

Trailsedge East Commercial Block 75

Site Servicing and Stormwater Management Report



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Trailsedge East Commercial Block 75

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Trailsedge East Commercial Block 75

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1 Introduction

Stantec Consulting Ltd. (Stantec) has been commissioned by Richcraft Homes (Richcraft) to prepare the following Servicing and Stormwater Management Report for Commercial Block 75 at the east of Phase 4 of the Trailsedge East Subdivision. The subject property is located southwest of the intersection of Mer Bleue Road and Brian Coburn Boulevard within the southwest quadrant of the Gloucester East Urban Community (EUC) Phase 3 Community Design Plan (CDP) study area in the City of Ottawa. Block 75 is bordered by Phase 4 of the Trailsedge East Subdivision to the west, Mer Bleue Road to the east, Phases 1-3 of the Trailsedge East Subdivision to the south, and Brian Coburn Boulevard and a future mixed-use block to the north, as shown in **Figure 1.1** below.

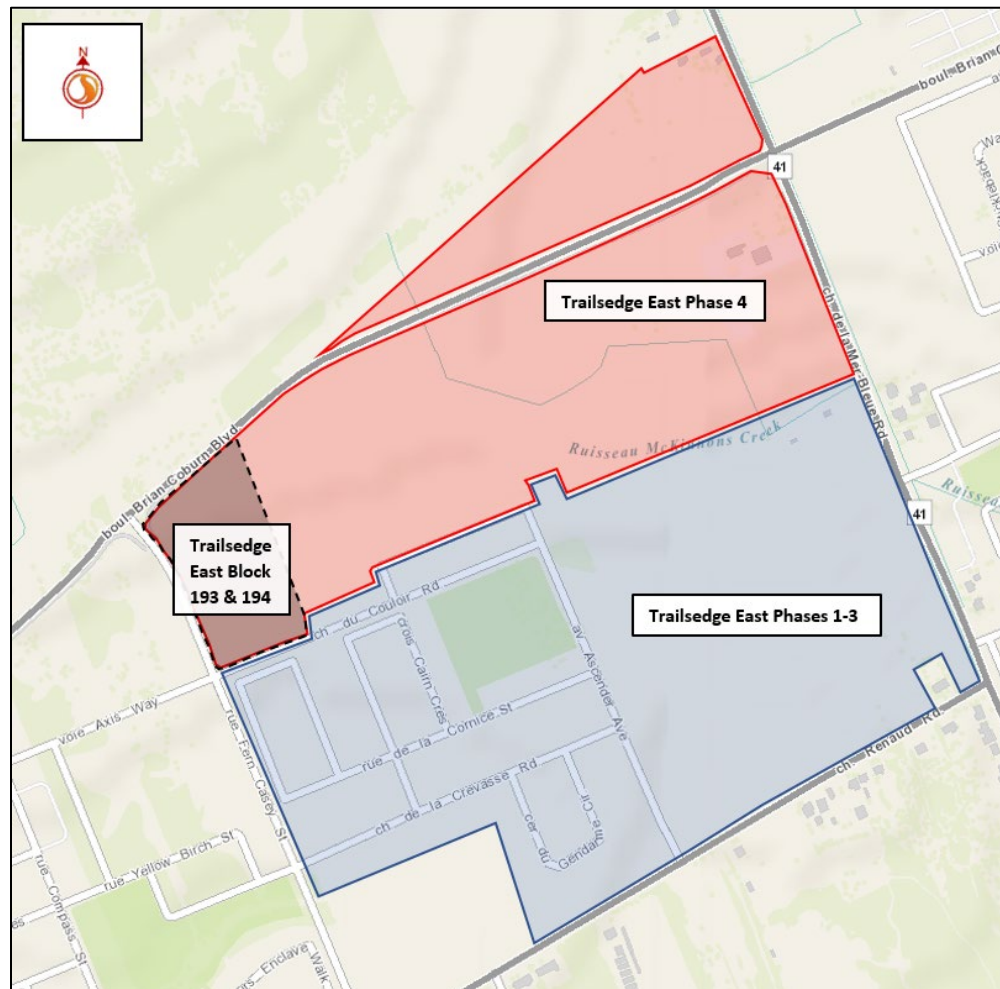


Figure 1.1: Key Map of the Trailsedge East Subdivision

The site is approximately 4.25 ha and consists of retail units, a grocery store, an anchor retail unit, and surface parking.

Stantec Consulting Ltd. has been retained by Richcraft to provide a detailed servicing design for the development. The intent of this report is to provide a servicing scenario that is free of conflicts and utilizes the existing and future infrastructure in accordance with the applicable background studies.

1.1 Background

The Trailsedge East subdivision is located within the Gloucester EUC and is subject to the Phase 3 CDP. Servicing criteria for the site are outlined in the Master Servicing Study (MSS) for East Urban Community Phase 3 Area Community Design Plan (DSEL June 2020) and the Site Servicing and Stormwater Management Report – Trailsedge East Phase 4 (Stantec, December 2024). The subdivision reports outline the design criteria for the development and the required trunk sewer and watermain infrastructure.

1.2 Objective

This servicing and stormwater management report is being prepared in support of the detailed design of Block 75 commercial development within Phase 4 of the Trailsedge East Subdivision. This report details the servicing required to support development of the subject property and is based on the approved Phase 4 design and the EUC MSS.

This servicing design is free of conflicts, meets all applicable Ottawa design guidelines and recommendations included in the various background studies.



2 References

The following documents were referenced in the preparation of this report:

- Gloucester East Urban Community (EUC) Infrastructure Servicing Study Update (In Support of the EUC Community Design Plan), Stantec Consulting Ltd., March 2005.
- Gloucester and Cumberland East Urban Community Expansion Area and Bilberry Creek Industrial Park Master Servicing, Stantec Consulting Ltd., July 2006.
- East Urban Community Pond No.1 Design Brief, Stantec Consulting Ltd., April 2008.
- Geotechnical Investigation – Proposed Residential Development Eden Park – East Portion – Renaud Road, Paterson Group Inc., November 15, 2016.
- Proposed Trailsedge East Development Community Transportation, Castleglenn Consultants, October 2016.
- Design Brief for the Reconstruction of the East Urban Community Stormwater Management Pond 1 for the Trailsedge Subdivision, David Schaeffer Engineering Ltd., December 2012.
- Stormwater Management Report for the Trailsedge West Subdivision, J.F. Sabourin and Associates Inc., January 2015.
- Trailsedge Subdivision / Proposed Modification of Mud Creek Channel Between Belcourt Extension and Compass Street, J.F. Sabourin and Associates, January 2015.
- Design Brief for the Trailsedge West, David Schaeffer Engineering Ltd., January 26, 2015.
- Design Brief – Minto Trailsedge Phase II, IBI Group, February 2015.
- City of Ottawa Sewer Design Guidelines, City of Ottawa, 2014 (and all subsequent technical bulletins).
- City of Ottawa Design Guidelines – Water Distribution, Infrastructure Services Department, City of Ottawa, First Edition, July 2010 (and all subsequent technical bulletins).
- East Urban Community (EUC) Phase 3 Master Servicing Study, David Schaeffer Engineering Ltd. (DSEL), June 2020.
- Trailsedge East – Functional Servicing Report, Stantec Consulting Ltd., August 2017.
- Trailsedge East Phase 1 – Detailed Servicing and Stormwater Management Report, Stantec Consulting Ltd., August 2019.
- Trailsedge East Phase 2-3 – Detailed Servicing and Stormwater Management Report, Stantec Consulting Ltd., 2021.
- Trailsedge East Block 193-194 – Servicing and Stormwater Management Report, Stantec Consulting Ltd., 2021.
- Trailsedge East Phase 4 – Functional Servicing Report, Stantec Consulting Ltd., July 2023
- Geotechnical Investigation – East Urban Community, Revision 1, Paterson Group Inc., February 7, 2024.
- Trailsedge East Phase 4 – Site Servicing and Stormwater Management Report, Stantec Consulting Ltd., December 2024.



3 Water Servicing

3.1 Existing Conditions

The proposed development is located within Pressure Zone 2E of the City of Ottawa's water distribution system. This pressure zone is fed by the Forest Ridge Pump Station, with the Innes Road elevated storage tank balancing storage for peak flows and demands. An existing 200 mm diameter watermain stub connects the commercial block to the 300 mm diameter watermain within Waddington Circle in the Phase 4 development area to the west, and a 400 mm diameter watermain exists along Mer Bleue Road between Renaud Road and Brian Coburn Boulevard.

3.2 Proposed Watermain Sizing and Layout

The proposed watermain alignment and sizing for the development is demonstrated on **Drawing SSP-1**. Proposed watermain diameters of 200 mm have been used for the commercial block. The East Urban Community Master Servicing Study (DSEL, 2020) was referenced to determine existing connections to the watermains within Waddington Circle from Phase 4 of the Trailsedge East development and the 400 mm diameter watermain along Mer Bleue Road. The watermain network plan from this master servicing study is included in **Appendix A**.

3.2.1 Connection to Existing Infrastructure

Potable water supply for the commercial block of the Trailsedge East Subdivision is to be provided via existing mains located on Street No. 7 of Phase 4 and Mer Bleue Road. **Drawing SSP-1** shows the locations of the connection points to the existing and proposed infrastructure.

3.2.2 Ground Elevations

The proposed ground elevations of the development range from approximately 87.1 m to 89.6 m. The elevations shown on **Figure 3.1** were interpolated from the proposed grading plan and assigned to the nodes in the hydraulic model.





Figure 3.1: Proposed Ground Elevations at Nodes

3.2.3 Water Demand

The City of Ottawa's Water Distribution Guidelines (July 2010) as well as the ISD-2010-02 and ISTB 2021-03 Technical Bulletins were used to determine water demands based on the commercial floor areas and peaking factors. Based on the draft plan phasing breakdown summarized in Section 1, the Trailsedge East commercial block is estimated to have a total building footprint area of 8,639 m².

A daily rate of 28,000 L/building footprint ha/day has been used to estimate average daily (AVDY) potable water demand for commercial areas. Maximum-day (MXDY) demands were determined by multiplying the AVDY demands by a factor of 1.5 for commercial areas. Peak hourly (PKHR) demands were determined by multiplying the MXDY by a factor of 1.8 for commercial areas. The estimated demands are shown in **Table 3.1** below and detailed in **Appendix A.1**.



Table 3.1: Water Demands

Area (ha)	AVDY (L/s)	MXDY (L/s)	PKHR (L/s)
0.86	0.3	0.4	0.8

3.3 Level of Service

3.3.1 Allowable Pressures

The City of Ottawa Water Distribution Design Guidelines state that the desired range of system pressures under normal demand conditions (i.e. basic day, maximum day and peak hour) should be in the range of 350 to 552 kPa (50 to 80 psi) and no less than 275 kPa (40 psi) at the ground elevation in the streets (i.e. at hydrant level). The maximum pressure at any point in the distribution system in occupied areas outside of the public right-of-way is 552 kPa (80 psi). As per the Ontario Building Code (OBC) & Guide for Plumbing, if pressures greater than 552 kPa (80 psi) are anticipated, pressure relief measures are required. The maximum pressure at any point in the distribution system in unoccupied areas shall not exceed 689 kPa (100 psi). Under emergency fire flow conditions, the minimum pressure objective in the distribution system is 138 kPa (20 psi).

3.3.2 Fire Flow

A maximum fire flow of 6,000 L/min (100 L/s) was determined for the commercial block based on the Fire Underwriters Survey (FUS) requirements using the grocery store floor area of 3086 m². Refer to **Appendix A.2** for FUS fire flow calculations.

3.4 Hydraulic Analysis

A hydraulic model was built by Stantec using the boundary conditions summarized in sections below.

The anticipated pressures in this development were assessed to meet minimum servicing requirements (average day and peak hour demands). A fire flow analysis was also performed under maximum day conditions. This analysis herein only assesses conditions for the commercial block. Refer to the hydraulic model prepared for the EUC Master Servicing Update (July 2006) for additional details of the watermain network within the EUC area at large or to the hydraulic model prepared for Phase 1 of the Trailsedge East Subdivision (Stantec, 2019), Phases 2-3 (Stantec, 2022), and Phase 4 (Stantec, 2024) for details of the watermain network within the previous phases.

3.4.1 Boundary Conditions

The boundary conditions below were provided by City of Ottawa staff based on previously supplied domestic demand and fire flow demand rates. Boundary conditions were based on computer model simulations and are summarized in **Table 3.2**. Boundary condition requests and correspondence with the City are included in **Appendix A.3**.



Table 3.2: Boundary Conditions

Location	AVDY (m)	PKHR (m)	MXDY+FF (m)
Mer Bleue	130.5	126.6	126.6
Waddington Circle *	130.5	126.1	119.9

*Note: No boundary conditions were requested for the connection at Waddington Circle in Phase 4, as the watermain has not yet been constructed. As such the boundary conditions for this connection were interpolated from Node 43 of the Trailsedge East Phase 4 hydraulic analysis. The sourced HGLs from Node 43 are provided in **Appendix A.3**.

3.4.2 Model Development

New watermains were added to the hydraulic model to simulate the proposed distribution system. Hazen-Williams coefficients (“C-Factors”) were applied to the new watermain in accordance with the City of Ottawa’s Water Distribution Design Guidelines (**Table 3.3**).

Table 3.3: C-Factors Applied Based on Watermain Diameter

Pipe Diameter (mm)	C-Factor
150	100
200 to 250	110
300 to 600	120
Over 600	130

3.4.3 Hydraulic Modeling Results

The watermain hydraulic analysis was conducted using the EPANET2.2 engine of PCSWMM. The hydraulic analysis assesses the water network under average day, peak hour, and maximum day plus fire flow conditions using the boundary conditions provided by the City of Ottawa and the Phase 4 Trailsedge East Servicing and SWM Report.

3.4.3.1 Average Day Demand (AVDY)

The hydraulic model results show that the maximum pressures under AVDY conditions are anticipated to be approximately 401 kPa to 412 kPa (58.2 psi to 59.8 psi). These pressures fall within the normal serviceability pressure range of 350 kPa to 552 kPa (50 psi to 80 psi) and no less than 275 kPa (40 psi) per the City of Ottawa Water Distribution Design Guidelines.





Figure 3.2: Maximum Pressures in the commercial block during AVDY Conditions

3.4.3.2 Peak Hour Demand (PKHR)

The hydraulic model results show that the minimum pressures under PKHR conditions are anticipated to be approximately 360 kPa to 373 kPa (52.2 psi to 54.1 psi). These pressures fall within the normal serviceability pressure range of 350 kPa to 552 kPa (50 psi to 80 psi) and no less than 275 kPa (40 psi) per the City of Ottawa Water Distribution Design Guidelines.



Figure 3.3: Minimum Pressures in the commercial block during PKHR Conditions

3.4.3.3 Maximum Day Plus Fire Flow (MXDY + FF)

An analysis was carried out using the hydraulic model to determine if the proposed development, under maximum day demands, could achieve the required fire flow of 6,000 L/min (100 L/s) in proximity to the grocery store. It was determined that the proposed watermain layout could provide the required fire flow while maintaining a residual pressure of 138 kPa (20 psi). This was accomplished using a steady-state maximum day demand scenario along with the automated fire flow simulation feature of the software. Fire flow analysis was only performed for the commercial block.

Results of the modeling analysis indicate that adequate flows are available under emergency fire demand conditions while maintaining a residual pressure of 138 kPa (20 psi). The residual pressures and



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available flows for the fire flow analysis are demonstrated in **Figure 3.4**. Results of the hydraulic modeling are included in Appendix.



Figure 3.4: Residual Pressures and Available Fire Flows during MXDY Conditions (100 L/s)



4 Wastewater Servicing

4.1 Background

As indicated in the Master Servicing Study (MSS) for EUC Phase 3 Area CDP, wastewater servicing for the Trailsedge East Development is conveyed to the Forest Valley Trunk Sewer (FVT) via a free flow gravity trunk running along Renaud Road to the Forest Valley Pumping Station. The MSS outlines the sanitary servicing requirements for the subject property, which identify an integrated network within Minto and further along Richcraft lands to the west, eventually connecting to the newly constructed 600 mm trunk sewer extension along Renaud Road recommended by the MSS. The Trailsedge East Phase 1 – Servicing and Stormwater Management Report in addition to the Trailsedge East Phase 4 Functional Servicing Report further detailed contributing flows from Phase 4 through sewers Arete Way. The Sanitary Drainage Plan and sanitary sewer design sheet for the existing Trailsedge East Phase 1 development is included in **Appendix B**.

The Design Brief – Minto Trailsedge Phase II report (IBI Group, 2015) identifies an external contribution to their subdivision based on a future population of 4,212, which includes drainage from the entirety of the Trailsedge East development Phases 1-4, as well as future lands forming a mixed-use community (MUC) identified within the EUC MSS.

4.2 Design Criteria

As outlined in the City's Sewer Design Guidelines, the following design parameters were used to calculate estimated wastewater flow rates and to preliminarily size on-site sanitary sewers:

- Minimum Full Flow Velocity – 0.6 m/s
- Maximum Full Flow Velocity – 3.0 m/s
- Manning's roughness coefficient for all smooth walled pipes – 0.013
- Extraneous Flow Allowance – 0.33 L/s/ha
- Commercial/Mixed Use Flows – 28,000 L/ha/day
- Harmon Correction Factor – 0.8
- Maintenance hole Spacing – 120 m
- Minimum Cover – 2.5 m

In addition, a commercial area peaking factor of 1.5 was assigned for commercial areas that contribute greater than 20% of the total tributary area to each sanitary sewer per Ottawa's Sewer Design Guidelines.



4.3 Proposed Servicing

Phase 1 of the Trailsedge East Subdivision is currently serviced by a network of gravity sewers which direct wastewater flows westerly to the trunk sewer within Fern Casey Street within the adjacent Minto development. Sanitary sewers within the commercial block are to flow westerly and connect to the sanitary sewer network established in Phase 4. The proposed sanitary sewer design sheet and associated Sanitary Drainage Area Plan can be found in **Appendix B.1**. The proposed sanitary sewer design indicates one connection point to the sanitary sewer stub within Phase 4 connecting to Waddington Circle.

The connection points and associated flows are summarized in **Table 4.1** below. Sanitary flows have been reduced based on the proposed building areas within the site as opposed to the conservative assumption of 28,000L/gross ha.

Table 4.1: Wastewater Connections to Trailsedge East Phase 4

Stub ID	Overall Area (ha)	Commercial Area (ha)	Total Flow (L/s)	Sewer Dia. (mm)
430	4.25	0.86	1.8	200

The Phase 4 Trailsedge East Subdivision wastewater analysis assigned a sanitary peak flow of 3.5 L/s for the sanitary sewer stub connecting the commercial block to Waddington Circle, as shown in **Table 4.2** below and detailed in subdivision sanitary sewer design sheet in **Appendix B.2**.

Table 4.2: Trailsedge East Subdivision Phase 4 Allocated Wastewater Flow to Commercial Block Sanitary Sewer Stub

Stub ID	Sanitary Drainage Area ID	Commercial Area (ha)	Total Flow (L/s)	Sewer Dia. (mm)
430	R430A	4.25	3.5	200

As demonstrated in the table above, peak flows from the proposed development can be accommodated within the downstream sewer network.



5 Stormwater Management

The following sections describe the stormwater management (SWM) design for the Trailsedge Commercial development.

5.1 Objectives

The objective of this stormwater management plan is to determine the measures necessary to control the quantity/quality of stormwater released from the proposed development to criteria established by the *Trailsedge East Phase 4 – Servicing and Stormwater Management Report* (Stantec, 2025) for the area as well as other applicable background documents, and to provide sufficient detail for approval and construction.

5.2 Design Constraints and Regulatory Requirements

The following summarizes the SWM criteria and constraints that will govern the detailed design of the proposed development as per the governing background studies.

- Design using the dual drainage principle.
- 'Enhanced' level of treatment (80% long term TSS removal) has been provided in the existing downstream EUC Pond 1.
- Maximum 100-year water depth of 0.35 m in road sags, including overflow spill depth.
- Proposed commercial/mixed-use blocks to provide on-site storage for all storms up to the 100-year storm.
- 100-year hydraulic grade line (HGL) to be a minimum 0.30 m below lowest building underside of footing elevation.
- Design inlets to capture the 2-year peak flow with a minimum inlet time of 10 minutes, and using 2004 City of Ottawa IDF parameters.
- Provide adequate emergency overflow conveyance.
- Limit minor and major system outflows to that identified within the Trailsedge East Phase 4 Servicing and Stormwater Management Report (Stantec, 2025).
- Design and submit a detailed erosion control plan.

With respect to overall water balance and servicing approach performance criteria as noted within Appendix A of the City of Ottawa's CLI-ECA for the stormwater management system, infiltration measures have been identified within section 11.6 of the Master Servicing Study (MSS) for East Urban Community Phase 3 (DSEL, 2020). Per the Master Servicing Study, infiltration within the subject lands is restricted by the low permeability of clay soils and perched water table, and infiltration volumes for any Low Impact Development (LID) measures would be limited. The MSS continues that the Mud Creek Cumulative Impact Study (Stantec, 2020) found that the implementation of LIDs would have little impact on the erosion protection requirements for Mud Creek, and as such recommended that the requirement for LIDs in the EUC area west of Mer Bleue Road solely include:



- A tree planting program in parkland, which is addressed in the CDP (Fotenn, May 2020).
- Using infiltration trenches in the backyards of singles and townhomes where feasible, which is addressed in the MSS.
- Setting right-of-way widths for the majority of local roadways at 18m and not 16.5m to ensure healthy street trees will be effective in providing evapotranspiration in post-development conditions, which is addressed in the CDP (Fotenn, 2020) and MTS (Castleglenn, 2020).

The MSS concludes that the subject area “can be considered as non-conductive to stormwater infiltration and recharge” per the Existing Conditions Report: Hydrogeology: East Urban Community MUC CDP (Paterson Group, 2014). As no specific LID requirements have been made for the subject commercial property to meet pre-development conditions, a conventional design approach has been considered for the site.

5.3 Stormwater Management

5.3.1 Allowable Release Rate

Allowable release rates for the site are based on the *Trailsedge East Phase 4 – Servicing and Stormwater Management Report* (Stantec, 2025), section 5.3.5.4 in consideration of a 4.26 ha site at 100% imperviousness. Peak release rates for the subdivision at this connection point are summarized in **Table 5.1** below.

Table 5.1: Allowable Release Rates

Storm Event	Minor System Flow (L/s) (MH 4030 Phase 4)	Major System Flow (L/s)
2-Year, 3 Hour Chicago	905	0.0
100-Year, 3 Hour Chicago	905	0.0

5.3.2 Proposed Conditions

The Trailsedge commercial block encompasses approximately 4.25 ha of land at approximately 79% imperviousness (runoff coefficient as determined by conservative estimate of hard surface coverage measured from site servicing plans). Storm sewers from the site will outlet to the existing storm sewer system on Waddington Circle.

Inlet control devices at low points within paved areas will be used to restrict inflow rates to suit the allowable release rate, and ensure no surface ponding during the 2-year storm event.

Major system peak flows from the majority of the block will be directed east onto Mer Bleue Road, with the exception of the northwestern portion of the site which will be directed north to Brian Coburn Boulevard as shown on **Drawing SD-1**.



5.3.3 Water Quantity Control

The Modified Rational Method (MRM) was used to assess the flow rate and volume of runoff generated from the development under post-development conditions. The site was subdivided into sub-catchments tributary to separate quantity control measures and subject to different inlet controls. **Drawing SD-1** details the subcatchment areas used in the analysis of the proposed site and outlines the major overland flow paths. The MRM spreadsheet is included in **Appendix C.2**.

5.3.3.1 Rooftop Storage

It is proposed to retain stormwater on the building rooftop by installing restricted flow roof drains. The following calculations assume the roof will be equipped with Zurn Model Z-105 Control-flo Roof Drains.

The Zurn Control-flow roof drain weir data has been used to calculate a practical roof release rate and detention storage volume for the rooftops. It should be noted that the Zurn drains have been used as an example only, and that other products may be specified for use, provided that the total roof drain release rate is restricted to match the maximum rate of release indicated in Table 5-2, and that sufficient storage is provided to meet (or exceed) the resulting volume of detained stormwater. Storage volume and controlled release rates are summarized in **Table 5.2**, and details for the Zurn drains are provided in **Appendix C.3**.

Table 5.2: Summary of Roof Controls

Area ID	Area (ha)	Depth (mm)	Discharge (L/s)	Volume (m³)
R1000A	0.11	138	8.5	34.4
R1001A	0.31	139	23.4	99.9
R1002A	0.03	128	3.9	8.1
R1003A	0.15	137	12.6	47.2
R1004A	0.04	131	4.0	10.0
R1005A	0.06	139	4.3	18.1
R1006A	0.07	136	6.3	22.7
R1007A	0.03	129	4.0	8.5
R1008A	0.03	129	4.0	8.8
R1009A	0.02	135	2.1	6.8

5.3.3.2 Surface Storage

In addition to rooftop storage, it is proposed to detain stormwater on the surface in parking lot areas using inlet control devices (ICDs) in associated catch basins. Additional runoff in excess of the 100-year storm event that exceeds available on-site storage will be directed overland towards adjacent municipal rights-of-way.



The Modified Rational Method was employed to determine the peak volume stored in the catch basins and surface storage areas. The site was subdivided into subcatchments (subareas) as defined by the proposed grades and the location, nature, or presence/absence of inlet control devices (ICDs). Each subcatchment was assigned a runoff coefficient based on the proposed finished surface. **Drawing SD-1** illustrates the proposed subcatchments. The inlet control devices were sized based on the available target release rate from the site during the 5-year storm event. Storage volume and controlled release rates from the on-site catch basins during the 5 and 100-year events are summarized in Table 5-3 below

Table 5.3 presents the individual parameters that vary for each of the proposed subcatchments.

Table 5.3: 2 and 100-Year Peak Surface Volume and Controlled Discharge Summary

Area ID	ICD (mm)	2-Year Event			100-Year Event		
		Release Rate (L/s)	Vrequired (m ³)	Vavailable (m ³)	Release Rate (L/s)	Vrequired (m ³)	Vavailable (m ³)
L101A	95	22.1	0	0	26.6	22.5	41.6
L103A	95	8.9	0	0	21.6	2.2	4.4
L103B	83	11.6	0	0	16.7	10	12.4
L103C	83	10.0	0	0	15.1	8.4	12.0
L104A	83	8.5	0	0	17.1	4.6	14.8
L104B	108	20.4	0	0	28.7	18.4	19.3
L105A	83	8.8	0	0	16.7	5.4	5.5
L106A	200	33.7	0	0	90.2	3.9	4.8
L109A	95	15.8	0	0	22.2	14.3	16.3
L111A	83	10.5	0	0	16.9	8.2	15.3
L112A	152	23.8	0	0	55.6	8.1	9.4
L112B	152	44.8	0	0	58.2	39.5	91.5
L112C	178	56.2	0	0	78.0	48.8	134.3
L112D	152	44.3	0	0	58.4	35.2	79.4
L114A	127	32.3	0	0	40.8	31.2	78.9
L114B	250	119.1	0	0	165.4	91.7	140.4
L107A	None	4.2	0	0	10.9	0	0

5.3.3.3 Uncontrolled Area

A small portion of the overall site (see areas UNC-1 and UNC-2 on **Drawing SD-1**) could not be graded to enter the site's storm system and as such will drain uncontrolled towards Brian Coburn Boulevard and Mer Bleue Road. Peak discharges from uncontrolled areas have been considered in the overall SWM plan and have been balanced through over-controlling proposed site discharge rates to meet target levels.



Table 5.4 summarizes the 2-year and 100-year uncontrolled release rates from the proposed development.

Table 5.4: Peak Uncontrolled 2-Year and 100-Year Release Rate

Area ID	Area (ha)	T _c (min)	2-Year Release (L/s)	100- Year Release (L/s)
UNC-1	0.24	10	23.4	68.1
UNC-2	0.15	10	8.4	24.4

5.3.4 Results

Table 5.5 demonstrates that the proposed stormwater management plan provides adequate attenuation storage to meet the target peak outflow rates for the site.

Table 5.5: Storm Event Peak Discharge Rates

Subcatchment Areas	2-Year Peak Discharge (L/s)	100-Year Peak Discharge (L/s)
Controlled	523	812
Uncontrolled	32	92
Total	555	905
Target	905	905



6 Grading

Phase 4 of the Trailsedge East Subdivision has been cleared of topsoil and deleterious material. Phase 4 of the site is expected to be subjected to a soil preloading program as per previous phases of development.

Historically, the topography across the site has been relatively flat, generally sloping from northeast to southwest, with an existing ditch running parallel to the western property line conveying drainage from the site and connecting to the subdrains constructed as part of Phases 2 and 3 to the south of the site.

The objective of the grading design strategy is to satisfy the stormwater management requirements, adhere to permissible grade raise restrictions where possible for the site, and provide for minimum cover requirements for storm and sanitary sewers. The grading design also follows the recommendations outlined in the EUC Phase 3 MSS where possible, and endeavors to provide an emergency overland route to the existing rights-of-way. An emergency overland flow route will be provided to Brian Coburn / Mer Bleue per requirements outlined in the MSS.

Refer to **Drawing GP-1** for detailed grading plans of the block.



7 Approvals

An Environmental Compliance Approval (ECA) from the Ontario Ministry of Environment, Conservation, and Parks (MECP) will not be required for the proposed site servicing works related to storm sewers and sanitary sewers so long as the property is retained under singular ownership. The Rideau Valley Conservation Authority (RVCA) and South Nation Conservation Authority (SNCA) will be circulated on detailed design submissions in conjunction with municipal approval of the subdivision. DFO/RVCA approval has already been given with regards to fish habitat compensation for the Mud Creek headwaters during design of Trailsedge West.

An MECP Permit to Take Water (PTTW) may be required for the development if more than 400,000 L/day of ground or surface water is to be pumped during the construction works. If between 50,000 L/day and 400,000 L/day are expected to be pumped during the construction works, registration on the Environmental Activity and Sector Registry (EASR) are required. The geotechnical consultant shall confirm prior to construction whether a PTTW or EASR registry are required for construction dewatering purposes.

8 Utilities

Utility providers have been upgrading their infrastructure as required to service the development area in conjunction with Phase 4 of the Trailsedge east subdivision.

We are working with the utility providers to develop their utility designs for the subject site in coordination with the servicing and landscaping for the subdivision development. Coordinated designs from the electrical, gas and telecom providers as well as any City streetlighting, OC Transpo, streetscape and servicing constraints will be reflected on the composite utility plan.



9 Erosion Control During Construction

To protect downstream water quality and prevent sediment build-up in catch basins and storm sewers, erosion and sediment control measures must be implemented during construction. The following recommendations will be included in the contract documents and communicated to the Contractor.

1. Until the local storm sewer is constructed, groundwater in trenches will be pumped into a filter mechanism prior to release to the environment. After construction of required downstream systems, any construction dewatering will be routed to the nearest storm sewer.
2. Seepage barriers to be constructed in any temporary drainage ditches.
3. Install a silt fence along the site perimeter.
4. Limit the extent of the exposed soils at any given time.
5. Re-vegetate exposed areas as soon as possible.
6. Minimize the area to be cleared and grubbed.
7. Protect exposed slopes with geotextiles, geogrid, or synthetic mulches.
8. Install silt barriers/fencing around the perimeter of the site as indicated in **Drawing ECDS-1** to prevent the migration of sediment offsite.
9. Install trackout control mats (mud mats) at the entrance/egress to prevent migration of sediment into the public ROW.
10. Provide sediment traps and basins during dewatering works.
11. Install sediment traps (such as SiltSack® by Terrafix) between catch basins and frames.
12. Schedule the construction works at times which avoid flooding due to seasonal rains.

The Contractor will also be required to complete inspections and guarantee the proper performance of their erosion and sediment control measures at least after every rainfall. The inspections are to include:

- Verification that water is not flowing under silt barriers.
- Cleaning and changing the sediment traps placed on catch basins.

Refer to **Drawing ECDS-1** for the proposed location of silt fences, sediment traps, and other erosion control measures.



10 Geotechnical Considerations and Grading

10.1 Geotechnical Investigation

A geotechnical investigation report for the development was completed by Paterson Group on February 7, 2024, which expanded on the previously prepared geotechnical investigation reports for Eden Park – East Portion – Renaud Road and Trailsedge East – Renaud Road, which were prepared on December 29, 2008, and July 26, 2018 (Paterson Group), respectively. Field testing throughout the subject site of the Trailsedge development was completed between March 2002 and June 2018. Excerpts from the geotechnical report are included in **Appendix D**.

The subsurface profile within the Trailsedge East Phase 4 lands consists of a shallow bedrock and deep silty clay deposits. More specifically, the shallow bedrock was found beneath a cultivated organic zone/topsoil overlain by a silty sand, and/or a clayey silt layer within the north portion of the site. The remainder of the subject site was underlain by a sensitive silty clay deposit.

Groundwater levels, determined via piezometers, varied in depths ranging from 0.2 to 7.3 m below original ground surface (elevations of 86.8 to 87.62 m) based on monitoring in October 2008. These groundwater levels can be influenced by surface water perched within the borehole backfill material and are subject to seasonal fluctuations.

A 0.5 to 1.5m permissible grade raise restriction (above original ground surface) is recommended within the Trailsedge East Phase 4 development, per the Paterson Group's permissible Grade Raise Plan (Drawing PG3130-7) in Appendix 2 of the Geotechnical – Existing Conditions Report (Paterson Group).

10.1.1 Recommendations for Settlement and Grade Raise

As outlined in the Geotechnical Investigation Report (Paterson Group, 2024) and the Completion of Surcharge Settlement Monitoring Program Memo (Paterson Group, 2023) the development will be subjected to grade restrictions and residential buildings should be design in accordance with Part 4 of the current Ontario Building Code (OBC). Given permissible grade raise restrictions are expected to be exceeded, a 2.0 m settlement surcharge monitoring program was initially selected to achieve the desirable settlements within the residential blocks prior to construction of Phase 4, inclusive of the subject site. Additionally, municipal services within the subject site will be completed mostly through OHSA Type 2 and 3 soils.

In subsequent survey events, Paterson has observed a decrease in the rate of settlement with minor total settlement recorded. As such, Paterson has deemed the surcharge settlement monitoring program to be complete from a geotechnical perspective.



10.1.2 Recommendations for Groundwater Control

Groundwater infiltration into excavated areas is expected to be low, given site soil properties, and can typically be controlled via open sumps. In instances where perched groundwater conditions produce high temporary groundwater infiltration levels, pumping from open sumps are expected to sufficiently control groundwater inflows into the excavated area. Under circumstances where the pumping flowrate exceeds 50,000 L/day during construction of the development, a temporary MOE permit to take water will be required.

The surficial geology across the site is primarily glaciomarine clay. The infiltration potential on the subject area is considered to be low due to the low permeability silty clay and shallow bedrock of relatively high competence. As such, the subject area can be considered as non-conductive to stormwater infiltration and recharge per the *Existing Conditions Report: Hydrogeology: East Urban Community MUC CDP* (Paterson Group, October 2, 2014).

10.2 Grading Plan

A detailed Grading Plan has been prepared for the commercial block and is detailed on **Drawing GP-1**. As shown on the grading plan, road grades for Phase 4 with direct tie-in to on-site grading exceed the permissible grade raise identified in the Geotechnical Investigation Report, dated February 7, 2024 (Report No. PG3130-2). As specified above, the geotechnical report allows for grade raises between 0.5 m and 1.5 m depending on the subsurface soil conditions. Since the proposed grading indicates portions of the site above the permissible grade raise, a surcharge settlement monitoring program was completed by Paterson Group, with the findings provided in the 2023 memo (Memo No. PG2384-MEMO.142) attached with the submission package.



11 Conclusion

11.1 Water Servicing

The proposed piping alignment and sizing can achieve the required level of service in the proposed commercial block of the subdivision. Based on the hydraulic analysis, the following conclusions were made:

- The proposed water distribution system in the commercial block is recommended to include 200mm diameter pipes;
- During average day (AVDY) conditions, the block's proposed watermain network will operate under the maximum pressure objective of 552 kPa (80 psi);
- During peak hour (PKHR) conditions, the block's proposed watermain network is capable of operating above the minimum pressure objective of 276 kPa (40 psi);
- During fire conditions with a fire demand of 6,000 L/min (100 L/s) in proximity to the proposed grocery store, the commercial block's proposed piping is capable of providing sufficient fire flow while maintaining a residual pressure of 138 kPa (20 psi) in all areas within the block;

11.2 Wastewater Servicing

The Trailsedge East commercial block will be serviced by a network of gravity sewers which will direct wastewater flows westerly to the sanitary sewer in Waddington Circle, and ultimately joining the subdivision wastewater flows to Renaud Road and the Forest Valley Pumping Station. The proposed sanitary sewer design for the commercial block indicates one connection point to the existing sewer network within Trailsedge East Phase 4. The preferred cover requirement of 2.5 m for the sanitary sewer system has been satisfied where possible (with the use of insulation on shallow building services), and requirements for slope and velocities have been met within the local internal sewers. Downstream sewers have been adequately sized to receive peak sanitary discharge from the proposed subdivision.

11.3 Stormwater Management

Rooftop and surface storage on the parking lot have been proposed to limit the post-development stormwater discharge rate for all rainfall events up to and including the 100-year event to their respective allowable release rates.

The downstream SWM Pond 1 has sufficient volume capacity to receive runoff volumes from the proposed site and provide the required water quantity and quality control as outlined in the SWM facility design report submitted by others.



11.4 Grading

The grading for this site has been designed to allow for an emergency overland flow outlet to downstream rights-of-way as per City standards and to minimize the grade raise restrictions as recommended by the Geotechnical Investigation by Paterson Group (February 7, 2024).

11.5 Approvals/Permits

An MECP Environmental Compliance Approval (ECA) will not be required for the installation of the proposed storm and sanitary sewers within the site so long as it is retained under singular ownership. A Permit to Take Water or registration on the EASR may be required for dewatering works during sewer/watermain installation pending confirmation by the geotechnical consultant. The Rideau Valley Conservation Authority and South Nation Conservation Authority will need to be consulted in order to obtain municipal approval for site development. No other approval requirements from other regulatory agencies are anticipated.



Appendices



Appendix A Water Servicing

A.1 Domestic Water Demand



Trailsedge Commercial Block, Ottawa, ON - Domestic Water Demand Estimates

Site Plan provided by Hobin Architecture. (December 2024)

Project No. 160401884

Designed by: MW

Revision: 01

Checked by:

Date: 22-Jan-2025

Date Checked:

City File No.

**Demand conversion factors per Table 4.2 of the City of Ottawa
Water Design Guidelines and Technical Bulletin ISTB-2021-03:**

Commercial

28000

L/area/day



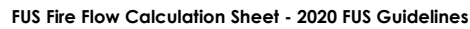
Commercial Phase	Building Footprint Area (m ²)	Avg Day Demand		Max Day Demand ¹		Peak Hour Demand ¹	
		(L/min)	(L/s)	(L/min)	(L/s)	(L/min)	(L/s)
Phase 1	5233	10.2	0.17	15.3	0.25	27.5	0.46
Phase 2	3407	6.6	0.11	9.9	0.17	17.9	0.30
Total Site :	8639	16.8	0.3	25.2	0.4	45.4	0.8

Notes:

- ¹ Water demand criteria used to estimate peak demand rates for commercial areas are as follows:
- maximum daily demand rate = 1.5 x average day demand rate
 - peak hour demand rate = 1.8 x maximum day demand rate (as per Technical Bulletin ISD-2010-02)

A.2 FUS Fire Flow Demands





Date: 2025-01-28

Description: Single-storey Grocery Store, 3086 m² floor area.

Notes: Site Plan (December 2024) provided by Hobin Architecture.

Step	Task	Notes									Value Used	Req'd Fire Flow (L/min)
1	Determine Type of Construction	Type II - Noncombustible Construction / Type IV-A - Mass Timber Construction									0.8	-
2	Determine Effective Floor Area	Sum of Two Largest Floors + 50% of Eight Additional Floors				Vertical Openings Protected?					NO	-
		3086									3086	-
3	Determine Required Fire Flow	(F = 220 x C x A ^{1/2}). Round to nearest 1000 L/min									-	10000
4	Determine Occupancy Charge	Combustible									0%	10000
5	Determine Sprinkler Reduction	Conforms to NFPA 13									-30%	-5000
		Standard Water Supply									-10%	
		Fully Supervised									-10%	
		% Coverage of Sprinkler System									100%	
6	Determine Increase for Exposures (Max. 75%)	Direction	Exposure Distance (m)	Exposed Length (m)	Exposed Height (Stories)	Length:Height Factor (m x stories)	Construction of Adjacent Wall	Firewall / Sprinklered ?		-	-	
		North	20.1 to 30	25.65	1	21-49	Type I-II - Unprotected Openings	YES		0%	1000	
		East	> 30	0	0	0-20	Type V	NO		0%		
		South	20.1 to 30	22.56	1	21-49	Type I-II - Unprotected Openings	YES		0%		
		West	20.1 to 30	57.14	2	> 100	Type V	NO		10%		
7	Determine Final Required Fire Flow	Total Required Fire Flow in L/min, Rounded to Nearest 1000L/min									6000	
		Total Required Fire Flow in L/s									100.0	
		Required Duration of Fire Flow (hrs)									2.00	
		Required Volume of Fire Flow (m ³)									720	

A.3 Boundary Conditions



Boundary Conditions

Trailsedge Commercial Block

Provided Information

Scenario	Demand	
	L/min	L/s
Average Daily Demand	18	0.30
Maximum Daily Demand	24	0.40
Peak Hour	48	0.80
Fire Flow Demand #1	6,000	100.00

Location



Results

Connection 1 – Mer Bleue

Demand Scenario	Head (m)	Pressure¹ (psi)
Maximum HGL	130.5	60.8
Peak Hour	126.6	55.3
Max Day plus Fire Flow #1	126.6	55.3

¹ Ground Elevation = 87.7 m

Notes

1. *Any connection to a watermain 400 mm or larger should be approved by DWS as per the Water Design Guidelines Section 2.4 Review by Drinking Water Services.*

Disclaimer

The boundary condition information is based on current operation of the city water distribution system. The computer model simulation is based on the best information available at the time. The operation of the water distribution system can change on a regular basis, resulting in a variation in boundary conditions. The physical properties of watermains deteriorate over time, as such must be assumed in the absence of actual field test data. The variation in physical watermain properties can therefore alter the results of the computer model simulation. Fire Flow analysis is a reflection of available flow in the watermain; there may be additional restrictions that occur between the watermain and the hydrant that the model cannot take into account.

