

Innovation Hub & Mixed-Use Development 121 Hume Street

Functional Servicing & Stormwater Management Report

Town of Collingwood, ON

June 28, 2020

Project: 16006



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**Innovation Hub & Mixed-Use Development
121 Hume Street**

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Innovation Hub & Mixed-Use Development

121 Hume Street, Collingwood

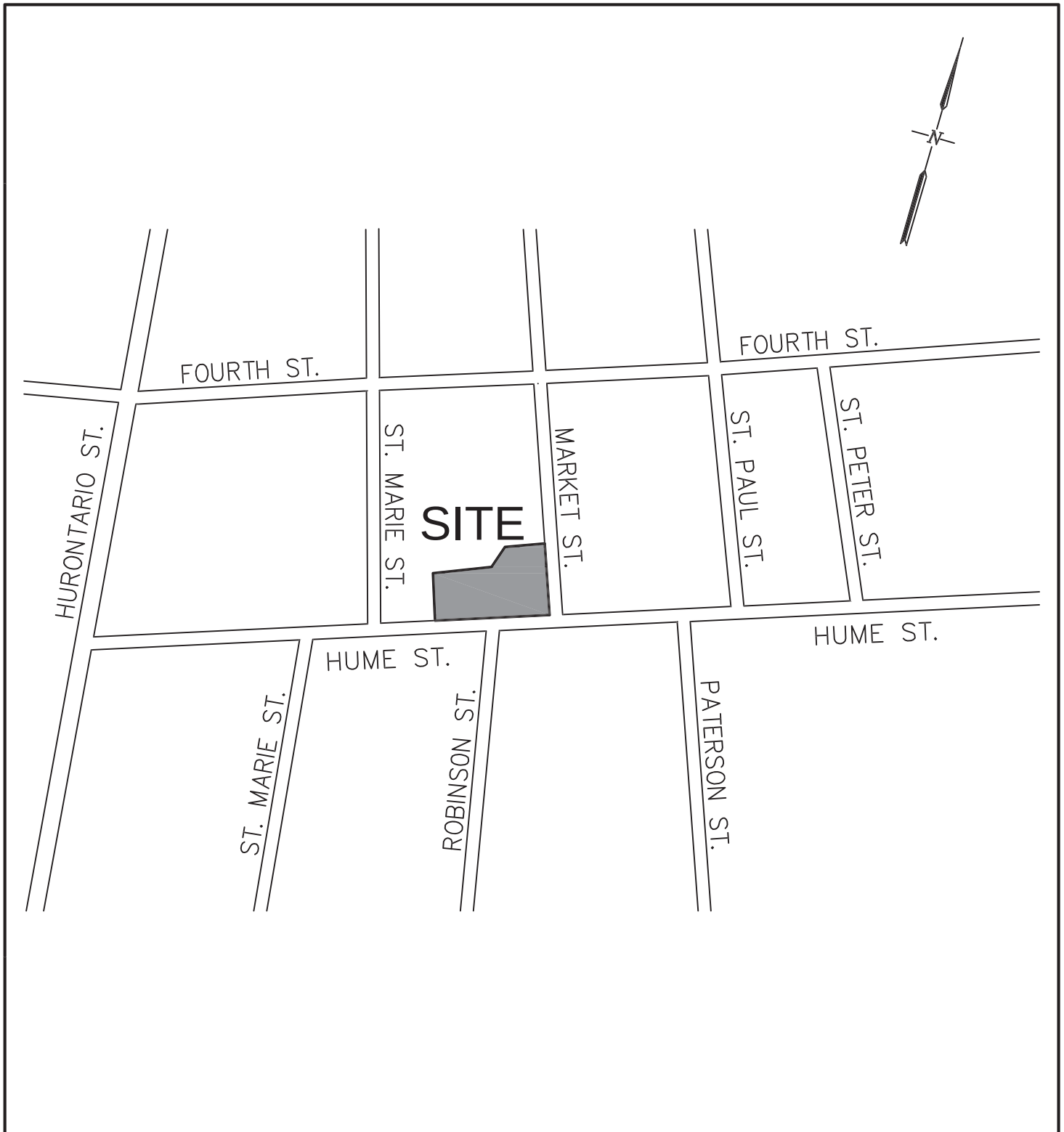
1.0 INTRODUCTION

The following Functional Servicing and Stormwater Management Report has been prepared by Urban Watershed Group Ltd. (UWGL), in support of the Site Plan Approval (SPA) application for a proposed mixed-use building at 121 Hume Street in the Town of Collingwood. The 0.246-hectare property is legally described as Lots 13 and 14 West of Market street and Part of Lot 14 East of St. Marie Street, Registered Plan 144. The site location is more generally identified in **Figure No. 1**.

The site is located at the northwest corner of the intersection of Hume Street and Market Street and is bound to the north and west by residential properties, Market Street to the East and Hume Street to the South. The existing lot has no structures, and largely consists of a gravel surface with a few trees along the perimeter. There are no known easements, natural heritage or drainage features or other encumbrances on the property. It is proposed to construct a 1,099 sq.m four (4) storey mixed-use building consisting of commercial, and office spaces. The building will have one (1) storey of underground parking.

Municipal water, storm, and sanitary services are available on Hume street. Municipal water and sanitary services are also available on Market Street. A proposed storm service is to be installed to the Hume Street municipal storm sewer.

This report aims to demonstrate the sanitary flows and water requirements for the proposed development. As well, this report identifies a stormwater management plan for the property to satisfy the criteria set out for the Town of Collingwood Development Standards (2007). In order to satisfy these criteria, an analysis of the runoff conditions on site during storm events up to a 100-year return period has been completed, and controls for stormwater quantity and quality control are proposed.



Drawing Title

SITE LOCATION PLAN

COMMERCIAL PROPERTY
COLLINGWOOD, ONTARIO

Client

PROPOSED MIXED USE DEVELOPMENT



15955 AIRPORT ROAD, SUITE 304
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Drawn By

B. KLESS

Scale

N.T.S.

Checked By

P. ELLIS

Project No.

16006

Drawing No.

FIG1

2.0 SITE STATISTICS

The existing condition of the property consists of a 2,465.5 sq.m lot on which there is an existing gravel surface, as shown in **Figure No. 2**. The site generally slopes from a central high point towards the property limits in all directions, and across an average change in grade of 0.65 meters at an average slope of approximately 2.5%. The site has been considered as two sub-catchments in the existing conditions to represent the drainage divide of flows to the northern neighbouring residential lots, and to the Hume Street ROW to the south. As the proposed development consists of a major infill type development, the rational method shall be used to determine the allowable release rate for storm drainage. The peak rate of runoff which is received by the Hume street storm sewer for a 5-year storm event under existing conditions shall be considered, and is calculated based on the site statistics and composite runoff coefficient provided below:

Existing Drainage Areas (Figure No. 2)

Land Use	Area (sq.m)	C	Ac
Semi-Pervious (Gravel)	878.0	0.50	439.0
Pervious (Grass)	152.0	0.25	38.0
Drainage Area to Hume Street	1,030.0	0.46	477.0
Semi-Pervious (Gravel)	1,126.6	0.50	563.3
Pervious (Grass)	308.9	0.25	77.2
Drainage Area to North	1,435.5	0.45	640.5
Total Site Area	2,465.5	0.45	1,117.5

The property in its post-development condition will capture and control runoff for discharge to the Hume Street storm sewer at the allowable release rate. For 100-year storm events runoff coefficients have been adjusted by a factor of 1.25 to a maximum value of 1.00, to account for soil saturation. The post-development site statistics are as follows:

Post-Development Drainage Areas (Figure No. 3)

Land Use	Area (sq.m)	C	Ac
Building Roof	1,099.0	0.90	989.1
Impervious (Asphalt and Concrete)	944.5	0.95	897.3
Pervious (Grass)	422.0	0.25	105.5
Total Site Area (to Hume Street)	2,465.5	0.81	1,991.9

