (m^3/s)	DESIGN FLOWS TOTAL Q_{TOTAL} (m^3/s)	PIPE DATA LENGTH (m)	PIPE DATA SIZE (mm)	PIPE DATA PIPE MAT.	PIPE DATA GRADE (%)	PIPE DATA Q_{FULL} (m^3/s)	PIPE DATA V_{FULL} (m/s)	PIPE DATA V_{ACTUAL} (m/s)	TIME SECT. TIME (min.)	TIME TOTAL TIME (min.)	CAPACITY CHECK Q_{DESIGN} / Q_{FULL}	COMMENTS
301	SHORE STREET	PR. STM MH 1	PR. STM MH 2	0.37	0.77	0.286	0.286	10	66.67	0.053	0.053	82.1	375	PVC	0.35	0.104	0.94	0.95	1.46	11.46	51.0%	
302	SUBJECT PROPERTY	SITE	PR. STM MH2	0.21	-	-	-	-	-	0.010	0.010	12.2	250	PVC	0.30	0.033	0.66	0.53	-	-	31.9%	Flow from subject property controlled to 10.4 L/s
303	SHORE STREET	PR. STM MH 2	CBMH RR050-0347	0.34	0.77	0.261	0.547	11.46	62.34	0.105	0.105	39.5	375	PVC	0.40	0.111	1.00	1.10	0.66	12.11	94.8%	

Urbanworks Engineering Corporation
1945 Dundas Str. E., Suite 200, Mississauga, ON L4X 2T8
t. 416.503.4500 | e. general@urbanworkseng.com

JELLYFISH SIZING REPORT

STANDARD OFFLINE
Jellyfish Filter Sizing Report

Project Information

Date	Thursday, September 12, 2024
Project Name	15, 21, 27 Shore St.
Project Number	24-0002CA
Location	Caledon

Jellyfish Filter Design Overview

This report provides information for the sizing and specification of the Jellyfish Filter. When designed properly in accordance to the guidelines detailed in the Jellyfish Filter Technical Manual, the Jellyfish Filter will exceed the performance and longevity of conventional horizontal bed and granular media filters.

Please see ***www.ImbriumSystems.com*** *for more information.*

Jellyfish Filter System Recommendation

The Jellyfish Filter model JF4-1-1 is recommended to meet the water quality objective by treating a flow of 7.6 L/s, which meets or exceeds 90% of the average annual rainfall runoff volume based on 18 years of TORONTO CENTRAL rainfall data for this site. This model has a sediment capacity of 85 kg, which meets or exceeds the estimated average annual sediment load.

Jellyfish Model	Number of High-Flo Cartridges	Number of Draindown Cartridges	Manhole Diameter (m)	Treatment Flow Rate (L/s)	Sediment Capacity (kg)
JF4-1-1	1	1	1.2	7.6	85

The Jellyfish Filter System

The patented Jellyfish Filter is an engineered stormwater quality treatment technology featuring unique membrane filtration in a compact stand-alone treatment system that removes a high level and wide variety of stormwater pollutants. Exceptional pollutant removal is achieved at high treatment flow rates with minimal head loss and low maintenance costs. Each lightweight Jellyfish Filter cartridge contains an extraordinarily large amount of membrane surface area, resulting in superior flow capacity and pollutant removal capacity.

Maintenance

Regular scheduled inspections and maintenance is necessary to assure proper functioning of the Jellyfish Filter. The maintenance interval is designed to be a minimum of 12 months, but this will vary depending on site loading conditions and upstream pretreatment measures. Quarterly inspections and inspections after all storms beyond the 5-year event are recommended until enough historical performance data has been logged to comfortably initiate an alternative inspection interval.

Please see ***www.ImbriumSystems.com*** *for more information.*

Thank you for the opportunity to present this information to you and your client.

