FUNCTIONAL SERVICING AND STORMWATER MANAGEMENT REPORT

DAVIS TOWNS
1015 DAVIS DRIVE
TOWN OF NEWMARKET

PREPARED FOR:
LULU HOLDINGS INC.
36 FAIRWAY HEIGHTS DRIVE
THORNHILL, ON L3T 3A8

DATE: June 2023

PROJECT NO. 17932

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1.0 INTRODUCTION

The purpose of this report is to provide site servicing and stormwater management (SWM) design information in support of the Official Plan, Zoning By-law Amendment and Site Plan Approval applications for the proposed development at 1015, 1025 and 1029 Davis Drive and 22 Hamilton Drive in the Town of Newmarket.

Specifically, this report will demonstrate the SWM measures that will be undertaken to deal with the quantity, quality and water balance requirements for the site. As well, the proposed servicing for the site will be reviewed to confirm the impact on the existing municipal drainage systems.

1.1 Site Description

The site bordered by detached residential development to the west and north, Davis Drive to the south, Hamilton Drive to the east. There are four existing detached residential houses on the site that will be demolished as part of the development. The site area is approximately 6,350m^2.

It is proposed to construct a 28-townhouse residential development on the property. The site location is shown on **Figure 1**.

1.2 Background

The SWM design for the site has been prepared to meet the requirements of the Town of Newmarket and Lake Simcoe Region Conservation Authority (LSRCA). The following materials were referenced in the preparation of this report:

- The Town of Newmarket's Site Plan Approval Process Manual.
- Town of Newmarket, Engineering Design Standards and Criteria, dated August 2019.
- The LSRCA Technical Guidelines for Stormwater Management Submissions, dated April 2022.
- The Phase II Environmental Site Assessment, dated March 2014, prepared by G2S Environmental Consulting Inc.
- The Stormwater Management Planning and Design Manual (MECP Guidelines), prepared by the Ministry of the Environment, Conservation and Parks, March 2003, were referenced in the preparation of the stormwater management plan.
- As-constructed plan and profile drawings for Davis Drive and Hamilton Drive, as provided by the Town of Newmarket.
- Town of Newmarket's Wayne and Waratah Area Stormwater Management Study, prepared by AECOM Canada Ltd., revised January 2018.

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FIGURE 1
1015 DAVIS DRIVE
SITE LOCATION PLAN

DATE: JUNE 2023 SCALE: N.T.S. PROJECT: 17932

2.0 STORMWATER MANAGEMENT CRITERIA

As outlined in the LSRCA Guidelines, a treatment train approach is required to meet the multiple SWM objectives of water quality, water balance, erosion and flood control. Lot level and conveyance controls are best for achieving water balance objectives.

The requirements are as follows:

Water Balance – the LSRCA Watershed Development Policies state that a SWM plan must make every feasible effort to maintain the pre-development infiltration and evapotranspiration rates and temperatures to the receiving watershed. A water balance assessment is required as per the MECP Guidelines.

Water Quality – the LSRCA Guidelines state that 80% of total suspended solids should be removed in the post development scenario, and that phosphorus loading should be reduced as much as feasible and not exceed predevelopment levels. The proposed landscaping and infiltration measures, along with the roof area, result in water quality requirements generally being addressed through the design.

Water Quantity – The flood flow requirement is to control the 2 through 100-year post development flows to the pre-development levels. For discharge to municipal infrastructure all flows up to the 100-year event are required to be controlled to the 5-year pre-development target.

3.0 SITE GRADING

All grading will be completed in a manner to satisfy the following goals:

- Enable gravity servicing connections (as required) to the existing sewers on Davis Drive.
- Provide minimal impact to abutting properties, as feasible.
- Achieve stormwater management and environmental objectives required for the site

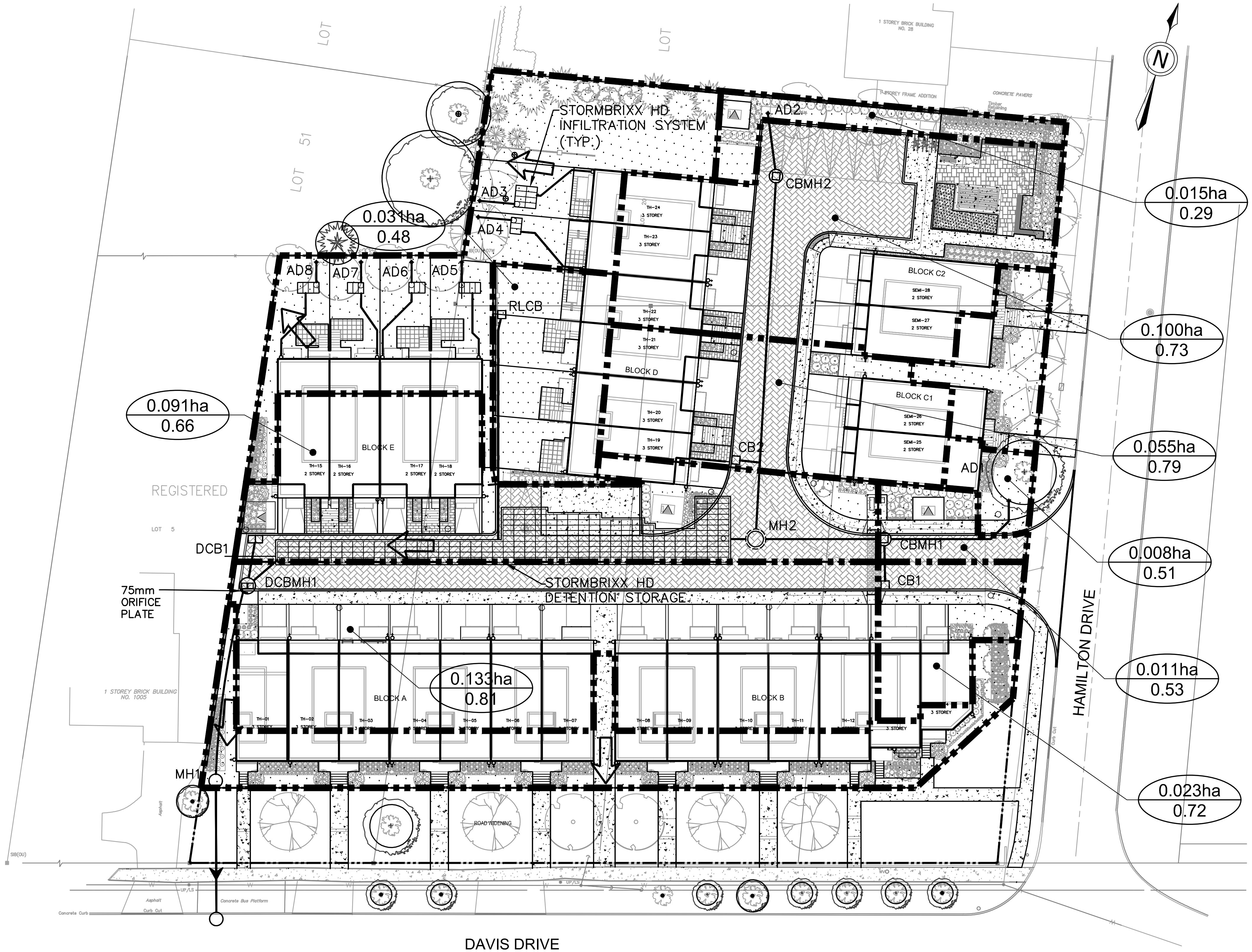
The site will be graded to suit the Town's design criteria and accommodate any constraints that may be imposed by the storm drainage and servicing objectives. Details can be referenced on **Drawing SW1.**

3.1 Minor System Drainage

The majority of the site grading will direct flows overland toward internal roadways. The roadways generally slope from east to west and from north to south. All drainage directed towards the roadways will be captured by the onsite storm system for all storms up to the 100-year event. A rear lot catchbasin is also proposed to the west of Block D to capture additional drainage. The northwest corner of the site is too low to be picked up in the internal site storm system, therefore, the design includes infiltration systems in the backyard of six townhouse units to provide controls on site with only the emergency overland flow being directed to the adjacent property to match existing condition. Additionally, the south and east perimeter of the site will drain overland towards the adjacent right-of-ways.

3.1.1 Catchbasin Inlet Capacity

The inlet capacity of each catchbasin was evaluated to ensure the 100-year storm event could be captured with 50 percent blockage. Flows to catchbasins as well as their inlet capacities are shown in **Table 1**. The catchment areas to each catchbasin are shown in **Figure 3**.



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LEGEND

CATCHMENT BOUNDARY

0.133ha — CATCHMENT AREA

0.81 — 5-YR RUNOFF COEFFICIENT

OVERLAND FLOW DIRECTION

PERVIOUS AREA

PROPOSED STORM SEWER

FIGURE 3
1015 DAVIS DRIVE
STORM DRAINAGE PLAN

DATE: JUNE 2023 SCALE: 1:500 PROJECT: 17932

Table 1. Catchbasin Inlet Capacity

Catchbasin	Catchment Area (ha)	Post-Development Peak Flow (L/s)	Inlet Capacity (With 50% blockage) (L/s)*
DCBMH1	0.133	89.51	87.5
CBMH1	0.011	5.68	22
CBMH2	0.100	62.84	55
DCB1	0.091	54.16	87.5
CB1	0.023	13.32	22
CB2	0.055	36.22	25
RLCB	0.031	14.07	60
AD1	0.008	3.65	21.5
AD2	0.015	5.05	16.5

*Catchbasin and Area Drains flow taken from MTO Drainage Manual Design, and Nyloplast Inlet Capacity Chart, respectively.

The inlet capacity has been calculated based on the ponding depth of each catchbasin/catchbasin manhole except for CBMH1, CB1 and CB2, as per *MTO Design Chart 4.19: Inlet Capacity at Road Sag*. CBMH1, CB1, and CB2 are located on a continuous grade; therefore; *MTO Design Chart 4.14* has been used to determine the inlet capacity. The upstream CB3 and DCBMH1 have a surplus run-off of 13.23L/S from the 100-year event which will flow to DCB1 with excess capacity of 33.34 L/s in total. Thus, all the runoff will be captured before leaving the site. As in Table 1, after accounting for 50 percent blockage, all the catchbasins and catchbasin manholes have adequate capacity to capture the 100-year event.

The inlet capacity of each AD has been calculated based on the maximum ponding depth. Based on the Design Chart of Nyloplast Area Drains, ADs will have a total inlet capacity of 76L/s. After accounting for 50 percent blockage, the inlet capacity will be reduced to 38L/s which exceeds the 100-year peak flow. Therefore, the Area Drains have adequate capacity to capture the 100-year event.

The maximum amount of ponding will occur at DCBMH1 and DCB1 which are located at the same low end of the site. The total storage available on the surface before spilling overland based on a depth of 0.15m is $0.9m^3$. The average release rate from both outlets was calculated to be 70L/s with 50 percent blockage accounted for. Therefore, the worst-case scenario for ponding will be able to drain down in approximately 13 seconds and there are no concerns with extended surface ponding.

The calculations are provided in **Appendix A**.

3.2 Major System Drainage

In the event of blockage or a storm event exceeding the 100-year intensity, a major overland flow route has been designed to safely convey the flows towards the Davis Drive right-of-way. In this event, all drainage will be directed overland with a maximum ponding depth of 0.18m along the roadways to the southwest. Upon reaching the southwest corner of the roadway, it will overtop and flow along the western side yard of Block A towards Davis Drive.

In the event of a full blockage, the uncontrolled 100-year storm events peak flow would be 330L/s. The channel along the western side of Block A has been designed to have a retaining wall 0.3m high along one side, a bottom width of 0.5m and a side slope adjacent to the block of approximately 5:1. Based on these parameters and a channel slope of 3.8 percent, the critical section where the channel is the narrowest can convey 684L/s. If the entire 100-year storm event's peak flows down this side lot, the maximum flow depth would be 0.205m, which would not overtop the 0.3m retaining wall.

Manning's flow for the open channel calculations is provided in **Appendix A**.

4.0 STORMWATER MANAGEMENT PLAN

The proposed stormwater management plan for the site is based on meeting the requirements of the LSRCA and Town of Newmarket.

The site has been designed to limit the peak release rate from the proposed development to meet the existing flows from the site. The existing site is composed of a number of split draining lots, with the front lots draining towards Davis Drive and the rear lots which make up the majority of the lot area, draining toward the rear and neighboring properties. The existing site catchment parameters are outlined in **Table 2** and **3** below.

