Consolidated Report
Swan Lake Flow Diversion Assessment

City of Markham

Project Number: 60721132

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AECOM: 2024-12-21

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Revision History

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1	2025-03-26	Menghui Xu	Consolidated pervious technical memos into a final report
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1. Introduction

Swan Lake, located at the intersection of 16th Avenue and Williamson Road in the City of Markham (the City), has approximately 5.5 hectares of open water surface area with a maximum depth of 4.5 metres. Over the past several years, numerous studies have identified water quality issues in Swan Lake, including elevated chloride and phosphorus levels, the occurrence of algal blooms, and reduced dissolved oxygen levels. In response, the City launched a Long-Term Management Plan in 2021 (Markham, 2021) which outlines a phased adaptive strategy to improve the lake's ecological sustainability. For chlorides specifically, the Plan prioritizes source control to reduce chloride levels within Swan Lake, which have already contributed to significant improvements in the water quality of Swan Lake. As a part of the Long-Term Management plan, the City is exploring potential mitigation options which may include redirection of some urban stormwater runoff from the Lake to the local storm sewer system.

To support the City's stormwater diversion evaluation, AECOM has been retained to develop a one-dimensional dual drainage hydraulic model using InfoWorks ICM to assess the feasibility of diverting stormwater runoff from Swan Lake without increasing the flood risk within the study area or locations downstream. The City has proposed that the following flow diversion alternatives be assessed and modelled as part of this assignment:

1. Existing conditions (combine the existing Swan Lake catchment Infoworks model with the downstream area Markham Village and Unionville models).
2. Redirecting minor system flow from the AMICA oil grit separator (OGS) and Swan Lake Blvd. OGS units to the 16th Ave. sewers.
3. Redirecting minor system flow from AMICA OGS and Swan Lake Blvd. OGS units to the Lake outlet.
4. Redirecting the "first flush" portion of the minor system flow from the AMICA OGS and Swan Lake Blvd. OGS units to the 16th Ave. sewer (i.e., redirect the most pollutant-laden runoff in a small diversion sewer).
5. Redirecting minor system flow from Swan Club OGS to the North Pond.
6. Adjusting the flow splitter weir for the East Pond and North Pond to reduce flow bypass to the Lake.
7. Expanding the storage capacity in the East Pond and North Pond to reduce flow bypass to the Lake (to consider if the flow redirection scenarios increase flood risk).
8. Creating underground storage capacity to attenuate the flows from AMICA OGS and Swan Lake Blvd. OGS before they enter the local sewer system (consider if there is a feasible candidate site and if the redirecting scenarios increase flood risk).
9. Redirecting/pumping flows from some foundation drain collectors (FDCs) toward Swan Lake (i.e., supply potentially cleaner, cool groundwater to the Lake).

The primary goal of the project is to develop a dual drainage model to estimate annual flow volume that can be diverted under each scenario, and to identify a preferred scenario for diverting runoff away from Swan Lake without increasing flood risk in the study area and locations downstream.

The main objectives of this project are:

- Develop a dual drainage hydrologic and hydraulic model to represent the integrated storm and overland drainage systems, and to calibrate and validate the model using available flow monitoring data;
- Assess the performance of storm and overland systems under existing conditions;
- Develop models for eight City-proposed scenarios which divert flow away from Swan Lake;

- Assess the annual volume reduction and downstream impact for these scenarios; and
- Evaluate these scenarios based on the cost of implementation, anticipated downstream impacts, annualized reduced flow volume, and presumed reduction in chloride loading to Swan Lake, estimated using winter runoff volumes as a proxy

This report provides a summary of the methods used and assumptions applied in the development of the Swan Lake dual drainage hydraulic model. The model was built using InfoWorks ICM Version 2021.1, following the procedures specified in the City of Markham Stormwater Modelling Guidelines Version 1 (Cole Engineering, 2020).

2. Background Information

2.1 Study Area

As shown in **Figure 1**, the study area covers approximately 148 hectares, and includes residential, commercial, and parkland land uses. The study area consists of the catchment areas of Swan Lake, City Pond 103, and the area south of 16th Avenue that drains directly to Mount Joy Creek. Most streets in the study area are serviced by conventional storm sewers.

The Swan Lake catchment area is 42.9 ha (excluding the lake itself). In the Swan Lake catchment, runoff is collected by local storm sewers and conveyed to the North and East Ponds. Low intensity rainfall events drain directly into these ponds, while high intensity events are diverted to Swan Lake when flow levels are high enough to spill over weirs located at the inlet of each pond. The outflow from both Swan Lake and the East Pond is then collected by downstream storm sewers on Lakeside Vista Way, Lehman Crescent, Larkin Street, and Fincham Avenue, ultimately discharging to the creek. The outflow from the North Pond is received by the 375 mm sewers on Williamson Rd, which eventually discharge to City Pond 102.

The major industrial, commercial, and institutional (ICI) zone in the study area is along the south shoreline of Swan Lake. Runoff in this area is pre-treated by three OGS units before draining to Swan Lake.

The area to the south of 16th Avenue was included in the City's existing hydraulic model developed for the Markham Village and Unionville Flood Control Study (RVA, 2021). The Swan Lake hydraulic model focuses on the catchment areas of Swan Lake and City Pond 103. The completed model was then integrated with the City's existing downstream model to assess the overall downstream impact of flow diversion options.

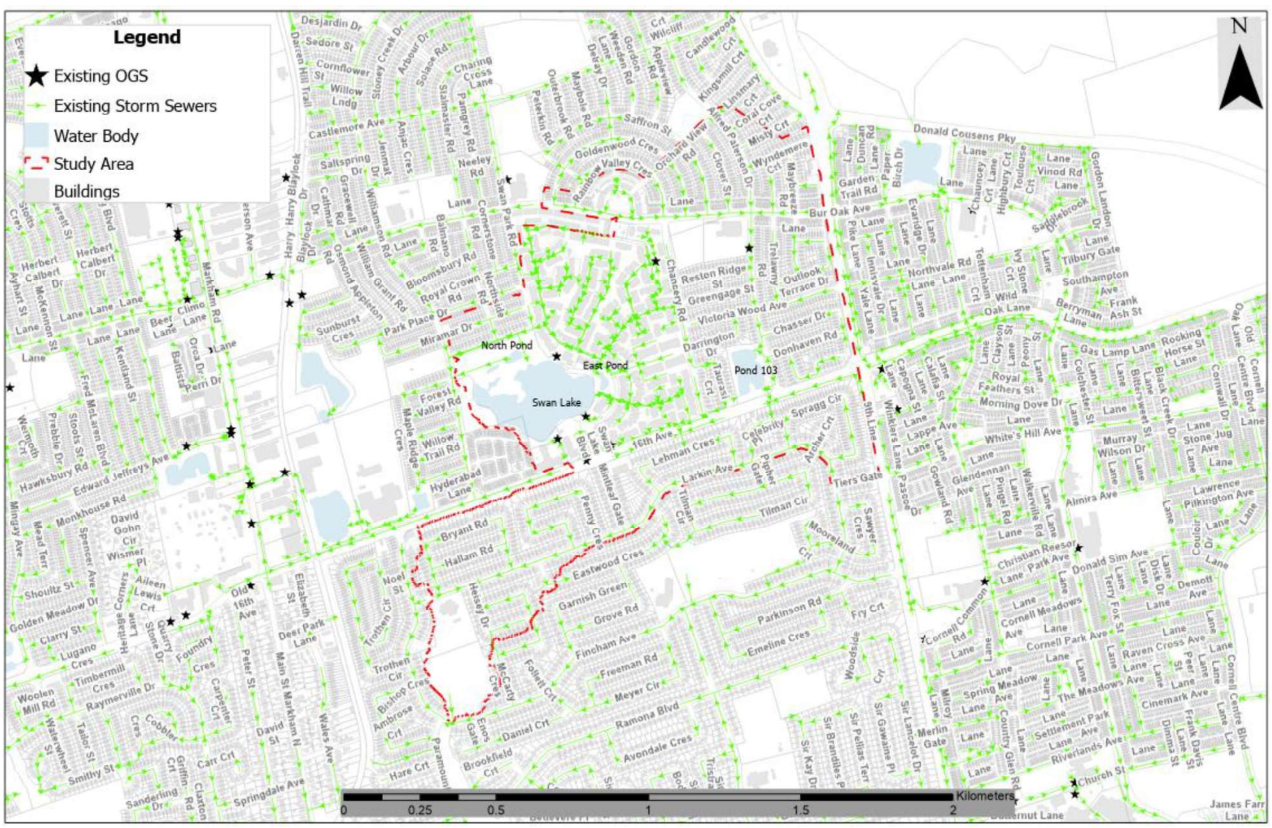


Figure 1: Study Area

2.2 Data Collection and Review

2.2.1 Storm Network

The storm network shapefiles were provided by the City in geodatabase format. The provided dataset consists of all stormwater assets within the City. AECOM filtered these assets to focus on the study area for a more detailed review and integration into the model.

For modeling purposes, the necessary asset features include asset ID, pipe upstream and downstream invert elevation, pipe upstream and downstream asset ID, pipe material, maintenance hole lid elevation, maintenance hole diameter, maintenance hole depth, catch basin grate type, and roof downspout connection. The provided shapefiles were generally reliable and contain corresponding attributes for most of the required features.

A comprehensive review of the GIS data was undertaken to gain an understanding of the storm system and to identify any data gaps. **Table 1** outlines a summary of the storm features in the study area, and **Section 3** highlights the identified data gaps.

Table 1: Storm Network

Data	Received Data	Data Source	Format	Quantity in Study Area (units)	Notes
Storm Maintenance Hole	12-11-2023	City of Markham	GIS shapefile	444 (-)	■ Depth available (“Depth” attribute) ■ Diameter available (“Width” attribute”) ■ Lid elevation NOT available ■ FDC maintenance holes are included in the shapefile of Storm maintenance hole.
Storm Sewer	12-11-2023	City of Markham	GIS shapefile	20.2 (km)	■ Inverts level available (“UPELEV and “DOWNELEV” attributes) ■ Corresponding maintenance hole data available (“UPSTREAM” and “DOWNSTREAM” attributes) ■ Material available (“Material” attributes)
Storm OGS	12-11-2023	City of Markham	GIS shapefile	6 (-)	■ Model number available (“Model Number” attribute)
Storm Catch Basin	12-11-2023	City of Markham	GIS shapefile	543 (-)	■ Catch basin type available - included ditch inlets, rear yard and private catch basins. ■ Grate type NOT available
Roof Downspout Survey	12-11-2023	City of Markham	GIS shapefile	0 (-)	■ Downspout survey did not include the area north of 16th Avenue.
Flow Monitoring	12-11-2023	City of Markham	Spreadsheet, Shapefiles	6 locations	■ Time Interval – 11/1/2022 to 10/31/2023 ■ 3 Storm Locations – MH # M718W (2.5 ha), S304 (Pond 104 inflow, 11.6 ha), Y030 (Pond 104 outflow) ■ 1 FDC locations - MH # J689 (Tributary includes 69 lots, along Swan Park Rd) ■ 2 Mixed locations MH # M718N, F973

2.2.2 Stormwater Management Facilities

There were three wet ponds in the study area, and all of them discharged to downstream storm sewers. For modeling purposes, these wet ponds that discharged into storm sewers required the incorporation of a stage-storage relation into the model. Stormwater management reports and available drawings were reviewed to identify these features of ponds and any data gaps, as summarized in **Table 2**. There is no missing data required for these three ponds and Swan Lake.

Table 2: Storm Ponds

City Asset ID	Designed Drainage Area (ha)	Volume (m^3)		Normal Water Level (m)	Stage-Storage Available in Report	Outlet Structures
		Permanent	Active			
Pond 103	48.7	12635	45334	206	Yes	■ 280 mm orifice at 206.47 m ■ 2 DICBs at 207.1 m
Pond104	12.6	1558	810	208.3	Yes	■ 100 mm orifice at 208.3 m
Pond 105	19.3	2051	1096	208.3	Yes	■ 66 mm Orifice at 206.8 m
Swan Lake	42.9	62,640 (at normal depth)	99,380 (at maximum depth)	208.3	Yes	■ 1.3 m (crest length) weir at 208.3 m discharge to 165 mm orifice at 207 m

2.3 Background Reports

2.3.1 City of Markham Stormwater Modelling Guideline (Cole Engineering Group Limited, 2020)

This report outlined the best practices for storm system hydraulic modeling, covering asset naming conventions, catchment discretization, runoff routing, high point and sag point identification, data requirements, hydraulic model parameterization, and model validation procedures. Guideline values and procedures for the following items, as outlined in the report, were reviewed and applied in order to develop the InfoWorks hydraulic model for the current study:

- Manning's roughness coefficients for pipes and roads.
- Standard conduit shapes for streets.
- Catchment geometries.
- Catchment hydrologic properties for each type of land use.
- Catch basin rating curve for different types of grates.
- Flag and naming format.
- Model reporting format.

2.3.2 Markham Village & Unionville Flood Remediation Plan, and the Correlated InfoWorks Model (RVA, 2021)

Three significant storm events occurred in the City of Markham between June and July 2017, leading to 350 flood reports. In response, the city recognized the need to assess and mitigate flood risks in the Markham Village and Unionville areas. The purpose of this study was to evaluate the existing stormwater drainage system in these areas and develop a comprehensive plan for implementation.

An InfoWorks model was developed for Markham Village area, and this model will be used as the base model for the current study; the current study will extend this model to include the study area north of 16th Avenue. Several system deficiencies were identified in this study, including surcharging and overflowing to ground level during smaller, 2-year storm events; failure to meet current level of service criteria; and an elevated risk of street ponding.

The proposed final flood remediation plan encompasses system upgrade recommendations, risk priorities, financial planning, and regulatory approvals for implementation.

AECOM reviewed the existing InfoWorks model following standard model review procedures outlined in the City of Markham Stormwater Modelling Guidelines (Cole Engineering Group Limited, 2020). The model parameters were determined to be accurate and aligned with Guideline values. Sub-catchment areas are derived from property parcel fabrics and are verified to be appropriately characterized to represent each type of runoff surface in the study area. Although the model was not calibrated due to a lack of site-specific data, the model results were compared with the storm event on July 16th, 2019. The predicted problematic areas were generally consistent with the recorded flooding locations along Church Street, but the predicted flooding locations were fewer than recorded in other areas.

2.3.3 Swan Lake Long-Term Management Plan (City of Markham, 2021)

This study outlined issues, opportunities, and a strategy for improving the water quality of Swan Lake in Markham. The report analyzed the current state of the lake, identifying issues such as high phosphorus levels, geese-related nutrient inputs, and elevated chloride concentrations. The report presented a phased approach with core measures for the first five years, including continued water quality monitoring, enhanced geese management, and the use of chemical treatments. Complementary measures, such as fish management plans and the installation of shoreline plantings, were introduced in the second phase, while the third phase considered adapted core measures and potential alternative strategies, such as investigating groundwater contributions and stormwater redirection. The 25-year plan aimed to achieve a low eutrophic condition in the lake, improve water clarity, and reduce algal bloom frequency.

The water balance study outlined in the report and the correlated PCSWMM model provide insights into the flow contribution from ponds and oil grit separators (OGSs) to Swan Lake, as well as the hydrologic characteristics of Swan Lake catchments critical to the current study.

2.4 Evaluation Criteria

Criteria outlined in the City of Markham Stormwater Modelling Guidelines (Cole Engineering Group Limited, 2020) are summarized as follows. These criteria were followed to evaluate existing system and feasibility of diversion scenarios:

Storm Sewers

- Surcharge state 1 **–** No surcharge will be considered as low risk.
- Surcharge state 2- The pipe is surcharged, but the slope of the HGL is flatter than the pipe slope (i.e. it is surcharged due to downstream conditions), which will be considered as moderate risk.
- Surcharge state 3- The pipe is surcharged, and the slope of the HGL is steeper than the pipe slope (i.e. the surcharge is at least in part caused by the pipe capacity), which will be considered as high risk.

Storm Maintenance Hole

- Maximum HGL is greater than 2.0 m below ground elevation will be considered as low risk.
- Maximum HGL is within 2.0 m of ground elevation will be considered as moderate risk.
- Maximum HGL exceeds ground elevation will be considered as high risk.

Overland

- Overland flow depth lower than 150 mm will be considered as low risk.
- Overland flow depth between 150 mm and 300 mm will be considered as moderate risk.
- Overland flow depth exceeds 300 mm will be considered as high risk.

2.5 Data Gaps

A background review of City-provided data was completed prior to undertaking model updates, and several information gaps were identified. Several data gaps were identified:

a) Storm Sewer Invert Elevations (Including FDC pipes)

Catch basins are marked On Roy Grove Way and Town Villa Way without corresponding pipes and maintenance holes. As shown in the **Figure 2**.

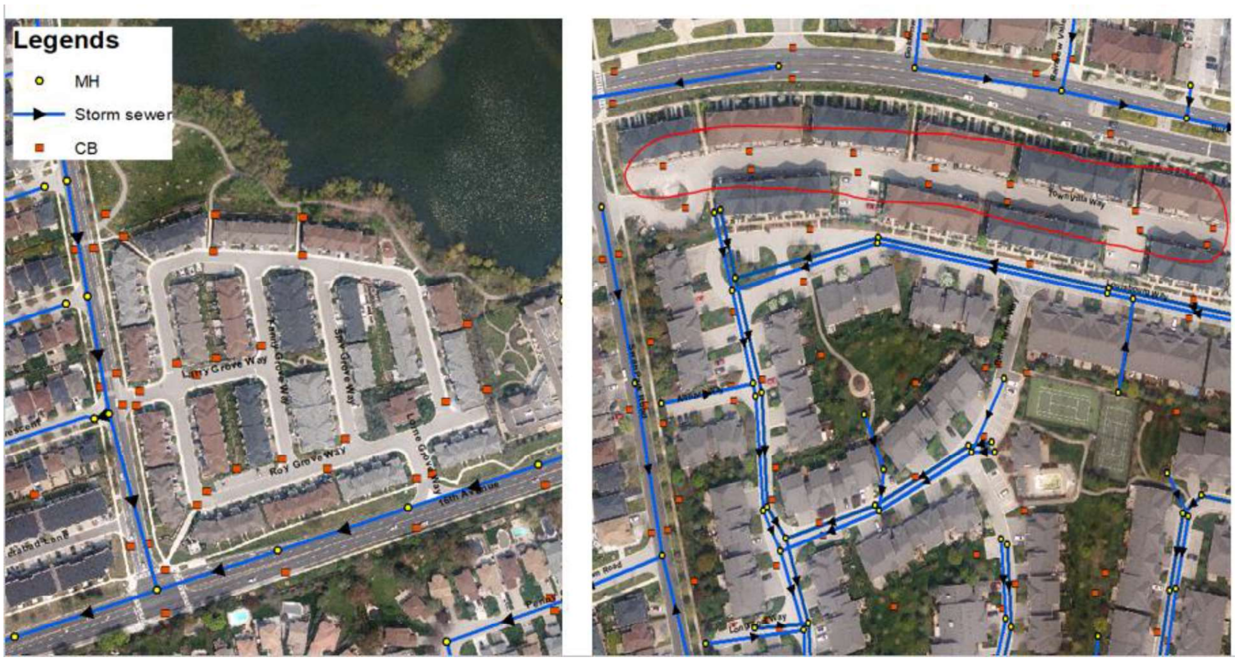


Figure 2: Catch Basin with out Corresponding Pipes

Data gaps in storm sewer invert elevation are listed as follows and shown in **Figure 3**.

- 6 of 467 (1.2%) pipe sections did not have upstream elevations but have downstream elevations.
- 17 of 467 (3.6%) pipe sections did not have downstream elevation but have upstream elevations.
- 129 of 467 (27.6%) pipe sections did not have elevation at both ends.

Note that the pipes with missing elevations with both ends are primarily in the private development area (see sketch below).

Figure 3: Pipe Missing Elevation with Both Ends

While most of the missing inverts were covered in the provided scanned as-built drawings, some of the data in the drawing were not readable due to scanning and resolution issues. As advised by the City, invert interpolations were applied to infer the missing elevation from the available upstream or downstream pipe invert elevations, or slope data available from either the drawing or GIS records. After the initial screening, eight pipe sections listed in **Table 3** were identified as not being available in any of the provided drawings, and located at the most upstream point of a sewer branch. Diameter data are also missing for these pipes.

Table 3: Pipes that Require Invert Survey

Asset ID	Upstream MH ID	Downstream MH ID
Q693Q692	Q693	Q692
Q688Q687	Q688	Q687
Q691Q690	Q691	Q690
Q692Q691	Q692	Q691
Q689Q688	Q689	Q688
Q694Q693	Q694	Q693
Q695Q693	Q695	Q693
Q690Q687	Q690	Q687

Field work was completed to measure the invert levels for these pipes and the collected data was incorporated when developing the Infoworks hydraulic model.

b) Storm Sewer Diameter

15 of 476 (3.2%) pipe sections did not have diameter information, diameters for 7 pipe sections were filled were identified from the provided site servicing plan drawings. Assumptions would not be accurate for the eight pipes listed in **Table 3**, as they are located at the most upstream of a sewer branch, and there is no available drawing for them. Field surveys were completed to measure the diameter for these pipes and the collected data was incorporated when developing the Infoworks hydraulic model.

c) Catch Basin Leads

The catch basin lead layer was not available in the provided geodatabase. AECOM has reviewed the provided drawings to identify catch basin downstream connections. Catch basins which are not included in any of the drawings were assumed to connect to their closest stormwater maintenance holes.

d) Catch Basin Grate Type

Only ditch inlets in catch basin layer were differentiated in the available attribute tables; grate opening numbers and types were not identified for right-of-way and rear yard catch basins. AECOM reviewed street view on Google to identify the type of grate for these catch basins. For areas where street view was not available, AECOM assumed all right-of-way and rear yard catch basins have single grid grates, and catch basins located in sag (depression) areas will be assigned twin herringbone grates, as per instruction provided by the City. This assumption were verified by the field visit. A field survey was conducted to check the grate types of catch basins visible from the right-of-way. A total of 143 catch basins were inspected. Except for 16 rear yard catch basins that have beehive grates for ditch inlets, the remaining 127 catch basins have herringbone grates, with double inlets located at the identified sag locations, which were consistent with the initial assumption.

e) Maintenance Hole Lid (Rim) Elevation

Lid elevations were not available for most the maintenance hole in the study area attributes; the DEM were used to collect lid elevations for maintenance holes to ensure consistency.

f) Maintenance Hole Chamber Diameter

379 of 470 (81%) maintenance holes did not have diameters. An assumption was made that the maintenance hole diameter is 600 mm larger than the largest pipe diameter, with a minimum diameter of 1200 mm.

g) Roof Connection

A downspout survey was provided but did not include the area north of 16^{th} Avenue. Therefore, a visual survey of downspout connectivity was conducted from the right-of-way (ROW) for properties within the Swan Lake catchment area. The results are discussed in Section 3.3.2 of this report.

3. Model Development

The dual drainage model was created with InfoWorks ICM software. Dual drainage represents the surface (major) and underground (minor) flow systems as an interconnected network. Subcatchments were discretized from maintenance hole to maintenance hole. Major overland flow is conveyed to the minor system through "gullies" in the model, representing catchbasins. The details of the dual drainage model are explained in the following sections.

3.1 Minor System

Minor system assets included in the Swan Lake InfoWorks model are shown in **Figure 4** and listed in **Table 4**. The outfalls at manholes F973 and G401 were converted to storm nodes when the Swan Lake hydraulic model was integrated with the existing downstream model.

Table 4: Minor System Assets in the Model

Item	Quantity
Storm Nodes	444
Storm Conduits	446 sections, total length approximately 20.2 km (Including lengths of FDC pipes)
Flow Control Structures	Three flow splitter weirs: ■ North Pond flow splitter weir ■ East Pond flow splitter weirs at both two sewer inlets ■ Swan Lake outlet control weir Five orifices: ■ 100 mm orifice plate at North Pond outlet ■ 66 mm orifice plate at East Pond outlet ■ 165 mm orifice at Swan Lake outlet ■ Two 100 mm orifice in the pipes on Swan Lake Blvd. to control the outflow from ICI area
Outfalls	Three outfalls: ■ Outlet to Markham Village Area at Manhole F973 ■ Outlet to Markham Village Area at Manhole G401 ■ North Pond outlet to downstream system at manhole J681
Storage Nodes	Four major storage nodes: ■ Swan Lake ■ East Pond ■ North Pond ■ Pond 103

Figure 4: Swan Lake Model Extent

3.1.1 Data Source

The primary source of information for model development is the geodatabase provided by the City, which contains GIS data for existing storm sewers, property parcels, storm manholes, catch basins, ponds and the LiDAR derived digital terrain model (DTM). Data gaps, including incomplete and inconsistent information, were identified in the previous technical memo (TM#1, Background Review). A comprehensive data validation was then performed in InfoWorks using the built-in engineering validation and tracing tools to identify connectivity errors. These data gaps and inconsistencies were resolved primarily using the as-built documents provided by the City. As discussed in the previous section, eight (8) pipe sections in the residential area between Chancery Road and Augusta Drive, as shown in **Figure 5**, are not included in any provided drawings. Since these pipes are the most upstream sections of a sewer branch, their invert levels can not be reasonably assumed. Therefore, fieldwork was conducted to collect dimensional and invert data for these assets.

Figure 5: Surveyed Pipes

3.1.2 Manholes

There were 198 manholes in the study area that did not have elevation data available in the provided geodatabase. For the manholes with available elevation in the provided GIS data, a comparison was made between the available data and the LiDAR obtained manhole lid elevation. This comparison shows that the difference between the GIS data and the LiDAR data was generally within 0.2 m. However, in some areas, elevation discrepancies ranged from ±0.5 m to ±2 m, affecting 11% of the total number of manholes. Since the GIS data originated from multiple sources, including as-built drawings, design drawings, inspection reports, historical surveys, and Google Earth, the lid elevations for all manholes were extracted from the LiDAR data to ensure consistency. The available elevations in the GIS data were not used for model development. For pipes that had their invert levels collected from field survey, the surveyor measured the depth from the top of the manholes to the pipe invert. The invert elevation was then calculated by subtracting the measured depth from the LiDAR elevation at those manholes.

3.1.3 Storm Sewers

The properties of storm sewers (conduits), including upstream and downstream invert levels, diameters, and pipe material, were obtained from the data sources outlined in the previous sections. A Manning's n value of 0.013 was applied for all types of concrete and PVC sewers.

3.1.4 Control Structure

A detailed review of all provided as-built drawings and site control plans was conducted to identify all flow control structures in the study area. Overflow weirs and orifice plates listed in **Table 4** were identified from the available drawings and incorporated into the InfoWorks model accordingly.

3.1.5 Catch Basins

In InfoWorks, defining a node as a “gully” enables it to perform like a catch basin, connecting the storm and overland systems. Each “gully” node in the model was assigned the specific number of catch basin within the subcatchment, along with a flow rating curve that allows the gully to represent the actual flow rate that can enter the minor system. The attribute “number of inlets” represents the corresponding number of catch basins connected to each manhole, and “gully type” represents the depth-discharge relation specific to each catch basin grate type. This set-up enables flow accumulated on the overland surface to enter the storm sewer system at a specific flow rate that correlated to the depth defined by a depth-discharge curve (rating curve). Conversely, stormwater in the collection system can surcharge to the overland network through gully nodes

The quantity and location of catch basins were obtained from the City-provided GIS data, and the catch basin inlet grate type for each inlet was gathered from field observations and Google Streetview. Due to the absence of service line connection data, catch basins were assigned to their closest manhole. The survey results show that the study area is primarily serviced by herringbone catch basin inlet grate types. The specified head-discharge curve developed by City of Ottawa, which accounts for on-sag and on-grade inlets, were incorporated into the hydraulic model.

3.1.6 Rear Yard Catch Basins

The number and location of rear yard catch basins were obtained from the City-provided GIS geodatabase. The field survey checked the grate types of rear yard catch basins that are visible from the right-of-way and identified them as either Ditch Inlets (DICB) or herringbone catch basins. Since rear yards in the gated community are primarily located in sag areas with no overland outlet, and the rating curve of a herringbone catch basin in sag areas is identical to that of a DICB, each rear yard catch basin was modeled as a gully node and assigned a storage-discharge curve specific to herringbone catch basins situated in sag areas. The provided subdivision plan drawings were used to identify the connection for rear yard catch basins. For rear yard catch basins that tee into a conduit without a manhole, a dummy node was implemented to represent the connection junctions. In cases where rear yard catch basins were not available in the provided drawings, lead pipes were assumed to be circular, with a diameter of 250 mm and a 1% slope, and were connected to the nearest appropriate stormwater manhole as determined by engineering judgment.

3.1.7 Storage Nodes

In Infoworks models, storage units, including stormwater management ponds, natural water bodies and major sag location, are represented by storage nodes. At each storage node, a stage-area relation is required to represent the storage volume at various elevations. Storage nodes were placed at Swan Lake, the North pond, East pond and City pond # 103. The stage-area relation for the North Pond, East Pond and Swan Lake were obtained from the City provided PCSWMM water balance model. The stage-area relationship for Pond 103 was gathered from Appendix C of the Stormwater Management Design Brief: Pond A, Pond E, and Aviva Pond (Revised by Stantec, 2004). The stage-area relationships applied in the Swan Lake InfoWorks model are summarized in **Table 5**.