Performance

Jellyfish efficiently captures a high level of Stormwater pollutants, including:

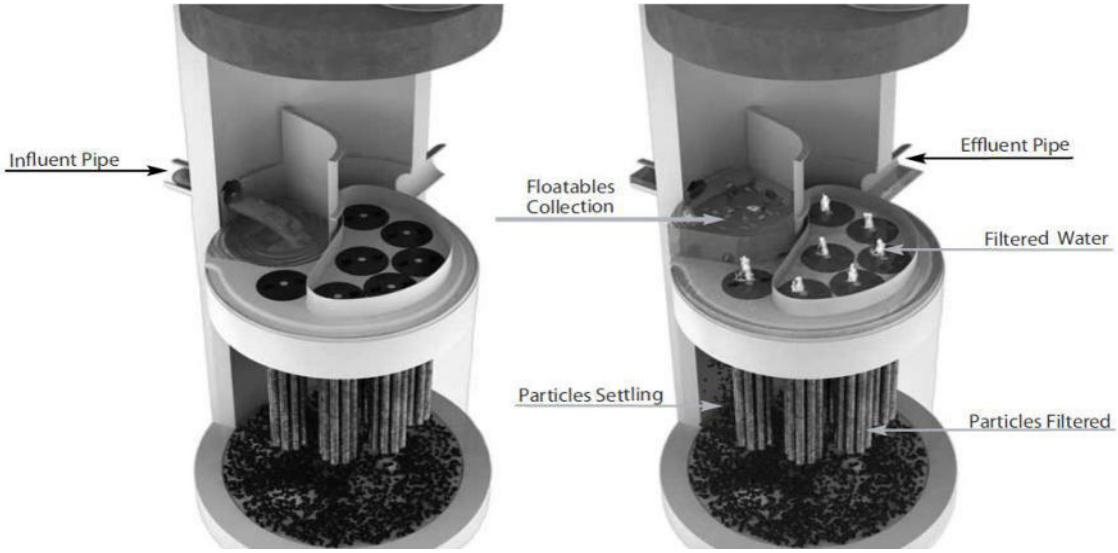
- ☑ 89% of the total suspended solids (TSS) load, including particles less than 5 microns
- ☑ 77% TP removal & 51% TN removal
- ☑ 90% Total Copper, 81% Total Lead, 70% Total Zinc
- ☑ Particulate-bound pollutants such as nutrients, toxic metals, hydrocarbons and bacteria
- ☑ Free oil, Floatable trash and debris

Field Proven Peformance

The Jellyfish filter has been field-tested on an urban site with 25 TAPE qualifying rain events and field monitored according to the TAPE field test protocol, demonstrating:

- A median TSS removal efficiency of 90%, and a median SSC removal of 99%;
- The ability to capture fine particles as indicated by an effluent d50 median of 3 microns for all monitotred storm events, and a median effluent turbidity of 5 NTUs;
- A median Total Phosphorus removal of 77%, and a median Total Nitrogen removal of 51%.

Jellyfish Filter Treatment Functions



Pre-treatment and Membrane Filtration

Jellyfish® Filter

Project Information

Date:	Thursday, September 12, 2024
Project Name:	15, 21, 27 Shore St.
Project Number:	24-0002CA
Location:	Caledon

Designer Information

Company:	Urbanworks Engineering Corp.
Contact:	Deven Verma
Phone #:	

Notes

Rainfall

Name:	TORONTO CENTRAL
State:	ON
ID:	100
Record:	1982 to 1999
Co-ords:	45°30'N, 90°30'W

Drainage Area

Total Area:	0.2 ha
Imperviousness:	82%

Upstream Detention

Peak Release Rate:	n/a
Pretreatment Credit:	n/a

Design System Requirements

Flow Loading	90% of the Average Annual Runoff based on 18 years of TORONTO CENTRAL rainfall data:	**5.2 L/s**
Sediment Loading	Treating 90% of the average annual runoff volume, 988 m³, with a suspended sediment concentration of 60 mg/L.	**59 kg**

