FUNCTIONAL SERVICING &
PRELIMINARY STORMWATER MANAGEMENT REPORT
FOR
1884 LIVERPOOL ROAD

CITY OF PICKERING

March 2026

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LIST OF FIGURES AND TABLES

REFERENCES:

- Drawing No. 2-F-1 – Glenanna Road (Liverpool Road to Glendale Drive), Town of Pickering, dated December 22, 1978, revised March 18, 1979
- Drawing No. T-6-96 – Glendale Drive (between Glenanna Road and Finch Avenue), dated August 1996
- Architectural Drawings including site statistics, prepared by Micacchi Architecture Inc., dated March 4, 2026.

1.0 INTRODUCTION

On behalf of Louisville Homes Ltd. ("the owner"), Politis Engineering Ltd. ("PEL") has been retained to prepare a functional servicing and preliminary stormwater management (SWM) report in support of a proposed residential development located at 1884 Liverpool Road and 1885 Glendale Drive, in the City of Pickering.

2.0 SITE DESCRIPTION

The subject site is 0.28 hectares (2,768.8 sq.m.) in area and is located on the north side of Glenanna Road from Liverpool Road to Glendale Drive as shown in **Figure 1** below. It is made up of part of Lot 18 and all of Lot 39, Registered Plan 492.

The property is occupied by a 1-storey house (1884 Liverpool Road) and a 2-storey house (1885 Glendale Drive), both with pitched roofs and paved driveways, accessory buildings and an inground swimming pool and associated concrete patio.

Figure 1 – Key Plan (Not to Scale)

Road widenings on Liverpool Road and Glenanna Road including a daylight triangle will be conveyed to the Region of Durham ("the Region") and City of Pickering ("the City"), which will reduce the development area to 2,240.7 sq.m.

3.0 PROPOSED DEVELOPMENT

The proposed development consists of demolishing the existing dwellings to re-develop the property and construct three blocks of stacked townhouses, containing a total of 51 residential units, comprised of the following unit mix:

- 1-bedroom units = 4
- 2-bedroom units = 47

One level of underground parking is proposed and vehicular access to the site and underground parking will be provided via a private driveway from Glenanna Road.

4.0 EXISTING MUNICIPAL INFRASTRUCTURE

4.1 Sanitary Sewer System

Sanitary servicing is available via existing sanitary sewers located along the east, south, and west frontages of the site. The surrounding sanitary infrastructure includes:

- A 200 mm diameter AC sanitary sewer within the Liverpool Road right-of-way
- A 200 mm diameter AC sanitary sewer within the Glenanna Road right-of-way
- A 250 mm diameter AC sanitary sewer within the Glendale Drive right-of-way

Any existing unused sanitary service connections will be disconnected at the main and capped with a watertight plug to the satisfaction of the Region.

4.2 Water Distribution System

Water supply is available from existing watermains along all frontages, including:

- A 200 mm diameter ductile iron watermain within the Liverpool Road right-of-way
- A 250 mm diameter ductile iron watermain within the Glenanna Road right-of-way
- A 200 mm diameter PVC watermain within the Glendale Drive right-of-way

The Region has advised that no connections will be permitted to the existing 600 mm diameter CPP feedermain within the Glenanna Road right-of-way.

All existing unused water service connections will be disconnected at the main and plugged with a brass plug to the satisfaction of the Region.

4.3 Storm Sewer System

There are existing storm sewers located along all frontages of the site. The surrounding storm infrastructure includes:

- A 300 mm diameter concrete storm sewer within the Liverpool Road right-of-way
- A 600 mm diameter concrete storm sewer within the Glenanna Road right-of-way, which transitions to a 900 mm diameter concrete storm sewer west of Glendale Drive
- A 750 mm diameter concrete storm sewer within the Glendale Drive right-of-way

The existing detached dwellings do not have storm service connections and currently drain overland toward the adjacent municipal roadways and the neighbouring property to the northwest.

All stormwater runoff from site is ultimately captured by existing road catchbasins on Glenanna Road and Glendale Drive and conveyed to existing Storm Manhole 10, which outlets to the 900 mm diameter storm sewer on Glenanna Road.

Refer to **Figure 2** for the Pre-Development Drainage Plan and **Table 1** for calculation of the pre-development composite runoff coefficient.

LEGEND

PROPERTY LINE	POOL
ROOF	GRAVEL
ASPHALT	INTERLOCK
CONCRETE	LANDSCAPE

0.7982 — TRIBUTARY AREA (ha)

0.30 — RUNOFF COEFFICIENT (-)

← EXISTING SURFACE DRAINAGE

PRE-DEVELOPMENT SURFACES PLAN

(SCALE = 1:500)

Table 1 – Pre-Development Runoff Coefficient

Description	Area (m2)	Runoff Coeff.	C x A
Roof	287.8	0.95	273.4
Asphalt	404.0	0.95	383.8
Concrete	118.1	0.95	112.2
Pool	38.4	0.95	36.5
Gravel	29.6	0.70	20.7
Interlock (Open Joints)	29.0	0.65	18.9
Lawn	1333.8	0.20	266.8
Totals =	2240.7		1112.2
Composite Runoff Coefficient =		0.50	

Per the pre-consultation minutes/meeting summary, dated June 10, 2024, post-development peak flows are to be controlled to the 5-year pre-development discharge rate using a runoff coefficient of C = 0.35 and the “old” IDF parameters. The pre-development flow calculation is provided for reference and demonstrates that the proposed allowable release rate represents a reduction from existing conditions.

