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Cranberry Marsh Estates

FUNCTIONAL SERVICING REPORT

Hill Ridge Homes

File 120181 | July 28, 2023

Document Control

File:	Prepared by:	Prepared for:
120181	**Tatham Engineering Limited** 115 Sandford Fleming Drive, Suite 200 Collingwood, Ontario L9Y 5A6	**Hill Ridge Homes** 110 Jardin Crescent, Suite 14 Vaughan, Ontario L4K 2T7
Date: **July 28, 2023**	**T** 705-444-2565 **tathameng.com**	

Authored by:	Reviewed by:
	
John Birchard, B.Eng., EIT Engineering Intern	Doris Casullo, B.A.Sc., P. Eng. Project Manager, Senior Engineer

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Issue	Date	Description
1	March 4, 2022	Draft Plan Submission
2	December 1, 2022	2nd Submission
3	July 28, 2023	3rd Submission

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1 Introduction

Tatham Engineering Limited has been retained by Hill Ridge Homes to prepare a Functional Service Report in support of Draft Plan Approval for the proposed Cranberry Marsh Estates Residential Development in the Town of Collingwood, County of Simcoe. The primary objective of this report is to address the servicing requirements of the Town of Collingwood and Simcoe County with respect to the existing and proposed sanitary servicing, water supply and distribution, drainage and stormwater management (SWM), safe vehicular access to the site and utilities common to support a residential development (phone, hydro, cable, TV, gas, etc.).

Additional reports have been prepared in conjunction with this report in support of the proposed residential development and are summarized below.

- *Cranberry Marsh Estates Preliminary Stormwater Management Report* prepared by C.C. Tatham & Associates Ltd. (October 2011).
- *Cranberry Marsh Estates Development Traffic Review* prepared by C.C. Tatham & Associates Ltd. (August 2011).
- *Cranberry Marsh Estates Stormwater Management Report* prepared by Tatham Engineering (July 2023).
- *Cranberry Marsh Estates Traffic Impact Brief* prepared by Tatham Engineering (January 2022).

2 Development Site

2.1 SITE LOCATION & ZONING

The subject property consists of approximately 1.29 ha of undeveloped land located south of Highway 26 in the Town of Collingwood. The municipal address of the subject property is 11589 Highway 26. The subject property is bounded by Highway 26 to the north, Trafalgar Road and private residences to the west, Greentree Gardens and Emporium to the east and Cranberry Marsh to the south.

The subject property is zoned as R3-34 (H10) – *Residential Third Density Exception 34*, excluding the southern portion of the site, which is zoned as EP-11 – *Environmental Protection Exception 11*.

2.2 EXISTING CONDITIONS

A topographic survey of the subject property was completed by C.C. Tatham & Associates Ltd. in 2012. The existing grading of the 170 m deep segment of land fronting Highway 26 generally slopes from the south to the north at an average gradient of 0.6%, feeding an existing interceptor ditch, ultimately discharging onto the roadside ditch adjacent to Highway 26. The remainder of the subject property generally slopes from the north to south at an average slope of 0.3%, discharging into the Cranberry Marsh. Refer to the Pre-Development Drainage Plan (DP-1) for details on existing drainage areas.

The site is currently vacant, and primarily tree covered with an environmentally protected marsh area at the south end of the property.

2.3 SUBSURFACE CONDITIONS

A geotechnical investigation, submitted under separate cover, completed by Peto MacCallum Ltd. dated January 2022. Fieldwork was conducted on November 22, 2021, consisting of four boreholes. The boreholes advanced to auger refusal, 3.4 m to 3.7 m below existing ground surface. Subsurface conditions are as follows:

- 50 mm to 200 mm of surficial topsoil;
- Borehole 1 showed a 650 mm layer of silt that was found to be very moist;
- Boreholes 2 - 4 showed a 0.5 m to 1.3 m layer of loose sand with trace amounts of silt and organics. The sand was found to be wet;
- A major till deposit extends below the silt or sand layers to the termination of the boreholes at 3.4 m for Borehole 3, and 3.7 m for Boreholes 1, 2 and 4. The till matrix varied from a silt

and sand with trace gravel and trace clay to a sandy silt with some gravel and trace clay. The till density was loose to very compact; and

- Auger refusal could have been due to boulders in the till or a shallow bedrock common in the area.

Groundwater was measured in the monitoring wells (Boreholes 1, 3 and 4) one month after installation (December 17, 2021). The geotechnical investigation established that the stabilized groundwater table is within 0.5 m of the ground surface.

The soil has been classified as Parkhill loam or silt loam (Type BC), as per the *Soil Survey of Simcoe County - Report No. 29 of the Ontario Soil Survey*, completed by the Ontario Department of Agriculture. This soil group has low to moderate infiltration rates when thoroughly wetted.

2.4 PROPOSED DEVELOPMENT

The proposed development features a 7.2 m wide private road and cul-de-sac, beginning at Highway 26 and extending 220 m towards Cranberry Marsh, followed by a turning circle. The development will feature 5 buildings fronting the private road and cul-de-sac, which will comprise of 26 freehold townhomes. The majority of lots will have 6 m frontages and are 28 m deep. The proposed development is shown on the Site Grading Plan (SG-1).

3 Water Supply & Distribution

3.1 EXISTING INFRASTRUCTURE

There is an existing 300 mm watermain running east-west on the north side of Highway 26, across from the proposed Cranberry Marsh Estates development.

The Collingwood water treatment plant has a rated capacity of 31,140 m^3/day per the Town of Collingwood 2020 Water Compliance Report. The 2019 and 2020 Water Compliance Reports showed maximum day flows of 25,576 m^3/day (82% of rated capacity) and 24,576 m^3/day (79% of rated capacity), respectively. Due to the lack of water supply that could sustain current growth, the Town has recently passed an Interim Control Bylaw (ICBL) that effectively limits the number of new connections for up to one year, while solutions are considered.

The Town is in the process of initiating an environmental assessment for expanding the treatment plant's rated capacity which they estimate will be online in 5 years.

3.2 WATER DEMANDS ASSESSMENT

Water supply demands for the proposed development have been calculated based on the Ministry of the Environment Conservation and Parks (MECP) guidelines and the Town of Collingwood design standards as noted below:

Table 1: Water Supply Design Criteria

DESIGN CRITERIA		SOURCE
Residential Population	2.40 persons/unit	Town of Collingwood
Average Daily Demand Per Person	260 L/person/day	Town of Collingwood
Maximum Daily Demand Factor	8.3	MECP
Peak Hourly Demand Factor	12.4	MECP
Minimum Fire Flow	57 L/s	Town of Collingwood
Allowable Pressure Ranges		
Maximum Pressure	550 kPa (80 psi)	Town of Collingwood
Peak Hour Minimum Pressure	275 kPa (40 psi)	Town of Collingwood
Fire Suppression Minimum Pressure	138 kPa (20 psi)	Town of Collingwood

A population density of 2.40 persons/unit was determined from the Amendment to Town of Development Standards Section 4.4.3.2 which specifies a design population of 2.4 people per unit for 2 to 3 bedroom townhouse units.