Table 5: Stage-Storage Relation in the model

Stage (m)	Area (m^2)	Average Area (m^2)	Volume (m^3)	Storage (m^3)	Active Storage (m^3)
East Pond					
205.8	100	0	0	0	0
206.8	521	311	311	311	0
207.8	1339	930	930	1241	0
208	1675	1507	301	1542	0
208.3	1949	1812	544	2086	0
208.5	2150	2050	410	2495	410
208.8	2446	2298	689	3185	1099
North Pond					
205.8	20	0	0	0	0
206.8	304	162	162	162	0
207.8	1090	697	697	859	0
208.3	1500	1295	648	1507	0
208.8	1740	1620	810	2317	810
209	2200	1970	394	2711	1204
Swan Lake					
204.5	0	0	0	0	0
205	320	160	80	80	0
205.5	1880	1100	550	630	0
206	5800	3840	1920	2550	0
206.5	12000	8900	4450	7000	0
207	20000	16000	8000	15000	0
207.5	34000	27000	13500	28500	0
208	46000	40000	20000	48500	0
208.3	48267	47134	14140	62640	0
208.5	52600	50433	10087	72727	10087
209	54000	53300	26650	99377	26650
Pond 103					
204.5	6532	0	0	0	0
205	7717	7125	3562	3562	0
205.5	8856	8287	4143	7706	0
206	10902	9879	4940	12645	0
206.5	12425	11664	5832	18477	5832
207	13719	13072	6536	25013	12368
207.5	15025	14372	7186	32199	19554
208	16479	15752	7876	40075	27430
208.5	17911	17195	8598	48672	36027
209	19317	18614	9307	57979	45334

3.1.8 Naming Conventions

Most minor system assets in the Infoworks model were named using the City's Asset ID. For objects without an Asset ID, as well as dummy objects and duplicate objects, asset names were assigned in compliance with the naming conventions listed in Table 4.1 of the City of Markham Stormwater Modeling Guidelines Version 1 (Cole Engineering, 2020).

3.2 Major System

The major (overland) system in the model consists of streets with flow constrained by the curb and gutter along both sides, and rear yard channels. The following data sources were used when creating major system.

- The City-provided GIS geodatabase
- LiDAR DEM
- Aerial imagery

3.2.1 Overland Conduit and Channel Geometry

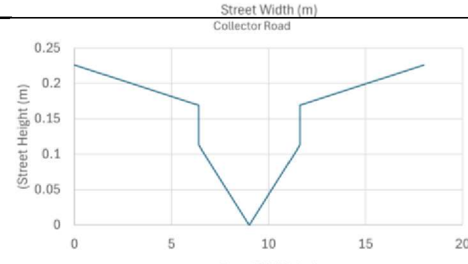
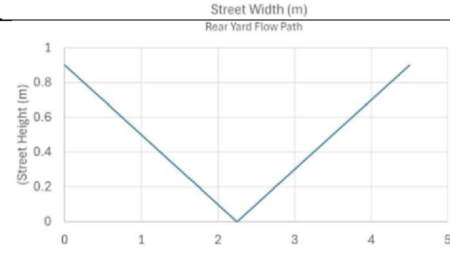
The streets were modelled as wide shallow open channel conduits with irregular cross-sectional shape to reflect the appropriate geometry, flow area and channel roughness. The overland conduit invert levels were set at the maintenance hole lid elevation such that flows can transfer between the minor and major systems if there is flooding out of the maintenance holes from the minor drainage system or when the flow is restricted into the minor system at individual catch basin inlets based on the catch basin inlet capture capacity.

An initial overland network was created by duplicating the minor system network, converting the system type to “overland” adjusting conduit invert levels to the ground elevations of their connected manholes, and reversing direction of conduits when the road slope is opposite of the pipe slope. Pipes with slope reversed to street slope are shown in **Figure 6**. Then, the overland flow path on streets was generated using the Esri ArcHydro tool based on the LiDAR data and compared against the original network. Additional overland conduits were added to the network where the minor system was not continuous. Flow splits at intersections were determined by the model based on the physical network layout topography. Local high points were identified using LiDAR data and added as flow split points, which may not follow the minor system direction.

The major system in the model was primarily defined by three types of roads in the study area: arterial road, collector and local road. Further, flow paths on rural and rear yard lands were also included, as shown in **Table 6**. The typical cross-sections for these three road types were obtained from the provided Markham Village Infoworks Model. Each cross-section was defined by unit width at unit height, then multiplied by the actual road width and height. Roadside ditches do not exist in the study area.

Figure 6: Pipes with Slope Reversed to Street Slope

Table 6: Cross-section Geometry – Arterial Road

Road Type	Unit Height	Unit Width	InfoWorks Conduit Cross-section
Arterial Road	0	0	
	0.5	0.17	
	0.75	0.17	
	1	1	
Collector Road	0	0	
	0.32	0.29	
	0.57	0.29	
	1	1	
Rear Yards	0	0	
	1	1	
	3	3	

3.2.2 Naming Conventions

Major system objects were named following the naming convention outlined in Table 4.1 and 4.2 of City of Markham Stormwater Modelling Guideline Version 1.

3.3 Subcatchments and Hydrology

The following data sources were used when creating the subcatchment areas in the model.

- LiDAR DEM (GIS)
- AECOM visual downspout inspection
- Property parcel (GIS)
- Building roof footprint

3.3.1 Catchment Delineation

Subcatchments were delineated on a manhole-to-manhole basis referring to the topography and property boundaries. Reverse driveway surveys were not conducted for this project, instead, all property front lots were assumed to be graded towards the street and confirmed with the flow path generated using ArcHydro tool. Rear yards were incorporated into the main catchment in cases where specific rear yard flow paths were absent; where flow paths were present, rear yards and the roof area draining towards rear yards were separated from the main catchment and routed to rear yard catch basins if available, or to dummy overland nodes located on flow paths.

The imperviousness of each subcatchment was initially determined by processing aerial images using the ESRI Raster Classification tool. This process involved analyzing the color spectrum of the aerial photos in GIS. The initial estimates were further refined by incorporating known impervious surfaces, such as roads and roofs. **Figure 7** shows an example of this process. Since runoff generated from impervious surfaces (roof leader, driveway, sidewalk, etc.) that drains to pervious surfaces may not be captured by stormwater catch basins, the imperviousness in the catchment areas with flow monitors was further calibrated using flow monitoring data. Roof areas were assumed to be equal to the area of the building footprint. The directly connected roof area to the minor system was calculated based on the downspout status information obtained during the field surveys. Roof downspouts directed into the ground were connected to the minor system. Subcatchments were then further adjusted into three categories based on runoff surfaces as follows:

- Main Storm Subcatchments (Named as: Asset ID_S): were established for the three runoff surfaces: impervious surface (street pavement, sidewalk, driveways, and parking lots), disconnected sloped roofs, and pervious surface (bare soil and green areas). Main subcatchments were assigned to the street gully nodes based on the topography, road grade and overall lot drainage direction.
- Connected Sloped Roofs (Named as: Asset ID_RC): were established for sloped roof areas connected directly to the storm sewer. A separate subcatchment was created from each storm subcatchment containing only directly connected sloped roofs. Total and contributing area of the dummy subcatchments were assumed to equal to the total connected sloped roof areas within the same main storm subcatchment. Connected roofs were discharged to the storm sewer system directly through a lateral. The flow discharged to the storm sewer should be limited to the capacity of the roof drainage system.
- Flat Roofs (Named as: Asset ID_FRC): were established for flat roof areas connected directly to the storm sewer. Flat roofs are normally associated with Industrial, Commercial or Institutional Land Use (ICI) or high-rise residential areas, and typically have large areas that drain to internal plumbing. To

account for the ancillary structures, the connected flat roofs were modelled as separate subcatchments and drained to a dummy node with a storage area equal to the roof area and a head-discharge curve for a flat roof downspout to control flow was used to drain rooftop flows to the storm sewer. Unless specific downspout numbers were available from the building drawings, a general assumption of 1 downspout per 160 m^2 of flat roof area was applied to estimate the discharge limit of flat roof drainage system (in accordance with CoT Infoworks modelling guidelines).

Figure 7: Example – Imperviousness Determination

3.3.2 Roof Connectivity

A visual survey for downspout connectivity was conducted from the right of way (ROW) for properties within the Swan Lake catchment area. Rear yard downspout connections could not be confirmed.

The results of the downspout survey are presented in **Figure 8**. From the 534 properties surveyed:

- 14 properties (2%) have downspouts connected into the ground (storm, sanitary or FDC pipes); and
- 503 properties (95%) have downspout draining to the surface, connected to the overland system; and
- 18 (3%) properties did not have downspouts visible from the ROW.

Figure 8: Roof Downspout Connectivity Survey Results

3.3.3 Roof Area Separation

As shown in **Figure 9**, flat and sloped roofs are identified using aerial imagery. The City's modeling guidelines (Cole Engineering, 2020) recommend using a "split-rainfall method," which assumes that a typical home's downspouts can capture flows from a rainfall event with up to a 5-year peak intensity, while any excess would overflow to the ground. This method involves creating three hyetographs and duplicating the connected roof catchment as overflow catchments. The hyetograph for the full storm rainfall is assigned to the general storm catchments, the hyetograph for up-to 5-year design storm is assigned to the roof catchment, and the overflow catchment is assigned the difference between the rainfall intensities for intervals where the storm rainfall exceeds the peak 5-year design storm rainfall. This method was not applied in the development of the Swan Lake hydraulic model for the following reasons:

- More than 95% of residential roofs in the study area were confirmed to be disconnected from the storm sewers
- The peak intensity of a 5-year design storm varies by event duration, making it difficult to select an appropriate duration for the 5-year design rainfall to match historical events when calibrating the model.

Large flat roofs of these commercial buildings within the study area were incorporated as separate subcatchments and routed to a dummy node with a storage area equal to the roof area and a head-discharge curve for a typical flat roof inlet. It is assumed that each 160 m^2 of flat roof area will be served by one inlet. The detailed runoff surface configuration for flat roofs follows Table 4.9 in the *City of Markham Stormwater Modelling Guidelines*.

Figure 9: Flat Roofs in the Study Area

3.3.4 Hydrologic Conditions

In InfoWorks, the hydrologic characteristics of a catchment are defined by the land use, which encompasses a list of runoff surfaces. The total runoff generated from each subcatchment is quantified by summing the runoff generated by each surface during a storm event. **Table 7** presents the runoff surface IDs, descriptions, and initial guideline values for the associated hydrological parameters for these runoff surfaces. These parameters were further adjusted based on flow monitoring data during the model calibration process.

Table 7: Runoff Surfaces Hydrologic Parameters

Runoff Surface ID	10	20	30	40	50
Runoff Surface Description	General pervious area	General impervious area	Connected slopped roof area	Disconnected roof area	Flat roof area
Runoff Routing Value	0.025	0.013	0.033	0.013	0.013
Runoff Volume Type	Horton	Fixed	Fixed	Fixed	Fixed
Surface Type	Perv.	Imp.	Imp.	Imp.	Imp.
Ground Slope (m/m)	0.01	0.01	0.33	0.33	0.005
Initial Loss Value	0.005	0.01	0.001	0.001	0.001
Fixed Runoff Coefficient	-	1	1	1	1
Horton Initial Infiltration	125	-	-	-	-
Horton Limiting Infiltration	5	-	-	-	-
Horton Decay	2	-	-	-	-
Horton Recovery	1	-	-	-	-

3.3.5 Naming Conventions

Subcatchments were named following the naming conventions outlined in Table 4.3 of City of Markham Stormwater modelling Guidelines, Version 1.(Cole Engineering, 2020)

3.4 Boundary Conditions

- Outlets to the downstream system are shown in **Figure 10**. Downstream boundary conditions can significantly impact the operation of the dual drainage system, as backwater effects may constrain flow in the storm sewers when water levels are high. Water levels in storm manhole M724 and G401 were not applied in the Swan Lake hydraulic model because the model was integrated with the City's existing downstream model after completion, and these two outfalls will be converted to storm nodes in the combined model.

- Outflow from the north pond exits the study area at Manhole J689, and the flow is further conveyed to City Pond 102 via the existing 250- 350 mm storm sewer pipes, with an invert level of 206.9 m. The LiDAR DTM indicates that the current water level in City Pond 102 is at 194 m, and the spill level is at 197 m. It has been determined that the pond does not restrict the outflow from the study area. However, the relatively small pipe sizes and runoff from the surrounding residential development may cause surcharge in the manhole, potentially limiting outflow from the study area. Further investigation is required to assess whether this could constrain outflow from the study area. Currently, manhole MH689 is modeled as a free flow outfall.

- Study area outflow through manhole M724 and G401 drains to the creek outlet by 1050-1800 mm sewers, spanning approximately 1.7 km along Lehman Crescent, Larkin Avenue, and Heisey Drive, as shown in **Figure 10**. This reach was identified as having insufficient capacity in the previous Markham Village and Unionville Flood Control Study (RVA, 2021). To address capacity deficiencies, a 520 m relief sewer, ranging in diameter from 1200 mm to 1800 mm, was proposed, extending from Manhole A095 to the outfall through the parkland, as shown in **Figure 11** (**RCA, 2021**). However, the previous study did not account for external flow from the Swan Lake area.

3.5 Updated Base Case

- After combining the two models, the updated model results indicate that the original proposed solution is insufficient to address all capacity constraints. The surcharge level would rise to above 1.8 m below the ground (assumed basement level) during a 100-year design storm event when flow from the Swan Lake area is included, as shown in **Figure 12**. To maintain flow within pipe capacity, and to address the capacity limit which would affect Swan Lake diversion options, additional upgrades were identified that consisted of upsizing a 790 m section of 1350 mm sewers on Larkin Avenue to 1800 mm, as shown in **Figure 13**. Please note that upsizing the entire 790 m length to 1800 mm is a conceptual scenario that removes downstream restriction at the critical locations; spacing constraints and constructability have not been assessed at this stage. During design stages, the pipe sizes may be gradually increased from upstream to downstream. The upsized pipes were incorporated into the base model and scenario models as a baseline condition, and costs of implementation for this external upgrade will not be considered as part of the implementation costs for the Swan Lake flow diversion scenarios.

Figure 10: Swan Lake Connections locations to downstream sewers

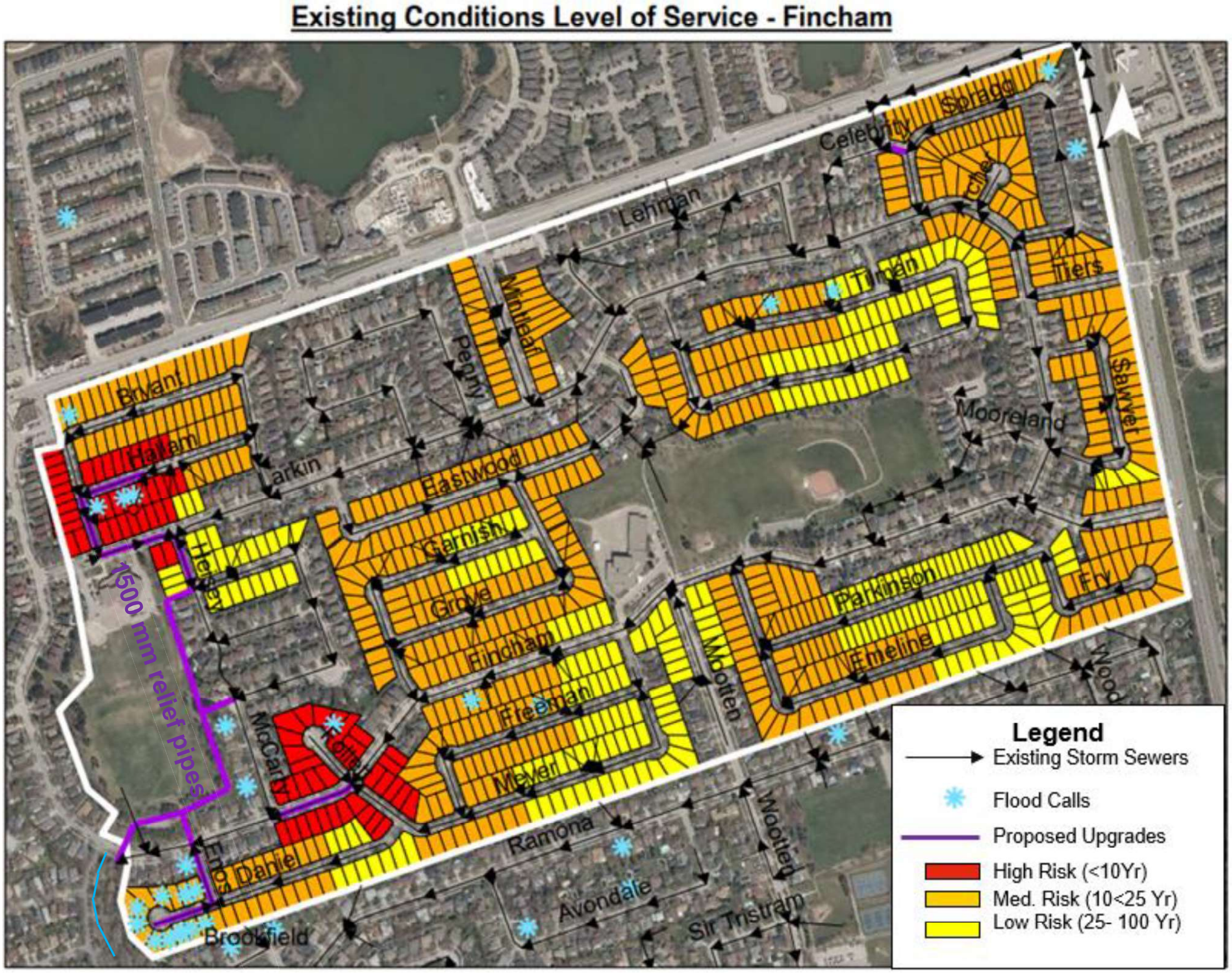


Figure 11: Proposed Solution from the Previous Study (RVA, 2021)

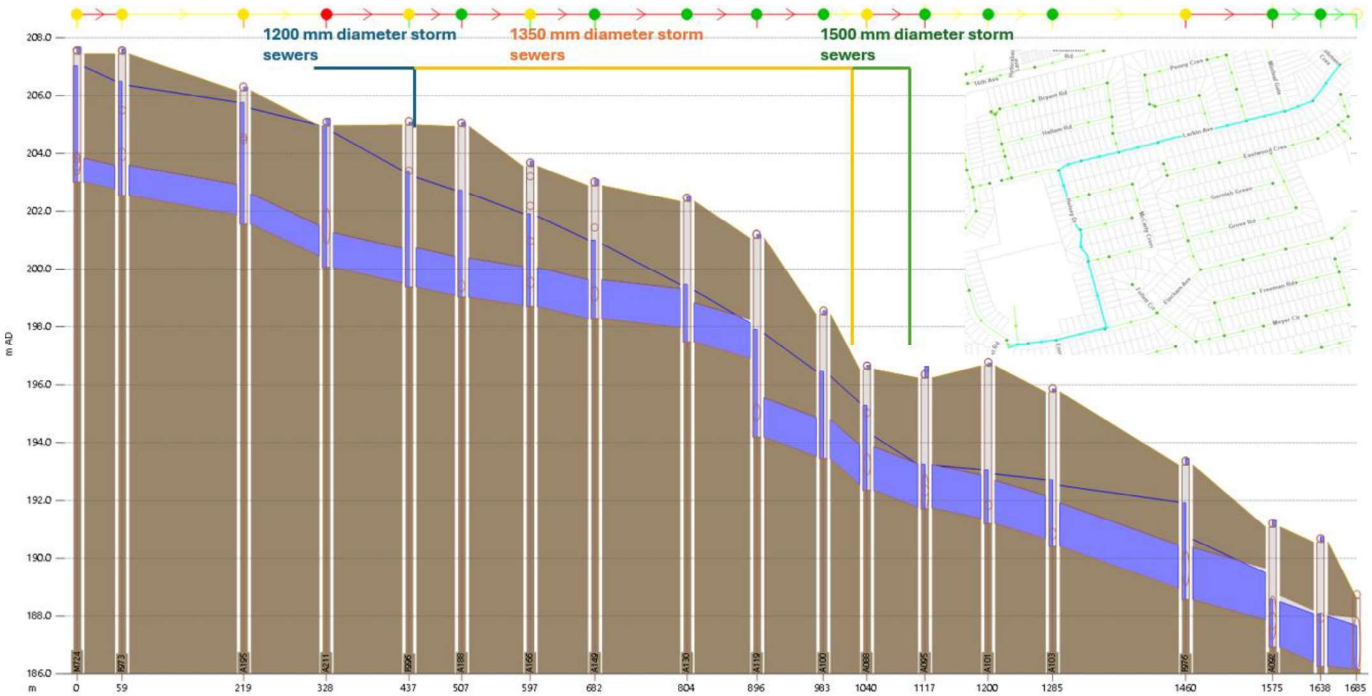


Figure 12: HGL Profile - Original Solution with Additional Flow from Swan Lake

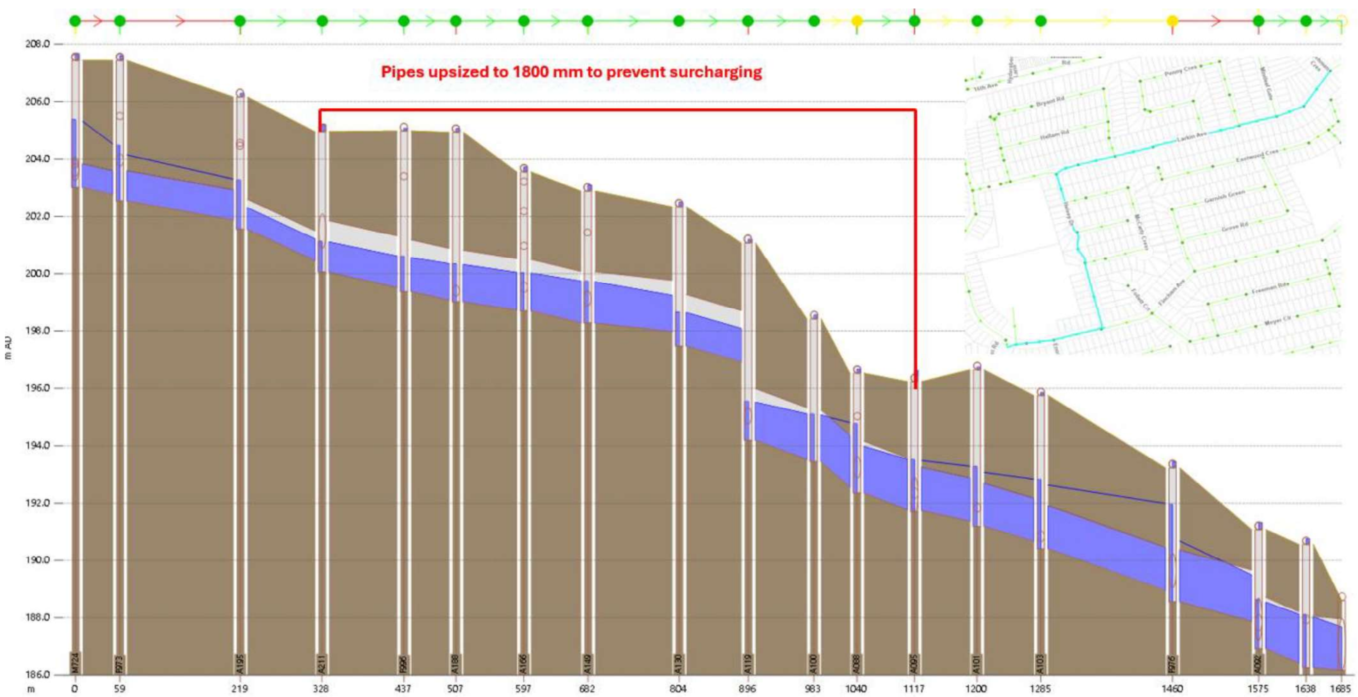


Figure 13: HGL Profile with Additional System Upgrades on Larkin Avenue

3.6 Rainfall and Flow Monitoring Data

Rainfall data from November 1, 2022, to November 4, 2023, was provided by the City for two rain gauges:

- MA-12 located in the open space at the southeast end of Monkhouse Rd, approximately 1.2 km west of Swan Lake.
- MA-20 located at Black Walnut Public School, approximately 2.5 km southeast Swan Lake.

A review of the rain gauge data was conducted to assess the quality and suitability of the recorded significant storm events for flow data analysis and model calibration. As shown in **Table 8**, twenty events with accumulated rainfall depths exceeding 10 mm were identified during the morning period.

Table 8: Summary of Rainfall Data

Event	Total Volume (mm)		Peak 5-min Intensity (mm/hr)	
	MA12	MA20	MA12	MA20
2022-11-30	24.0	26.6	9.6	14.4
2023-03-25	20.8	21.6	7.2	7.2
2023-04-01	19.6	19.4	12.0	12.0
2023-04-05	12.7	12.5	33.6	26.4
2023-04-17	12.8	12.0	9.6	9.6
2023-04-22	17.4	15.6	12.0	12.0
2023-04-29	20.8	17.6	7.2	9.6
2023-05-02	12.8	8.6	12.0	4.8
2023-05-20	33.8	34.6	26.4	24.0
2023-06-12	64.0	0.0	19.0	0.0
2023-06-24	12.6	15.0	7.2	9.6
2323-06-26	35.6	53.6	67.2	127.2
2023-07-01	12.8	13.0	55.2	86.4
2023-07-13	28.8	24.0	31.2	28.8
2023-07-16	9.6	9.6	9.6	12.0
2023-07-24	49.6	72.0	74.4	84.0
2023-08-25	11.0	19.4	31.2	50.4
2023-09-06	30.6	22.4	136.8	55.2
2023-09-12	11.2	12.6	12.0	14.0
2023-10-06	27.8	28.2	50.4	43.2

Flow monitoring in the study area was conducted by the City from November 2022 to November 2023. The locations of flow monitors in the study area are shown in Figure 14, and additional details are provided in Table 9.

Of the six flow gauges provided in the study area, the gauges at Manhole S304 and Manhole M718 (west leg) were selected for storm flow analysis, while Gauge J689 was chosen for analyzing FDC flow. Gauges in the mixed (storm and FDC) sewers were not used for analysis due to the potential uncertainty in FDC flow, which could compromise the accuracy of the storm flow analysis and vice versa.

Table 9: Flow Monitor Summary

MH ID	Street Location	System Type	Comment	Sewershed Area (ha)
J689	Swan Lake Road & Williamson Road	FDC	FDC flow from 63 buildings	12
M718 (North Leg)	39 Kingfisher Cover	Mixed	FDC flow from all 212 buildings in the gated community + Swan Lake outflow	26.4
M718 (West Leg)	39 Kingfisher Cover	Storm	Storm flow from ICI area along 16th Avenue	12
F973	38 Lehman Crescent	Mixed	Mixed	24
S304	18th Swan Park Road	Storm	North pond inlet	84
Y30	East to the water park	Storm	North pond outflow	43.2

Figure 14: Locations of Flow and Rainfall Monitors

3.7 Model Calibration

The following criteria have been used to select the events utilized for model calibration:

- No evidence of large spatial variability of rainfall when comparing the intensities at two gauges;
- Flow monitor data shows a clear response to the rainfall events;
- No impact of snow melt; and
- Largest event in the year (July 24th, 2023) to be analyzed as requested, although there is considerable difference between records of gauge MA12 and MA20 in this event. MA20 is more reasonable for this event, as discussed in the progress meeting with the City on April 4th, 2024.

By applying these criteria, six events were selected for model calibration, as shown in **Table 10**. The recent storm occurred on June 20th, 2024, will be used to validate the results, as suggested by the City.

Table 10: Selected Events for Calibration

Event	Total Volume (mm)		Peak 5-min Rainfall Intensity (mm/hr)	
	MA12	MA20	MA12	MA20
2023-04-01	19.6	19.4	12.0	12.0
2023-04-05	12.7	12.5	33.6	26.4
2023-04-22	17.4	15.6	12.0	12.0
2023-05-20	33.8	34.6	26.4	24.0
2023-07-24	49.6	72.0	74.4	84.0
2023-10-06	27.8	28.2	50.4	43.2

Model calibration is achieved by changing model parameters to produce results matching the flow monitoring data within a reasonable accuracy in the selected events. Model validation tests the calibrated model performance using measurements different than the calibration period to ensure the repeatability of the model results.

Model calibration procedure involves the following actions:

- Adjusting the percentage of connection (fixed runoff coefficient) of other impervious surfaces to match the total monitored flow volume.
- Reclassifying a portion of roof areas as pervious surfaces to account for volume losses due to roof leaders draining onto pervious areas. Adjust this portion until the modeled runoff volume matches the flow monitoring data. Modifying the "initial abstraction" value to align with the shape of flow monitoring hydrographs.

Following the initial model set up, simulated runoff volumes were generally too large compared to the monitored flows. The contribution of impervious areas was large relative to the volumetric runoff coefficient. Since impervious surface areas were calculated using aerial imagery within ArcGIS, there may be cases wherein a pervious surface was mistakenly considered as impervious, or the runoff generated from some impervious areas are not able to be conveyed to the storm management system. The adjusted impervious runoff surface parameters are listed in **Table 11**.

Table 11: Calibrated Imperviousness Rate

Gauge ID	Residential Imperviousness Rate	ICI Area Imperviousness Rate	Calibrated Impervious Rate
S304	74%	N/A	56%
M718 West Leg	72%	96%	83%

Table 12 compares the calibrated peak flow and flow volume with the monitored records. Detailed hydrograph comparisons are provided in **Appendix A**. In most events, errors between observed and simulated flow volume are within 20%. Discrepancies and relatively high percentage differences in peak flow and flow volume in some events may be attributed to the following:

- Due to the variability of rainfall within the study area the rainfall volumes and pattern could be different for some areas as compared to the distribution used in the model, which could have an impact on the simulation.
- The size of the rainfall events which have been used for model calibration raises some doubt about the accuracy of the level measurements. Accuracy of the level meter usually decreases appreciably at flow depths below 25 mm.
- Unexpected field conditions, such as blocked catch basins, leaking pipes, and broken manholes, result in reductions in the observed flow.

Table 12: Comparison between Observation and Simulation

Event	Gauge S304						Gauge M724 (West Leg)					
	Event Flow Volume Accumulation (m^3)			Event Peak Flow (L/s)			Event Flow Volume Accumulation (m^3)			Event Peak Flow (L/s)		
	Simu-lated	Observed	Differ-ence	Simu-lated	Observed	Differ-ence	Simu-lated	Observed	Differ-ence	Simu-lated	Observed	Differ-ence
2023-04-01	1171	1198	-2%	125	157	-26%	533	718	-35%	49	64	-31%
2023-04-05	783	730	7%	191	191	0%	350	360	-3%	65	61	6%
2023-04-22	935	770	18%	77	79	-3%	424	426	-0.1%	31	32	-3%
2023-05-20	1380	1200	13%	224	154	31%	630	630	0%	80	57	29%
2023-07-24	1880	1502	20%	595	626	-5%	851	795	7%	202	238	-18%
2023-10-06	1129	1045	7%	292	412	-41%	518	430	17%	106	93	12%

Hydrograph comparisons for validation event are shown in **Figure 15** and **Figure 16**. Note that in 2024 flow monitors S304 and M718 (west leg) were relocated to MH 50606 and the east leg, respectively, as shown in **Figure 17**. MH50606 is upstream of the original S304 monitor location, and M718's east leg receives additional uncalibrated flows from lake areas and surrounding properties, creating some observation-simulation discrepancies. The validation results show that at manhole MH50606, simulated flow matches well with the observed data.