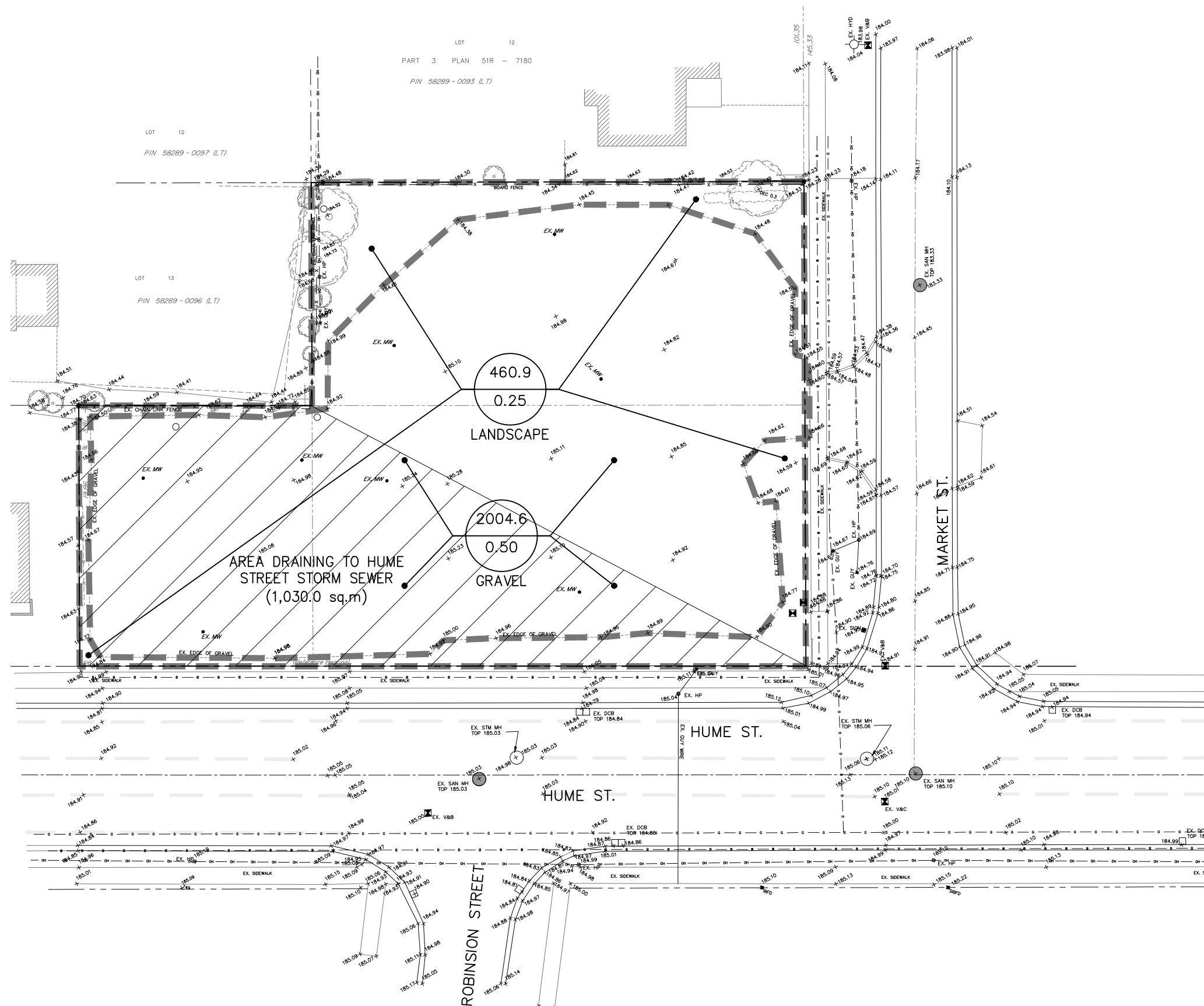


FIGURE No. 2

PROPOSED MIXED USE DEVELOPMENT

121 HUME STREET
COLLINGWOOD, ONTARIO



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PRE DEVELOPMENT DRAINAGE AREA PLAN

SCALE 1:400

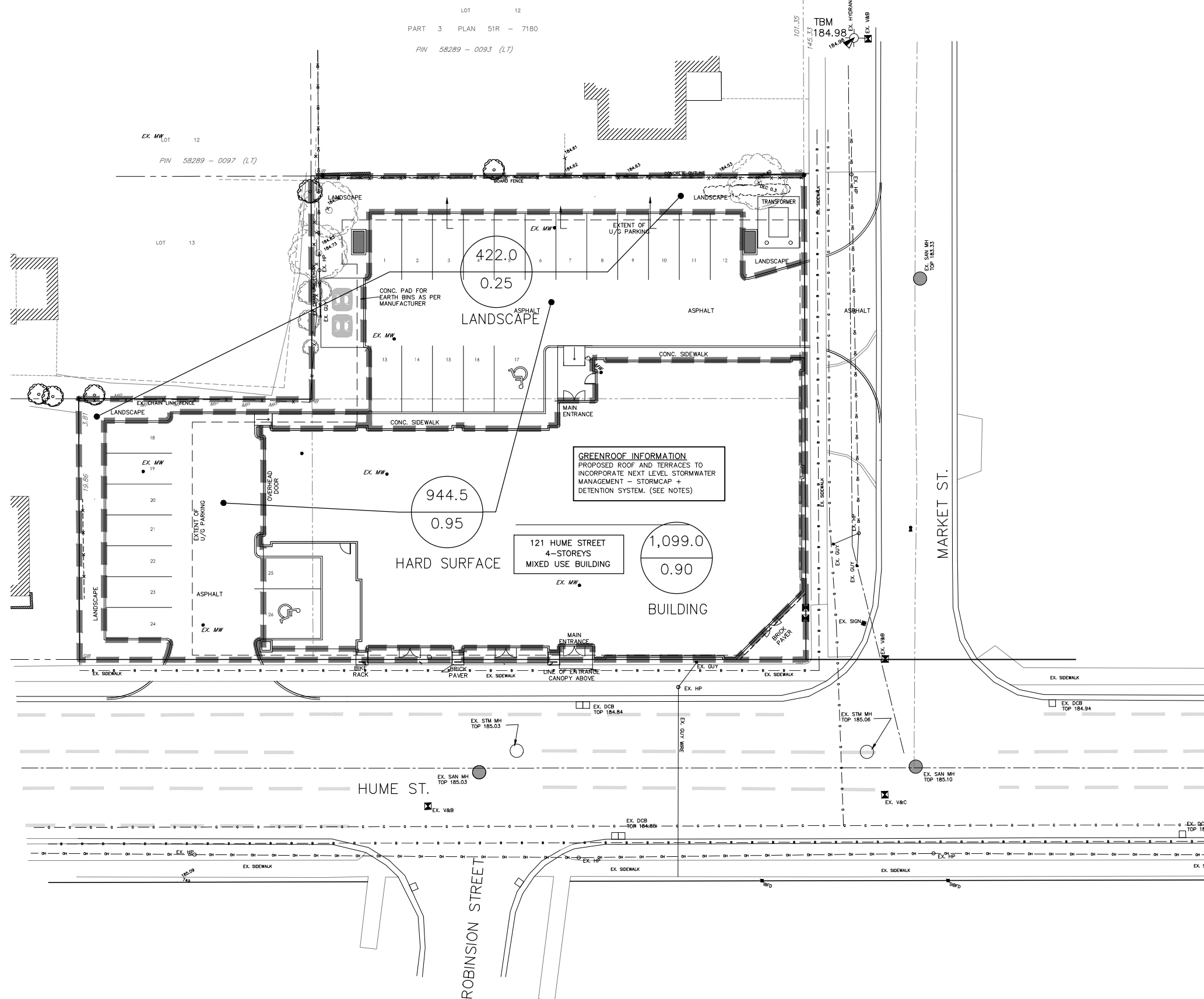


FIGURE No. 3

PROPOSED MIXED USE DEVELOPMENT

121 HUME STREET
COLLINGWOOD, ONTARIO



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POST DEVELOPMENT DRAINAGE AREA PLAN

SCALE 1:400

3.0 STORMWATER MANAGEMENT CRITERIA

The property is located within and unregulated area of the Nottawasaga Valley Conservation Authority (NVCA) Watershed. Runoff from the property is conveyed by sheet flow onto adjacent properties to the north and west and the municipal Right-of-Way (ROW) for Hume Street and Market Street to the south and east. Catchbasins at the intersection of Hume Street and Market Street collect storm runoff into an existing 500mm diameter westward draining storm sewer on Hume Street.