Junction Results - Basic Day

ID	Demand (L/s)	Elevation (m)	Head (m)	Pressure (m)	Pressure (psi)2	Pressure (kPa)
33	0.09	89.64	130.53	40.89	58.14	400.86
34	0.00	89.63	130.53	40.90	58.16	400.98
32	0.09	89.60	130.52	40.92	58.19	401.18
31	0.03	89.58	130.52	40.93	58.21	401.31
35	0.04	89.59	130.53	40.94	58.21	401.37
36	0.18	89.57	130.53	40.96	58.24	401.56
37	0.16	89.50	130.53	41.03	58.34	402.26
43	1.38	89.49	130.53	41.04	58.36	402.39
30	0.15	89.27	130.51	41.24	58.64	404.31
40	0.18	89.25	130.56	41.31	58.75	405.04
29	0.14	89.17	130.50	41.33	58.76	405.17
38	0.08	89.21	130.55	41.34	58.79	405.31
28	0.16	89.08	130.43	41.35	58.80	405.43
25	0.08	89.02	130.40	41.38	58.84	405.66
39	0.05	89.17	130.55	41.39	58.85	405.78
15	0.24	88.98	130.40	41.42	58.89	406.04
24	0.00	88.97	130.40	41.43	58.91	406.20
22	0.09	88.95	130.40	41.45	58.94	406.36
13	0.22	88.94	130.40	41.46	58.95	406.48
14	0.22	88.88	130.40	41.52	59.03	407.02
23	0.00	88.88	130.40	41.52	59.04	407.09
11	0.09	88.84	130.40	41.56	59.09	407.43
21	0.04	88.78	130.40	41.62	59.18	408.03
12	0.06	88.78	130.40	41.62	59.19	408.09
20	0.14	88.74	130.40	41.66	59.24	408.41
10	0.11	88.73	130.40	41.67	59.25	408.53
18	0.02	88.69	130.40	41.71	59.31	408.96
9	0.04	88.69	130.40	41.71	59.31	408.96
5	0.00	88.65	130.40	41.75	59.37	409.34
3	0.08	88.64	130.40	41.76	59.38	409.40
19	0.11	88.59	130.40	41.81	59.45	409.87
7	0.11	88.57	130.40	41.83	59.48	410.07
4	0.22	88.56	130.40	41.84	59.50	410.22
1	0.32	88.37	130.40	42.03	59.77	412.08
2	0.00	88.35	130.40	42.05	59.80	412.27
26	2.96	88.30	130.40	42.10	59.86	412.72
0	0.04	88.14	130.40	42.26	60.09	414.32
27	2.78	88.10	130.53	42.43	60.33	415.95

Link Results - Basic Day

ID	FROM	TO	Length (m)	Diameter (mm)	Roughness	Flow (L/s)	Velocity (m/s)
1032	32	31	9.03	204	110	2.413	0.074
1030	30	29	59.71	204	110	5.251	0.161
1029	29	28	84.99	204	110	9.904	0.303
1028	4-Ascender	15	61.79	297	120	3.243	0.047
1027	15	26	168.84	297	120	2.960	0.043
1026	13	25	111.22	204	110	0.082	0.003
1025	25	24	9.94	204	110	0.002	0.000
1031	31	30	47.50	204	110	2.383	0.073
1024	24	23	41.78	204	110	0.002	0.000
1023	23	22	10.39	204	110	0.002	0.000
1022	22	14	112.30	204	110	-0.088	0.003
1021	14	21	72.01	204	110	-0.307	0.009
1020	21	20	9.91	204	110	-0.347	0.011
1019	20	19	115.13	204	110	-0.487	0.015
1018	19	18	8.25	204	110	-0.597	0.018
1017	18	3	36.94	204	110	-0.617	0.019
1016	3	3-Arrete	42.50	204	110	-0.603	0.018
C5	2-Randkluft_S	0	14.50	204	110	0.608	0.019
1014	15	14	84.50	204	110	0.043	0.001

Junction Results - Peak Hour

ID	Demand (L/s)	Elevation (m)	Head (m)	Pressure (m)	Pressure (psi)2	Pressure (kPa)
33	0.48	89.64	126.13	36.49	51.88	357.71
34	0.00	89.63	126.13	36.50	51.90	357.83
32	0.48	89.60	126.13	36.53	51.94	358.13
35	0.19	89.59	126.13	36.54	51.96	358.23
31	0.18	89.58	126.13	36.54	51.96	358.26
36	0.99	89.57	126.13	36.56	51.99	358.48
43	7.58	89.49	126.13	36.64	52.10	359.24
37	0.91	89.50	126.15	36.65	52.11	359.28
30	0.84	89.27	126.13	36.85	52.40	361.30
29	0.39	89.17	126.13	36.95	52.55	362.29
40	0.97	89.25	126.24	36.99	52.60	362.65
38	0.43	89.21	126.21	37.00	52.61	362.72
28	0.91	89.08	126.11	37.04	52.66	363.11
39	0.27	89.17	126.22	37.05	52.69	363.25
25	0.43	89.02	126.15	37.13	52.79	363.99
24	0.00	88.97	126.15	37.18	52.87	364.53
15	1.30	88.98	126.17	37.19	52.88	364.59
22	0.48	88.95	126.15	37.20	52.90	364.71
13	1.20	88.94	126.15	37.21	52.91	364.80
14	1.20	88.88	126.16	37.27	53.00	365.42
23	0.00	88.88	126.15	37.27	53.00	365.44
11	0.48	88.84	126.14	37.29	53.03	365.64
12	0.34	88.78	126.14	37.36	53.13	366.31
21	0.24	88.78	126.16	37.38	53.15	366.48
10	0.58	88.73	126.13	37.40	53.18	366.66
20	0.77	88.74	126.16	37.42	53.21	366.87
9	0.09	88.69	126.13	37.44	53.24	367.08
5	0.00	88.65	126.12	37.48	53.29	367.42
18	0.10	88.69	126.18	37.49	53.31	367.58
7	0.58	88.57	126.10	37.53	53.37	367.94
3	0.43	88.64	126.19	37.54	53.39	368.09
4	0.91	88.56	126.12	37.56	53.41	368.28
19	0.58	88.59	126.18	37.58	53.44	368.48
1	1.73	88.37	126.18	37.82	53.77	370.74
26	16.28	88.30	126.13	37.83	53.79	370.84
2	0.00	88.35	126.19	37.84	53.80	370.95
27	12.12	88.10	126.12	38.02	54.06	372.72
0	0.24	88.14	126.19	38.05	54.11	373.09

Link Results - Peak Hour

ID	FROM	TO	Length (m)	Diameter (mm)	Roughness	Flow (L/s)	Velocity (m/s)
1032	32	31	9.03	204	110	-0.929	0.028
1030	30	29	59.71	204	110	0.247	0.008
1029	29	28	84.99	204	110	3.821	0.117
1028	4-Ascender	15	61.79	297	120	22.003	0.318
1027	15	26	168.84	297	120	16.280	0.235
1026	13	25	111.22	204	110	-1.149	0.035
1025	25	24	9.94	204	110	-1.579	0.048
1031	31	30	47.50	204	110	-1.109	0.034
1024	24	23	41.78	204	110	-1.579	0.048
1023	23	22	10.39	204	110	-1.579	0.048
1022	22	14	112.30	204	110	-2.059	0.063
1021	14	21	72.01	204	110	-2.811	0.086
1020	21	20	9.91	204	110	-3.051	0.093
1019	20	19	115.13	204	110	-3.821	0.117
1018	19	18	8.25	204	110	-4.401	0.135
1017	18	3	36.94	204	110	-4.501	0.138
1016	3	3-Arrete	42.50	204	110	-6.181	0.189
C5	2-Randkluft_S	0	14.50	204	110	6.677	0.204

Fire Flow Results - Max Day + 200 L/s

ID	Static Demand (L/s)	Static Pressure (m)	Static Pressure (psi)	Static Pressure (kPa)	Static Head (m)	Fire Flow Demand (L/s)	Residual Pressure (m)	Residual Pressure (psi)	Available Flow (L/s)	Available Pressure (psi)
27	5.82	31.75	45.14	311.26	119.85	200.00	25.42	36.15	357.97	20
40	0.44	30.82	43.82	302.14	120.07	200.00	29.82	42.40	1240.23	20
39	0.12	30.86	43.88	302.57	120.03	200.00	29.40	41.80	879.30	20
38	0.20	30.80	43.80	301.97	120.01	200.00	29.17	41.47	808.96	20
37	0.41	30.39	43.22	297.96	119.90	200.00	27.66	39.33	563.44	20
36	0.45	30.29	43.07	296.98	119.86	200.00	26.48	37.65	458.34	20
35	0.09	30.27	43.04	296.72	119.85	200.00	25.51	36.28	402.03	20
34	0.00	30.22	42.98	296.31	119.85	200.00	25.30	35.98	393.31	20
33	0.22	30.21	42.96	296.19	119.85	200.00	25.26	35.92	391.89	20
32	0.22	30.18	42.92	295.89	119.78	200.00	20.76	29.52	269.19	20
31	0.08	30.19	42.93	295.99	119.77	200.00	21.09	30.00	274.57	20
30	0.38	30.48	43.34	298.81	119.75	200.00	24.34	34.61	345.40	20
29	0.21	30.47	43.32	298.71	119.64	200.00	26.03	37.01	411.37	20
28	0.41	30.06	42.74	294.66	119.13	200.00	29.10	41.38	876.92	20
26	7.40	35.01	49.79	343.27	123.31	200.00	27.40	38.96	365.98	20
25	0.20	30.71	43.67	301.09	119.73	200.00	15.15	21.54	209.70	20
24	0.00	30.79	43.78	301.86	119.76	200.00	15.09	21.46	209.20	20
23	0.00	30.98	44.06	303.76	119.86	200.00	15.33	21.80	211.53	20
22	0.22	30.93	43.99	303.28	119.89	200.00	15.45	21.97	212.77	20
21	0.11	31.24	44.43	306.31	120.02	200.00	20.06	28.53	267.08	20
20	0.35	31.26	44.45	306.50	120.00	200.00	19.96	28.38	263.75	20
19	0.26	31.19	44.36	305.82	119.79	200.00	22.94	32.62	318.68	20
18	0.04	31.09	44.20	304.76	119.77	200.00	23.42	33.30	332.99	20
15	0.59	34.34	48.83	336.67	123.32	200.00	31.82	45.24	858.66	20
14	0.55	31.28	44.48	306.70	120.17	200.00	23.13	32.90	368.94	20
13	0.55	30.53	43.41	299.29	119.47	200.00	20.85	29.64	294.35	20
12	0.15	29.09	41.37	285.24	117.87	200.00	17.42	24.77	238.38	20
11	0.22	28.67	40.77	281.10	117.51	200.00	16.92	24.07	232.59	20
10	0.26	26.79	38.09	262.60	115.52	200.00	16.88	24.00	234.95	20
9	0.05	26.61	37.84	260.92	115.30	200.00	17.13	24.35	239.14	20
7	0.26	22.01	31.30	215.80	110.59	200.00	21.23	30.19	644.34	20
0	0.11	28.43	40.43	278.72	116.57	200.00	26.60	37.83	655.81	20
5	0.00	24.95	35.47	244.57	113.59	200.00	20.49	29.13	356.11	20
4	0.42	24.97	35.50	244.79	113.53	200.00	20.86	29.66	376.69	20
3	0.20	31.06	44.17	304.55	119.71	200.00	26.96	38.33	520.68	20
2	0.00	28.79	40.93	282.22	117.14	200.00	24.64	35.04	450.41	20
1	0.79	28.33	40.29	277.79	116.70	200.00	24.83	35.31	511.61	20
43	3.45	30.40	43.23	298.06	119.89	200.00	19.49	27.71	250.74	20

A.4 Watermain Hydraulic Analysis



Junction Results - Basic Day

ID	Demand (L/s)	Elevation (m)	Head (m)	Pressure (m)	Pressure (psi)2	Pressure (kPa)
23	0.00	89.58	130.52	40.94	58.21	401.35
22	0.00	89.58	130.52	40.94	58.22	401.39
28	0.02	89.54	130.52	40.98	58.27	401.76
33	0.03	89.54	130.52	40.98	58.27	401.78
16	0.05	89.52	130.52	41.01	58.31	402.04
6	0.04	89.50	130.53	41.02	58.33	402.21
20	0.00	89.47	130.52	41.06	58.38	402.51
14	0.00	89.46	130.52	41.06	58.39	402.59
15	0.00	89.45	130.52	41.08	58.41	402.71
12	0.01	89.44	130.53	41.08	58.42	402.78
21	0.00	89.43	130.52	41.09	58.43	402.88
0	0.00	89.42	130.52	41.10	58.44	402.96
19	0.00	89.41	130.52	41.11	58.46	403.06
11	0.00	89.39	130.53	41.14	58.49	403.29
27	0.00	89.38	130.52	41.14	58.50	403.34
32	0.00	89.37	130.52	41.15	58.52	403.47
18	0.00	89.35	130.52	41.17	58.54	403.60
17	0.01	89.34	130.52	41.19	58.57	403.81
31	0.00	89.28	130.52	41.24	58.65	404.35
29	0.00	89.26	130.52	41.26	58.67	404.52
9	0.00	89.25	130.53	41.27	58.69	404.66
4	0.00	89.25	130.53	41.28	58.70	404.70
10	0.00	89.23	130.53	41.29	58.72	404.84
5	0.00	89.23	130.53	41.29	58.72	404.85
8	0.00	89.21	130.53	41.31	58.74	405.02
13	0.00	89.18	130.52	41.34	58.79	405.33
3	0.10	89.17	130.53	41.36	58.81	405.50
1	0.00	89.12	130.53	41.40	58.87	405.90
30	0.00	89.11	130.52	41.41	58.89	406.03
2	0.00	89.11	130.53	41.42	58.89	406.06
24	0.01	89.01	130.52	41.50	59.02	406.91
25	0.02	88.46	130.51	42.05	59.79	412.24

Link Results - Basic Day

ID	FROM	TO	Length (m)	Diameter (mm)	Roughness	Flow (L/s)	Velocity (m/s)
1000	1	0	66.95	204	110	1.924	0.059
1001	2	1	18.12	204	110	1.934	0.059
1002	3	2	3.00	204	110	1.934	0.059
1003	4	3	67.57	204	110	2.034	0.062
1004	5	4	17.29	204	110	-3.102	0.095
1005	6	5	60.80	204	110	-0.040	0.001
1006	Node_43	4	7.80	204	110	5.136	0.157
1007	8	1	33.24	204	110	-0.010	0.000
1008	9	8	5.60	204	110	-0.010	0.000
1009	10	9	10.44	204	110	-0.010	0.000
1010	11	10	3.00	204	110	-0.010	0.000
1011	12	11	2.38	204	110	-0.010	0.000
1012	0	13	86.97	204	110	1.874	0.057
1013	14	0	37.78	204	110	-0.050	0.002
1014	15	14	2.59	204	110	-0.050	0.002
1015	16	15	2.17	204	110	-0.050	0.002
1016	17	5	32.22	204	110	-3.062	0.094
1017	18	17	36.38	204	110	-3.052	0.093
1018	20	19	6.00	204	110	0.000	0.000
1019	21	20	50.62	204	110	0.000	0.000
1020	22	21	13.27	204	110	0.050	0.002
1021	23	22	2.00	204	110	0.050	0.002
1022	18	23	78.18	204	110	0.050	0.002
1023	13	18	18.58	204	110	-3.002	0.092
1024	24	13	12.62	204	110	-4.876	0.149

1025	25	24	47.65	204	110	-4.866	0.149
1027	21	27	26.32	204	110	0.050	0.002
1028	29	28	2.59	204	110	0.020	0.001
1029	30	29	2.00	204	110	0.020	0.001
1030	27	30	20.51	204	110	0.020	0.001
1031	31	27	43.80	204	110	-0.030	0.001
1032	32	31	2.00	204	110	-0.030	0.001
1033	33	32	2.59	204	110	-0.030	0.001
C1	25	C01	28.53	204	110	4.846	0.148

Junction Results - Basic Day

ID	Demand (L/s)	Elevation (m)	Head (m)	Pressure (m)	Pressure (psi)2	Pressure (kPa)
6	0.19	89.50	126.18	36.67	52.15	359.55
23	0.00	89.58	126.27	36.68	52.16	359.65
22	0.00	89.58	126.27	36.69	52.17	359.68
16	0.27	89.52	126.24	36.72	52.22	360.04
28	0.10	89.54	126.27	36.73	52.22	360.06
33	0.17	89.54	126.27	36.73	52.22	360.08
12	0.06	89.44	126.20	36.76	52.27	360.38
14	0.00	89.46	126.24	36.78	52.30	360.59
15	0.00	89.45	126.24	36.79	52.32	360.71
20	0.00	89.47	126.27	36.80	52.33	360.80
11	0.00	89.39	126.20	36.81	52.34	360.89
0	0.00	89.42	126.24	36.82	52.35	360.96
21	0.00	89.43	126.27	36.84	52.38	361.17
19	0.00	89.41	126.27	36.86	52.41	361.35
17	0.07	89.34	126.22	36.88	52.45	361.60
27	0.00	89.38	126.27	36.89	52.45	361.64
32	0.00	89.37	126.27	36.90	52.47	361.76
4	0.00	89.25	126.15	36.90	52.48	361.81
18	0.00	89.35	126.27	36.91	52.49	361.89
5	0.00	89.23	126.18	36.94	52.53	362.19
9	0.00	89.25	126.20	36.95	52.54	362.25
10	0.00	89.23	126.20	36.97	52.57	362.44
8	0.00	89.21	126.20	36.99	52.59	362.62
31	0.00	89.28	126.27	36.99	52.60	362.65
29	0.00	89.26	126.27	37.01	52.62	362.81
3	0.55	89.17	126.19	37.02	52.65	362.98
1	0.00	89.12	126.20	37.08	52.72	363.50
2	0.00	89.11	126.19	37.08	52.73	363.55
13	0.00	89.18	126.29	37.12	52.78	363.88
30	0.00	89.11	126.27	37.16	52.84	364.32
24	0.06	89.01	126.34	37.32	53.07	365.92
25	0.10	88.46	126.50	38.04	54.09	372.97

Link Results - Basic Day

ID	FROM	TO	Length (m)	Diameter (mm)	Roughness	Flow (L/s)	Velocity (m/s)
1000	1	0	66.95	204	110	-8.344	0.255
1001	2	1	18.12	204	110	-8.284	0.253
1002	3	2	3.00	204	110	-8.284	0.253
1003	4	3	67.57	204	110	-7.734	0.237
1004	5	4	17.29	204	110	12.793	0.391
1005	6	5	60.80	204	110	-0.190	0.006
1006	Node_43	4	7.80	204	110	-20.527	0.628
1007	8	1	33.24	204	110	-0.060	0.002
1008	9	8	5.60	204	110	-0.060	0.002
1009	10	9	10.44	204	110	-0.060	0.002
1010	11	10	3.00	204	110	-0.060	0.002
1011	12	11	2.38	204	110	-0.060	0.002
1012	0	13	86.97	204	110	-8.614	0.264
1013	14	0	37.78	204	110	-0.270	0.008
1014	15	14	2.59	204	110	-0.270	0.008
1015	16	15	2.17	204	110	-0.270	0.008
1016	17	5	32.22	204	110	12.983	0.397
1017	18	17	36.38	204	110	13.053	0.399
1018	20	19	6.00	204	110	0.000	0.000
1019	21	20	50.62	204	110	0.000	0.000
1020	22	21	13.27	204	110	0.270	0.008
1021	23	22	2.00	204	110	0.270	0.008
1022	18	23	78.18	204	110	0.270	0.008
1023	13	18	18.58	204	110	13.323	0.408
1024	24	13	12.62	204	110	21.937	0.671

1025	25	24	47.65	204	110	21.997	0.673
1027	21	27	26.32	204	110	0.270	0.008
1028	29	28	2.59	204	110	0.100	0.003
1029	30	29	2.00	204	110	0.100	0.003
1030	27	30	20.51	204	110	0.100	0.003
1031	31	27	43.80	204	110	-0.170	0.005
1032	32	31	2.00	204	110	-0.170	0.005
1033	33	32	2.59	204	110	-0.170	0.005
C1	25	C01	28.53	204	110	-22.097	0.676

Fire Flow Results - Max Day + 100 L/s

ID	Static Demand (L/s)	Static Pressure (m)	Static Pressure (psi)	Static Pressure (kPa)	Static Head (m)	Fire Flow Demand (L/s)	Residual Pressure (m)	Residual Pressure (psi)	Available Flow (L/s)	Available Pressure (psi)
0	0.00	32.14	45.70	315.08	121.56	100.00	29.24	41.58	308.36	20
1	0.00	31.87	45.32	312.49	121.00	100.00	29.52	41.97	315.30	20
2	0.00	31.74	45.13	311.16	120.85	100.00	29.66	42.17	329.40	20
3	0.25	31.66	45.01	310.36	120.82	100.00	29.63	42.13	332.01	20
4	0.00	31.01	44.10	304.06	120.26	100.00	30.64	43.56	887.35	20
5	0.00	31.36	44.60	307.50	120.60	100.00	30.49	43.36	531.24	20
6	0.09	31.09	44.21	304.85	120.60	100.00	26.73	38.01	212.64	20
8	0.00	31.78	45.19	311.60	121.00	100.00	27.52	39.13	221.62	20
9	0.00	31.75	45.14	311.24	121.00	100.00	27.16	38.62	212.36	20
10	0.00	31.77	45.17	311.43	121.00	100.00	26.58	37.80	198.17	20
11	0.00	31.61	44.94	309.88	121.00	100.00	26.25	37.33	193.59	20
12	0.03	31.56	44.87	309.37	121.00	100.00	26.06	37.06	190.55	20
13	0.00	33.11	47.09	324.65	122.29	100.00	30.80	43.79	486.36	20
14	0.00	32.10	45.64	314.70	121.56	100.00	27.03	38.44	211.88	20
15	0.00	32.11	45.66	314.83	121.56	100.00	26.90	38.25	208.03	20
16	0.12	32.04	45.56	314.15	121.56	100.00	26.70	37.97	204.42	20
17	0.03	31.89	45.34	312.61	121.22	100.00	30.29	43.06	431.28	20
18	0.00	32.57	46.32	319.36	121.93	100.00	30.44	43.29	438.96	20
19	0.00	32.52	46.24	318.82	121.93	100.00	21.78	30.96	137.84	20
20	0.00	32.46	46.16	318.27	121.93	100.00	22.06	31.37	140.34	20
21	0.00	32.50	46.22	318.64	121.93	100.00	25.00	35.55	172.63	20
22	0.00	32.35	46.00	317.15	121.93	100.00	25.61	36.42	184.24	20
23	0.00	32.35	45.99	317.11	121.93	100.00	25.72	36.58	186.30	20
24	0.03	33.89	48.19	332.26	122.90	100.00	31.12	44.25	478.51	20
25	0.05	36.76	52.27	360.36	125.21	100.00	34.10	48.49	599.85	20
27	0.00	32.55	46.28	319.11	121.93	100.00	23.54	33.47	153.68	20
28	0.05	32.39	46.05	317.53	121.93	100.00	21.94	31.20	139.61	20
29	0.00	32.67	46.45	320.28	121.93	100.00	22.37	31.81	142.06	20
30	0.00	32.82	46.67	321.79	121.93	100.00	22.64	32.19	143.71	20
31	0.00	32.65	46.43	320.11	121.93	100.00	21.13	30.04	132.69	20
32	0.00	32.56	46.30	319.23	121.93	100.00	20.92	29.75	131.54	20
33	0.08	32.39	46.06	317.55	121.93	100.00	20.60	29.30	129.84	20

Appendix B Wastewater Servicing

B.1 Sanitary Sewer Design Sheet





SUBDIVISION:

Trailsedge East Commercial Block

DATE: 2025-04-30
REVISION: 1
DESIGNED BY: MJS
CHECKED BY: -

SANITARY SEWER
DESIGN SHEET
(City of Ottawa)

FILE NUMBER: 160401884

MAX PEAK FACTOR (RES.)=	4.0	AVG. DAILY FLOW / PERSON	280 l/p/day	MINIMUM VELOCITY	0.60 m/s
MIN PEAK FACTOR (RES.)=	2.0	COMMERCIAL	28,000 l/ha/day	MAXIMUM VELOCITY	3.00 m/s
PEAKING FACTOR (INDUSTRIAL):	2.4	INDUSTRIAL (HEAVY)	55,000 l/ha/day	MANNINGS n	0.013
PEAKING FACTOR (ICI >20%):	1.5	INDUSTRIAL (LIGHT)	35,000 l/ha/day	BEDDING CLASS	B
PERSONS / SINGLE	3.4	INSTITUTIONAL	28,000 l/ha/day	MINIMUM COVER	2.50 m
PERSONS / TOWNHOME	2.7	INFILTRATION	0.33 l/s/ha	HARMON CORRECTION FACTOR	0.8
PERSONS / APARTMENT	1.8				

LOCATION			RESIDENTIAL AREA AND POPULATION										COMMERCIAL		INDUSTRIAL (L)		INDUSTRIAL (H)		INSTITUTIONAL		GREEN / UNUSED		C+I+I	INFILTRATION			TOTAL	PIPE							
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (ha)	SINGLE	UNITS TOWN	APT	POP.	CUMULATIVE AREA (ha)	POP.	PEAK FACT.	PEAK FLOW (l/s)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	PEAK FLOW (l/s)	TOTAL AREA (ha)	ACCU. AREA (ha)	INFILT. FLOW (l/s)	FLOW (l/s)	LENGTH (m)	DIA (mm)	MATERIAL	CLASS	SLOPE (%)	CAP. (FULL) (l/s)	CAP. V PEAK FLOW (%)	VEL. (FULL) (m/s)	
C3A	3A	3	0.00	0	0	0	0	0.00	0	3.80	0.0	0.11	0.11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.1	0.11	0.11	0.0	0.1	6.0	150	PVC	DR 28	1.00	15.3	0.58%	0.86	
G3	3	2	0.00	0	0	0	0	0.00	0	3.80	0.0	0.00	0.11	0.00	0.00	0.00	0.00	0.00	0.00	0.25	0.25	0.1	0.25	0.36	0.1	0.2	37.5	250	PVC	SDR 35	0.30	33.2	0.52%	0.67	
C10A	10A	10	0.00	0	0	0	0	0.00	0	3.80	0.0	0.03	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.03	0.03	0.0	0.0	4.7	150	PVC	DR 28	1.00	15.3	0.18%	0.86	
C10B	10B	10	0.00	0	0	0	0	0.00	0	3.80	0.0	0.02	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.02	0.02	0.0	0.0	13.5	150	PVC	DR 28	1.00	15.3	0.12%	0.86	
G10	10	9	0.00	0	0	0	0	0.00	0	3.80	0.0	0.00	0.06	0.00	0.00	0.00	0.00	0.00	0.00	0.17	0.17	0.0	0.17	0.22	0.1	0.1	41.5	250	PVC	SDR 35	0.30	33.2	0.31%	0.67	
C9A	9A	9	0.00	0	0	0	0	0.00	0	3.80	0.0	0.03	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.03	0.03	0.0	0.0	13.4	150	PVC	DR 28	1.00	15.3	0.17%	0.86	
G9	9	8	0.00	0	0	0	0	0.00	0	3.80	0.0	0.00	0.09	0.00	0.00	0.00	0.00	0.00	0.00	0.71	0.87	0.0	0.71	0.96	0.3	0.3	26.7	250	PVC	SDR 35	0.30	33.2	1.04%	0.67	
C12A	12A	12	0.00	0	0	0	0	0.00	0	3.80	0.0	0.07	0.07	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.07	0.07	0.0	0.1	8.3	150	PVC	DR 28	1.00	15.3	0.40%	0.86	
G12	12	11	0.00	0	0	0	0	0.00	0	3.80	0.0	0.00	0.07	0.00	0.00	0.00	0.00	0.00	0.00	0.08	0.08	0.0	0.08	0.15	0.0	0.1	30.0	250	PVC	SDR 35	0.30	33.2	0.26%	0.67	
C11A	11A	11	0.00	0	0	0	0	0.00	0	3.80	0.0	0.06	0.06	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.06	0.06	0.0	0.0	8.3	150	PVC	DR 28	1.00	15.3	0.30%	0.86	
G11	11	8	0.00	0	0	0	0	0.00	0	3.80	0.0	0.00	0.13	0.00	0.00	0.00	0.00	0.00	0.00	1.02	1.09	0.0	1.02	1.22	0.4	0.4	79.3	250	PVC	SDR 35	0.30	32.9	1.35%	0.66	
	8	7	0.00	0	0	0	0	0.00	0	3.80	0.0	0.00	0.22	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.96	0.1	0.00	2.18	0.7	0.8	37.9	250	PVC	SDR 35	0.30	33.2	2.38%	0.67	
C7A	7A	7	0.00	0	0	0	0	0.00	0	3.80	0.0	0.04	0.04	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.04	0.04	0.0	0.0	13.4	150	PVC	DR 28	1.00	15.3	0.20%	0.86	
G7	7	2	0.00	0	0	0	0	0.00	0	3.80	0.0	0.00	0.26	0.00	0.00	0.00	0.00	0.00	0.00	0.22	2.19	0.1	0.22	2.44	0.8	0.9	36.7	250	PVC	SDR 35	0.30	33.2	2.68%	0.67	
	2	1	0.00	0	0	0	0	0.00	0	3.80	0.0	0.00	0.37	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.44	0.1	0.00	2.80	0.9	1.0	11.4	250	PVC	SDR 35	0.30	33.2	3.14%	0.67	
C6A	6A	6	0.00	0	0	0	0	0.00	0	3.80	0.0	0.15	0.15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.1	0.15	0.15	0.1	0.1	7.5	200	PVC	SDR 35	1.00	33.4	0.37%	1.05	
G6	6	5	0.00	0	0	0	0	0.00	0	3.80	0.0	0.00	0.15	0.00	0.00	0.00	0.00	0.00	0.00	0.47	0.47	0.1	0.47	0.63	0.2	0.3	67.1	250	PVC	SDR 35	0.30	33.2	0.84%	0.67	
C5B	5B	5	0.00	0	0	0	0	0.00	0	3.80	0.0	0.31	0.31	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.1	0.31	0.31	0.1	0.3	10.6	200	PVC	SDR 35	1.00	33.4	0.75%	1.05	
C5A	5A	5	0.00	0	0	0	0	0.00	0	3.80	0.0	0.03	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.03	0.03	0.0	0.0	15.2	150	PVC	DR 28	1.00	15.3	0.17%	0.86	
G5	5	4	0.00	0	0	0	0	0.00	0	3.80	0.0	0.00	0.49	0.00	0.00	0.00	0.00	0.00	0.00	0.29	0.77	0.2	0.29	1.26	0.4	0.7	31.3	250	PVC	SDR 35	0.30	33.2	1.97%	0.67	
G1A	4	1	0.00	0	0	0	0	0.00	0	3.80	0.0	0.00	0.49	0.00	0.00	0.00	0.00	0.00	0.00	0.19	0.96	0.2	0.19	1.45	0.5	0.7	75.8	250	PVC	SDR 35	0.30	33.2	2.16%	0.67	
	1	430	0.00	0	0	0	0	0.00	0	3.80	0.0	0.00	0.86	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.40	0.4	0.00	4.26	1.4	1.8	4.8	250	PVC	SDR 35	0.30	33.2	5.49%	0.67	
																												250							

B.2 Sanitary Excerpts from Background Reports



sewer on Renaud Road. The Renaud Road sanitary sewer ultimately outlets to the Forest Valley Pump Station to the west. The existing sanitary sewer network is shown in **Drawing 7**.

10.4.2 South West Wastewater Design

The MSU (Stantec, July 2006) reviewed the required infrastructure to service the wastewater of the South West quadrant. The South West quadrant is tributary to the Forest Valley Pump Station. The MSU (Stantec, July 2006) considered the South West quadrant to be serviced by a 375 mm diameter trunk sanitary sewer to the south of the quadrant, running westward before ultimately draining into a 600 mm diameter sewer on Renaud Road (formerly named Fourth Line Road). The sanitary collector sewers considered in the MSU (Stantec, July 2006) can be seen in **Appendix D**. A peak sanitary flow of 29.40 L/s was considered within the MSU (Stantec, July 2006). Note that no commercial flows were applied to the quadrant within the MSU (Stantec, July 2006), although the area was contemplated as mixed-use.

More recently the wastewater servicing of the South West quadrant and its surrounding area has been considered within the *Servicing Report for Trails Edge and Orléans Business Park* (DSEL, July 2017). See **Appendix D** for excerpts from the *Servicing Report for Trails Edge and Orléans Business Park* (DSEL, July 2017).

Since the completion of the MSU (Stantec, July 2006), the wastewater servicing of the South West quadrant has been considered during the construction of downstream infrastructure. The *Design Brief – Minto Trailsedge Phase II* (IBI Group, May 2015) includes the wastewater drainage for the portion of the South West quadrant that is east of Fern Casey Street. Per the *Design Brief – Minto Trailsedge Phase II* (IBI Group, May 2015), the downstream wastewater infrastructure anticipated a total peak flow allowance of 45.97 L/s at the South West quadrant's outlet, MH35A. See **Appendix D**, for sanitary drainage information and flow allowance calculations. It can be concluded that the capacity in the constructed downstream infrastructure exceeds the capacity that was considered within the MSU (Stantec, July 2006).

The wastewater servicing for the South West quadrant has also been considered as part of an approved development application neighbouring the South West quadrant. Per the *Trails Edge East – Functional Servicing Report* (Stantec, August 11, 2017), an additional 2.96 ha area drains to the existing MH35A along with the portion of the South West quadrant east of Fern Casey Street. The Trailsedge East lands are undergoing detailed design and construction. Per the *Trails Edge East Phase 1 Servicing and Stormwater Management Report* (Stantec, August 2018) and subsequent reports associated with the development application, the additional area has been refined to a 1.92 ha area with a population of 105 persons draining to the existing MH35A along with the portion of the South West quadrant that is east of Fern Casey Street. Note that the *Trails Edge East Phase 1 Servicing and Stormwater Management Report* (Stantec, August 2018) proposes to accommodate the wastewater flows from the South West quadrant through 2 inlet locations. All wastewater flows from the South West quadrant east of Fern Casey Street are still proposed to be directed towards existing manhole MH35A, despite the multiple inlet locations. See **Appendix D** for details.

The *Design Brief for the Trails Edge West Richcraft Group of Companies* (DSEL, January 26, 2015) includes the wastewater drainage for the portion of the South West quadrant west of Fern Casey Street. A

peak flow allowance of 4.07 L/s at existing MH37A was considered in the constructed wastewater infrastructure for the portion of the South West quadrant that is west of Fern Casey Street. See **Appendix D** for sanitary drainage information and flow allowance calculations.

Respecting the MSU (Stantec, July 2006), existing wastewater infrastructure and development applications within the South East quadrant, the proposed sanitary servicing strategy is to have all flows from the quadrant drain to the Forest Valley Pump Station via the existing 600 mm diameter sanitary sewer on Renaud Road. The proposed sanitary sewer network is shown in **Drawing 5**. A sewer is proposed to cross Brian Coburn Boulevard at an existing gap in underground services – coordination will be required for work in the ROW, including utility coordination.

At this stage of analysis, only the trunk sanitary sewer within the quadrant is shown. To demonstrate servicing feasibility, the trunk sewer is carried back at minimum possible slopes while accounting for drops at manholes, existing infrastructure sizing, and possible conflicts with crossing other sewers. As the design of the South West quadrant advances, the sanitary sewer network details are subject to change; for example, to be raised where appropriate to offer construction cost savings, provided that the conditions in **Section 14** related to minor changes are met. Additional springline connections and/or reduced drops across maintenance holes may be proposed as part of detailed design, to assist in minimizing grade raise requirements, provided that the conditions in **Section 14** related to minor changes are met. These are currently considered deviations from City Standards, and will require review on a case-by-case basis.

As shown in the design sheet included in **Appendix D**, the anticipated peak sanitary flow from the portion of the quadrant east of Fern Casey Street is 38.08 L/s. Including the additional 1.92 ha and population of 105 as per *Trails Edge East Phase 1 Servicing and Stormwater Management Report* (Stantec, August 2018), the total peak sanitary flow to existing MH35A is 39.66 L/s. This represents roughly 86% of the 45.97 L/s allowance reported within the *Design Brief – Minto Trailsedge Phase II* (IBI Group, May 2015). Therefore, the existing downstream wastewater infrastructure is demonstrated to adequately service the portion of the South West quadrant east of Fern Casey Street.

Please note that consistent with the *Trails Edge East – Functional Servicing Report* (Stantec, August 11, 2017), the wastewater flows from the South West quadrant east of Fern Casey Street are shown directed towards one outlet in this MSS for the purpose of high-level serviceability review. Outlets and routing alternatives may be refined as the design process advances, based on the constructed downstream infrastructure (e.g. to direct flows to the constructed two outlets).

The anticipated peak sanitary flow from the portion of the South West quadrant that is west of Fern Casey Street is 7.03 L/s, which is larger than the 4.07 L/s allowance considered in the *Design Brief for the Trails Edge West Richcraft Group of Companies* (DSEL, January 26, 2015). The primary reason for the increase in wastewater flows is a proposed increase of the population from 184 to 532 based on the high-level demand estimate assumptions adopted in this MSS. A check of the downstream sanitary sewers considered in the *Design Brief for the Trails Edge West Richcraft Group of Companies* (DSEL, January 26, 2015) shows that the constraining segment (MH160A – MH17A) still has 22% residual capacity with the addition of the proposed population increase of 348 persons. See **Appendix D** for detailed calculations. Therefore it is concluded that the existing downstream wastewater infrastructure can adequately service

the portion of the South West quadrant that is west of Fern Casey Street. It is expected that capacity will be confirmed as part of detailed design and operational requirements for the downstream Forest Valley Pump Station and Forest Valley Trunk will be assessed by the City of Ottawa for the contributing flows described in this MSS, as part of their City-wide growth assessments, infrastructure management, etc.

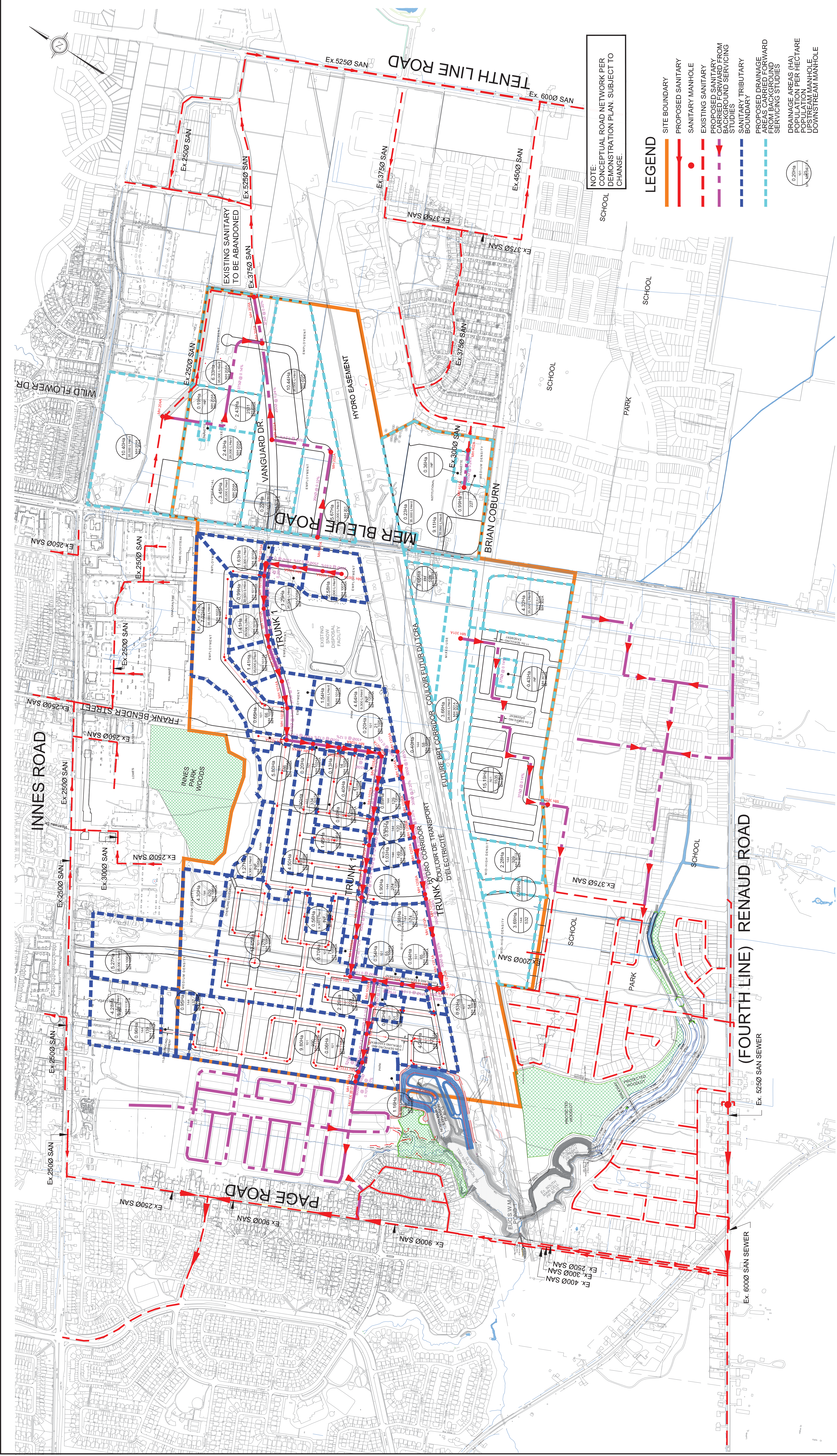
A summary of the wastewater servicing for the South West quadrant is provided in **Table 17**.

Table 17: Summary of Wastewater Servicing – South West Quadrant

	Governing Servicing Study	Additional Background Servicing Study	Proposed MSS
Study Name	Gloucester and Cumberland East Urban Community Expansion Area and Bilberry Creek Industrial Park Master Servicing Update (Stantec, July 2006)	Design Brief for the Trails Edge West Richcraft Group of Companies (DSEL, January 26, 2015) and Design Brief – Minto Trailsedge Phase II (IBI Group, May 2015)	EUC Phase 3 Area CDP MSS (June 2020)
Servicing Strategy	Quadrant to be serviced by the Forest Valley Pump Station and the 600 mm diameter sewer on Renaud Road via the sanitary sewer network to the south west.	Quadrant to be serviced by the Forest Valley Pump Station and the existing 600 mm diameter sewer on Renaud Road via the existing sanitary sewer network to the south west.	Quadrant to be serviced by the Forest Valley Pump Station and the existing 600 mm diameter sewer on Renaud Road via the existing sanitary sewer network to the south west.
Total Drainage Area to Ex MH37A	N/A	3.84 ha.	3.69 ha.
Residential Peak Flow to Ex MH37A	N/A	2.98 L/s	5.81 L/s.
ICI Peak Flow to Ex MH37A	N/A	0 L/s.	0 L/s.
Peak Total Flow to Ex MH37A	N/A	4.07 L/s.	7.03 L/s.
Total Drainage Area to Ex MH35A	N/A	42.80 ha.	29.54 ha. (31.46 ha including future area to the south).
Residential Peak Flow to Ex MH35A	N/A	31.97 L/s	23.41 L/s. (24.36 L/s including future area to the south).
ICI Peak Flow to Ex MH35A	N/A	2.01 L/s.	4.92 L/s.
Peak Total Flow to Ex MH35A	N/A	45.97 L/s.	38.08 L/s. (39.66 L/s including future area to the south).