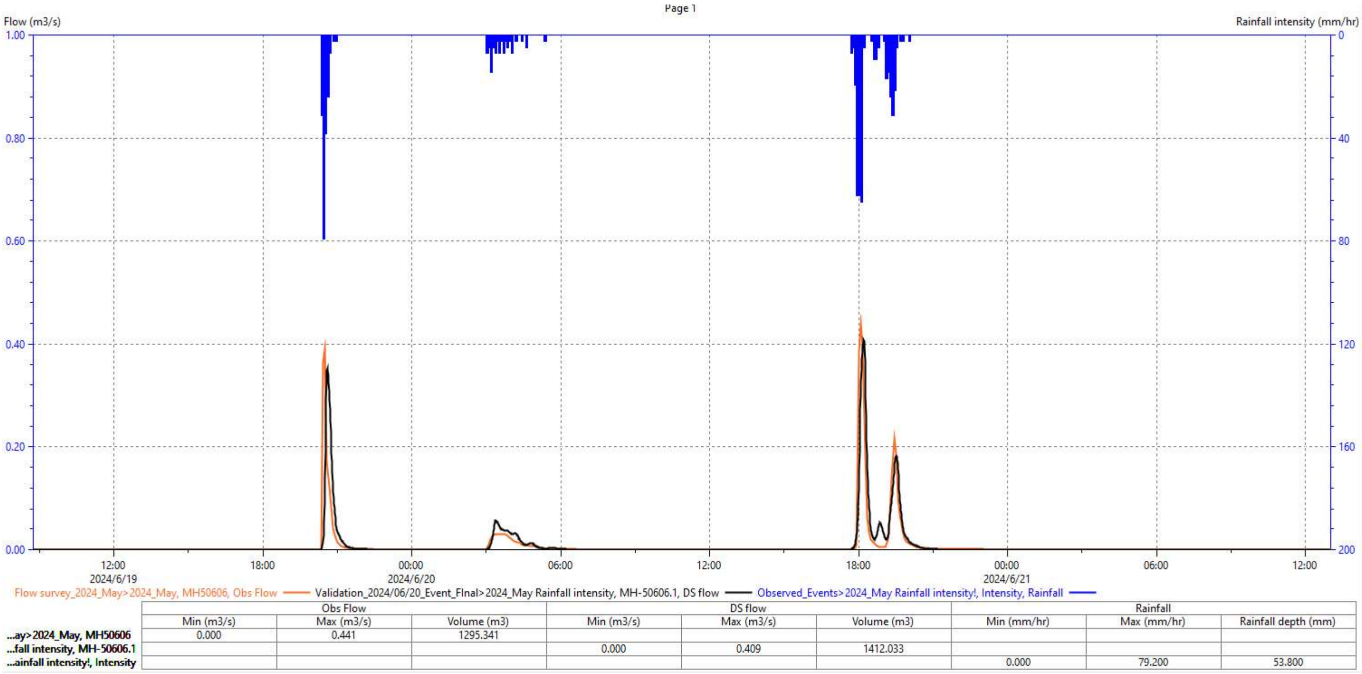


Figure 15: Model Validation - Gauge MH50606

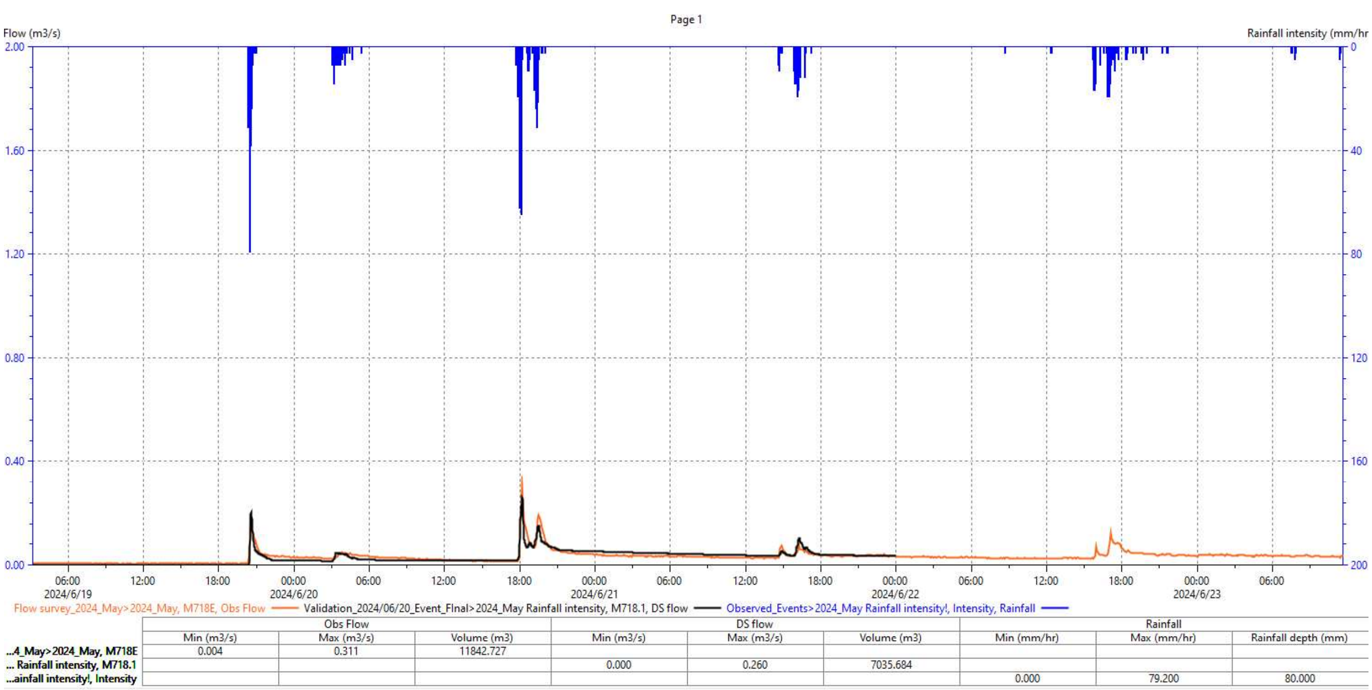


Figure 16: Model Validation - Gauge M718 East Leg

Figure 17: Location of 2024 Flow Gauges

3.8 System Evaluation

The 3-hour Chicago Design storm, obtained from City of Markham Engineering Design Criteria (Markham, 2014), were used for simulations conducted for 2-, 5-, 25-, and 100-year design storms to evaluate the storm drainage system's performance.

An existing conditions stormwater management and drainage assessment for all stormwater infrastructure within the study area was completed utilizing the calibrated hydrologic modelling to assess existing storm drainage system capacity deficiencies The calibrated model, including minor system (catch basin inlet, manholes, storm sewers, ditch/ swales) and major system (roadways, overland flow), were simulated for the 2- to 100-year design storm events.

The results of this assessment provided an indication of the stormwater management infrastructure with capacity and flooding issues (including surcharging and flooding/overland flow conditions), identified existing levels of service for the storm drainage system and provided an indication of existing spare capacity for future development and whether the existing storm system HGL is below basement levels. Areas with storm system capacity deficiencies such as surcharged nodes, and pipes under capacity were identified from simulation results. System capacity deficiency locations for the 5- and 100-year design storm events are shown in **Figure 18** and **Figure 19**.

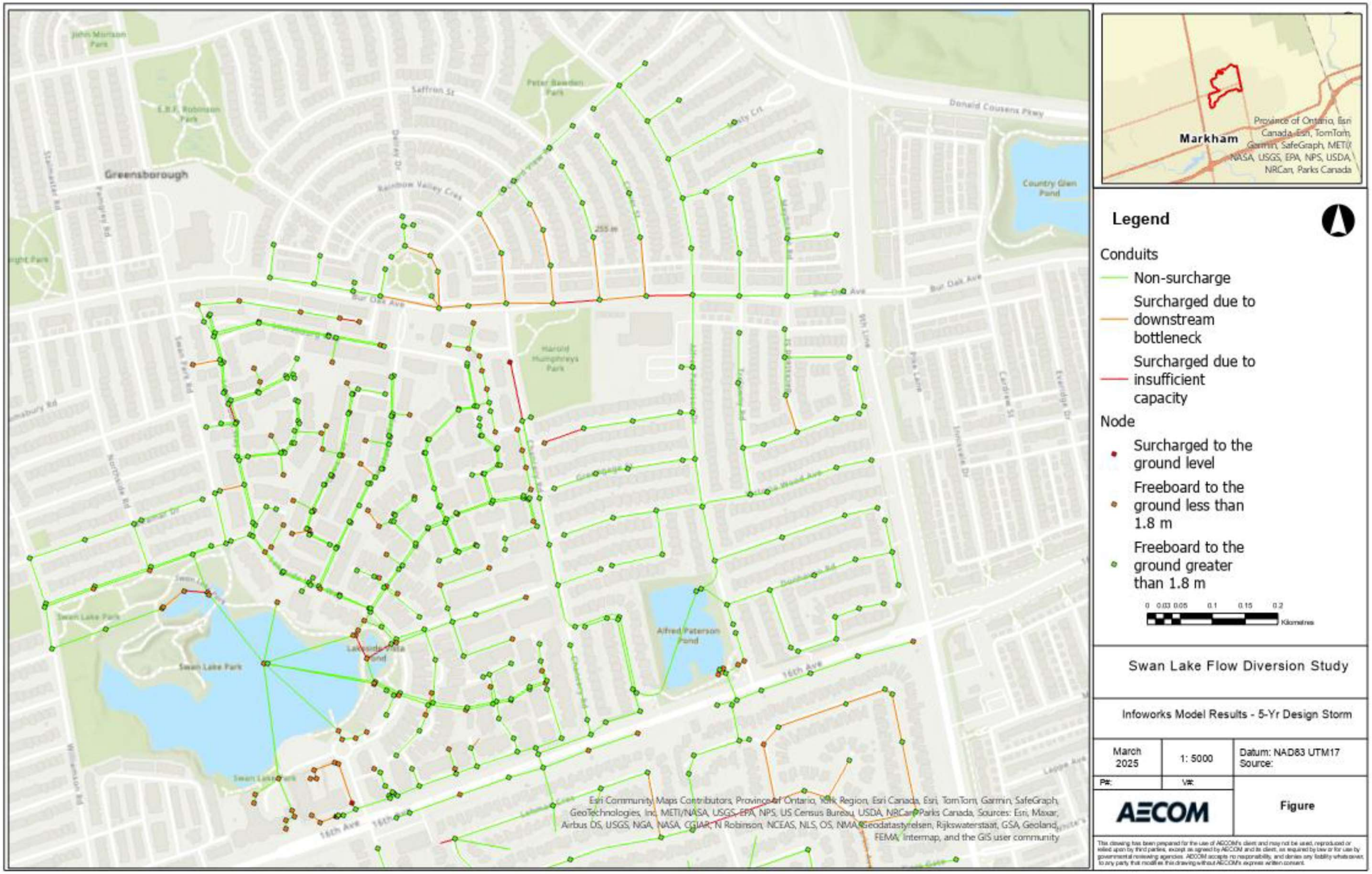


Figure 18: System Performance - 5-Year Design Storm

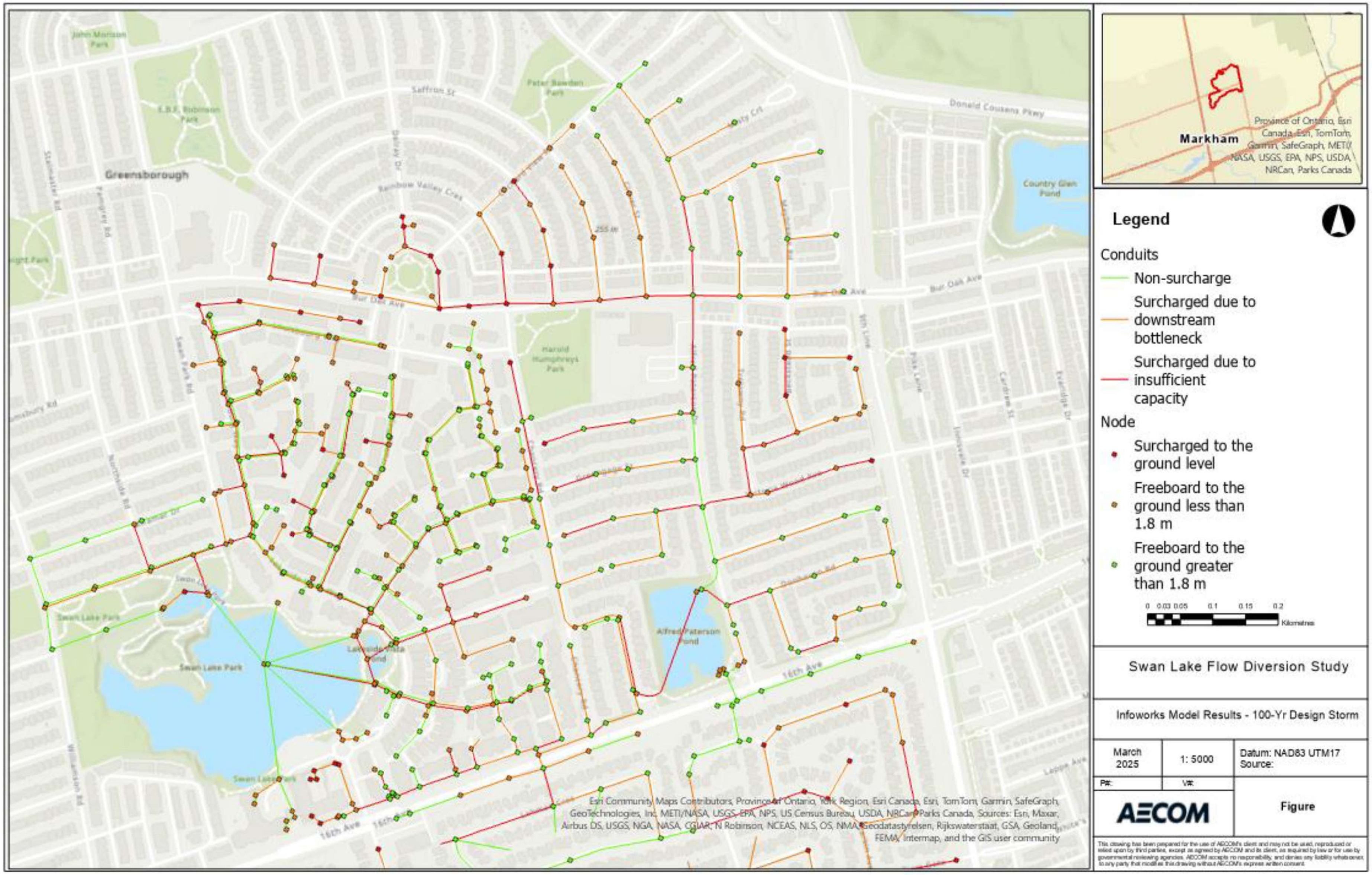


Figure 19: System Performance – 100-Year Design Storm

4. Flow Diversion Scenarios

4.1 General

This section presents the hydraulic analysis results for each diversion scenario, summarizing the reduction efficiency, cost, downstream impacts, and required infrastructure to implement the diversions as illustrated in **Figure 20**. The required new infrastructure and upgrades to the existing system were sized using the validated InfoWorks ICM model results.

City proposed flow diversion scenarios are listed as follows:

- Existing conditions (combine the existing Swan Lake catchment Infoworks model with the downstream area Markham Village and Unionville models).
- Redirecting minor system flow from the AMICA oil grit separator (OGS) and Swan Lake Blvd. OGS units to the 16th Ave. sewers.
- Redirecting minor system flow from AMICA OGS and Swan Lake Blvd. OGS units to the Lake outlet.
- Redirecting the “first flush” portion of the minor system flow from the AMICA OGS and Swan Lake Blvd. OGS units to the 16th Ave. sewer (i.e., redirect the most pollutant-laden runoff in a small diversion sewer).
- Redirecting minor system flow from Swan Club OGS to the North Pond.
- Adjusting the flow splitter weir for the East Pond and North Pond to reduce flow bypass to the Lake.
- Expanding the storage capacity in the East Pond and North Pond to reduce flow bypass to the Lake (to consider if the flow redirection scenarios increase flood risk).
- Creating underground storage capacity to attenuate the flows from AMICA OGS and Swan Lake Blvd. OGS before they enter the local sewer system (to consider if there is a feasible candidate site and if the redirecting scenarios increase flood risk).
- Redirecting/pumping flows from some foundation drain collectors (FDCs) toward Swan Lake (i.e., supply potentially cleaner, cool groundwater to the Lake).

Hydrologic information for the catchment areas for diversion is detailed as follows:

1. **Amica OGS catchment:** Amica OGS catchment is approximately 0.82 hectares in size with an imperviousness rate exceeding 90%. Total flat roof area in the catchment is approximately 0.32 ha and drains directly to the 300 mm local storm sewers. There are no sloped roofs in the Amica OGS catchment. It is assumed that runoff generated from the flat roof area will be attenuated by roof drain inlets, each with a capacity of 3 L/s at a depth of 5 cm, with one inlet per 160 m² of roof area. Overland flow is directed north to Swan Lake via Lakeside Vista Way, with no additional overland outlet to the downstream streets.
2. **Swan Lake Blvd. OGS Catchment:** Swan Lake Blvd. OGS Catchment is approximately 0.66 ha in size. This catchment contains two buildings with a combined roof area of 0.12 ha, with two out of six visible roof downspouts connected to the storm sewers. The overall imperviousness rate is approximately 75%. Overland flow travels westward and flow to Swan Lake through the double inlet catchbasin at the north end of Swan Lake Blvd.
3. **Swan Club OGS Catchment:** Swan Club OGS collects runoff from the parking lot west of the club building from a catchment area of 0.21 ha. The catch basin east of the building is connected to the 825 mm storm sewer on Lake Side Vista Way rather than the OGS. The building has one visible disconnected roof downspout on the northeast corner, which drains into the catch basin instead of the OGS. Thus, the Swan Club OGS only receives overland runoff from the parking lot through the herringbone opening.

4. **The North Pond catchment:** The North Pond catchment is approximately 12.6 hectares. This catchment consists of single-family residential areas with an imperviousness rate of 74%. A flow splitter weir located at the upstream pond inlet manholes (with a crest elevation of 208.8 m, standing 0.5 m high) diverts low flows to the North Pond and bypasses high flows to Swan Lake.
5. **East Pond Catchment:** The East Pond catchment is approximately 19.3 ha. The East Pond includes two inlets with flow splitter weirs at a crest elevation of 208.7 m (0.4 m high), designed to divert low flows to the North Pond and direct high flows to Swan Lake.

Figure 20: Catchment Area of Diversion Scenarios

4.2 Scenario Analysis

4.2.1 Scenario 1: Redirecting Minor System Flow from AMICA OGS and Swan Lake Blvd. OGS to Sewers on 16th Avenue

Diverting flow from the Amica and Swan Lake Blvd. OGS units requires installing new 450-600 mm pipe, totaling 220 m. In the ‘typical’ hydrologic year (2013 dataset, as provided by the City), the total inflow to Swan Lake was approximately 18,100 m³ under existing conditions. Note that 18,100 m³ represents the inflow to Swan Lake via the minor system, which does not include the rainfall volume directly received by the Lake and the flow that drains to the North and East pond. The hydraulic model results suggest that this diversion is expected to reduce the typical year flows to Swan Lake by 8,310 m³, with 5,780 m³ coming from the Amica OGS and 2,530 m³ from the Swan Lake Blvd. OGS, based on 2013 rainfall data. The diversion will increase peak flows to the downstream system by 420 L/s during a 100-year design storm event.

New storm sewers will be connected to existing sewer on Kingfisher at the intersection of Swan Lake Blvd. and 16th Ave. Existing sewers on Kingfisher Cove Way / 16th Ave range from 450 mm to 750 mm and provide a capacity of approximately 150 L/s to 920 L/s from upstream to downstream. Under existing conditions, the capacities of these sewers are exceeded during a 100-year design storm event, and the additional flow from the diversion will cause these pipes to surcharge to ground level, as shown in **Figure 21**. To mitigate flood risk, all pipes on Kingfisher Cove will need to be upgraded to sizes 750 to 1200 mm for a length of 645 m as shown in **Figure 22**. 100-Year HGL levels in the system with proposed upgrades are shown in **Figure 23**.

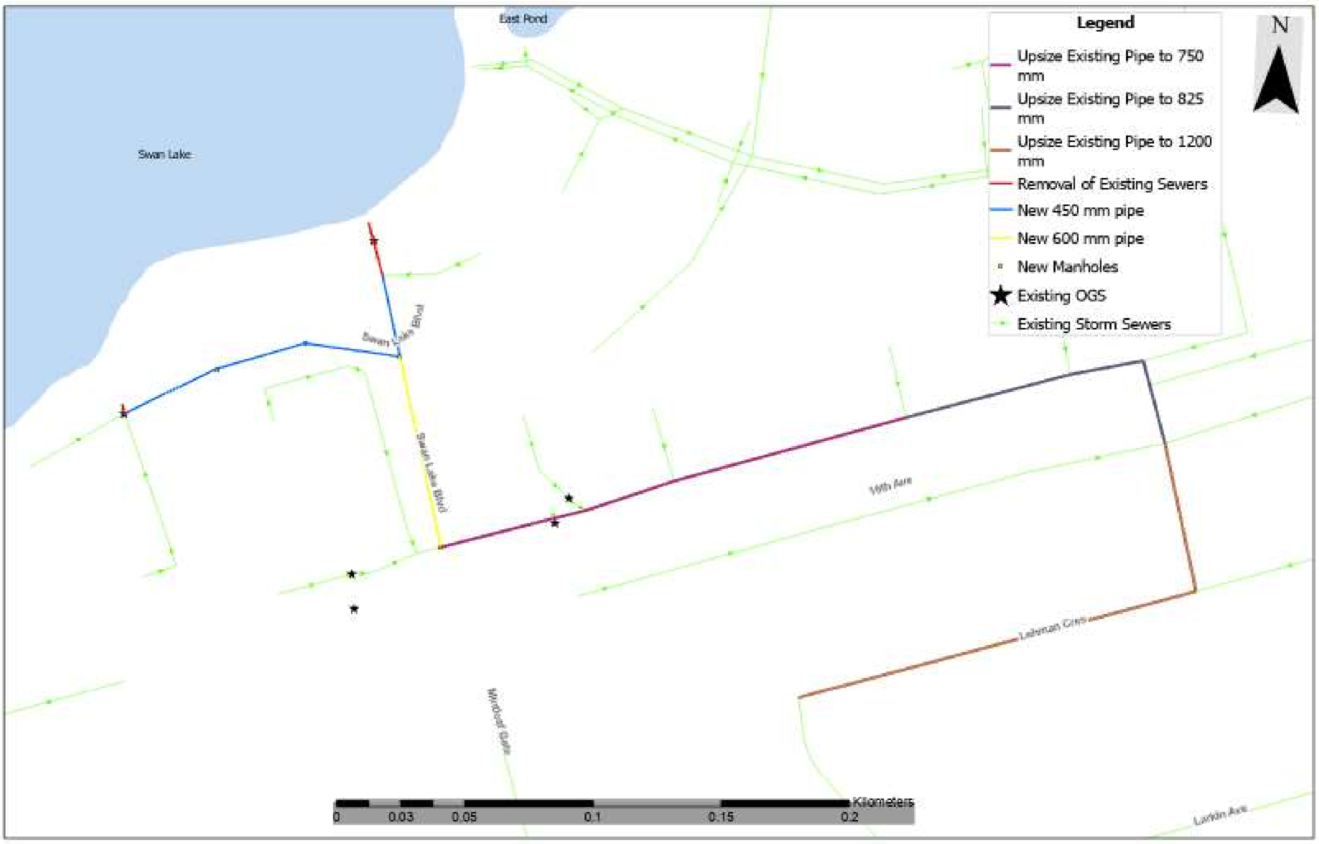


Figure 21: System Upgrade Requirement to Achieve Scenario 1

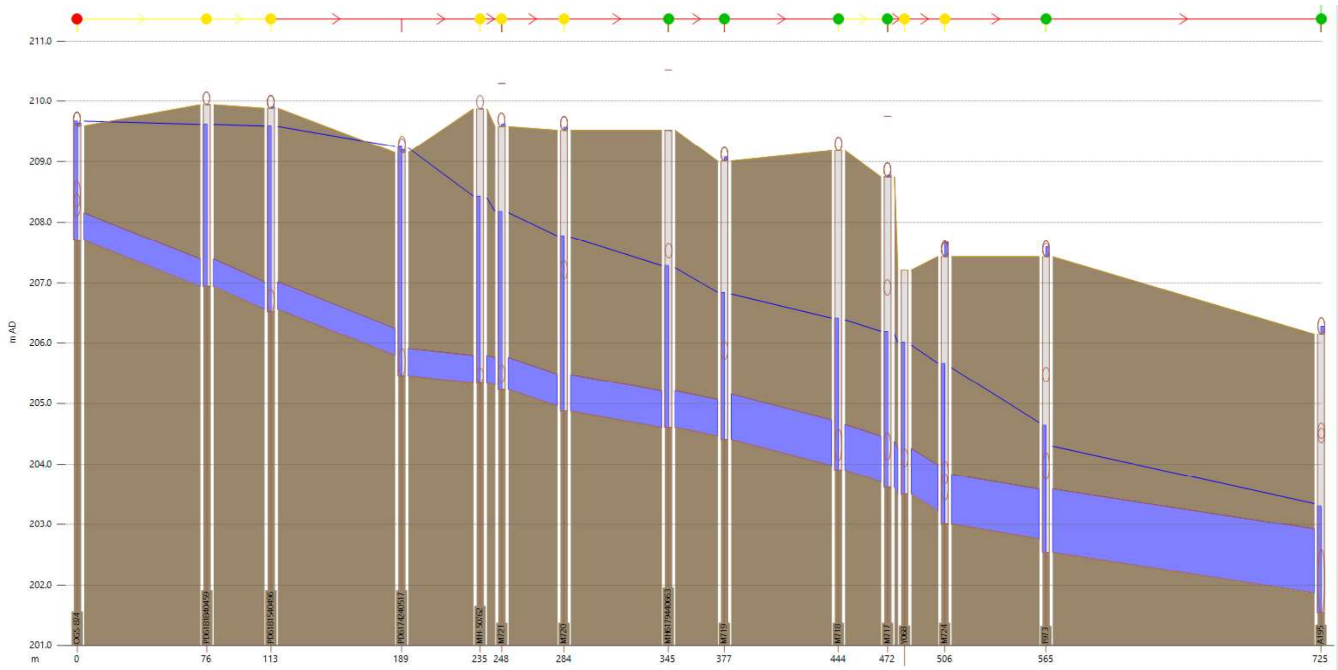


Figure 22: HGL Profile - Scenario 1 Existing Condition – 100-Year Design Storm

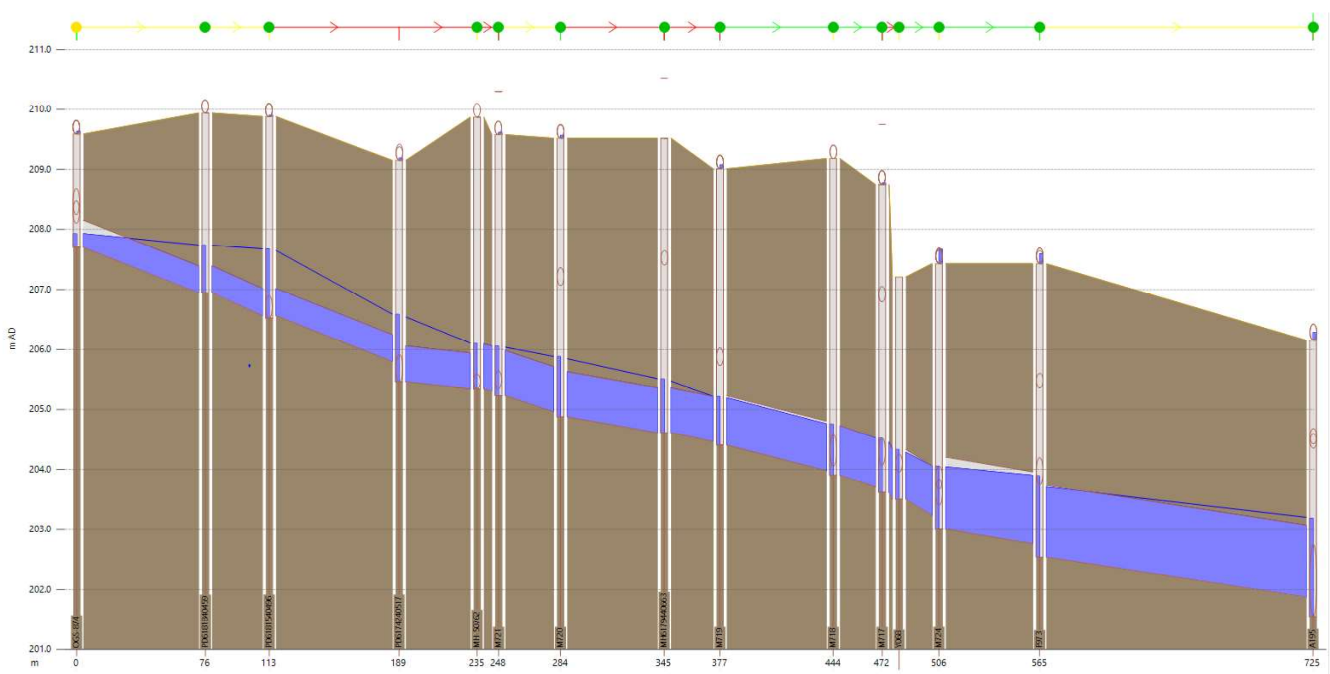


Figure 23: HGL Profile - Scenario 1 with System Upgrades - 100-Year Design Storm

4.2.2 Scenario 2: Redirecting the "First Flush" Portion of Minor System Flow from AMICA OGS and Swan Lake Blvd. OGS to 16th Avenue Sewer (i.e., redirect the most pollutant-laden runoff in a small diversion sewer)

The "first flush" refers to the initial runoff from the first 25 mm of rainfall during a storm event, which often contains higher pollutant concentrations, especially in urban areas with significant impervious surfaces. This runoff can carry significant pollutants into surface waters, including chloride from road salts in winter.