A plan and profile drawing, stormwater catchment area plan and storm sewer design sheet (5-year storm) for Hume Street have been prepared by R.J. Burnside & Associates Ltd., in June of 2017 (included in **Appendix A**), and detail the available capacities within the municipal storm sewer allotted for storm runoff from the subject property. Additional SWM criteria are outlined in a July 4th, 2017 pre-consultation memorandum from Herb Lemon of the Town of Collingwood Engineering Services Department. The proposed development will therefore be subject to the following SWM criteria:

- Control runoff to the existing 2-year storm event release rate from the property as a whole for all storms up to and including the 100-year event (Town of Collingwood);
- Maintain existing 5-year storm event release rate from the property to Hume Street storm sewers for all storms up to and including the 100-year event (Town of Collingwood);
- Provide a “Normal” level (70%) of water quality treatment for runoff (Town of Collingwood); and,
- Capture and infiltrate a volume of stormwater equivalent to a 5mm rainfall on the site (NVCA).

4.0 WATER QUANTITY CONTROL

4.1 Allowable Release Rate

The 2-year existing rate of runoff from the property as a whole has been calculated below to determine the peak allowable flow rate from the property.

2-Year Allowable Release Rate

$$\begin{aligned} I_{2 \text{ YR}} &= A / (T_c + C)^B \\ &= 807.44 / (10 + 6.75)^{0.828} \\ &= 78.28 \text{ mm/hr} \end{aligned}$$

$$\begin{aligned} Q_{\text{EXISTING}} &= CIA / 3600 \\ &= 0.45 \times 78.28 \text{ mm/hr} \times 2,465.5 \text{ m}^2 / 3600 \\ &= 24.12 \text{ L/s} \end{aligned}$$

The 5-year existing rate of runoff to the Hume Street storm sewer has been calculated below for comparison to the allowable 2-year release rate from the property.

5-Year Allowable Release Rate

$$\begin{aligned}
 I_{5 \text{ YR}} &= A / (T_c + C)^B \\
 &= 1135.4 / (10 + 7.5)^{0.841} \\
 &= 102.27 \text{ mm/hr} \\
 Q_{\text{EXISTING}} &= CIA / 3600 \\
 &= 0.46 \times 102.27 \text{ mm/hr} \times 1,030 \text{ m}^2 / 3600 \\
 &= 13.46 \text{ L/s}
 \end{aligned}$$

Therefore, the release of stormwater runoff from the site in the post-development condition shall be controlled to a rate of 13.46 L/s for all storms (up to and including the 100-year event), based on the existing rate of runoff to the Hume Street storm sewer during a 5-year storm event.

4.2 Stormwater Detention Storage

The proposed design will provide SWM storage on the building roof, and the driveway surface. The total stormwater detention volume required was calculated using the modified rational method detailed in **Figures 4 & 5 in Appendix A**. The necessary volumes to detain the runoff rates for the 100-year post-development scenario to the 5-year pre-development level is 77.5 cu.m. This storage volume required comprises the following:

Upper Level Roof Top Storage	24.0 m ³
Lower Level Roof Top Storage	22.1 m ³
Surface Ponding Storage	31.4 m ³
Total Required and Provided Storage	77.5 m³

4.3 Roof Drainage

The design of the proposed buildings incorporates a total roof area of 1,099 sq.m. Rooftop storage will be provided on the 592 sq.m fourth floor (upper level) roof area for stormwater detention storage. A green roof will be implemented on the 507 sq.m third floor (lower level) roof area for stormwater detention and retention storage. Based on a maximum release rate of 42 L/s/ha of roof area, the proposed release rate from the roof shall be no greater than 4.62 L/s. The roof designs will incorporate overflow scuppers in accordance with the Ontario Building Code (OBC).

4.3.1 Control Flo- Roof Drains

Conventional control-flo roof drains shall be implemented on the proposed upper level roof. The proposed release rate from the upper level roof shall be no greater than 2.49 L/s. Two (2) drains are proposed to control the roof drainage with one (1) weir provided for each drain. Each weir notch is expected to release approximately 0.383 L/s per inch of ponding, and at an 78mm (3.08") depth will produce a release rate of 1.18 L/s per notch (total release rate of 2.36 L/s). To produce the required 24.0 cu.m of rooftop storage a roof slope with a 51mm (2") rise is required. The drain down time will be approximately 17.5 hours.

4.3.2 StormCap + Detention Green Roof

A green roof garden will be implemented on approximately 50% of the lower level roof (with the remainder occupied by walkways and patios draining onto the green roof surfaces). The proposed green roof system is divided into two (2) assemblies to maximize the provided retention and detention storage volumes. The green roof garden will be comprised of a 267 sq.m vegetated assembly (for combined retention and detention). The remaining 240 sq.m of walkways and hard surfaces will be underlain with a non-vegetated assembly for additional detention storage. The total provided retention and detention storage volumes for the proposed StormCap system will be 11.17 cu.m and 33.91 cu.m, respectively (which is greater than the required 22.1 cu.m). The proposed green roof garden will have four (4) roof drains controlled to a combined peak release rate of 1.6 L/s by the proposed StormCap system. Product specifications and design calculations for the proposed StormCap + Detention green roof system can be found in **Appendix B**.

4.4 **Surface Storage Volumes**

Additional water quantity storage is to be provided on the surface of the parking lot and vegetative swale, above the area drain tops. There will be two surface storage areas (above AD1, AD2 and CB2), at a ponding elevation of 184.92 m during a 100-year storm. A total storage volume of 31.4 cu.m at a maximum depth of 0.27 m will be provided.

4.5 **Orifice Sizing**

An orifice plate, shall be installed on the downstream side of Control Manhole 3 (CMH3) to control the release of runoff from the stormwater detention volumes. The required orifice size based on the release rate and head necessary to achieve the proposed ponding elevation and surface storage volumes has been calculated as 69.2mm. However, due to maintenance concerns related to clogging of the orifice, MOE guidelines recommend orifice sizes below 75mm not be used. A minimum sized 75mm diameter orifice will provide a controlled release rate as calculated below:

100-year Ponding Elevation (m)	=	184.92
Orifice Spring line (m)	=	<u>183.32</u>
H (head)	=	1.60 m

$$\begin{aligned}\text{Orifice Area} &= \pi/4 \times (0.075\text{m})^2 \\ &= 0.00442 \text{ m}^2\end{aligned}$$

$$\begin{aligned}\text{Orifice Flow} &= cA\sqrt{2g(\Delta h)} \\ &= 0.64 \times 0.00442 \times \sqrt{(2 \times 9.81 \times 1.60)} \\ &= 15.80 \text{ L/s}\end{aligned}$$

The minimum plausible release rate from the property during a 100-year storm event is 15.8 L/s. While this rate exceeds the allowable controlled flowrates calculated using the Town's Criteria (13.46 L/s), based on the small catchment area the site's SWM design is constrained by the minimum recommended orifice size.

5.0 WATER QUALITY CONTROL

To provide a Normal level of water quality protection (corresponding to a 70% TSS removal rate), as set by MECP criteria, a multi-component approach will be utilized. The total impervious surface area (which will be exposed to sediment loading) for the site will be treated by a combination of pervious pavers and CB Shields or the perforated pipe system.

Pervious pavers will be installed in the proposed parking areas, and will promote infiltration of runoff prior to being collected by the proposed catchbasins. In accordance with the Low Impact Development Stormwater Management Planning and Design Guide (2010), pervious pavers with an underdrain system can be considered to reduce the average annual volume of runoff for a catchment by 45%. Furthermore, considering the additional filtration provided by the sand filter layer of the pervious pavers, a sediment removal efficiency of 50% is common for these features.

