



CITY OF ST. ALBERT

# MASTER PLAN UPDATE

## WASTEWATER COLLECTION AND STORMWATER MANAGEMENT SYSTEM UTILITIES

### FINAL REPORT

March 2022





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FINAL REPORT

PROJECT NO.: 20M-01312-00  
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# EXECUTIVE SUMMARY

## GENERAL

WSP was retained by the City of St. Albert to update its Wastewater and Stormwater Utilities Master Plan (UMP). The objective of the UMP is to provide a comprehensive plan that will guide the City in prioritizing wastewater and stormwater infrastructure upgrades for the next ten years in a strategic and cost-effective manner, to support future growth and development. The update and calibration of the wastewater and stormwater hydraulic models were carried out as a critical working tool to establish the improvement projects and servicing strategies identified in this report. WSP undertook technical assessments of the hydraulic infrastructure to determine the capacity of the existing wastewater and stormwater networks and recommend improvements. A 10-year capital plan was prepared to provide prioritized infrastructure projects to support the anticipated growth from the City's municipal development plans.

The wastewater and stormwater system models were updated as a part of the 2020 UMP Update. These models were used as tools to:

- Assess the hydraulic performance and current operational settings for the City's existing wastewater and stormwater networks. Thus, determining the necessary short-term capital improvements required in the system.
- Assess the existing systems' capacity to service the City's future projected flows as envisioned by the City's planning department through population growth and expansion of the City's wastewater and stormwater service areas. Thus, determining the upgrade works necessary to serve the projected growth.

This final report describes the existing system improvements and recommends future system servicing strategies for the stormwater and wastewater infrastructure. Improvement projects and servicing strategies will be subject to further refinement at the respective preliminary and detailed design phases for each.

## WASTEWATER SYSTEM

Overall, the existing wastewater conveyance system is performing well and is providing an acceptable level of service under the examined conditions with some exemptions of a limited extent.

The wastewater model was updated and reviewed based on all existing information relating to the wastewater system, such as studies, reports, drawings, operational data, etc., received from the City. The PCSWMM model was calibrated using the provided flow monitoring data and the model. The hydraulic modelling parameters, including the wet weather RTK analysis, were verified. The system performance was evaluated under the 1:5-year 4-hour and 1:25-year 24-hour design storm conditions. All simulation results are presented in [Appendix A](#). In addition, the future system was modelled and assessed based on the ultimate development prospects as predicted by the City's Planning and Development Department and inputs from the development industry.

Wastewater system capacity issues under the existing and the future condition scenarios were identified based on the design criteria established for the review of the system's peak flow conditions. A total of thirteen upgrades are recommended to overcome the most persistent problems of the existing wastewater collection system, reduce potential risks of surface flooding and improve the overall system conveyance. [Figure A-8](#) illustrates the locations of all proposed projects. An implementation plan was prepared to establish high-level cost estimates and prioritize system improvement projects required within a 10-year horizon. Additional investigations will be required on a project-level basis in order to further determine the priority of proposed projects and finalize cost estimates for each project.



## STORMWATER SYSTEM

In general, the existing stormwater sewer system is providing an acceptable level of service under the examined conditions with some local issues, as noted in the respective section of this report.

The stormwater model was updated and reviewed based on the provided information, including previous studies, reports, drawings, operational data, etc., by the City. Design and assessment criteria for the review of the stormwater system performance were established with the City. The MIKE URBAN model was updated, rectified, calibrated and migrated to the new MIKE+ software package, results are attached in [Appendix D](#). The storm system performance was assessed under the 1.5-year 4-hour, 1:100-year 4-hour and 100-year 24-hour design storm events. All simulation results are presented in [Appendix B](#). Moreover, a stormwater management concept was planned for the future system, as shown in [Figure B-21](#), based on the ultimate development prospects for the City of St. Albert. A new modelling scenario was developed for the future system to assess the system performance under the same rainfall events. In addition, a stormwater capacity assessment was conducted for the proposed development of St. Albert Trail, results of this analysis are presented in [Appendix C](#).

The stormwater system surface ponding and capacity issues were identified for the existing conditions and future build-out scenarios based on the assessment criteria established with the City. A total of six improvement projects are recommended to increase the conveyance of the existing stormwater system, reduce the potential flooding risks, and boost system capacity. Maps that illustrate the location of all recommended projects are shown in [Appendix B](#). A stormwater implementation plan was also prepared in which high-level cost estimates and prioritization of recommended system projects required over the next ten years. Additional investigation will be needed on a project-level basis in order to further refine the implementation priority and respective cost estimates for each project.



## ABBREVIATIONS

<b>ADD</b>	Average Day Water Demand
<b>ADWF</b>	Average Dry Weather Flow
<b>ASP</b>	Area Structure Plans
<b>DWF</b>	Dry Weather Flow
<b>EP</b>	Equivalent Population
<b>EP/ha</b>	Equivalent Population per Hectare
<b>GDA</b>	Gross Developable Area
<b>GIS</b>	Geographic Information Systems
<b>HGL</b>	Hydraulic Grade Line
<b>HWL</b>	High Water Level
<b>HSE</b>	Health, Safety & Environment
<b>ICI</b>	Institutional, Commercial and Industrial
<b>I&amp;I</b>	Inflow and Infiltration
<b>MDP</b>	Municipal Development Plan
<b>NIT</b>	North Interceptor Trunk
<b>NWL</b>	Normal Water Level
<b>p/ha</b>	People per Hectare
<b>PDWF</b>	Peak Dry Weather Flow
<b>PF</b>	Peaking Factor
<b>PWWF</b>	Peak Wet Weather Flow
<b>RG</b>	Rain Gauge
<b>RTC</b>	Real-time Controls
<b>SAPS</b>	St. Albert Pump Station
<b>SCADA</b>	Supervisory Control and Data Acquisition
<b>SIT</b>	South Interceptor Trunk
<b>SWMF</b>	Stormwater Management Facility
<b>UMP</b>	Utility Master Plan
<b>WWF</b>	Wet Weather Flow

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# 1 INTRODUCTION

The City of St. Albert (City) retained WSP Canada Inc. (WSP) to update its Wastewater and Stormwater Utilities Master Plan (UMP). The objective of the UMP is to provide a comprehensive plan that will guide the City in prioritizing wastewater and stormwater infrastructure upgrades for the next ten years in a strategic and cost-effective manner, to support future growth and development. The update and calibration of wastewater and stormwater hydraulic models were carried out as a critical working tool to establish the improvement projects and servicing strategies identified in this report.

Technical assessments of the hydraulic infrastructure were undertaken by WSP to determine the capacity of the existing wastewater and stormwater networks and recommend improvements. A 10-year capital plan has been prepared to provide prioritized infrastructure projects to support the anticipated growth from the City's municipal development plans (MDP).

The hydraulic models updated as a part of the 2020 UMP Update are used as tools for analyses to:

- Assess the hydraulic performance and current operational settings for the City's existing wastewater and stormwater networks. Thus, determining the necessary short-term capital improvements required in the system.
- Assess the existing systems' capacity to service the City's future projected flows as envisioned by the City's planning department through population growth and expansion of the City's wastewater and stormwater service areas. Thus, determining the upgrade works necessary to serve the projected growth.

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## 1.1 SCOPE OF WORK

The following is the scope of work undertaken by WSP for the 2020 UMP Update:

### **WASTEWATER**

- Gather and review all existing information relating to the wastewater system, such as studies, reports, drawings, operational data, etc. from the City.
- Meet with City staff to obtain and compile all relevant operational data.
- Calibrate the wastewater model based on provided flow monitoring data.
- Update the hydraulic model and verify modelling parameters which includes wet weather RTK analysis.
- Develop modelling scenarios under 1:5-year 4-hour and 1:25-year 24-hour design storm conditions.
- Establish design criteria for the review of the system's peak flow conditions.
- Identify system capacity issues under existing and future build-out scenarios, according to the design criteria established, and propose recommendations for system improvements.
- Prepare cost estimates and prioritize system improvement projects required within a 10-year year horizon.

### **STORMWATER**

- Gather and review all existing information related to the stormwater system, including previous studies, reports, drawings, operational data, etc. from the City.
- Meet with City staff to obtain and compile all relevant operational data.
- Validate the stormwater model based on provided rainfall and flow monitoring data.
- Upgrade the stormwater modelling software to the new MIKE+ package.

- Update the stormwater model and verify modelling parameters.
  - Develop modelling scenarios under 1:5-year 4-hour, 1:100-year 4-hour and 100-year 24-hour design storm conditions.
  - Establish design criteria for the review of the stormwater system performance.
  - Identify system capacity issues under existing and future build-out scenarios, according to the design criteria established, and propose recommendations for system improvements.
  - Prepare cost estimates and prioritize system improvement projects required.
- 

## 1.2 DATA COLLECTION AND REVIEW

The following key documents were referenced in the development of the UMP. These studies should be read in conjunction with the current report, as the recommendations provided in these studies form part of the overall 2020 UMP Update.

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### 1.2.1 WASTEWATER UTILITY

#### **2013 UTILITY MASTER PLAN UPDATE (STANTEC, 2014)**

The 2013 UMP describes the existing and future system improvements and servicing strategies for the water, wastewater and stormwater utilities. Development plans and build-out staging were based on predictions by the City's Planning and Development department and input from the local development industry.

The wastewater system capacity analysis identified potential risks of basement flooding during a 1:25-year 24-hour storm under the existing (2013) system. Several capital projects were identified to improve the performance of the wastewater system, some of which have been implemented by the City since the study. The recommended future servicing strategy included new gravity sewers within development areas, with four new lift stations and associated force mains. Recommendations for the North Interceptor Trunk Sewer Phase 2 Extension, and Phase 3 were outlined in the memo. Assumptions and recommendations from the 2013 UMP were considered in the development of the 2020 UMP update.

Key recommendations from the 2013 UMP include:

- Phase 3 of the NIT is recommended, as there is limited capacity in the existing system to service growth areas.
- Phase 2A and 2B of the NIT will be needed to service the Employment Lands growth areas.
- A new lift station in the northeast area of the City is recommended to relieve the capacity requirements of the Oakmont Lift Station.

#### **CITY OF ST. ALBERT MUNICIPAL ENGINEERING STANDARDS (2015)**

The Municipal Standards are referenced in this 2020 UMP Update in the determination of the servicing level requirements for the City's wastewater system. This includes the estimation of wastewater generation rates, I&I rates, and servicing assessment criteria for the pipes and lift station facilities.



## **ST. ALBERT SANITARY UTILITY MODEL TECHNICAL MOMORANDUM (ASSOCIATED ENGINEERING, 2017)**

The technical memorandum included updating, reviewing and validating the sanitary utility model with the 2016 data. This study was referred to for insights into the provided PCSWMM model assumptions and background settings.

## **PHASE 3 NORTH INTERCEPTOR SANITARY TRUNK PRELIMINARY DESIGN REPORT (ASSOCIATED ENGINEERING, 2018)**

Based on preliminary recommendations from the 2013 UMP, a preliminary design study was conducted in 2018 to assess the required capacity of, and review alignment options for, Phase 3 of the NIT. Assumptions for future servicing strategies, such as future I&I reductions, flow control locations, etc., from the preliminary design report were considered in the development of the 2020 UMP update.

Key takeaways from the 2018 NIT Phase 3 Report include:

- Recommended alignment for Phase 3 of the NIT.
- The future system was modelled using a 0.28 L/s/ha I&I rate, based on the premise that the City will be implementing I&I mitigation and reduction strategies.

## **INFLOW AND INFILTRATION ASSESSMENT PROGRAM (STANTEC, 2019)**

In order to manage the capacity requirements of the City's trunk sewers, and SAPS capacity during wet weather conditions, the City initiated an I&I Assessment Program. The initiative aims to assess the amount of I&I generated by the City and recommend strategies to reduce I&I to 0.28 L/s/ha, as committed to the ACRWC. As part of the program, areas with I&I above the ACRWC's targets were identified through flow monitoring and hydraulic simulations of 1:5-year 4-hour and 1:25-year 24-hour events. I&I calibration values and I&I mitigation strategies and projects recommended in the 2019 I&I report were considered in the development of the 2020 UMP update.

Key takeaways from the 2019 I&I Study were incorporated into the 2020 UMP Update, which include:

- The future system was modelled using 0.202 L/s/ha I&I for the 1:25-year 24-hour storm and 0.222 L/s/ha I&I for the 1:5-year 4-hour storm, based on the premise that newly built areas should exhibit lower I&I rates. WSP notes that this is less than the ACRWC threshold, and therefore could contribute to the dilution of the calculated City-wide I&I rates. For the 2020 UMP Update, a conservative rate of 0.28 L/s/ha has been applied for the purpose of hydraulic modelling.
- Recommendations for I&I reduction initiatives (e.g. source control, replacement of ageing infrastructure etc.) are summarized in the 2020 UMP Update.
- The study suggested that increased HGLs due to orifice controls should not pose a risk of basement properties flooding. However, the City may choose to consider overflow weirs to mitigate this risk. This has been considered in the 2020 UMP Update.
- The study suggested that the orifices in the trunk sewers should be upgraded so that they can be real-time controlled. While this would increase the complexity of the available storage in the trunk system, it would allow the City to control the conveyance of dry weather flows and activate the orifices when storage of wet weather flows is needed. The main drawback of this approach is the additional maintenance and operation efforts that would be required. Further details on the cost and operation of the real-time orifice controls are provided in the 2019 I&I Study.
- Reducing the opening of existing control orifice plates, as well as the addition of new orifice control plates, to optimize wet weather storage in the trunk sewers. It is noted that reducing the orifice opening size



is likely to increase the risk of solids setting. These control plates will need to be modified again in the future when the trunk sewers accept more flows from future developments. For the purpose of the 2020 UMP update, it is assumed that the orifice plates are static, and are set as per the provided PCSWMM model.

#### **CITY OF ST. ALBERT WASTEWATER FACILITIES STUDY (STANTEC, 2019)**

The 2019 Facilities Study was a multi-disciplinary assessment completed for the City's wastewater and stormwater lift stations. The assessment included field evaluations, condition ratings, and recommendations for station improvement works.

Key takeaways from this study were reviewed for inclusion in this 2020 UMP update include:

- On average, the condition and performance ratings of the lift station facilities were found to correlate with the age of the station, where new stations generally scored higher.
- For several lift stations, the measured pump capacity did not match the recorded design point for the lift station. Lift stations in the provided PCSWMM model were updated to reflect the recently measured discharge flow rates from the 2019 Facilities Study. This was done to provide a more accurate simulation of the actual system performance.
- A 1:25-year capital plan budget for repairs and replacements of the wastewater facilities is presented (in 2018 dollars).

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### **1.2.2 STORMWATER UTILITY**

#### **STORM SEWER IMPROVEMENTS FLOODED AREAS (FOCUS, 2009)**

This report investigated the existing storm sewer collection system capacity to increase the level of service for problematic neighbourhoods within the City of St. Albert. The study examined ten specific areas where flooding had occurred in the preceding years. It also highlighted:

- The role of adopting polydrain systems as a solution for the flooding problem, especially in mitigating construction.
- The flooding problems in the low area on Hebert Road east of St. Albert Trail. It attributed the problem to the lack of a major drainage system. As a solution, regrading of the parking area at Sturgeon Plaza was recommended to create an onsite pond.

#### **2013 UTILITY MASTER PLAN UPDATE (STANTEC, 2014)**

The existing stormwater system capacity analysis identified performance deficiencies in terms of capacity, flooding and available storage under the 1:5-year 4-hour and 1:100-year 24-hour storm events. Several capital projects were identified to improve the performance of the wastewater system, some of which have been implemented by the City, since the study. Assumptions and recommendations from the 2013 UMP were considered in the development of the 2020 UMP update. The development plans and build-out staging for the future system scenario were based on predictions by the City's Planning and Development department and input from the local development industry.

Key results and recommendations from this study were reviewed for inclusion in this 2020 UMP update, which covered:

- Construction of a new SWMF in Campbell Business Park
- Upgrade the storm sewer in the downtown area along Perron Street.
- Existing system deficiencies in the Deer Ridge area.
- The future servicing strategy included two options:



- Option 1: with stormwater trunk piped parallel to Carrot Creek to Big Lake.
- Option 2: with multi-stage SWMF outlets to Carrot Creek.

#### **CITY OF ST. ALBERT MUNICIPAL ENGINEERING STANDARDS (2015)**

The Municipal Standards are referenced in the determination of the servicing level requirements for the City's stormwater system, including the servicing assessment criteria for the system performance.

#### **ST. ALBERT FLOOD HAZARD STUDY: STURGEON RIVER (MATRIX SOLUTIONS, 2020)**

The flood hazard study delineated the 100-year inundation boundaries for the Sturgeon River. Hydrologic analysis was conducted to provide flood water levels for Big Lake and flow frequency estimates for the Sturgeon River. A HEC-RAS model was developed and applied in creating water surface profiles and preparing floodway criteria maps. This study is referenced in the determination of the flood levels for different stormwater outlets on the Sturgeon River.

#### **WATER DISTRIBUTION SYSTEM MASTER PLAN 2020 UPDATE (ASSOCIATED ENGINEERING, 2021)**

A preliminary study was conducted in 2020 to update the City's water master plan. The performance of the existing water distribution system was assessed, and system deficiencies and constraints were identified. The primary outcomes of this study included:

- A growth plan for future development and population increase.
- A 10-year Capital Plan for the City infrastructure investment.

#### **CARROT CREEK REGIONAL MASTER PLAN (STANTEC, 2021) – IN PROGRESS**

Based on recommendations from the 2013 UMP, the Carrot Creek regional master plan is going to provide guidance on the accommodation of future development within the Carrot Creek watershed. Watershed stakeholders are participating in forming guiding policies and design criteria. A plan is set to control bank erosion and rehabilitation. This study will also incorporate sustainable development strategies for the long-term environmental protection of the watershed.

Primary recommendations will include:

- Stormwater servicing options within the watershed.
- Maximum discharge rates for future stormwater management.

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## **1.3 RELEVANT TECHNICAL MEMORANDA**

The following Technical Memoranda have been prepared by WSP over the course of this project. Key information presented in these memos has been updated and included in this final report document.

- Design Criteria Establishment (WSP, December 2020).
- Wastewater System Existing Capacity Assessment (WSP, March 2021).
- Stormwater System Existing Capacity Assessment (WSP, March 2021).



## 2 WASTEWATER SYSTEM

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### 2.1 WASTEWATER SYSTEM ASSESSMENT BASIS

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#### 2.1.1 WASTEWATER MODEL UPDATE

The wastewater model was updated to better represent the City's current wastewater system. The City provided GIS shapefiles which were used to set the initial physical location and attributes of gravity sewers updated since 2017. Record drawings for some of the areas were used to refine the elevation and invert information of the wastewater assets. Information was gathered from numerous sources, and where conflicting data was present, these were presented to City staff for confirmation.

#### ASSUMPTIONS AND LIMITATIONS

The following assumptions and limitations apply to the City's 2020 PCSWMM Model of the wastewater system. Model flows and wet weather estimation assumptions are discussed in subsequent sections of this memo.

- The PCSWMM model provided did not include forcemain pipes downstream of the lift stations (i.e. modelled pumps are connected directly into the discharging manhole instead of into a pipe). While this approach could be enough for estimating the flow capacity within downstream systems (as the lift station pumps would be discharging at peak flows), it may slightly overestimate the downstream volumes, as pipes inherently have some amount of storage and attenuation function. Additionally, the pumping head required at the related lift stations will be difficult to accurately assess without representing the friction losses within the forcemains. For example, the 2017 Technical Memorandum notes that the Gate Avenue lift station is almost ineffective and that the head-discharge curve for the pump should be confirmed. The head-discharge from the pump should be sized to overcome static head requirements (elevation difference) in addition to friction losses within the system (flows within pipe). These limitations were discussed and confirmed with City staff during the model update phase of the 2020 UMP Update.
- Conduits in the existing PCSWMM model have been assigned a roughness coefficient of 0.015, which is noted to be higher than the typical 0.013 value used for simulations. Based on the UMP, the higher value was assigned to conservatively reflect the condition/characteristics of the current system. New pipes added as part of the 2020 update have been assigned a roughness coefficient of 0.013, as it is expected that new infrastructure would have less friction losses. This is also compliant with the St. Albert Municipal Standards.
- Junctions in the provided PCSWMM model were simulated with a 10 m<sup>2</sup> Ponding Area value. This parameter allows for water to “pool” above the manhole when the pipes are full, and then drain back into the system when there is available capacity (e.g. ponding would occur within an allowable 10 m<sup>2</sup> area around the manhole during peak flows, after which the water would drain back into the system as flows subside). WSP notes this is typically applied in stormwater modelling applications, where stormwater runoff may collect at the surface of manholes during high flows before re-entering the collection system. However, ponding (surface storage) may not be appropriate for simulating wastewater flows. This attenuation should not be “counted on” as it is a condition we are striving to eliminate. WSP has instead updated the model to allow for increased surcharging at manholes, which will have a similar effect of mitigating “losing” wastewater volumes from the simulation during high flows. The intent is to more accurately represent the peak flows that would be experienced in the system

during wet weather. Recommended system improvements will be targeted to reduce surcharging in pipes that are simulated to have HGLs above allowable elevations.

- There are three (3) locations with weirs in the provided PCSWMM model. Based on discussions with City staff, the weirs located at St. Vital Avenue and Malmo Avenue have been removed. However, a review of the sizing and details of the weirs was not conducted as part of this study, but it is recommended they be confirmed in the next master plan update.
- There are five (5) orifices modelled along the NIT to control peak flows. A detailed review of orifice sizing and locations was not conducted as part of this study. Operational details of the orifice control plates (i.e. the controlled flow rates) were not available at the time of this study. It is recommended that these be confirmed in the next master plan update, as they significantly influence the operation and capacity of the City's trunk sewer system.
- The model was updated according to GIS information available, supplemented by record drawings provided by the City. In some cases (e.g. Erin Ridge North, Meadowview, and Jensen Lakes), gaps occurred between the different information sources resulting in orphaned sections within the wastewater model. It is expected that these pipes may have been constructed at the time of development but may not yet be connected to the greater wastewater network as of the last GIS database update. WSP notes that these areas represent small sections of upstream areas, where capacity issues are not expected. These orphaned sections have not been included in the model update, and their projected wastewater flows have been loaded onto the next existing downstream node.
- It is assumed that wastewater flows from Erin Ridge North and Jensen Lakes will be captured by the 1200 mm trunk main diversion at the intersection of Enchanted Way N and Executive Way N and will not flow to the Oakmont Lift Station. Instead, there is a new lift station currently being constructed east of Erin Ridge North (location currently assumed to be near Coal Mine Road) that will discharge directly to the SAPS facility via forcemain. Therefore, it is assumed that the 1200 mm trunk main is disconnected from the 450 mm gravity sewer along Everett Drive. Pump curves and operating levels for the temporary Erin Ridge North and Jensen Lake lift stations were not available.
- The Riverside siphon is assumed to be disconnected from the NIT. The four existing siphon pipes (160 mm, 200 mm, 250 mm, and 300 mm) have been modelled as a single pipe with a calculated equivalent diameter of 450 mm. The siphon collects flows from the Meadowview area and discharges into the SIT.
- Pumping capacities of the lift stations are based on the measured capacities reported in the 2019 Facilities Study. However, the measured capacity of several lift stations are noted to be less than the reported design capacity. No further facilities investigations were conducted as a part of this study. Through discussions with City staff, it was noted that most of the lift stations were designed as duplex systems, however they are not operated in duty/standby, but rather as lead/lag. WSP notes that this operation may not conform to provincial and the Municipal Standards, which outline the requirement for 100% redundancy. For the purpose of consistency between previous studies and the current UMP update, the reported measured capacities were applied to the updated model, to reflect recent conditions in the system.
- The two temporary lift stations in the Riverside area are assumed to still be in place for the existing scenario. Due to limited record drawing and GIS shapefile information, it is recommended that the network connectivity and elevations for the Riverside area be reviewed and updated for accuracy, as more details become available. Seasonal lift stations have not been included in the model. These facilities include Meadowview lift station, Kinsmen RV Park, and Riel Sports Field.
- A review of the SAPS available and required capacity is not included in the current study. However, WSP notes from the 2018 NIT Phase 3 Report, the expected capacity is 1,800 L/s for the 2020 horizon and 2,400 L/s in the 2040 horizon (assumed to be the ultimate buildout horizon).