Table 2. Site Imperviousness and Runoff Coefficient – Drainage to Davis Drive

Surface Type	Area (ha)	Imperviousness (%)	Runoff Coefficient (RC)	RC x A
Pervious Area	0.07	0%	0.20	0.014
Impervious Area	0.07	100%	0.90	0.063
Total	**0.14**	**50%**	**0.55**	0.077

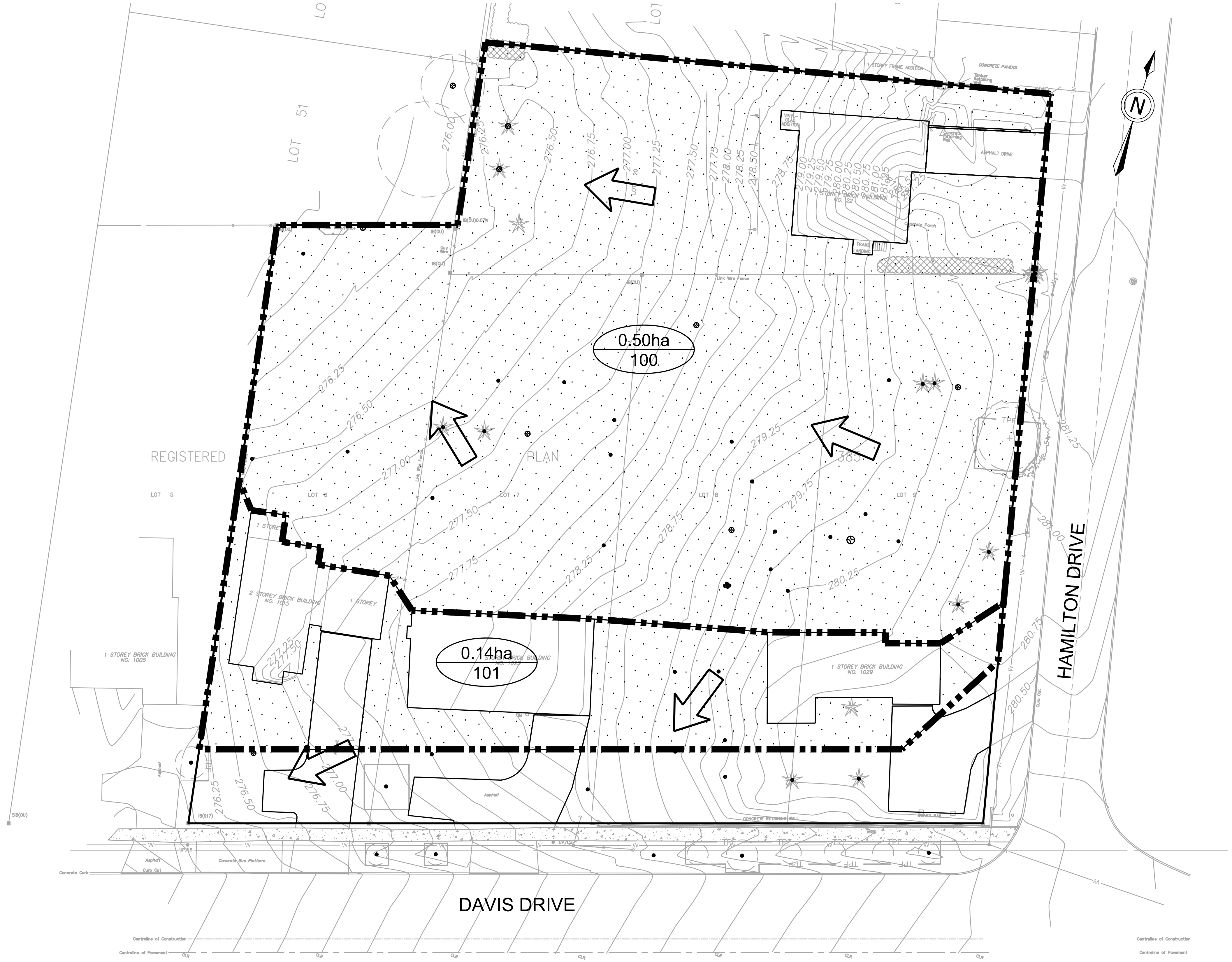
Table 3. Site Imperviousness and Runoff Coefficient – Drainage to Rear Lots

Surface Type	Area (ha)	Imperviousness (%)	Runoff Coefficient (RC)	RC x A
Pervious Area	0.47	0%	0.20	0.094
Impervious Area	0.03	100%	0.90	0.027
Total	**0.50**	**5%**	**0.24**	0.121

Refer to **Figure 2** for details of the existing site conditions.

4.1 Water Balance

The guidelines require retention of water on site, to the extent possible, to match pre-development runoff volumes. As no detailed water balance study has been completed at this time, an initial abstraction comparison will be used to determine the suitability of the proposed low impact development techniques.



CATCHMENT 100 - TO REAR LOTS			
	AREA	R.C.	AxR.C.
PERV.	0.47ha	0.20	0.094
IMPV.	0.03ha	0.90	0.027
TOTAL	**0.50ha**	**0.24**	0.121

CATCHMENT 101 - TO DAVIS DRIVE			
	AREA	R.C.	AxR.C.
PERV.	0.07ha	0.20	0.014
IMPV.	0.07ha	0.90	0.063
TOTAL	**0.14ha**	**0.55**	0.077

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LEGEND

- CATCHMENT BOUNDARY
- 0.14ha — CATCHMENT AREA
- 101 — RUNOFF COEFFICIENT
- OVERLAND FLOW DIRECTION
- PERVIOUS AREA

FIGURE 2
1015 DAVIS DRIVE
EXISTING DRAINAGE PLAN

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Underground storage chambers for infiltration are proposed, this will ensure that the post development water balance and groundwater recharge criteria are satisfied. To determine the initial abstraction from site runoff, the following assumptions have been made:

- For rooftop areas, the initial abstraction is generally 1 to 1.5mm. The depression storage is based on the roughness of the surface area and will increase as the rooftop and terrace surfaces degrade with time. To be conservative, 1mm is assumed for the calculation. This volume of water will generally evaporate.
- For permeable pavement areas, the total volume available in the storage reservoir was used as this volume will be retained on site and infiltrate.
- For landscaped areas, an initial abstraction of 5mm was used. This is generally conservative as based on the curve number, the initial abstraction will usually range from 5mm to 8mm.

A summary of the initial abstraction estimates and resulting retention volumes for the predevelopment site can be seen below in **Table 4**.

Table 4. Predevelopment On-Site Retention - Water Balance

Catchment	Area (m^2)	% of Total Area	IA (mm)	Retention (m^3)
Landscaped Areas	5,388	85%	5.0	26.94
Impervious Areas	962	15%	1.0	0.96
Total	**6,350**	**100%**	**4.39**	**27.90**

The total retention volume provided in the predevelopment scenario is 27.9m^3. A summary of the initial abstraction estimates and resulting retention volumes for the post development site can be seen below in **Table 5**.

Table 5. Post Development On-Site Retention - Water Balance

Catchment	Area (m^2)	% of Total Area	IA (mm)	Retention (m^3)
Landscaped Areas	1,698	27%	5.0	8.49
Impervious Areas	4,652	73%	1.0	4.65
Paver Storage Reservoir	-	-	-	138.68
Total	**6,350**	**100%**	**6.18**	**151.83**

Retention is provided in the 0.350m storage reservoir located below the permeable pavers. The total retention volume provided in the post development scenario is 151.83m^3, which exceeds the predevelopment target of 27.90m^3. Therefore, the water balance and groundwater recharge criteria for the site is satisfied.

4.2 Quantity Control

4.2.1 Peak Flow Controls

The guidelines require that the post development flows not exceed the predevelopment flows up to the 100-year storm event. Additionally, for discharge to municipal infrastructure the captured flow from the post development 100-year event must be controlled to the predevelopment 5-year target. The post development catchment plan is shown on **Figure 4** and parameters are summarized below in **Table 6**.

Table 6. Post Development Catchment Parameters

Catchment	Area (ha)	Runoff Coefficient (RC)- 5 Year	Runoff Coefficient (RC) 100 Year
Controlled Area (200)	0.47	0.72	0.86
North Uncontrolled Area to Rear Lots (201)	0.09	0.37	0.54
South Uncontrolled Area to Davis Drive (202)	0.06	0.61	0.75
East Uncontrolled Area To Hamilton Drive (203)	0.02	0.47	0.63
Total	**0.64**	**0.65**	**0.80**

The overall post development runoff coefficient is 0.65 for 5-year storm. Runoff coefficient calculations can be referenced in **Appendix A**. The runoff coefficients are the same for the 2-year and 5-year storm events.

On-site quantity controls are required to limit the site release rates to meet the target flows. Three quantity control options were reviewed as follows:

- **Parking Lot Surface Storage** – An orifice could be installed near the site outlet to limit flows. Storage will be provided in depressions in the parking lot to attenuate the flow.

- **Underground Pipe and Structure Storage** – Storage can be provided in storm sewer pipes, structures and underground chamber systems to allow for flows to be controlled to meet target release rates.

Due to site grading and the allowable release rates for the 2 through 100-year storm events, providing an underground chamber system for detention is the preferred option.

<u>*Underground Storage*</u>

The underground storage chambers will provide approximately 212m^3 of storage. To be conservative storage provided in the pipes and structures was not taken into consideration. A 75mm diameter orifice plate will be used to attenuate release rates.

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LEGEND

CATCHMENT BOUNDARY

0.01ha / 203 — CATCHMENT AREA / RUNOFF COEFFICIENT

OVERLAND FLOW DIRECTION

PERVIOUS AREA

PROPOSED STORM SEWER

FIGURE 4
1015 DAVIS DRIVE
PROPOSED SWM PLAN

DATE: JUNE 2023 SCALE: 1:500 PROJECT: 17932

A modified rational method calculation was prepared for the Town of Newmarket 2, 5 and 100 year storm events to determine the peak flows and storage volumes for the controlled and uncontrolled catchments. **Tables 7** and **8** below summarize the site release rates in the post development scenario.

Table 7. Post Development Release Rates (Municipal)

Catchment	100 Year Target (L/s) (Surface Drainage)	5 Year Target (L/s) (Connection to Sewer)	100 Year Post Development Release Rate (L/s)	Storage* Required (m^3)	Storage Provided (m^3)
Controlled Area (200)	117.4	30.1	**13.6(Sewer)**	201.3	212
South Uncontrolled Area to Davis Drive (202) and Hamilton Drive (203)			**72.7(Overland)**	-	-
North & East Uncontrolled Area to Rear Lots (201)	259.3	-	**57.2**	-	-

*- Storage is equal to quantity control plus 20m^3/ha as per *Wayne and Waratah Stormwater Study.*

Table 8. Post Development Release Rates (LSRCA)

Catchment	2-Year Target (L/s)	2-Year Post Development Release Rate (L/s)	5-Year Target (L/s)	5-Year Post Development Release Rate (L/s)	100-Year Target (L/s)	100-Year Post Development Release Rate (L/s)
Controlled Area (200) South Uncontrolled Area to Davis Drive (202)	21.7	20.9	30.1	28.1	117.4	86.3
North & East Uncontrolled Area to Rear Lots (201) Hamilton Drive (203)	33.7	9.1	46.7	12.6	259.3	57.2

It should be noted that according to the *"Town of Newmarket's Wayne and Waratah Area Stormwater Management Study"*, an additional storage volume of 20m^3/ha (20m^3/ha x 0.64ha=12.8m^3) has been considered in the storge provided for the site. As show above in **Tables 6** and **7** all post development flows can be controlled to predevelopment levels. The 100-year flow from the site discharging via the site connection to the municipal storm sewer will be controlled to less than the 5 year pre-development flow, and the total flow directed towards the Davis Drive right-of-way in the 100 year storm will be 86.3L/s(13.6L/s via sewer connection + 72.7L/s via overland drainage), which is less than the predevelopment flow of 117.4L/s. Flows to the rear lot will be less than the predevelopment flows and have been minimized to the extent feasible. Existing drainage patterns have been maintained, and there will be no adverse impacts on neighboring properties. Refer to **Figure 3** for post development drainage conditions.

Underground storage chamber specifications are included in **Appendix B**.

4.2.2 *Runoff Reduction*

In addition to the peak flow controls proposed, additional runoff reduction measures are proposed to reduce runoff directed overland to adjacent properties. Infiltration facilities are proposed to be constructed in the rear lot for each dwelling of Block E and two rear lots of Block D. As shown on **Figure 5**, the rooftop and rear yard area directed to the rear of each dwelling ranges from 86.8 to 356.4m^2 as summarized on **Drawing SW4**. Infiltration facilities are sized accordingly to accommodate the first 25mm of rainfall from the catchment area. This would result in approximately 90 percent of average annual rainfall being captured.

Therefore, the proposed infiltration facilities for Block E and two lots of Block D will reduce runoff from the rooftop by 90 percent on an annual basis. A hydrogeological investigation has been completed which determined the estimate infiltration rate of the soils to be 15mm/hour. With a factor of safety of 2.5 applied, this results in a design infiltration rate of 6mm/hour. The critical infiltration system will be 550mm full after a 25mm event. Therefore, in the worst-case scenario the drawdown time of a 25mm event will be 92 hours. With a drawdown time of 72 hours this system will be capable of retaining a 20mm storm event with additional capacity for a longer drawdown time. This is equal to 83 percent of annual rainfall. Given the additional capacity provided and the low likelihood of consecutive events exceeding 25mm, the system can be credited with 90 percent annual runoff reduction.