The concept of redirecting the first flush is to divert low flows to downstream sewers while allowing high flows to discharge into Swan Lake. According to the City's GIS database, the current outfall elevations of the Amica and Swan Club OGS units to Swan Lake are 208.2 m and 208.3 m, respectively. Swan Lake's normal water level is 208.3 m, and a 100-year design storm raises the Lake level to 208.65 m, indicating that lake water will back up into the upstream sewers during large storm event. To bypass low flows effectively and prevent backup from Swan Lake entering the upstream sewers, the flow split weir must be set at a minimum elevation of 208.6 m. Additionally, flow control should be provided at the inlet to the downstream sewers, allowing high flows to enter Swan Lake by surcharging above 208.6 m. The schematic of this configuration is shown in **Figure 24**.

As shown in the model results, setting the weir crest elevation at 208.7 m and installing 150 mm and 200 mm orifice plates at Amica and Lakeside Vista Blvd OGSs' downstream pipes will allow all runoff generated by a 25 mm, 4 hour Chicago rainfall event with a peak 5-minute intensity of 62 mm/hr to bypass Swan Lake. Any flow exceeding the peak flow from this event will be diverted to Swan Lake. This setup reduces the typical year flow to Swan Laake by 8305 m^3, achieving approximately 99% of the reduction effect of a complete disconnection of the OGS units with Swan Lake (i.e., Scenario 1 provided a reduction of 8,310 m^3). This high reduction could be attributed to the absence of extreme rainfall events in the suggested typical year (2013), as the peak intensity of a 25-mm, 4-hour Chicago rainfall event - 62 mm/hr - exceeds most events in this year. Winter storms are generally smaller, and the reduction in winter runoff, which has high chloride content, is identical to that in Scenario 1; overflow through the weir only occurred during intensive summer events, as shown in **Figure 25**.

New storm sewers required for this scenario are required, and pipes are sized to 300 mm, as per the minimum required storm sewer in the City of Markham Engineering Design Criteria. The HGL in the proposed system is shown in **Figure 26**.

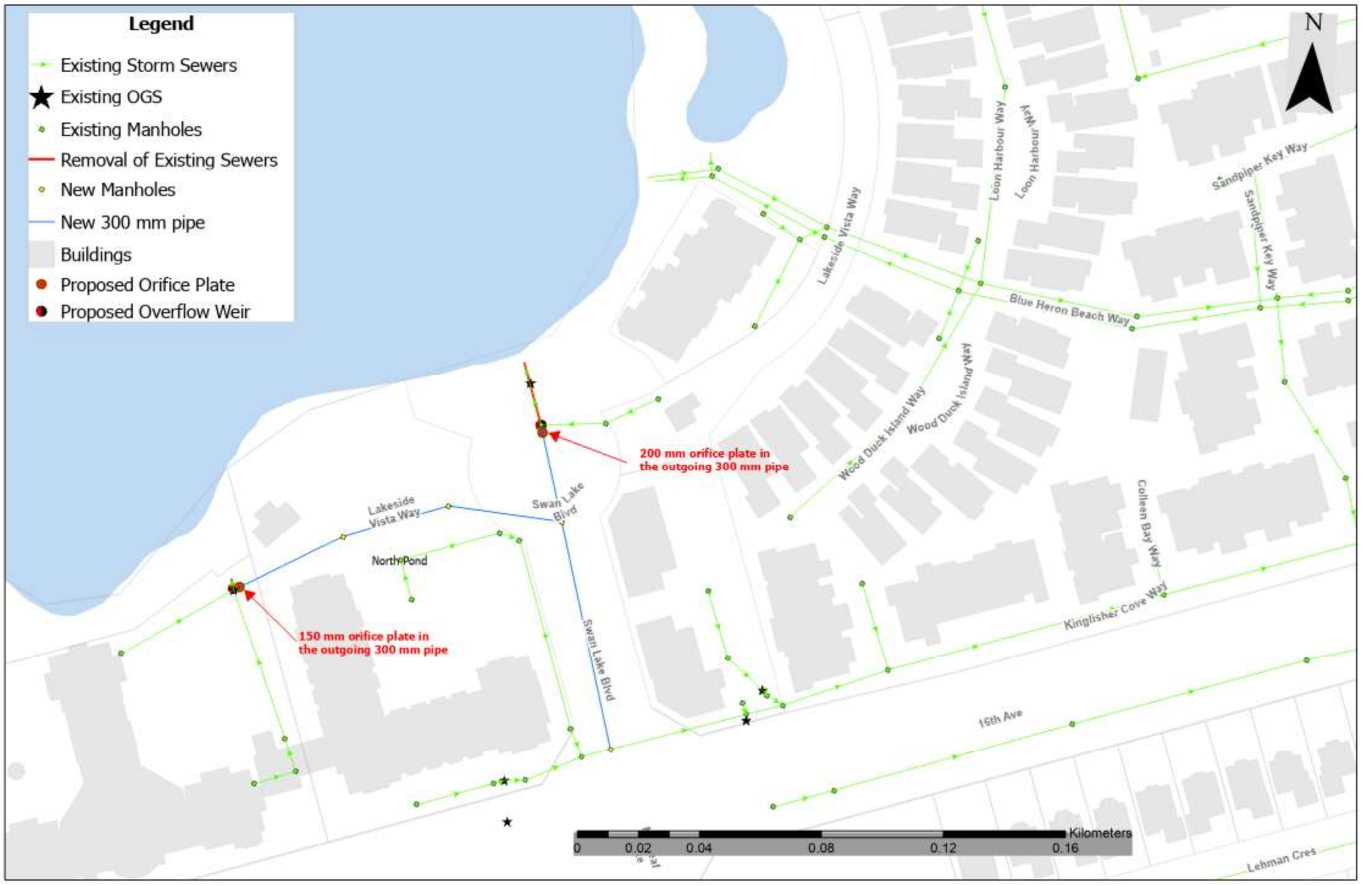


Figure 24: Schema - Diverting the First Flush

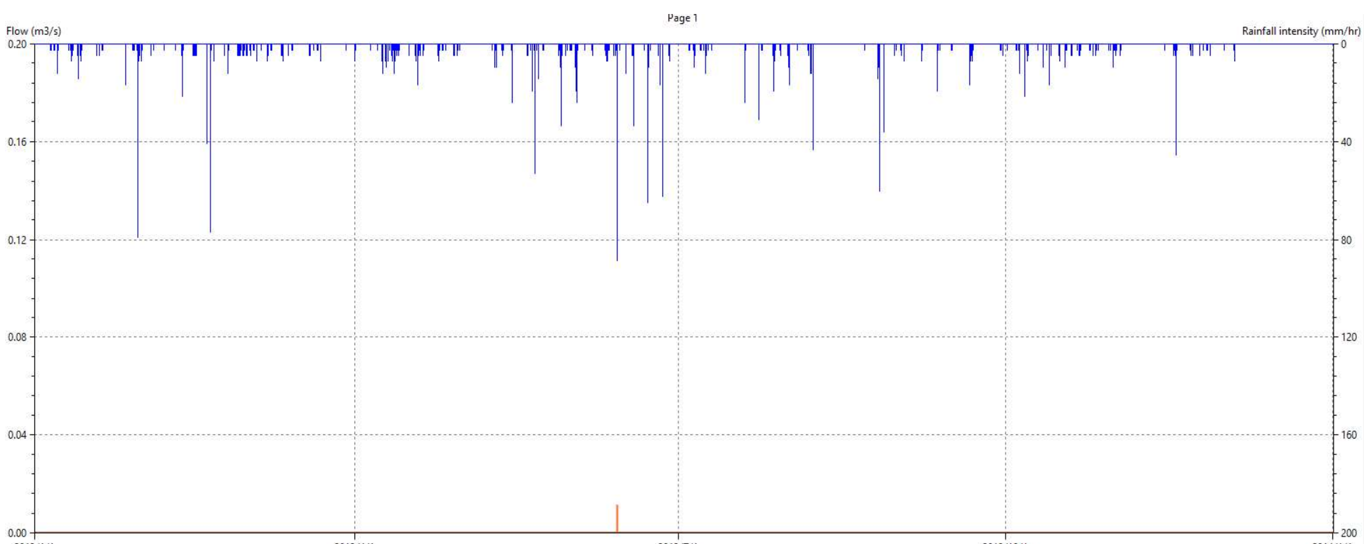


Figure 25: Typical Year Inflow to Swan Lake after Implementing Scenario 2

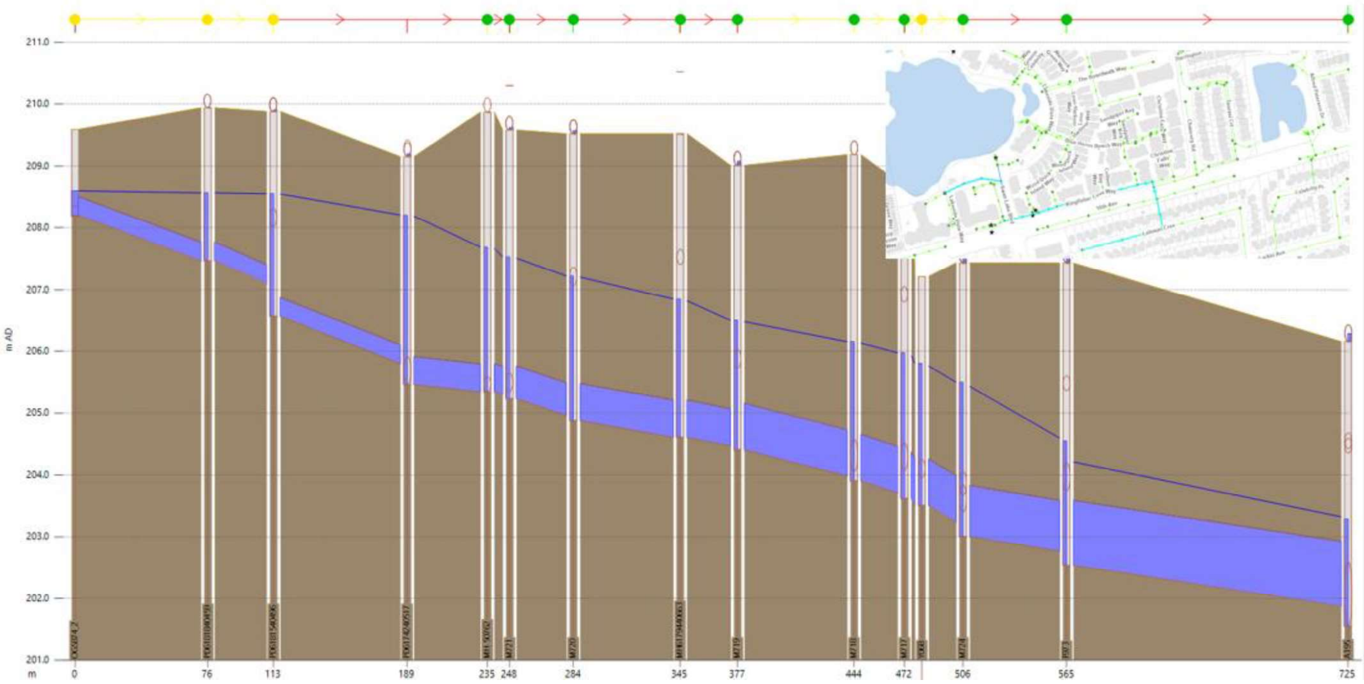


Figure 26: Scenario 2- Downstream HGL Profile

4.2.3 Scenario 3: Redirecting Minor System Flow from AMICA OGS and Swan Lake Blvd. OGS to the Lake Outlet

Identical to Scenario 1, Scenario 3 can achieve a typical year flow reduction of 8310 m^3. Scenario 3 involves the installation of 200 metres of 525 mm pipes, and 4 new manholes, as shown in **Figure 27**. New pipes will be connected to the existing 450 mm sewer on Lakeside Vista way at manhole MH-50688, which in turn connect to a 525 mm lake outlet sewer on Blue Heron Beach way. This route offers the advantage of bypassing the undersized sewers on Kingfisher Cove Way. Additionally, the existing sewers on Lakeside Vista Way are buried approximately 4-5 metres below the ground, providing adequate clearance above the assumed basement level of 1.8 metres below ground.

Additional flow from the diversion will increase the HGL level in the downstream sewer however, it would still be below assumed basement level, which is 1.8 m below the ground level, in a 100-year design storm event. Therefore, compared to Scenario 1, Scenario 3 would require fewer downstream improvements, the total required improvements to downstream pipes includes upsizing 95 m of 375 mm pipe to 525 mm, as illustrated in **Figure 28**.

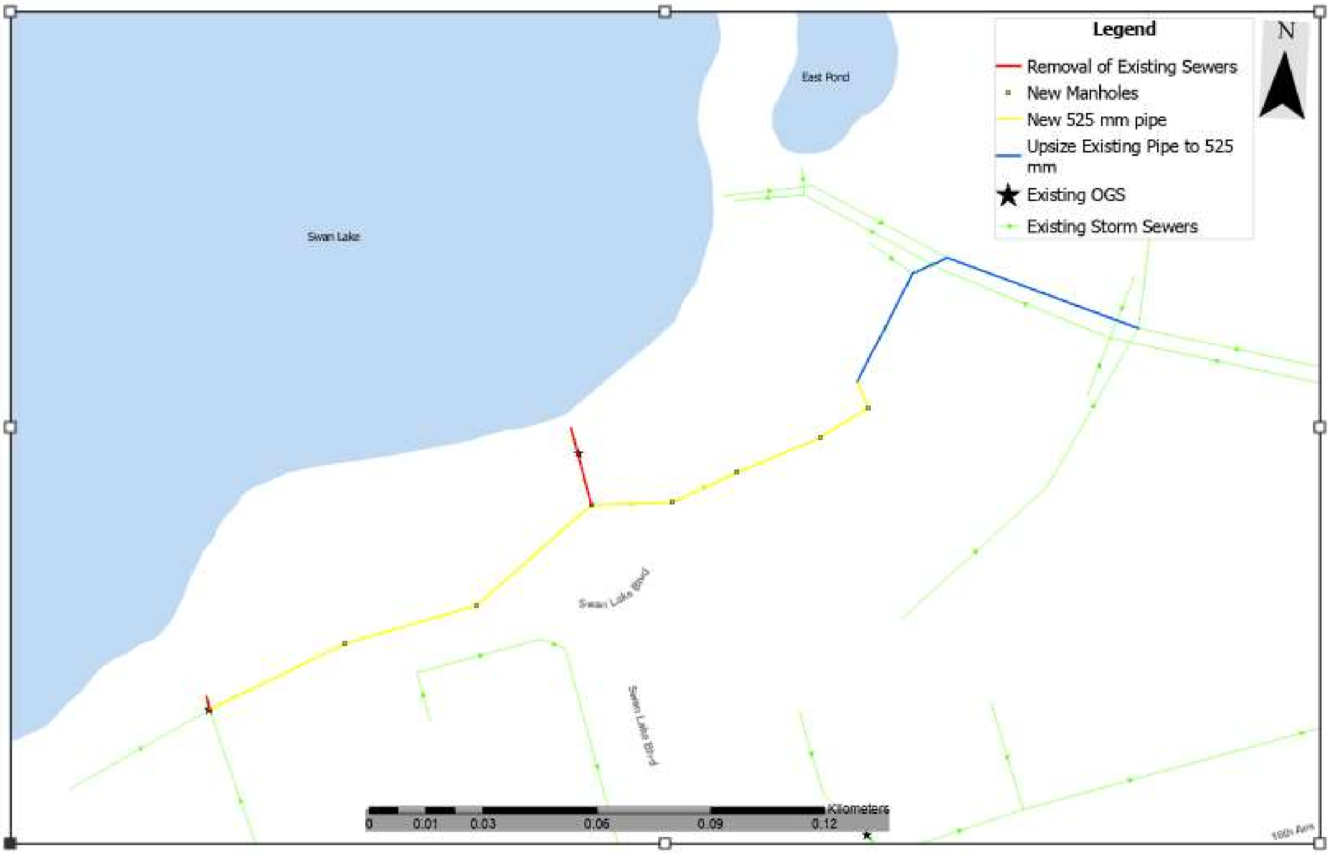


Figure 27: System Upgrade Requirement to Accommodate Scenario 3

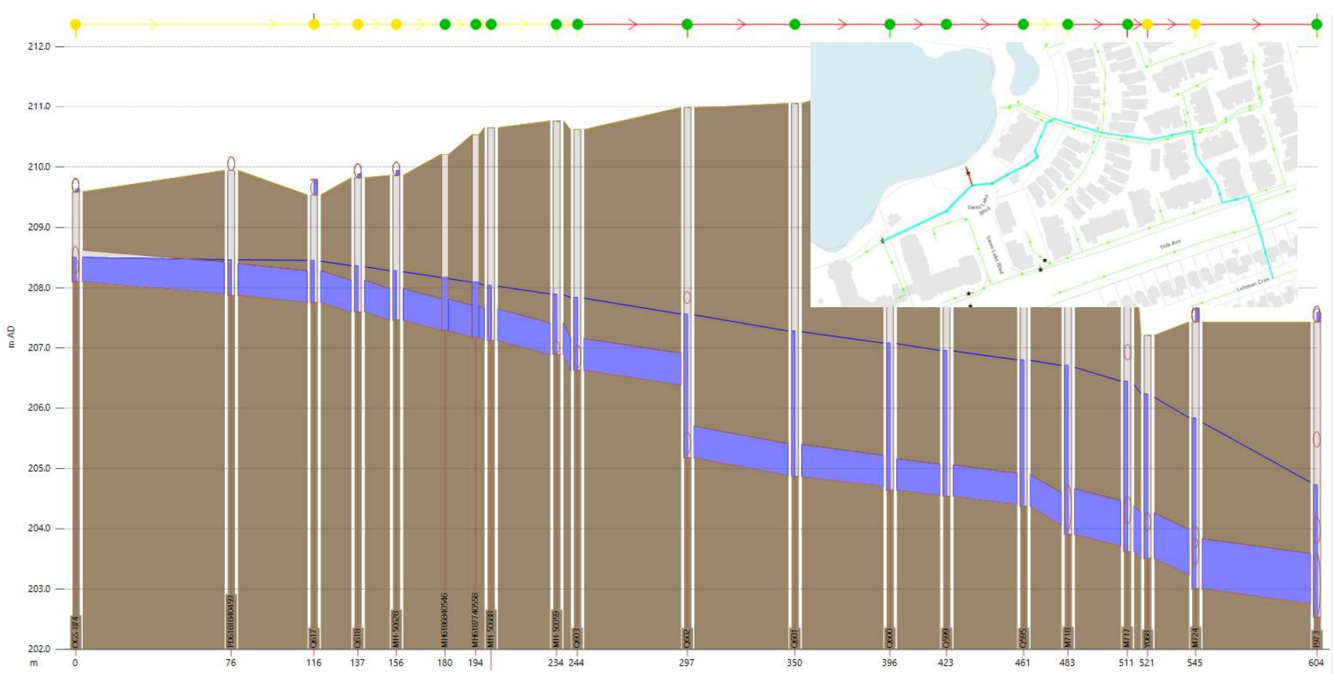


Figure 28: HGL- Scenario 3 - 100-Year Design Storm

4.2.4 Scenario 4: Redirecting Minor System Flow from Swan Club OGS to the North Pond

The new infrastructure required for this diversion includes one new manhole and 110 m of 300 mm pipe, as shown in **Figure 29**. The active storage in the north pond is approximately 810 m^3, at 208.8m, which is the level of the flow splitter weir at the pond inlet. Under existing conditions, 25 mm rainfall will generate approximately 960 m^3 of flow, additional flow will cause more spill from the weir. The 450 mm inlet pipe of the North Pond has a capacity of 200 L/s. Any flow exceeding this capacity will surcharge the pipe, diverting high flows above 208.8 m to Swan Lake.

Under existing conditions, a 25 mm rainfall event will cause the surcharge level to reach 208.9 m. Additional flow from the Swan Club OGS will cause the pond capacity to be exceeded during a 25mm event, and increase the weir overflow frequency, resulting in an increase of stormwater flows to Swan Lake from the weir.

This diversion is expected achieve a typical year inflow reduction of 1230 m^3, however, additional diverted flow will increase the overflow through the flow control weir at the pond inlet by approximately 240 m^3 per year, limiting the net typical year flow volume reduction to 990 m^3.

Figure 29: System Upgrade Requirement to Achieve Scenario 4

4.2.5 Scenario 5: Adjusting the Flow Splitter Weir for the East Pond and North Pond to Reduce Flow Bypass to the Lake

The existing configuration of the North and East Ponds are shown in **Figure 30**. Under existing condition, the flow splitter weirs at the inlets direct low flows to the North and East Ponds, while high flows are diverted to Swan Lake. Both weirs back flows up to the 450 mm inlet pipes to each pond, which are both approximately 200 L/s (under free outfall condition before the ponds are filled).

- **East Pond:** The East Pond has an active storage volume of 1,100 m³, at the weir elevation of 208.75 m. When this capacity is reached, any additional inflow overflows the weir at the flow splitter location and discharges directly into Swan Lake. The 1,100 m³ active storage volume represents about 60% of the runoff from a 25 mm storm. The pond initially receives inflow at a peak rate of 200 L/s. Flows exceeding this rate bypass the pond and go directly to Swan Lake due to the 450 mm inlet pipe's capacity. Once the pond fills to its maximum volume of 1,100 m³ (reaching the weir elevation at the flow splitter), nearly all additional flow is directed to Swan Lake.
- **North Pond:** Similarly, the North Pond has an active storage volume of 800 m³ at 208.8 m. When this capacity is reached, excess inflow bypasses the pond, overflowing the weir at the flow split and discharging to Swan Lake. The 800 m³ volume represents approximately 80% of the runoff from a 25 mm storm. Inflows initially reach the pond at a peak rate of 200 L/s, with any flow exceeding this rate directed to Swan Lake due to the inlet pipe's capacity. Once the pond fills to 800 m³, essentially all additional flow is diverted to Swan Lake.

Under existing conditions, the primary function of these ponds is to divert initial stormwater volumes from Swan Lake at the start of each rainfall event. However, the ponds have minimal impact on peak flow control for larger storm events, as they fill quickly and provide no further attenuation once full.

The East Pond has a spill elevation of 209.25 m, at which point it spills to Swan Lake. The weir height at the flow splitter location is 208.7 m, limiting the East Pond level from rising higher than this. There is an opportunity to raise the weir by approximately 0.3 m, thereby increasing the maximum water level in the East Pond to 209.0 m while maintaining 0.25 m of freeboard before spilling into Swan Lake. This would increase the active storage in the pond to approximately 1,200 m^3 (existing active storage = 1100 m^3).

The north pond has a spill elevation of 209.0 m when it spills to Swan Lake, with the weir height at the flow splitter set at 208.8 m, therefore raising the north pond weir would not significantly increase storage within the North Pond.

By raising the north and east pond weirs to 208.9 m (0.1 m rise) 209 m (0.3 m rise), respectively, the additional storage in the pond would enable a reduction of approximately 5016 m^3 of flow to Swan Lake in a typical year.

However, without increasing the 450 mm inlet pipes to the ponds, the flow rate to the ponds would still be limited to about 200 L/s, leading to occasional bypass of flows to Swan Lake during short durations of intense rain.

An additional scenario was analyzed by upsizing the pond inlet pipes to 600 mm, resulting in a reduction of flow to Swan Lake by approximately 5499 m^3 in a typical year. The increase in flow to the downstream sewers caused by pond upgrades is negligible; raising the weirs would increase the allowable water depth in the north and east ponds by 0.1 m and 0.3 m, respectively. This would result in approximately 2.5 L/s and 1.9 L/s of additional flow through the 100 mm and 66 mm orifice plates at the north and east pond outlets, respectively. However, it remains uncertain whether the surcharge conditions in the downstream sewer of the north pond, along Williamson Road, would impose any restrictions on pond outflows. Such restrictions could potentially impact the effectiveness of the pond upgrade options. The capacity of the downstream sewer on Willamson Road is not analyzed in this study; further investigation is required to confirm the downstream condition.

Potential upstream catchment impacts as a result of the raised weir height are discussed as a part of the Scenario 6 section.

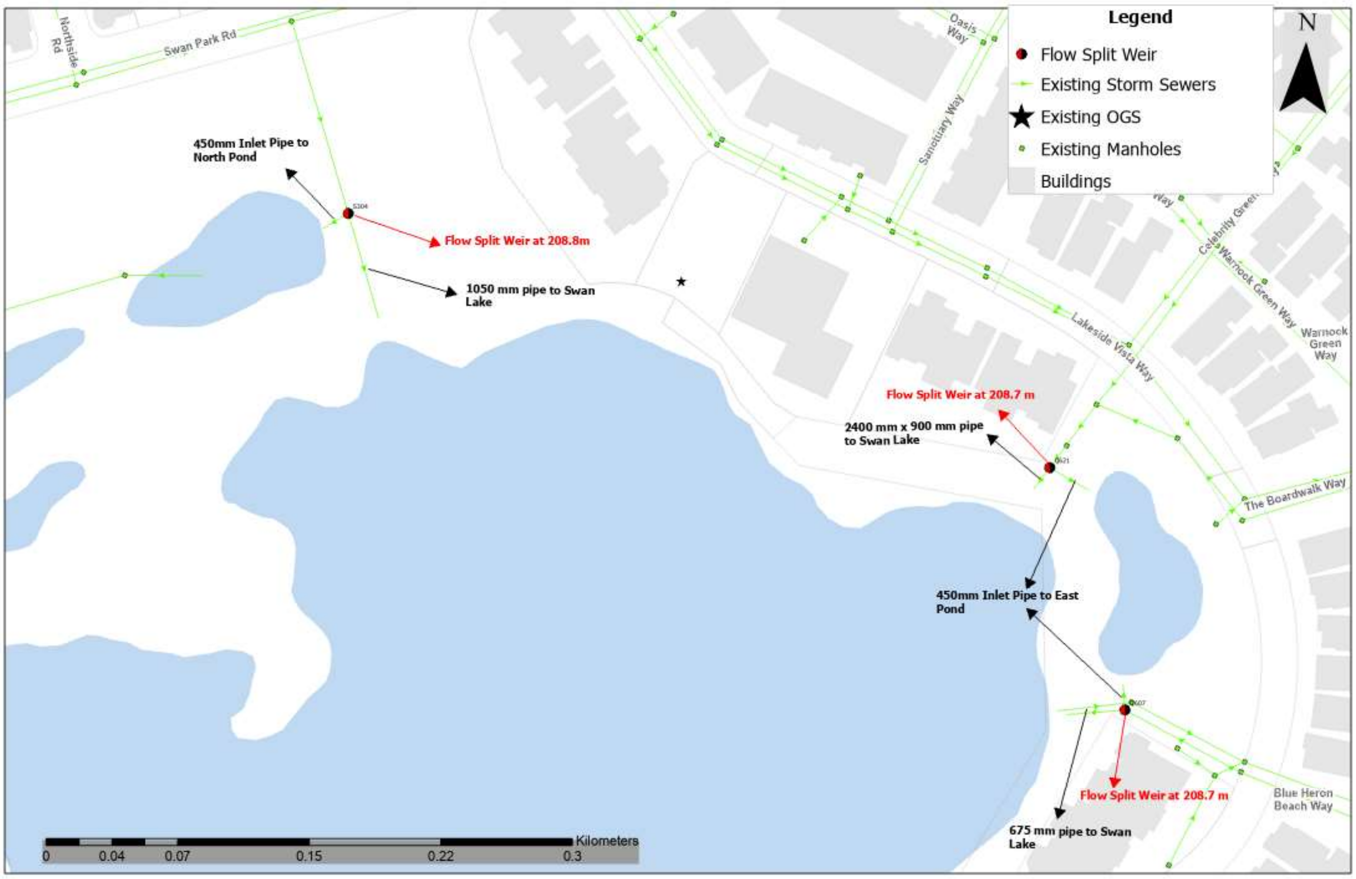


Figure 30: Existing Configurations of North and East Ponds

4.2.6 Scenario 6: Expand Storage Capacity in the East and North Ponds to Reduce Flow Bypass to the Lake (to consider if the redirection scenarios increase flood risk)

In this scenario, in addition to raising the weir and upsizing the inlet pipes, the North and East ponds were proposed to be retrofitted to provide an active storage of 3140 m^3 and 3126 m^3 respectively. The proposed pond layouts, and stage-storage relation are shown in **Figure 31** and **Table 13**, respectively. Enlargement of the north pond would require elimination of a small section of trail, but residual connections would generally offset any potential negative connectivity consequences. The enlarged facility would still be able to make beneficial use of the local park space for short-term sediment drying during construction and maintenance operations (for example, directly to the northeast of the north pond), although and major maintenance operations would likely necessitate trail restoration.

Enlargement of the east pond in the manner shown would consume the majority of the open space in the SWMF block and would require realignment of the existing trail on the on the south, east and northern sides. While a trail connection still appears to be feasible, the remaining SWM block area is generally understood not to provide sufficient space for sediment drying. This means that bulking would need to occur within the eastern SWMF, or that high-moisture content material may need to be transported offsite as liquid waste (in accordance with O.Reg.

406/19) as part of any future sediment removal operations completed under an expanded pond scenario. This added maintenance complexity can generally be expected to increase the cost of SWMF maintenance.