	SUBDIVISION:		SANITARY SEWER DESIGN SHEET (City of Ottawa) FILE NUMBER: 160401250										DESIGN PARAMETERS																								
	TrailsEdge East																																				
	DATE: 24/7/2019 REVISION: 2 DESIGNED BY: MJS/SG CHECKED BY: DT																																				
			MAX PEAK FACTOR (RES.)= 4.0 MIN PEAK FACTOR (RES.)= 2.0 PEAKING FACTOR (INDUSTRIAL): 2.4 PEAKING FACTOR (ICI >20%): 1.5 PERSONS / SINGLE 3.4 PERSONS / TOWNHOME 2.7 PERSONS / APARTMENT 1.8										AVG. DAILY FLOW / PERSON COMMERCIAL 28,000 l/ha/day INDUSTRIAL (HEAVY) 55,000 l/ha/day INDUSTRIAL (LIGHT) 35,000 l/ha/day INSTITUTIONAL 28,000 l/ha/day INFILTRATION 0.33 l/s/ha					MINIMUM VELOCITY 0.60 m/s MAXIMUM VELOCITY 3.00 m/s MANNINGS n 0.013 BEDDING CLASS B MINIMUM COVER 2.50 m HARMON CORRECTION FACTOR 0.8																			
LOCATION			RESIDENTIAL AREA AND POPULATION										COMMERCIAL		INDUSTRIAL (L)		INDUSTRIAL (H)		INSTITUTIONAL		GREEN / UNUSED		C+H	INFILTRATION			TOTAL	PIPE									
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (ha)	SINGLE	UNITS TOWN	APT	POP.	CUMULATIVE AREA (ha)	POP.	PEAK FACT.	PEAK FLOW (l/s)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	PEAK FLOW (l/s)	TOTAL AREA (ha)	ACCU. AREA (ha)	INFILT. FLOW (l/s)	FLOW (l/s)	LENGTH (m)	DIA (mm)	MATERIAL	CLASS	SLOPE (%)	CAP. (FULL) (l/s)	CAP. V PEAK FLOW (%)	VEL. (FULL) (m/s)	VEL. (ACT.) (m/s)				
R106B	106	105	0.28	0	12	0	32	0.28	32	3.68	0.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.28	0.28	0.1	0.5	85.6	200	PVC	SDR 35	0.50	23.6	2.03%	0.74	0.24				
R105A	105	104	0.03	0	0	0	0	0.31	32	3.68	0.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.03	0.31	0.1	0.5	10.5	200	PVC	SDR 35	0.50	23.6	2.07%	0.74	0.26				
FUTURE MUC2	Stub	104	29.23	0	0	0	1389	29.23	1389	3.16	14.2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	29.23	29.23	9.6	23.9	20.4	375	PVC	SDR 35	0.15	62.9	37.99%	0.60	0.47				
	104	103	0.00	0	0	0	0	29.54	1421	3.16	14.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00	29.54	9.7	24.3	41.8	375	PVC	SDR 35	0.15	62.9	38.64%	0.60	0.47				
R107B	107	103	0.41	0	12	0	32	0.41	32	3.68	0.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.41	0.41	0.1	0.5	93.6	200	PVC	SDR 35	0.50	23.6	2.21%	0.74	0.26				
FUTURE MUC 1	STUB	101	3.11	0	0	0	152	3.11	152	3.55	1.7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	3.11	3.11	1.0	2.8	46.3	200	PVC	SDR 35	0.50	23.6	11.74%	0.74	0.42				
R103A	103	102	0.51	0	10	0	27	30.47	1481	3.15	15.1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.51	30.47	10.1	25.2	80.5	375	PVC	SDR 35	0.15	62.9	40.02%	0.60	0.47				
R102A	102	101	0.23	0	5	0	14	33.80	1646	3.12	16.6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.23	33.80	11.2	27.8	47.0	375	PVC	SDR 35	0.15	62.9	44.23%	0.60	0.49				
R101A	101	100	0.46	0	0	0	0	34.26	1646	3.12	16.6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.46	34.26	11.3	28.0	105.0	375	PVC	SDR 35	0.24	79.5	35.15%	0.75	0.58				

	SUBDIVISION:		SANITARY SEWER DESIGN SHEET (City of Ottawa)								DESIGN PARAMETERS																							
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	DATE: 24/7/2019 REVISION: 2 DESIGNED BY: MJS/SG CHECKED BY: DT																																	
FILE NUMBER: 160401250			MAX PEAK FACTOR (RES.)= 4.0 MIN PEAK FACTOR (RES.)= 2.0 PEAKING FACTOR (INDUSTRIAL): 2.4 PEAKING FACTOR (ICI >20%): 1.5 PERSONS / SINGLE 3.4 PERSONS / TOWNHOME 2.7 PERSONS / APARTMENT 1.8 AVG. DAILY FLOW / PERSON 280 l/p/day COMMERCIAL 28,000 l/ha/day INDUSTRIAL (HEAVY) 55,000 l/ha/day INDUSTRIAL (LIGHT) 35,000 l/ha/day INSTITUTIONAL 28,000 l/ha/day INFILTRATION 0.33 l/s/ha MINIMUM VELOCITY 0.60 m/s MAXIMUM VELOCITY 3.00 m/s MANNINGS n 0.013 BEDDING CLASS B MINIMUM COVER 2.50 m HARMON CORRECTION FACTOR 0.8																															
LOCATION			RESIDENTIAL AREA AND POPULATION								COMMERCIAL		INDUSTRIAL (L)		INDUSTRIAL (H)		INSTITUTIONAL		GREEN / UNUSED		C+H	INFILTRATION			TOTAL	PIPE								
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (ha)	SINGLE	UNITS TOWN	APT	POP.	CUMULATIVE AREA POP.	PEAK FACT.	PEAK FLOW (l/s)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	PEAK FLOW (l/s)	TOTAL AREA (ha)	ACCU. AREA (ha)	INFILT. FLOW (l/s)	FLOW (l/s)	LENGTH (m)	DIA (mm)	MATERIAL	CLASS	SLOPE (%)	CAP. (FULL) (l/s)	CAP. V PEAK FLOW (%)	VEL. (FULL) (m/s)	VEL. (ACT.) (m/s)
RENAUD, L237A	237	236	5.19	4	0	0	245	5.19 245	3.49	2.8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	5.19	5.19	1.7	4.5	54.7	200	PVC	SDR 35	0.35	19.8	22.65%	0.62	0.42
R236A	236	235	0.24	5	0	0	17	5.43 262	3.48	3.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.24	5.43	1.8	4.7	26.9	200	PVC	SDR 35	0.35	19.8	23.98%	0.62	0.42
R235A	235	206	0.86	17	0	0	58	6.29 319	3.45	3.6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.86	6.29	2.1	5.6	120.0	200	PVC	SDR 35	0.35	19.8	28.56%	0.62	0.45
Phase 2	Stub	218	3.84	38	47	0	256	3.84 256	3.49	2.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	3.84	3.84	1.3	4.2	43.4	200	PVC	SDR 35	0.40	21.1	19.67%	0.67	0.43
R107A	107	225	0.43	0	14	0	38	0.43 38	3.67	0.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.43	0.43	0.1	0.6	120.0	200	PVC	SDR 35	0.50	23.6	2.50%	0.74	0.27
R227A	227	226	0.14	0	5	0	14	0.14 14	3.72	0.2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.14	0.14	0.0	0.2	18.5	200	PVC	SDR 35	1.00	33.4	0.62%	1.05	0.26
R106A	106	226	0.36	0	14	0	38	0.36 38	3.67	0.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.36	0.36	0.1	0.6	120.0	200	PVC	SDR 35	0.50	23.6	2.41%	0.74	0.26
R226A	226	225	0.18	0	6	0	16	0.68 68	3.63	0.8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.18	0.68	0.2	1.0	49.3	200	PVC	SDR 35	0.50	23.6	4.31%	0.74	0.31
R225A	225	224	0.42	0	14	0	38	1.53 143	3.56	1.7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.42	1.53	0.5	2.2	110.5	200	PVC	SDR 35	0.75	29.0	7.44%	0.91	0.44
Phases 2, 3	Stub	224	10.13	13	279	0	798	10.13 798	3.29	8.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	10.13	10.13	3.3	11.8	17.8	200	PVC	SDR 35	0.50	23.6	50.09%	0.74	0.64
G224A	Stub	224	0.00	0	0	0	0	0.00 0	3.80	0.0	0.00	0.00	0.00	0.00	0.00	0.00	2.22	2.22	0.0	0.0	0.0	2.22	2.22	0.7	0.7	18.1	200	PVC	SDR 35	0.50	23.6	3.10%	0.74	0.28
R224A	224	218	0.35	5	0	0	17	12.01 958	3.25	10.1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.22	0.0	0.0	0.0	0.35	14.23	4.7	14.8	64.8	200	PVC	SDR 35	0.50	23.6	62.52%	0.74	0.68
R218A	218	206	0.16	0	0	0	0	16.01 1214	3.20	12.6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.22	0.0	0.0	0.0	0.16	18.23	6.0	18.6	79.0	250	PVC	SDR 35	0.50	42.9	43.35%	0.86	0.71
Phase 2	207	206	7.16	0	0	0	429	7.16 429	3.41	4.7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	7.16	7.16	2.4	7.1	43.6	200	PVC	SDR 35	0.40	21.1	33.57%	0.67	0.51	
R206A	206	205	0.53	11	0	0	37	29.99 2000	3.07	19.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.22	0.0	0.0	0.0	0.53	32.21	10.6	30.5	81.0	300	PVC	SDR 35	0.30	52.6	58.02%	0.75	0.67
R232B	232	233	0.17	1	0	0	3	0.17 3	3.76	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.17	0.17	0.1	0.1	9.5	200	PVC	SDR 35	0.65	27.0	0.37%	0.85	0.16	
R233A	233	234	0.49	9	0	0	31	0.67 34	3.68	0.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.49	0.67	0.2	0.6	68.0	200	PVC	SDR 35	0.50	23.6	2.65%	0.74	0.27	
R234A	234	205	0.25	4	0	0	14	0.92 48	3.66	0.6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.25	0.92	0.3	0.9	58.0	200	PVC	SDR 35	1.05	34.3	2.53%	1.08	0.39	
R205A	205	204	0.55	11	0	0	37	31.46 2085	3.06	20.7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.22	0.0	0.0	0.0	0.55	33.68	11.1	31.8	79.0	300	PVC	SDR 35	0.30	52.6	60.41%	0.75	0.68
R232A	232	231	0.33	5	0	0	17	0.33 17	3.71	0.2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.33	0.33	0.1	0.3	64.4	200	PVC	SDR 35	0.65	27.0	1.17%	0.85	0.24	
R231A	231	230	0.23	1	0	0	3	0.56 20	3.70	0.2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.23	0.56	0.2	0.4	11.8	200	PVC	SDR 35	0.65	27.0	1.60%	0.85	0.26	
R230A	230	229	0.55	10	0	0	34	1.11 54	3.65	0.6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.55	1.11	0.4	1.0	67.3	200	PVC	SDR 35	0.50	23.6	4.27%	0.74	0.31	
R229A	229	204	0.37	6	0	0	20	1.49 75	3.62	0.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.37	1.49	0.5	1.4	67.3	200	PVC	SDR 35	1.90	46.1	2.97%	1.45	0.55	
R204A	204	203	0.43	4	8	0	35	33.37 2195	3.04	21.6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.22	0.0	0.0	0.0	0.43	35.59	11.7	33.4	68.9	300	PVC	SDR 35	0.30	52.6	63.49%	0.75	0.69
R203A	203	202	0.40	6	4	0	31	33.77 2226	3.04	21.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.22	0.0	0.0	0.0	0.40	35.99	11.9	33.8	65.8	300	PVC	SDR 35	0.30	52.6	64.27%	0.75	0.69
R213A	213	212	0.52	0	17	0	46	0.52 46	3.66	0.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.52	0.52	0.2	0.7	114.3	200	PVC	SDR 35	0.50	23.6	3.03%	0.74	0.28	
R212A	212	211	0.18	0	5	0	14	0.70 59	3.64	0.7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.18	0.70	0.2	0.9	34.1	200	PVC	SDR 35	0.50	23.6	3.94%	0.74	0.31	
R211A	211	210	0.09	0	2	0	5	0.79 65	3.63	0.8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.09	0.79	0.3	1.0	12.6	200	PVC	SDR 35	0.40	21.1	4.84%	0.67	0.28	
R210A	210	209	0.35	0	1																													





The Contractor shall verify and be responsible for all dimensions. Do NOT scale the drawing - any errors or omissions shall be reported to Stantec without delay.

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Diagram of a circular manhole with the following labels:

- PROPOSED SANITARY SEWER
- SANITARY DRAINAGE AREA ID
- POPULATION
- SANITARY DRAINAGE AREA ho
- SANITARY DRAINAGE AREA ho

8	ISSUED FOR APPROVAL	AJ	DT	19.09.18
7	REVISED ASCENDER OFFSITE SERVICING	AJ	DT/SG	19.05.17
6	ISSUED FOR GRADING APPROVAL	AJ	DT	19.03.11
5	ISSUED FOR CONSTRUCTION	MJS	GR	18.10.25
4	REVISED FIRE HYDRANT LOCATIONS	MJS	DT	18.10.24
3	REVISED AS PER CITY COMMENTS	MJS	DT	18.09.26
2	REVISED AS PER CITY COMMENTS AND DRAFT PLAN	MJS	DT	18.08.23
1	REVISED AS PER CITY COMMENTS	MJS	DT	18.05.30
0	ISSUED TO CITY FOR REVIEW	MJS	DT	18.03.06

Revision		By	Appd.	YY.MM.DD
File Name: 160401250 SA.DWG	JP	MJS	JP	18.02.14
	Drawn	Chkd	Desn	YY.MM.DD

Client/Project

RICHCRAFT GROUP OF COMPANIES

2280 ST. LAURENT BLVD
OTTAWA, ON, K1G 4K1

TRAILSEDGE EAST SUBDIVISION

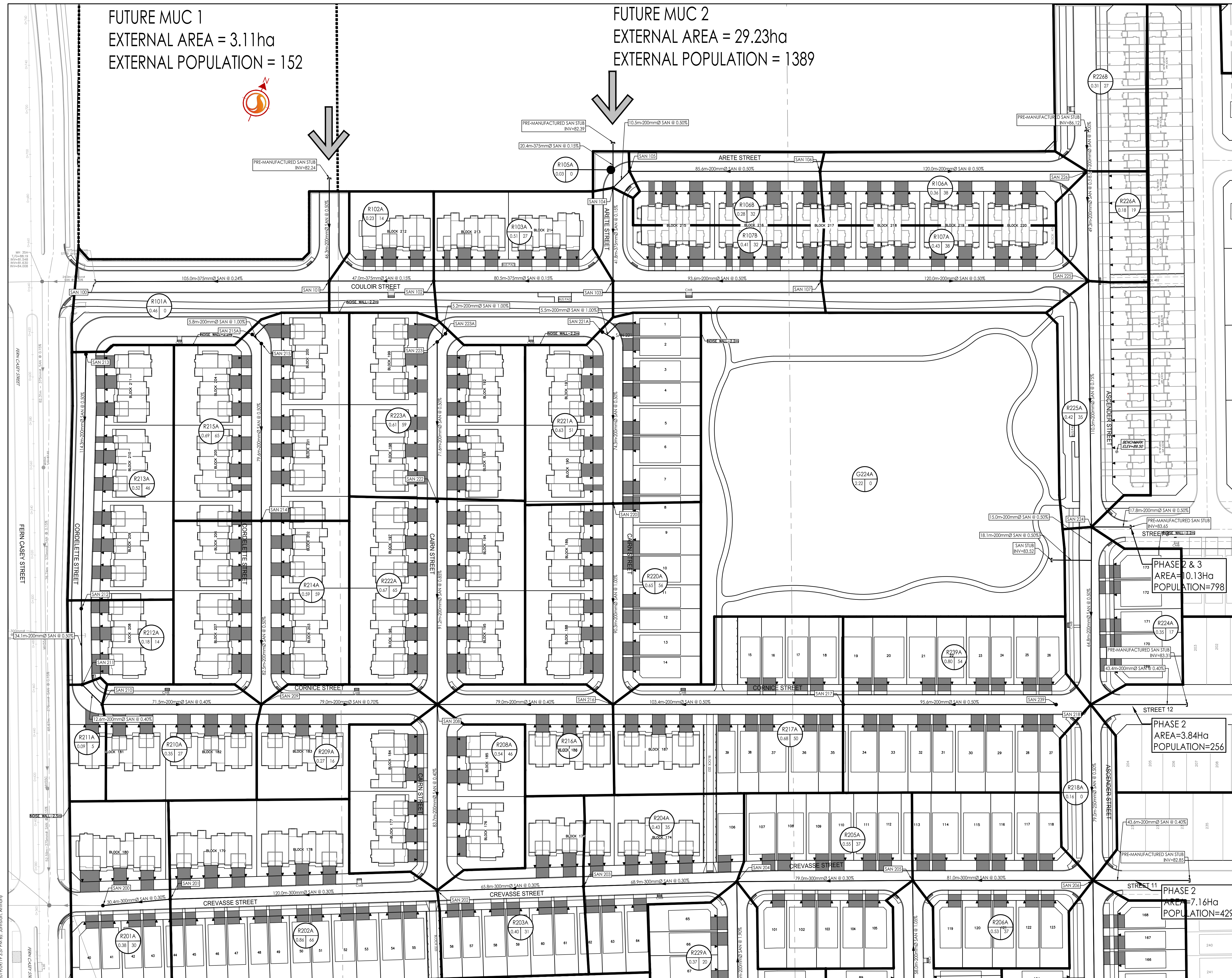
OTTAWA, ON, CANADA

Title
SANITARY DRAINAGE PLAN

Project No. 160401250	Scale 0 7.5 22.5 37.5 1:750
Drawing No.	Sheet Revision

SA-1 42 of 46 8

DWG# 17807



4 Wastewater Servicing

4.1 Background

As indicated in the Master Servicing Study (MSS) for EUC Phase 3 Area CDP, wastewater servicing for the Trailsedge East Development is conveyed to the Forest Valley Trunk Sewer (FVT) via a free flow gravity trunk running along Renaud Road to the Forest Valley Pumping Station. The MSS outlines the sanitary servicing requirements for the subject property, which identify an integrated network within Minto and further along Richcraft lands to the west, eventually connecting to the newly constructed 600 mm trunk sewer extension along Renaud Road recommended by the MSS. The Trailsedge East Phase 1 – Servicing and Stormwater Management Report in addition to the Trailsedge East Phase 4 Functional Servicing Report further detailed contributing flows from Phase 4 through sewers Arete Way. The Sanitary Drainage Plan and sanitary sewer design sheet for the existing Trailsedge East Phase 1 development is included in **Appendix B**.

The Design Brief – Minto Trailsedge Phase II report (IBI Group, 2015) identifies an external contribution to their subdivision based on a future population of 4,212, which includes drainage from the entirety of the Trailsedge East development Phases 1-4, as well as future lands forming a mixed-use community (MUC) identified within the EUC MSS.

4.2 Design Criteria

As outlined in the City's Sewer Design Guidelines, the following design parameters were used to calculate estimated wastewater flow rates and to preliminarily size on-site sanitary sewers:

- Minimum Full Flow Velocity – 0.6 m/s
- Maximum Full Flow Velocity – 3.0 m/s
- Manning's roughness coefficient for all smooth walled pipes – 0.013
- Residential density for single detached unit – 3.4 ppu
- Residential density for townhouse unit – 2.7 ppu
- Residential density for apartment unit – 1.8 ppu
- Extraneous Flow Allowance – 0.33 L/s/ha
- Residential Average Flows – 280 L/cap/day
- Commercial/Mixed Use Flows – 28,000 L/ha/day
- Harmon Correction Factor – 0.8
- Maintenance hole Spacing – 120 m



- Minimum Cover – 2.5 m

In addition, a residential peak factor based on Harmon's Equation was used to determine the peak design flows. Institutional and commercial areas were assigned a peaking factor of 1.5 where commercial areas contribute greater than 20% of the total tributary area to each sanitary sewer per Ottawa's Sewer Design Guidelines.

4.3 Proposed Servicing

Phase 1 of the Trailsedge East Subdivision is currently serviced by a network of gravity sewers which direct wastewater flows westerly to the trunk sewer within Fern Casey Street within the adjacent Minto development. Sanitary sewers within Phase 4 of the Trails Edge East Subdivision are to flow westerly and connect to the sanitary sewer network established in Phase 1. Flows from mixed use lands to the north will be conveyed through Phase 4 sewers crossing Brian Coburn Boulevard under direction of the MSS. The proposed sanitary sewer design sheet can be found in **Appendix B**, while the associated Sanitary Area Drainage Plan and sanitary servicing are shown in **Drawings SSP-1 to SSP-4 and SA-1 to SA-2**. The proposed sanitary sewer design indicates one connection point to the recently constructed sanitary sewers within Phase 1 on Arete Way.

The tables below do not consider additional flows from 8 single units (Lots 1 to 8 fronting Ascender) and 32 townhomes (Blocks 53 to 57 and 64) within Phase 4 that discharge directly to previously constructed sewers within Phase 1, as well as units along Randkluff Terrace, and whose flows have been previously included within detailed design reports for Phases 1-3 and Trailsedge Blocks 193/194 respectively.

The connection points and associated flows are summarized in **Table 4.1** below. Allocated flows for the available connection points based on the Phase 4 Functional Servicing Report are noted in **Table 4.2**.

Table 4-1: Wastewater Connections to Existing Network

MH ID	Residential Area (ha)	Commercial Area (ha)	Residential Population	Total Flow (L/s)	Sewer Dia. (mm)
104	13.61	11.30	2118 ¹	34.9	375

1. Population for Mixed Use block 151/152 per *EUC Phase 3 MSS*. Remainder of sanitary contributions per unit configuration as shown on current draft plan.

Table 4-2: Wastewater Connections per Trailsedge Phase 4 FSR

MH ID	Total Area (ha)	Commercial Area (ha)	Residential Population	Total Flow (L/s)	Sewer Dia. (mm)
104	14.31*	11.29	2102	34.8	375

*Includes 0.69 ha area R400B on Functional Servicing Report drawings now attributed to design of Block 193/194.

As demonstrated in the tables above, peak flows from the proposed development can be accommodated within the downstream sewer network.



Stantec	SUBDIVISION:		SANITARY SEWER DESIGN SHEET (City of Ottawa)										DESIGN PARAMETERS																																									
	Trailseige East Phase 4																																																					
	DATE: 2024-12-04 REVISION: 1 DESIGNED BY: MW CHECKED BY: -																																																					
FILE NUMBER: 160401473		MAX PEAK FACTOR (RES.)= 4.0 MIN PEAK FACTOR (RES.)= 2.0 PEAKING FACTOR (INDUSTRIAL): 2.4 PEAKING FACTOR (ICI >20%): 1.5 PERSONS / SINGLE: 3.4 PERSONS / TOWNHOME: 2.7 PERSONS / APARTMENT: 1.8 AVG. DAILY FLOW / PERSON: 280 l/p/day COMMERCIAL: 28,000 l/ha/day INDUSTRIAL (HEAVY): 55,000 l/ha/day INDUSTRIAL (LIGHT): 35,000 l/ha/day INSTITUTIONAL: 28,000 l/ha/day INFILTRATION: 0.33 l/s/ha MINIMUM VELOCITY: 0.60 m/s MAXIMUM VELOCITY: 3.00 m/s MANNINGS n: 0.013 BEDDING CLASS: B MINIMUM COVER: 2.50 m HARMON CORRECTION FACTOR: 0.8																																																				
LOCATION			RESIDENTIAL AREA AND POPULATION										COMMERCIAL		INDUSTRIAL (L)		INDUSTRIAL (H)		INSTITUTIONAL		GREEN / UNUSED		C+H	INFILTRATION			TOTAL	PIPE																										
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (ha)	SINGLE	UNITS TOWN	APT	POP.	CUMULATIVE AREA (ha)	POP.	PEAK FACT.	PEAK FLOW (l/s)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	PEAK FLOW (l/s)	TOTAL AREA (ha)	ACCU. AREA (ha)	INFILT. FLOW (l/s)	FLOW (l/s)	LENGTH (m)	DIA (mm)	MATERIAL	CLASS	SLOPE (%)	CAP. (FULL) (l/s)	CAP. V PEAK FLOW (%)	VEL. (FULL) (m/s)	VEL. (ACT.) (m/s)																			
R419A, R419B	419	418	0.05	0	1	0	3	0.05	3	3.76	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.04	0.0	0.10	0.10	0.0	0.1	7.5	200	PVC	SDR 35	1.00	33.4	0.20%	1.05	0.17																			
R418A	418	412	0.63	0	27	0	73	0.68	76	3.62	0.9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.0	0.63	0.73	0.2	1.1	107.0	200	PVC	SDR 35	0.40	21.1	5.33%	0.67	0.29																			
R421A	421	420	0.06	0	1	0	3	0.06	3	3.76	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.06	0.06	0.0	0.1	7.8	200	PVC	SDR 35	1.00	33.4	0.16%	1.05	0.17																			
R443A	443	422	1.01	1	32	0	90	1.01	90	3.60	1.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	1.01	1.01	0.3	1.4	120.6	200	PVC	SDR 35	1.50	41.0	3.37%	1.29	0.51																			
R430A, R430B	430	429	0.00	0	0	0	0	0.00	0	3.80	0.0	4.26	4.26	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.05	2.1	4.30	4.30	1.4	3.5	42.8	250	PVC	SDR 35	0.30	33.2	10.50%	0.67	0.36																			
R441B	441	440	0.23	0	10	0	27	0.23	27	3.69	0.3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.23	0.23	0.1	0.4	78.3	200	PVC	SDR 35	1.50	41.0	0.97%	1.29	0.34																			
R440A	440	439	0.05	0	1	0	3	0.28	30	3.68	0.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.05	0.28	0.1	0.4	7.2	200	PVC	SDR 35	0.40	21.1	2.11%	0.67	0.23																			
R31AA	431	439	7.04	0	0	564	1014	7.04	1014	3.24	10.6	7.04	7.04	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.4	7.04	7.04	2.3	16.4	57.6	375	PVC	SDR 35	0.15	62.9	26.07%	0.60	0.42																			
R439A	439	438	0.22	0	7	0	19	7.54	1063	3.23	11.1	0.00	7.04	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.4	0.22	7.54	2.5	17.0	44.4	375	PVC	SDR 35	0.15	62.9	27.09%	0.60	0.42																			
R437B	437	438	0.26	0	10	0	27	0.26	27	3.69	0.3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.26	0.26	0.1	0.4	81.3	200	PVC	SDR 35	1.50	41.0	0.99%	1.29	0.34																			
R438A	438	429	0.43	4	8	0	35	8.23	1125	3.21	11.7	0.00	7.04	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	3.4	0.43	8.23	2.7	17.9	62.4	375	PVC	SDR 35	0.15	62.9	28.41%	0.60	0.43																			
	429	428	0.00	0	0	0	0	8.23	1125	3.21	11.7	0.00	11.30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.05	5.5	0.00	12.53	4.1	21.3	7.3	375	PVC	SDR 35	0.15	62.9	33.95%	0.60	0.45																			
R436B	436	428	0.36	5	0	0	17	0.36	17	3.71	0.2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.36	0.36	0.1	0.3	78.5	200	PVC	SDR 35	0.65	27.0	1.20%	0.85	0.24																			
R428A	428	427	0.45	0	13	0	35	9.03	1177	3.20	12.2	0.00	11.30	0.00	0.00	0.00	0.00	0.00	0.00	0.05	5.5	0.45	13.33	4.4	22.1	73.7	375	PVC	SDR 35	0.15	62.9	35.18%	0.60	0.46																				
R427A	427	426	0.09	0	2	0	5	9.13	1183	3.20	12.3	0.00	11.30	0.00	0.00	0.00	0.00	0.00	0.00	0.05	5.5	0.09	13.43	4.4	22.2	7.4	375	PVC	SDR 35	0.15	62.9	35.31%	0.60	0.46																				
R426A	426	425	0.29	3	3	0	18	9.42	1201	3.20	12.4	0.00	11.30	0.00	0.00	0.00	0.00	0.00	0.00	0.05	5.5	0.29	13.72	4.5	22.5	37.8	375	PVC	SDR 35	0.15	62.9	35.74%	0.60	0.46																				
R425A	425	424	0.92	11	17	0	83	10.34	1284	3.18	13.2	0.00	11.30	0.00	0.00	0.00	0.00	0.00	0.00	0.05	5.5	0.92	14.64	4.8	23.6	120.0	375	PVC	SDR 35	0.15	62.9	37.49%	0.60	0.47																				
R437A	437	433	0.26	0	10	0	27	0.26	27	3.69	0.3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.26	0.26	0.1	0.4	81.8	200	PVC	SDR 35	0.65	26.9	1.51%	0.85	0.26																			
R441A	441	435	0.22	0	10	0	27	0.22	27	3.69	0.3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.22	0.22	0.1	0.4	75.3	200	PVC	SDR 35	0.65	26.9	1.47%	0.85	0.26																			
R435A	435	434	0.05	1	0	0	3	0.27	30	3.68	0.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.05	0.27	0.1	0.5	7.0	200	PVC	SDR 35	1.00	33.4	1.35%	1.05	0.30																			
R434A	434	433	0.21	4	0	0	14	0.48	44	3.66	0.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.21	0.48	0.2	0.7	44.7	200	PVC	SDR 35	0.40	21.1	3.22%	0.67	0.25																			
R433A	433	432	0.45	9	0	0	31	1.18	102	3.59	1.2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.45	1.18	0.4	1.6	64.3	200	PVC	SDR 35	0.41	21.4	7.35%	0.67	0.32																			
G436AA	436A	436	0.00	0	0	0	0	0.00	0	3.80	0.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.41	0.41	0.1	0.1	11.0	150	PVC	DR 28	1.00	15.3	0.89%	0.86	0.23																			
R436A	436	432	0.36	5	0	0	17	0.36	17	3.71	0.2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.36	0.78	0.3	0.5	84.5	200	PVC	SDR 35	0.65	27.0	1.71%	0.85	0.28																			
R432A	432	424	0.25	4	0	0	14	1.79	132	3.57	1.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.41	0.0	0.25	2.20	0.7	2.3	79.0	200	PVC	SDR 35	1.05	34.3	6.58%	1.08	0.51																				
R424A	424	423	0.39	5	3	0	25	12.52	1442	3.15	14.7	0.00	11.30	0.00	0.00	0.00	0.00	0.00	0.00	0.46	5.5	0.39	17.23	5.7	25.9	79.7	375	PVC	SDR 35	0.15	62.9	41.22%	0.60	0.48																				
R423A	423	422	0.47	9	0	0	31	12.98	1472	3.15	15.0	0.00	11.30	0.00	0.00	0.00	0.00	0.00	0.00	0.46	5.5	0.47	17.70	5.8	26.4	73.8	375	PVC	SDR 35	0.15	62.9	41.92%	0.60	0.48																				
R422A	422	411	0.00	0	0	0	0	13.99	1562	3.13	15.9	0.00	11.30	0.00	0.00	0.00	0.00	0.00	0.00	0.58	5.5	0.12	18.83	6.2	27.6	84.3	375	PVC	SDR 35	0.15	63.0	43.79%	0.60	0.49																				
R417A	417	416	0.67	0	20	0	54	0.67	54	3.65	0.6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.67	0.67	0.2	0.9	95																											

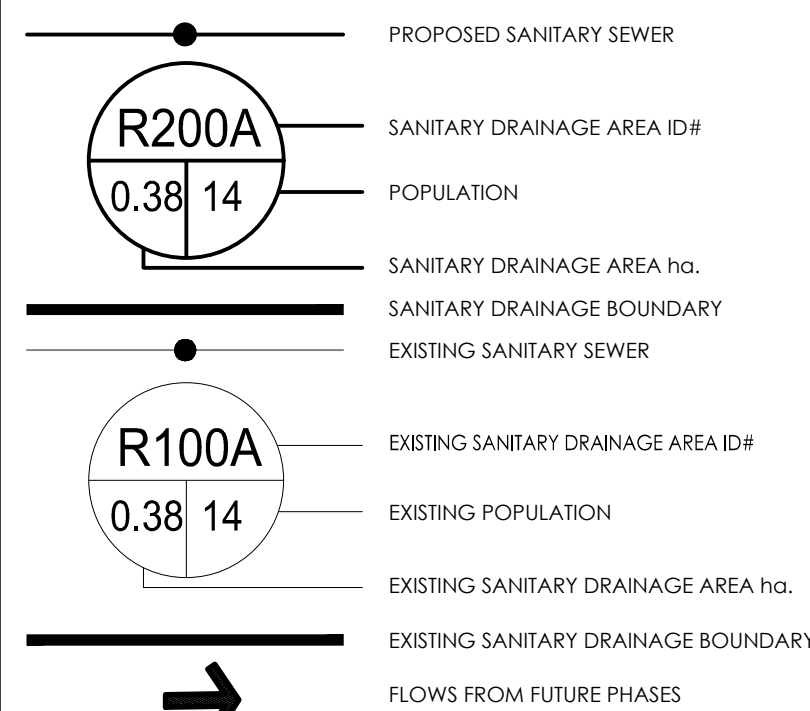


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KEY PLANTS

2	ISSUED FOR CITY REVIEW	JP	DT	25.03.18
1	ISSUED FOR CITY REVIEW	JP	DT	24.12.20
Revision		By	Appd.	YY.MM.DD
File Name: 16401473-TEE-PH 4-5A.dwg		JP	DT/SGG	JP
		Dwn.	Chkd.	Dsgn.
				YY.MM.DD

Permit-Seal

Client/Project

RICH CRAFT GROUP OF COMPANIES
 2280 ST. LAURENT BLVD
 OTTAWA, ON, K1G 4K1
 TRAILS EDGE EAST SUBDIVISION
 PHASE 4

Title

SANITARY DRAINAGE PLAN

Project No. 160401473

Drawing No. Sheet Revision

Scale 0 7.5 22.5 37.5m 1:750

SA-2

34 of 36

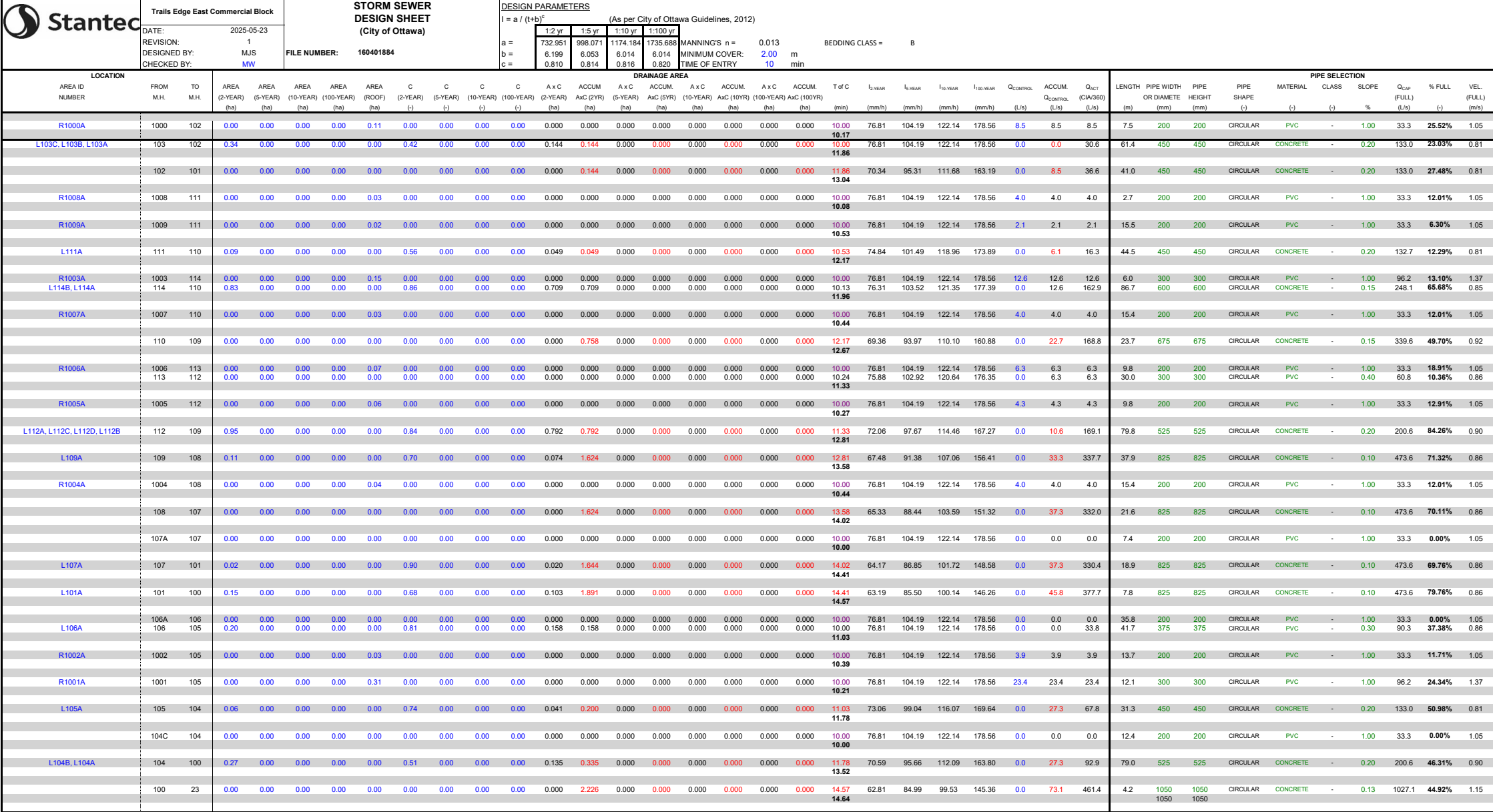
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DWG# XXXXX

Appendix C Stormwater Management

C.1 Storm Sewer Design Sheet





C.2 MRM Sheet



Roof Drain Design Calculation Sheet

Project #160401884, Richcraft Mer Bleue Road - Commercial Development Roof Drain Design Sheet, Area R1003 Standard Zurn Model Z-105-5 Control-Flo Single Notch Roof Drain

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0004	0.0023	0	0.025	34	0	0	0.025
0.050	0.0008	0.0046	2	0.050	135	2	2	0.050
0.075	0.0012	0.0069	8	0.075	304	5	8	0.075
0.100	0.0015	0.0092	18	0.100	540	10	18	0.100
0.125	0.0019	0.0115	35	0.125	844	17	35	0.125
0.150	0.0023	0.0138	61	0.150	1216	26	61	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
2.0	427.6	2.0	0.11878
7.3	773.7	5.3	0.33371
17.7	1130.1	10.4	0.64762
34.9	1490.5	17.2	1.06164
60.5	1852.9	25.6	1.57634

Rooftop Storage Summary

Total Building Area (sq.m)	1519.999951	
Assume Available Roof Area (sq. 80%)	1215.999961	
Roof Imperviousness	0.99	
Roof Drain Requirement (sq.m/Notch)	232	
Number of Roof Notches*	6	
Max. Allowable Depth of Roof Ponding (m)	0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)	61	
Estimated 100 Year Drawdown Time (h)	1.3	

From Zurn Drain Catalogue

Head (m)	L/min	L/s	Notch Rating
0.051	45.5	0.00076	232

* Note: Number of drains can be reduced if multiple-notch drain used.

Calculation Results	5yr	100yr	Available
Qresult (cu.m/s)	0.008	0.013	-
Depth (m)	0.090	0.137	0.150
Volume (cu.m)	13.8	47.2	60.8
Drain time (hrs)	0.5	1.3	

Roof Drain Design Calculation Sheet

Project #160401884, Richcraft Mer Bleue Road - Commercial Development Roof Drain Design Sheet, Area R1002 Standard Zurn Model Z-105-5 Control-Flo Single Notch Roof Drain

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0004	0.0008	0	0.025	7	0	0	0.025
0.050	0.0008	0.0015	0	0.050	28	0	0	0.050
0.075	0.0012	0.0023	2	0.075	64	1	2	0.075
0.100	0.0015	0.0031	4	0.100	114	2	4	0.100
0.125	0.0019	0.0038	7	0.125	178	4	7	0.125
0.150	0.0023	0.0046	13	0.150	256	5	13	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
0.4	270.1	0.4	0.07502
1.5	488.7	1.1	0.21076
3.7	713.7	2.2	0.40902
7.3	941.4	3.6	0.67051
12.7	1170.3	5.4	0.99558

Roof Storage Summary

Total Building Area (sq.m)	320.0000152	
Assume Available Roof Area (sq. 80%)	256.0000122	
Roof Imperviousness	0.99	
Roof Drain Requirement (sq.m/Notch)	232	
Number of Roof Notches*	2	
Max. Allowable Depth of Roof Ponding (m)	0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)	13	
Estimated 100 Year Drawdown Time (h)	0.7	

* Note: Number of drains can be reduced if multiple-notch drain used.