## 2.1.2 WASTEWATER SYSTEM ASSESSMENT CRITERIA

The following subsections outline the wastewater servicing design and evaluation criteria to be applied in the development of the master plan, and assessment of the City's PCSWMM wastewater model.

### DIURNAL PATTERNS AND PEAK FLOWS

Under the 2020 assessment scenario, existing diurnal patterns established for previously calibrated areas have been maintained, as the 2019 I&I Study recommended they are a suitable representation of the flow patterns throughout the City. However, areas that have been flow-monitored in 2020 will be updated with the most recent flow patterns observed.

For future development areas, peaking factors will be calculated according to the City's Municipal Standards.

The peaking factor for residential land use is as follows:

$$Pf_{res} = 1 + \frac{14}{4 + p^{0.5}}$$

where:

Pf = Peaking factor

p = Equivalent population in 1,000s

The peaking factor for institutional, commercial, and industrial (ICI) land uses used in this study is 3.0, and the  $PDWF_{ICI}$  is calculated as follows:

$$PDWF_{ICI} = 3.0 \times ADWF_{ICI}$$

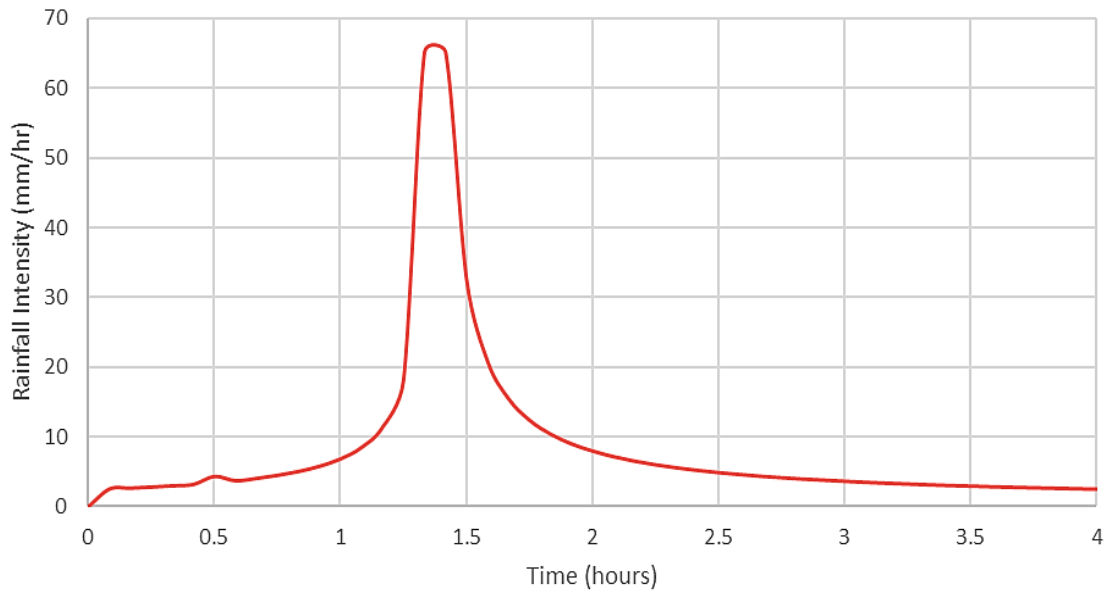
A synthetic diurnal pattern, based on the existing diurnal patterns throughout the City for similar land use types, will be applied to future development areas to simulate a varied flow throughout the day.

### WET WEATHER FLOWS – INFLOW AND INFILTRATION, DESIGN STORMS

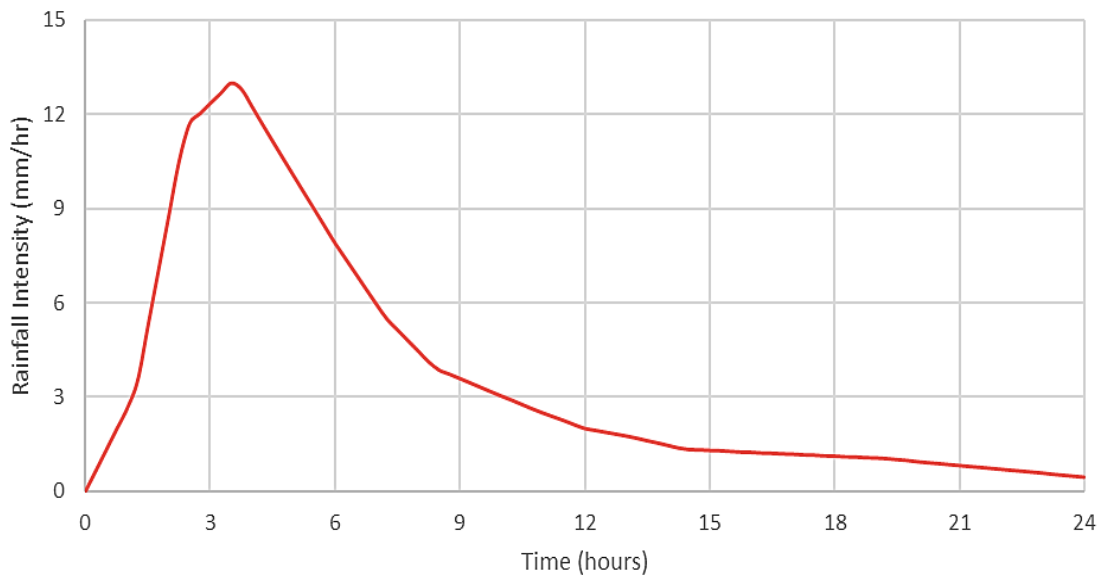
I&I for uncalibrated nodes (e.g. future development) will be according to the City's design criteria: 0.28 L/s/ha. The synthetic I&I Peaking Pattern will be applied to these nodes.

For calibrated areas, the following simulated storm events will be applied, which are based on EPCOR's design storms, in accordance with the City's design criteria (Figure 2-1 and Figure 2-2):

- 1:5-year 4-hour storm, (Modified Chicago Distribution)
- 1:25-year 24-hour storm, (Huff Distribution)



**Figure 2-1: 1:5-year 4-hour Design Storm (Modified Chicago Distribution)**



**Figure 2-2: 1:25-year 24-hour Design Storm (Huff Distribution)**

## MINIMUM AND MAXIMUM VELOCITIES

The flow velocity should not be less than 0.6 m/s (for self-cleansing purposes), and the maximum flow velocity should not be greater than 3.0 m/s (risk of turbulence and erosion).

## GRAVITY SEWER MINIMUM DIAMETERS

Minimum diameters for wastewater mains shall be at minimum 200 mm.

## MANNING'S FORMULA

A roughness coefficient of 0.015 has been applied to all existing pipes, as the provided model has been previously calibrated to this roughness value. For all future piping in the hydraulic model, a roughness coefficient of "N" value of 0.013 has been assigned, as per the City's Municipal Standards.



## MAXIMUM FLOW AND DEPTH CAPACITY

According to the City's Municipal Standards, the required capacity of sewer pipes is as follows:

$$q_{design} / Q_{max\ flow} \leq 0.86$$

where:

$q_{design}$  = maximum flow in the pipe during the simulation

$Q_{max\ flow}$  = full flow of the pipe, as calculated from the full pipe area, full hydraulic radius, roughness, and pipe slope

It should be noted that the full flow capacity of the pipe reported in PCSWMM ( $Q_{max\ flow}$ ) is calculated from Manning's formula (as a pipe property, not as a simulation result) based on the assumption of steady uniform flow. However, flows through the pipe are not uniform during the computer simulation. Under certain circumstances, the simulated depth of flow in the upstream or downstream end of a pipe may exceed the crown of the pipe (i.e. greater than the diameter of the pipe), even if the full flow of the pipe (Manning's formula) has not been reached.

When the flow depth within a pipe (or section of pipe) is higher than the crown of the pipe, service connections may not be able to flow freely into the gravity sewer. While this may not manifest into visible flooding into buildings, it could affect normal operation of basement facilities (i.e. high gravity sewer flow levels "block" the service connections). Based on this limitation, the City's updated hydraulic model has also been reviewed for peak HGLs (flow depth):

$$d_{design} / D_{max\ flow} \leq 0.86$$

where:

$d_{design}$  = maximum depth of flow within the pipe during the simulation

$D_{max\ flow}$  = maximum diameter of the closed pipe (flow depth at "full" flow)

## LIFT STATIONS

As per the City's design criteria, developments should be designed to rely on gravity for conveyance wherever possible in order to minimize operation and maintenance requirements and life cycle costs of the system. Where lift stations are absolutely required, pumping capacity of the lift station should be designed for peak flows with 100% redundancy. It should be noted that in many municipalities an N+1 approach is utilized instead, as 100% redundancy on lift stations with 2 or more main pumps could be considered unnecessary. This standard is recommended to be reviewed on a case-by-case basis.

It should also be noted that, based on previous studies and reports, the normal operation for most lift stations is lead/lag rather than duty/standby. Therefore, the capacity assessments summarized in this study are based on the measured capacity of the lift station (as reported in the 2019 Wastewater Facilities Study). However, it is recommended that opportunities for operational redundancy be reviewed for the City's lift stations.

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## 2.2 EXISTING WASTEWATER SYSTEM

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### 2.2.1 EXISTING POPULATION

#### HISTORICAL RESIDENTIAL POPULATION

Based on census data reviewed as part of the concurrent City's Waster Master Plan (WMP), the average annual population growth rate between 2008 – 2018 is 1.3%. The Edmonton Metropolitan Region Growth Board (EMRB) document published in 2017 estimated that the City will have a future population of approximately 90,000 – 118,000 persons by the year 2044, or a 1.2% to 2.3% per year growth rate. By comparison, the 2020 Draft Municipal Development Plan (MDP) projected a future population of 100,000 by 2044, which is equivalent to a 1.6% growth rate. As noted in the WMP, this growth rate exceeds the historical growth rate observed for the City.

Therefore, both the WMP and the current master plan study will be adopting a growth rate of 1.3% applied through to 2044, which results in a population of 92,455 by that time. An ultimate population of 100,000 is projected to be reached in 2050, based on a growth rate of 1.3%.

#### EXISTING RESIDENTIAL AND ICI POPULATION DENSITY

The 2018 census data showed a population of 66,082 persons within the City of St. Albert. Therefore, the projected 2020 current population within the City is estimated to be 67,811 persons, based on an annual 1.3% growth rate.

In the WMP, the existing land use densities were estimated using water meter data. The total area per land use was estimated based on the City's GIS land use maps. The WMP calculates existing population densities for Low Density Residential (LDR) and Multi-Family Residential (MFR) areas to be 29.0 p/ha and 88.2 p/ha, respectively. Population equivalents (EP) for Commercial/Industrial and Institutional areas are calculated to be 23.1 p/ha and 22.1 p/ha, respectively. These population densities were used to develop the existing population distribution within the City for the 2020 assessment scenario.

However, WSP notes that these population densities differ from the derived population densities of the 2017 Model Update. The 2017 Technical Memorandum estimates that 2016 population densities within the City's neighbourhoods range between 36 p/ha to 245 p/ha, which deviates from what has been calculated as part of the concurrent WMP. A potential reason for this could be the use of overall land use areas to calculate densities instead of individual property/unit parcels. This approach includes road areas and doesn't consider neighbourhoods with more units occupied than others (the effect of this on I&I allocation is further discussed in subsequent sections of this report).

The 2017 Technical Memorandum also shows that the per-unit residential density ranges between 1.8 p/unit to 3.0 p/unit. Estimating wastewater generation based on a rate-of-return on measured water meter data, with per-unit densities applied to specific parcels, may allow for a more refined population distribution throughout the City. This could lend itself to a more accurate representation of areas that have a denser residential population than others within the same land use designation.

For example, the 2017 Technical Memorandum shows Lacombe Park had a population density of 245 p/ha versus the adjacent neighbourhood of Deer Ridge, which was calculated to have a population density of 52 p/ha (both of which are higher than the single-family population densities calculated in the WMP). By comparison, a per-unit calculation shows that Lacombe Park and Deer Ridge have similar unit-based densities of 2.8 p/unit and 3.0 p/unit, respectively. This shows that there are potentially more single-family units currently existing within Lacombe Park than Deer Ridge, and it is expected that the wastewater generation density between these two areas would be different as opposed to an even distribution across the land use area. An added benefit of a parcel-based approach is in the event of redevelopment, population and



flow allocations in the model can be more easily replaced; the estimated wastewater generation rates of neighbouring properties will not be affected.

Based on discussions with City staff, it was determined that using the total land use area approach would be appropriate for this study as it will allow for a consistent population distribution between the concurrent master plan projects. The City's November 2020 land use districts map was used to establish the contribution populations and resulting wastewater generation rates, and flows for the existing developed areas.

## EXISTING POPULATION DISTRIBUTION

Total land use areas were used to assign land use, along with population equivalents and I&I contributing areas, to all the manholes throughout the City using the Thiessen Polygon Method. Where a parcel-based approach assigns parcel properties and wastewater service connections to the nearest manhole, the Thiessen Polygon method is based on using perpendicular bisectors between manholes to develop 'catchment' areas for each manhole. These 'catchments' were used to calculate the contributing population and effective I&I area for each manhole for this study. **Table 2-1** summarizes the estimated existing population with the City.

WSP notes that the application of blanket values for residential and ICI population densities has resulted in a conservative population estimate of 72,600 existing residents, in comparison to the 67,811 persons projected in the WMP.

**Table 2-1: 2020 Population Estimates**

NEIGHBOURHOOD	2020 ESTIMATED RESIDENTIAL POPULATION	2020 ESTIMATED ICI POPULATION EQUIVALENT
Akinsdale	4,114	603
Braeside	3,178	265
Campbell Business Park <sup>1</sup>	-	-
Deer Ridge	5,718	301
Downtown	386	715
Erin Ridge	5,519	695
Erin Ridge North	2,581	1143
Forest Lawn	3,000	409
Grandin	8,832	734
Heritage Lakes	3,499	161
Inglewood	1,180	1378
Jensen Lakes	1,484	662
Kingswood	4,914	197
Lacombe Park	8,150	578
Mission	1,711	640
Northridge	3,674	167
Oakmont	3,020	155
Pineview	2,429	73
Riel Business Park <sup>1</sup>	-	-
Riverside	3,732	290

NEIGHBOURHOOD	2020 ESTIMATED RESIDENTIAL POPULATION	2020 ESTIMATED ICI POPULATION EQUIVALENT
Sturgeon Heights	1,762	515
Ville Giroux	8,15	117
Woodlands	2,903	161
<b>Total</b>	<b>72,600</b>	<b>9,959</b>

<sup>1</sup> Equivalent population values were not assigned to Campbell Business Park and Riel Business Park. Wastewater flows from these areas are calculated according to an area-based generation rate, which was calibrated in the City's 2019 I&I Study.

## 2.2.2 EXISTING WASTEWATER FLOWS AND SCENARIO DEVELOPMENT

### EXISTING FLOWS – MODEL ALLOCATION

Flows under the existing scenario are based on the population allocation discussed in [Section 2.2.1](#) and the design flowrates discussed in the subsections below.

### EXISTING DRY WEATHER FLOWS

In combination with the population distribution and allocation approach discussed in the Design Criteria Establishment Technical Memorandum (Design Criteria Memo), DWF will be based on the 2019 I&I Study, as in [Table 2-2](#). For non-residential land uses within neighbourhoods outside of Campbell Business Park and Riel Business Park, the institutional, commercial, and industrial (ICI) population equivalents were estimated based on the land use area, and an assumed density of 25 p/ha. For newly developed areas (i.e. Jensen Lakes), a design rate of 220 L/c/d will be applied, as outlined in the Design Criteria Memo.

WSP notes that neighbourhoods without separate residential (L/c/d) and ICI (L/d/ha) generation rates might have overestimated flows, due to residential rates being applied to ICI population equivalents. This introduces an extra layer of conservatism into this model.

**Table 2-2: 2019 Calibrated DWF Rates**

NEIGHBOURHOOD	2019 CALIBRATED RESIDENTIAL DWF (L/c/d)	2019 CALIBRATED ICI DWF (L/d/ha)
Akinsdale	200	-
Braeside	424	-
Campbell Business Park	-	5,092
Deer Ridge	267	-
Downtown	254	-
Erin Ridge	182	-
Erin Ridge North	186	-
Forest Lawn	481	-
Grandin	415	-
Heritage Lakes	176	-
Inglewood	424	-
Jensen Lakes	-	-
Kingswood	238	-

NEIGHBOURHOOD	2019 CALIBRATED RESIDENTIAL DWF (L/c/d)	2019 CALIBRATED ICI DWF (L/d/ha)
Lacombe Park	343	-
Mission	328	-
Northridge	194	-
Oakmont	168	-
Pineview	200	-
Riel Business Park	-	4,356
Riverside	287	-
Sturgeon Heights	880	-
Ville Giroux	328	-
Woodlands	234	-

### EXISTING WET WEATHER FLOWS

Existing areas flow monitored in 2020 have been calibrated according to observed rainfall events closest to 1:25-year 24-hour and/or 1:5-year 4-hour storm events. All other existing areas will be assigned the area-based I&I rates based on 2019 I&I Study, according to the applicable neighbourhood, as shown in [Table 2-3](#). Highlighted values indicate an I&I rate that exceeds the allowable (design) rate. Highlighted values in [Table 2-3](#) indicate an I&I rate that exceeds the allowable (design) rate. For newly developed areas that were not covered in the 2019 I&I Study (i.e. Jensen Lakes), the City's design I&I rate of 0.28 L/s/ha will be applied.

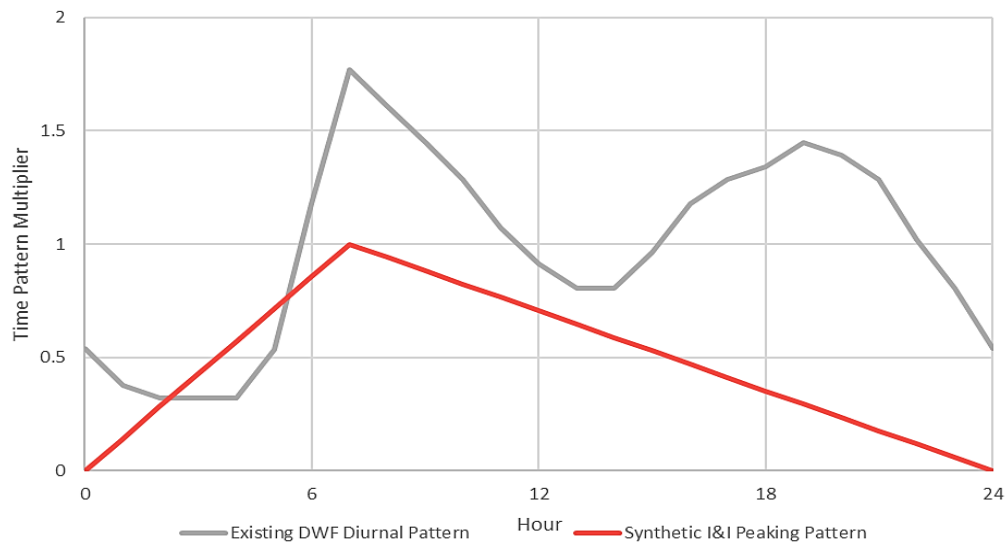
**Table 2-3: 2019 Calibrated I&I Rates**

NEIGHBOURHOOD	2019 CALIBRATED 1:25-YEAR 24-HOUR I&I (L/s/ha)	2019 CALIBRATED 1:5-YEAR 4-HOUR I&I (L/s/ha)
Akinsdale	0.533	0.520
Braeside	0.470	0.558
Campbell Business Park	0.234	0.270
Deer Ridge	0.433	0.588
Downtown	0.941	1.102
Erin Ridge	0.263	0.423
Erin Ridge North	0.178	0.296
Forest Lawn	0.178	0.296
Grandin	0.561	0.571
Heritage Lakes	0.329	0.397
Inglewood	0.180	0.299
Jensen Lakes	-	-
Kingswood	0.391	0.573
Lacombe Park	0.563	0.665
Mission	0.634	0.775
Northridge	0.164	0.273

NEIGHBOURHOOD	2019 CALIBRATED 1:25-YEAR 24-HOUR I&I (L/s/ha)	2019 CALIBRATED 1:5-YEAR 4-HOUR I&I (L/s/ha)
Oakmont	0.323	0.534
Pineview	0.748	0.875
Riel Business Park	0.518	0.677
Riverside	0.518	0.677
Sturgeon Heights	0.297	0.494
Ville Giroux	0.368	0.442
Woodlands	0.294	0.493

The sewershed areas for existing developed neighbourhoods, in the provided 2017 PCSWMM, were maintained for the 2020 Master Plan model. I&I contribution areas (existing and future development areas) are delineated based on the Thiessen polygon method. WSP notes a limitation to the catchment delineation approach, discussed in the population density distribution section, is that the I&I calculated for these areas may not be exactly representative of the actual I&I experienced by the system. It is recommended they be refined in the future as inclusion of roads may overestimate the inflow (fast response) effects of I&I, where it would otherwise be captured by the stormwater system. By comparison, it could skew the simulated results of infiltration (slow response) due to lawns or other grassed areas on parcel properties.

In the absence of RTK parameters, a synthetic time pattern will be applied to these areas based I&I rates to simulate peak I&I flows occurring at the same time as PDWF. This approach may not accurately illustrate whether a catchment experiences predominantly slow or fast responses to I&I but timing the peak I&I with PDWF can be considered a conservative approach to simulating PWWF. The synthetic curve for the area based I&I rates is shown in [Figure 2-3](#).



**Figure 2-3 Synthetic Wet Weather Flow Peaking Pattern**

## 2.2.3 EXISTING CAPACITY ASSESSMENT

### 2.2.3.1 ASSUMPTIONS AND LIMITATIONS

- The PCSWMM model provided did not include forcemain pipes downstream of the lift stations (i.e. modelled pumps are connected directly into the discharging manhole instead of into a pipe). While this approach could be enough for estimating the flow capacity within downstream systems (as the lift

station pumps would be discharging at peak flows), it may slightly overestimate the downstream volumes, as pipes inherently have some amount of storage. Additionally, the pumping head required at the related lift stations will be difficult to accurately assess without representing the friction losses within the forcemains. For example, the 2017 Technical Memorandum notes that the Gate Avenue lift station is almost ineffective and that the head-discharge curve for the pump should be confirmed. The head-discharge from the pump should be sized to overcome static head requirements (elevation difference) in addition to friction losses within the system (flows within the pipe).

- The model was updated according to GIS information available, supplemented by record drawings provided by the City. In some cases (e.g. Erin Ridge North, Meadowview, and Jensen Lakes), gaps occurred between the different information sources resulting in orphaned sections within the wastewater model. It is expected that these pipes may have been constructed at the time of development but may not yet be connected to the greater wastewater network as of last GIS database update. WSP notes that these areas represent small sections of upstream areas, where capacity issues are not expected. These orphaned sections have not been included in the model update, and their projected wastewater flows have been loaded onto the next existing downstream node.
- It is assumed that wastewater flows from Erin Ridge North and Jensen Lakes will be captured by the 1200 mm trunk main diversion at the intersection of Enchanted Way N and Executive Way N and will not flow to the Oakmont Lift Station. Instead, there is a new lift station currently being constructed east of Erin Ridge North (location currently assumed to be near Coal Mine Road) that will discharge directly to the SAPS facility via forcemain. Therefore, it is assumed that the 1200 mm trunk main is disconnected from the 450 mm gravity sewer along Everett Drive. Pump curves and operating levels for the temporary Erin Ridge North and Jensen Lake lift stations were not available.
- The Riverside siphon is assumed to be disconnected from the NIT. The four existing siphon pipes (160 mm, 200 mm, 250 mm, and 300 mm) have been modelled as a single pipe with a calculated equivalent diameter of 450 mm. The siphon collects flows from the Meadowview area and discharges into the SIT.
- The two temporary lift stations in the Riverside area are assumed to still be in place for the existing scenario. Due to limited record drawing and GIS shapefile information, it is recommended that the network connectivity and elevations for the Riverside area be reviewed and updated for accuracy, as more details become available.
- Seasonal lift stations have not been included in the model. These facilities include Meadowview lift station, Kinsmen RV Park, and Riel Sports Field.
- The model shows an existing orifice control at the connection from the NIT into the SAPS. Details of this orifice control were not investigated as part of this study, and the results presented in the existing system capacity analysis are provided based on the provided model's simulated flow controls.