Note that peaking factors were interpolated from Table 3-3 of the *Design Guidelines for Drinking-Water Systems* (MOE 2008) based on the design population.

Water demands have been based on 26 units and are calculated as follows:

Design Population (P)	= 26 units x 2.4 persons/units
	= 62.4 persons
Average day demand (ADD)	= P x Average daily demand per person
	= 62.4 persons x 260 L/day
	= 16,224 L/day
	= 16.2 m^3/day (0.19 L/s)
Peak Hour	= ADD x Peak hourly factor
	= 16.2 m^3/day x 12.4
	= 201.8 m^3/day (2.34 L/s)
Maximum day demand (MDD)	= ADD x Maximum daily factor
	= 16.2 m^3/day x 8.3
	= 134.0 m^3/day (1.55 L/s)

3.2.1 Fire Flow Demands

The recommended fire flow demands for the subject property are outlined in the Town of Collingwood Development Standards, as noted in Table 1. Watermains are required to accommodate the minimum required fire flow plus the maximum day use, and is calculated as follows:

MDD plus fire flow	= MDD + Minimum Fire Flow
	= 1.6 L/s + 57 L/s
	= 58.6 L/s

3.3 PROPOSED INFRASTRUCTURE

The proposed water strategy for the Cranberry Marsh Estates development includes tapping into the existing 300 mm diameter watermain on the north side of Highway 26 and extending a 150 mm diameter ductile iron watermain south into the proposed development. Water service connections will extend into each townhouse unit and will be metered separately. In total, 261 m of the proposed 150 mm diameter watermain will run underneath the internal road along the west side. At the cul-de-sac past Block 5, the watermain will reduce to 50mm and complete a loop around the cul-de-sac. Water services to Block 5 will connect to the 50mm watermain loop to prevent water stagnation within the loop. The proposed water system can be seen in the Site Servicing Plan and Profile (PP-1).

3.3.1 Water Service Connections

Each townhouse will connect to the proposed 150 mm diameter watermain via 20 mm diameter copper type K water service connections with curb stop valves as per the Town of Collingwood Development Standards.

Water meters will be installed internal to each individual unit to record water consumption. The proposed buildings will be equipped with backflow prevention devices in accordance with the Ontario Building Code and the Town's water by-law. The backflow prevention devices will also be installed internal to the building to allow for testing and maintenance as may be required.

3.3.2 Fire Protection

Fire hydrants external to the site do not provide sufficient coverage of the proposed development. Three fire hydrants spaced at a maximum of 45m, have been proposed between Blocks 1 and 2, Blocks 3 and 4 and south of Block 5 provide the requisite fire flows for the development. See drawing Site Servicing Plan and Profile (PP-1) for further details pertaining to hydrant location.

Table 2: Proposed Fire Hydrant Flow Rates

HYDRANT	LOCATION DESCRIPTION	MDD + FIRE FLOW (L/S)	REQUIRED PRESSURE (KPA)	PROVIDED FIRE FLOW* (L/S)
North	Block 1/2 Property Line	58.6	138	122.7
Central	Block 3/4 Property Line	58.6	138	81.5
South	South of Block 5	58.6	138	63.8

*Estimated fire flow. Hydrant flow test should be conducted to validate available flow rates

As shown in Table 2, the proposed watermain will provide 63.8 L/s of flow to the proposed south hydrant with a residual pressure of 138 kPa, which is sufficient to meet the minimum required fire flow of 59.6 L/s. The flows in Table 2 are estimates and assume a minimum pressure at watermain along Highway 26 of 413 kPa (60 psi). A hydrant flow test should be conducted to validate the available flow rates to the hydrants. Refer to Appendix B for the supporting water service calculations.

3.3.3 Water Availability

The proposed 150 mm diameter watermain has sufficient capacity for the required peak hour demand, providing 43.0 L/s of flow. The peak hour demand of the subject site is 3.9 L/s. Refer to Appendix B for Watermain Capacity Calculations.

As outlined above, the Town of Collingwood drinking water treatment plant has a rated capacity of 31,140 m^3/day and the maximum day flow in 2020 was 24,576 m^3/day. The proposed maximum day flow of 223.5 m^3/day for the proposed Cranberry Marsh Estates Development could be accommodated by the treatment plant (excess capacity is 6,564 m^3/day).

The Town of Collingwood operates a hydraulic model of their water distribution system. We request that the Town add the proposed development to their model and confirm that the above demand flows can be achieved within the applicable pressure ranges as outlined above.

4 Sanitary Sewage Collection & Conveyance System

4.1 EXISTING INFRASTRUCTURE

An existing 750 mm diameter sanitary sewer is located north of the proposed property, flowing west to east under the north side of Highway 26. Drawings have indicated that there is a 200 mm diameter sanitary sewer stub that services the subject property, however it's existence will require confirmation. The ultimate discharge location is the Collingwood Wastewater Treatment Plant (WWTP).

In reviewing the Collingwood Wastewater Treatment Plant Annual Compliance Reports for 2019 and 2020, the plant has an average day flow rated capacity of 24,548 m^3/day and a maximum day flow rate capacity of 60,900 m^3/day. The 2019 reports show an average day flow and maximum day flow of 16,202 m^3/day and 33,460 m^3/day respectively, whereas the 2020 reports show an average day flow and maximum day flow of 18,854 m^3/day and 48,370 m^3/day respectively. Taking the more conservative 2020 numbers, the plant is operating at 74% of its average day flow rated capacity and 79% of its maximum day flow rated capacity. The available average day and maximum day surplus capacities are 5,694 m^3 and 12,530 m^3 respectively.

4.2 SANITARY DEMANDS ASSESSMENT

4.2.1 Sewage Demands

Design Population (P)	= 26 units x 2.4 persons/units
	= 62.4 persons
Infiltration (I)	= Infiltration Flow x Site Area
	= 0.23 L/ha/s x 1.29 ha
	= 0.30 L/s = 25,635 L/day = 25.6 m^3/day
Average day flow (ADF)	= P x Average daily demand per person + I
	= 62.4 persons x 260 L/day + 25,635 L/day
	= 16,224 L/day + 25,635 L/day
	= 41,859 L/day
	= 41.9 m^3/day = 0.48 L/s
Harmon's Peaking Factor (M)	= 1 + 14 ÷ (4 + $\sqrt{P}$/1000)

$= 1 + 14 \div (4 + \sqrt{(62.4/1000)})$

$= 4.29$

Maximum Day Flow $= (ADF\text{-}I) * PF + I$

$= (41.9\ m^3/day - 25.6\ m^3/day) \times 4.29 + 25.6\ m^3/day$

$= 95.3\ m^3/day = 95{,}305\ L/day = 1.1\ L/s$

Note the maximum day peaking factor was calculated using the Harmon Formula as per Town standards.