Each catchbasin on-site will be equipped with a CB Shield to provide an initial average annual sediment removal rate of 53%. Product specifications and design charts for the CB Shield are provided in **Appendix D**.

For the western parking area, the required Normal level (70% TSS removal) of water quality protection will be provide in series, as demonstrated below:

$$\begin{aligned}
 \text{Net TSS Removal Efficiency} &= 1 - [(1 - \text{Pervious Pavers Efficiency}) \times \text{CB Shield Efficiency}] \\
 &= 1 - [(1 - 0.50) \times 0.53] \\
 &= 1 - [0.50 \times 0.53] \\
 &= 1 - 0.27 \\
 &= 0.73 = \mathbf{73\%}
 \end{aligned}$$

Area drains installed along the north perimeter of the development will be connected to a perforated pipe system to promote infiltration and filter sediment from runoff. Studies of the TSS removal efficiency for various perforated pipe systems have demonstrated an average 88% removal efficiency.

Goss traps are to be installed in each catchbasin to capture floatables and oils which may be present within surface runoff.

6.0 WATER BALANCE

NVCA criteria require a stormwater runoff volume from a 5mm rainfall event across the total site area be captured and infiltrated on site. The required rainfall retention volume required is calculated as follows:

$$\text{Rainfall to be Captured for Retention} = 2,465.5 \text{ sq.m} \times 5 \text{ mm} = 12.3 \text{ cu.m}$$

The proposed 422 sq.m landscape surfaces have been considered to provide a 5mm initial abstraction of rainfall runoff. This will result in a 2.1 cu.m (422 sq.m x 0.005m), contribution to the required water balance capture volume.

The proposed 312 sq.m impervious areas will drain onto a permeable pavement surface to promote infiltration. A 25mm runoff abstraction from the permeable pavers has been considered for water balance calculations. This will result in 7.8 cu.m (311 sq.m x 0.025 m) of runoff retention for infiltration during frequent rainfall events.

An additional 2.4 cu.m will be captured within the perforated pipe system. A 632.5 sq.m area from the northern parking lot will drain into the perforated pipe system and provide a 3.8mm abstraction for infiltration retention storage.

Therefore, a total retention volume of 12.3 cu.m (2.1 cu.m + 7.8 cu.m + 2.4 cu.m) is provided. Additional retention storage provided on the lower level green roof garden will further reduce the sites contribution of stormwater runoff to the municipal storm sewer.

The existing site soils were observed to be predominately a granular fill. Based on literature an estimated coefficient of permeability of 1.0×10^{-5} m/sec has been made for these soils based on a clayey gravel mixture classification. Using the correlation of hydraulic conductivity to percolation time presented in Appendix C of the CVC Low Impact Development Stormwater Management Planning and Design Guide, an infiltration rate for the existing soils can be calculated as follows:

$$\begin{aligned}
 \text{Hydraulic Conductivity} &= 6e-11 \times (\text{Infiltration Rate})^{3.7363} \\
 \text{Infiltration Rate} &= (\text{HC} / 6e-11)^{1/3.7363} \\
 &= (10e-5 / 6e-11)^{1/3.7363} \\
 &= 46 \text{ mm/hr} \\
 \text{Design Infiltration Rate} &= \text{Measured Infiltration Rate} / \text{Safety Factor} \\
 &= 46 \text{ mm/hr} / 2.5 \\
 &= 18.4 \text{ mm/hr}
 \end{aligned}$$

6.1 Permeable Pavement Design

A portion of the proposed parking lot shall consist of a surface of permeable paving stones. A storage bed is to be provided beneath the permeable pavers to provide the necessary stormwater retention volumes. The storage bed is to be constructed of a 19mm diameter clear stone with an average porosity of 40%. The proposed pervious pavement structure will be constructed as Eco-Priora pavers (or approved equivalent), placed on 50mm of open graded granular material meeting ASTM No. 8 specifications and underlain by 470 mm of 19mm diameter clear stone. Based on the 116 sq.m parking surface to be provided, a minimum retention volume of 21.8 cu.m will be provided. An under-drain system is to be installed within the storage bed and is to be connected to the storm manholes.

The capacity of the permeable pavement has been verified by calculating the maximum rainfall depth which can be infiltrated within a 2-hour period for the 220mm depth of clear stone bedding installed beneath the subdrain (assuming a 50% clogging factor). Design calculations have been completed using the formula given in the LID Design Guide:

$$\begin{aligned}
 d &= [Q_r(R) + P - I(T)] / V_r && \text{For } Q_r = 0 \text{ and rearranging for } P \text{ (rainfall depth)} \\
 P &= dV_r - iT \\
 &= 0.22\text{m} \times 0.40 - 0.441 \text{ m/day} \times 0.0833 \text{ days} \times 50\% \text{ clogging factor} \\
 &= 0.0256 \text{ m (25.6mm)}
 \end{aligned}$$

Therefore, it is expected that the 220mm depth of clear stone bedding will draw down a 25 mm rainfall depth within 2 hours following a rainfall event.

6.2 Infiltration Facility Design

Area drains will be used to direct clean runoff from the proposed impervious areas into the perforated pipe system. The perforated pipe system is to be constructed with 50mm diameter clear stone with a perforated 200mm diameter PVC sub-drain system and wrapped in filter cloth. 100mm high baffles will be provided at each area drain to promote runoff retention. A 210mm depth of clear stone will be provided beneath the subdrain. Detailed infiltration trench design calculations, following Section 4.5.8 of the MOECC Stormwater Planning and Design Manual (2003), have been completed below:

$$\begin{aligned}\text{Equation 4.2} \quad \text{depth} &= P_t \\ t &= \text{depth} / P \\ &= 0.31 / 18.4 \times 1000 \\ &= 16.8 \text{ hr}\end{aligned}$$

$$\begin{aligned}\text{Equation 4.3} \quad \text{Area} &= 1000V / (Pnt), \\ V &= \text{Area} \times Pnt / 1000 \\ &= 19.5 \times (18.4 \times 0.40 \times 16.8) / 1000 \\ &= 2.4 \text{ cu.m}\end{aligned}$$

Therefore, for the perforated pipe system with an area of 19.5 sq.m (39.0 m long x 0.5 m wide) and 0.31 m depth (below the baffles within the subdrain) will allow a 2.4 cu.m volume of rainwater to infiltrate within 16.8 hours.

7.0 EROSION AND SEDIMENT CONTROL

A temporary silt fence (as per OPSD 219.130) is to be installed along all property lines where there is a risk of sediment runoff onto the adjacent property during construction. This fencing shall be installed prior to site grading and shall be maintained throughout the construction period. Once the storm sewer is installed the catchbasins and area drains shall be protected from sediment. The erosion and sediment controls described above shall be maintained until all exposed soil is stabilized on the site. The Erosion and Sediment Control Plan provides complete details of the above.