The pre-development peak discharge rate is calculated using the rational method as follows:

$$Q = 0.00278\ x\ A\ x\ I\ x\ C$$

where,

Q = Peak runoff rate (cu.m./s)
A = Drainage area (ha)
I = Rainfall intensity (mm/hr)
C = Runoff coefficient (-)

using the latest City IDF parameters, the 5-year intensity is equal to 106.3 mm/hr:

$$Q = 0.00278\ x\ \left(\frac{2240.7}{10000}\right) x\ 106.3\ x\ 0.50$$

$$Q = 0.0331\ cu.m./s\ or\ 33.1\ L/s$$

5.0 SANITARY DRAINAGE SYSTEM

5.1 Existing Conditions

Under existing conditions, the site is occupied by two detached dwellings. Using a population equivalent of 3.5 persons per dwelling, the total existing population is estimated to be 7 persons.

Using an average daily sanitary flow of 364 L/person/day and a peaking factor of 3.8 (maximum) based on the Harmon equation, the total existing average daily sanitary flow is estimated to be:

$$Q = 7\ x\ 364\ x\ 3.8 = 9{,}682.4\ L/day\ or\ 0.112\ L/sec$$

The infiltration allowance is 0.26 L/s/ha of gross area:

$$Q = 0.26\ x\ 0.22407 = 0.058\ L/s$$

Therefore, the total pre-development sanitary flow is equal to:

$$Q = 0.112 +\ 0.058 = 0.170\ L/s$$

5.2 Proposed Peak Flows

The proposed development includes 51 residential units, resulting in the following population estimate based on the Region's design criteria:

- 1-bedroom units: 4 x 1.5 persons/unit = 6 persons
- 2-bedroom units: 47 x 2.5 persons/unit = 117.5 persons

Total proposed population = 123.5 persons

The harmon peaking factor is:

$$K_H = 1 + \frac{14}{4 +\ P^{\ 1/2}}$$

where,

K_H = Harmon peaking factor (Min = 1.5 | Max = 3.8)
P = Population in thousands

$$K_H = 1 + \frac{14}{4 +\ 0.1235^{\ 1/2}} = 4.217$$
$$K_H = 3.8$$

The proposed peaking factor exceeds the Region's maximum, therefore 3.8 has been used.

Using a daily average flow of 364 L/person/day the dry weather flow is:

$$Q = 123.5\ x\ 364\ x\ 3.8 = 170{,}825.2\ L/day\ or\ 1.977\ L/sec$$

The infiltration allowance is 0.26 L/s/ha of gross area:

$$Q = 0.26\ x\ 0.22407 = 0.058\ L/s$$

The total post-development sanitary flow is equal to:

$$Q = 1.977 +\ 0.058 = 2.035\ L/s$$

Therefore, the proposed development will increase the sanitary peak flow from the pre-development level by 1.865 L/s. The Region will confirm the available capacity within the downstream sanitary sewer system at the time of the development agreement.

5.3 Proposed Sanitary Service

A new 150 mm diameter PVC sanitary service connection will be extended to the existing 250 mm diameter sanitary sewer on Glenanna Road at a slope of 2.0%. The proposed service connection will have a capacity of 15.2 L/s. An inspection manhole will be provided at street line.

6.0 WATER DISTRIBUTION SYSTEM

The Region of Durham does not have specific design criteria for residential domestic water systems, therefore the latest Ministry of the Environment, Conservation and Parks (MECP) criteria as well as the Fire Underwriter's Survey will be used.

6.1 Domestic Water Demand

Daily domestic water demand is assumed to be equal to the average daily sanitary flow rate of 364 L/person/day. Based on a proposed population of 123.5 persons, the total post-development average daily domestic water demand is estimated to be 44,954 L/day or 31.2 L/min.

6.2 Fire Flow Demand

The underground garage is to be sprinklered and a Siamese connection will be provided and will be located within 45 m of a municipal fire hydrant. Above the underground garage, the building is considered to be 3-storey wood frame and will have a fire break between Block 2 and 3.

Fire flow calculations based on the Fire Underwriter's Survey "Water Supply for Public Fire Protection" dated 2020 have determined the maximum required fire flow to be 16,000 L/min for Block 1, which governs for this site.

The detailed FUS calculations can be found appended to this report in **Appendix 1.**

6.4 Total Water Demand

The required flow required is the greater of the maximum daily demand plus the fire flow or the peak hourly demand. In this case, the maximum day demand plus fire flow is critical and based on a maximum day peaking factor of 1.65, the design flow is 16,051.5 L/min.