4.3 PROPOSED INFRASTRUCTURE

Sanitary discharge from the proposed units fronting the internal road will drain to the existing sanitary sewer via 226 m of 200 mm diameter PVC sanitary sewer that originates north of the cul-de-sac, and flows north, where it will connect to the existing 750 mm diameter sanitary sewer on Highway 26. A maintenance structure will be provided at the property line. The connection point to the trunk sewer was investigated and it was confirmed that the existing sanitary sewer is at sufficient depth for a sanitary service connection to the proposed buildings.

The proposed sanitary sewer system can be seen of the Site Servicing Plan and Profile (PP-1).

4.3.1 Sanitary Service Connections

Each unit will connect to the proposed 200 mm diameter sanitary sewer via 125 mm diameter PVC sanitary service as per the Town of Collingwood Development Standards.

4.3.2 Sewage Capacity

As noted above, the Collingwood WWTP has average day and maximum day available capacities of 24,548 m^3/day and 60,900 m^3/day respectively whereas the proposed average day and maximum day flows for the proposed development are 41.9 m^3/day and 95.3 m^3/day respectively (average day and maximum day surplus capacities are 5,694 m^3 and 12,530 m^3 respectively). Therefore, the Collingwood WWTP can accommodate the increased sanitary flows from the Cranberry Marsh Estates Development.

The Town of Collingwood operates a hydraulic model of their sanitary sewer system. We request that the Town add the proposed development to their model and confirm that the downstream sewer system can accommodate the proposed development.

5 Stormwater Management

A separate Stormwater Management (SWM) Report has been prepared by Tatham Engineering to address drainage and stormwater management requirements for the development. A summary of the SWM servicing strategy is as follows:

- Enhanced water quality protection will be provided by an enhanced swale that will retain first flush runoff up to the 25 mm storm event to remove suspended solids before draining into the Cranberry Marsh. Runoff from events greater than the 25 mm storm will bypass the enhanced swale and be conveyed directly to the Cranberry Marsh. Runoff directed towards the Highway 26 roadside ditch is considered clean and does not require quality treatment.
- Proposed condition peak flow rates discharging into the Highway 26 roadside ditch north of the site will be controlled to existing condition rates for all storms up to and including the 100-year event to ensure no adverse impacts for downstream landowners. As the Cranberry Marsh is the ultimate receiving waterbody for site drainage to the south, quantity control is not required.
- Emergency overland flow routes will be conveyed towards the cul-de-sac, which will convey peak flows to various low points. The maximum ponding depth will not exceed 150 mm before spilling over the east road curb, and flowing overland towards the enhanced swale, and ultimately into the Cranberry Marsh.
- Siltation and erosion controls will be implemented for all construction activities, including topsoil stripping, material stockpiling, road construction and grading operations.

6 Transportation

A Traffic Impact Brief has been completed by Tatham Engineering under separate cover. A summary of the conclusions and recommendations are as follows:

- There is adequate spacing between the site access and to the immediate east the neighbouring Greentree Gardens and Emporium access.
- A left turn lane currently exists on Highway 26 to serve the site and thus there are no further requirements in this regard.
- The available sight lines on Highway 26 to the east and west of the site access exceed the minimum stopping sight distance requirement for a design speed of 70 km/h.
- Given the limited traffic volume to be generated by the development there will not be any significant operational impacts on the operations of Highway 26 and the surrounding lands. The operational assessment of the site access indicates that the intersection will experience adequate levels of service and average traffic delays for the northbound movements exiting the site through the 2031 horizon year. Therefore, no operational improvements are required.

7 Utilities

7.1 ELECTRICAL SERVICES

Tatham Engineering has reviewed the proposed development from an electrical servicing standpoint and has provided an electrical distribution plan based on EPCOR standards as part of the second submission.

7.2 GAS SERVICES

Enbridge Gas was contacted about their existing gas mains in the area and their ability to service the proposed development. Enbridge has previously noted that there is an ongoing pressure increase project along the Barrie to Collingwood line is complete. This project is expected to be completed in the fall of 2022.

7.3 TELEPHONE AND INTERNET SERVICES

Bell has been contacted regarding available services in the area.

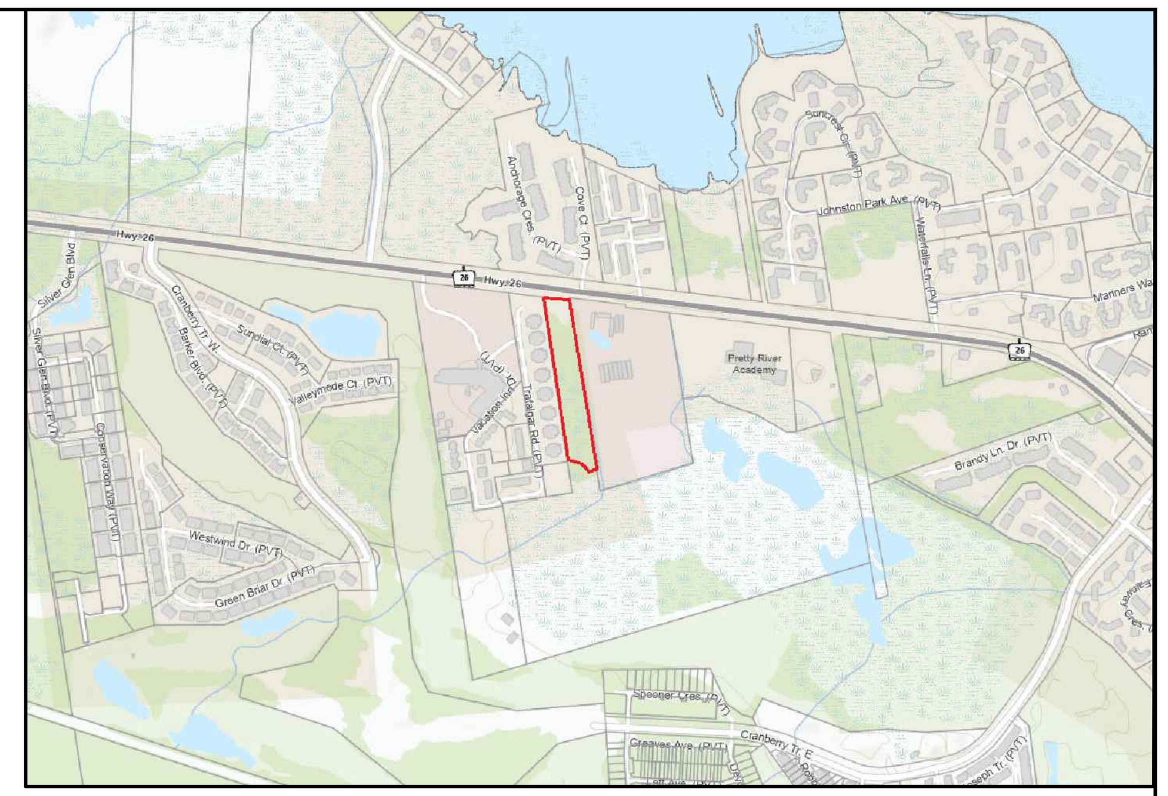
Rogers has been contacted regarding available services in the area. They have indicated that Roger's has infrastructure along the section of Highway 26 adjacent to the proposed site and would be able to service the site.

