



Final

City of Hamilton

Interim Water, Wastewater and Stormwater Master Servicing Strategy

Interim Stormwater Master Servicing Strategy

2026-07-10

WSP Project Number: CA-EI-TPB17090



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City of Hamilton

Project No.: CA-EI-TPB178090

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- 2 Approximate Development Plan (2007-2022)
- 3 Existing Stormwater Management Facilities Plan
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- 5a Existing Storm Sewer Plan
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- A** Policy and Guidelines Recommendations (May 2020)
- B** Six Cities Results Maps
- C** Dundas Results Maps
- D** Stoney Creek Results Maps
- E** Ancaster Results Maps
- F** SWM Pond Summary Table
- G** Overland Flow Analysis Summaries Recommended
- H** Upgrades and Capital Cost Estimates

1 INTRODUCTION AND SCOPE

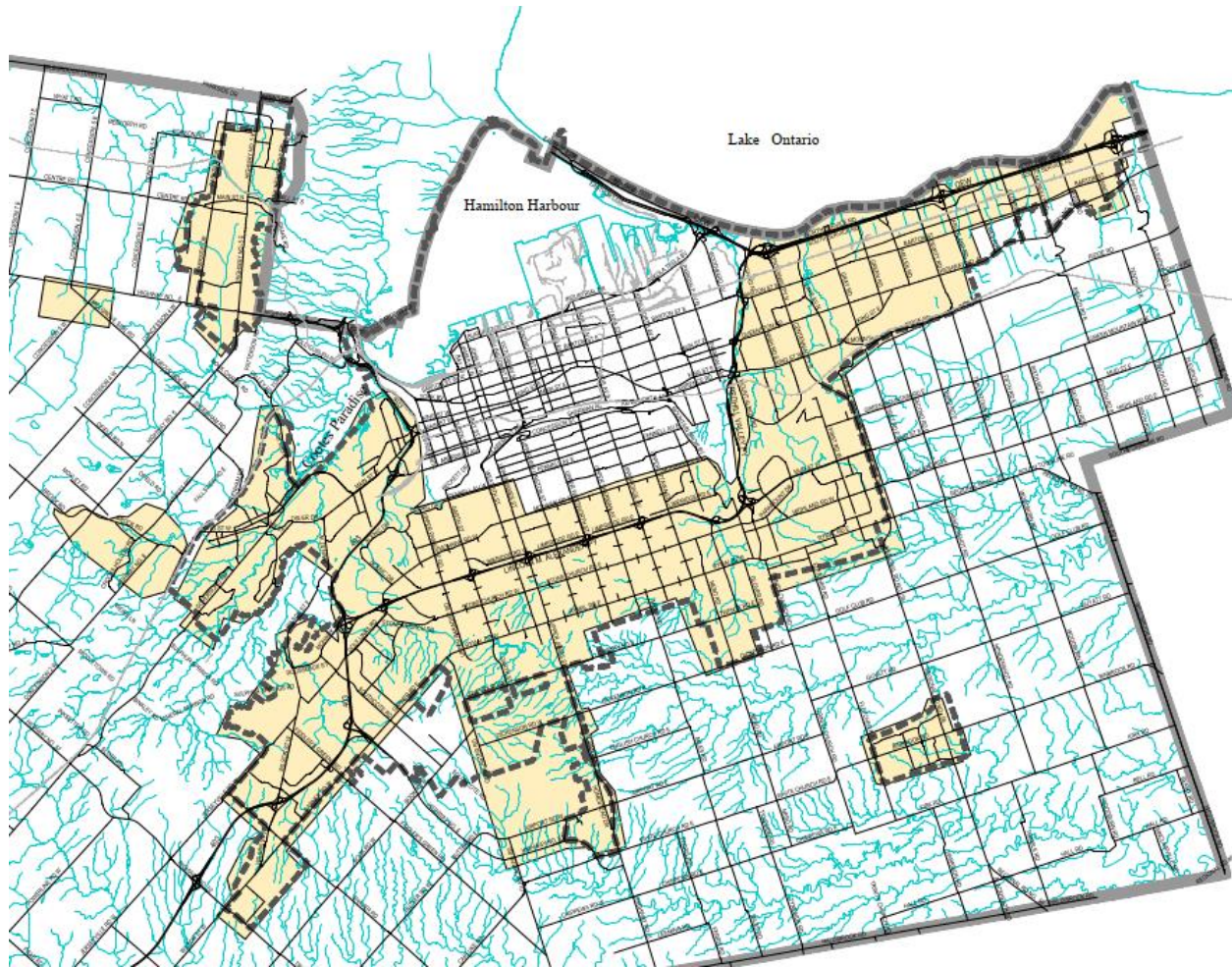
WSP E&I Canada Limited (WSP) has been retained by the City of Hamilton as part of an overall team to prepare integrated Interim Water, Wastewater, and Stormwater Interim Master Servicing Strategies (the “Study” and/or “IMSS”). The team includes GEI (overall consultant team lead and lead for water and wastewater MSS), DHI (hydrologic \ hydraulic modelling support for the stormwater IMSS) and Scheckenberger & Associates (project advisory services for the stormwater IMSS).

The City of Hamilton is undertaking this study as part of a strategic, comprehensive, and integrated growth planning and infrastructure planning initiative. This integrated process provides the City with a basis to make sound decisions to shape infrastructure planning to support Hamilton’s future growth. The City of Hamilton is building on a progressive process originally established and completed in 2006 when the City’s Infrastructure Master Plans were integrated under GRIDS (Growth Related Integrated Development Strategy) to provide better planning and engineering decisions. The City’s current GRIDS 2 update project builds upon previous work to determine the growth targets to 2051.

Notably, the current Study forms part of a comprehensive rescope of the original project work which was initiated in 2017. The rescope has been driven by the 2024 No Urban Expansion (NoUBE) Traffic Survey Zone (TSZ) data which provide details on intensification plans across the City and is thereby essential for addressing the City’s infrastructure needs up to 2051. The scope change, supported by this new data, has adjusted the overall City of Hamilton Master Planning process, which was originally planned to follow the formal MEA Class EA process. The modified study represents a “Streamlined” Technical Strategy Project also known as the Interim Water, Wastewater and Stormwater Master Servicing Strategy (or IMSS). The IMSS will produce similar final outcomes in terms of Servicing Strategies and Capital Program; however, it is being completed outside of the MEA Class EA Process due to the City’s updated project timelines and requirements. The study is considered Interim as it is expected that it may be updated at a later date into a more formal Master Plan process but is being advanced now to meet the City’s current requirements.

The study area for the Interim Stormwater Master Servicing Study is specific to those areas within the City’s urban boundary limits which are serviced by a separated storm sewer system rather than the combined sewer system; the limits are presented in **Figure 2.1.1** (ref. **Drawing 1** for full extents).

Figure 2.1.1: Separated Storm Sewer Area (Study Area)



2 BACKGROUND REVIEW

2.1 City of Hamilton Stormwater Master Plan (2007)

2.1.1 Summary

The City of Hamilton completed a Stormwater Master Plan (SWMP) in 2007 (City of Hamilton Stormwater Master Plan – Class Environmental Assessment Report (City-Wide), Aquafor Beech Limited, May 2007). The current update study builds upon the work completed as part of this study and continue the work initiated as part of the original 2017 master plan update scope. As such, a comprehensive review of this previous work is considered key. The project scope area is generally common with the 2007 SWMP, namely the urban portion of the City of Hamilton, specifically those areas which are serviced by a separated storm sewer system. **Drawing 1** presents the associated study areas limits corresponding to the separated system.

It should be noted that the 2007 SWMP also included the entire City of Hamilton as a secondary study area, with respect to the assessment of watersheds, creeks and receiving bodies of water for consideration of existing environmental conditions or environmental impacts. Based on the Terms of Reference for the current IMSS study, this secondary study area will not be re-assessed unless necessary based on expected stormwater-related re-development impacts. As such, the focus area of the current update study is outlined on **Drawing 1**.

The 2007 SWMP included a number of different components. A brief summary of the key undertakings, findings, and recommendations is provided as follows:

Characterization and Assessment

- a) A general characterization and summary of existing conditions (topography, surficial soils and geology, groundwater, surficial watersheds, aquatic and terrestrial resources, degree of urbanization, etcetera), including development of watershed “fact sheets”
- b) A summary of available watercourse-based water quality data
- c) Development of hydrologic / hydraulic models of trunk storm sewers (generally $\geq$ 525 mm diameter) in MOUSE (MIKE URBAN)
 - a) Corresponding assessment of existing trunk storm sewer flow conveyance capacity \ level of service (no assessment of overland flow \ dual drainage in any models at that time)
 - b) Incorporation of stormwater management facilities based on available data

- d) Development of a City-wide spreadsheet-based water balance/water quality mass balance model, with the following general steps:
- Each watershed (15 in City) modeled as a single catchment
 - Determine typical contaminant loadings (event mean concentrations) for urban and rural land uses for dry and wet weather conditions
 - Use daily flow measurements or prorating to estimate dry / wet flows
 - Daily time step, simulation for 1989 - 2003 period using observed daily rainfall

Preferred Stormwater Management Strategy

- e) Summary of five different growth options from GRIDS
- a) Evaluation and ranking based on stormwater criteria (“triple bottom line” of social / economic / environmental)
- f) Consideration of qualitative impacts to environmental features (water quality, groundwater, terrestrial / aquatic habitat)
- g) Development of a general preferred stormwater management strategy for the preferred GRIDS option (including mass balance / water quality spreadsheet model assessment), with differing strategies presented for individual subwatersheds depending on the expected development impact including:
- a) Scenario 1 - Do Nothing
- b) Scenario 2 - Business as usual (standard SWM)
- c) Scenario 3 - Comprehensive Urbanization Approach (additional source and conveyance controls)
- d) Scenario 4 - Business as Usual with Urban Retrofits (additional source and conveyance controls)
- e) Scenario 5 - Business as Usual with Rural Retrofits (additional source and conveyance controls)
- h) Typically, Scenario 3, 4 or 5 was recommended based on the results from the spreadsheet- based mass balance model
- i) An evaluation of detailed alternatives was undertaken, and summary costs were estimated for the overall proposed measures
- a) Source Controls for existing urban areas (\$35 M)
- b) Conveyance Controls for Existing Urban Areas, largely implemented into roadway reconstruction programs (\$32 M)
- c) Retrofitting of 29 priority existing (2007) SWM facilities (\$19 M) – based on a general triple bottom line evaluation (environmental / social / economic benefits)

- d) Prioritization of 53 stream restoration projects (\$22 M) – based on a similar triple bottom line type evaluation, and including information from the 2006 Development Charges Study
- e) Storm Sewer upgrades to offset intensification – only required if source / conveyance controls are not implemented (\$50 to \$ 100 M based on replacing between 5 to 10 percent of trunk storm sewers at \$3,000/m)
- f) Rural Stewardship Programs (\$40 M, assuming all costs are borne by the City, \$20 M assuming 50% cost sharing with rural landowners)

Implementation

- j) Recommended development of working groups of affected stakeholders to advance the preferred strategy, including modifications to policy and criteria as required
- k) Consideration of alternative funding sources to fund proposed projects

2.1.2 Review

The 2007 SWMP is extensive and provides a large amount of information related to the City's separated stormwater system, including area watercourses. Notwithstanding, based on WSP's review of the report, and experience with its use in the application to various stormwater and water resources projects in the City of Hamilton, there are a number of components of the SWMP which bear further consideration, or could be improved upon as part of the current update process. Note that issues specific to the hydrologic / hydraulic modelling (MIKE URBAN) are discussed separately in Section 4 of this TM.

- a) Although there is an extensive review of background information and data, there is no summary of existing flood-prone or at-risk areas. Typically, this information is key to prioritizing works for existing areas. This information can also be used to informally verify or assess modelling results with actual reported flooding. It is unclear if this information was not available from the City, or not included due to privacy concerns, or not included for some other reason.
- b) The modelling approach used for the assessment of water quality (spreadsheet-based mass balance model) is overly simplified and considered of limited utility to guide retrofits and SWM planning. While it does provide a general assessment at a high level, its utility in making more resolute decisions related to stormwater quality controls is limited. It is unknown whether this model has ever been used by City staff or its external consultants since its development, however this appears unlikely. Overall, there may be greater value in assessing water quality as part of any future updates to hydrologic \ hydraulic models for the City, to ensure a consistent approach.

- c) In general, based on WSP's experience, it appears that few of the recommendations provided in the SWMP are being followed by the City. As discussed in subsequent sections of this technical memorandum, little to no SWM facility retrofits or stream rehabilitation projects have been considered or undertaken. Further, the fundamental recommendations of the SWMP, which related to a shift in focus towards low impact development best management practices (LID BMPs, i.e. source or conveyance controls) also appear not to have been followed given the lack of any record of such installations in the City's database, and the number of developments and road reconstructions which have occurred since the completion of the 2007 SWMP, without these measures. The reason for this is unknown and could potentially involve a number of factors, including various technical constraints or issues with funding and implementation, policy, or other factors. Ultimately however, this suggests a disconnect in stormwater management policy and direction for the City.
- d) While individual projects with respect to stormwater management facility retrofits and stream restoration works are ranked and prioritized, there is no such ranking for the implementation of other recommended measures, or an overall prioritization for all projects combined. Such a prioritization or ranking system would likely better assist City staff in planning the works proposed in the SWMP. The lack of this prioritization may also contribute to many of the proposed measures not being implemented to date, given the inherent uncertainty in the need and benefit of implementing the subject works. In general, the 2007 SWMP does not provide sufficient specifics on project prioritization and direction.
- e) While the SWMP recommends a general / high-level shift towards more LID BMPs for quantity control management, it does not directly address a typical concern with respect to these approaches in that they are most suitable for more frequent, less formative storm events, but of more limited benefit for less frequent, more formative storm events. This is particularly the case in the City of Hamilton, which has an abundance of relatively low permeability surficial soils (i.e. clays), which functionally limits the infiltration ability of LID BMPs. In general, considerations of quantity control for more formative storm events are not addressed quantitatively in the 2007 SWMP.
- f) There is limited documentation on the methodology used for the assessment of potential storm sewer upgrades to accommodate future development, which were noted to have been assessed in the text (i.e., suggested to have been used to develop the estimated capital cost of \$50 to \$100 M). While it is noted that the preferred solution in the SWMP involves an increased focus on LID BMPs over storm sewer upgrades, mapping or a summary of this work would be useful to assess differences in upgrade requirements across the City, or locations where a combined approach (source / conveyance controls with selected infrastructure

upgrades) may be the most logical solution, which based on experience is often the case.

- g) There is no discussion with respect to long term maintenance requirements related to the separated stormwater system. Operations and maintenance are key components of the overall management of a stormwater system, including CCTV inspection and condition tracking of storm sewers, flushing and repair of sewers, catchbasin and OGS cleanouts, SWM facility inspections and cleanouts, etcetera. Furthermore, there is no dialogue on the aspect of future operations and maintenance for the LID BMPs recommended in the plan.
- h) In conjunction with operations and maintenance, there is no discussion related to the requirements for ongoing field monitoring work, including inspections, water level and flow monitoring, and water quality sampling to support enhanced assessments and programs in the future based on ground-truthed performance data.
- i) Funding of the overall preferred solution is a key consideration. While there is a general discussion within the 2007 SWMP, it is more general and high-level in nature. A more fulsome discussion is considered warranted, including reference to the City's Development Charges program, as well as operations and maintenance works. Ultimately without funding, there is likely to be little progress on the recommended solutions.

2.2 Background Reports

2.2.1 City Documents

2.2.1.1 General City Documents

I) Hamilton Harbour Remedial Action Plan (Stage 1 and 2 Reports – 1992, and Stage 2 Update – 2002)

- a) Numerous studies have been undertaken for Hamilton Harbour, primarily focusing on remediation activities to have the harbour delisted as one of the “Areas of Concern” around the Great Lakes. To specifically address this, the Hamilton Harbour Remedial Action Plan (1992) was prepared. Both Stage 1 (Environmental Conditions and Problem Definition) and Stage 2 (Goals, Options and Recommendations) reports were prepared. An update to the Stage 2 report was also prepared in 2002.
- b) The 2002 report has a focus on combined sewer overflows and sets a first priority recommendation that the City eliminate or minimize CSO discharges (which is beyond the scope of the current separated Stormwater Master Plan). However, of note is the 1992 report (Stage 2) which states the overall principle of zero discharge, namely, that “The philosophy be adopted whereas control of

inputs of persistent toxic substances shall eventuate to zero discharge”. No specific targets related to development sites are evident in the reporting; specific targets are primarily focused on WWTP effluent and industry sources.

m) Stormwater Quality Management Strategy, Community of Stoney Creek (Philips Engineering, 2004)

- a) The Stormwater Quality Management Strategy was developed to address concerns relating to the aquatic habitat quality of natural systems within the former City of Stoney Creek. The objective of the document was to establish a Master Plan capable of maintaining or enhancing existing stream system water quality, ensuring no net loss of habitat caused by growth, and ensuring compliance with all applicable legislation, mandates and policies. The document outlined a preferred solution and implementation strategy to address the study objectives. A short list of preferred alternatives was developed, consisting of source and conveyance controls, management practices, indirect opportunities, financial contributions, retrofit opportunities, and new stormwater facilities.

n) Storm Drainage Policy (Philips Engineering, May 2004)

- a) The Storm Drainage Policy was developed by the City of Hamilton to provide planning and design process guidance for land development and storm drainage infrastructure within the Municipality. The intent of the policy document is to outline the storm drainage policy to be applied within the City, specify storm drainage requirements for land development and Capital Works projects, specify requirements for storm drainage design and reporting, and provide reference/context to federal, provincial, and municipal policies and regulations to be considered for storm drainage systems. The policy document is supported by the Criteria and Guidelines for Stormwater Infrastructure Design (2007) and also superseded by the Comprehensive Development Guidelines and Financial Policies Manual (2019) described below.

o) Stormwater Master Plan (Aquafor Beech Limited, May 2007)

- a) The Stormwater Master Plan is a fundamental document for the City of Hamilton and is also the basis for the current update study. This document was reviewed in greater detail previously as part of Technical Memorandum #2 (Stormwater Systems Review and Capacity Verification). In general, the Stormwater Master Plan (completed for the separated storm sewer system area) provides an inventory and characterization of existing stormwater systems, assesses system deficiencies as well as the impact of future planned growth, and makes recommendations for drainage system improvements as well as setting the general strategy (a large focus on source and conveyance controls; low impact development best management practices). As part of this study, a spreadsheet-

based water quality model (mass balance approach lumped by watershed), was developed, along with a MOUSE / MIKE Urban model for trunk storm sewers.

p) Criteria and Guidelines for Stormwater Infrastructure Design (Philips Engineering, September 2007)

- a) The City of Hamilton prepared the Criteria and Guidelines document for use in the design of storm drainage infrastructure within the Municipality. The document provides specific direction on how the City requires stormwater infrastructure to be designed and implemented by proponents, including design requirements for the minor and major conveyance systems, stormwater quality and quantity controls, analytical methods of hydrologic and hydraulic modelling, and several other elements. The document has been superseded by the Comprehensive Development Guidelines and Financial Policies Manual (2019) described below.

q) Landscape Design Guidelines for SWM Facilities (City of Hamilton, May 2009)

- a) The document was developed by the City of Hamilton as a guideline to siting and landscaping SWM Facilities, with the goal of integrating the SWM Facilities as aesthetic amenities. SWM facilities covered by the guidelines consist of end-of-pipe controls (i.e. SWM Ponds), at-source controls (i.e. bioretention, infiltration, filtration), and conveyance controls (i.e. grassed swales). The document provides guidance on the design, approval, and maintenance / monitoring process requirements of the City.

r) Engineering Guidelines for Servicing Land Under Development Applications (City of Hamilton, December 2012)

- a) The document is a guideline to assist all proponents through the land development process for sites within the City of Hamilton. The document outlines the engineering submission requirements to support a development application. The document has been superseded by the Comprehensive Development Guidelines and Financial Policies Manual (2019) described below.

s) Innovative Stormwater Source Control Policy for Industrial, Commercial and Institutional Land Uses (AMEC Environment & Infrastructure, April 2013)

- a) The document provides guidance for source controls to be implemented into the development / re-development of industrial, commercial and institutional land types. The document aims to provide guidance on the selection of source controls that provide water quantity, quality, and erosion controls beyond the current Provincial and Federal legislative requirements.

t) Hamilton IDF Assessment (Cole Engineering, October 2015)

- a) This project was intended to review the current IDF Curves (i.e. Mount Hope), and to determine whether that IDF standard was still appropriate given recent major rainfall events in the City. The study concluded that the current IDF parameters generate more rainfall than those of most adjacent municipalities, and that they also did not change notably when an appended data set (including data beyond 2003) was included. As such, no changes to the current IDF were recommended, although ongoing monitoring and review was recommended.

u) Comprehensive Development Guidelines and Financial Policies Manual (City of Hamilton, 2019)

- a) The Comprehensive Development Guidelines document amalgamates various development guideline documents pertaining to subdivision planning, site plans, infrastructure design (i.e. roads, watermain, sanitary / storm sewers, stormwater management, etc.), and financial policies pertaining to cost sharing and cost recovery. The guidelines supersede several of the previous documents as noted in the foregoing.

v) Climate Change Impact Adaptation Plan (City of Hamilton, 2022)

- a) The plan is described as an evidence based, action-oriented plan to help minimize the impacts of climate change on Hamilton residents, businesses and infrastructure. The plan was created with ICLEI Canada through their Building Adaptive and Resilient Communities (BARC) framework.
- b) Relevant key objectives include incorporating climate change into future land use, development and construction, reducing transportation disruption and improve the safety of travel on roads, and proactively conserving and protecting surface water and groundwater resources.

w) Complete Streets Design Manual (WSP, June 2022)

- a) The manual provides a set of consistent guidelines and tools to inform the design, implementation, maintenance and monitoring of complete streets. Complete streets are roadways that are planned and designed to balance the needs of all road users.
- b) The guidelines include prioritizing Low Impact Stormwater Management features as part of its design principles

x) Intensity-Duration-Frequency Curve Update and Climate Change Considerations (Matrix Solutions Inc, July 2022)

- a) The report was completed to review and recommend updates (if necessary) to the City's existing IDF curves in consideration of recent climate records and future climate projections.
- b) Report recommendations included that the City:

- Continue using the current IDF curves (Philips Engineering published in September 2007)
- Continue rainfall monitoring network and implement a system to manage climate data and introduce QA/QC processes
- Repeat the process to update the design IDF curves every 5 years, consistent with the Canadian Standards Association (CSA) technical guide (CSA Group 2019). The significance of winter rainfall should also be considered in future IDF curve updates.
- Not update its design IDF curves to account for the potential effects of climate change, as they are generally within the range of the future ensemble and nearly always greater than the median of the ensemble.
- Repeat the process of computing future IDF curves using its best available future climate data every 5 years.
- Introduce risk-based climate resilience assessment into its stormwater or flood related projects to assess the ability for new or existing infrastructure to withstand extreme rainfall events that are greater than that reflected by the IDF curves.

y) Stormwater and Drainage Asset Guidance Document (WSP, January 2024)

- a) This document was generated to provide Hamilton Water Stormwater and Drainage with a guidance document to outline the operational and maintenance needs of the assets under the purview of Hamilton Water (including SWM facilities, Low Impact Development, oil / grit separators, watercourses, shorelines, municipal drains and drainage easements).
- b) The guideline specifies technical requirements for maintenance as well as frequencies for inspections.

z) City of Hamilton – 2024 Development Charges Update – Stormwater Background Study (WSP, March 2024)

- a) This report was part of the overall review of development charges in the City and reflected updates that had occurred in the 2019-2024 period. Updated projects and cost estimates were included. The 2024 DC Update made a distinction between including GRIDS (Growth Related Integrated Strategy) and excluding GRIDS in the overall totals. The overall total excluding GRIDS identified \$478M for stormwater projects, with \$287M estimated to be allocated to new development, while the overall total including GRIDS identified \$889M for stormwater projects, with \$315M estimated to be allocated to new development.

aa) Green Standards and Guidelines (GSG) for Site Servicing (City of Hamilton, 2024)

- a) The City of Hamilton (the City) has, under the guidance of Council, prepared City-wide Green Standards and Guidelines (GSG) to guide private development applications.
- b) The GSG creates a guideline and approach that tailors to the specific needs and conditions within the City, the applicable watershed and subwatersheds, and area-specific stormwater management (SWM) criteria.
- c) The GSG provides developers with a decision methodology and implementation considerations to guide private development applications. This decision methodology / matrix allows development proponents to systematically evaluate their development applications to identify best management practice options and onsite control requirements.
- d) The City is separately reviewing the approach to establishing stormwater management requirements for public works / lands, and this is expected to be a future complement to these GSG.

bb) Stormwater Fee Guidance Manual (City of Hamilton, April 2025)

- a) The document provides general information and assistance associated with the City's stormwater fee (to commence in April 2026) whereby owners are charged based on the load (i.e. imperviousness) they place on the stormwater system. The document outlines incentives and credits that property owners may obtain from on-site stormwater management measures.

2.2.1.2 Area-Specific City Documents

cc) City of Hamilton Watercourse Erosion Assessment Study (Aquafor Beech Limited, August 30, 2011)

- a) This study was intended to build off the background work and prioritization of stream reaches in the 2007 SWMP, by undertaking an inventory of watercourses and erosion sites, and then developing a subsequent prioritization for restoration works and alternative measures.

dd) West Central Mountain Drainage Assessment (AMEC Environment & Infrastructure, October 2011)

- a) The West Central Mountain Drainage Assessment was completed to determine appropriate stormwater management requirements for the Mewburn and Sheldon Neighbourhood (generally located between Garth Street and Upper James Street between Rymal Road and Stone Church Road) that would protect the receiving drainage system (storm sewer system in particular). The drainage assessment was completed subsequent to the Mewburn Sheldon Neighbourhood MSP Class EA (SNC Lavalin, 2004) due to issues related to the capacity of the surrounding receiving drainage system, as well as land ownership and impervious coverage

associated with development. The drainage assessment provided recommendations on SWM Facility locations and corresponding rating curves (storage-discharge).

ee) Watercourse Erosion Investigation (Hamilton Conservation Authority, 2016)

- a) This report summarizes the results of a visual monitoring program for 22 watercourse sites in the City Hamilton (as identified based on previous studies). Based on the photo monitoring conducted in 2016 and a comparison to results from previous documentation (largely 2011), a prioritized list of sites for remediation works and further monitoring were identified.

ff) Riverdale East Flood Investigation – Technical Memo #2 – Additional Flow Monitoring, Model Calibration and Flooding Verification (Stantec Consulting Ltd., February 24, 2017)

- a) This memo is specific to the Riverdale East neighbourhood in lower Stoney Creek, which has experienced extensive basement flooding in the past decade (nearly 500 reports). Technical Memo #2 in particular summarizes the additional rainfall and flow monitoring work completed, as well as subsequent model calibration and assessment (MIKE URBAN). The modelling however appears to have been completed for the sanitary sewer system only, and does not appear to include the separate storm sewer system.

gg) Airport Employment Growth District – AEGD Subwatershed Study & Stormwater Master Plan (SWMP) – Implementation Document (Aquafor Beech Limited, April 2017)

- a) This document is intended as an implementation plan for subwatershed strategies and stormwater targets for the AEGD around John C. Munro International Airport, which were established as part of the previous 2011 Subwatershed and Stormwater Master Plan Studies. This document considers policy and regulation changes that occurred following the completion of the 2011 studies, as well as the Ontario Municipal Board (OMB) decision of 2015 related to the AEGD Secondary Plan.

2.2.2 Provincial and Conservation Authority Documents

In addition to the preceding City-wide and area specific studies, there are a number of other guidelines by Provincial Agencies / Ministries and Conservation Authorities which are relevant in the review of stormwater policy for the City of Hamilton. These documents are outlined briefly in this section.

hh) Policies Guidelines Provincial Water Quality Objectives (Ministry of Environment and Energy, 1994)

- a) The Provincial Water Quality Objectives (PWQOs) set general targets for water quality parameters for surface water systems to satisfy the needs of aquatic life and recreational uses. Targets are not however provided for all water quality parameters. The targets are generalized based on research and may be superseded by more applicable site-specific data if available. In general, however, PWQOs are the primary basis for water quality comparatives in Ontario.

ii) Drainage Management Manual (Ministry of Transportation, 1997)

- a) The Drainage Management Manual was developed to assist in both planning and design of highway drainage. The manual provides design objectives / criteria for the hydraulic design of all major components of highway drainage (i.e. culverts, bridges, ditches, stormwater ponds, sewers, etc.). Design tools provided within the manual are commonly referenced and used throughout the design aspects of the land development process.

jj) Technical Guide - River & Stream Systems: Flooding Hazard Limit (Ministry of Natural Resources, 2002)

- a) This document outlines Provincial policies with respect to flooding hazards, including design standards, and requirements for hydrologic and hydraulic analyses. This guideline is a key document applied by Conservation Authorities in the development of regulated area mapping, and also in the management of Regional Storm flows. It is understood that MNRF is currently updating this document.

kk) Stormwater Management Planning and Design Manual (Ministry of the Environment, 2003)

- a) The Stormwater Management Planning and Design Manual has been the primary document used by stormwater practitioners in Ontario. The document provides an overview of stormwater management measures, as well as design criteria and parameters to be used, including quality control treatment based on the protection level required, as well as specific criteria and considerations for measures such as dry and wet ponds, as well as source and conveyance control measures. Guidance and forecasting data on operations and maintenance are also provided.

II) Highway Drainage Design Standards (Ministry of Transportation, 2008)

- a) The Highway Drainage Design Standards document is a manual containing standards for use in the design and rehabilitation of highway drainage infrastructure under the jurisdiction of the MTO. The document was developed / intended to be used as a tool to assist designers in all hydrologic and hydraulic

aspects of highway drainage design, including highway surface drainage, water crossings, stormwater management, and temporary works during construction.

mm) Low Impact Development Stormwater Management Planning and Design Guide (TRCA / CVC, 2010)

- a) The Low Impact Development (LID) guidelines were developed to assist proponents with incorporating LID technologies within their development proposals. The document provides guidance on the understanding, planning, selection, design, construction and monitoring of a large number of lot-level and conveyance type LID practices. The goal of the document is to continue the evolution of watershed management, which is critical in maintaining the health of watercourses and water bodies.

nn) Water Opportunities Act (Province of Ontario, 2010)

- a) The Water Opportunities Act was developed by the Province of Ontario to establish the plans and objectives of municipal water sustainability and public sector requirements relating to water conservation within the province. The Act was developed with the intention to conserve and sustain water resources for present and future generations, foster and innovate water, wastewater and stormwater technologies, services and practices in the private and public sectors, and create opportunities for economic development and clean-technology jobs in Ontario.

oo) Interpretation Bulletin, Ontario Ministry of Environment and Climate Change, Expectations re: Stormwater Management (MOECC, 2015)

- a) The Interpretation Bulletin was prepared to clarify the Ministry's expectations regarding stormwater management. The document provides justification for the implementation of source controls (i.e. Low-Impact Development techniques) to assist in achieving the Ministry's stormwater management expectation of mimicking the natural (pre-development) hydrology of development sites. The Interpretation Bulletin was released in advance of the planned update to the Ministry's expected update to the 2003 SWM Planning and Design Manual, which is currently in draft form (summarized further within this section).

pp) Approaches to Manage Regulatory Event Flow Increases Resulting from Urban Development (TRCA, June 2016)

- a) This document was developed to address the increasing risk of Regulatory Event flooding in existing and planned urban areas, in advance of pending direction from the MNRF. The document provides guidance on methods to manage Regulatory flooding at both a policy and planning level. Recommendations for the planning level include on-line and off-line stormwater control facilities sized to control Regulatory Flood flows to levels occurring prior to urbanization.

qq) Low Impact Development Stormwater Management Guidance Manual – Draft for Consultation (Ministry of the Environment, January 2022)

- a) This document was prepared as a supplement to the 2003 SWM Planning and Design Manual and focuses on the guidance of the design and implementation of Low Impact Development (LID) measures for source control.
- b) A key component of the LID Guidance manual is the proposed runoff volume control target. The 90th percentile precipitation event is proposed based on local rainfall patterns (for Hamilton this would be approximately 28 mm). Management of the 90th percentile rainfall is then required through a hierarchical approach for on site measures, generally retention, then filtration, then conventional end of pipe measures. This hierarchal approach has also been adopted in the City's GSG.
- c) The document remains in draft form; it was posted on the environmental registry for comment between January 27 and April 27, 2022. As per the most recent update (July 29, 2025) it is noted that the ministry is reviewing comments, and it remains under active review.
- d) Notwithstanding, many municipal Consolidated Linear Infrastructure Environmental Compliance Approvals (CLI-ECAs) include explicit reference to the 90th percentile runoff volume control target, despite the guidelines remaining in draft.
- e) It is understood that the City is currently preparing a CLI-ECA agreement with the MECP, but that this has not yet been finalized. As such, this criterion does not yet apply, in addition to the preceding consideration of the draft status of the LID guidelines.

2.3 Additional Information Reviewed

The additional information and data listed below have been reviewed in the preparation of this technical memorandum. In 2022, the City provided updated GIS data (from the original 2017 data) that has been incorporated into the current analyses.

Mapping and Base Data

rr) Storm Sewers

ss) Catchbasins

tt) Maintenance Holes

uu) Separated Storm Sewer Drainage Areas

vv) Stormwater Management Facilities

ww) Public Works Capital Projects Listing (Right-of-way, Parks, Bridges)

xx) Sewer Flooding Date Summary (by Parcel)

yy) Sewer Backflow Valve Mapping

Meteorology and Monitoring Data

zz) Rainfall gauge data and summaries from Calder gauges – 2004 onwards

aaa) Rainfall gauge data from multiple stations – 2015 to 2017

bbb) Stormwater Pumping Station Flow Data (HSS01 – Grafton and HSS02 – Centennial) – 2015 to 2017

Hydrologic and Hydraulic Modelling

ccc) MIKE URBAN Modelling (Six Cities Model and Dundas Model)

2.4 Stormwater Drainage Policy Review

2.4.1 Quantity Control Policies

2.4.1.1 Flood Management and Watercourse Erosion Control

The current storm drainage policy requires proponents to provide flood management and erosion control in accordance with the specified level of control outlined within a Subwatershed Study (SWS) or Master Drainage Plan (MDP) that encompasses the proponent's site. Where a SWS or MDP does not exist, the proponent is required to consult with the City and Agencies regarding the acceptable level of runoff controls to be applied to the proponent's site. For flood and erosion control, the proponent is required to mitigate any potential flood and erosion impacts in accordance with Provincial guidelines. Typically, this results in a standard post-development to pre-development peak flow control for flooding, and standard extended detention control (detention of the 25 mm storm event over 24 – 48 hours) for erosion. Notably however, while the standard approach is simple and direct, it may result in unforeseen issues in the receiving watercourse system(s), particularly if those systems are sensitive to changes in runoff volume and duration of elevated flows.

Post-development to pre-development peak flow control is typically mandated for the 2 through 100-year return periods. As per current City policy, quantity controls are not required for the Regional Storm (Hurricane Hazel). This is consistent with the approved direction of the MNRF (2002), which does not credit SWM facilities in the establishment of Regional Storm impacts. It has been previously assumed that due to saturated soils under the Regional Storm, no significant flow increases would be expected from development (i.e. impervious surfaces would respond generally the same as saturated pervious areas). This approach has been increasingly questioned by practitioners and Provincial Conservation Authorities, as documented in the TRCA's 2016 summary. It is

understood that the MNRF is currently undertaking an update of its 2002 criteria and that this update will further consider this issue. In the interim however, the City may wish to consider referring to the TRCA document in its guidelines, including increasing the capacity of off-line SWM facilities to incorporate Regional Storm storage capacity, particularly where impacts to private property and/or flood vulnerable areas is prevalent.

Overall, the preferred approach to flood management establishes stormwater management control criteria for the entire planned development area within the City through the completion of comprehensive plans (a SWS or MDP). Should the City allow proponents to proceed through the development process for a site that is not encompassed in a SWS or MDP (i.e. if such a document does not exist), the current policy should consider the sensitivity of the ultimate receiver (watercourse) and the proximity of the site to the receiver as part of an overall system-based approach.

Further, consistent with the expected direction from the Ministry of the Environment Conservation and Parks (i.e. 2019 Draft LID Guidelines and 90th percentile rainfall), as well as the City's 2007 SWMP, runoff volume reduction management should be considered through the implementation of source and conveyance controls. Such controls have been shown to assist in mimicking the natural hydrology of a site compared to traditional (end-of-pipe) stormwater management techniques (which do not promote infiltration or volume reduction).

It is also highly recommended that these broader scale studies apply continuous simulation hydrologic analysis approaches based on long-term data, rather than event-based analyses, as this is much more reflective of real-world conditions accounting for seasonal variability in precipitation

2.4.1.2 Conveyance System

The current storm drainage policy specifies that the minor system (storm sewers) shall be designed to convey the 5-year storm event (85% full flow design capacity) as per the 2019 Comprehensive Design Guidelines. This is generally consistent with most municipalities in Ontario; some municipalities incorporate a lesser standard (2-year storm event – Welland), while others use a somewhat higher standard (10-year storm event – Mississauga, Brampton).

It is noted that prior to amalgamation, each of the individual municipalities had differing standards, generally between a 2- and 5-year storm event (the former City of Hamilton used an 18-year storm event between 1942 and 1992, and a 50-year storm event thereafter). Ancaster applied a 2-year storm event previously. Due to these historic differences (not including changes in rainfall data and analytical methods) older areas may have a lesser design capacity than more recently designed areas.

The current City policy requires the major system to be designed to safely convey overland flows through roadways, swales, and channels to the receiving watercourse. The major system is to be designed to the 100-year storm event, less the capacity of the minor system. The current requirement of major system conveyance is considered to be appropriate, and generally consistent with the standards of other local municipalities.

2.4.1.3 Combined Sewer System

The scope of the Stormwater Master Servicing Strategy is focused upon the separated storm sewer system area (i.e. does not include areas serviced by combined sewers, which is covered by the Wastewater Master Servicing Strategy). Notwithstanding, a brief review of combined sewer policy is provided herein.

Current City policy prohibits the implementation of new combined sewers. In an effort to eliminate existing combined sewers, the current policy recommends the disconnection of storm sewers from combined sewers wherever sewer reconstruction / replacement is proposed. In many areas of the combined sewer system, parallel “relief” storm sewers have been constructed, which could ultimately permit a full separation in the future should downstream infrastructure permit it.

Reference is also made to the Flooding and Drainage Improvement Framework (FDIF) study prepared by GM BluePlan (now GEI) and Wood (now WSP) in February 2022 which also prepared a high-level roadmap for improvements in the combined sewer area, including priority sewer separation works and other capital improvements.

As noted, recommendations for modified policy for the combined sewer system will be part of the Wastewater Master Servicing Strategy updates. Notwithstanding, a sewer separation policy is considered appropriate and beneficial.

2.4.2 Quality Control Policies

Where a SWS or MDP exist that outlines the requirements for quality control, a development site, regardless of if it is new greenfield or re-development, is required to adhere to the requirements of the SWS / MDP. Policy revisions should carry this requirement forward, and the preferred approach to quality control would be to establish quality control criteria for the entire planned development area within the City through the completion of a SWS or MDP.

Where a SWS or MDP does not encompass a new development site, proponents are required to consult with the City regarding the level of quality control required. Typically, proponents are required to adhere to existing provincial policies (i.e. typically MECP Level of Protection 1 (Enhanced – 80% Average Annual Removal of Total Suspended Solids) or Level of Protection 2 (Normal – 70% Average Annual Removal of Total

Suspended Solids) and municipal policies. Table 6.3 in the 2019 policy document provides a summary of current water quality treatment requirements by watershed, either Enhanced, Normal, or a reference to contact the Conservation Authority.

The City's policy notes that all areas discharging to Hamilton Harbour are required to provide Enhanced Protection (80% average annual TSS removal). Notwithstanding, the City's current policy does not provide explicit reference to the Hamilton Harbour Remedial Action Plan. It is noted that the 1992 report (Stage 2) outlines the overall principle of zero impact, namely, that "*The philosophy be adopted whereas control of inputs of persistent toxic substances shall eventuate to zero discharge*". This approach (no net increase in contaminants due to development) would in fact suggest a more stringent target than Level 1 Protection (80% TSS removal) for areas draining to Hamilton Harbour. This should be confirmed and more clearly stated as part of the policy.

Historically, the City has considered the opportunity for re-development sites to provide cash-in-lieu where quality control is not feasible or practical to implement, or where storm runoff discharges to a combined sewer. The cash-in-lieu policy was not formally defined but the 2019 guidelines stated that consultation with City staff would be required. It is also not clear whether or not the City utilized pre-determined cash-in-lieu charges depending on the development area. City staff have noted that the cash-in-lieu provision has not yet been applied to their knowledge.

In general, the cash-in-lieu policy should be less preferred over on-site controls (including source / conveyance controls, which can be more readily implemented in sites with limited areas), as is suggested by the City's current policy and the recent GSG. Notwithstanding a clearer policy for those areas where it is considered feasible, should be considered. Reference is made to the Stormwater Quality Management Strategy for Stoney Creek (Philips Engineering, 2004). City staff should confirm whether or not the cash in lieu practice is still accepted or not. The City has also noted that the cash-in-lieu provision will be reviewed as a future update to the City's Comprehensive Development Guidelines, planned for 2026.

As noted in the introduction, the City enacted its Green Standards and Guidelines in January 2025. These GSG have established a Minimum Retention Target (MRT) which varies from 2.5 mm to 10 mm based on the drainage system (combined or separate), size (greater or less than 0.5 ha) and whether the area is covered by a SWS or equivalent, or not. These GSG promote the MECP hierarchical approach to applying the management of the 90-percentile rainfall (29 mm in Hamilton), with the first priority focussed on infiltrative measures followed by filtration and lastly through conventional means (end-of-pipe). Hamilton further emphasizes the use of "green" practices which are surface based as part of the first priority.