8.0 WATER DISTRIBUTION SERVICING

8.1 Domestic Water & Fire Flow Demand Assessment

Domestic water demands have been estimated using the Town of Collingwood Development Standards (2007). For the 4,314 sq.m gross floor area (1,017.9 sq.m + 1,099 sq.m + 1,099 sq.m + 1,099 sq.m), the domestic water demands have been estimated for a water usage of 3,750 L/day/1,000 sq.m GFA (as suggested by the MOE Guidelines for the Design of Water Distribution Systems), a maximum daily demand factor of 2 and a peak hourly demand factor of 4.5 (based on Town of Collingwood Development Standards).

$$\begin{aligned}\text{Daily Water Consumption} &= \text{Gross Floor Area} \times \text{Water Usage} \\ &= 4,314 \text{ sq.m} \times 3,750 \text{ L/day/1,000 sq.m GFA} \\ &= 16,200 \text{ L/day}\end{aligned}$$

$$\begin{aligned}\text{Maximum Daily Demand} &= \text{Daily Water Consumption} \times \text{Maximum Day Factor} \\ &= 16,200 \text{ L/day} \times 2 \\ &= 32,400 \text{ L/day} \\ &= 0.374 \text{ L/s}\end{aligned}$$

$$\begin{aligned}\text{Peak Hourly Demand} &= \text{Daily Water Consumption} \times \text{Peak Hour Factor} \\ &= 16,200 \text{ L/day} \times 4.5 / 24 \text{ hours/day} \\ &= 3,033 \text{ L/hour} \\ &= 0.843 \text{ L/s}\end{aligned}$$

The fire flow requirements for the building have been estimated using the NFPA Fire Underwriters Survey's Water Supply for Public Fire Protection (1999). The proposed 4,314 sq.m gross floor area above grade (1,017.9 sq.m + 1,099 sq.m + 1,099 sq.m + 1,099 sq.m), has been used to calculate the fire flow requirements. Detailed calculations of the fire flow demand are included in **Appendix D** and summarized in the table below:

Water Demand	Mixed-Use	Units
Daily Water Consumption	16,200	L/day
Max Day Demand	0.374	L/s
Fire Flow Demand	236.07	L/s
Total Water Demand (Fire Flow + Max Day)	236.444	L/s

The maximum anticipated water demand from the development will be 236.444 L/s, or 3,748 GPM. The proposed water system will be added to the Town water model to ensure adequate water flow and pressure for all demands including fire.

9.0 SANITARY SERVICING

9.1 Proposed Sanitary Flow Rates

The peak sanitary flow rate from the proposed mixed-use building has been considered the same as the peak hourly water demand calculated in **Section 8.1**. An infiltration allowance of 0.23 L/s/ha has been considered as per Town Standards. The expected sanitary flows from the property are calculated as follows:

$$\text{Peak Hourly Sanitary Flow Rate} = 0.843 \text{ L/s}$$

$$\begin{aligned}\text{Peak Sanitary Flow Rate} &= \text{Peak Hourly Flow Rate} + \text{Infiltration Allowance} \\ &= 0.843 \text{ L/s} + (0.25 \text{ ha} \times 0.23 \text{ L/s/ha}) \\ &= 0.843 \text{ L/s} + 0.058 \text{ L/s} \\ &= 0.901 \text{ L/s}\end{aligned}$$

Therefore, the peak sanitary flow rate expected from the property is 0.901 L/s.

10.0 CLOSURE

The proposed Stormwater Management Plan demonstrates that the development will meet the established criteria with respect to stormwater management set forth in governing documents. The internal storm sewer systems will control the release rate to the existing levels based on the observed site conditions. On-site stormwater detention storage has been provided up to a quantity of 77.5 cu.m in order to facilitate the proposed SWM controls. A StormCap green roof system will provide on site stormwater runoff retention to reduce the total volume of runoff received by the municipal storm sewer. Water quality control is to be provided by the proposed pervious pavers, CB Shields, perforated pipe system and goss traps. A rainfall depth equivalent to a 5.9 mm event from the property will be capture by the proposed permeable pavers and perforated pipe system to meet the water balance criteria.

Proposed maximum anticipated water demand and peak sanitary demand have been calculated as 3748 GPM and 0.901 L/s, respectively. These values will be used in the Town's water supply model to determine whether adequate service capacity exists within the municipal infrastructure.

In conclusion, the proposed stormwater management and servicing plan supports the concept of an environmentally sustainable development. The proposed plan will mitigate anticipated stormwater impacts associated with the construction of the proposed development and aims to mimic the pre-development hydrological characteristics of the site. Additionally, the proposed water and sanitary demand requirements for the development have been calculated in conformance with Town standards to asses the plausibility of providing new service connections to the property.

Appendix A

Figures No. 4 & 5 - Modified Rational Method Calculations

STORMWATER MANAGEMENT ANALYSIS - FIGURE No. 4

PROJECT NUMBER:	16006
CLIENT:	GREENLAND CONSULTING
LOCATION:	121 HUME ST
IDF STATION:	OWEN SOUND

SITE INFORMATION

Number of Catchment Areas: 1

This Analysis is for Subcatchment Area No's. 1

	AREA (m2)	RUNOFF COEFFICIENT*
Upper Level Roof	592.0	1.00
Lower Level Roof	507.0	1.00
Impervious (Pavement)	944.5	1.00
Pervious Landscaped Area	422.0	0.31
Overall Area	2,465.5	0.88
Paved + Landscaped	1,366.5	0.79

*runoff coefficients have been adjusted by a factor of 1.25 to a maximum value of 1.00

Allowable Release Rate from Site (L/s):

Total Allowable Release Rate=	13.46
Release Rate from Upper Roof=	2.36
Release Rate from Lower Roof=	1.60
Allowable Release from Other Areas=	9.50

DETERMINATION OF NECESSARY STORAGE VOLUME - FIGURE No. 5

This Analysis is for Catchment Area No. 1

Modified Rational Method used to determine storage volume:

$$Q = CiA/360, \quad i = A / (tc + C)^B \text{ for: } A = 2193.1, \quad B = 0.871, \quad C = 9.04$$

TIME (MIN.)	i (mm/hr)	INFLOW (m3/sec) (Q=CiA/360)			ACCUMULATED RUNOFF VOLUME (m3) (Vacc = Q x tc)			ALLOWABLE RELEASE VOL. (m3) (Vrel = Q100yr x tc)			REQ'D STORAGE (m3) (Vreq'd = Vacc - Vrel)			
		UPPER ROOF	LOWER ROOF	SURFACE	UPPER ROOF	LOWER ROOF	SURFACE	UPPER ROOF	LOWER ROOF	SURFACE	UPPER ROOF	LOWER ROOF	SURFACE	TOTAL
1	294.1	0.048	0.041	0.088	2.9	2.5	5.3	0.1	0.1	0.6	2.8	2.4	4.7	9.9
2	270.8	0.045	0.038	0.081	5.3	4.6	9.7	0.3	0.2	1.1	5.1	4.4	8.6	18.0
3	251.1	0.041	0.035	0.075	7.4	6.4	13.5	0.4	0.3	1.7	7.0	6.1	11.8	24.9
4	234.2	0.039	0.033	0.070	9.2	7.9	16.8	0.6	0.4	2.3	8.7	7.5	14.5	30.7
5	219.6	0.036	0.031	0.066	10.8	9.3	19.7	0.7	0.5	2.8	10.1	8.8	16.9	35.8
6	206.9	0.034	0.029	0.062	12.2	10.5	22.3	0.9	0.6	3.4	11.4	9.9	18.8	40.2
7	195.6	0.032	0.028	0.058	13.5	11.6	24.6	1.0	0.7	4.0	12.5	10.9	20.6	44.0
8	185.5	0.031	0.026	0.055	14.6	12.5	26.6	1.1	0.8	4.6	13.5	11.8	22.1	47.4
9	176.6	0.029	0.025	0.053	15.7	13.4	28.5	1.3	0.9	5.1	14.4	12.6	23.4	50.3
10	168.4	0.028	0.024	0.050	16.6	14.2	30.2	1.4	1.0	5.7	15.2	13.3	24.5	53.0
15	137.5	0.023	0.019	0.041	20.3	17.4	37.0	2.1	1.4	8.5	18.2	16.0	28.4	62.7
20	116.6	0.019	0.016	0.035	23.0	19.7	41.8	2.8	1.9	11.4	20.2	17.8	30.4	68.4
25	101.6	0.017	0.014	0.030	25.0	21.5	45.5	3.5	2.4	14.2	21.5	19.1	31.3	71.9
30	90.1	0.015	0.013	0.027	26.7	22.8	48.5	4.3	2.9	17.1	22.4	20.0	31.4	73.8
35	81.1	0.013	0.011	0.024	28.0	24.0	51.0	5.0	3.4	19.9	23.1	20.6	31.0	74.7
40	73.9	0.012	0.010	0.022	29.2	25.0	53.0	5.7	3.8	22.8	23.5	21.1	30.2	74.9
50	62.9	0.010	0.009	0.019	31.0	26.6	56.4	7.1	4.8	28.5	23.9	21.8	27.9	73.6
60	54.9	0.009	0.008	0.016	32.5	27.8	59.0	8.5	5.8	34.2	24.0	22.1	24.8	70.9
70	48.8	0.008	0.007	0.015	33.7	28.8	61.2	9.9	6.7	39.9	23.8	22.1	21.3	67.2
80	44.0	0.007	0.006	0.013	34.7	29.7	63.1	11.3	7.7	45.6	23.4	22.0	17.5	62.9
90	40.1	0.007	0.006	0.012	35.6	30.5	64.7	12.8	8.6	51.3	22.8	21.8	13.4	58.0
100	36.8	0.006	0.005	0.011	36.3	31.1	66.1	14.2	9.6	57.0	22.2	21.5	9.1	52.8