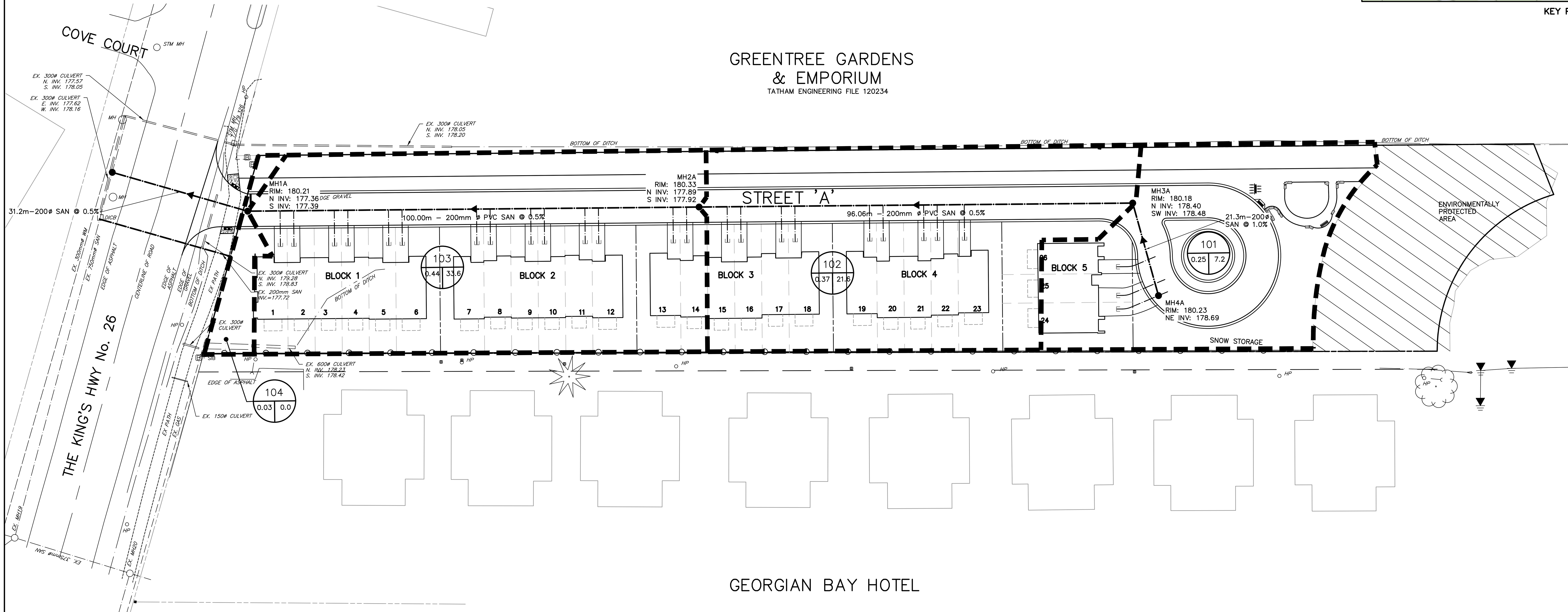
8 Summary

As outlined above, existing infrastructure surrounding the subject property can adequately service the development for sanitary sewage, potable water, hydro, natural gas, and telecommunications. A Stormwater Management Brief submitted under separate cover confirms that applicable runoff, quantity, quality, and erosion targets will be met. Additionally, a Traffic Impact Brief submitted under separate cover confirms that the proposed development will not adversely affect the existing surrounding road network. A summary of the servicing strategy is as follows:

- Potable water will be provided by connecting into the existing 300 mm diameter watermain on the north side of Highway 26 with a proposed 150 mm ductile iron watermain that extends into the subject site. The proposed watermain has sufficient capacity for the required peak hour demand.
- Fire hydrants will be required between Blocks 1 and 2, Blocks 3 and 4 and south of Block 5. The proposed watermain will loop around the cul-de-sac with a 50 mm line.
- Sanitary flows from the proposed development will drain to the existing municipal 750 mm diameter sanitary sewer along Highway 26. A 200mm sanitary sewer and maintenance structure at the property line is proposed to service the development.
- Tatham Engineering has reviewed the proposed development from an electrical servicing standpoint and has provided an electrical distribution plan based on EPCOR standards.
- Rogers has confirmed telephone and internet infrastructure along the section of Highway 26 adjacent to the proposed site and will be able to service the site.

Additional details related to the various servicing components will be provided at the detailed design stage. Detailed drawings will be completed for approval by the Town and relevant regulatory agencies to clear the conditions of Draft Plan Approval and associated Site Plan Agreement.





LEGEND

SANITARY AREA BOUNDARY	
AREA IDENTIFICATION NUMBER	101
AREA IN HECTARES	0.25
POPULATION BASED ON 2.4 PERSONS PER UNIT	7.2

TOWN APPROVAL

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BENCHMARKS

ELEVATIONS SHOWN ON THIS PLAN ARE RELATED TO GEODETIC DATUM AND ARE DERIVED FROM BENCH MARK No. 0011972U311 HAVING A PUBLISHED ELEVATION OF 181.032 METRES.

NOTES

LEGAL SURVEY INFORMATION AND LOT DIMENSIONS SHOWN ON THIS PLAN ARE TAKEN FROM A SURVEY PLAN PREPARED BY PATTEN & THOMSEN LTD, DATED, JANUARY 2, 2012 JOB No. 66-170-6

TOPOGRAPHIC SURVEY COMPLETED BY TATHAM ENGINEERING OCTOBER, 2022.