2.4.3 Climate Change

The City has a Climate Change Impact Adaptation Plan (2022) in place. It is noted that the City also completed a review of its rainfall IDF curves in 2022 (Matrix). That study concluded that the City's IDF curves (as per Philips Engineering, 2007) were adequate in terms of current historical data. However, it was recommended that this be re-evaluated every 5 years, both in terms of updated historic data, and more current future climate change projections. Based on the timing of the previous study, it is expected that the next review would occur in approximately 2027.

Currently, there are three primary online tools available to obtain climate change influenced IDF data, which are as follows. Other data tools and forecasting information is also available (such as climatedata.ca).

ddd) University of Western Ontario IDFC Tool

- a) The University of Western Ontario has developed a "Computerized Tool for the Development of Intensity-Duration-Frequency Curves under a Changing Climate" (available at <http://www.idf-cc-uwo.ca/>). This computerized web-based IDF tool integrates a user interface with a Geographic Information System (GIS). By creating or selecting a station the user is able to carry out statistical analysis on historical data, as well as generate and verify possible future change based on a methodology using a combination of global climate modelling outputs and locally observed weather data.
- b) There is currently a total of 24 different Global Circulation Models (GCM's) available in the tool. The RCP's (as defined in the Fifth Assessment Report (AR5, IPCC, 2013)), represent various future emission scenarios. Three different RCP scenarios are available in the tool. RCP 2.6 represents a low greenhouse gas concentration scenario, followed by RCP 4.5 as an intermediate scenario and RCP 8.5 as a high concentration scenario.

eee) Ontario Climate Change Data Portal (OCCDP) IDF Rainfall Tool

- a) The Ontario Climate Change Data Portal (Ontario CCDP - <http://www.ontarioccdp.ca/>) has incorporated the high-resolution (25 km x 25 km) climate projections developed by the Institute for Energy, Environment and Sustainable Communities (IEESC) at the University of Regina using the PRECIS model (under the A1B emissions scenario) and the RegCM model (under RCP 8.5 emissions scenario). Presently, projected IDF data are only available based on climate projections under the A1B emissions scenario. The A1B scenario aligns generally with the RCP 4.5 scenario.

fff) MTO IDF Curve Lookup Tool

- a) The IDF Curve Lookup (http://www.mto.gov.on.ca/IDF_Curves) is a web-based application provided by the Ontario Ministry of Transportation (MTO) for the purpose of retrieving Intensity-Duration-Frequency (IDF) curves. The time trend analysis was completed using observations from 1960 to 2014. A linear trend was observed and extrapolated from this period to 2060. Significantly less sources were available for data after 2010, so 2010 is the reference year used in this tool. IDF curve projections are extrapolated from the 2010 base year.

Each tool allows users to obtain forecasted IDF information within the Province of Ontario. As evident based upon the number of different climate models and emission scenarios, impacts are typically presented as a range, from low to high impacts. Ultimately infrastructure managers must determine an acceptable level of uncertainty and risk based on the range of potential impacts.

Other simplified approaches to climate change altered rainfall include simply uniformly increasing the IDF curves by some set value (i.e. 10%, 20%, etcetera) to attempt to account for future variability.

An alternative method of assessing temporal climate variability based on currently available data (i.e. not including forecasted / simulated information) is to undertake regular updates of approved IDF data as additional annual extreme datasets become available. For the City of Hamilton, this is based on Environment Canada's Hamilton Airport (Mount Hope) gauge. As additional annual IDF data become available, the City should continue to update and analyze its approved data.

As noted previously, the 2022 Matrix report noted that additional adjustments are not currently required, however this will be reviewed every 5 years, as per the recommendations of that study.

Beyond these IDF updates, policy revisions should consider the influence of climate change and the resulting effects on the City's infrastructure. This could be required as a potential "stress test" of drainage system resiliency to ensure functionality, consistent with the recommendations of the 2022 Matrix Report.

Should the City ultimately decide to require climate change analyses in some form as part of its drainage policy and criteria in the future, it is suggested that the approach to be employed should be formally defined (i.e. X% increase, or a specific GCM / RCP scenario to be employed). This would provide clear guidance to practitioners and help to avoid any uncertainty or ambiguity on the methodology to be employed, while providing consistent guidelines across the community.

As noted, based on the recommendations of the 2022 Matrix report for an updated assessment every 5 years, it is expected that a subsequent review of existing and future IDF will occur in approximately 2027.

2.5 Stormwater Infrastructure Design Criteria Review

2.5.1 Hydrologic Analysis

2.5.1.1 Rainfall

Current Intensity-Duration-Frequency (IDF) curves / hyetographs, based on rainfall data from both the Hamilton Airport and Royal Botanical Gardens, are now over two decades old (last updated based on 2004 data; issued in 2007). Notwithstanding, the City undertook an IDF assessment in 2022 (Matrix) and determined that the currently specified IDF data were representative and did not require updating. This should however continue to be re-assessed every 5 years (including climate change considerations), as discussed in Section 2.4.3 and as per the recommendations of the 2022 Matrix Report.

Consideration should also be given to assessing multiple area rainfall gauge sources as part of subsequent IDF updates, rather than a single gauge location. A single station may frequently miss larger more formative events, as such regional IDF updates in many cases are more appropriate, particularly for larger municipalities such as the City of Hamilton.

2.5.1.2 Analytical Methods

Current City standards provide proponents the opportunity to select between Rational Method, event-based (i.e. design storm), and continuous simulation hydrologic modelling methods, depending on the applicability of the method to their undertaking. Each method should be well understood, along with the applicability and associated limitations.

The Rational Method is typically limited to a maximum drainage area of 5 ha +/- (as per current City Comprehensive Development Guidelines and Financial Policy Manual (CDEGFP), Section G.5.1.2.) due to its conservatism in estimating peak flows. Consideration should be given to further restricting the situations in which the Rational Method can be applied due to its limited accuracy. In particular, City CDEGFP Section G.5.1.2 notes that the Rational Method can be used “*for storm sewer design regardless of the total contributing area*”, which is considered potentially problematic for larger sewershed areas. The Rational Method makes use of runoff coefficients, which are provided in current City standards for various land uses. Consideration should be given to reviewing recommended Runoff Coefficients and the appropriateness of applying a

generalized coefficient to any specific land use. City staff have noted they will also accept detailed calculations of Runoff Coefficients based on actual proposed impervious coverage.

Notwithstanding, for small sites and local storm sewers, the Rational Method will typically generate a conservative drainage system design, which provides an added degree of resiliency to the drainage system. However, the City may wish to consider a formal maximum limit on the applicability of the Rational Method (including storm sewer design), such that larger more complex drainage systems are better represented through the use of event-based or continuous hydrologic modelling.

Event-based (design storm) hydrologic modelling is applicable to studies of the same and larger scale compared to the Rational Method. Typically, event-based hydrologic modelling is used for large residential and commercial developments and is best applied in areas with a SWS or MDP that specifies the level of stormwater control and temporal distribution to be utilized in the study. Event-based hydrologic models are more robust than the Rational Method (particularly in assessing temporal variability of hydrographs), however there are limitations in the accuracy due to the synthetic nature of design storms.

The City of Hamilton currently provides a number of different design storm temporal distributions and durations as part of its IDF data, including the SCS (6, 12, and 24 hour) Chicago (3 and 6 hour) and AES (1 and 12 hour) design storm distributions. These distributions are commonly used by local practitioners; however longer duration Chicago distributions (12 and 24 hour) are also commonly used (including the Town of Oakville and MTO, which specify the 24-hour Chicago distribution as its standard design storm). Currently, the City does not mandate a specific design storm distribution, with the decision left to the practitioner typically based on a sensitivity analysis. In the case of designs influenced by volume considerations (i.e. storage and quantity control) a longer duration storm event (such as a 24-hour distribution) should potentially be mandated to avoid under sizing such infrastructure (i.e. by using a short duration, lower volume storm such as the 3-hour Chicago). Further, the City may wish to consider mandating a sensitivity analysis to ensure that proposed stormwater infrastructure designs evaluate all design storm distributions to determine the most conservative or mandate a conservative distribution (such as the 24-hour Chicago, which typically generates both high peak flows and high volumes).

Requirements for event-based models are provided in Section G.5.1.3 of the 2019 Comprehensive Development Guidelines. Appendix J of the guidelines provides a list of approved hydrologic models. The list appears quite dated; considerations for these models are noted below in **Table 2.5.1**.

Table 2.5.1: Assessment of Current City-Accepted Hydrologic Models

Model	Recommendation	Note
SWMHYMO	Preserve subject to using latest version (5.5 or above)	Older software that has been challenging to run on modern computers. Most recently available version suggests it has been updated to run on current versions (i.e. Windows 11). Provided this is confirmed and proponents use latest version, can be preserved.
Visual OTTHYMO	Preserve	Currently used in industry
SWMM (EPA-SWMM and PCSWMM)	Update	Update to clarify EPA-SWMM and PCSWMM are acceptable
XP-SWMM	Remove	No longer commonly used in industry
MOUSE (DHI)	Update	MOUSE has been superseded by MIKE software; change reference to MIKE+
HSP-F	Preserve	Not commonly used but has application for continuous simulation specifically
MIDUSS	Remove	Older software that is not easily verifiable; recommend removing and suggesting alternative programs
HEC-HMS	Add	Freeware that is publicly supported (USACE) and used in industry, suggest adding
InfoWorks ICM	Add	Not typically used for small scale assessments but commonly used software in industry

Continuous simulation is preferred particularly for broader based system analyses, due to its use of actual observed rainfall data. Continuous simulation modelling also provides useful data with respect to water balance, erosion data (through the use of critical duration analysis), drawdown time for storage facilities (including LID BMPs) and the impact of “back-to-back” storms, baseflow information, and other useful data. Notwithstanding, continuous simulation modelling requires much more data and effort to apply. Continuous simulation modelling is considered more suitable for larger scale watershed/subwatershed assessments and more natural, rural systems (as was noted in Section 3.1.1). The City may however consider specifying drainage area thresholds for which continuous simulation modelling is to be implemented.

2.5.1.3 Climate Change

As discussed previously, climate change altered rainfall is not currently considered in the City’s IDF, however this is appropriate based on the findings of the 2022 Matrix Report and assessment. As per that study, this should be re-assessed every 5 years (both existing climate data and future projected climate data results). The next review

should therefore occur in approximately 2027. A supplementary “stress test” of drainage system resiliency for works should be considered, as noted in the 2022 Matrix Report. Ultimately, should the City implement any such modifications in the future, clear direction on the methodology to be applied would be required to guide practitioners.

2.5.2 Minor System

2.5.2.1 Storm Sewers, Catchbasins and Inlets

Current City standards require storm sewers (minor system) to be sized to convey the 1 in 5-year storm event at 85% of the full flow capacity. This level of conveyance is considered appropriate and generally consistent with other area municipalities. Notwithstanding, the City of Hamilton may wish to consider an increased design standard (i.e. 1 in 10-year storm) to account for future uncertainties. Any increase in minor system conveyance capacity would lead to a reduction in major system (overland) flows and would also provide a further degree of resilience; potentially as an alternative to direct climate change directives (i.e. IDF modifications, as discussed in Section 3.3). Any additional minor system capacity would also be particularly beneficial in areas with foundation drains serviced by a gravity connection to the storm sewer (i.e. reduced frequency of surcharge). Notwithstanding, the existing storm sewer system would largely have been designed to a 5-year standard in most cases, thus any new storm sewers designed to a higher standard (i.e. 10-year storm event) would be somewhat restricted by downstream tailwater conditions regardless. Larger diameter storm sewers would also result in increased construction / capital costs. Any modified design standard would therefore need to be carefully considered by City staff and reviewed with the development industry.

Locally based storm sewer peak flows are typically generated using the Rational Method, which is noted to be generally conservative in estimating peak flows. Notwithstanding, the limitations of the analytical methods should be understood (refer to Section 4.1.2 for a discussion of the Rational Method). Further to that discussion, it is suggested that a formal maximum contributing drainage area for the use of the Rational Method should be considered (the City guidelines currently suggest a nominal threshold of 5 ha, but no maximum on storm sewer designs). With respect to Runoff Coefficients, while there are numerous guidelines available to the selection of appropriate values, the connection between this value and the actual proposed imperviousness should be confirmed, particularly given the increased density and imperviousness associated with more recent residential developments.

In conjunction with the Rational Method, pipe sizing is typically completed using a spreadsheet/table approach using Manning’s Equation. This approach does not easily account for dynamic tailwater / backwater conditions due to downstream restrictions or water levels of receiving systems. Similarly to the discussion in Section 4.1.2, a

maximum drainage area limit on the applicability of the Rational Method and Manning's Equation should be considered, potentially formalizing the limits noted in the City guidelines (i.e. 5 ha).

The City's current guidelines require that a hydraulic grade line (HGL) be completed for "the inlet capacity and/or one in five-year standard and one in 100-year standard", and that the 100-year HGL in the sewer is to remain below the ground surface to avoid interference with overland flows (unless it is demonstrated that this cannot be practically achieved). It is noted that a dual drainage model (integrated minor and major system modelling) will be required in those cases where the 100-year HGL cannot be maintained below ground surface. However, no technical guidance on acceptable methods for establishment of the HGL are otherwise provided in the guidelines, and there is no further mention of dual drainage modelling beyond this one consideration.

It is suggested that the City provide formal requirements for HGL analyses and consider requiring integrated dual drainage modelling for at a minimum, areas of a certain size or critical areas (i.e. those with foundation drains with gravity connection to the storm sewer), and potentially in all cases. Numerous software programs are available which can undertake this analysis, including PCSWMM, MIKE+, and InfoWorks ICM, all of which are listed on the suggested approved software list in **Table 2.5.1**.

Potential alternative approaches for defining the HGL should be easily verifiable and appropriately account for the inlet capacity of catchbasins (using standard inlet capacity curves available in the MTO's Drainage Management Manual or other verified sources). Simplified methods (such as assuming the storm sewer flow is restricted to the 5-year design capacity) are not appropriate for the calculation of the HGL; this should be explicitly stated in the City's guidelines.

Further to the background review work completed as part of Technical Memorandum 2.4 (July 2025), it is understood that the City does have limited inlet control devices (ICDs) in place in its catchbasins. City staff have noted that the City has approved ICDs for certain subdivisions, particularly where superpipes are proposed. However, ICDs are not generally in place in older areas of the City.

Without ICDs in place, the inlet capacity available typically exceeds the full flow capacity of the storm sewer system. This can lead to extensive storm sewer surcharging for more formative storms (such as the 100-year event). This may result in a 100-year sewer HGL above the ground surface (which is contrary to current City policy), or potential impacts to foundation drainage in areas with direct gravity connections (foundation drains are discussed further in Section 4.2.2). In conjunction with more formal requirements for HGL calculations, the City should also consider requiring or implementing ICDs in areas where HGL impacts would be expected. Where implemented, ICDs should be tracked in the City's drainage inventory databases, and

also be reflected in hydrologic / hydraulic models where present. A formal list of suitable ICDs would also be required (which could be added to the City's approved products list). Although not part of the scope for the separated storm sewer system assessment, it should be noted that ICDs are particularly advantageous in restricting storm flows for the combined sewer system, where the impacts of sewer surcharging are particularly detrimental (i.e. surcharge through residential basement floor drains).

In general, the 100-year simulated HGL should be at least 0.30 m below the bottom of the proposed building footing \ slab \ lowest elevation.

2.5.2.2 Building Drainage Connections

Foundation Drains

Current City standards generally promote connection to a storm sewer or combined sewer (never a sanitary sewer). The use of sump pumps discharging to surface is also noted to be acceptable, subject to several considerations with respect to grading and available space. In general, it is suggested that drainage to the surface should be the primary option for foundation drainage (sump pumps with "gooseneck" connections above surface), given the known limitations of sewer capacity (particularly combined sewers, and the need to restrict inflow and infiltration to these systems and the impacts of surcharging).

Reference is again given in the City's guidelines for the need for a hydraulic analysis for areas with foundation gravity drains, which is logical (noting the discussion on HGL analysis techniques presented in Section 4.2.1). City guidelines should also consider the use of backflow prevention on building foundation gravity drains connected to the storm sewer system, or areas with identified HGL concerns, in order to address the risk of basement flooding.

Roof Drains

Current City standards require that all roof leaders discharge to surface. While in general, this is considered a sound policy (to minimize direct inflows to the storm sewer system and promote at source infiltration), this criterion should be expanded to address two particular considerations:

ggg) Low Impact Development Best Management Practices (LID BMPs) As roof water is typically considered "clean", roof leaders can be connected to a number of source control measures such as rear-yard soakaway pits or other sub-surface measures. City criteria should promote these approaches for roof leader flows, consistent with the direction of the 2007 SWMP (as well as the MOEE's 2003 SWM Planning and Design Manual). Currently, City guidelines indicate that soakaway pits are "discouraged due to maintenance and impacts on use of rear yards".

Maintenance requirements could potentially be addressed through requirements to place these features on public lands or easements.

hhh) Areas of High-Density Development The City should re-consider the restriction of roof leader disconnection in high density development. Roof leader disconnection should only occur when discharge is directed to a pervious area of sufficient size to eliminate local surface flooding issues (a similar approach is noted in the City's requirements for sump pump discharges, but not in the discussion of roof leaders, which would be expected to have considerably higher flows). Given the increase in higher density developments in recent years (with associated minimal pervious areas), in some instances it may be more logical to encourage connected downspouts and implement conveyance controls (LID BMPs) to safely infiltrate this water further downstream.

Rear-Yard Catch Basins

Rear yard catch basins (RYCBs) are commonly implemented as part of residential developments in the City of Hamilton to overcome local grading constraints or where consistent overland flow routes cannot be achieved. The City's guidelines provide requirements for RYCBs, including requirements for maximum ponding depth and overland flow routes. In general, it is suggested that the City consider putting a greater emphasis on overland flow conveyance routes for rear yard drainage and limiting the creation of sag points which require RYCBs. Although RYCBs are typically required to be private services, they are frequently poorly maintained and may be impacted by re-development and hard scaping of backyard areas.

There is no reference to requiring an HGL analysis as part of the design of RYCBs; the guidelines should be revised to incorporate this requirement. Based on WSP's experience, in many cases RYCBs may experience surcharge and flooding during the 100-year storm event, which could impact private property (notwithstanding the City's requirements for maximum ponding depths and overland flow relief routes).

2.5.3 Major System

2.5.3.1 Roadway Conveyance and Overland Flow Routes

Current City standards restrict the 100-year flow depth of roadway conveyance to various levels depending on the classification of the roadway (0 mm above the crown for arterials and emergency routes, 150 mm above the crown for all other roadways). This is generally considered to be reasonable and should be sufficient for vehicle access. Notwithstanding, based on WSP's experience there has been an emerging direction that emergency vehicles will no longer enter or cross roadways covered in water, irrespective of the depth. As such, the City of Hamilton may wish to consider

stricter criteria for overland flow depths, particularly for roadways identified as emergency routes.

Consistent with the discussion in Section 4.2, the City's guidelines do not specify how flooding depths are to be assessed / confirmed. Further to Section 2.5.2.1, dual drainage modelling provides greater accuracy in estimating 100-year hydraulic grade lines, including surface flooding depths. Accurate representation of inlet capacity is a key consideration in estimating overland flow depths, including the differences in inlet capacity of at-grade and sag point catchbasins.

The City's guidelines include reference to the consideration of obstruction (and by inference, debris blockage, which is a consideration under high intensity storm events) and the associated impact on inlet capacity specifically for rear yard catchbasins in Section G.2.2 in the 2019 CDEGFG. However, there is no corresponding statement requirement for road catchbasins. The City may wish to similarly and conservatively include some debris restriction (i.e. 50% of full flow capacity) in the assessment of road catchbasins, in order to build additional capacity and resiliency into the overland flow system.

In older areas of the City, where overland flow routes may be poorly defined or restricted, consideration should also be given to the use of 2-dimensional (2D) hydraulic modelling. 2D models can be very useful in analyzing overland flow routes in these types of areas, as they provide the ability to model the lateral movement of water, which may locate unexpected overland flow paths and areas at risk from surface flooding / ponding. The City's guidelines should be amended to reflect its acceptance of these modelling methods.

2.5.3.2 Roadway Crossings (Culverts and Bridges)

Current City standards require proponents to design watercourse crossings to meet Ontario Ministry of Transportation Drainage Design Standards and Ontario Ministry of Natural Resources and Forestry ingress and egress requirements. These standards are the current guidelines on watercourse crossing design and should continue to be referenced in City standards.

Consideration should be given to flow conveyance depths and vehicle passage as per Section 4.3.1.

City guidelines should consider including the requirement for development plans to include access to watercourse crossings that are to be owned / maintained by the City. This would largely benefit maintenance activities, but also potentially emergency access if required.

2.5.3.3 Natural Watercourse Systems

Current City standards require proponents to design natural watercourse systems to meet all applicable agency standards. These requirements should be carried forward, however should be revised to reference current agency names and technical documents. The specific requirements for headwater drainage features (HWDFs) should also be considered as required.

Consistent with Section 4.3.1, no mention of 2-dimensional (2D) hydraulic modelling is included in the City's guidelines. The guidelines should be updated to note that 2D modelling is an accepted methodology and may in fact be more appropriate in areas with flat floodplains, or in particular for assessing spills (rather than the typical 1-dimensional approach of simply indicating a spill will occur).

2.5.4 Quantity and Quality Controls

2.5.4.1 Source Controls and End of Pipe Controls

In general, based on a review of the City's comprehensive development guidelines (2019) the City's guidance includes cautious or somewhat negative wording around source and conveyance controls (i.e. LID BMPs):

iii) Discouraged (soakaway pits)

jjj) Applicable in specialized applications (porous / pervious pavement, pervious pipes and catchbasins)

kkk) Limited applicability in the City of Hamilton (infiltration basins and trenches)

Further, the development guidelines state in a discussion on infiltration methods that:

"In general, there are relatively few areas within the City of Hamilton where stormwater infiltration is critical to maintain downstream ecosystem integrity. Furthermore, the database regarding the impact of stormwater infiltration on groundwater quantity and quality is limited."

The preceding would appear to run contrary to the direction of the 2007 SWMP, which advocated a shift in the stormwater management approach within the City to an increased focus on LID BMPs (source and conveyance controls, which focus largely on infiltration), as well as the 2022 Draft LID Guidance document (MECP) and the requirements generally outlined for typical municipal CLI-ECAs (MECP), as well as the City's Green Standards and Guidelines (GSG, 2024).

Notwithstanding, City staff have noted that this statement will be corrected as part of the pending (2026) updates to the City's Comprehensive Development Guidelines. It is expected that the next iteration of the document will more clearly promote and advocate LID BMP approaches, given the direction of the 2007 SWMP as well as LID guidelines

from the MECP. Notwithstanding, LID BMP implementation needs to be logical and consider specific site constraints to ensure long-term functionality (as is suggested in the City's current guidelines).

As noted, the City has established its GSG which promote a variable source control target of between 2 and 10 mm premised on various site conditions and variables. The list of source controls, conveyance controls, and end-of-pipe techniques provided in the City guidelines omits a number of standard techniques, such as those discussed in the Low Impact Development Stormwater Management Planning and Design Guide (Credit Valley Conservation and Toronto and Region Conservation Authorities, 2010).

Additional controls not listed in the current City standards should include (but are not limited to) the following (Note: the list should also align with the GSG and in particular those practices which are surface-based and which the City considers "green"):

III) Bioretention areas

mmm) Rainwater harvesting (rain barrels, cisterns, rain gardens, etcetera)

nnn) Modular pavement / soil support and tree systems (i.e. Silva Cells™)

ooo) Increased topsoil depth

In addition, there are limited references to "treatment train" as part of the 2019 guidelines (with respect to filter and buffer strips, and again for oil / grit separators). It is recommended that this approach be mandated as per MECP (2022) and the City's GSG; requiring at least two different techniques for water quality treatment, particularly if one of these techniques is an oil / grit separator unit. Providing explicit direction on the amount of TSS removal credited for different techniques may also be considered by the City for consistency in evaluating submissions, however it may also be logical to continue to assess approaches on a case-by-case basis.

With respect to more typical engineered quality control measures, it is noted that there is no reference to more recent water quality treatment technologies, including:

ppp) Filtration based measures (Filterrra™ catchbasins, Jellyfish™ treatment devices, etcetera)

qqq) Tree planter cells (Silva Cells™ and similar products)

rrr) Catchbasin inserts (CB Shield™)

sss) Maintenance hole baffle inserts (SAFL Baffle™)

It is noted that several manufacturers are developing manufactured sand filter technologies, however the City's current guidelines noted that sand filters have "limited application". This should potentially be reviewed.

With respect to quantity control measures, underground storage tanks are also not explicitly listed in the City's guidelines. We note that these should be clearly included, as

storage chambers are common across Ontario for both private and public stormwater quantity control (and in some cases infiltration) purposes, subject to utility (right-of-way, parking, etcetera). Underground storage systems are typically employed for commercial and industrial site developments, or smaller residential infills. The City should also consider a review of technologies in an effort to specify whether any specific underground storage tank technologies are or are not considered acceptable. Furthermore, the City should conduct a life cycle analysis of systems in order to protect and inform its asset management systems.

Overall, as new technologies for stormwater management and control become available, the City should be responsive in considering these measures and providing updates accordingly. Given the uncertainty as to timing, a more general overview of the City's approach to technology verification and approval could be considered. The existing CA-ETV program should be mentioned as part of this approach.

2.5.4.2 Cash-in-Lieu Policy

The City's cash-in-lieu policy (where developers may provide a financial payment in lieu of quantity or quality controls where not considered feasible or effective) was discussed previously in Section 3.3. As noted in Section 3.3, it is recommended that the City develop a clearer policy for those areas where it is considered feasible, assuming that the City still accepts or considers cash-in-lieu, as the current status has not been confirmed. In general, it is suggested that cash-in-lieu approaches be minimized to those locations where controls are completely infeasible or the benefits are negligible; otherwise, on-site controls should be preferred as a minimum. It is understood that this was also recommended to the City as part of the Green Standards and Guidelines document.

2.6 Background and Policy Review Summary and Recommendations

The preceding summary provides a review of applicable stormwater management policies and criteria for the City of Hamilton. This review has been conducted in advance of assessing the impacts of planned growth and development, such that any potential areas of concern are identified beforehand. Considerations have been noted below. Note that WSP previously prepared a summary of draft Stormwater Policies in May 2020 (refer to **Appendix A**) for the City, however that summary does not reflect more recent changes in Provincial and industry stormwater policies, which are reflected in the summary below.

1 Overall Policy Considerations. The following policy and design criteria items were noted:

- a) The City's policy should incorporate reference to emerging direction on Regional Storm Management, including potentially requiring additional storage volumes within off-line stormwater management facilities.
- a) The City should consider undertaking or incorporating results from regional IDF assessments that utilize data from multiple gauges rather than a single station.
- b) The City may wish to consider stricter standards on acceptable overland flow depths for roadways, given emerging direction on emergency vehicle procedures.
- c) Stormwater quality control policy for watersheds draining to Hamilton Harbour should be more clearly stated, including the "no net impact" requirement which is stricter than the typically applied "Enhanced" protection level (80% average annual TSS removal).

2 Climate Change. The impacts of climate change influenced rainfall are not incorporated into the City's current IDF, however this is consistent with the findings of the most recent assessment (2022) by Matrix Solutions. That study recommended re-assessing the data every 5 years; as such the next review should occur in approximately 2027.

Depending on the outcomes of that study, further updates may be considered to the City's IDF if necessary. A number of potential options have been reviewed in the current report which could be considered as part of the 2027 review, including:

- a) Requiring the use of various future IDF tools
- b) Requiring the use of a set percentage increase in IDF
- c) Increasing design standards (i.e. 10-year flow capacity requirement for storm sewers)
- d) Implementing a "stress test" verification to drainage system designs

3 Modelling and Analysis Techniques. Hydrologic and hydraulic modelling approaches outlined in the City's guidelines have been reviewed, and a number of different suggestions have been included:

- a) The use of the simplified Rational Method should be further restricted to areas 5 ha +/- or less, including storm sewer designs (which are currently not included in the 5 ha restriction)
- b) The City may wish to more formally require a sensitivity analysis by practitioners in the selection of an appropriate design storm distribution for event-based analyses. Alternatively, the City could consider mandating a single, conservative distribution (such as the 24-Hour Chicago Storm).

- c) The City should update its list of acceptable hydrologic models to reflect current practices.
- d) Continuous simulation should be further promoted or required for larger assessments of rural/natural drainage areas, in particular for subwatershed studies or master drainage plans. Similar to the approach for the Rational Method, a drainage area limit could potentially be considered.
- e) The City should require hydraulic gradeline (HGL) analyses for developments and should explicitly state accepted methods for undertaking these analyses. Dual drainage models should be more explicitly promoted accordingly.
- f) The City may also wish to conservatively consider a debris blockage factor for road catchbasin inlets as part of these analyses.
- g) The City may also consider requiring inlet control devices (ICDs) in areas where HGL impacts to residential properties would be expected without restrictors. Where implemented, ICDs should be tracked as part of the City's inventory databases, and acceptable types should be added to the City's approved products list.
- h) 2-Dimensional (2D) hydraulic models are not referenced in the City's guidelines. 2D modelling approaches are now relatively common and are a more robust method to assess surface flooding extents in both rural and urban applications. The City should indicate acceptance of these methods and may potentially wish to list acceptable modelling platforms.

4 Building Drainage Connections. The following has been noted with respect to building drainage connections:

- a) The City's guidelines should consider putting a greater emphasis on discharge of foundation drainage to surface rather than sewered connections given potential HGL impacts, while still taking into account site restrictions (i.e. available pervious area space for discharge suitably far away from the residence).
- b) The City's guidelines should incorporate additional details with respect to roof drain discharge. While discharge to surface (pervious area) should still be promoted, consideration should be given to infiltration measures (i.e. rear yard soakaway pits) as well as high density developments where a piped connection (with downstream conveyance controls) may be more appropriate.
- c) The City's guidelines should promote positive surface grading over the implementation of rear-yard catchbasins (RYCBs) where feasible. RYCBs should also be incorporated into HGL analyses to ensure that potential impacts to private property are assessed.

5 Low Impact Development Best Management Practices (LID BMPs). The City's recently implemented GSG should be used to update its current policy and design

criteria which are cautious on the widespread application of LID BMP measures. It is understood this will be updated as part of the City's 2026 revisions to the Comprehensive Development Guidelines. It is suggested that:

- a) The policy and criteria wording should include additional accepted LID BMP, and attempt to promote these approaches further, while still requiring an analysis of site-specific constraints to ensure long-term function.
- b) The City should harmonize its guidance in the GSG on minimum infiltration targets.
- c) More explicitly promote the "treatment train" approach with multiple measures for quality treatment as per the GSG, MECP (2022 Draft) and its requirements through its Draft CLI ECA.

6 Development Stormwater Management (SWM) Measures. The following has been noted for site measures in addition to the preceding discussion on LID BMPs:

- a) The City should include more current engineered stormwater quality technologies as they become accepted; these technologies should also potentially be screened and listed as part of the City's approved products list (including oil / grit separator units and related technologies).
- b) The City's guidelines should also include reference to underground storage tanks and undertake a review of available technologies in order list acceptable products as part of the City's approved products list.
- c) The City should continue to review a cash-in-lieu policy for stormwater management measures, as per the guidance in the WSP report for the GSG.

3 EXISTING STORMWATER SYSTEM

3.1 Recent Development Areas

In order to understand potential changes in the stormwater system since the completion of the 2007 Stormwater Master Plan (SWMP), the extents of development which have occurred since 2007 to 2022 have been estimated. This assessment has been completed using historical (2007 era), aerial photography as available in Google Earth™ and GIS data provided by the City for this project in 2022. Based on this assessment, recent developments (2007 - 2017) and new development area (2017-2022) have been generally estimated. These areas are indicated on **Drawing 2**. It should be noted however that the previous SWMP began in 2005, as such it may have used earlier (2004 +/-) data. As such, there may be some additional area of development from this era (2004-2007) which may not be represented, however this is likely very minor.

Notwithstanding, a total area of 998 ha +/- of development was estimated for the 2007 - 2017 time period plus an additional 696 ha +/- up to 2022. This reflects a total development over the 2007 - 2022 period of 1,694 ha. The limitations to the approximate estimation methodology should be noted. In general, recent and new development is clustered in the following areas:

- Waterdown (north of Parkside Drive);
- Ancaster (Garner Road area);
- West Mountain (Garth Road and Rymal Road area);
- East Mountain (area south of Rymal Road east of Trinity Church Road and along Upper Centennial Road); and
- Binbrook

Other smaller localized developments are also indicated on the mapping.

3.2 Stormwater Management Facilities and Oil Grit Separators

3.2.1 Overview of Facilities

GIS layers (shapefiles) of Stormwater Management Facilities (SWMFs) have been provided by the City of Hamilton in 2017 and 2022. Separately, the City includes a summary of oil / grit separators as part of its GIS layer (shapefile) of maintenance holes

and chambers. **Drawing 3** summarizes this information. Summary statistics on the types and numbers of facilities is presented both in **Drawing 3** and in **Table 3.2.1**.

Note that the City's SWM Facility identification numbers have not been included in **Drawing 3** given the number of SWM facilities and the drawing space restrictions. Note that summary totals are City-wide, not just within the separated storm sewer area \ study area limits. Private SWM facilities within the City's database have also been included.

Table 3.2.1: Summary of Stormwater Management Facility Inventory Data (City-Wide)

Facility Type	Number of Facilities Estimated to have been Constructed by 2007 or earlier	Number of Facilities Estimated to have been Constructed in 2007 - 2017 Period	Number of Facilities Estimated to have been Constructed in 2017 - 2022 Period	Removals in 2017-2022 Period ²	Total
Dry Pond	69	1	8	1	77
Wet Pond	66	33	36	14	121
Wetland	5	6	0	0	11
LID BMP	3	1	2	1	5
OGS	52	5	0	0	57
TOTAL¹	143 (195)	41 (46)	46 (46)	16 (16)	214 (271)

¹ First values omit OGS units, second (bracketed) value includes OGS units.

² Removal generally does not mean any loss of function but typically the decommissioning of a temporary facility and replacement with a new or retrofitted facility. Refer to details below.

As evident, there are currently estimated to be 271 SWMFs (including OGS units; 214 excluding OGS units) within the overall City of Hamilton as of the 2022 dataset.

It is noted that the 2022 dataset includes 16 separate ponds within the St. Elizabeth Village property at Garth Street and Rymal Road which were not included in the 2017 dataset, although they have been in place since before this time.

Wet pond removals in the 2017 - 2022 period appear to represent temporary ponds for interim construction works. The LID BMP removal appears to actually be a correction for the facility around the Canada Bread site (Nebo Road and Twenty Road) which was more correctly a dry pond (quantity control only) facility. Note that this facility was subsequently replaced by a new wet pond SWM facility as part of the construction of the Atlantic Packaging site to the south in approximately 2023, but this is not reflected in the dataset given the vintage of the data (2022).

It should be noted that a few of the noted SWMFs are located within the combined sewer area of the City (3; 1 at Gage Park and 2 SWMFs in the industrial area north of Nikola Tesla Boulevard), however the vast majority are located within the separated sewer area of the City.

The 2007 SWMP indicated that there were “over 130 stormwater management facilities” at the time of that report, which would appear generally consistent with the totals presented in Table 3.2.1 (143 SWM facilities and 52 Oil / Grit Separators). A complete inventory listing though is not provided in the 2007 SWMP. Based on the summary provided in the 2007 SWMP, it appears that an inventory of facilities was completed in 2005 (as noted in the Background Information Review Summary in Section 2.2). However, this document has not been received or reviewed by WSP to confirm what additional information could be contained therein.

The GIS layer supplied by the City of Hamilton does not summarize the construction date. As such, the dates summarized in **Table 3.2.1** have been estimated using the development era information previously presented in **Drawing 2** (attached). In general, it appears that SWM facilities constructed since 2007 are primarily wet ponds.

The majority (92%) of the SWM facilities are publicly owned (assumed) or likely planned for public ownership (unassumed) based on the information supplied by the City. The balance (8% - 20 SWMFs) are privately owned.

There is also a limited amount of information on SWM Facility characteristics within the supplied GIS layer (i.e. key facility elevations and depths, outlet structure, etcetera). Some SWM facilities do provide information related to surface area, however these data are generally limited. There is some SWM facility information included as part of the MIKE+ models (as discussed in Section 4). The 2007 SWMP indicates that this information was, in part, developed using the 2005 report cited earlier, as well as subsequent field verifications and where necessary, field survey and data collection.

As noted in Section 2, the 2007 SWMP recommended a number (29) of prioritized SWMF retrofits (primarily dry ponds which would become wet ponds or wetlands). Based on the limited information available in the City GIS data, it appears that only one facility has been retrofitted since this time, within the Ancaster Industrial Pond (ID 5). Based on Google Earth™, it appears this retrofit was completed sometime between 2006 and 2009. There are a number of other facilities which have a relative priority ranking for retrofitting (identified as “RETROFIT”) which appears to be consistent with the 2007 SWMP, however there is also a separate “RANK_13223” which does not match the preceding data; it is unclear what this field represents.

As noted, the 2007 SWMP recommended a general shift in the SWM strategy for the City, with an increased focus on low impact development best management practices

(LID BMPs), in the form of source controls (largely for individual developments) and conveyance controls (primarily for roadway reconstructions). Based on the information provided and summarized in **Drawing 3** and **Table 3.2.1**, only one LID BMP installation has been completed since 2007. This installation is located in the Nebo Road and Twenty Road (system of bioswales for the Canada Bread Plant). No conveyance controls are indicated as being constructed since 2007, which is noteworthy given the recommendations of the SWMP. Based on the summary of Public Works Capital Projects (right-of-way – roadway reconstructions), an estimated 316 roadway projects have occurred in the study area from 2007 onwards. While it should be acknowledged that many of these projects would likely not have involved full replacement of infrastructure (i.e. asphalt rehabilitation only), there would have been a significant number of reconstruction projects for which conveyance controls were not implemented. A detailed review has not been conducted to determine for which projects stormwater quality controls were (or were not) implemented, as it is also uncertain as to whether or not this information is readily available. This should potentially be considered by the City as part of future studies and initiatives.

3.2.2 Water Quality Treatment

The GIS data supplied by the City of Hamilton do not include information as to the designed level of water quality treatment provided by the various practices, wet pond, wetland, and oil/grit separator features in particular (it has been assumed that dry ponds would provide little to no water quality treatment). In addition, there is no information on whether or not any of these features have excess treatment capacity, however this is generally considered to be unlikely.

Further, there is no information for these features with respect to the drainage area serviced / treated. As noted in Section 2, the previous SWMP focused on a mass balance approach at a broad-scale watershed / subwatershed level and provided minimal consideration of existing levels of water quality treatment for the urban areas of the City of Hamilton.

Using storm drainage area / subcatchment boundary data mapping supplied by the City of Hamilton, and the previously noted SWMF inventory mapping, a summary map of water quality treatment has been prepared and is presented in **Drawing 4**. It should be noted that for the purposes of the current assessment, the drainage areas included in the summary do not account for downstream SWM facilities (such as the Mountain Brow SWM Facility which re-directs and treats low flow from Red Hill Creek; it is unknown if the City has similar types of SWMFs in other locations).

In addition, for some areas of the City, no drainage boundary information was available based on the data supplied by the City of Hamilton. As such, in these areas, a SWMF

may be indicated, but no corresponding drainage area is shown [including areas of Flamborough (Freelton, Carlisle and Waterdown], Dundas\West Hamilton and Ancaster in particular). As such, the estimated extents of stormwater quality treatment are considered to be lower than the actual treated extents.

Notwithstanding the preceding limitations, as indicated in **Drawing 4**, approximately 3,862 ha +/- (20% +/-) of the separated storm sewer system area is estimated to receive some form of stormwater quality treatment prior to discharge. In general, these areas typically align with newer development areas (as per **Drawings 2 and 3**), as would be expected. Areas with minimal to no end of pipe stormwater quality treatment (noting the preceding limitations) include:

- Dundas and West Hamilton
- West Mountain (Scenic Drive to Upper James Street, Fennell Avenue to Stone Church Road)
- Central Mountain (Upper James Street to Upper Ottawa Street, Mohawk Road to Rymal Road)
- Stoney Creek

Notwithstanding, the preceding does not consider private and on-site SWM quality controls, which are not included or available in the information provided to WSP. Newer developments which were subject to Site Plan Control, would be expected to have such controls, but these cannot be mapped given the lack of available information.

Of particular concern would be areas with high commercial and industrial lands uses; this would include the Upper James area on the West Mountain, as well as the lower Stoney Creek area (with the extensive industrial development to the north of Barton Street).

As noted previously, no simulated water quality data (i.e. further characterizing the expected level of contaminant generation and associated treatment) are available for any of the separated storm sewer area. As noted, the previous SWMP considered lumped watershed areas only as part of a mass-balance spreadsheet model.

It is considered that there may be a benefit to incorporating stormwater quality modelling directly into the primary hydrologic / hydraulic modelling tools developed by this study (i.e. MIKE+), rather than relying on a separate simplified approach. This is discussed further as part of Section 4, but this modelling analysis is beyond the current scope.

3.3 Storm Sewer System

A GIS inventory (shapefile) of all of the sewer pipes, maintenance holes, and catchbasin units has been provided by the City of Hamilton for the entire City, including the current study area (separated storm sewer system). In addition, the previously modelled storm sewer extents from the 2007 SWMP (the “Six Cities” model – refer to Section 4 for further details) have been extracted from the MOUSE \ MIKE URBAN modelling and reviewed. The results are presented in **Table 3.3.1** and presented graphically in the attached drawings: **Drawing 5a** (Existing Storm Sewer Plan) and **Drawings 5b to 5d** (Modelled Storm Sewer Plan).

