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REPORT

City of Wetaskiwin

Stormwater Master Plan Update



MAY 2020

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EXECUTIVE SUMMARY

The City of Wetaskiwin (the City) retained Associated Engineering to complete an update to their 2014 Stormwater Master Plan. This update will consider all lands within the City and some areas to the west within the County of Wetaskiwin, which drain towards the City.

The report objective is to provide the City with a comprehensive Stormwater Master Plan which will identify limitations within the existing system, outline recommended upgrades, and present servicing concepts for future development. The scope of the project includes:

- Provide the City with an updated and calibrated stormwater model;
- Identify existing system deficiencies and opportunities for repairs or system improvements;
- Provide a future stormwater management plan for future developments; and
- Provide estimated upgrading costs.

A coupled 1D-2D hydrologic and hydraulic model was developed for the proposed Stormwater Master Plan using the MIKE software modelling package developed and marketed by Danish Hydraulic Institute (DHI). The model included all of the City's stormwater infrastructure including pipes, manholes, catchbasins, ditches, culverts, and stormwater management facilities.

A flow monitoring program was conducted between June and October 2019. The objective of the monitoring program was to collect reliable flow data within the storm sewer system for model calibration and validation of flows within the pipe network. The 2019 flow monitoring program consisted of five monitoring sites in strategically selected locations within the storm sewer system, as well as two tipping-bucket style rain gauges.

The largest storm event during the flow monitoring period occurred on July 6, 2019 and this event was chosen for the calibration of the developed model. This is approximately a 1:10-year design storm event.

The calibration results show that peak flows are overestimated on average by 30%. However, the timing and shape of the simulated peaks during the calibration event achieved a good fit at all monitor locations between the observed and simulated data. This can be attributed to the physically based modelling approach, estimated infiltration rates, lack of catchbasin locations, catchbasin type, and the coupling locations.

Validation runs were simulated for the July 18 and June 7, 2019 storm events. The July 18 event was approximately equivalent to a 1:2-year return period design storm event, while the June 7 event was less than a 1:2-year design storm event. Similar to the calibration results, validation flows overestimated on average by 30%. Note, the timing and shape of the simulated hydrographs remains consistent between simulated results and observed data.

The existing system performance was assessed by simulating the 1:5-year and 1:100-year 4 hour design storm events using the coupled 1D-2D model. The majority of the City's culverts have capacity for up to the 1:5-year design storm event. Significant surface ponding was simulated during the 1:5-year and 1:100-year 4 hour design storm events. Currently, the City's ditch network does not provide capacity for the 1:100-year 4 hour design storm event.

To assess the impact of the overestimation within the existing minor system assessment, the 1:2-year 4 hour design storm event was also simulated. This event is equivalent to approximately 30% less than the 1:5-year 4 hour design storm event. Results show that there are still minor surface ponding areas within the City during this event. Approximately half of the City's pipes are still running over capacity. Therefore, the majority of the City's storm networks (both minor and major) provide capacity up to approximately the 1:2-year 4 hour design storm event.

Associated Engineering assessed several upgrades to address excessive flooding not confined to streets during a 1:5-year 4 hour design storm event. These upgrades consisted of upsizing of pipes within the downtown area of the City. Upgrades were proposed assuming ditch modifications will be completed to mitigate backup into the City's minor system. A capital cost estimate of **\$ 28 Million** was developed in 2020 dollars to complete these upgrades. Note, this cost includes 30% Engineering and Contingency.

An Ultimate Future System Expansion Storm Servicing Concept was developed to service the three development stages and ultimate expansion for the City of Wetaskiwin. The concept outlines major storm trunks, and proposed stormwater management facilities for stormwater runoff quantity and quality control.

Some key conclusions and recommendations are as follows:

- The majority of the City's storm networks (both minor and major) provide capacity up to approximately the 1:2-year 4 hour design storm event. In addition, the majority of the City's culverts have capacity for up to the 1:5-year design storm event. Significant surface ponding was simulated during the 1:5-year and 1:100-year 4 hour design storm events. Currently, the City's ditch network does not provide capacity for the 1:100-year 4 hour design storm event.
- The City implement a continuous storm sewer flow monitoring program and undertake a CCTV inspection program of their storm system to ensure no blockages or pipe deterioration exists.
- Additional calibration and validation of the developed coupled 1D-2D model as more observed data becomes available.
- Further refinement of each proposed existing minor system upgrade should be completed during design. The solution in each location should be based upon site constraints, construction method (open-cut, trenchless), budget, etc. In addition, upgrades may incorporate a combination of solutions such as storage, diversion, Low Impact Development (LID), raingardens, soil cells, etc.
- The Ultimate Future System Expansion Storm Servicing Concept be followed as development within the City progresses. Development should progress in an orderly manner with discouragement of "leap-frogging".
- It is recommended that the City coordinate stormwater runoff works with Wetaskiwin County to avoid adverse impacts occurring from offsite drainage areas.

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1 INTRODUCTION

1.1 Background

The City of Wetaskiwin (the City) retained Associated Engineering to complete an update to their 2014 Stormwater Master Plan. The City of Wetaskiwin is located in Central Alberta, approximately 70 kilometers south of the City of Edmonton. The present corporate boundaries encompass approximately 20 square kilometers. Refer to **Figure 1-1**.

As part of this update, Associated Engineering proposed to develop a coupled One Dimensional (1D) - Two Dimensional (2D) hydrologic and hydraulic model. In addition to the lands within the City, this model includes 90 square kilometres to the west within the County of Wetaskiwin which drains towards the City.

The proposed model was developed using the MIKE software modelling package developed and marketed by Danish Hydraulic Institute (DHI). This software is widely used and contains 1D, 2D, and Three Dimensional (3D) modules for urban and rural environments. The 1D MIKE URBAN, 1D MIKE 11, and 2D MIKE 21 Flexible Mesh (FM) modules were used for this study.

An urban stormwater system consists of minor and major systems. The minor system within the City includes pipes ranging in size from 200 to 2100 mm in diameter, manholes, catchbasins, and outfall structures. Meanwhile, the major system consists of streets, roads, a regional Stormwater Management Facility (SWMF), parkland, and overland runoff routes. As shown on **Figure 1-1**, there are three major ditches flowing through the City which include a number of culverts.

1.2 Study Area

Figure 1-2 shows the project area including the estimated 90 square kilometres of contributing area west of the City. This offsite area was delineated using a 15 metre Digital Elevation Model (DEM).

The topography of the area generally falls toward the east. The highest elevations within the City occur in the west portion of the City, and the lowest elevations are located in the northeast.

The City Zoning Map (January 2018) was used for existing developed areas to establish land cover roughness, infiltration potential, and resulting stormwater runoff. The City includes residential areas of various densities from single family homes to condominium or apartment complexes. There is an airport located within the west section of the City. In addition, the southeast section includes an industrial area. Concentrated commercial developments are located within the downtown area (along 50 Street) and along Highway 13/2A (56 Street).

1.3 Report Objectives and Scope

The report objective is to provide the City with a comprehensive Stormwater Master Plan Update which will identify limitations within the existing system, outline recommended upgrades and present servicing concepts for future development. The scope of the project includes:

- Providing the City with an updated and calibrated stormwater model;
- Identifying existing system deficiencies and opportunities for repairs or system improvements;
- Providing a future stormwater management plan for future developments; and
- Providing estimated upgrading costs.

1.4 References

The following information has been reviewed in preparation of this report:

1. Area Structure Plans;
2. City of Wetaskiwin Stormwater Master Plan Study Final Report, Stantec Consulting Ltd., January 2004;
3. City of Wetaskiwin Utilities System Overall Master Plan, MPE Engineering Ltd., April 2014;
4. City of Wetaskiwin Municipal Engineering standards, December 2018;
5. GIS data (including land use);
6. Infrastructure grid map 2018;
7. LiDAR contour data; and
8. Stormwater Treatment & Storage Facility Stormwater Management Report, EXH Engineering Services Ltd., March 2006.

1.5 Abbreviations

ASP	Area Structure Plan
cm	centimetre
DEM	digital elevation model
DHI	Danish Hydraulic Institute
ERP	Emergency Response Plan
FM	flexible mesh
GLF	Grade Line Factors
GST	Goods and Services Tax
ha	hectares
HGL	hydraulic grade line
IDF	Intensity-Duration-Frequency
km	kilometre
km ²	square kilometres
L/s	Litres per second
L/s/ha	Litres per second per hectare
LiDAR	Light Detection and Ranging
LID	Low Impact Development
m	metre
m/s	metres per second
m ³ /s	cubic metres per second
m ²	square metres
m ³	cubic metres
mm	millimetre
mm/hr	millimetres per hour
SWMF	Stormwater Management Facility
1D	one dimensional
2D	two dimensional
3D	three dimensional

1.6 Previous Studies

1.6.1 Stormwater Management Master Plan Study Final Report – Stantec Consulting Ltd. 2004

The Stormwater Management Master Plan Study Final Report (2004) assessed the City of Wetaskiwin's existing storm drainage system and proposed a stormwater management master plan for future growth. As part of this study, an XPSWMM model was developed. However, this model was not calibrated due to lack of flow monitoring data at the time. The City's drainage ditch network was surveyed and cross sections were included within the model.

Survey in 2004 indicated that there was accumulated silt and debris within the ditches, reducing their channel capacities. Model results showed that structures within the ditch network were able to convey peak flows generated during a 1:5-year design storm event, while most of the storm sewers were designed to convey a 1:2-year design storm event which did not meet the City's Design Standards.

Critical areas within the City were identified where the existing storm system did not have adequate capacity for runoff generated in excess of a 1:2-year design storm event. In addition, some improvements to the City's drainage network were recommended. These improvements included sewer upgrades, ditch/culvert construction, surge pond construction, ditch maintenance, and topographic survey. Flow monitoring was recommended in order to calibrate the developed model.

1.6.2 Stormwater Treatment and Storage Facility Stormwater Management Report – EXH Engineering Services Ltd. 2006

The 2006 Stormwater Management Report summarized drainage within the City's North Central Drainage System (minor and major system) and assessed stormwater treatment and storage facilities. This assessment was conducted to alleviate flooding of the Home Hardware site and other parcels within the northeast section of the City.

As part of this study, the 2004 Master Plan XPSWMM model was modified within the vicinity of the North Central Drainage System. A storm event in June 2005 was approximated as a 1:100-year 4 hour Chicago design storm event and modelled flood levels were compared to an observed flood water level at Clearwater Crescent. Similar to the Stormwater Management Master Plan Study (2004), additional flow monitoring was recommended to provide calibration data for the model.