From Zurn Drain Catalogue

Head (m)	L/min	L/s	Notch Rating
0.051	45.5	0.00076	232

Calculation Results

	5yr	100yr	Available
Qresult (cu.m/s)	0.003	0.004	-
Depth (m)	0.082	0.128	0.150
Volume (cu.m)	2.2	8.1	12.8
Drain time (hrs)	0.3	0.7	

Roof Drain Design Calculation Sheet

Project #160401884, Richcraft Mer Bleue Road - Commercial Development Roof Drain Design Sheet, Area R1001 Standard Zurn Model Z-105-5 Control-Flo Single Notch Roof Drain

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0004	0.0042	1	0.025	69	1	1	0.025
0.050	0.0008	0.0084	5	0.050	275	4	5	0.050
0.075	0.0012	0.0127	15	0.075	618	11	15	0.075
0.100	0.0015	0.0169	37	0.100	1099	21	37	0.100
0.125	0.0019	0.0211	72	0.125	1717	35	72	0.125
0.150	0.0023	0.0253	124	0.150	2472	52	124	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
4.0	474.1	4.0	0.13171
14.9	858.0	10.9	0.37003
36.0	1253.1	21.2	0.71811
71.0	1652.7	34.9	1.1772
123.0	2054.6	52.1	1.74793

Roof Storage Summary

Total Building Area (sq.m)	3089.999855	
Assume Available Roof Area (sq. 80%	2471.999884	
Roof Imperviousness	0.99	
Roof Drain Requirement (sq.m/Notch)	232	
Number of Roof Notches*	11	
Max. Allowable Depth of Roof Ponding (m)	0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)	124	
Estimated 100 Year Drawdown Time (h)	1.5	

From Zurn Drain Catalogue

Head (m)	L/min	L/s	Notch Rating
0.051	45.5	0.00076	232

* Note: Number of drains can be reduced if multiple-notch drain used.

Calculation Results	5yr	100yr	Available
Qresult (cu.m/s)	0.016	0.023	-
Depth (m)	0.092	0.139	0.150
Volume (cu.m)	29.7	99.9	123.6
Drain time (hrs)	0.6	1.5	

Roof Drain Design Calculation Sheet

Project #160401884, Richcraft Mer Bleue Road - Commercial Development Roof Drain Design Sheet, Area R1000 Standard Zurn Model Z-105-5 Control-Flo Single Notch Roof Drain

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0004	0.0015	0	0.025	24	0	0	0.025
0.050	0.0008	0.0031	2	0.050	96	1	2	0.050
0.075	0.0012	0.0046	5	0.075	216	4	5	0.075
0.100	0.0015	0.0061	13	0.100	384	7	13	0.100
0.125	0.0019	0.0077	25	0.125	600	12	25	0.125
0.150	0.0023	0.0092	43	0.150	864	18	43	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
1.4	455.7	1.4	0.12659
5.2	824.7	3.8	0.35566
12.6	1204.4	7.4	0.69022
24.8	1588.5	12.2	1.13149
43.0	1974.8	18.2	1.68005

Rooftop Storage Summary

Total Building Area (sq.m)	1080.000028	
Assume Available Roof Area (sq. 80%	864.0000224	
Roof Imperviousness	0.99	
Roof Drain Requirement (sq.m/Notch)	232	
Number of Roof Notches*	4	
Max. Allowable Depth of Roof Ponding (m)	0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)	43	
Estimated 100 Year Drawdown Time (h)	1.4	

From Zurn Drain Catalogue

Head (m)	L/min	L/s	Notch Rating
0.051	45.5	0.00076	232

* Note: Number of drains can be reduced if multiple-notch drain used.

Calculation Results	5yr	100yr	Available
Qresult (cu.m/s)	0.006	0.008	-
Depth (m)	0.091	0.138	0.150
Volume (cu.m)	10.2	34.4	43.2
Drain time (hrs)	0.6	1.4	

Roof Drain Design Calculation Sheet

Project #160401884, Richcraft Mer Bleue Road - Commercial Development Roof Drain Design Sheet, Area R1004 Standard Zurn Model Z-105-5 Control-Flo Single Notch Roof Drain

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0004	0.0008	0	0.025	8	0	0	0.025
0.050	0.0008	0.0015	1	0.050	33	0	1	0.050
0.075	0.0012	0.0023	2	0.075	74	1	2	0.075
0.100	0.0015	0.0031	4	0.100	132	3	4	0.100
0.125	0.0019	0.0038	9	0.125	206	4	9	0.125
0.150	0.0023	0.0046	15	0.150	296	6	15	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
0.5	312.3	0.5	0.08674
1.8	565.0	1.3	0.24369
4.3	825.3	2.5	0.47293
8.5	1088.4	4.2	0.77528
14.7	1353.1	6.2	1.15114

Roof Storage Summary

Total Building Area (sq.m)	370.0000048	
Assume Available Roof Area (sq. 80%)	296.0000038	
Roof Imperviousness	0.99	
Roof Drain Requirement (sq.m/Notch)	232	
Number of Roof Notches*	2	
Max. Allowable Depth of Roof Ponding (m)	0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)	15	
Estimated 100 Year Drawdown Time (h)	0.9	

From Zurn Drain Catalogue

Head (m)	L/min	L/s	Notch Rating
0.051	45.5	0.00076	232

* Note: Number of drains can be reduced if multiple-notch drain used.

Calculation Results	5yr	100yr	Available
Qresult (cu.m/s)	0.003	0.004	-
Depth (m)	0.084	0.131	0.150
Volume (cu.m)	2.7	10.0	14.8
Drain time (hrs)	0.3	0.9	

Roof Drain Design Calculation Sheet

Project #160401884, Richcraft Mer Bleue Road - Commercial Development Roof Drain Design Sheet, Area R1005 Standard Zurn Model Z-105-5 Control-Flo Single Notch Roof Drain

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0004	0.0008	0	0.025	12	0	0	0.025
0.050	0.0008	0.0015	1	0.050	50	1	1	0.050
0.075	0.0012	0.0023	3	0.075	112	2	3	0.075
0.100	0.0015	0.0031	7	0.100	199	4	7	0.100
0.125	0.0019	0.0038	13	0.125	311	6	13	0.125
0.150	0.0023	0.0046	22	0.150	448	9	22	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
0.7	472.6	0.7	0.13128
2.7	855.2	2.0	0.36883
6.5	1249.0	3.8	0.71579
12.9	1647.4	6.3	1.17339
22.3	2048.0	9.4	1.74227

Rooftop Storage Summary

Total Building Area (sq.m)	560.0000173	
Assume Available Roof Area (sq. 80%)	448.0000138	
Roof Imperviousness	0.99	
Roof Drain Requirement (sq.m/Notch)	232	
Number of Roof Notches*	2	
Max. Allowable Depth of Roof Ponding (m)	0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)	22	
Estimated 100 Year Drawdown Time (h)	1.5	

From Zurn Drain Catalogue

Head (m)	L/min	L/s	Notch Rating
0.051	45.5	0.00076	232

* Note: Number of drains can be reduced if multiple-notch drain used.

Calculation Results	5yr	100yr	Available
Qresult (cu.m/s)	0.003	0.004	-
Depth (m)	0.092	0.139	0.150
Volume (cu.m)	5.4	18.1	22.4
Drain time (hrs)	0.6	1.5	

Roof Drain Design Calculation Sheet

Project #160401884, Richcraft Mer Bleue Road - Commercial Development Roof Drain Design Sheet, Area R1006 Standard Zurn Model Z-105-5 Control-Flo Single Notch Roof Drain

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0004	0.0012	0	0.025	16	0	0	0.025
0.050	0.0008	0.0023	1	0.050	66	1	1	0.050
0.075	0.0012	0.0035	4	0.075	148	3	4	0.075
0.100	0.0015	0.0046	9	0.100	263	5	9	0.100
0.125	0.0019	0.0058	17	0.125	411	8	17	0.125
0.150	0.0023	0.0069	30	0.150	592	12	30	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
1.0	416.3	1.0	0.11565
3.6	753.4	2.6	0.32493
8.6	1100.3	5.1	0.63058
17.0	1451.3	8.4	1.0337
29.5	1804.2	12.5	1.53486

Roof Storage Summary

Total Building Area (sq.m)	740.0000095	
Assume Available Roof Area (sq. 80%)	592.0000076	
Roof Imperviousness	0.99	
Roof Drain Requirement (sq.m/Notch)	232	
Number of Roof Notches*	3	
Max. Allowable Depth of Roof Ponding (m)	0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)	30	
Estimated 100 Year Drawdown Time (h)	1.3	

From Zurn Drain Catalogue

Head (m)	L/min	L/s	Notch Rating
0.051	45.5	0.00076	232

* Note: Number of drains can be reduced if multiple-notch drain used.

Calculation Results	5yr	100yr	Available
Qresult (cu.m/s)	0.004	0.006	-
Depth (m)	0.089	0.136	0.150
Volume (cu.m)	6.6	22.7	29.6
Drain time (hrs)	0.5	1.3	

Roof Drain Design Calculation Sheet

Project #160401884, Richcraft Mer Bleue Road - Commercial Development Roof Drain Design Sheet, Area R1007 Standard Zurn Model Z-105-5 Control-Flo Single Notch Roof Drain

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0004	0.0008	0	0.025	7	0	0	0.025
0.050	0.0008	0.0015	0	0.050	29	0	0	0.050
0.075	0.0012	0.0023	2	0.075	66	1	2	0.075
0.100	0.0015	0.0031	4	0.100	117	2	4	0.100
0.125	0.0019	0.0038	8	0.125	183	4	8	0.125
0.150	0.0023	0.0046	13	0.150	264	6	13	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
0.4	278.5	0.4	0.07736
1.6	504.0	1.2	0.21735
3.8	736.0	2.3	0.4218
7.6	970.8	3.7	0.69146
13.1	1206.8	5.6	1.0267

Roof Storage Summary

Total Building Area (sq.m)	329.9999982	
Assume Available Roof Area (sq. 80%)	263.9999986	
Roof Imperviousness	0.99	
Roof Drain Requirement (sq.m/Notch)	232	
Number of Roof Notches*	2	
Max. Allowable Depth of Roof Ponding (m)	0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)	13	
Estimated 100 Year Drawdown Time (h)	0.7	

From Zurn Drain Catalogue

Head (m)	L/min	L/s	Notch Rating
0.051	45.5	0.00076	232

* Note: Number of drains can be reduced if multiple-notch drain used.

Calculation Results	5yr	100yr	Available
Qresult (cu.m/s)	0.003	0.004	-
Depth (m)	0.082	0.129	0.150
Volume (cu.m)	2.3	8.5	13.2
Drain time (hrs)	0.3	0.7	

Roof Drain Design Calculation Sheet

Project #160401884, Richcraft Mer Bleue Road - Commercial Development Roof Drain Design Sheet, Area R1008 Standard Zurn Model Z-105-5 Control-Flo Single Notch Roof Drain

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0004	0.0008	0	0.025	8	0	0	0.025
0.050	0.0008	0.0015	1	0.050	30	0	1	0.050
0.075	0.0012	0.0023	2	0.075	68	1	2	0.075
0.100	0.0015	0.0031	4	0.100	121	2	4	0.100
0.125	0.0019	0.0038	8	0.125	189	4	8	0.125
0.150	0.0023	0.0046	14	0.150	272	6	14	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
0.4	286.9	0.4	0.07971
1.6	519.2	1.2	0.22394
4.0	758.3	2.3	0.43459
7.8	1000.2	3.8	0.71242
13.5	1243.4	5.7	1.05781

Roof Storage Summary

Total Building Area (sq.m)	340.0000185	
Assume Available Roof Area (sq. 80%)	272.0000148	
Roof Imperviousness	0.99	
Roof Drain Requirement (sq.m/Notch)	232	
Number of Roof Notches*	2	
Max. Allowable Depth of Roof Ponding (m)	0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)	14	
Estimated 100 Year Drawdown Time (h)	0.8	

* Note: Number of drains can be reduced if multiple-notch drain used.

From Zurn Drain Catalogue

Head (m)	L/min	L/s	Notch Rating
0.051	45.5	0.00076	232

Calculation Results

	5yr	100yr	Available
Qresult (cu.m/s)	0.003	0.004	-
Depth (m)	0.083	0.129	0.150
Volume (cu.m)	2.4	8.8	13.6
Drain time (hrs)	0.3	0.8	

Roof Drain Design Calculation Sheet

Project #160401884, Richcraft Mer Bleue Road - Commercial Development Roof Drain Design Sheet, Area R1009 Standard Zurn Model Z-105-5 Control-Flo Single Notch Roof Drain

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0004	0.0004	0	0.025	5	0	0	0.025
0.050	0.0008	0.0008	0	0.050	20	0	0	0.050
0.075	0.0012	0.0012	1	0.075	46	1	1	0.075
0.100	0.0015	0.0015	3	0.100	82	2	3	0.100
0.125	0.0019	0.0019	5	0.125	128	3	5	0.125
0.150	0.0023	0.0023	9	0.150	184	4	9	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
0.3	388.2	0.3	0.10784
1.1	702.5	0.8	0.30297
2.7	1026.0	1.6	0.58797
5.3	1353.2	2.6	0.96386
9.2	1682.3	3.9	1.43115

Rooftop Storage Summary

Total Building Area (sq.m)	230.0000004	
Assume Available Roof Area (sq. 80%)	184.0000004	
Roof Imperviousness	0.99	
Roof Drain Requirement (sq.m/Notch)	232	
Number of Roof Notches*	1	
Max. Allowable Depth of Roof Ponding (m)	0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)	9	
Estimated 100 Year Drawdown Time (h)	1.2	

From Zurn Drain Catalogue

Head (m)	L/min	L/s	Notch Rating
0.051	45.5	0.00076	232

* Note: Number of drains can be reduced if multiple-notch drain used.

Calculation Results	5yr	100yr	Available
Qresult (cu.m/s)	0.001	0.002	-
Depth (m)	0.088	0.135	0.150
Volume (cu.m)	2.0	6.8	9.2
Drain time (hrs)	0.5	1.2	

Stormwater Management Calculations

File No: 160401884
 Project: Richcraft Mer Bleue Road - Commercial Development
 Date: 23-May-25

SWM Approach:
 Post-development to Allowable per Trailledge East Phase 4 Study

Post-Development Site Conditions:

Overall Runoff Coefficient for Site and Sub-Catchment Areas

Sub-catchment Area		Runoff Coefficient Table			Overall Runoff Coefficient	
Catchment Type	ID / Description	Area (ha) "A"	Runoff Coefficient "C"	"A x C"		
Controlled - Tributary	L101A	Hard Soft	0.104	0.9	0.094	
	Subtotal		0.048	0.2	0.010	0.103
Controlled - Tributary	L103A	Hard Soft	0.045	0.9	0.041	
	Subtotal		0.006	0.2	0.001	0.042
Controlled - Tributary	L103B	Hard Soft	0.058	0.9	0.053	
	Subtotal		0.009	0.2	0.002	0.054
Controlled - Tributary	L103C	Hard Soft	0.003	0.9	0.003	
	Subtotal		0.220	0.2	0.044	0.047
Controlled - Tributary	L104A	Hard Soft	0.035	0.9	0.032	
	Subtotal		0.040	0.2	0.008	0.040
Controlled - Tributary	L104B	Hard Soft	0.082	0.9	0.074	
	Subtotal		0.109	0.2	0.022	0.096
Controlled - Tributary	L105A	Hard Soft	0.043	0.9	0.039	
	Subtotal		0.013	0.2	0.003	0.041
Controlled - Tributary	L106A	Hard Soft	0.170	0.9	0.153	
	Subtotal		0.025	0.2	0.005	0.158
Uncontrolled - Tributary	L107A	Hard Soft	0.022	0.9	0.020	
	Subtotal		0.000	0.2	0.000	0.020
Controlled - Tributary	L109A	Hard Soft	0.076	0.9	0.068	
	Subtotal		0.030	0.2	0.006	0.074
Controlled - Tributary	L111A	Hard Soft	0.045	0.9	0.041	
	Subtotal		0.043	0.2	0.009	0.049
Controlled - Tributary	L112A	Hard Soft	0.119	0.9	0.107	
	Subtotal		0.022	0.2	0.004	0.111
Controlled - Tributary	L112B	Hard Soft	0.229	0.9	0.206	
	Subtotal		0.021	0.2	0.004	0.210
Controlled - Tributary	L112C	Hard Soft	0.284	0.9	0.256	
	Subtotal		0.037	0.2	0.007	0.263
Controlled - Tributary	L112D	Hard Soft	0.229	0.9	0.206	
	Subtotal		0.007	0.2	0.001	0.208
Controlled - Tributary	L114A	Hard Soft	0.163	0.9	0.147	
	Subtotal		0.024	0.2	0.005	0.151
Controlled - Tributary	L114B	Hard Soft	0.614	0.9	0.552	
	Subtotal		0.027	0.2	0.005	0.558
Uncontrolled - Tributary	UNC-2	Hard Soft	0.013	0.9	0.012	
	Subtotal		0.138	0.2	0.028	0.039
Uncontrolled - Tributary	UNC-1	Hard Soft	0.087	0.9	0.078	
	Subtotal		0.157	0.2	0.031	0.110
Roof	R1009A	Hard Soft	0.023	0.9	0.021	
	Subtotal		0.000	0.2	0.000	0.021
Roof	R1008A	Hard Soft	0.034	0.9	0.031	
	Subtotal		0.000	0.2	0.000	0.031
Roof	R1007A	Hard Soft	0.033	0.9	0.030	
	Subtotal		0.000	0.2	0.000	0.030
Roof	R1006A	Hard Soft	0.074	0.9	0.067	
	Subtotal		0.000	0.2	0.000	0.067
Roof	R1005A	Hard Soft	0.056	0.9	0.050	
	Subtotal		0.000	0.2	0.000	0.050
Roof	R1004A	Hard Soft	0.037	0.9	0.033	
	Subtotal		0.000	0.2	0.000	0.033
Roof	R1000A	Hard Soft	0.108	0.9	0.097	
	Subtotal		0.000	0.2	0.000	0.097
Roof	R1001A	Hard Soft	0.309	0.9	0.278	
	Subtotal		0.000	0.2	0.000	0.278
Roof	R1002A	Hard Soft	0.032	0.9	0.029	
	Subtotal		0.000	0.2	0.000	0.029
Roof	R1003A	Hard Soft	0.152	0.9	0.137	
	Subtotal		0.000	0.2	0.000	0.137
Total			4.255		3.147	
Overall Runoff Coefficient= C:						0.74

Total Roof Areas	0.858 ha
Total Tributary Surface Areas (Controlled and Uncontrolled)	3.002 ha
Total Tributary Area to Sewer	3.860 ha
Total Uncontrolled Areas	0.395 ha
Total Site	4.255 ha

Stormwater Management Calculations

Project #160401884, Richcraft Mer Bleue Road - Commercial Development Modified Rational Method Calculations for Storage

2 yr Intensity City of Ottawa	$I = a/(t + b)^c$	a = 732.951	t (min)	I (mm/hr)
	b = 6.199		10	76.81
	c = 0.81		20	52.03
			30	40.04
			40	32.86
			50	28.04
			60	24.56
			70	21.91
			80	19.83
			90	18.14
			100	16.75
			110	15.57
			120	14.56

2 YEAR Predevelopment Target Release from Portion of Site

Subdrainage Area: Allowable per Trallsedge East Phase 4 Study
Area (ha): 4.2550
C: 0.75

Typical Time of Concentration

tc (min)	I (5 yr) (mm/hr)	Qtarget (L/s)
10	104.19	905.0

2 YEAR Modified Rational Method for Entire Site

Subdrainage Area: L101A
Area (ha): 0.15
C: 0.68

Controlled - Tributary

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	76.8	22.1	22.1	0.0	0.0
20	52.0	15.0	15.0	0.0	0.0
30	40.0	11.5	11.5	0.0	0.0
40	32.9	9.4	9.4	0.0	0.0
50	28.0	8.1	8.1	0.0	0.0
60	24.6	7.1	7.1	0.0	0.0
70	21.9	6.3	6.3	0.0	0.0
80	19.8	5.7	5.7	0.0	0.0
90	18.1	5.2	5.2	0.0	0.0
100	16.7	4.8	4.8	0.0	0.0
110	15.6	4.5	4.5	0.0	0.0
120	14.6	4.2	4.2	0.0	0.0

Storage: Above CB

Orifice Equation: $Q = C_d A (2gh)^{0.5}$ Where C = 0.572

Orifice Diameter: 95.00 mm
Invert Elevation: 87.08 m
T/G Elevation: 89.08 m
Max Ponding Depth: 0.00 m
Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
2-year Water Level	88.64	1.51	22.1	0.0	41.6 OK

Subdrainage Area: L103A
Area (ha): 0.05
C: 0.82

Controlled - Tributary

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	76.8	8.9	8.9	0.0	0.0
20	52.0	6.0	6.0	0.0	0.0
30	40.0	4.7	4.7	0.0	0.0
40	32.9	3.8	3.8	0.0	0.0
50	28.0	3.3	3.3	0.0	0.0
60	24.6	2.9	2.9	0.0	0.0
70	21.9	2.5	2.5	0.0	0.0
80	19.8	2.3	2.3	0.0	0.0
90	18.1	2.1	2.1	0.0	0.0
100	16.7	1.9	1.9	0.0	0.0
110	15.6	1.8	1.8	0.0	0.0
120	14.6	1.7	1.7	0.0	0.0

Storage: Above CB

Orifice Equation: $Q = C_d A (2gh)^{0.5}$ Where C = 0.572

Orifice Diameter: 95.00 mm
Invert Elevation: 87.90 m
T/G Elevation: 89.28 m
Max Ponding Depth: 0.00 m
Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
2-year Water Level	88.19	0.25	8.9	0.0	4.4 OK

Subdrainage Area: L103B
Area (ha): 0.07
C: 0.81

Controlled - Tributary

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	76.8	11.6	11.6	0.0	0.0
20	52.0	7.8	7.8	0.0	0.0
30	40.0	6.0	6.0	0.0	0.0
40	32.9	5.0	5.0	0.0	0.0
50	28.0	4.2	4.2	0.0	0.0
60	24.6	3.7	3.7	0.0	0.0

Project #160401884, Richcraft Mer Bleue Road - Commercial Development Modified Rational Method Calculations for Storage

100 yr Intensity City of Ottawa	$I = a/(t + b)^c$	a = 1735.688	t (min)	I (mm/hr)
	b = 6.014		10	178.56
	c = 0.820		20	119.95
			30	91.87
			40	75.15
			50	63.95
			60	55.89
			70	49.79
			80	44.99
			90	41.11
			100	37.90
			110	35.20
			120	32.69

100 YEAR Predevelopment Target Release from Portion of Site

100 YEAR Modified Rational Method for Entire Site

Subdrainage Area: L101A
Area (ha): 0.15
C: 0.85

Controlled - Tributary

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	178.6	64.1	26.6	37.5	22.5
20	120.0	43.1	26.6	16.5	19.8
30	91.9	33.0	26.6	6.4	11.5
40	75.1	27.0	26.6	0.4	1.0
50	64.0	23.0	23.0	0.0	0.0
60	55.9	20.1	20.1	0.0	0.0
70	49.8	17.9	17.9	0.0	0.0
80	45.0	16.2	16.2	0.0	0.0
90	41.1	14.8	14.8	0.0	0.0
100	37.9	13.6	13.6	0.0	0.0
110	35.2	12.6	12.6	0.0	0.0
120	32.9	11.8	11.8	0.0	0.0

Storage: Surface Storage Above CB

Orifice Equation: $Q = C_d A (2gh)^{0.5}$ Where C = 0.572

Orifice Diameter: 95.00 mm
Invert Elevation: 87.08 m
T/G Elevation: 89.08 m
Max Ponding Depth: 0.24 m
Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	89.32	2.19	26.6	22.5	41.6 OK

0.00

Subdrainage Area: L103A
Area (ha): 0.05
C: 1.00

Controlled - Tributary

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	178.6	25.3	21.6	3.7	2.2
20	120.0	17.0	17.0	0.0	0.0
30	91.9	13.0	13.0	0.0	0.0
40	75.1	10.7	10.7	0.0	0.0
50	64.0	9.1	9.1	0.0	0.0
60	55.9	7.9	7.9	0.0	0.0
70	49.8	7.1	7.1	0.0	0.0
80	45.0	6.4	6.4	0.0	0.0
90	41.1	5.8	5.8	0.0	0.0
100	37.9	5.4	5.4	0.0	0.0
110	35.2	5.0	5.0	0.0	0.0
120	32.9	4.7	4.7	0.0	0.0

Storage: Surface Storage Above CB

Orifice Equation: $Q = C_d A (2gh)^{0.5}$ Where C = 0.572

Orifice Diameter: 95.00 mm
Invert Elevation: 87.90 m
T/G Elevation: 89.28 m
Max Ponding Depth: 0.12 m
Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	89.40	1.45	21.6	2.2	4.4 OK

0.00

Subdrainage Area: L103B
Area (ha): 0.07
C: 1.00

Controlled - Tributary

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	178.6	33.3	16.7	16.6	10.0
20	120.0	22.3	16.7	5.7	6.8
30	91.9	17.1	16.7	0.4	0.8
40	75.1	14.0	14.0	0.0	0.0
50	64.0	11.9	11.9	0.0	0.0
60	55.9	10.4	10.4	0.0	0.0

Stormwater Management Calculations

Project #160401884, Richcraft Mer Bleue Road - Commercial Development Modified Rational Method Calculations for Storage

70	21.9	3.3	3.3	0.0	0.0
80	19.8	3.0	3.0	0.0	0.0
90	18.1	2.7	2.7	0.0	0.0
100	16.7	2.5	2.5	0.0	0.0
110	15.6	2.3	2.3	0.0	0.0
120	14.6	2.2	2.2	0.0	0.0

Storage: Above CB

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.572
 Orifice Diameter: 83.00 mm
 Invert Elevation: 87.97 m
 T/G Elevation: 89.35 m
 Max Ponding Depth: 0.00 m
 Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
2-year Water Level	88.73	0.71	11.6	0.0	12.4 OK

Subdrainage Area: L103C Controlled - Tributary
Area (ha): 0.22
C: 0.21

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.8	10.0	10.0	0.0	0.0
20	52.0	6.8	6.8	0.0	0.0
30	40.0	5.2	5.2	0.0	0.0
40	32.9	4.3	4.3	0.0	0.0
50	28.0	3.7	3.7	0.0	0.0
60	24.6	3.2	3.2	0.0	0.0
70	21.9	2.9	2.9	0.0	0.0
80	19.8	2.6	2.6	0.0	0.0
90	18.1	2.4	2.4	0.0	0.0
100	16.7	2.2	2.2	0.0	0.0
110	15.6	2.0	2.0	0.0	0.0
120	14.6	1.9	1.9	0.0	0.0

Storage: pe Storage

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.572
 Orifice Diameter: 83.00 mm
 Invert Elevation: 87.46 m
 T/G Elevation: 88.71 m
 Max Ponding Depth: 0.00 m
 Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
2-year Water Level	88.03	0.53	10.0	0.0	12.0 OK

Subdrainage Area: L104A Controlled - Tributary
Area (ha): 0.08
C: 0.53

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.8	8.5	8.5	0.0	0.0
20	52.0	5.7	5.7	0.0	0.0
30	40.0	4.4	4.4	0.0	0.0
40	32.9	3.6	3.6	0.0	0.0
50	28.0	3.1	3.1	0.0	0.0
60	24.6	2.7	2.7	0.0	0.0
70	21.9	2.4	2.4	0.0	0.0
80	19.8	2.2	2.2	0.0	0.0
90	18.1	2.0	2.0	0.0	0.0
100	16.7	1.9	1.9	0.0	0.0
110	15.6	1.7	1.7	0.0	0.0
120	14.6	1.6	1.6	0.0	0.0

Storage: Above CB

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.572
 Orifice Diameter: 83.00 mm
 Invert Elevation: 87.67 m
 T/G Elevation: 89.05 m
 Max Ponding Depth: 0.00 m
 Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
2-year Water Level	88.09	0.38	8.5	0.0	14.8 OK

Subdrainage Area: L104B Controlled - Tributary
Area (ha): 0.19
C: 0.50

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.8	20.4	20.4	0.0	0.0
20	52.0	13.8	13.8	0.0	0.0
30	40.0	10.6	10.6	0.0	0.0
40	32.9	8.7	8.7	0.0	0.0
50	28.0	7.4	7.4	0.0	0.0
60	24.6	6.5	6.5	0.0	0.0
70	21.9	5.8	5.8	0.0	0.0
80	19.8	5.3	5.3	0.0	0.0
90	18.1	4.8	4.8	0.0	0.0
100	16.7	4.4	4.4	0.0	0.0
110	15.6	4.1	4.1	0.0	0.0
120	14.6	3.9	3.9	0.0	0.0

Storage: Above CB

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.572

Project #160401884, Richcraft Mer Bleue Road - Commercial Development Modified Rational Method Calculations for Storage

70	49.8	9.3	9.3	0.0	0.0
80	45.0	8.4	8.4	0.0	0.0
90	41.1	7.7	7.7	0.0	0.0
100	37.9	7.1	7.1	0.0	0.0
110	35.2	6.6	6.6	0.0	0.0
120	32.9	6.1	6.1	0.0	0.0

Storage: Surface Storage Above CB

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.572
 Orifice Diameter: 83.00 mm
 Invert Elevation: 87.97 m
 T/G Elevation: 89.35 m
 Max Ponding Depth: 0.14 m
 Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	89.49	1.48	16.7	10.0	12.4 OK

0.00

Subdrainage Area: L103C Controlled - Tributary
Area (ha): 0.22
C: 0.26

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.6	29.1	15.1	14.0	8.4
20	120.0	19.5	15.1	4.4	5.3
30	91.9	15.0	15.0	0.0	0.0
40	75.1	12.2	12.2	0.0	0.0
50	64.0	10.4	10.4	0.0	0.0
60	55.9	9.1	9.1	0.0	0.0
70	49.8	8.1	8.1	0.0	0.0
80	45.0	7.3	7.3	0.0	0.0
90	41.1	6.7	6.7	0.0	0.0
100	37.9	6.2	6.2	0.0	0.0
110	35.2	5.7	5.7	0.0	0.0
120	32.9	5.4	5.4	0.0	0.0

Storage: In-Pipe Storage

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.572
 Orifice Diameter: 83.00 mm
 Invert Elevation: 87.46 m
 T/G Elevation: 88.71 m
 Max Ponding Depth: 0.00 m
 Downstream W/L: 0.00 m

Pipe Storage: Length 244.4 Diameter 250 Volume 12.0

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	88.71	1.21	15.1	8.4	12.0 OK

0.00

Subdrainage Area: L104A Controlled - Tributary
Area (ha): 0.08
C: 0.66

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.6	24.7	17.1	7.6	4.6
20	120.0	16.6	16.6	0.0	0.0
30	91.9	12.7	12.7	0.0	0.0
40	75.1	10.4	10.4	0.0	0.0
50	64.0	8.8	8.8	0.0	0.0
60	55.9	7.7	7.7	0.0	0.0
70	49.8	6.9	6.9	0.0	0.0
80	45.0	6.2	6.2	0.0	0.0
90	41.1	5.7	5.7	0.0	0.0
100	37.9	5.2	5.2	0.0	0.0
110	35.2	4.9	4.9	0.0	0.0
120	32.9	4.5	4.5	0.0	0.0

Storage: Surface Storage Above CB

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.572
 Orifice Diameter: 83.00 mm
 Invert Elevation: 87.67 m
 T/G Elevation: 89.05 m
 Max Ponding Depth: 0.21 m
 Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	89.26	1.55	17.1	4.6	14.8 OK

0.00

Subdrainage Area: L104B Controlled - Tributary
Area (ha): 0.19
C: 0.63

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.6	59.3	28.7	30.6	18.4
20	120.0	39.8	28.7	11.1	13.4
30	91.9	30.5	28.7	1.8	3.3
40	75.1	24.9	24.9	0.0	0.0
50	64.0	21.2	21.2	0.0	0.0
60	55.9	18.5	18.5	0.0	0.0
70	49.8	16.5	16.5	0.0	0.0
80	45.0	14.9	14.9	0.0	0.0
90	41.1	13.6	13.6	0.0	0.0
100	37.9	12.6	12.6	0.0	0.0
110	35.2	11.7	11.7	0.0	0.0
120	32.9	10.9	10.9	0.0	0.0

Storage: Surface Storage Above CB

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.572

Stormwater Management Calculations

Project #160401884, Richcraft Mer Bleue Road - Commercial Development Modified Rational Method Calculations for Storage

Orifice Diameter:	108.00	mm
Invert Elevation	87.36	m
T/G Elevation	88.72	m
Max Ponding Depth	0.00	m
Downstream W/L	0.00	m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
2-year Water Level	88.19	0.77	20.4	0.0	19.3 OK

Subdrainage Area: L105A Controlled - Tributary
Area (ha): 0.06
C: 0.74

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.8	8.8	8.8	0.0	0.0
20	52.0	6.0	6.0	0.0	0.0
30	40.0	4.6	4.6	0.0	0.0
40	32.9	3.8	3.8	0.0	0.0
50	28.0	3.2	3.2	0.0	0.0
60	24.6	2.8	2.8	0.0	0.0
70	21.9	2.5	2.5	0.0	0.0
80	19.8	2.3	2.3	0.0	0.0
90	18.1	2.1	2.1	0.0	0.0
100	16.7	1.9	1.9	0.0	0.0
110	15.6	1.8	1.8	0.0	0.0
120	14.6	1.7	1.7	0.0	0.0

Storage: Above CB

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.572
Orifice Diameter: 83.00 mm
Invert Elevation: 87.53 m
T/G Elevation: 88.91 m
Max Ponding Depth: 0.00 m
Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
2-year Water Level	87.99	0.42	8.8	0.0	5.5 OK

Subdrainage Area: L106A Controlled - Tributary
Area (ha): 0.20
C: 0.81

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.8	33.7	33.7	0.0	0.0
20	52.0	22.8	22.8	0.0	0.0
30	40.0	17.6	17.6	0.0	0.0
40	32.9	14.4	14.4	0.0	0.0
50	28.0	12.3	12.3	0.0	0.0
60	24.6	10.8	10.8	0.0	0.0
70	21.9	9.6	9.6	0.0	0.0
80	19.8	8.7	8.7	0.0	0.0
90	18.1	8.0	8.0	0.0	0.0
100	16.7	7.4	7.4	0.0	0.0
110	15.6	6.8	6.8	0.0	0.0
120	14.6	6.4	6.4	0.0	0.0

Storage: Above CB

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.610
Orifice Diameter: 200.00 mm
Invert Elevation: 87.73 m
T/G Elevation: 88.81 m
Max Ponding Depth: 0.00 m
Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
2-year Water Level	87.99	0.16	33.7	0.0	4.8 OK

Subdrainage Area: L107A Uncontrolled - Tributary
Area (ha): 0.02
C: 0.90

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.8	4.2	4.2	0.0	0.0
20	52.0	2.9	2.9	0.0	0.0
30	40.0	2.2	2.2	0.0	0.0
40	32.9	1.8	1.8	0.0	0.0
50	28.0	1.5	1.5	0.0	0.0
60	24.6	1.4	1.4	0.0	0.0
70	21.9	1.2	1.2	0.0	0.0
80	19.8	1.1	1.1	0.0	0.0
90	18.1	1.0	1.0	0.0	0.0
100	16.7	0.9	0.9	0.0	0.0
110	15.6	0.9	0.9	0.0	0.0
120	14.6	0.8	0.8	0.0	0.0

Subdrainage Area: L109A Controlled - Tributary
Area (ha): 0.11
C: 0.70

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.8	15.8	15.8	0.0	0.0
20	52.0	10.7	10.7	0.0	0.0
30	40.0	8.3	8.3	0.0	0.0
40	32.9	6.8	6.8	0.0	0.0
50	28.0	5.8	5.8	0.0	0.0
60	24.6	5.1	5.1	0.0	0.0
70	21.9	4.5	4.5	0.0	0.0

Project #160401884, Richcraft Mer Bleue Road - Commercial Development Modified Rational Method Calculations for Storage

Orifice Diameter:	108.00	mm
Invert Elevation	87.36	m
T/G Elevation	88.72	m
Max Ponding Depth	0.22	m
Downstream W/L	0.00	m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	88.94	1.53	28.7	18.4	19.3 OK

Subdrainage Area: L105A Controlled - Tributary
Area (ha): 0.06
C: 0.93

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.6	25.7	16.7	9.0	5.4
20	120.0	17.3	16.7	0.5	0.7
30	91.9	13.2	13.2	0.0	0.0
40	75.1	10.8	10.8	0.0	0.0
50	64.0	9.2	9.2	0.0	0.0
60	55.9	8.0	8.0	0.0	0.0
70	49.8	7.2	7.2	0.0	0.0
80	45.0	6.5	6.5	0.0	0.0
90	41.1	5.9	5.9	0.0	0.0
100	37.9	5.5	5.5	0.0	0.0
110	35.2	5.1	5.1	0.0	0.0
120	32.9	4.7	4.7	0.0	0.0

Storage: Surface Storage Above CB

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.572
Orifice Diameter: 83.00 mm
Invert Elevation: 87.53 m
T/G Elevation: 88.91 m
Max Ponding Depth: 0.15 m
Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	89.06	1.49	16.7	5.4	5.5 OK

Subdrainage Area: L106A Controlled - Tributary
Area (ha): 0.20
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.6	96.8	90.2	6.6	3.9
20	120.0	65.0	65.0	0.0	0.0
30	91.9	49.8	49.8	0.0	0.0
40	75.1	40.7	40.7	0.0	0.0
50	64.0	34.7	34.7	0.0	0.0
60	55.9	30.3	30.3	0.0	0.0
70	49.8	27.0	27.0	0.0	0.0
80	45.0	24.4	24.4	0.0	0.0
90	41.1	22.3	22.3	0.0	0.0
100	37.9	20.5	20.5	0.0	0.0
110	35.2	19.1	19.1	0.0	0.0
120	32.9	17.8	17.8	0.0	0.0

Storage: Surface Storage Above CB

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.610
Orifice Diameter: 200.00 mm
Invert Elevation: 87.73 m
T/G Elevation: 88.81 m
Max Ponding Depth: 0.15 m
Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	88.96	1.13	90.2	3.9	4.8 OK

Subdrainage Area: L107A Uncontrolled - Tributary
Area (ha): 0.02
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.6	10.9	10.9	0.0	0.0
20	120.0	7.3	7.3	0.0	0.0
30	91.9	5.6	5.6	0.0	0.0
40	75.1	4.6	4.6	0.0	0.0
50	64.0	3.9	3.9	0.0	0.0
60	55.9	3.4	3.4	0.0	0.0
70	49.8	3.0	3.0	0.0	0.0
80	45.0	2.8	2.8	0.0	0.0
90	41.1	2.5	2.5	0.0	0.0
100	37.9	2.3	2.3	0.0	0.0
110	35.2	2.2	2.2	0.0	0.0
120	32.9	2.0	2.0	0.0	0.0

Subdrainage Area: L109A Controlled - Tributary
Area (ha): 0.11
C: 0.88

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.6	46.0	22.2	23.9	14.3
20	120.0	30.9	22.2	8.8	10.5
30	91.9	23.7	22.2	1.5	2.8
40	75.1	19.4	19.4	0.0	0.0
50	64.0	16.5	16.5	0.0	0.0
60	55.9	14.4	14.4	0.0	0.0
70	49.8	12.8	12.8	0.0	0.0

Stormwater Management Calculations

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80	19.8	4.1	4.1	0.0	0.0
90	18.1	3.7	3.7	0.0	0.0
100	16.7	3.5	3.5	0.0	0.0
110	15.6	3.2	3.2	0.0	0.0
120	14.6	3.0	3.0	0.0	0.0

Storage: : Above CB

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.572
 Orifice Diameter: 95.00 mm
 Invert Elevation: 87.87 m
 T/G Elevation: 89.25 m
 Max Ponding Depth: 0.00 m
 Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
2-year Water Level	88.70	0.78	15.8	0.0	16.3 OK

Subdrainage Area: L111A Controlled - Tributary
Area (ha): 0.09
C: 0.56

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.8	10.5	10.5	0.0	0.0
20	52.0	7.1	7.1	0.0	0.0
30	40.0	5.5	5.5	0.0	0.0
40	32.9	4.5	4.5	0.0	0.0
50	28.0	3.8	3.8	0.0	0.0
60	24.6	3.4	3.4	0.0	0.0
70	21.9	3.0	3.0	0.0	0.0
80	19.8	2.7	2.7	0.0	0.0
90	18.1	2.5	2.5	0.0	0.0
100	16.7	2.3	2.3	0.0	0.0
110	15.6	2.1	2.1	0.0	0.0
120	14.6	2.0	2.0	0.0	0.0

Storage: : Above CB

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.572
 Orifice Diameter: 83.00 mm
 Invert Elevation: 87.52 m
 T/G Elevation: 88.90 m
 Max Ponding Depth: 0.00 m
 Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
2-year Water Level	88.15	0.59	10.5	0.0	15.3 OK

Subdrainage Area: L112A Controlled - Tributary
Area (ha): 0.14
C: 0.79

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.8	23.8	23.8	0.0	0.0
20	52.0	16.1	16.1	0.0	0.0
30	40.0	12.4	12.4	0.0	0.0
40	32.9	10.2	10.2	0.0	0.0
50	28.0	8.7	8.7	0.0	0.0
60	24.6	7.6	7.6	0.0	0.0
70	21.9	6.8	6.8	0.0	0.0
80	19.8	6.1	6.1	0.0	0.0
90	18.1	5.6	5.6	0.0	0.0
100	16.7	5.2	5.2	0.0	0.0
110	15.6	4.8	4.8	0.0	0.0
120	14.6	4.5	4.5	0.0	0.0

Storage: : Above CB

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.572
 Orifice Diameter: 152.00 mm
 Invert Elevation: 87.66 m
 T/G Elevation: 89.04 m
 Max Ponding Depth: 0.00 m
 Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
2-year Water Level	88.00	0.27	23.8	0.0	9.4 OK

Subdrainage Area: L112B Controlled - Tributary
Area (ha): 0.25
C: 0.84

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.8	44.8	44.8	0.0	0.0
20	52.0	30.4	30.4	0.0	0.0
30	40.0	23.4	23.4	0.0	0.0
40	32.9	19.2	19.2	0.0	0.0
50	28.0	16.4	16.4	0.0	0.0
60	24.6	14.3	14.3	0.0	0.0
70	21.9	12.8	12.8	0.0	0.0
80	19.8	11.6	11.6	0.0	0.0
90	18.1	10.6	10.6	0.0	0.0
100	16.7	9.8	9.8	0.0	0.0
110	15.6	9.1	9.1	0.0	0.0
120	14.6	8.5	8.5	0.0	0.0

Storage: : Above CB

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.572
 Orifice Diameter: 152.00 mm

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80	45.0	11.6	11.6	0.0	0.0
90	41.1	10.6	10.6	0.0	0.0
100	37.9	9.8	9.8	0.0	0.0
110	35.2	9.1	9.1	0.0	0.0
120	32.9	8.5	8.5	0.0	0.0

Storage: Surface Storage Above CB

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.572
 Orifice Diameter: 95.00 mm
 Invert Elevation: 87.87 m
 T/G Elevation: 89.25 m
 Max Ponding Depth: 0.19 m
 Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	89.44	1.52	22.2	14.3	16.3 OK