Appendix B

StormCap + Detention Product Specifications



MEMORANDUM

Date: 12 December 2019
Subject: 121 Hume Street – StormCap™+Detention results
To: Sasha Aguilera, Next Level Stormwater Management
From: Brad Garner, Researcher/Software Engineer

Green Roof Diagnostics has reviewed the proposed StormCap™+Detention green roof assembly for stormwater performance at the aforementioned project. Areas are as follows:

Area (m²)	Profile	Retention Storage (m³)	Detention Storage (m³)
267 vegetated assembly	50mm GM, 50mm MW, 50mm RC	11.17	33.91
240 non-vegetated assembly	Hard surfaces, 50 mm RC		

**The outflow from the downspout of a 15 m² stairwell roof into the 4th floor green roof has been incorporated in the calculations. It is critical to ensure the downspout is not located very close to any 4th floor drain and that it's outflow is well distributed upon entering the green roof.*

StormCap™+Detention with Reservoir Cell (2 different assemblies on certain areas of the roof):

Complete vegetated assembly:

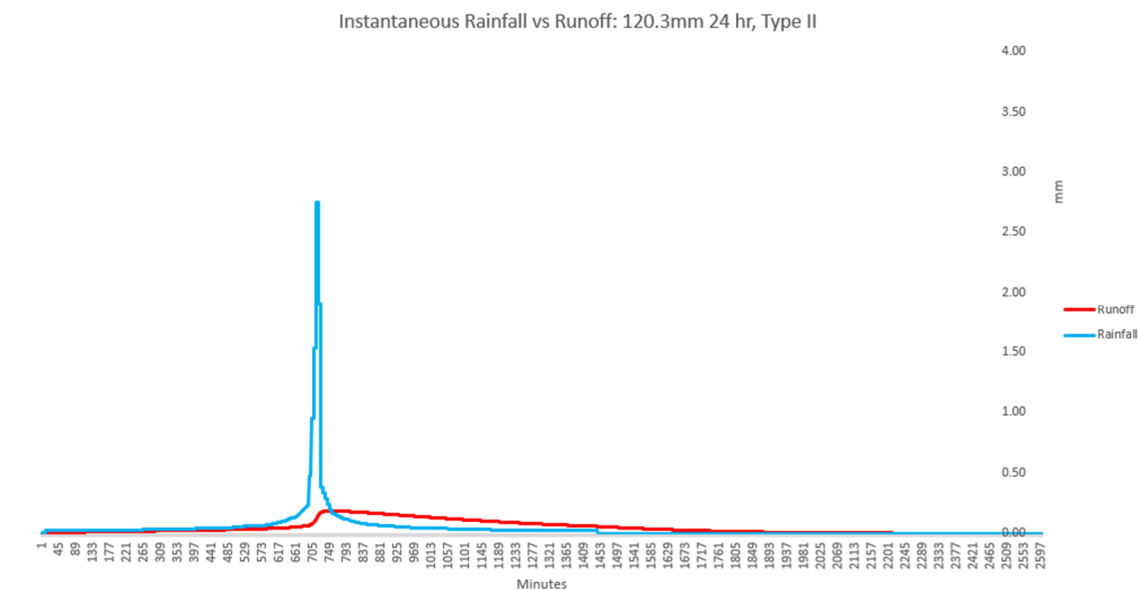
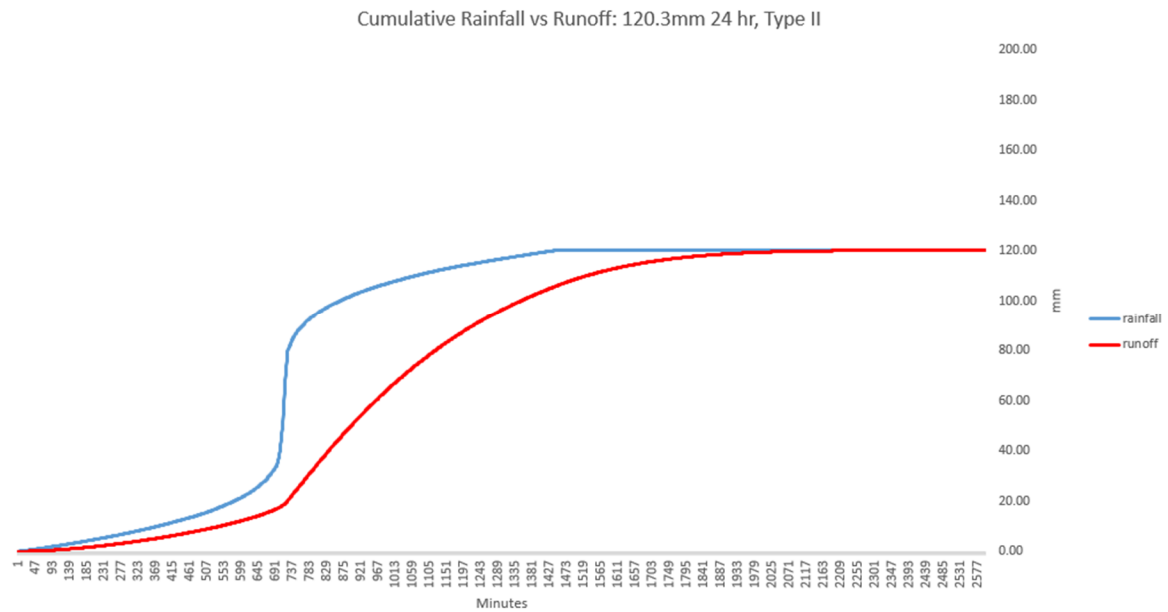
- 25 mm Pre-grown Sedum Blanket
- 50 mm Growing Media
- 50 mm Needled Mineral Hydro Blanket
- 50 mm Reservoir Cell
- 5 mm Detention Mat
- 0.5 mm Root Barrier

Full non-vegetated assembly:

- Walkway/deck/planter/seating area
- 50 mm Reservoir Cell
- 5 mm Detention Mat
- 0.5 mm Root Barrier
-

More Information:

- Maximum modeled outflow rate of the 501 m² roof = 30.72 L/s/ha
- Dead load (dry): 54 kg/m²
- Dead load (at maximum retention value): 125 kg/m²
- Detention live load (only) at maximum capacity: 59 kg/m²
- Total weight at maximum capacity (live load + dead load): 184 kg/m²
- Peak Flow Delay time (centroid delay): 4.1 hours
- Peak Flow Reduction: 93%
- Recharge timeframe: 19.7 hours