6.5 Proposed Water Connection

A new 150 mm diameter combined water service will be extended from the existing 200 mm watermain on Liverpool Road. The combined service will split before street line into a 150 mm diameter fire line and a 100 mm diameter domestic line, with shutoffs located within the boulevard to the Region's approval.

Appropriate backflow prevention and meters are to be provided and located within a meter room acceptable to the Region.

7.0 STORMWATER MANAGEMENT

The stormwater management criteria applicable to the proposed development include:

1. <u>Water Quantity Control:</u> Provide control of post-development peak flow rates to the 5-year pre-development flow for all storm events up to the 100-year return period.
2. <u>Water Quality Control:</u> Development must provide water quality control measures designed to provide enhanced (Level 1) water quality control as defined by the MOE Design Manual.
3. <u>Erosion Control:</u> A minimum of 5mm of runoff must be infiltrated on the site. Low Impact Development (LID) measures shall be designed in accordance with the TRCA & CVC LID SWM Planning & Design Guide and to the City's SWM Design Guidelines.

7.1 Pre-Development Peak Flows

Post-development peak flows will be controlled to the 5-year pre-development discharge rate using a pre-development runoff coefficient of C = 0.35 and the "old" IDF parameters (A = 2464, B = 16, C = 1).

The allowable discharge rate has been calculated using the rational method as follows:

$$Q = 0.00278\ x\ A\ x\ I\ x\ C$$

$$Q = 0.00278\ x\ \left(\frac{2240.7}{10000}\right) x\ 94.8\ x\ 0.35$$

$$Q = 0.0207\ cu.m./s\ or\ 20.7\ L/s$$

Therefore, the allowable discharge rate from the site is 20.7 L/s.

7.2 Post-Development Storm Drainage

The proposed development will introduce additional impervious area associated with building roofs, driveway access, and surface hardscaping.

All stormwater runoff generated from the site will be self-contained and will be collected via catchbasins and roof drains and conveyed to a private underground stormwater detention facility. Before being discharged into the existing municipal storm sewer system, the stormwater will be controlled to the allowable discharge rate and will pass through an oil grit separator (OGS) unit sized to provide enhanced water quality control.

7.3 Post-Development Peak Flows

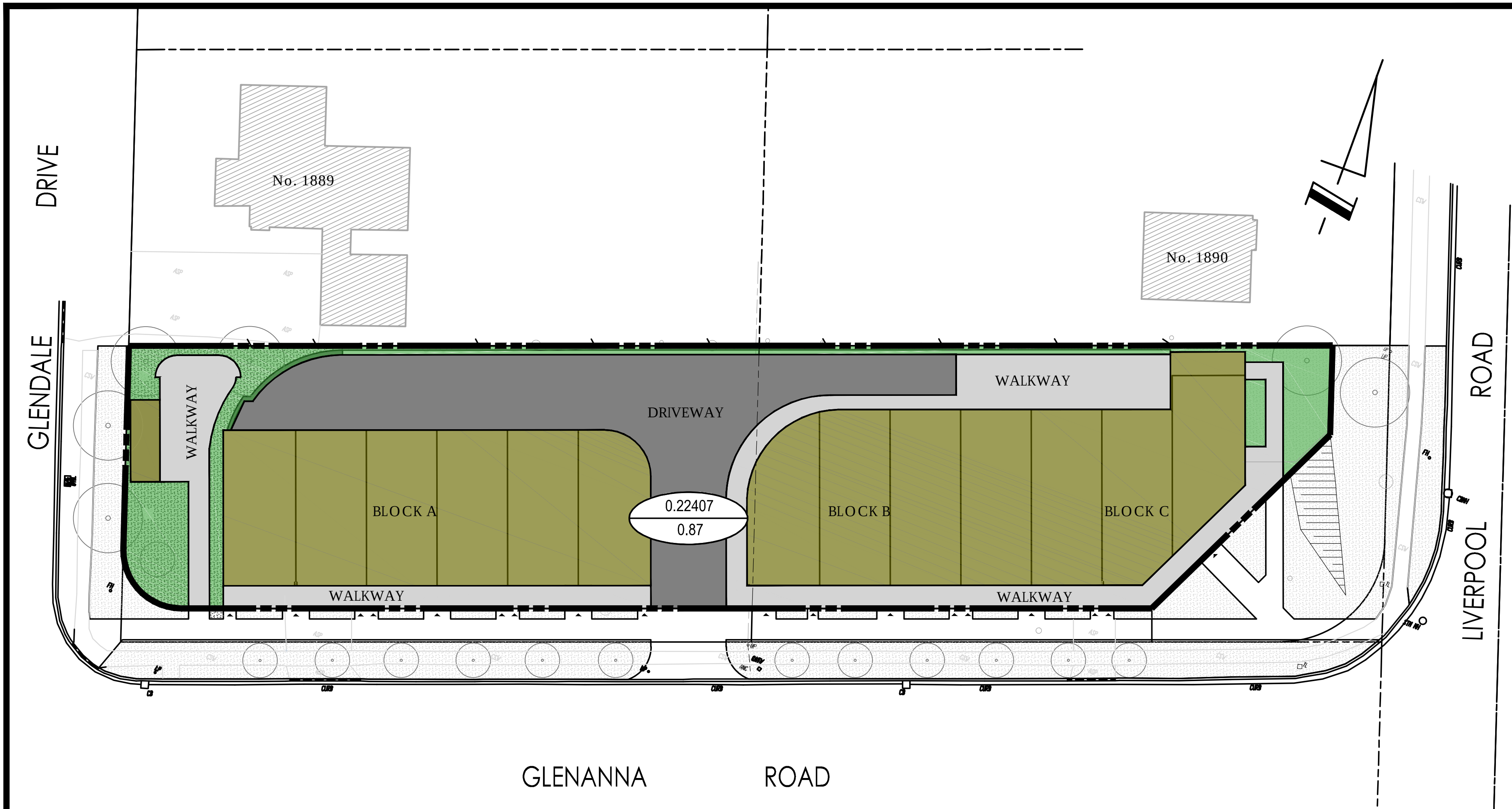
All stormwater generated by the proposed development will be self-contained on-site. Using the Proposed Site Plan, prepared by Micacchi Architecture Inc., dated March 4, 2026, a post-development runoff coefficient has been calculated using the various proposed surfaces across the site, including roofs, driveways, landscaping, and walkways.