Table 3.3.1: Summary of Separated Storm Sewer Data for Study Area

Pipe Diameter Range (mm)	Existing Length (2022)		Previously Modeled Length ¹		Potential Additional Length to be Modeled	
	Sections	Length (km)	Sections	Length (km)	Sections	Length (km)
< 525 ³	9,357	512	228	12	-	-
525 to <1200 ²	7,623	390	2,501	143	5,122	247
>=1200 ²	3,103	179	1,012	73	2,091	106
TOTAL	20,083	1,081	3,741	228	7,213	353

¹ From “Six Cities” model developed from 2007 SWMP – refer to Section 4 for further discussion

² 1200 mm was used as a cutoff for previous model data, but 1000 mm was used for 2022 data (i.e. 1050 mm diameter and greater, thus values may differ slightly due to this difference.

³ For 2022 pipe data, pipes with no diameter information (0) are assumed to be less than 525 mm for the purposes of calculating a breakdown.

Based on the results presented in **Table 3.3.1**, approximately 38% of the current (2022) “trunk” storm sewer lengths (525 mm in diameter or greater) were included in the 2007 SWMP modelling (by length), or 21% of all storm sewers. It should be noted however, based on discussions with City of Hamilton staff, that following the 2007 SWMP, more detailed hydrologic / hydraulic models were developed for the communities of Dundas, Stoney Creek, and Ancaster, which would increase the coverage summarized previously. This is discussed in greater detail in Section 4.

Based on the above noted assessment, there are approximately 353 km (or 7,213 sections) of additional “trunk” storm sewers that could potentially be added to the modelling as compared to the 2007 modelling. As noted however, this may not accurately reflect additional piping in the more detailed community-specific modelling completed after the 2007 SWM MP.

In general, the storm sewer GIS dataset provided by the City of Hamilton includes only limited storm sewer invert elevations, or invert/rim elevations for maintenance holes. It is

unknown whether the City has these data available in a database form or not. If not, any potential model revisions and additions would likely be more labour-intensive (i.e. would need to rely on approximate methods (interpolation) or a review of available plan and profile drawings). The 2007 SWMP indicates that this information was supplied by the City as part of the model development process, which would suggest that this information is available. However, no such information has been supplied to the Project Team.

The previously prepared Technical Memorandum #2 (2017) had stated that mapping data on storm sewer outfalls were also not included in the information supplied by the City (at that time) and that if available, this information would be of use in establishing outfall characteristics (i.e. those to SWM facilities versus uncontrolled discharges to watercourses), in particular as part of an assessment of stormwater quality treatment. Subsequent reporting sections include a brief discussion on Water Quality Simulation as a potential model enhancement.

3.4 Overland Flow

Existing hydrologic \ hydraulic modelling for the City (as discussed further in Section 4) is all generally minor system (storm sewer) only, with the exception of the most recently completed modelling for the Ancaster community. As such, the modelling provides no direct insight into potential overland flow drainage issues and deficiencies.

To provide an initial high-level understanding of overland flow for the separated storm sewer area in the City, a GIS-based analysis has been undertaken. Areas identified as having a ponding depth of 0.3 m area have been assessed, given this is a typical threshold above which surface flooding is a concern, such as the depth of the municipal roadway right-of-way.

The analysis was completed in 2021 / 2022 and utilized the best publicly available Digital Elevation Model (DEM) at the time, namely the SWOOP 2015 dataset. This dataset was subsequently superseded by the Hamilton-Niagara 2021 DTM, however the data were not available at the time of the original assessment. Given the nature of the assessment, it was not considered necessary to revise or update the data to reflect the updated topography.

It should be noted that the bare earth DEM was not modified to “burn” through watercourse embankments (i.e. bridges and culverts). As such, watercourse areas are generally identified as having depression storage, when this may not be the case in reality. Where available, floodplain mapping limits (as available from area Conservation Authorities) have been included on the mapping to identify watercourse areas.

Regardless, the focus of the assessment is on urban areas and any areas which indicate accumulation of surface water.

Summary figures have been included in **Appendix G** (refer to Figures G1 to G7). Urban areas with notable depression storage\ sag points include:

- **Ancaster (Figure G1)**
 - a) Seminole Road, Nakoma Road Spring Trail and Cornwallis Road area
 - b) Thornwood Drive and April Court (rear yard areas)
 - c) Wilson Street at Fiddlers Green Road
 - d) Oneida Boulevard
- **Binbrook (Figure G2)**
 - a) Flinton Lane
 - b) Odonnell Drive
- **Dundas and West Hamilton (Figure G3)**
 - a) West Hamilton
 - i) Glenmount Avenue and Thorndale Street
 - ii) Between Emerson and Bowman Streets
 - b) Dundas
 - i) Tweedsmuir Avenue
 - ii) Pleasant Avenue
 - iii) Governors Road at Moss Blvd
- **Flamborough\Waterdown (Figure G4)**
 - a) Royaledge Way
 - b) Glaceport Ave and Culotta Drive (rear yard area)
- **Glanbrook (Figure G5)**
 - a) No particular areas of concern for Glanbrook
- **Hamilton Mountain (Figure G6)**
 - a) Green Cedar Drive and Guildwood Drive
 - b) Golden Orchard Drive and Brigadoon Drive
 - c) Bendamere Avenue
 - d) Hayden Street
 - e) Apple Blossom Drive to West Fifth Street
 - f) Stone Church Road west of Upper James Street
 - g) Upper Wentworth Street north of Stone Church Road

- h) Rowntree Drive, Upper Sherman and Rexford Drive
- i) Birchview Drive and Limeridge Road East
- j) Several sections of the Lincoln Alexander Parkway
- **Stoney Creek (Figure G7)**
 - a) Paramount Drive
 - b) National Drive
 - c) Areas between Arvin Avenue and the railway tracks (attributable to culverts not being represented in the DEM assessment)

The preceding areas may warrant further consideration or assessment, in combination with any available City records of historic flooding or drainage issues.

3.5 Open Channels and Watercourses

The previous 2007 SWMP included a limited assessment of open channels and watercourses. This work included a general characterization of each watershed through the development of fact sheets, as well as an assessment and prioritization of stream reaches for restoration activities, as part of the overall recommendations. Further as stated earlier, water quality impacts were considered in a lumped manner through the development of a mass balance model for watersheds as a whole.

The Project Team requested mapping of watercourse features (including regulated areas / floodplain extents, drainage boundaries, aquatic and terrestrial feature mapping) from the four local Conservation Authorities, which include the Hamilton Conservation Authority, Conservation Halton, the Grand River Conservation Authority, and the Niagara Peninsula Conservation Authority.

It is likely that there has been minimal change since the completion of the 2007 SWMP. Notwithstanding, the most current flood hazard mapping is generally available online directly from the subject Conservation Authorities.

It is not known whether any of the recommended stream restoration works identified in the previous SWMP have been designed or constructed; City staff has been unable to confirm. Based on the findings with respect to SWMF retrofits, it is considered likely that minimal stream restoration work has occurred since that time.

Overall, it is expected that watercourse feature mapping (hazards and regulated features) will be generally used to identify potential conflicts with planned development, growth, and intensification areas identified as part of GRIDS 2. Overall, given that the current study is focused on infill and intensification areas within the existing urban boundary, impacts to hazard lands are expected to be minimal.

Note that detailed hydrologic and hydraulic modelling for watercourse areas is not part of the current scope of work; rather the focus is on the development and use of the existing models for urban areas and infrastructure systems (i.e. MIKE+, as detailed in Section 4).

3.6 Reported Flooding

3.6.1 Data Supplied in 2017

Included in the information supplied by the City of Hamilton at the outset of this study (2017) was a GIS summary (shapefile) of sewer flooding dates and locations. This mapping includes a list of properties (by parcel) which have experienced sewer flooding. A separate layer, indicating the dates of compassionate grants by the City, has also been provided.

Based on the data provided, a total of 2,254 instances of sewer flooding have been reported within the separated storm sewer area (as per the data originally supplied in 2017). These instances are categorized as being either “EFLOD” (unclear terminology) or “SBU” or “SLBU” (assumed to be sewer backup or sewer line backup). Further clarification from City staff would assist in interpreting this information. It is assumed that the majority of these instances of flooding may be attributable to the sanitary sewer system (i.e. floor drain backup from sanitary sewer surcharging), however this cannot be definitively confirmed based on the currently available data. A more detailed summary distinguishing between sanitary and storm sewer system backup (i.e. in particular where foundation drainage is connected to the storm sewer system) would be useful to identify areas of higher relative risk / vulnerability, specific to the separated storm drainage system.

A summary map of the provided information has not been generated, as it is assumed that this information should be treated as confidential, given the potential impact to private property valuation. In general, however, a few clusters of frequent sewer flooding are evident from the available data; these include:

- East Mountain (Stone Church and Upper Gage, Limeridge and Upper Ottawa)
- Stoney Creek Mountain (between 2nd Road West and Upper Centennial and Highland and Rymal)
- Lower Stoney Creek (RHVP to Millen Road, the Niagara Escarpment to Barton Street)

As noted, the sewer flooding information summarized is listed by the parcel. It should be confirmed whether the City maintains a separate list of roadway flooding and storm

sewer surcharging, as this information would be valuable for model corroboration and to confirm areas of higher relative risk / vulnerability. The City of Hamilton operates a Storm Event Response Group (SERG) which may have such mapping available; this should be confirmed with City staff.

Several historic flooding dates are noted based on the information reported:

- August 19-25, 2005
- June 13 to 15, 2006
- July 26 to August 1, 2006
- December 1 to 7, 2006
- June 22 to 29, 2008
- July 11 to 28, 2008
- August 5 to 12, 2008
- July 26 to 31, 2009
- August 29 to September 4, 2009
- September 28 to October 6, 2010
- July 22 to 27, 2012
- Numerous dates between August 2012 and January 2013

There do not appear to be any additional reports beyond 2013, as such the provided database may be out of date (as it is likely that some flooding events have occurred between 2013 and 2017, and between 2017 and the current time (2025)).

It has been assumed that the range in dates may reflect the delay in residential reporting times. In general, the July 26, 2009 storm event in particular appears to have been the most formative on record (this storm event resulted in the flooding and closure of the Red Hill Valley Parkway and the QEW exceeding a 100-year return period for that area).

The City's GIS information also includes a list of backflow valves installed for private residences. This information would however be more directly applicable to the assessment of the sanitary and combined sewer system (since only foundation drainage would potentially be connected to the separated storm sewer system, no other residential services).

3.6.2 Supplemental Data

The City subsequently provided more current (2025) data based on its records of flood calls and reported incidents. The City provided a "heat map" of flood calls, aggregated

for all calls but rendered on the basis of number of calls by area. Results are presented in **Figure 3.6.1**.

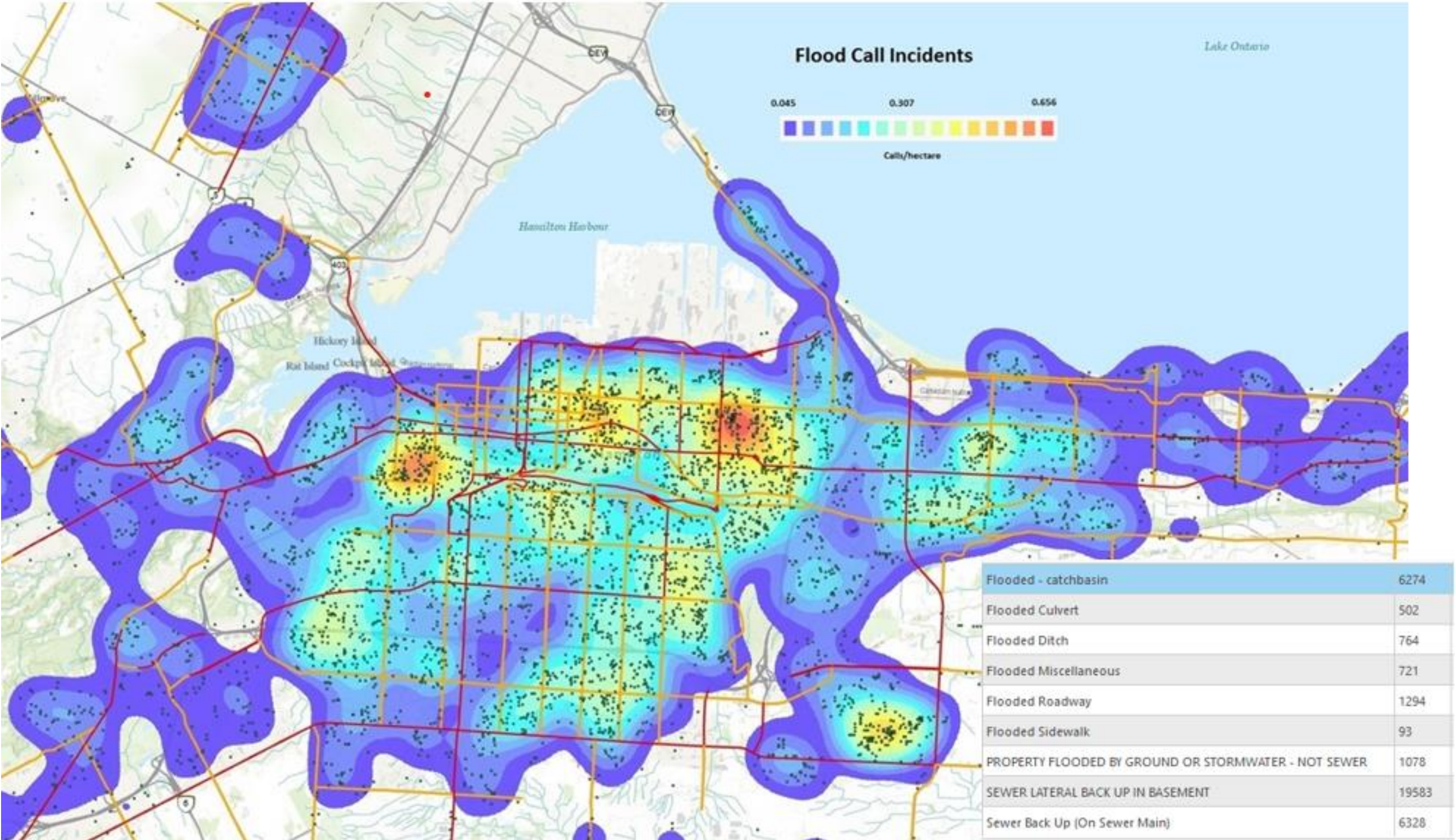
As evident from **Figure 3.6.1**, flood call incidents are particularly clustered in the lower portion of the City (i.e. the combined sewer area, which is outside the scope of the current study). Similarly, sewer backups would be expected to be primarily related to combined sewer or sanitary sewer impacts, as these are the services that would be directly connected to residences.

To facilitate the understanding of higher stormwater flood risk areas, the preceding data were filtered to remove sewer back-ups and sewer lateral back-ups. The balance of reported incidents would be expected to be focused on surface flooding (culverts, ditches, roadways, sidewalks). The resulting filtered data are presented in **Drawing 6**. For the purposes of the current study area of interest (separated stormwater area), areas of the City which reported higher surface flooding potential include:

- Upper Paradise Road area (Stone Church Road to Scenic Drive)
- Stone Church Road East area (Upper Sherman to Upper Ottawa)
- Lisgar Neighbourhood (south of Mohawk Road between Upper Ottawa and Upper Kenilworth Streets)
- Leckie Park Neighbourhood (First Road West and Highland Road West area)
- Downtown Stoney Creek (north of King Street East in vicinity of Lake Ave N)
- Gray Road between Queenston Road and Barton Street

The preceding areas should be considered in conjunction with development pressures and other factors (as discussed in Section 5) to guide the prioritization of potential drainage system improvements.

Figure 3.6.1: 2025 City Summary of Flood Call Incidents



3.7 Additional Information

The preceding review of available stormwater infrastructure data has provided an overall basis of understanding of the existing storm drainage system and is useful to understand prior to applying the hydrologic / hydraulic modelling of the storm sewer system (as detailed in Section 4).

A number of potential additional sources of information have been discussed in the preceding sections. The following additional information (not previously discussed) was noted as being of value in further understanding the existing separated stormwater system (if available):

- Foundation drain type mapping (i.e. areas where foundations are directly connected via gravity drainage as compared to areas on separated sump pumps)
- Rooftop downspout connection mapping (i.e. areas where rooftops are directly connected to the storm sewer system, or discharge to surface)

As per the previous review and discussion, it is understood that the City does not have any inlet control devices (ICDs) installed, thus there would be no mapping of these features.

Ultimately, the preceding information was determined to not be available from the City and has therefore not been considered further in the current study.

4 REVIEW OF EXISTING HYDROLOGIC/HYDRAULIC MODELS

The City currently has four hydrologic/hydraulic models of the separated storm sewer system:

- “Integrated_Six_City_MOUSE_Model” (referred to as “Six Cities model”)
- “558-Dundas_NOV2014_5Yr_CB” (referred to as “Dundas model”)
- “StoneyCreekStormSewer_Calibrated” (referred to as “Stoney Creek model”)
- “60507157-Ancaster-201910300Dual20201028_f_915_2100” (referred to as “Ancaster model”)

These models simulate the hydrologic (rainfall runoff) and hydraulic response of the stormwater drainage system for a range of natural and design storm events.

The current Six Cities model, as shown in **Drawing 7** (attached), covers an area of approximately 10,357 ha with 236 km of storm sewer pipes. The Six Cities model was originally developed as a skeletal model of the trunk storm sewer system for the six communities that now make up the City of Hamilton (including the former City of Hamilton, Town of Dundas, City of Stoney Creek, Town of Ancaster, Township of Glanbrook, and Town of Flamborough). The model was originally developed using DHI’s MOUSE modelling software package. In 2007, it was upgraded to DHI’s MIKE URBAN modelling software package and a more detailed pipe network was added in selected locations for the Hamilton Stormwater Master Plan project (Aquafor Beech Ltd, 2007). Neither the original MOUSE model nor the 2007 MIKE URBAN model was calibrated to any measured flow or water level data.

During the initial phase of the IMSS study (i.e. prior to 2019), the 2007 MIKE URBAN model was upgraded to run in the 2017 version of MIKE URBAN and the model setup was updated as per initial recommendations described in the Task 13 Technical Memorandum (DHI, 2019). During the recent phase of the IMSS study (i.e. after 2022), the 2017 MIKE URBAN model was upgraded to run in the 2025 version of MIKE+ and the model was updated to include the latest available stormwater infrastructure data.

The current Dundas model (refer to attached **Drawing 8**) covers an area of approximately 1,514 ha with 77 km of storm sewer pipes. The Dundas model was developed using DHI’s MIKE URBAN (2014) modelling software package and it was calibrated using rainfall and flow data collected in 2013 (Aquafor Beech Ltd, 2014). It includes the entire drainage pipe network and stormwater management facilities for the Dundas area. During the initial phase of the IMSS, the model was updated to run in the

2017 version of MIKE URBAN but no meaningful changes were made to the model setup. For the current phase of the IMSS, the Dundas model has been upgraded to run in the 2025 version of MIKE+ and the model was updated to include the latest available stormwater infrastructure data.

The Stoney Creek model (refer to attached **Drawing 9**) covers an area of approximately 2,460 ha with 233 km of storm sewer pipes discharging to major (overland) drainage systems (refer to **Drawing 9** attached). The Stoney Creek model was developed using DHI's MIKE URBAN (2011) modelling software package and it was calibrated using rainfall and flow data collected in 2012 (Dillon Consulting, 2014). It includes the entire drainage pipe network and stormwater management facilities for the Stoney Creek area. During the initial phase of the IMSS, the Stoney Creek model was upgraded to run in the 2017 version of MIKE URBAN and the model setup was updated as described in Task 13 Technical Memorandum (DHI, 2019). For the recent phase of the IMSS, the 2017 MIKE URBAN model was upgraded to run in the 2025 version of MIKE+ and the model was updated to include the latest available stormwater infrastructure data.

The Ancaster model (refer to attached **Drawing 10**) covers an area of approximately 1,582 ha with 112 km of storm sewer pipes and 124 km of overland flow channels (refer to **Drawing 10** attached). The Ancaster model was developed using DHI's MIKE URBAN (2019) modelling software package (AECOM, 2019). It is a dual drainage model with both the surface and pipe drainage networks represented. This model was not available during the initial phase of the IMSS. For the current phase of the IMSS, the Ancaster model has been upgraded to run in the 2025 version of MIKE+, and it was updated to include the latest available stormwater infrastructure data. A review of the model setup revealed inconsistencies in the assignment of the hydrologic parameters which could compromise the system capacity assessments (see Section 4.1.4).

The following sections provide a description of the hydrologic and hydraulic model setup for each of the Hamilton stormwater models.

4.1 Hydrology Model Setup

The Six Cities model, the Dundas model, the Stoney Creek model, and the Ancaster model use the Kinematic Wave hydrology model to calculate the rainfall runoff hydrograph generated by each catchment. The Kinematic Wave model provides a comprehensive representation of the main processes influencing rainfall runoff to the stormwater collection system (i.e. runoff from impervious areas, and runoff and infiltration from pervious areas).

Surface runoff is computed as shallow, laminar sheet flow taking into account the gravitational and friction forces as described in the following equation.

$$Q(t) = M * W * I^{1/2} * y_R(t)^{5/3}$$

Where:

M	=	Manning's roughness
W	=	Channel width (calculated as Area/Length)
I	=	Surface slope
$y_R(t)$	=	Runoff depth at time t

The amount of runoff is controlled by the size of the contributing area, the proportion of impervious cover in the contributing area, and the hydrological losses. The shape of the runoff hydrograph is controlled by the catchment parameters including the average drainage path length and slope, and the roughness of the catchment surface. These parameters form the basis of the kinematic wave computation (Manning's equation). A summary and brief description of all the parameters associated with Kinematic Wave model is presented in **Table 4.1.1**. The Primary Catchment Parameters are generally unique to each catchment and are typically the parameters adjusted during calibration, while the Secondary Catchment Parameters can often be summarized in fewer groupings of parameters where each group applies to a number of catchments. The secondary parameters are generally used for fine-tuning the model calibration.

Of these hydrology model parameters, the drainage path length is most frequently misunderstood. It is a conceptual (empirical) parameter rather than a parameter that can be physically measured. The reason it is conceptual is because the hydrology model assumes that all runoff from the catchment occurs as shallow, laminar sheet flow over a uniform length with a uniform depth across the entire catchment. In reality, the runoff only flows a relatively short distance as sheet flow before it is channelized in a ditch, a gutter or the sewer. In general, the length is larger for larger catchment areas but it is not directly proportional because it depends how well-serviced the catchment is with respect to engineered drainage systems. For example, a large residential catchment will have a much smaller drainage path length than a forested catchment of the same size. Given the empirical nature of the drainage path Length, it is typically determined during the model calibration process. It is one of the key 'calibration' parameters used to adjust the shape of the rainfall runoff curve (i.e. the longer the drainage path length, the more the peak runoff rate is attenuated).

Table 4.1.1: Summary Kinematic Wave Hydrology Model Parameters

Parameter	Description
Primary Catchment Parameters	
Length (m)	Conceptual length of surface drainage path.
Slope (%)	Average slope of surface drainage as shallow laminar sheet flow.
Steep Impervious Area (%)	Fraction of catchment surface containing steep impervious surfaces (e.g. residential rooftops)
Flat Impervious Area (%)	Fraction of catchment surface containing flat impervious surfaces (e.g. roads and parking lots)
High Pervious Area (%)	Fraction of catchment surface containing high permeability surfaces (e.g. gardens, sandy soils)
Medium Pervious Area (%)	Fraction of catchment surface containing medium permeability surfaces (e.g. silty soils, weathered till)
Low Pervious Area (%)	Fraction of catchment surface containing low permeability surfaces (e.g. clay soils)
Secondary Hydrologic Parameters	
Wetting loss [mm]	Initial wetting of the catchment surface
Storage loss [mm]	Precipitation depth required for filling the depressions on the catchment surface prior to runoff
Max infiltration [mm/hr]	The maximum rate of infiltration (Horton) for the specific surface type
Min infiltration [mm/hr]	The minimum rate of infiltration (Horton) for the specific surface type
Horton's Exponent [/hr]	Determines the rate of reduction of the infiltration rate reduction over time during rainfall
Inv. Horton's Exponent [/hr]	Determines the rate of recovery of the infiltration rate over time after rainfall has stopped
Manning's n [-]	Roughness of the catchment surface

The impervious area characterization allows each catchment area to represent impervious cover from steep sloping surfaces (e.g. residential rooftops) and flat surfaces (e.g. roads, driveways, parking lots). Depending on the proportional coverage of each type of surface in each catchment, the runoff response time can be influenced. However, in many cases, the high granularity of catchments combined with relatively sparse flow monitoring stations makes it difficult to identify this level of detail in the flow monitoring data. As a result, the total impervious area is often defined using only the flat impervious area. Similarly, the pervious area of each catchment can be defined using a combination of high, medium and low pervious cover, but the catchment areas are usually small enough that there is no significant change in the pervious soil type. So, the total pervious area is often defined using only a single pervious area type.

The following sections provide a summary of the hydrology model setup for the Six Cities model, the Dundas model, the Stoney Creek model and the Ancaster model.

4.1.1 Six Cities Hydrology Model

The Six Cities hydrology model consists of 4,896 catchments representing a total drainage area of 10,357 ha (104 km²). The individual catchment drainage areas range from .001 to 105 hectares. The unique IDs for the catchments are defined by the manholes to which they are connected.

% Imperviousness

The methodology used to estimate the % of impervious and pervious surface for each catchment is described in the Task 13 Technical Memorandum (DHI, 2019). It involved the development of representative % imperviousness values for steep (roofs) and flat (paved surface) impervious areas for 51 different land use types. The original estimates were then adjusted during calibration to achieve a reasonable match between the measured and modelled flows at the available flow monitoring locations.

The Low, Medium and Large Pervious areas for each catchment are determined based on soil classifications (DHI, 2019).

Drainage Path Length

The catchment drainage path Length values range from 1 to 746 m. A visual inspection of the distribution of catchment length values indicates it is consistent with expectations whereby residential areas are dominated by relatively short drainage path length values (<100 m) while the larger length values (> 500 m) are assigned to the larger, undeveloped catchments.

Catchment Slope

The slope values range from 0.001 to 7% and have been estimated based on the average slope of the sewer segments inside each catchment.

Secondary Hydrologic Parameters

The Six Cities model uses only one set of Secondary Hydrologic parameters for all catchments in the model. The parameter values are given in **Table 4.1.2**.

Table 4.1.2: Summary of Secondary Hydrologic Model Parameters for Six Cities Model

Parameter	Steep Impervious	Flat Impervious	Low Pervious	Medium Pervious	High Pervious
Wetting loss [mm]	0.05	0.05	0.05	0.05	0.05
Storage loss [mm]	-	0.6	1	1	2
Max infiltration [mm/hr]	-	-	15	36	72
Min infiltration [mm/hr]	-	-	5	3.6	18
Horton's Exponent [/hr]	-	-	5.4	5.4	5.4
Inverse Horton's Exponent [/hr]	-	-	0.018	0.036	0.18
Manning's n [-]	0.0125	0.014	0.033	0.033	0.083

4.1.2 Dundas Hydrology Model

The Dundas hydrology model consists of 897 catchments representing a total area of 1,514 ha (15.14 km²). The catchment drainage areas in the Dundas model range from 0.002 to 12 ha (with an additional 8 catchments having drainage areas ranging from 20 to 272 ha). The unique IDs for the catchments are the same as the manholes to which they are connected. The catchments are represented in the model using georeferenced shapes with most catchments delineated around 1 - 2 pipe segments.

The model was calibrated to 2013 rainfall and flow monitoring data as described in the report “Expansion and Completion of the Separate Storm Sewer System Modelling for Dundas Area by Using MIKE URBAN” (Aquafor Beech Ltd, 2014). Neither the rainfall nor the flow monitoring data associated with the model calibration were available to check the existing model setup and results against the data.

% Imperviousness

The majority of catchments contain values for both the Steep and Flat Impervious Area with the remaining catchment area (i.e. the pervious area) assigned to Low, Medium or High Pervious areas. The majority of catchments are located in residential areas and have a combined (i.e. Flat and Steep) impervious area value of less than 20% of the catchment area. This calibrated imperviousness is lower than expected for residential areas but the report shows the model has been calibrated and is producing results that match the flow measurements.

Drainage Path Length Calculations

The drainage path Length values range from 12 to 40,000 m. In general, Length values are typical of what would normally be expected in a detailed urban drainage model. The exceptions are the catchments with the drainage areas larger than 2.0 hectares. These

catchments are generally unserviced greenspaces draining into the storm sewer system. In many of these catchments, the drainage path length is much greater than the actual length of the catchment itself and should be revisited since it could significantly over-estimate peak flow attenuation in these catchments.

Secondary Hydrologic Parameters

The Dundas model included 2 different sets of Secondary Hydrologic parameters, but both sets of data were identical. It appears as though there may have been some intention of developing different parameter sets based on different soil types but it has not been implemented in the current version of the model. The Secondary Hydrologic parameter values are indicated in **Table 4.1.3**.

Table 4.1.3: Summary of Secondary Hydrologic Model Parameters for Dundas Model

Parameter	Steep Impervious	Flat Impervious	Low Pervious	Medium Pervious	High Pervious
Wetting loss [mm]	1.0	3.0	6.0	8.0	0.05
Storage loss [mm]	-	1.0	2.0	4.0	2.0
Max infiltration [mm/hr]	-	-	125	200	72
Min infiltration [mm/hr]	-	-	5	13	18
Horton's Exponent [/hr]	-	-	5.4	5.4	5.4
Inverse Horton's Exponent [/hr]	-	-	0.018	0.036	0.18
Manning's n [-]	0.0125	0.014	0.033	0.033	0.083

4.1.3 Stoney Creek Hydrology Model

The Stoney Creek hydrology model consists of 3,022 catchments representing a total area of 2,460 ha (24.6 km²). The catchment drainage areas range from 0.01 to 26 hectares, whereby most catchments are delineated around 1 - 2 pipe segments. The setup for the Stoney Creek model does not follow the typical naming conventions for Hamilton's collection system model whereby the Catchments are named according to the manholes to which they are connected. In the Stoney Creek model, the unique IDs for the catchments are not the same as the manholes to which they are connected, for example catchment "SC-1006" is connected to manhole "HQ08B061".

% Imperviousness

The model uses only the Flat impervious area and the Medium pervious area values to describe the pervious nature of the catchments. The impervious area was initially estimated using default values for specific land use types as per the Task 13 Technical Memorandum (DHI, 2019) while the pervious area was calculated as the remaining area of the catchment. The imperviousness of residential catchments is generally within the expected range from 25 - 35% of the catchment area.

Drainage Path Length Calculations

The drainage path Length values range from 6 to 439 m and are within the expected range for well-serviced and unserviced catchments throughout the model domain.

Remaining Hydrologic Parameters

The Stoney Creek model includes four different sets of Secondary Hydrologic parameters, but only one of these, the “Brampton” set was used in the model. The Secondary Hydrologic parameter values are shown in **Table 4.1.4**.

Table 4.1.4: Summary of Secondary Hydrologic Model Parameters for Stoney Creek Model

Parameter	Steep Impervious	Flat Impervious	Low Pervious	Medium Pervious	High Pervious
Wetting loss [mm]	0.05	0.05	0.05	0.05	0.05
Storage loss [mm]	-	0.6	1.0	1.0	2.0
Max infiltration [mm/hr]	-	-	3.6	75	72
Min infiltration [mm/hr]	-	-	1.8	7.5	18
Horton's Exponent [/hr]	-	-	5.4	2	5.4
Inverse Horton's Exponent [/hr]	-	-	0.018	0.036	0.18
Manning's n [-]	0.0125	0.014	0.033	0.033	0.083

4.1.4 Ancaster Hydrology Model

The Ancaster hydrology model consists of 1,600 catchments representing a total drainage area of 1,582 ha (15.82 km²). The catchment drainage areas in the Ancaster model range from 0.022 to 79 ha. The catchments are named after the minor system manhole to which they are connected. The general naming convention is ‘cat_[minor system manhole ID]_[connection index]’ where ‘connection index’ is the integer index number for all contributing catchments when there are multiple catchments connected to the same manhole.

The catchment delineation appears to have been done manually – possibly according to stormwater drainage designs (no description of how the catchment were delineated was provided in the AECOM report). The catchment boundaries generally follow roads and storm sewers. In residential areas, the resolution of the catchment delineation is often set up with one catchment per manhole, but there are many places where this is not followed. In addition, some large green spaces were subdivided into catchments with straight lines that appear to be somewhat arbitrary. However, in general, the catchment delineation appears to follow logical drainage patterns and is considered acceptable.

% Imperviousness

The model uses both the Steep and Flat Impervious as well as the Low, Medium and High Pervious areas to characterise the catchments. According to the AECOM report (AECOM, 2019), the initial estimates of catchment imperviousness were calculated based on land use maps while the pervious area allocations were determined based on a surficial soils map. The model was calibrated by adjusting the imperviousness of the catchments contributing to the flow monitoring station but the catchments outside of the flow monitoring station contributing area were not adjusted from the original estimates. As a result, the calibrated imperviousness of residential catchments ranges from 10% to more than 50% of the catchment area throughout the model.

Slope

The slope of each catchment was calculated using a DEM (AECOM, 2019). The values range from 0.1% to 15.86%.

Drainage Path Length Calculations

The drainage path Length values range from 2 to 541 m and are within the expected range for well-serviced and unserviced catchments throughout the model domain.

Secondary Hydrologic Parameters

The Ancaster model included 17 different sets of Secondary Hydrologic Parameters. The maximum and minimum infiltration rate of each subcatchment was first estimated based on the corresponding soil type and the results were grouped into 17 sets of parameters (AECOM, 2019).

The maximum infiltration rate ranges from 80 mm/hr to 150 mm/hr while the minimum infiltration rates vary from 3.3 to 13.1 mm/hr. These ranges are relatively consistent with the other models.

The Secondary Hydrologic parameter values are summarized in **Table 4.1.5**.

Table 4.1.5: Summary of Secondary Hydrologic Model Parameters for Ancaster Model

Parameter	Steep Impervious	Flat Impervious	Low Pervious	Medium Pervious	High Pervious
Wetting loss [mm]	0.05	0.05	0.05	0.05	0.05
Storage loss [mm]	-	2.0	3.0	5.5	7.2
Max infiltration [mm/hr]	-	-	80	83 - 150	131 - 150
Min infiltration [mm/hr]	-	-	3.3 - 6.6	4.5 - 13.1	9.9 - 13.1
Horton's Exponent [/hr]	-	-	5.4	5.4	5.4
Inverse Horton's Exponent [/hr]	-	-	0.018	0.036	0.18
Manning's n [-]	0.0125	0.014	0.033	0.033	0.083

Model Calibration

The model was calibrated at four flow monitoring sites. The adjusted parameters include Horton's infiltration, storage loss, impervious percentage, evapotranspiration, and flow length. Two calibration events and one validation event were used for calibration at each site. The calibrated model results provided in the AECOM report (AECOM, 2019) demonstrate a good fit between the measured and modelled flows. However, the adjustments made to the hydrologic parameters were only applied to the catchments contributing to each flow monitoring site. This results in significant inconsistencies in imperviousness between two adjacent catchments even though they have the same land use.

An example of this difference is shown in **Figure 4.1.1** for the Site 3 flow monitoring station. The storm sewers upstream of the Site 3 station are highlighted in green and the catchments are color-coded based on the total % imperviousness value (i.e. Steep and Flat impervious). Most of the catchments in the Site 3 drainage area are single-family residential with an imperviousness value below 20% of the area, while the neighboring catchments with essentially the same residential land use have percent imperviousness values greater than 50%. This type of inconsistency is evident throughout the model domain and will result in inconsistent and unreliable system capacity assessments.

Unfortunately, this problem was relatively discovered late in the project and there was not sufficient time to address it as part of the current study; however, this should be revisited by the City as part of future work.

Figure 4.1.1: Channel Cross-Sections Definitions use for Cross-Sections



4.2 Hydraulic Model Setup

The Six Cities model, the Dundas model, the Stoney Creek model and the Ancaster model all use the MIKE1D numerical engine to calculate the pipe flow hydraulics in the storm sewer system. The numerical engine solves the complete St. Venant (dynamic flow) equations throughout the drainage network (looped and dendritic) and allows for modelling of backwater effects, flow reversal, surcharging in manholes, free-surface and pressure flow, tidal outfalls and storage basins. The flow in the hydraulic model is generated by rainfall runoff from the catchment hydrology model.

The Six Cities, Dundas and Stoney Creek models are only representing the minor drainage system (i.e. underground pipes) while the Ancaster model is a dual drainage model that represents both major (i.e. roadways and ditches) and minor system drainage. A summary of the hydraulic model network features for each model is provided in **Table 4.2.1**, while a breakdown of the size, length and volume of circular pipes is provided in **Table 4.2.2**.

Table 4.2.1: Summary of Model Network Features

Type	Six Cities	Dundas	Stoney Creek	Ancaster
Manholes	4640	1399	4071	2125
Pipes	4711	1442	4153	2093
Overland Node	0	0	0	1932
Overland Channel	0	0	0	2164
Curb Inlet	0	0	0	1931
Basins	85	5	13	19
Outfalls	321	116	112	170
Pumps	0	0	0	0
Weirs - V-Notch	56	3	0	0
Weirs - Rectangular	2	0	3	8
Orifices - Circular	64	1	3	7
Orifices - Rectangular	4	1	0	6

Table 4.2.2: Summary of Pipe Length and Volume

Diameter [m]	Six Cities Model		Dundas Model		Stoney Creek Model		Ancaster Model	
	Length [km]	Volume [m³]	Length [km]	Volume [m³]	Length [km]	Volume [m³]	Length [km]	Volume [m³]
0-0.25	0.2	6	1.1	34	1.8	60	0.2	6
0.25-0.5	10.3	1,144	52	5,135	96.5	10,856	51.6	5,571
0.5-0.75	64.8	18,705	14.1	3,715	60.4	16,949	25.5	6,961
0.75-1	63.8	36,002	5.5	2,908	33.9	18,591	18.2	10,099
1-1.25	51.2	52,080	2.1	2,241	19.1	19,060	9.6	10,084
1.25-1.5	20	28,549	1.1	1,583	5.1	7,254	2.2	3,102
1.5-1.75	19.5	36,807	0.3	620	6.6	12,161	0.9	1,593
1.75-2	17.3	46,250	0.2	446	5.1	13,045	1.9	4,906
2-2.25	4	13,876	0.1	213	1.5	5,106	0.5	1,892
2.25-2.5	7.7	31,250	0.03	136	3.1	13,139	1.1	4,840
2.5-2.75	3	16,534	0.2	1,268	-	-	-	-
2.75-3	1.5	9,675	0.02	139	0.2	844	-	-
3-3.25	1.2	6,982		-	0.4	1,761	0.01	72
3.25-3.5	-	-	-	-	-	-	-	-
3.5-3.75	1	10,035	-	-	-	-	-	-
3.75-4	1	11,661	-	-	-	-	-	-
4-4.25	0.4	4,177	0.1	817	-	-	-	-
4.25-4.5	-	-	-	-	-	-	-	-
4.5-4.75	0.4	7,435	-	-	-	-	-	-

Table 4.2.2: Summary of Pipe Length and Volume - Continued

Diameter [m]	Six Cities Model		Dundas Model		Stoney Creek Model		Ancaster Model	
	Length [km]	Volume [m ³]	Length [km]	Volume [m ³]	Length [km]	Volume [m ³]	Length [km]	Volume [m ³]
4.75-5	-	-	-	-	-	-	-	-
5-5.25	-	-	-	-	-	-	-	-
5.25-5.5	3.1	71,311	-	-	-	-	-	-
Total	270.4	402,479	76.8	19,255	233.7	18,826	111.7	49,126

4.2.1 Six Cities Hydraulic Model

The Six Cities model was originally a skeletal model of only the trunk sewer system (i.e. pipes with a diameter larger than 0.525 m) but additional detail was added for the 2007 Hamilton Stormwater Master Plan project (Aquafor Beech Ltd, 2007). For the current IMSS, the model was updated to reflect the latest available stormwater infrastructure data. The following is a summary and discussion of the main elements of the model.

Pipes

Of the 4,711 pipes, 108 are rectangular in shape while the remaining pipes are circular. As indicated in **Table 4.2.2**, only 10.5 km of the sewer (3.9% of the total length of sewer) has a diameter less than 0.5 m and a total in-pipe storage of 1,150 m³ (less than 0.3% of the total available in-pipe storage). Pipes with a diameter between 0.5 to 1.25 m make up almost 66% of the total length of the sewer in the model but only provides 27% of the in-pipe storage, while pipes with a diameter of 5.4 m make up a length of less than 1.1% but provide 18% of the available in-pipe storage.

The pipes range in length from 0.1 to 300 m. The pipes with a specified length of 0.1 m are fictitious pipes used to connect the stormwater management pond basins to the sewer system. In reality, the sewer system outlets to the edge of the pond so this is where the linear asset data stops. However, the basin node that is used to represent the pond is located in the middle of the pond area, so the fictitious pipe is used to connect the end of the sewer pipe to the basin node.

A summary of the pipe materials is provided in **Table 4.2.3**.

Table 4.2.3: Summary of Pipe Materials for Six Cities Model

Material ID	Manning's n	# of pipes
Concrete (Smooth) MOUSE	0.0118	3,727
Concrete (Normal) MOUSE	0.0133	2
Other MOUSE	0.0200	1
Plastic MOUSE	0.0125	5
PVC	0.0125	7
RCP	0.0133	969

Manholes

The available storage in each individual manholes is accounted for in the model, and the size of the manhole is also used in the calculation of entry and exit head losses. The diameter of the manholes represented in the model range from 1.0 m to 5.4 m.