The proposed additional storage (include raising the weir and upsizing the inlet pipes) could reduce the typical year bypass flow to swan lake by 8,226 m^3. The comparison of typical year flow reduction at each inlet between existing condition, raising the weir, raising the weir while upsizing the inlet pipe, and raising the weir while upsizing the inlet pipe and upsizing the pond is summarized in **Table 14**. To analyze the cost efficiency of retrofitting each pond, **Table 15** provides a comparison of the reduction efficiencies between the North Pond and East Pond upgrades.

Table 13: S6 Pond Expansion - Stage-Storage Relation

Stage (m)	Area (m^2)	Average Area (m^2)	Volume (m^3)	Storage (m^3)
Proposed North Pond				
208.3	2916	0	0	0
208.4	3036	2976	298	298
208.6	3280	3158	632	929
208.8	3530	3405	681	1610
209	3787	3659	732	2342
209.2	4051	3919	784	3126
East Pond				
208.2	2569	0	0	0
208.4	2790	2680	536	536
208.6	3018	2904	581	1117
208.8	3252	3135	627	1744
209	3492	3372	674	2418
209.2	3738	3615	723	3141

Table 14: Comparison of Flow Reduction at Inlets

Scenarios	Typical Year Inflow to Swan Lake at Each Inlet (m^3)				Typical Year Reduction Compared to Existing Condition (m^3)	Typical Year Reduction as % of Total Swan Lake Inflow
	North Pond Inlet	East Pond North Inlet	East Pond South Inlet	Total		
Existing	2583	4310	1360	8253	n/a	n/a
Raise Weir (Scenario 5a)	1152	1395	690	3237	5016	27.7%
Raise the weir, upsize the inlet pipe (Scenario 5b)	902	1102	750	2754	5499	30.4%
Raise the weir, upsize the inlet pipe, expand the pond (Scenario 6)	10	17	0	27	8226	45.4%

Table 15: Comparison of Flow Reduction between Upgrading North and East Pond

Scenarios	Reduction at North Pond (m^3)	Reduction at East Pond (m^3)
Existing	0	0
Raise Weir (Scenario 5a)	1431	3585
Raise the weir, upsize the inlet pipe (Scenario 5b)	1681	3818
Raise the weir, upsize the inlet pipe, expand the pond (Scenario 6)	2573	5653

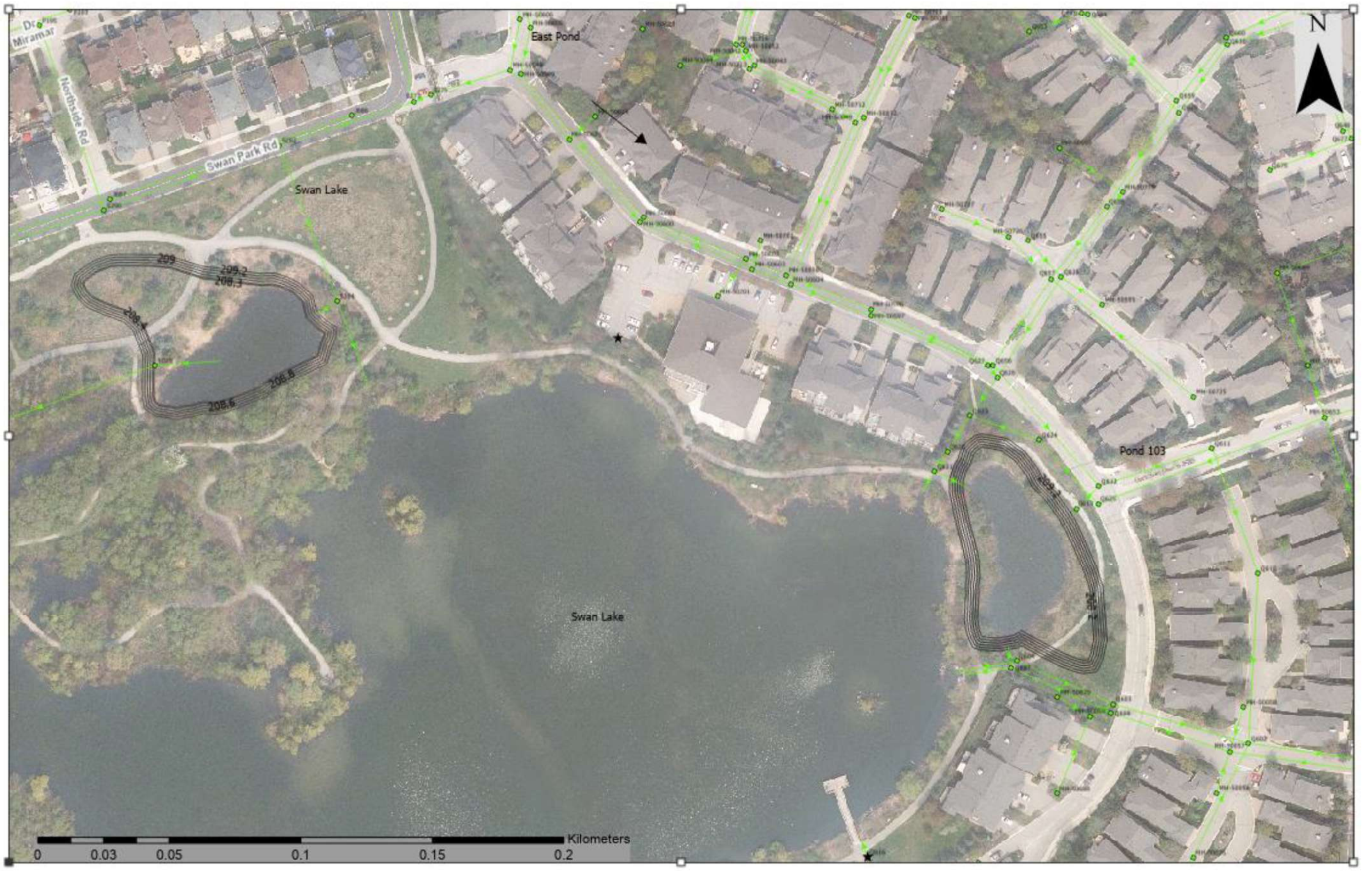
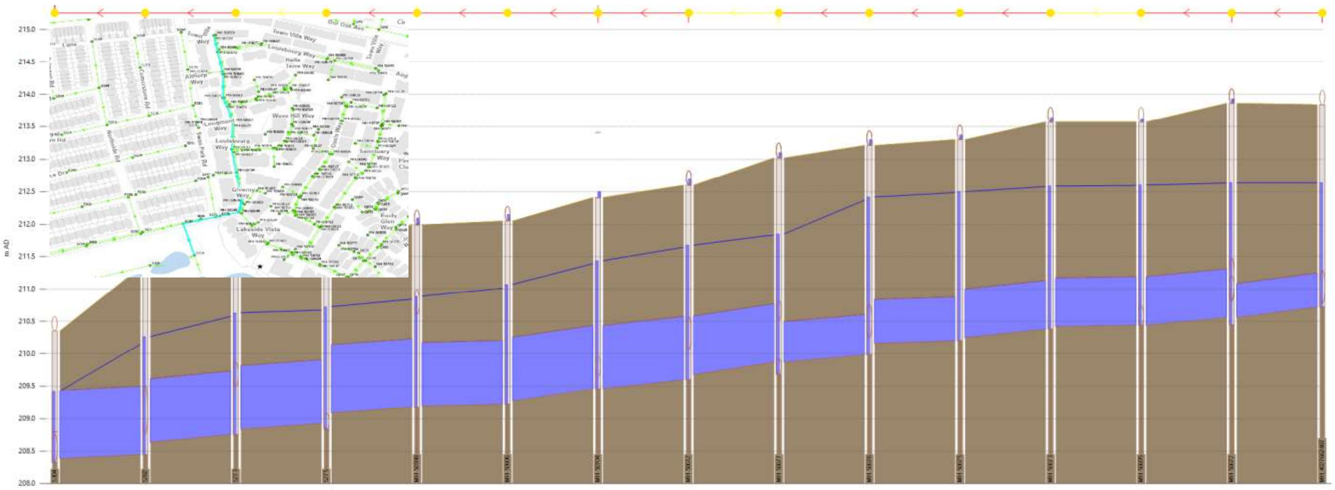


Figure 31: Proposed Pond Contour (Active Storage)

Upstream Catchment Hydraulic Grade line Impacts as a Result of Increased Pond Depths

Among all three pond retrofit scenarios (5a, 5b, and 6), solely raising the weir will result in the highest increase of water level during extreme storm events (e.g., the 100-year design storm), as additional flow are diverted to the ponds without corresponding increase in storage capacity. The increased flow will lead to a rise in the hydraulic grade line (HGL) in upstream sewers. As shown in **Figure 30** to **Figure 35**, the HGL in the sewers upstream of the north pond remained largely unchanged, with only a 0.02 m increase after the weir was raised. This is because the 0.1 m increase in the pond water level does not significantly increase the flow rate through the north pond inlet pipes. Additionally, the north pond has a lower spill level (209 m), and this level is exceeded under existing condition during a 100-year storm, which limits the volume of flow that can be diverted into it.

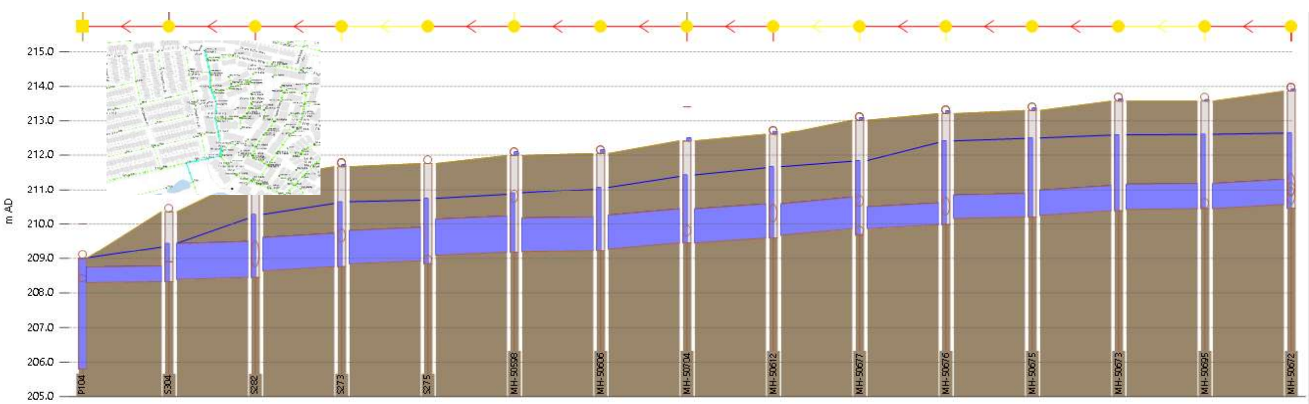
The HGL in the sewers upstream of the east pond's north and south inlets increased by 0.2 m and 0.15 m, respectively. While this increase will not cause additional manholes to surcharge to the ground surface, it will slightly elevate the basement flooding risk for properties connected to these pipes. This is because the pipes are relatively shallow and do not provide 1.8 m of freeboard to the ground surface. However, since foundation drain collector (FDC) pipes are presented in the catchment area of the east pond, it is possible that basements of these properties are connected to the FDC system rather than directly to the storm sewers. Connectivity tests are recommended to confirm basement connections.



Link	S282.1	S273.1	S275.1	MH-50598.1	MH-50606.1	MH-50704.E	MH-50612.E	MH-50677.1	MH-50676.1	MH-50675.1	MH-50673.1	MH-50695.E	MH-50672.E	MH_4027662487.M
US node ID	S282	S273	S275	MH-50598	MH-50606	MH-50704	MH-50612	MH-50677	MH-50676	MH-50675	MH-50673	MH-50695	MH-50672	MH_4027662487
ds node	S304	S282	S273	S275	MH-50598	MH-50606	MH-50704	MH-50612	MH-50677	MH-50676	MH-50675	MH-50673	MH-50695	MH-50672
length (m)	63.3	53.6	7.1	31.7	19.5	59.2	44.8	50.8	33.3	17.9	45.8	5.0	38.2	26.7
Shape ID	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC
width (mm)	1050	975	975	1050	975	975	975	900	625	675	750	750	750	525
height (mm)	1050	975	975	1050	975	975	975	900	625	675	750	750	750	525
us inv (m AD)	208.450	208.760	208.930	209.180	209.230	209.460	209.600	209.870	209.980	210.200	210.380	210.440	210.560	210.720
ds inv (m AD)	208.390	208.640	208.840	209.090	209.200	209.280	209.460	209.680	209.870	210.160	210.250	210.420	210.440	210.560
grad (m/m)	0.00095	0.00224	0.01276	0.00284	0.00154	0.00304	0.00313	0.00374	0.00331	0.00224	0.00284	0.00398	0.00314	0.00600
surc	2.00	2.00	1.00	2.00	2.00	2.00	2.00	1.00	2.00	2.00	2.00	1.00	2.00	2.00
r.pfc (m3/s)	0.841	1.061	2.532	1.456	0.879	1.236	1.253	1.107	0.394	0.398	0.593	0.702	0.624	0.333
US total head (m AD)	210.642	210.812	210.894	211.009	211.270	211.607	211.825	211.944	212.687	212.578	212.630	212.644	212.659	212.637
DS flow (m3/s)	2.84208	1.69707	1.64729	1.64689	1.77747	1.63285	1.50730	1.00274	0.87637	0.70016	0.68370	0.67707	0.65374	0.52039
DS velocity (m/s)	3.279	2.161	2.089	2.291	2.312	2.132	1.903	2.019	2.706	1.922	1.649	1.740	1.600	2.276

Node	S304	S282	S273	S275	MH-50598	MH-50606	MH-50704	MH-50612	MH-50677	MH-50676	MH-50675	MH-50673	MH-50695	MH-50672	-
System type	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm
Asset ID	S304	S282	S273	S275	MH-50598	MH-50606	MH-50704	MH-50612	MH-50677	MH-50676	MH-50675	MH-50673	MH-50695		MH_6
ground (m AD)	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
HD table	-	-	herringbone_On_Sag	herringbone_On_Sag	-	herringbone_On_Sag	herringbone_On_Sag	herringbone_On_Sag	herringbone_On_Sag	herringbone_On_Sag	-	herringbone_On_Sag	herringbone_On_Sag	-	-
Num gullies	0.000	2.000	2.000	1.000	0.000	3.000	1.000	5.000	2.000	5.000	1.000	1.000	3.000	1.000	1.000
expr:Freeboard	0.927454	1.069914	1.043871	1.039587	1.107722	0.983075	0.977963	0.926996	1.169948	0.792291	0.799359	0.982857	0.966622	1.223083	1.203083

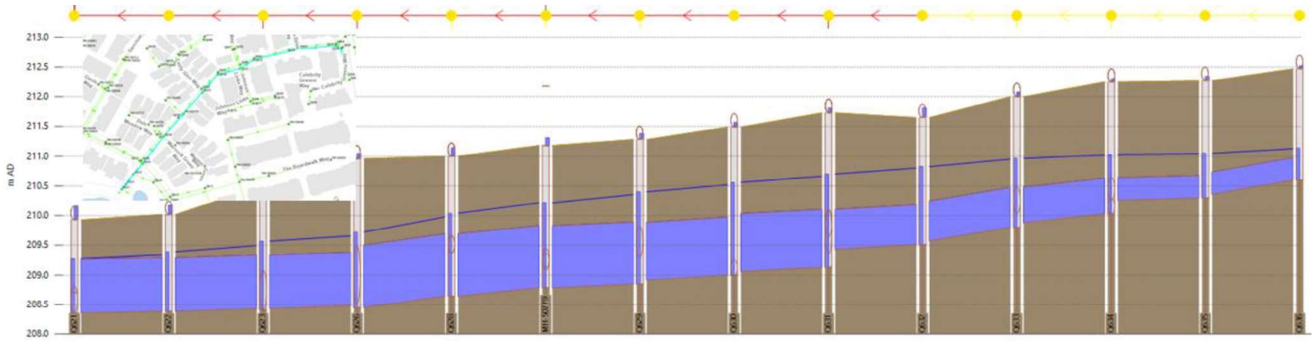
Figure 32: 100 Yr HGL Level – Pipes Upstream to North Pond – Base Scenario



Link	S304.2	S282.1	S273.1	S275.1	MH-50598.1	MH-50606.1	MH-50704.E	MH-50612.E	MH-50677.1	MH-50676.1	MH-50675.1	MH-50673.1	MH-50695.E	MH-50672.E
US node ID	S304	S282	S273	S275	MH-50598	MH-50606	MH-50704	MH-50612	MH-50677	MH-50676	MH-50675	MH-50673	MH-50695	MH-50672
ds node	P104	S304	S282	S273	S275	MH-50598	MH-50606	MH-50704	MH-50612	MH-50677	MH-50676	MH-50675	MH-50673	MH-50695
length (m)	9.5	63.3	53.6	7.1	31.7	19.5	59.2	44.8	50.8	33.3	17.9	45.8	5.0	38.2
Shape ID	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC
width (mm)	450	1050	975	975	1050	975	975	975	900	625	675	750	750	750
height (mm)	450	1050	975	975	1050	975	975	975	900	625	675	750	750	750
us inv (m AD)	208.330	208.450	208.760	208.930	209.180	209.230	209.460	209.600	209.870	209.980	210.200	210.380	210.440	210.560
ds inv (m AD)	208.300	208.390	208.640	208.840	209.090	209.200	209.280	209.460	209.680	209.870	210.160	210.250	210.420	210.440
grad (m/m)	0.00316	0.00095	0.00224	0.01276	0.00284	0.00154	0.00304	0.00313	0.00374	0.00331	0.00224	0.00284	0.00398	0.00314
surc	2.00	2.00	2.00	1.00	2.00	2.00	2.00	2.00	1.00	2.00	2.00	2.00	1.00	2.00
r.pfc (m3/s)	0.160	0.841	1.061	2.532	1.456	0.879	1.236	1.253	1.107	0.394	0.398	0.593	0.702	0.624
US total head (m AD)	210.053	210.651	210.822	210.904	211.018	211.274	211.605	211.820	211.938	212.678	212.573	212.627	212.641	212.659
DS flow (m3/s)	0.64608	2.83262	1.69360	1.64672	1.64614	1.76618	1.62273	1.49813	0.99698	0.87390	0.69782	0.68254	0.67610	0.65202
DS velocity (m/s)	3.993	3.239	2.156	2.088	2.260	2.293	2.111	1.891	2.012	2.699	1.918	1.643	1.734	1.609

Node	P104	S304	S282	S273	S275	MH-50598	MH-50606	MH-50704	MH-50612	MH-50677	MH-50676	MH-50675	MH-50673	MH-50695	-
System type	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm
Asset ID	-	S304	S282	S273	S275	MH-50598	MH-50606	MH-50704	MH-50612	MH-50677	MH-50676	MH-50675	MH-50673	MH-50695	-
ground (m AD)	-	210.350	211.330	211.670	211.760	211.990	212.050	212.410	212.600	213.020	213.210	213.300	213.570	213.570	-
HD table	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Num gullies	1.000	0.000	2.000	2.000	1.000	0.000	3.000	1.000	5.000	2.000	5.000	1.000	1.000	3.000	1.000
expr:Freeboard	-	0.908624	1.058470	1.033281	1.029455	1.097910	0.976926	0.977810	0.930109	1.175045	0.800622	0.805905	0.986656	0.969902	-

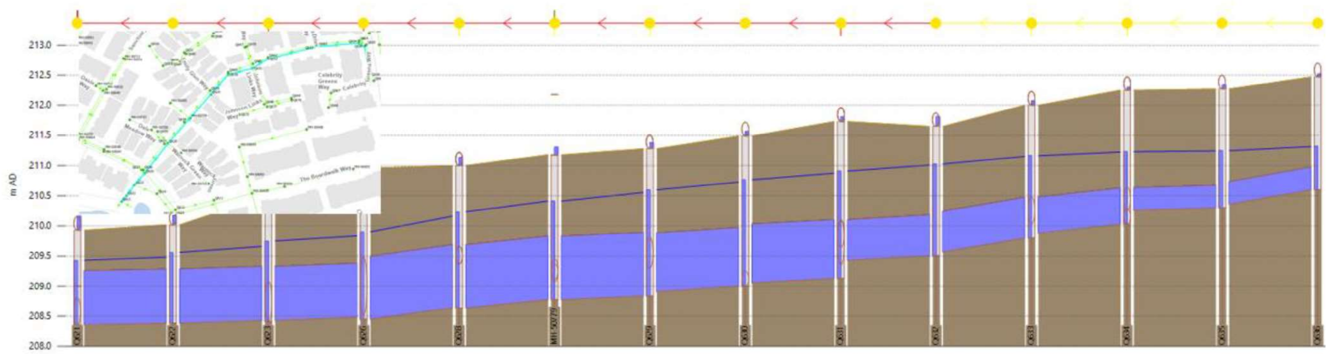
Figure 33: 100 Yr HGL level – Pipes upstream to North Pond – Scenario 5: Raising the Overflow Weir



Link	Q622.1	Q623.1	Q626.1	Q628.1	MH-50779.1	Q629.1	Q630.1	Q631.1	Q632.1	Q633.1	Q634.1	Q635.1	Q636.1
US node ID	Q622	Q623	Q626	Q628	MH-50779	Q629	Q630	Q631	Q632	Q633	Q634	Q635	Q636
ds node	Q621	Q622	Q623	Q626	Q628	MH-50779	Q629	Q630	Q631	Q632	Q633	Q634	Q635
length (m)	8.9	16.1	17.9	45.2	40.1	37.1	31.3	28.9	21.3	56.0	49.9	7.7	34.9
Shape ID	RECT	RECT	RECT	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC
width (mm)	1800	1800	1800	1050	1050	1050	975	975	675	675	600	375	375
height (mm)	900	900	900	1050	1050	1050	975	975	675	675	600	375	375
us inv (m AD)	208.385	208.430	208.480	208.640	208.780	208.840	209.000	209.130	209.510	209.800	210.030	210.300	210.600
ds inv (m AD)	208.360	208.385	208.430	208.450	208.640	208.780	208.910	209.060	209.430	209.570	209.880	210.260	210.360
grad (m/m)	0.00280	0.00280	0.00280	0.00420	0.00349	0.00162	0.00287	0.00243	0.00375	0.00410	0.00301	0.00519	0.00688
surc	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	1.00	1.00	1.00	1.00
r.pfc (m3/s)	2.954	2.954	2.954	1.770	1.613	1.098	1.201	1.104	0.515	0.539	0.337	0.126	0.145
US total head (m AD)	209.692	209.838	209.918	210.242	210.419	210.523	210.675	210.804	210.916	211.007	211.046	211.065	211.159
DS flow (m3/s)	4.37515	4.37603	3.83664	2.05689	1.80563	1.65082	1.37052	1.31109	0.57270	0.39709	0.23471	0.10022	0.10129
DS velocity (m/s)	2.708	2.607	2.277	2.284	2.047	1.897	1.825	1.974	1.988	1.494	1.253	1.387	1.408

Node	Q621	Q622	Q623	Q626	Q628	MH-50779	Q629	Q630	Q631	Q632	Q633	Q634	Q635	Q636
System type	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm
Asset ID	Q621	Q622	Q623	Q626	Q628	MH-50779	Q629	Q630	Q631	Q632	Q633	Q634	Q635	Q636
ground (m AD)	209.930	210.020	210.470	210.960	211.000	211.180	211.280	211.490	211.730	211.650	212.000	212.250	212.270	212.470
HD table	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Num gullies	0.000	0.000	0.000	3.000	2.000	0.000	3.000	1.000	2.000	2.000	2.000	1.000	0.000	3.000
expr:Freeboard	-	0.633495	0.907531	1.245416	0.964844	0.963600	0.882966	0.931895	1.032673	0.826163	1.036179	1.223450	1.223613	-

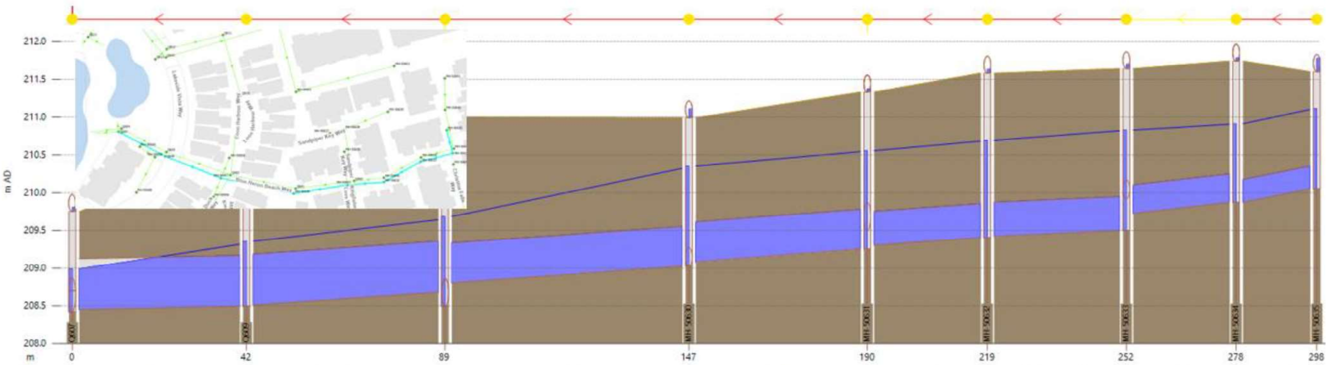
Figure 34: 100 Yr HGL Level – Pipes Upstream to East Pond North Inlet – Base Scenario



Link	Q622.1	Q623.1	Q626.1	Q628.1	MH-50779.1	Q629.1	Q630.1	Q631.1	Q632.1	Q633.1	Q634.1	Q635.1	Q636.1
US node ID	Q622	Q623	Q626	Q628	MH-50779	Q629	Q630	Q631	Q632	Q633	Q634	Q635	Q636
ds node	Q621	Q622	Q623	Q626	Q628	MH-50779	Q629	Q630	Q631	Q632	Q633	Q634	Q635
length (m)	8.9	16.1	17.9	45.2	40.1	37.1	31.3	28.9	21.3	56.0	49.9	7.7	34.9
Shape ID	RECT	RECT	RECT	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC
width (mm)	1800	1800	1800	1050	1050	1050	975	975	675	675	600	375	375
height (mm)	900	900	900	1050	1050	1050	975	975	675	675	600	375	375
us inv (m AD)	208.385	208.430	208.480	208.640	208.780	208.840	209.000	209.130	209.510	209.800	210.030	210.300	210.600
ds inv (m AD)	208.360	208.385	208.430	208.450	208.640	208.780	208.910	209.060	209.430	209.570	209.880	210.260	210.360
grad (m/m)	0.00280	0.00280	0.00280	0.00420	0.00349	0.00162	0.00287	0.00243	0.00375	0.00410	0.00301	0.00519	0.00688
surc	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	2.00	1.00	1.00	1.00	1.00
r.pfc (m3/s)	2.954	2.954	2.954	1.770	1.613	1.098	1.201	1.104	0.515	0.539	0.337	0.126	0.145
US total head (m AD)	209.819	209.994	210.089	210.423	210.613	210.726	210.889	211.025	211.124	211.209	211.252	211.266	211.340
DS flow (m3/s)	4.31790	4.31844	3.78087	2.05924	1.81122	1.67576	1.41644	1.35794	0.58043	0.40106	0.22354	0.09851	0.10027
DS velocity (m/s)	2.566	2.560	2.235	2.277	1.996	1.845	1.801	1.860	1.951	1.472	1.231	1.378	1.407

Node	Q621	Q622	Q623	Q626	Q628	MH-50779	Q629	Q630	Q631	Q632	Q633	Q634	Q635	Q636
System type	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm	storm
Asset ID	Q621	Q622	Q623	Q626	Q628	MH-50779	Q629	Q630	Q631	Q632	Q633	Q634	Q635	Q636
ground (m AD)	209.930	210.020	210.470	210.960	211.000	211.180	211.280	211.490	211.730	211.650	212.000	212.250	212.270	212.470
HD table	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Num gullies	0.000	0.000	0.000	3.000	2.000	0.000	3.000	1.000	2.000	2.000	2.000	1.000	0.000	3.000
expr:Freeboard	-	0.464626	0.725646	1.061410	0.770889	0.768150	0.684541	0.727396	0.822377	0.623938	0.836182	1.018509	1.021282	-

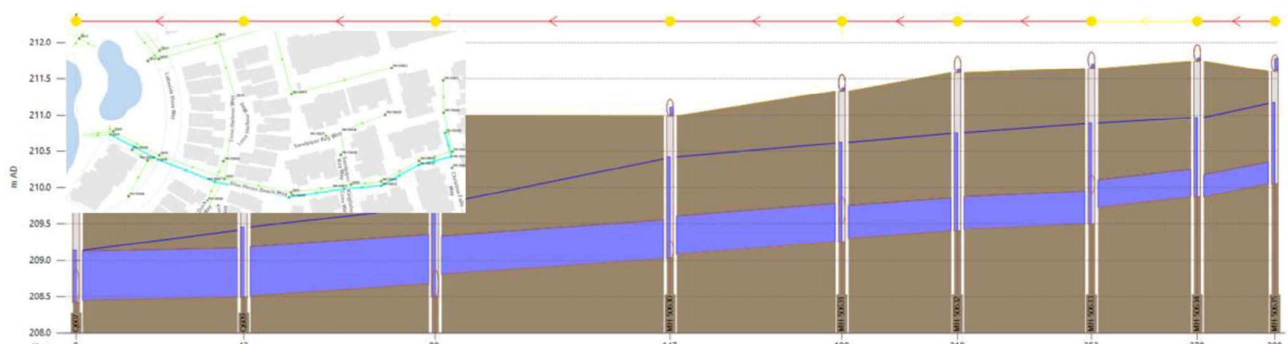
Figure 35: 100 Yr HGL level – Pipes Upstream to East Pond North Inlet – Raising the Overflow Weir