Refer to **Figure 3** for the post-development surfaces plan and **Table 2** for the post-development runoff coefficient.

Table 2 – Post-Development Runoff Coefficient

Description	Area (m2)	Runoff Coeff.	C x A
Roof	1144.6	0.95	1087.4
Asphalt Driveway	450.3	0.95	427.8
Walkways	410.2	0.95	389.7
Landscape	235.6	0.20	47.1
Totals =	2240.7		1952.0
Composite Runoff Coefficient =		0.87	

Therefore, the post-development composite runoff coefficient for the proposed development is equal to 0.87.



LEGEND

PROPERTY LINE

ROOF

WALKWAY

DRIVEWAY

LANDSCAPE

0.7982 — TRIBUTARY AREA (ha)

0.30 — RUNOFF COEFFICIENT (-)

POST-DEVELOPMENT SURFACES PLAN

(SCALE = 1:500)

To control all post-development storm events to the allowable release rate, underground detention storage is required on-site. The necessary detention volumes have been calculated using the post-development runoff coefficient and the allowable release rate.

The post-development peak flows and the required detention volumes are compared below in **Table 3** and detailed detention storage calculations are provided in **Appendix 2**.

Table 3 – Underground Detention Volumes

Return Period	Area (ha)	Runoff Coeff.	Precip. Factor	Factored Coeff.	Req. Storage (m3)
2-Year	0.22407	0.87	1.00	0.87	12.8
5-Year	0.22407	0.87	1.00	0.87	22.8
10-Year	0.22407	0.87	1.00	0.87	30.3
25-Year	0.22407	0.87	1.10	0.96	46.6
50-Year	0.22407	0.87	1.20	1.00	58.9
100-Year	0.22407	0.87	1.25	1.00	68.7

7.4 Temporary Detention Storage

Underground detention storage will be provided to mitigate the post-development peak flows to the allowable release rate. Storage will be provided within an underground stormwater detention facility, subject to confirmation during detailed engineering design.

7.6 Proposed Storm Connection

A 200 mm PVC storm connection is proposed to existing Storm Manhole 10, which outlets into the existing 900 mm diameter storm sewer main on Glenanna Road. The capacity of the proposed connection laid at 1.0% is 32.8 L/s, which exceeds the 5 year allowable release rate (pre-development peak flow).

An inspection manhole will be provided at street line.

7.7 In-Stream Erosion Control and Water Balance

For small infill sites and site plans less than 5 hectares the minimum erosion control requirements are:

- extended detention of the 4 hour, 25 mm Chicago distribution rainfall event for a minimum of 24 hours, or
- runoff reduction from the site through infiltration, evapotranspiration and reuse of a minimum 5mm of rainfall depth across all impervious surfaces.

The total impervious area of the site is 2,005.1 sq.m. and therefore a total volume of 10.0 cu.m. will need to be retained to meet the 5mm requirement.

Since the proposed development includes an underground parking structure that covers most of the site, use of infiltration techniques is not feasible, therefore other techniques will need to be considered and implemented.

The following is a list of possible uses of retained storm runoff will be determined during the detailed engineering design:

- Irrigation for landscaped areas and terrace/ rooftop landscape features
- Provide green roofs
- Building mechanical systems which will be specified by the Mechanical Engineer at the detailed design stage (i.e. evaporative cooling)

7.8 Permanent Water Quality

The water quality control criteria requires that the development must provide water quality control measures designed to provide Enhanced (Level 1) water quality control as defined by the MOE Design Manual.

Runoff from the rooftop and landscaped areas, are considered to be clean and will not require treatment prior to being discharged from the site. Flows from paved areas will require treatment to meet the TSS removal criteria.