### 2.2.3.2 LIFT STATION CAPACITY ASSESSMENT

Table 2-4 compares the required existing lift station capacities (based on the PCSWMM simulation) against the available/design capacities, as reported in the 2019 wastewater facilities study. Overall, the wastewater conveyance system is performing well and have adequate capacities under the existing system for the examined conditions.

**Table 2-4: Existing Lift Station Capacity Assessment Summary**

LIFT STATION <sup>1</sup>	NO. OF PUMPS / OPERATION	REQUIRED CAPACITY <sup>2</sup> (L/s)	DESIGN CAPACITY (L/s)	EXCESS (L/s)	DEFICIENT
Deer Ridge	Lead / Lag	19	22	3	No
Oakmont	2 Duty / 1 Standby	152	152	0	Yes
Rivercrest	Lead / Lag	30	51	21	No
Riel	1 Duty / 1 Standby	140	180 <sup>3</sup>	40	No
Riverside <sup>4</sup>	-	-	182	-	-
Twilight <sup>5</sup>	Lead / Lag	155	15	- 140	Yes
Gate Ave	Lead / Lag	41	40 <sup>6</sup>	- 1	Yes
Firehall	Lead / Lag	8	13	5	No
Erin Ridge	Lead / Lag	9	22	13	No
Erin Ridge North	Lead / Lag	9	32 <sup>7</sup>	23	No
Jensen Lakes	N/A	22	-	-	-

<sup>1</sup> Characteristics of the new lift station (Future Erin Ridge) is not included in this table as it is still in the preliminary design phase. Station is set to serve Erin Ridge North and Jensen Lakes and pump directly to the SAPS.

<sup>2</sup> Capacity required is the greater modelled peak wet weather inflow, based on the 1:5-year 4-hour or 1:25-year 24-hour storm.

<sup>3</sup> Measured capacity of the Riel Lift Station not available in the 2019 Facilities Study and was unavailable in the provided SCADA data. The excess capacity is calculated based on the reported design capacity of one pump at 180 L/s.

<sup>4</sup> Measured flows for the Riverside Lift Station were not available at the time of this study. As discussed with City staff, the existing scenario does not include a capacity assessment of the Riverside Lift Station. 182 L/s is based on a pump curve design point provided by the City.

<sup>5</sup> Required Capacity of the Twilight Lift Station is based on the substantial overflow from the upstream catchments, which occurs during wet weather events.

<sup>6</sup> SCADA data provided for the Gate Avenue lift station shows that the current maximum discharge rate of the Gate Avenue Lift Station is recorded as approximately 36 L/s.

<sup>7</sup> Under a separate project by WSP, the Erin Ridge North Lift Station was noted to have a design capacity of 43 L/s, but a measured pumping capacity of 32 L/s.

The model simulations show that the model may currently be underestimating the dry weather inflows into the lift stations, as the May 2020 recorded inflows appear to be greater than the simulation DWF (Rivercrest Lift Station and Gate Avenue Lift Station). Conversely, the Riel Lift Station is modelled to have a greater ADWF inflow than what is recorded in the City's SCADA data. These discrepancies could be due to inaccurate SCADA data, assumed population distribution, or wastewater loading in the model, or there are other flows contributing to the system on these dry weather days (e.g. system flushing activities, etc.). It is recommended to investigate the discrepancies in the DWF for the lift station catchments.

While the modelling results suggest the current pumps at these stations have sufficient capacity to convey peak DWF (based on the applied population loading approach), two lift stations are calculated to be over capacity for the design storm conditions (Twilight Lift Station and Gate Ave Lift Station). Further monitoring activities should be conducted to improve wet weather representation of the model prior to assessing options for system improvements, as needed.

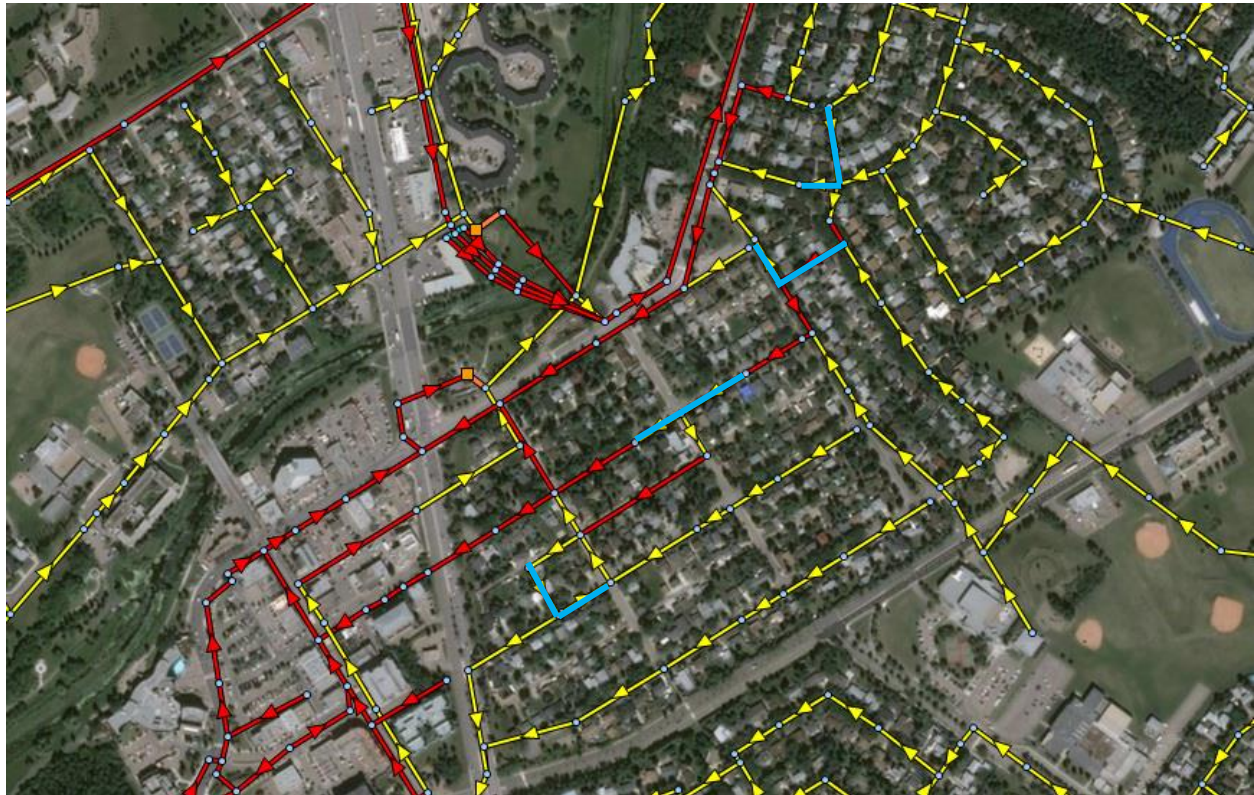
The Twilight Lift Station capacity issues appears to be due to receiving overflows at several locations from upstream catchments. In order to avoid oversizing the Twilight Lift Station, further analysis is required to determine if diverting flows from an upstream catchment to the North Interceptor or South Interceptor Sanitary trunk mains, or preventing/throttling overflows to the Twilight Lift Station could improve the scenario. For example, investigation of the PCSWMM model results shows that significant overflow occurs during wet weather events at several locations in the Braeside neighbourhood area, as shown in [Figure 2-4a](#).

Connections to the overflowing trunk could be closed either fully or throttled with a gate. Redirection of flow from upstream areas can be another possible solution. For instance, as shown in [Figure 2-4b](#), it appears that the weir is directing all the upstream flow to the Twilight catchment and almost no flow to the downstream 450 mm pipe under all simulated events. More real-system investigation of the potential additional surcharging within the trunk can reduce the total flow to the Twilight Lift Station, and determine if the wastewater system needs to be upgraded to contain the additional overflow.

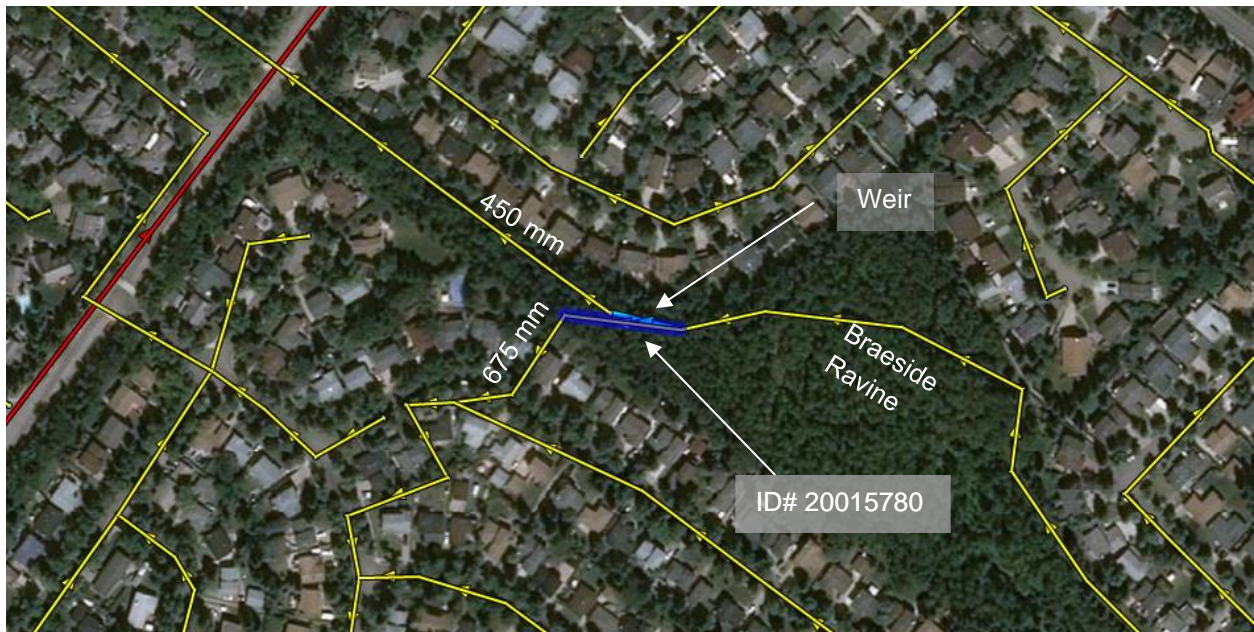
It is also worth noting that another significant factor in this overflow is elevated I/I rates. Peak dry weather inflows into the station (7 L/s) are modelled to be significantly lower than peak WWF (155 L/s, 1:5-year 4-hour storm), indicating that the upstream catchments have severe I&I levels. As per the 2019 I&I Study, the estimated 1:5-year 4-hour I&I rate for the downtown area, alone, is 1.1 L/s/ha. Other upstream catchments such as Akinsdale, Forest Lawn, Braeside and Pineview were all reported to have I&I rates greater than 0.5 L/s/ha.

The most recent life cycle assessment for the existing lift stations in the wastewater system was conducted in 2019 by the City of St. Albert, in which a multi-disciplinary assessment was completed for eighteen wastewater facilities (Deer Ridge, Erin Ridge Sanitary, Erin Ridge Storm, Firehall #1, Gate Avenue, Glacier Place, Kingswood Park, Lacombe Lake, Meadowview, Mission, Oakmont, Riel, Riel Sports Field, Rivercrest, Riverview, Twilight, Walmart/Jensen Lake Storm Lift Station). The wastewater lift stations were inspected and evaluated for different disciplines, including architectural, civil, electrical and instrumentation, building-mechanical, process-mechanical, structural and HSE. The assessment included field evaluations, condition ratings, and recommendations for station improvement works.

On average, the condition and performance ratings of the lift station facilities were found to correlate with the age of the station, where new stations generally scored higher. The overall lift station condition assessment was found to range between 50% to 91%. For example, the Gate Avenue, Riel, Kinsmen and Meadowview lift station facilities were all in good condition scoring above %80 overall and with consistent scores in the mechanical, electrical and process discipline ratings. Kingswood Day Park and Lacombe Lake Day Park lift stations both scored below 60%. The Kingswood facility had many problems in the process, structural and HSE disciplines receiving the lowest score of 50.3%; therefore, it was recommended to replace the existing facility rather than following up with upgrades to the Kingswood lift station.



(a) Overflow Locations in Braeside Area



(b) Overflow Weir Location Adjacent to Braeside Ravine

**Figure 2-4 Model Results for Overflow to Twilight Lift Station Service Area**

### 2.2.3.3 COLLECTION SYSTEM CAPACITY ASSESSMENT

Several areas of the wastewater system were modelled to be under capacity under the existing system scenario. Figures illustrating all locations are provided in the attached [Appendix A](#).

A number of areas were also modelled to be above capacity:

- Significant portions of Lacombe Park are modelled to have surcharging and above-ground flooding under the design storms. As wastewater issues have not been observed at surface level in this area, this suggests that the modelled capacity deficiencies may be due to the simulated I&I responses in the system.
- Under wet weather conditions, the pipes within the St. Albert Centre area are modelled to be surcharging and flooding above-ground. While current DWF are not modelled to result in capacity issues in this area, capacity constraints presented during wet weather events may be exacerbated if not addressed prior to intensification.

WSP understands orifice controls have been installed on the upstream west end of the NIT, likely as an approach to control flow volumes and reduce size requirements in the downstream sections of the trunk sewer. Modelling shows that during wet weather events, the orifice controls result in a relatively flat but surcharged HGLs within the trunk sewer. Under these conditions, gravity sewers that are connected to the trunk sewer at invert-to-invert elevation will likely experience backwater effects. The simulation results suggest that surcharging could occur in upstream sections of connected catchments, due to wastewater volumes collecting at these connection sites because they are unable to freely discharge into the trunk sewer.

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### 2.2.4 EXISTING FLOWS TO SAPS

The existing peak flows into SAPS is approximately 1,496 L/s – 1689 L/s between the two design storms, and prior to any system improvement works. According to the 2018 NIT Phase 3 Report, the SAPS station is expected to have a capacity of 1,800 L/s in 2020. In the wet weather flow data provided and the 2020 ACRWC Annual Report (November 2020), the peak wet weather flow observed was 819.5 L/s observed between April and September 2020. Based on the provided model, currently there is a flow control from Phase 3 of the NIT into SAPS. Increasing the amount of flow through the NIT into SAPS may relieve the high HGLs modelled in the NIT. However, the capacity of the SAPS station should be confirmed prior to increasing the City's total discharge rate into the facility. Further discussion on improvement options is provided in [Section 2.4.2](#).

## 2.3 ULTIMATE WASTEWATER SYSTEM

### 2.3.1 FUTURE POPULATION

The following sources were referenced in the development of the future population projection:

- Area Structure Plans (ASP)
- City of St. Albert Open Data portal (as of February 2021)
- Flourish – Growing to 100K” Municipal Development Plan (Flourish MDP)
- Water Distribution System Master Plan 2020 Update (Draft)

#### AREA STRUCTURE PLANS

The following ASPs were reviewed in the development of future population projections of growth areas:

- Downtown Area Redevelopment Plan (DARP), 2010
- Ville Giroux, 2012
- Jensen Lakes ASP, 2014
- Range Road 260 ASP (Avenir and Elysian Fields), 2014
- South Riel ASP, 2015
- North Ridge ASP (Phase 2), 2015
- Erin Ridge North ASP (Phase 2), 2019
- Riverside ASP, 2020

**Table 2-5** summarizes the projected residential population and percentage of Gross Developable Area (GDA) to be dedicated to ICI. Detailed unit density information can be found in the respective ASP documents.

**Table 2-5: City of St. Albert Future Developments – Approved Area Structure Plans**

APPROVED AREA STRUCTURE PLAN	LOW DENSITY RESIDENTIAL UNITS	MEDIUM DENSITY RESIDENTIAL UNITS	HIGH DENSITY RESIDENTIAL UNITS	PROJECTED RESIDENTIAL POPULATION	DEDICATED ICI AREA
Downtown ARP	-	-	2800	N/A <sup>1</sup>	20%
Ville Giroux	85	236	425	1409	16%
Jensen Lakes	1584	360	239	5727	9%
Range Road 260 – Avenir	538	461	791	4284	1%
Range Road 260 – Elysian Fields	309	204	209	1653	41%
South Riel	86	324	709	2442	56%
North Ridge – Phase 2	283	47	226	1324	6%
Erin Ridge North – Phase 2	240	-	405	1409	26%
Riverside	2513	652	780	9861	3%

<sup>1</sup> Proposed population projection not available within the approved ASP.

## OPEN DATA PORTAL

Residential occupancy projections for identified residential developments within the City were extracted from the City's Open Data portal (as of February 2021). WSP understands the list may not be inclusive of all future lands to be developed. However, they were reviewed in order to compare the density and population projections proposed in other planning documents. The future residential developments identified through the City's Open Data Portal are summarized in [Table 2-6](#).

**Table 2-6: City of St. Albert Future Developments – Open Data Portal**

DEVELOPMENT	AREA (ha)	ICI	RESIDENTIAL
Erin Ridge North	6.2	Commercial	-
Jensen Lakes Crossing	6.5	Commercial	-
Shoppes at Giroux	4.7	Commercial	-
Riverside Landing	5.6	Commercial	-
Anthony Henday Business Park Retail	4.7	Commercial	-
The Urban District	4.9	Commercial	Unknown <sup>1</sup>
Riverbank Landing	4.0	Commercial	Multi-Family <sup>2</sup>
Grandin Parc Village	4.6	Commercial	Multi-Family <sup>2</sup>
Midtown	18.1	Commercial	Multi-Family <sup>2</sup>
Bellevue Village	5.8	Commercial	Multi-Family <sup>2</sup>
Lakeview Business District	254.7	Industrial	-
Anthony Henday Business Park	39.1	Industrial	-
Campbell Business Park	136.4	Industrial	-
Erin Ridge North	94.6	-	Low Density Multi-Family
Jensen Lakes	135.9	-	Low Density Multi-Family
North Ridge Phase 2	54.5	-	Low Density Multi-Family
Elysian Fields	99.4	-	Low Density Multi-Family
Ville Giroux	21.7	-	Low Density Multi-Family
Riverside	187.0	-	Low Density Multi-Family
Downtown	19.7	N/A	Low Density Multi-Family

<sup>1</sup> The Urban District is a proposed mixed-use development. Proposed residential unit counts were not available.

<sup>2</sup> Proposed townhomes have been included in the multi-family unit count.

WSP notes that the population per unit density varies between residential land use types (e.g. low density, high density, etc.) as well as between the various development areas. To maintain a basis of comparison to the densities developed as part of the WMP, the area-based residential population densities were calculated, and are summarized in [Table 2-7](#).

In general, the population projections within the ASP documents are consistent with those outlined in the Open Data Portal, with the exception of the Riverside development. As shown in the table above, the general residential population per hectare for the outward growth areas 17 p/ha to 65 p/ha.

**Table 2-7: Residential Unit Densities – Open Data Portal**

DEVELOPMENT	AREA (ha)	LD UNITS	MF UNITS <sup>1</sup>	PROJECTED POPULATION	DENSITY (p/ha)
Erin Ridge North	94.64	940	982	4381	46
Jensen Lakes	135.94	1761	417	5727	42
North Ridge Phase 2	54.52	1264	695	1324	24
Elysian Fields	99.38	513	209	1653	17
Ville Giroux	21.70	85	661	1409	65
Riverside	186.99	3165	780	5400 <sup>2</sup>	29
<b>Total</b>	<b>593.17</b>	<b>7728</b>	<b>3744</b>	<b>19877</b>	<b>-</b>

<sup>1</sup> Proposed townhomes have been included in the multi-family unit count.

<sup>2</sup> The Riverside ASP outlines a projected residential population of 9,861.

## FLOURISH MDP

Key growth targets from the City's Flourish MDP are as follows:

- North Transit Oriented Development Centre (located in the Erin Ridge North area), is projected to create 140 EP/ha (jobs).
- Areas between 400 m – 800 m from the St. Albert Trail (Rapid Transit Area / Corridor), and areas adjacent to Downtown are targeted to have 50 – 125 residential units/ha. WSP assumes this to represent Medium Density / Residential / Mixed Use Intensification areas.
- Areas within 400 mm of the St. Albert Trail (Rapid Transit Area / Corridor) are targeted to have at least 200 units/ha. WSP assumes this to represent High Density Residential / Mixed Use Intensification areas.
- Commercial / institutional areas within 800 m of the St. Albert Trail (Rapid Transit Area / Corridor) are projected to have 140 EP/ha (jobs).
- A population equivalent specifically for the commercial/institutional portion of mixed-use parcels within the Downtown area was not provided in the Flourish MDP. Table 2-8 summarizes the ultimate density targets for the City, as per the Flourish MDP.

**Table 2-8: Population Densities – Flourish**

LAND USE TYPE	OUTWARD GROWTH	INTENSIFICATION
Single Family Residential	Minimum 40 units/ha	30% increase in density
Medium Density Residential	-	50 – 125 units/ha
High Density Residential	-	200 units/ha
Mixed Use	50 – 60 units/ha	-
Commercial / Institutional	140 EP/ha	140 EP/ha
Downtown Area	-	100 units/ha (residential)

WSP notes that target population densities (e.g. p/ha and ppu) were not outlined in the Flourish MDP, apart from EP for ICI land uses. Due to the generality of the regions identified for future growth (e.g. along the entire length of the Regional Transit Corridor) we would not recommend using these large areas in the calculation of future population projections. Additionally, based on discussions with City staff, the ICI growth targets are significantly higher than the EP currently observed throughout the City.



## 2020 WATER MASTER PLAN

Table 2-9 summarizes the recommended future population densities, as per the City's updated WMP. Overall, the high-density target growth areas identified in the WMP are in line with key development areas outlined in the City's Flourish MDP, Open Data Portal, and ASPs.

**Table 2-9: Future Population Densities – 2020 Water Master Plan**

LAND USE TYPE	OUTWARD GROWTH	INTENSIFICATION	EXISTING DEVELOPED AREAS
Single Family Residential	60 p/ha	-	40 p/ha
Medium Density Residential	-	-	80 p/ha
High Density Residential	-	-	200 p/ha
Mixed Use	200 p/ha	200 p/ha	-
Commercial / Institutional	37 EP/ha	-	37 EP/ha

For the purpose of infrastructure planning, the City may choose to develop a per unit population density for each neighbourhood and land use type based on recent Census data. This may provide a more applicable population distribution, as a blanket population density may result in overestimating in one neighbourhood and underestimating in another.

## FUTURE POPULATION SUMMARY

Table 2-10 below summarizes the future developments. WSP notes the following key discrepancies between the population planning documents:

- The WMP applied a 60 p/ha density for outward growth of low-density residential areas, whereas the approved ASPs are calculated to have a range between 17-65 p/ha.
- High density residential areas were proposed to have 200 p/ha in the WMP, whereas the target growth outlined in the Flourish MDP is 200 units/ha. Combining these two documents would equate to a density 1 person/unit, which is lower than the unit densities proposed in any of the approved ASPs. For example, high density units in Erin Ridge North are anticipated to have 1.76 persons/unit.
- Equivalent population densities for ICI areas is proposed to be 37 EP/ha in the WMP, but 140 EP/ha in the Flourish MDP. For comparison, the City of Edmonton standards suggests 40,000-120,000 L/ha/day for ICI land uses such as restaurants, offices, departments stores, etc. The standard City of Edmonton per capita generation rate is 220 L/c/d, which would equate to approximately 182-545 EP/ha for ICI land uses.