No.	REVISION DESCRIPTION	DATE
1.	1ST SUBMISSION	03/22
2.	2ND SUBMISSION	12/22
3.	3RD SUBMISSION	07/23

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2023.07.28
D. M. CASULLO
PROVINCE OF ONTARIO

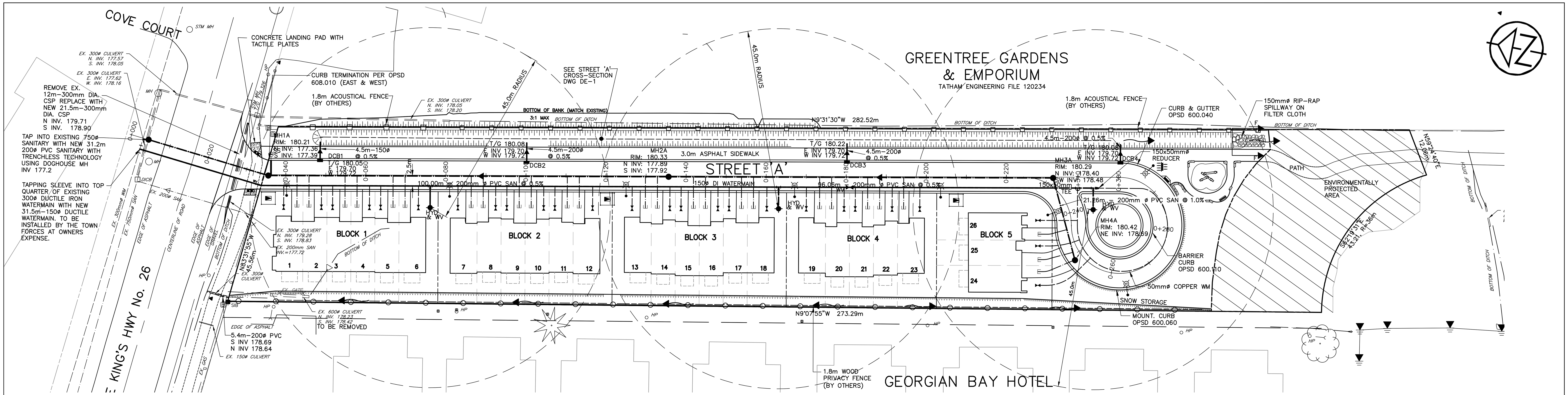
CRANBERRY MARSH ESTATES
TOWN OF COLLINGWOOD

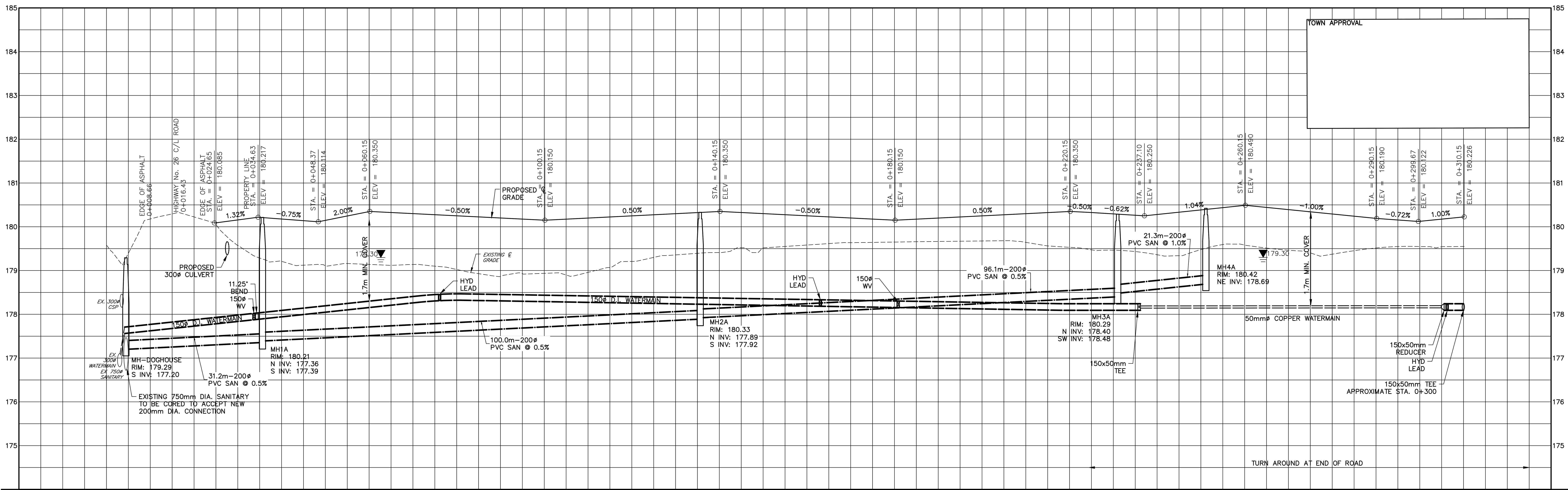
SANITARY DRAINAGE PLAN

TATHAM ENGINEERING

DESIGN: KG	FILE: 120181	DWG:
DRAWN: KH/SBU/AP	DATE: NOV 2021	SAN-1
CHECK: DC	SCALE: 1:500	

Drawing Name: 120181 – SAN01.dwg, Plotted: Aug 01, 2023





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EXISTING C/L ELEV	179.58	180.22	179.21	179.14	179.02	178.94	179.21	179.41	179.58	179.65	179.68	179.53	179.36	179.58	179.37	179.55		
CHAINAGE	0+000	0+020	0+040	0+060	0+080	0+100	0+120	0+140	0+160	0+180	0+200	0+220	0+240	0+260	0+280	0+300	0+320	

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CRANBERRY MARSH ESTATES
TOWN OF COLLINGWOOD

TATHAM ENGINEERING

SITE SERVICING
PLAN AND PROFILE

DESIGN: KG	FILE: 120181	DWG:
DRAWN: KH/SBU/AP	DATE: MAR 2022	PP-1
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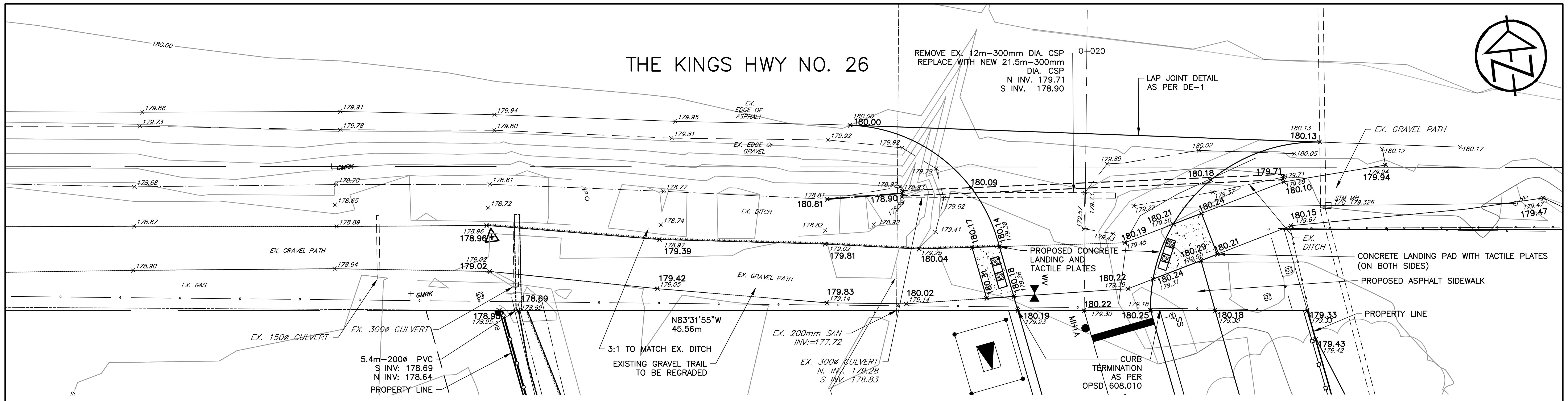