4.2.3 *Volume Control*

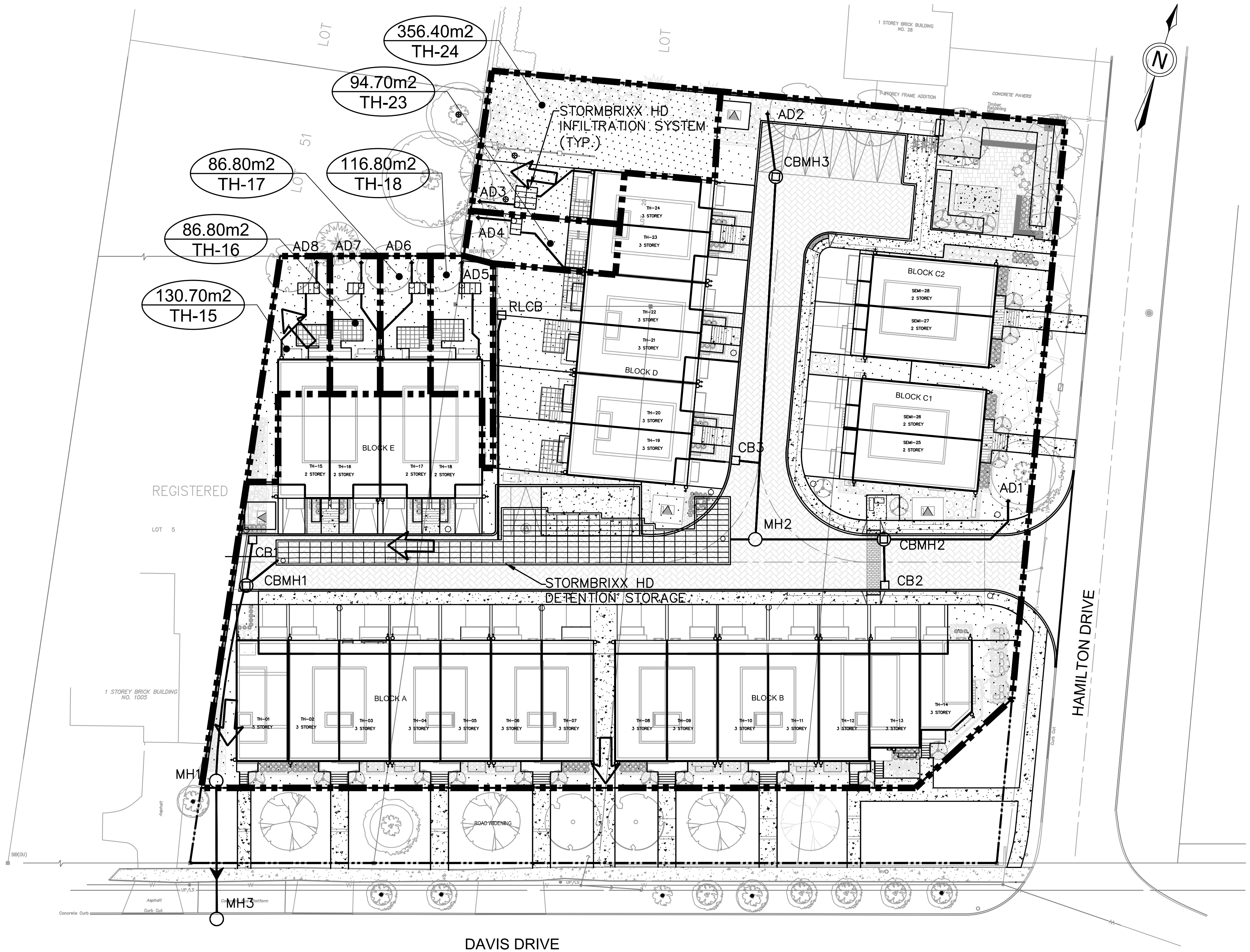
The LSRCA requires the new developments, on sites without restrictions, capture and retain or treat on site, the post development direct runoff volume from 25mm of rainfall from all impervious surfaces. The proposed development will consist of 2,180m^2 of rooftop area, 991m^2 of permeable pavers and an additional 1,481m^2 of impervious surfaces. This results in a total impervious surface area of 4,652m^2. Therefore, it is required to retain the runoff volume for a 25mm rainfall event from these surfaces. The runoff volume from a 25mm event for this area is 105m^3 (0.025m x 0.90 [Runoff coefficient] x 4,652m^2). To the extent feasible, almost all of the impervious surface area is directed toward the permeable pavers. A stone reservoir is provided with a total retention volume of 138.7m^3. Therefore, sufficient storage is provided to meet the volume control requirements.

The design infiltration rate is 6mm/hour and the total storage depth is 350mm. Therefore, the total storage volume will drawdown in 58 hours which satisfied the 72 hour drawdown time requirement.

4.3 Quality Control

4.3.1 *TSS Removal*

Based on the LSRCA's requirements, the water quality criteria for this site is 80 percent average annual TSS removal from runoff originating onsite. The stormwater management design incorporates permeable pavers providing sufficient retention volume within the stone reservoir to allow for quality requirements to be satisfied by infiltration. The sizing of the stone reservoir for infiltration was based on the MECP Guidelines. The post development imperviousness for the total site is approximately 86 percent, therefore a storage volume of 40.33m^3/ha was used in accordance with Table 3.2 of the MECP Guidelines. **Table 9** below summarizes the infiltration volume requirements for quality control.



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CATCHMENT BOUNDARY

0.01m2 — CATCHMENT AREA

203 — CATCHMENT I.D.

OVERLAND FLOW DIRECTION

PERVIOUS AREA

PROPOSED STORM SEWER

FIGURE 5

1015 DAVIS DRIVE

REAR LOT CATCHMENT PLAN

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Table 9. MECP Infiltration Volume Requirements

Site Area (Controlled)	Site Imperviousness	Storage Volume Requirement for 80% TSS Removal*	Infiltration Volume Required	Infiltration Volume Provided
0.47ha	86	40.33m^3/ha	**19.0m^3**	**138.7m^3**

* - Storage volume requirement is taken from Table 3.2 in the MECP SWM Planning & Design Manual and corresponds to an overall site imperviousness of 86%.

The drawdown time requirements, as per the MECP SWM Planning & Design Manual, section 3.3.2 Water Quality Sizing Criteria, are stated as a maximum of 24 hours. The Hydrogeological Assessment estimated the hydraulic conductivity of the soil to be 2.25 x 10^{-8}m/s. Based on Appendix C of the LID SWM Planning and Design Guide, this is equal to an infiltration rate of approximately 15mm/hour. Once a factor of safety of 2.5 is applied in accordance with the LID SWM Guide, a design infiltration rate of 6mm/hour was calculated. The total storage depth of the stone reservoir is 0.35m, however only 0.05m of depth is needed to provide the required infiltration storage for quality. Therefore, this portion will drawdown in approximately 8 hours, which will satisfy the drawdown time requirement of 24 hours.

The infiltration facilities are also required to be located a minimum of 1.0m above the groundwater table. The Hydrogeological Assessment concluded that the groundwater levels on site ranged from 2.10m to 6.02m below grade. Therefore, as the bottom of the stone reservoir will not exceed 1.10m in depth, a minimum of 1.0m clearance between the bottom of the proposed infiltration facility and the groundwater table is provided. This will satisfy the MECP sizing criteria for infiltration SWMP types, and provides the site with 80% TSS removal.

4.3.2 Phosphorus Removal

Based on the LSRCA's requirements, the water quality criteria for this site is for post development phosphorus loading to be reduced to zero if feasible and to not exceed predevelopment loads. The Low Impact Development Treatment Train Tool (LID TTT) was used to calculate the total outgoing phosphorus loads in the post development scenario. Catchments were entered as shown in **Figure 3**. Permeable pavers were entered as impervious surface and were credited with phosphorus removal based on annual infiltration. No additional removal was credited for the permeable pavers other than 100% of the infiltrated volume. No credit was given to detention chambers.

The total annual phosphorus load generated by the site in the post development scenario is 1.315kg per year. After receiving treatment from permeable pavers and landscaped areas, the total annual phosphorus load leaving the site was calculated to be 0.084kg per year, resulting in a reduction of 94 percent.

Based on an offsetting ratio of 2.5:1 and an offsetting cost of $35,770/kg/year the total phosphorus offsetting fee for this development would be $7,512 (2.5 x 0.084kg/year x $35,770/kg/year).

Refer to **Appendix C** for LID TTT outputs.

4.4 Low Impact Development Measures

4.4.1 Underground Storage Chambers

Underground storage chambers are proposed to detain water on site to provide sufficient volume to attenuate peak flows. The chambers will be wrapped in an impermeable liner due to their proximity to the groundwater table. The chamber configuration is long and slender and will allow for additional suspended solids to settle out while water is detained in the system.

The maintenance requirements for the underground chamber system are listed below in **Section 4.5.1**.

4.4.2 Rear Lot Infiltration Facilities

As discussed in Section 4.2.2, infiltration facilities are provided in the rear lots of Block E and two of the rear lots of Block D. These infiltration facilities will be constructed with Stormbrixx HD chambers to minimize the footprint and allow for the systems to be maintained. Runoff from the adjacent rooftop area and an area drain picking up the drainage from the backyard is directed to the facilities. As previously discussed, these facilities will reduce rooftop runoff by 90 percent on an annual basis and reduce the amount of drainage directed overland to adjacent lots.

Maintenance requirements for the chamber systems are outlined in section 4.5.1. The depth of water within the system can be determined by measuring via the inspection port. If a notable depth water is observed after 72 hours without rainfall, this could indicate a blockage or clogging of the system. A qualified professional can be consulted to determine if system maintenance or removal and replacement is required.

4.4.3 Permeable Pavers

Permeable pavers are proposed for the amenity area of the development. These will increase infiltration and provide additional treatment of runoff, improving water quality. Runoff from the amenity area will be directed towards the treatment train proposed.

Permeable Paver maintenance is to be in accordance with manufacture's specifications and recommendations.

The use of bio-retention facilities was reviewed for the site; however, they were determined not to be feasible based on the following:

- The lots are freehold, therefore, it would be difficult to ensure that they are maintained by the homeowners.
- It would take away from the usable area of the amenity space.
- This cannot be provided within the roadway block.

Therefore, low impact development features are proposed to address stormwater quality, quantity and water balance requirements in accordance with the Town's LID requirements.

4.5 Maintenance & Monitoring

4.5.1 Underground Storage Chambers

The chambers and catchbasins should be inspected every 6 months for the first two years and annually after that, once the sediment loading rate is determined. The catchbasins and chambers should be cleaned out when any accumulation of sediment is noted during inspection. Two inspection ports are located by the inlet and the outlet. Chamber cleanout should be completed by use of jetting equipment and vacuuming.

Extra care should be taken during the construction of the storm drainage system to prevent sediment from entering the storage chambers until the site has been stabilized. The connection from the catchbasins to the chambers should be blocked during construction.

4.5.2 Permeable Pavement

To maintain the proper function of the permeable pavement, it is necessary to minimize sediment from entering the joints. Fine sediments will clog the pores and reduce the flow of rainwater between the pavers. A variety of measures can be taken to maintain the performance of the pavers:

1. Clear debris, lawn clippings, etc, weekly or bi-weekly, with regular gardening and lawn maintenance
2. Clean the surface regularly. This is typically completed with a regenerative air or pure vacuum sweeper.
3. Prevent storage of snow, soil, sand, or other materials that could clog the pores.
4. Remove any weed or vegetation growing between the cracks annually or semi-annually.
5. Do not use sand as part of the winter maintenance.

The performance of the permeable pavement can generally be determined by visual inspection immediately following a rainfall event. Surface ponding and significant runoff from the surface are signs of reduced function of the facility and the need for maintenance.

5.0 WASTEWATER

There is an existing 200mm diameter sanitary sewer under the north boulevard on Davis Drive which drains west across the frontage of the site. There is also a 200mm diameter sanitary sewer draining north on Hamilton Drive, which starts near the north limit of the property. It is proposed to provide service connection from the Davis Drive connection between two townhouse blocks to the private road.

Table 10 provides a comparison of the pre and post development peak flows from the site.

Table 10. Sanitary Flow Comparison

Land Use	Equivalent Population[1]	Peaking Factor[2]	Maximum Flow (L/s)[3]
Pre-development (4 single detached)	14	4.0	0.23
Post-development (28 townhouse units)	81	4.0	1.35

[1]Equivalent population as per the Newmarket *Engineering Standards and Design Criteria*.

(3.38 persons/single detached dwelling; 2.88 persons/townhouse unit)

[2]Peaking Factor as per Harmon's formula (Maximum – 4.0; Minimum – 1.5)

[3]Average Wastewater Flow as per Newmarket *Engineering Standards and Design Criteria*

(Average wastewater flow = 360 litres/capita/day)

As shown above there is an increase in population of 67 persons in the post development scenario. The sanitary flows are summarized in **Table 11** below.

Table 11. Sanitary Flow Summary

Scenario	Population	Peaking Factor	Sanitary Flow (L/s)	Infiltration Flow (L/s)*	Total (L/s)
Pre-Development	14	4.0	0.23	0.19	**0.42**
Post Development	81	4.0	1.35	0.19	**1.54**

*Infiltration flows are based off a site area of 0.64ha and a design flow rate of 0.30 L/s/ha.

Therefore, the proposed redevelopment of the site will result in an additional peaked sanitary contribution of 1.12L/s, an increase of 3.7 times compared to the existing scenario. The total increase of 1.12L/s accounts for 3 percent of the receiving pipe capacity.