For the subject development, a "treatment train" approach will be taken which will ultimately include an oil/grit separator in order to meet the TSS removal criteria prior to discharge to the municipal storm sewer system, subject to the detailed design process.

7.9 Water Quality During Construction

Temporary erosion and sediment control will be required during the construction period. It would be prudent to make provisions to provide “good housekeeping” measures to mitigate the transportation of silt from the site during the construction phases. These measures include, but are not limited to the following:

- Provide silt fences around the perimeter of the site to reduce silt from leaving the site.
- Provide silt filters at catchbasins upon their installation to reduce the amount of silt entering the sewer system during construction.
- Use of a “mud mat” or temporary tracking control at the entrance of the site to minimize mud tracking from the site.
- Stabilize the site as soon as possible, that is, re-establish vegetative ground cover and avoid bare soil areas.

All the above erosion and siltation control measures should be monitored and maintained on a regular basis to ensure maximum benefit and minimum silt migration off-site and shall be in accordance with the GTA CA's Erosion & Sediment Control Guidelines for Urban Construction (2019).

8.0 SUMMARY

The area of the site is 2,768.8 sq.m. (0.28 ha). The property is located on the north side of Glenanna Road and bounded by Liverpool Road to the east and Glendale Drive to the west.

A road widening on Liverpool Road and Glenanna Road will reduce the development area to 2,240.7 sq.m.

The intention is to demolish the 2 existing houses to construct three blocks of stacked townhouses, containing a total of 51 residential units.

The total pre-development sanitary flow, including peaking and infiltration, is equal to 0.170 L/s.

The total post-development sanitary flow, including peaking and infiltration, is 2.035 L/s, which represents an increase of 1.865 L/s from the pre-development level.

The Region will confirm the available capacity within the downstream sanitary sewer system.

A 150 mm diameter PVC sanitary service with a slope of 2.0% will connect to the existing 250 mm diameter sanitary sewer on Glenanna Road, with an inspection manhole at street line.

The post-development average daily domestic water demand is estimated to be 44,954 L/day or 31.2 L/min.

The maximum fire flow demand is 16,000 L/min for Block A, which governs for the site.

The total required water demand is equal to the maximum day demand (PF = 1.65) plus the fire flow, or 16,051.5 L/min.

A 150 mm diameter combined water service will connect to the existing 200 mm diameter watermain on Liverpool Road. The service will split before streetline into a 150 mm diameter watermain fire line and a 100 mm diameter domestic line, with shutoffs in the municipal boulevard.

Appropriate backflow prevention and meters are to be located within a meter room acceptable to the Region. A siamese connection will be provided within a 45 m distance of a fire hydrant.

The allowable discharge rate was calculated using C = 0.35 and the “old” City IDF parameters, and is equal to 20.7 L/s.

All stormwater runoff generated from the site will be self-contained and stored underground on-site. Before discharging, the stormwater will be controlled to the allowable discharge rate and will pass through an oil/grit separator (OGS) unit.

The post-development composite runoff coefficient is equal to 0.87.

To control all post-development storm events to the allowable release, detention storage is required on-site. The maximum volume required is 68.7 cu.m. for the 100-year return period.

Storage will be provided within an underground stormwater detention facility.

A 200 mm PVC storm connection is proposed to existing Storm Manhole 10, which outlets to the existing 900 mm diameter storm sewer on Glenanna Road, with an inspection manhole at street line.

To provide in-stream erosion control and water balance, 5 mm of rainfall is to be retained on site which is 10.0 cubic meters.

In order to meet the water quality criteria, an oil/grit separator will be sized to provide enhanced (Level 1) water quality control.

Temporary erosion and sediment control measures will be provided in accordance with the Erosion & Sediment Control Guidelines for Urban Construction (2019), as part of the detailed design.

Based on the foregoing, the existing municipal infrastructure is adequate to service the proposed development.

Respectfully submitted

Politis Engineering Ltd.

Per: Tim Politis, P.Eng.

APPENDIX 1

FIRE FLOW REQUIREMENTS

Based on "Water Supply for Public Fire Protection - 2020", Fire Underwriters Survey

Date:	2026-03-06
Address:	1884 Liverpool Road, Pickering, Ontario
NBC Occupancy:	Group C
Construction Class:	Wood Frame Construction (Type V)
Notes:	Block A

STEP 1 - DETERMINE FIRE FLOW:

F = 220 x C x A^0.5

Effective Floor Area (A) =	1409.1	m2
Construction Coefficient (C) =	1.5	
F =	**13000**	**L/min (Round up to nearest 1,000 L/min)**

STEP 2 - OCCUPANCY ADJUSTMENT: (Weighted-Average based on GFA and Occupancy)

Reduction for Residential Occupancies =	15%	
Decrease for Occupancy Factor =	1950	L/min
Adjusted Fire Flow =	**11050**	**L/min**

STEP 3 - AUTO SPRINKLER FACTOR:

	Y/N	Credit
NFPA 13 Sprinkler Standard	YES	0%
Standard Water Supply	YES	0%
Fully Supervised System	YES	0%