The Maximum Inflow control is used for manholes connected to catchments whereby a maximum inflow rate is calculated as 0.03 m³/s per catchbasin located inside the connected catchment area.

The majority of the manholes calculate the head loss using the “MOUSE Classic (Engelund)” setting, while head losses for manholes connected to pipes with a diameter of 5.4 m or greater were ignored by selecting the “MOUSE_No_CRS_Changes_1” setting.

All of the manholes use a “Normal” setting for the manhole cover. The Normal cover setting means that any surcharge occurring above the rim of the manhole will be stored in a conceptual large diameter basin located immediately above the surcharged manhole. While this setting ensures that all rainfall runoff eventually runs through the collection system, the additional build-up of hydraulic head at the surcharged manholes can lead to higher hydraulic head conditions upstream of these locations. This is not necessarily an error in the model setup but, rather, it is just something to be aware of when interpreting the results. Ideally, if there is a significant amount of manholes surcharging, the model should be extended to a dual drainage model to properly account for major and minor drainage.

Stormwater Management Facilities (SWMF)

The Six Cities model includes 85 SWMFs (only a portion of the total SWMFs which the City operates as per the previously presented inventory), which are represented in the model as a Basin node with a specified Level-Area curve used to calculate the available storage volume. Spilling from the ponds is represented using a weir that either connects to another pond, or its outlets externally from the model. Outlet control devices in the

ponds are represented using an orifice. A summary of the stormwater management pond type and storage volume is provided in **Table 4.2.4**.

A comparison of the modelled SWMF features (e.g. storage volume, bottom and top elevation, outlet control devices) vs. the design specifications was not possible due to lack of information in the data provided for this study and the limited scope of the project. However, the model was checked against available documents and drawings, when possible, to clarify apparent inconsistencies in the data.

It should be noted that not all SWMFs were defined the same way. Some used a weir with a crest level at the top of the pond geometry curve to effectively 'spill' the flooding, while other SWMFs allowed the flooding to accumulate in a virtual storage area above the top of the pond. Both approaches are potentially valid depending on the drainage conditions at the site, but a consistent approach is recommended to facilitate the interpretation and reporting of the results.

Boundary Conditions

All of the 59 stormwater outlets that discharge into Lake Ontario are associated with a constant water level boundary condition with a constant value of 75.2 m (longer term average water level of Lake Ontario). All remaining outlets that are discharging into a creek or river and are defined as outlets and use a free outflow boundary condition by default.

In practice, the creeks and rivers will have elevated water levels during a design storm event, and this may cause backwater conditions at the outlets and create flooding and surcharging in the storm sewer system. However, defining the outlet water levels associated with each design storm event is beyond the scope of this study and is recommended for future considerations.

Model Calibration

In 2019, the Six Cities model was updated as described in the Task 13 Technical Memorandum (DHI, 2019), including calibrating the model to measured rainfall and flow from the 2012 Stoney Creek monitoring campaign. No other flow monitoring data were available at the time. The calibrated model showed a good agreement between the simulated and measured flow data within the Stoney Creek area (DHI, 2019). In the absence of any other relevant flow and rainfall monitoring data, the calibrated % imperviousness values associated with the relevant land uses in the Stoney Creek area were applied to the catchments throughout the entire Six Cities model.

Table 4.2.4: Summary of Ponds and Connected Weirs and Orifices in the Six Cities Model Basins

Basins						Weirs				Orifices						
Model ID	Type	Invert level (m)	Ground level (m)	Top of geometry (m)	Volume geometry based (m³)	Model ID	Down-stream node	Crest Level (m)	Crest Width (m)	Model ID	Down-stream node	Invert Level (m)	Shape	Diameter (m)	Height (m)	Width (m)
AH12SP01	Dry	239.54	244.04	244.04	17000	AH12SP01_W1	spill	242.85	-	AH12SP01_O1	spill	239.54	Circular	0.3	-	-
AH12SP01	Dry	239.54	244.04	244.04	17000	AH12SP01_W1	spill	242.85	-	AH12SP01_O2	Spill	240.54	Circular	0.2	-	-
AH12SP02	Dry	240.4	242.4	242.1	2800	AH12SP02_W1	spill	242.1	5	AH12SP02_O1	spill	240.4	Circular	0.3	-	-
AH18SP01	Wet	217.18	220.18	220.18	54600	AH18SP01_W1	spill	219.08	-	AH18SP01_O1	spill	217.18	Circular	0.825	-	-
AI08SP01	Wet	194.98	202.52	202.52	67300	AI08SP01_W1_Overflow	spill	200.38	7.66	AI08SP01_O1	AI08B030	196.31	Circular	0.11	-	-
AI08SP01	Wet	194.98	202.52	202.52	67300	AI08SP01_W2_Emergency	spill	200.77	4.46	AI08SP01_O1	AI08B030	196.31	Circular	0.11	-	-
AI08SP01	Wet	194.98	202.52	202.52	67300	AI08SP01_W3_Overflow	spill	200.66	1.35	AI08SP01_O1	AI08B030	196.31	Circular	0.11	-	-
AI13SP01	Dry	242.11	246.67	246.67	6500	AI13SP01_W1	spill	246.17	10	AI13SP01_O1	spill	242.11	Circular	0.525	-	-
AJ09SP01	Dry	227.5	229.83	229.53	900	AJ09SP01_W2	spill	229.5	100	AJ09SP01_O1	spill	227.5	Circular	0.28	-	-
AJ13SP01	Dry	245.27	247.38	247.38	5600	AJ13SP01_W1	spill	247.38	10	-	-	-	-	-	-	-
AJ13SP02_1	Dry	238.5	242	242	19495	AJ13SP02_1_W1	spill	239.8	-	AJ13SP02_1_O1	spill	238.5	Circular	0.075	-	-
AJ13SP02_1	Dry	238.5	242	242	19495	AJ13SP02_1_W1	spill	239.8	-	AJ13SP02_1_O1	spill	238.5	Circular	0.075	-	-
AJ13SP02_2	Dry	238.4	240.4	240.4	2943	AJ13SP02_2_W1	spill	239.8		AJ13SP02_2_O1	spill	238.5	Circular	0.25	-	-
AJ13SP03	Dry	244.31	246.36	246.06	570	AJ13SP03_W1	spill	246.02	10	-	-	-	-	-	-	-
AK06SP01	Dry	182.15	182.85	182.55	275	AK06SP01_W2	spill	182.55	10	AK06SP01_O1	spill	182.15	Circular	0.25	-	-
AK09SP01	Dry	239.7	240.87	240.87	345	AK09SP01_W1	spill	240.87	10	AK09SP01_O1	spill	239.7	Circular	0.3	-	-
AK12SP01	Dry	246.35	247.98	247.98	1800	AK12SP01_W1	spill	247.98	10	AK12SP01_O1	spill	247.29	Circular	0.3	-	-
AK12SP01	Dry	246.35	247.98	247.98	1800	AK12SP01_W1	spill	247.98	10	AK12SP01_O2	spill	246.36	Circular	0.3	-	-
AK13SP01	Dry	245.6	246.505	246.505	5500	AK13SP01_W1	spill	246.5	-	AK13SP01_O1	spill	245.86	Circular	0.1	-	-
AL07SP01	Dry	223.82	225.57	225.57	1350	AL07SP01_W1	spill	225.43	5	-	-	-	-	-	-	-
AL12SP01	Dry	236.49	240.8	240.8	51000	AL12SP01_W1	spill	238.98	-	AL12SP01_O1	spill	236.49	Circular	1.6	-	-
AM05SP01	Dry	188	191	191	19700	AM05SP01_W1	spill	189	4.5	AM05SP01_O1	spill	188	Circular	0.3	-	-
AM07SP01	Dry	228.27	231.43	231.43	2200	AM07SP01_W1	spill	228.99	-	AM07SP01_O1	spill	228.47	Circular	0.3	-	-
AM10SP01	Dry	236.66	238.66	238.36	990	AM10SP01_W1	spill	237.65	-	AM10SP01_O1	spill	236.66	Circular	0.525	-	-
AM10SP02	Dry	235.32	241.72	241.72	1400	AM10SP02_W1	spill	241.72	10	-	-	-	-	-	-	-
AN07SP01	Dry	220	225	225	82000	AN07SP01_W1	spill	225	10	AN07SP01_O1	AN07SP02	220	Rectangular		2	3.4
AN07SP02	Dry	219.48	225.35	225.35	288000	AN07SP02_W1	spill	221.32	-	AN07SP02_O1	spill	219.48	Circular	1.1	-	-
AO04SP01	Wetland	195.46	203.41	203.41	196000	AO04SP01_W1	spill	201.91	-	AO04SP01_O1	spill	195.46	Circular	0.3	-	-
AO07SP01	Dry	220.5	225.3	225	43500	AO07SP01_W1	spill	225	10	AO07SP01_O1	AN07SP01	220.5	Rectangular		2	3.4
AO07SP02_Qual	Wet	221.59	222.89	222.59	2642	AO07SP02_Qual_W1	AO07SP02_Quan	222.59	37	AO07SP02_Qual_O1	AO07SP02_Quan	222.59	Circular	0.525	-	-
AO07SP02_Qual	Wet	221.59	222.89	222.59	2642	AO07SP02_Qual_W1	AO07SP02_Quan	222.59	37	AO07SP02_Qual_O2	AO07SP02_Quan	222.59	Circular	0.525	-	-
AO07SP02_Qual	Wet	221.59	222.89	222.59	2642	AO07SP02_Qual_W1	AO07SP02_Quan	222.59	37	AO07SP02_Qual_O3	AO07SP02_Quan	222.59	Circular	0.525	-	-
AO07SP02_Quan	Wet	220.87	222.57	222.27	5275	AO07SP02_Quan_W1	AO07SP01	222.27	50	AO07SP02_Quan_O1	AO07B026	222.27	Circular	0.3	-	-
DE09SP01	Dry	123.9	127.5	127.5	30000	DE09SP01_W1	spill	125.5	-	DE09SP01_O1	spill	123.9	Rectangular	0.49	0.7	0.7

Table 4.2.4: Summary of Ponds and Connected Weirs and Orifices in the Six Cities Model Basins - Continued

Basins						Weirs				Orifices						
Model ID	Type	Invert level (m)	Ground level (m)	Top of geometry (m)	Volume geometry based (m³)	Model ID	Down-stream node	Crest Level (m)	Crest Width (m)	Model ID	Down-stream node	Invert Level (m)	Shape	Diameter (m)	Height (m)	Width (m)
DE10SP01	Dry	136.01	139.25	139.25	3100	DE10SP01_W1	spill	138.87	-	DE10SP01_O1	spill	136.01	Circular	0.75	-	-
DH05SP01	Dry	92.88	94.88	94.88	2800	DH05SP01_W1	DH05B032	93.85	10	-	-	-	-	-	-	-
FA06SP01	Dry	264.9	268.04	268.04	9250	FA06SP01_W1	spill	268.04	10	-	-	-	-	-	-	-
FB06SP01	Wet	260.05	261.2	260.9	470	FB06SP01_W2	spill	260.85	100	FB06SP01_O1	spill	260.05	Circular	0.1	-	-
FB06SP02	Wet	263.4	265.5	265.5	4200	FB06SP02_W1	spill	264.3	-	FB06SP02_O1	spill	263.4	Circular	0.1	-	-
FB09SP01	Dry	254	255	255	1100	FB09SP01_W1	spill	254.01	-	-	-	-	-	-	-	-
FB09SP02	Dry	254	257.14	257.14	9250	FB09SP02_W1	spill	254.01	-	-	-	-	-	-	-	-
FC08SP01	Dry	255.4	257.4	257.4	2100	FC08SP01_W1	spill	255.8	-	FC08SP01_O1	spill	255.4	Circular	0.1	-	-
FD36SP01	Wet	226.16	229.3	229.3	9250	FD36SP01_W1	spill	228.81	30	FD36SP01_O1	spill	226.16	Circular	0.15	-	-
FK25SP01	Wet	212.9	214.2	214.2	16000	FK25SP01_W1	spill	213.9	-	FK25SP01_O1	spill	213	Circular	0.7	-	-
FL22SP01_1	Dry	217.9	219.5	219.5	3008	FL22SP01_1_W1	spill	219.5	-	FL22SP01_1_O1	spill	217.9	Circular	0.093	-	-
FL22SP01_2	Dry	218.39	219.44	219.44	1853	FL22SP01_2_W1	spill	219.2	-	FL22SP01_2_O1	spill	218.45	Circular	0.2	-	-
FL22SP02_1	Dry	220.6	222.2	222.2	1818	FL22SP02_1_W1	spill	222.05	-	FL22SP02_1_O1	spill	220.6	Circular	0.068	-	-
FL22SP02_2	Dry	220.97	222.17	222.17	1643	FL22SP02_2_W1	spill	221.8	-	FL22SP02_2_O1	spill	221	Circular	0.2	-	-
FM21SP01	LID	223.67	225.17	225.17	200	FM21SP01_W1	spill	225.17	10	FM21SP01_O1	spill	223.68	Circular	0.08	-	-
FM21SP01	LID	223.67	225.17	225.17	200	FM21SP01_W1	spill	225.17	10	FM21SP01_O2	spill	224.28	Circular	0.15	-	-
FM21SP01	LID	223.67	225.17	225.17	200	FM21SP01_W1	spill	225.17	10	FM21SP01_O3	spill	224.28	Circular	0.15	-	-
FM21SP01	LID	223.67	225.17	225.17	200	FM21SP01_W1	spill	225.17	10	FM21SP01_O4	spill	224.68	Circular	0.3	-	-
FT10SP01	Dry	273	274.2	274.2	400	FT10SP01_W1	spill	274.2	10	-	-	-	-	-	-	-
FT11SP01	Dry	273.75	275.85	275.85	14000	FT11SP01_W1	spill	274.2	-	FT11SP01_O1	spill	273.75	Circular	0.3	-	-
FW10SP01	Dry	276.75	278.93	278.93	2300	FW10SP01_W1	spill	278.45	-	FW10SP01_O1	spill	276.75	Circular	0.3	-	-
GB01SP01	Dry	221.06	222.36	222.36	13258	GB01SP01_W1	spill	222.36	10	-	-	-	-	-	-	-
GC01SP01_Qual	Wet	221.05	223.95	223.95	9878	GC01SP01_Qual_W1	GC01SP01_Quan	223.55	20	GC01SP01_Qual_O1	GC01SP01_Quan	222.55	Circular	0.31	-	-
GC01SP01_Quan	Wet	222.13	224.17	224.17	15780	GC01SP01_Quan_W1	spill	223.11	10	GC01SP01_Quan_O1	spill	222.13	Circular	0.75	-	-
GD06SP01	Dry	226.75	228	228	3800	GD06SP01_W1	spill	227.5	-	GD06SP01_O1	spill	226.75	Circular	0.25	-	-
GE02SP01	Dry	220.06	223.62	223.62	14500	GE02SP01_W1	GE02B023	221.74	0.61	-	-	-	-	-	-	-
GP14SP01	Wet	205.94	206.7	206.7	19300	GP14SP01_W1	spill	206.7	10	GP14SP01_O1	spill	206	Circular	0.15	-	-
GP14SP02	Wet	201.6	205.25	205.25	16500	GP14SP02_W1	GQ14SP01	204.2	15	GP14SP02_O1	GQ14SP01	203.1	Circular	0.11	-	-
GP14SP03	Wet	200.85	204.5	204.5	16500	GP14SP03_W1	GQ14SP01	204.2	15	GP14SP03_O1	GQ14SP01	202.6	Circular	0.26	-	-
GQ14SP01	Dry	201.5	205.5	205.5	165000	GQ14SP01_W1	spill	202.5	-	GQ14SP01_O1	spill	201.5	Circular	0.45	-	-
HD14SP01	Dry	190.801	194.801	194.801	22300	HD14SP01_W1	spill	192.43	-	HD14SP01_O1	HD14B087	190.81	Circular	0.18	-	-
HE18SP01	Dry	215.43	221.43	221.43	5000	HE18SP01_W1	spill	221.43	10	-	-	-	-	-	-	-
HG21SP01	Wet	228	229.8	229.8	2500	HG21SP01_W1	spill	229.15	-	HG21SP01_O1	spill	228.6	Circular	0.525	-	-
HG22SP01	Wet	219	222.7	222.7	11400	HG22SP01_W1	spill	220	3	-	-	-	-	-	-	-
HI18SP02	Dry	200.95	204.45	204.45	18200	HI18SP02_W1	spill	204.45	10	-	-	-	-	-	-	-

Table 4.2.4: Summary of Ponds and Connected Weirs and Orifices in the Six Cities Model Basins - Continued

Basins						Weirs				Orifices						
Model ID	Type	Invert level (m)	Ground level (m)	Top of geometry (m)	Volume geometry based (m³)	Model ID	Down-stream node	Crest Level (m)	Crest Width (m)	Model ID	Down-stream node	Invert Level (m)	Shape	Diameter (m)	Height (m)	Width (m)
HN18SP01	Wetland	180	182.9	182.9	13000	HN18SP01_W1	spill	181.5	5	HN18SP01_O1	spill	181.5	Circular	0.3	-	-
HN18SP01	Wetland	180	182.9	182.9	13000	HN18SP01_W1	spill	181.5	5	HN18SP01_O2	spill	181.5	Circular	0.3	-	-
NEW_001	-	215.89	218.156	218.156	1317	NEW_001_W1	spill	217.68	1	NEW_001_O1	spill	215.89	Circular	0.4	-	-
NEW_002	Wet	220.9	224.9	224.9	40133	NEW_002_W1	spill	224.9	10	-	-	-	-	-	-	-
NEW_003	Wet	219.4	225.9	225.9	40635	NEW_003_W1	spill	225.9	10	-	-	-	-	-	-	-
NEW_004	Wet	219.9	225.15	225.15	69000	NEW_004_W1	spill	225.15	10	-	-	-	-	-	-	-
NEW_005	Wet	220.4	224.65	224.65	57750	NEW_005_W1	spill	224.65	10	-	-	-	-	-	-	-
NEW_033	-	231.59	235.03	234.73	9249	NEW_033_W1	spill	234.73	10	NEW_033_O1	spill	231.59	Circular	0.525	-	-
SB16SP01	Dry	188	193.5	193.5	44000	SB16SP01_W1	spill	191	2.4	SB16SP01_O1	spill	188	Rectangular	-	1.2	1.8
SB18SP01	Wet	206.3	209.15	209.15	11800	SB18SP01_W1	spill	208.4	-	-	-	-	-	-	-	-
SD17SP01	Wet	208.92	210.02	210.02	4000	SD17SP01_W1	spill	210.02	-	SD17SP01_O1	spill	208.92	Circular	0.7	-	-
SD19SP01	Wet	211.13	213.125	213.125	10091	SD19SP01_W1	spill	212.08	-	SD19SP01_O1	spill	211.13	Circular	0.525	-	-
SI07SP01	Dry	94	97	97	13500	SI07SP01_W1	spill	94.55	-	SI07SP01_O1	spill	94	Circular	0.8	-	-
SI07SP01	Dry	94	97	97	13500	SI07SP01_W1	spill	94.55	-	SI07SP01_O2	spill	94.55	Circular	1.2	-	-
SI07SP01	Dry	94	97	97	13500	SI07SP01_W1	spill	94.55	-	SI07SP01_O3	spill	94.55	Circular	1.2	-	-
SL04SP01	Dry	85.79	88.29	88.29	5100	SL04SP01_W1	spill	87.9	6	SL04SP01_O1	spill	85.79	Circular	0.3	-	-
SO01SP01	Wet	75.1	77.05	76.75	11800	SO01SP01_W2	spill	76.7	100	SO01SP01_O1	SO01IL01	75.25	Circular	0.2	-	-
SWMF181	Wet	194.7	197.65	197.65	27872	SWMF181_W1	SA17B042	197.35	20	SI07SP01_O1	SA17B042	194.7	Circular	0.135	-	-
SWMF181	Wet	194.7	197.65	197.65	27872	SWMF181_W1	SA17B042	197.35	20	SI07SP01_O2	SA17B042	195.45	Circular	0.235	-	-
SWMF181	Wet	194.7	197.65	197.65	27872	SWMF181_W1	SA17B042	197.35	20	SI07SP01_O3	SA17B042	196.78	Circular	0.29	-	-
SWMF199	Wet	234.2	237.2	237.2	50280	SWMF199_W1	spill	237.2	10	SWMF199_O1	spill	234.2	Circular	0.075	-	-
SWMF199	Wet	234.2	237.2	237.2	50280	SWMF199_W1	spill	237.2	10	SWMF199_O2	spill	234.2	Circular	0.26	-	-
SWMF199	Wet	234.2	237.2	237.2	50280	SWMF199_W1	spill	237.2	10	SWMF199_Central_O1	spill	234.2	Circular	0.127	-	-
SWMF199	Wet	234.2	237.2	237.2	50280	SWMF199_W1	spill	237.2	10	SWMF199_Central_O2	spill	234.2	Circular	0.6	-	-
SWMF199	Wet	234.2	237.2	237.2	50280	SWMF199_W1	spill	237.2	10	SWMF199_Central_O3	spill	234.2	Circular	0.53	-	-
SWMF199	Wet	234.2	237.2	237.2	50280	SWMF199_W1	spill	237.2	10	SWMF199_East_O1	spill	234.2	Circular	0.075	-	-
SWMF199	Wet	234.2	237.2	237.2	50280	SWMF199_W1	spill	237.2	10	SWMF199_East_O2	spill	234.2	Circular	0.26	-	-
SWMF217	Wet	210.4	213.7	213.7	37360	SWMF217_W1	GO01B059	212.52	7.4	SWMF217_O1	GO01B059	210.4	Circular	0.53	-	-
SWMF218	Wet	208.7	211.5	211.5	70758	SWMF218_W1	GP02B037	210.53	16	SWMF218_O1	GP02B037	208.7	Circular	0.3	N/A	N/A
SWMF218	Wet	208.7	211.5	211.5	70758	SWMF218_W1	GP02B037	210.53	16	SWMF218_O2	GP02B037	209.22	Rectangular	N/A	1.2	1.2
SWMF220	Wet	220.85	223.58	223.58	42003	SWMF220_W1	spill	223.58	10	SWMF220_O1	HG20B067	221.85	Circular	0.45	N/A	N/A
SWMF220	Wet	220.85	223.58	223.58	42003	SWMF220_W1	spill	223.58	10	SWMF220_O2	HG20B067	222.016	Generic Shape	N/A	0.73	0.73
SWMF220	Wet	220.85	223.58	223.58	42003	SWMF220_W1	spill	223.58	10	SWMF220_O3	HG20B067	222.89	Generic Shape	N/A	0.73	0.74
SWMF222		218	221.25	221.25	21146	SWMF_222_W1	spill	221.25	10	-	-	-	-	-	-	-

Table 4.2.4: Summary of Ponds and Connected Weirs and Orifices in the Six Cities Model Basins - Continued

Basins						Weirs				Orifices						
Model ID	Type	Invert level (m)	Ground level (m)	Top of geometry (m)	Volume geometry based (m³)	Model ID	Down-stream node	Crest Level (m)	Crest Width (m)	Model ID	Down-stream node	Invert Level (m)	Shape	Diameter (m)	Height (m)	Width (m)
SWMF226	Wet	234.7	237	237	8489	SWMF226_W1	FJ19B047	236.55	7.8	SWMF226_O1	FJ19B047	234.68	Circular	0.106	N/A	N/A
SWMF226	Wet	234.7	237	237	8489	SWMF226_W1	FJ19B047	236.55	7.8	SWMF226_O2	FJ19B047	236.05	Rectangular	N/A	0.25	2.6
SWMF227	Wet	244.4	246.8	246.8	21266	SWMF227_W1	FO15OF04	246.28	4.6	SWMF227_O1	FO15OF04	244.4	Circular	0.407	-	-
SWMF227	Wet	244.4	246.8	246.8	21266	SWMF227_W1	FO15OF04	246.28	4.6	SWMF227_O2	FO15OF04	244.4	Circular	0.407	-	-
SWMF227	Wet	244.4	246.8	246.8	21266	SWMF227_W1	FO15OF04	246.28	4.6	SWMF227_O3	FO15OF04	244.4	Circular	0.26	-	-
SWMF227	Wet	244.4	246.8	246.8	21266	SWMF227_W1	FO15OF04	246.28	4.6	SWMF227_O4	FO15OF04	244.4	Circular	0.075	-	-
SWMF227	Wet	244.4	246.8	246.8	21266	SWMF227_W1	FO15OF04	246.28	4.6	SWMF227_O5	FO15OF04	244.4	Circular	0.075	-	-
SWMF227	Wet	244.4	246.8	246.8	21266	SWMF227_W1	FO15OF04	246.28	4.6	SWMF227_O6	FO15OF04	243.9	Circular	0.31	-	-
SWMF227	Wet	244.4	246.8	246.8	21266	SWMF227_W1	FO15OF04	246.28	4.6	SWMF227_O7	FO15OF04	243.9	Circular	0.32	-	-
SWMF227	Wet	244.4	246.8	246.8	21266	SWMF227_W1	FO15OF04	246.28	4.6	SWMF227_O8	FO15OF04	243.9	Circular	0.32	-	-
SWMF229		213.45	215.7	215.7	44850	SWMF229_W1	SWMF229_Outfall	215.7	10	SWMF229_O1	SWMF229_Outfall	213.45	Generic shape	-	-	-
SWMF230		213.6	216.7	216.7	23624	SWMF_230_W1	spill	216.7	10	SWMF230_O1	HWY6 CUL-In	214.316	Circular	0.75	-	-
SWMF90	Wet	208	209	209	20738	SWMF90_W1	SWMF90_OF	208.5	15	SWMF90_O1	SWMF90_OF	208	Circular	0.275	-	-
SWMF_Flamborough	Wet	215.65	220.1	219.8	38456	SWMF_Flamborough	spill	219.75	100	SWMF_Flamborough	FK23B055	217.65	Circular	0.3	-	-
PowerCentreNorth	Wet	215.65	220.1	219.8	38456	PowerCentreNorth_W2	spill	219.75	100	PowerCentreNorth_O1	FK23B055	217.65	Circular	0.3	-	-

4.2.2 Dundas Hydraulic Model

The Dundas model is a detailed storm sewer model representing the entire minor storm sewer system in the Dundas study area, including all pipes, manholes, stormwater management ponds, structures and outlets. The following is a summary and discussion of the main elements of the model.

Pipes

Of the 1,444 pipes in the Dundas model, one is rectangular in shape and the rest are circular. As indicated in **Table 4.2.2**, more than 65% of the length of pipe in the system has a diameter less than 0.5 m providing 26% of the approximately 19,000 m³ of storage in the sewer system. The majority of the remaining in-pipe storage volume is provided by pipes ranging in diameter from 0.5 to 1.5 m.

The pipes range in length from 0.1 to 300 m, but only 55 (<5%) have a length less than 10 m. All of the pipes are assigned a Material classification of “Concrete (Smooth)” with a Manning’s n value of 0.012.

Manholes

Of the 1,522 manholes, 607 have a diameter of 1.2 m while the remaining manholes have a diameter of 1.5 m.

679 manholes use an inlet control to restrict the inflow of runoff from the connected catchment. The maximum inflow values range from 2 to 448 L/s, but the majority of manholes use a value of either 28 or 56 L/s. If the runoff rate from the catchment exceeds the specified inlet capacity, the runoff will collect in a fictitious storage area and will be released into the manhole at the maximum rate until such time as all the volume in the storage area has been released.

All of manholes use the default “MOUSE Classic (Engelund)” settings for calculating head losses and the use a “Normal” setting for the manhole cover. The Normal cover setting means that any surcharge occurring above the rim of the manhole will be stored in a conceptual large diameter basin located immediately above the surcharged manhole.

Stormwater Management Facilities

There are five stormwater management facilities (SWMFs) in the Dundas model catchment area and each of them are represented in the model as a Basin node with a specified Level-Area curve. In two of the five ponds, flooding caused by spilling over the edge of the ponds is represented using a weir while outlet control devices are represented using an orifice. In one pond the overflow weir is connected back into the

collection system but there is no outlet control orifice. Two of the ponds do not have an overflow weir or an outlet control orifice represented in the model. A summary of the SWMFs is provided in **Table 4.2.5**.

The naming convention of the ponds in the model does not appear to correspond to the naming conventions used in the GIS data for Storm Storage Basins, but the facilities represented in the model were matched to the GIS representation of ponds based on the locations.

Boundary Conditions

Of the 116 stormwater outlets represented in the model, 96 were associated with a constant water level boundary condition. No documentation was available to provide an explanation for the values used as constant water levels.

In practice, the creeks and rivers will have elevated water levels during a design storm event and this may cause backwater conditions at the outlets and create flooding and surcharging in the storm sewer system. However, defining the outlet water levels associated with each design storm event is beyond the scope of this study and is recommended for future considerations.

Model Calibration

According to the Aquafor Beech report (Aquafor Beech Ltd, 2014), the model was calibrated using rainfall and flow monitoring data collected in the Dundas area during 2013. The report provides an explanation of the methodology used to develop initial estimates of the hydrology model parameters, but it does not provide any details regarding which parameters were adjusted during the calibration process. The calibration results presented in the report show a good agreement between the modelled and measured flows at all five flow monitoring stations.

Table 4.2.5: Summary of Ponds and Connected Weirs and Orifices in the Dundas Model Basins

Basins						Weirs				Orifices						
Model ID	Type	Invert level (m)	Ground level (m)	Top of geometry (m)	Volume geometry based (m3)	Model ID	Down-stream node	Crest Level (m)	Crest Width (m)	Model ID	Down-stream node	Invert Level (m)	Shape	Diameter (m)	Height (m)	Width (m)
AH12SP02	Dry	240.4	242.4	242.1	2800	AH12SP02_W1	spill	242.1	5	AH12SP02_O1	spill	240.4	Circular	0.3	-	-
AH18SP01	Wet	217.18	220.18	220.18	54600	AH18SP01_W1	spill	219.08		AH18SP01_O1	spill	217.18	Circular	0.825	-	-
AI13SP01	Dry	242.11	246.67	246.67	6500	AI13SP01_W1	spill	246.17	10	AI13SP01_O1	spill	242.11	Circular	0.525	-	-
AJ09SP01	Dry	227.5	229.83	229.53	900	AJ09SP01_W2	spill	229.5	100	AJ09SP01_O1	spill	227.5	Circular	0.28	-	-
AJ13SP01	Dry	245.27	247.38	247.38	5600	AJ13SP01_W1	spill	247.38	10	-	-	-	-	-	-	-

4.2.3 Stoney Creek Hydraulic Model

Pipes

The model consists of 4,158 pipes where 37 are rectangular and the rest are circular. The model includes smaller pipes down to 13.5 cm and 42% of the pipe are less than 0.5 m in diameter.

The pipes range in length from 0.1 to 249 m, with 209 pipes having a length less than 10 m. The pipes have a number of different materials with Manning's n values ranging from 0.0117 to 0.0143.

Table 4.2.6: Summary of Pipe Materials for Stoney Creek Model

Material ID	Manning's n	# of pipes
ACP	0.0133	2
ACS	0.0133	7
CIPPS	0.0143	38
CON	0.0133	34
CONC	0.0133	344
CONCRB	0.0133	4
Concrete (Smooth)	0.0117	13
CONCRF	0.0133	1,337
CONCRP	0.0133	82
CONCS	0.0133	1,090
CPP	0.0133	2
CSP	0.0133	24
PVC	0.0125	85
PVC-35	0.0125	115
PVC-RB	0.0125	4
PVCS	0.0125	83
PVC-UR	0.0125	56
RCP	0.0133	149
RPM	0.0125	1
UNKN	0.0133	398
VCP	0.0133	27
VCS	0.0133	257
ZZZ	0.0133	6

Manholes

There are 4071 manholes in the model with a diameter ranging from 1.0 m to 5.4 m.

All manholes use an inlet control to restrict the inflow of runoff from the connected catchment to a maximum rate of 70 L/s. If the runoff from the catchment exceeds the specified maximum inlet capacity, the runoff will collect in a fictitious storage volume above the manhole and will be released into the manhole at the maximum rate until such time as all the volume in the fictitious storage has been released.

All of manholes use the default “MOUSE Classic (Engelund)” settings for calculating head losses and the use a “Normal” setting for the manhole cover. The Normal cover setting means that any surcharge occurring above the rim of the manhole will be stored in a conceptual large diameter basin located immediately above the surcharged manhole.

Stormwater Management Facilities

There are 13 stormwater management facilities (SWMFs) represented in the Stoney Creek model catchment area and each of them are represented in the model as a Basin node with a specified Level-Area curve. Flooding caused by spilling over the edge of the ponds is represented using a weir while outlet control devices are represented using an orifice that discharges into a surface drainage channel. A summary of the SWMFs is provided in **Table 4.2.7**.

Boundary Conditions

There are 117 stormwater outlets in the model and 18 of them discharge to Lake Ontario. The outlets discharging to Lake Ontario have a constant water level boundary condition of 75.2 m while the remaining are free flow outlets.

In practice, the creeks and rivers will have elevated water levels during a design storm event and this may cause backwater conditions at the outlets and create flooding and surcharging in the storm sewer system. However, defining the outlet water levels associated with each design storm event is beyond the scope of this study and is recommended for future considerations.

Model Calibration

The Stoney Creek model was calibrated as described the Task 13 Technical Memorandum (DHI, 2019). The model was calibrated using rainfall and flow monitoring data collected from three rain gauges and six flow monitoring stations in a 2012 flow monitoring campaign. The calibrated model was able to provide a good to very good representation of flow at all six of the flow monitoring stations for a range of rainfall events. The water level data from the flow monitoring stations was not available so no water level calibration was performed.

Table 4.2.7: Summary of Ponds and Connected Weirs and Orifices in the Stoney Creek Model Basins

Basins							Weirs				Orifices						
Model ID	Type	Invert level (m)	Ground level (m)	Depth (m)	Top of geometry (m)	Volume geometry based (m3)	Model ID	Down-stream node	Crest Level (m)	Crest Width (m)	Model ID	Down-stream node	Invert Level (m)	Shape	Diameter (m)	Height (m)	Width (m)
SWMF103	Wetland	76	79.5	79.5	79.5	3188	SWMF103-W1	Spill	79.3	3	SWMF103-O1	SK02IL02	77.5	Circular	0.375	-	-
SWMF104	Wet	77.3	79.7	79.7	79.7	3694	SWMF104-W1	SK02B002	79.4	4	-	-	-	-	-	-	-
SWMF105	Wet	76.4	79.8	79.8	79.8	2899	SWMF105_W1	Spill	79.75	100	-	-	-	-	-	-	-
SWMF106	Wet	73.25	77.1	77.1	77.1	5673	SWMF106-W1	SN01IL01	76.9	5	SWMF106-O1	SN01IL01	75.24	Circular	0.3	-	-
SWMF142	Wet	80	83.3	83.3	83.3	8265	SWMF142-W1	Spill	83.25	10	-	-	-	-	-	-	-
SWMF164	Wet	181.8	185.4	185.4	185.4	10018	SWMFSD12_Weir1	SWMF80	184.3	7.54	SWMFSD12_Orifice1	SWMF80	183.3	Circular	0.15	-	-
SWMF164	Wet	181.8	185.4	185.4	185.4	10018	SWMFSD12_Weir2	SWMF80	184.95	8.5	SWMFSD12_Orifice1	SWMF80	183.3	Circular	0.15	-	-
SWMF179	Wet	180.75	186.15	186.15	186.15	39504	SWMF179_W1	Spill	186.1	100	SWMF179OF1	SD12B145	183.19	Rectangular	N/A	0.2	0.15
SWMF179	Wet	180.75	186.15	186.15	186.15	39504	SWMF179_W1	Spill	186.1	100	SWMF179OF2	SD12B145	184.28	Rectangular	N/A	0.2	1.05
SWMF179	Wet	180.75	186.15	186.15	186.15	39504	SWMF179_W1	Spill	186.1	100	SWMF179OF3	SD12B145	184.62	Rectangular	N/A	0.8	2.1
SWMF179	Wet	180.75	186.15	186.15	186.15	39504	SWMF179_W1	Spill	186.1	100	SWMF179OF4	SD12B145	182.29	Circular	0.105	N/A	N/A
SWMF198	Dry	85.25	89.34	89.34	89.34	9480	SWMF198_W1	Spill	89.3	100	-	-	-	-	-	-	-
SWMF216	Wet	180.6	185.6	185.6	185.6	36772	SWMF216_W1	Spill	185.55	100	SWMF216OF1	SC13OF06	182.1	Circular	0.14	N/A	N/A
SWMF216	Wet	180.6	185.6	185.6	185.6	36772	SWMF216_W1	Spill	185.55	100	SWMF216OF2	SC13OF06	182.75	Circular	N/A	0.45	0.4
SWMF216	Wet	180.6	185.6	185.6	185.6	36772	SWMF216_W1	Spill	185.55	100	SWMF216OF3	SC13OF06	183.36	Circular	N/A	0.25	0.4
SWMF216	Wet	180.6	185.6	185.6	185.6	36772	SWMF216_W1	Spill	185.55	100	SWMF216OF4	SC13OF06	184.15	Rectangular	N/A	2.4	2.4
SWMF66	Wetland	73.5	76.1	76.1	76.1	9799	SWMF66-W1	Spill	76	100	SWMF66-O1	SO01IL01	75	Circular	0.15	-	-
SWMF68	Dry	94	97	3	97	13489	SWMF68-W1	Spill	94.55	-	SWMF68-O1	Spill	94.005	Circular	0.8	-	-
SWMF80	Wet	182.5	183.9	183.9	183.9	9667	SWMF80	Spill	183.85	100	-	-	-	-	-	-	-
SWMF82	Dry	85.5	88	2.5	88	5116	SWMF82-W1	Spill	87.7	7	SWMF82-O1	Spill	85.5	Circular	0.3	-	-

4.2.4 Ancaster Hydraulic Model

The Ancaster model is a detailed dual-drainage model representing all of the minor storm sewer system and major drainage system in the Ancaster study area. The following is a summary and discussion of the main elements of the model.

Pipes

Of the 2,125 pipes in the Ancaster model, 28 are rectangular in shape and the rest are circular. As indicated in **Table 4.2.2**, more than 46% of the length of pipe in the system has a diameter less than 0.5 m providing 11% of the approximately 49,126 m³ of storage in the sewer system. Pipes with a diameter between 0.5 to 1.5 m make up almost 50% of the total length of the sewer in the model and provides 62% of the in-pipe storage, while pipes with a diameter larger than 1.5 m make up a length of less than 4% but provide 27% of the available in-pipe storage.

The pipes range in length from 2.1 to 264.7 m, but only 114 (5.4%) have a length less than 10 m. All of the pipes are assigned a local Manning's n value of 0.012.

Manholes

2,042 of 2,152 manholes have a diameter of 1.2 m and the rest are enlarged to accommodate the diameter of the connected pipes. Inlet flow controls are not used. All of manholes use the default "MOUSE Classic (Engelund)" settings for calculating head losses and they use a "Normal" setting for the manhole cover. The Normal cover setting means that any surcharge occurring above the rim of the manhole will be stored in a conceptual large diameter basin located immediately above the surcharged manhole. However, with a dual drainage model the water surcharging above the ground level will be transferred to the major drainage network.

Major System Channels

The major system was created parallel to the minor system. Most of the major system links and nodes are named after the corresponding minor system link and node with a suffix "-S" while the default feature name (e.g., Link_10) was used for some additional major links.

The major system links are flagged as "Major System" in the Description field. The geometry of the major system links is defined using cross-sections that are representative of roadways. In total, 18 cross sections were created for the typical roads in the study area and assigned to the major links. Each roadway cross-section has a vertical 'wall' at each end with a minimum height of 1.35 m. Of the total 2164 major system links, there are 2 circular and 91 rectangular links. No explanation was provided in the AECOM report for these anomalous links. It appears they were either mislabelled

as Major System drainage, or they were created for green space areas where there is no road.

The roughness of the major system links was defined using a Local Manning n coefficient with values of 0.012 for the roadways, and either 0.028 or 0.033 for the rectangular links.

All 1,932 major nodes have a diameter of 1.2 m. The bottom level is set to the ground elevation and ground level is set to 0.15 m above the ground elevation. All major nodes use the default “MOUSE Classic (Engelund)” settings for calculating head losses and use a “Normal” setting for the manhole cover. Using the MOUSE Classic (Engelund) head loss calculation at these intersections could introduce unrealistic head loss across the node. To avoid this, the model was updated such that the nodes connecting the major drainage links ignored head losses (i.e. the node head loss method was set to “No cross-section changes”).

Curb Inlets

The curb inlets were named after the connected manhole with a suffix of “-IC”. All 1,931 curb inlets use a Sag type inlet with the same depth vs. flow (DQ) curve (see Figure 4.2). In the original model from AECOM there was an error in the second line of DQ curve where a depth of 0.1 m was entered instead of 0.01 m. This error restricted the flow from entering the Curb Inlets until the depth of water exceeded 0.1 m so it produced results that are inconsistent with how the major to minor drainage system should function. The error was fixed for the system capacity assessment. The invert level at each Curb Inlet is set to the ground elevation and the freeboard is set to 0.15 m. The option to define a blockage % was not used.

Figure 4.2.1: Depth vs. Flow (Q) Relation for Curb Inlets

	Depth [m]	Q [m ³ /s]
1	0	0
2	0.1	0.001
▶ 3	0.02	0.003
4	0.03	0.007
5	0.04	0.015
6	0.06	0.038
7	0.08	0.066
8	0.1	0.099
9	0.15	0.2
10	0.2	0.238
11	0.25	0.269
12	0.3	0.296

Stormwater Management Facilities

As mentioned in “Ancaster All Pipes Storm Sewer Network Model Development and Calibration Report” (AECOM, 2019), there are 20 stormwater management facilities (SWMFs) in the Ancaster model catchment area. All of them are included in the model and each of them are represented in the model as a Basin node with a specified Level-Area curve. Flooding caused by spilling over the edge of the ponds is represented using a weir while outlet control devices are represented using an orifice. A summary of the SWMFs is provided in **Table 4.2.8**.