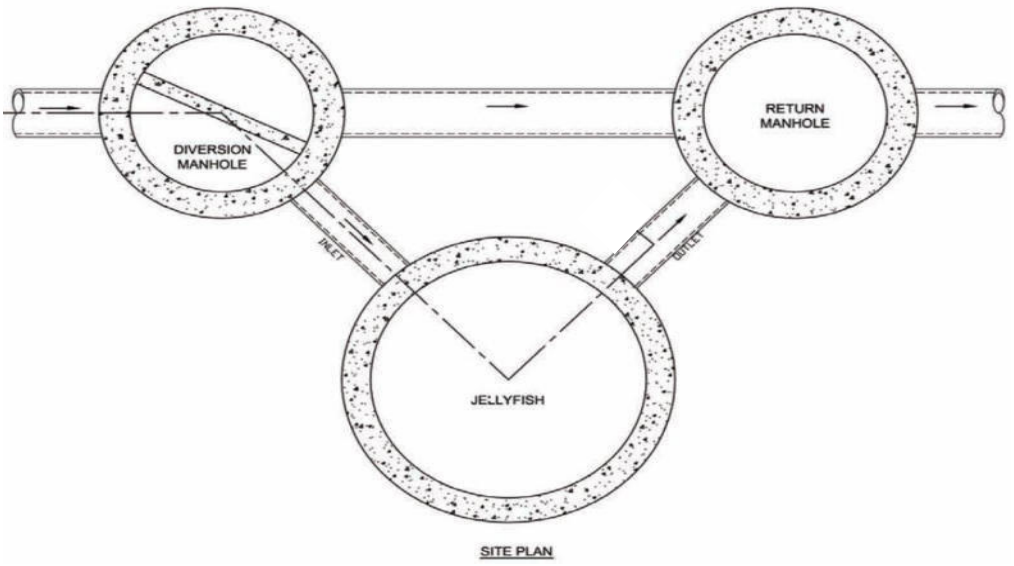
Recommendation

The Jellyfish Filter model JF4-1-1 is recommended to meet the water quality objective by treating a flow of 7.6 L/s, which meets or exceeds 90% of the average annual rainfall runoff volume based on 18 years of TORONTO CENTRAL rainfall data for this site. This model has a sediment capacity of 85 kg, which meets or exceeds the estimated average annual sediment load.

Jellyfish Model	Number of High-Flo Cartridges	Number of Draindown Cartridges	Manhole Diameter (m)	Wet Vol Below Deck (L)	Sump Storage (m³)	Oil Capacity (L)	Treatment Flow Rate (L/s)	Sediment Capacity (kg)
JF4-1-1	**1**	**1**	**1.2**	**2313**	**0.34**	**379**	**7.6**	**85**
JF4-2-1	2	1	1.2	2313	0.34	379	12.6	142
JF6-3-1	3	1	1.8	5205	0.79	848	17.7	199
JF6-4-1	4	1	1.8	5205	0.79	848	22.7	256
JF6-5-1	5	1	1.8	5205	0.79	848	27.8	313
JF6-6-1	6	1	1.8	5205	0.79	848	28.6	370
JF8-6-2	6	2	2.4	9252	1.42	1469	35.3	398
JF8-7-2	7	2	2.4	9252	1.42	1469	40.4	455
JF8-8-2	8	2	2.4	9252	1.42	1469	45.4	512
JF8-9-2	9	2	2.4	9252	1.42	1469	50.5	569
JF8-10-2	10	2	2.4	9252	1.42	1469	50.5	626
JF10-11-3	11	3	3.0	14456	2.21	2302	63.1	711
JF10-12-3	12	3	3.0	14456	2.21	2302	68.2	768
JF10-12-4	12	4	3.0	14456	2.21	2302	70.7	796
JF10-13-4	13	4	3.0	14456	2.21	2302	75.7	853
JF10-14-4	14	4	3.0	14456	2.21	2302	78.9	910
JF10-15-4	15	4	3.0	14456	2.21	2302	78.9	967
JF10-16-4	16	4	3.0	14456	2.21	2302	78.9	1024
JF10-17-4	17	4	3.0	14456	2.21	2302	78.9	1081
JF10-18-4	18	4	3.0	14456	2.21	2302	78.9	1138
JF10-19-4	19	4	3.0	14456	2.21	2302	78.9	1195
JF12-20-5	20	5	3.6	20820	3.2	2771	113.6	1280
JF12-21-5	21	5	3.6	20820	3.2	2771	113.7	1337
JF12-22-5	22	5	3.6	20820	3.2	2771	113.7	1394
JF12-23-5	23	5	3.6	20820	3.2	2771	113.7	1451
JF12-24-5	24	5	3.6	20820	3.2	2771	113.7	1508
JF12-25-5	25	5	3.6	20820	3.2	2771	113.7	1565
JF12-26-5	26	5	3.6	20820	3.2	2771	113.7	1622
JF12-27-5	27	5	3.6	20820	3.2	2771	113.7	1679