TRANSMISSIVITY TEST RESULTS

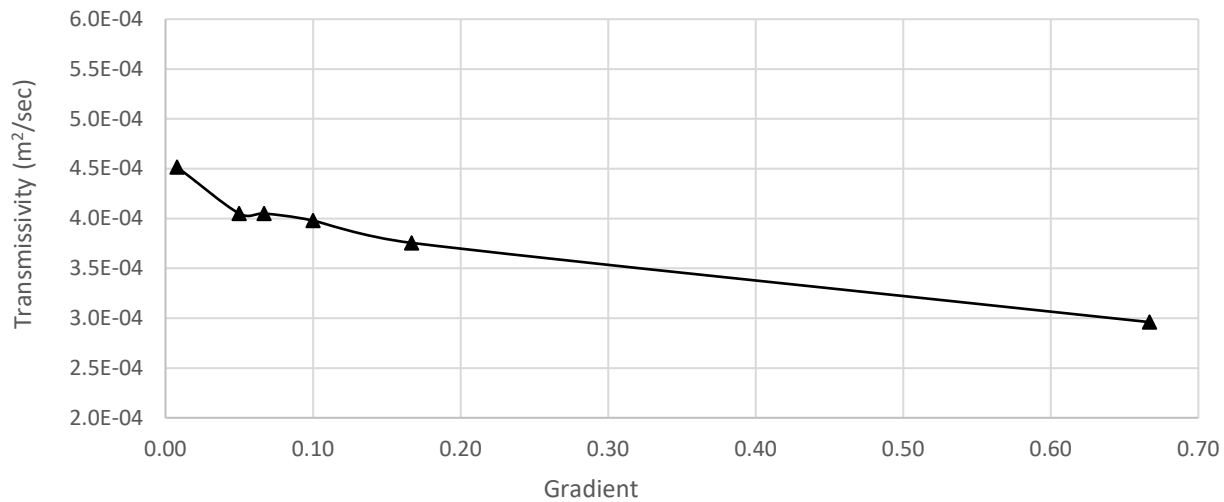
ASTM D4716

PROJECT NAME: GREEN ROOF/2019 LAB TESTING/VA
 SAMPLE NUMBER: 5mmc HC50 MW

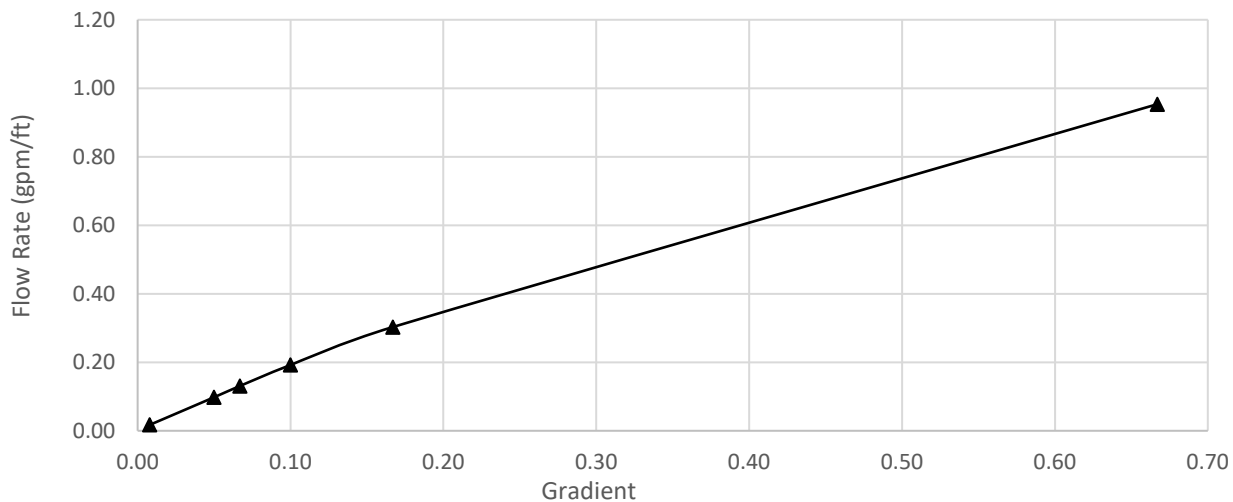
Configuration: Plate / 5mmc (perpendicular to stripes) / 50 mm Honeycomb / Mineral Wool / Plate
 Seating Time: 15 minutes
 Pressure: 50 psf

Gradient	Transmissivity (m ² /sec)				Flow Rate (gpm/ft)		
	Specimen 1	Specimen 2	Average		Specimen 1	Specimen 2	Average
0.008	4.58E-04	4.45E-04	4.52E-04		0.018	0.017	0.017
0.050	4.39E-04	3.71E-04	4.05E-04		0.106	0.090	0.098
0.067	4.37E-04	3.73E-04	4.05E-04		0.141	0.121	0.131
0.100	4.23E-04	3.73E-04	3.98E-04		0.204	0.180	0.192
0.167	4.01E-04	3.50E-04	3.76E-04		0.323	0.283	0.303
0.667	3.15E-04	2.77E-04	2.96E-04		1.015	0.892	0.954

Transmissivity vs Gradient



Flow Rate vs Gradient



StormCap[™]+Detention

Maximum Water-Retaining and Water-Detaining
Pre-Vegetated System

Best ROI

All your stormwater
managed on the roof!



No more buried cost

Reduce or eliminate
the cistern

Sedum Blanket

Growing Media

Needled Mineral
Hydro Blanket

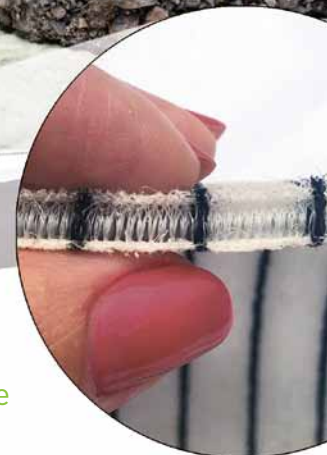
Detention Mat

Root Barrier

Conventional Roofing
Assembly (CRA)



The Detention Mat replaces the
Filter Fabric and Drainage Mat
in the StormCap[™] System

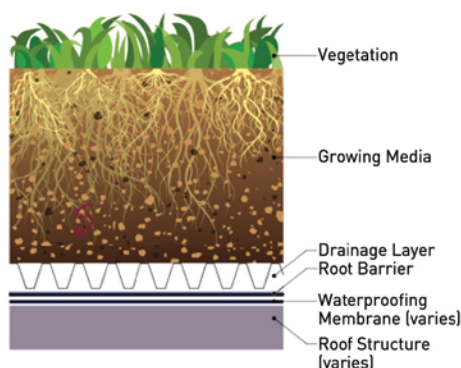


StormCap[™]+Detention manages stormwater on the roof by providing *retention* and *detention*, making it a full-fledged stormwater tool ready to face second day and extra-large storms. Detention provides a clear return on investment. StormCap[™]+Detention reduces or eliminates cisterns and bioretention facilities and frees up valuable space for income-generating development. It even works on a low-sloped roof!

What to do with all that space?

- Park cars
- Bicycle storage
- Build a fitness centre
- Open a coffee shop
- Childcare space
- More!

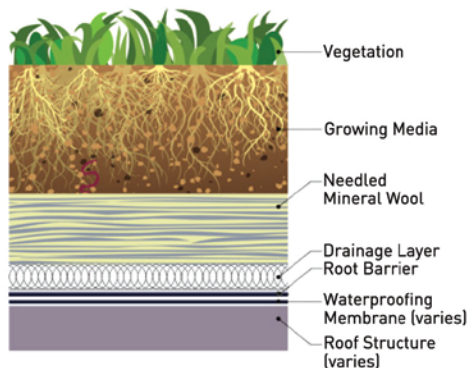
How does StormCap™+Detention work?



Traditional

- ☒ Retention
- ☐ Enhanced Retention
- ☐ Detention

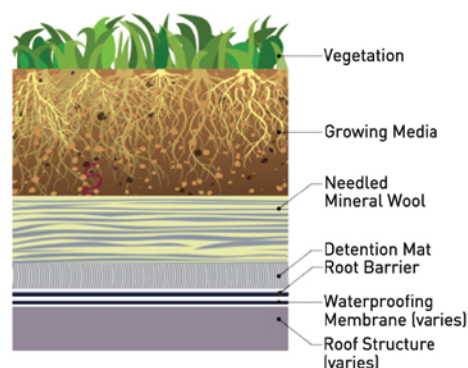
All green roofs provide retention. Retention can be achieved and replicated only when the system dries to a certain level. To become dry, water must leave as vapour through the slow process of evapotranspiration.



StormCap™

- ☒ Retention
- ☒ Enhanced Retention
- ☐ Detention

Incorporates needled mineral wool for enhanced retention. The rest of the water runs off, particularly in large storms and second-day storms.