The model also includes three additional SWMFs (STM_AL13IL01-S, STM_AM08055 and STM_AJ12B028) but there is no documentation available.

Table 4.2.8: Summary of Stormwater Management Facilities for Ancaster Model

Model ID	Type	Invert (m)	Ground (m)	Volume (m ³)
AN10SP01	Wet	238	241	19,947
AH15SP01	Wet	227.3	231	6,875
AH16SP01	Wet	224.3	229	23,588
AG16SP01	Dry	225.3	228.3	18,059
AI15SP01	Wet	233.7	235	3,169
AJ15SP01	Wet	233.2	235.6	5,131
AO09SP01	Wet	232	236	12,785
AH18SP01	Wet	216.8	220.3	67,460
AN10SP02	Wet	236	238	1,163
AN09SP01	Wet	233	237	15,070
OF_AI13OF02	Dry	242.7	246	3,752
AH18SP01_W	Wet	216.3	219.3	4,037
AH18SP01_E	Wet	215.8	218.8	30,884
STM_AL13IL01-S	Unknown	239	242	37,200
OF_AP08OF01	Wet	227	231	28,318
OF_AG16OF03_02	Dry	223.9	227	19,808
AO07SP02	Wet	222	227	60,465
STM_AM08055	Unknown	231.92	232.42	2,342
STM_AJ12B028	Unknown	247	247.5	3,650
SWMF233	Unknown	216.5	220.5	54,260

Boundary Conditions

Of the 170 stormwater outlets represented in the model, 157 were associated with a constant water level boundary condition. The constant water level at some of the locations were based on Hamilton Conservation Authority floodplain mapping and

estimated water level from Water Survey of Canada flow monitoring stations. However, the water levels were only used for 100-year design storm simulation that was run by AECOM. A free outfall condition was used in 2-, 5-, 10-, 25- and 50-year rainfall event simulations evaluated for this study.

Model Calibration

The model was calibrated at four flow monitoring sites. The adjusted parameters include Horton's infiltration, storage loss, impervious percentage, evapotranspiration, and flow length. Two calibration events and one validation event were used for calibration at each site. The calibrated model results provided in the AECOM report (AECOM, 2019) demonstrate a good fit between the measured and modelled flows. However, the adjustments made to the hydrologic parameters were only applied to the catchments contributing to each flow monitoring site. This results in significant inconsistencies in imperviousness between adjacent catchments even though they have the same land use.

An example of this difference is shown in **Figure 4.4.2** for the Site 3 flow monitoring station (lower left corner). The storm sewers upstream of the Site 3 station are highlighted in green and the catchments are color-coded based on the total % imperviousness value (i.e. Steep and Flat impervious). Most of the catchments in the Site 3 drainage area are single-family residential with an imperviousness value below 20% of the area (i.e. blue and purple color), while the neighboring catchments with essentially the same residential land use have percent imperviousness values greater than 50% (i.e. green color). This type of inconsistent application of imperviousness is evident throughout the model domain and will result in inconsistent and unreliable system capacity assessments. Unfortunately, this problem was discovered late in the project and there was not sufficient time to address it.

In addition, the monitoring data included water levels, but this data was not used for the model calibration.

Figure 4.2.2: Example map of inconsistent imperviousness values for catchments in the Ancaster model



4.3 Potential Model Enhancements

Water Quality Simulation

Although a separate consideration from the simulation of storm sewer conveyance capacity, there is also the potential to incorporate a stormwater quality component into the MIKE+ modelling. As noted in previous sections, the 2007 SWMP addressed stormwater quality through a broad-based spreadsheet (mass balance) model, which was lumped for the primary watersheds and subwatersheds in the City. This approach is by its nature lumped/coarse and is not able to properly account for expected stormwater quality impacts from infill and intensification on a more meaningful scale. In addition, it is considered more valuable to be able to assess both quantity and quality considerations as part of a single tool, rather than undertaking these assessments separately. Depending on the preferences of City staff and the intended outcomes from this study, it is considered that there is value in incorporating a stormwater quality component to the existing MIKE+ models. This would allow for a more nuanced assessment of existing conditions and expected under future development, including the most appropriate mitigation strategies, which could be assessed at a more resolute scale than a lumped watershed approach.

Outfall Water Levels

With the exception of the Ancaster model, none of the models consider the water level in the receiving water course during a storm event, and the Ancaster model only considered the water course water level for the 100-year event. If the water level in the receiving water course is not accounted for, the drainage capacity may be over-estimated in areas where urban water courses have a similar hydrologic response time as the stormwater drainage system. To get a more accurate reflection of the drainage system performance during design storm events, the water level of the receiving water courses should be considered for those water courses where existing floodplain modelling and mapping are available.

4.3.1 Six Cities Model Specific Enhancements

Model Retirement

In its current form, the Six Cities model is mostly uncalibrated and lacks sufficient representation of the drainage system details to make informed and reliable planning and operational assessments of the system performance. It is recommended to continue to phase out the Six Cities model and replace it with detailed stormwater drainage models for the remaining areas of the City, including Flamborough, Glanbrook, and Hamilton Mountain. Although the City's current stormwater drainage system GIS data still have some gaps, the initiative to develop detailed models will serve as a catalyst for addressing these gaps. In addition, the intention of preparing these models should be taken into consideration when developing the flow monitoring programs. Finally, given the drainage system deficiencies identified in this study, the detailed models should be developed as dual drainage models to facilitate the integrated evaluation of minor and major drainage systems.

Model Calibration

If the Six Cities model will continue to be used while detailed models are still being developed for Flamborough, Glanbrook and Hamilton Mountain areas of the City, the existing Six Cities model should be calibrated to available flow monitoring data that are collected in these areas.

4.3.2 Dundas Model Specific Enhancements

Add Surface Drainage Systems

Consideration should be given to the incorporation of major surface drainage networks (primarily roadways) to the model to provide a more accurate representation of the level of service provided by the storm sewer system. Including roadways as a drainage

pathway allows a more realistic representation of the performance of the drainage system during extreme events when the sewer system is surcharged and provides more useful information related to road ponding and flooding, and areas of concern. It would also be understood to be more consistent with the approach currently being pursued by the City for the combined sewer system as part of on-going initiatives for this area.

Update Manhole Cover Classification

This recommendation only applied if the recommendations to add surface drainage systems are NOT incorporated. The current model assumes that any surcharge / flooding from a manhole collects at the manhole location in a virtual basin above the manhole and then returns to the collection system once the surcharge condition subsides. While this setting is not necessarily incorrect, excessive surface ponding at a manhole can create local bias in the system water levels. Instead, the overland drainage analysis discussed in Section 3.4 should be used to identify the manholes located in areas where pooling will occur and these should continue to use the current setting where flooding pools above the manhole, while all other manhole should adopt a spilling setting where flooding above the manhole exits the model.

Update Model Calibration

The original model was calibrated to measured flows more 10 years ago and the measured water levels were not used to calibrate the model. It is recommended to collect six months of flow and level monitoring data for locations with different land use combinations and then use the data to calibrate the model to both flow and water levels.

4.3.3 Stoney Creek Model Specific Enhancements

Add Surface Drainage Systems

Consideration should be given to the incorporation of major surface drainage networks (primarily roadways) to the model to provide a more accurate representation of the level of service provided by the storm sewer system. Including roadways as a drainage pathway allows a more realistic representation of the performance of the drainage system during extreme events when the sewer system is surcharged and provides more useful information related to road ponding and flooding, and areas of concern. It would also be more consistent with the approach currently being pursued by the City for the combined sewer system as part of the on-going City-Wide Flooding study.

Update Manhole Cover Classification

This recommendation only applied if the recommendations to add surface drainage systems are NOT incorporated. The current model assumes that any surcharge / flooding from a manhole collects at the manhole location in a virtual basin above the

manhole and then returns to the collection system once the surcharge condition subsides. While this setting is not necessarily incorrect, excessive surface ponding at a manhole can create local bias in the system water levels. Instead, the overland drainage analysis discussed in Section 3.4 should be used to identify the manholes located in areas where pooling will occur and these should continue to use the current setting where flooding pools above the manhole, while all other manhole should adopt a spilling setting where flooding above the manhole exits the model.

Update Model Calibration

The original model was calibrated to measured flows more than 10 years ago and the measured water levels were not used to calibrate the model. It is recommended to collect six months of flow and level monitoring data for locations with different land use combinations and then use the data to calibrate the model to both flow and water levels.

4.3.4 Ancaster Model Specific Enhancements

Update Model Calibration

The Ancaster model provides a good foundation for a dual drainage model of the area, but it needs an update to the model calibration before it can reliably be applied for system capacity assessments (see discussion in Section 4.2.4). In its current state, the results of a system capacity assessment will be unreliable and should be used with caution.

The model should be calibrated using reasonable ranges of imperviousness for each land use category, and it should be calibrated to both flow and water level data to ensure it is properly capturing head losses and pipe filling throughout the system.

4.4 Capacity Assessment

An updated storm sewer capacity assessment has been performed by running the Six Cities model, the Dundas model, the Stoney Creek model, and the Ancaster model using a series of 6-hour design storms including the 2-, 5-, 10-, 25- and 50-Year design storm events. In addition, one observed storm event (April 20, 2017) has also been simulated. The April 2017 event was run to see if the Dundas model would be able to reproduce the significant flooding that was reported for this event.

The results from the Six Cities model are presented in **Appendix B**, the results from the Dundas model are presented in **Appendix C**, the results from the Stoney Creek model are presented in **Appendix D**, and the results from the Ancaster model are presented in **Appendix E**. Due to the size of the Six Cities model area, the results for each storm event are presented using six separate maps; one map for each of the six former

municipalities (e.g. Ancaster, Dundas, Flamborough, Glanbrook, Hamilton, and Stoney Creek).

The results for each event have been plotted on summary figures indicating how full the pipes are flowing based on the ratio of the maximum modelled flow in the pipes (Q) vs. the theoretical full pipe flow (q). In addition, the maps indicate manhole locations where the model predicts surface flooding where the maximum hydraulic grade line (HGL) is more than 0.3 m above ground surface, and where there is a potential risk of basement flooding (i.e. the HGL is at an elevation higher than 1.8 m below ground surface). These figures have been included in **Appendix B** (Six Cities model), **Appendix C** (Dundas model), **Appendix D** (Stoney Creek model) and **Appendix E** (Ancaster model).

The manhole HGL threshold values of 0.3 m above ground surface for surface flooding and -1.8 m below ground surface for basement flooding were used in order to maintain consistency with the reporting values used in the City-Wide Flooding and Drainage Study (Aquafor Beech Limited). However, it should be noted that the bottoms of many of the manholes are less than 1.8 m below ground surface, so many of these will be reported as 'at risk' regardless of whether they actually pose a real risk. In addition, the 0.3 m threshold used to indicate surface flooding problems is being used in the Flooding and Drainage Study because that study is using a dual drainage model where surface drainage in the streets is explicitly represented in the model. However, neither of the Six Cities, Dundas or Stoney Creek models account for surface flow when a manhole surcharges, so the effect of 0.3 m of flooding above ground is quite different for these models. The threshold criteria may therefore warrant revisiting prior to finalizing the analyses, depending on any potential model enhancements considered (as per Section 4.3).

When interpreting the results of the system capacity assessments, the following points should be kept in mind:

- The Six Cities model was only calibrated for the Stoney Creek area of the model and it was calibrated using data from 2012. For the purposes of this project, it has been assumed that the other areas of the model will respond similarly in terms of rainfall runoff for similar land uses.
- The Dundas model was calibrated for well-serviced areas, but the storm events were quite small and the runoff parameters used for unserviced catchments is likely to significantly under-estimate peak flow runoff.
- The Ancaster model was calibrated to measured flow at four locations, but the calibration process only adjusted hydrologic model parameters in catchments upstream of the flow monitoring stations. As such, there is a significant level of inconsistency in the hydrologic parameters throughout the model, and this leads to uncertainty with the modelling results. In addition, the model was only calibrated to

measured flow and not to measured water levels. The model should be calibrated to both flow and water level, particularly when flooding is indicated by water levels in the system. These deficiencies in the model should be kept in mind during the analysis and discussion of the capacity assessment. As discussed in Section 4.3.3, this further supports a consideration of additional model calibration and validation.

4.4.1 Six Cities Model

The design storm modelling results for the Six Cities model are summarized in **Table 4.4.1** and **Table 4.4.2**. The model indicates the majority of the storm sewer system is capable of effectively conveying the 2-Year design storm event with less than 6% of the manholes experiencing flooding. While the flooding is scattered throughout the study area there are some areas where there is a concentration of flooded manholes. For the sake of consistency, the following list does not include potential stormwater drainage issues from Ancaster, Dundas and Stoney Creek areas since those areas are covered by more detailed models.

Flamborough

- Clappison Ave.
- Thornledge Dr.

Glanbrook

- South end of Homestead Dr. from Airport Rd.
- Penfold Ct. to Fulmar Way
- Carpenter area near Glancaster Road and Twenty Road

Hamilton

- From Millwood and Harington to Hester and Duchesne
- From Greencedar and Atkins to Appleford and Wendover
- On Gemini from Upper Paradise to Cranbrook
- On Brigadoon from Duncairn to Galloway
- On Glenhollow from Highland to Dalegrove
- Along Kenora Ave from QEW to King Street East
- Along Greencedar Drive and Forestgate Drive

Stoney Creek (South)

- On Eringate near Willowridge
- At the south end of First Rd W

The number of flooded manholes for the 5-year through to the 50-year design storm events significantly increase to more than 28% of the total manholes in the model. The locations of the flooded manholes for these events are spread throughout most areas of the system such that there are very few areas without flooding. The volume of flooding from the surcharged manholes is shown in **Figure 4.4.1**.

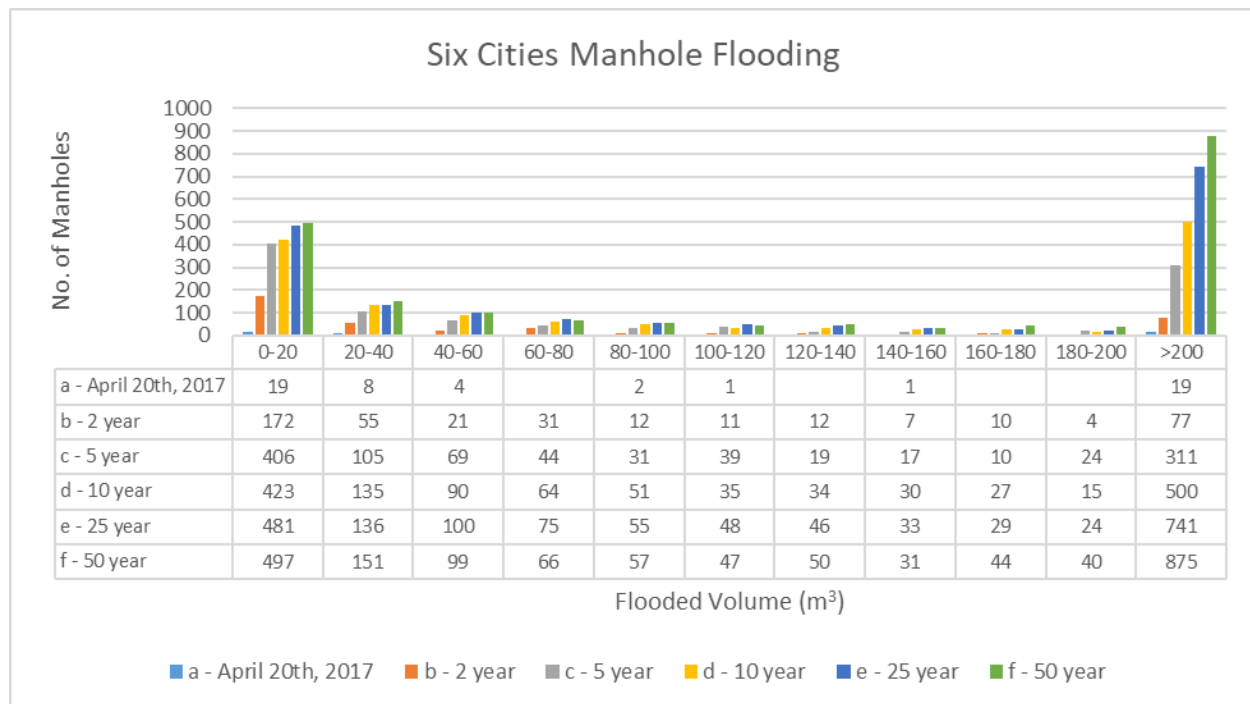
Table 4.4.1: Summary of Design Storm Manhole Flooding in the Six Cities Model (number of flooded manholes)

COMMUNITY	Description	No Flood Risk [<-1.8m]	Flood Risk [-1.8m-0.3m]	Out of Manhole Flood [>0.3m]
ANCASTER	April 20th, 2017	354	67	3
	2 year	260	145	19
	5 year	180	162	82
	10 year	144	173	107
	25 year	106	175	143
	50 year	97	162	165
DUNDAS	April 20th, 2017	213	70	19
	2 year	201	87	14
	5 year	169	84	49
	10 year	140	100	62
	25 year	131	91	80
	50 year	128	84	90
FLAMBOROUGH	April 20th, 2017	763	47	3
	2 year	542	243	28
	5 year	348	383	82
	10 year	280	386	147
	25 year	229	384	200
	50 year	201	380	232
GLANBROOK	April 20th, 2017	446	94	8
	2 year	239	269	40
	5 year	140	291	117
	10 year	89	305	154
	25 year	65	295	188
	50 year	55	278	215
HAMILTON	April 20th, 2017	1629	26	1
	2 year	1304	252	100
	5 year	901	490	265
	10 year	699	581	376
	25 year	574	583	499
	50 year	538	554	564
STONEY CREEK	April 20th, 2017	809	85	3
	2 year	539	284	74
	5 year	369	357	171
	10 year	298	377	222
	25 year	245	365	287
	50 year	226	358	313

Table 4.4.2: Summary of Design Storm Pipe Discharge Capacity in the Six Cities Model (km of pipe)

COMMUNITY	Description	Discharge/Capacity <85% [km]	Discharge/Capacity 85%-100% [km]	Discharge/Capacity >100% [km]
ANCASTER	April 20th, 2017	21.3	0.2	0.7
	2 year	16.3	2.5	3.4
	5 year	11.1	2.9	8.1
	10 year	9.6	3.2	9.4
	25 year	8.3	2.8	11.1
	50 year	7.8	2.6	11.7
DUNDAS	April 20th, 2017	11.4	1.0	2.5
	2 year	11.3	1.5	2.2
	5 year	9.1	2.0	3.8
	10 year	8.1	1.6	5.2
	25 year	7.2	2.2	5.5
	50 year	7.0	1.8	6.0
FLAMBOROUGH	April 20th, 2017	40.9	0.1	0.5
	2 year	33.1	3.1	5.3
	5 year	25.5	4.7	11.3
	10 year	21.9	5.1	14.5
	25 year	19.9	4.3	17.2
	50 year	18.8	4.4	18.3
GLANBROOK	April 20th, 2017	28.5	0.3	0.8
	2 year	19.0	4.1	6.5
	5 year	14.2	3.8	11.7
	10 year	12.7	3.4	13.5
	25 year	11.8	2.7	15.1
	50 year	11.2	3.2	15.3
HAMILTON	April 20th, 2017	103.3	0.6	0.7
	2 year	79.3	10.7	14.6
	5 year	58.7	15.7	30.2
	10 year	50.7	17.6	36.3
	25 year	43.6	17.9	43.1
	50 year	42.3	17.3	44.9
STONE CREEK	April 20th, 2017	49.7	0.1	1.4
	2 year	36.7	5.9	8.5
	5 year	28.7	6.4	16.1
	10 year	26.0	5.8	19.3
	25 year	23.7	6.5	21.0
	50 year	22.9	6.4	21.9

Figure 4.4.1.1: Six Cities Model – Manholes flooded with 20 m³ volume intervals



A summary of the ponds where flooding is indicated is provided in **Table 4.4.3**. The ponds are indicated as flooded when the maximum water level in the pond exceeds the top of the pond as described by the basin geometry curve.

Table 4.4.3: Summary of Flooded Ponds Results for the Six Cities Model

Pond ID	2-year	5-year	10-year	25-year	50-year	April 20, 2017
	Flood Status	Flood Status	Flood Status	Flood Status	Flood Status	Flood Status
AH12SP02	-	-	-	Flood	Flood	-
AJ09SP01	-	-	-	-	Flood	-
AJ13SP03	-	-	-	-	Flood	-
AO07SP02_Qual	-	-	-	Flood	Flood	-
AO07SP02_Quan	-	-	-	Flood	Flood	-
FB06SP01	-	Flood	Flood	Flood	Flood	-
NEW_033	-	-	-	-	Flood	-
SO01SP01	-	-	-	Flood	Flood	-
SWMF_Flamborough PowerCentreNorth	-	-	-	Flood	Flood	-

4.4.2 Dundas Model

The design storm modelling results for the Dundas model are summarized in **Table 4.4.4** and **Table 4.4.5**. The model indicates the majority of the storm sewer system is capable of effectively conveying the 2 through 50-year design storm events with only a relatively small number of manholes experiencing any meaningful flooding. In addition, none of the overflow weirs in the SWM facilities were active during any of the design storm events.

The manhole flooding for all of the events is relatively well scattered throughout the Dundas area, with only a few areas with 3 or more clusters of flooded manholes appearing for the 25-year and 50-year design storms:

- King Street West northeast of Bond Street South
- At the intersection of York Road and Willett Hill
- Along Alma Street
- At the west end of Tally Ho
- Along Shirley Street
- On Park Street west of Market Street
- Along Ann Street near Creighton Road

An analysis of the volume of flooding from surcharged manholes (see **Figure 4.4.2.2**) shows that the flooding is usually less than 20 m³ for the majority of surcharged manholes, but there are some manholes where significant surcharged volumes are predicted. In addition, while King Street West between Bond Street South and Market Street North does not show any cluster of flooded manholes, it does show the sewer pipes are flowing more than full for almost the entire length beginning with the 10-year design storm.

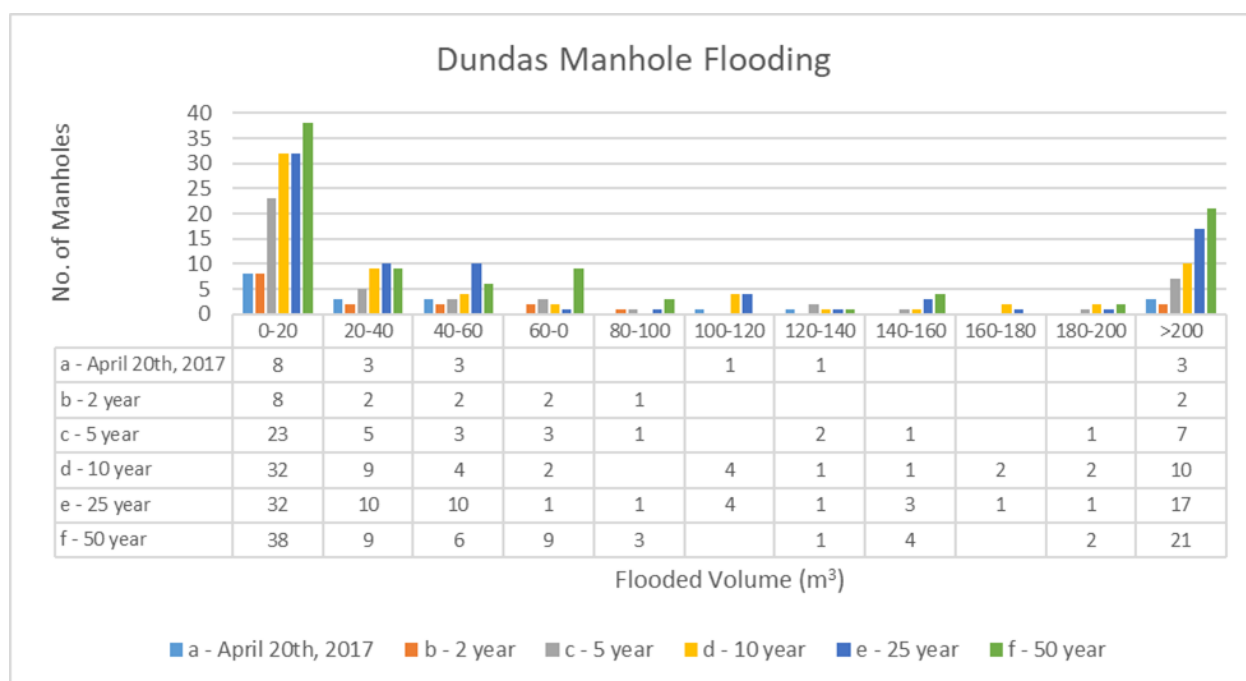
Table 4.4.4: Summary of Design Storm Manhole Flooding in the Dundas Model

COMMUNITY	Description	No Flood Risk [<-1.8m]	Flood Risk [-1.8m-0.3m]	Out of Manhole Flood [>0.3m]
Dundas	April 20th 2017	1212	174	13
	2 year	1180	209	10
	5 year	1087	283	29
	10 year	1051	304	44
	25 year	1021	320	58
	50 year	1004	334	61

Table 4.4.5: Summary of Design Storm Pipe Discharge Capacity in the Dundas Model

COMMUNITY	Description	Discharge/Capacity <85% [km]	Discharge/Capacity 85%-100% [km]	Discharge/Capacity >100% [km]
DUNDAS	April 20th 2017	72.7	0.9	3.3
	2 year	71.8	1.4	3.8
	5 year	67.9	2.4	6.7
	10 year	66.1	2.8	8.0
	25 year	65.6	2.9	8.4
	50 year	64.8	3.4	8.7

Figure 4.4.2.2: Dundas Model – Manholes flooded with 20 m³ flood volume intervals



The pond results are given in Table A.2. All stormwater management facilities have enough capacity for all design storms and are not flooded.

Overall, the modelling results indicate the storm sewer has capacity to effectively handle the 50-year design storm event with only a small percentage of manholes experiencing meaningful flooding. However, the extensive flooding reported on April 20, 2017 indicates the model is not able to account for flooding from the creeks that flow through the area. As discussed earlier in this report, the models are not intended to represent flow in the watercourses. However, if the watercourses are flooded, then the storm sewers that outfall to these watercourses will also be flooded, and current model is not able to represent this because the outfalls are set as free flow outfalls rather than using

a water level that reflects the expected water level in the watercourse. The absence of surcharging in the system may also be attributed to the very large drainage path length values used for the unserviced catchments, causing the peak flow from these catchments to be extremely attenuated. In addition, the significance of the potential flooding from the surcharged manholes is difficult to assess because the model does not represent surface routing of the surcharged stormwater.

4.4.3 Stoney Creek Model

The design storm modelling results for the Stoney Creek model are summarized in **Table 4.4.6** and **Table 4.4.7** below.

Table 4.4.6: Summary of Design Storm Manhole Flooding in the Stoney Creek Model

COMMUNITY	Description	No Flood Risk [<1.8m]	Flood Risk [-1.8m-0.3m]	Out of Manhole Flood [>0.3m]
Stoney Creek	2 year	2765	1188	118
	5 year	2138	1657	276
	10 year	2003	1703	365
	25 year	1887	1729	455
	50 year	1833	1718	520

Table 4.4.7: Summary of Design Storm Pipe Discharge Capacity in the Stoney Creek Model

COMMUNITY	Description	Discharge/Capacity <85% [km]	Discharge/Capacity 85%-100% [km]	Discharge/Capacity >100% [km]
Stoney Creek	2 year	177.9	22.1	33.7
	5 year	158.8	27.5	47.4
	10 year	153.0	28.0	52.6
	25 year	149.3	29.0	55.3
	50 year	147.6	28.8	57.3

The model indicates the majority of the collection system is able to convey the 2 to 50-year design storm events but surface flooding will occur at almost 3% of the manholes during a 2-Year design storm event, at almost 7% of manholes during a 5-Year design storm event, and at 13% of the manholes during a 50-Year design storm event. The flooding mainly occurs in the upstream sections of the model and does not generally occur along the main trunks with diameter larger than 600 mm.

An analysis of the volume of flooding produced is provided in **Figure 4.4.3.3**. The majority of surcharging manholes produce less than 20 m³ of flood volume for each design storm event, but there are more than 50 manholes producing more than 200 m³ of flood volume in all design storm events except the 2-year design storm event and 5-year design storm. In some cases, the flood volume is more than 1000 m³.

The flooding is relatively well distributed throughout the model but there is a high concentration of flooding in the area south of Barton Street and North of Ridge Road and Lawrence Road, with pockets of flooding in a few other areas:

- The area west of St. Andrews Drive and Quigley Road
- The area adjacent to the intersection of Trevor Drive and Wardrop Avenue
- The area west of Seabreeze Park
- The area adjacent to Lake Point Park

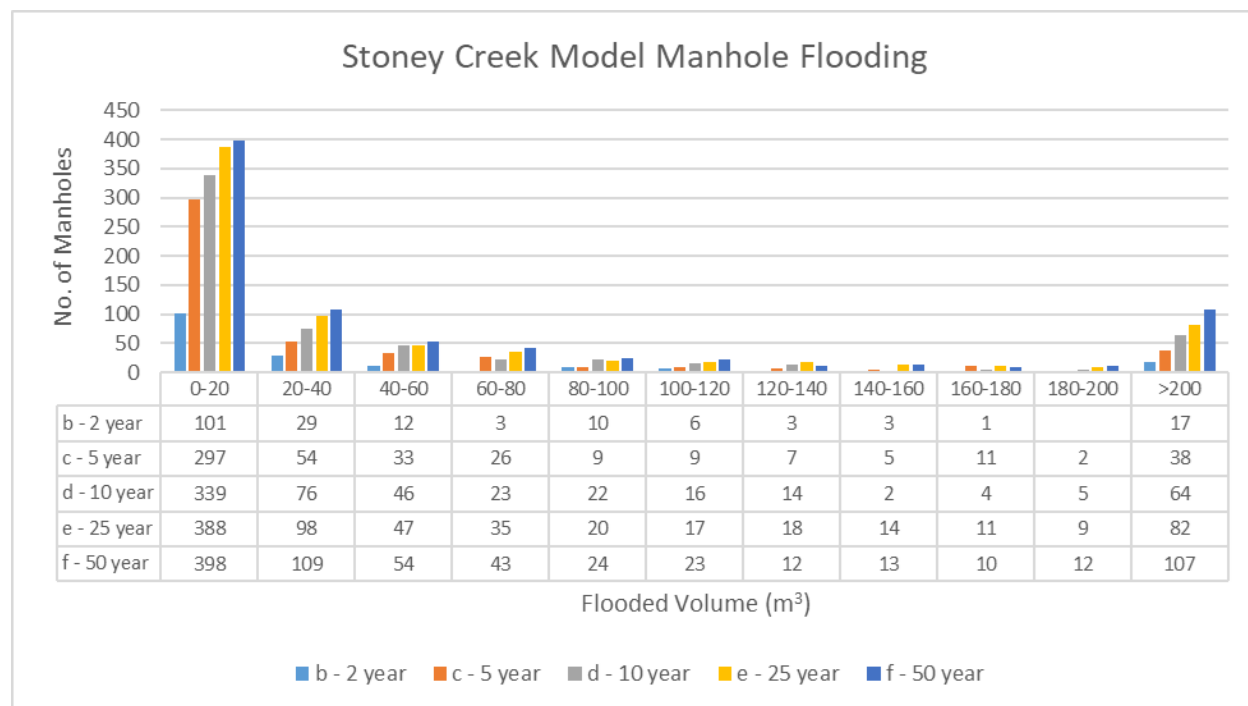
It should be noted that some of the flooding observed in the model may be a result by errors in the City's GIS database. In particular, the surcharge indicated south of Memorial Park is caused by a 250 mm pipe connecting Cannon Drive to Green Road. This pipe is connected to an upstream 675 mm pipe and a downstream 1050 mm pipe, so actual size of this pipe should be verified with either as-built drawings or a field investigation.

Overall, the modelling results indicate the storm sewer experiences notable capacity issues in the area south of Barton Street and north of Ridge Road and Lawrence Road beginning with the 2-year design storm event. However, the significance of the potential flooding from the surcharged manholes is difficult to assess because there is no surface routing of the surcharged stormwater.

Table 4.4.8: Summary of Flooded Ponds Results for the Stoney Creek Model

Pond ID	2-year	5-year	10-year	25-year	50-year
	Flood Status	Flood Status	Flood Status	Flood Status	Flood Status
SWMF142	-	-	-	Flood	Flood
SWMF66	-	-	Flood	Flood	Flood

Figure 4.4.3.3: Stoney Creek Model – Manholes Flooded with 20 m³ Flood Volume Intervals



4.4.4 Ancaster Model

The design storm modelling results for the Ancaster model are summarized in **Table 4.4.9** and **Table 4.4.10**. The model indicates the majority of the storm sewer system is capable of effectively conveying the 2 to 10-year design storm events with only a relatively small number of manholes (<3%) experiencing any meaningful flooding. It should be noted that the legacy storm sewer design standard for the former Town of Ancaster was the 2-year storm event, given the presence of more permeable soils in this area.

However, the number of manholes that are in flood risk increases significantly during 5 design storm events. The number of flooding manholes are more than doubled during 25 to 50-year design storm event.

One overflow weir in the SWM facilities (AN10SP02) were active during 2-year design storm event. During the 5-year design storm, all overflow weirs were active except AN09SP01, AP08OF1, SWMF233, AH18SP01_E.

The flooded manholes clusters at following locations for the 25-year and 50-year design storms:

- Along Springbrook Avenue and Regan Drive
- Along Sandhill Drive

- Bridgeport Crescent from Sable Drive to Kitty Murray Lane
- Raymond Road from Fair Street to Whittington Drive
- Along Daffodil Crescent
- Along Broad Leaf Crescent
- Along Seymour Drive near Viscount Drive
- Along Deerview Avenue
- Norma Crescent near Coventry Court
- Valmont Count near Moore Crescent
- Along Devonshire Terrace
- Amberly Boulevard from Cornwallis Road to Sunnymeade Drive

Note that the system along Springbrook Avenue and Regan Drive was added in 2025 based on 2022 GIS data and subdivision design documents. The catchment parameters and connection may need further refinement when more information is made available.

Overall, the modelling results indicate the storm sewer has capacity to effectively handle the 2 to 10-year design storm event with only a small percentage of manholes experiencing meaningful flooding. Clustered flooding appears starting from 10-year design storm.

Table 4.4.9: Summary of Design Storm Manhole Flooding in the Ancaster Model

COMMUNITY	Description	No Flood Risk [<1.8m]	Flood Risk [-1.8m-0.3m]	Out of Manhole Flood [>0.3m]
Ancaster	April 20th 2017	1768	325	0
	2 year	1013	1075	5
	5 year	547	1520	26
	10 year	419	1618	56
	25 year	317	1656	120
	50 year	248	1691	154

Table 4.4.10: Summary of Design Storm Pipe Discharge Capacity in the Ancaster Model

COMMUNITY	Description	Discharge/Capacity <85% [km]	Discharge/Capacity 85%-100% [km]	Discharge/Capacity >100% [km]
Ancaster	April 20th 2017	101.5	3.3	6.9
	2 year	71.7	12.5	27.5
	5 year	56.4	16.8	38.5
	10 year	51.6	18.9	41.2
	25 year	47.3	20.8	43.6
	50 year	44.6	20.9	46.3

5 PLANNING AND GROWTH PROJECTIONS

5.1 Overview

The Integrated Interim Water, Wastewater and Stormwater Master Servicing Strategies will address servicing requirements to the year 2051. The servicing needs are intended to be identified and evaluated within the GRIDS2 process that will establish the framework of overall effective and efficient growth across the City.

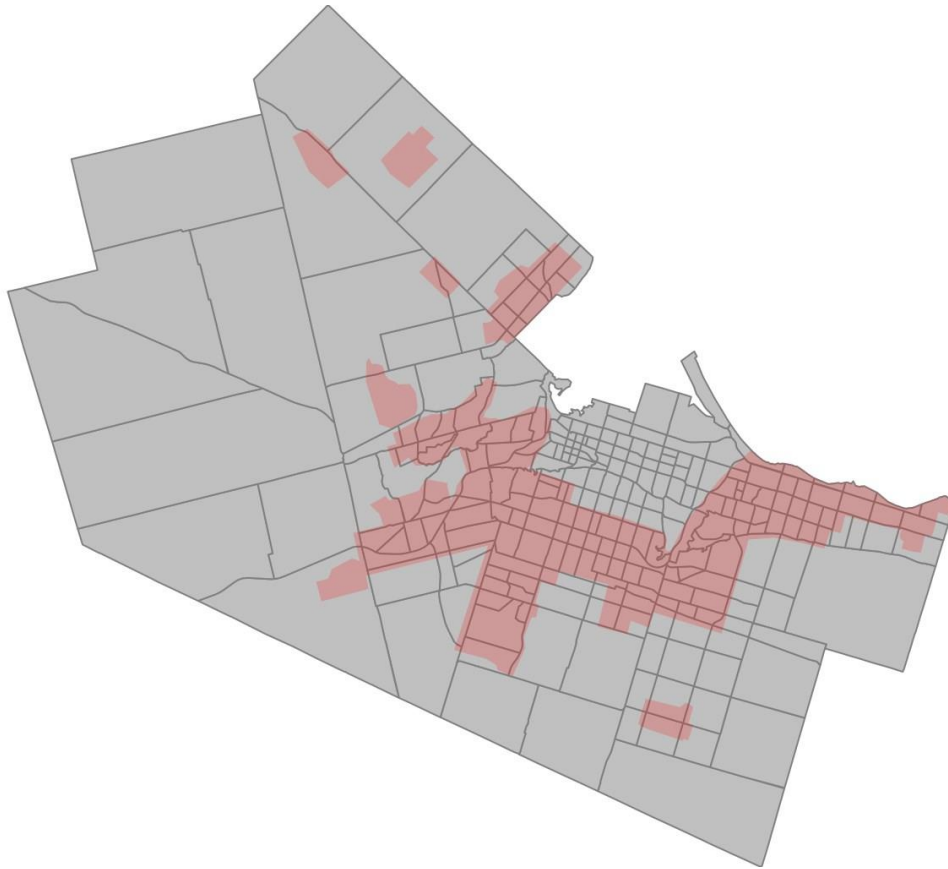
GRIDS2 is an integrated planning process that identifies a land use structure, infrastructure, economic development strategy and final implications for growth across the City of Hamilton. This planning process will determine how and where population growth can be accommodated, which will directly impact the servicing strategies and infrastructure locations identified in the Interim Water, Wastewater and Stormwater Master Servicing Strategies.

Integration with GRIDS2 will require an iterative analytical / assessment process. The City will be balancing greenfield and built boundary densities. These developments will continue to place demands upon the capacity of existing infrastructure. Given the concurrent nature of the GRIDS2 process and other key studies, the approach for these Integrated Interim Master Servicing Strategies requires flexibility and agility to adjust the servicing impact evaluation as the growth needs are defined and refined over the course of the study.

Due to the changes in planning scenarios over the course of this study, several different iterations and versions of the planned growth/future land use data have been reviewed since study inception. The data ultimately applied were provided by the City in August 2025 and is based on the no urban boundary (NUBE) scenario, thus all growth is based within the existing urban boundary through infill and intensification.

There are 265 traffic zones (community areas/blocks typically bounded by arterial roadways) across the City (113,043 ha), ranging from 11 ha to 6,910 ha (average area of 427 ha). The specific overlap / coverage of the traffic zones with the separated storm sewer area is presented in **Figure 5.1.1**.

Figure 5.1.1: Traffic Zone Limits and Extents of Separated Storm Sewer System



Based on the data provided, there are 195 traffic zones which intersect to some degree with the separated storm sewer area. In many cases however, the difference in boundaries is slight (i.e. only a minor portion of a traffic zone is within the separated storm sewer drainage area or vice versa).

Based on previous discussions with City staff (July 8, 2021), it was understood that the City may have more refined data at a parcel or block level to support a more reasonable / accurate assessment of land use changes, given the scale disparity between subcatchments and traffic zones. An example of parcel level data is presented in **Figure 5.1.2.**

Figure 5.1.2: Example of Property Level Growth and Intensification Data



Ultimately, more refined data (in the form of intensification areas) could not be provided at a property level to support stormwater analyses, and in particular mapping of areas of change.

As a further option, the City was able to provide data with respect to future development “nodes” and “corridors” – where intensification and re-development within the existing urban boundary is expected to be concentrated. This information is presented on **Drawing 6**. As evident, there are 12 different intensification nodes, 10 of which are located within the separated storm sewer area, identified as:

- Waterdown
- Dundas
- Ancaster
- Meadowlands
- James \ Rymal
- Limeridge
- Heritage Green
- West Elfrida
- Stoney Creek

- Centennial

The majority of the arterial roadways within the separated storm sewer area are also noted as intensification corridors, including:

- Main Street
- Queenston Road
- Barton Street
- Mohawk Road
- Upper James Street
- Rymal Road
- Centennial and Upper Centennial Parkway

Based on previous review, the nodes and corridors information is considered the best estimate of areas where re-development may occur and thus where storm sewer capacity issues may have the greatest impact, due to the additional runoff generated by these zones. As such, these data have been used to support the identification and prioritization of stormwater infrastructure improvements, as discussed in subsequent sections.

6 STORMWATER IMPACT ASSESSMENT AND STRATEGY DEVELOPMENT

6.1 Objectives and Rationale

As a growth-focused study, the proposed stormwater strategy should prioritize and focus on stormwater infrastructure improvements and measures that coincide with areas of expected future growth and development. As noted in Section 5, for the purposes of the stormwater assessment, the nodes and corridors data (refer to **Drawing 6**) is considered the best indicator of the locations and form of planned intensification, given that other planning data do not adequately meet the needs of expected surface cover changes; notably, the focus of this assessment is on changes in population and employment numbers (which affect water\wastewater) rather than land cover (which affects stormwater).