Jellyfish Filter Design Notes

- Typically the Jellyfish Filter is designed in an offline configuration, as all stormwater filter systems will perform for a longer duration between required maintenance services when designed and applied in off-line configurations. Depending on the design parameters, an optional internal bypass may be incorporated into the Jellyfish Filter, however note the inspection and maintenance frequency should be expected to increase above that of an off-line system. Speak to your local representative for more information.



Jellyfish Filter Typical Layout

- Typically, 18 inches (457 mm) of driving head is designed into the system, calculated as the difference in elevation between the top of the diversion structure weir and the invert of the Jellyfish Filter outlet pipe. Alternative driving head values can be designed as 12 to 24 inches (305 to 610mm) depending on specific site requirements, requiring additional sizing and design assistance.
- Typically, the Jellyfish Filter is designed with the inlet pipe configured 6 inches (150 mm) above the outlet invert elevation. However, depending on site parameters this can vary to an optional configuration of the inlet pipe entering the unit below the outlet invert elevation.
- The Jellyfish Filter can accommodate multiple inlet pipes within certain restrictions.
- While the optional inlet below deck configuration offers 0 to 360 degree flexibility between the inlet and outlet pipe, typical systems conform to the following:

Model Diameter (m)	Minimum Angle Inlet / Outlet Pipes	Minimum Inlet Pipe Diameter (mm)	Minimum Outlet Pipe Diameter (mm)
1.2	**62º**	**150**	**200**
1.8	59º	200	250
2.4	52º	250	300
3.0	48º	300	450
3.6	40º	300	450

- The Jellyfish Filter can be built at all depths of cover generally associated with conventional stormwater conveyance systems. For sites that require minimal depth of cover for the stormwater infrastructure, the Jellyfish Filter can be applied in a shallow application using a hatch cover. The general minimum depth of cover is 36 inches (915 mm) from top of the underslab to outlet invert.
- If driving head caclulations account for water elevation during submerged conditions the Jellyfish Filter will function effectively under submerged condtions.
- Jellyfish Filter systems may incorporate grated inlets depending on system configuration.
- For sites with water quality treatment flow rates or mass loadings that exceed the design flow rate of the largest standard Jellyfish Filter manhole models, systems can be designed that hydraulically connect multiple Jellyfish Filters in series or alternatively Jellyfish Vault units can be designed.

STANDARD SPECIFICATION
STORMWATER QUALITY – MEMBRANE FILTRATION TREATMENT DEVICE

PART 1 – GENERAL

1.1 WORK INCLUDED

Specifies requirements for construction and performance of an underground stormwater quality membrane filtration treatment device that removes pollutants from stormwater runoff through the unit operations of sedimentation, floatation, and membrane filtration.