It is important to note that the wastewater system is more sensitive to population densities and distributions in comparison to the water system. This is because the water distribution system design is typically governed by fire flow requirements (i.e. overall land use designation), which is typically much higher than the domestic demands (consumption flow rates per person).

These discrepancies were discussed with the City, and the following approach was decided for the population projection:

- Future land areas to be developed will be consistent with the parcels outlined in the WMP.

- Residential population projections outlined in the approved ASPs will be applied to the future development areas for this study. For development parcels without ASPs, unit and population densities recommended in the WMP will be applied.
- ICI land areas will be based on the approved ASPs where available, at a population density of 37 EP/ha as per the WMP. Medium- and high-density units are assumed to be within the mixed-use land areas (i.e. areas outlined as solely residential in the WMP are considered to be the low-density/single family residential areas).
- Populations mixed use areas that do not have an available ASP for reference will be estimated based on 200 EP/ha as per the WMP.

**Table 2-10: 2020 UMP Update – Future Development Population Projections**

AREA ID	PROPOSED LAND USE	AVAILABLE ASP	RESIDENTIAL POPULATION	ICI POPULATION
0	Employment Lands	-	-	1286
1	Employment Lands	-	-	3781
2	Employment Lands	-	-	2020
3	Employment Lands	-	-	2657
4	Residential	Ville Giroux	1409	139
5	Residential	Riverside	3326	-
6	Residential	Riverside	963	-
7	Mixed Use	Riverside	2573	740
8	Residential	Riverside	2999	-
9	Residential	None	366	-
10	Residential	Elysian Fields	299	-
11	Residential	Avenir	1318	-
12	Residential	Elysian Fields	810	-
13	Residential	Elysian Fields	327	-
14	Mixed Use	Elysian Fields	217	496
15	Residential	Avenir	865	-
16	Residential	Avenir	1259	-
17	Mixed Use	Avenir	242	210
18	Employment Lands	-	-	1788
18	Employment Lands	-	-	1788
19	Residential	North Ridge Phase 2	509	-
20	Residential	North Ridge Phase 2	815	59
21	Residential	Jensen Lakes	2864	204
22	Residential	None	3871	-
22	Residential	None	3871	-
23	Mixed Use	Erin Ridge North Phase 2	713	247
24	Residential	Erin Ridge North Phase 2	696	-
27	Residential	-	965	-
28	Mixed Use	-	2499	2499
29	Mixed Use	-	3754	3754
30	Residential	-	2990	-

AREA ID	PROPOSED LAND USE	AVAILABLE ASP	RESIDENTIAL POPULATION	ICI POPULATION
31	Mixed Use	-	1769	1769
27	Residential	-	965	-
28	Mixed Use	-	2499	2499
29	Mixed Use	-	3754	3754
30	Residential	-	2990	-
31	Mixed Use	-	1769	1769
37	Residential	-	362	-
38	Mixed Use	-	4524	4524
39	Employment Lands	-	-	1876
40	South Riel	South Riel	-	1449
41	South Riel	South Riel	2442	1160
42	Downtown	Downtown	3921	3921

## 2.3.2 WASTEWATER FLOWS AND SCENARIO DEVELOPMENT

### FUTURE FLOWS – MODEL ALLOCATION

The approach for allocated projected flows under the Ultimate Scenario is, as follows, based on the population allocation discussed in [Section 2.3.1](#) and the design flow rates discussed in the subsections below:

- Flows from all existing developed areas (that have not been identified for intensification or further development) are assumed to remain unchanged for the ultimate scenario.
- Flows from future development parcels (new areas) that have not been included in the existing scenario (e.g. Avenir, Elysian Fields, etc.) have been allocated to new model nodes, at approximately the centroid of the land area. Where a development parcel encompasses more than one model node, the projected flows have been evenly allocated between the nodes.
- Flows from future development parcels that encompass areas with existing manholes have been allocated to existing model nodes, replacing the wastewater loads that were developed in the 2020 system scenario. (e.g. Ville Giroux, South Riel, Intensification Lands, etc.). Where the development parcel and intensification lands span over multiple existing manholes, the projected flows have been evenly distributed across the model nodes within the area.

### FUTURE DRY WEATHER FLOWS

The 2019 I&I Study recommended a future residential rate of 250 L/c/d as it appeared to be closer to the flowrates observed in the newer existing areas, compared to the City’s design criteria document that outlines an average dry weather flow (ADWF) of 320 L/c/d for residential land use areas. WSP notes that the concurrent WMP proposes to use an average day water demand (ADD) of 250 L/c/d, which is less than the ADWF outlined in the City’s Municipal Standards. Typically, systems do not experience higher wastewater generation than domestic water consumption. In our experience, ADWF generation rates range from 80% to 90% return on ADD. Based on discussions with City staff, this study will apply a 250 L/c/d ADWF generation rate for future development areas, as recommended in the 2019 I&I Study.

WSP notes that the City’s WMP applies an area-based population equivalent for future ICI areas. Based on discussions with City staff, in order to maintain consistency between the master plan documents, future flows from ICI areas are calculated based on estimated population equivalent, and a per capita consumption

of 250 L/c/d instead of 6,170 L/ha/d (this equates to approximately 25 EP/ha, in comparison to the 37 EP/ha estimated in the WMP for future ICI areas).

WSP notes that a future diurnal curve has not been designed for the Ultimate Scenario model; the flows have been allocated into the model to represent a static PDWF.

## FUTURE WET WEATHER FLOWS

Unit hydrographs for areas calibrated as part of this study are carried forward into the Ultimate scenario. For future development and intensification areas, an I&I rate of 0.28 L/s/ha is applied. Similarly, all other existing areas have been assigned I&I rates based on the 2019 I&I Study, according to the applicable neighbourhood, as shown in [Table 2-3](#). Sewershed areas developed for existing neighbourhoods in the provided 2017 PCSWMM model were maintained for the Ultimate Scenario. For future development and intensification lands, the contributing area used to calculate I&I flows was measured using GIS, based on the gross area of the proposed development.

WSP notes that a synthetic curve for the I&I flows has not been designed for the Ultimate Scenario model; the flows have been allocated into the model to represent a 0.28 L/s/ha static I&I for wet weather simulations.

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### 2.3.3 ULTIMATE CAPACITY ASSESSMENT

This section of the report covers the hydraulic analysis of the existing wastewater system under the current and future design horizons. The objective of this analysis is to assess the system's compliance with the level of service criteria discussed and agreed upon with City staff, as detailed in [Section 2.1](#). Existing and future deficiencies are highlighted in order to determine appropriate upgrade options to accommodate the ultimate system conditions.

#### 2.3.3.1 ASSUMPTIONS AND LIMITATIONS

- Rim elevations for future model nodes are based on the provided DEM information. Invert elevations have been interpolated based first on the on the minimum slope required, and second on the minimum depth required (as available), according to the City's Municipal Standards. WSP notes that these elevations and slopes are only preliminary and may not be representative of final developed grades. More detailed modelling should be conducted as more development information becomes available.
- Forcemains have not been modelled in the current study. Therefore, pump capacity requirements have been assessed based on the required flow capacity. Head requirements for sizing pumps within the lift stations should be reviewed when the forcemain size and alignment information become available.
- Preliminary network layouts for future development areas are based on the City's 2013 UMP report. Based on discussions with City staff, the capacity assessment for the Ultimate Scenario is aimed to confirm the future servicing strategies outlined in the 2013 UMP, and provide additional comments and recommendations following the approaches introduced in that report.
- As flows for future development areas have been loaded into the model as static flows (PDWF and I&I), the volume of wastewater over time (i.e. over a 24-hour period) may be overestimated in the hydraulic model. However, peak flows within the system, which is the basis for the following capacity assessment, is represented through this approach.
- The model shows an existing orifice control at the connection from the NIT into the SAPS. Details of this orifice control were not investigated as part of this study, and the results presented in the existing system capacity analysis are provided based on the provided model's simulated flow controls.

### 2.3.3.2 LIFT STATION CAPACITY ASSESSMENT

Table 2-11 compares the required future lift station capacities (based on the PCSWMM simulation) against the measured capacities, as reported in the 2019 wastewater facilities study.

**Table 2-11: Ultimate Lift Station Capacity Assessment Summary**

LIFT STATION	NO. OF PUMPS / OPERATION	CAPACITY REQUIRED <sup>1</sup> (L/s)	MEASURED CAPACITY (L/s)	EXCESS (L/s)	DEFICIENT
Deer Ridge	Lead / Lag	19	22	3	No
Oakmont	2 Duty / 1 Standby	153	152	- 1	Yes
Rivercrest	Lead / Lag	53	51	- 2	No
Riel	1 Duty / 1 Standby	198	180 <sup>2</sup>	- 18	Yes
Riverside	-	323	182 <sup>3</sup>	-	-
Twilight	Lead / Lag	161	15	- 146	Yes
Gate Ave	Lead / Lag	106	40 <sup>4</sup>	- 70	Yes
Firehall	Lead / Lag	23	13	- 10	Yes
Erin Ridge	Lead / Lag	9	22	13	No
Erin Ridge North	Lead / Lag	65	32 <sup>5</sup>	- 33	Yes
Jensen Lakes	N/A	69	-	-	-
New Lift Station <sup>6</sup>	N/A	169	-	-	-
Southwest	N/A	137	-	-	-
South Riel	N/A	31	-	-	-

<sup>1</sup> Capacity required is the greater modelled peak wet weather inflow, based on the 1:5-year 4-hour or 1:25-year 24-hour storm.

<sup>2</sup> Measured capacity of the Riel Lift Station not available in the 2019 Facilities Study and was unavailable in the provided SCADA data. The excess capacity is calculated based on the reported design capacity of one pump at 180 L/s.

<sup>3</sup> Measured flows for the Riverside Lift Station were not available at the time of this study. 182 L/s is based on a pump curve design point provided by the City.

<sup>4</sup> SCADA data provided for the Gate Avenue lift station shows that the current maximum discharge rate of the Gate Avenue Lift Station is recorded as approximately 36 L/s.

<sup>5</sup> Under a separate project by WSP, the Erin Ridge North Lift Station was noted to have a design capacity of 43 L/s, but a measured pumping capacity of 32 L/s.

<sup>6</sup> New Lift Station located in the NE of the City, that will collect flows from Erin Ridge North and Jensen Lakes and pump directly to the SAPS.

Modelling results suggest that some lift stations are calculated to be under capacity for the projected future loads, under the design storms and existing I&I rates.

The capacity issues of the Twilight Lift Station are further worsened in the future scenario when I&I is not addressed due to increased domestic flows from new developments and intensification of upstream areas. As discussed in the assessment of the existing conditions, the increase in the required capacity is driven by overflows from upstream catchments and can be rectified with investigations to close the overflowing pipe connections or redirection of upstream flows. Additionally, the Oakmont, Riel, Gate Ave, Fire Hall, and Erin Ridge North Lift Stations are modelled to be under capacity in the ultimate scenario.

The New Lift Station is modelled to have a peak inflow of 169 L/s in this scenario; however, this is based on upstream lift stations (Erin Ridge North, Jensen Lakes) pumping at rates lower than their required capacity. It is expected that under the improvement scenario (Section 2.4), the ultimate capacity requirement for the New Lift Station will be higher when the upstream lift stations are upgraded.

### 2.3.3.3 COLLECTION SYSTEM CAPACITY ASSESSMENTS

Significant portions of Lacombe Park are modelled to have surcharging and above-ground flooding under the design storms. Under wet weather conditions, the pipes within the St. Albert Centre area and upstream are modelled to be surcharging and flooding above-ground. This scenario shows that capacity constraints presented during wet weather events may be exacerbated if not addressed prior to intensification. In the rest of the system, surcharging and surface flooding is shown.

WSP understands orifice controls have been installed in Phase 1 of the NIT, likely as an approach to provide wet weather storage and reduce size requirements in the downstream sections of the trunk sewer. These orifices have not been verified as a part of this study; however, modelling shows that during wet weather events, the current orifice controls result in high HGLs within the trunk sewer. Under these conditions, gravity sewers that are connected to the trunk sewer at invert-to-invert elevation will likely experience backwater effects. The simulation results suggest that surcharging could occur in upstream sections of connected catchments, due to backwater effects because they are unable to freely discharge into the trunk sewer.

### 2.3.4 FUTURE FLOWS TO SAPS

According to 2018 NIT Phase 3 Report, the SAPS station is expected to have a capacity of 2,400 L/s in 2040. Based on the PCSWMM model and previous reports, flow controls along the NIT may be optimized to relieve high HGLs modelled in the NIT, maximizing the capacity of the SAPS. A wholistic review of system recommendations, which includes optimizing system improvements upstream, is further discussed in [Section 2.4](#).

## 2.4 WASTEWATER SYSTEM IMPROVEMENTS (IMPROVEMENT SCENARIOS)

### 2.4.1 IMPROVEMENT SCENARIOS DEVELOPMENT

#### DRY WEATHER FLOWS

Under the existing and ultimate improvement scenarios, the allocation of domestic flows is unchanged.

#### WET WEATHER FLOWS

Under the existing improvement scenario, the WWF are assumed to be consistent with recent calibration activities for each storm event, as city-wide I&I reduction in the short term would not be feasible. Therefore, the upgrade recommendations provided for the existing improvement scenario take the existing I&I rates into consideration. Under the ultimate improvement scenario, the WWF have been replaced with the threshold I&I rate of 0.28 L/s/ha. As a result, there is only one wet weather scenario for the ultimate improvement scenario.

#### 2.4.1.1 INFLOW AND INFILTRATION

Simulations show that a major source of WWF is due to the high estimated I&I throughout the system. PCSWMM modelling shows that the system generally has sufficient capacity to convey the current WWF in most areas of the City under existing conditions. However, the high I&I is simulated to put significant strain on the wastewater collection system under the ultimate scenario, with the simulation showing surface flooding in many areas of the City as a result of unmitigated I&I during wet weather events.



While newer developments in the City will likely have lower area based I&I rates than in older neighbourhoods, thereby “diluting” the City’s overall L/s/ha I&I into the SAPS, capacity issues within existing neighbourhoods would persist. Upsizing the lift stations according to the higher I&I rates may result in the lift stations being ultimately oversized in the long term, after the City reduces the WWF influences on the system (I&I reduction).

A separate study was conducted in 2019 to recommend I&I management and source control initiatives to reduce I&I throughout the City. Based on updated flow monitoring and calibration of the existing system, the simulation shows that the following areas are at risk for surface flooding during threshold storm events, most likely due to I&I effects. Lacombe Park, Liberton Drive, and Downtown are areas that should be prioritized for additional flow monitoring and/or mitigation activities.

As a first step, WSP would recommend additional flow monitoring to ensure the model parameters are representative of how the system is currently behaving, and how it would behave during a significant storm event. Once confidence in the data and model parameters is increased, a tailored solution can be developed (e.g. confirm if deficiencies exist and determine if either increased capacity and/or I&I reduction strategies are the appropriate solution). The long-term I&I reduction recommendations are provided in the 2019 I&I Study.

As WSP understands that the City has undertaken an I&I mitigation initiative to relieve wet weather stresses on its wastewater system, the system improvement recommendations discussed in the subsequent sections of this report (e.g. infrastructure upgrades) are based on a City-wide I&I rate of 0.28 L/s/ha.

#### **2.4.1.2 LIFT STATIONS**

Based on the updated modelling, several of the City’s lift stations are undersized when compared to the existing and future projected peak flows. A detailed lift station facilities assessment was completed in 2019, including prioritization for upgrades as well as cost estimates. While the capital plan provides an indicative timing and budgetary estimate for when facility upgrades are required, design requirements such as peak flows (according to domestic flows and I&I rates at the time) should be re-confirmed prior to initiating the capital project.

##### **TWILIGHT LIFT STATION (EXISTING SCENARIO)**

The existing Twilight lift station is heavily influenced by overflowing wet weather flow from a trunk normally draining to the SIT. This trunk backs up during wet weather event, draining to the Twilight Lift Station. The existing capacity of the Twilight lift station is currently measured to be 15 L/s, which is significantly less capacity than required for the wet weather scenario (155L/s). Under the ultimate scenario, this flow increases to 161 L/s under the ultimate scenarios. If I&I rates were to be improved to the standard of 0.28 L/s/ha, a peak inflow of 47 L/s is modelled for the ultimate improvement scenario, given the overflow from other contributory catchments is resolved.

Additionally, the inlet pipe into the Twilight lift station currently discharges at the base (invert) level of the wet well, based on the PCSWMM model. It is recommended to confirm the wet well base elevation; the wet well may have to be deepened during upgrades, to avoid backwater effects into the inlet pipes.

As an alternative to a large capacity increase, connections to the overflowing trunk could be closed either fully, or throttled with a gate. This will require a more in-depth investigation of the potential additional surcharging within the trunk as result of reduced flow to the Twilight lift station, and may necessitate downstream trunk upgrades to the SIT line.

##### **NEW LIFT STATION**

The New Lift Station near Coal Mine Road is modelled to have a peak inflow of 169 L/s under the ultimate improvement scenario. However, as discussed in previous sections, this is based on the modelled peak wet



weather flow capacity requirements of upstream lift stations (Jensen Lakes, Erin Ridge North). WSP understands that the contribution area to the New Lift Station is also subject to change, following this 2020 UMP Update. A detailed design and sensitivity study should be conducted to optimize the design point for the New Lift Station (e.g. need to also consider downstream constraints of the SAPS capacity, based on peak flows from the NIT, SIT, and Oakmont lift station as well).

#### **GATE AVENUE LIFT STATION (EXISTING SCENARIO)**

According to SCADA data and previous assessments, the Gate Avenue lift station appears to have a measured maximum flow rate of 36 L/s, which is less than the existing modelled inflow for the lift station. Under the existing scenario, the modelled inflow is 41 L/s; however, and the upstream pipes are simulated to be highly surcharged due to the inflow being greater than the lift station output. Under the improvement scenario, the upstream pipes are upsized to accommodate existing high flows (domestic and high I&I) and should also have enough capacity to convey future flows (increased densification, with reduced I&I). For the interim improvement scenario, it would be recommended to increase capacity of the Gate Avenue Lift Station (in combination with gravity sewer upgrades described in the following sections) to accommodate the simulated peak WWF of 51 L/s (simulated peak flows when upstream pipes are upsized), as shown in [Table 2-12](#). Under the ultimate scenario (with I&I reduction and pipe improvements), the simulated inflow into the Gate Avenue Lift Station is modelled to be 97 L/s due to development intensification of the area. It is recommended that the lift station be upgraded again at that time (e.g. additional pumps) to service the increased population.

**Table 2-12: Gate Avenue Lift Station Design Discharge**

	EXISTING CONDITIONS	EXISTING CONDITIONS (IMPROVED)	FUTURE (REDUCED I&I, DENSIFICATION)
Design Discharge (L/s)	41	51	97

#### **2.4.1.3 GRAVITY SEWERS**

Modelling results show that most capacity issues (flow depth) within the City's system can be addressed through I&I reduction as well as through lift station upgrades. For other areas, gravity sewer sizing should be considered based on their ability to service the City's ultimate scenario (e.g. future growth, reduced I&I, etc.). This is to avoid the need for upsizing any section of the system twice. Upsizing recommendations to resolve existing deficiencies should be sized such that they are not deficient again in the ultimate scenario. Therefore, the system improvements have been evaluated based on their ultimate requirements and then prioritized based on the horizon in which capacity issues are triggered.

In the improvement scenario, trunk sewers and sub-trunk sewers with d/D rates greater than 0.86 (but less than 1.0) were permitted, to avoid oversizing the system under normal conditions. In some areas of the model, the slopes and grading of the pipes should be confirmed, as adverse slopes (reverse sloping pipes) in the model are triggering high d/D results.

WSP notes that proposed upgrades are sized to take current I&I rates into consideration. It is expected that as I&I rates throughout the City are reduced over time, these upgraded pipes will have sufficient capacity to convey future flows with a 0.28 L/s/ha I&I rate. The upgrade recommendations below should be confirmed through development servicing reviews, as the capacity requirements simulated in the model are heavily dependent on the population projections and flow allocations according to information currently available.

The following recommendations are for system improvements within the City's existing wastewater system, and do not include sizing or recommendations for future development areas:

#### **GS-01 ST. ALBERT TRAIL AND MCKENNEY AVENUE UPGRADES (EXISTING)**



Upsize 490 m of existing gravity sewers along McKenney Avenue, from Liberton Drive to Bellerose Drive, and St. Albert Trail from Bellerose Drive to St. Vital Avenue (connection into NIT), as well as from Liberton Drive to St. Albert Trail, to 450 mm. The grading of the gravity sewers should be adjusted such that the connection into the NIT is a ‘drop’ connection, instead of invert-to-invert. This will help mitigate backwater effects during high HGLs in the NIT, that could cause backups in the upstream pipes.

**GS-02 LIBERTON DRIVE – PHASE 1 (EXISTING)**

Upsize 335 m of gravity sewer along Liberton Drive, from Liberton Park to McKenney Avenue, to 400 mm.

**GS-03 LIBERTON DRIVE – PHASE 2 (EXISTING)**

Upsize 535 m of gravity sewer along Liberton Drive, from Laval Drive to Lennox Drive, to 325 mm. WSP notes that the modelled pipes along this road have a shallow slope, but there is also a drop in the manhole connection south of Langholm Park. The connectivity in this area should be confirmed prior to initiating upgrade designs.

**GS-04 OAKLAND WAY AND OAKLAND DRIVE (EXISTING)**

Upsize 395 m of gravity sewer along Oakland Way and Oakland Drive, to 250 mm.

**GS-05 GLADSTONE CRESCENT (EXISTING)**

Upsize 280 m of gravity sewer (MH 2380 east to MH 1775002 west, upstream of Grosvenor Park) to 250 mm, see [Figure 2-5\(a\)](#). Downstream pipe segments can be kept at 200 mm as the pipe slopes are steeper, as shown in [Figure 2-5\(b\)](#). The upgrade can be extended to MH 1867000 north to maintain a constant sewer size of 250 mm.

**GS-06 GRANGE DRIVE (EXISTING)**

Upsize 460 m of gravity sewer along Grange Drive, from Gervais Road to the Gate Avenue Lift Station, to 400 mm.

**GS-07 GRANDIN ROAD, FROM FORCEMAIN CONNECTION TO GROSVENOR BOULEVARD (ULTIMATE)**

Upsize 600 m of gravity sewer along Grandin Road, from the Gate Avenue forcemain to Grosvenor Boulevard to 400 mm. WSP notes that the sizing of these sewers should be confirmed following investigation into the pump rate from the Gate Avenue Lift Station and forcemain.