Decrease for Sprinkler Credit = 0 L/min

STEP 4 - EXPOSURE FACTORS: Type V Construction

Exposure 1	0%	South Exposure	(> 30m)
Exposure 2	17%	East Exposure	(3.1 - 10m; LHF = 41-60)
Exposure 3	19%	North Exposure	(3.1 - 10m; LHF = 81-100)
Exposure 4	0%	West Exposure	(> 30m)
Total	36%	(Maximum 75%)	
Increase for Exposure =	**4680**	**L/min**	

STEP 5 - TOTAL REQUIRED FIRE FLOW 15730 L/min

16000 L/min (Rounded to nearest 1,000 L/min)

FIRE FLOW REQUIREMENTS

Based on "Water Supply for Public Fire Protection - 2020", Fire Underwriters Survey

Date:	2026-03-06
Address:	1884 Liverpool Road, Pickering, Ontario
NBC Occupancy:	Group C
Construction Class:	Wood Frame Construction (Type V)
Notes:	Block B

STEP 1 - DETERMINE FIRE FLOW:

F = 220 x C x A^0.5

Effective Floor Area (A) =	761.8	m2
Construction Coefficient (C) =	1.5	
F =	**10000**	**L/min (Round up to nearest 1,000 L/min)**

STEP 2 - OCCUPANCY ADJUSTMENT: (Weighted-Average based on GFA and Occupancy)

Reduction for Residential Occupancies =	15%	
Decrease for Occupancy Factor =	1500	L/min
Adjusted Fire Flow =	**8500**	**L/min**

STEP 3 - AUTO SPRINKLER FACTOR:

	Y/N	Credit
NFPA 13 Sprinkler Standard	YES	0%
Standard Water Supply	YES	0%
Fully Supervised System	YES	0%

Decrease for Sprinkler Credit = 0 L/min

STEP 4 - EXPOSURE FACTORS: Type V Construction

Exposure 1	0%	South Exposure	(> 30m)
Exposure 2	22%	East Exposure	(0 - 3m; LHF = 41-60)
Exposure 3	12%	North Exposure	(10.1 - 20m; LHF = 41-60)
Exposure 4	17%	West Exposure	(3.1 -10m; LHF = 41-60)
Total	51%	(Maximum 75%)	
Increase for Exposure =	**5100**	**L/min**	

STEP 5 - TOTAL REQUIRED FIRE FLOW 13600 L/min

14000 L/min (Rounded to nearest 1,000 L/min)

FIRE FLOW REQUIREMENTS

Based on "Water Supply for Public Fire Protection - 2020", Fire Underwriters Survey

Date:	2026-03-06
Address:	1884 Liverpool Road, Pickering, Ontario
NBC Occupancy:	Group C
Construction Class:	Wood Frame Construction (Type V)
Notes:	Block C

STEP 1 - DETERMINE FIRE FLOW:

F = 220 x C x A^0.5

Effective Floor Area (A) =	1036.2	m2
Construction Coefficient (C) =	1.5	
F =	**11000**	**L/min (Round up to nearest 1,000 L/min)**

STEP 2 - OCCUPANCY ADJUSTMENT: (Weighted-Average based on GFA and Occupancy)

Reduction for Residential Occupancies =	15%	
Decrease for Occupancy Factor =	1650	L/min
Adjusted Fire Flow =	**9350**	**L/min**

STEP 3 - AUTO SPRINKLER FACTOR:

	Y/N	Credit
NFPA 13 Sprinkler Standard	YES	0%
Standard Water Supply	YES	0%
Fully Supervised System	YES	0%

Decrease for Sprinkler Credit = 0 L/min

STEP 4 - EXPOSURE FACTORS: Type V Construction

Exposure 1	0%	South Exposure	(> 30m)
Exposure 2	0%	East Exposure	(> 30m)
Exposure 3	19%	North Exposure	(3.1 - 10m; LHF = 81-100)
Exposure 4	22%	West Exposure	(0 - 3m; LHF = 41-60)
Total	41%	(Maximum 75%)	
Increase for Exposure =	**4510**	**L/min**	

STEP 5 - TOTAL REQUIRED FIRE FLOW	13860	L/min
	14000	**L/min (Rounded to nearest 1,000 L/min)**