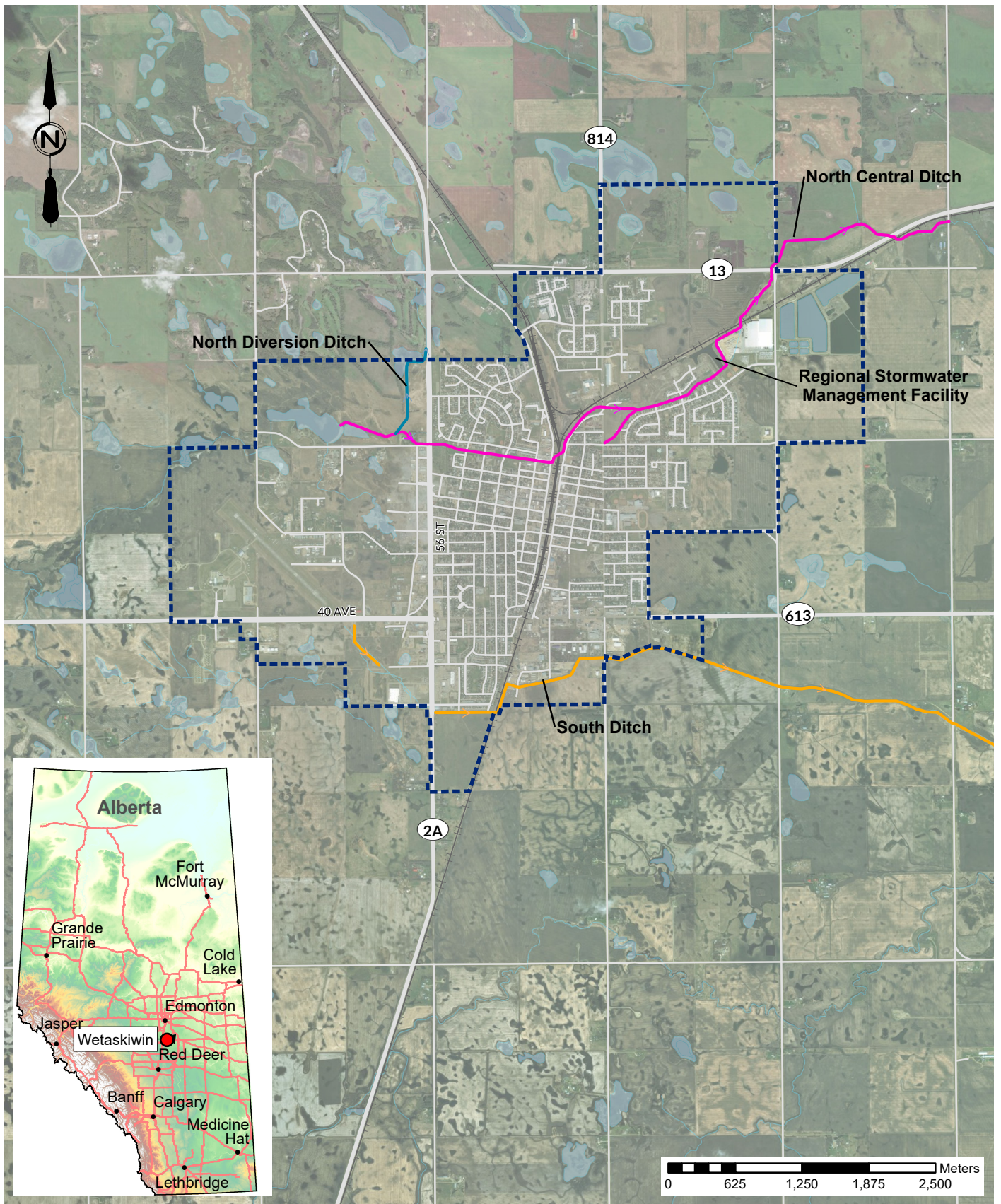
Construction of three stormwater ponds and maintenance/upgrading of the North Central Ditch was recommended and proposed to be undertaken in several phases. The regional surge pond near Home Hardware was constructed in 2006. This pond services a large area of land within the northeast central area of the City.

1.6.3 Utilities System Overall Master Plan – MPE Engineering Ltd. 2014

The stormwater drainage section within the 2014 Utilities System Overall Master Plan discusses the existing overland drainage system, previous recommendations, modelling, and recommendations. The modelling included using Bentley CulvertMaster to look at ditch capacity within the City's drainage ditch network. The 2004 XPSWMM model was not updated as part of this work.

The overall master plan recommended stormwater management facility controls for future developments according to release rates outlined within the Stormwater Management Master Plan Study (2004). In addition, it was found that the existing ditches have capacity for up to the 1:25-year design storm event. Therefore, it was recommended that all future development discharge into the ditch network.

PLOT DATE: 4/14/2020 12:15:28 PM
SAVE DATE: 4/14/2020 12:15:28 PM
DWG PATH: I:\s\2019\2019-3447\00_UMP_Update\Engineering\03.00_Conceptual_Feasibility_Design_Master_Plans\010_GIS\map_Location.mxd



AE PROJECT No.	2019-3447
SCALE	1:50,000
APPROVED	
DATE	2020 APR 14
REV	
DESCRIPTION	ISSUED FOR REPORT

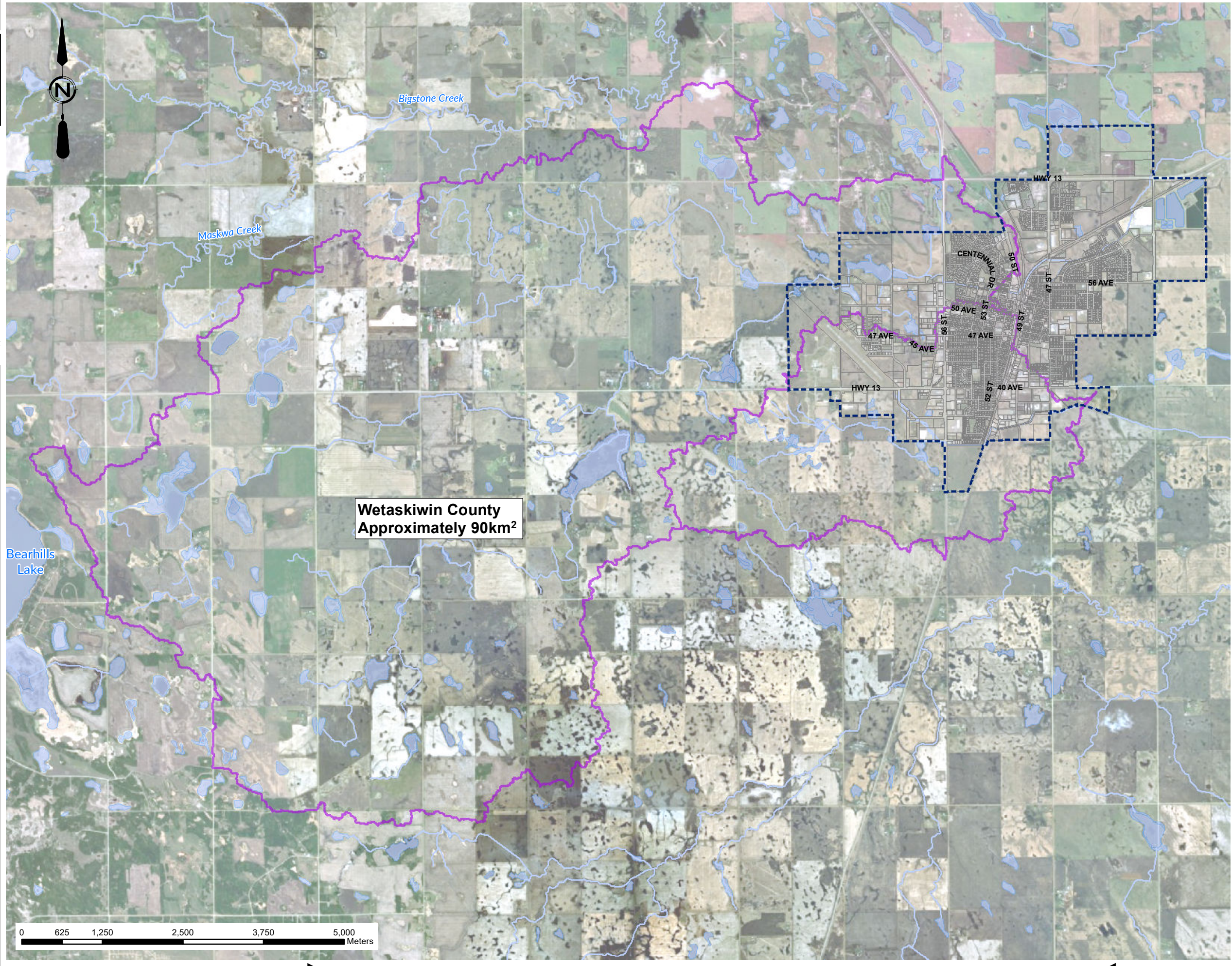
FIGURE 1-1

CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE

LOCATION

PLOT DATE: 4/1/2020 11:07:10 AM
SAVE DATE: 4/1/2020 11:07:10 AM
DWG PATH: \\s-01\projects\2019\344700_UWP_Update\Engineering\03_00_Conceptual_Feasibility_Design_Master_Plans\010_GIS\map_fig_1-1_Project_Area.mxd

IF NOT 25 mm ADJUST SCALES
SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE



Legend

- Watercourse
- Delineated Offsite Catchments
- Water Body
- City of Wetaskiwin Boundary



FIGURE 1-2
CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE
PROJECT AREA

AE PROJECT No.	2019-3447
SCALE	1:57,000
APPROVED	
DATE	2020APR01
REV	
DESCRIPTION	ISSUED FOR DRAFT

2 DESIGN CRITERIA

The City of Wetaskiwin Municipal Engineering Standards (2018) state that minor system elements should be designed to accommodate runoff flow rates and volumes from the 1:5-year design storm event while the major system elements should accommodate the 1:100-year design storm event plus 20%. However, these standards are rarely achievable in established urban areas that were developed to previous standards. The Stormwater Management Master Plan Study (2004) stated that the original design of the City's minor storm sewer system was based on the Rational Method using the 1:1-year and 1:2-year design storm events for residential and commercial/industrial areas, respectively.

The Engineering Standards include Intensity-Duration-Frequency (IDF) values obtained from the Edmonton City Centre Airport curve data for 1984-2010. However, in 2018, EPCOR updated their IDF curves for the City of Edmonton encompassing 32 years of records (1984-2015). Therefore, it is recommended that the City of Wetaskiwin update their Municipal Engineering Standards to include the use of the most up-to-date IDF data for the region. The 2018 EPCOR IDF Curve document has been included in [Appendix A](#). Associated Engineering will use the 2018 EPCOR IDF curves for design storm event simulations within this study.

The current Municipal Engineering Standards (2018) for the minor and major systems have been summarized below. In addition, any exceptions have been identified in bold and italicized.