Overall, it should be noted that re-developments (infill / intensification) that require Site Plan Control would be expected to also require on-site stormwater management, including quantity controls. As per current City standards (i.e. the City of Hamilton Comprehensive Development Guidelines and Financial Policies Manual; see reference in Section 2.5.4), this would require over-control of the 100-year post-development peak flow to the 5-year pre-development flow, for areas where there is no defined and acceptable overland flow route (which is considered to be the vast majority of the locations within the study area). This would then inherently have the net effect of reducing discharges to the stormwater system rather than increasing them, through planned re-development/intensification and policy-driven SWM requirements.

It is noted however that smaller re-developments that do not meet the requirements for Site Plan Control would be expected to proceed without stormwater controls and thus could potentially increase flows. Notwithstanding, many infill/intensification areas would potentially already have some existing development on the subject lands, thus the actual net increase in surface cover imperviousness may actually be more limited.

Regardless, the overall objective of the stormwater strategy is to address identified (modelled) drainage system deficiencies in areas of expected re-development, to provide a more consistent level of service and limit drainage and flooding risks.

A secondary priority of the stormwater assessment (beyond areas of expected infill / intensification) is given to locations that also overlap with City-reported higher surface flooding risk (also shown on **Drawing 6**).

The priority for the stormwater strategy is on storm sewer upgrades. Notwithstanding, stormwater storage facilities may be considered where potentially feasible and preferable, but limited opportunities are expected, given available space \ land and typical storm sewer grades (deep below ground). Right-of-way improvements, road re-grading \ re-profiling and other such measures should still be considered as part of future detailed design planning and work for rehabilitation works.

6.2 Strategy Development – Storm Sewer Upgrades

This section describes the recommended storm sewer upgrade projects as per the modelling results based on the 5-year design storm event (as discussed in Section 4). For each project, a map is provided to indicate the locations where the respective model indicates that the 5-year design storm is causing surface flooding.

While it is recognized that the City's design criteria require the storm sewers to handle the 5-year design storm event at 85% full flow capacity, it was decided as part of this assessment, to focus the strategy on locations with surface flooding issues within and adjacent to the areas prioritized for development for the following reasons:

- 1 The number of projects required to address pipe surcharging above 85% full flow capacity throughout the entire storm sewer system would be substantial and considered impractical (and unaffordable) to address, particularly given the relatively limited risk.
- 2 As noted earlier, the models being employed for the assessment are either poorly calibrated, or the calibration is based on monitoring locations with data that are more than 10 years old. Furthermore, none of the models were calibrated to measured water levels which are the main indicator of pipe surcharge and flooding.

The projects are organized according to the areas covered by the respective stormwater models, including the Ancaster model, the Dundas model, the Stoney Creek model, and the Six Cities model. The individual projects are labelled using the street name(s) where the project is located, a letter code to reflect the name of the modelled area, and a number to reflect the sequence (e.g. 'Kitty Lane (A1)' is located along Kitty Lane, and it is the first project recommended from the Ancaster model).

A profile plot is also provided for each project to show the maximum hydraulic grade line (HGL) for the 5-year design storm for existing and recommended storm sewer infrastructure conditions. The pipes shown in the profile plots represent the recommended pipe diameters and slope.

The following sections identify each location, characterize the existing system deficiencies, and provide a project recommendation for stormwater drainage system upgrades. Due to the relative uncertainty associated with the model results, the storm sewer system upgrade recommendations provided herein are intended to be used as a means for identifying the problem areas, the causes of the problems, and potential solutions for those problems. However, in each case, it is recommended to collect flow and water level data in these locations to better characterise the system response to rainfall events and to update the model calibration to measured flows and water levels in the near vicinity of the proposed projects.

In addition to the preceding, it has been assumed that the focus would be on storm sewers less than 2 m in diameter, as the costs associated with replacing or upgrading storm sewers 2 m diameter or greater is substantial. As per previous recommendations, additional flow monitoring and model calibration \ validation should be considered, as well as a cost-benefit analysis before considering any such upgrades to larger diameter systems. As such, the focus of this study has been on storm sewers less than 2 m in diameter.

Because of the limited availability of overland flow modelling (dual drainage) within the majority of the available models (other than Ancaster), the potential to implement Inlet Control Devices (ICDs) to limit storm sewer surcharging has not been considered, as it would be necessary to evaluate the overland flow capacity. Notwithstanding, this could be considered as part of subsequent detailed design work.

Drawing 11 presents an overview of the storm sewer upgrade areas while **Drawings 12a to 12g** present the nodes and corridor areas, the areas with high incidents of surface flooding, and the recommended stormwater upgrades throughout the study area. A detailed listing of the upgrades is available in **Appendix H**.

6.2.1 Ancaster

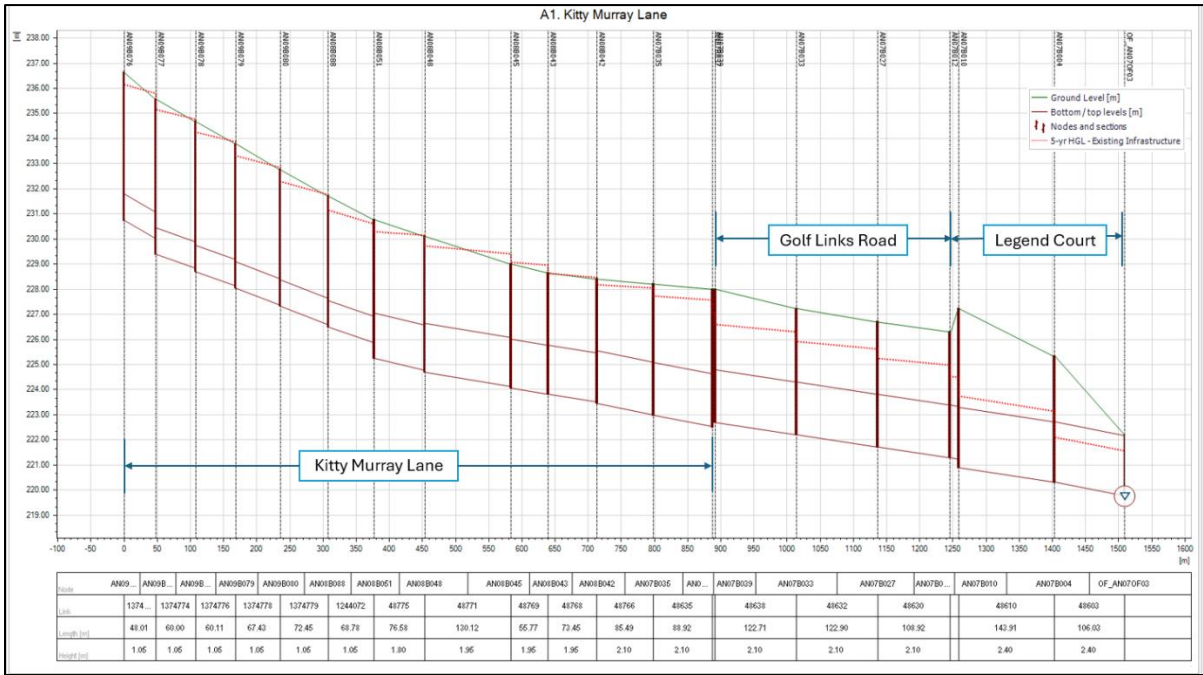
6.2.1.1 Kitty Murray Lane (A1)

The project site is located along a section of storm sewer running along Kitty Murray Lane through to the outfall (see **Drawing 12a** and **Figure 6.2.1**). This section has surcharged pipes along the entire length, with the exception of the outfall, and there is moderate surface flooding at a few manholes located immediately outside of the Ancaster node area. The maximum hydraulic grade line (HGL) shows significant jumps at manhole locations where there are 90-degree changes in flow direction with lateral pipes also contributing inflow. While it is reasonable to expect larger head losses at these locations, the relatively large head losses suggest the model may be exaggerating the value. While the model was calibrated to flow at a monitoring location immediately upstream of the outfall, it was not calibrated to water level. A comparison of

available modelled and measured water levels at the flow monitoring station indicated the model is over-predicting the water level.

Given the already large pipe sizes (>2 m) through most of this section, and due to the uncertainty of the model prediction, **no upsizing is recommended at this time** until further investigation of this site been completed (including additional monitoring data collection and model calibration refinement).

Figure 6.2.1: Profile for Kitty Lane (A1)



6.2.1.2 Wilson Street East (A2)

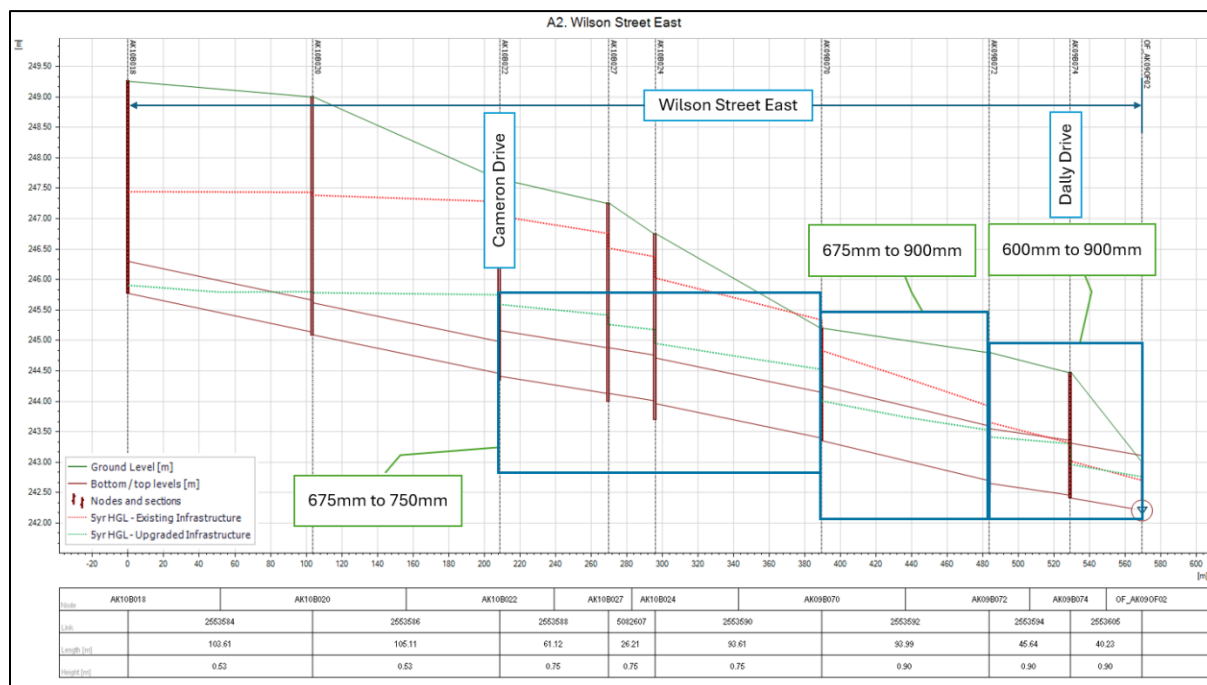
This project runs along Wilson Street from St. Margaret Street to Dalley Street and is located entirely within the Ancaster node area (see **Drawing 12a** and **Figure 6.2.2**). The last two pipes before the outfall are 600 mm diameter while the 4 pipes immediately upstream are all 675 mm diameter, so there is a slight constraint at the end of this section. Although the outfall pipe is only slightly surcharged, the maximum HGL shows a combination of a steep HGL plus large head losses at manholes cause the pipes to be surcharged along the remainder of the upstream section with moderate flooding at one manhole. Although the degree of surcharging is significantly affected by the magnitude of the head losses calculated along this section, the steep HGL also indicates there is a capacity constraint in the vicinity where the pipe diameter narrows from 675 to 600 mm. As a result, a modelling exercise was conducted to make recommendations for upsizing the pipes.

The modelling primarily focussed on upsizing the pipes upstream of the outfall up to Cameron Drive. The recommended pipe upgrades and the resulting reduction in the Interim Stormwater Master Servicing Strategy
Project No. CA-EI-TPB178090
City of Hamilton

maximum HGL are shown in **Figure 6.2.2** while the details of the recommended pipe upgrades are provided in **Appendix H**. The recommended upgrades eliminate surface flooding at this site and significantly reduce the maximum HGL.

In addition to the recommended upgrades, it is also recommended to collect additional flow and water level monitoring data and revisit the model calibration before confirming the required pipe diameters.

Figure 6.2.2: Profile for Wilson Street East (A2)



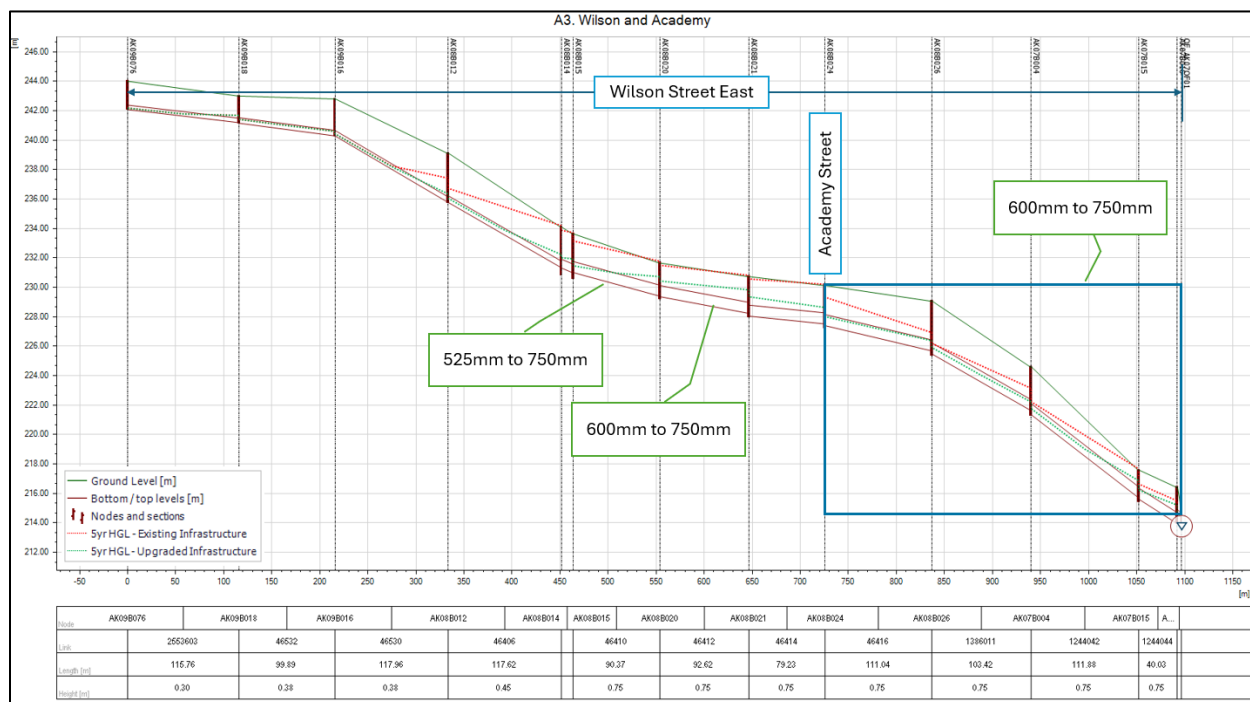
6.2.1.3 Wilson and Academy (A3)

The project site is located along a section of storm sewer running along Wilson Street from Stoneham Park to the outfall at Rousseaux Street (see **Drawing 12a** and **Figure 6.2.3**). Along this section the slope of the pipes increases significantly after Academy Street, and the pipe diameter decreases from 750 mm upstream of Academy to 600 mm downstream of Academy. The modelling shows the storm sewer is surcharged along the entire length and surface flooding is predicted at manholes from Church Street to Academy Street. While the large head losses in the steep section of pipe appears to be the main cause of the pipe surcharge in this section, the steep HGL in the first section of the 600 mm diameter pipe also indicates there is a capacity issue in this pipe. As a result, a modelling exercise was conducted to make recommendations for upsizing the pipes.

The modelling primarily focussed on upsizing the pipes upstream of the outfall and, in particular, the flat section of pipes upstream of Academy Street. The recommended pipe

upgrade and the resulting reduction in the maximum HGL are shown in **Figure 6.2.3** while the details of the recommended pipe upgrades are provided in **Appendix H**. The recommended upgrades eliminate surface flooding at this site and significantly reduce the maximum HGL.

Figure 6.2.3: Profile for Wilson and Academy (A3)



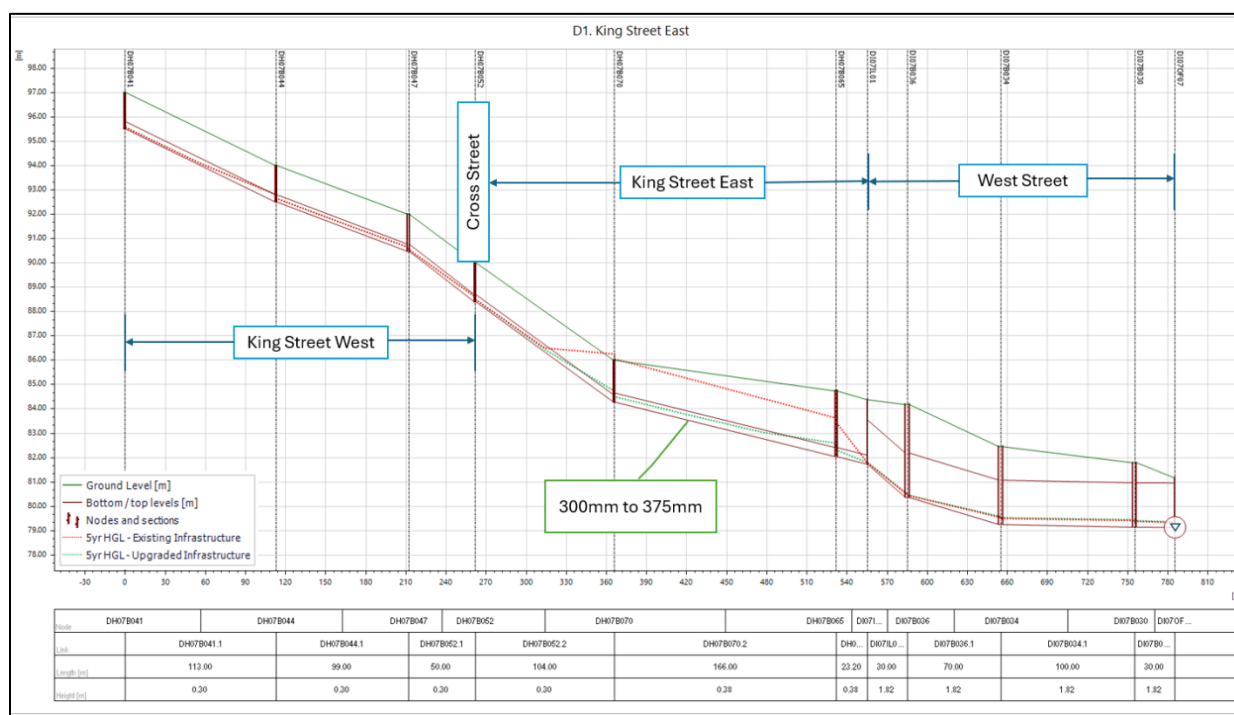
6.2.2 Dundas

6.2.2.1 King Street East (D1)

The project site is located along a section of storm sewer running along King Street East from Sydenham Street to West Street and then south along West Street where it discharges to an outfall on Spencer Creek (see **Drawing 12b** and **Figure 6.2.4**). The problem occurs in the section of pipe between Cross Street and York Street where the pipe transitions from very steep to very flat. The modelling results show that this transition significantly slows the flow and causes a capacity issue along the flat section of pipe. Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the flooding condition.

The modelling primarily focussed on upsizing the long, flat pipe on King Street East and lowering the downstream invert to create more slope. The recommended pipe upgrade and the resulting reduction in the maximum HGL are shown in **Figure 6.2.4** while the details of the recommended pipe upgrades are provided in **Appendix H**. The recommended upgrades eliminate surface flooding at this site and significantly reduce the maximum HGL.

Figure 6.2.4: Profile for King Street East (D1)

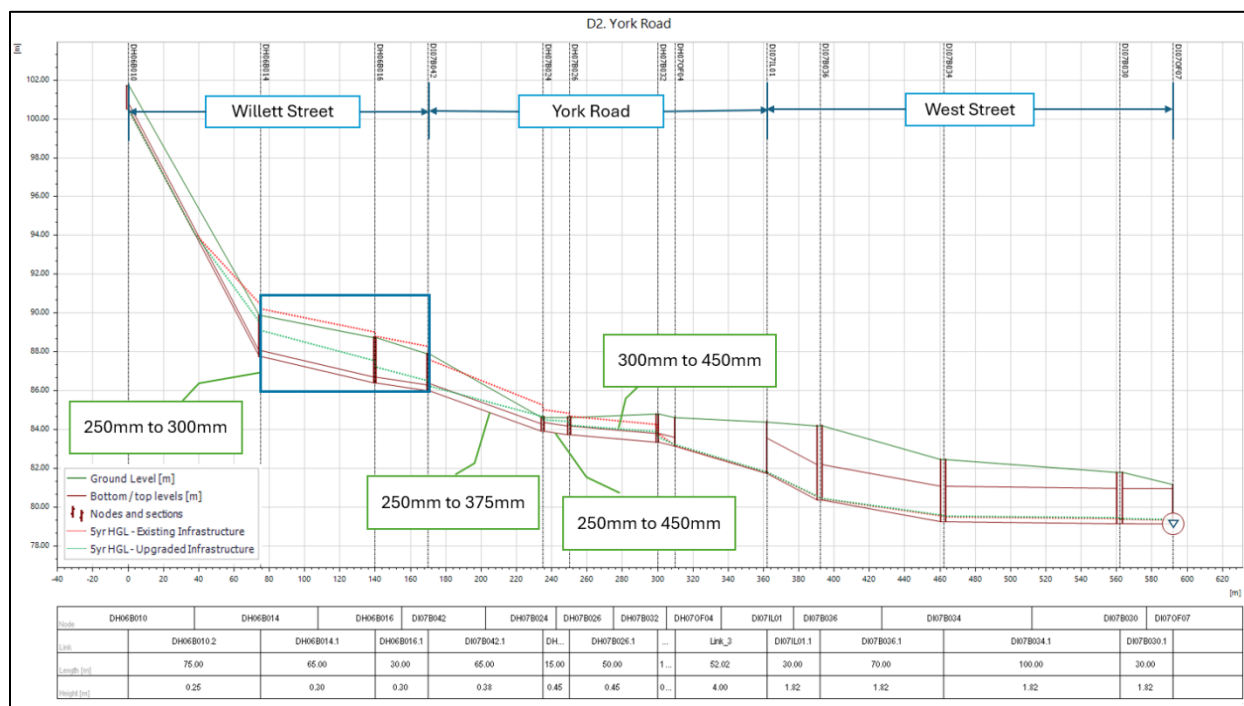


6.2.2.2 York Road (D2)

The project site is located along a section of storm sewer on York Road from Willet Road to King Street (see **Drawing 12b** and **Figure 6.2.5**). The section of pipe immediately south of Park Street East is surcharged and the manholes at the intersection of Park Street East and York Road, and along Willet Road are flooded. The surcharged pipe is located inside the Dundas node area while the flooded manholes are immediately north of the node area boundary. The flooding at the intersection of York Road and Willet Road is caused by a combination of small pipe diameters (<300 mm) and the very shallow manhole depths (<1 m), while the flooding along Willet Road is caused by small pipe diameters (<300 mm). Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the flooding condition.

The modelling primarily focussed on upsizing the undersized pipes on York Road and Willet Street that do not meet the City's current design standard of a minimum 300 mm pipe diameter. The recommended pipe upgrades and the resulting reduction in the maximum HGL are shown in **Figure 6.2.5** while the details of the recommended pipe upgrades are provided in **Appendix H**. The recommended upgrades eliminate surface flooding at this site and significantly reduce the maximum HGL.

Figure 6.2.5: Profile for York Road (D2)

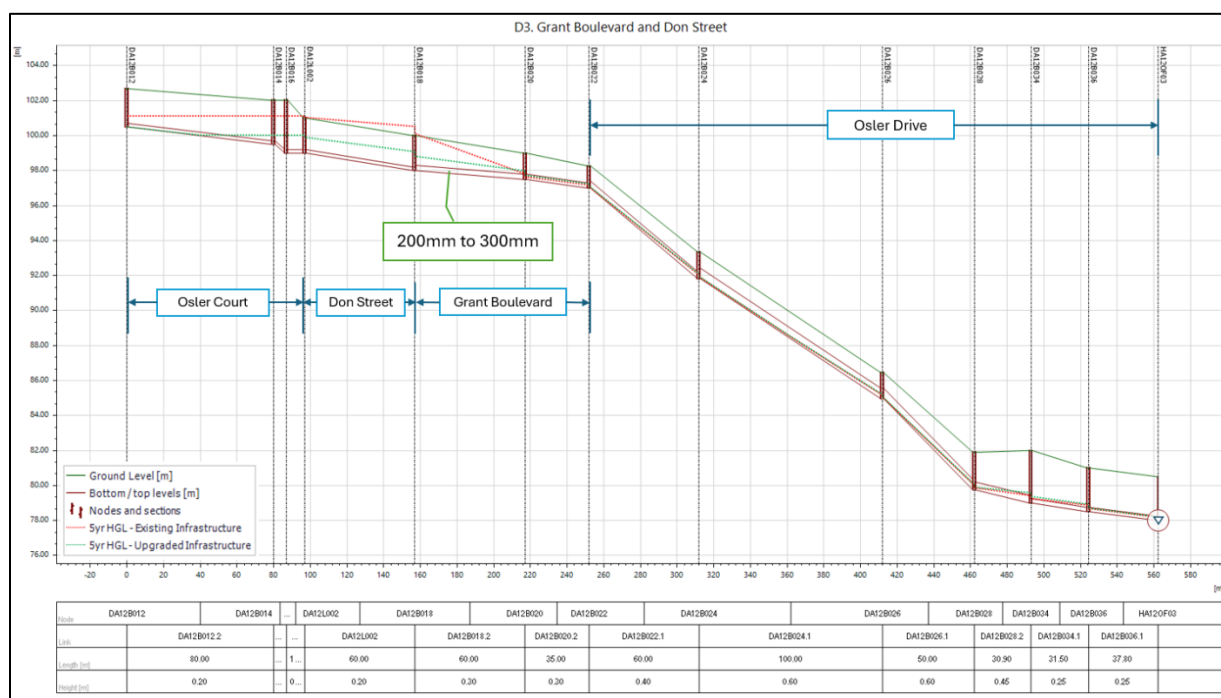


6.2.2.3 Grant Boulevard and Don Street (D3)

This project site is located along a section of storm sewer on Grant Boulevard from Don Street to Osler Drive and is located inside the Corridor area (see **Drawing 12b** and **Figure 6.2.6**). This section of sewer along Grant Boulevard is relatively flat and has a pipe diameter of only 200 mm and the modelling results show the pipe along Grant Boulevard is surcharged and the manhole at the intersection of Grant Boulevard and Don Street is surcharged. Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the flooding condition.

The modelling primarily focussed on upsizing the undersized pipes on Grant Boulevard that do not meet the City's current design standard of a minimum 300 mm pipe diameter. The recommended pipe upgrades and the resulting reduction in the maximum HGL are shown in **Figure 6.2.6** while the details of the recommended pipe upgrades are provided in **Appendix H**. The recommended upgrades eliminate surface flooding at this site and significantly reduce the maximum HGL.

Figure 6.2.6: Profile for Grant Boulevard and Don Street (D3)



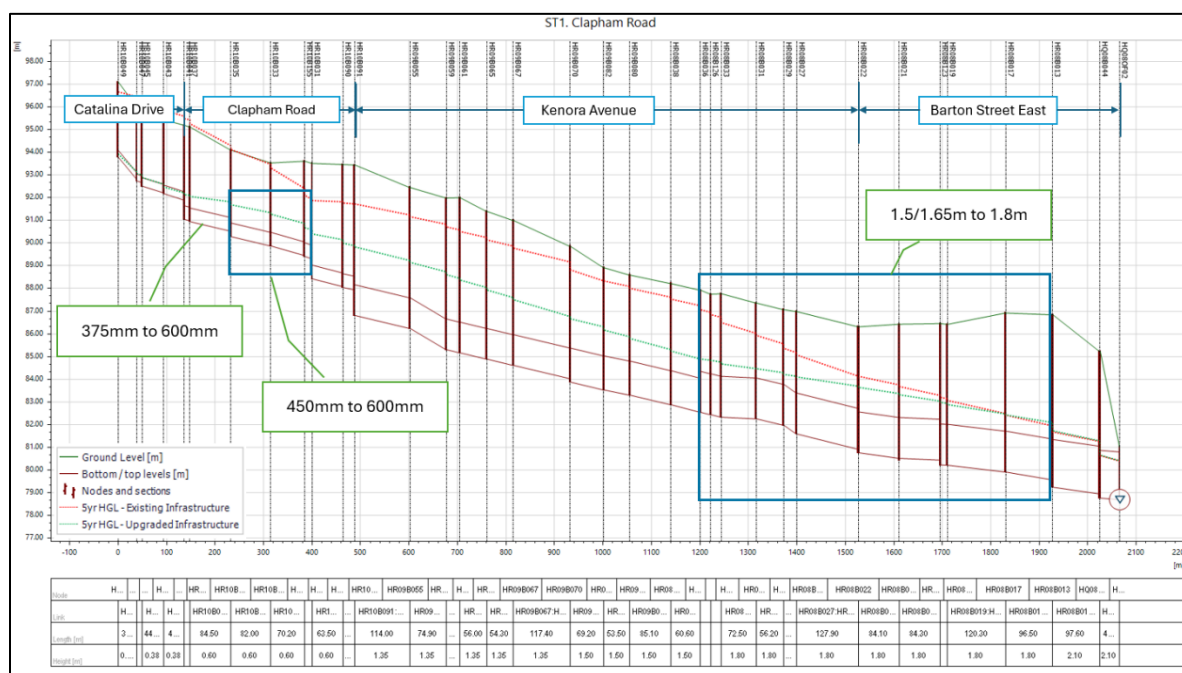
6.2.3 Stoney Creek

6.2.3.1 Clapham Road and Kenora Avenue (ST1)

This project site is located along a section of storm sewer that runs north along Clapham Road from Catina Drive to Highway 8, and then along Kenora Avenue from Highway 8 to Bancroft Street, and then west along Bancroft Street to the outfall at Red Hill Creek (see **Drawing 12c** and **Figure 6.2.7**). The modelling results indicate this section of sewer is surcharged along the entire length and some surface flooding occurs on Clapham Road. Given the steep maximum HGL in the pipes along Bancroft Drive and at the north end of Kenora Avenue, there appears to be a capacity limitation along this section, as well as on Clapham Road. Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the surcharging and flooding conditions.

The modelling primarily focussed on upsizing the pipes immediately upstream of the outfall along Barton Street East and Kenora Avenue, and the pipes on Clapham Road. The recommended pipe upgrades and the resulting reduction in the maximum HGL are shown in **Figure 6.2.7** while the details of the recommended pipe upgrades are provided in **Appendix H**. The recommended upgrades eliminate surface flooding at this site and significantly reduce the maximum HGL.

Figure 6.2.7: Profile for Clapham Road and Kenora Avenue (ST1)

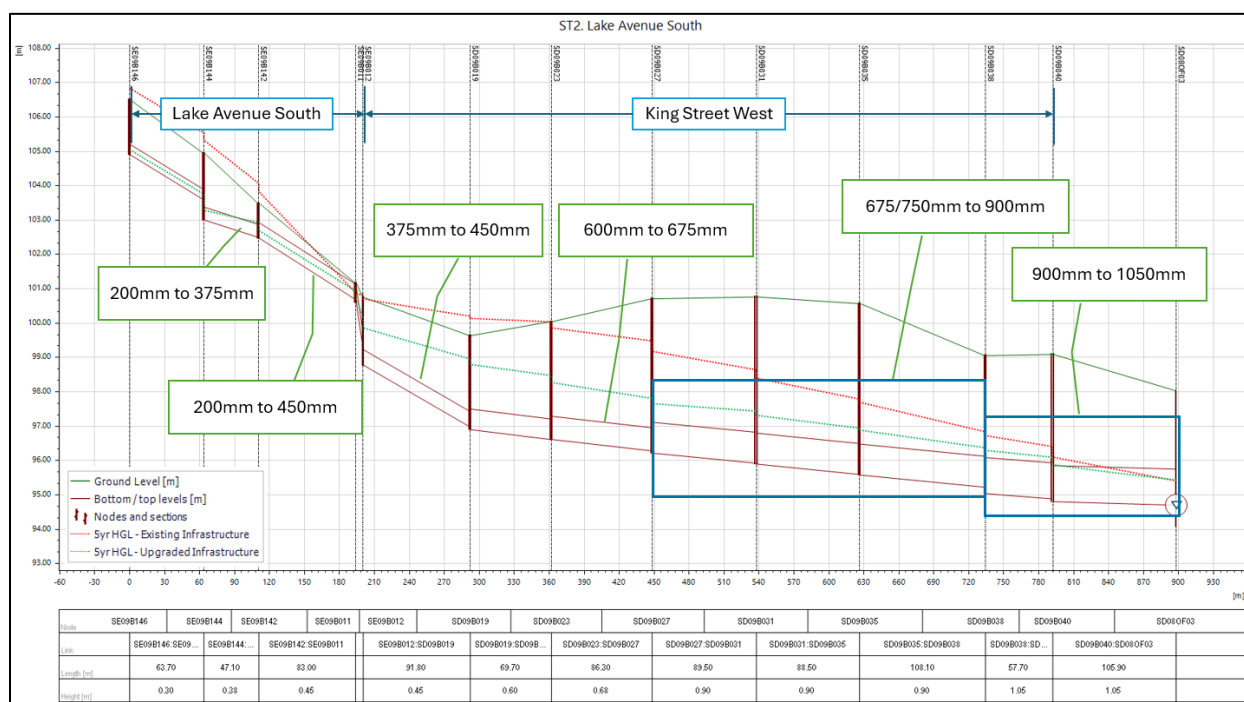


6.2.3.2 Lake Avenue South and King Street West (ST2)

This project site is located along a section of storm sewer that runs north along Lake Avenue South and then west along King Street West and then north to the outfall in Faircourt Park (see **Drawing 12c** and **Figure 6.2.8**). The modelling results indicate this section of sewer is surcharged along the entire length and is causing flooding on King Street West at the intersection of First Street South, and along Lake Avenue South. The steep maximum HGL along King Street and along Lake Avenue South indicates there are capacity constraints along these sections. Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the surcharging and flooding conditions.

The modelling primarily focussed on upsizing the pipes along the entire project site. The recommended pipe upgrades and the resulting reduction in the maximum HGL are shown in **Figure 6.2.8** while the details of the recommended pipe upgrades are provided in **Appendix H**. The recommended upgrades eliminate surface flooding at this site and significantly reduce the maximum HGL.

Figure 6.2.8: Profile of Lake Avenue South (ST3)

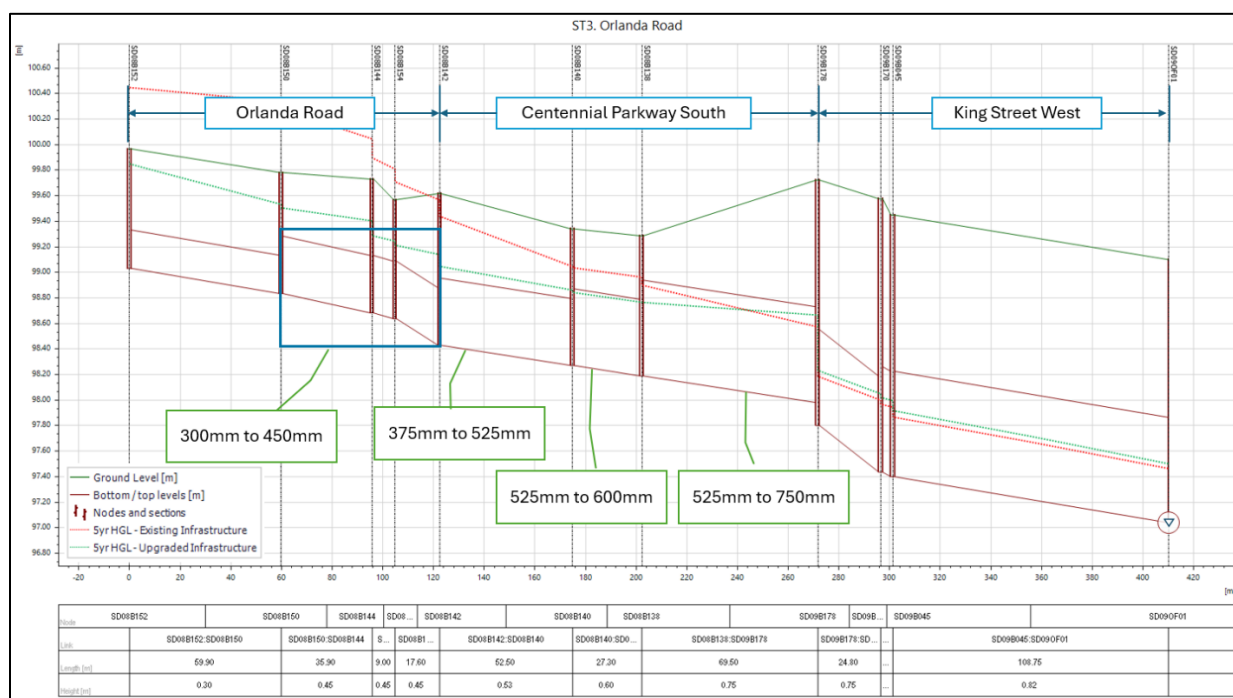


6.2.3.3 Orlanda Road and Centennial Parkway (ST3)

The site is located along a section of storm sewer that runs east along Orlanda Road from Grand Avenue to Centennial Parkway South, and then south along Centennial Parkway South to King Street West and then east along King Street West until it discharges into Battlefield Creek (see **Drawing 12c** and **Figure 6.2.9**). This entire section of storm sewer is located inside the Corridor Area. The modelling results indicate this section of pipe is surcharged from the beginning at Orlanda Road to the intersection of Centennial Parkway South and King Street West, and surface flooding is occurring in the section along Orlanda Road. The steep maximum HGL along several sections of pipe indicates there are capacity limitations in some of the pipes. While there are some significant jumps in the maximum HGL due to head losses at manhole locations, the majority of the problem appears to be related to capacity limitations. Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the surcharging and flooding conditions.

The modelling primarily focussed on upsizing the pipes along Orlanda Road and Centennial Parkway South. The recommended pipe upgrades and the resulting reduction in the maximum HGL are shown in **Figure 6.2.9** while the details of the recommended pipe upgrades are provided in **Appendix H**. The recommended upgrades eliminate surface flooding at this site and significantly reduce the maximum HGL.

Figure 6.2.9: Profile of Orlanda Road and Centennial Parkway (ST3)

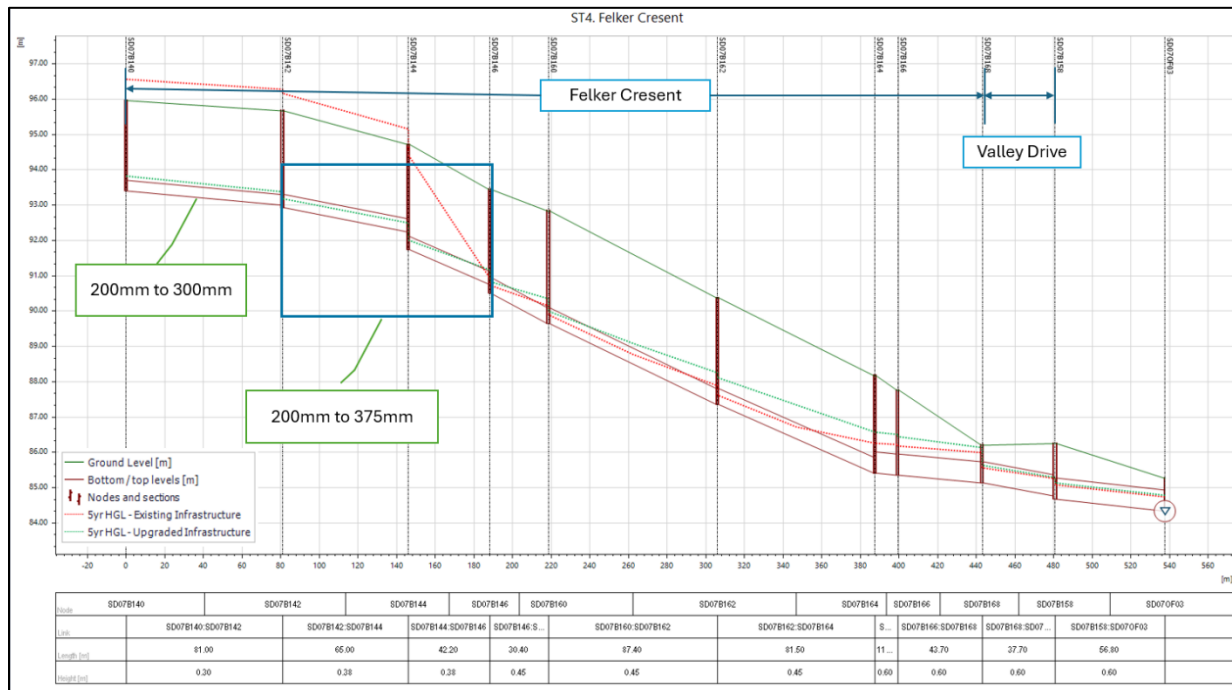


6.2.3.4 Felker Crescent (ST4)

The project site is located along a section of storm sewer that runs east along Felker Avenue and then north along Felker Crescent where it discharges into Battlefield Creek (see **Drawing 12c** and **Figure 6.2.10**). The section of the storm sewer on Felker Avenue is located inside the Corridor area. The modelling results indicate that the pipes are surcharged near the outfall and along Felker Avenue, and the section along Felker Avenue is experiencing significant surface flooding. The pipes on Felner Avenue all have a diameter of only 200 mm so the problem is clearly related to a capacity limitation. At the downstream end of this section, at north end of Felker Crescent, the slope of the pipes flattens out, and the pipe diameters increase from 450 mm to 600 mm, but the model shows there is still some pipe surcharging occurring. However, the model is also showing a relatively large head loss at this location because the pipes change direction by 90 degrees. Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the surcharging and flooding conditions along Felker Avenue but the issue with the surcharged pipes near the outfall should be studied further to ensure the simulated head losses are reasonable.