The sanitary sewer will be extended internally within the private road. The townhomes will be serviced by individual connections to a private sanitary sewer. The sewer design will follow the Town's design criteria.

Refer to Drawing SW2 for the proposed sanitary design.

5.1 Water and Wastewater Master Plan

The Town's Water and Wastewater Master Plan was reviewed to confirm whether there are any impacts to the existing infrastructure as a result of the development.

The proposed development falls outside of the Urban Centres Secondary Plan Area. The Master Plan allows for future growth of 8,129 equivalent population between the years 2015 to 2021 for areas outside of the Urban Centre. The additional population for the development of 67 people (81 -14) would account for less than 1% of the future growth allowance for up to 2021.

The development is part of the Wayne Drive Sub-trunk which connects to the Elgin Street Sub-Trunk before discharging to the Newmarket pumping station. Based on the analysis that was completed surcharge conditions were not predicted in either the Wayne Drive or Elgin Street Sub-Trunks in either the existing or future development scenarios which projected growth up to 2041.

Based on this there is capacity in the existing infrastructure for the proposed development.

6.0 WATER DISTRIBUTION

There is a 150mm diameter watermain on the north side and a 400mm diameter watermain on the south side of Davis Drive and a 150mm diameter watermain on Hamilton Drive across the frontages of the site. There is a hydrant on the north side of Davis Drive near the middle of the site and a hydrant on the east side of Hamilton Drive on the north side of the site.

It is proposed to service the site off of the 150mm diameter watermain on Hamilton Drive. The site watermain will be extended internally on the south side of the private road. The townhomes will be serviced by individual service connections. The watermain design will follow the Town's design criteria.

Refer to Drawing SW2 for the proposed water design.

6.1 Water Design Criteria

Following the Town's design criteria, the watermains are to be designed to meet the greater of the maximum day plus fire flow or the maximum hour demand.

Criteria for Watermains:

Persons per unit (ppu):	Townhouse	2.88
Residential per capita demand (row housing (3–6 units):		300L/cap/day
Peaking Factor (pf):	Peak Hour	3.0
	Maximum Day	2.0
Minimum Pressure (under non-fire demand scenario)		350kPa
Minimum Pressure (under max day + fire demand scenario)		140kPa

Fire Flow:

Minimum residential fire flow for townhouse development	10,000L/min

6.2 Watermain Analysis

Average Daily Demand:

Townhouses = 28(townhouses) x 2.88ppu x 300L/cap/day

=24,192L/day

Peak Hour Demand:

Townhouses = 24,192L/day x 3.0(pf)

= 50.4L/min (13.3gal/min)

Maximum Day Demand:

Townhouses = 24,192L/day x 2.0(pf)

= 33.6L/min (8.9gal/min)

Interpolating on the hydrant flow test graph demonstrates that under peak hour and maximum day flow conditions the pressure in the watermain will be 414kPa (60psi); therefore, the proposed site will meet the Town's minimum pressure of 350kPa (50psi).).

Fire Demand:

Minimum residential fire flow for townhouse development: 10,000L/min

The detailed fire formula on page 17 of the FUS was used to calculate the minimum fire flow. The architect provided the following background information for the development:

- Building GFA, refer to Architectural drawings.
- Type of building construction can be flexible, if necessary, to meet minimum fire demand as required by the City (ie. 'wood frame construction' requires up to 50% more fire flow than 'ordinary construction').
- Sprinklers are not proposed.

Table 12. Fire Flow Estimates

Population Type	Area (m^2)	Construction Coefficient	Occupancy Increase/ Decrease	Sprinkler	Exposure	Required Flow (L/min.)
Block A	1,741	1.0	0%	0%	60%	15,000
Block B	1,672	1.0	0%	0%	50%	14,000
Block C	773	1.0	0%	0%	55%	10,000
Block D	1,526	1.0	0%	0%	55%	13,000
Block E	791	1.0	0%	0%	55%	10,000

When using this information, the minimum required fire flows are between 10,000 to 15,000L/min if the townhouses are constructed using 'ordinary construction' materials, refer to detailed calculations attached in **Appendix D**.

A hydrant flow test was completed on May 7, 2021. According to the hydrant flow test with two ports with a residual pressure of 50 psi and hydrant flow rate of 2014 UGSPM, the theoretical fire flow at the minimum Town pressure of 140kPa (20psi) was determined to be 16,117L/min (4,258gal/min). Based on the required fire flow of 15,000L/min, the proposed development will be protected as required. The hydrant flow test result is provided in **Appendix D**.

6.3 Water Turnover

Watermains shall be designed so that water shall not remain unused in the watermain under the daily average demand.

To determine the water turnover rate, the amount of water in the pipe is calculated and then compared to the average water demand.

Watermain Pipe Diameter: 0.15m

Watermain Pipe Length: 101.5m

Watermain Volume $= \pi \times (0.15)^2/4 \times 101.5$

$= 1,793.6L$

Average Daily Demand = 24,192L/day (from Section 6.2)

Water Turnover Rate = 1,793.6/24,192 x 24

= 1.78hours

Therefore, the frequent water turnover of 1.78hrs shows that water in the watermain is fresh and indicates less potential for taste and odour problems since the water is not stagnant.

7.0 CONCLUSIONS

The proposed development meets the LSRCA and Town of Newmarket criteria as follows:

- Permeable pavers will provide retention storage to enable infiltration to satisfy the quality and water balance requirements.
- Underground storage chambers will be used in conjunction with an orifice plate to control up to the 100-year storms to predevelopment levels and flows to municipal infrastructure will be controlled to meet the allowable release rate for the minor system.
- A gravity sanitary sewer is proposed to the existing sanitary sewer on Davis Drive. The sewer will extend along the private road to service the individual units.
- A water service will be provided to the existing watermain on Hamilton Drive. The watermain will extend internally to each unit.
- The water system has been analyzed and adequate fire and domestic flows can be provided to the site from the municipal main

With the proposed controls in place, the site design will meet the requirements of the LSRCA Guidelines and Town of Newmarket Standards. Therefore, based on the information provided herein, the Site Plan application is appropriate in respect of these matters.

APPENDIX A

STORMWATER MANAGEMENT CALCULATIONS

Rational Method Calc.

Project:	1015 Davis Drive
Project No.:	17932
Municipality:	Town of Newmarket
Catchment:	Pre-Development

	Catchment 100			Catchment 101		
	2 Year	5 Year	100 Year	2 Year	5 Year	100 Year
Runoff Coefficient (C) =	0.24	0.24	0.43	0.55	0.55	0.70
Area (A) =	0.50	0.50	0.50	0.14	0.14	0.14
A:	648.00	930.00	1770.00	648.00	930.00	1770.00
B:	4.00	4.00	4.00	4.00	4.00	4.00
C:	0.78	0.80	0.82	0.78	0.80	0.82
Tc:	6.50	6.50	6.50	6.50	6.50	6.50
Intensity (I) mm/hr =	102.6	142.4	435.0	102.6	142.4	435.0
Peak Flow (Q) L/s =	33.7	46.7	259.3	21.7	30.1	117.4

Composite RC (Catchment 100) - Drainage to Rear Lots (5 Year)

	Area (m2)	C	CxA
Landscape	4687	0.2	937.4
Impervious Area	271	0.9	243.9
	4958	0.24	1181.3
Imperviousmess	5		

Composite RC (Catchment 100) - Drainage to Rear Lots (100 Year)

	Area (m2)	C	CxA
Landscape	4687	0.4	1874.8
Impervious Area	271	1	271.0
	4958	0.43	2145.8
Imperviousmess	5		

Composite RC (Catchment 101) - Drainage to Davis Drive (5 Year)

	Area (m2)	C	CxA
Landscape	701	0.2	140.2
Impervious Area	691	0.90	621.9
	1392	0.55	762.1
Imperviousmess	50		

Composite RC (Catchment 101) - Drainage to Davis Drive (100 Year)

	Area (m2)	C	CxA
Landscape	701	0.4	280.4
Impervious Area	691	1.00	691.0
	1392	0.70	971.4
Imperviousmess	50		

Post-Development Runoff Coefficients (5 Year)

Project:	1015 Davis Drive
Project No.:	17932
Municipality:	Town of Newmarket
Catchment:	Proposed Site

Composite RC - Catchment 201 (5 Year)

	Area (m2)	C	CxA
Landscpae	665.81	0.20	133.16
Building	127.12	0.90	114.41
Impervious	79.93	0.90	71.94
Total	872.86	0.37	319.51

Composite RC - Catchment 202 (5 Year)

	Area (m2)	C	CxA
Landscpae	264.16	0.20	52.83
Building	261.33	0.90	235.20
Impervious	114.34	0.90	102.91
Total	639.83	0.61	390.94

Composite RC - Catchment 203 (5 Year)

	Area (m2)	C	CxA
Landscpae	116.17	0.20	23.23
Building	44.33	0.90	39.90
Impervious	29.40	0.90	26.46
Total	189.90	0.47	89.59

Composite RC - Catchment 200 (5 Year)

	Area (m2)	C	CxA
Landscpae	651.85	0.20	130.37
Building	1747.61	0.90	1572.849
Permeable Paver	990.60	0.50	495.3
Impervious	1257.37	0.90	1131.633
Total	4647.43	0.72	3330.152

Composite RC - Total Site (5 Year)

	Area (m2)	C	CxA
Landscpae	1697.99	0.20	339.60
Building	2180.39	0.90	1962.35
Permeable Paver	990.60	0.50	495.30
Impervious	1481.04	0.90	1332.94
Total	6350	0.65	4130.19
Total Site Imperviousness	58%		

Post-Development Runoff Coefficients (100 Year)

Project:	1015 Davis Drive
Project No.:	17932
Municipality:	Town of Newmarket
Catchment:	Proposed Site

Composite RC - Catchment 201 (100 Year)

	Area (m2)	C	CxA
Landscpae	665.81	0.40	266.32
Building	127.12	1.00	127.12
Impervious	79.93	1.00	79.93
Total	872.86	0.54	473.37

Composite RC - Catchment 202 (100 Year)

	Area (m2)	C	CxA
Landscpae	264.16	0.40	105.66
Building	261.33	1.00	261.33
Impervious	114.34	1.00	114.34
Total	639.83	0.75	481.33

Composite RC - Catchment 203 (100 Year)

	Area (m2)	C	CxA
Landscpae	116.17	0.40	46.47
Building	44.33	1.00	44.33
Impervious	29.40	1.00	29.40
Total	189.90	0.63	120.20

Composite RC - Catchment 200 (100 Year)

	Area (m2)	C	CxA
Landscpae	651.85	0.40	260.74
Building	1747.61	1.00	1747.61
Permeable Paver	990.60	0.75	742.95
Impervious	1257.37	1.00	1257.37
Total	4647.43	0.86	4008.67

Composite RC - Total Site (100 Year)

	Area (m2)	C	CxA
Landscpae	1697.99	0.40	679.20
Building	2180.39	1.00	2180.39
Permeable Paver	990.60	0.75	742.95
Impervious	1481.04	1.00	1481.04
Total	6350	0.80	5083.58
Total Site Imperviousness	58%		

Rational Method Calc.

Project:	1015 Davis Drive
Project No.:	17932
Municipality:	Town of Newmarket

Catchment: 201 - Uncontrolled (To North)

	2 Year	5 Year	100 Year
Runoff Coefficient (C) =	0.37	0.37	0.54
Area (A) =	0.09	0.09	0.09
A:	648.00	930.00	1770.00
B:	4.00	4.00	4.00
C:	0.78	0.80	0.82
Tc:	6.500	6.500	6.500
Intensity (I) mm/hr =	102.6	142.4	435.0
Peak Flow (Q) L/s =	9.1	12.6	57.2

Catchment: 202 - Uncontrolled (To Davis)

	2 Year	5 Year	100 Year
Runoff Coefficient (C) =	0.61	0.61	0.75
Area (A) =	0.06	0.06	0.06
A:	648.00	930.00	1770.00
B:	4.00	4.00	4.00
C:	0.78	0.80	0.82
Tc:	6.500	6.500	6.500
Intensity (I) mm/hr =	102.6	142.4	435.0
Peak Flow (Q) L/s =	11.1	15.5	58.2

Catchment: 203 - Uncontrolled (To Hamilton)

	2 Year	5 Year	100 Year
Runoff Coefficient (C) =	0.47	0.47	0.63
Area (A) =	0.02	0.02	0.02
A:	648.00	930.00	1770.00
B:	4.00	4.00	4.00
C:	0.78	0.80	0.82
Tc:	6.500	6.500	6.500
Intensity (I) mm/hr =	102.6	142.4	435.0
Peak Flow (Q) L/s =	2.6	3.5	14.5

Modified Rational Method

Project:	1015 Davis Drive		
Project No.:	17932		
Municipality:	Town of Newmarket		
Catchment:	Controlled Area		
			100-Year
Area:	0.47 ha	Rainfall $I=A*(T+B)^C$	
Runoff Coefficient:	0.72	A:	648.00
		B:	4.00
Release Rate:	0.0072 m^3/s	C:	-0.78
Storage Required	58.1 m^3		