0.00

Subdrainage Area: L111A Controlled - Tributary
Area (ha): 0.09
C: 0.70

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.6	30.6	16.9	13.7	8.2
20	120.0	20.5	16.9	3.6	4.4
30	91.9	15.7	15.7	0.0	0.0
40	75.1	12.9	12.9	0.0	0.0
50	64.0	11.0	11.0	0.0	0.0
60	55.9	9.6	9.6	0.0	0.0
70	49.8	8.5	8.5	0.0	0.0
80	45.0	7.7	7.7	0.0	0.0
90	41.1	7.0	7.0	0.0	0.0
100	37.9	6.5	6.5	0.0	0.0
110	35.2	6.0	6.0	0.0	0.0
120	32.9	5.6	5.6	0.0	0.0

Storage: Surface Storage Above CB

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.572
 Orifice Diameter: 83.00 mm
 Invert Elevation: 87.52 m
 T/G Elevation: 88.90 m
 Max Ponding Depth: 0.18 m
 Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	89.08	1.52	16.9	8.2	15.3 OK

0.00

Subdrainage Area: L112A Controlled - Tributary
Area (ha): 0.14
C: 0.99

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.6	69.1	55.6	13.5	8.1
20	120.0	46.4	46.4	0.0	0.0
30	91.9	35.6	35.6	0.0	0.0
40	75.1	29.1	29.1	0.0	0.0
50	64.0	24.8	24.8	0.0	0.0
60	55.9	21.6	21.6	0.0	0.0
70	49.8	19.3	19.3	0.0	0.0
80	45.0	17.4	17.4	0.0	0.0
90	41.1	15.9	15.9	0.0	0.0
100	37.9	14.7	14.7	0.0	0.0
110	35.2	13.6	13.6	0.0	0.0
120	32.9	12.7	12.7	0.0	0.0

Storage: Surface Storage Above CB

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.572
 Orifice Diameter: 152.00 mm
 Invert Elevation: 87.66 m
 T/G Elevation: 89.04 m
 Max Ponding Depth: 0.16 m
 Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	89.20	1.46	55.6	8.1	9.4 OK

0.00

Subdrainage Area: L112B Controlled - Tributary
Area (ha): 0.25
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.6	124.1	58.2	65.9	39.5
20	120.0	83.4	58.2	25.1	30.2
30	91.9	63.8	58.2	5.6	10.1
40	75.1	52.2	52.2	0.0	0.0
50	64.0	44.4	44.4	0.0	0.0
60	55.9	38.8	38.8	0.0	0.0
70	49.8	34.6	34.6	0.0	0.0
80	45.0	31.3	31.3	0.0	0.0
90	41.1	28.6	28.6	0.0	0.0
100	37.9	26.3	26.3	0.0	0.0
110	35.2	24.5	24.5	0.0	0.0
120	32.9	22.9	22.9	0.0	0.0

Storage: Surface Storage Above CB

Orifice Equation: $Q = CdA(2gh)^{0.5}$ Where C = 0.572
 Orifice Diameter: 152.00 mm

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Invert Elevation	87.64	m
T/G Elevation	89.02	m
Max Ponding Depth	0.00	m
Downstream W/L	0.00	m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
2-year Water Level	88.67	0.95	44.8	0.0	91.5 OK

Subdrainage Area: L112C Controlled - Tributary
Area (ha): 0.32
C: 0.82

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.8	56.2	56.2	0.0	0.0
20	52.0	38.1	38.1	0.0	0.0
30	40.0	29.3	29.3	0.0	0.0
40	32.9	24.0	24.0	0.0	0.0
50	28.0	20.5	20.5	0.0	0.0
60	24.6	18.0	18.0	0.0	0.0
70	21.9	16.0	16.0	0.0	0.0
80	19.8	14.5	14.5	0.0	0.0
90	18.1	13.3	13.3	0.0	0.0
100	16.7	12.3	12.3	0.0	0.0
110	15.6	11.4	11.4	0.0	0.0
120	14.6	10.7	10.7	0.0	0.0

Storage: Above CB

Orifice Equation: $Q = C_d A (2gh)^{0.5}$ Where C = 0.572
Orifice Diameter: 178.00 mm
Invert Elevation: 87.74 m
T/G Elevation: 89.06 m
Max Ponding Depth: 0.00 m
Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
2-year Water Level	88.62	0.79	56.2	0.0	134.3 OK

Subdrainage Area: L112D Controlled - Tributary
Area (ha): 0.24
C: 0.88

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.8	44.3	44.3	0.0	0.0
20	52.0	30.0	30.0	0.0	0.0
30	40.0	23.1	23.1	0.0	0.0
40	32.9	19.0	19.0	0.0	0.0
50	28.0	16.2	16.2	0.0	0.0
60	24.6	14.2	14.2	0.0	0.0
70	21.9	12.7	12.7	0.0	0.0
80	19.8	11.4	11.4	0.0	0.0
90	18.1	10.5	10.5	0.0	0.0
100	16.7	9.7	9.7	0.0	0.0
110	15.6	9.0	9.0	0.0	0.0
120	14.6	8.4	8.4	0.0	0.0

Storage: Above CB

Orifice Equation: $Q = C_d A (2gh)^{0.5}$ Where C = 0.572
Orifice Diameter: 152.00 mm
Invert Elevation: 87.71 m
T/G Elevation: 89.10 m
Max Ponding Depth: 0.00 m
Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
2-year Water Level	88.72	0.93	44.3	0.0	79.4 OK

Subdrainage Area: L114A Controlled - Tributary
Area (ha): 0.19
C: 0.81

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.8	32.3	32.3	0.0	0.0
20	52.0	21.9	21.9	0.0	0.0
30	40.0	16.9	16.9	0.0	0.0
40	32.9	13.8	13.8	0.0	0.0
50	28.0	11.8	11.8	0.0	0.0
60	24.6	10.3	10.3	0.0	0.0
70	21.9	9.2	9.2	0.0	0.0
80	19.8	8.4	8.4	0.0	0.0
90	18.1	7.6	7.6	0.0	0.0
100	16.7	7.1	7.1	0.0	0.0
110	15.6	6.6	6.6	0.0	0.0
120	14.6	6.1	6.1	0.0	0.0

Storage: Above CB

Orifice Equation: $Q = C_d A (2gh)^{0.5}$ Where C = 0.572
Orifice Diameter: 127.00 mm
Invert Elevation: 87.43 m
T/G Elevation: 88.81 m
Max Ponding Depth: 0.00 m
Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
2-year Water Level	88.51	1.02	32.3	0.0	78.9 OK

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Invert Elevation	87.64	m
T/G Elevation	89.02	m
Max Ponding Depth	0.30	m
Downstream W/L	0.00	m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	89.32	1.60	58.2	39.5	91.5 OK

Subdrainage Area: L112C Controlled - Tributary
Area (ha): 0.32
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.6	159.3	78.0	81.3	48.8
20	120.0	107.0	78.0	29.0	34.8
30	91.9	82.0	78.0	4.0	7.1
40	75.1	67.1	67.1	0.0	0.0
50	64.0	57.1	57.1	0.0	0.0
60	55.9	49.9	49.9	0.0	0.0
70	49.8	44.4	44.4	0.0	0.0
80	45.0	40.1	40.1	0.0	0.0
90	41.1	36.7	36.7	0.0	0.0
100	37.9	33.8	33.8	0.0	0.0
110	35.2	31.4	31.4	0.0	0.0
120	32.9	29.4	29.4	0.0	0.0

Storage: Surface Storage Above CB

Orifice Equation: $Q = C_d A (2gh)^{0.5}$ Where C = 0.572
Orifice Diameter: 178.00 mm
Invert Elevation: 87.74 m
T/G Elevation: 89.06 m
Max Ponding Depth: 0.30 m
Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	89.36	1.53	78.0	48.8	134.3 OK

Subdrainage Area: L112D Controlled - Tributary
Area (ha): 0.24
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.6	117.1	58.4	58.7	35.2
20	120.0	78.7	58.4	20.3	24.3
30	91.9	60.3	58.4	1.9	3.4
40	75.1	49.3	49.3	0.0	0.0
50	64.0	42.0	42.0	0.0	0.0
60	55.9	36.7	36.7	0.0	0.0
70	49.8	32.7	32.7	0.0	0.0
80	45.0	29.5	29.5	0.0	0.0
90	41.1	27.0	27.0	0.0	0.0
100	37.9	24.9	24.9	0.0	0.0
110	35.2	23.1	23.1	0.0	0.0
120	32.9	21.6	21.6	0.0	0.0

Storage: Surface Storage Above CB

Orifice Equation: $Q = C_d A (2gh)^{0.5}$ Where C = 0.572
Orifice Diameter: 152.00 mm
Invert Elevation: 87.71 m
T/G Elevation: 89.10 m
Max Ponding Depth: 0.30 m
Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	89.40	1.61	58.4	35.2	79.4 OK

Subdrainage Area: L114A Controlled - Tributary
Area (ha): 0.19
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.6	92.8	40.8	52.0	31.2
20	120.0	62.4	40.8	21.6	25.9
30	91.9	47.8	40.8	7.0	12.5
40	75.1	39.1	39.1	0.0	0.0
50	64.0	33.2	33.2	0.0	0.0
60	55.9	29.1	29.1	0.0	0.0
70	49.8	25.9	25.9	0.0	0.0
80	45.0	23.4	23.4	0.0	0.0
90	41.1	21.4	21.4	0.0	0.0
100	37.9	19.7	19.7	0.0	0.0
110	35.2	18.3	18.3	0.0	0.0
120	32.9	17.1	17.1	0.0	0.0

Storage: Surface Storage Above CB

Orifice Equation: $Q = C_d A (2gh)^{0.5}$ Where C = 0.572
Orifice Diameter: 127.00 mm
Invert Elevation: 87.43 m
T/G Elevation: 88.81 m
Max Ponding Depth: 0.30 m
Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	89.11	1.62	40.8	31.2	78.9 OK

Stormwater Management Calculations

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Subdrainage Area: L114B		Controlled - Tributary	
Area (ha): 0.84			
C: 0.87			

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	76.8	119.1	119.1	0.0	0.0
20	52.0	80.7	80.7	0.0	0.0
30	40.0	62.1	62.1	0.0	0.0
40	32.9	51.0	51.0	0.0	0.0
50	28.0	43.5	43.5	0.0	0.0
60	24.6	38.1	38.1	0.0	0.0
70	21.9	34.0	34.0	0.0	0.0
80	19.8	30.7	30.7	0.0	0.0
90	18.1	28.1	28.1	0.0	0.0
100	16.7	26.0	26.0	0.0	0.0
110	15.6	24.1	24.1	0.0	0.0
120	14.6	22.6	22.6	0.0	0.0

Storage: Above CB

Orifice Equation: $Q = C_d A (2gh)^{0.5}$ Where C = 0.610

Orifice Diameter: 250.00 mm

Invert Elevation: 87.46 m

T/G Elevation: 88.84 m

Max Ponding Depth: 0.00 m

Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
2-year Water Level	88.39	0.81	119.1	0.0	140.4 OK

Subdrainage Area: UNC-2		Uncontrolled - Tributary	
Area (ha): 0.15			
C: 0.26			

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	76.8	8.4	8.4		
20	52.0	5.7	5.7		
30	40.0	4.4	4.4		
40	32.9	3.6	3.6		
50	28.0	3.1	3.1		
60	24.6	2.7	2.7		
70	21.9	2.4	2.4		
80	19.8	2.2	2.2		
90	18.1	2.0	2.0		
100	16.7	1.8	1.8		
110	15.6	1.7	1.7		
120	14.6	1.6	1.6		

Subdrainage Area: UNC-1		Uncontrolled - Tributary	
Area (ha): 0.24			
C: 0.45			

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	76.8	23.4	23.4		
20	52.0	15.9	15.9		
30	40.0	12.2	12.2		
40	32.9	10.0	10.0		
50	28.0	8.6	8.6		
60	24.6	7.5	7.5		
70	21.9	6.7	6.7		
80	19.8	6.1	6.1		
90	18.1	5.5	5.5		
100	16.7	5.1	5.1		
110	15.6	4.8	4.8		
120	14.6	4.4	4.4		

Subdrainage Area: R1009A		Roof	
Area (ha): 0.02		Maximum Storage Depth: 150 mm	
C: 0.90			

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)	Depth (mm)
10	76.8	4.4	1.3	3.1	1.9	86.2 0.00
20	52.0	3.0	1.4	1.6	2.0	88.0 0.00
30	40.0	2.3	1.3	1.0	1.8	85.2 0.00
40	32.9	1.9	1.2	0.6	1.5	81.2 0.00
50	28.0	1.6	1.2	0.4	1.3	77.2 0.00
60	24.6	1.4	1.1	0.3	1.1	72.6 0.00
70	21.9	1.3	1.0	0.2	0.9	67.9 0.00
80	19.8	1.1	1.0	0.2	0.8	63.7 0.00
90	18.1	1.0	0.9	0.1	0.7	60.0 0.00
100	16.7	1.0	0.9	0.1	0.6	56.7 0.00
110	15.6	0.9	0.8	0.1	0.5	53.8 0.00
120	14.6	0.8	0.8	0.1	0.4	51.1 0.00

Storage: Roof Storage

Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
2-year Water Level	88.0	0.09	1.4	2.0	9.2 0.00

Subdrainage Area: R1008A		Roof	
Area (ha): 0.03		Maximum Storage Depth: 150 mm	
C: 0.90			

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)	Depth (mm)
10	76.8	6.5	2.5	4.0	2.4	82.5 0.00
20	52.0	4.4	2.5	1.9	2.3	81.5 0.00
30	40.0	3.4	2.4	1.0	1.9	76.9 0.00
40	32.9	2.8	2.2	0.6	1.5	70.7 0.00
50	28.0	2.4	2.0	0.4	1.2	64.6 0.00
60	24.6	2.1	1.8	0.3	1.0	59.4 0.00

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Subdrainage Area: L114B		Controlled - Tributary	
Area (ha): 0.84			
C: 1.00			

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	178.6	318.2	165.4	152.8	91.7
20	120.0	213.7	165.4	48.4	58.0
30	91.9	163.7	163.7	0.0	0.0
40	75.1	133.9	133.9	0.0	0.0
50	64.0	114.0	114.0	0.0	0.0
60	55.9	99.6	99.6	0.0	0.0
70	49.8	88.7	88.7	0.0	0.0
80	45.0	80.2	80.2	0.0	0.0
90	41.1	73.3	73.3	0.0	0.0
100	37.9	67.5	67.5	0.0	0.0
110	35.2	62.7	62.7	0.0	0.0
120	32.9	58.6	58.6	0.0	0.0

Storage: Surface Storage Above CB

Orifice Equation: $Q = C_d A (2gh)^{0.5}$ Where C = 0.610

Orifice Diameter: 250.00 mm

Invert Elevation: 87.46 m

T/G Elevation: 88.84 m

Max Ponding Depth: 0.30 m

Downstream W/L: 0.00 m

Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	89.14	1.56	165.4	91.7	140.4 OK

Subdrainage Area: UNC-2		Uncontrolled - Tributary	
Area (ha): 0.15			
C: 0.33			

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	178.6	24.4	24.4		
20	120.0	16.4	16.4		
30	91.9	12.5	12.5		
40	75.1	10.3	10.3		
50	64.0	8.7	8.7		
60	55.9	7.6	7.6		
70	49.8	6.8	6.8		
80	45.0	6.1	6.1		
90	41.1	5.6	5.6		
100	37.9	5.2	5.2		
110	35.2	4.8	4.8		
120	32.9	4.5	4.5		

Subdrainage Area: UNC-1		Uncontrolled - Tributary	
Area (ha): 0.24			
C: 0.56			

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)
10	178.6	68.1	68.1		
20	120.0	45.8	45.8		
30	91.9	35.1	35.1		
40	75.1	28.7	28.7		
50	64.0	24.4	24.4		
60	55.9	21.3	21.3		
70	49.8	19.0	19.0		
80	45.0	17.2	17.2		
90	41.1	15.7	15.7		
100	37.9	14.5	14.5		
110	35.2	13.4	13.4		
120	32.9	12.6	12.6		

Subdrainage Area: R1009A		Roof	
Area (ha): 0.02		Maximum Storage Depth: 150 mm	
C: 1.00			

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)	Depth (mm)
10	178.6	11.4	2.0	9.5	5.7	127.3 0.00
20	120.0	7.7	2.1	5.6	6.7	134.1 0.00
30	91.9	5.9	2.1	3.8	6.8	134.8 0.00
40	75.1	4.8	2.0	2.8	6.6	133.3 0.00
50	64.0	4.1	2.0	2.1	6.2	130.9 0.00
60	55.9	3.6	2.0	1.6	5.8	128.0 0.00
70	49.8	3.2	1.9	1.3	5.3	124.9 0.00
80	45.0	2.9	1.9	1.0	4.9	120.9 0.00
90	41.1	2.6	1.8	0.8	4.5	117.0 0.00
100	37.9	2.4	1.7	0.7	4.1	113.3 0.00
110	35.2	2.3	1.7	0.6	3.7	109.7 0.00
120	32.9	2.1	1.6	0.5	3.4	106.3 0.00

Storage: Roof Storage

Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
100-year Water Level	134.8	0.13	2.1	6.8	9.2 0.00

Subdrainage Area: R1008A		Roof	
Area (ha): 0.03		Maximum Storage Depth: 150 mm	
C: 1.00			

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)	Depth (mm)
10	178.6	16.9	3.8	13.0	7.8	124.7 0.00
20	120.0	11.3	4.0	7.4	8.8	129.2 0.00
30	91.9	8.7	3.9	4.8	8.6	128.0 0.00
40	75.1	7.1	3.8	3.3	7.8	124.8 0.00
50	64.0	6.0	3.7	2.4	7.1	119.9 0.00
60	55.9	5.3	3.5	1.8	6.3	114.9 0.00

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70	21.9	1.9	1.7	0.2	0.7	54.9	0.00
80	19.8	1.7	1.6	0.1	0.6	51.1	0.00
90	18.1	1.5	1.5	0.1	0.5	47.5	0.00
100	16.7	1.4	1.4	0.1	0.4	44.2	0.00
110	15.6	1.3	1.3	0.1	0.4	41.4	0.00
120	14.6	1.2	1.2	0.0	0.3	38.9	0.00

Storage: Roof Storage

Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
82.5	0.08	2.5	2.4	13.6	0.00

Subdrainage Area: R1007A
Area (ha): 0.03
C: 0.90
Maximum Storage Depth: Roof 150 mm

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)
10	76.8	6.3	2.5	3.8	2.3	82.1
20	52.0	4.3	2.5	1.8	2.2	80.8
30	40.0	3.3	2.3	1.0	1.7	76.1
40	32.9	2.7	2.1	0.6	1.4	69.4
50	28.0	2.3	1.9	0.4	1.1	63.3
60	24.6	2.0	1.8	0.2	0.9	58.2
70	21.9	1.8	1.7	0.2	0.7	53.8
80	19.8	1.6	1.5	0.1	0.5	50.0
90	18.1	1.5	1.4	0.1	0.4	46.2
100	16.7	1.4	1.3	0.1	0.4	43.0
110	15.6	1.3	1.2	0.0	0.3	40.3
120	14.6	1.2	1.2	0.0	0.3	37.9

Storage: Roof Storage

Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
82.1	0.08	2.5	2.3	13.2	0.00

Subdrainage Area: R1006A
Area (ha): 0.07
C: 0.90
Maximum Storage Depth: Roof 150 mm

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)
10	76.8	14.2	4.0	10.2	6.1	87.0
20	52.0	9.6	4.1	5.5	6.6	89.4
30	40.0	7.4	4.0	3.4	6.1	87.0
40	32.9	6.1	3.8	2.2	5.4	83.3
50	28.0	5.2	3.7	1.5	4.6	79.4
60	24.6	4.5	3.5	1.1	3.8	75.6
70	21.9	4.1	3.3	0.8	3.3	71.1
80	19.8	3.7	3.1	0.6	2.8	66.8
90	18.1	3.4	2.9	0.5	2.5	63.0
100	16.7	3.1	2.7	0.4	2.1	59.7
110	15.6	2.9	2.6	0.3	1.8	56.7
120	14.6	2.7	2.5	0.2	1.5	54.0

Storage: Roof Storage

Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
89.4	0.09	4.1	6.6	29.6	0.00

Subdrainage Area: R1005A
Area (ha): 0.06
C: 0.90
Maximum Storage Depth: Roof 150 mm

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)
10	76.8	10.8	2.7	8.1	4.8	88.2
20	52.0	7.3	2.8	4.5	5.4	91.7
30	40.0	5.6	2.8	2.8	5.1	90.1
40	32.9	4.6	2.7	1.9	4.6	87.0
50	28.0	3.9	2.6	1.4	4.1	83.4
60	24.6	3.4	2.5	1.0	3.6	79.9
70	21.9	3.1	2.3	0.7	3.0	76.5
80	19.8	2.8	2.2	0.5	2.6	72.7
90	18.1	2.5	2.1	0.4	2.3	68.8
100	16.7	2.3	2.0	0.3	2.0	65.3
110	15.6	2.2	1.9	0.3	1.8	62.2
120	14.6	2.0	1.8	0.2	1.6	59.3

Storage: Roof Storage

Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
91.7	0.09	2.8	5.4	22.4	0.00

Subdrainage Area: R1004A
Area (ha): 0.04
C: 0.90
Maximum Storage Depth: Roof 150 mm

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)
10	76.8	7.1	2.6	4.5	2.7	83.6
20	52.0	4.8	2.6	2.3	2.7	83.4
30	40.0	3.7	2.4	1.3	2.3	79.3
40	32.9	3.0	2.3	0.8	1.8	74.4
50	28.0	2.6	2.1	0.5	1.5	68.2
60	24.6	2.3	1.9	0.3	1.2	62.9
70	21.9	2.0	1.8	0.2	1.0	58.4
80	19.8	1.8	1.7	0.2	0.8	54.5
90	18.1	1.7	1.6	0.1	0.6	51.0
100	16.7	1.6	1.5	0.1	0.5	47.7
110	15.6	1.4	1.4	0.1	0.4	44.7

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70	49.8	4.7	3.4	1.3	5.6	110.0	0.00
80	45.0	4.3	3.2	1.0	4.9	105.4	0.00
90	41.1	3.9	3.1	0.8	4.2	101.1	0.00
100	37.9	3.6	3.0	0.6	3.7	96.5	0.00
110	35.2	3.3	2.8	0.5	3.3	92.1	0.00
120	32.9	3.1	2.7	0.4	2.9	88.0	0.00

Storage: Roof Storage

Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
129.2	0.13	4.0	8.8	13.6	0.00

Subdrainage Area: R1007A
Area (ha): 0.03
C: 1.00
Maximum Storage Depth: Roof 150 mm

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)
10	178.6	16.4	3.8	12.6	7.5	124.3
20	120.0	11.0	4.0	7.1	8.5	128.7
30	91.9	8.4	3.9	4.5	8.1	127.2
40	75.1	6.9	3.8	3.1	7.4	123.6
50	64.0	5.9	3.6	2.2	6.7	118.5
60	55.9	5.1	3.5	1.6	5.9	113.4
70	49.8	4.6	3.3	1.2	5.2	108.5
80	45.0	4.1	3.2	0.9	4.5	103.9
90	41.1	3.8	3.1	0.7	3.9	99.5
100	37.9	3.5	2.9	0.6	3.4	94.6
110	35.2	3.2	2.8	0.5	3.0	90.2
120	32.9	3.0	2.6	0.4	2.7	86.2

Storage: Roof Storage

Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
128.7	0.13	4.0	8.5	13.2	0.00

Subdrainage Area: R1006A
Area (ha): 0.07
C: 1.00
Maximum Storage Depth: Roof 150 mm

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)
10	178.6	36.7	5.9	30.8	18.5	127.8
20	120.0	24.7	6.2	18.5	22.1	135.1
30	91.9	18.9	6.3	12.6	22.7	136.2
40	75.1	15.5	6.2	9.2	22.2	135.1
50	64.0	13.2	6.1	7.0	21.1	132.9
60	55.9	11.5	6.0	5.5	19.8	130.3
70	49.8	10.2	5.9	4.4	18.4	127.5
80	45.0	9.3	5.7	3.5	16.9	124.4
90	41.1	8.5	5.6	2.9	15.7	120.6
100	37.9	7.8	5.4	2.4	14.4	117.0
110	35.2	7.2	5.2	2.0	13.3	113.5
120	32.9	6.8	5.1	1.7	12.2	110.2

Storage: Roof Storage

Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
136.2	0.14	6.3	22.7	29.6	0.00

Subdrainage Area: R1005A
Area (ha): 0.06
C: 1.00
Maximum Storage Depth: Roof 150 mm

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)
10	178.6	27.8	3.9	23.8	14.3	128.6
20	120.0	18.7	4.2	14.5	17.4	136.7
30	91.9	14.3	4.3	10.0	18.1	138.6
40	75.1	11.7	4.2	7.5	17.9	138.1
50	64.0	10.0	4.2	5.8	17.3	136.5
60	55.9	8.7	4.1	4.6	16.5	134.3
70	49.8	7.8	4.1	3.7	15.5	131.8
80	45.0	7.0	4.0	3.0	14.6	129.2
90	41.1	6.4	3.9	2.5	13.6	126.6
100	37.9	5.9	3.8	2.1	12.6	123.6
110	35.2	5.5	3.7	1.8	11.8	120.3
120	32.9	5.1	3.6	1.5	11.0	117.1

Storage: Roof Storage

Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
138.6	0.14	4.3	18.1	22.4	0.00

Subdrainage Area: R1004A
Area (ha): 0.04
C: 1.00
Maximum Storage Depth: Roof 150 mm

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)
10	178.6	18.4	3.9	14.5	8.7	125.6
20	120.0	12.3	4.0	8.3	10.0	130.7
30	91.9	9.4	4.0	5.5	9.8	130.0
40	75.1	7.7	3.9	3.8	9.2	127.4
50	64.0	6.6	3.8	2.8	8.3	123.7
60	55.9	5.7	3.7	2.1	7.5	118.9
70	49.8	5.1	3.5	1.6	6.8	114.3
80	45.0	4.6	3.4	1.3	6.0	109.8
90	41.1	4.2	3.2	1.0	5.3	105.6
100	37.9	3.9	3.1	0.8	4.7	101.6
110	35.2	3.6	3.0	0.6	4.1	97.5

Stormwater Management Calculations

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120	14.6	1.3	1.3	0.1	0.4	42.1	0.00
Storage: Roof Storage							
		Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
2-year Water Level		83.6	0.08	2.6	2.7	14.8	0.00
Subdrainage Area: R1000A Roof Area (ha): 0.11 Maximum Storage Depth: 150 mm C: 0.90							
tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	76.8	20.8	5.4	15.4	9.2	87.9	0.00
20	52.0	14.1	5.6	8.5	10.2	91.1	0.00
30	40.0	10.8	5.5	5.3	9.6	89.2	0.00
40	32.9	8.9	5.3	3.6	8.6	85.9	0.00
50	28.0	7.6	5.1	2.5	7.6	82.3	0.00
60	24.6	6.6	4.8	1.8	6.5	78.7	0.00
70	21.9	5.9	4.6	1.3	5.5	75.2	0.00
80	19.8	5.4	4.4	1.0	4.8	71.0	0.00
90	18.1	4.9	4.1	0.8	4.2	67.1	0.00
100	16.7	4.5	3.9	0.6	3.7	63.7	0.00
110	15.6	4.2	3.7	0.5	3.2	60.6	0.00
120	14.6	3.9	3.5	0.4	2.8	57.8	0.00
Storage: Roof Storage							
		Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
2-year Water Level		91.1	0.09	5.6	10.2	43.2	0.00
Subdrainage Area: R1001A Roof Area (ha): 0.31 Maximum Storage Depth: 150 mm C: 0.90							
tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	76.8	59.4	14.9	44.5	26.7	88.3	0.00
20	52.0	40.2	15.5	24.7	29.7	91.8	0.00
30	40.0	31.0	15.2	15.7	28.3	90.2	0.00
40	32.9	25.4	14.7	10.7	25.7	87.1	0.00
50	28.0	21.7	14.1	7.6	22.7	83.5	0.00
60	24.6	19.0	13.5	5.5	19.7	80.0	0.00
70	21.9	16.9	12.9	4.0	16.8	76.6	0.00
80	19.8	15.3	12.3	3.0	14.5	72.8	0.00
90	18.1	14.0	11.7	2.4	12.8	69.0	0.00
100	16.7	12.9	11.1	1.9	11.3	65.5	0.00
110	15.6	12.0	10.5	1.5	9.9	62.3	0.00
120	14.6	11.3	10.0	1.2	8.7	59.5	0.00
Storage: Roof Storage							
		Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
2-year Water Level		91.8	0.09	15.5	29.7	123.6	0.00
Subdrainage Area: R1002A Roof Area (ha): 0.03 Maximum Storage Depth: 150 mm C: 0.90							
tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	76.8	6.1	2.5	3.6	2.2	81.7	0.00
20	52.0	4.2	2.5	1.7	2.0	80.1	0.00
30	40.0	3.2	2.3	0.9	1.6	75.2	0.00
40	32.9	2.6	2.1	0.5	1.3	68.1	0.00
50	28.0	2.2	1.9	0.3	1.0	62.0	0.00
60	24.6	2.0	1.7	0.2	0.8	56.9	0.00
70	21.9	1.8	1.6	0.1	0.6	52.5	0.00
80	19.8	1.6	1.5	0.1	0.5	48.6	0.00
90	18.1	1.5	1.4	0.1	0.4	44.9	0.00
100	16.7	1.3	1.3	0.1	0.3	41.8	0.00
110	15.6	1.2	1.2	0.0	0.3	39.1	0.00
120	14.6	1.2	1.1	0.0	0.3	36.8	0.00
Storage: Roof Storage							
		Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
2-year Water Level		81.7	0.08	2.5	2.2	12.8	0.00
Subdrainage Area: R1003A Roof Area (ha): 0.15 Maximum Storage Depth: 150 mm C: 0.90							
tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	76.8	29.2	8.0	21.2	12.7	87.2	0.00
20	52.0	19.8	8.3	11.5	13.8	89.9	0.00
30	40.0	15.2	8.1	7.2	12.9	87.7	0.00
40	32.9	12.5	7.8	4.7	11.4	84.1	0.00
50	28.0	10.7	7.4	3.3	9.8	80.3	0.00
60	24.6	9.3	7.1	2.3	8.2	76.5	0.00
70	21.9	8.3	6.7	1.7	7.0	72.3	0.00
80	19.8	7.5	6.3	1.3	6.1	68.0	0.00
90	18.1	6.9	5.9	1.0	5.3	64.2	0.00
100	16.7	6.4	5.6	0.8	4.6	60.8	0.00
110	15.6	5.9	5.3	0.6	3.9	57.8	0.00
120	14.6	5.5	5.1	0.5	3.3	55.1	0.00
Storage: Roof Storage							
		Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check

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120	32.9	3.4	2.9	0.5	3.7	93.4	0.00
Storage: Roof Storage							
		Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
100-year Water Level		130.7	0.13	4.0	10.0	14.8	0.00
Subdrainage Area: R1000A Roof Area (ha): 0.11 Maximum Storage Depth: 150 mm C: 1.00							
tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	178.6	53.6	7.9	45.7	27.4	128.3	0.00
20	120.0	36.0	8.4	27.6	33.2	136.2	0.00
30	91.9	27.6	8.5	19.1	34.4	137.9	0.00
40	75.1	22.6	8.4	14.1	33.9	137.2	0.00
50	64.0	19.2	8.3	10.9	32.6	135.5	0.00
60	55.9	16.8	8.2	8.6	31.0	133.2	0.00
70	49.8	14.9	8.0	6.9	29.1	130.6	0.00
80	45.0	13.5	7.9	5.6	27.1	127.9	0.00
90	41.1	12.3	7.7	4.7	25.1	125.2	0.00
100	37.9	11.4	7.5	3.9	23.4	121.7	0.00
110	35.2	10.6	7.3	3.3	21.8	118.4	0.00
120	32.9	9.9	7.1	2.8	20.2	115.1	0.00
Storage: Roof Storage							
		Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
100-year Water Level		137.9	0.14	8.5	34.4	43.2	0.00
Subdrainage Area: R1001A Roof Area (ha): 0.31 Maximum Storage Depth: 150 mm C: 1.00							
tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	178.6	153.4	21.7	131.7	79.0	128.6	0.00
20	120.0	103.0	23.1	79.9	95.9	136.7	0.00
30	91.9	78.9	23.4	55.5	99.9	138.6	0.00
40	75.1	64.6	23.3	41.2	98.9	138.1	0.00
50	64.0	54.9	23.1	31.9	95.6	136.6	0.00
60	55.9	48.0	22.7	25.3	91.1	134.4	0.00
70	49.8	42.8	22.3	20.5	86.0	131.9	0.00
80	45.0	38.6	21.9	16.8	80.6	129.4	0.00
90	41.1	35.3	21.4	13.9	75.1	126.7	0.00
100	37.9	32.6	20.9	11.6	69.9	123.8	0.00
110	35.2	30.2	20.4	9.9	65.2	120.5	0.00
120	32.9	28.3	19.8	8.4	60.8	117.3	0.00
Storage: Roof Storage							
		Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
100-year Water Level		138.6	0.14	23.4	99.9	123.6	0.00
Subdrainage Area: R1002A Roof Area (ha): 0.03 Maximum Storage Depth: 150 mm C: 1.00							
tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	178.6	15.9	3.8	12.1	7.2	123.9	0.00
20	120.0	10.7	3.9	6.7	8.1	128.1	0.00
30	91.9	8.2	3.9	4.3	7.7	126.4	0.00
40	75.1	6.7	3.8	2.9	7.0	122.3	0.00
50	64.0	5.7	3.6	2.1	6.3	117.1	0.00
60	55.9	5.0	3.4	1.5	5.5	111.9	0.00
70	49.8	4.4	3.3	1.1	4.8	107.0	0.00
80	45.0	4.0	3.1	0.9	4.1	102.3	0.00
90	41.1	3.7	3.0	0.7	3.6	97.5	0.00
100	37.9	3.4	2.8	0.5	3.1	92.7	0.00
110	35.2	3.1	2.7	0.4	2.8	88.3	0.00
120	32.9	2.9	2.6	0.3	2.4	84.3	0.00
Storage: Roof Storage							
		Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
100-year Water Level		128.1	0.13	3.9	8.1	12.8	0.00
Subdrainage Area: R1003A Roof Area (ha): 0.15 Maximum Storage Depth: 150 mm C: 1.00							
tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	178.6	75.5	11.8	63.7	38.2	127.9	0.00
20	120.0	50.7	12.5	38.2	45.8	135.4	0.00
30	91.9	38.8	12.6	26.2	47.2	136.7	0.00
40	75.1	31.8	12.5	19.2	46.2	135.7	0.00
50	64.0	27.0	12.3	14.7	44.1	133.7	0.00
60	55.9	23.6	12.1	11.5	41.5	131.2	0.00
70	49.8	21.0	11.8	9.2	38.7	128.4	0.00
80	45.0	19.0	11.6	7.4	35.7	125.5	0.00
90	41.1	17.4	11.2	6.1	33.1	122.0	0.00
100	37.9	16.0	10.9	5.1	30.6	118.4	0.00
110	35.2	14.9	10.6	4.3	28.3	114.9	0.00
120	32.9	13.9	10.3	3.6	26.0	111.6	0.00
Storage: Roof Storage							
		Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check

Stormwater Management Calculations

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	(mm)	(m)	(L/s)	(cu. m)	(cu. m)	Check
2-year Water Level	89.9	0.09	8.3	13.8	60.8	0.00
SUMMARY TO OUTLET						
				Vrequired	Vavailable*	
Tributary Area			3,860 ha			
Total 2yr Flow to Sewer			523 L/s		77	1,023 m ³ Ok
Non-Tributary Area			0.395 ha			
Total 2yr Flow Uncontrolled			32 L/s			
Total Area			4,255 ha			
Total 2yr Flow			555 L/s			
Target			905 L/s			

Project #160401884, Richcraft Mer Bleue Road - Commercial Development Modified Rational Method Calculations for Storage

	(mm)	(m)	(L/s)	(cu. m)	(cu. m)	Check
100-year Water Level	136.7	0.14	12.6	47.2	60.8	0.00
SUMMARY TO OUTLET						
				Vrequired	Vavailable*	
Tributary Area			3,860 ha			
Total 100yr Flow to Sewer			812 L/s		617	1,023 m ³ Ok
Non-Tributary Area			0.395 ha			
Total 100yr Flow Uncontrolled			92 L/s			
Total Area			4,255 ha			
Total 100yr Flow			905 L/s			
Target			905 L/s			

C.3 Zurn Roof Drains





SPECIFICATION DRAINAGE

Control-Flo

Roof Drainage System



www.zurn.com



Control-Flo...Today's Successful Answer to More

THE ZURN "CONTROL-FLO CONCEPT"

Originally, Zurn introduced the scientifically-advanced "Control-Flo" drainage principle for dead-level roofs. Today, after thousands of successful applications in modern, large dead-level roof areas, Zurn engineers have adapted the comprehensive "Control-Flo" data to **sloped roof** areas.

WHAT IS "CONTROL-FLO"?

It is an advanced method of removing rain water off dead-level or sloped roofs. As contrasted with conventional drainage practices, which attempt to drain off storm water as quickly as it falls on the roof's surface, "Control-Flo" drains the roof at a controlled rate. Excess water accumulates on the roof under controlled conditions...then drains off at a lower rate after a storm abates.

CUTS DRAINAGE COSTS

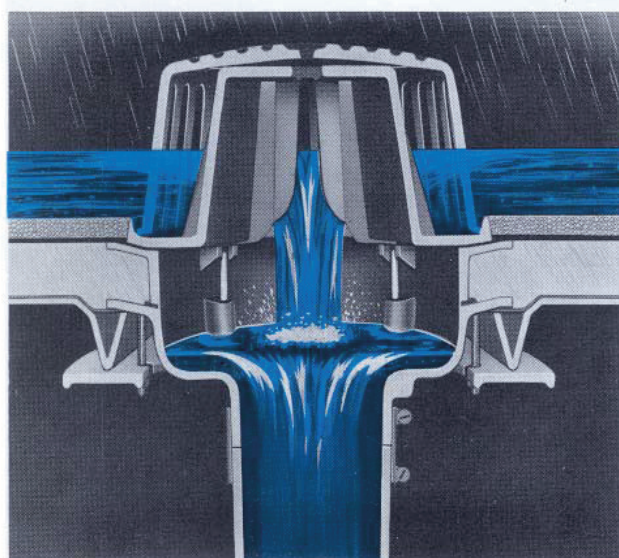
Fewer roof drains, smaller diameter piping, smaller sewer sizes, and lower installation costs are possible with a "Control-Flo" drainage system because roof areas are utilized as temporary storage reservoirs.

REDUCES PROBABILITY OF STORM DAMAGE

Lightens load on combination sewers by reducing rate of water drained from roof tops during severe storms thereby reducing probability of flooded sewers, and consequent backflow into basements and other low areas.

THANKS TO EXCLUSIVE ZURN "AQUA-WEIR" ACTION

Key to successful "Control-Flo" drainage is a unique scientifically-designed weir containing accurately calibrated notches with sides formed by parabolic curves which provide flow rates directly proportional to the head. Shape and size of notches are based on predetermined flow rates, and all factors involved in roof drainage to assure permanent regulation of drainage flow rates for specific geographic locations and rainfall intensities.

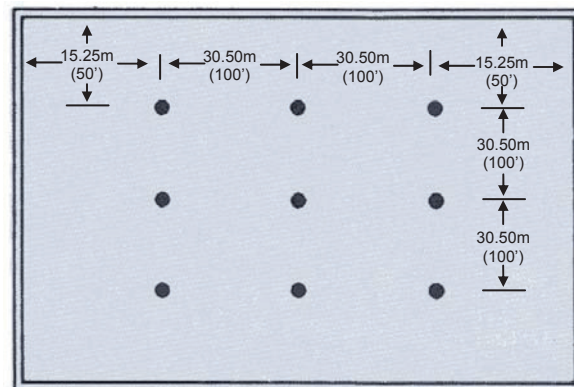


DEFINITION

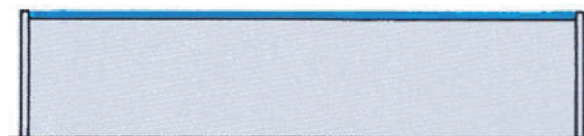
DEAD LEVEL ROOFS

DIAGRAM "A"

A dead-level roof for purposes of applying the Zurn "Control-Flo" drainage principle is one which has been designed for zero slope across its entire surface. Measurements shown are for maximum distances.



(Plan View)



(Section View)

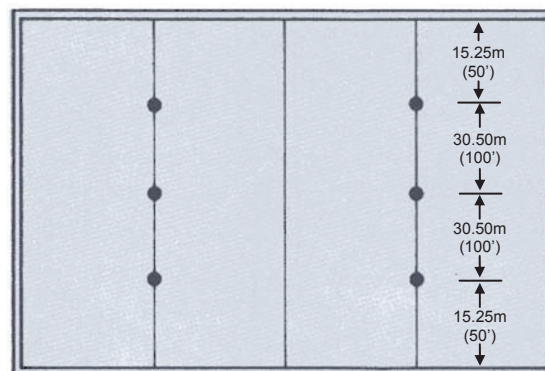
SLOPED ROOFS

DIAGRAM "B"

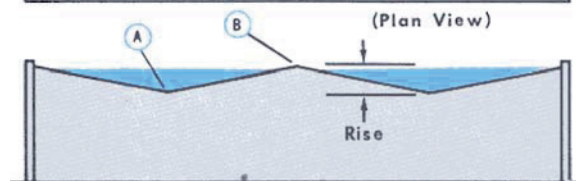
A sloped roof is one designed commonly with a shallow slope. The Zurn "Control-Flo" drainage system can be applied to any slope which results in a total rise up to 152mm (6").

The total rise of a roof as calculated for "Control-Flo" application is defined as the vertical increase in height in inches, from the low point or valley of a sloping roof (A) to the top of the sloping section (B). (Example: a roof that slopes 3mm (1/8") per foot having a 7.25m (24') span would have a rise of 7.25m x 3mm or 76mm (24' x 1/8" or 3").

Measurements shown are for maximum distances.