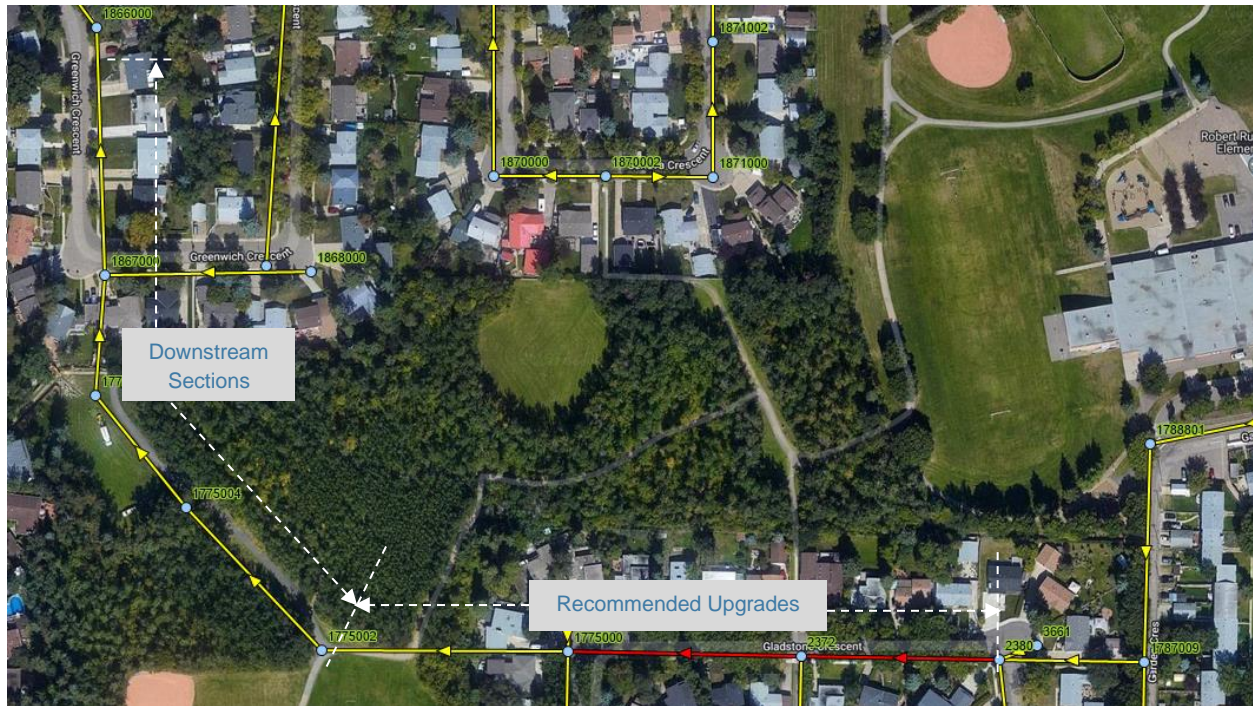
WSP understands that the twinned pipes downstream of Grosvenor Boulevard were recently upgraded, and the engineering design of twinned pipes was selected due to utility congestion in the area. While modelling results show that there could potentially be surcharging issues in the area, this may be due to how the flows were allocated in the model (i.e. the domestic flows may have been loaded unproportionally to one pipe instead of the other) during the spatial allocation. Between the twinned pipes, it is expected that there is sufficient capacity to convey the total simulated flows. However, flow monitoring at the diversion manhole may provide better clarity on the actual flow split.

**GS-08 BELLROSE DRIVE (ULTIMATE)**

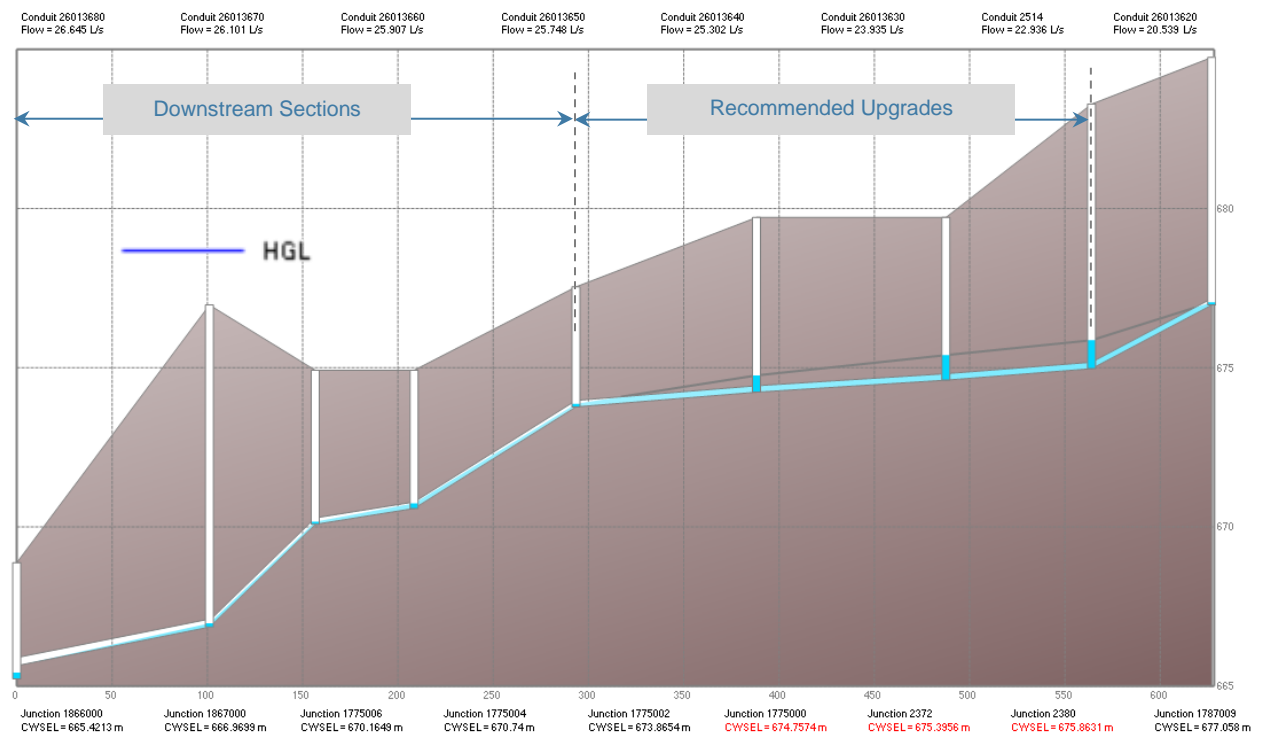
Upsize 285 m of gravity sewer along Bellerose Drive, east of St. Albert Avenue, to 300 mm.

**GS-09 ST. ANNE STREET, UPSTREAM OF TWILIGHT LIFT STATION (ULTIMATE)**

Upsize 580 m of gravity sewer, upstream of the Twilight Lift Station, to 300 mm to accommodate future projected flows into the Twilight Lift Station.



(a) Location of Recommended Pipe Upgrades in Gladstone Crescent Area



(b) Longitudinal Profiles Gladstone Crescent Area

Figure 2-5 Existing System Improvements (GS-05) in Gladstone Crescent Area

GS-10 SHERIDAN DRIVE, FROM SAVOY PLACE TO SCARBORO PLACE (ULTIMATE)



Upsize 385 m of gravity sewer to 250 mm, and 485 m of gravity sewer to 375 mm to accommodate future projected flows.

#### GS-11 KINGSWOOD DEVELOPMENT AREAS (ULTIMATE)

985 m of gravity sewer along the new development parcels in Kingswood (Kingswood Boulevard to Sir Winston Churchill Avenue) should be upsized to 250 mm to accommodate future flows, pending final layout design of the development.

#### GS-12 FALSTAFF AVENUE AND FOREST DRIVE (ULTIMATE)

Upsize 185 m of gravity sewer along Falstaff Avenue and Forest Drive to 250 mm to accommodate future projected flows.

#### GS-13 RIVERSIDE LIFT STATION INLET PIPING (ULTIMATE)

The capacity of the Riverside Lift Station wasn't assessed as part of the existing scenario. The current diameters of the inlet pipes are 525 mm; however, the model simulation shows surface flooding in the inlet pipes of the lift station under the ultimate scenario. The high flows in the inlet pipe may be due to the model having limited gravity sewers modelled within the new development parcels (which would provide some storage). Under the ultimate improvement scenario, upsizing 355 m of piping upstream of the lift station to 650 mm shows a reduction of the peak HGLs to below surface elevation, based on an ultimate pumping rate of 300 L/s at the lift station. A more detailed analysis of the catchment flows is recommended prior to designing the Riverside Lift Station and inlet piping.

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## 2.4.2 TRUNK SEWERS – ORIFICE FLOW CONTROLS

### NIT FLOW CONTROLS (ULTIMATE SCENARIO)

WSP notes that in the existing system scenario, the peak flow into the SAPS is modelled to be approximately 1,700 L/s during a 1:25-year 24-hour storm, which is approaching the noted 2020 design capacity of the facility of 1,800 L/s. The model shows that there is an orifice flow control between the NIT and the SAPS facility, which is currently set to 325 mm. High flows in the downstream sections of the NIT (south of the river) due to high I&I appears to necessitate upstream flow controls along the NIT. Under the existing improvement scenario, I&I rates are assumed to be unchanged, and therefore increasing the opening of the orifice controls along the NIT north of the river is not recommended. Though this may result in higher HGLs in the upstream sections of the NIT, it will help provide some relief for the downstream sections of the trunk sewer. [Table 2-13](#) lists the orifice control size for the existing and ultimate scenarios.

Modelling of the ultimate improvement area shows that orifice control openings in the upstream sections of the trunk sewer could be increased when the I&I rates are reduced throughout the City. Without increasing the upstream orifice controls, modelling shows that the increase in flows from the future development areas will result in very high HGLs in the NIT, that could cause further backups in the Ville Giroux and Riverside areas.

**Table 2-13: Control Orifice Sizes**

ORIFICE CONTROL	LOCATION	EXISTING SCENARIO SIZE (mm)	ULTIMATE IMPROVEMENT SCENARIO SIZE (mm)
Fixed	NIT upstream of SAPS	325	650

Under both the existing and ultimate improvement scenarios, increasing the orifice opening upstream of the SAPS facility from 325 mm to 650 mm appears to relieve high HGL levels in the NIT, while keeping the total inflow into the SAPS below the projected design capacities. Increasing the opening of the orifice could allow for more flows into the SAPS, but the capacity available is also dependent on the flow rates from the New Lift Station. It is currently assumed that the New Lift Station will convey flows from Jensen Lakes and Erin Ridge North; however, if more development lands are added to this catchment area for the New Lift Station, this could limit how much additional flows could be allowed from the NIT into the facility. Once the design flow rate of the New Lift Station is confirmed, a sensitivity analysis regarding the orifice flow controls along the NIT should be conducted to optimize the flows within the NIT such that the SAPS capacity is maximized.

Flow monitoring along the trunk sewer is recommended prior to sensitivity analyses and initiating designs so that they consider actual conditions in the system.

## 2.5 WASTEWATER SYSTEM IMPLEMENTATION PLAN

### 2.5.1 COST ESTIMATE BASIS

This section details the cost estimating approach for improvement projects recommended in this 2020 UMP Update. **Table 2-14** summarizes the unit costs used in the development of the Capital Projects Plan.

The estimated unit costs for the wastewater utility are provided for gravity sewer upgrades, including allowances of 40% for engineering fees and contingency. Construction costs for typical 1200 mm size manholes at 100 m intervals were incorporated in the unit pipe costs. For replacement installations or new constructions in developed areas, the respective unit price comprises excavation, compaction and restoration costs to typical city road conditions. Costs of land acquisitions, drainage easements, right of way access are not included. Costs for other improvement works (I&I reduction and lift station upgrades) are summarized in the attached 2019 I&I Study and 2019 Facilities Study.

**Table 2-14: System Improvements Unit Costs**

ASSET TYPE	SIZE	UNIT	UNIT COST
Wastewater System – Gravity Sewers	250 mm	Lineal meter (m)	\$4,250
	275 mm	Lineal meter (m)	\$4,385
	300 mm	Lineal meter (m)	\$4,490
	325 mm	Lineal meter (m)	\$4,575
	350 mm	Lineal meter (m)	\$4,650
	375 mm	Lineal meter (m)	\$4,795
	400 mm	Lineal meter (m)	\$4,825
	450 mm	Lineal meter (m)	\$5,100
	650 mm	Lineal meter (m)	\$7,045

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## 2.5.2 FUNDING IMPROVEMENTS

Part of the proposed improvements for the wastewater system could be funded by a combination of grant funding, local improvement tax (LIT) and development levies.

Development Levies could cover part of the improvements costs that is directly servicing new developments. However, municipal funds should cover the costs to resolve the issues in the drainage system that serves existing neighbourhoods.

According to the Alberta Municipal Government Act, an LIT may be imposed to raise revenue to pay for a project that the council considers to be of greater benefit to an area of the municipality than to the whole municipality. The LIT is based on several factors including, but not limited to:

1. Financial assistance (grant funding) received from the Crown in right of Canada or Alberta or from other sources.
2. The number of subdivisions (parcels) of land within the affected area of the improvement project.

In all cases, the LIT must be calculated such that each benefitting parcel bears an appropriate share of the updated tax rate. The Government of Alberta and the federal government offer grants to support the planning and development of municipalities. Current grants available include the Municipal Sustainability Initiative, Municipal Asset Management Program, Investing in Canada Infrastructure Program, and Green Municipal Funds.

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## 2.5.3 CAPITAL PROJECTS PRIORITIZATION

A detailed 10-year infrastructure upgrade plan, with a forecast of improvement works to the 2031 horizon for the City of St. Albert, has been developed to lay out an implementation schedule for system improvement works required within existing developed areas. Infrastructure required to accommodate proposed developments in new areas (i.e. trunk sewers and sub trunk sewers) are not included in the prioritization list; the timeline by which these developments are expected to come online (e.g. development staging) has not been included in the current study.

The Capital Projects List, which prioritizes the infrastructure improvements from this study, is broken down by 3-year intervals until 2031. The capital projects are grouped by utility, and broken down by time frame. The prioritized capital list focuses on improvement works within the existing network that require upgrades in order to resolve existing deficiencies and accommodate future flows. Moreover, project implementation priority has been allocated based on the urgency to address the existing system deficiencies and their impacts on the future wastewater system conveyance. Also, the priority level took into account the system review conducted with the City of St. Albert during the Future Ready Workshop and over several follow-up meetings. The staged implementation plans described in this section also considered the wastewater design and assessment criteria established with the City so as to maintain proper level of service and reduce the potential risks of surface and basement flooding, as shown in [Table 2-15](#).

It is worth noting that infrastructure renewal and replacements are not included in the capital projects list. Lift station upgrades and rehabilitation works (i.e. I&I mitigation and reduction) have not been included in this UMP update. Prioritization and cost estimates for these works are detailed in the 2019 Facilities Study and 2019 I&I Study.

**Table 2-15: Wastewater System Improvement Costs and Prioritizations**

ID	DESCRIPTION	PRIORITIZATION	COST
GS-01	St. Albert Trail and McKenney Avenue Upgrades	2021 - 2024	\$2,499,000
GS-02	Liberton Drive - Phase 1	2025 - 2028	\$1,616,375
GS-03	Liberton Drive - Phase 2	2025 - 2028	\$2,447,625
GS-04	Oakland Way and Oakland Drive	2029 - 2032	\$1,678,750
GS-05	Gladstone Crescent	2029 - 2032	\$1,190,000
GS-06	Grange Drive	2021 - 2024	\$2,219,500
GS-07	Grandin Road, from Forcemain connection to Grosvenor Boulevard	After 2032	\$2,895,000
GS-08	Bellrose Drive	After 2032	\$1,279,650
GS-09	St. Anne Street, Upstream of Twilight lift Station	After 2032	\$2,604,200
GS-10	Sheridan Drive, from Savoy place to Scarboro place	After 2032	\$3,961,825
GS-11	Kingswood Development Areas	After 2032	\$4,186,250
GS-12	Falstaff Avenue and Forest Drive	After 2032	\$786,250
GS-13	Riverside Lift Station Inlet Piping	After 2032	\$2,500,975
<b>Total</b>			<b>\$29,865,400</b>

## 2.5.4 OTHER RECOMMENDATIONS

- Update the hydraulic model and resolve anomalies (reverse sloping, etc.).
- Conduct flow monitoring in key areas of the wastewater system to confirm dry and wet weather flows and update the hydraulic model accordingly (e.g. at Grosvenor Boulevard, at trunk sewer connections, etc.).
- Continue with I&I mitigation measures to reduce I&I impacts on the collection system.
- Investigate potential bottlenecks within the lift station and forcemains that are causing the lift stations to perform below their design rates.
- Review the wet weather overflow draining to the Twilight lift station, as discussed previously.
- Conduct feasibility studies for improving redundancy at the lift stations where there is currently no operational redundancy.
- Refine the population projections for existing parcels according to Census information and update the existing hydraulic model accordingly.

## 3 STORMWATER SYSTEM

### 3.1 STORMWATER SYSTEM ASSESSMENT BASIS

This section sets the goals and performance assessment criteria for the existing and future stormwater systems with respect to service requirements, stormwater model assumptions and limitations, model validation, and the stormwater system performance assessment criteria. Figures illustrating the stormwater system characteristics are provided in [Appendix B](#).

#### 3.1.1 STORMWATER SERVICE REQUIREMENTS

The assessment criteria applied in this study was established in the Design Criteria Technical Memorandum (December 2020, WSP) which draws largely from the City's Municipal Engineering Standards (2015) and input received from City technical staff. The assessment criteria for the required capacity of sewer pipes and lift stations was detailed in the Design Criteria Memo. A System Performance Review workshop was held with City staff on December 1<sup>st</sup>, 2020. The purpose of the workshop was to collect operational experience of the City's operational staff, and enhance WSP's understanding of the boundary conditions, key issues, and constraints in the system. Key system observations and issues identified are as follows:

- Flooding: Drainage issues at the corner of Sturgeon Road and Highway 2 as well as a lack of drainage in west ditch at Highway 2 and Coal Mine Road (Petro Canada).
- Outfalls: The City has conducted assessments on outfall structural integrity and functionality. The assessment covered most but not all of the existing outfalls on the Sturgeon River, as some outfalls were not accessible during high-flow events. Furthermore, the Perron Street Bridge and Twilight Station outfalls have required frequent steaming to release (thaw) blockages, and The Perron Street Bridge has experienced surcharge and washout (bridge abutment).
- SWM Ponds: Stormwater management facilities (SWMFs) located in the City's north-end are new and relatively low-maintenance. The Riel Wet Ponds have control structures but lack maintenance manuals to help identify ideal/target pond levels. SWM Ponds require more in-depth assessment with respect to handling algae, etc. The City also stated concerns with the public's perception of SWM Ponds.

The stormwater system assessment criteria was then selected in alignment with the input received at the System Performance Review Workshop, and the City's established and overarching flood mitigation and water quality control objectives outlined in the Municipal Engineering Standards (2015) which outline the following objectives:

- prevent all property damage and flooding and minimize disruption to public activity due to runoff from a 1:5-year return frequency, or more frequent rainfall event.
- prevent significant damage, physical injury, and loss of life due to runoff from a 1:100-year return frequency, or more frequent, rainfall event.
- improve stormwater quality, through filtering of contaminants, prior to discharge to receiving watercourses and prevent erosion of the receiving watercourse.

It is not unusual for the stormwater drainage system in mature neighbourhoods to not have a defined major drainage system. New developments are required to include a dual drainage system consisting of a major overland system and a minor sewer system. In addition, newer stormwater servicing standards require higher storage requirement. Also, Alberta Environment currently enforces lower release rates, adopts higher measures for water quality of stormwater effluents, and recommends different on-site stormwater treatment

options. **Table 3-1** summarizes the criteria applied in the current stormwater system performance assessment.

**Table 3-1: Stormwater System Assessment Criteria**

ITEM	DESCRIPTION
Peak Flows	<ul style="list-style-type: none"> <li>The minor system consisting of underground storm sewers, is to accommodate runoff generated by a 1:5-year rainfall event, and without surcharge for storm sewer mains.</li> <li>The major system, consisting of roadways, ditches and swales, is to accommodate runoff generated by a 1:100-year rainfall event. (particularly within new developments)</li> </ul>
Gravity Sewer Flow Capacity	<ul style="list-style-type: none"> <li>Storm trunk sewers, which are defined as storm sewer mains which serve drainage areas greater than 30 ha, must have capacity to convey flows up to 1.25 times greater than design flows to account for potential future changes in land use.</li> <li>For storm trunk sewers receiving both uncontrolled flow from a drainage area greater than 30 ha and controlled discharge from stormwater management facilities, the storm trunk must have capacity for 1.25 times the uncontrolled design flow plus the design maximum outflow rates from the stormwater management facilities.</li> </ul>
Flow Velocity	<ul style="list-style-type: none"> <li>The flow velocity should not be less than 0.6 m/s and the maximum flow velocity should not be greater than 3.0 m/s. Gravity sewer mains must maintain a mean flow velocity between 0.9 to 1.0 m/s.</li> </ul>
Surface Ponding	<ul style="list-style-type: none"> <li>The minor system must accommodate runoff generated by a 1:5-year rainfall event, and without surcharge for storm sewer mains. The major system must accommodate runoff generated by a 1:100-year rainfall event.</li> <li>The surface ponding depth should not exceed 0.35 m above ground and a maximum of 0.15 m on arterial roadways is permissible where necessary.</li> </ul>
Storage Facilities	<ul style="list-style-type: none"> <li>SWMFs must accommodate the 1:100 year 24-hour storm event and must limit post-development peak runoff flows to 2.5 L/s/ha, with the exception being post development discharge into Carrot Creek which is set to a rate to be confirmed in the Carrot creek report.</li> </ul>
Outlet Controls	<ul style="list-style-type: none"> <li>Stormwater flows from all future stormwater management facilities are to be restricted to the maximum allowable release rates in order to prevent surcharging the downstream system or inducing erosion in the downstream surface water bodies.</li> </ul>
Water Quality	<ul style="list-style-type: none"> <li>To meet Alberta Environment water quality guidelines, future stormwater management facilities should be designed to accommodate the removal of 85% of sediments for particle sizes of 75 microns and larger.</li> </ul>

### 3.1.2 STORM FLOW MONITORING PROGRAM

Flow monitoring (storm and sanitary) and rainfall survey data collected between 2016 and 2020 was provided by the City to facilitate validation and verification of the stormwater model. Exploratory data analysis found the 2017 and 2019 datasets to contain overlapping records for both storm flows and rainfall records and were thus considered in the assessment. Monitoring data comprised of recordings at 5-minute intervals (depth, velocity, and flow) and spanned across 15 flow monitoring locations and 8 rain gauge sites. **Figure B-1** illustrates the areas of the stormwater network where pipe flow was monitored and shows the rain gauge locations. Additional details of monitoring location and available record of the flow monitoring and rainfall data are listed in **Table 3-2** and **Table 3-3**, respectively. Recorded sewer pipe flows over the fifteen monitoring stations and the collected rainfall data over the 2017 and 2019 summer months are shown in **Appendix D**.

**Table 3-2: Stormwater Flow Monitoring Stations (2017 and 2019)**

SITE ID	PROGRAM ID	LOCATION	NODE ID	DIAMETER (mm)
ST-1	2017: ST-1 2019: N/A	St. Albert Trail at Green Grove Drive	60300	525
ST-2	2017: ST-2 2019: N/A	St. Thomas Street by City Hall	62300	900
ST-3	2017: ST-3 2019: A18-017-13	Heritage Lane II at Levasseur Road	19000	900
ST-4	2017: ST-4 2019: N/A	40 Highland Crescent	16000	1,200
ST-5	2017: ST-5 2019: N/A	Forest Drive (MH in trail west of Forest Drive)	118000	1,800
ST-6	2017: ST-6 2019: N/A	Braeside Ravine (behind 43 Brunswick Crescent)	122100	1,650
ST-7	2017: ST-7 2019: A18-017-15	Butterfield Crescent	262800	1,650
ST-8	2017: ST-8 2019: A18-017-16	Canadian Tire Lot (at Bellerose Drive)	J57	1,800
ST-9	2017: ST-9 2019: A18-017-17	20 Meadowview Drive	231300	900
ST-10	2017: ST-10 2019: A18-017-18	26 Desmarais Crescent	173700	1,050
ST-11	2017: ST-11 2019: A18-017-19	Sir Winston Churchill Avenue at Perron Street	61500	750
ST-12	2017: ST-12 2019: N/A	6 Spruce Crescent	69300	525
ST-13	2017: N/A 2019: A18-017-14	11 Beaverbrook Crescent	122800	1,200
ST-14	2017: N/A 2019: A18-017-24	Sturgeon Road (north of Berrymore Drive)	J442	1,000
ST-15	2017: N/A 2019: A18-017-12	Perron Street Bridge	72300	900

**Table 3-3: Rain Gauge Locations (2017 and 2019)**

SITE ID	PROGRAM ID	LOCATION
RG-1	2017: RG-1 2019: A18-017-RG05	Oakmont Reservoir
RG-2	2017: RG-2 2019: A18-017-RG07	Riel Lift Station
RG-3	2017: RG-3 2019: A18-017-RG06	Public Works Yard
RG-4	2017: RG-4 2019: A18-017-RG02	Deer Ridge Lift Station
RG-5	2017: RG-5 2019: A18-017-RG04	Lacombe Park Reservoir
RG-6	2017: RG-6 2019: A18-017-RG08	Salisbury Park
RG-7	2017: RG-7 2019: A18-017-RG01	Alpine Park
RG-8	2017: RG-8 2019: A18-017-RG03	Grosvenor Park

### 3.1.3 MODELLING SOFTWARE

In the assessment of the City of St. Albert stormwater system of the City of St. Albert, three versions of the 1D stormwater model were used and referred to as follows:

- The *Source Model*: the original model received at the start of this study. This version of the model utilized MIKE URBAN version 2017. Both the hydrologic and hydraulic analyses were conducted with the DHI MOUSE computation engine implemented in the software package. This version required updates of newer developments constructed after 2014.
- The *Existing Conditions Model*: this version of the stormwater model was first developed in MIKE URBAN version 2017 and after incorporating the newer developments migrated to the newer version of the software, MIKE+ 2021 update 1. The MOUSE computation engine incorporated in the old City model is no longer supported in the MIKE+ model which only offers the MIKE1D engine. Therefore, all hydrologic and hydraulic analyses were carried out using MIKE1D.
- The *Future Conditions Model*: this model was developed by adding the future development areas to the Existing Conditions Model. All modelling and results presentation is based on the MIKE+ 2021 software.
- All the results presented in the assessment section and incorporated in the study recommendations are solely based on the simulation results conducted with the MIKE+ model.