1.2 REFERENCE STANDARDS

ASTM C 891: Specification for Installation of Underground Precast Concrete Utility Structures
ASTM C 478: Specification for Precast Reinforced Concrete Manhole Sections
ASTM C 443: Specification for Joints for Concrete Pipe and Manholes, Using Rubber Gaskets
ASTM D 4101: Specification for Copolymer steps construction

CAN/CSA-A257.4-M92
Joints for Circular Concrete Sewer and Culvert Pipe, Manhole Sections and Fittings Using Rubber Gaskets

CAN/CSA-A257.4-M92
Precast Reinforced Circular Concrete Manhole Sections, Catch Basins and Fittings

Canadian Highway Bridge Design Code

1.3 SHOP DRAWINGS

Shop drawings for the structure and performance are to be submitted with each order to the contractor. Contractor shall forward shop drawing submittal to the consulting engineer for approval. Shop drawings are to detail the structure's precast concrete and call out or note the fiberglass (FRP) internals/components.

1.4 PRODUCT SUBSTITUTIONS

No product substitutions shall be accepted unless submitted 10 days prior to project bid date, or as directed by the engineer of record. Submissions for substitutions require review and approval by the Engineer of Record, for hydraulic performance, impact to project designs, equivalent treatment performance, and any required project plan and report (hydrology/hydraulic, water quality, stormwater pollution) modifications that would be required by the approving jurisdictions/agencies. Contractor to coordinate with the Engineer of Record any applicable modifications to the project estimates of cost, bonding amount determinations, plan check fees for changes to approved documents, and/or any other regulatory requirements resulting from the product substitution.

1.5 HANDLING AND STORAGE

Prevent damage to materials during storage and handling.

PART 2 – PRODUCTS

2.1 GENERAL

2.1.1 The device shall be a cylindrical or rectangular, all concrete structure (including risers), constructed from precast concrete riser and slab components or monolithic precast structure(s), installed to conform to ASTM C 891 and to any required state highway, municipal or local specifications; whichever is more stringent. The device shall be watertight.

2.1.2 Cartridge Deck The cylindrical concrete device shall include a fiberglass deck. The rectangular concrete device shall include a coated aluminum deck. In either instance, the insert shall be bolted and sealed watertight inside the precast concrete chamber. The deck shall serve as: (a) a horizontal divider between the lower treatment zone and the upper treated effluent zone; (b) a deck for attachment of filter cartridges such that the membrane filter elements of each cartridge extend into the lower treatment zone; (c) a platform for maintenance workers to service the filter cartridges (maximum manned weight = 450 pounds (204 kg)); (d) a conduit for conveyance of treated water to the effluent pipe.

2.1.3 Membrane Filter Cartridges Filter cartridges shall be comprised of reusable cylindrical membrane filter elements connected to a perforated head plate. The number of membrane filter elements per cartridge shall be a minimum of eleven 2.75-inch (70-mm) diameter elements. The length of each filter element shall be a minimum 15 inches (381 mm). Each cartridge shall be fitted into the cartridge deck by insertion into a cartridge receptacle that is permanently mounted into the cartridge deck. Each cartridge shall be secured by a cartridge lid that is threaded onto the receptacle, or similar mechanism to secure the cartridge into the deck. The maximum treatment flow rate of a filter cartridge shall be controlled by an orifice in the cartridge lid, or on the individual cartridge itself, and based on a design flux rate (surface loading rate) determined by the maximum treatment flow rate per unit of filtration membrane surface area. The maximum design flux rate shall be 0.21 gpm/ft² (0.142 lps/m²).

Each membrane filter cartridge shall allow for manual installation and removal. Each filter cartridge shall have filtration membrane surface area and dry installation weight as follows (if length of filter cartridge is between those listed below, the surface area and weight shall be proportionate to the next length shorter and next length longer as shown below):

Filter Cartridge Length (in / mm)	Minimum Filtration Membrane Surface Area (ft2 / m2)	Maximum Filter Cartridge Dry Weight (lbs / kg)
15	106 / 9.8	10.5 / 4.8
27	190 / 17.7	15.0 / 6.8
40	282 / 26.2	20.5 / 9.3
54	381 / 35.4	25.5 / 11.6

2.1.4 Backwashing Cartridges The filter device shall have a weir extending above the cartridge deck, or other mechanism, that encloses the high flow rate filter cartridges when placed in their respective cartridge receptacles within the cartridge deck. The weir, or other mechanism, shall collect a pool of filtered water during inflow events that backwashes the high flow rate cartridges when the inflow

event subsides. All filter cartridges and membranes shall be reusable and allow for the use of filtration membrane rinsing procedures to restore flow capacity and sediment capacity; extending cartridge service life.