Link	Q609.1	MH-50057.1	MH-50630.1	MH-50631.1	MH-50632.1	MH-50633.1	MH-50634.1	MH-50635.1
US node ID	Q609	MH-50057	MH-50630	MH-50631	MH-50632	MH-50633	MH-50634	MH-50635
ds node	Q607	Q609	MH-50057	MH-50630	MH-50631	MH-50632	MH-50633	MH-50634
length (m)	41.6	47.5	58.4	42.6	28.7	33.3	26.2	19.3
Shape ID	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC
width (mm)	675	675	525	525	450	450	375	300
height (mm)	675	675	525	525	450	450	375	300
us inv (m AD)	208.494	208.680	209.027	209.255	209.406	209.499	209.874	210.053
ds inv (m AD)	208.451	208.511	208.808	209.085	209.303	209.425	209.728	209.874
grad (m/m)	0.00103	0.00356	0.00375	0.00399	0.00359	0.00222	0.00556	0.00928
surc	2.00	2.00	2.00	2.00	2.00	2.00	1.00	2.00
r.pfc (m3/s)	0.270	0.502	0.263	0.272	0.171	0.134	0.131	0.093
US total head (m AD)	209.503	209.793	210.512	210.622	210.744	210.873	210.938	211.177
DS flow (m3/s)	0.71513	0.64770	0.44727	0.28748	0.18824	0.17533	0.09478	0.09494
DS velocity (m/s)	2.342	1.738	1.964	1.284	1.096	1.160	1.298	1.497

Node	Q607	Q609	MH-50057	MH-50630	MH-50631	MH-50632	MH-50633	MH-50634	-
System type	storm	storm	storm	storm	storm	storm	storm	storm	storm
Asset ID	Q607	Q609	MH-50057	MH-50630	MH-50631	MH-50632	MH-50633	MH-50634	-
ground (m AD)	209.750	210.660	211.000	210.990	211.330	211.580	211.640	211.740	[illegible]
HD table	-	herringbone_On_Sag	Herringbone/Horizonal_Bar	herringbone_On_Sag	Herringbone/Horizonal_Bar	herringbone_On_Sag	-	-	-
Num gullies	0.000	5.000	0.000	1.000	0.000	2.000	0.000	0.000	1.000
expr:Freeboard	0.762207	1.306805	1.321030	0.638514	0.774305	0.888014	0.812165	0.829157	-

Figure 36: 100 Yr HGL Level – Pipes Upstream to East Pond South Inlet – Base Scenario



Link	Q609.1	MH-50057.1	MH-50630.1	MH-50631.1	MH-50632.1	MH-50633.1	MH-50634.1	MH-50635.1
US node ID	Q609	MH-50057	MH-50630	MH-50631	MH-50632	MH-50633	MH-50634	MH-50635
ds node	Q607	Q609	MH-50057	MH-50630	MH-50631	MH-50632	MH-50633	MH-50634
length (m)	41.6	47.5	58.4	42.6	28.7	33.3	26.2	19.3
Shape ID	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC	CIRC
width (mm)	675	675	525	525	450	450	375	300
height (mm)	675	675	525	525	450	450	375	300
us inv (m AD)	208.494	208.680	209.027	209.255	209.406	209.499	209.874	210.053
ds inv (m AD)	208.451	208.511	208.808	209.085	209.303	209.425	209.728	209.874
grad (m/m)	0.00103	0.00356	0.00375	0.00399	0.00359	0.00222	0.00556	0.00928
surc	2.00	2.00	2.00	2.00	2.00	2.00	1.00	2.00
r.pfc (m3/s)	0.270	0.502	0.263	0.272	0.171	0.134	0.131	0.093
US total head (m AD)	209.593	209.883	210.576	210.686	210.804	210.929	210.996	211.234
DS flow (m3/s)	0.70767	0.63984	0.43977	0.28454	0.18562	0.17278	0.09469	0.09501
DS velocity (m/s)	2.006	1.711	1.923	1.278	1.077	1.146	1.296	1.491

Node	Q607	Q609	MH-50057	MH-50630	MH-50631	MH-50632	MH-50633	MH-50634	-
System type	storm	storm	storm	storm	storm	storm	storm	storm	storm
Asset ID	Q607	Q609	MH-50057	MH-50630	MH-50631	MH-50632	MH-50633	MH-50634	-
ground (m AD)	209.750	210.660	211.000	210.990	211.330	211.580	211.640	211.740	[illegible]
HD table	-	herringbone_On_Sag	Herringbone/Horizonal_Bar	herringbone_On_Sag	Herringbone/Horizonal_Bar	herringbone_On_Sag	-	-	-
Num gullies	0.000	5.000	0.000	1.000	0.000	2.000	0.000	0.000	1.000
expr:Freeboard	0.611786	1.205486	1.228088	0.568537	0.708403	0.825987	0.754182	0.771479	-

Figure 37: 100 Yr HGL level – Pipes Upstream to East Pond South Inlet – Raising the Overflow Weir

4.2.7 Scenario 7: Combine Scenario S4 (Redirecting Minor System Flow from Swan Club OGS to the North Pond) with North Pond Upgrade Options

The previous section (Scenario S4) has illustrated that diverting flow from Swan Club OGS to the north pond would increase the bypass flow to Swan Lake through the overflow weir. This occurs because the additional flow from the Swan Club OGS not only increases the total volume entering the pond but also raises the peak flow rate in the inlet pipe, elevating the HGL level and causing bypass flow to occur more frequently. As a result, Scenario S4 is a less effective option. However, north pond upgrades, including raising the weir elevation, upsizing the pond inlet pipes, and pond expansion, would reduce significantly this overflow. Diverting more flow to the north pond will increase the inflow through 450 mm pipe, potentially increasing the risk of bypass due to insufficient pipe conveyance capacity, making upsizing the pond inlet pipe more necessary.. **Table 16** shows the reduction effect of combining Scenario S4 with different North Pond upgrade options.

Table 16: Comparison of Flow Reduction Between Combined Scenarios

Scenarios	Flow Volume Drains Swan Lake Through Swan Club OGS (m^3)	Flow Volume Drains Swan Lake Through North Pond Overflow Weir (m^3)	Typical Year Flow Reduction (m^3)
Existing	1210	2573	0
S7a- Combining Scenario 4 and the North pond portion of Scenario 5a (Raising the weirs)	0	1335	2468
S7b- Combining Scenario 4 and the North pond portion of Scenario 5b (Raising the weirs and upsizing pond inlet pipes)	0	1083	2720
S7c- Combining Scenario 4 and the North pond portion of Scenario 6b (Pond expansion)	0	13	3790

4.2.8 Scenario 8: Creating Underground Storage Capacity (to attenuate the flows from AMICA OGS and Swan Lake Blvd. OGS before they enter the local sewer system)

Mintleaf Gate was selected as the site for the presumed construction of underground storage pipes to manage the diverted flow from the Amica and Swan Lake Blvd. OGS units before discharging into the downstream pipes. This site was chosen based on the following considerations:

- The existing sewers are undersized, as shown in the model results.
- There will be no constraint in the downstream sewers after the system upgrades on Larkin Street are implemented.

The required size of this storage pipe depends largely on the roof areas directly connected to the storm sewers. The original 2021 model assumes that 44% of the roofs on Mintleaf Gate are directly connected to the storm sewers. However, the actual connection rate may be significantly lower, as observed through Google Earth. Due to this consideration, AECOM conducted a visual inspection from the right-of-way. The results indicate that, of approximately 70 visible downspouts, 7 are connected to the storm or sanitary sewers, while the remainder discharge directly onto the ground, as shown in **Appendix B**. This data has been incorporated into the InfoWorks model to accurately quantify the runoff generated by roof area that drains directly to storm sewers.

Model results indicate that a 208 m long, 2400 x 1200mm box storage pipe with a 550 mm orifice plate for outlet control will accommodate the existing and diverted flow generated by a 100-year design storm event. This setup controls the post-diversion flow to 350 L/s, which is lower than the existing minor system flows on Mintleaf Gate (420 L/s), providing a slight benefit to downstream conveyance capacity.

Through discussions with the City, AECOM was requested to investigate the feasibility of replacing the existing east and north stormwater management ponds with underground storage. Since these ponds were designed not only to provide runoff quantity management but also to deliver water quality benefits (through the inclusion of such design elements as sediment forebays, for example) it is recommended that, rather than constructing an underground box culvert system as proposed for Mintleaf Gateway, subsurface storage chambers - complete with an isolator row -could be constructed to allow sediments to settle.

To provide the same total storage volume (active storage plus permanent storage) as proposed for the north and east pond expansions in Scenario 6, the required chamber sizes are approximately 6,630 m^3 for the east pond and 7,370 m^3 for the north pond. Applying an estimated unit cost of $850 per cubic metre, the total cost to replace both ponds would be approximately $11.9 million, which is significantly higher than the cost of pond expansion. Furthermore, the water surface elevation within Swan Lake and either stormwater management pond indicates that hydrostatic uplift of any subsurface chambers may make such techniques infeasible. While the City may review available groundwater elevation data in this area in order to make an informed decision, the significant cost associated with such works further suggests that this option is infeasible.

Figure 38: System Upgrade Requirement to Achieve Scenario 7 (Box Storage Implementation on Mintleaf Gate)

4.2.9 Scenario 9: Redirecting/Pumping Flows from Some Foundation Drain Collectors (FDCs) toward Swan

Based on the available flow monitoring data in the FDC system at maintenance hole J689, which services approximately 63 properties along Miramar Drive and Swan Park Road, the annual measured flow volume is only 69 m³. This indicates that diverting FDC flow to Swan Lake will not significantly impact the overall chloride levels in the lake. However, this volume may be underestimated due to reduced monitoring accuracy when flow levels are below the sensor detection threshold, especially considering that foundation drainage is typically continuous, uniform, and low in flow rate. Additional flow monitoring is recommended to confirm the impact of this scenario.

5. Scenario Evaluation

Table 17 lists the proposed diversion options and describes the advantages, disadvantages, cost of implementation, flow reduction and cost of downstream improvements for each. This table is used as a screening method to evaluate the overall effectiveness of flow reduction, the impact of each option, and the ease of implementation for each diversion alternative.

Table 17: Scenario Evaluation

Scenario ID	Description	Typical Year Flow Reduction (m^3)	Cost of Implementation	Required Access to Private Property	Comment
S1	Redirecting minor system flow from AMICA OGS and Swan Lake Blvd. OGS to sewers on 16th Ave..	8310	$7,062,688.10	Y	**Benefits:** ■ Completely divert all flows from these 2 OGSs to Swan Lake **Disadvantages** ■ All the downstream sewers on Kingfisher Cove need to be upsized to accommodate the additional flow. ■ Requiring road excavation
S2	Redirecting the “first flush” portion of minor system flow from AMICA OGS and Swan Lake Blvd. OGS to 16th Ave. sewer.	8305	$1,109,178.53	Y	**Benefits:** ■ Lower peak flow contribution to the downstream sewers by allowing high flows to be diverted to Swan Lake ■ Winter flows with high chloride content can be fully diverted to the downstream; intense summer storms that do not create a chloride concern would still go to Swan Lake. ■ Preventing backwater at AMICA OGS **Disadvantages** ■ Local sewers are lower than normal Lake level, Potential risk of back water from the Lake to enter downstream system during extreme event. ■ Requiring road excavation
S3	Redirecting minor system flow from AMICA OGS and Swan Lake Blvd. OGS to the Lake outlet	8310	$1,757,024.75	Y	**Benefits:** ■ Able to bypass the undersize sewers on Kingfisher Cove ■ Downstream pipes are buried deep, additional flow to the system is less likely to raise a concern of basement flooding **Disadvantages** ■ Implementation require access to work on gated private properties ■ Requiring road excavation
S4	Redirecting minor system flow from Swan Club OGS to the North Pond	990	$275,071.88	Y	**Benefits:** ■ This large parking lot may be subject to high winter salt usage. ■ Excavation is on open space. **Disadvantages** ■ Spacing for such construction is very limited. ■ Additional flow to North Pond will increase the spill of water to Swan Lake from the pond. ■ Installing new pipes will require the removal of trees.

Scenario ID	Description	Typical Year Flow Reduction (m^3)	Cost of Implementation	Required Access to Private Property	Comment
S5a	Adjusting the flow splitter weir for the East Pond and North Pond to reduce flow bypass to the Lake	5016	$30,420.00	N	**Benefits:** ■ Minor construction work is required **Disadvantages** ■ Increases basement flooding risks for properties in the east pond catchment area ■ Raising the weir at North Pond won't significantly increase the active storage as the spill level is at 209 m and current weir is at 208.8 m ■ Small inlet pipe size will limit flow to the pond, causing the flow to bypass to the Lake during short durations of intense rain.
S5b	Raising the flow splitter weir at the North and East Ponds and upsizing the inflow pipes	5499	$124,787.00	N	**Benefits:** ■ Reduces the backwater caused by the limit of the 450mm pipe to enter Swan Lake **Disadvantages** ■ Diverting additional flow will cause the ponds to spill to the Lake more frequently ■ Installing new pipes will require the removal of trees.
S6	Expand storage capacity in the East and North Ponds to reduce flow bypass to the Lake	8266	$2,963,987.00	N	**Benefits:** ■ Provide more storage **Disadvantages** ■ Requiring additional open spaces to be converted to pond area. ■ Will require the removal of existing lakeside trails and trees ■ East pond lot may not have sufficient space to realign the trail after expansion
S6b	Expand storage capacity in North Pond to reduce flow bypass to the Lake	2573	$1,662,872.25		**Benefits:** ■ Provide more storage **Disadvantages** ■ Requiring additional open spaces to be converted to pond area. ■ Will require the removal of existing lakeside trails and trees ■ Not cost-effective as the catchment area is relatively small
S6c	Expand storage capacity in East Pond to reduce flow bypass to the Lake	5653	$1,301,114.75		**Benefits:** ■ Provide more storage ■ Large catchment area **Disadvantages** ■ Spacing in the pond lot is limited for expansion. ■ Requiring additional open spaces to be converted to pond area. ■ Will require the removal of existing lakeside trails and trees

Scenario ID	Description	Typical Year Flow Reduction (m^3)	Cost of Implementation	Required Access to Private Property	Comment
S7a	Combining S4 (Divert minor system flow from Swan Club OGS to the North Pond) with S5a (raising the North Pond flow split weir)	2468	$285,211.88	N	**Benefits:** ■ This large parking lot may be subject to high winter salt usage. ■ Excavation is on open space. ■ Compared to Scenario 4, bypass to the lake is reduced. **Disadvantages** ■ Spacing for such construction is very limited. ■ Installing new pipes will require the removal of trees. ■ Diverting additional flow will cause the pond to spill more frequently, and the additional flow from Swan Club OGS will increase further increase the spill frequency.
S7b	Combining S4 (Divert minor system flow from Swan Club OGS to the North Pond) with S5b (Raising the North Pond flow splitter weir and upsizing the inflow pipes)	2720	$315,544.13	N	**Benefits:** ■ This large parking lot may be subject to high winter salt usage. ■ Excavation is on open space. ■ Significantly reduces the backwater caused by the flow increased from Swan Club OGS and the limit of the 450mm pipe. **Disadvantages** ■ Spacing for such construction is very limited. ■ Installing new pipes will require the removal of trees. ■ Additional flow from Swan Club OGS will cause the pond to spill more frequently , and the additional flow from Swan Club OGS will increase further increase the spill frequency.
S7c	Combining S4 (Divert minor system flow from Swan Club OGS to the North Pond) with S6 (Expanding North Pond)	3770	$1,897,359.75	N	**Benefits:** ■ This large parking lot may be subject to high winter salt usage. ■ Bypass flow to Swan Lake caused by the additional flow from Swan Club OGS is reduced. **Disadvantages** ■ Spacing for such construction is very limited. ■ Installing new pipes will require the removal of trees Requiring additional open spaces to be converted to pond area. ■ Not cost-effective as the catchment area is relatively small
S8	Creating underground storage capacity to attenuate the flows from AMICA OGS and Swan Lake Blvd. OGS before they enter the local sewer system (to consider if there is a feasible candidate site and if the redirection scenarios increase flood risk)	8310 (46% of total typical year flow to Swan Lake)	$6,098,470.63	Y	**Benefits:** ■ No impact to downstream sewers ■ Able to mitigate the basement flood risks on Mint Leaf Gate Way, as these properties were identified as “Med Risk” in the Markham Village and Unionville Study (RVA, 2021) **Disadvantages** ■ New storage sewers need to be buried deep to match the invert level of the downstream sewer, which makes construction more expensive. ■ Excavation will occur on roads.

Scenario ID	Description	Typical Year Flow Reduction (m^3)	Cost of Implementation	Required Access to Private Property	Comment
S9	Redirecting/pumping flows from some foundation drain collectors (FDCs) toward Swan Lake	None	Not Analyzed	N	**Benefits:** ■ Foundation seepage water is considerable cleaner than stormwater runoff. Pumping clean water into the Lake will dilute chloride concentration in the Lake. **Disadvantages** ■ The flow monitor at J689 shows a flow volume of only 64 m^3 per year for FDC. Diverting it into Swan Lake would not provide water quality benefits.

6. Continuous Simulation for Water Budget

A continuous simulation from 2009 to 2024, using rainfall data collected at gauge MA 12, has been conducted to support the City's water and chloride budget analysis. Results from the calibrated model, including the runoff inflows to Swan Lake, the north pond, and the east pond, as well as lake levels and outflows over the simulation period, have been provided to the City. Evapotranspiration and groundwater recharge will be calculated separately by the City, and the results of the water and chloride budget analysis will be provided in a separate memo prepared by the City.

7. Conclusions

The conclusions of the study are:

For the four scenarios that redirect flows from Amica and Swan Lake Blvd. OGS, the conclusions are:

- **S1- Directing flows to 16th Avenue** is the most expensive option, reducing typical year inflow by 8,310 m³ at a cost of $7.06 million This high cost is primarily due to the undersized existing pipes on Kingfisher Cove Way, which would be unable to convey the additional flow without causing surface overflow during a 100-year design storm event. To accommodate the increased flow, upgrading approximately 645 metres of storm sewer along this route would be necessary.
- **S8- Directing flows from the Amica and Swan Lake Blvd. OGS units to underground storage pipes** on Mintleaf Gateway is also a costly option with an estimated cost of $6.10 million to divert 8,310 m³. So, while this option provides a significant advantage by preventing negative downstream impacts, and the storage pipe could mitigate basement flooding risks for properties along Mintleaf Gate, the cost per volume of water redirected from Swan Lake is amongst the highest.
- **S-3 Directing flows to the Lake outlet** avoids the constraints of undersized pipes on Kingfisher Cove Way. This scenario is more cost-effective than diverting flows to 16th Avenue, achieving the same reduction effect at only 25% of the cost ($1.75 million). While the additional flow will still cause surcharge in downstream pipes, due to the depths of downstream pipes, HGL remains below the assumed basement level (1.8 m underground). As a result, fewer downstream improvements are required compared to diverting flows to 16th Avenue.
- **S2- Directing "first flush" portion of the minor system flow from the Amica and Swan Lake Blvd. OGS units to 16th Avenue** is the most cost-effective option among the four. It requires smaller local pipes and achieves 99% (8,305 m³) of the reduction compared to a complete disconnection of the OGS units, but at a significantly lower cost. The total estimated cost of implementation - $1.11 million - is only 16% of the cost of diverting to 16th Avenue and 63% of the cost of diverting to the Lake outlet. Furthermore, it avoids the need for downstream upgrades. However, this scenario introduces a risk of Lake water backing up into the upstream and downstream sewers, as the invert levels of these OGS units are at the lake's normal water level. Additional measures to prevent backflow should be considered during implementation.

All four of these options would require additional upgrades to the original proposed solution on Larkin Avenue for the Markham Village area. The costs of these upgrades are not included in the current estimates, potentially making these three scenarios more expensive.

For the three scenarios which involve pond upgrades, the conclusions are:

- **S5a- Raising flow splitter weirs** at pond inlets is the most cost-effective option, reducing approximately 5,016 m³ of stormwater inflow at a cost of approximately $30,000, however, this option would slightly increase the basement flooding risks for properties in the east pond catchment area. Additionally, due to the limiting size of inlet pipes to the ponds, the flow rate to the ponds would still be limited, leading to occasional bypass of flows to Swan Lake during short durations of intense rain. Diverting additional flow will also cause the ponds to spill to the Lake more frequently
- **S5b- Raising the flow splitter weirs and upsizing the inflow pipes** will increase the reduction effect to 5,499 m³, however, diverting additional flow will cause the ponds to spill to the Lake more frequently. The cost is approximately $125 thousand.

- **S6a/6b/6c- Expanding the North and East ponds** increases the typical year flow reduction to 8,226 m³. However, this comes at a significantly higher cost—around $2.96 million. When comparing the expansion of the two ponds, upgrading the east pond is more cost-effective than upgrading the north pond. This is because, for a similar increase in storage volume, the catchment area of the east pond is twice that of the north pond, allowing it to collect more runoff and achieve a greater flow reduction potential. In additional, the complex terrain at the North Pond would require more volume of soil excavation. Upsizing the east pond will cost $1.30 million and result in a typical year inflow reduction of 5,653 m³, whereas upsizing the north pond will cost $1.66 million and achieve a lower typical year inflow reduction of 2,573 m³. Notwithstanding this conclusion, the East Pond is constrained by the adjacent roadway, trail network and limited residual pond block size. Further expansion of this facility may be hampered by constraints in realigning local trails and maintaining setbacks from Lakeside Vista Way. Long-term maintenance of the pond will be impacted by limited space for staging and sediment drying area. Future investigation of these matters is recommended.

For the scenarios that redirect Swan Club OGS to the North Pond, the conclusions are:

- **S4- Diverting the Swan Club OGS** is not considered to be a cost-effective option due to its small catchment area (0.21 ha). This OGS is expected to reduce direct discharge to Swan Lake by 1,210 m³ in a typical year, at a cost of $275,000, however, without upgrading the North Pond, additional flow directed into the pond will increase bypass flows through the flow splitter weirs, reducing the effectiveness of this solution.
- **S7a- Combining Scenario 4 (diverting Swan Club OGS to the North pond) with the North Pond portion of Scenario 5a (raising the weir)** retains the advantages of both individual scenarios. The benefits include the minimized construction work primarily occurring in open spaces and the cost-effectiveness. Additionally, this combination slightly mitigates the main drawback of Scenario 4, where the additional flow diverted to the north pond increases bypass flow to the Lake. The combined scenario reduces bypass flow to the lake by 2,468 m³. Compared to only raising the weir at the North pond, the combined scenario achieves an additional inflow reduction of 1,037 m³ at a cost of $285,000.
- **S7b- Combining Scenario 4 with the North Pond portion of Scenarios 5b (raising the weir and upsizing the pond inlet pipe)**, moderately reduces the additional bypass to Swan Lake caused by the extra flow from the Swan Club OGS. This scenario achieves a typical year inflow reduction of 2,720 m³ at a cost of $315,000. Compared to the Scenario 5b (the North pond portion) this Scenario has increased flow typical year reduction by 1,040 m³.
- **S7c- Combining Scenario 4 (diverting Swan Club OGS) with Scenario 6 (North pond expansion)** provides the highest typical year inflow reduction of 3,770 m³. However, this option comes at a significantly higher cost of approximately $1.88 million and requires additional long-term maintenance and involves the same constraints as Scenario 6.

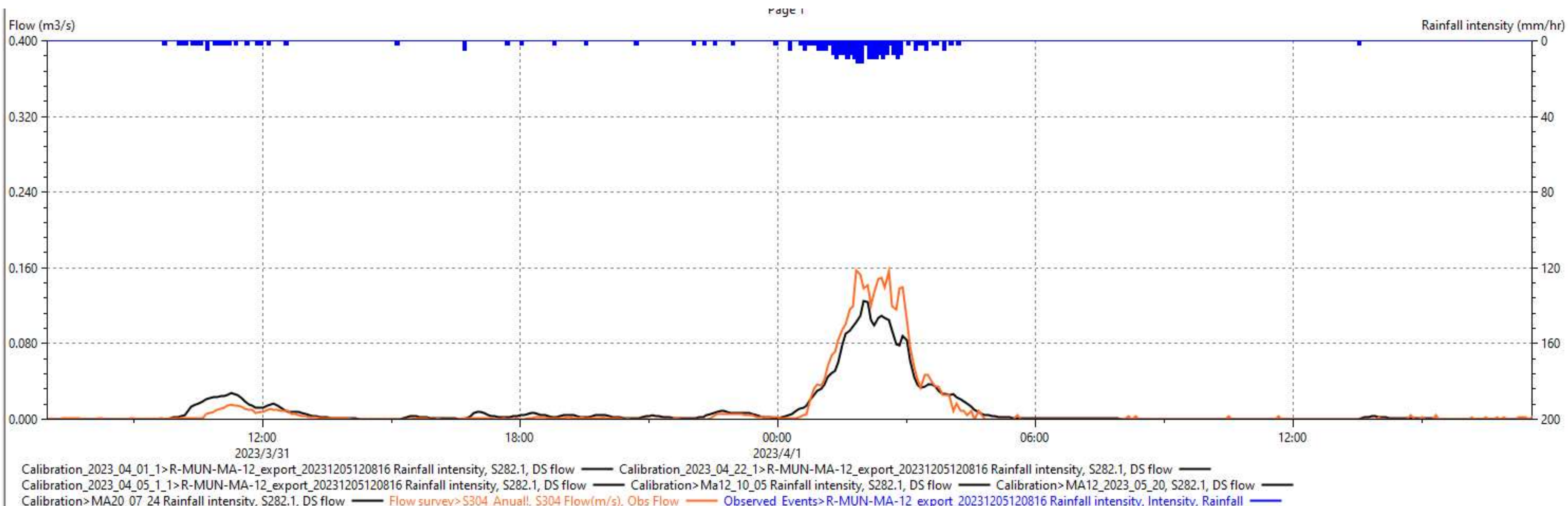
One scenario was considered involving redirecting Foundation Drain Collector flows to the Lake:

- **S9- Pumping Foundation Drain Collector (FDC) flow to Swan Lake** would not significantly impact chloride levels in the Lake. Flow monitoring data indicates an annual FDC flow of only 69 m³, which may be underestimated due to reduced monitoring accuracy at low water levels. Further FDC flow monitoring at different locations may be required to calibrate the FDC flow parameters.