### 3.1.4 MODEL VALIDATION PROCESS

The validation process involved comparison of simulated flow hydrographs generated by the stormwater system model to observed flow and rainfall records from various flow meter locations within the drainage

system. For a model to be considered calibrated, the variance between predicted and recorded hydrographs is expected to fall within acceptable limits as summarized below.

Storm flows should be calibrated to be accurate within:

- Peak depth -100mm to +100 mm. In cases of surcharged flow, up to +500 mm is considered acceptable (e.g. backwater effects from trunk sewers).
- Peak flow rate -15 % to +15 %.
- Average volume of flow -10 % to + 20 % over the period for which the observed flows are expected to be accurate.
- In general, the shape of the two hydrographs (modelled versus observed) should be similar and should continue until substantial recession has occurred.

If the above criteria is satisfied for two out of three selected rainfall events (or where a satisfactory explanation can be given for disparity in the results), the model is considered to be verified. It should be noted that due to the inherent limitations of the flow monitoring equipment, a complete verification for all the events cannot be achieved.

For example, accurate data cannot be obtained at low flow depths (e.g. less than 50 mm). This is because an unacceptable disturbance is caused by the flow passing over the depth sensor. Similarly, measurement by the velocity sensor at low flow velocities (e.g. less than 0.2 m/s) can be unreliable.

Furthermore, there may be locations where the criteria cannot be met for any of the rainfall events. Therefore, for this study, as well as the use of the above criteria, overall percentage confidence is assumed to the monitor site, based on the following:

- Correlation between predicted and observed peak flows and depths.
- Correlation between predicted and observed volumes.
- Correlation between overall shape of predicted and observed flows.
- Justification of selected storm events.
- Range of intensities and storm durations.
- Uniformity across rain gauge stations to avoid spatially isolated events.
- Events based on available storm sewer monitoring data from 2017 and 2019 study periods.

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### 3.1.5 STORMWATER MODEL MIGRATION TO MIKE+

The scope of work for the Utility Master Plan update also included updating the stormwater model from MIKE URBAN 2017 to the current version of the software, now named MIKE+. The MOUSE computation engine that was incorporated into the MIKE URBAN software is no longer supported in the MIKE+ software. As the new MIKE+ only utilizes the MIKE1D module for surface runoff routing and pipe hydrodynamic computations, some revisions to get the City model running in the new modelling environment were needed. The computability issues might be triggered by 1) different representation of some databases and system features, 2) tolerance to inconsistencies, 3) numerical instabilities, and 4) Geodatabase configurations and formats, between the MOUSE and MIKE1D computation engines.

Hence, all simulation results presented in this report are derived from the new stormwater model migrated to the MIKE+ software. The model results from the two software packages were compared to determine whether or not they generated comparable predictions under the same hydrological and hydraulic conditions. The performance comparison included different parameters such as link discharge, node water level, flooding, pipe capacity, storage nodes and structures across different neighbourhoods. [Appendix D](#) includes a brief comparison for the results of the MIKE+ and MIKE URBAN models.

WSP examined the results of the two models beyond the comparison mentioned in this report and it was found that the overall predictions of the MOUSE and MIKE1D engines incorporated into the MIKE URBAN and MIKE+ models respectively, were relatively close. Some of the encountered differences in results can be attributed to potential variance in the routing mechanisms and numerical computations. Also, the two models are not identical considering the updates implemented to rectify GIS database errors and discrepancies in the MIKE URBAN model, in addition to fixing the capacity issues for Braeside Ravine cross-sections. The difference in predictions of the two software packages was larger in the major system compared to the minor system. Also, the variance was more obvious near the upstream ends of the system and comparably more moderate further downstream in the system towards the system outlet. Overall, the difference between the results of the two models was found acceptable and the MIKE+ models generated relatively similar results in most cases.

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### 3.1.6 MODEL VALIDATION

The objective of model validation is to compare the model predictions using real rainfall with flow characteristics recorded at selectively located monitoring stations for a period long enough to ideally capture diverse records with diverse magnitude, duration, and pattern. The stormwater model performance was assessed by comparing the model predictions under selected storm events with the observed pipe flows at the respective monitoring stations. The hydrological routing results, such as inflow and runoff volumes, were similar to reference storm events with the same characteristics. The continuity error values over the simulated events were within acceptable ranges. Considering the issues in the collected sewer flow data and the lack of storm records with high return periods, the stormwater system model performance was overall acceptable and representative of the real storm system.

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### 3.1.7 STORMWATER SYSTEM ASSESSMENT CRITERIA

This section details the stormwater system performance assessment, including the application of the assessment criteria to the minor system and major systems conveyance capacities, surface ponding, and capacities and release rates of the stormwater management facilities within the system.

#### 3.1.7.1 STORMWATER ASSESSMENT EVENTS

Given the above-mentioned issues with the flow monitoring data, three simulation scenarios were adopted to assess the performance of the stormwater network under standardized worst-case conditions. The design events are based on Edmonton City Centre Airport rainfall IDF Curve (1914 – 1995) per the City of St. Albert Municipal Engineering Standards (2015) and are summarized in [Table 3-4](#). The stormwater system's capacity will be assessed under selected statistical storm events:

**Table 3-4: Assessment Design Events**

DISTRIBUTION	RETURN PERIOD (year)	STORM DURATION (hr)	INTENSITY (mm/hr)	TOTAL DEPTH (mm)
Chicago	5	4	9.27	37
Chicago	100	4	17.23	69
Huff	100	24	5.29	127

### 3.1.7.2 MINOR SYSTEM SURFACE PONDING

The maximum hydraulic grade line (HGL) during a specific storm event is compared to the ground surface at each node in the stormwater system. The difference between the ground surface elevation and the elevation of the maximum HGL is categorized into three groups, as shown in Table 3-5. The potential for surface ponding depth is determined by the stormwater model predictions for water elevation at different nodes in the minor system. Flooding from the minor system nodes is taken into the major system. The stormwater simulation model stores the flooding volumes in the corresponding major system nodes, where they will either flow along the major system or drain back into the minor system when the capacity becomes available.

**Table 3-5: Surface Ponding Assessment**

ASSESSMENT	SURFACE PONDING DEPTH (mm)
Low Risk → Desirable	Value $\leq 0$
Medium Risk → Acceptable	$0.35 \geq \text{Value} > 0$
High Risk → Unacceptable	Value $> 0.35$

### 3.1.7.3 MINOR SYSTEM CAPACITY

The minor system pipe capacity ratio is determined by the  $Q_{\max}$  to  $Q_{\text{full}}$  ratio, the ratio of the maximum simulated flow over the full pipe (pipe capacity) flow rate for each link in the stormwater system. Capacity ratios are categorized into three groups as shown in Table 3-6. In the stormwater simulation model, the full discharge of a link is estimated by applying the full manning flow and is referred to as  $Q_{\max}$ . When the capacity ratio is greater than one, a pipe is surcharged at a rate that is much more than the design capacity. Noting that the acceptability of flows with a ratio between 100% and 150% is also dependent on the elevation of the HGL relative to the ground elevation, as shown in Table 3-6.

**Table 3-6: Pipe Capacity Assessment**

ASSESSMENT	$Q_{\max} / Q_{\text{FULL}}$
Low Risk → Desirable	Value $\leq 1.0$
Medium Risk → Acceptable	$1.5 \geq \text{Value} > 1.0$
High Risk → Unacceptable	Value $> 1.5$

### 3.1.7.4 PIPE VELOCITY

The maximum flow velocity should not be less than 0.6 m/s and should not be greater than 3.0 m/s for optimal flow conditions that prevent sedimentation or scouring, respectively. Maximum velocity values are categorized as shown in [Table 3-7](#).

**Table 3-7: Pipe Capacity Assessment**

ASSESSMENT	MAXIMUM VELOCITY (m/s)
Sediments Risk → Undesirable	Value $\leq$ 0.6
Low Risk → Acceptable	$3.0 \geq$ Value $>$ 0.6
Scour Risk → Unacceptable	Value $>$ 3.0

### 3.1.7.5 MAJOR SYSTEM MAXIMUM DEPTH

The water depths of the major system are indicative of potential surface ponding. Surface ponding on arterial roads should not exceed 0.15 m and elsewhere the ponding should not exceed 0.35 m. Hence, the major system was categorized into three groups, as shown in [Table 3-8](#). The stormwater simulation model calculates the maximum water depth for a given storm event and does not typically allow surcharging of open conduits.

**Table 3-8: Major System Ponding Assessment**

ASSESSMENT	MAXIMUM PONDING DEPTH (m)
Low Risk → Desirable	Value $\leq$ 0.15
Medium Risk → Acceptable	$0.35 \geq$ Value $>$ 0.15
High Risk → Unacceptable	Value $>$ 0.35

## 3.2 EXISTING STORMWATER SYSTEM

The following section details the characteristics of the existing stormwater system, including a description of the existing drainage features, the updates made in the source stormwater model, simulation scenarios, simulation modelling results, system performance assessment, identified system deficiencies, recommended system improvements and prioritization based on the City goals for level of service. All figures presenting the existing stormwater system, its performance and recommended improvements are attached in [Appendix B](#).

### 3.2.1 EXISTING STORMWATER SYSTEM DESCRIPTION

The Sturgeon River provides a natural destination for surface runoff for most of the drainage from the City of St. Albert. It runs diagonally from its source in Big Lake, located at the southwest corner of the City of St. Albert, and then travels through the City toward its northeast boundary. Higher ground on both sides of the river naturally drains into it through a series of constructed outfalls, refer to [Figure B-2](#) for the topography of the City.

The typical stormwater management system consists of two different systems, the major (overland) and minor (underground) systems. A major drainage network usually incorporates roadways, ditches, gutters, urban streams, floodways and floodplains. In the City of St. Albert, the most mature neighbourhoods were developed before the requirement for well-established major drainage systems and stormwater management facilities. However, the major system exists along various roads in the City and is more prevalent in newer neighbourhoods, see [Figure B-3](#) for the major system in the City of St. Albert.

Stormwater management facilities (SWMF) provide storage during high-intensity storm events, reduce system requirements downstream, control stormwater release rates and improve water quality. Following the recommendations of the 2008 Master Plan Study, the number of SWMFs has increased considerably in the City, primarily within newly constructed neighbourhoods such as Riverside, North Ridge, South Riel and Erin Ridge. Existing stormwater ponds in the City are shown in [Figure B-4](#).

Other underground components provide much of the basic conveyance of the stormwater drainage system such as sewer pipes, manholes and system structures. A map of the minor system is shown in [Figure B-4](#).

### 3.2.2 SOURCE STORMWATER MODEL REVIEW

The model received from the City of St. Albert was developed using the MIKE URBAN software package by DHI, as previously elaborated in [Section 3.1.3](#). The Source Model contained the main features of the stormwater system, including both the major and minor systems based on the sewer system GIS database.

The City's drainage area was delineated into sub-basins and catchments using built-in spatial analysis tools to outline surface runoff catchments to respective storm system outlets. Catchments were generally connected to the closest major system node. The major drainage system generally runs parallel to the stormwater pipe network. The majority of the system was defined using typical roadway cross-sections from the 2013 City of St. Albert Engineering Standards. Some generic cross-sections were also employed to represent more natural open overland flow paths in the system. The minor drainage system modelled only includes pipes larger than 150 mm in size. The connection between the major and minor systems was formulated as orifices representing different curb inlets and catch basins that connect the two systems in the storm network. Storage of existing SWMFs was defined based on available design drawings. Wet ponds were modelled above the NWL, and their storages were defined accordingly. The modelled extents of major and minor systems are shown in [Figure B-3](#) and [Figure B-4](#).

Verification of the existing stormwater system components and their data attributes were not part of the current study. Such information was assumed accurate unless otherwise evident based on reviewing as-built plans or other model data.

A brief summary of the additions to this model in the various neighbourhoods follows.

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### 3.2.3 STORMWATER SYSTEM UPDATES

Stormwater infrastructure constructed after the 2013 Utility Master Plan Update was added to the City's Source Model, including storm sewers, major system features and stormwater management facilities. The model was updated using available record drawings and previous study data. The SWMFs included in the hydraulic model are shown on [Figure B-4](#). System configuration for new sewer pipes and structures was based on as-built drawings and respective information provided by the City. The system update areas, based on the corresponding neighbourhood boundaries, are shown in [Figure B-5](#) and are as follows:

- Erin Ridge North, including a new Storm Trunk and Sturgeon River Outfall
- Jensen Lakes (Villeneuve Road and at St. Albert Trail)
- Riverside
- South Riel

The model updates completed, and the data sources used (including assumptions and limitations), are summarized below. The update process was primarily done through the use of record drawings and GIS datasets provided by the City. Confirmation of the stormwater network attributes outside of the areas identified in [Figure B-5](#) was not included in this study. However, WSP has conducted a high-level review of these areas for the purposes of integrating the new inputs into the existing model. Where incomplete information exists, specifically with respect to conveyance infrastructure (e.g. the northern section of Erin Ridge North), WSP has included hydrological components (i.e. sub-catchments) to adequately consider runoff flows entering the system at the appropriate downstream receiving node.

- Minor and Major Storm Systems: Minor system elements updated in the stormwater system model included manholes/catch basin manholes and conduits. Individual catch basins and catch basin leads were omitted. The Major system update included overland flow routes and catch basin inlets. Overland flow routes situated on roadways applied transect cross-sections consistent with the corresponding road classifications (Table 3.1 in Municipal Engineering Standards (2015)).
- Surface (Rim) Elevations: GIS shapefiles (manholes, gravity sewers, etc.) representing the existing stormwater network were provided by the City. With respect to stormwater assets that were either recently constructed or not considered in the last model update, WSP found that elevation measurements in the GIS records were limited and often missing. To address this data gap, record drawings and as-builts were used to populate the missing rim and invert elevations for system elements built after 2017.
- Sub-catchment Areas: For model nodes located in new development areas (e.g. Jensen Lakes and Erin Ridge North), sub-catchments have been delineated in GIS using the 1.0 m St. Albert Digital Terrain Model (2018). Since earthworks were not yet completed in 2018 for these areas, some catchment areas may be under or overestimated; however, Sub-catchments were checked visually for any discrepancies and modified.
- Land Use: Land use characteristics were determined based on a combination of engineering drawings, such as as-built records, and datasets obtained through the City of St. Albert Open Data Portal. Specifically, datasets obtained through the Open Data Portal consisted of district mapping and municipal by-laws to inform sub-catchment imperviousness, a key consideration in modelling runoff.



## ERIN RIDGE NORTH

### INTERIM SWMF

An interim SWMF was built to service the initial stages of development beginning with Erin Ridge Stage 8. In the ultimate configuration, the SWMF will be expanded to service all remaining stages of Erin Ridge North, north of Neil Ross Road. While development is not yet complete, the entire Erin Ridge North area was captured in the model and the ultimate SWMF was represented to capture and retain flows. A full hydrologic/hydraulic system is represented where construction drawings and/or City-provided GIS data was available. The remainder of the land area was captured using hydrological routing only (i.e. Sub-catchment routing).

### PUMP STATION

A new pump station in Erin Ridge currently under design located along Element Drive North, north of Neil Ross Road. It is assumed that the entirety of the flows generated from Erin Ridge North, north of Neil Ross Road, will be collected in the ultimate SWMF and directed to the pump station through a 1,050 mm pipe. Flows from outside of Erin Ridge North are also planned to be pumped through the pump station but are outside of the bounds of this study and therefore not included. Ultimate design of the pump station is incomplete at this time. A basic pump curve has been assumed at 2.5 L/s/ha of the upstream area. From the pump station, a 700 mm force main carries flows to the Erin Ridge North Stage 3 SWMF south of Neil Ross Road.

### ERIN RIDGE STORM TRUNK AND OUTFALL

A new storm trunk, currently in the design phase, will convey flows from Erin Ridge North at Eastgate Way and Coal Mine Road to the Sturgeon River. The 1,200 mm trunk is represented in the model based on the preliminary trunk alignment and concept plan (WSP, 2020). Invert elevations were interpolated between the outfall elevation (651 m) and the connection at Eastgate Way and Coal Mine Road (679.76 m), resulting in an average slope of 1.56%.

## JENSEN LAKES

A full hydrologic/hydraulic system is represented where drawings and/or City-provided GIS data was available. Future areas are to be included as development progresses.

## RIVERSIDE

A full hydrologic/hydraulic system is represented where construction drawings and/or City-provided GIS data was available. Future areas are to be included as development progresses. A SWMF located north of Rankin Drive was incorporated into the stormwater model.

## SOUTH RIEL

A full hydrologic/hydraulic system is represented where construction drawings and/or City-provided GIS data was available. The South Riel area has been represented in the model in its entirety, including updates to the two-pond SWMF located south-east of Ray Gibbon Drive and LeClair Way.



## ST. ALBERT TRAIL CORRIDOR DEVELOPMENT

Within the development limits of the North St. Albert Trail Corridor Plan (Phase 2 & 3), the existing St. Albert Trail is a 4-lane divided arterial roadway with a rural cross-section. The proposed new design concepts for St. Albert Trail from Boudreau/Giroux Road to the north City limit envision the evolution of the corridor as a modern 6-lane urban boulevard that accommodates commercial developments, commuter roads, and pedestrian and cycling paths. In addition, the corridor should also include Bus Rapid Transit (BRT) in the medium term and Light Rail Transit (LRT) in the long term. WSP was retained to review the effects of the proposed development of the St. Albert Trail Corridor on the downstream drainage system, particularly for the SWMFs in Element Park and Edgewater Park within the Erin Ridge Neighbourhood.

Utilizing the City's stormwater model, no changes were found to the flooding status of the existing stormwater ponds and downstream sewer trunks down to the system outfall on Sturgeon River. However, minor changes in water levels and volumes of the receiving stormwater systems downstream were evident, but do not raise concerns in the short term. The downstream trunk system can experience a limited increase in HGL, not causing significant changes to system capacity or flooding. Also, considering the new Sturgeon River stormwater outfall (Project 5, currently under design), the response of the local and extended stormwater system is expected to improve. More information about the existing stormwater system capacity assessment for the St. Albert Trail proposed development can be found in [Appendix C](#).

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### 3.2.4 MODEL ASSUMPTIONS AND LIMITATIONS

The following assumptions and limitations apply to the capacity assessment of the City's stormwater system:

- The stormwater model for the City of St. Albert is bound by the limitation of 1D modelling methodologies in urban settings and consequently, its results are subject to the same restrictions.
- The model was updated according to GIS information available, supplemented by record drawings provided by the City. In some cases, gaps between the different information sources resulted in orphaned sections within the stormwater model, such as the Erin Ridge North Interim SWMF. It is expected that the information for some pipe segments may not have been updated in the GIS database. WSP notes that these areas represent small sections of upstream areas, where capacity issues are not expected. These isolated and disconnected sections have not been included in the model update.
- In applying the rainfall data records from the flow monitoring program as a hydrological input in the stormwater model, data of all rain monitoring stations was set to True. Thus, the runoff computation engine applied the specified boundary condition using the principle of geographical proximity to the catchment's centre point, which is similar to the Thiessen Polygon Method.
- The kinematic wave routing method was adopted for all runoff computations in which GIS-based applications were used to estimate some catchment properties such as length and imperviousness. Other module constants were set to the default values of the City of St. Albert, as provided with the model data.
- The dynamic wave routing method was used for all network computations in which the respective time step settings were set to match the default settings in the model with minimum and maximum time steps set to 5 and 10 seconds, respectively.
- The runoff routing parameters were kept to the default values for the City of St. Albert set in the Source Model, as listed in [Table 3-9](#).
- Pipe roughness assumptions and transect definitions followed the set of assumptions and naming convention of the source model.

- The stormwater modelling results are subject to the level of accuracy regarding the major system inclusion in various neighbourhoods and the assumed parameters for modelling its connection to the minor system.
- Sturgeon River flood levels were set to the 1:5-year levels as per the St. Albert Flood Hazard Study (2020). As there is no direct relation between the return period of the applied storm event in the stormwater model and the return period of river flood, these levels were kept constant for all simulation events.

**Table 3-9: Runoff Computations: Default Perviousness Parameters**

MODEL PARAMETER		IMPERVIOUS COVER		PERVIOUS COVER		
		Steep	Flat	Low	Medium	High
<b>Infiltration losses</b> (mm)	Wetting	1.0	1.0	1.0	2.0	4.0
	Storage	--	3.2	6.4	6.4	6.4
<b>Infiltration Capacity</b> (mm/hr)	Maximum	--	--	4.0	40.0	80.0
	Minimum	--	--	3.6	7.2	18.0
<b>Horton Exponent</b> (1/hr)	Wet conditions	--	--	4.14	4.14	4.14
	Dry conditions	--	--	0.018	0.036	0.18
<b>Manning Roughness</b>	n	0.013	0.017	0.1	0.1	0.25

### 3.2.5 EXISTING SYSTEM MODELLING RESULTS

This section presents the simulation results of the existing stormwater system model. Performance assessments for the major, minor systems, and SWMFs are discussed in terms of flooding potential, deficiency, level of service and capacity. The stormwater simulations were executed for the minor system in conjunction with the major system allowing for the flow to be conveyed between the two systems. Thus, despite adopting a 1D modelling approach in the City's stormwater model, the connection between the two systems provides a more accurate representation of the actual performance of the stormwater management system.

Potential remediations and recommendations for problematic areas identified will be addressed in the following section.

#### 3.2.5.1 SIMULATION SCENARIOS

The existing storm sewer and major drainage systems were analyzed for the 1:5 year 4-hour and 1:100 year 4-hour Chicago storm events and the 1:100 year- 24-hour Huff storm event. The selected return periods align with the performance requirements of the minor and major systems, respectively, as outlined in the Municipal Engineering Standards (2015). The 4-hour duration storm events are mainly used to analyze the overall hydraulic performance and flood potential, particularly in terms of pipe capacity and surface ponding in both the major and minor systems. While the 24-hour storm event is primarily used to assess the storage requirements in the system, including the review of SWMF performance as well as release rates. The results summary of the model simulations is listed in [Table 3-10](#), where the continuity volume differences, and balance errors were deemed acceptable.

**Table 3-10: Existing Condition Model Simulation Results Summary**

DESIGN STORM	1:5-YEAR 4-HOUR	1:100-YEAR 4-HOUR	1:100-YEAR 24-HOUR
<b>Total Rainfall (m<sup>3</sup>)</b>	1,306,369	2,422,813	4,435,681
<b>Continuity Difference (m<sup>3</sup>)</b>	-19,838	18,456	41,515
<b>Continuity Error (%)</b>	-1.51	0.76	0.93
<b>Surface Runoff (m<sup>3</sup>)</b>	558,692	1,431,907	1,930,723
<b>Runoff (%)</b>	42.8	59.1	43.3

### 3.2.5.2 SURFACE PONDING

The maximum surface ponding depth was simulated for the St. Albert stormwater network under the 1:5-year 4-hour and 1:100-year 4-hour design storm events. Referring to [Figure B-6](#) for the 1:5-year Chicago storm, the minor stormwater system performed relatively well, experiencing limited surface ponding over the City. Some locations encountered higher ponding depths (>0.35 m) in the neighbourhoods of Downtown, Mission, Sturgeon Heights, Braeside and Lacombe Park. Also, in Akinsdale, the commercial area at the intersection of St. Albert Trail and Hebert Road experienced surface ponding with the HGL above ground at many manholes in the area.