2.1 Minor System

- The minor system shall be designed to accommodate runoff generated by a 1:5year design storm event and flows from the foundation drain discharge system.
- The City does not accept the direct connection of sump pumps into the storm system for new development.
- Minimum gravity sewer main diameter: 300 mm with minimum and maximum flow velocities of 0.6 m/s and 3.0 m/s, respectively. Manning's roughness coefficient of at least 0.013 for calculation.
- Storm trunk sewers shall be designed to accommodate, without surcharge, the anticipated design flow (1:5-year) multiplied by a safety factor of 1.25 in order to account for potential future changes in land use and ensure trunk sewers do not surcharge before upstream lateral storm sewers.
- Manholes should be spaced no greater than 150 m apart, and be installed at all changes in size, grade or alignment. The crowns of the pipes must be matched to maintain a continuous hydraulic grade line and regardless of the design flow and pipe slope, pipe diameter shall not be permitted to decrease through the downstream direction.
- Catchbasin leads shall be a minimum 250 mm diameter and connect directly to a manhole. All leads shall have a minimum grade of 1.0% and maximum length of 30 m.
- To prevent the occurrence of ponding, the maximum distance between manholes will be 150 m. In addition, the maximum distance between catchbasins will be 120 m.
- A minimum depth of cover of 1.5 m shall be provided for catchbasin leads and 1.8 m for sewer mains as measured from the top of the pipe to the final finished grade at the surface.
- ***For existing developments, surcharging of the storm pipes may be permitted, provided that surcharge levels are below ground surface in a 1:5-year design storm event and the risk to private property is minimized in a 1:100-year design storm event. Manhole and pipe details will be required to meet the specifications if they are re-built or replaced for other reasons. These upgrades should be modelled to check the possibility of downstream impacts.***

2.2 Major System

- The maximum depth of peak flows and ponding shall not exceed 150 mm on arterial roadways.
- Major system conveyance components must provide continuous overland flow routes to a designated receiving watercourse. In addition, the design must include an analysis of the capacity and characteristics of the downstream receiving drainage course and identification of any measures to be completed to prevent downstream flooding and/or erosion and sediment control.
- The maximum depth of peak flows and ponding along major system conveyance components at adjacent lots and buildings shall be a minimum of 350 mm below the lowest anticipated landscape grade or opening. Manning's roughness coefficients to be applied for roadways and grassed boulevards shall be 0.013 and 0.050, respectively.
- Culverts may be provided along swales and drainage channels with a minimum 400 mm diameter, including gratings for public safety and energy dissipation, sediment and erosion control while maintaining a maximum flow velocity of 1.0 m/s.
- Commercial, institutional, and high-density residential developments require onsite stormwater management facilities. These stormwater management facilities shall be designed based on the critical volume determined from the 1:25-year design storm event. Suitable overland flow routes shall be provided from the facility to accommodate overflows for more significant rainfall events. The design must limit post-development peak runoff flows from the development to 2.5 L/s/ha. ***Previous Master Plans used different pre-development release rates for future development. A more detailed analysis may be required prior to stormwater management facility designs to confirm this rate.***
- ***For existing development areas, the goal should be to prevent flooding of buildings. This requires assessment on an individual basis, but, generally, can be met by limiting the depth of ponding in the low areas to a maximum of 150 mm.***

2.3 Computer Modelling

The City requires the use of computer modelling for analysis of areas greater than 65 ha. ***It is recommended that the storm hyetographs developed by EPCOR in 2018 for the 4-hour Chicago and the 24-hour Huff design storms are adopted to provide consistency for modelling purposes. Tables providing the EPCOR hyetograph values are included in [Appendix A](#).***

2.4 Storage Components

Storage components (i.e. SWMFs) may be required to reduce peak flow rates to downstream sewer systems and/or watercourses, or to attenuate runoff for major drainage flows.

- SWMFs are to be sized for the 1:100-year design storm event plus 20%.
- Wet ponds or constructed wetlands are recommended where storage is required within residential subdivisions to promote settling of solid particles.
- Wet ponds should have a minimum surface area of 2 ha at normal water level and minimum permanent depth of 2.5 m with 0.5 m of freeboard above the high water level. Recommended slopes shall not exceed 5:1 and shall meet recommendations of the geotechnical report. Inlets and outlets must be located as far from each other as possible and fully submerged with obverts 1 m below the normal water level.
- Constructed wetlands must have a minimum drainage area of 5 ha and use a variety of water depths (0.1 to 0.6 m) with average water depth of 0.3 m. The surface area at normal water level shall be a minimum of 1 ha. A forebay with slopes of 7:1 is required to trap suspended sediment at inlets and outlet.

- Dry ponds are not recommended unless wet ponds and constructed wetlands are impractical. The maximum storage depth shall be 1.5 m with minimum bottom slope of 2% to promote proper drainage.

2.5 Outfalls

Outfall pipe obverts shall be above the 1:5-year design flood level of the receiving channel and inverts shall be above the normal ice level under average annual precipitation conditions. Drop structures and energy dissipation works shall be included where necessary to prevent erosion and gratings shall be included for public safety.

3 MODEL DEVELOPMENT

A coupled 1D-2D hydrologic and hydraulic model was developed for the proposed **Stormwater Master Plan Update**. This model used the **MIKE URBAN**, **MIKE 11**, and **MIKE 21 Flexible Mesh (FM)** modules within the MIKEFLOOD interface to evaluate the capacity of the existing storm sewer system. The model simulates the actual physical processes of a major and minor stormwater system by applying a design storm event as a boundary condition to the developed 2D model surface. Stormwater runoff is then simulated to flow through the overland drainage system into the coupled 1D pipe network model. Culverts within the City's major drainage ditch network were represented and simulated within the 1D MIKE 11 and MIKE URBAN models.

Note that the City's minor storm system was modelled down to manhole level and coupled to the major system at manhole locations.

Areas with trap lows, depressions and sags were represented within the model. Overland flooding within the major system was simulated as well as the flow conditions within the minor system (storm pipes). Backwater effects that increased water levels and restricted flows within the pipe and ditch networks were simulated.

The following sections provide details of the model development, calibration, validation, design storm event simulations, and options considered to improve and enhance the hydraulic performance of the existing and future storm drainage systems, respectively.

3.1 Modelling Assumptions

The following assumptions were used to develop the coupled 1D-2D model:

- Manholes within the model were represented as catchbasins.
- Based on limited information, manholes represented as catchbasins were assumed to have type F-51 characteristics.
- Allowable depth of ponding 0.15 m.
- Coupling locations were assumed to have a maximum inlet capacity ranging between 0.2552 m³/s and 0.366 m³/s with a clear open area of 0.3 m². This was based on the report *Capacities of Catch Basin Inlets Based on Use in Alberta*, Wilson, D.
- The pipe network was modelled based on the 2004 XPSWMM storm model and updated with the City's 2018 Infrastructure Grid Map.
- Manhole diameters were assumed to be 1.2 m.
- The entire storm system was assumed to be dry at model start-up.
- Rainfall intensity was assumed to be constant over the entire model domain.
- Existing and future land uses were based on the Cadastre Zonings (shapefiles provided by the City) and the Municipal Development Plan (2011), respectively.
- Infiltration rates were initially estimated based on the Soil Conservation Service (SCS) Curve Numbers for Urban Areas with an antecedent moisture condition (AMC) II for Hydrologic Soil Group C.
- The control gate at the North Diversion/North Central Ditch confluence is closed at all times.

3.2 Available Modelling Background Information

For the coupled 1D-2D model, the following information was available:

- Detailed (1 m) resolution Light Detection and Ranging (LiDAR) data was purchased to represent the topography of the City of Wetaskiwin.
- Coarse (15 m) resolution Digital Elevation Model (DEM) data was purchased to represent the topography of the areas within the County of Wetaskiwin to the west of the City contributing stormwater runoff.
- The City's 2018 Infrastructure Grid Map (DWG file), June 2018.
- City of Wetaskiwin Stormwater Management Master Plan Study XPSWMM storm model (2004).

3.3 Modelling Methodology

The following summarizes the over-arching methodology used in the model development:

- Data pre-processing and post-processing.
- 1D model development of the City's storm sewer network and culverts within the ditch network using MIKE URBAN and MIKE 11, respectively.
- 2D model development of the City's overland flow network using MIKE 21 FM.
- Coupled 1D-2D model development within MIKE FLOOD.
- Model simulation.

3.4 Off-site Drainage

An area of approximately 90 km² flows west to east from Wetaskiwin County towards the City of Wetaskiwin. However, the By the Lake Park and North Diversion Ditch were constructed in 1988 as part of the West Wetaskiwin Flood Diversion Project to attenuate this offsite drainage area and divert flows around the City into Bigstone Creek. As part of this study, this offsite drainage area was assessed.

In order to understand the contribution from the offsite area, Associated Engineering completed a pilot test. This test compared the results from PCSWMM and MIKE 21 FM models to determine the best approach to represent and simulate offsite runoff contributions. As part of the background data collection phase, Associated Engineering purchased 15 m LiDAR DEM data surrounding the City to ensure all offsite drainage areas were captured and represented as part of the [Stormwater Master Plan Update](#). Catchment areas were delineated using this DEM data and are shown in [Figure 3-1](#). These catchment areas were applied within a PCSWMM model using typical parameters. In addition, the delineated areas were included in a 2D MIKE 21 FM model. These models simulated the runoff generated during a 1:100-year 24 hour Huff distribution design storm event.

Results indicated that the 2D MIKE 21 FM model was over-estimating runoff flows due to the coarse resolution of DEM data. Flows generated within the 2D model were approximately two to three times greater than flows generated from the PCSWMM model. This is most likely due to the modelling approach within the two models. PCSWMM uses an implicit approach (lumped) in catchment representation and MIKE 21 FM uses an explicit approach for representing the terrain.

The coarse resolution 15 m DEM may not accurately represent the numerous depressions within the terrain. These depressions would capture runoff and reduce peak flows during and after a storm event. However, due to the implicit

modelling approach in PCSWMM, parameters can be included to reflect this phenomenon. Whereas, the explicit approach in MIKE 21 FM is not able to reflect this.

Associated Engineering concluded that the best representation of the off-site drainage to the west and south of the City would be simulating catchment runoff within PCSWMM. Generated runoff hydrographs would then be applied as point source inflows into the developed 2D MIKE 21 FM model within the coupled model interface. The detailed PCSWMM catchment parameters used for boundary conditions within the coupled 1D-2D model are included in [Appendix B](#).

3.5 Mike Urban Model

MIKE URBAN is a 1D model developed with modules simulating phenomenon within urban pipe networks. A 1D model was created using the latest City Infrastructure Grid Map obtained on April 3, 2019. The storm network layers within the map were converted to shapefiles. These shapefiles were superimposed and compared with the storm network included within the Stormwater Management Master Plan Study (2004) XPSWMM models provided by the City. Based on the Stormwater Management Master Plan Study (2004), the existing model did not include pipes less than 375 mm in diameter. Therefore, the model developed for this study included the pipe network within the existing XPSWMM model, pipes with diameters less than 375 mm, and areas developed within the City since 2004. The existing minor system within the MIKE URBAN model is illustrated in [Figure 3-2](#).

As previously stated, the City's minor storm system was modelled down to manhole level and coupled to the major system at manhole locations. Manholes within the model were represented as catchbasins based on limited information.

The MIKE URBAN model includes network data consisting of pipes, manholes, weirs, orifices, etc., hydrologic, and hydraulic parameters. For this assessment, the hydraulic parameters for the system such as pipe size and slope were obtained from previous studies and City data. However, a critical parameter that needs to be included within the model is the head loss, which must be properly estimated.