GRADING WITHIN HWY#26 RIGHT OF WAY
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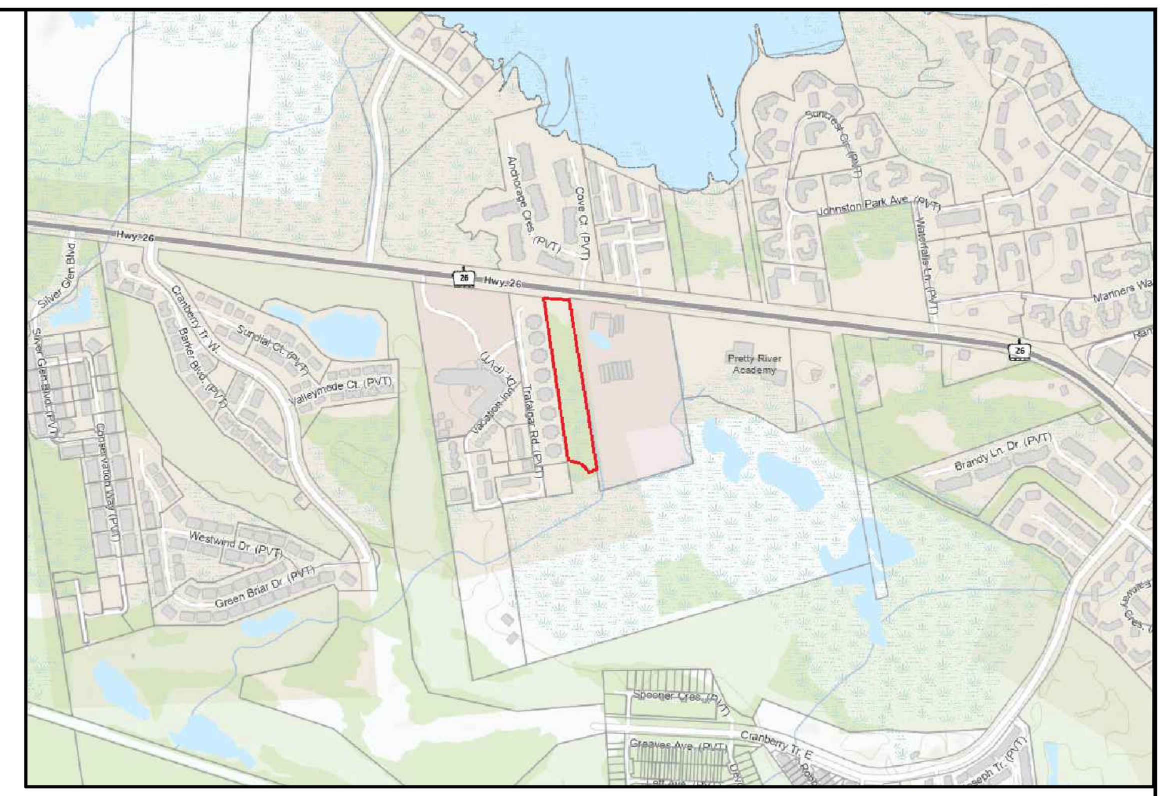
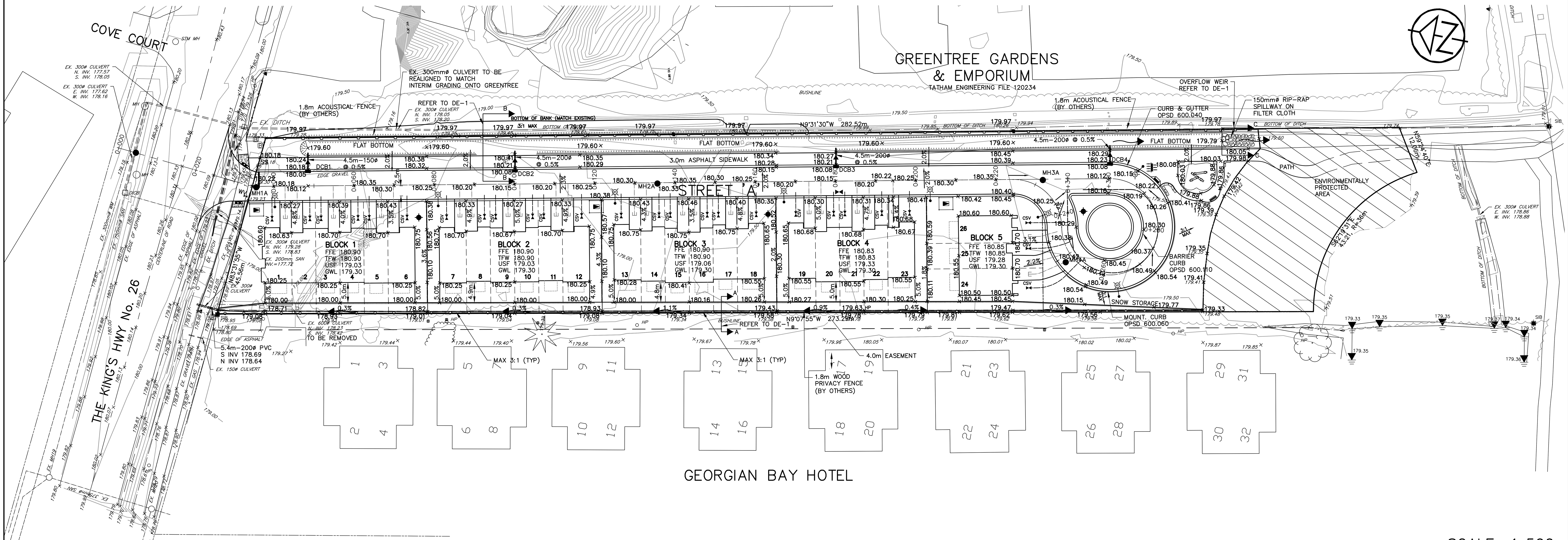