The modelling primarily focussed on upsizing the undersized pipes at the upstream end of Felker Crescent. The recommended pipe upgrades and the resulting reduction in the maximum HGL are shown in **Figure 6.2.10** while the details of the recommended pipe upgrades are provided in **Appendix H**. The recommended upgrades eliminate surface flooding at this site and significantly reduce the maximum HGL.

Figure 6.2.10: Profile of Felker Crescent (ST4)

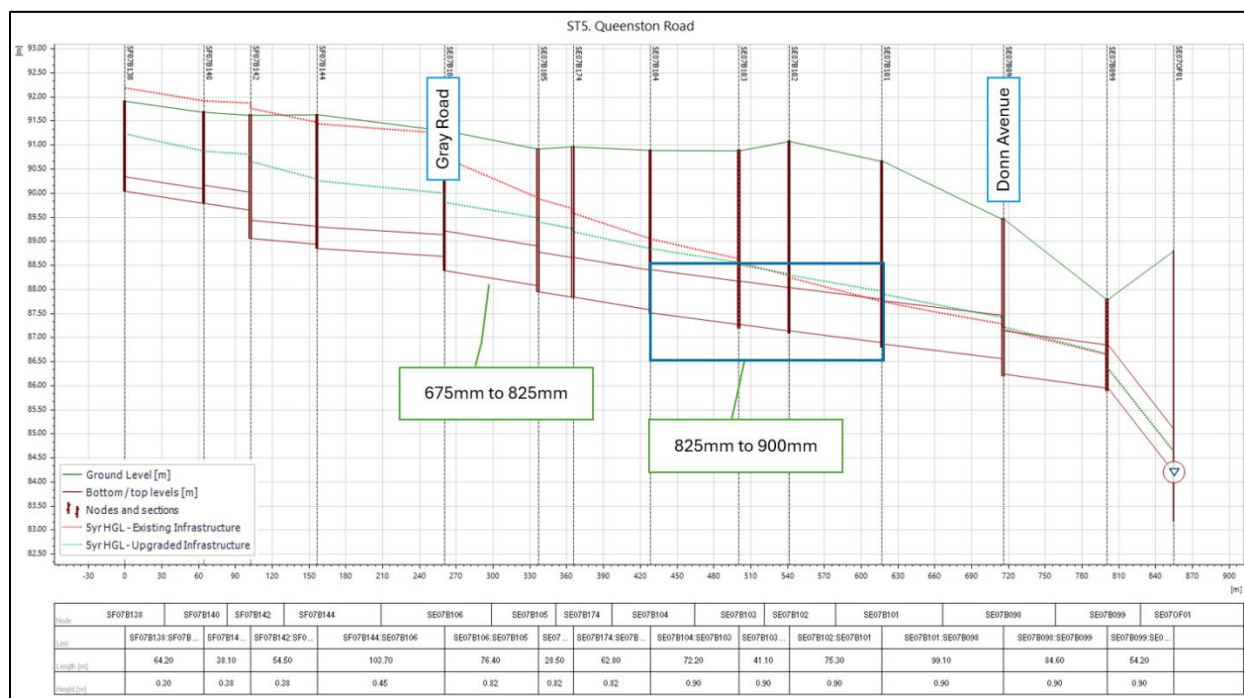


6.2.3.5 Queenston Road (ST5)

The project site is located along a section of storm sewer that runs west along Queenston Road from Durham Road to the outfall at Red Hill Creek (see **Drawing 12c** and **Figure 6.2.11**). This section of storm sewer is located entirely inside the Corridor area. The modelling results indicate that the pipes are surcharged along most of this section and the steep maximum HGL from Gray Road to Donn Avenue indicates the majority of the problem is related to capacity constraints. Some surface flooding is indicated between Durham Road and Gray Road. Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the surcharging and flooding conditions.

The modelling primarily focussed on upsizing the pipes on Queenston Road between Gray Road and Donn Avenue. The recommended pipe upgrades and the resulting reduction in the maximum HGL are shown in **Figure 6.2.11** while the details of the recommended pipe upgrades are provided in **Appendix H**. The recommended upgrades eliminate surface flooding at this site and significantly reduce the maximum HGL.

Figure 6.2.11: Profile of Queenston Road (ST5)



6.2.3.6 Berkindale Drive and Gray Road (ST6)

The project site is located along the section of storm sewer that runs north along Berkindale Drive from Federal Street to Bellamy Street and then continues north on Gray Road to South Service Road and then west on South Service Road to the outfall along Stoney Creek (see **Drawing 12c** and **Figure 6.2.12**). The south portion of the storm sewer on Berkindale Drive is immediately adjacent to the Corridor area and the entire section along Berkindale Drive is located within a flooding hotspot area. The modelling results show that the storm sewer pipes are surcharged for the entire section south of the railway corridor with some surface flooding around the intersection of Berkindale Drive and Roxborough Avenue. The steep maximum HGL along Gray Road north of the railway corridor indicates that surcharging is caused by capacity limitations along this section of the storm sewer. Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the surcharging and flooding conditions.

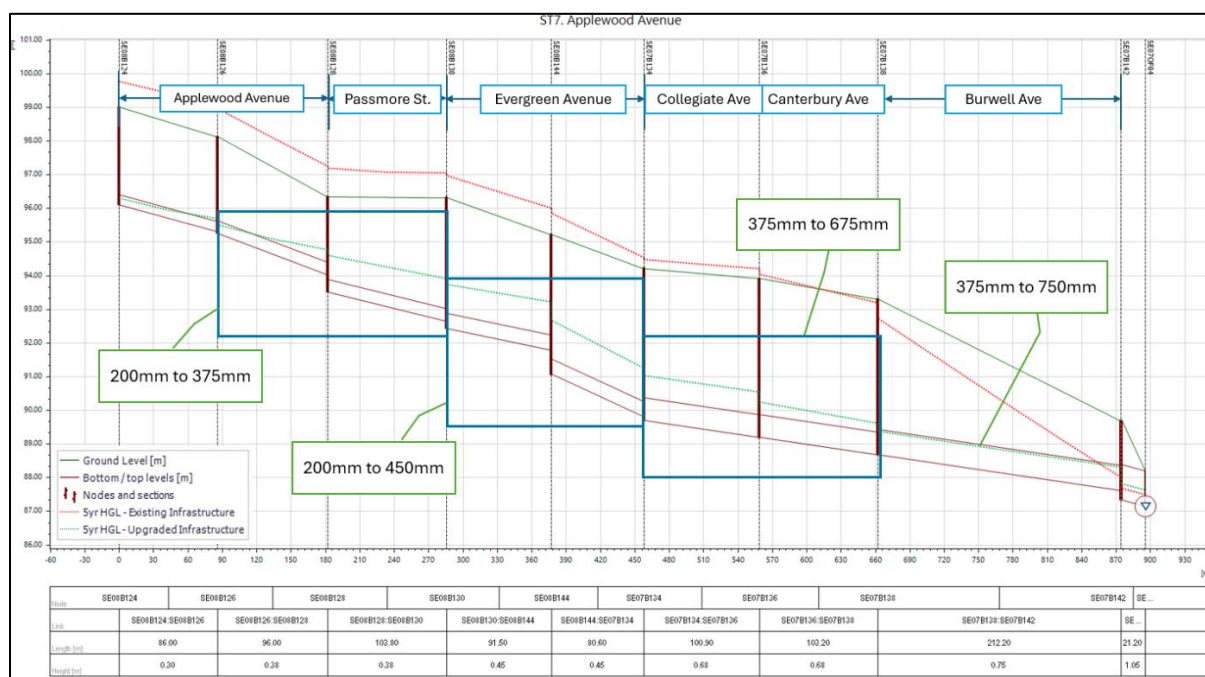
The modelling primarily focussed on upsizing the pipes along Greys Road. The recommended pipe upgrades and the resulting reduction in the maximum HGL are shown in **Figure 6.2.12** while the details of the recommended pipe upgrades are provided in **Appendix H**. The recommended upgrades eliminate surface flooding at this site and significantly reduce the maximum HGL.

[illegible]

The project site is located along the section of storm sewer that runs north along Applewood Avenue, then east along Passmore Street, then north on Evergreen Avenue, then west on Collegiate Avenue, then north on Canterbury Avenue, and west on Burwell Avenue where it discharges to an outfall on Stoney Creek (see **Drawing 12c** and **Figure 6.2.13**). The upstream end of the storm sewer on Applewood Avenue is immediately adjacent to the Stoney Creek node and the entire section of storm sewer along Applewood Avenue is within flooding hotspot. The pipes from Applewood Avenue to Evergreen Avenue are all 200 mm diameter. The modelling results show that the storm sewer pipes are surcharged for the entire section, except at the outfall, and there is surface flooding at the majority of manholes upstream of Burwell Avenue. The steep maximum HGL in the pipe on Burwell Avenue indicates that surcharging is caused by capacity limitations along this section of the storm sewer. Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the surcharging and flooding conditions.

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Figure 6.2.13: Profile for Applewood Avenue (ST7)

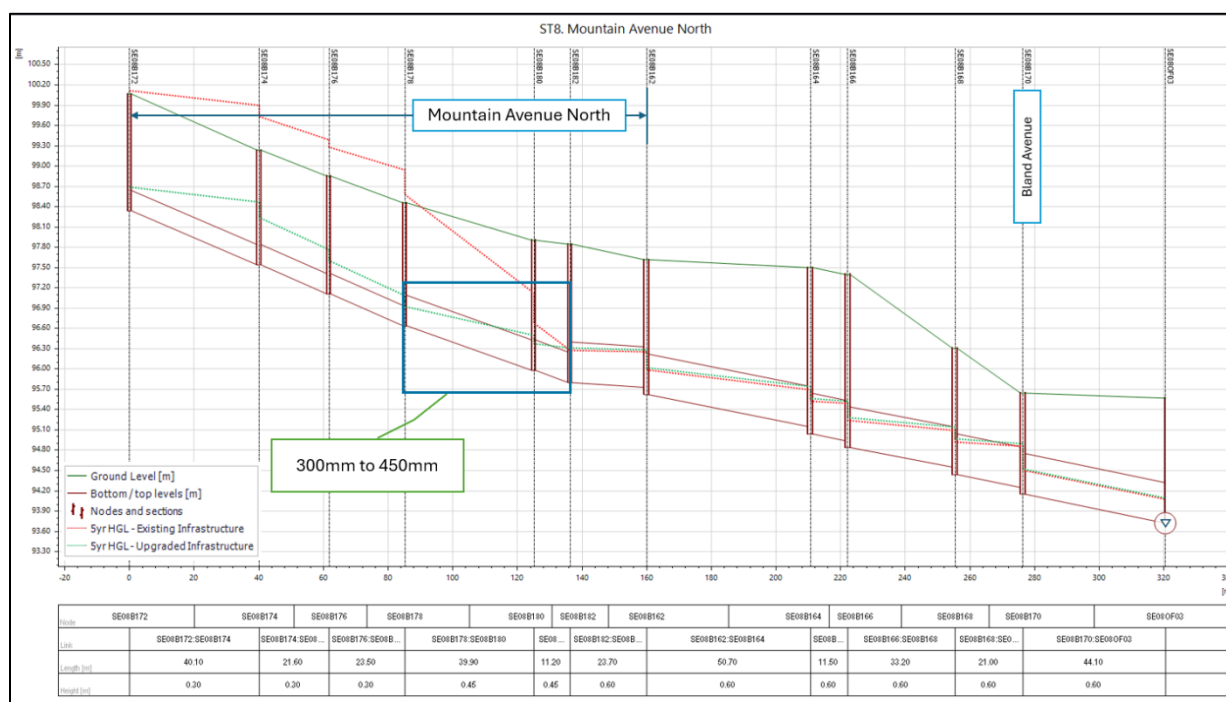


6.2.3.8 Mountain Avenue North (ST8)

The project site is located along the section of storm sewer that runs north along Mountain Avenue North and then east at the Village Court Apartments where it crosses Bland Avenue and discharges to Stoney Creek (see **Drawing 12c** and **Figure 6.2.14**). The south end of this section is adjacent to the Stoney Creek node boundary. The modelling results indicate that the majority of pipes along Mountain Avenue North are surcharged, and the manholes are flooding. The steep maximum HGL in pipes SE08B180:SE08B182 and SE08B178:SE08B180 indicate these pipes are undersized and should be upgraded. Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the surcharging and flooding conditions.

The modelling primarily focussed on upsizing the pipes on Mountain Avenue North where the slope of maximum HGL is much steeper than the pipe. The recommended pipe upgrades and the resulting reduction in the maximum HGL are shown in **Figure 6.2.14** while the details of the recommended pipe upgrades are provided in **Appendix H**. The recommended upgrades eliminate surface flooding at this site and significantly reduce the maximum HGL.

Figure 6.2.14: Profile for Mountain Avenue North (ST8)

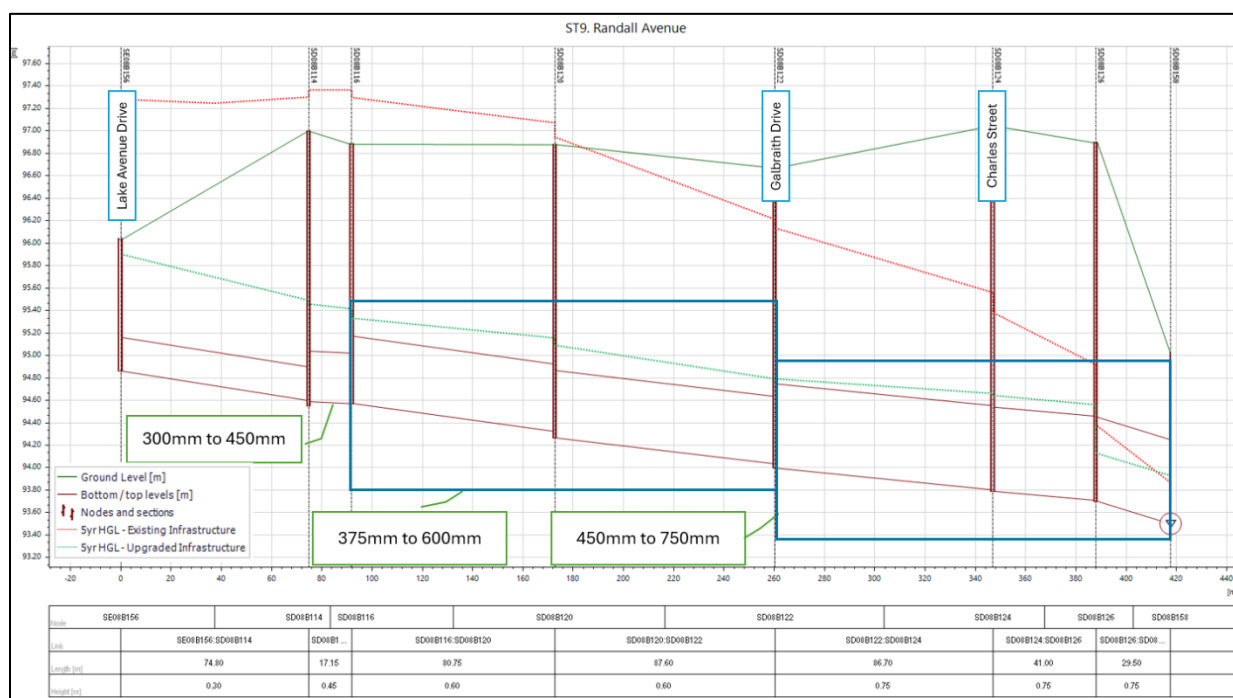


6.2.3.9 Randall Road (ST9)

The project site is located along the section of storm sewer that runs west along Randall Road from Lake Avenue Drive to the outfall into Stoney Creek (see **Drawing 12c** and **Figure 6.2.15**). This entire section of Storm Sewer is located outside of the Stoney Creek Node, but it is in the middle of a flooding hotspot. The modelling results indicate the pipes are surcharged along the entire section and significant surface flooding is occurring from Lake Avenue Drive to 2nd Street North. The steep maximum HGL indicates this is caused by lack of capacity in the pipes from 2nd Street North to the outfall. Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the surcharging and flooding conditions.

The modelling primarily focussed on upsizing the pipes upstream of the outfall to eliminate surface flooding and reduce the maximum HGL. The recommended pipe upgrades and the resulting reduction in the maximum HGL are shown in **Figure 6.2.15** while the details of the recommended pipe upgrades are provided in **Appendix H**. The recommended upgrades eliminate surface flooding at this site and significantly reduce the maximum HGL.

Figure 6.2.15: Profile for Randall Road (ST9)



6.2.4 Six Cities (Hamilton Mountain and Waterdown)

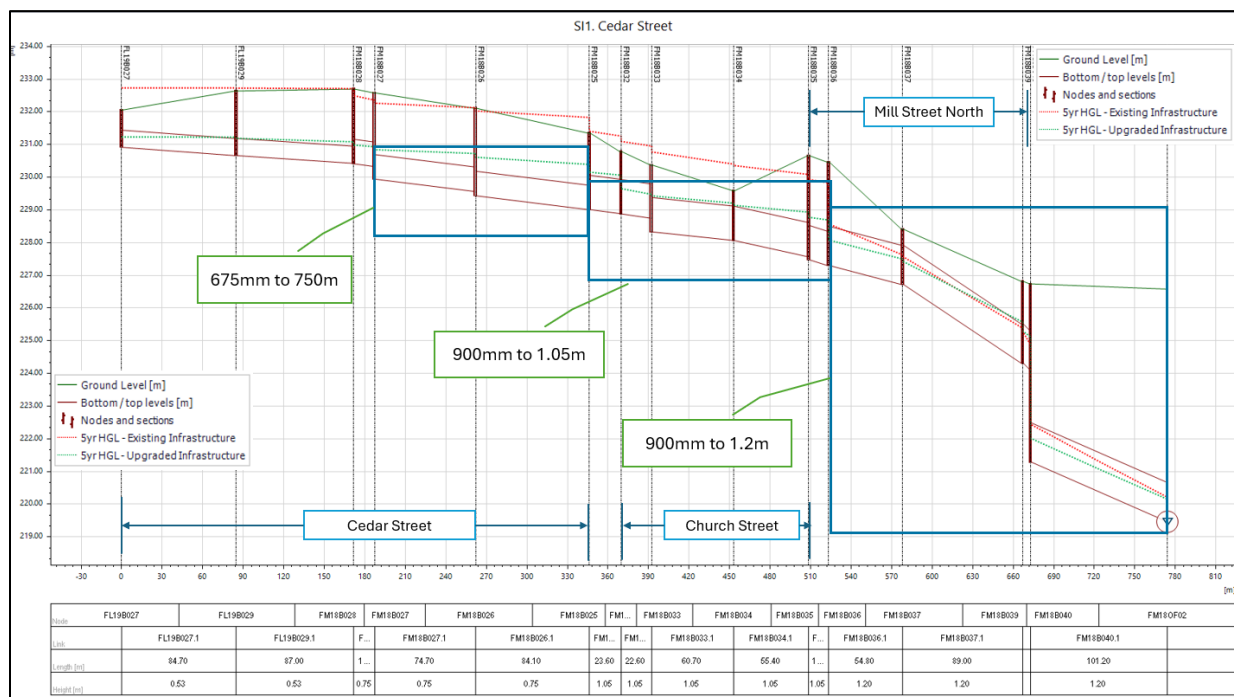
6.2.4.1 Cedar Street and Mill Street (SI1) - Waterdown

The project site is located along the section of storm sewer that runs north along Cedar Street from Hamilton Street North to Main Street North, and then north on Church Street and east on Mill Street North where it discharges into Grindstone Creek (see **Drawing 12d** and **Figure 6.2.16**). The southern section of storm sewer from Hamilton Street North to Fern Avenue is located within the Waterdown Node area. The modelling results indicate that the pipes are surcharged along the entire section of storm sewer and significant surface flooding is occurring near the intersection of Cedar Street and Main Street North, and at the intersection of Cedar Street and Hamilton Street North. Since the slope of the maximum HGL is generally in line with the slope of the pipes, it does not appear to be a pipe capacity issue but, rather, it's an issue with the large head loss being calculated at a few manhole locations where the flow direction changes sharply and there is inflow from another pipe. However, in spite of the fact that pipe capacity does not appear to be the main problem, a modelling exercise was conducted to evaluate alternatives for addressing the surcharging and flooding conditions, but the recommendations should be considered together with a closer evaluation of the head loss calculations in the model.

The modelling primarily focussed on upsizing the pipes along the majority of the site upstream of the outfall. The recommended pipe upgrades and the resulting reduction in the maximum HGL are shown in **Figure 6.2.16** while the details of the recommended

pipe upgrades are provided in **Appendix H**. The recommended upgrades eliminate surface flooding at this site and significantly reduce the maximum HGL.

Figure 6.2.16: Profile for Cedar Street and Mill Street (SI1)

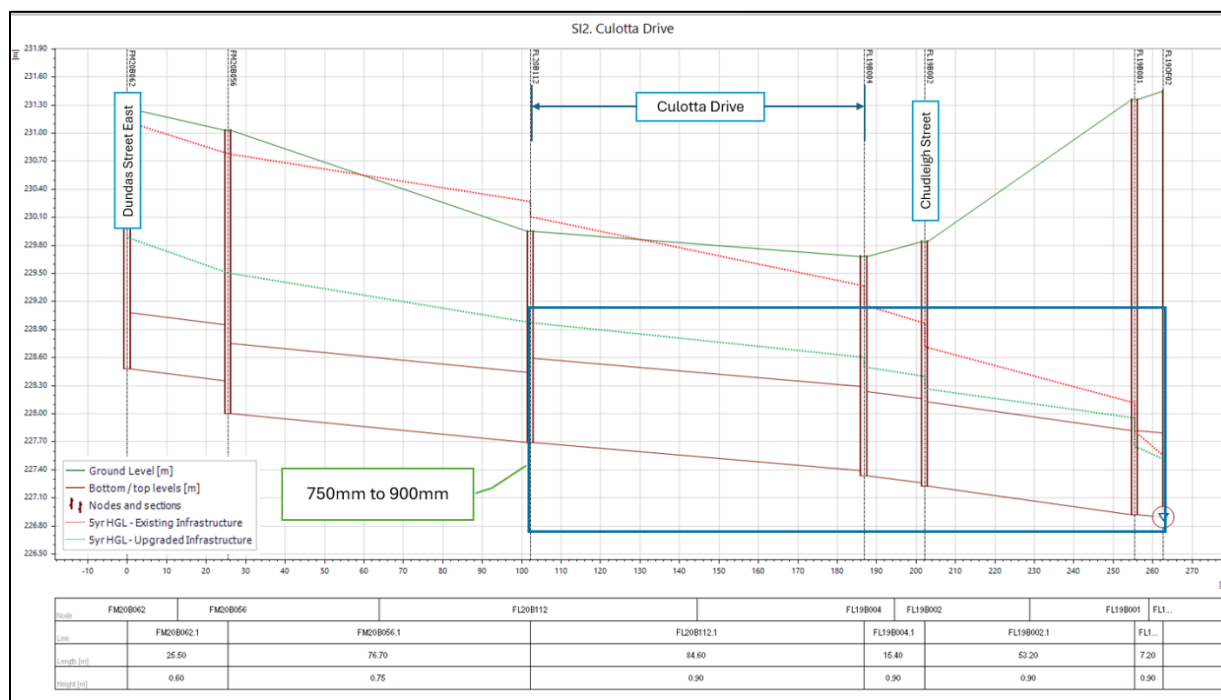


6.2.4.2 Culotta Drive (SI2) - Waterdown

The project site is located along the section of storm sewer that runs north along Culotta Drive from Dundas Street East to Chudleigh Street where it discharges to Bowers Creek (see **Drawing 12d** and **Figure 6.2.17**). The south end of this section falls inside the Waterdown Node area. The modelling results show the pipes along the entire section are surcharged and some surface flooding is occurring at the southeast corner of Culotta Drive. The steep maximum HGL indicates that pipe capacity is the main problem along this section of storm sewer. Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the surcharging and flooding conditions.

The modelling primarily focussed on upsizing the pipes upstream of the outfall to eliminate surface flooding and reduce the maximum HGL. The recommended pipe upgrades and the resulting reduction in the maximum HGL are shown in **Figure 6.2.17** while the details of the recommended pipe upgrades are provided in **Appendix H**. The recommended upgrades eliminate surface flooding at this site and significantly reduce the maximum HGL.

Figure 6.2.17: Profile for Culotta Drive (SI2)



6.2.4.3 Parkside Drive (SI3) - Waterdown

The project site is located along the section of storm sewer that runs west along Parkside Drive from Main Street North to the outfall at Borers Creek (see **Drawing 12d** and **Figure 6.2.18**). A short portion of this section of storm sewer runs adjacent to the north end of the Waterdown Node boundary. The modelling results show that this entire section of pipes is surcharged and surface flooding is occurring at the locations adjacent to the Waterdown Node boundary. The steep maximum HGL from Keewaydin Street to Braeheid Avenue indicate there is a capacity limitation along this section of pipe. Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the surcharging and flooding conditions.

The modelling primarily focussed on upsizing the pipes upstream of the outfall to eliminate the surface flooding. The recommended pipe upgrades and the resulting reduction in the maximum HGL are shown in **Figure 6.2.18** while the details of the recommended pipe upgrades are provided in **Appendix H**.

Based on further discussions with City staff it understood that Parkside Drive is undergoing reconstruction in 2026. As such, it is assumed that this work will address the identified storm sewer deficiencies and that this project does not need to be included in the capital projects list. It is shown as greyed out on the summary drawings accordingly and is not included in the capital costing.

S13, Parkside Drive

Ground Level [m]
Bottom / top levels [m]
Nodes and sections
Syr HGL - Existing Infrastructure
Syr HGL - Upgraded Infrastructure

Hamilton Street North
Parkside Drive
Keewaydin Street
Braeheid Avenue

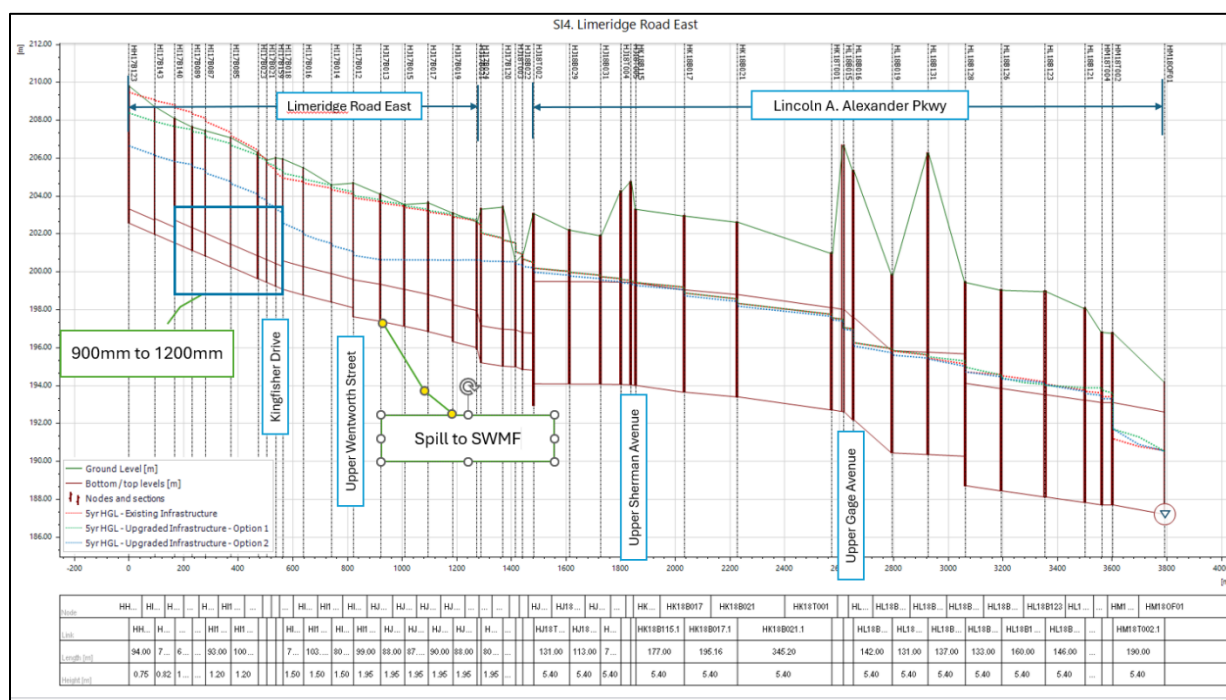
0.97m to 1.2m
1.05m to 1.5m
1.2m to 1.5m
1.35m to 1.5m

Node	FL18B085	FL18B087	FL18B089	FL18B091	FL18B093	FL18B095	FL18B097	FL18B099	FL18B101	FL18B103	FL18B105	FL18B107	FL18B109	FL18B111	FL18B113	FL18B115	FL18B117	FL18B119	FL18B121	FL18B123	FL18B125	FL18B127	FL18B129	FL18B131	FL18B133	FL18B135	FL18B137	FL18B139	FL18B141	FL18B143	FL18B145	FL18B147	FL18B149	FL18B151	FL18B153	FL18B155	FL18B157	FL18B159	FL18B161	FL18B163	FL18B165	FL18B167	FL18B169	FL18B171	FL18B173	FL18B175	FL18B177	FL18B179	FL18B181	FL18B183	FL18B185	FL18B187	FL18B189	FL18B191	FL18B193	FL18B195	FL18B197	FL18B199	FL18B201	FL18B203	FL18B205	FL18B207	FL18B209	FL18B211	FL18B213	FL18B215	FL18B217	FL18B219	FL18B221	FL18B223	FL18B225	FL18B227	FL18B229	FL18B231	FL18B233	FL18B235	FL18B237	FL18B239	FL18B241	FL18B243	FL18B245	FL18B247	FL18B249	FL18B251	FL18B253	FL18B255	FL18B257	FL18B259	FL18B261	FL18B263	FL18B265	FL18B267	FL18B269	FL18B271	FL18B273	FL18B275	FL18B277	FL18B279	FL18B281	FL18B283	FL18B285	FL18B287	FL18B289	FL18B291	FL18B293	FL18B295	FL18B297	FL18B299	FL18B301	FL18B303	FL18B305	FL18B307	FL18B309	FL18B311	FL18B313	FL18B315	FL18B317	FL18B319	FL18B321	FL18B323	FL18B325	FL18B327	FL18B329	FL18B331	FL18B333	FL18B335	FL18B337	FL18B339	FL18B341	FL18B343	FL18B345	FL18B347	FL18B349	FL18B351	FL18B353	FL18B355	FL18B357	FL18B359	FL18B361	FL18B363	FL18B365	FL18B367	FL18B369	FL18B371	FL18B373	FL18B375	FL18B377	FL18B379	FL18B381	FL18B383	FL18B385	FL18B387	FL18B389	FL18B391	FL18B393	FL18B395	FL18B397	FL18B399	FL18B401	FL18B403	FL18B405	FL18B407	FL18B409	FL18B411	FL18B413	FL18B415	FL18B417	FL18B419	FL18B421	FL18B423	FL18B425	FL18B427	FL18B429	FL18B431	FL18B433	FL18B435	FL18B437	FL18B439	FL18B441	FL18B443	FL18B445	FL18B447	FL18B449	FL18B451	FL18B453	FL18B455	FL18B457	FL18B459	FL18B461	FL18B463	FL18B465	FL18B467	FL18B469	FL18B471	FL18B473	FL18B475	FL18B477	FL18B479	FL18B481	FL18B483	FL18B485	FL18B487	FL18B489	FL18B491	FL18B493	FL18B495	FL18B497	FL18B499	FL18B501	FL18B503	FL18B505	FL18B507	FL18B509	FL18B511	FL18B513	FL18B515	FL18B517	FL18B519	FL18B521	FL18B523	FL18B525	FL18B527	FL18B529	FL18B531	FL18B533	FL18B535	FL18B537	FL18B539	FL18B541	FL18B543	FL18B545	FL18B547	FL18B549	FL18B551	FL18B553	FL18B555	FL18B557	FL18B559	FL18B561	FL18B563	FL18B565	FL18B567	FL18B569	FL18B571	FL18B573	FL18B575	FL18B577	FL18B579	FL18B581	FL18B583	FL18B585	FL18B587	FL18B589	FL18B591	FL18B593	FL18B595	FL18B597	FL18B599	FL18B601	FL18B603	FL18B605	FL18B607	FL18B609	FL18B611	FL18B613	FL18B615	FL18B617	FL18B619	FL18B621	FL18B623	FL18B625	FL18B627	FL18B629	FL18B631	FL18B633	FL18B635	FL18B637	FL18B639	FL18B641	FL18B643	FL18B645	FL18B647	FL18B649	FL18B651	FL18B653	FL18B655	FL18B657	FL18B659	FL18B661	FL18B663	FL18B665	FL18B667	FL18B669	FL18B671	
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The project site is located along a section of storm sewer that runs east along Limeridge Road East from Upper Wellington Street to just east of Limeridge Mall where it turns south and then runs east along Lincoln Alexander Parkway where it eventually discharges into the headwaters of Redhill Creek (see **Drawing 12f** and **Figure 6.2.19**). The section of storm sewer on Limeridge Road cuts right through the south end of the Limeridge Node area. The modelling results show that this section of pipes surcharges along the entire length with surface flooding occurring at various locations along Limeridge Road. Although the slope of the maximum HGL is steeper than the pipes in some locations, there are several locations with large head losses at manholes that appear to be influencing the majority of the pipe surcharging along Lincoln Alexander Parkway. However, regardless of the head loss calculations, it's clear that the section of pipe along Limeridge Road is going to be surcharged because the pipes are much smaller and they are connected at the invert of the much larger downstream pipes along the Lincoln Alexander Parkway. Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the surface flooding conditions.

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Figure 6.2.19: Profile for Limeridge Road East (SI4)



Although the storm sewer pipes along Lincoln Alexander Parkway are showing surcharging at some locations, no upsizing of these pipes was considered because they are already greater than 2 m diameter (which as noted in Section 6.2, have been excluded from the current analysis). Furthermore, upsizing the pipes along Limeridge Road alone would not significantly alleviate the surface flooding problems because the water level at the connection with the Lincoln Alexander Pkwy large diameter sewer is already quite high.

Based on this information, it was determined that the most effective way to improve the drainage performance would be to add a stormwater management facility (SWMF) for quantity control purposes in the area immediately east of the ramp connecting Upper Wentworth Street north to Lincoln Alexander Parkway west. There is already an existing SWMF (City ID#85) in a similar location south of Lincoln Alexander Parkway and west of the ramp connecting Upper Wentworth Street south to Lincoln Alexander Parkway east (see **Figure 6.2.20**). That SWMF is a dry pond with 2 separate cells on either side of the Lincoln Alexander Parkway.

Figure 6.2.20: Location of Recommended Upper Wentworth Ramp SWMF

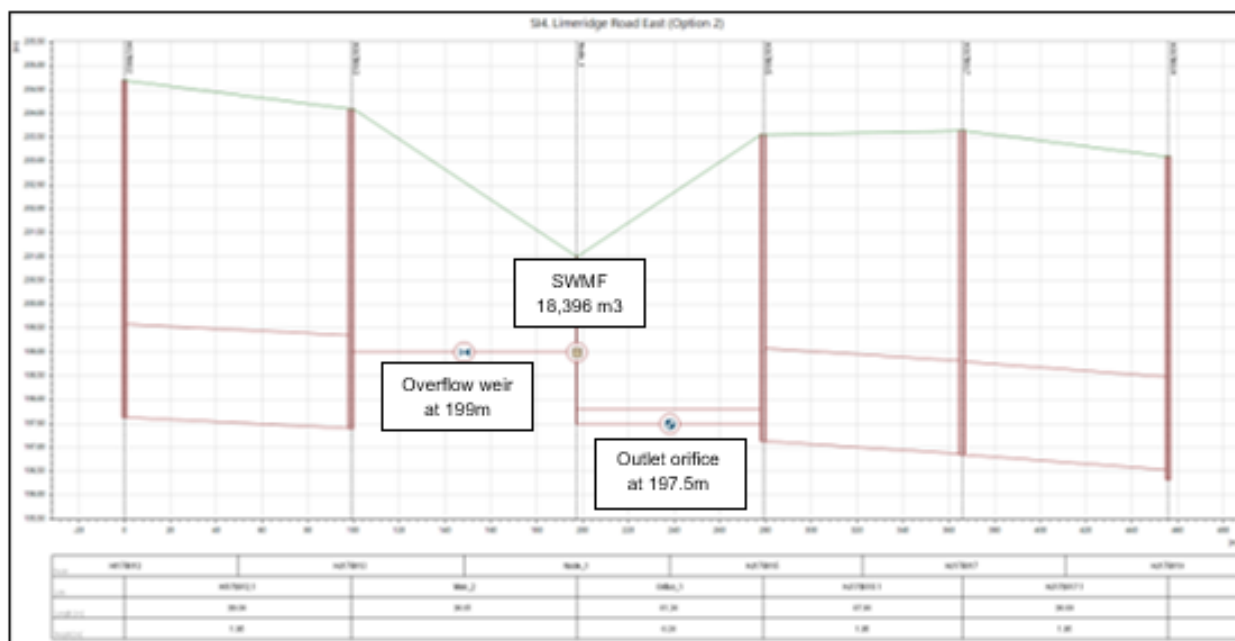


The preliminary recommended SWMF has assumed a similar design as the existing SWMF with a bottom elevation of 197.5 m, a depth of 3.5 m, and a volume of 18,376 m³. The existing storm sewer would drain into the pond when the water level at manhole HJ17B013 exceeds 199 m and the pond would drain back into the storm sewer via a 300 mm diameter orifice connected to manhole HJ17B015. This design is shown the profile plot in **Figure 6.2.21**.

Although the recommended SWMF does not resolve the pipe surcharge conditions, it significantly reduces the maximum HGL along Limeridge Road and it eliminates surface flooding.

As noted, a scoped storm sewer upgrade along Limeridge Road East has also been proposed to further address simulated HGL levels. A detailed summary of the recommended pipe upgrades is provided in **Appendix H**.

Figure 6.2.21: Profile of the Upper Wentworth Ramp SWMF

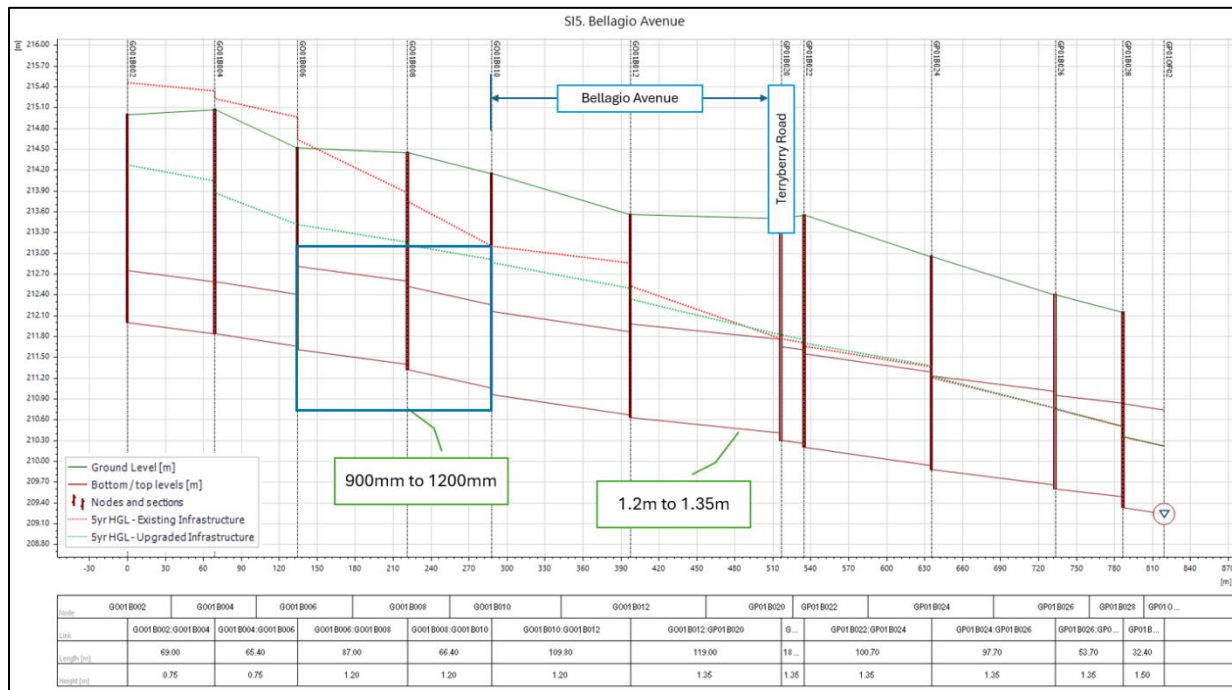


6.2.4.5 Bellagio Avenue (SI5) – Mountain East

The site is located along a section of storm sewer that runs east along Bellagio Avenue past Canary Grass Boulevard where it discharges into a headwater tributary of Sinkhole Creek (see **Drawing 12g** and **Figure 6.2.22**). The entire section runs immediately adjacent to or inside the West Elfrida Node area. The modelling results show the pipes are surcharged west of Terryberry Road with some surface flooding at the west end of the section. The steep maximum HGL in a few of the pipes indicate that the surcharging is caused by pipe capacity limitations but there are also a few locations where the manhole head losses are very large and should be investigated. Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the surcharging and flooding conditions.