(Plan View)

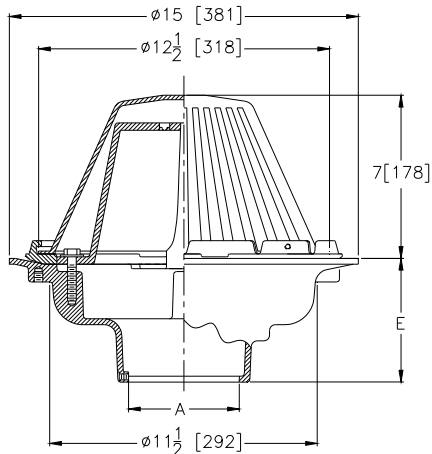


(Section View)

Economical Roof Drainage Installations



SPECIFICATION DATA



ENGINEERING SPECIFICATION: ZURN Z-105 "Control-Flo" roof drain for dead-level or sloped roof construction, Dura-Coated cast iron body. "Control-Flo" weir shall be linear functioning with integral membrane flashing clamp/gravel guard and Poly-Dome. All data shall be verified proportional to flow rates.

ROOF DESIGN RECOMMENDATIONS

Basic roofing design should incorporate protection that will prevent roof overloading by installing adequate overflow scuppers in parapet walls.

GENERAL INFORMATION

The "Control-Flo" roof drainage data is tabulated for four areas (232.25m² (2500 sq. ft.), 464.502m² (5000 sq. ft.), 696.75m² (7500 sq. ft.), 929m² (10,000 sq. ft.) notch areas ratings) for each locality. For each notch area rating the maximum discharge in L.P.M. (G.P.M.) - draindown in hours, and maximum water depth at the drain in inches for a dead level roof — 51mm (2 inch) rise — 102mm (4 inch) rise and 152mm (6 inch) rise—are tabulated. The rise is the total change in elevation from the valley to the peak. Values for areas, rise or combination thereof other than those listed, can be arrived at by extrapolation. All data listed is based on the fifty-year return frequency storm. In other words the maximum conditions as listed will occur on the average of once every fifty years.

NOTE: The tabulated "Control-Flo" data enables the individual engineer to select his own design limiting condition. The limiting condition can be draindown time, roof load factor, or maximum water depth at the drain. If draindown time is the limiting factor because of possible freezing conditions, it must be recognized that the maximum time listed will occur on the average of once every 50 years and would most likely be during a heavy summer thunder storm. Average winter draindown times would be much shorter in duration than those listed.

GENERAL RECOMMENDATIONS

On sloping roofs, we recommend a design depth referred to as an equivalent depth. An equivalent depth is the depth of water attained at the drains that results in the same roof stresses as those realized on a dead-level roof. In all cases this equivalent depth is almost equal to that attained by using the same notch area rating for the different rises to 152mm (6"). With the same depth of water at the drain the roof stresses will decrease with increasing total rise. Therefore, it would be possible to have a depth in excess of 152mm (6") at the drain on a sloping roof without exceeding stresses normally encountered in a 152mm (6") depth on a dead-level roof. However, it is recommended that scuppers be placed to limit the maximum water depth on any roof to 152mm (6") to prevent the overflow of the weirs on the drains and consequent overloading of drain piping. In the few cases where the data shows a flow rate in excess of 136 L.P.M. (30 G.P.M.) if all drains and drain lines are sized according to recommendations, and the one storm in fifty years occurs, the only consequence will be a brief flow through the scuppers or over-flow drains.

NOTE: An equivalent depth is that depth of water attained at the drains at the lowest line or valley of the roof with all other conditions such as notch area and rainfall intensity being equal. For Toronto, Ontario a notch area rating of 464.50m² (5,000 sq. ft.) results in a 74mm (2.9 inch) depth on a dead level roof for a 50-year storm. For the same notch area and conditions, equivalent depths for a 51mm (2"), 102mm (4") and 152mm (6") rise respectively on a sloped roof would be 86mm (3.4"), 104mm (4.1") and 124mm (4.9"). Roof stresses will be approximately equal in all cases.



Control-Flo Drain Selection Is Quick and Easy...

The exclusive Zurn "Selecta-Drain" Chart (pages 8—11) tabulates selection data for 34 localities in Canada. Proper use of this chart constitutes your best assurance of sure, safe, economical application of Zurn "Control-Flo" systems for your specific geographical area. If the "Selecta-Drain" Chart does not cover your specific design criteria, contact Zurn Industries Limited, Mississauga, Ontario, for additional data for your locality. Listed below is additional information pertinent to proper engineering of the "Control-Flo" system.

ROOF USED AS TEMPORARY RETENTION

The key to economical "Control-Flo" is the utilization of large roof areas to temporarily store the maximum amount of water without overloading average roofs or creating excessive draindown time during periods of heavy rainfall. The data shown in the "Selecta-Drain" Chart enables the engineer to select notch area ratings from 232.25 m² (2,500 ft.²) to 929m² (10,000 ft.²) and to accurately predict all other design factors such as maximum roof load, L.P.M. (G.P.M.) discharge, draindown time and water depth at the drain. Obviously, as design factors permit the notch area rating to increase the resulting money saved in being able to use small leaders and drain lines will also increase.

ROOF LOADING AND RUN-OFF RATES

The four values listed in the "Selecta-Drain" Chart for notch area ratings for different localities will normally span the range of good design. If areas per notch below 232.25m² (2,500 ft.²) are used considerable economy of the "Control-Flo" concept is being lost. The area per notch is limited to 929m² (10,000 ft.²) to keep the drain-down time within reasonable limits. Extensive studies show that stresses due to water load on a sloping roof for any fixed set of conditions are very nearly the same as those on a dead-level roof. A sloping roof tends to concentrate more water in the valleys and increase the water depth at this point. The greater depth around the drain leads to a faster run-off rate, particularly a faster early run-off rate. As a result, the total volume of water stored on the roof is less, and the total load on the sloping roof is less. By using the same area on the sloping roof as on the dead-level roof the increase in roof stresses due to increased water depth in the valleys is offset by the decrease in the total load due to less water stored. The net result of the maximum roof stress is approximately the same for any single span rise and fixed set of conditions. A fixed set of conditions, would be the same notch area, the same frequency store, and the same locality.

SPECIAL CONSIDERATIONS FOR STRUCTURAL SAFETY: Normal practice of roof design is based on 18kg (40 lbs.) per 929 cm² (sq ft.). (Subject to local codes and by-laws.) Thus it is extremely important that design is in accordance with normal load factors so deflection will be slight enough in any bay to prevent progressive deflection which could cause water depths to load the roof beyond its design limits.

ADDITIONAL NOTCH RATINGS

The 'Selecta-Drain' Chart along with Tables I and II enables the engineer to select "Control-Flo" Drains and drain pipe sizes for most Canadian applications. These calculations are computed for a proportional flow weir that is sized to give a flow of 23 L.P.M. (5 G.P.M.) per inch of head. The 23 L.P.M. (5 G.P.M.) per inch of head notch opening is selected as the bases of design as it offers the most economical installation as applied to actual rainfall experienced in Canada.

Should you require design criteria for locations outside of Canada or for special project applications please contact Zurn Industries Limited, Mississauga, Ontario.

LEADER AND DRAIN PIPE SIZING

Since all data in the "Selecta-Drain" Chart is based on the 50-year-storm it is possible to exceed the water depth listed in these charts if a 100-year or 1000-year storm would occur. Therefore, for good design it is recommended that scuppers or other methods be used to limit water depth to the design depth and tables I and II be used to size the leaders and drain pipes. If the roof is capable of supporting more water than the design depth it is permissible to locate the scuppers or other overflow means at a height that will allow a greater water depth on the roof. However, in this case the leader and drain pipes should be sized to handle the higher flow rates possible based on a flow rate of 23 L.P.M. (5 G.P.M.) per inch of depth at the drain.

PROPER DRAIN LOCATION

The following good design practice is recommended for selecting the proper number of "Control-Flo" drains for a given area. **On dead-level roofs**, drains should be located no further than 15.25m (50 feet) from edge of roof and no further than 30.50m (100 feet) between drains. See diagram "A" page 2. **On sloping roofs**, drains should be located in the valleys at a distance no greater than 15.25m (50 feet) from each end of the valleys and no further than 30.50m (100 feet) between drains. See diagram "B" page 2. Compliance with these recommendations will assure good run off regardless of wind direction.



Saves Specification Time, Assures Proper Application

QUICK, EASY SELECTION

Using the "Selecta-Drain" Chart (pages 9—13) in combination with the steps and examples appearing below, should save you countless hours in engineering specification time. This vast compilation of data is related to the proper selection of drains for 34 cities. All cities in alphabetical order by province. If a specific city does not appear in the tabulation, chooses the city nearest your area and select the proper drain using these factors.

3 EASY STEPS...

AND 3 TYPICAL EXAMPLES FOR APPLICATION OF SURE, SCIENTIFIC CONTROL OF DRAINAGE FROM DEAD-LEVEL AND SLOPING ROOFS WITH THE ZURN CONCEPT.

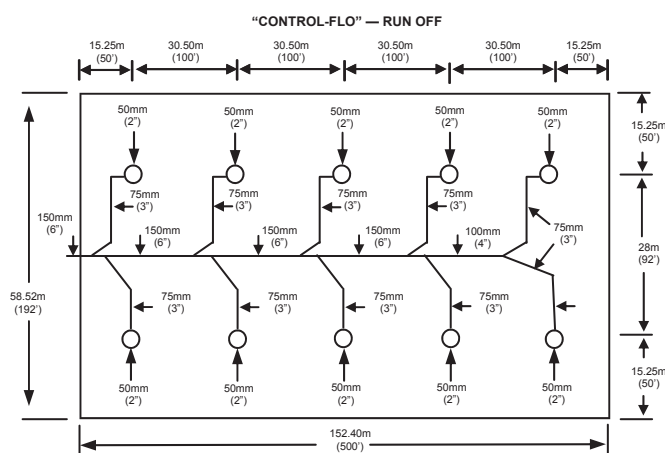
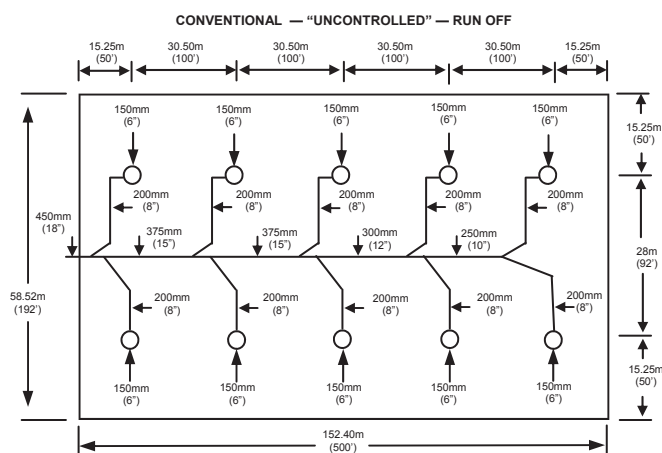
NOTE: Where roof area to be drained is adjacent to one or more vertical walls projecting above the roof, then a percentage of the of the wall(s) must be added to the roof area in determining total roof area to be drained.

TORONTO, ONTARIO		DEAD-LEVEL ROOF	102mm (4 INCH) SLOPE	152mm (6 INCH) SLOPE
1	Determine total roof area or individual areas when roof is divided by expansion joints or peaks in the case of sloping roof.	Roof Area: 56.52m x 152.40m = 8918.40m ² (192ft x 500ft = 96,000 sq. ft.) (See Z105 layout bottom of this page.)	3 Individual Roof Areas: 19.50m x 152.40m = 2972.80m ² (64ft x 500ft = 32,000 sq. ft.) Valleys 152.40m (500ft) long 3 x 2972.80 = 8918.40m ² (3 x 32,000 = 96,000 sq. ft.)	2 Individual Roof Areas: 29.87m x 152.40m = 4552m ² (98ft x 500ft = 49,000 sq. ft.) Valleys 152.40m (500ft) long 2 x 4552 = 9104m ² (2 x 49,000 = 98,000 sq. ft.)
2	Divide roof area or individual areas by Zurn Notch Area Rating selected to obtain the total number of notches required.	Zurn Notch Area Rating selected for Toronto = 464.50m ² (5,000 sq. ft.) from "Selecta-Drain Chart, page 11. Total Roof Area = 8918.40m ² (96,000 sq. ft.) Entire roof. 464.50m ² (5,000 sq. ft.) notch area = 19.2 notches—USE 20.	Zurn Notch Area Rating selected for Toronto = 464.50m ² (5,000 sq. ft.) from "Selecta-Drain Chart, page 11. Total Roof Area = 2972.80m ² (32,000 sq. ft.) Each area. 464.50m ² (5,000 sq. ft.) notch area = 6.4 notches—USE 7 PER AREA.	Zurn Notch Area Rating selected for Toronto = 464.50m ² (5,000 sq. ft.) from "Selecta-Drain Chart, page 11. Total Roof Area = 4552m ² (49,000 sq. ft.) Each area. 464.50m ² (5,000 sq. ft.) notch area = 9.8 notches—USE 10 PER AREA.
3	Determine total number of drains required by not exceeding maximum spacing dimensions in the preceding instructions. See Diagrams "A" or "B", page 2. Divide total number of notches required to determine the number of notches per drain. Note maximum water depth at drain and use this dimension to determine scupper height. Maximum scupper height to be used is 152mm (6"). <i>Use this flow rate to size leaders and drain lines.</i>	*10 drains required. All drains must have two notches each for a total of 20 notches. Flow rate is 66 L.P.M. (14.5 G.P.M.) per notch. Size leaders for 2 notch weirs for a flow rate of 66 L.P.M. (14.5 G.P.M.) 50 mm (two inch) pipe size leaders required. Maximum water depth and scupper height is 74mm (2.9"). Requires 19 hours drain-down time maximum. For drain, vertical and horizontal pipe sizing data see Tables I and II on page 6 and 7.	**5 drains per area required located in the valleys 15.25m (50ft.) from each end with 3 in the middle at 30.50m (100ft.) spacings. Two drains on ends with two notches—3 drains in middle on notch each for a total of 7 notches. Maximum flow rate 93 L.P.M. (20.5 G.P.M.) per notch. Leader size 50mm (2") for single notch weirs—75mm (3") notch weirs. Maximum water depth and scupper height is 104mm (4.1"). Requires 11 hours draindown time maximum. For drain, vertical and horizontal pipe sizing data see Tables I and II on page 6 and 7.	**5 drains per area required located in the valleys 15.25m (50ft.) from each end with 3 in the middle at 30.50m (100ft.) spacing in the middle. 10 notches are required therefore all drains must have two notches. Flow rate is 111 L.P.M. (24.5 G.P.M.) per notch. Size all leaders for 2 notch weirs. 75mm (3") pipe size required. Maximum water depth and scupper height is 124mm (4.9"). Requires 9 hours draindown time maximum. For drain, vertical and horizontal pipe sizing data see Tables I and II on page 6 and 7.

*See Diagram "A" page 2 for recommended drain placement.
**See Diagram "B" page 2 for recommended drain placement.

*See Diagram "A" page 2 for recommended drain placement.
**See Diagram "B" page 2 for recommended drain placement.

DEAD LEVEL ROOF 6mm (1/4") PER FT. SLOPE STORM DRAIN





Select The Proper Vertical Drain Leaders

ROOF DRAINAGE DATA

The flow rate for any design condition can be easily read from the data contained on the following pages; the tabulations shown below (and on the opposite page) can be used to simplify selection of drain line sizes.

TABLE 1 - SUGGESTED RELATION OF DRAIN OUTLET AND VERTICAL LEADER SIZE TO ZURN CONTROL-FLO ROOF DRAINS (BASED ON NATIONAL PLUMBING CODE ASA -A40.8 DATA ON VERTICAL LEADERS).

No. of Notches in Drain	Max. Flow per Notch in L.P.M. (G.P.M.)		
	Pipe Size		
	50mm (2")	75mm (3")	100mm (4")
1	136* (30*)	—	—
2	68 (15)	136* (30*)	—
3	45 (10)	136* (30*)	—
4	—	105 (23)	136* (30*)
5	—	82 (18)	136* (30*)
6	—	68 (15)	136* (30*)

*Maximum flow obtainable from 1 notch with 152mm (6") water depth at drain.

Table 1 should be used to select **vertical drain** leaders which at the same time establishes the drain outlet size. This table illustrates the minimum flow per notch in L.P.M. (G.P.M.) Since the Z-105 drain is available with a minimum of one and a maximum of six notches, calculations have already been made and are listed in this table for any quantity of weir notch openings established in your design. It was determined ten drains with two notches each weir would be required in the Dead-Level Roof example on page 5. A 66 L.P.M. (14.5 G.P.M.) discharge per notch flow rate was also established.

Once this design criteria has been determined it will be the key to the proper selection of all drain outlet sizes, vertical and horizontal storm drain sizes in Table I and II. Enter the column "Number of Notches in Drain", Table I, read down the column to the figure 2 which indicates two notches in weir, then read across until you reach a figure equal to or closest figure in excess of 66 L.P.M. (14.5 G.P.M.) You will find fifteen in the column under 50mm (2") which represents the pipe size. Therefore all drain outlets and vertical leaders are 50mm (2") size.

Let us digress for a moment assuming a specific structure requires a total of six drains each containing a weir with a different number of notches. One with 1, one with 2, etc. Table 1 discloses the pipe size for one notch is 50mm (2"), two notch is 50mm (2"), three notch is 75mm (3"), four notch is 75mm (3"), five notch is 75mm (3") and six notch is 75mm (3") as they all equal or closely exceed the 66 L.P.M. (14.5 G.P.M.) design.

NOTE: Although pipe size calculations should be based on accumulated flow rate, local by-laws should be referred to for minimum pipe size requirements and roof drain spacing.

TABLE II should be used to select **horizontal** storm drain piping. Use the same flow rate 66 L.P.M. (14.5 G.P.M.) used to establish the vertical leaders to size the storm drainage system and main storm drain. Let us assume the ten drains each with two notch weirs were actually on the roof in two separate lines of five drains each and joined at a common point before leaving the building. Since Table II includes 3mm (1/8"), 6mm (1/4") and 13mm (1/2") per foot slope, let us **use 6mm (1/4") as our basis for selection** which will take us to the centre section. Starting with the first of five drains we enter the extreme left column in Table II and read down to the figure 2 since this drain has two notches in weir, read across horizontally and the size of first section of horizontal storm drain is 75mm (3") between 1st and 2nd drain, return to left hand column proceed reading down until you reach figure 4 then read across horizontally and the pipe size will be 100mm (4") between 2nd and 3rd drain, 100mm (4") between 3rd and 4th and 125mm (5") (if available) between 4th and 5th. If not available use 150mm (6"). (You may be tempted to use 100mm (4") since the capacity is close. We recommend you go to the larger size.) Pipe size leaving 5th drain would be 150mm (6"). The same sizing would hold true for the second line of five drains. Since both columns of five drains each are being joined together before leaving the building there will be total of twenty notches discharging into the main building storm sewer. Enter left hand column Table II, read down until you reach the figure twenty, then read across horizontally to the 6mm (1/4") per 305mm (1') slope column and you will see a 150mm (6") storm drain will handle the job adequately. The same procedure should be followed for sloped roof installations. The above method of sizing was done to better acquaint you with Table II and its use. The more economical and practical way of laying out and installing this same job is illustrated in the control-flo layout shown on bottom of page 5.

NOTE: Although pipe size calculations should be based on accumulated flow rates, local by-laws should be referred to for minimum pipe size requirements and roof drain spacing.

Select Proper Horizontal Storm Drain Piping



Table II — SUGGESTED RELATION OF HORIZONTAL STORM DRAIN SIZE TO ZURN CONTROL-FLO ROOF DRAINAGE

Total No. of Notches Discharging to Storm Drain	MAX. FLOW PER NOTCH IN L.P.M. (G.P.M.)								MAX. FLOW PER NOTCH IN L.P.M. (G.P.M.)								MAX. FLOW PER NOTCH IN L.P.M. (G.P.M.)							
	Storm Drain Size 3mm (1/8") per 305mm (1') Slope								Storm Drain Size 6mm (1/4") per 305mm (1') Slope								Storm Drain Size 13mm (1/2") per 305mm (1') Slope							
	75 (3")	100 (4")	125 (5")	150 (6")	200 (8")	250 (10")	300 (12")	375 (15")	75 (3")	100 (4")	125 (5")	150 (6")	200 (8")	250 (10")	300 (12")	75 (3")	100 (4")	125 (5")	150 (6")	200 (8")	250 (10")	300 (12")		
1	136* (30*)	—	—	—	—	—	—	—	136* (30*)	—	—	—	—	—	—	136* (30*)	—	—	—	—	—	—		
2	77 (17)	136* (30*)	—	—	—	—	—	—	109 (24)	136* (30*)	—	—	—	—	—	136* (30*)	—	—	—	—	—	—		
3	50 (11)	118 (26)	136* (30*)	—	—	—	—	—	73 (16)	136* (30*)	—	—	—	—	—	100 (22)	136* (30*)	—	—	—	—	—		
4	36 (8)	86 (19)	136* (30*)	—	—	—	—	—	55 (12)	127 (28)	136* (30*)	—	—	—	—	77 (17)	136* (30*)	—	—	—	—	—		
5	—	65 (15)	127* (28*)	136* (30*)	—	—	—	—	—	100 (22)	136* (30*)	—	—	—	—	59 (13)	136* (30*)	—	—	—	—	—		
6	—	59 (13)	105 (23)	136* (30*)	—	—	—	—	—	82 (18)	136* (30*)	—	—	—	—	50 (11)	118 (26)	136* (30*)	—	—	—	—		
7	—	50 (11)	91 (20)	136* (30*)	—	—	—	—	—	73 (16)	127 (28)	136* (30*)	—	—	—	—	100 (22)	136* (30*)	—	—	—	—		
8	—	—	77 (17)	127 (28)	136* (30*)	—	—	—	—	64 (14)	114 (25)	136* (30*)	—	—	—	—	86 (19)	136* (30*)	—	—	—	—		
9	—	—	68 (15)	114 (25)	136* (30*)	—	—	—	—	55 (12)	100 (22)	136* (30*)	—	—	—	—	77 (17)	136* (30*)	—	—	—	—		
10	—	—	64 (14)	100 (22)	136* (30*)	—	—	—	—	—	91 (20)	136* (30*)	—	—	—	—	68 (15)	123 (27)	136* (30*)	—	—	—		
11	—	—	55 (12)	91 (20)	136* (30*)	—	—	—	—	—	82 (18)	132 (29)	136* (30*)	—	—	—	64 (14)	114 (25)	136* (30*)	—	—	—		
12	—	—	—	82 (18)	136* (30*)	—	—	—	—	—	73 (16)	118 (26)	136* (30*)	—	—	—	59 (13)	105 (23)	136* (30*)	—	—	—		
13	—	—	—	77 (17)	136* (30*)	—	—	—	—	—	68 (15)	109 (24)	136* (30*)	—	—	—	55 (12)	95 (21)	136* (30*)	—	—	—		
14	—	—	—	73 (16)	136* (30*)	—	—	—	—	—	64 (14)	100 (22)	136* (30*)	—	—	—	—	86 (19)	136* (30*)	—	—	—		
15	—	—	—	68 (15)	136* (30*)	—	—	—	—	—	59 (13)	95 (21)	136* (30*)	—	—	—	—	82 (18)	132 (29)	136* (30*)	—	—		
16	—	—	—	64 (14)	136* (30*)	—	—	—	—	—	—	91 (20)	136* (30*)	—	—	—	—	77 (17)	123 (27)	136* (30*)	—	—		
17	—	—	—	59 (13)	127 (28)	136* (30*)	—	—	—	—	—	82 (18)	136* (30*)	—	—	—	—	73 (16)	118 (26)	136* (30*)	—	—		
18	—	—	—	55 (12)	118 (26)	136* (30*)	—	—	—	—	—	77 (17)	136* (30*)	—	—	—	—	68 (15)	109 (24)	136* (30*)	—	—		
19	—	—	—	—	114 (25)	136* (30*)	—	—	—	—	—	73 (16)	136* (30*)	—	—	—	—	64 (14)	105 (23)	136* (30*)	—	—		
20	—	—	—	—	109 (24)	136* (30*)	—	—	—	—	—	68 (15)	136* (30*)	—	—	—	—	59 (13)	100 (22)	136* (30*)	—	—		
23	—	—	—	—	91 (20)	136* (30*)	—	—	—	—	—	64 (14)	132 (29)	136* (30*)	—	—	—	55 (12)	86 (19)	136* (30*)	—	—		
25	—	—	—	—	86 (19)	136* (30*)	—	—	—	—	—	59 (13)	123 (27)	136* (30*)	—	—	—	—	77 (17)	136* (30*)	—	—		
30	—	—	—	—	73 (16)	127 (28)	136* (30*)	—	—	—	—	—	100 (22)	136* (30*)	—	—	—	—	64 (14)	136* (30*)	—	—		
35	—	—	—	—	59 (13)	109 (24)	136* (30*)	—	—	—	—	—	86 (19)	136* (30*)	—	—	—	—	55 (12)	123 (27)	136* (30*)	—		
40	—	—	—	—	55 (12)	95 (21)	136* (30*)	—	—	—	—	—	77 (17)	136* (30*)	—	—	—	—	—	105 (23)	136* (30*)	—		
45	—	—	—	—	—	86 (19)	136* (30*)	—	—	—	—	—	68 (15)	123 (27)	136* (30*)	—	—	—	—	95 (21)	136* (30*)	—		
50	—	—	—	—	—	77 (17)	123 (27)	136* (30*)	—	—	—	—	59 (13)	109 (24)	136* (30*)	—	—	—	—	86 (19)	136* (30*)	—		
55	—	—	—	—	—	68 (15)	114 (25)	136* (30*)	—	—	—	—	—	100 (22)	136* (30*)	—	—	—	—	77 (17)	136* (30*)	—		
60	—	—	—	—	—	64 (14)	105 (23)	136* (30*)	—	—	—	—	—	91 (20)	136* (30*)	—	—	—	—	68 (15)	127 (28)	136* (30*)		
65	—	—	—	—	—	59 (13)	95 (21)	136* (30*)	—	—	—	—	—	82 (18)	136* (30*)	—	—	—	—	64 (14)	118 (26)	136* (30*)		
70	—	—	—	—	—	55 (12)	91 (20)	136* (30*)	—	—	—	—	—	77 (17)	127 (28)	—	—	—	—	59 (13)	109 (24)	136* (30*)		

*Maximum flow obtainable from 1 notch with 152mm (6") water depth at drain.



Select Proper Horizontal Storm Drain Piping

TABLE III - TO BE USED WHEN ROOF STORM WATER RUN OFF AND OTHER SURFACE WATER RUN OFF IS BEING CONSOLIDATED INTO ONE COMMON MAIN HORIZONTAL STORM SEWER.

Flow capacity of vertical leaders litres per minute (gallons per minute)

Pipe Size	Maximum Capacity L.P.M. (G.P.M.)
50mm (2")	136 (30)
75mm (3")	409 (90)
100mm (4")	864 (190)
†125mm (5")	1582 (348)
150mm (6")	2550 (561)

†In some areas 125mm (5") drainage pipe may not be available.

Flow capacity of horizontal storm sewers litres per minute (gallons per minute).

Pipe Size	Slope per 305mm (1'0")		
	3mm (1/8")	6mm (1/4")	13mm (1/2")
75mm (3")	163 (36)	232 (51)	327 (72)
100mm (4")	355 (78)	505 (111)	714 (157)
†125mm (5")	646 (142)	914 (201)	1291 (284)
150mm (6")	1050 (231)	1487 (327)	2100 (462)
200mm (8")	2264 (498)	3205 (705)	4528 (996)
250mm (10")	4100 (902)	5796 (1275)	8201 (1804)
300mm (12")	6669 (1467)	9437 (2076)	13338 (2934)
375mm (15")	12120 (2666)	17157 (3774)	24239 (5332)

Note: Although pipe size calculations should be based on accumulated flow rate, local by-laws should be referred to for minimum pipe size requirements and roof drain spacing.

SCUPPER AND OVERFLOW DRAINS

Roofing members and understructures, weakened by seepage and rot resulting from improper drainage and roof construction can give away under the weight of rapidly accumulated water during flash storms. Thus, it is recommended, and often required by building codes, to install scuppers and overflow drains in parapet-type roofs. Properly selected and sized scuppers and overflow drains are vital to a well-engineered drainage system to prevent excessive loading, erosion, seepage and rotting.

Selecta-Drain Chart



LOCATION	SQUARE METRE (SQUARE FOOT)	ROOF LOAD FACTOR KGS. (LBS.)	TOTAL ROOF SLOPE											
			DEAD LEVEL			51mm (2") RISE			102mm (4") RISE			152mm (6") RISE		
			L.P.M. (G.P.M.) Discharge	Draindown Time Hrs.	mm (in.) Water Depth	L.P.M. (G.P.M.) Discharge	Draindown Time Hrs.	mm (in.) Water Depth	L.P.M. (G.P.M.) Discharge	Draindown Time Hrs.	mm (in.) Water Depth	L.P.M. (G.P.M.) Discharge	Draindown Time Hrs.	mm (in.) Water Depth
Calgary, Alberta	232 (2,500)	4.7 (10.4)	45.5 (10)	7	51 (2)	57 (12.5)	6	63.5 (2.5)	72.5 (16)	4	81.5 (3.2)	86.5 (19)	3.2	96.5 (3.8)
	465 (5,000)	5.9 (13)	57 (12.5)	17	63.5 (2.5)	66 (14.5)	14	73.5 (2.9)	82 (18)	9	91.5 (3.6)	97.5 (21.5)	7.5	109 (4.3)
	697 (7,500)	6.4 (14)	61.5 (13.5)	28	68.5 (2.7)	72.5 (16)	22	81.5 (3.2)	88.5 (19.5)	15	99 (3.9)	104.5 (23)	12	117 (4.6)
	929 (10,000)	6.8 (15.1)	66 (14.5)	38	73.5 (2.9)	77.5 (17)	31	86.5 (3.4)	93 (20.5)	22	104 (4.1)	109 (24)	17	122 (4.8)
Edmonton, Alberta	232 (2,500)	4.5 (9.9)	43 (9.5)	7	48.5 (1.9)	57 (12.5)	6	63.5 (2.5)	72.5 (16)	4	81.5 (3.2)	82 (18)	3	91.5 (3.6)
	465 (5,000)	5.9 (13)	57 (12.5)	17	63.5 (2.5)	68 (15)	14.5	76 (3)	84 (18.5)	9.5	94 (3.7)	97.5 (21.5)	7.5	109 (4.3)
	697 (7,500)	6.6 (14.5)	63.5 (14)	28	71 (2.8)	75 (16.5)	24	84 (3.3)	97.5 (21.5)	16	104 (4.1)	107 (23.5)	12	119.5 (4.7)
	929 (10,000)	7.1 (15.6)	68 (15)	38	76 (3.0)	79.5 (17.5)	32	89 (3.5)	100 (22)	22	112 (4.4)	113.5 (25)	18	127 (5.0)
Penticton, British Columbia	232 (2,500)	3.8 (8.3)	36.5 (8)	6	40.5 (1.6)	38.5 (8.5)	4	43 (1.7)	52.5 (11.5)	3	58.5 (2.3)	61.5 (13.5)	2.3	68.5 (2.7)
	465 (5,000)	4.0 (8.8)	38.5 (8.5)	13	43 (1.7)	41 (9)	9	45.5 (1.8)	57 (12.5)	6	63.5 (2.5)	68 (15)	5	76 (3)
	697 (7,500)	4.2 (9.3)	41 (9)	21	45.5 (1.8)	43 (9.5)	14.5	48.5 (1.9)	61.5 (13.5)	10.5	68.5 (2.7)	72.5 (16)	8	81.5 (3.2)
	929 (10,000)	4.2 (9.3)	41 (9)	27	45.5 (1.8)	45.5 (10)	20	51 (2)	63.5 (14)	14	71 (2.8)	75 (16.5)	11	84 (3.3)
Vancouver, British Columbia	232 (2,500)	3.3 (7.3)	32 (7)	5.5	35.5 (1.4)	38.5 (8.5)	4	43 (1.7)	47.5 (10.5)	2.8	53.5 (2.1)	57 (12.5)	2	63.5 (2.5)
	465 (5,000)	4.0 (8.8)	38.5 (8.5)	13	43 (1.7)	45.5 (10)	10	51 (2)	57 (12.5)	6	63.5 (2.5)	68 (15)	5	76 (3)
	697 (7,500)	4.5 (9.9)	43 (9.5)	22	48.5 (1.9)	50 (11)	17	56 (2.2)	63.5 (14)	11	71 (2.8)	75 (16.5)	8.5	84 (3.3)
	929 (10,000)	4.9 (10.9)	47.5 (10.5)	30	53.5 (2.1)	54.5 (12)	24	61 (2.4)	68 (15)	15	76 (3)	79.5 (17.5)	12	89 (3.5)
Victoria, British Columbia	232 (2,500)	3.3 (7.3)	32 (7)	5.5	35.5 (1.4)	38.5 (8.5)	4	43 (1.7)	43 (9.5)	2.5	48.5 (1.9)	54.5 (12)	2	61 (2.4)
	465 (5,000)	4.0 (8.8)	38.5 (8.5)	13	43 (1.7)	45.5 (10)	10	51 (2)	54.5 (12)	6	61 (2.4)	68 (15)	5	76 (3)
	697 (7,500)	4.5 (9.9)	43 (9.5)	22	48.5 (1.9)	50 (11)	16	56 (2.2)	59 (13)	10	66 (2.6)	75 (16.5)	8	84 (3.3)
	929 (10,000)	4.7 (10.4)	45.5 (10)	30	51 (2)	54.5 (12)	23	61 (2.4)	63.5 (14)	14	71 (2.8)	79.5 (17.5)	12	89 (3.5)
Brandon, Manitoba	232 (2,500)	5.9 (13)	57 (12.5)	8	63.5 (2.5)	68 (15)	7	76 (3)	82 (18)	4.5	91.5 (3.6)	92.5 (21)	3.5	106.5 (4.2)
	465 (5,000)	7.3 (16.1)	73 (16)	20	81.5 (3.2)	84 (18.5)	17	94 (3.7)	97.5 (21.5)	11	109 (4.3)	113.5 (25)	8.5	127 (5)
	697 (7,500)	8.3 (18.2)	79.5 (17.5)	32	89 (3.5)	93 (20.5)	27	104 (4.1)	107 (23.5)	19	119.5 (4.7)	125 (27.5)	15	139.5 (5.5)
	929 (10,000)	9.0 (19.8)	86.5 (19)	43	96.5 (3.8)	100 (22)	38	112 (4.4)	113.5 (25)	26	127 (5.0)	132 (29)	21	147.5 (5.8)
Winnipeg, Manitoba	232 (2,500)	4.7 (10.4)	45.5 (10)	7	51 (2)	57 (12.5)	6	63.5 (2.5)	75 (16.5)	4	84 (3.3)	86.5 (19)	3.2	96.5 (3.8)
	465 (5,000)	5.9 (13)	57 (12.5)	17	63.5 (2.5)	68 (15)	15	76 (3)	84 (18.5)	10	94 (3.7)	100 (22)	7.5	112 (4.4)
	697 (7,500)	6.6 (14.5)	63.5 (14)	28	71 (2.8)	75 (16.5)	24	84 (3.3)	93 (20.5)	16	104 (4.1)	107 (23.5)	12	119.5 (4.7)
	929 (10,000)	7.1 (15.6)	68 (15)	39	76 (3)	82 (18)	32	91.5 (3.6)	97.5 (21.5)	22	109 (4.3)	113.5 (25)	17	127 (5.0)
Campbellton, New Brunswick	232 (2,500)	6.4 (14)	62 (13.5)	9	68.5 (2.7)	70.5 (15.5)	7	78.5 (3.1)	79.5 (17.5)	4.5	89 (3.5)	91 (20)	3.5	101.5 (4.0)
	465 (5,000)	9.0 (19.8)	86.5 (19)	22	96.5 (3.8)	91 (20)	18	101.5 (4)	102.5 (22.5)	12	115 (4.5)	113.5 (25)	9	127 (5.0)
	697 (7,500)	10.4 (22.9)	100 (22)	35	112 (4.4)	102.5 (22.5)	28	114.5 (4.5)	118 (26)	20	132 (5.2)	132 (29)	15	147.5 (5.8)
	929 (10,000)	11.3 (25)	109 (24)	47	122 (4.8)	111.5 (24.5)	40	124.5 (4.9)	127.5 (28)	29	142 (5.6)	141 (31)	22	157.5 (6.2)



Selecta-Drain Chart

LOCATION	SQUARE METRE (SQUARE FOOT)	ROOF LOAD FACTOR KGS. (LBS.)	TOTAL ROOF SLOPE											
			DEAD LEVEL			51mm (2") RISE			102mm (4") RISE			152mm (6") RISE		
			L.P.M. (G.P.M.) Discharge	Draindown Time Hrs.	mm (in.) Water Depth	L.P.M. (G.P.M.) Discharge	Draindown Time Hrs.	mm (in.) Water Depth	L.P.M. (G.P.M.) Discharge	Draindown Time Hrs.	mm (in.) Water Depth	L.P.M. (G.P.M.) Discharge	Draindown Time Hrs.	mm (in.) Water Depth
Chatham, New Brunswick	232 (2,500)	4.5 (9.9)	43 (9.5)	7	48.5 (1.9)	52.5 (11.5)	5.5	58.5 (2.3)	63.5 (14)	3.5	71 (2.8)	77.5 (17)	2.9	86.5 (3.4)
	465 (5,000)	5.7 (12.5)	54.5 (12)	16	61 (2.4)	63.5 (14)	13	71 (2.8)	77.5 (17)	9	86.5 (3.4)	91 (20)	7	101.5 (4.0)
	697 (7,500)	6.4 (14)	61.5 (13.5)	27	68.5 (2.7)	68 (15)	22	76 (3)	84 (18.5)	14	94 (3.7)	102.5 (22.5)	12	114.5 (4.5)
	929 (10,000)	6.6 (14.6)	63.5 (14)	37	71 (2.8)	75 (16.5)	30	84 (3.3)	91 (20)	20	101.5 (4.0)	107 (23.5)	16	119.5 (4.7)
Moncton, New Brunswick	232 (2,500)	4.3 (9.4)	41 (9)	7	45.5 (1.8)	54.5 (12)	6	61 (2.4)	63.5 (14)	3.5	71 (2.8)	72.5 (16)	2.7	81.5 (3.2)
	465 (5,000)	5.9 (13)	57 (12.5)	17	63.5 (2.5)	68 (15)	14	76 (3)	82 (18)	9	91.5 (3.6)	93 (20.5)	7	104 (4.1)
	697 (7,500)	6.6 (14.6)	63.5 (14)	28	71 (2.8)	79.5 (17.5)	24	89 (3.5)	93 (20.5)	16	104 (4.1)	104.5 (23)	12	117 (4.6)
	929 (10,000)	7.5 (16.6)	73.5 (16)	39	81.5 (3.2)	84 (18.5)	34	94 (3.7)	100 (22)	23	112 (4.4)	113.5 (25)	17	127 (5.0)
Saint John, New Brunswick	232 (2,500)	5.7 (12.5)	54.5 (12)	8	61 (2.4)	57 (12.5)	6	63.5 (2.5)	75 (16.5)	4	84 (3.3)	86.5 (19)	3	96.5 (3.8)
	465 (5,000)	7.5 (16.6)	72.5 (16)	20	81.5 (3.2)	79.5 (17.5)	16	89 (3.5)	95.5 (21)	11	106.5 (4.2)	104.5 (23)	8	117 (4.6)
	697 (7,500)	8.7 (19.2)	84 (18.5)	32	94 (3.7)	93 (20.5)	27	104 (4.1)	107 (23.5)	19	119.5 (4.7)	118 (26)	13.5	132 (5.2)
	929 (10,000)	9.7 (21.3)	93 (20.5)	44	104 (4.1)	104.5 (23)	38	117 (4.6)	113.5 (25)	27	127 (5.0)	127.5 (28)	20	142 (5.6)
Gander, Newfound- land	232 (2,500)	3.5 (7.8)	34 (7.5)	5.5	38 (1.5)	45.5 (10)	5	51 (2.0)	57 (12.5)	3.5	63.5 (2.5)	68 (15)	2.5	76 (3.0)
	465 (5,000)	4.7 (10.4)	45.5 (10)	15	51 (2.0)	57 (12.5)	12	63.5 (2.5)	72.5 (16)	8	81.5 (3.2)	82 (18)	6.5	91.5 (3.6)
	697 (7,500)	5.7 (12.5)	54.5 (12)	25	61 (2.4)	63.5 (14)	21	71 (2.8)	79.5 (17.5)	13.5	89 (3.5)	93 (20.5)	11	104 (4.1)
	929 (10,000)	6.1 (13.5)	59 (13)	35	66 (2.6)	70.5 (15.5)	29	78.5 (3.1)	84 (18.5)	19	94 (3.7)	100 (22)	15	112 (4.4)
St. Andrews, Newfound- land	232 (2,500)	3.5 (7.8)	34 (7.5)	5.5	38 (1.5)	45.5 (10)	5	51 (2.0)	59 (13)	3.5	66 (2.6)	63.5 (14)	2.5	71 (2.8)
	465 (5,000)	5.2 (11.4)	47.5 (10.5)	15	53.5 (2.1)	59 (13)	13	66 (2.6)	72.5 (16)	8	81.5 (3.2)	79.5 (17.5)	6	89 (3.5)
	697 (7,500)	5.9 (13)	57 (12.5)	26	63.5 (2.5)	66 (14.5)	21	73.5 (2.9)	82 (18)	14	91.5 (3.6)	88.5 (19.5)	10	99 (3.9)
	929 (10,000)	6.6 (14.6)	63.5 (14)	36	71 (2.8)	72.5 (16)	30	81.5 (3.2)	86.5 (19)	20	96.5 (3.8)	95.5 (21)	14.5	106.5 (4.2)
St. John's, Newfound- land	232 (2,500)	5.9 (13)	57 (12.5)	8	63.5 (2.6)	68 (15)	7	76 (3.0)	77.5 (17)	4.5	86.5 (3.4)	86.5 (19)	3.2	96.5 (3.8)
	465 (5,000)	8.5 (18.7)	82 (18)	21	91.5 (3.6)	91 (20)	18	101 (4.0)	100 (22)	11	112 (4.4)	113.5 (25)	9	127 (5.0)
	697 (7,500)	10.6 (23.4)	102.5 (22.5)	34	114.5 (4.5)	109 (24)	29	122 (4.8)	122.5 (27)	21	137 (5.4)	132 (29)	15	147.5 (5.8)
	929 (10,000)	11.8 (26)	113.5 (25)	48	127 (5.0)	129.5 (28.5)	43	145 (5.7)	143 (31.5)	33	160 (6.3)	150 (33)	24	167.5 (6.6)
Torbay, Newfound- land	232 (2,500)	4.9 (10.9)	47.5 (10.5)	7.5	53.5 (2.1)	61.5 (13.5)	6.5	68.5 (2.7)	75 (16.5)	4	84 (3.3)	84 (18.5)	3	94 (3.7)
	465 (5,000)	6.4 (14)	61.5 (13.5)	18	68.5 (2.7)	75 (16.5)	15.5	84 (3.3)	88.5 (19.5)	10	99 (3.9)	102.5 (22.5)	8	114.5 (4.5)
	697 (7,500)	7.3 (16.1)	70.5 (15.5)	29	78.5 (3.1)	84 (18.5)	25	94 (3.7)	100 (22)	17.5	112 (4.4)	113.5 (25)	13	127 (5)
	929 (10,000)	8.0 (17.7)	77.5 (17)	40	86.5 (3.4)	88.5 (19.5)	34	99 (3.9)	107 (23.5)	24	119.5 (4.7)	122.5 (27)	19	137 (5.4)
Halifax, Nova Scotia	232 (2,500)	5.9 (13)	57 (12.5)	8	63.5 (2.5)	68 (15)	7	76 (3.0)	77.5 (17)	4.5	86.5 (3.4)	86.5 (19)	3.2	96.5 (3.8)
	465 (5,000)	8.5 (18.7)	82 (18)	21	91.5 (3.6)	91 (20)	18	101.5 (4.0)	100 (22)	11	112 (4.4)	113.5 (25)	9	127 (5.0)
	697 (7,500)	10.6 (23.4)	102.5 (22.5)	34	114.5 (4.5)	109 (24)	29	122 (4.8)	122.5 (27)	21	137 (5.4)	132 (29)	15	147.5 (5.8)
	929 (10,000)	11.8 (26)	113.5 (25)	48	127 (5.0)	129.5 (28.5)	43	145 (5.7)	143 (31.5)	33	160 (6.3)	150 (33)	24	167.5 (6.6)