KEY PLAN



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TOWN APPROVAL

CRANBERRY MARSH ESTATES
TOWN OF COLLINGWOOD

TATHAM ENGINEERING

SITE GRADING PLAN

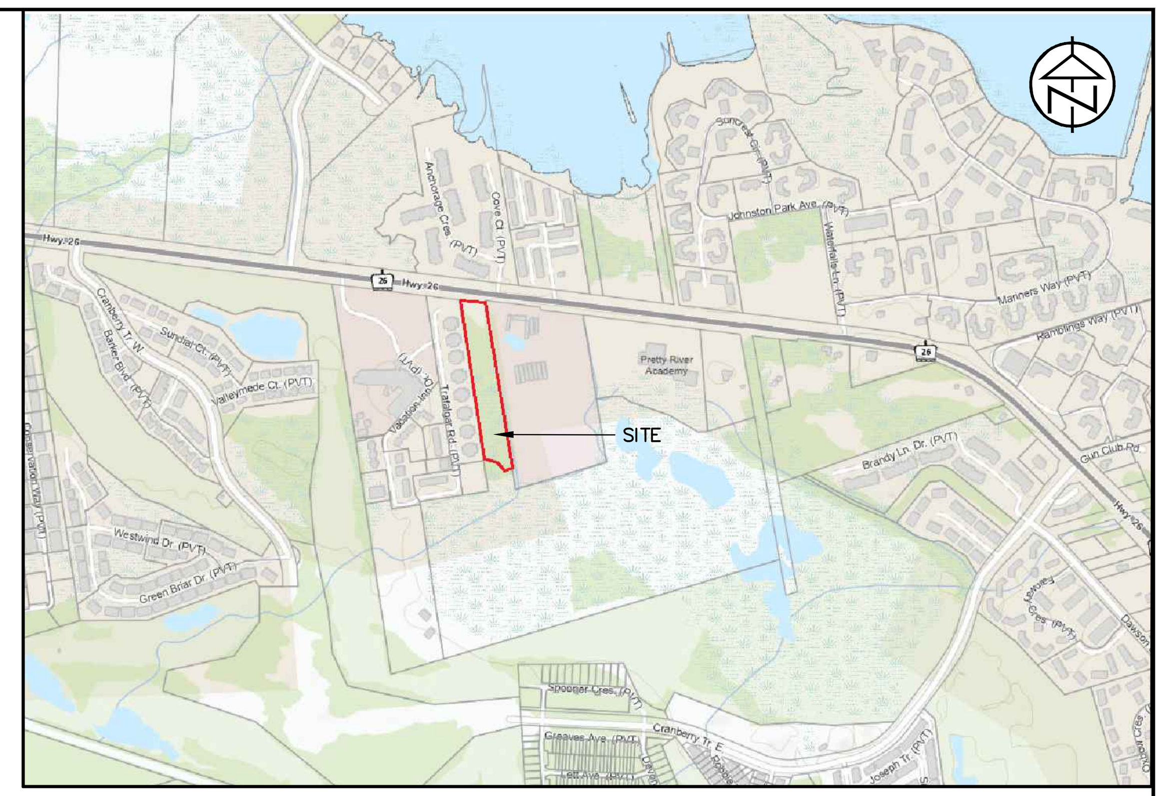
DESIGN: KH	FILE: 120181	DWG:	SG-1
DRAWN: KH/SBU/AP	DATE: OCT 2021		
CHECK: DC	SCALE: AS NOTED		

Drawing Name: 120181 – SG01.dwg, Plotted: Aug 02, 2023

LEGEND

- AREA BOUNDARY
- AREA IDENTIFICATION NUMBER (101)
- AREA IN HECTARES (1.40)
- RUNOFF COEFFICIENT (0.25)
- EXISTING MAJOR OVERLAND FLOW DIRECTION
- EXISTING DITCH FLOW DIRECTION

TOWN APPROVAL



KEY PLAN

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PROVINCE OF ONTARIO

CRANBERRY MARSH ESTATES
TOWN OF COLLINGWOOD

TATHAM ENGINEERING

PRE-DEVELOPMENT DRAINAGE PLAN

DESIGN: KG	FILE: 120181	DWG:
DRAWN: KH/SBU/AP	DATE: DEC 2021	**DP-1**
CHECK: DC	SCALE: 1:500	

Drawing Name: 120181 - DP01.dwg, Plotted: Aug 01, 2023

Appendix A:
Sanitary Design Sheet

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Sanitary Sewer Design Sheet

Version Number: 3 Version Date: July 25, 2023

Project Information

Cranberry Marsh Estates	120181

Drawing Reference

SAN01	July 25/23

Prepared By

John Birchard	July 25/23

Reviewed By

Doris Casullo	July 25/23

Municipality

Town of Collingwood

Population Density

Capita per Unit	Low	Medium	High
	-	2.40	-

Infiltration

Infiltration (L/s/ha)	0.23

Flow

Development Type	Average (L/cap/day)	Peaking Factor
Residential	260	Harmon
Development Type	**Average (L/ha/day)**	**Peaking Factor**
Institution	-	-
Commercial	-	-
Industrial High Intensity	-	-
Industrial Low Intensity	-	-

Manning's Coefficient

Pipe Material	Value
Concrete	0.013
PVC	0.013
Applied	0.013

Engineer Stamp

Street Name	Area Label/ID	Upstream Maintenance Hole	Downstream Maintenance Hole	Development Type	Population Density	Number of Units	Population (cap)	Accumulated Population (cap)	Peaking Factor	Area (ha)	Cumulative Area (ha)	Average Flow (L/s) Development	Average Flow (L/s) Infiltration	Average Flow (L/s) Total	Peak Flow (L/s) Development	Peak Flow (L/s) Infiltration	Peak Flow (L/s) Total	Proposed Sanitary Sewer Sewer Length (m)	Sewer Slope (%)	Actual Sewer Diameter (mm)	Full Flow Velocity (m/s)	Full Flow Capacity (L/s)	Actual Velocity (m/s)	Calculated Sewer Diameter (mm)	Percentage of Full Flow Capacity (%)
Street A	101	MH4A	MH3A	Residential	Med.	3	7.2	7.2	4.43	0.25	0.25	0.02	0.06	0.08	0.10	0.06	0.15	21.1	1.0%	200	1.04	32.80	0.28	27	0.5%
Street A	102	MH3A	MH2A	Residential	Med.	9	21.6	28.8	4.36	0.37	0.62	0.09	0.14	0.23	0.38	0.14	0.52	96.1	0.5%	200	0.74	23.19	0.30	48	2.2%
Street A	103	MH2A	MH1A	Residential	Med.	14	33.6	62.4	4.29	0.44	1.06	0.19	0.24	0.43	0.81	0.24	1.06	100.0	0.5%	200	0.74	23.19	0.37	63	4.6%
Street A	104	MH1A	EX SAN	Residential	Med.	0	0.0	62.4	4.29	0.03	1.09	0.19	0.25	0.44	0.81	0.25	1.06	31.2	0.5%	200	0.74	23.19	0.37	63	4.6%