StormCap™+Detention

- ☒ Retention
- ☒ Enhanced Retention
- ☒ Detention

Manages heavy rain events typically controlled by tanks or ponds. Incorporates a surface roughness layer - detention - that helps fill and submerge the system in rainwater, and then slowly empties afterwards.

SAMPLE PROJECT using a custom assembly of StormCap™+Detention

<u>1696 m²</u>	Size	<u>6.5 hours</u>	Runoff delay (centroid)
<u>2%</u>	Slope	<u>7.6 hours</u>	Recharge timeframe
<u>Type II 24-hr 157mm</u>	Design Storm	<u>97%</u>	Peak flow reduction
<u>16.8 L/s/ha</u>	Max Outflow Rate	<u>104 m³</u>	Detention storage
<u>6</u>	Roof drains	<u>46 m³</u>	Retention storage

Project-specific ROI:

- Eliminated stormwater tank
- Dramatically reduced excavation and foundation costs

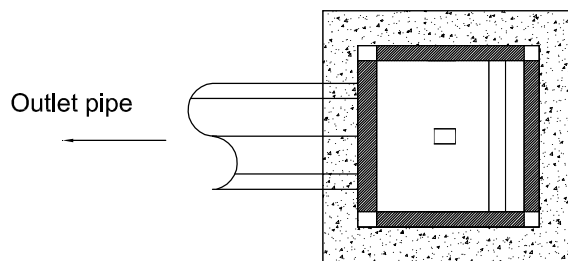
Contact Sasha Aguilera, Design Ambassador, who will help run calculations specific for your project. Sasha@nlsm.ca or 416 637 5772 ext 2.

Appendix C

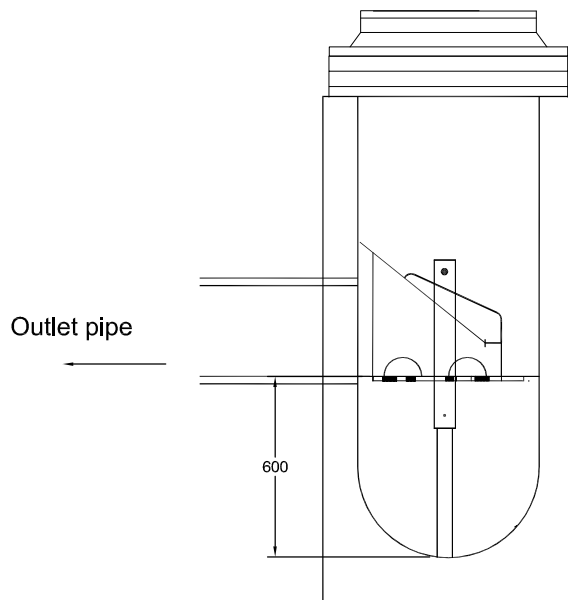
CB Shield Product Specifications

Notes

1. CB Shield can be installed at any time. In a non frozen condition.
2. The frame and cover should be well aligned with the catchbasin for proper installation
3. The catchbasin sump must be clean before installation
4. The grate should be at the same level as the standing water in the sump.



Top view



Profile view

CB Shield (600mm Sump)

**Average Annual Sediment Removal Rates (%) using a CB Shield
(based on ETV Sediment - 1 to 1000 micron Particle Size Distribution)**

Area to CB (ha)	Imperviousness ¹ (%)					
	20%	35%	50%	65%	80%	100%
0.02	57%	57%	57%	57%	56%	56%
0.05	56%	56%	56%	55%	55%	54%
0.10	56%	55%	54%	53%	52%	51%
0.20	54%	53%	51%	49%	48%	46%
0.30	53%	50%	48%	46%	45%	43%
0.40	51%	48%	46%	44%	42%	40%
0.50	50%	47%	44%	42%	40%	38%
0.60	49%	45%	43%	40%	39%	36%

Notes:

1. Runoff Coefficient 'C' is approximately equal to $0.05 + 0.9 \times \text{Impervious Fraction}$.
2. Above chart is based on long term continuous hydrologic analysis of Toronto, Ontario (Bloor St) rainfall data.
3. Assumes 0.6 m sump in CB and that maintenance is performed (i.e. CB cleaning) when required by sediment/pollutant build-up or otherwise.
4. See accompanying chart for suggested maintenance scheduling - AND - get CB Shield Inc. to monitor it for you in field.
5. Sediment/Pollutant removal rates based on third party certified laboratory testing using ETV sediment (PSD analysis available on request).
6. See additional discussion regarding scour protection from CB Shield during more infrequent runoff events.

CB SHIELD

info@cbshield.com | 519-212-9161 | www.cbshield.com

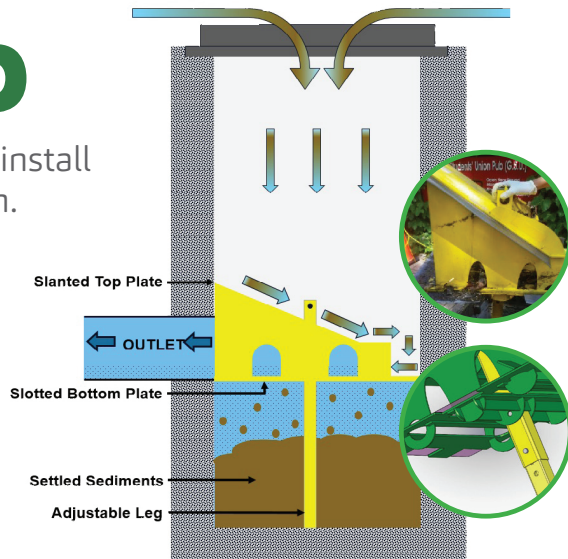
NOW ETV VERIFIED



CB Shield is the most cost-effective device to install and maintain in a stormwater treatment train.

**50%
OR GREATER
TSS CAPTURE**

**REDUCE
SCOURING BY
AS MUCH AS
92%**



Appendix D

Fire Flow Demand Calculations

121 Hume Street - Innovation Hub & Mixed Use Development

1. Required Fire Flow Calculation

$$F=220C(A)^{0.5}$$

F= the required fire flow in litres per minute
C= 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 and interior)
= 0.8 for non-combustible construction (unprotected metal structural components, masonry or metal walls)
= 0.6 for fire resistive construction (fully protected frame, floors, roof)
A= the total floor area in square metres in the building being considered

$$F= 14450 \text{ l/min}$$

Type of Construction = Non- Combustible
C= 1.0

Total GFA = 4,314 m²

2. Determine if occupancy type has a low contents fire hazard or high contents fire hazard.

Contents Classification

- | | |
|------------------------|------|
| 1) Non-Combustible | -25% |
| 2) Limited Combustible | -15% |
| 3) Combustible | 0% |
| 4) Free Burning | 15% |
| 5) Rapid Burning | 25% |

$$F= 10875 \text{ l/min}$$

Per Appendix A - Occupancy is Considered Low Fire Hazard

1	-25%
0	0%
0	0%
0	0%
0	0%
Total	-25%

3. Automatic Sprinkler Protection Reduction

Sprinkler Reduction Ratings

- | | |
|---|------|
| 1) Sprinkler System Conforms to NFPA 13 and other NFPA Sprinkler Standards | -30% |
| 2) Water supply standard for both the sprinkler system and fire department hose lines | -10% |
| 3) Fully supervised sprinkler system | -5% |

$$F= -4896 \text{ l/min}$$

Sprinkler Reduction Ratings - Building is not Sprinklered

1	-30%
1	-10%
1	-5%
Total	-45%

4. Exposure to adjacent buildings

Separation	Charge
1) 0 to 3.0m	25%
2) 3.1 to 10.0m	20%
3) 10.1 to 20.0m	15%
4) 20.1 to 30.0m	10%
5) 30.1 to 45.0m	5%

The total % shall be the sum of the % of all sides but shall not exceed 75%.

$$F= 8160 \text{ l/min}$$

Number of Walls within Exposure Limits

0	0%
0	0%
1	15%
8	80%
	0%
Total	75%

THEREFORE TOTAL FIRE FLOW REQUIRED = 14164 l/min
or 236.07 l/s