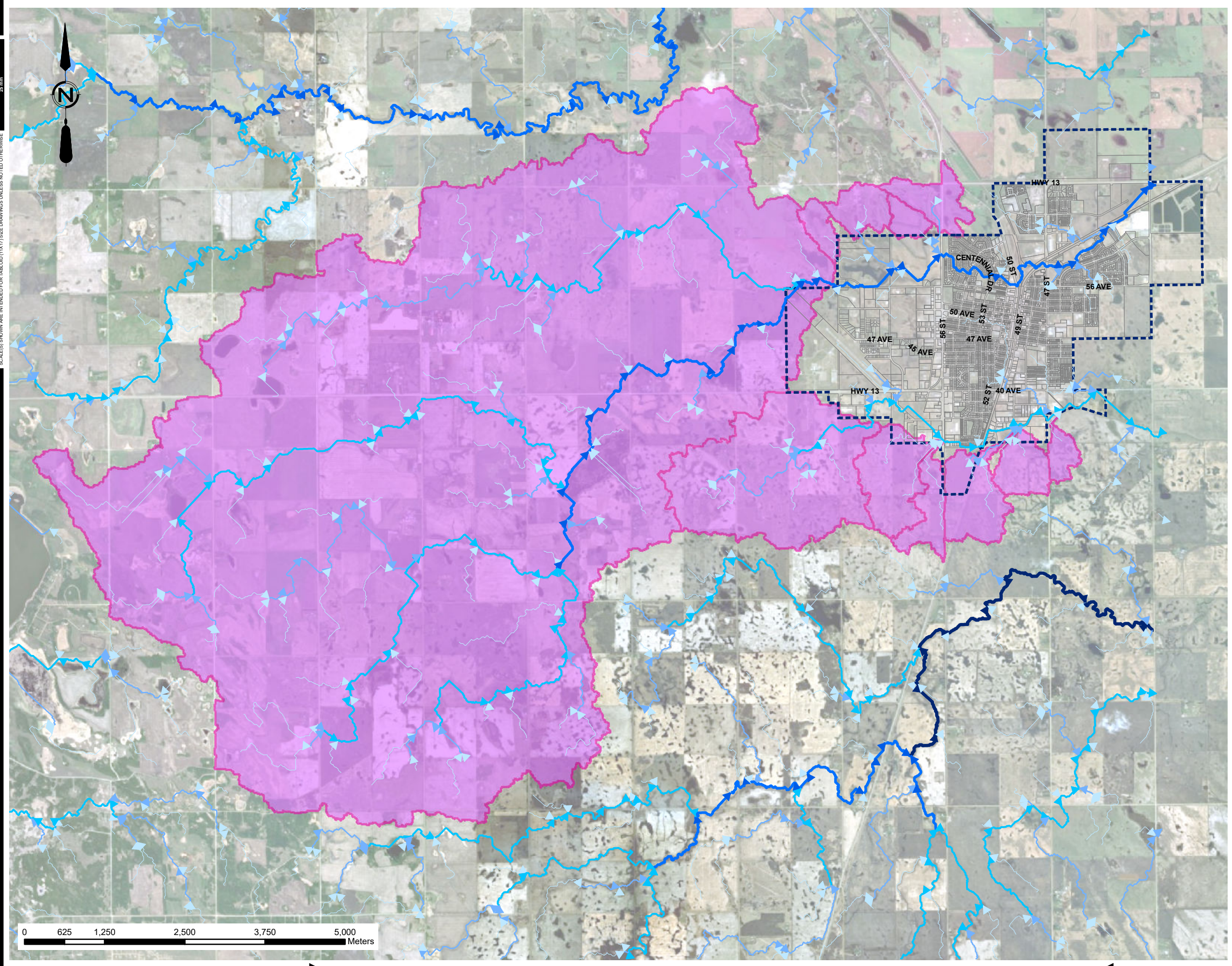


In general, head loss is an energy loss when water flows into a manhole or catchbasin. Head loss usually occurs when there is a change in water elevation, water flow direction, expansion, or contraction of the flow. There are several head loss types within MIKE URBAN. Head losses tend to be over-estimated within MIKE URBAN. Therefore, within the City of Wetaskiwin storm model, a “No Cross Section Changes” head loss type was used. This means that no head loss calculations were applied within the 1D pipe network. However, the outlets to the City’s existing drainage channels have a “MOUSE Classic Modified” head loss calculation with an outlet loss coefficient (K_m) of 1.0 and a full node area effective flow calculation.



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IF NOT 25 mm ADJUST SCALES
SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE



Legend

Delineated Streamlines (DEM15)

 Delineated 15m Catchments

 City of Wetaskiwin Boundary

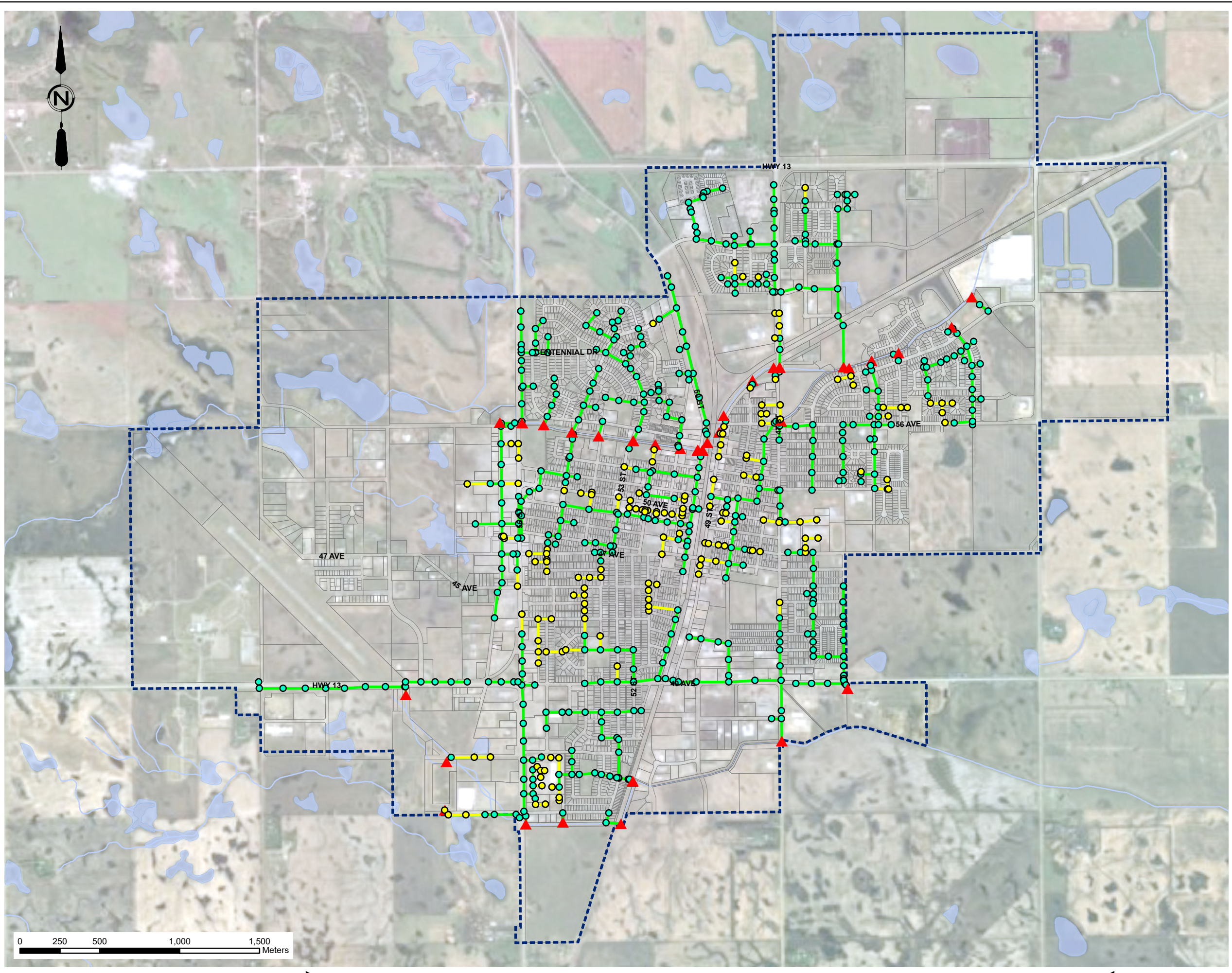


FIGURE 3-1

CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE

OFFSITE DRAINAGE AREAS

AE PROJECT No.	2019-3447
SCALE	1:57,000
APPROVED	
DATE	2020APR01
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Legend

- Storm Manhole/CB
- Storm Outfall
- Manhole added 2019
- Storm Pipe
- Storm Pipe added 2019
- Water Body
- City of Wetaskiwin Boundary



FIGURE 3-2

CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE

MIKE URBAN MODEL SCHEMATIC

AE PROJECT No.	2019-3447
SCALE	1:23,000
APPROVED	
DATE	2020APR09
REV	
DESCRIPTION	ISSUED FOR DRAFT

3.6 Mike 11 Model



A 1D MIKE 11 model was developed to represent and simulate flows through the culverts within the City of Wetaskiwin major drainage ditch network. MIKE 11 is a 1D model developed with a variety of basic modules each simulating a particular phenomenon within a river system. The hydrodynamic module within MIKE 11 uses an implicit finite difference scheme to solve the non-linear equations of open channel flow. It can simulate a fully dynamic wave approximation that accounts for backwater effects. In addition, the model can simulate a kinematic or diffusive wave approximation that simulates the principal routing processes which does not account for backwater effects or highly dynamic flow conditions. This study used the high order fully dynamic mode to simulate the flow through the large diameter culverts.

The City's major system includes a major ditch network which consists of three ditches: The North Central Ditch, the North Diversion Ditch, and the South Ditch as presented in [Figure 3-3](#). The North Central Ditch flows from the By the Lake outlet, east through the City, and eventually into Pipestone Creek. The North Diversion Ditch also flows from the By the Lake outlet. However, this diversion flows north and discharges into Bigstone Creek, north of the City limits. Bigstone Creek is a tributary of Pipestone Creek. The South Ditch flows east and eventually into an unnamed tributary to Pipestone Creek.

The flows within the ditch network were simulated in 2D which is discussed below. However, flows through the culverts within this network were represented and simulated within 1D using a combination of MIKE 11 and MIKE URBAN. The culverts within the City's ditch network included in the model are shown in [Figure 3-3](#). For this study, each culvert was given a unique identifier (ID). Detailed culvert characteristics are summarized in [Table 3-1](#). Culverts were surveyed by Associated Engineering in October 2019. Where data could not be collected, information was adopted from the City of Wetaskiwin Stormwater Management Master Plan Study (2004). Note, culverts located outside the City boundary were not included within the master plan model.