Selecta-Drain Chart

LOCATION	SQUARE METRE (SQUARE FOOT)	ROOF LOAD FACTOR KGS. (LBS.)	TOTAL ROOF SLOPE											
			DEAD LEVEL			51mm (2") RISE			102mm (4") RISE			152mm (6") RISE		
			L.P.M. (G.P.M.) Discharge	Draindown Time Hrs.	mm (in.) Water Depth	L.P.M. (G.P.M.) Discharge	Draindown Time Hrs.	mm (in.) Water Depth	L.P.M. (G.P.M.) Discharge	Draindown Time Hrs.	mm (in.) Water Depth	L.P.M. (G.P.M.) Discharge	Draindown Time Hrs.	mm (in.) Water Depth
Sydney, Nova Scotia	232 (2,500)	4.3 (9.4)	41 (9)	6.5	45.5 (1.8)	45.5 (10)	5	51 (2.0)	57 (12.5)	3.5	6.5 (2.5)	68 (15)	2.5	76 (3)
	465 (5,000)	5.7 (12.5)	54.5 (12)	16	61 (2.4)	59 (13)	13	66 (2.6)	75 (16.5)	8	84 (3.3)	84 (18.5)	6.5	94 (3.7)
	697 (7,500)	6.4 (14)	61.5 (13.5)	28	68.5 (2.7)	68 (15)	22	76 (3)	84 (18.5)	14	94 (3.7)	97.5 (21.5)	11	109 (4.3)
	929 (10,000)	7.1 (15.6)	68 (15)	38	76 (3)	75 (16.5)	30	84 (3.3)	91 (20)	20	101.5 (4)	104.5 (23)	16	117 (4.6)
Yarmouth, Nova Scotia	232 (2,500)	6.4 (14)	61.5 (13.5)	9	68.5 (2.7)	70.5 (15.5)	7.5	78.5 (3.1)	82 (18)	4.5	91.5 (3.6)	91 (20)	3.5	101.5 (4)
	465 (5,000)	8.3 (18.2)	79.5 (17.5)	21	89 (3.5)	88.5 (19.5)	18	99 (3.9)	104.5 (23)	12	117 (4.6)	116 (25.5)	9	129.5 (5.1)
	697 (7,500)	9.4 (20.8)	91 (20)	34	101.5 (4)	102.5 (22.5)	29	114.5 (4.5)	118 (26)	21	132 (5.2)	132 (29)	15	147.5 (5.8)
	929 (10,000)	10.4 (22.9)	100 (22)	45	112 (4.4)	109 (24)	41	122 (4.8)	129.5 (28.5)	29	145 (5.7)	141 (31)	22	157.5 (6.2)
Thunder Bay, Ontario	232 (2,500)	4.9 (10.9)	47.5 (10.5)	7.5	53.5 (2.1)	61.5 (13.5)	6.5	68.5 (2.7)	75 (16.5)	4	84 (3.3)	88.5 (19.5)	3.5	91.5 (3.6)
	465 (5,000)	6.1 (13.5)	59 (13)	18	66 (2.6)	72.5 (16)	15	81.5 (3.2)	86.5 (19)	9.5	96.5 (3.8)	102.5 (22.5)	7.5	114.5 (4.5)
	697 (7,500)	6.6 (14.6)	63.5 (14)	28	71 (2.8)	77.5 (17)	24	86.5 (3.4)	93 (20.5)	16	104 (4.1)	109 (24)	13	122 (4.8)
	929 (10,000)	7.1 (15.6)	68 (15)	38	76 (3)	84 (18.5)	33	94 (3.7)	97.5 (21.5)	22	109 (4.3)	116 (25.5)	18	129.5 (5.1)
Guelph, Ontario	232 (2,500)	5.7 (12.5)	54.5 (12)	8	61 (2.4)	63.5 (14)	7	71 (2.8)	86.5 (19)	5	96.5 (3.8)	100 (22)	3.7	112 (4.4)
	465 (5,000)	6.6 (14.6)	63.5 (14)	19	71 (2.8)	75 (16.5)	15.5	84 (3.3)	97.5 (21.5)	11	109 (4.3)	116 (25.5)	9	129.5 (5.1)
	697 (7,500)	7.3 (16.1)	70.5 (15.5)	29	78.5 (3.1)	82 (18)	25	91.5 (3.6)	104.5 (23)	18	117 (4.6)	125 (27.5)	14	139.5 (5.5)
	929 (10,000)	8.0 (17.7)	77.5 (17)	40	86.5 (3.4)	84 (18.5)	34	94 (3.7)	109 (24)	26	122 (4.8)	132 (29)	20	147.5 (5.8)
Hamilton, Ontario	232 (2,500)	5.9 (13)	57 (12.5)	8.5	63.5 (2.5)	72.5 (16)	7.5	81.5 (3.2)	93 (20.5)	5	104 (4.1)	109 (24)	4	122 (4.8)
	465 (5,000)	6.6 (14.6)	63.5 (14)	19	71 (2.8)	79.5 (17.5)	16	89 (3.5)	104.5 (23)	12	117 (4.6)	122.5 (27)	9	137 (5.4)
	697 (7,500)	6.8 (15.1)	66 (14.5)	28	73.5 (2.9)	84 (18.5)	26	94 (3.7)	111.5 (24.5)	20	124.5 (4.9)	127.5 (28)	15	142 (5.6)
	929 (10,000)	7.1 (15.6)	68 (15)	39	76 (3)	86.5 (19)	34	96.5 (3.8)	116 (25.5)	27	129.5 (5.1)	134 (29.5)	21	150 (5.9)
Kingston, Ontario	232 (2,500)	6.4 (14)	61.5 (13.5)	9	68.5 (2.7)	77.5 (17)	8	86.5 (3.4)	91 (20)	5	101.5 (4)	109 (24)	4	122 (4.8)
	465 (5,000)	7.5 (16.6)	72.5 (16)	20	81.5 (3.2)	86.5 (19)	18	96.5 (3.8)	104.5 (23)	12	117 (4.6)	122.5 (27)	9.5	137 (5.4)
	697 (7,500)	8.5 (18.7)	82 (18)	31	91.5 (3.6)	93 (20.5)	28	104 (4.1)	111.5 (24.5)	20	124.5 (4.9)	132 (29)	15	147.5 (5.8)
	929 (10,000)	8.7 (19.2)	86.5 (19)	42	96.5 (3.8)	97.5 (21.5)	38	109 (4.3)	116 (25.5)	27	129.5 (5.1)	68 (15)	21	152.5 (6)
London, Ontario	232 (2,500)	6.1 (13.5)	59 (13)	8.5	66 (2.6)	72.5 (16)	7.5	81.5 (3.2)	88.5 (19.5)	5	99 (3.9)	107 (23.5)	4	119.5 (4.7)
	465 (5,000)	7.1 (15.6)	68 (15)	20	76 (3)	84 (18.5)	17	94 (3.7)	102.5 (22.5)	12	114.5 (4.5)	122.5 (27)	9.5	137 (5.4)
	697 (7,500)	8.0 (17.7)	77.5 (17)	30	86.5 (3.4)	88.5 (19.5)	27	99 (3.9)	109 (24)	19	122 (4.8)	129.5 (28.5)	15	145 (5.7)
	929 (10,000)	8.5 (18.7)	82 (18)	41	91.5 (3.6)	91 (20)	36	101.5 (4)	113.5 (25)	27	127 (5)	134 (29.5)	21	150 (5.9)
North Bay, Ontario	232 (2,500)	5.7 (12.5)	54.5 (12)	8	61 (2.4)	68 (15)	7	76 (3)	86.5 (19)	5	96.5 (3.8)	100 (22)	3.8	112 (4.4)
	465 (5,000)	6.6 (14.6)	63.5 (14)	19	71 (2.8)	79.5 (17.5)	16	89 (3.5)	97.5 (21.5)	11	109 (4.3)	113.5 (25)	9	127 (5)
	697 (7,500)	7.5 (16.6)	72.5 (16)	30	81.5 (3.2)	86.5 (19)	26	96.5 (3.8)	107 (23.5)	19	119.5 (4.7)	122.5 (27)	14	137 (5.4)
	929 (10,000)	8.3 (18.2)	77.5 (17)	40	86.5 (3.4)	93 (20.5)	36	104 (4.1)	111.5 (24.5)	26	124.5 (4.9)	127.5 (28)	20	142 (5.6)

Selecta-Drain Chart



LOCATION	SQUARE METRE (SQUARE FOOT)	ROOF LOAD FACTOR KGS. (LBS.)	TOTAL ROOF SLOPE											
			DEAD LEVEL			51mm (2") RISE			102mm (4") RISE			152mm (6") RISE		
			L.P.M. (G.P.M.) Discharge	Draindown Time Hrs.	mm (in.) Water Depth	L.P.M. (G.P.M.) Discharge	Draindown Time Hrs.	mm (in.) Water Depth	L.P.M. (G.P.M.) Discharge	Draindown Time Hrs.	mm (in.) Water Depth	L.P.M. (G.P.M.) Discharge	Draindown Time Hrs.	mm (in.) Water Depth
Ottawa, Ontario	232 (2,500)	4.7 (10.4)	45.5 (10)	7	51 (2)	59 (13)	6.5	66 (2.6)	77.5 (17)	4.5	86.5 (3.4)	86.5 (19)	3.2	96.5 (3.8)
	465 (5,000)	5.9 (13)	57 (12.5)	17	63.5 (2.5)	68 (15)	14	76 (3)	86.5 (19)	10	96.5 (3.8)	100 (22)	7.5	112 (4.4)
	697 (7,500)	6.4 (14)	61.5 (13.5)	27	68.5 (2.7)	75 (16.5)	23	84 (3.3)	93 (20.5)	16	104 (4.1)	107 (23.5)	12	119.5 (4.7)
	929 (10,000)	6.6 (14.6)	63.5 (14)	36	71 (2.8)	79.5 (17.5)	32	89 (3.5)	97.5 (21.5)	22	109 (4.3)	113.5 (25)	18	127 (5)
St. Thomas, Ontario	232 (2,500)	5.7 (12.5)	54.5 (12)	8	61 (2.4)	68 (15)	7	76 (3.0)	86.5 (19)	5	96.5 (3.8)	104.5 (23)	4	117 (4.6)
	465 (5,000)	6.6 (14.6)	63.5 (14)	19	71 (2.8)	77.5 (17)	16	86.5 (3.4)	97.5 (21.5)	11	109 (4.3)	118 (26)	9	132 (5.2)
	697 (7,500)	7.1 (16.6)	68 (15)	29	76 (3.0)	82 (18)	26	91.5 (3.6)	102.5 (22.5)	18	114.5 (4.5)	125 (27.5)	15	139.5 (5.5)
	929 (10,000)	7.5 (16.6)	72.5 (16)	40	81.5 (3.2)	86.5 (19)	34	96.5 (3.8)	107 (23.5)	24	119.5 (4.7)	132 (29)	20	147.5 (5.8)
Timmins, Ontario	232 (2,500)	4.3 (9.4)	41 (9)	7	45.5 (1.8)	57 (12.5)	6	63.5 (2.5)	72.5 (16)	4	81.5 (3.2)	86.5 (19)	3.3	96.5 (3.8)
	465 (5,000)	5.7 (12.5)	54.5 (12)	16	61 (2.4)	63.5 (14)	14	71 (2.8)	82 (18)	9	91.5 (3.6)	97.5 (21.5)	7.5	109 (4.3)
	697 (7,500)	6.4 (14)	61.5 (13.5)	27	68.5 (2.7)	70.5 (15.5)	22	78.5 (3.1)	86.5 (19)	15	96.5 (3.8)	104.5 (23)	12	117 (4.6)
	929 (10,000)	6.6 (14.6)	63.5 (14)	36	71 (2.8)	72.5 (16)	30	81.5 (3.2)	91 (20)	21	101.5 (4.0)	109 (24)	17	122 (4.8)
Toronto, Ontario	232 (2,500)	5.7 (12.5)	54.5 (12)	8	61 (2.4)	66 (14.5)	7	73.5 (2.9)	82 (18)	4.5	91.5 (3.6)	97.5 (21.5)	3.5	109 (4.3)
	465 (5,000)	6.8 (15.1)	66 (14.5)	19	73.5 (2.9)	77.5 (17)	16	86.5 (3.4)	93 (20.5)	11	104 (4.1)	111.5 (24.5)	9	124.5 (4.9)
	697 (7,500)	8.0 (17.7)	77.5 (17)	30	86.5 (3.4)	84 (18.5)	26	94 (3.7)	100 (22)	18	112 (4.4)	120.5 (26.5)	14	134.5 (5.3)
	929 (10,000)	8.7 (19.2)	82 (18)	42	91.5 (3.6)	86.5 (19)	34	96.5 (3.8)	104.5 (23)	24	117 (4.6)	127.5 (28)	20	142 (5.6)
Windsor, Ontario	232 (2,500)	6.1 (13.5)	59 (13)	8.5	66 (2.6)	70.5 (15.5)	7.5	78.5 (3.1)	84 (18.5)	4.5	94 (3.7)	107 (23.5)	4	119.5 (4.7)
	465 (5,000)	7.1 (15.6)	68 (15)	20	76 (3.0)	79.5 (17.5)	16	89 (3.5)	97.5 (21.5)	11	109 (4.3)	118 (26)	9	132 (5.2)
	697 (7,500)	8.0 (17.7)	77.5 (17)	30	86.5 (3.4)	86.5 (19)	26	96.5 (3.8)	107 (23.5)	18	119.5 (4.7)	125 (27.5)	15	139.5 (5.5)
	929 (10,000)	8.7 (19.2)	82 (18)	42	91.5 (3.6)	91 (20)	36	101.5 (4.0)	113.5 (25)	26	127 (5.0)	129.5 (28.5)	20	145 (5.7)
Charlottetown, Prince Edward Island	232 (2,500)	4.9 (10.9)	47.5 (10.5)	7.5	53.5 (2.1)	57 (12.5)	6	63.5 (2.5)	68 (15)	3.8	76 (3.0)	79.5 (17.5)	3	89 (3.5)
	465 (5,000)	6.6 (14.6)	63.5 (14)	19	71 (2.8)	75 (16.5)	15.5	84 (3.3)	88.5 (19.5)	10	99 (3.9)	100 (22)	7.5	112 (4.4)
	697 (7,500)	7.8 (17.2)	75 (16.5)	31	84 (3.3)	86.5 (19)	26	96.5 (3.8)	102.5 (22.5)	18	114.5 (4.5)	113.5 (25)	13	127 (5.0)
	929 (10,000)	8.7 (19.2)	84 (18.5)	42	94 (3.7)	97.5 (21.5)	37	106.5 (4.2)	111.5 (24.5)	26	124.5 (4.9)	125 (27.5)	20	139.5 (5.5)
Montreal, Quebec	232 (2,500)	5.2 (11.4)	50 (11)	7.5	56 (2.2)	61.5 (13.5)	7	68.5 (2.7)	79.5 (17.5)	4.5	89 (3.5)	97.5 (21.5)	3.5	109 (4.36)
	465 (5,000)	5.9 (13)	57 (12.5)	17	63.5 (2.5)	70.5 (15.5)	15	78.5 (3.1)	88.5 (19.5)	10	99 (3.9)	109 (24)	8	122 (4.8)
	697 (7,500)	6.1 (13.5)	59 (13)	27	66 (2.6)	72.5 (16)	23	81.5 (3.2)	93 (20.5)	16	104 (4.1)	113.5 (25)	13	127 (5.0)
	929 (10,000)	6.4 (14)	61.5 (13.5)	36	68.5 (2.7)	77.5 (17)	31	86.5 (3.4)	95.5 (21)	22	106.5 (4.2)	120.5 (26.5)	19	134.5 (5.3)
Quebec City, Quebec	232 (2,500)	5.4 (12)	52.5 (11.5)	8	58.5 (2.3)	63.5 (14)	7	71 (2.8)	79.5 (17.5)	4.5	89 (3.5)	97.5 (21.5)	3.5	109 (4.3)
	465 (5,000)	6.4 (14)	61.5 (13.5)	18	68.5 (2.7)	70.5 (15.5)	15	78.5 (3.1)	84 (18.5)	10	94 (3.7)	104.5 (23)	8	117 (4.6)
	697 (7,500)	6.6 (14.6)	63.5 (14)	28	71 (2.8)	72.5 (16)	23	81.5 (3.2)	86.5 (19)	15	96.5 (3.8)	107 (23.5)	12	119.5 (4.7)
	929 (10,000)	7.1 (15.6)	68 (15)	37	76 (3.0)	77.5 (17)	31	86.5 (3.4)	88.5 (19.5)	20	99 (3.9)	109 (24)	17	122 (4.8)



Selecta-Drain Chart

LOCATION	SQUARE METRE (SQUARE FOOT)	ROOF LOAD FACTOR KGS. (LBS.)	TOTAL ROOF SLOPE											
			DEAD LEVEL			51mm (2") RISE			102mm (4") RISE			152mm (6") RISE		
			L.P.M. (G.P.M.) Discharge	Draindown Time Hrs.	mm (in.) Water Depth	L.P.M. (G.P.M.) Discharge	Draindown Time Hrs.	mm (in.) Water Depth	L.P.M. (G.P.M.) Discharge	Draindown Time Hrs.	mm (in.) Water Depth	L.P.M. (G.P.M.) Discharge	Draindown Time Hrs.	mm (in.) Water Depth
Regina, Saskatchewan	232 (2,500)	4.5 (9.9)	43 (9.5)	7	48.5 (1.9)	54.5 (12)	6	61 (2.4)	72.5 (16)	4	81.5 (3.2)	79.5 (17.5)	3	89 (3.5)
	465 (5,000)	6.4 (14)	61.5 (13.5)	18	68.5 (2.7)	68 (15)	14	76 (3.0)	86.5 (19)	10	96.5 (3.8)	97.5 (21.5)	7.5	109 (4.3)
	697 (7,500)	7.3 (16.1)	70.5 (15.5)	29	78.5 (3.1)	77.5 (17)	24	86.5 (3.4)	100 (22)	17	112 (4.4)	109 (24)	12	122 (4.8)
	929 (10,000)	8.3 (18.2)	79.5 (17.5)	40	89 (3.5)	82 (18)	32	91.5 (3.6)	104.5 (23)	24	117 (4.6)	118 (26)	18	132 (5.2)
Saskatoon, Saskatchewan	232 (2,500)	4.0 (8.8)	38.5 (8.5)	6	43 (1.7)	57 (12.5)	6	63.5 (2.5)	66 (14.5)	3.8	73.5 (2.9)	77.5 (17)	2.8	86.5 (3.4)
	465 (5,000)	5.7 (12.5)	54.5 (12)	16	61 (2.4)	68 (15)	14.5	76 (3.0)	82 (18)	9	91.5 (3.6)	95.5 (21)	7	106.5 (4.2)
	697 (7,500)	6.6 (14.6)	63.5 (14)	28	71 (2.8)	75 (16.5)	24	84 (3.3)	91 (20)	16	101.5 (4.0)	104.5 (23)	12	117 (4.6)
	929 (10,000)	7.1 (15.6)	68 (15)	38	76 (3.0)	82 (18)	32	91.5 (3.6)	97.5 (21.5)	22	109 (4.3)	113.5 (25)	18	127 (5.0)



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C.4 Storm Excerpts from Background Reports



Major System Segment	Flow Rate (L/s)	Flow Velocity (m/s)	Flow Depth (m)	d x v
C16	104	0.20	0.16	0.033

5.3.5.4 Development Block

Peak discharge rates and required storage volumes for development blocks within the subdivision are noted in **Table 5-12** below.

Table 5-12: Development Block Release Rates

Catchment	2-Year Storm (L/s)	100-Year Storm (L/s)	100-Year Storage Volume (m3)
L4031D (Future Mixed-Use)	1394	1394	1085
L4030B (Commercial Block)	905	905	642

5.3.5.5 Results

Table 5-13 demonstrates the modeled peak minor system and major system outflows from Trailsedge phase 4 compared to the and the major system flows in the approved Phase 2/3 model. Expects from Trailsedge Phase 1 report regarding the allowable release rates can be found in **Appendix C.5**.

Table 5-13: Storm Event Peak Discharge Rates

Storm Event	Minor System Discharge (m ³ /s)	Major System Discharge (m ³ /s)			Total Discharge (m ³ /s)	Per Approved PCSWMM Model (m ³ /s)	
		North	South	West		Minor System	Major System
2-Year, 3 Hour Chicago	3.52	0.00	0.00	0.00	3.52	5.08	0.00
5-Year, 3 Hour Chicago	4.17	0.00	0.00	0.00	4.17	7.24	0.00
100-Year, 3 Hour Chicago	4.89	0.00	0.00	0.00	4.89	8.11	0.00
100-Year, 3 Hour Chicago + 20%	5.11	0.02	0.23	0.11	5.46	8.23	1.23
100-Year, 24 Hour SCS	4.88	0.00	0.00	0.00	4.88	5.10	0.00



9.0 STORMWATER MANAGEMENT

As introduced in **Section 8.0**, stormwater modeling to support the development of Trails Edge and the Orleans Business Park has been completed by J.F. Sabourin and Associates (JFSA) to:

- Determine the adequacy of the proposed minor system to convey the 5-year and 100-year storm flows from the development to the EUC Pond 1;
- Determine the capacity of the proposed major system to safely convey the excess 100-year flows to EUC Pond 1; and
- Determine the storage required in EUC Pond 1 to meet established quality and quantity control requirements.

The updated stormwater management design is detailed in the *Preliminary Stormwater Management* document that is enclosed in **Appendix J**. The report details the following:

- Assumptions and Sources of Data Used;
- Hydraulic Simulation Results;
- Stormwater Management Facility Characteristics, including facility controls and sediment forebay calculations; and
- Major and Minor System Results (see **Section 8**).

As discussed in **Sections 1.1** and **1.2**, the existing stormwater pond - East Urban Community Pond 1 - was recommended as part of the *East Urban Community Sanitary and Storm Drainage Master Plan Update* (Gore & Storrie, 1990) and the *East Urban Community Master Drainage Plan* (Gore & Storrie, 1992). The design for the stormwater facility was later modified by the *East Urban Community Master Drainage Plan Addendum* (Stantec, 2000) and it was finally constructed as per the *East Urban Community Pond No. 1 Design Brief* (Stantec, 2008).

The existing SWM facility was designed with:

- **Permanent Pool Volume = 27,200 m³;**
- **Level of Protection = Normal, 70% long-term suspended solids removal;**
- **Drainage Area = 326 ha.; and**
- **Drainage Area Imperviousness = 57%.**

The summary of existing outlet control structures is presented in **Table 8**, as per the *EUC Pond No.1 Pagé Road Stormwater Management Facility As-Built Drawings* (Stantec, January 2012). The existing structure consists of a 600 mm reverse-slope pipe with an equivalent orifice opening of 400mm for quality control (set by adjusting the outlet sluice gate to a 270 mm invert to bottom of sluice opening position). Quantity control is provided by: two rectangular weirs on the front (east) face of the outlet structure, measuring 1.0m long and 3.0m long respectively; one 1.5m overflow weir on the south face of the outlet control structure; and one 1.5m overflow weir on the north face of the outlet control structure.

Table 8: Existing Conditions - Summary of Existing Outlet Structure Design

Quality Control 1	Quantity Control 1	Quantity Control 2	Quantity Control 3	Quantity Control 4
Vertical Orifice	Rectangular Weir	Rectangular Weir	Rectangular Weir	Rectangular Weir
400mm dia	1.00m wide	3.00m long	1.5m long	1.5m long
INV = 80.10	INV = 81.65	INV = 82.35	INV = 82.35	INV = 82.35

A schematic of existing EUC Pond 1 is depicted on **Figure 17**, referencing as-built information.

9.1 Quantity Control Objectives

The design of the existing EUC Pond 1 is contained in *East Urban Community Pond 1 Design Brief* (Stantec, April 2008). Quantity control objectives were based on the Master Drainage Plan (MDP) to control the post-development flows for storms greater than the 5- year return period and up to the 100-year return period to pre-development levels. The recommended maximum water level in the facility during the 100-year storm is 83.0 m.

9.2 Quality Control Objectives

The design of the existing EUC Pond 1 was completed in the *East Urban Community Pond 1 Design Brief* (Stantec, April 2008). The recommended quality control objective from the RVCA was a Normal Level of fish habitat protection, corresponding to 70% average long term removal of total suspended solids.

For EUC Pond 1, erosion control requirements were based on both Master Drainage Plan (MDP) and SWMP Design Manual guidelines. This was satisfied by detention of the 25mm storm for 24 to 48 hours and over control of the 2- and 5-year storms for the existing design storms.

9.3 Stormwater Modeling

For the purpose of this 2014 Servicing Report, the existing EUC Pond 1 and previous development plan (Stantec, 2005) were modeled according to current standards.

The stormwater management models discussed in **Appendix J** (JFSA, February 2013) indicate that the anticipated storage requirement in EUC Pond 1 is higher than the 100-year volume estimated in the previous design brief (Stantec, 2008) as a result of the following changes to updated standards:

1. Revised modeling of the pond storage-discharge relationship;
2. Revised drainage areas and percent imperviousness;
3. Minor system capture of the 5-year flow plus 12%;
4. Use current City of Ottawa IDF curves for the 100-year Chicago storm;
5. Employing five minute time steps in routine routing;
6. Modeling parameters selected according to current City of Ottawa standards; and
7. Use Horton's equation in modeling infiltration.

DDSWMM and SWMHYMO were used to simulate hydrology of the major system flows and minor system inflows for the drainage area to EUC Pond 1, and to estimate the pond storage required to meet quality and quantity control targets. XPSWMM was used to model the minor system hydraulics (as described in **Section 8.0**). The schematic of the XPSWMM model as well as the digital DDSWMM, SWMHYMO and XPSWMM models are enclosed in **Appendix J**.

Stormwater modeling completed as part of this Servicing Report (details described herein) has demonstrated that the existing EUC Pond 1 does not have the capacity to support the proposed development of Trails Edge, the Orleans Business Park and the Transitway/Brian Coburn Boulevard. As such, detailed modeling has been undertaken to identify the required additional storage requirements for various scenarios.

- **Interim Condition A:** Pond with upgrades to south cell and south forebay (as required), supporting build-out of the Trails Edge development.
- **Interim Condition B:** Pond with upgrades to south cell and south forebay (as required), supporting build-out of the Trails Edge development south of the Hydro Easement and the development of the Transitway and Brian Coburn Boulevard.
- **Ultimate Conditions:** Pond with potential upgrades to north cell (as required), supporting build-out of the Trails Edge development, the development of the Transitway and Brian Coburn Boulevard, and the development of the Orleans Business Park to the north of the Hydro Easement.

As detailed in the following sections, **it is recommended that the South Cell and South Forebay be expanded in the short term, to provide the required storage for the Interim B Condition.** A separate detailed design exercise is currently underway for these modifications. This allows for the development of Trails Edge, the Transitway, and

Brian Coburn to proceed in the short term (subject to required approvals), without downstream stormwater management capacity constraints. Note that there are no proposed changes to the outlet structure as compared to the existing pond, in order to avoid any impacts on erosion or stability in the downstream Mud Creek.

9.4 Stormwater Management Facility

The detailed analysis of each scenario is included in **Appendix J**. As detailed in the JFSA analysis, the drainage area to be serviced by EUC Pond 1 under ultimate conditions is 371 ha at 74% imperviousness. Preliminary storage volumes have been based on the 24-hour SCS Type II design storm, which was determined to be the critical storm.

The summary of parameters for each scenario and the minimum required storage volumes (per the MOE SWMP Design Manual) is presented in **Table 9**.

Table 9: Summary of Minimum Storage Requirements for Drainage Areas to SWM Pond

Scenario	Area (ha)	Imperviousness (%)	Storage Volume for Impervious Level ¹ (m ³ /ha)	Permanent Pool Required Volume (m ³)	Quality Control Required Volume ² (m ³)
Existing Conditions (Pre-development)	367	18	73	12,111	14,680
Interim A Conditions	370	33	88	17,760	14,800
Interim B Conditions	370	36	91	18,870	14,800
Ultimate Conditions	371	74	135	35,245	14,840

Notes:

1. Protection Level for Wet Pond: Normal 70% long term suspended solids removal. SWM Planning & Design Manual, Table 3.2, p. 3-10 (March 2003)
2. Required quality control volume based on Storage Volume for Impervious Level less 40m³/ha required permanent pool volume.

9.4.1 Existing Conditions

Under existing conditions, drainage boundaries generally match the ultimate conditions, with about 367 hectares draining to EUC Pond 1. The existing storm condition is depicted in **Figures 12A** and **12B**.

Under existing conditions, approximately 276 ha of the 367 ha drainage area to SWM Facility 1 is undeveloped, resulting in an existing imperviousness of approximately 18%. Undeveloped areas were modeled with a SCS Curve Number of 78 (CN* = 70) based on hydrologic soil group D and a combination of woods, meadow and brush in fair condition. Time to peak values for the undeveloped areas were calculated using existing contour data and the Bransby Williams equation. The outlets were modeled as per existing conditions outlined in **Table 8**. The summary of pond operational characteristics for the existing pond – as built per the *East Urban Community Pond 1 Design Brief* (Stantec, April 2008) - under existing pre-development conditions is presented in **Table 10** and **Appendix O**.

Table 10: Existing Conditions - Summary of SWM Pond Operating Characteristics

Pond Component	Allowable Outflow (m ³ /s)	Pond Level – Main Cell (m)	Pond Outflow (m ³ /s)	Volume Used (m ³)
Permanent Pool	N/A	80.10	N/A	30,710
Quality Control	N/A	80.711	0.221	14,698
Extended Detention	N/A	81.650	0.401	40,838
2yr/24hr SCS	1.000	81.60	0.392	38,907
5yr/24hr SCS	2.300	82.10	0.955	58,020
10yr/24hr SCS	3.800	82.37	1.445	70,417
25hr/24hr SCS	5.600	82.66	2.688	83,856
50yr/24hr SCS	6.700	82.84	4.107	92,812
100yr/24hr SCS	8.000	83.01	5.652	101,206
July 1 st , 1979 Event	N/A	83.02	5.688	101,829

Note that the permanent pool volume used exceeds the minimum permanent pool volume required for treatment of existing pre-development runoff shown in **Table 8**. This is because the existing EUC Pond 1 has been built to support full-buildout of 326 hectares of development, so the permanent pool is over-sized for the requirements of pre-development conditions.

9.4.2 Interim A Conditions

Under interim A conditions, the modeling considers build-out of the Trails Edge development. The following modifications to EUC Pond 1 would be required:

- Mud Creek would need to be reconstructed as a stormwater conveyance channel from Belcourt Extension to the South Forebay; and
- South forebay and south cell would need to be reconstructed within the existing City of Ottawa pond block, to provide the required storage volumes.

The summary of outlet control structures under interim conditions is presented in **Table 11**. Note that there are no proposed changes to the outlet structure as compared to the existing pond, in order to avoid any impacts on erosion or stability in the downstream Mud Creek.

Table 11: Interim Conditions - Summary of Existing Outlet Structure Design

Quality Control 1	Quantity Control 1	Quantity Control 2	Quantity Control 3	Quantity Control 4
Vertical Orifice	Rectangular Weir	Rectangular Weir	Rectangular Weir	Rectangular Weir
400mm dia	1.00m wide	3.00m long	1.5m long	1.5m long
INV = 80.10	INV = 81.65	INV = 82.35	INV = 82.35	INV = 82.35

The summary of pond operational characteristics under interim A conditions is presented in **Table 12**. These characteristics are based on the tributary storm drainage areas and runoff coefficients under interim pond conditions for Trails Edge Community, as shown in **Figure 13A**. Refer to **Figure 13B** for the storm drainage areas for the undeveloped Orleans Business Park under interim conditions. The allowable outflow rates are based on the MOE Certificate of Approval provided in **Appendix E**, which are consistent with the design values provided in the *East Urban Community Pond No. 1 Design Brief* (Stantec, 2008). The stage-storage relationship modeled is based on preliminary proposed modifications to the existing EUC Pond 1 design, assuming the south cell and south forebay are widened to provide the required additional permanent pool volume.

Table 12: Interim A Conditions - Summary of SWM Pond Operating Characteristics

Pond Component	Allowable Outflow (m ³ /s)	Pond Level – North Cell (m)	Pond Level – South Cell (m)	Pond Outflow ¹ (m ³ /s)	Volume Used (m ³)
Permanent Pool	N/A	80.10	80.10	N/A	35,639
Quality Control	N/A	80.68	80.68	0.203	14,802
Extended Detention	N/A	81.65	81.65	0.383	44,047
2yr/24hr SCS	1.000	81.38	81.38	0.343	38,660
5yr/24hr SCS	2.300	81.87	81.87	0.595	55,559
10yr/24hr SCS	3.800	82.09	82.09	0.916	64,376
25hr/24hr SCS	5.600	82.38	82.38	1.447	77,305
50yr/24hr SCS	6.700	82.58	82.58	2.586	87,242
100yr/24hr SCS	8.000	82.76	82.76	4.149	96,212
July 1 st , 1979 Event	N/A	82.89	82.89	5.446	102,680

As shown in **Table 12**, to achieve the same outflows as existing conditions, a permanent pool of 35,639 m³ is required; therefore, 8,429 m³ of additional storage is required beyond the existing permanent pool volume of 27,200 m³.

To minimize throw-away construction and provide for construction efficiencies, it is recommended that EUC Pond 1 be modified according to the Interim B Condition (**Section 9.4.3**), to provide for the future development of the Transitway and Brian Coburn Boulevard in addition to the development of the Trails Edge community.

9.4.3 Interim B Conditions

Under interim B conditions, the modeling considers build-out of development south of the Hydro easement, including build-out of the Transitway and Brian Coburn Boulevard. The following modifications to EUC Pond 1 would be required:

- Mud Creek would need to be reconstructed as a stormwater conveyance channel from Belcourt Extension to the South Forebay; and
- South forebay and south cell would need to be reconstructed within the existing City of Ottawa pond block, to provide the required storage volumes.

The summary of pond operational characteristics under interim B conditions is presented in **Table 13**. These characteristics are based on the tributary storm drainage areas and runoff coefficients under interim pond conditions for Trails Edge Community, combined with the drainage from the Transitway and Brian Coburn Boulevard (shown in **Figure 14A/ 14B**). Note that the Orleans Business Park remains undeveloped in this scenario. The drainage area characteristics are further described in **Appendix J**, specifically in **Appendix J – Figure 3**. The same preliminary stage-storage relationship described in **Section 9.4.2** has been applied.

The proposed alignment of the Transitway and Brian Coburn Boulevard will cross EUC Pond 1. In the absence of preliminary design information, a culvert of 3000mm diameter has been sized to connect the south main cell to the north main cell, to achieve the water levels described in **Table 13** below. See **Appendix J** for additional information.

The allowable outflow rates are based on the MOE Certificate of Approval provided in **Appendix E**, which are consistent with the design values provided in the *East Urban Community Pond No. 1 Design Brief* (Stantec, 2008).

In the detailed design phase, the design of the stormwater conveyance channel will be completed by a geomorphologist. Based on a preliminary assessment by Coldwater Consulting (**Appendix M**), the proposed channel width will be adequate for flexibility in future detailed design exercises.

The future modifications to the South Forebay must ensure that a length to width ratio of between 3:1 to 4:1 is achieved and that the forebay size does not exceed one third of the permanent pool volume, according to Ministry of the Environment standards. Assuming the South Forebay is modified to achieve the required storage through widening, it is expected to have an average velocity of 0.21 m/s, which is below the velocity of 0.40 m/s where erosion is expected to occur for the local soils. Refer to **Appendix J**.

The detailed design and cost estimate for the required pond modifications will be subject to a separate study.

Table 13: Interim B Conditions - Summary of SWM Pond Operating Characteristics

Pond Component	Allowable Outflow (m ³ /s)	Pond Level – North Cell (m)	Pond Level – South Cell (m)	Pond Outflow ¹ (m ³ /s)	Volume Used (m ³)
Permanent Pool	N/A	80.10	80.10	N/A	35,639
Quality Control	N/A	80.68	80.68	0.203	14,802
Extended Detention	N/A	81.65	81.65	0.383	44,047
2yr/24hr SCS	1.000	81.84	81.84	0.550	53,361
5yr/24hr SCS	2.300	82.24	82.24	1.180	70,810
10yr/24hr SCS	3.800	82.47	82.47	1.840	81,738
25hr/24hr SCS	5.600	82.70	82.70	3.585	93,231
50yr/24hr SCS	6.700	82.86	82.86	5.131	101,185
100yr/24hr SCS	8.000	83.03	83.03	7.057	110,069
July 1 st , 1979 Event	N/A	83.13	83.14	8.367	114,777

The Transitway and Brian Coburn Boulevard will outlet directly to the main cell of EUC Pond 1, and will not receive the sediment removal benefits of the designed forebays. As such, an oil and grit separator unit should be implemented before stormwater is discharged to the main cell of EUC Pond 1 to ensure the required 70% average TSS removal is achieved. As an example, a CDS PSWC5678-10 model with a treatment flowrate of 708 L/s could be installed (or equivalent) to achieve the required TSS removal. TSS removal calculations, pricing information, and a sample cut sheet drawing of the sample unit are provided in **Appendix N**, for information. The ultimate dimensions and unit specifications would be determined at the detailed design stage.

Note that the permanent pool and quality control volumes in **Table 13** exceed the minimum volumes required for treatment of interim condition B runoff shown in **Table 9**, therefore the modified EUC Pond 1 design will meet Ministry of the Environment requirements for a normal protection level (70% long-term suspended solids removal).

The future modifications to the South Cell and South Forebay (being completed as a separate design exercise) must be designed to provide the storage requirements for the Interim B Condition (e.g. an additional permanent pool volume of 8,439 m³ beyond the existing 27,200m³ permanent pool), so that the development of Brian Coburn and the Transitway can occur without additional modifications to the South Cell and South Forebay

9.4.4 Ultimate Conditions

Under ultimate conditions, the modeling has been updated from interim conditions to consider build-out of development north of the Hydro easement, including the full development of the Orleans Business Park.

Approximately 57,157 m³ of permanent pool volume is required in EUC Pond 1 under ultimate conditions – meaning that **21,528m³ of additional storage** is required beyond the existing permanent pool volume of 27,200 m³ and the proposed interim expansion of 8,439 m³. The required active storage volumes, simulated in SWMHYMO based on the sum of major and minor system flows to the pond, are contained in the JFSA memo enclosed in **Appendix J**. The required extended detention storage volume is approximately 76,695 m³ in EUC Pond 1 under ultimate conditions.

Design alternatives to meet these additional storage requirements are to be evaluated as part of a future study, in conjunction with the future East Urban Community Mixed-Use Centre Community Design Plan that will refine the proposed development plan north of the Hydro Corridor. Design options for consideration include:

- Doing nothing;
- Limiting runoff from development and maintaining the existing pond;
- Expanding pond and forebay (North of the Hydro Corridor); and,
- Building new pond.

In order to estimate the significance of the required upgrades, a model for the potential ultimate EUC Pond 1 was prepared, on the basis of potential future expansion of the EUC Pond 1 North Cell and North Forebay. The summary of potential outlet control structures under ultimate conditions is presented in **Table 14**. As in interim conditions, the outlets have not been modified from existing conditions, in order to avoid any impacts on erosion or stability in the downstream Mud Creek. This results in extended detention drawdown times that are greater than the 24 to 48 hours specified in the original *East Urban Community Pond 1 Design Brief* (Stantec, April 2008).

Table 14: Ultimate Conditions - Summary of Existing Outlet Structure Design

Quality Control 1	Quantity Control 1	Quantity Control 2	Quantity Control 3	Quantity Control 4
Vertical Orifice	Rectangular Weir	Rectangular Weir	Rectangular Weir	Rectangular Weir
400mm dia	1.00m wide	3.00m long	1.5m long	1.5m long
INV = 80.10	INV = 81.65	INV = 82.35	INV = 82.35	INV = 82.35

The summary of pond operational characteristics for this potential ultimate expansion condition is presented in **Table 15**. The characteristics described below are based on the ultimate storm conditions shown for the Trails Edge Development and Orleans Business Park in **Figure 15A** and **Figure 15B**, respectively. As indicated in **Section 9.4.1**, the allowable outflow rates are based on the MOE Certificate of Approval provided in **Appendix E**, which are consistent with the design values provided in the *East Urban Community Pond No. 1 Design Brief* (Stantec, 2008).

As per **Section 9.4.3**, a culvert of 3000mm diameter has been sized to connect the south main cell to the north main cell (under the Transitway and Brian Coburn Boulevard), to achieve the water levels described in **Table 15** below.