Initial Time 6.5 min   Increment 5 min

Time (min)	Intensity (mm/hr)	Peak Flow (m^3/s)	Runoff Volume (m^3)	Discharge Volume (m^3)	Storage Volume (m^3)
6.5	102.6	0.096	37.42	2.81	34.6
11.5	75.6	0.071	48.78	4.97	43.8
16.5	60.7	0.057	56.21	7.13	49.1
21.5	51.2	0.048	61.73	9.29	52.4
26.5	44.5	0.042	66.12	11.45	54.7
31.5	39.5	0.037	69.78	13.61	56.2
36.5	35.6	0.033	72.92	15.78	57.1
41.5	32.5	0.030	75.67	17.94	57.7
46.5	29.9	0.028	78.13	20.10	58.0
51.5	27.8	0.026	80.36	22.26	58.1
56.5	26.0	0.024	82.40	24.42	58.0
61.5	24.4	0.023	84.28	26.58	57.7
66.5	23.0	0.022	86.02	28.74	57.3
71.5	21.8	0.020	87.65	30.90	56.8
76.5	20.8	0.019	89.18	33.06	56.1
81.5	19.8	0.019	90.63	35.22	55.4
86.5	18.9	0.018	92.00	37.39	54.6
91.5	18.2	0.017	93.30	39.55	53.8
96.5	17.5	0.016	94.54	41.71	52.8
101.5	16.8	0.016	95.72	43.87	51.9
106.5	16.2	0.015	96.86	46.03	50.8
111.5	15.6	0.015	97.95	48.19	49.8
116.5	15.1	0.014	98.99	50.35	48.6
121.5	14.7	0.014	100.00	52.51	47.5

Orifice Flow Calculation

Pipe Diameter	75	mm
Area	0.004	m^2
Maximum WL	275.42	m
Invert	275.03	m
Head (h)	0.35	m
Co-efficient	0.62	
Flow (Q) = $CA(2gh)^{0.5}$		
	0.0072	m^3/s

Uncontrolled Flow	0.0137	m^3/s
Controlled Flow	0.0072	m^3/s
Total Peak Flow	**0.0209**	**m^3/s**

Modified Rational Method

Project:	1015 Davis Drive		
Project No.:	17932		
Municipality:	Town of Newmarket		
Catchment:	Controlled Area		
			100-Year
Area:	0.47 ha	Rainfall I=A*(T+B)^C	
Runoff Coefficient:	0.72	A:	1021.00
		B:	3.00
Release Rate:	0.0091 m^3/s	C:	-0.79
Storage Required	99.2 m^3		

Initial Time 6.5 min Increment 5 min

Time (min)	Intensity (mm/hr)	Peak Flow (m^3/s)	Runoff Volume (m^3)	Discharge Volume (m^3)	Storage Volume (m^3)
6.5	173.6	0.162	63.34	3.55	59.8
11.5	124.5	0.116	80.34	6.28	74.1
16.5	98.6	0.092	91.30	9.01	82.3
21.5	82.4	0.077	99.40	11.74	87.7
26.5	71.2	0.067	105.86	14.47	91.4
31.5	62.9	0.059	111.24	17.20	94.0
36.5	56.6	0.053	115.88	19.93	95.9
41.5	51.5	0.048	119.95	22.66	97.3
46.5	47.4	0.044	123.60	25.39	98.2
51.5	43.9	0.041	126.91	28.12	98.8
56.5	41.0	0.038	129.94	30.85	99.1
61.5	38.5	0.036	132.73	33.58	99.2
66.5	36.3	0.034	135.33	36.31	99.0
71.5	34.3	0.032	137.77	39.04	98.7
76.5	32.6	0.031	140.05	41.77	98.3
81.5	31.1	0.029	142.21	44.50	97.7
86.5	29.7	0.028	144.26	47.23	97.0
91.5	28.5	0.027	146.21	49.96	96.3
96.5	27.3	0.026	148.07	52.69	95.4
101.5	26.3	0.025	149.85	55.42	94.4
106.5	25.4	0.024	151.55	58.15	93.4
111.5	24.5	0.023	153.19	60.88	92.3
116.5	23.7	0.022	154.76	63.61	91.2
121.5	22.9	0.021	156.28	66.33	89.9

Orifice Flow Calculation

Pipe Diameter	75	mm
Area	0.004	m^2
Maximum WL	275.63	m
Invert	275.03	m
Head (h)	0.56	m
Co-efficient	0.62	
Flow (Q) =CA(2gh)$^{0.5}$		
	0.0091	m^3/s

Uncontrolled Flow	0.0190	m^3/s
Controlled Flow	0.0091	m^3/s
Total Peak Flow	**0.0281**	**m^3/s**

Modified Rational Method

Project:	1015 Davis Drive		
Project No.:	17932		
Municipality:	Town of Newmarket		
Catchment:	200 - Controlled Area		
			100-Year
Area:	0.47 ha	Rainfall $I=A*(T+B)^C$	
Runoff Coefficient:	0.86	A:	1770.00
		B:	4.00
Release Rate:	0.0136 m^3/s	C:	-0.82
Storage Required	188.5 m^3		

Initial Time 6.5 min Increment 5 min

Time (min)	Intensity (mm/hr)	Peak Flow (m^3/s)	Runoff Volume (m^3)	Discharge Volume (m^3)	Storage Volume (m^3)
6.5	257.4	0.290	113.04	5.30	107.7
11.5	187.0	0.211	145.32	9.37	136.0
16.5	148.7	0.167	165.79	13.44	152.3
21.5	124.3	0.140	180.63	17.52	163.1
26.5	107.4	0.121	192.23	21.59	170.6
31.5	94.8	0.107	201.76	25.66	176.1
36.5	85.1	0.096	209.84	29.74	180.1
41.5	77.3	0.087	216.86	33.81	183.1
46.5	71.0	0.080	223.08	37.88	185.2
51.5	65.7	0.074	228.66	41.96	186.7
56.5	61.2	0.069	233.73	46.03	187.7
61.5	57.4	0.065	238.38	50.10	188.3
66.5	54.0	0.061	242.67	54.18	188.5
71.5	51.1	0.057	246.66	58.25	188.4
76.5	48.4	0.055	250.39	62.32	188.1
81.5	46.1	0.052	253.89	66.40	187.5
86.5	44.0	0.050	257.20	70.47	186.7
91.5	42.1	0.047	260.33	74.54	185.8
96.5	40.4	0.045	263.30	78.62	184.7
101.5	38.8	0.044	266.14	82.69	183.4
106.5	37.4	0.042	268.84	86.76	182.1
111.5	36.0	0.041	271.43	90.84	180.6
116.5	34.8	0.039	273.92	94.91	179.0
121.5	33.7	0.038	276.31	98.99	177.3

Orifice Flow Calculation

Pipe Diameter	75	mm
Area	0.004	m^2
Maximum WL	276.32	m
Invert	275.03	m
Head (h)	1.25	m
Co-efficient	0.62	
Flow (Q) = $CA(2gh)^{0.5}$		
	0.0136	m^3/s

Uncontrolled Flow	0.0727	m^3/s
Controlled Flow	0.0136	m^3/s
Total Peak Flow	**0.0863**	**m^3/s**

Rational Method Calc.

			100-Year
Project:	1015 Davis Drive	A:	1770
Project No.:	17932	B:	4
Municipality:	Town of Newmarket	C:	0.820
		Tc:	6.5

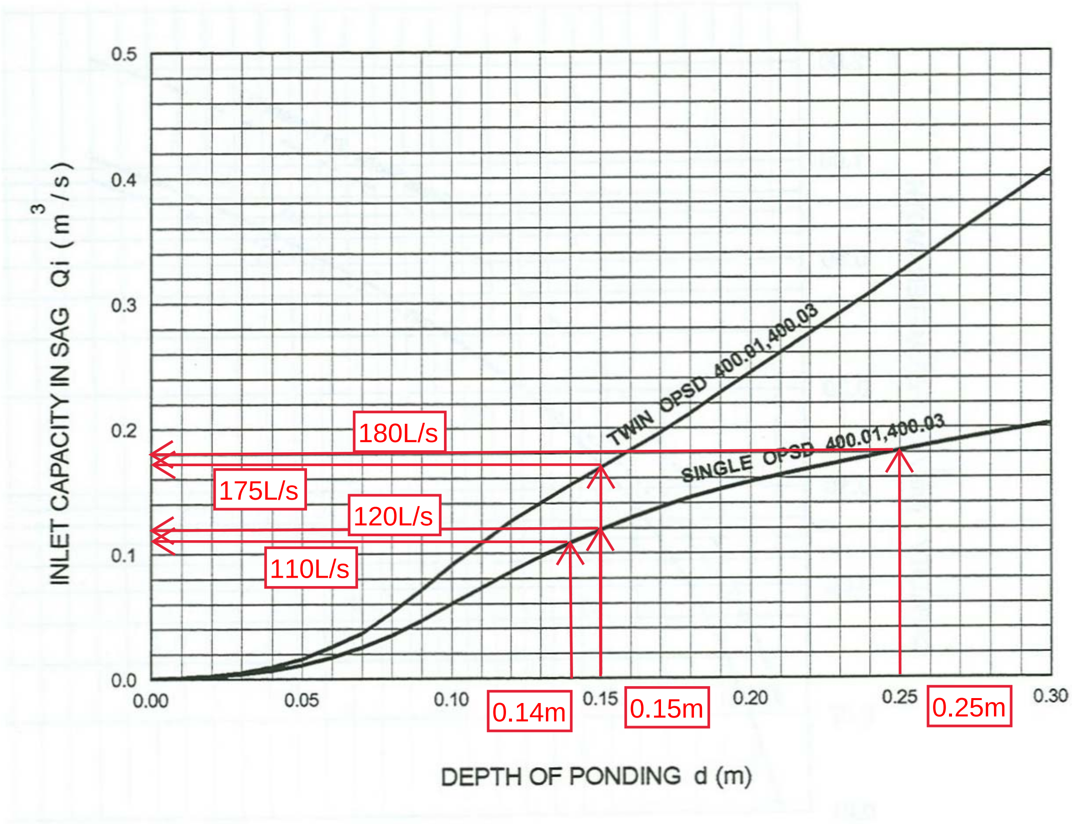
Catchbasin	DRAINAGE AREA (m^2)	COMPOSITE RUNOFF COEFFICIENT (C)	100yr. INTENSITY @ Tc=6.5min (mm/hr)	100yr. FLOW (L/s)
DCBMH1	1330.0	0.94	257.4	89.51
CBMH1	110.0	0.72	257.4	5.68
CBMH2	1000.0	0.88	257.4	62.84
DCB1	910.0	0.83	257.4	54.16
CB1	230.0	0.81	257.4	13.32
CB2	550.0	0.92	257.4	36.22
RLCB	306.8	0.64	257.4	14.07
AD1	76.8	0.67	257.4	3.65
AD2	149.0	0.47	257.4	5.05