Appendix A

Hydrograph Comparison

Gauge S304 Hydrograph comparison for Calibration events



	DS flow			Obs Flow			Rainfall		
	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (mm/hr)	Max (mm/hr)	Rainfall depth (mm)
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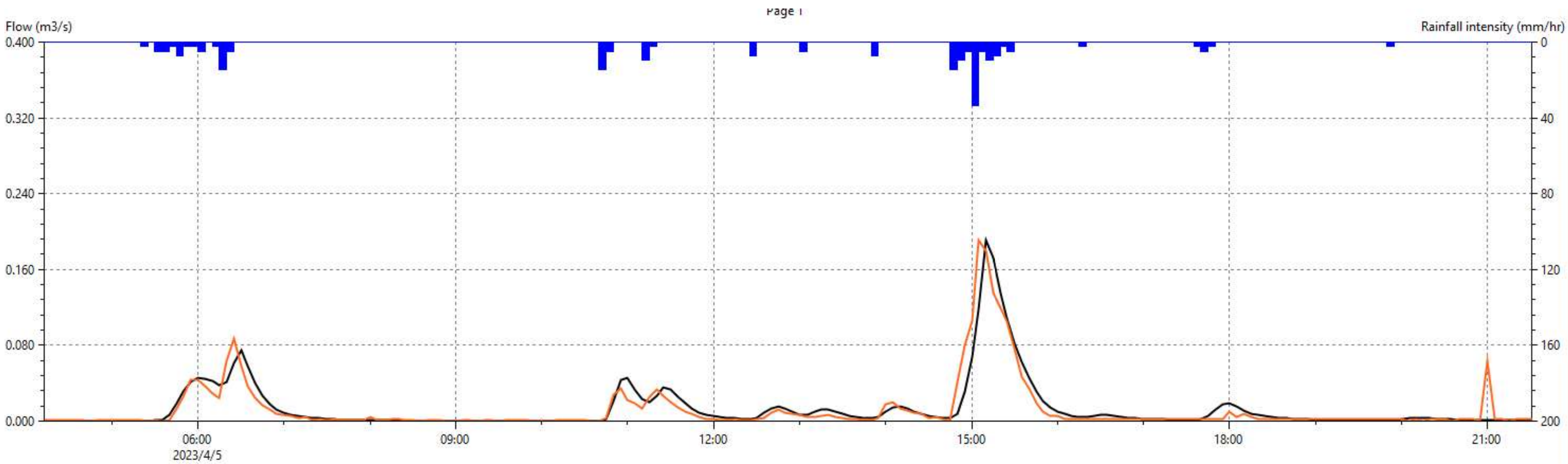
...6 Rainfall intensity, S282.1			0.000						
...6 Rainfall intensity, S282.1			0.000						
...5 Rainfall intensity, S282.1			0.000						
...MA12_2023_05_20, S282.1			0.000						
...4 Rainfall intensity, S282.1			0.000						
...4_Anual!, S304 Flow(m/s)				0.000	0.157	1198.646			
...ainfall intensity, Intensity							0.000	12.000	26.000



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Calibration_2023_04_05_1_1>R-MUN-MA-12_export_20231205120816 Rainfall intensity, S282.1, DS flow — Calibration>Ma12_10_05 Rainfall intensity, S282.1, DS flow — Calibration>MA12_2023_05_20, S282.1, DS flow —
Calibration>MA20_07_24 Rainfall intensity, S282.1, DS flow — Flow survey>S304_Anuall, S304 Flow(m/s), Obs Flow — Observed_Events>R-MUN-MA-12_export_20231205120816 Rainfall intensity, Intensity, Rainfall —

	DS flow			Obs Flow			Rainfall		
	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (mm/hr)	Max (mm/hr)	Rainfall depth (mm)
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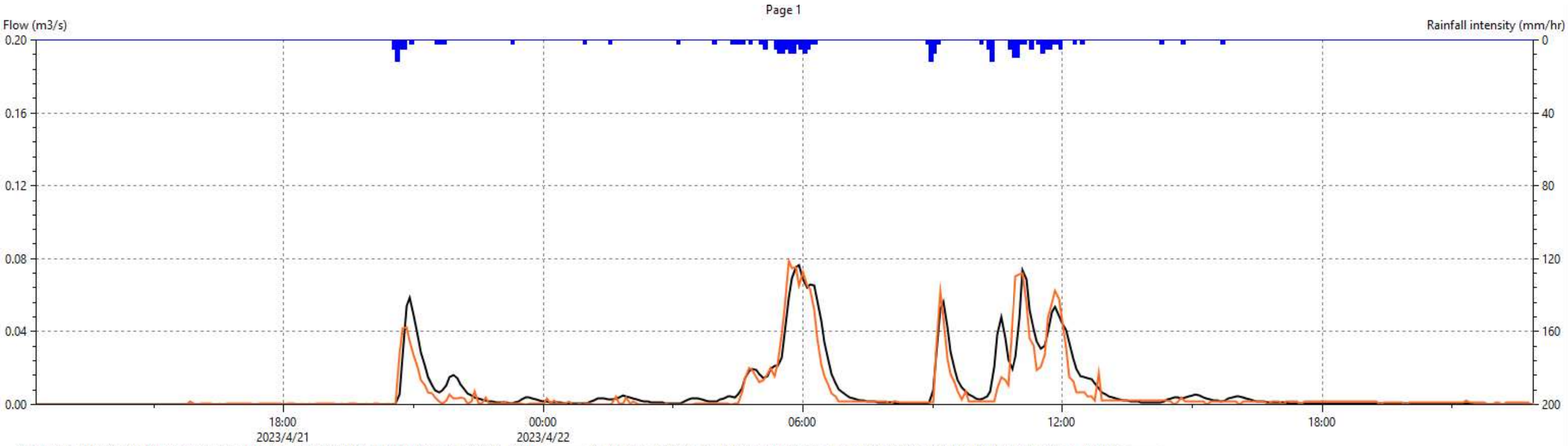
...6 Rainfall intensity, S282.1			0.000						
...6 Rainfall intensity, S282.1	0.000	0.191	784.603						
...5 Rainfall intensity, S282.1			0.000						
...MA12_2023_05_20, S282.1			0.000						
...4 Rainfall intensity, S282.1			0.000						
...4_Anuall, S304 Flow(m/s)				0.000	0.191	730.099			
...ainfall intensity, Intensity							0.000	33.600	17.400



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Calibration_2023_04_05_1_1>R-MUN-MA-12_export_20231205120816 Rainfall intensity, S282.1, DS flow — Calibration>Ma12_10_05 Rainfall intensity, S282.1, DS flow — Calibration>MA12_2023_05_20, S282.1, DS flow —
Calibration>MA20_07_24 Rainfall intensity, S282.1, DS flow — Flow survey>S304_Anual!, S304 Flow(m/s), Obs Flow — Observed_Events>R-MUN-MA-12_export_20231205120816 Rainfall intensity, Intensity, Rainfall —

	DS flow			Obs Flow			Rainfall		
	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (mm/hr)	Max (mm/hr)	Rainfall depth (mm)
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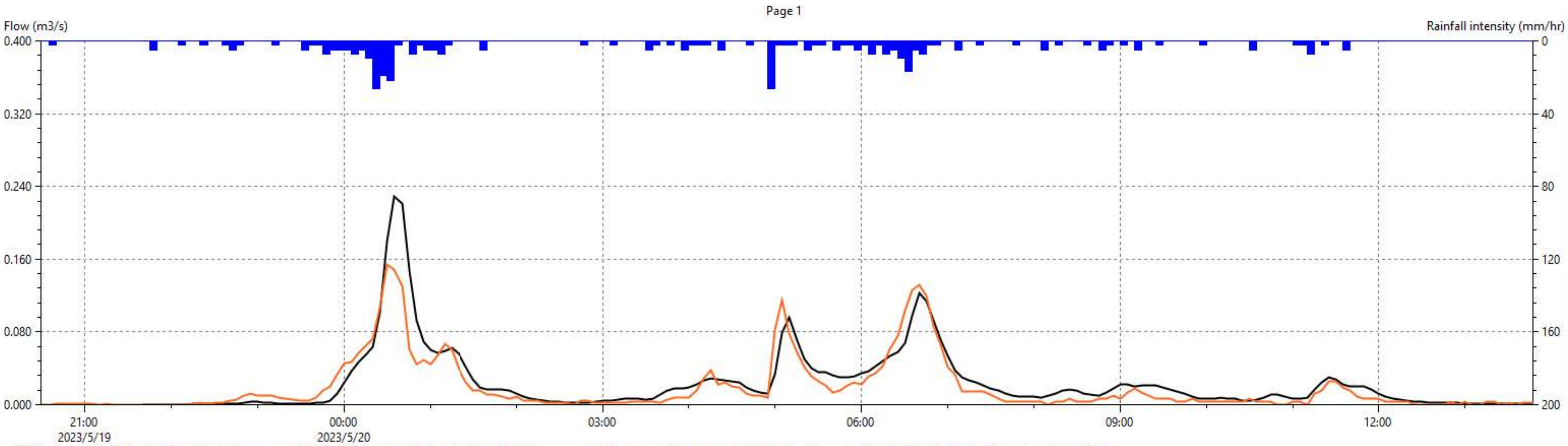
...16 Rainfall intensity, S282.1	0.000	0.077	935.491						
...16 Rainfall intensity, S282.1			0.000						
...05 Rainfall intensity, S282.1			0.000						
...>MA12_2023_05_20, S282.1			0.000						
...24 Rainfall intensity, S282.1			0.000						
...04_Anual!, S304 Flow(m/s)				0.000	0.079	770.753			
... Rainfall intensity, Intensity							0.000	12.000	20.600

Page 1



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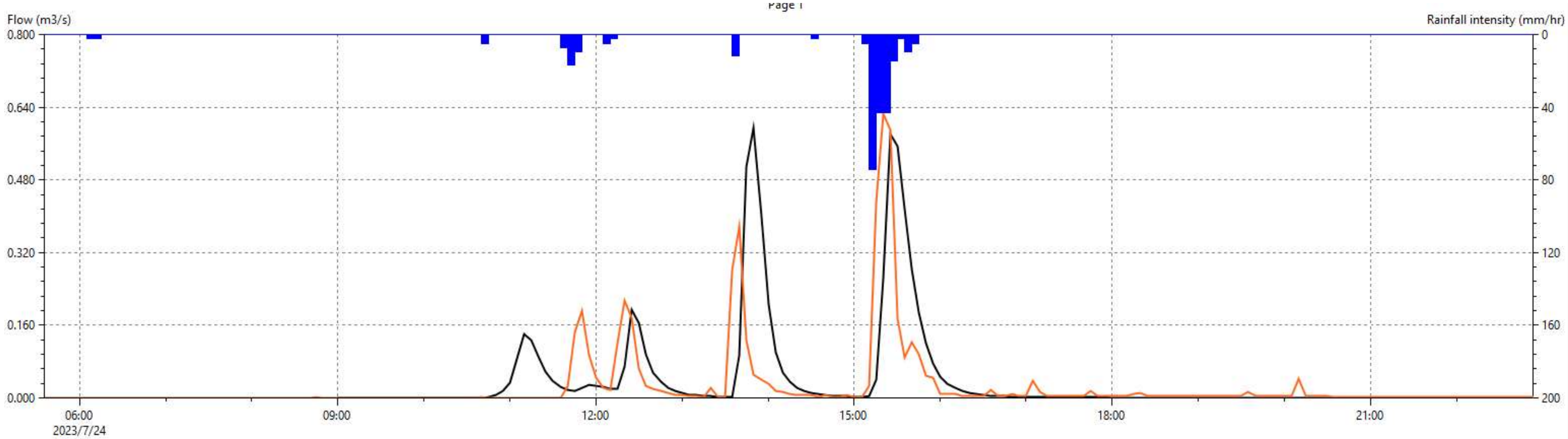
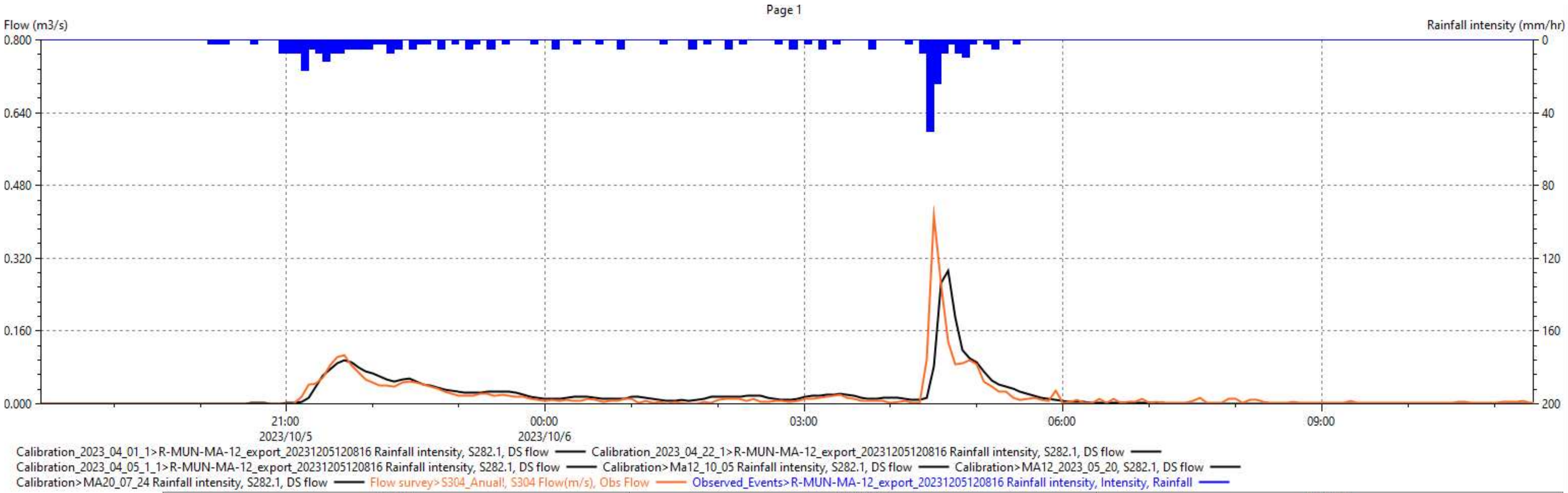
Calibration_2023_04_05_1_1>R-MUN-MA-12_export_20231205120816 Rainfall intensity, S282.1, DS flow —— Calibration>Ma12_10_05 Rainfall intensity, S282.1, DS flow —— Calibration>MA12_2023_05_20, S282.1, DS flow ——
Calibration>MA20_07_24 Rainfall intensity, S282.1, DS flow —— Flow survey>S304_Anual!, S304 Flow(m/s), Obs Flow —— Observed_Events>R-MUN-MA-12_export_20231205120816 Rainfall intensity, Intensity, Rainfall ——

	DS flow			Obs Flow			Rainfall		
	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (mm/hr)	Max (mm/hr)	Rainfall depth (mm)
...16 Rainfall intensity, S282.1			0.000						
...16 Rainfall intensity, S282.1			0.000						
...16 Rainfall intensity, S282.1			0.000						
...05 Rainfall intensity, S282.1			0.000						
...>MA12_2023_05_20, S282.1	0.000	0.230	1380.921						
...24 Rainfall intensity, S282.1			0.000						
...04_Anual!, S304 Flow(m/s)				0.000	0.154	1139.812			
... Rainfall intensity, Intensity							0.000	26.400	33.400

Page 1

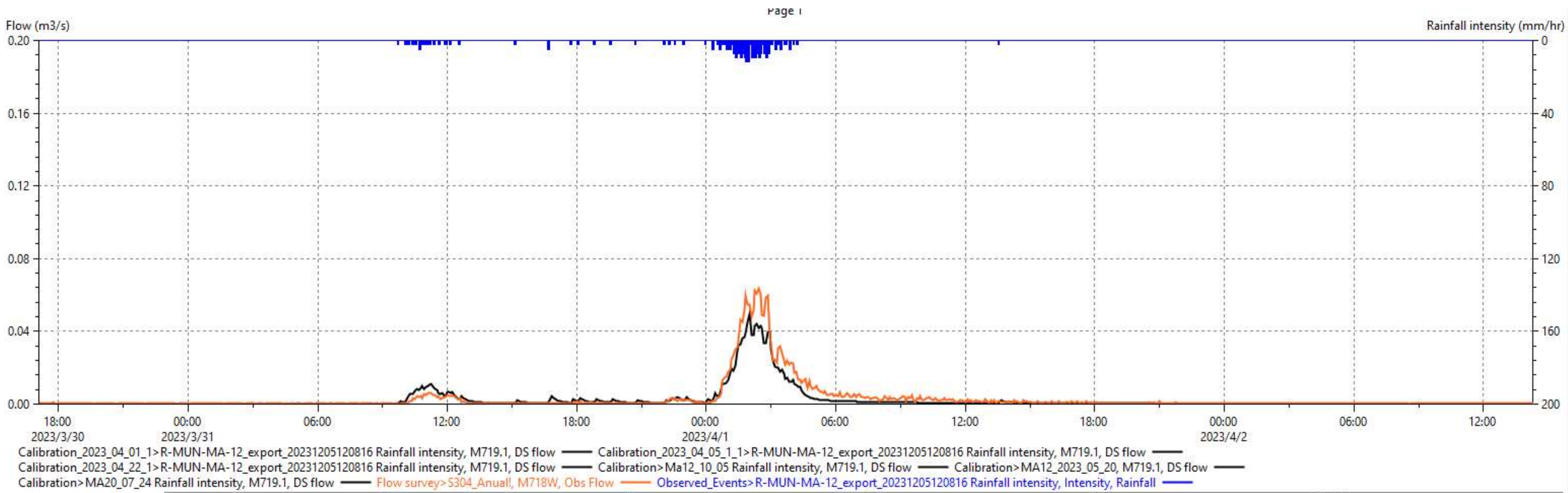


	DS flow			Obs Flow			Rainfall		
	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (mm/hr)	Max (mm/hr)	Rainfall depth (mm)
...16 Rainfall intensity, S282.1			0.000						
...16 Rainfall intensity, S282.1			0.000						
...16 Rainfall intensity, S282.1			0.000						
...05 Rainfall intensity, S282.1			0.000						
...>MA12_2023_05_20, S282.1			0.000						
...24 Rainfall intensity, S282.1	0.000	0.595	1895.050						
...04_Anual!, S304 Flow(m/s)				0.000	0.626	1502.086			
... Rainfall intensity, Intensity							0.000	74.400	21.800



	DS flow			Obs Flow			Rainfall		
	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (mm/hr)	Max (mm/hr)	Rainfall depth (mm)
...16 Rainfall intensity, S282.1			0.000						
...16 Rainfall intensity, S282.1			0.000						
...16 Rainfall intensity, S282.1			0.000						
...05 Rainfall intensity, S282.1	0.000	0.292	1129.129						
...>MA12_2023_05_20, S282.1			0.000						
...24 Rainfall intensity, S282.1			0.000						
...04_Anual!, S304 Flow(m/s)				0.000	0.412	1045.297			
... Rainfall intensity, Intensity							0.000	50.400	27.600

Gauge M719 Hydrograph comparison for Calibration events



	DS flow			Obs Flow			Rainfall		
	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (mm/hr)	Max (mm/hr)	Rainfall depth (mm)
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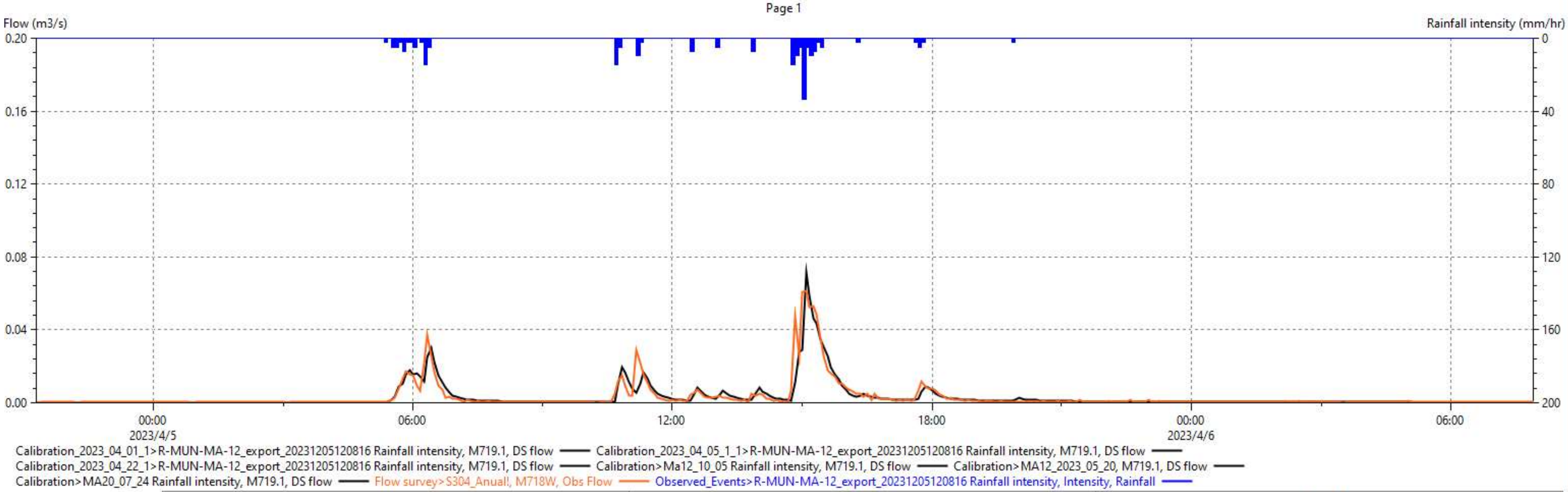
...6 Rainfall intensity, M719.1			0.000						
...6 Rainfall intensity, M719.1			0.000						
...5 Rainfall intensity, M719.1			0.000						
...MA12_2023_05_20, M719.1			0.000						
...4 Rainfall intensity, M719.1			0.000						
...rvey>S304_Anual!, M718W				0.000	0.064	719.880			
... Rainfall intensity, Intensity							0.000	12.000	26.000

Page 1



	DS flow			Obs Flow			Rainfall		
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...6 Rainfall intensity, M719.1	0.000	0.072	352.212						
...6 Rainfall intensity, M719.1			0.000						
...5 Rainfall intensity, M719.1			0.000						
...MA12_2023_05_20, M719.1			0.000						
...4 Rainfall intensity, M719.1			0.000						
...rvey>S304_Anual!, M718W				0.000	0.061	361.194			
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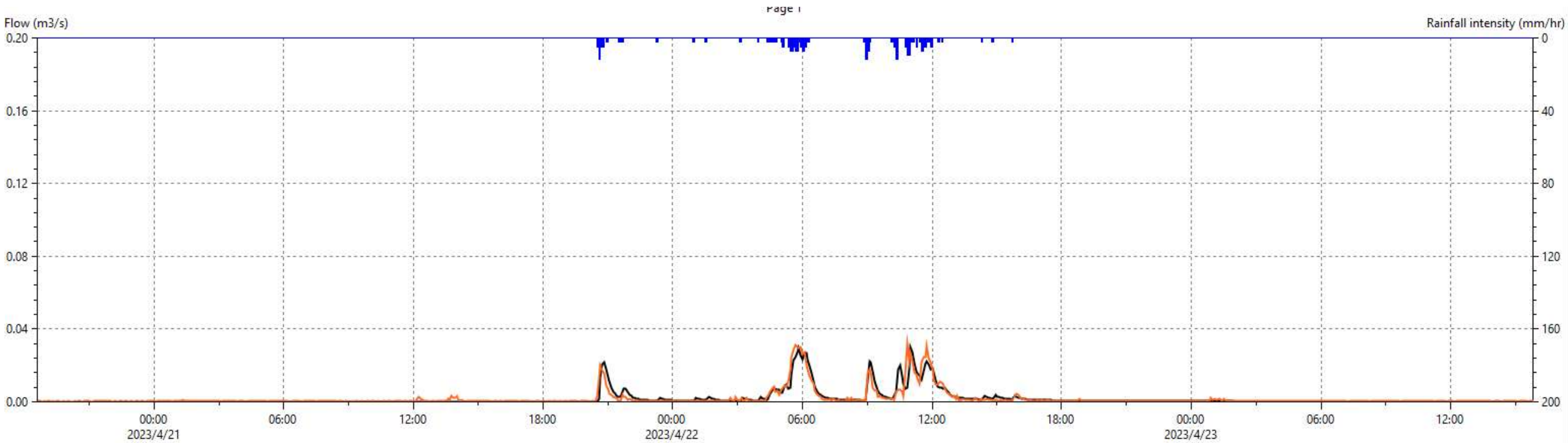

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Calibration>MA20_07_24 Rainfall intensity, M719.1, DS flow — Flow survey>S304_Anual!, M718W, Obs Flow — Observed_Events>R-MUN-MA-12_export_20231205120816 Rainfall intensity, Intensity, Rainfall —

	DS flow			Obs Flow			Rainfall		
	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (mm/hr)	Max (mm/hr)	Rainfall depth (mm)
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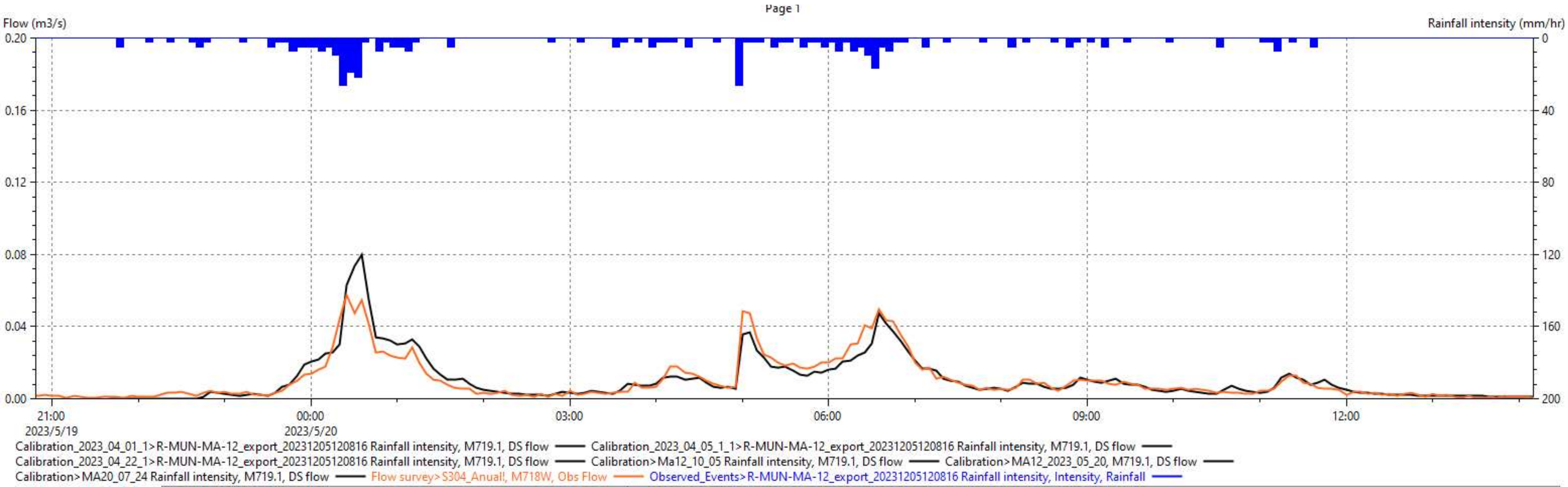
...6 Rainfall intensity, M719.1			0.000						
...6 Rainfall intensity, M719.1	0.000	0.030	424.887						
...5 Rainfall intensity, M719.1			0.000						
...MA12_2023_05_20, M719.1			0.000						
...4 Rainfall intensity, M719.1			0.000						
...rvey>S304_Anual!, M718W				0.000	0.032	426.743			
... Rainfall intensity, Intensity							0.000	12.000	20.600

Page 1



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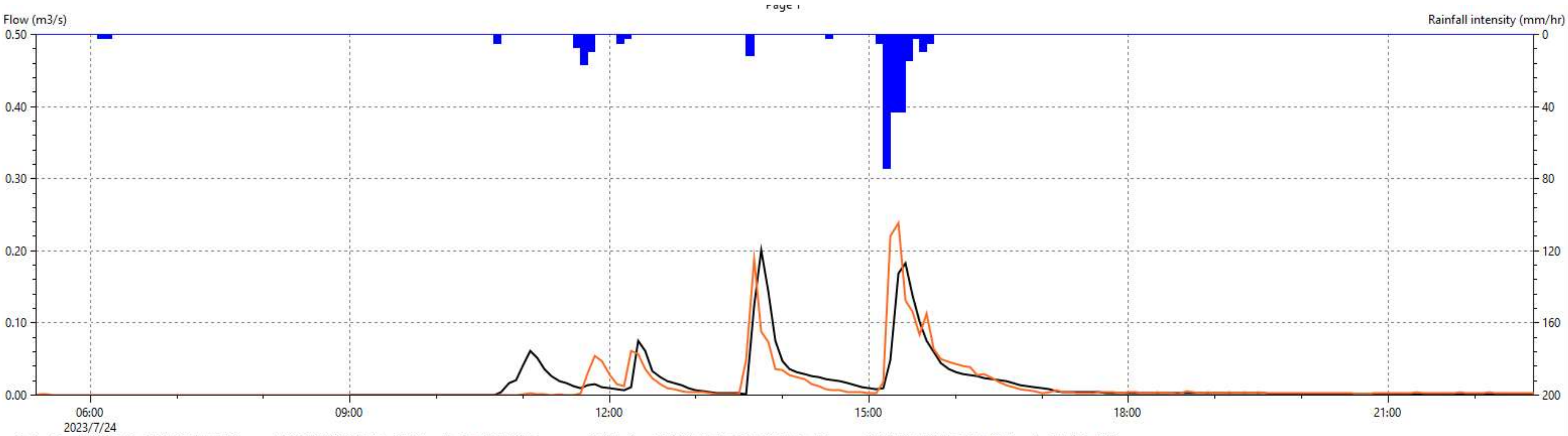
...6 Rainfall intensity, M719.1			0.000						
...6 Rainfall intensity, M719.1			0.000						
...6 Rainfall intensity, M719.1			0.000						
...5 Rainfall intensity, M719.1			0.000						
...MA12_2023_05_20, M719.1	-0.000	0.080	631.655						
...4 Rainfall intensity, M719.1			0.000						
...rvey>S304_Anuall, M718W				0.000	0.057	629.280			
... Rainfall intensity, Intensity							0.000	26.400	33.200



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Calibration>MA20_07_24 Rainfall intensity, M719.1, DS flow — Flow survey>S304_Anual!, M718W, Obs Flow — Observed_Events>R-MUN-MA-12_export_20231205120816 Rainfall intensity, Intensity, Rainfall —

	DS flow			Obs Flow			Rainfall		
	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (mm/hr)	Max (mm/hr)	Rainfall depth (mm)
...6 Rainfall intensity, M719.1			0.000						
...6 Rainfall intensity, M719.1			0.000						
...6 Rainfall intensity, M719.1			0.000						
...5 Rainfall intensity, M719.1			0.000						
...MA12_2023_05_20, M719.1			0.000						
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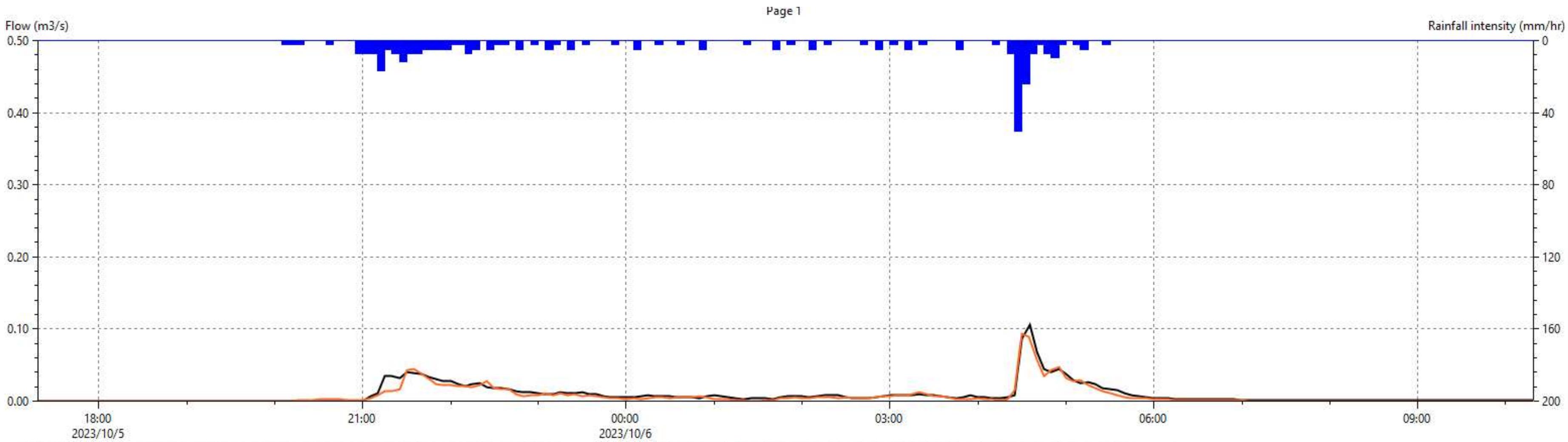
... Rainfall intensity, Intensity							0.000	74.400	21.800



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Calibration>MA20_07_24 Rainfall intensity, M719.1, DS flow — Flow survey>S304_Anual!, M718W, Obs Flow — Observed_Events>R-MUN-MA-12_export_20231205120816 Rainfall intensity, Intensity, Rainfall —

	DS flow			Obs Flow			Rainfall		
	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (mm/hr)	Max (mm/hr)	Rainfall depth (mm)
...6 Rainfall intensity, M719.1			0.000						
...6 Rainfall intensity, M719.1			0.000						
...6 Rainfall intensity, M719.1			0.000						
...5 Rainfall intensity, M719.1	0.000	0.106	518.221						
...MA12_2023_05_20, M719.1			0.000						
...4 Rainfall intensity, M719.1			0.000						
...rvey>S304_Anual!, M718W				0.000	0.093	429.946			
... Rainfall intensity, Intensity							0.000	50.400	27.600