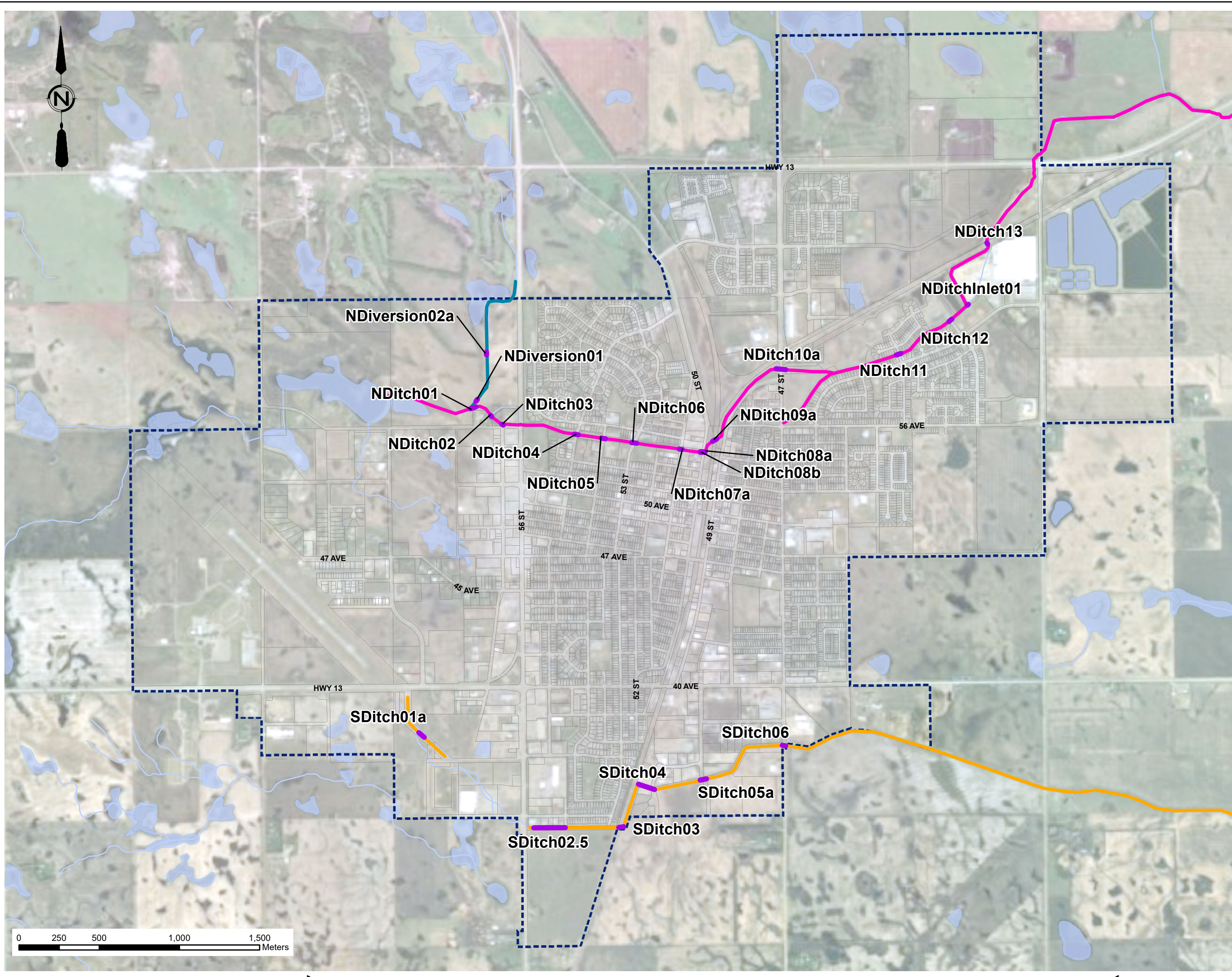
A control gate exists at the North Diversion/North Central Ditch confluence just east of By the Lake Park which controls flows into the North Central ditch. This gate appears to be manually operated in order to direct flows from the west (Wetaskiwin County) into the North Diversion Ditch and around the City. For this study, this gate was assumed to be closed and directing flows into the North Diversion Ditch for all modelled simulations.

The hydrodynamic module within the 1D MIKE 11 module requires that boundary conditions be specified upstream and downstream of the model extent. Water level boundaries upstream and downstream of each culvert within the City's ditch network were set to zero since the inlet and outlet conditions were governed by the water surface level within the 2D interface. The 2D model domain was depressed to elevations equal to the surveyed inverts at culvert inlets and outlets.



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IF NOT 25 mm ADJUST SCALES
SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE



Legend

- Modelled Culverts
- North Central Ditch
- North Diversion Ditch
- South Ditch
- Water Body
- City of Wetaskiwin Boundary



FIGURE 3-3

CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE

WETASKIWIN DITCH/CULVERT NETWORK

AE PROJECT No.	2019-3447
SCALE	1:23,000
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DATE	2020APR09
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DESCRIPTION	ISSUED FOR DRAFT

Table 3-1
City of Wetaskiwin Culvert Characteristics

Culvert ID	Size (mm)	No. of Barrels	Material	Length (m)	Upstream Invert (m)	Downstream Invert (m)	Slope (%)
NDitch01	900	1	CSP	36.83	760.590	760.460	0.35%
NDitch02	1500	1	CSP	9.54	760.266	760.395	-1.35%
NDitch03** (storm inlet)	1800	1	CSP	11.66	759.970	759.770	1.72%
NDitch04	1800	1	CSP	26.78	757.032	756.866	0.62%
NDitch05	1800	1	CSP	26.69	756.767	756.605	0.61%
NDitch06	1800	1	CSP	32.13	756.541	756.522	0.06%
NDitch07a	1500	2	CSP	18.68	756.201	756.154	0.25%
NDitch08**	1500	2	CSP	30.05	756.168	756.004	0.55%
NDitch09a	1500	2	CSP	30.28	755.665	755.425	0.79%
NDitch10a	1500	2	CSP	55.44	754.170	753.779	0.71%
NDitch11	2000	1	CSP	30.63	752.451	752.242	0.68%
NDitch12	2400W x 1800H	1	Concrete Box Section	24.11	751.146	751.181	-0.15%
NDitch13	2500W x 1500H*	1	Concrete Box Section*	10.50	751.075	751.043	0.30%
NDitchInlet01	2400W x 1800H	1	Concrete Box Section	6.92	751.700	751.451	3.60%
NDiversion01	3000	1	CSP	14.57	760.838	760.576	1.80%
NDiversion02a	2400	2	CSP	17.49	759.207	759.177	0.17%
SDitch01a	1000	2	CSP	44.70	756.797	756.679	0.26%
SDitch02.5	1300	1	CSP	196.01	754.820*	754.709	0.06%
SDitch03	1800	1	CSP	30.97	754.541	754.184	1.15%
SDitch04	2400	1	CSP	105.03	753.638	752.497	1.09%
SDitch05a	1200W x 1200H	2	Concrete Box Section	40.85	753.797	753.642	0.38%
SDitch06	1800W x 1200H*	1	CSP Pipe Arch Section	21.31	752.308	751.983	1.53%

*Data obtained from *City of Wetaskiwin Stormwater Management Master Plan Study (2004)*

**Modelled in MIKE URBAN

3.7 Mike 21 FM Model

The MIKE 21 FM model has a variety of basic modules each simulating a particular phenomenon within a river system or overland flow. The hydrodynamic module within MIKE 21 FM is based on solving the 2D shallow water equations (depth-integrated incompressible Reynolds averaged Navier-Stokes equations). Thus, the model consists of continuity, momentum, temperature, salinity and density equations.

The topography can be represented with a flexible mesh, quadrangular mesh, or a combination of both. These meshes are typically known as elements or cells. The 2D equations are solved from cell to cell to simulate flow, water level, velocity, and other hydraulic parameters. In general, it provides a more detailed representation of a domain (model extent) and result outputs when compared to 1D models. However, it requires a much longer computational time.

The developed 2D model encompasses the entire City of Wetaskiwin. As discussed above, the City's ditch network was modelled in 2D. The 1 m LiDAR data was used in model development which included stormwater within the ditch network as LiDAR does not penetrate water. Therefore, the surface was depressed at culvert, inlet, and outlet inverts for model stability. The ditches were not depressed in any other location. As previously mentioned, the culverts within this ditch network were modelled within the 1D MIKE 11 and MIKE URBAN models.

During a site visit conducted in April 2019, Associated Engineering personnel noted that there was standing water within the ditches and culverts. This supports the modelling assumptions. In addition, during the site visit, excessive vegetation growth and debris were found within some sections of the ditches and culverts. This has the potential to cause reductions in the ditch and culvert capacity.

The flexible element routine in MIKE 21 FM develops a combination of quadrangular and triangular elements in which the individual cell spacing is varied to better represent key features within the project area. The element sizes and arrangement affect the resolution of the final results and require a compromise between element resolution and computation times. This study used a combination of quadrangular and triangular elements within the developed model, with element sizes ranging from 5 to 75 m within the City domain. The ditches were modelled implicitly with breaklines along the ditch network at a 5 m spacing. **Figure 3-4** shows the 2D model domain and element routine used.

Spatially distributed roughness coefficients (Manning's n) and infiltration rates (for a 12 hour simulation) were applied to the 2D topography as shown in **Figure 3-5** and **Figure 3-6**. **Table 3-2** summarizes the roughness coefficients based on Chow et al (1988) and corresponding to the 2018 City of Wetaskiwin Municipal Engineering Standards and typical values used within the industry. Their locations are based on the City's land use (provided as shapefile). In addition,

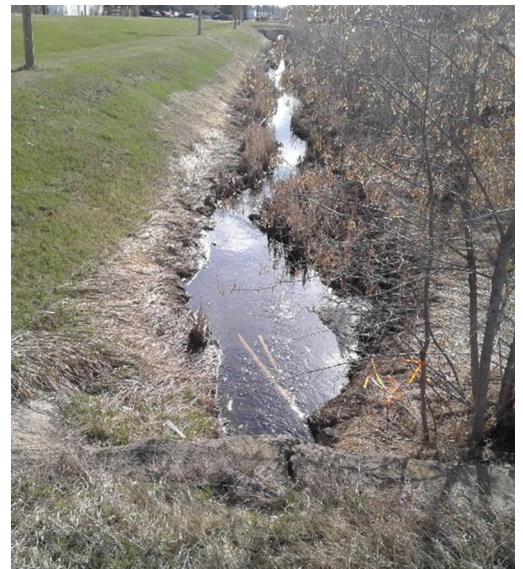


Table 3-2 includes the infiltration rates used within the model simulations. Initially these infiltration rates were based on the SCS Curve Numbers with an AMC II for Hydrologic Soil Group C. However, through an iterative process (described in [Section 5](#)), these values were refined to achieve a good fit between simulated and observed flows and levels.

Rainfall intensities were applied as a boundary condition on the entire 2D model surface, and, the stormwater system was dry at model start-up.