The modelling primarily focussed on upsizing the pipes upstream of Terryberry Road to reduce the maximum HGL. The recommended pipe upgrades and the resulting reduction in the maximum HGL are shown in **Figure 6.2.22** while the details of the recommended pipe upgrades are provided in **Appendix H**. The recommended upgrades eliminate surface flooding at this site and significantly reduce the maximum HGL.

Figure 6.2.22: Profile of Bellagio Avenue (SI5)



6.2.4.6 Rymal Road East (SI6) – Mountain East

The site is located along a section of storm sewer that runs west along Rymal Road East from Fletcher Road to Glover Road where it discharges into a tributary of Hannon Creek (see **Drawing 12g** and **Figure 6.2.23**). The entire section of storm sewer is located in the Corridor area. The modelling results show that the storm sewer pipes east of Trinity Church Road are surcharged with surface flooding occurring along the majority of the section. The steep maximum HGL in the two pipes immediately east of Trinity Church Road indicate that these pipes are presenting capacity constraints and need to be upsized. Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the surcharging and flooding conditions.

The modelling primarily focussed on upsizing the pipes upstream of Trinity Church Road to reduce the maximum HGL. The recommended pipe upgrades and the resulting reduction in the maximum HGL are shown in **Figure 6.2.23** while the details of the recommended pipe upgrades are provided in **Appendix H**. The recommended upgrades eliminate surface flooding at this site and significantly reduce the maximum HGL.

SIG. Rymal Road East

Rymal Road East

Fletcher Road

825mm to 900mm

Dakota Blvd

825mm to 1200mm

Trinity Church Road

Legend:

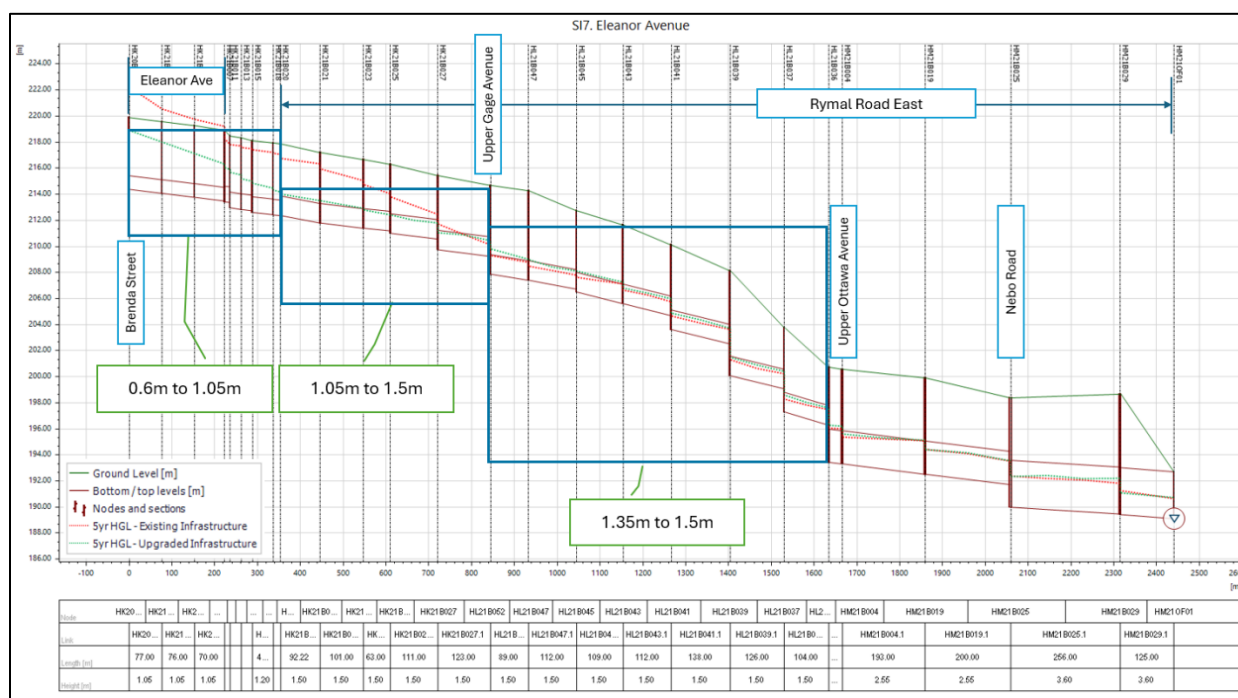
- Ground Level [m]
- Bottom/top levels [m]
- Nodes and sections
- Syr HGL - Existing Infrastructure
- Syr HGL - Upgraded Infrastructure

Node	SB19B007	SA19B001	SA19B003	SA19B005	SA19B007	SA19B009	SA19B011	SA19B013	SO22B003	SO22B005	SO22B007	SO22B009	HO22B002	HO22B004	HO22B006	HO22B008	HO22B010	HO22B012	HO22B014	HO22B016	HO22B018	HO22B020	HO22B022	HO22B024	HO22B026	HO22B028	HO22B030	HO22B032	HO22B034	HO22B036	HO22B038	HO22B040	HO22B042	HO22B044	HO22B046	HO22B048	HO22B050	HO22B052	HO22B054	HO22B056	HO22B058	HO22B060	HO22B062	HO22B064	HO22B066	HO22B068	HO22B070	HO22B072	HO22B074	HO22B076	HO22B078	HO22B080	HO22B082	HO22B084	HO22B086	HO22B088	HO22B090	HO22B092	HO22B094	HO22B096	HO22B098	HO22B100	HO22B102	HO22B104	HO22B106	HO22B108	HO22B110	HO22B112	HO22B114	HO22B116	HO22B118	HO22B120	HO22B122	HO22B124	HO22B126	HO22B128	HO22B130	HO22B132	HO22B134	HO22B136	HO22B138	HO22B140	HO22B142	HO22B144	HO22B146	HO22B148	HO22B150	HO22B152	HO22B154	HO22B156	HO22B158	HO22B160	HO22B162	HO22B164	HO22B166	HO22B168	HO22B170	HO22B172	HO22B174	HO22B176	HO22B178	HO22B180	HO22B182	HO22B184	HO22B186	HO22B188	HO22B190	HO22B192	HO22B194	HO22B196	HO22B198	HO22B200	HO22B202	HO22B204	HO22B206	HO22B208	HO22B210	HO22B212	HO22B214	HO22B216	HO22B218	HO22B220	HO22B222	HO22B224	HO22B226	HO22B228	HO22B230	HO22B232	HO22B234	HO22B236	HO22B238	HO22B240	HO22B242	HO22B244	HO22B246	HO22B248	HO22B250	HO22B252	HO22B254	HO22B256	HO22B258	HO22B260	HO22B262	HO22B264	HO22B266	HO22B268	HO22B270	HO22B272	HO22B274	HO22B276	HO22B278	HO22B280	HO22B282	HO22B284	HO22B286	HO22B288	HO22B290	HO22B292	HO22B294	HO22B296	HO22B298	HO22B300	HO22B302	HO22B304	HO22B306	HO22B308	HO22B310	HO22B312	HO22B314	HO22B316	HO22B318	HO22B320	HO22B322	HO22B324	HO22B326	HO22B328	HO22B330	HO22B332	HO22B334	HO22B336	HO22B338	HO22B340	HO22B342	HO22B344	HO22B346	HO22B348	HO22B350	HO22B352	HO22B354	HO22B356	HO22B358	HO22B360	HO22B362	HO22B364	HO22B366	HO22B368	HO22B370	HO22B372	HO22B374	HO22B376	HO22B378	HO22B380	HO22B382	HO22B384	HO22B386	HO22B388	HO22B390	HO22B392	HO22B394	HO22B396	HO22B398	HO22B400	HO22B402	HO22B404	HO22B406	HO22B408	HO22B410	HO22B412	HO22B414	HO22B416	HO22B418	HO22B420	HO22B422	HO22B424	HO22B426	HO22B428	HO22B430	HO22B432	HO22B434	HO22B436	HO22B438	HO22B440	HO22B442	HO22B444	HO22B446	HO22B448	HO22B450	HO22B452	HO22B454	HO22B456	HO22B458	HO22B460	HO22B462	HO22B464	HO22B466	HO22B468	HO22B470	HO22B472	HO22B474	HO22B476	HO22B478	HO22B480	HO22B482	HO22B484	HO22B486	HO22B488	HO22B490	HO22B492	HO22B494	HO22B496	HO22B498	HO22B500	HO22B502	HO22B504	HO22B506	HO22B508	HO22B510	HO22B512	HO22B514	HO22B516	HO22B518	HO22B520	HO22B522	HO22B524	HO22B526	HO22B528	HO22B530	HO22B532	HO22B534	HO22B536	HO22B538	HO22B540	HO22B542	HO22B544	HO22B546	HO22B548	HO22B550	HO22B552	HO22B554	HO22B556
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The site is located along a section of storm sewer that runs south along Eleanor Avenue from Brenda Street to Rymal Road and then east along Rymal Road to just past Nebo Road where it discharges into a tributary of Hannon Creek (see **Drawing 12f** and **Figure 6.2.24**). The majority of this section of storm sewer is located in the Corridor area. The modelling result show that the section of pipes west of Upper Gage Avenue, along Rymal Road and Eleanor Avenue are all surcharge with surface flooding occurring along Eleanor Avenue. Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the surcharging and flooding conditions.

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Figure 6.2.24: Profile of Rymal Road and Eleanor Avenue (SI7)



6.2.4.8 Hester Street and South Bend Road (SI8) – Mountain West

The site is located along the section of storm sewer that runs west along Milwood Drive, south along Cardinal Drive, west along Bobolink Road, west along Hester Street, north along Hayden Street, west on Mohawk Road East, north on Upper James Street, west on Richwill Road, north on West 5th Street, west on South Bend Road West, west on Bandamere Avenue, and finally north on West 23rd Street where it discharges in to Chedoke Creek (see **Drawing 12e** and **Figure 6.2.25**). Only a short portion of this section of storm sewer is located in the Corridor area. The modelling results indicate the entire section of storm sewer pipes are surcharged and there is significant surface flooding from Milwood Drive to Hayden Street. Although there are some manhole locations with large head losses, the majority of pipes would still be surcharged, regardless of the head loss calculations, due to capacity constraints in the pipes.

However, since the majority of the capacity constrained pipes have a diameter larger than 2 m (which as noted in Section 6.2, have been excluded from the current analysis), **no infrastructure upgrades are being considered.**

That being said, it is recommended to collect flow and water level monitoring data in this location to update the model calibration and head loss calculations and verify that the model is providing a realistic representation of flooding in this location.

— Ground Level [m]
 — Bottom / top levels [m]
 Nodes and sections
 --- S-yr HGL - Existing Infrastructure

Node	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80	81	82	83	84	85	86	87	88	89	90	91	92	93	94	95	96	97	98	99	100	101	102	103	104	105	106	107	108	109	110	111	112	113	114	115	116	117	118	119	120	121	122	123	124	125	126	127	128	129	130	131	132	133	134	135	136	137	138	139	140	141	142	143	144	145	146	147	148	149	150	151	152	153	154	155	156	157	158	159	160	161	162	163	164	165	166	167	168	169	170	171	172	173	174	175	176	177	178	179	180	181	182	183	184	185	186	187	188	189	190	191	192	193	194	195	196	197	198	199	200	201	202	203	204	205	206	207	208	209	210	211	212	213	214	215	216	217	218	219	220	221	222	223	224	225	226	227	228	229	230	231	232	233	234	235	236	237	238	239	240	241	242	243	244	245	246	247	248	249	250	251	252	253	254	255	256	257	258	259	260	261	262	263	264	265	266	267	268	269	270	271	272	273	274	275	276	277	278	279	280	281	282	283	284	285	286	287	288	289	290	291	292	293	294	295	296	297	298	299	300	301	302	303	304	305	306	307	308	309	310	311	312	313	314	315	316	317	318	319	320	321	322	323	324	325	326	327	328	329	330	331	332	333	334	335	336	337	338	339	340	341	342	343	344	345	346	347	348	349	350	351	352	353	354	355	356	357	358	359	360	361	362	363	364	365	366	367	368	369	370	371	372	373	374	375	376	377	378	379	380	381	382	383	384	385	386	387	388	389	390	391	392	393	394	395	396	397	398	399	400	401	402	403	404	405	406	407	408	409	410	411	412	413	414	415	416	417	418	419	420	421	422	423	424	425	426	427	428	429	430	431	432	433	434	435	436	437	438	439	440	441	442
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This project site is located along the section of storm sewer that runs north along Greencedar Drive, crosses the Lincoln Alexander Parkway and continues north on Daisy Street, then east and north on Forestgate Drive, east on Appleford Road, north on Wendover Drive, crosses Mohawk Rd, and north on Rice Avenue where it discharges into a tributary of Chedoke Creek (see **Drawing 12e** and **Figure 6.2.26**). The entire section of storm sewer is located in a flooding hotspot, but only the section north of Mohawk Road is located in the Corridor area. The modelling results indicate the pipes along the entire section are surcharged with many locations where surface flooding is occurring. Although there are some manhole locations with large head losses, it is clear that the majority of pipes would still be surcharged, regardless of the head loss calculations, due to capacity constraints in the pipes.

An initial upgrade analysis was included in previous versions of the reporting (refer to **Drawing 12e**). Based on subsequent discussions with City staff, it was noted that the upgrade recommendations of the previously completed “Fessenden Neighbourhood Stormwater Management Schedule B Class Environmental Assessment” (McCormick Rankin Corporation, November 2011) were actually implemented by City staff, and that the MIKE Urban modelling did not represent the final as-constructed details, as key details, including online storage chambers, were not included in GIS layers provided to the WSP team. It was considered likely that the upgrades and measures completed as part of the Fessenden Class EA (an excerpt has been included in **Appendix H** for

reference) should be effective in addressing the previously modelled deficiencies. As such, further hydrologic\hydraulic modelling or upgrade analysis is not considered warranted for area SI9.

6.2.4.10 Templemead Drive and Hempstead Drive (SI10) – Mountain Centre

This project site is located along a section of storm sewer that runs east along Presidio Drive and continues east along Royalvista Drive, the north, east and south along Templemead Drive, east along Unsworth Drive, then north and east on Hempstead Drive to Dartnell Road, north on Dartnell Road and east on Stone Church Road East where it discharges into Hannon Creek (see **Drawing 12f** and **Figure 6.2.26**). The entire section of storm sewer is located outside of any Nodes or Corridors, but a small portion of the upstream end is located within a flooding hotspot. The modelling results indicate the entire section of storm sewer is surcharged with surface flooding occurring along Royalvista Drive. Although there are a few locations with large head losses at manholes, the steep maximum HGL throughout almost the entire section indicates that the problem is mainly a pipe capacity issue. Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the surcharging and flooding conditions.

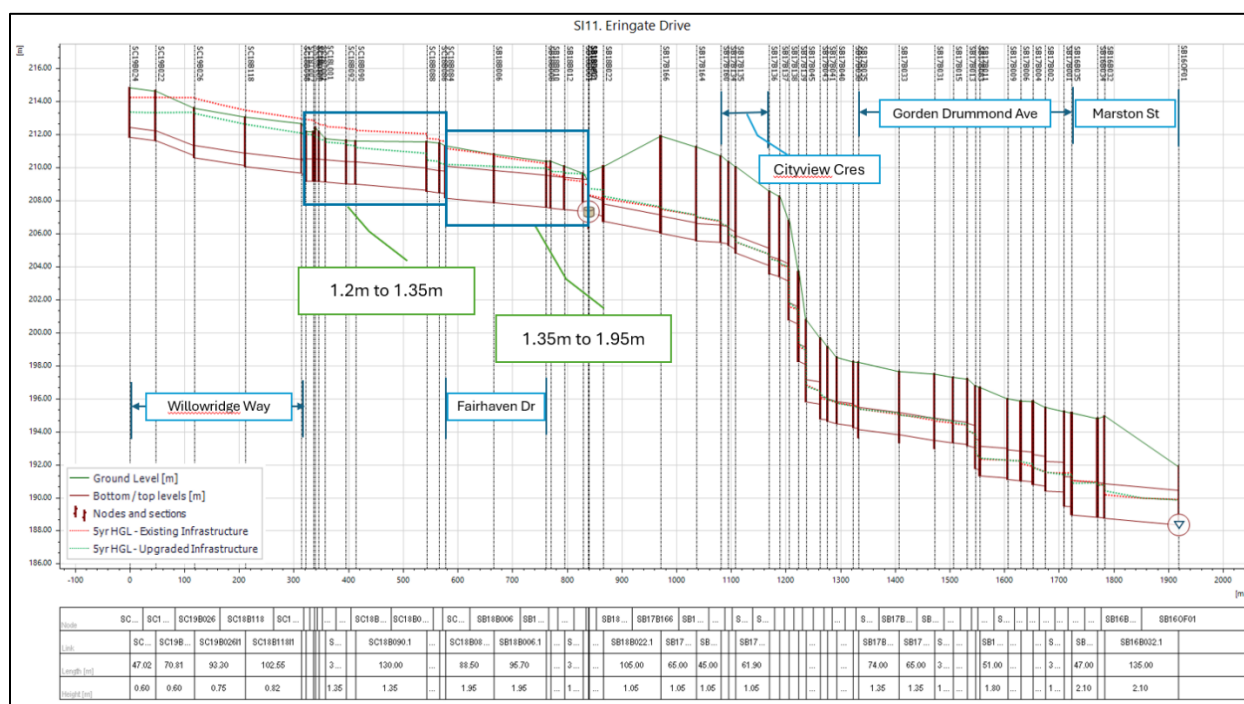
The modelling focussed on upsizing the pipes along the entire length of the project site to reduce the maximum HGL. The recommended pipe upgrades and the resulting reduction in the maximum HGL are shown in **Figure 6.2.26** while the details of the recommended pipe upgrades are provided in **Appendix H**. The recommended upgrades eliminate surface flooding at this site and significantly reduce the maximum HGL.

Figure 1: S110, Royalvasta Drive. This is a detailed cross-section profile of the S110 road project. The vertical axis represents elevation in meters, ranging from 1780 to 2240. The horizontal axis represents stationing, ranging from -200 to 3200. The profile shows the ground level (green line), existing infrastructure (red line), and upgraded infrastructure (blue line). Key features include: a large rectangular area labeled '0.6m to 1.05m' and '0.6/0.675/0.75m to 1.35m'; a section labeled '0.9/1.05/1.2/1.35m to 1.8m'; a section labeled '1.5m to 2m'; and a section labeled '1.8m to 2.4m'. Road names shown include: Darnall Rd, Stone Church Rd, Presidio Dr, Royalvasta Dr, Templemead Dr, Silverton Dr, Unsworth Dr, and Hempstead Drive. A legend indicates: Ground Level [m], Bottom/top levels [m], Nodes and sections, Svr HGL - Existing Infrastructure, and Svr HGL - Upgraded Infrastructure.

The site is located along a section of storm sewer that runs north on Willowridge Way, west on Eringate Drive, north on Second Road West, west on Fairhaven Drive and then north where it discharges into a SWMF (see **Drawing 12g** and **Figure 6.2.27**). The entire section of storm sewer is located outside of any Node or Corridor, but the section south of the SWMF is located entirely inside of a flooding hotspot. The modelling results indicate the section of pipes upstream of the SWMF are all surcharged with surface flooding indicated at most locations. The maximum HGL is steeper than the majority of the pipes indicating that the surcharge is caused by pipe capacity limitations. Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the surcharging and flooding conditions.

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Figure 6.2.27: Profile of Eringate Drive and Fairhaven Drive (SI11)



6.2.4.12 First Road West and Isaac Brock Drive (SI12) – Mountain East

The site is located along the section of storm sewer that runs north along First Road West up to Maplewood Green Park, then west through the park and north along Gatestone Drive, then west and north on Isaac Brock Drive where it crosses Mud Street West and discharges into Felker's Creek (See **Drawing 12g** and **Figure 6.2.28**). The entire section of storm sewer is located outside of any Node or Corridor but the section on First Road West is located entirely within a flooding hotspot. The modelling results show the majority of this section of storm sewer is surcharge with surface flooding occurring at many locations along First Road West. Although there are some manhole locations with large head losses, the maximum HGL is steeper than the pipes throughout most of the section of storm sewer, indicating that the problem is primarily related to capacity constraints. Given these results, a modelling exercise was conducted to evaluate alternatives for addressing the surcharging and flooding conditions.

The modelling focussed on upsizing the pipes along the entire length of the project site. The recommended pipe upgrades and the resulting reduction in the maximum HGL are shown in **Figure 6.2.28** while the details of the recommended pipe upgrades are provided in **Appendix H**. The recommended upgrades eliminate surface flooding at this site and significantly reduce the maximum HGL.

[illegible]

The graphical summary of recommended storm sewer upgrades is presented in Drawings 12a to 12g, attached. A tabular summary of recommended upgrades by City of Hamilton community area, is presented in **Table 6.2.1**. Upgrades have been presented \ divided on the basis of those associated with expected future re-development \ infill and intensification areas (nodes & corridors) and those which are more associated with identified hotspot\reported surface flooding.

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Table 6.2.1: Summary of all Assessed Storm Sewer Upgrades

Community Area	Length (m)	Number of Upgraded Pipe Segments	Range of Recommended Pipe Size Upgrades	Average Standard Pipe Size Upgrade
Ancaster (Node & Corridors)	927	14	600 to 900 mm	2
Dundas (Node & Corridors)	461	8	300 to 450 mm	1
Hamilton Mountain (Node & Corridors)	5,096	82	750 to 2400 mm	3
Stoney Creek (Node & Corridors)	2,782	47	300 to 1800 mm	2
Waterdown (Node & Corridors)	1,585	28	675 to 1500 mm	2
TOTAL (Nodes & Corridors)	10,851	179	300 to 2400 mm	2
Hamilton Mountain (Flooding Hotspots)	5,711	92	900 to 2400 mm	4
Stoney Creek (Flooding Hotspots)	1,977	24	300 to 1350 mm	3
TOTAL (Flooding Hotspots)	7,688	116	300 to 2400 mm	3
TOTAL (All)	18,539	295	300 to 2400 mm	3

The preceding results indicate more limited upgrades in Ancaster and Dundas, and relatively less in Waterdown. The recommended upgrades for these areas are all generally focused on identified deficiencies in the vicinities of planned infill/intensification areas.

By contract, a much higher number of recommended storm sewer upgrades have been identified for the Hamilton Mountain and Stoney Creek areas. These areas include not only recommended upgrades in the vicinity of infill / intensification areas but also upgrades to address City identified flooding hotspot areas.

The assessed upgrades are generally stand-alone sections as per the attached Drawings (12a to 12g), however in some cases the works could be coupled into a single construction project \ tender. In addition, the proposed works should be considered in conjunction with other required roadway and infrastructure (sanitary and water) upgrades as this would determine the overall priority. Preliminary construction costing has been presented in Section 7 along with a consideration of overall prioritization.

In addition to the proposed storm sewer upgrades, a preliminary recommendation has been made for a new SWM facility at the Upper Wentworth Street and Lincoln Alexander Parkway. This SWM facility was determined to be a preferred solution as compared to further storm sewer upgrades, given the size of trunk sewers in this area. Further assessment and preliminary design are required to review feasibility in this location including considerations of bedrock and traffic impacts, among other factors (groundwater, surficial soils, utilities, etcetera).

As noted previously, some concerns have been noted with the lack of robust model calibration for all of the community models. The City should also consider additional \ updated flow monitoring and data collection to support updated model development, including more refined modelling for areas where community-specific models have not yet been developed (i.e. the balance of the “Six Cities” modelling).

6.3 SWM Facilities

It is understood that existing stormwater management (SWM) facilities (ponds) will potentially receive increased runoff volume from infill / intensification areas (even though in many cases peak flows will reduce). While it is expected that any re-development areas that are subject to Site Plan Control (SPC) will require on-site quantity controls, these controls are typically based on peak flow controls only, not volume controls. Notwithstanding, the requirements of Hamilton’s recent Green Standards and Guidelines (GSG) do require a hierarchical application of controls that would consider volume control requirements, which would address a minimum retention target of between 2.5 and 10 mm, as well as the overall 90th percentile provincial guideline of 29 mm.

An initial assessment of SWM facilities which may potentially be impacted by infill / intensification has been undertaken by reviewing those SWM facilities which would receive drainage from these areas (i.e. nodes and corridors as per Drawing 6). SWM facilities which would be expected to potentially be impacted have been highlighted on **Drawings 12a to 12g**, attached. A summary is presented in **Table 6.3.1**, including the numbers of SWMFs and associated City ID numbers (refer to **Drawings 12a to 12g** for specific locations).

Table 6.3.1: Summary of SWMFs Potentially Impacted by Infill/Intensification

Community	Total SWMF	SWMF Not in Modelling	SWMF Included as Outfalls	SWMF Modelled as Storage Units
Ancaster	6	3 (2, 19, 156)	3 (70, 71, 157)	0
Dundas	0	0	0	0
Mountain	8	1 (146)	4 (99, 131, 147, 150)	3 (65, 67, 218)
Stoney Creek	5	1 (223)	1 (112)	3 (68, 80, 82)
Waterdown	1	0	1 (44)	0
Total	20	5	9	6

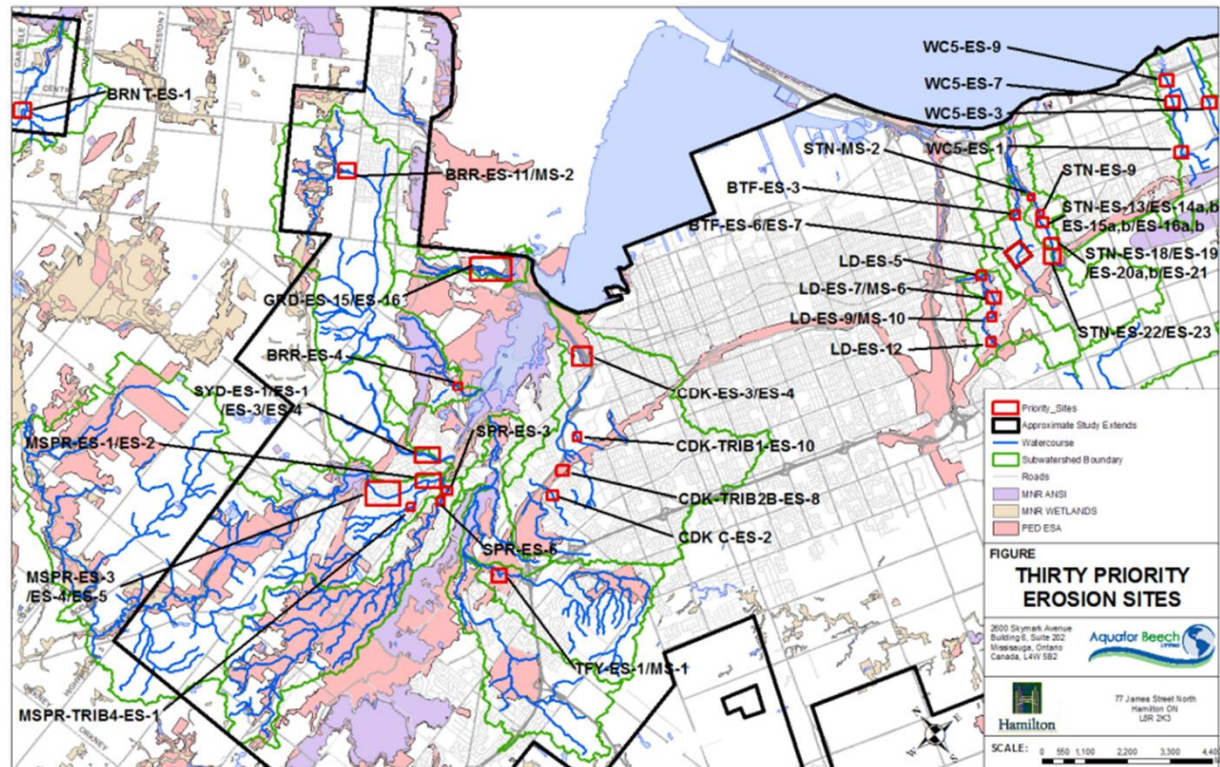
As evident from **Table 6.3.1**, 20 SWMFs would potentially be impacted by increased runoff volume from infill / intensification. However, only 6 of the 20 SWMFs are currently included in the available hydrologic \ hydraulic modelling (the balance is either for non-trunk sewer areas or modelled as outfalls rather than storage units). Given the preceding, as well as the uncertainty regarding land use changes under future re-development conditions, the actual potential impact to SWMF performance cannot currently be quantified. However, scoped assessments of these features should be considered in the future as the specifics of infill/intensification become clearer.

6.4 Watercourses

Similar to the high-level review of the potential impact of infill/intensification to SWM facilities, the potential impact of increased runoff volume to watercourse receivers should also be considered.

A City-Wide Watercourse Erosion Assessment Study was undertaken in 2011. This was then reviewed \ updated by the Hamilton Conservation Authority based on additional field review in 2016. These erosion sites (included updated recommendations from the 2016 review) have been assessed against the identified re-development areas (nodes and corridors as per **Drawing 6**), to assess areas of potential impact. An excerpted map from the original study is presented in **Figure 6.4.1**.

Figure 6.4.1: Excerpted map of Priority Erosion Sites (Aquafor Beech, 2011)



From a review of overlay of infill / intensification areas (nodes and corridors) with erosion sites, the following locations indicate overlap (location IDs reference the Hamilton Conservation Authority's 2016 Assessment):

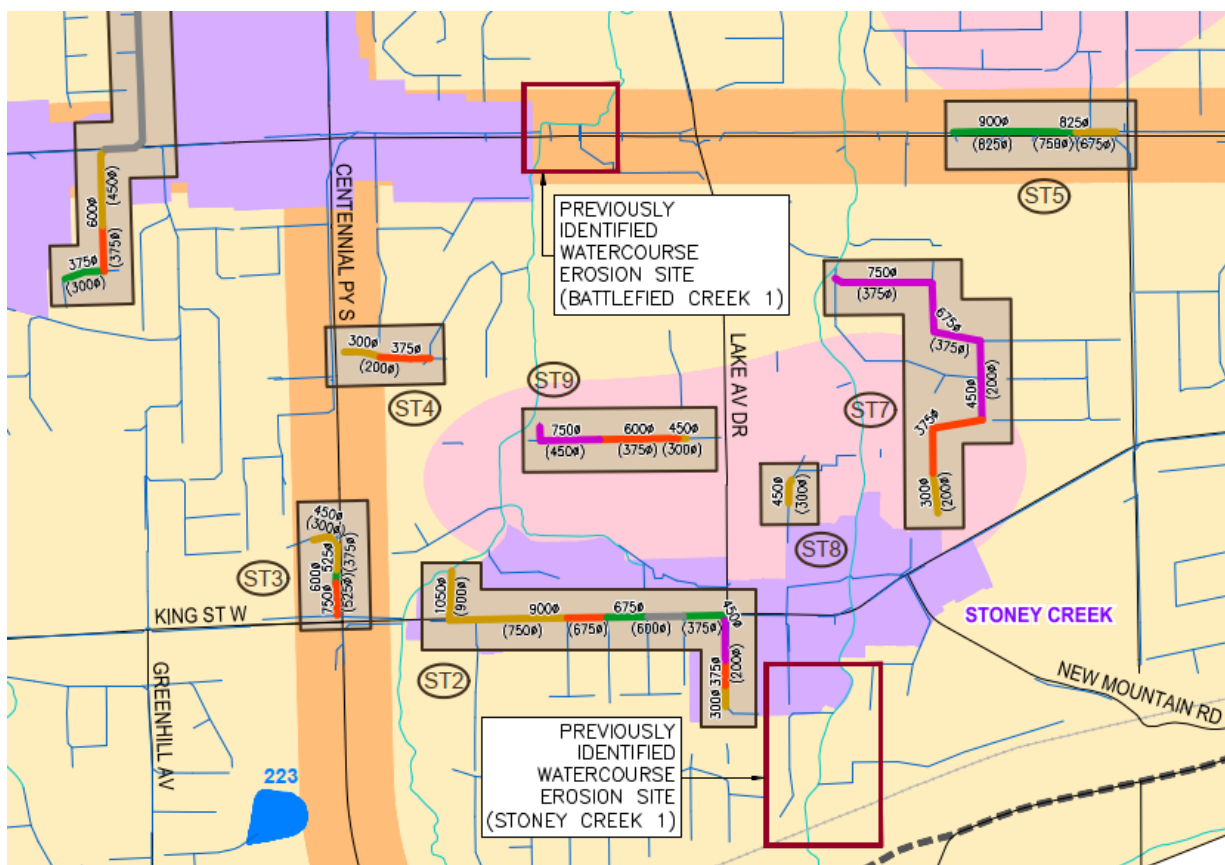
- Western Watercourses (4 total)
 - a) Mid-Spencer 2
 - b) Tiffany 1 (not in HCA list)
 - c) Chedoke 2
 - d) Chedoke Tributary 10 (not in HCA list)
- Eastern Watercourses (6 total)
 - a) **Battlefield Creek 1**
 - b) **Battlefield Creek 2**
 - c) **Stoney Creek 1**
 - d) **Stoney Creek 2**
 - e) *Watercourse 5 – 1 (not in HCA list)*
 - f) *Watercourse 5 – 3 (not in HCA list)*

The majority of the identified sites are towards the downstream limits of the watercourse and thus would be more affected by upstream flow contributions. Overall, the impact of local scale changes (intensification) would likely be comparatively minor for the majority of the identified watercourse locations accordingly.

Two sites appear more directly connected to, or adjacent to planned intensification areas and should be considered for remediation sooner. These locations are bolded in the above list (Battlefield Creek 1 and Stoney Creek 2). These locations have also been identified on **Drawing 12c**, an excerpt is presented in **Figure 6.4.2**.

These two locations would also be suggested as priority locations for rehabilitation works, given their connection to expected infill / intensification works.

Figure 6.4.2: Previously Identified Watercourse Erosion Sites



7 CAPITAL PROGRAM DEVELOPMENT

7.1 Storm Sewers

7.1.1 Storm Sewer Prioritization

The storm sewer upgrades assessed herein were discussed in previous sections, including Section 6.2. Storm sewer upgrade projects were developed based on a combination of hydrologic \ hydraulic modelling results, overlap with areas of identified infill\intensification potential (i.e. nodes and corridors) and overlap with City-reported surface flooding (i.e. hotspots).

A qualitative prioritization has been undertaken in consideration of the following general factors:

- Proportion of the project within a re-development area (i.e. a node or corridor)
- Proportion of the project within an identified surface flooding area (i.e. hotspot)
- Magnitude of the recommended storm sewer upgrade (pipe sizes and extents)

A detailed summary table has been included in **Appendix H** (refer to **Table H1**). Results are presented in **Table 7.1.1** below for projects identified as having medium or high priority (low priority projects are not included).

Table 7.1.1: Prioritization of Storm Sewer Projects

Project ID	Community	Area	Priority
ST5	Stoney Creek	Queenston Road (Durham Road to RHC)	High
A2	Ancaster	Wilson Street East (St Margaret to Dalley)	Medium
A3	Ancaster	Wilson and Academy (Stoneham Park to Rousseaux)	Medium
SI6	Mountain	Rymal Road East (Fletcher Road to Glover Road)	Medium
SI7	Mountain	Eleanor Ave (Brenda to Rymal) and Rymal Road East (to Nebo)	Medium
ST2	Stoney Creek	Lake Ave and King St W	Medium
ST3	Stoney Creek	Orlanda Road (Grand Ave to Centennial Pkwy) and Centennial and King St W	Medium

Of the 23 storm sewer projects assessed and previously discussed, only seven of the projects are considered to be either high (1) or medium (6) priority. The remainder are either low priority or not currently recommended.

WSP has undertaken a review of the City's 10-year capital works planning information. Overall, only one of the preceding projects appears on that list. Project SI7 (Rymal Road East, from Miles Road to Upper Ottawa Street) would coincide with City projects 11347 and 11348, which are separate road reviews for Rymal Road (Upper Sherman to Upper Gage, and Upper Gage to Upper Ottawa). It is not clear from the available information if these are studies or resurfacing projects or full depth \ infrastructure replacement projects.

Consideration should also be given to overlap with recommendations stemming from the water and wastewater master plans (being completed separately). Higher prioritization would be given to projects where water and sanitary infrastructure upgrades have also been recommended.

7.1.2 Storm Sewer Costing

Estimated costing for the proposed storm sewer works has employed a consistent methodology with that applied for the wastewater interim master servicing study (GEI). Storm sewers have all been assumed to be installed using open cut, however different costing has been applied for storm sewer projects where the average depth is 5 m or less, versus those where the depth is 5 to 7 m. The supplied unit costs consider excavation, bedding and backfill, pipe supply and installation, structures, connections and testing and restoration work to generate a unit cost.

An additional 25% surcharge is suggested to account for general construction items (9%), construction contingency (10%) and provisional and allowance (6%).

Soft costs related to field investigations and design costs, as well as contract administration and management, have not been included.

Detailed costs are presented in **Table H2 (Appendix H)** for all projects. Individual storm sewer segment costs have been rounded up to the nearest \$10,000, final costing has been rounded up to the nearest \$100,000. Resulting summary costs for identified high and medium priority storm sewer projects are presented in **Table 7.1.2**.

Table 7.1.2: Estimated Priority Storm Sewer Project Construction Costs

Project ID	Community	Area	Priority	Estimated Construction Cost
ST5	Stoney Creek	Queenston Road (Durham Road to RHC)	High	\$1,800,000
Total for High Priority Projects				\$1,800,000
A2	Ancaster	Wilson Street East (St Margaret to Dalley)	Medium	\$1,800,000
A3	Ancaster	Wilson and Academy (Stoneham Park to Rousseaux)	Medium	\$2,500,000
SI6	Mountain	Rymal Road East (Fletcher Road to Glover Road)	Medium	\$3,900,000
SI7	Mountain	Eleanor Ave (Brenda to Rymal) and Rymal Road East (to Nebo)	Medium	\$13,600,000
ST2	Stoney Creek	Lake Ave and King St W	Medium	\$3,700,000
ST3	Stoney Creek	Orlanda Road (Grand Ave to Centennial Pkwy) and Centennial and King St W	Medium	\$800,000
Total for Medium Priority Projects				\$26,300,000
Total for High + Medium Priority Projects				\$28,100,000

As per the results in **Table 7.1.2**, a total of approximately \$2M has been estimated for the 2 high priority projects, and a further \$26M for the additional six medium priority projects (\$28M total). Projects SI9 and SI7 constitute the majority of this cost given their size and extent.

The preceding does not include soft costs related to field investigations, design and construction management, as noted previously.

As per the results in **Table H3 (Appendix H)**, there is a further approximate \$102M construction cost associated with the remaining 16 low priority storm sewer upgrade projects.

7.2 Other Projects and Recommendations

A number of other miscellaneous project recommendations have been generated through the previous review and assessment. These are summarized below, along with estimated capital costs:

- **Creek Rehabilitation Projects**

- a) Battlefield Creek 1 Erosion Site Rehabilitation (Queenston Road)
 - i) Reach BTF 2, Erosion Site ES 3 (Battlefield Creek – Downstream of Queenston Road) in the 2011 assessment - estimated cost of \$280,000 (2011 dollars)
 - ii) Apply a 35% cost escalation, current cost estimate of \$380,000
 - iii) Does not include soft costs (design, field investigations, etcetera)
- b) Stoney Creek 2 Erosion Site Rehabilitation (Upstream of King Street)
 - i) Reach ST 8, Erosion Sites ES 22-23 (Stoney Creek – Upstream of King Street East) in the 2011 Assessment – estimated cost of \$200,000 (2011 dollars)
 - ii) Apply a 35% cost escalation, current cost estimate of \$270,000
 - iii) Does not include soft costs (design, field investigations, etcetera)
- **Studies and Assessments**
 - a) Preliminary Design Assessment for Upper Wentworth \ Linc SWMF
 - i) Review feasibility of the preliminary recommendation for a SWMF at the Upper Wentworth \ Lincoln Alexander Parkway interchange
 - ii) Field investigations and completion of a preliminary (30%) design
 - iii) Estimated cost of \$150,000
 - b) Storm Sewer Flow Monitoring Program
 - i) Suggest at least 2 flow monitors for each of the model community areas, and at least 6 months of data collection
 - ii) Include local rainfall data collection if not available from other source
 - iii) Estimated cost of \$150,000
 - c) Development of More Resolute Hydrologic \ Hydraulic Modelling from the “Six Cities” model
 - i) Split into individual models for the communities of Waterdown, Binbrook and the balance of Hamilton Mountain
 - ii) Incorporate major system (overland flow) into the modelling similar to the Ancaster model
 - iii) Estimated cost of \$150,000
 - d) Overall Model Update and Calibration\Validation Study
 - i) Incorporate major system (overland flow) into the community models which do not currently include this (i.e. Dundas and Stoney Creek)
 - ii) Using the storm sewer flow monitoring data above, update model calibration\validation for all of the community models
 - iii) Estimated cost of \$250,000

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- Dillon Consulting. (2014). Modelling Study Summary Report. London: Dillon Consulting.

APPENDIX

A

Policy and Guidelines
Recommendations (May 2020)

APPENDIX

B

Six Cities Results Maps

APPENDIX

C

Dundas Results Maps

APPENDIX

D

Stoney Creek Results Maps

APPENDIX

E

Ancaster Results Maps

APPENDIX

F

SWM Pond Summary Table

APPENDIX

G

Overland Flow Analysis
Summaries Recommended

APPENDIX

H

Upgrades and Capital Cost
Estimates