Appendix B:
Watermain Capacity Calculations

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	Project:	Cranberry Marsh Estates
	File No.:	122182
	Date:	July, 2023
	Design:	JB
	Checked:	DC
	Revision:	3

PEAK FLOW - CRANBERRY MARSH ESTATES

Calculation of Water Flow Rates for Different Pipe Sizes
(Hazen Williams Formula - S.I. units)

Pipe Material:	DI	
Hazen Williams Coefficient, **C** =	110	
Assumed static pressure in pipe =	413	kN/m^2
Assumed acceptable residual pressure =	275	kN/m^2
Pressure drop over the pipe length, DP =	**138**	**kN/m^2**

Pipe Length (m)	Water Flow Rate (m^3/hr)									
	Pipe Diameter (mm)									
	20	25	40	50	75	100	130	**150**	200	250
261	0.8	1.4	4.8	8.6	25.1	53.4	106.5	**155.1**	330.5	594.4

Flow rate provided by 150 mm dia. water service	=	155.1	m^3/hr
	=	**43.08**	**L/s**

Water Demand (from Section 3.2 Water Demand Assessment)

Peak Hour Demand	Residential Units	**2.34**	**L/s**

Check: $Q_{provided}$ = **43.08 L/s > $Q_{required}$ = 2.34 L/s** **Acceptable**

Hazen Williams Equation as used in this spreadsheet:

$$Q = (3.763 \times 10^{-6})\ C\ D^{2.63}(DP/L)^{0.54}$$

where Q is the water flow rate in m^3/hr
D is the pipe diameter in mm
L is the pipe length in m
DP is the pressure difference across pipe length L in kN/m^2

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FIRE FLOW - CRANBERRY MARSH ESTATES

Calculation of Water Flow Rates for Different Pipe Sizes
(Hazen Williams Formula - S.I. units)

Pipe Material:	DI	
Hazen Williams Coefficient, **C** =	110	
Assumed static pressure in pipe =	413	kN/m^2
Assumed acceptable residual pressure =	138	kN/m^2
Pressure drop over the pipe length, DP =	**276**	$\mathbf{kN/m^2}$

Hydrant Location	Pipe Length (m)	Water Flow Rate (L/s)								
		Pipe Diameter (mm)								
		25	40	50	75	100	130	**150**	200	250
North	75	1.1	3.8	6.8	19.8	42.2	84.2	**122.7**	261.5	470.3
Central	160	0.7	2.5	4.5	13.2	28.1	55.9	**81.5**	173.7	312.4
South	252	0.6	2.0	3.5	10.3	22.0	43.8	**63.8**	135.9	244.4

Minimum Required Fire Flow (from Table 2)

North Hydrant:	$Q_{req,N}$ =	58.6	L/s	<	$Q_{pro,N}$ =	122.7	L/s	**Acceptable**
Central Hydrant:	$Q_{req,C}$ =	58.6	L/s	<	$Q_{pro,C}$ =	81.5	L/s	**Acceptable**
South Hydrant:	$Q_{req,S}$ =	58.6	L/s	<	$Q_{pro,S}$ =	63.8	L/s	**Acceptable**

Hazen Williams Equation as used in this spreadsheet:

$$Q = (3.763 \times 10^{-6})\ C\ D^{2.63}(DP/L)^{0.54}$$

where Q is the water flow rate in m^3/hr
D is the pipe diameter in mm
L is the pipe length in m
DP is the pressure difference across pipe length L in kN/m^2

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SINGLE RESIDENTIAL UNIT

Calculation of Water Flow Rates for Different Pipe Sizes

(Hazen Williams Formula - S.I. units)

Pipe Material:	Copper	
Hazen Williams Coefficient, C =	140	
Minimum acceptable pressure in pipe =	275	kN/m^2
Assumed acceptable residual pressure =	138	kN/m^2
Pressure drop over the pipe length, DP =	**138**	**kN/m^2**

Pipe Length (m)	Water Flow Rate (m^3/hr)									
	Pipe Diameter (mm)									
	12	**20**	25	40	50	65	75	100	130	150
10	1.5	**5.7**	10.3	35.5	63.8	127.1	185.2	394.8	787.1	1146.7

Flow rate provided by 20 mm diameter water service

$Q_{provided}$ = 5.7 m^3/hr

$Q_{provided}$ = 1.59 L/s

Water Demand For Single Unit

Average Daily Demand (ADF) = (2.4 cap/unit) x (450L/cap/day) =	624	L/day
Peak Hour Factor per Design Guidelines for Drinking Water Systems (PF) =	11.5	
Peak Hour Demand, $Q_{required}$ = PF x ADF =	0.08	L/s

Check: $Q_{provided}$ = 1.59 L/s > $Q_{required}$ = 0.08 L/s Acceptable

Hazen Williams Equation as used in this spreadsheet:

$$Q = (3.763 \times 10^{-6})\ C\ D^{2.63}(DP/L)^{0.54}$$

where Q is the water flow rate in m^3/hr
D is the pipe diameter in mm
L is the pipe length in m
DP is the pressure difference across pipe length L in kN/m^2

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<u>Calculation of Sanitary Flow Rates</u>

(Mannings Formula - S.I. units)

Flow rate provided by 125 mm diameter sanitary service

Pipe Material:	PVC	
Manning's Coeff. =	0.013	
Slope =	0.01	m/m
Diameter =	0.125	m
Area =	0.012	m^2
Perimeter =	0.39	m
Hydraulic Radius =	0.03	m
Flow Rate, $Q_{provided}$ =	0.009	m^3/s
Flow Rate, $Q_{provided}$ =	9.37	L/s

Sanitary Loading

Peak Daily Flow =	0.08	L/s	(As per Water Demand Calculation)
Extraneuos Flow Criteria =	0.23	L/ha-s	
Contribuiting Area =	0.03	ha	(1.63ha developable boundary / 58 units)
Extraneuos Flow =	0.01	L/s	

Required Flow, $Q_{required}$ = Max. Daily Flow • Peak Factor + Extraneous Flow = 0.09 L/s

Check:	**$Q_{provided}$ =**	**9.37**	**L/s**	**>**	**$Q_{required}$ =**	**0.09**	**L/s**	**Acceptable**

Manning's Equation as used in this spreadsheet:

$Q = (A \bullet R^{2/3} \bullet S^{1/2})/n$

where Q is the water flow rate in m^3/s
A is the pipe area in m^2
R is the pipe hydraulic radius in m
S is the slope of the pipe
n is the Manning's Coefficient