Point source inflows representing the off-site drainage from Wetaskiwin County were applied to the 2D MIKE 21 FM model. The locations of these point source hydrographs are illustrated in [Figure 3-4](#). These hydrographs were generated within the PCSWMM model with the same rainfall intensities applied within the coupled model interface. In order to be conservative, a boundary inflow was located downstream of the By-the-Lake Park waterbody.

3.8 Coupled Models

The developed MIKE URBAN, MIKE 11 and MIKE 21 FM models were coupled within the MIKE FLOOD interface. MIKE URBAN was coupled to the MIKE 21 FM model at manhole locations using the orifice equation. Detailed information on catchbasin locations was not available. Therefore, we expect coupling at every manhole with a maximum inlet capacity of $0.2552 \text{ m}^3/\text{s}$ to allow more stormwater runoff into the minor system than may occur in reality.

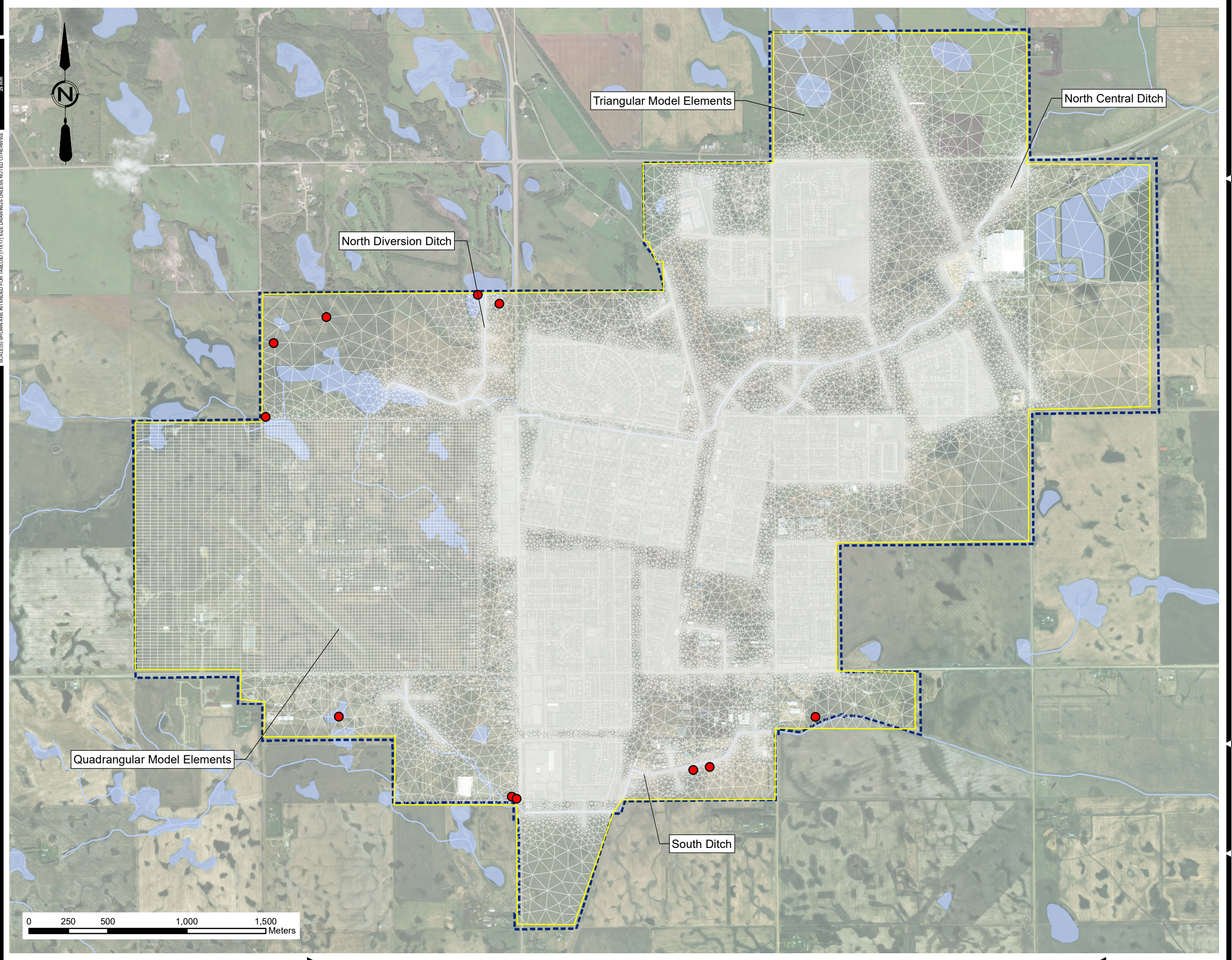
MIKE 11 culverts were coupled to the MIKE 21 FM model at upstream and downstream inverts. The MIKE 21 FM model mesh was depressed at each coupling location to the MIKE URBAN or MIKE 11 invert elevation to improve model stability. In addition, near the southeast model domain boundary, one MIKE URBAN outfall was coupled to a MIKE 11 culvert. Detailed link connections are included in [Appendix C](#).

As stated previously, it was assumed that the rule of operation for the control gate at the North Diversion/North Central Ditch confluence is to remain closed at all times in order to divert stormwater into the North Diversion Ditch.



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IF NOT 75 mm AS SHOWN ON THIS SCALE
SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE



- Legend**
- Off-site Drainage Inflow Location
 - Model Element
 - Model Domain
 - Water Body
 - City of Wetaskiwin Boundary

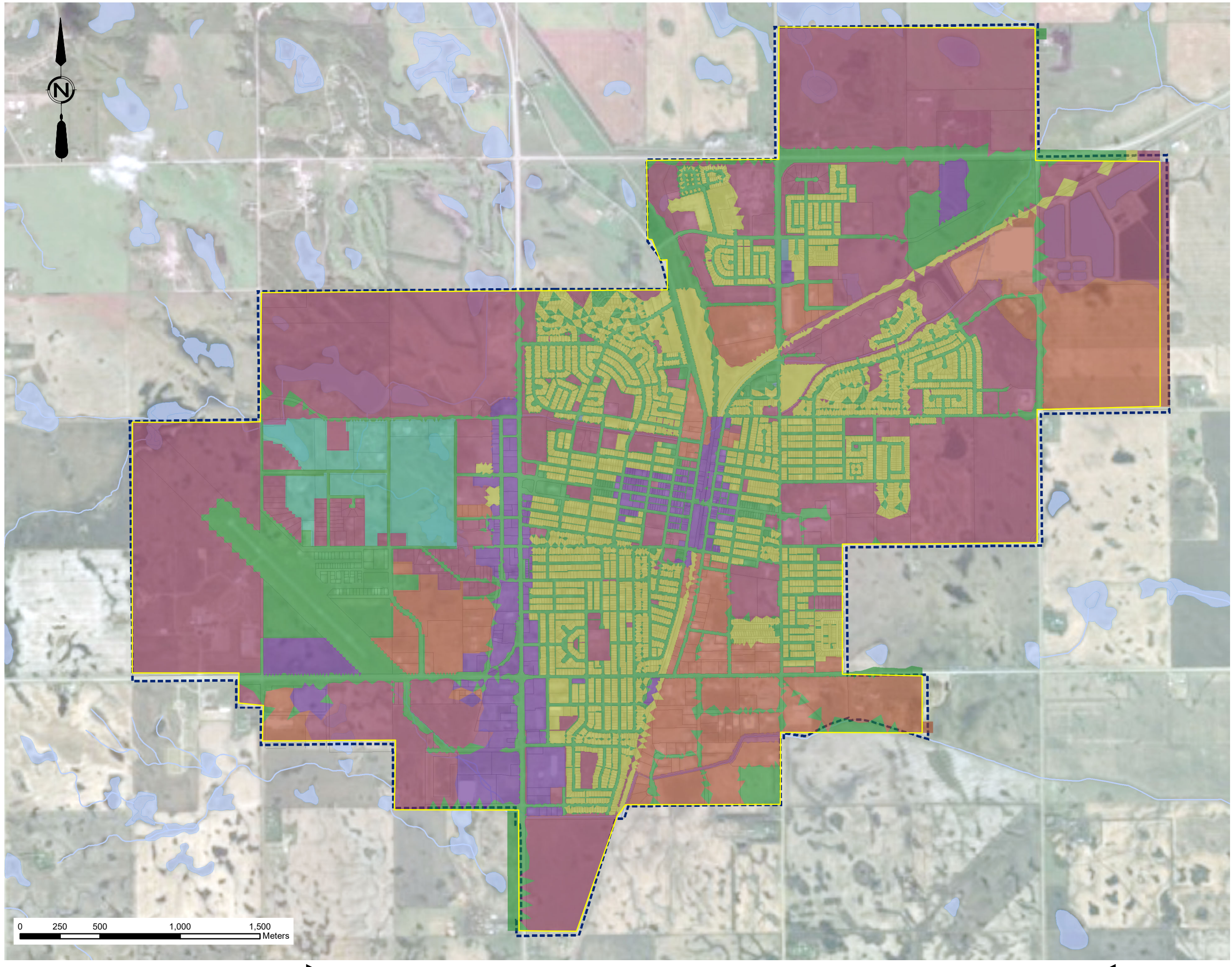


FIGURE 3-4
CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE
2D MODEL DOMAIN AND ELEMENT ROUTINE

AE PROJECT No.	2019-3447
SCALE	1:23,000
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DATE	2020APR09
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DESCRIPTION	ISSUED FOR DRAFT

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IF NOT 25 mm ASBESTUS SCALES
SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE



- Legend**
- Model Domain
 - Water Body
 - City of Wetaskiwin Boundary
- Manning's n**
- 0.010
 - 0.014
 - 0.020
 - 0.022
 - 0.025
 - 0.030
 - 0.080