For the 1:100-year event, most neighbourhoods in the City of St. Albert appear to have varying levels of surface ponding depth with the HGL above ground at many locations, as shown in [Figure B-7](#). Higher surface ponding (>0.35 m) occurred at many locations within the Grandin, Sturgeon Heights, Akinsdale, Lacombe Park, Mission, Braeside and Lacombe Park neighbourhoods. The lack of a well-defined major system played a role in the minor system performance in some areas within the Mission, Sturgeon, Downtown and Braeside neighbourhoods. Sag areas in Campbell Business Park and the Deer Ridge neighbourhoods. Higher surface ponding (>0.35 m) occurred at many locations within the Grandin, Sturgeon Heights, Akinsdale, Lacombe Park, Mission, Braeside and Lacombe Parks. The lack of a well-defined major system played a role in the minor system performance in some areas within the Mission, Sturgeon, Downtown and Braeside neighbourhoods. Sag areas in Campbell Business Park and the Deer Ridge neighbourhoods encountered flooding issues due to lower ground levels at these locations.

[Figure B-8](#) and [Figure B-9](#) depict the major system performance under the same events with 0.35 m set as the critical surface ponding zone limit, as per the St. Albert Municipal Engineering Standards (2015). The simulated flooding in the major system was minimal over the 1:5-year storm event, with some individual links exhibited ponding > 0.35 m in scattered throughout the City. For the 1:100-year 4-hour Chicago simulation, surface ponding > 0.15 m occurred for many major system links in almost every neighbourhood in the City of St. Albert. Higher surface ponding and, in turn, susceptibility to flooding were found in Lacombe Park, Sturgeon Heights, Mission, Jensen Lakes, Deer Ridge and Grandin.

### 3.2.5.3 PIPE CAPACITY

The stormwater model was used to estimate the peak flows in the stormwater system for the 1:5-year 4-hour and 1:100-year 4-hour design storms. [Figure B-10](#) and [Figure B-11](#) illustrate the results of the pipe capacity assessment of the stormwater system for those storm events. The pipes with capacity ratios ( $Q_{\max}/Q_{\text{full}}$ ) greater than 1.5 are highlighted in red, pipes with a ratio between 1.0 and 1.5 are highlighted in yellow, and sewer pipes with a ratio of 1.0 or less are highlighted in green.

For the 1:5-year event, most neighbourhoods exhibited local capacity issues with some pipe stretches running over capacity ( $Q_{\max}/Q_{\text{full}} > 1$ ), without significant effects on the overall system conveyance. The service level of some sewer pipes within the Downtown, Grandin, Sturgeon Heights, Akinsdale, Lacombe Park, Mission, and Campbell Business Park was less than the 1:5-year rainfall event as the pipe capacity ratio was  $\geq 1.5$ . For the 1:100-year 4-hour storm event, a pipe capacity ratio  $\geq 1.5$  occurred for many sewer pipe segments in various neighbourhoods throughout the City. The most vulnerable communities include Campbell Business Park, Lacombe Park, Downtown, Riverside and Akinsdale.

### 3.2.5.4 PIPE VELOCITY

The maximum pipe velocity was estimated for the St. Albert stormwater network using the 1:5-year 4-hour and 1:100-year 4-hour design storm events. Referring to [Figure B-12](#) and [Figure B-13](#), the velocity is colour-coded to distinctly illustrate the low-velocity areas ( $< 0.6$  m/s) and the high-velocity areas ( $> 3.0$  m/s), following the St. Albert Municipal Engineering Standards (2015).

The system had pipes which exceeded the maximum velocity limit of 3.0 m/s for both the 1:5-year and 1:100-year events. The high-velocity sewer pipes generally occurred in trunks leading to system outlets on the Sturgeon River at Meadowview Lane, Boudreau Road, Otter Crescent and Kingswood Drive. Downtown, Grandin, Sturgeon Heights, Lacombe Park, Mission, and Riel Business Park also showed high velocity in their sewers. In addition, the Inglewood and Oakmont areas seemed to have some local cases with high maximum flow velocities.

### 3.2.5.5 ASSESSMENT OF STORMWATER PONDS

The performance assessment of the stormwater ponds was conducted using the 1:100-year 24-hour Huff design storm as per the requirement of the St. Albert Municipal Engineering Standards (2015). The simulation results of the stormwater model were used to estimate the release rates of each stormwater pond, as shown in [Table 3-11](#). The total catchment area was estimated based on the connectivity of the sewer system upstream of each basin element as per the stormwater network defined in the stormwater model. Further, the maximum discharge over the simulated period was obtained for the outlet controlling structure(s) downstream of the stormwater pond. For some locations where there were no controls directly connected downstream, such as Riverside 5 and Heritage Lake 3, the first pipe link downstream of the stormwater pond was used instead.

Four new ponds were added to the source model received from the City of St. Albert. All ponds were modelled as wet ponds with initial water depth at the NWL of each pond, as per the record designs. In Riel South, the two ponds (Riel South 2A and 2B) are hydraulically connected through a 1200 mm pipe with a control manhole downstream of Riel South 2A. The invert level of the controlling rectangular orifice was set at 662.2 m and the crest level of the overflow weir at 664.25 m. The maximum release rate of the control orifice (1.66 L/s/ha) was less than the allowable limit of 2.5 L/s/ha. The Riverside 5 pond serves the drainage area of Riverside Stage 20 and part of Stage 9A, along with the collected upstream runoff through Riverside 4 pond. However, the Riverside 5 pond discharges directly to a 2.4x3.5 m box culvert, resulting in a release rate of 8.05 L/s/ha. The Erin Ridge North pond, Erin 3, was controlled with a 284 x 284 mm rectangular orifice and the NWL of the pond was set at 679.2 m, the discharge of this pond was ultimately pumped to the sewer system. Over the simulated event, the release rate of the Erin 3 pond peaked at 1.37 L/s/ha. No flooding was encountered at any of the four stormwater ponds.

The release rate of the existing ponds in the stormwater system varied in the 1:100-year 24-hour simulation, with some ponds exceeding the maximum allowable release rate, as shown in [Table 3-11](#). This table demonstrate a comparison between the source and existing conditions model results for the 1:100-year 24-hour storm event, in which the HWL, outflow, and release rate of each pond is indicated. The comparison shows that the outflow and release rates of both models were relatively close for all stormwater ponds. Some ponds such as Riverside 3, Lacombe Park 1, Erin 1, and North Ridge 3 and 5 generated higher flow rates in the existing conditions model compared to the source model, but they are still at or below the allowable rate of 2.5 L/s/ha.

Due to interactions in the stormwater system, the two ponds of Erin Ridge North Stage3 and North Ridge 5 showed lower release rates as a result of lower high-water levels (HWLs). The release rates were high compared to the allowable release limit of 2.5 L/s/ha for both the source and existing models for a group of stormwater ponds including Heritage Lake 3, Campbell 1, North Ridge 4 and Riverside 4. At Heritage Lake 3, the significantly high release rate can be attributed to the lack of controls downstream of the storage node which allowed the basin to act as a surge tank in the stormwater network. Over the 1:100-year Huff simulation, Heritage Lake 3 received water from the downstream sewer system at the beginning of the event, then later when the water levels in the pond started to increase and forced flow in the normal direction to the downstream trunk.

Further, the differences in HWL between the source and existing condition models ranged between 0.09 m at Riel South 1 and 0.61 m at Erin 1. Further analysis can be conducted to assess the performance of existing stormwater ponds, and to consider different options on a case-by-case basis, with the addition future developments to the system.

**Table 3-11: Existing Conditions Modelling Results for the Stormwater Management Facilities**

Stormwater Pond	POND PROPERTIES <sup>1</sup>					EXISTING MODEL				SOURCE MODEL			
	Drainage Area (ha)	Bed Level (m)	Rim Level (m)	Design Storage (m <sup>3</sup> ) <sup>2</sup>	Type	HWL (m)	Outflow (m <sup>3</sup> /s)	Release Rate (L/s/ha)	Storage (m <sup>3</sup> ) <sup>4</sup>	HWL (m)	Outflow (m <sup>3</sup> /s)	Storage (m <sup>3</sup> ) <sup>4</sup>	Release Rate (L/s/ha)
Riel South 2A <sup>1,2</sup>	56.03	659.20	664.60	20,200	Wet	663.71	0.093	1.66	13,800	N/A	N/A	N/A	N/A
Riel South 2B <sup>1,2</sup>	53.71	659.20	664.60	41,700	Wet	663.71	0.628	11.69	29,500	N/A	N/A	N/A	N/A
Riverside 5 <sup>1,3</sup>	561.95	651.70	656.57	2,600	Dry	655.40	4.524	8.05	1,250	N/A	N/A	N/A	N/A
Erin 3 <sup>1</sup>	58.19	676.40	681.80	60,800	Wet	680.65	0.080	1.37	40,500	N/A	N/A	N/A	N/A
Heritage Lake 1	50.76	677.19	681.05	44,700	Wet	679.26	0.106	2.09	27,400	679.24	0.106	27,200	2.09
Jensen Lake 1	65.81	683.44	687.00	31,400	Dry	684.86	0.200	3.04	8,100	684.72	0.200	6,900	3.04
Riverside 3	45.11	651.53	652.24	16,600	Wet	651.99	0.045	1.00	13,200	651.68	0.009	10,200	0.20
Riverside 4	551.49	654.11	656.01	24,000	Wet	655.74	4.424	8.02	18,700	655.61	4.942	14,600	8.96
Lacombe Park 1	75.31	658.10	662.60	143,800	Wet	660.19	0.068	0.90	127,100	660.15	0.003	125,400	0.04
North Ridge 5	176.10	676.40	677.00	12,300	Wet	676.60	0.085	0.48	8,800	676.55	0.047	8,200	0.27
North Ridge 4	166.65	676.40	678.00	65,100	Wet	677.50	1.570	9.42	46,500	677.41	1.519	44,600	9.11
North Ridge 3	73.54	679.30	681.20	80,400	Wet	679.95	0.184	2.50	41,500	679.82	0.166	38,700	2.26
North Ridge 2 <sup>1,2</sup>	35.97	681.20	682.70	34,700	Wet	681.61	0.069	1.92	20,200	681.52	0.064	19,300	1.78
North Ridge 6 <sup>1,2</sup>	15.98	681.20	682.70	46,800	Wet	681.61	0.030	1.88	27,600	681.53	0.033	26,500	2.07
Riverside Temp	70.23	653.70	656.60	9,200	Wet	654.83	0.621	8.84	3,400	654.76	0.568	3,200	8.09
Heritage Lake 2	51.44	671.00	673.00	15,200	Wet	671.79	0.076	1.48	13,800	671.74	0.076	13,500	1.48
Ray Gibbon 2	40.72	651.00	652.91	44,700	Wet	652.30	0.100	2.46	29,800	651.91	0.099	22,400	2.43
Grandin 1	25.77	681.73	683.63	42,000	Wet	683.16	0.087	3.38	8,900	683.04	0.082	6,800	3.18
Campbell 1	44.78	684.32	686.30	25,700	Dry	685.84	0.377	8.42	13,100	685.76	0.369	11,800	8.24
Riel 2 <sup>3</sup>	228.86	651.80	653.80	74,500	Wet	652.60	0.540	2.36	35,900	652.58	0.512	35,400	2.24
Oakmont 1	171.13	654.00	655.10	18,200	Dry	655.04	0.199	1.16	1,900	655.01	0.168	1,800	0.98
Erin Ridge 3	225.38	681.89	683.00	21,100	Wet	682.55	0.204	0.91	21,700	682.51	0.204	21,400	0.91
Erin Ridge 1	11.56	679.30	683.81	5,100	Dry	679.48	0.040	3.46	57	679.55	0.077	100	6.66
Riel South 1	124.91	653.20	656.30	33,100	Wet	654.05	0.323	2.59	4,700	654.14	0.321	5,500	2.57
Ern 01	50.63	680.40	682.10	83,300	Wet	681.57	0.083	1.64	66,700	680.94	0.058	55,650	1.15
Ern 02	97.72	682.30	684.30	91,000	Wet	683.18	0.073	0.75	70,000	682.91	0.061	65,000	0.62
Riel 01	242.82	651.80	653.80	145,700	Wet	652.33	0.330	1.36	102,200	652.13	0.313	86,200	1.29
Heritage Lake 3 <sup>3</sup>	0.38	665.94	668.00	7,300	Dry	666.67	0.066	173.68	2,600	666.63	0.133	2,500	350.00
Campbell South <sup>1</sup>	12.81	681.50	685.4	30,600	Dry	684.29	0.066	5.15	26,900	N/A	N/A	N/A	N/A

<sup>1</sup> Newly added to the model

<sup>3</sup> No direct control on the pond outlet

<sup>5</sup> Based on old UMP information

<sup>2</sup> Hydraulically connected SWMFs

<sup>4</sup> Based on the existing modelled SWMF

### 3.2.6 EXISTING SYSTEM IMPROVEMENTS

Based on the analysis of the simulation modelling results, this section highlights some recommended upgrades for the existing stormwater system. These upgrades are needed to achieve a 1:5-year level of service and mitigate the most pressing drainage issues. The primary focus is on areas where the susceptibility for flooding is high and may put the adjacent properties at potential risk. In addition, upgrades provided in the Storm Sewer Improvements report (Focus, 2009) and 2013 St. Albert Utility Master Plan were also investigated to determine the need for their inclusion in the present study. Moreover, prioritization of recommended upgrades or selection of the fittest option can be further enhanced by studying the cost/benefit feasibility.

The following sections provide more detail on the local improvements recommended for those locations.

#### 3.2.6.1 SHERIDAN DRIVE

The stormwater management in the Sheridan Drive area might be affected by the absence of major system links that connect the local area to the major system on St. Albert Trail and, in turn, prevent excessive flooding on private properties. The City has reported excessive water ponding on the roadway and into the lots with the potential for basement flooding. Examination the drainage pattern in this area shows that there might be some ditches missing to drain the Sheridan Drive area properly; possible connections can be placed between lots 72 and 76, or on lots 60 and 62 above the sewer easement, as shown in [Figure B-14](#). Considering there is a noise barrier between these lots and St. Albert Trail, Locations 1 and 2 would require excavating under the noise barrier to allow a major flow route to St. Albert Trail. The source model prepared in 2013 already includes these major system links and the simulation results do not show drainage problems in this area. Therefore, the addition of these links should help to mitigate the documented drainage challenges on Sheridan Drive.

Field investigations can further the understanding of the major system extent in this area and its overland drainage pattern.

#### 3.2.6.2 LACOMBE PARK IMPROVEMENTS

In the Lacombe Park neighbourhood, the 1375 mm storm pipe flowing south on Lancaster Crescent to the storm trunk on McKenny Avenue has obvious capacity issues. Some locations on this stretch of the minor system show HGL levels above the ground surface. Also, the major system in this area indicates a few localized flooding locations with depths above 0.15 m for the 1:5-year and above 0.35 m for the 1:100-year design storms. This storm trunk services different neighbourhoods, Deer Ridge (upstream), and Mission and Riverside (downstream). Consequently, the capacity and flooding problems in this area might be driven by factors outside its immediate drainage area. Moreover, based on available data, the storm trunk seems to be located in a 1.0 m sag on Lancaster Crescent, which should be confirmed in the field prior to making any improvements.

There are two potential options to improve the drainage issues in the minor system and address related problems in the major system as well. First, system performance can be improved by constructing a new surge pond to decrease the HGL levels in the main trunk, either close to Larose Drive or L'Hirondelle Court. Alternatively, a storage pond in Lacombe Park could also improve the sewer system capacity in this area and decrease the need for larger storm pipes downstream. The selection of the appropriate location for the proposed pond depends on the availability of land close enough to the affected areas. Second, there is also potential to improve the system capacity by upsizing the existing trunk sewer in this area down to McKenny Avenue. [Figure B-15](#) shows high-level plans for the proposed projects in the Lacombe Park neighbourhood.

### 3.2.6.3 STORMWATER SYSTEM PLAN IN MISSION NEIGHBOURHOOD

The Mission neighbourhood appears to lack major and minor drainage systems for most areas north of St Vital Avenue and east Mount Royal Drive. The addition of a piped minor drainage system can lessen the flooding risks in the Mission area and increase the system conveyance capacity. Field visits would be needed to verify the extent of an existing major system in the Mission neighbourhood and if any existing features are missing from the GIS database. A high-level concept of the proposed minor system is shown in [Figure B-16](#). Such upgrades can be executed and budgeted over a 10 to 20-year plan.

Moreover, the area west and southwest of Mission Park also lacks a minor drainage system which might be the cause for flooding exceeding 0.4 m in the major system. The potential of flooding can be reduced by adding three new storm pipes along Mount Royal Drive, Mill Drive, and between St Vital and Mission Avenues. Refer to [Figure B-16](#) for the proposed minor system pipe sizes and locations.

### 3.2.6.4 MINOR SYSTEM PLAN IN GRANDIN AND BRAESIDE NEIGHBOURHOODS

Areas within the Grandin and Braeside neighbourhoods do not have a minor drainage system. In Grandin, it is proposed to provide the area east of Grosvenor Boulevard and along Gillian Crescent with a new piped minor system. Similar concepts are applicable to the Braeside neighbourhood, particularly south of Burnham Avenue and west of Bernard Drive. These minor system additions can lessen the flooding risks in both neighbourhoods. High-level concepts of the proposed minor systems in the Grandin and Braeside areas are shown in [Figure B-17](#) and [Figure B-18](#), respectively. The proposed upgrades can be planned and budgeted over a 10 to 20-year plan.

### 3.2.6.5 DEER RIDGE IMPROVEMENTS

There is a major sag area in Deer Ridge in the north part of the neighbourhood bounded by Deer Ridge Drive, Dorchester Drive and Desmarais Crescent. This area needs attention to bring it up to the 1:5-year service level. The existing conditions model currently illustrates flooding problems for a major stretch of system along Dorchester Drive and Desmarais Crescent, with ponding depths over 0.35 m. Moreover, HGL levels are above the ground surface, with several locations of surface ponding evident in the areas adjacent to the storm trunk. The main trunk servicing this area, which flows through Deer Ridge, Lacombe Park and then into the Riverside neighbourhood, is exhibiting notable capacity concerns. Modelling results also show that the sewer pipes are at or above capacity in many locations within the sag area. Overall, this indicates that the sewer system in the area is at capacity and with the proposed future development servicing concept, additional areas north of Villeneuve Road would also drain to this trunk adding to the existing capacity issue, unless RTC is utilized.

The current drainage concerns with this sag area could be relieved by installing a new SWMF north of Villeneuve Road in the new development area west of Jensen Lakes, as shown in [Figure B-19](#). The stormwater pond should accommodate surge volumes flowing by gravity from the Deer Ridge area during large storm events, plus all surface runoff from its future-development catchments. The overflow pipe can also be designed to provide some underground storage during storm events increasing the capacity of the local system. Then, a lift station can be used to drain the proposed SWMF and direct water back to the Deer Ridge system when there is enough capacity. An RTC system could be employed to control the SWMF discharge at a 2.5 L/s/ha release rate after a rainfall event ends. Considering the surface flooding problem, the RTC can be set to eliminate the flooding at the lowest point in the sag (MH 173300 on Desmarais Crescent). In this case, the lift station can start emptying the SWMF when the pipe downstream of that manhole can accommodate the pumped flow plus approximately 10% capacity as a margin for travel time and freeboard. Moreover, as some storm trunks (in red) on Giroux Road are already showing capacity issues, the RTC system should only allow releases when the receiving downstream system along Giroux Road has available capacity. These recommendations are consistent with the 2008 and 2013 St. Albert Utility Master Plans. Characteristics of the proposed SWMF, lift station and sewer system are pending the

final layout for the stormwater area north of Villeneuve Road. Additional study will be required to investigate and determine the exact RTC settings to manage the local sewer system's surface flooding and pipe capacity.

### 3.3 FUTURE STORMWATER SYSTEM

The future growth areas and associated development prospects were estimated based on the 2042 Development Horizon, Area Structure Plans, and full development to the current City Limits, hereafter referred to as the Ultimate Conditions. The proposed future growth includes development of undeveloped areas around the periphery of the City boundaries. New neighbourhoods are expected to further develop with the construction of planned stages. Also, according to neighbourhoods, and land use changes within the new mature city areas, neighbourhoods were estimated based on the growth projections by the City and the Municipal Development Plan, the mature city areas are proposed to continue growing through intensification to commercial and mixed land uses. All figures illustrating the future stormwater system characteristics and its assessment results are provided in [Appendix B](#).

Refer to [Figure B-20](#) for the projected ultimate conditions. As illustrated in the ultimate conditions, new development areas are expected along the north boundary of the City extending north of Villeneuve Road from Jensen Lakes west to Carrot Creek, along the west side of St. Albert Trail north to the Carrot Creek tributary, and along Element Drive including the Erin Ridge North area. Additional developments are anticipated along the west boundary of the City between Ray Gibbon Drive and Carrot Creek, and on the south side between Ray Gibbon Drive and Big Lake. Furthermore, newer neighbourhoods that are only partially developed will further develop, including Riverside 1, 2 and 3, Campbell South, and South Riel, to the full development conditions.

#### 3.3.1 ULTIMATE CONDITIONS MANAGEMENT PLAN

In the Ultimate Conditions stormwater model, this study generally followed the drainage schemes provided in the St. Albert Utility Master Plan Update (2013), including the projected drainage areas and storage requirements. Based on the topography and natural drainage pattern of the development area, two options for the ultimate stormwater management plan were investigated. The two alternative drainage plans were mainly based on the available conveyance capacity of Carrot Creek and its related erosion concerns that can limit the release of stormwater flow to Carrot Creek.

The first option entailed utilizing a major storm trunk parallel to Carrot Creek with an outlet to Big Lake. This option would increase the capital cost required to construct the off-site storm trunk and outfall works. The parallel storm trunk would also need to be constructed before any development in the service areas. However, this option would minimize any limitations for storm discharges to Carrot Creek and any potential impacts. In turn, it would reduce the size of proposed SWMFs.

The second option incorporated multiple outfalls to Carrot Creek. If needed, RTC can be utilized to attenuate storm flows to avoid increasing the Carrot Creek peak flow at different release rates. This option will allow development of growth areas independent of downstream development. Another advantage of this option is the lower capital cost needed due to reduced offsite works. But, this option would require larger storage volumes for the SWMFs resulting from the need for lower release rates to accommodate and control storm releases to Carrot Creek.

At the time of compiling this report, the Carrot Creek Study has not finalized all the geotechnical considerations and geomorphology study in order to set the allowable release rate for Carrot Creek. Hence, in the present study, the second option was adopted to represent the future drainage plans with multiple outfalls to Carrot Creek, as shown in [Figure B-21](#). This option was adopted as it provided less dependency

among future growth areas and in turn their developers, no need for prior development of any downstream sections, and reduced capital costs for the City due to lower off-site capital drainage works.

The new drainage areas have been divided into basins based roughly on land parcel configuration and anticipated roads alignments. Each quarter section is generally serviced by a stormwater management facility (SWMF) to limit the total number of ponds and minimize the respective maintenance costs. The SWMFs parameters were estimated to achieve a maximum release rate of 2.5 L/s/ha. The approximate location of these SWMFs is shown in **Figure B-21**. These SWMF locations are conceptual only, based solely on the available topographic data. The exact location, size, and configuration will need to be determined during the neighbourhood planning stage for each individual basin area. Some basin areas along Big Lake that have been designated as environmentally sensitive were excluded and assumed to drain overland directly into Big lake. There are two main drainage areas based on the stormwater management concept plans. The added drainage area in the ultimate conditions can drain (1) directly to receiving surface waters or (2) to the existing sewer system.

The future development areas on the west side of Ray Gibbon Drive can directly discharge to either Carrot Creek, Big Lake or the Sturgeon River, whichever is closer. Examination of downstream boundary condition levels of receiving waters will determine if the outlet control can be designed to work by gravity. At the time of writing this report, the respective Carrot Creek levels and impacts of any additional stormwater flows on the stream were not available.