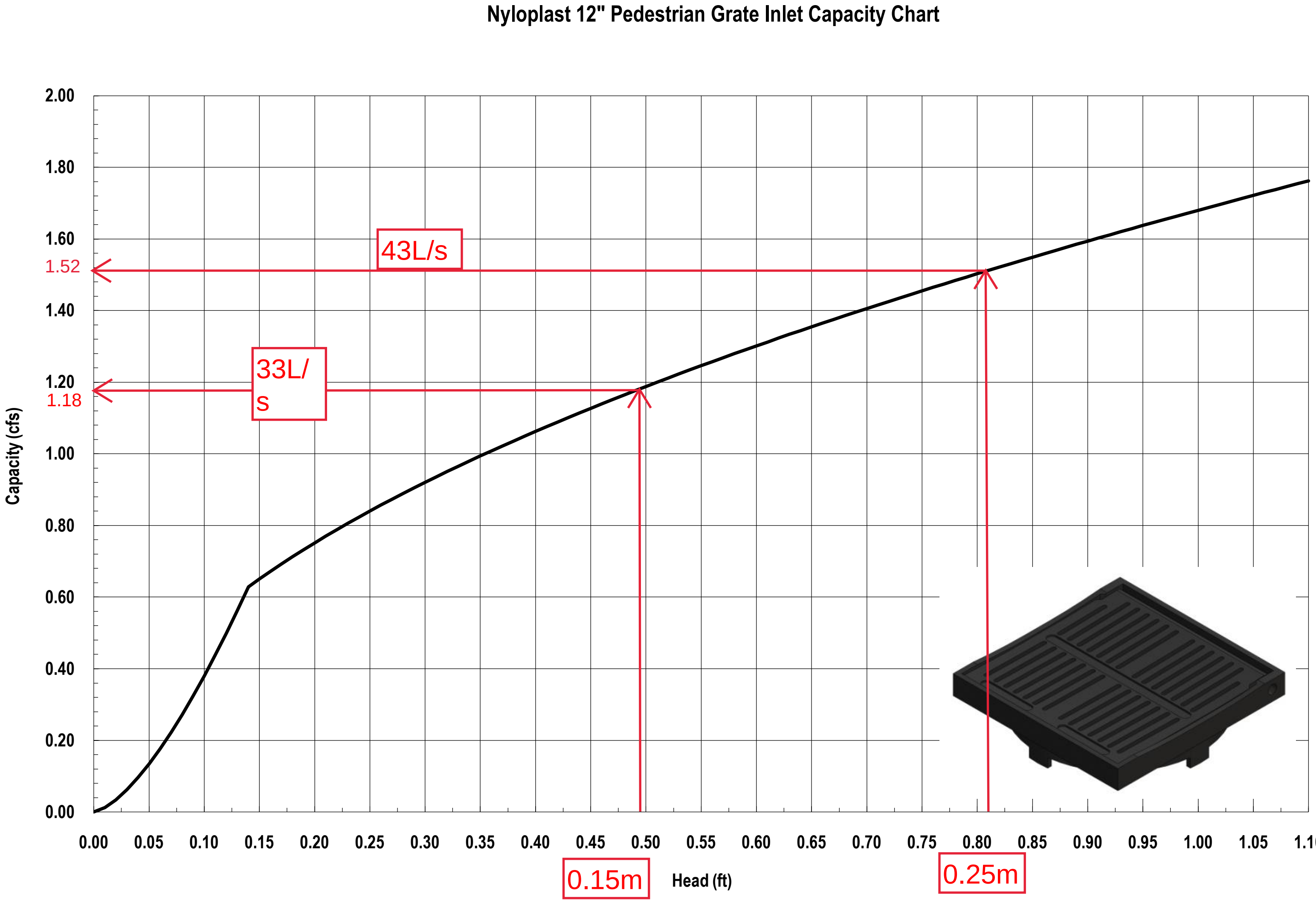
Inlet Capcity

Catchbasin	Maximum Ponding Depth (m) =	Inlet Capcity based on Chart (L/s) =	Actual Capcity assumed 50% Blockage(L/s) =	100yr. FLOW (L/s)
DCBMH1	0.15	175.0	87.50	89.51
CBMH2*	-	44.0	22.0	5.68
CBMH3	0.14	110.0	55.0	62.84
DCB1	0.15	175.0	87.5	54.16
CB2*	-	44.0	22.0	13.32
CB3*	-	50.0	25.0	36.22
RLCB	0.15	120.0	60.0	14.07
AD1	0.25	43.0	21.5	3.65
AD2	0.15	33.0	16.5	5.05

*Catchbasins on a continuous grade- Refer to Design Chart 4-14;"
MTO Drainage Management Manual"

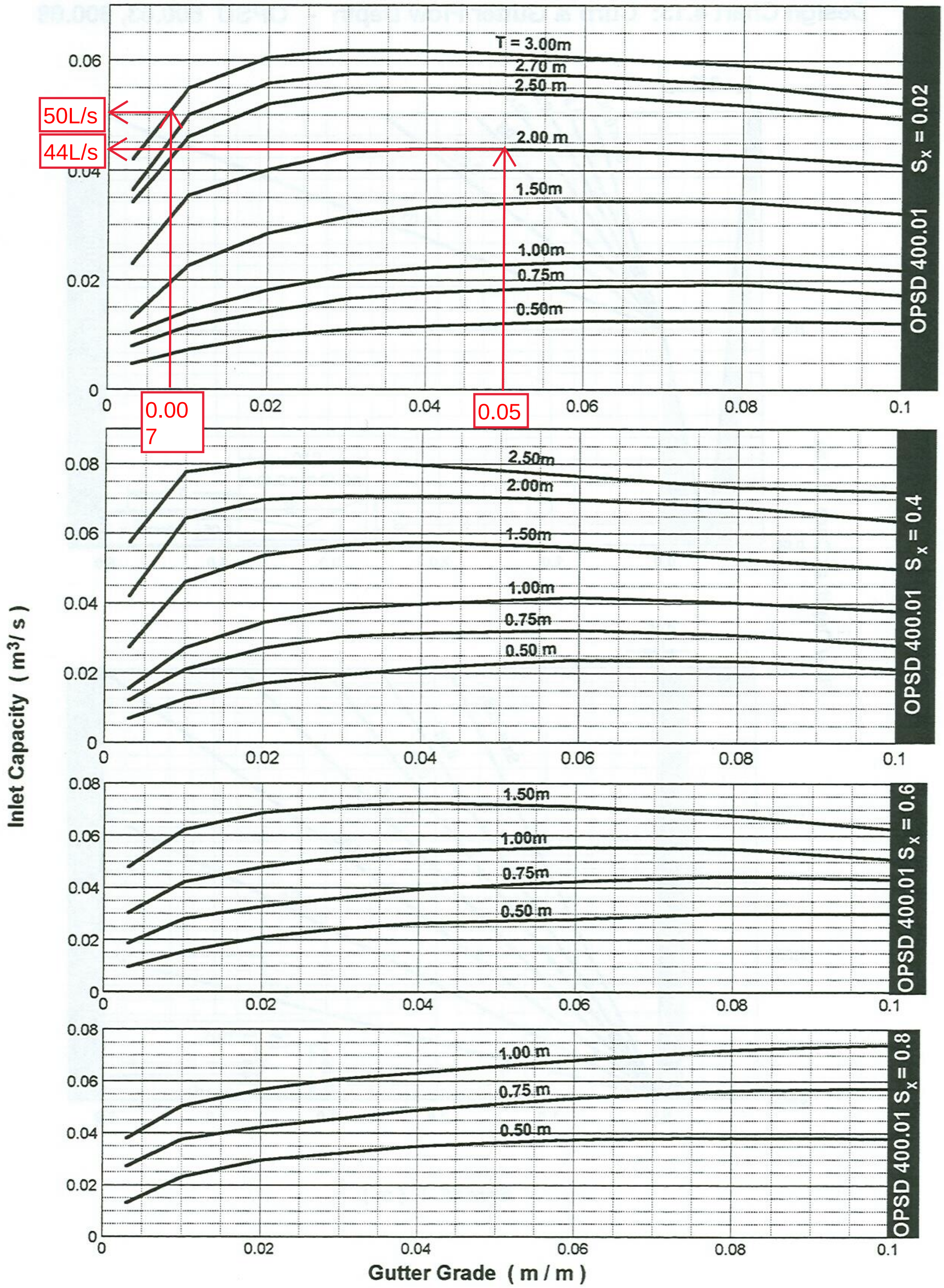
Design Chart 4.19: Inlet Capacity at Road Sag





Nyloplast
3130 Verona Avenue • Buford, GA 30518
(866) 888-8479 / (770) 932-2443 • Fax: (770) 932-2490

Design Chart 4.14: Inlet Capacity OPSD 400.01 (C & G OPSD 600.01)



Storm Design Sheet
Davis Towns

Rainfall Intensity = $\frac{A}{(Tc+B)^c}$

	5-Year	100-Year
A =	930	1770
B =	4	4
c =	0.8	0.82

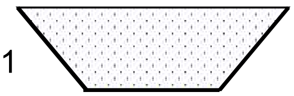
Starting Tc = 6.5 min

Project: Davis Towns
Project #: 17932
Date: Thursday, September 29, 2022
Designed by: MA

STREET	FROM MH	TO MH	No. CBs	CB FLOW (m3/s)	Accum. CB Flow (m3/s)	EXT or BLDG Area (ha)	EXT/BLDG FLOW RATE (l/s/ha)	EXT or BLDG FLOW (m3/s)	ACCUM. EXT/BLDG FLOW (m3/s)	Control Flow	Total Flow (m3/s)	LENGTH (m)	SLOPE (%)	PIPE DIAMETER (mm)	FULL FLOW CAPACITY (m3/s)	FULL FLOW VELOCITY (m/s)	TIME OF CONCENTRATION (min)	ACC. TIME OF CONC. (min)
Site	CB4	CBMH3								5-yr	0.001	8.1	13.00	250	0.214	4.368	0.031	6.531
Site	CBMH3	HAMBER1-I								5-yr	0.028	2.4	5.00	300	0.216	3.059	0.013	6.544
Site	AMBER1-C	MH2								5-yr	0.018	41.4	0.89	300	0.091	1.291	0.535	7.035
Site	AD1	CBMH2								5-yr	0.001	12.8	2.00	250	0.084	1.713	0.125	6.625
Site	CBMH2	MH2								5-yr	0.023	14.5	5.00	300	0.216	3.059	0.079	6.704
Site	MH2	CHAMBER2								5-yr	0.040	5.7	5.00	300	0.216	3.059	0.031	7.066
Site	CB1	CBMH1								5-yr	0.050	5.1	5.00	250	0.133	2.709	0.031	6.531
Site	CBMH1	CHAMBER2								5-yr	0.050	4.5	0.50	300	0.068	0.967	0.078	6.609
Site	CBMH2-D	MH1				1.000	35.000	0.035	0.035	External	0.035	22.1	0.60	250	0.046	0.938	0.393	6.893

Mannings Channel Flow

Project:	1015 Davis Drive
Project No.:	17932
Section Location:	Grassed Swale Critical Section Between Building and Retaining Wall



Input		
Channel Grade =	3.8%	
Side Slopes =	4.6:1	m/m
Bottom Width =	0.53	m
Manning's 'n' =	0.03	
γb	9810	N/m^3

Calculations		
Velocity =	1.68	m/s
Area=	0.198	m^2
Wetted Perimeter =	1.506	m
Hydraulic Radius =	0.131	m

Solve		
Estimated Flow Depth =	0.205	m
Flow =	0.333	cms

Shear Stress on Channel Bed		
b_w/y	2.6	m/m
K_b	1.534784	
τ_b	75.22748	Pa

Shear Stress on Side Slope		
K_{bk}	1.56225	
τ_s	76.57371	Pa

lhs-rhs = 0

K_b,K_{bk} = (tractive force coefficient)

0.095	
0.114	A
0.19	WP

HUSSON

APPENDIX B

UNDERGROUND STORAGE SPECIFICATIONS

ACO StormBrixx®

Geocellular Stormwater Storage Product Maintenance

Prevention
Inspection
Maintenance
Cleaning

ACO

ACO StormBrixx® SD and HD

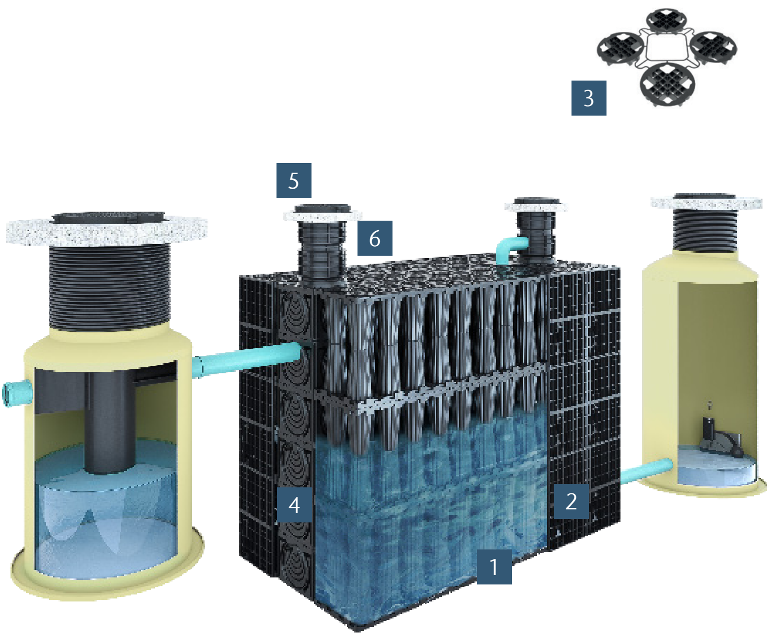
ACO StormBrixx® is a unique and patented geocellular stormwater management system for detention and infiltration usage.

Its versatile design allows the system to be used in configurations and applications across all construction environments as a standalone solution or as part of an integrated LID (Low Impact Development) or BMP (Best Management Practices). Systems may or may not include pre-treatment to remove sediment and/or contaminants prior to entering the storage area. Those without pre-treatment require greater attention to system functionality and may require additional maintenance.

In order to sustain proper system functionality, ACO offers the following general maintenance guidelines for the StormBrixx® product.

Contents

System Components



1. StormBrixx Tank Bodies*
2. Side Panel*
3. Top Cover*
4. Remote Access Unit*
5. Remote Access Cover - Ductile Iron
6. Extension Shaft*

* Image shown represents a StormBrixx® SD system. The Remote Access Unit may be swapped out with the Remote Access Plate. ACO offers vented and non-vented Remote Access Covers.

1 Prevention Measures

1.1 PRIOR TO & DURING CONSTRUCTION

Siltation Prevention of the Stormwater System
Conform to all local, state, and federal regulations for sediment and erosion control during construction.

Install site erosion and sediment BMP's (Best Management Practices) required to prevent siltation of the stormwater system.

Inspect and maintain erosion and sediment BMP's during construction.

1.2 POST CONSTRUCTION

Prior to Commissioning the ACO StormBrixx® System
Remove and properly dispose of construction erosion and sediment BMP's per all local, state, and federal regulations.

Care should be taken during removal of the BMP devices to prevent collected sediment or debris falling into the stormwater system.

Flush the ACO StormBrixx® system to remove any sediment or construction debris immediately after the BMP's removal. Follow the maintenance procedure outlined.

The prevention measures we recommend will increase the efficiency of the installed tank and the life of the entire system.