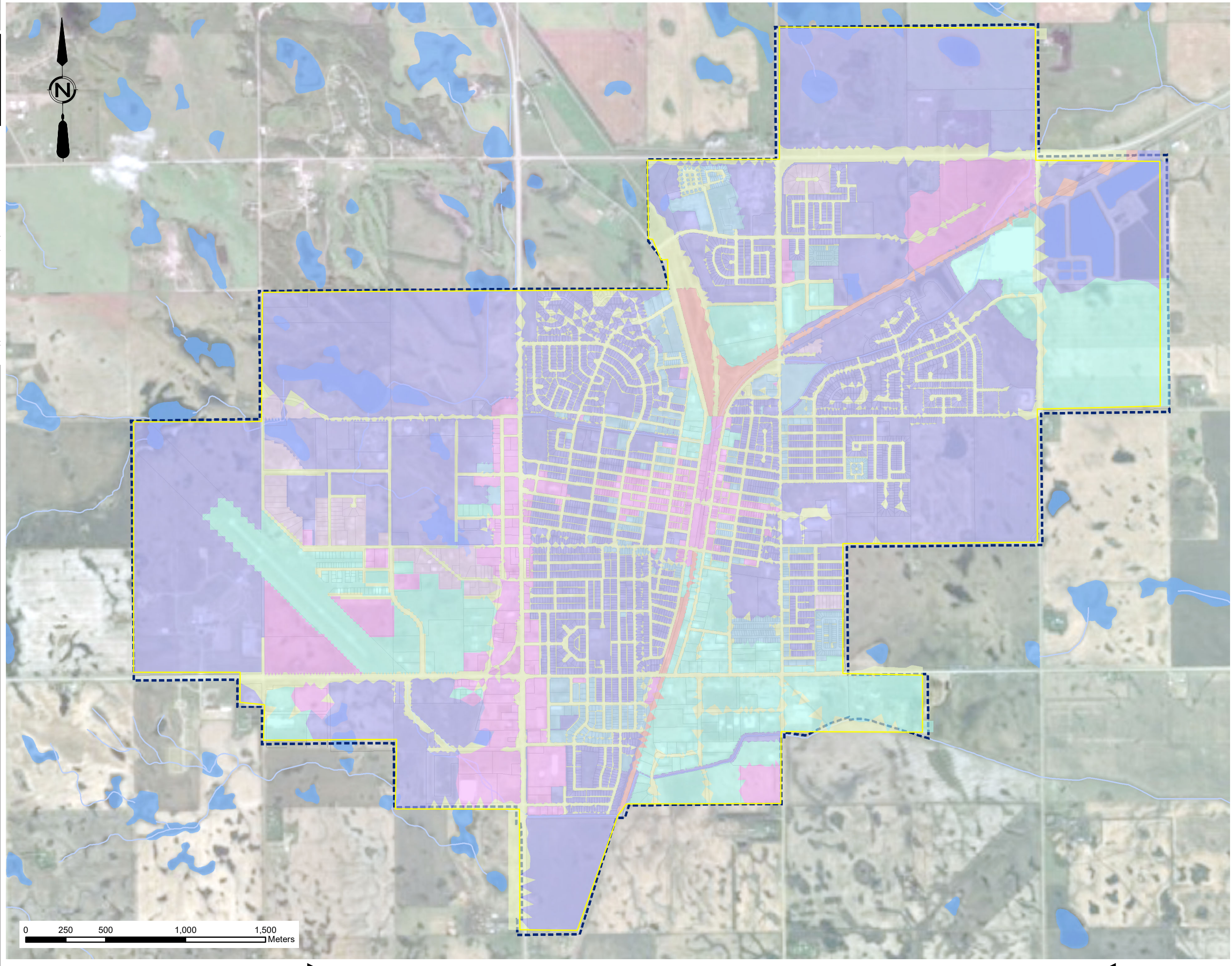
FIGURE 3-5
CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE

SPATIALLY DISTRIBUTED ROUGHNESS
COEFFICIENTS (MANNING'S N)

AE PROJECT No.	2019-3447
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DESCRIPTION	ISSUED FOR DRAFT

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IF NOT 25 mm ASBESTOS SCALES
SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE



Legend

- Model Domain
 - Water Body
 - City of Wetaskiwin Boundary
- Infiltration Rate mm/hr**
- 0.09
 - 0.27
 - 0.47
 - 0.52
 - 1.06
 - 1.34
 - 6.62



FIGURE 3-6

CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE

SPATIALLY DISTRIBUTED INFILTRATION
RATES

AE PROJECT No.	2019-3447
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DATE	2020APR09
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DESCRIPTION	ISSUED FOR DRAFT

Table 3-2
Model Roughness Coefficients and Infiltration Rates

Zoning Code	Zoning Description	Manning's n	Infiltration Rate (mm/hr)*
C1	Downtown Commercial	0.014	0.27
C2	General Commercial	0.014	0.27
C3	Highway Commercial	0.014	0.27
C4	Neighbourhood Commercial	0.022	0.27
C5	Shopping Centre Commercial	0.014	0.27
DC	Direct Control	0.010	0.27
DCEM	Direct Control Enviro Mgmt	0.080	6.62
DDO	Downtown District Overlay	0.014	0.27
M1	Light Industrial	0.020	1.34
M2	Heavy Industrial	0.020	1.34
M3	Airport Industrial	0.010	1.34
MH	Mobile Home in MHP – No Land	0.025	0.47
NZ	No Zoning	0.025	0.47
PUD	Planned Unit Develop DC	0.025	0.47
R1	Single Family Residential	0.025	6.62
R1A	Small Lot – Single Family Res	0.025	6.62
R1N	SFD Narrow Lot with Lane	0.025	6.62
R1R	Country Residential	0.03	1.06
R2	Low Density Residential	0.025	6.62
R3	Medium Density Residential	0.025	0.47
R4	High Density Residential	0.025	0.47
R5	Condominium Residential Dist	0.025	0.47
RE	Residential Estate	0.030	1.06
RMH	Mobile Home Park	0.025	0.47
RMX	Residential Mixed Use	0.030	1.06
TSC	Trans-Util Service Corridor	0.025	0.52
UR	Urban	0.030	6.62
US	Urban	0.030	6.62
	Roads	0.010	0.09
	Undefined	0.010	0.09

*Based on a 12 hour simulation

4 FLOW MONITORING PROGRAM

The City of Wetaskiwin retained Associated Engineering together with SFE Global Inc. (SFE) to conduct flow monitoring within the City between June and October 2019. The objective of the monitoring program was to collect reliable flow data within the storm sewer system for model calibration and validation of flows within the pipe network. The 2019 flow monitoring program consisted of five monitoring sites in strategically selected locations within the storm sewer system, as well as two tipping-bucket style rain gauges. SFE installed and operated the equipment. In addition, they collected the flow monitoring data and maintained an on-line data base. Associated Engineering provided the project management, final data processing, quality control and assurance, result analysis, and the final data base.

4.1 Flow Monitor and Rain Gauge Locations

Figure 4-1 shows the locations of flow monitors and rain gauges installed and operated in 2019. **Appendix D** provides detailed maps of each site. **Table 4-1** provides an overview of each site, its instrumentation and its operation. Associated Engineering chose flow monitor locations based on system characteristics (pipe size, slope), and catchment characteristics (land use, area). Monitor 1 was re-located in July due to backup of flows experienced early in the monitoring period.

4.2 Flow Monitor and Rain Gauge Catchment Areas

Figure 4-1 illustrates the estimated catchment areas delineated by Associated Engineering to each flow monitor. These catchments were based on the City's 1 m LiDAR data and Cadastre Zoning parcels shapefile. **Table 4-1** provides a summary of the estimated catchment area and land use for each flow monitor site.

The catchments for Flow Monitors 1, 2, 3, and 4 include a large proportion of residential area and Urban land use designation which includes public parks, schoolyards, and institutional land uses. Flow Monitor 4's catchment area also includes a large industrial area and some commercial land use. Conversely, Flow Monitor 5 services an almost exclusive commercial land use area along Highway 2A (56 Street).

In addition, **Figure 4-1** illustrates the approximate area of influence for Rain Gauges 1 and 2 determined using GIS tools. Due to the central location of Rain Gauge 2 on the City's Central Pump House, rain data from this gauge has been used for model calibration and validation as discussed in **Section 5**.

4.3 Instrumentation

4.3.1 Flow Monitors

Area-Velocity (AV) meters were used for each flow monitor site. These monitors use a pressure sensor to measure depth and an ultrasonic velocity sensor to measure velocity under both full and partial pipe flow gravity conditions. In addition, two meters were installed at each site in order to provide backup and quality control data where variable flows, sediment accumulations, or surcharge during severe storm events has the potential to occur.

All sites were inspected and cleaned during bi-weekly maintenance visits and manual depth and velocity measurements were taken to verify the meter operations and accuracy. Any

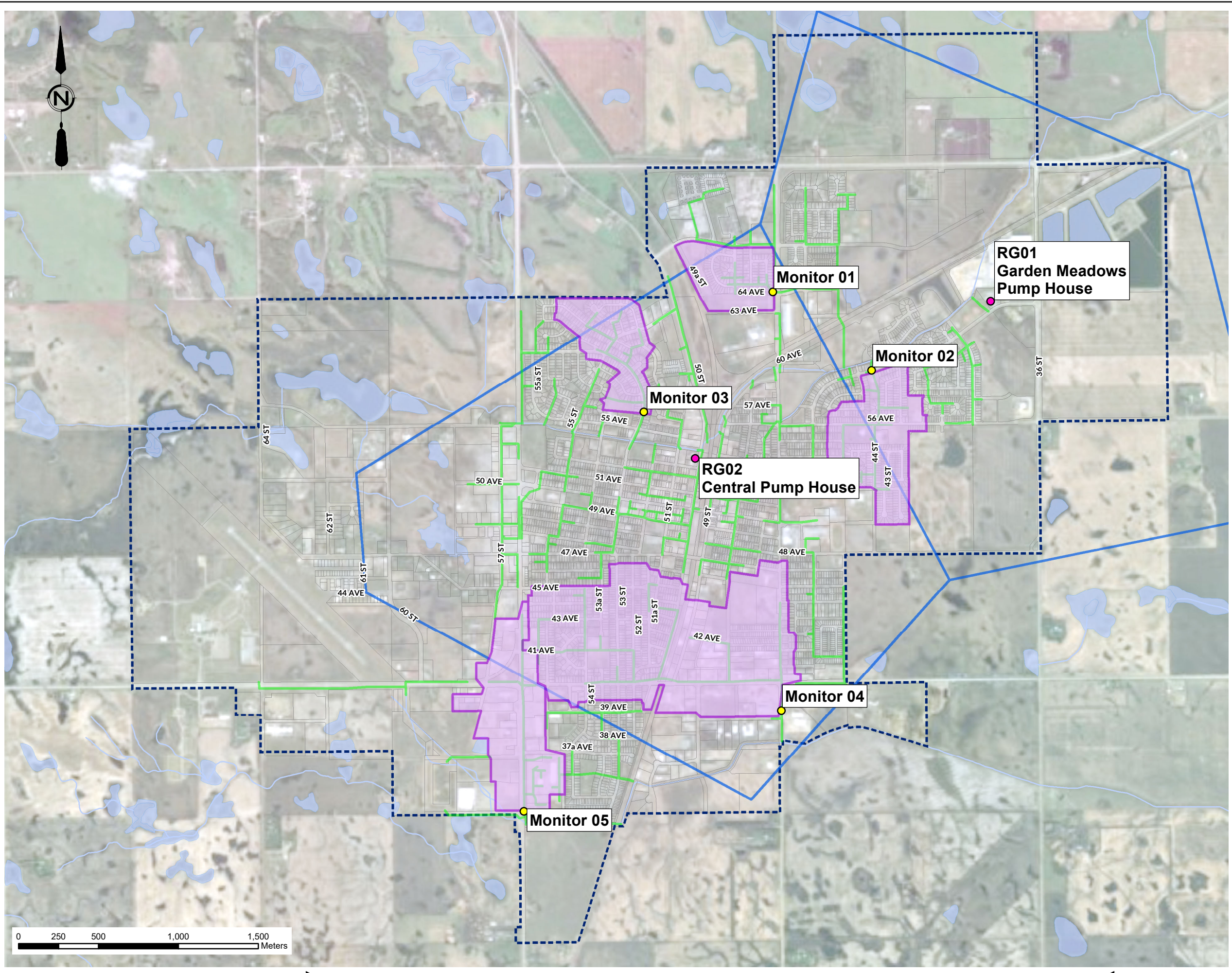


required repairs or replacements were made at these times. All monitor data was compiled automatically and posted online at least once per day. Associated Engineering reviewed this data bi-weekly for quality control and provided SFE with maintenance requests.