2.1.5 Maintenance Access to Captured Pollutants The filter device shall contain an opening(s) that provides maintenance access for removal of accumulated floatable pollutants and sediment, removal of and replacement of filter cartridges, cleaning of the sump, and rinsing of the deck. Access shall have a minimum clear vertical clear space over all of the filter cartridges. Filter cartridges shall be able to be lifted straight vertically out of the receptacles and deck for the entire length of the cartridge.

2.1.6 Bend Structure The device shall be able to be used as a bend structure with minimum angles between inlet and outlet pipes of 90-degrees or less in the stormwater conveyance system.

2.1.7 Double-Wall Containment of Hydrocarbons The cylindrical precast concrete device shall provide double-wall containment for hydrocarbon spill capture by a combined means of an inner wall of fiberglass, to a minimum depth of 12 inches (305 mm) below the cartridge deck, and the precast vessel wall.

2.1.8 Baffle The filter device shall provide a baffle that extends from the underside of the cartridge deck to a minimum length equal to the length of the membrane filter elements. The baffle shall serve to protect the membrane filter elements from contamination by floatables and coarse sediment. The baffle shall be flexible and continuous in cylindrical configurations, and shall be a straight concrete or aluminum wall in rectangular configurations.

2.1.9 Sump The device shall include a minimum 24 inches (610 mm) of sump below the bottom of the cartridges for sediment accumulation, unless otherwise specified by the design engineer. Depths less than 24 inches may have an impact on the total performance and/or longevity between cartridge maintenance/replacement of the device.

2.2 PRECAST CONCRETE SECTIONS

All precast concrete components shall be manufactured to a minimum live load of HS-20 truck loading or greater based on local regulatory specifications, unless otherwise modified or specified by the design engineer, and shall be watertight.

2.3 JOINTS All precast concrete manhole configuration joints shall use nitrile rubber gaskets and shall meet the requirements of ASTM C443, Specification C1619, Class D or engineer approved equal to ensure oil resistance. Mastic sealants or butyl tape are not an acceptable alternative.

2.4 GASKETS Only profile neoprene or nitrile rubber gaskets in accordance to CSA A257.3-M92 will be accepted. Mastic sealants, butyl tape or Conseal CS-101 are not acceptable gasket materials.

2.5 FRAME AND COVER Frame and covers must be manufactured from cast-iron or other composite material tested to withstand H-20 or greater design loads, and as approved by the

local regulatory body. Frames and covers must be embossed with the name of the device manufacturer or the device brand name.

2.6 DOORS AND HATCHES If provided shall meet designated loading requirements or at a minimum for incidental vehicular traffic.

2.7 CONCRETE All concrete components shall be manufactured according to local specifications and shall meet the requirements of ASTM C 478.

2.8 FIBERGLASS The fiberglass portion of the filter device shall be constructed in accordance with the following standard: ASTM D-4097: Contact Molded Glass Fiber Reinforced Chemical Resistant Tanks.

2.9 STEPS Steps shall be constructed according to ASTM D4101 of copolymer polypropylene, and be driven into preformed or pre-drilled holes after the concrete has cured, installed to conform to applicable sections of state, provincial and municipal building codes, highway, municipal or local specifications for the construction of such devices.

2.10 INSPECTION All precast concrete sections shall be inspected to ensure that dimensions, appearance and quality of the product meet local municipal specifications and ASTM C 478.