APPENDIX 2

2 YEAR STORAGE REQUIREMENTS

POST-DEVELOPMENT DATA
AREA (ha) = 0.22407
C = 0.87
Ca = 1.00
Ca x C = 0.87

ALLOWABLE DISCHARGE RATE (m3/s) = 0.0207

RAINFALL INTENSITY

I = A/(T+B) ^ C

Where A= 715.076
B= 5.262
C= 0.815

REQUIRED STORAGE VOLUME (m3) = 12.8

TIME (min)	INTENSITY (mm/hr)	PEAK FLOW (m3/s)	RUNOFF VOLUME (m3/s)	DISCHARGE VOLUME (m3/s)	STORAGE VOLUME (m3)
10.0	77.57	0.042	25.2	12.4	12.8
11.0	73.66	0.040	26.4	13.7	12.7
12.0	70.17	0.038	27.4	14.9	12.5
13.0	67.02	0.036	28.3	16.1	12.2
14.0	64.17	0.035	29.2	17.4	11.8
15.0	61.58	0.033	30.0	18.6	11.4
16.0	59.20	0.032	30.8	19.9	10.9
17.0	57.03	0.031	31.5	21.1	10.4
18.0	55.02	0.030	32.2	22.4	9.9
19.0	53.17	0.029	32.9	23.6	9.3
20.0	51.44	0.028	33.5	24.8	8.6
25.0	44.40	0.024	36.1	31.1	5.1
30.0	39.20	0.021	38.3	37.3	1.0
35.0	35.19	0.019	40.1	43.5	0.0
40.0	31.98	0.017	41.6	49.7	0.0
45.0	29.37	0.016	43.0	55.9	0.0
50.0	27.18	0.015	44.2	62.1	0.0
55.0	25.33	0.014	45.3	68.3	0.0
60.0	23.74	0.013	46.3	74.5	0.0

5 YEAR STORAGE REQUIREMENTS

POST-DEVELOPMENT DATA
AREA (ha) = 0.22407
C = 0.87
Ca = 1.00
Ca x C = 0.87

ALLOWABLE DISCHARGE RATE (m3/s) = 0.0207

RAINFALL INTENSITY

I = A/(T+B) ^ C

Where A= 1082.901
B= 6.007
C= 0.837

REQUIRED STORAGE VOLUME (m3) = 22.8

TIME (min)	INTENSITY (mm/hr)	PEAK FLOW (m3/s)	RUNOFF VOLUME (m3/s)	DISCHARGE VOLUME (m3/s)	STORAGE VOLUME (m3)
10.0	106.31	0.058	34.6	12.4	22.2
11.0	101.05	0.055	36.2	13.7	22.5
12.0	96.34	0.052	37.6	14.9	22.7
13.0	92.07	0.050	38.9	16.1	22.8
14.0	88.21	0.048	40.2	17.4	22.8
15.0	84.68	0.046	41.3	18.6	22.7
16.0	81.45	0.044	42.4	19.9	22.5
17.0	78.47	0.043	43.4	21.1	22.3
18.0	75.73	0.041	44.3	22.4	22.0
19.0	73.18	0.040	45.2	23.6	21.6
20.0	70.82	0.038	46.1	24.8	21.2
25.0	61.13	0.033	49.7	31.1	18.7
30.0	53.94	0.029	52.6	37.3	15.4
35.0	48.38	0.026	55.1	43.5	11.6
40.0	43.93	0.024	57.2	49.7	7.5
45.0	40.30	0.022	59.0	55.9	3.1
50.0	37.27	0.020	60.6	62.1	0.0
55.0	34.69	0.019	62.1	68.3	0.0
60.0	32.48	0.018	63.4	74.5	0.0

10 YEAR STORAGE REQUIREMENTS

POST-DEVELOPMENT DATA
AREA (ha) = 0.22407
C = 0.87
Ca = 1.00
Ca x C = 0.87

ALLOWABLE DISCHARGE RATE (m3/s) = 0.0207

RAINFALL INTENSITY

I = A/(T+B) ^ C

Where A= 1313.979
B= 6.026
C= 0.845

REQUIRED STORAGE VOLUME (m3) = 30.3

TIME (min)	INTENSITY (mm/hr)	PEAK FLOW (m3/s)	RUNOFF VOLUME (m3/s)	DISCHARGE VOLUME (m3/s)	STORAGE VOLUME (m3)
10.0	126.04	0.068	41.0	12.4	28.6
11.0	119.76	0.065	42.9	13.7	29.2
12.0	114.12	0.062	44.6	14.9	29.6
13.0	109.03	0.059	46.1	16.1	30.0
14.0	104.41	0.057	47.6	17.4	30.2
15.0	100.20	0.054	48.9	18.6	30.3
16.0	96.34	0.052	50.1	19.9	30.3
17.0	92.79	0.050	51.3	21.1	30.2
18.0	89.52	0.049	52.4	22.4	30.1
19.0	86.49	0.047	53.5	23.6	29.9
20.0	83.67	0.045	54.4	24.8	29.6
25.0	72.12	0.039	58.7	31.1	27.6
30.0	63.57	0.034	62.0	37.3	24.8
35.0	56.96	0.031	64.9	43.5	21.4
40.0	51.68	0.028	67.3	49.7	17.6
45.0	47.37	0.026	69.3	55.9	13.5
50.0	43.77	0.024	71.2	62.1	9.1
55.0	40.72	0.022	72.9	68.3	4.6
60.0	38.10	0.021	74.4	74.5	0.0