StormBrixx® is built to be used in areas in which protecting the environment is important. The prevention measures allow for the system as well as the locale it is installed to be sustainable.

StormBrixx® provides top of the line stormwater management solutions for detention, retention, reuse, and infiltration systems. The long term environmental focuses of StormBrixx® through LID, SuDS, MS4, and BMP will benefit the installer, the land owner, and the nearby environment.

Prevention measure

2 Inspections

Follow all local, state, and federal regulations regarding stormwater BMP inspection requirements. The results of the visual inspection, notes and repairs can be recorded in an operating logbook as a recommended best practice. These records will allow decisions to be made about the necessary frequency of future inspection and maintenance measures.

ACO makes the following recommendations:

2.1 VISUAL INSPECTION

Year One
During the first service year a visual inspection should be completed during and after each major rainfall event, in addition to every 6 month period to monitor and establish what sediment and debris buildup occurs.

Each ACO StormBrixx® system is unique to the application and multiple criteria can affect maintenance frequency as such:

- System Design: pre-treatment/no-treatment, inlet protection, stand-alone device.
- Surface area collecting from: hardscape, gravel, soil, or any other surface.
- Adjacent Area: soil runoff, gravel, trash.

2.2 ANNUAL INSPECTION

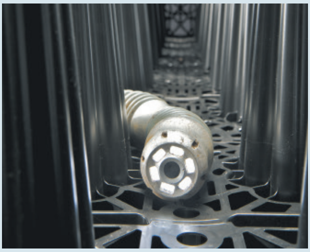
Year Two
Establish an annual inspection frequency based on the information collected during the first year. At a minimum an inspection should be performed at 6 month intervals.

2.3 ITEMS TO INSPECT

Components
- ACO StormBrixx® Remote Access Units/ Plates and inspection ports.
- Inlet and Outlet points.
- Discharge area.

2.4 IDENTIFY AND REPORT

Maintenance required if:
- Sediment and debris accumulation 6” or more.
- System backing up.
- Make operating logbook notes if needed.



Inspection camera

3

Maintenance Procedure

3.1 SURFACE ACCESS

Regulations
Conform to all local, state, and federal regulations.

Access Cover
Locate access cover(s) at the surface connected to the tank.

3.2 SAFETY

Access Cover
Once located, safely open lid and remove.

3.3 SYSTEM INSPECTION

System Debris
Perform an inspection of the tank to locate any debris. This can be done visually, with or without an inspection camera.

3.4 STANDING WATER

Remove Water
If the tank has standing water in it, you will need to vacuum the water first before visually inspecting the tank.

3.5 HIGH PRESSURE

System Clearing
Use the high pressure jet nozzle/wand to loosen and suspend any solid debris that has built up.

Access to high pressure water and vacuum will be needed to clear the tank of any built up debris.

A minimum water pressure of 2,500 PSI is recommended. The maximum pressure depends on the geotextile fabric chosen. Please check with fabric manufacturer for max PSI.

To ensure correct insertion angle of the high pressure jet nozzle, we recommend using a pipe elbow.

Alternatively, a nearby fire hydrant can be used to suspend debris within the StormBrixx® system before vacuuming up the water.

Vacuum removal of debris

Wand used to loosen debris

3 Maintenance Procedure

3.6 WATER LEVEL

Optimal Water Depth
Once the water level has reached 12” or more, shut off and remove high pressure jet nozzle/wand.

3.7 VACUUM HOSE

Remote Access Unit/Plate
Insert vacuum hose via the remote access unit/plate and begin removing all debris that is now suspended in water. Do this until all water has been removed.

3.8 REPEAT

Water and Debris
Not all water and debris may be removed in the first round, you may need to add and remove more water.

3.9 FINAL INSPECTION

Cleared Tank
Once all debris has been removed, inspect tank again to make sure everything has been cleared.

3.10 REMOVE EQUIPMENT

Replace Cover
Once the tank is clear of debris and water, remove all equipment and place the cover back on the tank. Secure cover accordingly.

Final inspection

Camera view of clean tank

For further information on ACO products, please visit the ACO USA website. This allows access to technical data, videos, images, specifications, and installation instructions.

www.acoswm.com/stormbrixx

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Toll free: (800) 543-4764
Fax: (440) 639-7235

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ACO. creating
the future of drainage

APPENDIX C

PHOSPHOROUS LOADING CALCULATIONS

Summary

Site	Project Name	Project Title	Storm Type
Pre-Development			
Post-Development	1015 Davis Drive	1015 Davis Drive	avg-annual

Water Balance | Post-Development

Catchment	Site Area	Site Rainfall In (mm) (m^3)	Site Infiltration (mm) (m^3)	Site Evapotranspiration (mm) (m^3)	External Outflow (mm) (m^3)	Rainfall Reduction (mm) (%)
1	0.63 ha	847.60 mm 5,365.31 m^3	534.39 mm 3,382.69 m^3	241.02 mm 1,525.63 m^3	43.76 mm 277.00 m^3	803.84 mm 94.84 %
TOTAL	**0.63 ha**	**847.60 mm** **5,365.31 m^3**	**534.39 mm** **3,382.69 m^3**	**241.02 mm** **1,525.63 m^3**	**43.76 mm** **277.00 m^3**	**803.84 mm** **94.84 %**

Map | Post-Development

Loading Summary TSS | Post Development

Catchment	Total Catchment TSS Removal	Peak Outflow	Generated Total Flow (m^3) Average Concentration (mg/l) Total Load (kg)	Outgoing Total Flow (m^3) Average Concentration (mg/l) Total Load (kg)
Catchment 1	92.345 %	0.005 m^3/s	3,769.124 m^3 90.221 mg/l 340.053 kg	277.000 m^3 93.978 mg/l 26.032 kg
Total	**92.345 %**	**0.005 m^3/s**	**3,769.124 m^3** **90.221 mg/l** **340.053 kg**	**277.000 m^3** **93.978 mg/l** **26.032 kg**

Loading Summary TP | Post Development

Catchment	Total Catchment TP Removal	Peak Outflow	Generated Total Flow (m^3) Average Concentration (mg/l) Total Load (kg)	Outgoing Total Flow (m^3) Average Concentration (mg/l) Total Load (kg)
Catchment 1	90.550 %	0.005 m^3/s	3,769.124 m^3 0.235 mg/l 0.886 kg	277.000 m^3 0.302 mg/l 0.084 kg
Total	**90.550 %**	**0.005 m^3/s**	**3,769.124 m^3** **0.235 mg/l** **0.886 kg**	**277.000 m^3** **0.302 mg/l** **0.084 kg**

Peak Flow | Post-Development

Catchment	Element	Description	Peak outflow
1	Site	PEAK RUNOFF FLOW from	0.03 m^3/s
	Rear Uncontrolled	PEAK RUNOFF FLOW from	0.00 m^3/s
	Front Uncontrolled	PEAK RUNOFF FLOW from	0.00 m^3/s
	Perm Pavers	PEAK RUNOFF FLOW from	0.00 m^3/s
	Davis Storm Sewer	MAXIMUM FLOW at	0.005 m^3/s
	Front Davis Uncontrolled	PEAK RUNOFF FLOW from	0.00 m^3/s

Loading TSS | Post Development

TSS - Catchment 1

Name	LID Type (removal)	Peak Outflow	Incoming Total Flow (m^3) Concentration (mg/l) Total Load (kg)	Outgoing Total Flow (m^3) Concentration (mg/l) Total Load (kg)
Site	0 %	0.03 m^3/s	3,331.068 m^3 90.000 mg/l 299.796 kg	2,650.000 m^3 90.000 mg/l 238.500 kg
Rear Uncontrolled	0 %	0 m^3/s	728.936 m^3 89.305 mg/l 65.098 kg	170.000 m^3 89.305 mg/l 15.182 kg
Front Uncontrolled	0 %	0 m^3/s	127.140 m^3 98.000 mg/l 12.460 kg	40.000 m^3 98.000 mg/l 3.920 kg
Front Davis Uncontrolled	0 %	0 m^3/s	339.040 m^3 99.000 mg/l 33.565 kg	70.000 m^3 99.000 mg/l 6.930 kg
Perm Pavers	80 %	0 m^3/s	3,489.124 m^3	0.000 m^3

			90.000 mg/l 314.021 kg	18.000 mg/l 0.000 kg
Davis Storm Sewer	0 %	0.005 m^3/s	277.000 m^3 93.978 mg/l 26.032 kg	277.000 m^3 93.978 mg/l 26.032 kg

Loading TP | Post Development

TP - Catchment 1

Name	LID Type	Peak Outflow	Incoming Total Flow (m^3) Concentration (mg/l) Total Load (kg)	Outgoing Total Flow (m^3) Concentration (mg/l) Total Load (kg)
Site	0 %	0.03 m^3/s	3,331.068 m^3 0.230 mg/l 0.766 kg	2,650.000 m^3 0.230 mg/l 0.610 kg
Rear Uncontrolled	0 %	0 m^3/s	728.936 m^3 0.294 mg/l 0.214 kg	170.000 m^3 0.294 mg/l 0.050 kg
Front Uncontrolled	0 %	0 m^3/s	127.140 m^3 0.302 mg/l 0.038 kg	40.000 m^3 0.302 mg/l 0.012 kg
Front Davis Uncontrolled	0 %	0 m^3/s	339.040 m^3 0.311 mg/l 0.105 kg	70.000 m^3 0.311 mg/l 0.022 kg
Perm Pavers	60 %	0 m^3/s	3,489.124 m^3	0.000 m^3

			0.230 mg/l 0.802 kg	0.092 mg/l 0.000 kg
Davis Storm Sewer	0 %	0.005 m^3/s	277.000 m^3 0.302 mg/l 0.084 kg	277.000 m^3 0.302 mg/l 0.084 kg

Detailed Report Parameters | Post Development

Site

Field	Value
Subcatchment name	Site
Catchment	1
Total AREA (HA)	0.393
Impervious area (HA)	0.393
Roof area (HA)	0
Landscaped area (HA)	0
Row Crop area (HA)	0
Open Space / Parkland area (HA)	0
Forest area (HA)	0
Wetland area (HA)	0
Other area (HA)	0
Manning's n for impervious areas	0.01
Manning's n for pervious areas	0.1
Depression storage for impervious areas (mm)	2
Depression storage for pervious areas (mm)	2.54
Weighted Curve Number	0

Rear Uncontrolled

Field	Value
Subcatchment name	Rear Uncontrolled
Catchment	1
Total AREA (HA)	0.086
Impervious area (HA)	0
Roof area (HA)	0.00989
Landscaped area (HA)	0.07611
Row Crop area (HA)	0
Open Space / Parkland area (HA)	0
Forest area (HA)	0
Wetland area (HA)	0
Other area (HA)	0
Manning's n for impervious areas	0.01
Manning's n for pervious areas	0.1
Depression storage for impervious areas (mm)	2
Depression storage for pervious areas (mm)	2.54
Weighted Curve Number	82

Front Uncontrolled

Field	Value
Subcatchment name	Front Uncontrolled
Catchment	1
Total AREA (HA)	0.015
Impervious area (HA)	0.003
Roof area (HA)	0
Landscaped area (HA)	0.012
Row Crop area (HA)	0
Open Space / Parkland area (HA)	0
Forest area (HA)	0
Wetland area (HA)	0
Other area (HA)	0
Manning's n for impervious areas	0.01
Manning's n for pervious areas	0.1
Depression storage for impervious areas (mm)	2
Depression storage for pervious areas (mm)	2.54
Weighted Curve Number	82

Perm Pavers

Field	Value
Name	Perm Pavers
LID type	perm-pavement

Catchment	1
Outlet (name)	5
% Imperv	100
Width (m)	100
Paved surface (HA)	0.099
Roof (HA)	0
Landscaped Area (HA)	0
Row Crop (HA)	0
Open Space/Parkland (HA)	0
Forest (HA)	0
Wetland (HA)	0
(HA)	0
Berm Height (mm)	1
Surface Slope (%)	4
Thickness (mm)	350
Void Ratio	0.4
Impervious Surface Fraction	0.5
Permeability (mm/hr)	1000
Clogging Factor	0
Soil	
Porosity (Fraction)	0.5

Field Capacity (Fraction)	0.3
Wilting Point (Fraction)	0.1
Conductivity (mm/hr)	10
Conductivity Slope (Dimensionless)	45
Suction Head (mm)	200
Seepage Rate (mm/hr)	6
Flow Coefficient	0
Flow Exponent	0
Offset Height (mm)	0
Mannings Roughness	

Davis Storm Sewer

Field	Value
Name	Davis Storm Sewer
Catchment	1
Outfall Elevation (m)	0

Front Davis Uncontrolled

Field	Value
Subcatchment name	Front Davis Uncontrolled
Catchment	1
Total AREA (HA)	0.04

Impervious area (HA)	0.004
Roof area (HA)	0
Landscaped area (HA)	0.036000000000000004
Row Crop area (HA)	0
Open Space / Parkland area (HA)	0
Forest area (HA)	0
Wetland area (HA)	0
Other area (HA)	0
Manning's n for impervious areas	0.01
Manning's n for pervious areas	0.1
Depression storage for impervious areas (mm)	2
Depression storage for pervious areas (mm)	2.54
Weighted Curve Number	82

APPENDIX D

WATER DEMAND CALCULATIONS

Fire Flow Requirements

Project:	1015 Davis Drive
Project No.:	17932
Municipality:	Town of Newmarket

Townhouse Development - Block A

GUIDE FOR DETERMINATION OF REQUIRED FIRE FLOW

(as per the Water Supply for Public Fire Protection 1999 manual by the Fire Underwriters Survey)

STEP 1

Determine the fire flow.