Appendix B

Mintleaf Gate Downspout Inspection Results

SW DIVERSIONS
SWAN LAKE
(City of Markham)

LEGEND

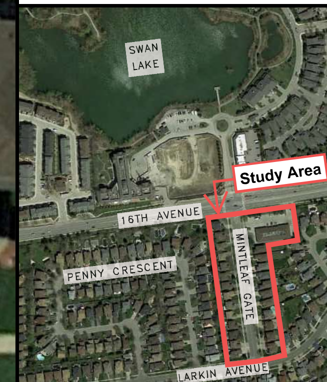
- Downspout discharging directly to the ground surface
- Downspout likely connected to some piping in the ground
- 17 House Number

AECOM

Mintleaf Gate,
Markham ON
Downspout Information

FIGURE 1
Sheet 1 of 3

SW DIVERSIONS
SWAN LAKE
(City of Markham)



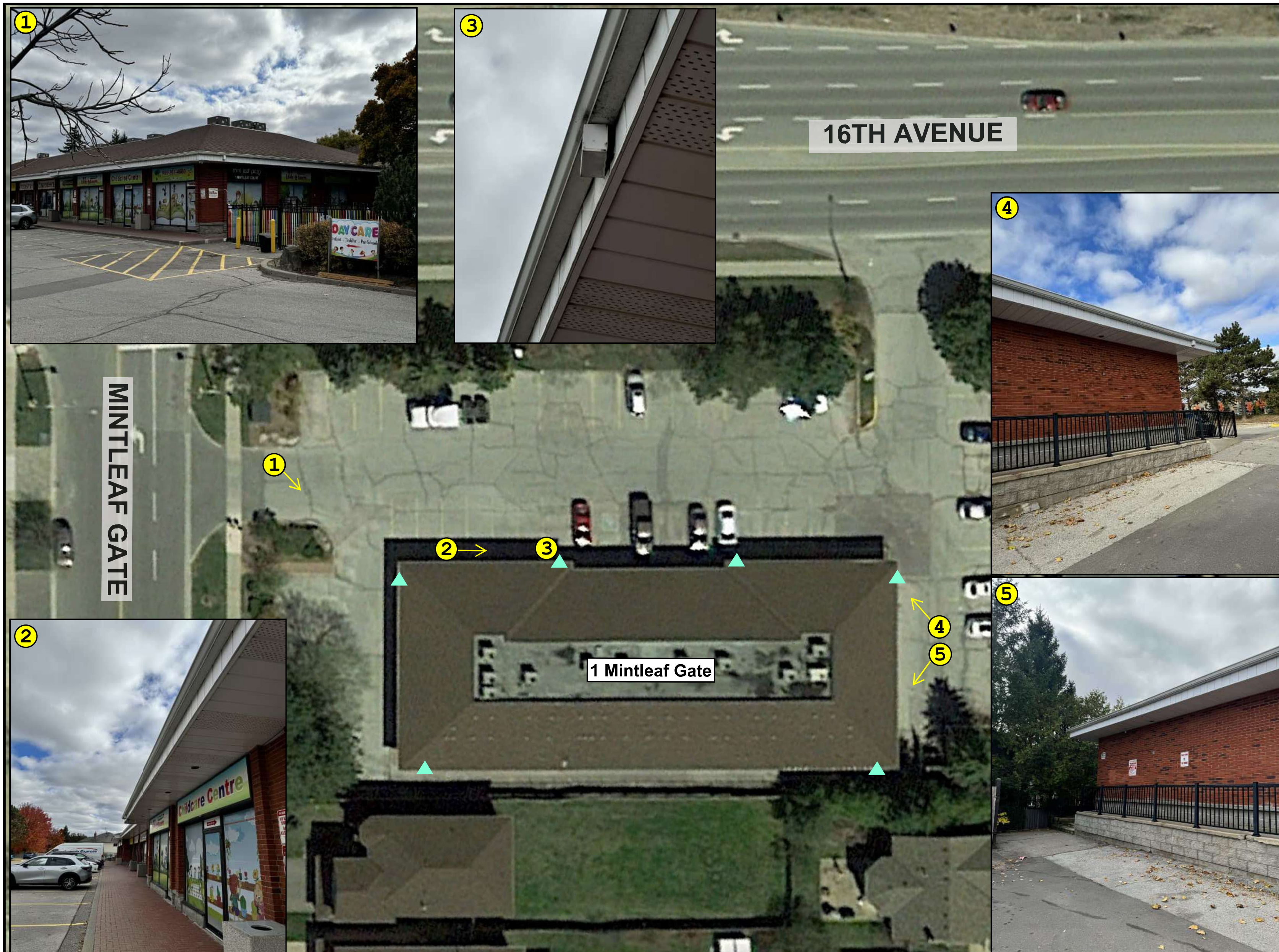
LEGEND

- Downspout discharging directly to the ground surface
- Downspout likely connected to some piping in the ground
- 17 House Number

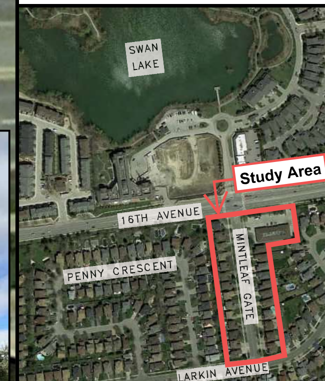
AECOM

Mintleaf Gate,
Markham ON
Downspout Information

FIGURE 1
Sheet 2 of 3



SW DIVERSIONS SWAN LAKE
(City of Markham)



LEGEND

▲ Most likely connection from the roof to interior of the building. (Refer Photo 3)

NOTE

No downspouts present at this property.

N

AECOM

Mintleaf Gate, Markham ON
Downspout Information

FIGURE 1
Sheet 3 of 3

AECOM

Appendix C

Cost Estimate Sheet

Scenario 1: Redirection of minor system flows from AMICA OGS and Swan Lake Blvd. OGS to sewers on 16th Ave.

Item	Part	Size/type	Depth	Quantity	Unit Cost	Cost of implementation	Contingency*	Totaling
Supply and install 375 mm diameter, 0-3 m deep, storm sewers	Part 1 - Storm sewers (unit m)	375 mm (Concrete)	0-3 m	32 m	$1,230.00	$39,360.00	$11,808.00	$51,168.00
Supply and install 450 mm diameter, 3-4.5 m deep, storm sewers	Part 1 - Storm sewers (unit m)	450 mm	3-4.5 m	113 m	$1,550.00	$175,150.00	$52,545.00	$227,695.00
Supply and install 600 mm diameter, 3-4.5 m deep, storm sewers	Part 1 - Storm sewers (unit m)	600 mm	3-4.5 m	76 m	$1,700.00	$129,200.00	$38,760.00	$167,960.00
Supply and install 750 mm diameter, 3-4.5 m deep, storm sewers	Part 1 - Storm sewers (unit m)	750 mm	3-4.5 m	95 m	$2,000.00	$190,000.00	$57,000.00	$247,000.00
Supply and install 750 mm diameter, 4.5-6 m deep, storm sewers	Part 1 - Storm sewers (unit m)	750 mm	4.5-6 m	94 m	$2,150.00	$202,100.00	$60,630.00	$262,730.00
Supply and install 825 mm diameter, 3-4.5 m deep, storm sewers	Part 1 - Storm sewers (unit m)	825 mm	3-4.5 m	33 m	$2,650.00	$87,450.00	$26,235.00	$113,685.00
Supply and install 825 mm diameter, 4.5-6 m deep, storm sewers	Part 1 - Storm sewers (unit m)	825 mm	4.5-6 m	95 m	$2,700.00	$256,500.00	$76,950.00	$333,450.00
Supply and install 1200 mm diameter, 4.5-6 m deep, storm sewers	Part 1 - Storm sewers (unit m)	1200 mm	4.5-6 m	328 m	$3,690.00	$1,210,320.00	$363,096.00	$1,573,416.00
Restoration of 0 - 3 m deep urban roads	Part 2 Pavement Restoration (Unit m)	Urban Road	0-3 m	32 m	$1,690.00	$54,080.00	$16,224.00	$70,304.00
Restoration of 3 - 4.5 m deep urban roads	Part 2 Pavement Restoration (Unit m)	Urban Road	3-4.5 m	317 m	$1,955.00	$619,735.00	$185,920.50	$805,655.50
Restoration of 4.5 - 6 m deep urban roads	Part 2 Pavement Restoration (Unit m)	Urban Road	4.5-6 m	517 m	$2,450.00	$1,266,650.00	$379,995.00	$1,646,645.00
Supply and install new maintenance holes and upsize existing maintenance holes to required sizes (minimal 600 mm larger than the largest connected pipe)	Part 3: Maintenance hole Installation and upgrade	30% of total pipe cost	-	-	-	$687,024.00	$206,107.20	$893,131.20
N/A	Part 4: Flow Control Structures	-	-	-	-	$0.00	$0.00	$0.00
Allowance for utility replacement and service reconnection	Part 5: Miscellaneous allowance	15% of total pipe cost	-	-	-	$343,512.00	$103,053.60	$446,565.60
Allowance for catchbasin replacement and Lateral reconnection	Part 5: Miscellaneous allowance	7.5% of total pipe cost	-	-	-	$171,756.00	$51,526.80	$223,282.80
							Total	$7,062,688.10

* 30% contingency accounts for the potential cost increase due to difficult site condition, design changes, market fluctuation, inflation, etc.

Scenario 2: Redirecting the "first flush" portion of minor system flow from AMICA OGS and Swan Lake Blvd. OGS to 16th Ave. sewer

Item	Type	Size/type	Depth	Quantity	Unit Cost	Cost of implementation	Contingency	Totaling
Supply and install 300 mm diameter, 0-3 m deep, storm sewers	Part 1 - Storm Sewers (unit m)	300 mm (PVC)	0-3 m	145 m	$1,250.00	$181,250.00	$54,375.00	$235,625.00
Supply and install 300 mm diameter, 3-4.5 m deep, storm sewers	Part 1 - Storm Sewers (unit m)	300 mm (PVC)	3-4.5 m	76 m	$1,420.00	$107,920.00	$32,376.00	$140,296.00
Restoration of 0 - 3 m deep urban roads	Part 2 Pavement Restoration (Unit m)	-	0-3 m	145 m	$1,690.00	$245,050.00	$73,515.00	$318,565.00
Restoration of 3 - 4.5 m deep urban roads	Part 2 Pavement Restoration (Unit m)	-	3-4.5 m	76 m	$1,955.00	$148,580.00	$44,574.00	$193,154.00
Supply and install new maintenance holes and upsize existing maintenance holes to required sizes (minimal 600 mm larger than the largest connected pipe)	Part 3: Maintenance hole Installation and upgrade	30% of total pipe cost	-	-	-	$86,751.00	$26,025.30	$112,776.30
Supply and install orifice plate	Part 4: Flow Control Structures	-	-	2	$1,500.00	$3,000.00	$900.00	$3,900.00
Supply and Install precast concrete overflow weir	Part 4: Flow Control Structures	-		2	$7,800.00	$15,600.00	$4,680.00	$20,280.00
Allowance for utility replacement and service reconnection	Part 5: Miscellaneous allowance	15% of total pipe cost	-	-	-	$43,375.50	$13,012.65	$56,388.15
Allowance for catchbasin replacement and Lateral reconnection	Part 5: Miscellaneous allowance	7.5% of total pipe cost	-	-	-	$21,687.75	$6,506.33	$28,194.08
								$1,109,178.53

Scenario 3: Redirecting minor system flow from AMICA OGS and Swan Lake Blvd. OGS to the Lake outlet

Item	Part	Size/type	Depth	Unit	Unit Cost	Cost of implementation	Contingency	Totaling
Supply and install 525 mm diameter, 0-3 m deep, storm sewers	Part 1 - Storm Sewers (unit m)	525 mm	0-3 m	179	1680	$300,720.00	$90,216.00	$390,936.00
Supply and install 525 mm diameter, 3-4.5 m deep, storm sewers	Part 1 - Storm Sewers (unit m)	525 mm	3-4.5 m	64	$1,750.00	$112,000.00	$33,600.00	$145,600.00
Supply and install 525 mm diameter, 4.5-6 m deep, storm sewers	Part 1 - Storm Sewers (unit m)	525 mm	4.5-6 m	54	$1,970.00	$106,380.00	$31,914.00	$138,294.00
Restoration of 0-3 m deep Urban Road	Part 2 Pavement Restoration (Unit m)	Urban Road	0-3 m	179	$1,690.00	$302,510.00	$90,753.00	$393,263.00
Restoration of 3-4.5 m deep Urban Road	Part 2 Pavement Restoration (Unit m)	Urban Road	3-4.5 m	64	$1,955.00	$125,120.00	$37,536.00	$162,656.00
Restoration of 4.5-6 m Deep Urban Road	Part 2 Pavement Restoration (Unit m)	Urban Road	4.5-6 m	54	$2,450.00	$132,300.00	$39,690.00	$171,990.00
Supply and install new maintenance hole s and upsize the existing maintenance hole to the require sizes (minimal 600 mm larger than the largest connected pipe)	Part 3: maintenance hole Installation and upgrade	30% of total pipe cost	-	-	-	$155,730.00	$46,719.00	$202,449.00
-	Part 4: Flow Control Structures	-	-	-	-	$0.00	$0.00	$0.00
							$0.00	
Allowance for utility replacement and service reconnection	Part 5: Miscellaneous allowance	15% of total pipe cost	-	-	-	$77,865.00	$23,359.50	$101,224.50
Allowance for catchbasin replacement and Lateral reconnection	Part 5: Miscellaneous allowance	7.5% of total pipe cost	-	-	-	$38,932.50	$11,679.75	$50,612.25
								$1,757,024.75

Scenario 4: Redirecting minor system flow from Swan Club OGS to the North Pond

Item	Part	Size/type	Depth	Quantity	Unit Cost	Cost of implementation	Contingency	Totaling
Supply and install 300 mm diameter, 0-3 deep, storm sewers	Part 1 - Storm Sewers (unit m)	300 mm (PVC)	0-3 m	111 m	$1,250.00	$138,750.00	$41,625.00	$180,375.00
Restoration of 0-3 deep Open Space	Part 2 Pavement Restoration (Unit m)	Open Space	0-3 m	0 m	$0.00	$0.00	$0.00	$0.00
Supply and install new maintenance holes and upsize existing maintenance holes to required sizes (minimal 600 mm larger than the largest connected pipe)	Part 3: Maintenance hole Installation and upgrade	30% of total pipe cost	-	-	-	$41,625.00	$12,487.50	$54,112.50
N/A	Part 4: Flow Control Structures	-	-	-	-	$0.00	$0.00	$0.00
Allowance for utility replacement and service reconnection	Part 5: Miscellaneous allowance	15% of total pipe cost	-	-	-	$20,812.50	$6,243.75	$27,056.25
Allowance for catchbasin replacement and Lateral reconnection	Part 5: Miscellaneous allowance	7.5% of total pipe cost	-	-	-	$10,406.25	$3,121.88	$13,528.13
								$275,071.88

Scenario 5: Adjusting the flow splitter weir for the East Pond and North Pond to reduce flow bypass to the Lake

Item	Part	Size/type	Depth	Quantity	Unit Cost	Cost of implementation	Contingency	Totaling
N/A	Part 1 - Storm Sewers (unit m)				$0.00	$0.00	$0.00	$0.00
N/A	Part 2 Pavement Restoration (Unit m)				$0.00	$0.00	$0.00	$0.00
N/A	Part 3: Maintenance hole Installation and upgrade				$0.00	$0.00	$0.00	$0.00
Supply and install precast concrete overflow weirs	Part 4: Flow Control Structures	-	-	3	$7,800.00	$23,400.00	$7,020.00	$30,420.00
N/A	Part 5: Miscellaneous allowance				$0.00	$0.00	$0.00	$0.00
							Total:	$30,420.00

Scenario 5b: Adjusting the flow splitter weir and upsize the pond inlet pipe

Item	Part	Size/type	Depth	Quantity	Unit Cost	Cost of implementation	Contingency	Totaling
Supply and install 600 mm diameter, 0-3 m deep, storm sewers	Part 1 - Storm Sewers (unit m)	600 mm	0-3 m	28 m	$1,700.00	$47,600.00	$14,280.00	$61,880.00
Restoration of 0-3 m Deep Open Space	Part 2 Pavement Restoration (Unit m)	Open Space	0-3 m	-	$0.00	$0.00	$0.00	$0.00
Supply and install new maintenance holes and upsize existing maintenance holes to required sizes (minimal 600 mm larger than the largest connected pipe)	Part 3: Maintenance hole Installation and upgrade	30% of total pipe cost	-	-	-	$14,280.00	$4,284.00	$18,564.00
Supply and install precast concrete overflow weirs	Part 4: Flow Control Structures	-	-	3	$7,800.00	$23,400.00	$7,020.00	$30,420.00
Allowance for utility replacement and service reconnection	Part 5: Miscellaneous allowance	15% of total pipe cost	-	-	-	$7,140.00	$2,142.00	$9,282.00
Allowance for catchbasin replacement and Lateral reconnection	Part 5: Miscellaneous allowance	7.5% of total pipe cost	-	-	-	$3,570.00	$1,071.00	$4,641.00
								$124,787.00

S6 Pond Expansion

Item	Part	Size/type	Depth	Quantity	Unit Cost	Cost of implementation	Contingency	Totaling
Supply and Install 600 mm diameter, 0-3 m deep, storm sewers	Part 1 - Storm Sewers (unit m)	600 mm	0-3 m	28	$1,700.00	$47,600.00	$14,280.00	$61,880.00
Restoration of 0-3 m Deep Open Space	Part 2 Pavement Restoration (Unit m)	Open Space	0-3 m	-	-	$0.00	$0.00	$0.00
Supply and install new maintenance hole s and upsize the existing maintenance hole to the require sizes (minimal 600 mm more than the largest connected pipe)	Part 3: maintenance hole Installation and upgrade	30% of total pipe cost	-	-	-	$14,280.00	$4,284.00	$18,564.00
Supply and install pre-casted concrete overflow weirs	Part 4: Flow Control Structures	-	-	3	$7,800.00	$23,400.00	$7,020.00	$30,420.00
Earth work (Cut and Fill) (Unit m3)	Part 4: Flow Control Structures	Wet Pond		14000	$120.00	$1,680,000.00	$504,000.00	$2,184,000.00
Allowance for utility replacement and service reconnection	Part 5: Miscellaneous allowance	15% of total pipe cost	-	-	-	$7,140.00	$2,142.00	$9,282.00
Allowance for catchbasin replacement and Lateral reconnection	Part 5: Miscellaneous allowance	7.5% of total pipe cost	-	-	-	$3,570.00	$1,071.00	$4,641.00
Allowance for equipment mobilization, demobilization, sediment testing, payment for bonds	Part 5: Miscellaneous allowance	30% of earth work cost				$504,000.00	$151,200.00	$655,200.00
							Total	$2,963,987.00

S6b Pond Expansion (North Pond Portion)

Item	Part	Size/type	Depth	Quantity	Unit Cost	Cost of implementation	Contingency	Totaling
Supply and Install 600 mm diameter, 0-3 m deep, storm sewers	Part 1 - Storm Sewers (unit m)	600 mm	0-3 m	9	$1,700.00	$15,300.00	$4,590.00	$19,890.00
Restoration of 0-3 m Deep Open Space	Part 2 Pavement Restoration (Unit m)	Open Space	0-3 m	-	-	$0.00	$0.00	$0.00
Supply and install new maintenance hole s and upsize the existing maintenance hole to the require sizes (minimal 600 mm more than the largest connected pipe)	Part 3: maintenance hole Installation and upgrade	30% of total pipe cost	-	-	-	$4,590.00	$1,377.00	$5,967.00
Supply and install pre-casted concrete overflow weirs	Part 4: Flow Control Structures	-	-	1	$7,800.00	$7,800.00	$2,340.00	$10,140.00
Earth work (Cut and Fill) (Unit m3)	Part 4: Flow Control Structures	Wet Pond		8000	$120.00	$960,000.00	$288,000.00	$1,248,000.00
Allowance for utility replacement and service reconnection	Part 5: Miscellaneous allowance	15% of total pipe cost	-	-	-	$2,295.00	$688.50	$2,983.50
Allowance for catchbasin replacement and Lateral reconnection	Part 5: Miscellaneous allowance	7.5% of total pipe cost	-	-	-	$1,147.50	$344.25	$1,491.75
Allowance for equipment mobilization, demobilization, sediment testing, payment for bonds	Part 5: Miscellaneous allowance	30% of earth work cost				$288,000.00	$86,400.00	$374,400.00
							Total	$1,662,872.25

S6c Pond Expansion (East Pond portion)

Item	Part	Size/type	Depth	Quantity	Unit Cost	Cost of implementation	Contingency	Totaling
Supply and Install 600 mm diameter, 0-3 m deep, storm sewers	Part 1 - Storm Sewers (unit m)	600 mm	0-3 m	19	$1,700.00	$32,300.00	$9,690.00	$41,990.00
Restoration of 0-3 m Deep Open Space	Part 2 Pavement Restoration (Unit m)	Open Space	0-3 m	-	-	$0.00	$0.00	$0.00
Supply and install new maintenance hole s and upsize the existing maintenance hole to the require sizes (minimal 600 mm more than the largest connected pipe)	Part 3: maintenance hole Installation and upgrade	30% of total pipe cost	-	-	-	$9,690.00	$2,907.00	$12,597.00
Supply and install pre-casted concrete overflow weirs	Part 4: Flow Control Structures	-	-	2	$7,800.00	$15,600.00	$4,680.00	$20,280.00
Earth work (Cut and Fill) (Unit m3)	Part 4: Flow Control Structures	Wet Pond		6000	$120.00	$720,000.00	$216,000.00	$936,000.00
Allowance for utility replacement and service reconnection	Part 5: Miscellaneous allowance	15% of total pipe cost	-	-	-	$4,845.00	$1,453.50	$6,298.50
Allowance for catchbasin replacement and Lateral reconnection	Part 5: Miscellaneous allowance	7.5% of total pipe cost	-	-	-	$2,422.50	$726.75	$3,149.25
Allowance for equipment mobilization, demobilization, sediment testing, payment for bonds	Part 5: Miscellaneous allowance	30% of earth work cost				$216,000.00	$64,800.00	$280,800.00
							Total	$1,301,114.75

S7a Combine Scenario S4 (Redirecting minor system flow from Swan Club OGS to the North Pond) with S5a (Rasing the weir)

Item	Part	Size/type	Depth	Quantity	Unit Cost	Cost of impleme ntation	Contingency	Totaling
Supply and Install 300 mm diameter, 0-3 deep, storm sewers	Part 1 - Storm Sewers (unit m)	300 mm	0-3	111	$1,250.00	$138,75 0.00	$41,625.00	$180,375.00
								$0.00
Pavement Restoration (unit m)	Part 2 Pavement Restoration (Unit m)	Open Space	0-3	0	$0.00	$0.00	$0.00	$0.00
								$0.00
Supply and install new maintenance hole s and upsize the existing maintenance hole to the require sizes (minimal 600 mm more than the largest connected pipe)	Part 3: maintenance hole Installation and upgrade	30% of total pipe cost	-	-	-	$41,625. 00	$12,487.50	$54,112.50
Supply and install pre-casted concrete overflow weirs	Part 4: Flow Control Structures	-	-	1	$7,800.00	$7,800.0 0	$2,340.00	$10,140.00
Allowance for utility replacement and service reconnection	Part 5: Miscellaneous allowance	15% of total pipe cost	-	-	-	$20,812. 50	$6,243.75	$27,056.25
Allowance for catchbasin replacement and Lateral reconnection	Part 5: Miscellaneous allowance	7.5% of total pipe cost	-	-	-	$10,406. 25	$3,121.88	$13,528.13
								$285,211.88

S7b Combine Scenario S4 (Redirecting minor system flow from Swan Club OGS to the North Pond) with S5b (Rasing the weir and upsizing the inlet pipe)

Item	Part	Size/type	Depth	Quantity	Unit Cost	Cost of implementation	Contingency	Totaling
Supply and Install 600 mm diameter, 0-3 m deep, storm sewers	Part 1 - Storm Sewers (unit m)	600 mm	0-3 m	9	$1,700.00	$15,300.00	$4,590.00	$19,890.00
Supply and Install 300 mm diameter, 0-3 deep, storm sewers	Part 1 - Storm Sewers (unit m)	300 mm	0-3	111	1250	138750	41625	180375
Restoration of 0-3 m Deep Open Space	Part 2 Pavement Restoration (Unit m)	Open Space	0-3 m	-	$0.00	$0.00	$0.00	$0.00
Supply and install new maintenance hole s and upsize the existing maintenance hole to the require sizes (minimal 60mm more than the largest connected pipe)	Part 3: maintenance hole Installation and upgrade	30% of total pipe cost	-	-	-	$46,215.00	$13,864.50	$60,079.50
Supply and install pre-casted concrete overflow weirs	Part 4: Flow Control Structures	-	-	1	$7,800.00	$7,800.00	$2,340.00	$10,140.00
Allowance for utility replacement and service reconnection	Part 5: Miscellaneous allowance	15% of total pipe cost	-	-	-	$23,107.50	$6,932.25	$30,039.75
Allowance for catchbasin replacement and Lateral reconnection	Part 5: Miscellaneous allowance	7.5% of total pipe cost	-	-	-	$11,553.75	$3,466.13	$15,019.88
								$315,544.13

S7c Combine Scenario S4 (Redirecting minor system flow from Swan Club OGS to the North Pond) with S6 (Pond Expansion)

Item	Part	Size/type	Depth	Quantity	Unit Cost	Cost of implementation	Contingency	Totaling
Supply and Install 600 mm diameter, 0-3 m deep, storm sewers	Part 1 - Storm Sewers (unit m)	600 mm	0-3 m	9	$1,700.00	$15,300.00	$4,590.00	$19,890.00
Supply and Install 300 mm diameter, 0-3 m deep, storm sewers	Part 1 - Storm Sewers (unit m)	300 mm	0-3 m	111	$1,250.00	$138,750.00	$41,625.00	$180,375.00
Restoration of 0-3 m Deep Open Space	Part 2 Pavement Restoration (Unit m)	Open Space	0-3 m	-	-	$0.00	$0.00	$0.00
Supply and install new maintenance hole s and upsize the existing maintenance hole to the require sizes (minimal 60mm more than the largest connected pipe)	Part 3: maintenance hole Installation and upgrade	30% of total pipe cost	-	-	-	$46,215.00	$13,864.50	$60,079.50
Supply and install pre-casted concrete overflow weirs	Part 4: Flow Control Structures	-	-	1	$7,800.00	$7,800.00	$2,340.00	$10,140.00
Earth work (Cut and Fill) (Unit m3)	Part 4: Flow Control Structures	-		8000	$120.00	$960,000.00	$288,000.00	$1,248,000.00
Allowance for utility replacement and service reconnection	Part 5: Miscellaneous allowance	15% of total pipe cost	-	-	-	$2,295.00	$688.50	$2,983.50
Allowance for catchbasin replacement and Lateral reconnection	Part 5: Miscellaneous allowance	7.5% of total pipe cost	-	-	-	$1,147.50	$344.25	$1,491.75
Allowance for equipment mobilization, demobilization, sediment testing, payment for bonds	Part 5: Miscellaneous allowance	30% of total earth work cost				$288,000.00	$86,400.00	$374,400.00
							Total	$1,897,359.75

S8 Creating underground storage capacity

Item	Part	Size/type	Depth	Quantify	Unit Cost	Cost of implementation	Contingency (30%)	Totaling
Supply and install 375 mm (Concrete) diameter, 0-3 m deep, storm sewers	Part 1 - Storm Sewers (unit m)	375 mm (Concrete)	0-3 m	33 m	$1,230.00	$40,590.00	$12,177.00	$52,767.00
Supply and install 450 mm diameter, 0-3 m deep, storm sewers	Part 1 - Storm Sewers (unit m)	450 mm	0-3 m	113 m	$1,270.00	$143,510.00	$43,053.00	$186,563.00
Supply and install 450 mm diameter, 3-4.5m deep, storm sewers	Part 1 - Storm Sewers (unit m)	450 mm	3-4.5m	107 m	$1,550.00	$165,850.00	$49,755.00	$215,605.00
Supply and install 450 mm diameter, 4.5 - 6m deep, storm sewers	Part 1 - Storm Sewers (unit m)	450 mm	4.5 – 6 m	36 m	$1,900.00	$68,400.00	$20,520.00	$88,920.00
Supply and install 2400 mm X 1200 mm diameter, 4.5 - 6m deep, storm sewers	Part 1 - Storm Sewers (unit m)	2400 mm X 1200 mm	4.5 – 6 m	207 m	$9,500.00	$1,966,500.00	$589,950.00	$2,556,450.00
Restoration of 0-3 m Deep Urban Roads	Part 2 Pavement Restoration (Unit m)	Urban Roads	0-3 m	146 m	$1,700.00	$248,200.00	$74,460.00	$322,660.00
Restoration of 3-4.5m Deep Urban Roads	Part 2 Pavement Restoration (Unit m)	Urban Roads	3-4.5m	107 m	$1,955.00	$209,185.00	$62,755.50	$271,940.50
Restoration of 4.5*6m Deep Urban Roads	Part 2 Pavement Restoration (Unit m)	Urban Roads	4.5-6 m	243 m	$2,450.00	$595,350.00	$178,605.00	$773,955.00
Supply and install new maintenance holes and upsize existing maintenance holes to required sizes (minimal 600 mm larger than the largest connected pipe)	Part 3: Maintenance hole Installation and upgrade	30% of total pipe cost	-	-	-	$715,455.00	$214,636.50	$930,091.50
Supply and install orifice plate	Part 4: Flow Control Structures	-	-	1	$1,500.00	$1,500.00	$450.00	$1,950.00
Allowance for utility replacement and service reconnection	Part 5: Miscellaneous allowance	15% of total pipe cost	-	-	-	$357,727.50	$107,318.25	$465,045.75
Allowance for catchbasin replacement and Lateral reconnection	Part 5: Miscellaneous allowance	7.5% of total pipe cost	-	-	-	$178,863.75	$53,659.13	$232,522.88
								$6,098,470.63

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