25 YEAR STORAGE REQUIREMENTS

POST-DEVELOPMENT DATA
AREA (ha) = 0.22407
C = 0.87
Ca = 1.10
Ca x C = 0.96

ALLOWABLE DISCHARGE RATE (m3/s) = 0.0207

RAINFALL INTENSITY

I = A/(T+B) ^ C

Where A= 1581.718
B= 6.007
C= 0.848

REQUIRED STORAGE VOLUME (m3) = 46.6

TIME (min)	INTENSITY (mm/hr)	PEAK FLOW (m3/s)	RUNOFF VOLUME (m3/s)	DISCHARGE VOLUME (m3/s)	STORAGE VOLUME (m3)
10.0	150.62	0.090	53.9	12.4	41.5
12.0	136.31	0.081	58.5	14.9	43.6
14.0	124.66	0.074	62.5	17.4	45.1
16.0	114.98	0.069	65.8	19.9	46.0
18.0	106.81	0.064	68.8	22.4	46.4
19.0	103.17	0.062	70.2	23.6	46.6
20.0	99.80	0.060	71.4	24.8	46.6
21.0	96.66	0.058	72.6	26.1	46.6
22.0	93.72	0.056	73.8	27.3	46.5
23.0	90.98	0.054	74.9	28.6	46.3
24.0	88.40	0.053	75.9	29.8	46.1
25.0	85.97	0.051	76.9	31.1	45.9
30.0	75.74	0.045	81.3	37.3	44.1
40.0	61.53	0.037	88.1	49.7	38.4
50.0	52.07	0.031	93.2	62.1	31.1
60.0	45.30	0.027	97.3	74.5	22.8
70.0	40.19	0.024	100.7	86.9	13.7
80.0	36.19	0.022	103.6	99.4	4.3
85.0	34.50	0.021	104.9	105.6	0.0

50 YEAR STORAGE REQUIREMENTS

POST-DEVELOPMENT DATA
AREA (ha) = 0.22407
C = 0.87
Ca = 1.20
Ca x C = 1.00

ALLOWABLE DISCHARGE RATE (m3/s) = 0.0207

RAINFALL INTENSITY

I = A/(T+B) ^ C

Where A= 1828.009
B= 6.193
C= 0.856

REQUIRED STORAGE VOLUME (m3) = 58.9

TIME (min)	INTENSITY (mm/hr)	PEAK FLOW (m3/s)	RUNOFF VOLUME (m3/s)	DISCHARGE VOLUME (m3/s)	STORAGE VOLUME (m3)
10.0	168.58	0.105	63.0	12.4	50.5
12.0	152.58	0.095	68.4	14.9	53.5
14.0	139.55	0.087	73.0	17.4	55.6
16.0	128.71	0.080	76.9	19.9	57.0
18.0	119.55	0.074	80.4	22.4	58.0
20.0	111.69	0.070	83.4	24.8	58.6
21.0	108.16	0.067	84.8	26.1	58.7
22.0	104.87	0.065	86.2	27.3	58.8
23.0	101.79	0.063	87.4	28.6	58.9
24.0	98.90	0.062	88.6	29.8	58.8
25.0	96.18	0.060	89.8	31.1	58.7
26.0	93.61	0.058	90.9	32.3	58.6
27.0	91.19	0.057	92.0	33.5	58.4
28.0	88.91	0.055	93.0	34.8	58.2
30.0	84.68	0.053	94.9	37.3	57.6
50.0	58.11	0.036	108.5	62.1	46.4
70.0	44.78	0.028	117.1	86.9	30.1
90.0	36.68	0.023	123.3	111.8	11.5
102.0	33.17	0.021	126.3	126.7	0.0

100 YEAR STORAGE REQUIREMENTS

POST-DEVELOPMENT DATA
AREA (ha) = 0.22407
C = 0.87
Ca = 1.25
Ca x C = 1.00

ALLOWABLE DISCHARGE RATE (m3/s) = 0.0207

RAINFALL INTENSITY

I = A/(T+B) ^ C

Where A= 2096.425
B= 6.485
C= 0.863

REQUIRED STORAGE VOLUME (m3) = 68.7

TIME (min)	INTENSITY (mm/hr)	PEAK FLOW (m3/s)	RUNOFF VOLUME (m3/s)	DISCHARGE VOLUME (m3/s)	STORAGE VOLUME (m3)
10.0	186.69	0.116	69.7	12.4	57.3
15.0	148.54	0.092	83.2	18.6	64.6
20.0	124.00	0.077	92.6	24.8	67.8
21.0	120.10	0.075	94.2	26.1	68.1
22.0	116.45	0.072	95.7	27.3	68.4
23.0	113.04	0.070	97.1	28.6	68.5
24.0	109.83	0.068	98.4	29.8	68.6
25.0	106.81	0.066	99.7	31.1	68.7
26.0	103.97	0.065	100.9	32.3	68.7
27.0	101.28	0.063	102.1	33.5	68.6
28.0	98.74	0.061	103.3	34.8	68.5
29.0	96.34	0.060	104.3	36.0	68.3
30.0	94.05	0.059	105.4	37.3	68.1
40.0	76.31	0.047	114.0	49.7	64.3
50.0	64.50	0.040	120.4	62.1	58.3
70.0	49.65	0.031	129.8	86.9	42.9
90.0	40.63	0.025	136.6	111.8	24.8
100.0	37.32	0.023	139.4	124.2	15.2
116.0	33.07	0.021	143.3	144.1	0.0