Required Fire Flow (F)	F = 220 x C x sqrt(A)	The required fire flow in litres per minute.
Maximum Floor Area (A) =	1741.00 m2	The total floor area in square metres (including all storeys, but excluding basements at least 50% below grade) in the building being considered.
Coefficient (C) =	1	Coefficient related to the type of construction. = 1.5 for wood frame construction (structure essentially all combustible). = 1.0 for ordinary construction (brick or other masonry walls, combustible floor = 0.8 for non-combustible construction (unprotected metal structural) = 0.6 for fire-resistive construction (fullyprotected frame,floors, roof).
F =	9200 L/min.	

STEP 2

Determine the increase or decrease for occupancy.

	0%	Reduction for Low Hazard Occupancy (Dwellings).
Decrease	0 L/min.	

STEP 3

Determine the decrease, if any, for automatic sprinkler protection.

	0%
Decrease	0 L/min.

STEP 4

Determine the total increase for exposures. 0 -3m (25%), 3-10m (20%), 10-20m (15%), 20-30m (10%), 30-45m (5%)

North - Block E (16m)	15%	
East - Block B (<3m)	25%	
South - Davis Drive (>45m)	0%	
West - Residential (6m)	20%	
	60.0%	Maximum exposure increase is 75%.
Increase	5520 L/min.	

STEP 5

Determine the minimum required fire flow.

F =	**15,000 L/min.**	
Q_R =	**4609 GPM**	Hydrant Flow Test dated May 7, 2021.
	17,447 L/min	

Fire Flow Requirements

Project:	1015 Davis Drive
Project No.:	17932
Municipality:	Town of Newmarket

Townhouse Development - Block B

GUIDE FOR DETERMINATION OF REQUIRED FIRE FLOW

(as per the Water Supply for Public Fire Protection 1999 manual by the Fire Underwriters Survey)

STEP 1

Determine the fire flow.

Required Fire Flow (F)	F = 220 x C x sqrt(A)	The required fire flow in litres per minute.
Maximum Floor Area (A) =	1672.00 m2	The total floor area in square metres (including all storeys, but excluding basements at least 50% below grade) in the building being considered.
Coefficient (C) =	1	Coefficient related to the type of construction. = 1.5 for wood frame construction (structure essentially all combustible). = 1.0 for ordinary construction (brick or other masonry walls, combustible floor = 0.8 for non-combustible construction (unprotected metal structural) = 0.6 for fire-resistive construction (fullyprotected frame,floors, roof).
F =	9000 L/min.	

STEP 2

Determine the increase or decrease for occupancy.

	0%	Reduction for Low Hazard Occupancy (Dwellings).
Decrease	0 L/min.	

STEP 3

Determine the decrease, if any, for automatic sprinkler protection.

	0%
Decrease	0 L/min.

STEP 4

Determine the total increase for exposures. 0 -3m (25%), 3-10m (20%), 10-20m (15%), 20-30m (10%), 30-45m (5%)

North - Block C (17m)	15%	
East - Hamilton Drive (28m)	10%	
South - Davis Drive (>45m)	0%	
West - Block A (<3m)	25%	
	50.0%	Maximum exposure increase is 75%.
Increase	4500 L/min.	

STEP 5

Determine the minimum required fire flow.

F =	**14,000 L/min.**	
Q_R =	**4609 GPM**	Hydrant Flow Test dated May 7, 2021.
	17,447 L/min	

Fire Flow Requirements

Project:	1015 Davis Drive
Project No.:	17932
Municipality:	Town of Newmarket

Townhouse Development - Block C

GUIDE FOR DETERMINATION OF REQUIRED FIRE FLOW

(as per the Water Supply for Public Fire Protection 1999 manual by the Fire Underwriters Survey)

STEP 1			
Determine the fire flow.			
Required Fire Flow (F)	F = 220 x C x sqrt(A)		The required fire flow in litres per minute.
Maximum Floor Area (A) =	773.00	m2	The total floor area in square metres (including all storeys, but excluding basements at least 50% below grade) in the building being considered.
Coefficient (C) =	1		Coefficient related to the type of construction. = 1.5 for wood frame construction (structure essentially all combustible). = 1.0 for ordinary construction (brick or other masonry walls, combustible floor = 0.8 for non-combustible construction (unprotected metal structural) = 0.6 for fire-resistive construction (fullyprotected frame,floors, roof).
F =	6200	L/min.	
STEP 2			
Determine the increase or decrease for occupancy.			
	0%		Reduction for Low Hazard Occupancy (Dwellings).
Decrease	0	L/min.	
STEP 3			
Determine the decrease, if any, for automatic sprinkler protection.			
	0%		
Decrease	0	L/min.	
STEP 4			
Determine the total increase for exposures.			0 -3m (25%), 3-10m (20%), 10-20m (15%), 20-30m (10%), 30-45m (5%)
North - residential (16m)	15%		
East - Hamilton Drive (35m)	5%		
South - Block B(17m)	20%		
West - Block D (16m)	15%		
	55.0%		Maximum exposure increase is 75%.
Increase	3410	L/min.	
STEP 5			
Determine the minimum required fire flow.			
F =	**10,000**	**L/min.**	
Q_R =	**4609**	**GPM**	Hydrant Flow Test dated May 7, 2021.
	17,447	**L/min**	

Fire Flow Requirements

Project:	1015 Davis Drive
Project No.:	17932
Municipality:	Town of Newmarket

Townhouse Development - Block D

GUIDE FOR DETERMINATION OF REQUIRED FIRE FLOW

(as per the Water Supply for Public Fire Protection 1999 manual by the Fire Underwriters Survey)

STEP 1

Determine the fire flow.

Required Fire Flow (F)	F = 220 x C x sqrt(A)		The required fire flow in litres per minute.
Maximum Floor Area (A) =	1526.00	m2	The total floor area in square metres (including all storeys, but excluding basements at least 50% below grade) in the building being considered.
Coefficient (C) =	1		Coefficient related to the type of construction. = 1.5 for wood frame construction (structure essentially all combustible). = 1.0 for ordinary construction (brick or other masonry walls, combustible floor = 0.8 for non-combustible construction (unprotected metal structural) = 0.6 for fire-resistive construction (fullyprotected frame,floors, roof).
F =	8600	L/min.	

STEP 2

Determine the increase or decrease for occupancy.

	0%		Reduction for Low Hazard Occupancy (Dwellings).
Decrease	0	L/min.	

STEP 3

Determine the decrease, if any, for automatic sprinkler protection.

	0%	
Decrease	0	L/min.

STEP 4

Determine the total increase for exposures. 0 -3m (25%), 3-10m (20%), 10-20m (15%), 20-30m (10%), 30-45m (5%)

North - residential (40m)	5%		
East - Block C(16m)	15%		
South -Block B(17m)	15%		
West - Block E (9m)	20%		
	55.0%		Maximum exposure increase is 75%.
Increase	4730	L/min.	

STEP 5

Determine the minimum required fire flow.

F =	**13,000**	**L/min.**	
Q_R =	**4609**	**GPM**	Hydrant Flow Test dated May 7, 2021.
	17,447	**L/min**	

HUSSON

Fire Flow Requirements

Project:	1015 Davis Drive
Project No.:	17932
Municipality:	Town of Newmarket

Townhouse Development - Block E

GUIDE FOR DETERMINATION OF REQUIRED FIRE FLOW

(as per the Water Supply for Public Fire Protection 1999 manual by the Fire Underwriters Survey)

STEP 1

Determine the fire flow.

Required Fire Flow (F)	F = 220 x C x sqrt(A)	The required fire flow in litres per minute.
Maximum Floor Area (A) =	791.00 m2	The total floor area in square metres (including all storeys, but excluding basements at least 50% below grade) in the building being considered.
Coefficient (C) =	1	Coefficient related to the type of construction. = 1.5 for wood frame construction (structure essentially all combustible). = 1.0 for ordinary construction (brick or other masonry walls, combustible floor = 0.8 for non-combustible construction (unprotected metal structural) = 0.6 for fire-resistive construction (fullyprotected frame,floors, roof).
F =	6200 L/min.	

STEP 2

Determine the increase or decrease for occupancy.

	0%	Reduction for Low Hazard Occupancy (Dwellings).
Decrease	0 L/min.	

STEP 3

Determine the decrease, if any, for automatic sprinkler protection.

	0%
Decrease	0 L/min.

STEP 4

Determine the total increase for exposures. 0 -3m (25%), 3-10m (20%), 10-20m (15%), 20-30m (10%), 30-45m (5%)

North - residential (>45m)	0%	
East - Block D (9m)	20%	
South - Block A (16m)	15%	
West - Residential (12m)	20%	
	55.0%	Maximum exposure increase is 75%.
Increase	3410 L/min.	

STEP 5

Determine the minimum required fire flow.

F = 10,000 L/min.

Q_R =	**4609 GPM**	Hydrant Flow Test dated May 7, 2021.
	17,447 L/min	

Hydrant Flow Test Report

SITE NAME:

SITE ADDRESS / MUNICIPALITY: **Hamilton Drive in Newmarket, ON**

TEST HYDRANT LOCATION : **Front of House # 15 Hamilton Street (Hydrant ID # H0460)**

BASE HYDRANT LOCATION: **By House # 49 Hamilton Street (Hydrant ID # H0461)**

TEST BY: Luzia Wood

TEST DATE: May 07,2021

TEST TIME: 10:50AM

TEST DATA

FLOW HYDRANT — Pipe Diam. (in / mm) **150mm** **D.T.**

	PITOT 1	**PITOT 2**
SIZE OPENING (inches):	2.5	2.5
COEFFICIENT (**note 1**):	0.90	0.90
PITOT READING (psi):	46	36 / 36
FLOW (usgpm):	**1138**	**2014**

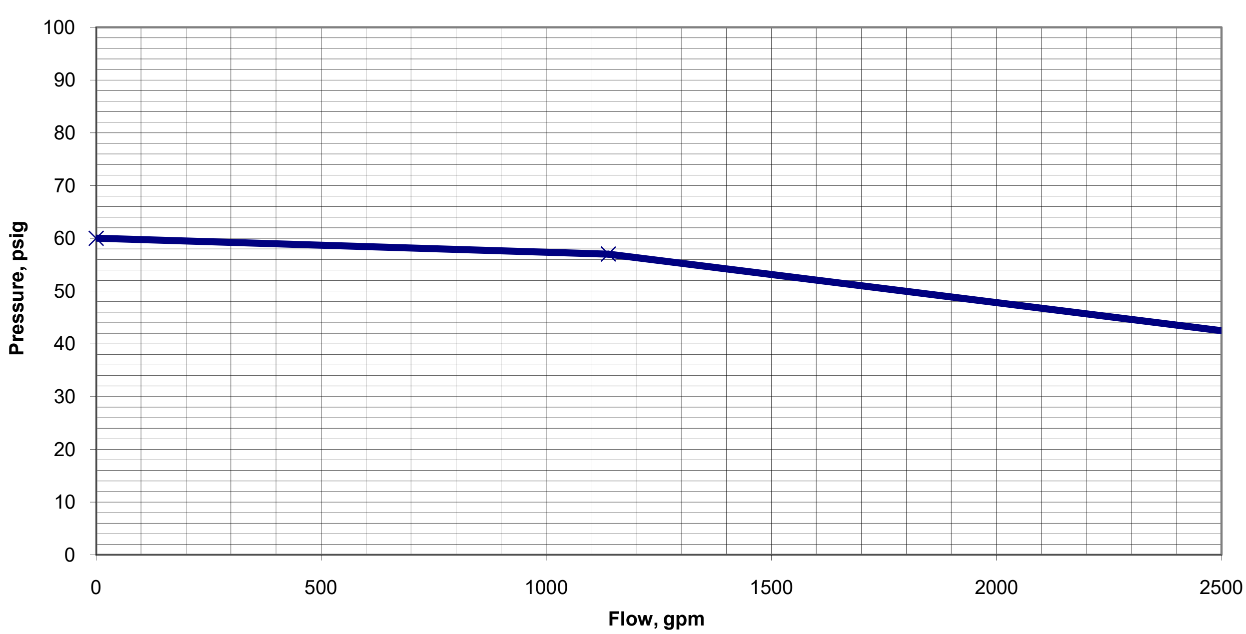
THEORETICAL FLOW @ 20 PSI **4609**

BASE HYDRANT — Pipe Diam. (in / mm) **150mm** **D.T.**

STATIC READING (psi): **60** **57** RESIDUAL 2 (psi): **50**

REMARKS:

NOTE 1 : Conversion factor of .90 used for flow calculation based on rounded and flush internal nozzle configuration. No appreciable difference in pipe invert between flow and base hydrants.



491 Port Maitland Rd