PART 3 – PERFORMANCE

3.1 GENERAL

3.1.1 Verification – The stormwater quality filter must be verified in accordance with ISO 14034:2016 Environmental management – Environmental technology verification (ETV).

3.1.2 Function - The stormwater quality filter treatment device shall function to remove pollutants by the following unit treatment processes; sedimentation, floatation, and membrane filtration.

3.1.3 Pollutants - The stormwater quality filter treatment device shall remove oil, debris, trash, coarse and fine particulates, particulate-bound pollutants, metals and nutrients from stormwater during runoff events.

3.1.4 Bypass - The stormwater quality filter treatment device shall typically utilize an external bypass to divert excessive flows. Internal bypass systems shall be equipped with a floatables baffle, and must avoid passage through the sump and/or cartridge filtration zone.

3.1.5 Treatment Flux Rate (Surface Loading Rate) – The stormwater quality filter treatment device shall treat 100% of the required water quality treatment flow based on a maximum design treatment flux rate (surface loading rate) across the membrane filter cartridges of 0.21 gpm/ft² (0.142 lps/m²).

3.2 FIELD TEST PERFORMANCE

At a minimum, the stormwater quality filter device shall have been field tested and verified with a minimum 25 TARP qualifying storm events and field monitoring shall have been conducted according to the TARP 2009 NJDEP TARP field test protocol, and have received NJCAT verification.

3.2.1 Suspended Solids Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median TSS removal efficiency of 85% and a minimum median SSC removal efficiency of 95%.

3.2.2 Runoff Volume – The stormwater quality filter treatment device shall be engineered, designed, and sized to treat a minimum of 90 percent of the annual runoff volume determined from use of a minimum 15-year rainfall data set.

3.2.3 Fine Particle Removal - The stormwater quality filter treatment device shall have demonstrated the ability to capture fine particles as indicated by a minimum median removal efficiency of 75% for the particle fraction less than 25 microns, an effluent d_{50} of 15 microns or lower for all monitored storm events.

3.2.4 Turbidity Reduction - The stormwater quality filter treatment device shall have demonstrated the ability to reduce the turbidity from influent from a range of 5 to 171 NTU to an effluent turbidity of 15 NTU or lower.

3.2.5 Nutrient (Total Phosphorus & Total Nitrogen) Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median Total Phosphorus removal of 55%, and a minimum median Total Nitrogen removal of 50%.

3.2.6 Metals (Total Zinc & Total Copper) Removal - The stormwater quality filter treatment device shall have demonstrated a minimum median Total Zinc removal of 55%, and a minimum median Total Copper removal of 85%.

3.3 INSPECTION and MAINTENANCE

The stormwater quality filter device shall have the following features:

3.3.1 Durability of membranes are subject to good handling practices during inspection and maintenance (removal, rinsing, and reinsertion) events, and site specific conditions that may have heavier or lighter loading onto the cartridges, and pollutant variability that may impact the membrane structural integrity. Membrane maintenance and replacement shall be in accordance with manufacturer's recommendations.

3.3.2 Inspection which includes trash and floatables collection, sediment depth determination, and visible determination of backwash pool depth shall be easily conducted from grade (outside the structure).

3.3.3 Manual rinsing of the reusable filter cartridges shall promote restoration of the flow capacity and sediment capacity of the filter cartridges, extending cartridge service life.

3.3.4 The filter device shall have a minimum 12 inches (305 mm) of sediment storage depth, and a minimum of 12 inches between the top of the sediment storage and bottom of the filter cartridge tentacles, unless otherwise specified by the design engineer. Variances may have an impact on the total performance and/or longevity between cartridge maintenance/replacement of the device.

3.3.5 Sediment removal from the filter treatment device shall be able to be conducted using a standard maintenance truck and vacuum apparatus, and a minimum one point of entry to the sump that is unobstructed by filter cartridges.

3.3.6 Maintenance access shall have a minimum clear height that provides suitable vertical clear space over all of the filter cartridges. Filter cartridges shall be able to be lifted straight vertically out of the receptacles and deck for the entire length of the cartridge.