The ultimate conditions developments north of Villeneuve Road were connected to the existing drainage system at three points in the North Ridge, Deer Ridge and Erin Ridge North neighbourhoods, as shown in **Figure B-21**. Review of the proposed levels for the system elements in basins 28d and 28e revealed that these areas can drain by gravity to the existing North Ridge stormwater system. The total drainage area of the two basins is about 55 ha and its attenuated flows should not present a problem to the downstream system, as later discussed with the modelling results.

Basins 18a, 18b and 19 all have stormwater ponds that provide for runoff storage before discharging to the exiting stormwater system in the Deer Ridge neighbourhood. Due to the existing elevations of the sewer system, the discharge of the downstream SWMF in Jensen Lakes will need to be pumped into the Deer Ridge system. Assessment of the available capacity of the receiving system can determine the need for RTC measures. Basins 16 and 14 are connected to the existing SWMFs in the Erin Ridge neighbourhood, which in turn is connected through a lift station to the existing downstream sewer system. Finally, Basin 13 is connected to the existing system through a proposed SWMF to limit downstream effects. Effects of Basins 16, 14 and 13 should be minimized by the connection to the new Oakmont outfall.

**Table 3-12** lists the characteristics of the Future system basins based on the projected development characteristics and existing land-use patterns, including the basin contributing area, projected ultimate conditions imperviousness ratios, storage volume requirements, and approximate normal and high water levels. Further refinement of the actual design parameters should be considered upon finalizing the individual neighbourhood development plans.

**Table 3-12: Properties of the Future System Basins**

BASIN ID	DEVELOPABLE AREA (ha)	IMPERVIOUS RATIO (%)	REQUIRED STORAGE (m <sup>3</sup> )	HWL (m)	NWL (m)	NWL AREA (ha)
28a	25.59	60.00	25,000	676.07	675.00	2.5
28b	6.10	50.00	6,000	670.64	669.50	0.6
29a	18.73	71.90	22,000	676.67	675.50	2.2
29b	15.85	70.00	18,000	666.62	665.50	1.8
30	62.31	72.10	71,000	667.64	666.60	6.9
31	69.21	72.00	79,000	665.10	664.00	7.7
50	57.29	80.00	72,000	660.11	659.00	7.0
35	82.24	75.50	82,000	653.99	652.90	8.0
36	11.92	50.30	9,000	652.25	650.70	0.9
53	36.13	80.00	45,000	657.12	656.00	4.4
60-39	43.69	50.00	31,000	655.21	654.00	3.0
28c	9.22	60.00	8,000	676.07	675.00	0.8
28d	27.24	60.00	22,500	684.02	682.95	2.2
28e	27.48	60.00	22,500	684.02	682.95	2.2
18b	21.60	50.10	19,000	682.99	681.10	4.5
19	103.65	50.10	86,000	681.29	680.40	11.5
18a	85.00	63.80	87,000	681.99	681.10	11.3
16	41.00	50.00	34,455	682.25	681.50	5.5
14	70.00	50.20	58,483	680.66	679.20	4.6
13	41.00	50.00	34,406	681.47	680.90	4.4
Riverside 1	57.37	55.00	43,500	653.50	655.73	4.3
Riverside 2	21.91	50.00	16,000	651.00	652.00	1.6
Riverside 3	39.33	50.00	28,000	651.00	652.01	2.8
Oakmont	18.37	45.00	13,000	660.00	661.25	1.3
Kingswood	30.46	45.00	20,500	679.70	680.57	2.0

### 3.3.2 FUTURE MODEL ASSUMPTIONS

In addition to the limitations and assumptions described in [Section 3.2.4](#), the following assumptions were applied in the modelling of the stormwater system for the ultimate conditions:

- It is important to note that the future growth areas in the northern portion of St. Albert, including the Jenson Lakes Area and the Erin Ridge areas are expected to be heavily influenced by the future annexation lands located further north. Further analysis is recommended within this area, taking into account the annexation lands, as these will heavily impact the design and management of the proposed RTC systems as well as downstream capacity.
- The existing conditions model was updated according to the future development prospects by the City the stormwater management plans in the 2013 UMP, and available GIS database records.
- High-level designs were prepared for the future sewer systems following the guidelines of Municipal Engineering Standards (2015) for sewer pipe sizing and slopes. The stormwater management system was designed to accommodate the surface runoff of the added areas and allow for proper drainage in the ultimate conditions model.
- The runoff routing parameters of the new developments were kept to the default values for the City of St. Albert set in the Source Model, as listed in [Table 3-9](#).

### 3.3.3 FUTURE SYSTEM MODELLING RESULTS

This section presents the model simulation results of the future stormwater system. Performance assessments for the major system, minor systems, and SWMFs are discussed in terms of flooding potential, deficiency, level of service and capacity. The assessment of the future system performance was mainly conducted by comparing it with the existing condition results.

#### 3.3.3.1 SIMULATION SCENARIOS

The stormwater system was assessed using the 1:5 year 4-hour and 1:100 year 4-hour Chicago storm events and the 1:100 year- 24-hour Huff storm event. The results summary of those simulations is listed in [Table 3-13](#), where the continuity volume differences, and balance errors were within acceptable range.

**Table 3-13: Ultimate Conditions Model Simulation Results Summary**

DESIGN STORM	1:5-YEAR 4-HOUR	1:100-YEAR 4-HOUR	1:100-YEAR 24-HOUR
<b>Total Rainfall (m<sup>3</sup>)</b>	1,696,592	3,146,526	5,760,654
<b>Continuity Difference (m<sup>3</sup>)</b>	-37.6	-38.10	-136.90
<b>Continuity Error (%)</b>	-0.002	0.00	0.00
<b>Surface Runoff (m<sup>3</sup>)</b>	823,731	1,980,856	2,951,018
<b>Runoff (%)</b>	48.55	62.95	51.23

### 3.3.3.2 FUTURE SYSTEM PERFORMANCE

The new developments should not have a significant impact on the existing stormwater system, as most of the added areas are not directly connected to the existing stormwater network. With all developments on the west side draining to Carrot Creek, and all north developments connected to the new Oakmont outfall, the impact of discharges into the existing system are expected to be confined to Deer Ridge.

The simulation results for the ultimate conditions model are illustrated in this section. Refer to [Figure B-22](#) and [Figure B-23](#) for the minor system flooding, [Figure B-24](#) and [Figure B-25](#) for the major system ponding, [Figure B-26](#) and [Figure B-27](#) for the sewer pipe capacity, and [Figure B-28](#) and [Figure B-29](#) for the maximum pipe velocity, under the 1:5-year 4-hour and 1:100-year 4-hour design storms, respectively.

As the release rate recommendations from the Carrot Creek Report (in progress) become available, the assessment of the stormwater management plan for the projected new developments west of Ray Gibbon Drive will be updated. In the ultimate conditions model, the SWMFs were sized to release a maximum of 2.5 L/s/ha. Model results showed no flooding or capacity issues. Final servicing plans and outfall levels for Carrot Creek would confirm the model results.

The model currently has Basins 18a, 18b and 19 connected to the existing stormwater system in the Deer Ridge neighbourhood through a proposed pump station without real-time controls. Comparing the results of the exiting and ultimate condition models, there was a relative increase in flooding potential in both the major and minor systems within Deer Ridge. More sewer pipes were at or above capacity under the ultimate conditions. The performance and flooding problems in the Deer Ridge stormwater system can be improved with the introduction of RTC at the SWMF in Basin 19, which could be set to only pump stormwater flows into the system when capacity becomes available in the system downstream. The final assessment of the Deer Ridge stormwater system is pending the final layout of the future systems within Basins 18a, 18b and 19, which may get transferred to drain east to the Oakmont outfall and not the existing Deer Ridge system.

In terms of the pipe capacity and surface ponding, limited changes were encountered around and downstream of the new developments in the Kingswood and Oakmont neighbourhoods. In Kingswood, the impacts were mitigated by the introduction of the new SWMF in the system. As detailed designs and sizing of the storage facility are finalized, the Impacts to the stormwater system in this area should not be of great concern.

Within the exiting stormwater system, the group of neighbourhoods that experienced flooding and capacity issues in the existing conditions model would encounter the same problems under the ultimate conditions model. Neighbourhoods including Mission, Braeside, Akinsdale, Grandin, Sturgeon Heights and Lacombe Park seem to have the same extent of problems and are relatively not affected by the future developments.

### 3.3.3.3 ASSESSMENT OF STORMWATER PONDS

The performance assessment of the future stormwater ponds was similarly conducted using the 1:100-year 24-hour Huff storm. The simulation results of both the existing and ultimate condition models were used to estimate the release rates of each stormwater pond. The total catchment areas, maximum outflow and maximum water levels over the simulated period were determined applying the same procedure adopted to assess the existing conditions. No flooding was encountered at any of the four stormwater ponds. Refer to [Table 3.14](#) for a comparison between the source and existing conditions model results for the 1:100-year 24-hour storm event, in which the HWL, outflow, and release rate of each pond are indicated.

Comparing the release rate values for the exiting and ultimate conditions, most of the stormwater management system experienced changes. Some of the changes can be attributed to changing the computation engine from MOUSE to MIKE1D supported in the old MIKE URBAN and new MIKE+ models. SWMFs such as South Riel 1 and 2, Grandin 1, Ern 2, Heritage Park Pond and Jensen Lake 1 encountered slight to no increase in their maximum release rates. Some SWMFs showed relatively lower release rates, including North Ridge 5 and 6, Riel 1, Erin 3 as a result of lower high-water levels (HWLs).



Another group showed comparatively more considerable changes due to being closer to projected future changes, such as Riverside 5, Lacombe Park 1 and Ray Gibbon 2.

Those changes can be attributed to changes in the relative conveyance capacity of the sewer system as new areas were connected to the system. Furthermore, catchments expected to further develop in the future system exhibited higher imperviousness values, which in turn changed the runoff volumes and drainage pattern.

At Heritage Lake 3, the release rates were very high compared to the allowable release limit of 2.5 L/s/ha for the ultimate and existing condition models. That can be associated with the lack of controls downstream of the storage node, which allowed the basin to act as a surge tank for both stormwater systems.

Further, the differences in HWL between the existing and ultimate condition models ranged between 0.02 m at Grandin 1 and 1.01 m at Lacombe Park 1. Further analysis can be conducted to assess the performance of existing stormwater ponds and to consider different mitigation options on a case-by-case basis.

**Table 3-14: Ultimate Conditions Modelling Results for the Stormwater Management Facilities**

Stormwater Pond	POND PROPERTIES <sup>1</sup>					EXISTING MODEL				ULTIMATE MODEL			
	Drainage Area (ha)	Bed Level (m)	Rim Level (m)	Design Storage (m <sup>3</sup> ) <sup>2</sup>	Type	HWL (m)	Outflow (m <sup>3</sup> /s)	Release Rate (L/s/ha)	Storage (m <sup>3</sup> ) <sup>4</sup>	HWL (m)	Outflow (m <sup>3</sup> /s)	Storage (m <sup>3</sup> ) <sup>4</sup>	Release Rate (L/s/ha)
Riel South 2A <sup>1,2</sup>	56.03	659.20	664.60	20,200	Wet	663.71	0.093	1.66	13,800	663.76	0.095	14,400	1.69
Riel South 2B <sup>1,2</sup>	53.71	659.20	664.60	41,700	Wet	663.71	0.628	11.69	29,500	663.76	0.649	30,600	12.07
Riverside 5 <sup>1,3</sup>	561.95	651.70	656.57	2,600	Dry	655.40	4.524	8.05	1,250	655.49	5.068	1,350	9.02
Erin 3 <sup>1</sup>	58.19	676.40	681.80	60,800	Wet	680.65	0.080	1.37	40,500	680.87	0.002	48,200	0.03
Heritage Lake 1	50.76	677.19	681.05	44,700	Wet	679.26	0.106	2.09	27,400	679.26	0.106	27,400	2.09
Jensen Lake 1	65.81	683.44	687.00	31,400	Dry	684.86	0.200	3.04	8,100	684.89	0.202	8,300	3.07
Riverside 3	45.11	651.53	652.24	16,600	Wet	651.99	0.045	1.00	13,200	652.01	0.058	13,400	1.28
Riverside 4	551.49	654.11	656.01	24,000	Wet	655.74	4.424	8.02	18,700	655.93	4.986	24,800	9.04
Lacombe Park 1	75.31	658.10	662.60	143,800	Wet	660.19	0.068	0.90	127,100	660.30	0.690	131,700	9.17
North Ridge 5	176.10	676.40	677.00	12,300	Wet	676.60	0.085	0.48	8,800	676.60	0.082	8,800	0.47
North Ridge 4	166.65	676.40	678.00	65,100	Wet	677.50	1.570	9.42	46,500	677.56	1.609	47,700	9.65
North Ridge 3	73.54	679.30	681.20	80,400	Wet	679.95	0.184	2.50	41,500	679.98	0.187	42,200	2.55
North Ridge 2 <sup>1,2</sup>	35.97	681.20	682.70	34,700	Wet	681.61	0.069	1.92	20,200	681.87	0.117	23,000	3.24
North Ridge 6 <sup>1,2</sup>	15.98	681.20	682.70	46,800	Wet	681.61	0.030	1.88	27,600	681.87	0.002	31,300	0.13
Riverside Temp	70.23	653.70	656.60	9,200	Wet	654.83	0.621	8.84	3,400	654.77	0.390	3,250	5.56
Heritage Lake 2	51.44	671.00	673.00	15,200	Wet	671.79	0.076	1.48	13,800	672.08	0.075	15,900	1.47
Ray Gibbon 2	40.72	651.00	652.91	44,700	Wet	652.30	0.100	2.46	29,800	653.08	0.308	47,000	7.58
Grandin 1	25.77	681.73	683.63	42,000	Wet	683.16	0.087	3.38	8,900	683.18	0.088	9,300	3.40
Campbell 1	44.78	684.32	686.30	25,700	Dry	685.84	0.377	8.42	13,100	686.17	0.440	18,500	9.82
Riel 2 <sup>3</sup>	228.86	651.80	653.80	74,500	Wet	652.60	0.540	2.36	35,900	652.64	0.590	37,100	2.58
Oakmont 1	171.13	654.00	655.10	18,200	Dry	655.04	0.199	1.16	1,900	655.02	0.177	1,850	1.04
Erin Ridge 3	225.38	681.89	683.00	21,100	Wet	682.55	0.204	0.91	21,700	682.62	0.220	22,300	0.98
Erin Ridge 1	11.56	679.30	683.81	5,100	Dry	679.48	0.040	3.46	57	679.83	0.073	478	6.33
Riel South 1	124.91	653.20	656.30	33,100	Wet	654.05	0.323	2.59	4,700	654.33	0.360	8,100	2.88
Ern 01	50.63	680.40	682.10	83,300	Wet	681.57	0.083	1.64	66,700	681.95	0.095	79,200	1.88
Ern 02	97.72	682.30	684.30	91,000	Wet	683.18	0.073	0.75	70,000	683.21	0.074	70,600	0.76
Riel 01	242.82	651.80	653.80	145,700	Wet	652.33	0.330	1.36	102,200	652.35	0.100	103,800	0.41
Heritage Lake 3 <sup>3</sup>	0.38	665.94	668.00	7,300	Dry	666.67	0.066	173.68	2,600	666.80	0.154	3,100	405.15
Campbell South <sup>1</sup>	12.81	681.50	685.4	30,600	Dry	684.29	0.066	5.15	26,900	683.97	0.51	21,000	39.83

<sup>1</sup> Newly added to the model

<sup>3</sup> No direct control on the pond outlet

<sup>5</sup> Based on old UMP information

<sup>2</sup> Hydraulically connected SWMFs

<sup>4</sup> Based on the existing modelled SWMF

## 3.4 STORMWATER SYSTEM IMPLEMENTATION PLAN

This section details the implementation plans for the proposed improvements including approximate cost estimates and capital projects prioritization with construction time frame.

### 3.4.1 COST ESTIMATE BASIS

For the stormwater utility sewer pipes, the estimated unit costs for different pipe sizes are provided in **Table 3-15**, including costs for new installations and replacements. The listed total costs take into account an allowance of 40% for contingency. Construction costs for typical 1200 mm size manholes at 100 m intervals were incorporated in the unit pipe costs. For replacement installations or new constructions in developed areas, the respective unit price comprises excavation, compaction and restoration costs to typical city road conditions. Costs of land acquisitions, drainage easements, right of way access are not included. In order to estimate an average unit price for the construction of stormwater ponds, a unit price of \$30 per cubic meter is estimated based on available construction fee records, including costs for excavation, compaction, landscaping, control structures and 40% engineering fees and contingency. Also, the following price estimates do not include the price of land for acquisition or drainage easements. These prices estimates are only to be used for the purpose of this study to deliver high-level approximate estimations for future project budgeting.

**Table 3-15: Unit Costs for Stormwater Gravity Sewer Installation**

SIZE	UNIT	UNIT COSTS	
		Green Field <sup>1</sup>	Existing Areas <sup>2</sup>
300 mm	Lineal meter (m)	\$1,160	\$4,490
375 mm	Lineal meter (m)	\$1,235	\$4,795
450 mm	Lineal meter (m)	\$1,310	\$5,100
525 mm	Lineal meter (m)	\$1,690	\$5,980
600 mm	Lineal meter (m)	\$1,765	\$6,860
675 mm	Lineal meter (m)	\$2,145	\$7,230
750 mm	Lineal meter (m)	\$2,910	\$7,595
900 mm	Lineal meter (m)	\$3,135	\$8,185
1050 mm	Lineal meter (m)	\$5,190	\$10,425
1200 mm	Lineal meter (m)	\$5,485	\$11,025

<sup>1</sup> Green field, open cut installation, 0 to 7m deep

<sup>2</sup> New construction or replacements in existing areas, 0 to 7m deep

### 3.4.2 CAPITAL PROJECTS PRIORITIZATION

The Capital Projects List, which prioritizes the infrastructure improvements from this study, is broken down by 3-year intervals until 2031. The capital projects are prioritized and broken down by time frame. **Table 3-16** identifies priority level and construction time frame for each neighbourhood. The proposed capital project improvements for the existing stormwater system are prioritized based on the flooding susceptibility within each neighbourhood, and the anticipated value of the proposed improvement for flood risk reduction and conveyance capacity increase. In addition, the priority level is based on the system review conducted with the City of St. Albert during the Future Ready Workshop and over several follow-up meetings. In addition, this prioritized capital list focuses on improvement works within the existing network that require upgrades in order to resolve existing deficiencies and accommodate future flows. However, the following capital projects plan does not include the infrastructure renewal and replacements are not included in this assessment.

**Table 3-16: Capital Projects Prioritization for the Existing Stormwater System**

PROJECT	ITEMS	PRIORITY LEVEL	PRIORITIZATION
<b>Deer Ridge Lift Station and SWMF</b>	New SWMF, lift station, overflow pipe and forcemain	High	2022-2023
<b>Mission Minor System</b>	Constructing two storm trunks	Low	2029-2031
<b>Grandin Minor System</b>	Constructing minor system	Medium	2026-2029
<b>Braeside Minor System</b>	Constructing minor system	Low	2029-2031
<b>Lacombe Park Upgrades</b>	Upsizing stormwater trunk and/or adding new surge ponds	Medium	2026-2029
<b>Sheridan Drive Major System</b>	Major system improvements	High	2022-2025

### 3.4.3 CAPITAL PROJECTS COST ESTIMATES

A breakdown of estimated costs for the proposed improvements under each capital project are provided below in **Table 3-17**.

### 3.4.4 FUNDING IMPROVEMENTS

Part of the proposed improvements could be funded by any combination of grant funding, local improvement tax (LIT) or development levies.

Development Levies could cover part of the improvements costs for infrastructure that is directly servicing new developments. However, municipal funds should cover the costs to resolve the issues in the drainage system that serves existing neighbourhoods.



According to the Alberta Municipal Government Act, an LIT may be imposed to raise revenue to pay for a project that the council considers to be of greater benefit to an area of the municipality than to the whole municipality. The LIT is based on several factors including, but not limited to:

1. Financial assistance (grant funding) received from the Crown in right of Canada or Alberta or from other sources.
2. The number of subdivisions (parcels) of land within the affected area of the improvement project.

In all cases, the LIT must be calculated such that each benefitting parcel bears an appropriate share of the updated tax rate. The Government of Alberta and the federal government offer grants to support the planning and development of municipalities. Current grants available include the Municipal Sustainability Initiative, Municipal Asset Management Program, Investing in Canada Infrastructure Program, and Green Municipal Funds.

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### 3.4.5 OTHER RECOMMENDATIONS

- The 2014 hydraulic model was built with an assumption of the major drainage components that generally follow the minor system. Some revisions are needed to verify that assumption compared with the real drainage system in order to ensure the accuracy of the simulation results.
- The calibration and validation of the stormwater model parameters would benefit from high-quality flow monitoring data. If the flow records extend over periods that include storm events with significant return periods, the collected data can be used to further validate the model results for larger storms.
- Storm sewers flow monitoring should be conducted in areas of the stormwater system to confirm dry and wet weather flows and update the hydraulic model accordingly (e.g. at Grosvenor Boulevard, at trunk sewer connections, etc.).
- As mentioned under assumptions, further analysis within the future development lands to the North of St. Albert, taking into account the future annexation lands, as these will heavily impact the design and management of the proposed RTC systems as well as downstream capacity within the Deer Ridge and the new Sturgeon River outfall draining the Erin Ridge area.



**Table 3-17: Approximate Costs for Proposed Capital Stormwater Project**

ID	ITEM	SIZE	QUANTITY	UNIT	UNIT PRICE	TOTAL
<b>1</b>	<b>Mission</b>					
1.1	Connection from St Vital Ave to Mission Ave	600	350	m	\$6,860	\$2,401,000
					Subtotal	\$2,401,000
<b>2</b>	<b>Grandin</b>					
2.1	Connection on Gillian Crs North	450	235	m	\$5,100	\$1,198,500
2.2	Connection on Gillian Crs South	450	235	m	\$5,100	\$1,198,500
2.3	Connection on Gillian Crs North to Gillian Crs South	525	180	m	\$5,980	\$1,076,400
2.4	Connection on Gillian Crs South to Levasseur Rd	600	60	m	\$6,860	\$411,600
2.5	Connection on Gordon Crs South	450	290	m	\$5,100	\$1,479,000
2.6	Connection on Gordon Crs North	450	220	m	\$5,100	\$1,122,000
2.7	Connection on Gillian Crs to Grandin Rd	525	315	m	\$5,980	\$1,883,700
2.8	Connection from Grandin Rd to Grandin Pond	750	125	m	\$7,595	\$949,375
2.9	Connection on Gandora	300	85	m	\$4,490	\$381,650
2.10.	Connection on Gandora North	450	240	m	\$5,100	\$1,224,000
2.11	Connection on Gandora South	450	235	m	\$5,100	\$1,198,500
2.12	Connection on Gillian Crs to Grandin Rd	525	95	m	\$5,980	\$568,100
					Subtotal	\$12,691,325
<b>3</b>	<b>Lacombe Park</b>					
3.1	SWMF at Location 1	1	20000	m <sup>3</sup>	\$30	\$600,000
3.2	SWMF at Location 2: Larose Park	1	15000	m <sup>3</sup>	\$30	\$450,000
					Subtotal	\$1,050,000
<b>4</b>	<b>Braeside</b>					
4.1	Connection on Burnham Ave	450	200	m	\$5,100	\$1,020,000
4.2	Connection on Bandon St	450	230	m	\$5,100	\$1,173,000
					Subtotal	\$2,193,000
<b>5</b>	<b>Deer Ridge</b>					
5.1	Force main from Proposed SWMF	*	490	m	N/A	N/A
5.2	Connection to Proposed SWMF	1050	690	m	\$10,425	\$7,193,250
5.3	Proposed SWMF	1	35000	m <sup>3</sup>	\$30	\$1,050,000
5.4	Proposed Lift Station	1	*	-	*	N/A
					Subtotal	\$8,243,250

\*Values pending the final layout of Jensen Lakes Drainage.