**Table 15: Ultimate Conditions –
Summary of SWM Pond Operating Characteristics**

Pond Component	Allowable Outflow (m ³ /s)	Pond Level – North Cell (m)	Pond Level – South Cell (m)	Pond Outflow (m ³ /s)	Volume Used (m ³)
Permanent Pool	N/A	80.10	80.10	N/A	57,157
Quality Control	N/A	80.43	80.43	0.123	14,828
Extended Detention	N/A	81.65	81.65	0.383	76,695
2yr/24hr SCS	1.000	81.85	81.85	0.571	92,736
5yr/24hr SCS	2.300	82.22	82.22	1.147	115,566
10yr/24hr SCS	3.800	82.44	82.44	1.682	130,004
25hr/24hr SCS	5.600	82.66	82.66	3.201	145,733
50yr/24hr SCS	6.700	82.81	82.81	4.610	156,784
100yr/24hr SCS	8.000	82.97	82.97	6.343	168,722
July 1 st , 1979 Event	N/A	83.06	83.07	7.494	176,215

As shown in **Table 15**, to achieve the same outflows as existing conditions 57,157 m³ of permanent pool volume is required; therefore, 21,528m³ of additional storage would be required beyond the existing permanent pool volume of 27,200 m³ and the proposed interim expansion of 8,429 m³. The design alternatives to accommodate the required additional storage will be addressed in a future servicing report associated with the future East Urban Community Mixed-Use Centre Community Design Plan.

Note that the permanent pool and quality control volumes in **Table 15** exceed the minimum volumes required for treatment of ultimate runoff shown in **Table 9**, ensuring the future EUC Pond 1 design will meet Ministry of the Environment requirements for a normal protection level (70% long-term suspended solids removal).

9.5 Stormwater Conclusions and Recommendations

Stormwater management measures have been designed to current City of Ottawa and MOE design guidelines and have been updated based on revised drainage areas, land uses and corresponding percent imperviousness. The analysis has also been updated to incorporate a storm conveyance channel following the alignment of the existing Mud Creek west of Belcourt Extension.

Under **interim conditions**, it is recommended that the Mud Creek channel, south forebay and south main cell of EUC Pond 1 be reconstructed to meet current SWM facility requirements for the Interim B condition (e.g. development of Trails Edge, the Transitway, and Brian Coburn Boulevard). The modeling results described in **Appendix J** indicate that sufficient permanent pool and quality control volumes can be provided according to MOE standards for a normal protection level (70% long term suspended solids removal). Target release rates for the 2-year to 100-year storms can be met via modifications to the south forebay and south main cell of EUC Pond 1, and the 100-year water level in the pond can be maintained from the original design. The proposed modifications to the south pond cell will involve minor reconstruction within the existing municipal property boundaries and would be pre-approved under Schedule A of the MCEA. Detailed designs and costing for the modifications to EUC Pond 1 are being completed as a separate study, but must provide the required storage without exceeding the target outflows or water levels. The proposed interim modifications can accommodate the development of the Transitway and Brian Coburn Boulevard.

The proposed modifications to the stormwater management facility do not change the quality or quantity targets approved as part of the construction of the existing facility (Stantec, 2008). As such, this 2014 Servicing Report does not include in its scope the further analysis of the downstream Mud Creek system, since it was addressed in the approved studies leading up to the approved *Gloucester and Cumberland EUC Expansion Area and Bilberry Creek Industrial Park Master Servicing Update* (Stantec, 2006).

However, during the course of this study, the City and RVCA have indicated larger downstream erosion control issues are being studied by the City and NCC in the ongoing *Mud Creek (Greens Creek) Fluvial Geomorphology and Risk Assessment Study*. Furthermore, an ongoing study of Mud Creek is being completed by Queen's University. Subject to the forthcoming conclusions of these on-going studies by others, the City, RVCA, and NCC are expected to revise targets for the receiving waterbody - and thus potential future revisions to the design of EUC Pond 1 are anticipated. There is also a potential requirement to implement stream rehabilitation/erosion protection works downstream of EUC Pond 1, should revisions to the pond design not be sufficient to mitigate the impacts of the development. These potential requirements will be pursued through the draft approval process for the residential lands, and future planning studies anticipated for the commercial lands (in conjunction with the future East Urban Community Mixed-Use Centre Community Design Plan).

For **ultimate conditions**, further study will be needed in conjunction with the future East Urban Community Mixed-Use Centre Community Design Plan, in order to provide additional storage volume to support the development of the lands north of the Hydro Easement to current stormwater management standards.

Existing developments to the southwest of the quadrant have brought a storm sewer network to the southern boundary of the South West quadrant. A 1050 mm diameter storm sewer stub borders the portion of the South West quadrant west of Fern Casey Street, and a 2700 mm diameter stub borders the portion of the quadrant east of Fern Casey Street. A 900 mm-1050 mm diameter storm sewer within Brian Coburn Boulevard is located within the South West quadrant. To the east, there is a 525 mm-900 mm diameter storm sewer on Mer Bleue Road. The existing storm sewer network is shown in **Drawing 7**.

11.4.2 South West Minor System Design

The MSU (Stantec, July 2006) reviewed the required infrastructure to service the South West quadrant of the EUC Phase 3 Area. The intended storm outlet for the quadrant is the existing South Forebay of EUC Pond 1.

The MSU (Stantec, July 2006) considered the South West quadrant to be serviced by trunk storm sewers ranging in diameter from 1250 mm to 2400 mm. The minor system generally drains east to west, through adjacent lands towards the EUC Pond 1 South Forebay. Five-year capture was used for all development lands and local roads, while 10-year capture was applied for arterial roads. The minor system did not consider flows from Brian Coburn Boulevard (then known as the Blackburn Hamlet By-Pass) as it was assumed that the drainage would be addressed through a separate sewer. The portion of the South West quadrant north of Brian Coburn was also assumed to drain separately from the rest of the quadrant. Details of the MSU (Stantec, July 2006) minor system can be found in **Appendix E**.

More recently, the South West quadrant was considered as part of the *Servicing Report for Trails Edge and Orléans Business Park* (DSEL, July 2017). Consistent with the MSU (Stantec, July 2006), the quadrant was to be serviced by a trunk sewer network draining towards the EUC Pond 1 South Forebay. A stormwater conveyance channel was detailed to direct outflows from the trunk storm sewers to the EUC Pond 1 South Forebay. The storm drainage plan can be seen in **Drawing 4**. Five-year capture was used for all development lands and local roads, while 10-year capture was applied for arterial roads. Deviating from the MSU (Stantec, July 2006), the portion of the South West quadrant north of Brian Coburn Boulevard and the segment of Brian Coburn Boulevard within the quadrant were assumed to drain through the quadrant's storm sewer network. The majority of the South West quadrant was considered as Mixed-Use and was assigned a runoff coefficient of 0.8. There were also Medium Density and Commercial land uses which were assigned runoff coefficients of 0.7 and 0.65, respectively.

Since the completion of the MSU (Stantec, July 2006), the stormwater management of the South West quadrant has been considered during the construction of downstream infrastructure. The *Design Brief – Minto Trailsedge Phase II* (IBI Group, May 2015) considers the stormwater drainage for the portion of the South West quadrant that is east of Fern Casey Street. Per the storm design information included in **Appendix E**, a 2700 mm diameter storm sewer stub exists south of the quadrant. Per the *Design Brief – Minto Trailsedge Phase II* (IBI Group, May 2015), this stretch of sewer anticipated a Rational Method peak flow of 5,424.84 L/s from undeveloped land to the east, which includes the portion of the South West quadrant that is east of Fern Casey Street. Five-year capture was assumed and an available Rational Method additional capacity of 5,040.86 L/s was reported available downstream in the constraining segment (MH55 - MH55B).

The *Design Brief – Minto Trailsedge Phase II* (IBI Group, May 2015) also considered the stormwater drainage for the portion of Fern Casey Street (formerly Belcourt Boulevard) within the South West quadrant. A portion of the street is to drain to Brian Coburn while a portion is being serviced by the storm sewers constructed as part of Trailsedge Phase II.

The stormwater servicing of the South West quadrant has also been considered as part of the approved *Trails Edge East – Functional Servicing Report* (Stantec, August 11, 2017). As shown in **Appendix E**, an additional 3.18 ha residential area drains to the existing 2700 mm diameter stub along with the stormwater flows from the portion of the South West quadrant east of Fern Casey Street. Per the *Trails Edge East – Functional Servicing Report* (Stantec, August 11, 2017), a rational method peak flow of 4,824.3 L/s was anticipated from the portion of the South West quadrant that is east of Fern Casey Street. 2-year capture was assumed for the portion of the South West quadrant that is east of Fern Casey Street, deviating from past studies, but consistent with current City of Ottawa and MECP standards. Brian Coburn Boulevard was expected to drain separately consistent with the MSU (Stantec, July 2006) and design information for Brian Coburn Boulevard.

The Trailsedge East lands are currently undergoing detailed design and construction. Per the *Trails Edge East Phase 1 Servicing and Stormwater Management Report* (Stantec, August, 2018), 5-year capture was assumed for the portion of the South West quadrant east of Fern Casey Street. Note that the *Trails Edge East Phase 1 Servicing and Stormwater Management Report* (Stantec, August, 2018) proposes to accommodate the stormwater flows from the portion of the South West quadrant east of Fern Casey Street through 2 inlet locations (MH1004 & MH1002). The anticipated Rational Method peak flow attributed to the portion of the South West quadrant east of Fern Casey Street to MH1004 and MH1002 are 4686.2 L/s and 741.52 L/s respectively. All stormwater flows from the South West quadrant east of Fern Casey Street are still proposed to ultimately be directed towards the existing 2700mm diameter stub. Brian Coburn Boulevard is reported to drain separately, which is consistent with the MSU (Stantec, July 2006) and with as-built information for Brian Coburn Boulevard. Details are provided in **Appendix E**. The total Rational Method peak flow anticipated to the existing 2700 mm diameter storm sewer stub is reported to be 6,154.9 L/s, which is above the 5,424.84 L/s allowance detailed in the *Design Brief – Minto Trailsedge Phase II* (IBI Group, May 2015). The anticipated increase in flows of 730.06 L/s is below the 5,040.86 L/s available Rational Method capacity downstream in the constraining segment (MH55 – MH55B).

The *Design Brief for the Trails Edge West Richcraft Group of Companies* (DSEL, January 26, 2015) includes the stormwater drainage for the portion of the South West quadrant west of Fern Casey Street. 5-year capture was assumed and a rational method peak flow of 845.8 L/s was determined to drain into existing Control MH2. See **Appendix E** for details.

Respecting the MSU (Stantec, July 2006), existing stormwater infrastructure, and ongoing development applications within the South West quadrant, the proposed stormwater servicing strategy is to have all flows from the quadrant drain to the EUC Pond 1 South Forebay via the existing stormwater conveyance channel and an extension of the existing trunk storm sewer network. The proposed storm sewer network is shown in **Drawing 4**. A trunk 2100 mm trunk storm sewer is shown servicing the majority of the quadrant by routing minor system stormwater flows to the proposed Trailsedge East storm sewer south of the quadrant. The remaining portions of the South West quadrant drain directly into existing infrastructure. A storm sewer

crosses Brian Coburn Boulevard at an existing gap in underground infrastructure – coordination will be required for work in the ROW, including utility coordination.

Note that at this stage of analysis, only the trunk storm sewer within the quadrant is shown. To demonstrate serviceability, the trunk sewer is carried back at minimum possible slopes accounting for drops at manholes, existing infrastructure sizing and possible conflicts with the sanitary sewer. As design of the South West quadrant advances, the storm sewer network details are subject to change for construction efficiencies, etc., provided that the conditions in Section 14 related to minor changes are met. Additional springline connections and/or reduced drops across maintenance holes may be proposed as part of detailed design, to assist in minimizing grade raise requirements, provided that the conditions in **Section 14** related to minor changes are met. These are currently considered deviations from City Standards, and will require review on a case-by-case basis.

2-year capture was assumed for all local roads and development lands in the South West quadrant. 5-year capture was used for collector roads in order to adhere to City Standards regarding allowable depth of flow on streets.

The rational method, with design criteria described in **Section 11.1**, was employed to size the storm sewer to accommodate all minor flows. Per the design sheet included in **Appendix E**, the anticipated rational method peak flow from the portion of the South West quadrant east of Fern Casey Street is 4,964 L/s. Please note that consistent with the *Trails Edge East – Functional Servicing Report* (Stantec, August 11, 2017), the stormwater flows from the North West quadrant east of Fern Casey Street are directed towards one outlet in this MSS for the purpose of high-level servicing review. Outlets and routing alternatives may be refined as the design and construction processes advance, based on constructed downstream infrastructure (e.g. direct flows to multiple constructed outlets, rather than a single outlet).

The proposed 4,964 L/s rational method peak flow to MH 302 is roughly 106% of the 4,686.2 L/s flow anticipated at this location within the *Trails Edge East Phase 1 Servicing and Stormwater Management Report* (Stantec, August, 2018). A flow of 741.5 L/s was also anticipated from the portion of the South West quadrant east of Fern Casey Street, to be collected further downstream per the *Trails Edge East Phase 1 Servicing and Stormwater Management Report* (Stantec, August, 2018). Per the *East Urban Community / Preliminary Hydraulic Gradeline Analysis and Pond Design* (JFSA, June 2019)(see **Appendix E**), the peak modelled flow to the existing stub is 5,800 L/s under the 100-year plus 20% storm event. The proposed design of the South West quadrant east of Fern Casey does not have a negative impact of the downstream stormwater management infrastructure. Therefore, it is concluded that the downstream infrastructure has adequate capacity to service the minor system stormwater flows from the portion of the South West quadrant, east of Fern Casey Street.

The portions of Brian Coburn Drive and Fern Casey Street within the South West quadrant are proposed to continue draining as they are at the time of this MSS, per asbuilt conditions.

The anticipated rational method peak flow from the portion of the South West quadrant west of Fern Casey Street is 853 L/s, roughly 101% of the 845.8 L/s flow anticipated in the in the *Design Brief for the Trails Edge West Richcraft Group of Companies* (DSEL, January 26, 2015). Seeing as the land is to provide 5-year capture consistent with the *Design Brief for the Trails Edge West Richcraft Group of Companies*

(DSEL, January 26, 2015), any discrepancy in anticipated rational method peak flow can be attributed to rounding. Per the *East Urban Community / Preliminary Hydraulic Gradeline Analysis and Pond Design* (JFSA, June 2019), see **Appendix E**, the proposed design of the South West quadrant west of Fern Casey is consistent with the design of the downstream stormwater management infrastructure. Therefore, it is concluded that the downstream infrastructure has adequate capacity to service the minor system stormwater flows from the portion of the South West quadrant, west of Fern Casey Street.

11.4.3 South West Hydraulic Grade Line Analysis

Given the preferred minor system design and land uses differ from background studies, a Hydraulic Grade Line analysis was completed for the South West quadrant, using downstream modelling information provided by Stantec from the *Trails Edge East Phase 1 Servicing and Stormwater Management Report* (Stantec, August, 2018). The suitability of the proposed trunk sewer network was analyzed in the *East Urban Community / Preliminary Hydraulic Gradeline Analysis and Pond Design* (JFSA, June 2019), provided in **Appendix E**. The simulated 100-year HGL results through the proposed trunk storm sewer network have been analyzed for suitability with the proposed road grades and anticipated underside of footing elevations, estimated to be 2.1 m below ground level.

The analysis was simulated with the 100-year 3-hour Chicago storm, 100-year 24-hour SCS Type II storm and July 1979, August 1988 and August 1996 historical events. The results indicated that the quadrant is serviceable per the preferred design as a freeboard of 0.3 m between the hydraulic gradeline and the estimated underside of footing is provided for the 100-year storms and a freeboard of 0 m has been provided for the historical events and the 100-year storm + 20% climate change stress test. As noted earlier, the storm sewer design is expected to change at detailed design (e.g. to include local sewers, to minimize earthworks costs, etc.), and an updated analysis will be required in conformance with all City of Ottawa and MECP guidelines.

11.4.4 South West Major System & Grading Design

The MSU (Stantec, July 2006) did not seem to include a detailed design of the major system for the South West quadrant. Based on the MSU (Stantec, July 2006) Macro Grading Plan, included in **Appendix E**, the major system flow is generally directed towards the EUC Pond 1 South Forebay. Grade raise restrictions for the South West quadrant are reported as 0-0.6 m and are respected in the grading plan.

Per the *Servicing Report for Trails Edge and Orléans Business Park* (DSEL, July 2017), the major flow servicing strategy was updated based on the proposed road network. Consistent with the MSU (Stantec, July 2006), the major flow was to be routed westward towards the EUC Pond 1 South Forebay. See **Appendix E** for details. The majority of the South West quadrant was reported to provide onsite storage up to the 100-year storm event as they were contemplated for Mixed-Use, High Density Residential and Commercial land uses. The *Servicing Report for Trails Edge and Orléans Business Park* (DSEL, July 2017) grading plan reports grade raises up to 3 m within the South West quadrant. This exceeds the grade raise restriction for the quadrant of 0.5 m to 1.5 m per the *Geotechnical – Existing Conditions Report East Urban Community Mixed Use CDP* (Paterson Group, June 28, 2018). As such, a surcharge program or lightweight fill was recommended to be considered as the design process continued.

Per the approved *Trails Edge East – Functional Servicing Report* (Stantec, August 11, 2017) and the *Trails Edge East Phase 1 Servicing Design Brief* (Stantec, August 2018), onsite storage up to the 100-year storm event was to be provided for the portion of the South West quadrant, east of Fern Casey Street. The *Design Brief for the Trails Edge West Richcraft Group of Companies* (DSEL, January 26, 2015) similarly states that onsite storage up to the 100-year storm event was to be provided for the portion of the South West quadrant, west of Fern Casey Street.

The proposed major system design is for commercial and mixed-use blocks within the South West quadrant to provide onsite storage up to the 100-year storm event, which is consistent with background servicing reports for the quadrant. The remaining areas are to have overland flow stored within the road network, then directed towards the EUC Pond 1 via the neighbouring lands to the south. The routing is to follow the proposed road network, as shown in **Drawing 2**. Note that the medium-high density block west of Fern Casey Street is to have major system flows directed towards Axis Way, while the other medium-high density block is to have major system flows directed towards Fern Casey Street.

Given that the updated preferred stormwater concept has not been detailed in background documents, a stormwater model has been prepared for the South West quadrant within the *East Urban Community / Preliminary Hydraulic Gradeline Analysis and Pond Design* (JFSA, June 2019) – see **Appendix E**. The modelling indicates that the main road network is expected to adequately convey major system flow per City of Ottawa and MECP standards.

As shown in the proposed grading plan, road grade elevations for the conceptual road network are anticipated to be greater than 1.5 m in some areas. The maximum permissible grade raise for the South West quadrant is 0.5 m - 1.5 m per the *Geotechnical – Existing Conditions Report East Urban Community Mixed Use CDP* (Paterson Group, June 28, 2018). Note that the conceptual road network is subject to change. The grading plan has been designed as low as possible to best respect the grade raise restrictions and was determined by providing minimum cover to the infrastructure (assuming basements for all land uses with gravity foundation drainage), facilitating major system flow to the EUC Pond 1, and respecting existing/proposed road grades surrounding the quadrant.

Since the proposed grading plan indicates portions of the South West quadrant to be above proposed grade raise restrictions, a surcharge program or lightweight fill program may be required to the satisfaction of a licensed Geotechnical Engineer in Ontario. As the design process advances for the quadrant, grading plans, grade raises, surcharge programs and fill specifications will be required from a Geotechnical Engineer.

A summary of the stormwater management strategy for the South West quadrant is provided in **Table 25**.

Table 25: Summary of Stormwater Management Strategy – South West Quadrant

	Governing Servicing Study	Additional Background Servicing Study	Proposed MSS
Study Name	Gloucester and Cumberland East Urban Community Expansion Area and Bilberry Creek Industrial Park Master Servicing Update (Stantec, July 2006)	Design Brief for the Trails Edge West Richcraft Group of Companies (DSEL, January 26, 2015) and the Trails Edge East Phase 1 Servicing and Stormwater Management Report (Stantec, August, 2018)	EUC Phase 3 Area CDP MSS (June 2020)
Minor System Stormwater Management Strategy	Quadrant to be serviced by trunk storm sewers running through southern adjacent lands to the South Forebay of the EUC Pond 1.	Quadrant to be serviced by trunk storm sewers running through southern adjacent lands and a stormwater conveyance channel to the South Forebay of the EUC Pond 1.	Quadrant to be serviced by trunk storm sewers running through southern adjacent lands and a stormwater conveyance channel to the South Forebay of the EUC Pond 1.
Total Area to Existing Control MH 2	N/A	3.65 ha.	3.68 ha.
Avg. C	N/A	0.8	0.8
Total Rational Method Peak Flow to Existing Control MH 2	N/A	845.8 L/s.	853 L/s.
100-year, 3-Hour Chicago Storm + 20% modelled Peak Flow to proposed Existing Control MH 2	N/A	N/A	1,189 L/s.
Total Area to Proposed MH STM 1004	N/A	27.89 ha to MH 1004 & 3.20 ha to MH 1002 downstream	29.28 ha.
Avg. C	N/A	0.8	0.78
Total Rational Method Peak Flow to Proposed STM MH 1004	N/A	4,686.2 L/s to MH 1004 & 757.8 L/s to MH 1002 downstream	4,964 L/s
100-year, 3-Hour Chicago Storm + 20% modelled Peak Flow to existing	N/A	N/A	5,800 L/s.

Appendix D External Reports



Geotechnical Investigation

East Urban Community

Mer Bleue Road
Ottawa, Ontario

Prepared for Richcraft Group of Companies

Report PG3130-3 Revision 1 dated February 7, 2024

4.0 Observations

4.1 Surface Conditions

Currently, the subject site consists of agricultural lands and lands formerly used for agricultural purposes. A portion of the east side is occupied by an existing snow disposal facility. The site and regional topography are relatively flat with a slight downslope towards the west and south. The site is approximately at grade with neighboring properties and adjacent roadways. It should be noted that an existing Hydro corridor runs roughly in an east-west direction through the south portion of the site.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations varies between shallow bedrock and a deep silty clay deposit across the subject site. Shallow bedrock was encountered below a cultivated organic zone/topsoil followed by a silty sand, stiff silty clay and/or clayey silt layer within the north portion of the site. The remainder of the subject site was underlain by a sensitive silty clay deposit. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Bedrock

Based on available geological mapping, the bedrock in this area mostly consists of interbedded limestone and dolomite of the Gull River formation with an overburden drift thickness of 0 to 30 m depth.

Atterberg Limit Tests

Atterberg limit testing of 36 samples was completed. The Plasticity Index of the underlying silty clay was measured to range from 24 to 51. The results of the Atterberg limit testing on select silty clay samples are presented in Appendix 1.

Grain Size Distribution Tests

Twelve (12) sieve analyses were completed to classify selected soil samples according to the Unified Soil Classification System (USCS). The results are presented in Appendix 1.

4.3 Groundwater

Generally, the groundwater levels recovered from the piezometers installed at the borehole locations varied between 0.2 and 7.3 m below existing ground surface. It is important to note that groundwater readings at piezometers can be influenced by surface water perched within the borehole backfill material. Long-term groundwater conditions can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, it is estimated that long-term groundwater level can be expected between **2.5 to 3.5 m** depth. Groundwater levels are subject to seasonal fluctuations and therefore could vary during time of construction. The groundwater conditions observed at the borehole and test pits were recorded in detail in the field. Our groundwater observations are presented in the Soil Profile and Test Data sheets in Appendix 1.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is adequate for the proposed development. Bedrock removal may require line drilling and blasting or hoe ramming depending on the depth of bedrock removal required. Due to the presence of the sensitive silty clay layer, residential buildings should be designed in accordance with Part 4 of the current Ontario Building Code (OBC). Also, due to the sensitive silty clay deposit, the proposed development will be subjected to grade raise restrictions.

Permissible grade raise recommendations have been designed for the subject site. The recommended permissible grade raise areas are presented in Drawing PG3130-7 – Permissible Grade Raise Plan in Appendix 2. If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

Municipal services are anticipated within the subject site and will generally be completed through OHSA Type 2 and 3 soils.

The above and other considerations are further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding, and other settlement sensitive structures.

Due to the relatively shallow depth of the bedrock surface within the north portion of the site and the anticipated founding level for the proposed buildings, bedrock removal may be required.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where only a small quantity of the bedrock needs to be removed. Sound bedrock may be removed by line drilling and controlled blasting and/or hoe ramming.

5.6 Pavement Structure

For design purposes, the pavement structure presented in the following tables could be used for the design of car only parking areas and local roadways.

Table 6 – Recommended Pavement Structure – Car Only Parking Areas/Driveways	
Thickness (mm)	Material Description
50	Wear Course – HL-3 or Superpave 12.5 Asphaltic Concrete
150	BASE – OPSS Granular A Crushed Stone
300	SUBBASE – OPSS Granular B Type II
SUBGRADE – Either in situ soil or OPSS Granular B Type I or II material placed over in situ soil.	

Table 7 – Recommended Pavement Structure – Local Roadways	
Thickness (mm)	Material Description
40	Wear Course – HL-3 or Superpave 12.5 Asphaltic Concrete
50	Binder Course – HL-8 or Superpave 19.0 Asphaltic Concrete
150	BASE – OPSS Granular A Crushed Stone
450	SUBBASE – OPSS Granular B Type II
SUBGRADE – Either in situ soil or OPSS Granular B Type I or II material placed over in situ soil.	

Table 8 – Recommended Pavement Structure – Arterial Roadways with Bus Traffic	
Thickness (mm)	Material Description
40	Wear Course – HL-3 or Superpave 12.5 Asphaltic Concrete
50	Upper Binder Course – HL-8 or Superpave 19.0 Asphaltic Concrete
50	Lower Binder Course – HL-8 or Superpave 19.0 Asphaltic Concrete
150	BASE – OPSS Granular A Crushed Stone
450	SUBBASE – OPSS Granular B Type II
SUBGRADE – Either in situ soil or OPSS Granular B Type I or II material placed over in situ soil.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and future details of the proposed development have been prepared:

- ☐ Review preliminary and detailed grading, servicing and landscaping plans from a geotechnical perspective.
- ☐ Review of the geotechnical aspects of the foundation drainage systems prior to construction, if applicable.

It is a requirement for the foundation design data provided herein to be applicable that a material testing and observation program be performed by the geotechnical consultant. The following aspects of the program should be performed by Paterson:

- ☐ Review and inspection of the installation of the foundation drainage systems.
- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Sampling and testing of the concrete and fill materials used.
- ☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ☐ Observation of all subgrades prior to backfilling and field density tests to determine the level of compaction achieved.
- ☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

All excess soil must be handled as per ***Ontario Regulation 406/19: On-Site and Excess Soil Management***.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with Paterson's present understanding of the project. Paterson requests permission to review the grading plan once available. Paterson's recommendations should be reviewed when the drawings and specifications are complete.

The client should be aware that any information pertaining to soils and all test hole logs are furnished as a matter of general information only. Test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests to be notified immediately in order to permit reassessment of the recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Richcraft Group of Companies or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.



Owen R. Canton, B.Eng.



Scott S. Dennis, P.Eng.

Report Distribution:

- ☐ Richcraft Group of Companies (Email Copy)
- ☐ Paterson Group (1 Copy)

re: Completion of Surcharge Settlement Monitoring Program
Proposed Residential Development – Trails Edge North
Brian Coburn Boulevard – Ottawa

to: Richcraft - **Mr. Jesse Ross** - jesser@richcraft.com
date: November 13, 2023
file: PG2384-MEMO.142

Further to your request, Paterson Group (Paterson) prepared the current memo to provide a completion summary of the settlement monitoring program for the surcharge material within Parcel A of the Trails Edge North residential development. This memo should be read in conjunction with Paterson Group Memorandum PG3130-MEMO.02 Revision 1 dated June 5, 2019.

Background

As presented in the above-noted memo, Paterson identified several grade raises which exceed the permissible grade raise restriction recommended for the subject site. Based on the grading requirements, anticipated construction schedule and correspondence with Richcraft, a **2.0 m** settlement surcharge monitoring program was initially selected to achieve the desired settlements within the residential lots prior to construction of the proposed residential development. Details of the settlement surcharge monitoring program and results of the periodic monitoring survey events are presented on the following pages.

The location of the settlement surcharge monitoring program is presented on Drawing PG2384-6 Revision 15 – Settlement Plate Location Plan attached to the current memorandum.

Surcharge Settlement Monitoring Program Summary

The 2.0 m settlement surcharge monitoring program was initiated in January/February 2018 by removing the overlying organic topsoil layer and stockpiling the material within the east portion of East Urban Community (EUC) South/Trails Edge North at Block 181 to Block 186 to be utilized as part of the settlement surcharge monitoring program. To compensate for the lighter topsoil material used for the above noted blocks, the surcharge height was increased to a **2.5 m** surcharge.

During January/February 2019, the surcharge material was imported from the excess generated fill from the service installation within Phase 1 of the current residential development. Upon completion of the placement of the fill material to satisfy the recommended 2.0 m settlement surcharge program for several portions of the site,



Paterson was notified, and 6 settlement plates identified as SP 40, SP 41, SP 42, SP 43, SP 44 and SP 45 were installed between March 6 and 8, 2019. The initial settlement surveys for SP 40 to SP 45 were completed on March 13, 2019.

Settlement plates SP 46, SP 47, SP 48 and SP 49 were installed on March 21, 2019 prior to the placement of the surcharge material within the surrounding areas and the initial settlement surveys for SP 46 to SP 49 were conducted on April 4, 2019.

Based on the ground surface elevations recovered at the time of the settlement surveys prior to January 2020, it appears that the loosely placed surcharge material that was placed during freezing conditions had thawed and had subsequently consolidated well below the recommended surcharge heights.

Our field observations during January and February 2020 indicated that the surcharge fill (topsoil) in the vicinity of settlement plates SP 40, SP 41 and SP 42 had been brought up to the recommended surcharge height of 2.5 m above finished grade. Furthermore, excess generated fill from Phase 2 had been placed during January and February 2020 in the vicinity of settlement plates SP 48 and SP 49 to satisfy the 2.0 m surcharge.

It should be noted that settlement plates SP 40, SP 41 and SP 49 were inaccessible at the time of our settlement survey on February 10, 2020, as a result of the recent placement of surcharge material to satisfy the 2.0 m surcharge. The protective PVC casings at SP 40, SP 41 and SP 42 were exposed and straightened on February 13, 2020.

At the time of our settlement survey on March 31, 2020, the majority of the settlement plates to the north of the relocated ditch were inaccessible. The earthwork contractor ASL had subsequently provided a pedestrian culvert crossing at two locations to provide access for future settlement surveys. Settlement plates SP 50 and SP 51 were installed on March 24, 2020, through the recently placed surcharge material, and the initial settlement surveys for these plates were conducted on March 31, 2020.

Furthermore, settlement plates SP 52, SP 53 and SP 54 were installed on February 1, 2021, and the initial settlement survey was conducted on February 1st, 2021. Settlement plates SP 55, SP 56 and SP 57 were installed on March 5th, 2021, and the initial settlement survey was completed on March 18th, 2021.

Elevations have been referenced to a temporary benchmark (TBM) consisting of the concrete in-filled steel well casing extending to the underlying bedrock surface. The TBM was assigned a geodetic elevation of 89.503 m, provided by Annis O'Sullivan Vollebekk Ltd.



Geotechnical Review and Commentary

Based on our recent correspondence with Richcraft and the contractor, it is our understanding that surcharge material is being placed around settlement plates SP 52, SP 53 and SP 54 within the North Phase to satisfy the recommended **2.0 m** settlement surcharge program. Settlement plates SP 52, SP 53 and SP 54 were installed on January 28, 2021, and the initial settlement surveys for SP 52 to SP 54 were conducted on February 1, 2021.

Following further correspondence with Richcraft and the contractor, it is our understanding that a significant amount of excess fill material, generated from the ongoing surcharge program, has been stockpiled in the western portion of the North Phase (west of settlement plates SP 44, SP 46 and SP 47). Settlement plates SP 55 and SP 56 were installed on March 2, 2021, and settlement plate SP 57 was installed on March 5, 2021, to accurately monitor settlement in the area of the surplus fill material. The initial settlement surveys for settlement plates SP 55 to SP 57 were conducted on March 18, 2021.

In reviewing the data from monitoring carried out to date, cumulative settlements of up to 106 mm, 100 mm, 70 mm, 39 mm, 138 mm and 117 mm have been recorded at settlement plates SP 40 to SP 45, as compared to the initial settlement surveys completed on March 13, 2019.

Cumulative settlements of up to 91 mm, 90 mm, 83 mm and 104 mm have been recorded at settlement plates SP 46 to SP 49 since the initial settlement surveys conducted on April 4, 2019.

Cumulative settlements of up to 141 mm and 124 mm have been recorded at settlement plates SP 50 and SP 51 since the initial settlement surveys conducted on March 31, 2020.

Cumulative settlements of up to 81 mm, 93mm, 117mm, 59mm, 91mm, and 68 mm have been recorded at settlement plates SP 52 to SP 57 since the initial settlement surveys conducted on March 18, 2021.

Furthermore, the rate of settlement has observed to be decreasing over the past two survey events with minor total settlement recorded. Based on Paterson's review of the monitoring results, **the surcharge settlement monitoring program is considered to be complete from a geotechnical perspective**. The settlement survey results are presented in Figure 1 – Surcharge Settlement Monitoring Program Summary attached to the memorandum.



Ms. Jesse Ross
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We trust that the current submission meets your immediate requirements.

Best Regards,

Paterson Group Inc.

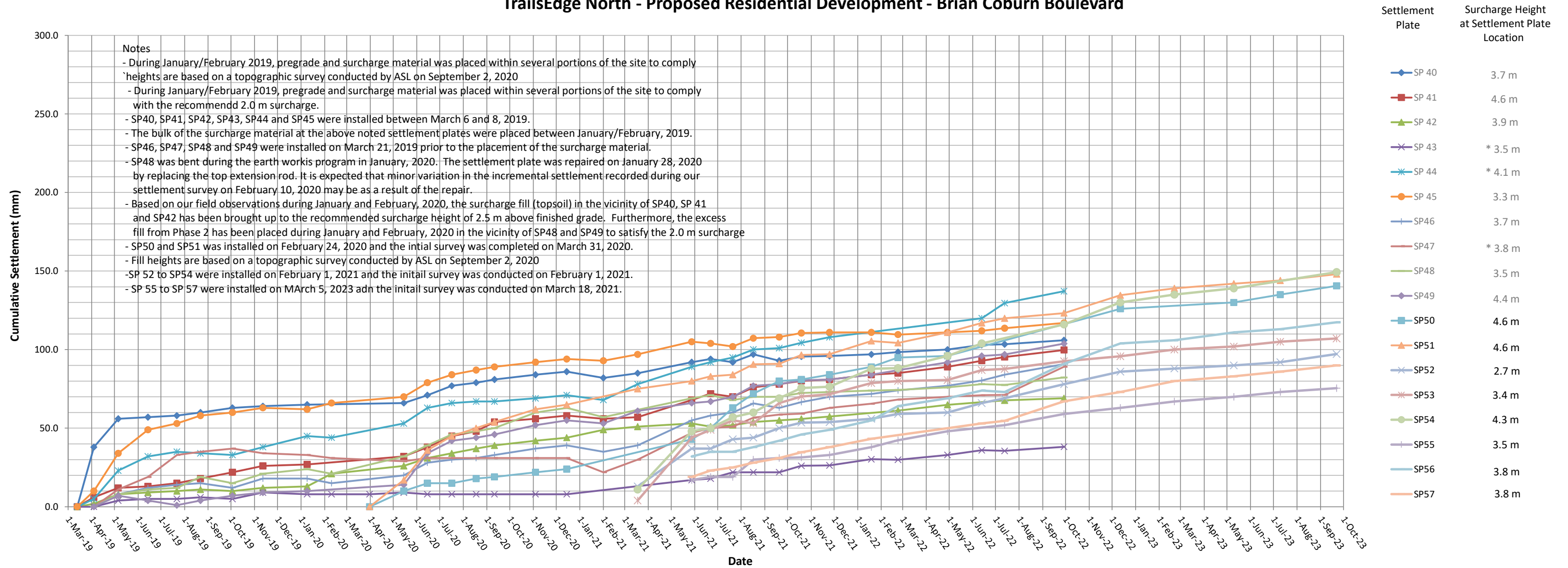
Owen Canton, EIT

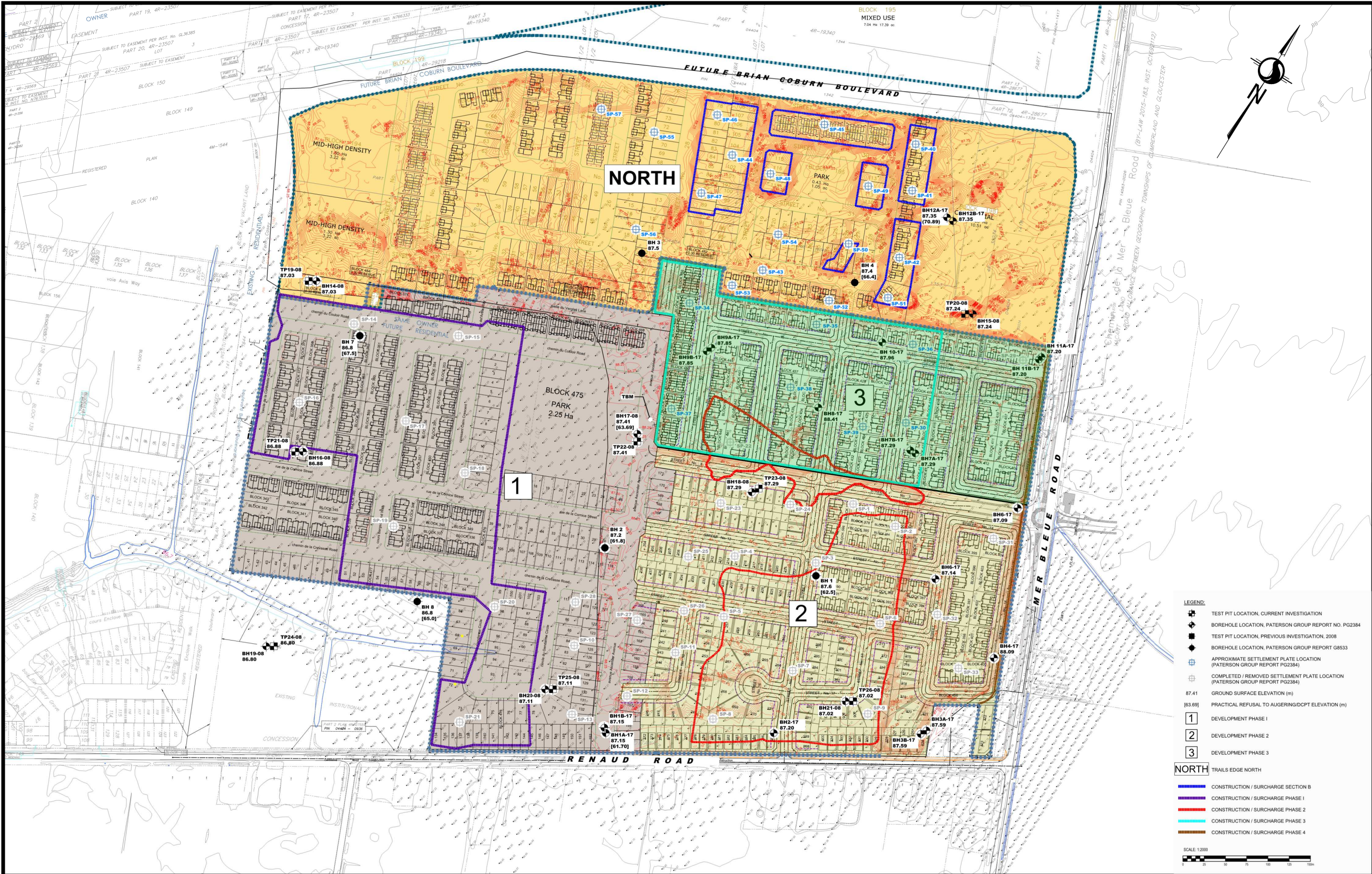


Scott S. Dennis, P.Eng.



Figure 1 - Surcharge Monitoring Program
TrailsEdge North - Proposed Residential Development - Brian Coburn Boulevard





LEGEND:

- TEST PIT LOCATION, CURRENT INVESTIGATION
- BOREHOLE LOCATION, PATERSON GROUP REPORT NO. PG2384
- TEST PIT LOCATION, PREVIOUS INVESTIGATION, 2008
- BOREHOLE LOCATION, PATERSON GROUP REPORT G8533
- APPROXIMATE SETTLEMENT PLATE LOCATION (PATERSON GROUP REPORT PG2384)
- COMPLETED / REMOVED SETTLEMENT PLATE LOCATION (PATERSON GROUP REPORT PG2384)
- GROUND SURFACE ELEVATION (m)
- PRACTICAL REFUSAL TO AUGERING/DCPT ELEVATION (m)
- 1 DEVELOPMENT PHASE 1
- 2 DEVELOPMENT PHASE 2
- 3 DEVELOPMENT PHASE 3
- NORTH TRAILS EDGE NORTH
- CONSTRUCTION / SURCHARGE SECTION B
- CONSTRUCTION / SURCHARGE PHASE 1
- CONSTRUCTION / SURCHARGE PHASE 2
- CONSTRUCTION / SURCHARGE PHASE 3
- CONSTRUCTION / SURCHARGE PHASE 4

SCALE: 1:2000

PATERSON GROUP

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NO.	REVISIONS	DATE	INITIAL
15	UPDATED PHASE 2 & 3 SITE PLAN	16/07/2021	OC
14	UPDATED TO INCLUDE SETTLEMENT PLATES SP95 TO SP97	05/04/2021	VD
13	UPDATED TO INCLUDE COMPLETED / REMOVED SETTLEMENT PLATES	08/02/2021	OC
12	UPDATED TO INCLUDE SETTLEMENT PLATES SP92 TO SP94	20/11/2020	RG
11	UPDATED WITH CONTOURS RECOVERED IN THE FIELD ON SEPT. 2, 2020	06/04/2020	RG
10	SETTLEMENT PLATES LOCATION UPDATED	21/02/2020	RG
9	MARCH 2019 SETTLEMENT PLATES ADDED (SP46 TO SP49)	03/04/2019	RG
8	TRAILS EDGE NORTH BASE PLAN UPDATED AND 2019 SETTLEMENT PLATES ADDED (SP39 TO SP45)	12/03/2019	RG

RICHRAFT GROUP

GEOTECHNICAL INVESTIGATION
TRAILS EDGE RESIDENTIAL DEVELOPMENT - RENAUD ROAD
OTTAWA, ONTARIO

SETTLEMENT PLATE LOCATION PLAN

Stamp:

Scale: 1:2000

Drawn by: RCG

Checked by: RG

Approved by: DJG

Date: 11/2016

Report No.: **PG2384-2**

Drawing No.: **PG2384-6**

Revision No.: 15

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