3.3.7 Filter cartridges shall be able to be maintained without the requirement of additional lifting equipment.

PART 4 – EXECUTION

4.1 INSTALLATION

4.1.1 PRECAST DEVICE CONSTRUCTION SEQUENCE

The installation of a watertight precast concrete device should conform to ASTM C 891 and to any state highway, municipal or local specifications for the construction of manholes, whichever is more stringent. Selected sections of a general specification that are applicable are summarized below.

4.1.1.1 The watertight precast concrete device is installed in sections in the following sequence:

- aggregate base
- base slab
- treatment chamber and cartridge deck riser section(s)
- bypass section
- connect inlet and outlet pipes
- concrete riser section(s) and/or transition slab (if required)
- maintenance riser section(s) (if required)
- frame and access cover

4.1.2 The precast base should be placed level at the specified grade. The entire base should be in contact with the underlying compacted granular material. Subsequent sections, complete with joint seals, should be installed in accordance with the precast concrete manufacturer's recommendations.

4.1.3 Adjustment of the stormwater quality treatment device can be performed by lifting the upper sections free of the excavated area, re-leveling the base, and re-installing the sections. Damaged sections and gaskets should be repaired or replaced as necessary to restore original condition and watertight seals. Once the stormwater quality treatment device has been constructed, any/all lift holes must be plugged watertight with mortar or non-shrink grout.

4.1.4 Inlet and Outlet Pipes Inlet and outlet pipes should be securely set into the device using approved pipe seals (flexible boot connections, where applicable) so that the structure is watertight, and such that any pipe intrusion into the device does not impact the device functionality.

4.1.5 Frame and Cover Installation Adjustment units (e.g. grade rings) should be installed to set the frame and cover at the required elevation. The adjustment units should be laid in a full bed of mortar with successive units being joined using sealant recommended by the manufacturer. Frames for the cover should be set in a full bed of mortar at the elevation specified.

4.2 MAINTENANCE ACCESS WALL
In some instances the Maintenance Access Wall, if provided, shall require an extension attachment and sealing to the precast wall and cartridge deck at the job site, rather than at the precast facility. In this instance, installation of these components shall be performed according to instructions provided by the manufacturer.

4.3 FILTER CARTRIDGE INSTALLATION Filter cartridges shall be installed in the cartridge deck only after the construction site is fully stabilized and in accordance with the manufacturer's guidelines and recommendations. Contractor to contact the manufacturer to schedule cartridge delivery and review procedures/requirements to be completed to the device prior to installation of the cartridges and activation of the system.

PART 5 – QUALITY ASSURANCE

5.1 FILTER CARTRIDGE INSTALLATION Manufacturer shall coordinate delivery of filter cartridges and other internal components with contractor. Filter cartridges shall be delivered and installed complete after site is stabilized and unit is ready to accept cartridges. Unit is ready to accept cartridges after is has been cleaned out and any standing water, debris, and other materials have been removed. Contractor shall take appropriate action to protect the filter cartridge receptacles and filter cartridges from damage during construction, and in accordance with the manufacturer's recommendations and guidance. For systems with cartridges installed prior to full site stabilization and prior to system activation, the contractor can plug inlet and outlet pipes to prevent stormwater and other influent from entering the device. Plugs must be removed during the activation process.

5.2 INSPECTION AND MAINTENANCE

5.2.1 The manufacturer shall provide an Owner's Manual upon request.

5.2.2 After construction and installation, and during operation, the device shall be inspected and cleaned as necessary based on the manufacturer's recommended inspection and maintenance guidelines and the local regulatory agency/body.

5.3 REPLACEMENT FILTER CARTRIDGES When replacement membrane filter elements and/or other parts are required, only membrane filter elements and parts approved by the manufacturer for use with the stormwater quality filter device shall be installed.

END OF SECTION

APPENDIX F

ENGINEERING DRAWINGS