4.3.2 Rain Gauges

Two tipping-bucket rain gauges were installed to measure rainfall rates which vary substantially over time and space during storm events. They were installed on the Garden Meadows Pump House (RG01) and Central Pump House (RG02) as shown in **Figure 4-1**.

A tipping-bucket rain gauge records rainfall in 0.25 mm increments over 5-minute time periods. The operation and accuracy of each rain gauge was verified with a measured volume of water during installation and upon removal.



- Legend**
- Rain Gauges
 - Flow Monitor Locations
 - Estimated Monitor Catchment
 - Estimated Rain Gauge Area of Influence
 - Water Body
 - City of Wetaskiwin Boundary



FIGURE 4-1
CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE
2019 MONITORING PROGRAM LOCATIONS

AE PROJECT No.	2019-3447
SCALE	1:23,000
APPROVED	
DATE	2020APR09
REV	
DESCRIPTION	ISSUED FOR DRAFT

Table 4-1 Wetaskiwin Storm Flow Monitor Summary													
Monitoring Site	Pipe Size (mm)	Asset ID	LOCATION	Neighbourhood	Install Date	Removal Date	Total Catchment Area (ha)	Landuse Area (ha)					
								Residential	Commercial	Industrial	Urban (Institutional, Parks)	Unspecified	Roadway
1	750	N128	64 Ave at 47A Street	Northmount	12-Jul-19	30-Oct-19	21.3	8.1	0	0	9.0	0	4.1
02	1050	N268	Garden Meadows Drive between Garnet Crescent and Garwood Drive	Norwood / Garden Meadows	30-May-19	30-Oct-19	39.8	23.1	0	0	7.0	0.4	9.3
03	750	N38	Centennial Drive north of 55 Avenue	North / East / South Centennial	30-May-19	30-Oct-19	26.5	16.6	0	0	3.4	0	6.6
04	1200	N329	47 Street north of 39 Avenue	East / West Central / Spatinow / Parkdale / Central Industrial Park	30-May-19	30-Oct-19	129.0	46.6	2.0	33.2	13.6	0.5	33.0
05	1050	N405	56 Street and 36 Avenue	Highway - South / Lynalta	30-May-19	30-Oct-19	46.6	0	36.3	0.8	0.4	0	9.1
RG01			Garden Meadows Pump House	Parkside	31-May-19	30-Oct-19							
RG02			Central Pump House	City Center - North West	31-May-19	30-Oct-19							

4.4 Data Review

4.4.1 Flow Data Processing

Equipping monitoring sites with two flow monitors provides backup in the event of a flow monitor malfunction as well as an independent check to ensure accuracy of the monitor. In addition, manual depth and velocity measurements are made during each site maintenance visit to verify accuracy of the flow monitor. This protocol generates a large amount of data that needs to be processed to generate the final flows.

During the data processing stage, flow data is analyzed under an intensive Quality Control program to compile the final data base. Associated Engineering uses a custom-designed spreadsheet to complete data processing to produce efficient and consistent results.

The data processing involves the following:

- Confirming pipe sizes and monitor site locations;
- Screening to identify device malfunctions, and velocity drop-outs during low-flow periods;
- Scatterplot comparison of depth and velocity readings to check for consistency and stability and to substitute for velocity drop-outs;
- Comparison with manual measurements and rainfall data;
- Discarding or correcting suspect or unreliable data or adjusting depth or velocity readings where indicated
- Comparing depth and velocity measurements to calculate the final flow readings; and
- Reviewing the final flow data to ensure consistency.

In general, the flow data was found to be of good quality. Throughout the course of the monitoring period, some connectivity issues were experienced making real-time data review difficult. However, after download, the data required little to no modification during quality control reviews.

Backing up of flows within the storm sewer network was experienced at Monitor 1 (at 47 Street) during the beginning of the monitoring season. Consequently, Monitor 1 was moved to 47 Street and 64 Avenue in July. After relocation, flow data improved.

Standing water was observed by SFE at Monitors 2, 3, and 4 during the monitoring season. In addition, the flow monitoring data included some negative flows and suspect hydrographs indicating a potential backflow from time to time. This increases difficulty in calibration of the model and is discussed in [Section 5](#).

4.4.2 Rain Gauge Data Processing

Rainfall data processing involves comparison of validation measurements with meter readings and comparing the two rain gauges to check for suspect data. Overall, no data adjustments were required. However, data inconsistencies were noted from the end of September to October.

The collection of rainfall data is important for correlation with flow response within the storm sewer system.

Table 4-2 provides a summary of the daily rainfall amounts observed at the two rain gauges installed in 2019. Days that have significant rainfall (more than 10 mm) are highlighted in yellow. As discussed above, data inconsistencies from the end of September to October were noted. Therefore, data collected during this period was discounted. In addition, the cumulative rainfall is plotted in **Figure 4-3**.

As shown in **Table 4-2**, 11 storm events had more than 10 mm of rainfall at one or both rain gauges. The largest event occurred on July 6, generating between 33 mm of rainfall at the Garden Meadows Pump House and 47 mm of rainfall at the Central Pump House. This is approximately a 1:10-year design storm event. This will be discussed further in **Section 5**.

Table 4-2
Wetaskiwin Flow Monitoring Program
Rain Gauge Summary

	Daily Total Rainfall (mm)			Daily Total Rainfall (mm)			Daily Total Rainfall (mm)			Daily Total Rainfall (mm)			Daily Total Rainfall (mm)	
	RG01	RG02		RG01	RG02		RG01	RG02		RG01	RG02		RG01	RG02
31-May-19			30-Jun-19	0	0	31-Jul-19	0.0625	0.25	30-Aug-19	0.0625	0	30-Sep-19	18	3
1-Jun-19	5	5.75	1-Jul-19	4.25	5.25	1-Aug-19	2.125	10.25	31-Aug-19	0.3125	1.25	1-Oct-19	18	0.25
2-Jun-19	0	0	2-Jul-19	0	0	2-Aug-19	0.125	0.5	1-Sep-19	0	0.25	2-Oct-19	18	0.75
3-Jun-19	0	0	3-Jul-19	0	0	3-Aug-19	0.0625	0	2-Sep-19	0	0	3-Oct-19	18	0.25
4-Jun-19	0.25	0.25	4-Jul-19	0	0	4-Aug-19	0	0	3-Sep-19	0	0	4-Oct-19	18	0
5-Jun-19	0	0	5-Jul-19	0	0	5-Aug-19	0	0	4-Sep-19	0.8125	3.5	5-Oct-19	18	0
6-Jun-19	1	1.5	6-Jul-19	33.25	47.25	6-Aug-19	0.25	1.25	5-Sep-19	1.8125	7.25	6-Oct-19	18	0
7-Jun-19	28.5	31	7-Jul-19	16.75	20.5	7-Aug-19	0.0625	0	6-Sep-19	0	0	7-Oct-19	18	0
8-Jun-19	1.75	3.25	8-Jul-19	19	19	8-Aug-19	0.5625	2	7-Sep-19	1.6875	7.25	8-Oct-19	18	0.25
9-Jun-19	3.5	4	9-Jul-19	0	0	9-Aug-19	0.625	2.75	8-Sep-19	1.875	7.5	9-Oct-19	18	1.25
10-Jun-19	0	0	10-Jul-19	0	0	10-Aug-19	0	0	9-Sep-19	0.1875	0.75	10-Oct-19	18	44
11-Jun-19	0	0	11-Jul-19	1.5	3.25	11-Aug-19	1	5.75	10-Sep-19	0.875	3.25	11-Oct-19	18	20.25
12-Jun-19	0	0	12-Jul-19	0.25	0	12-Aug-19	0.0625	0	11-Sep-19	0	0.25	12-Oct-19	18	1.5
13-Jun-19	7	4.75	13-Jul-19	0	0	13-Aug-19	0.3125	1.75	12-Sep-19	0	0.5	13-Oct-19	18	0
14-Jun-19	2.75	4	14-Jul-19	0	0	14-Aug-19	0	0	13-Sep-19	0	0	14-Oct-19	18	0
15-Jun-19	0	0	15-Jul-19	27.5	3	15-Aug-19	0.1875	0.75	14-Sep-19	0	0.5	15-Oct-19	18	0
16-Jun-19	0	0	16-Jul-19	0.75	1	16-Aug-19	0.1875	0	15-Sep-19	0	0	16-Oct-19	18	0
17-Jun-19	0	0	17-Jul-19	0.25	0.25	17-Aug-19	0	0	16-Sep-19	1.0625	0	17-Oct-19	18	0
18-Jun-19	4.5	8.75	18-Jul-19	29	39.75	18-Aug-19	0.0625	0.5	17-Sep-19	0.4375	0	18-Oct-19	18	0
19-Jun-19	16.75	16.25	19-Jul-19	21.5	23	19-Aug-19	1.4375	4.25	18-Sep-19	0	0	19-Oct-19	18	0
20-Jun-19	5	5.5	20-Jul-19	0.25	0.25	20-Aug-19	0	0	19-Sep-19	0	0	20-Oct-19	18	0
21-Jun-19	2	1.25	21-Jul-19	0	0	21-Aug-19	0	0	20-Sep-19	0.5625	0	21-Oct-19	18	0
22-Jun-19	0	0	22-Jul-19	3.75	0	22-Aug-19	0	0	21-Sep-19	0	0	22-Oct-19	18	0
23-Jun-19	1.5	3.25	23-Jul-19	0	0	23-Aug-19	0.875	3.5	22-Sep-19	0	0	23-Oct-19	18	0
24-Jun-19	0.75	0.5	24-Jul-19	2.75	5	24-Aug-19	1.5625	6.25	23-Sep-19	9.1875	0	24-Oct-19	18	0
25-Jun-19	0.5	0.75	25-Jul-19	10.25	9.5	25-Aug-19	0	0	24-Sep-19	18	0	25-Oct-19	18	0
26-Jun-19	0.25	0.25	26-Jul-19	0	0	26-Aug-19	0	0	25-Sep-19	18	0	26-Oct-19	18	0
27-Jun-19	10.5	10.75	27-Jul-19	2.25	2.75	27-Aug-19	0	0	26-Sep-19	18	0	27-Oct-19	18	0
28-Jun-19	1.75	0.5	28-Jul-19	2	1.5	28-Aug-19	0.0625	0.25	27-Sep-19	18	0	28-Oct-19	18	0
29-Jun-19	0.75	1	29-Jul-19	0.5	0.25	29-Aug-19	0	0	28-Sep-19	18	0	29-Oct-19	18	0
			30-Jul-19	0	0				29-Sep-19	18	618.75	30-Oct-19	17.9375	0
Month Total	94	103		176	182		10	40		19	32			

21

Rainfall exceeding 10 mm

Rainfall discounted

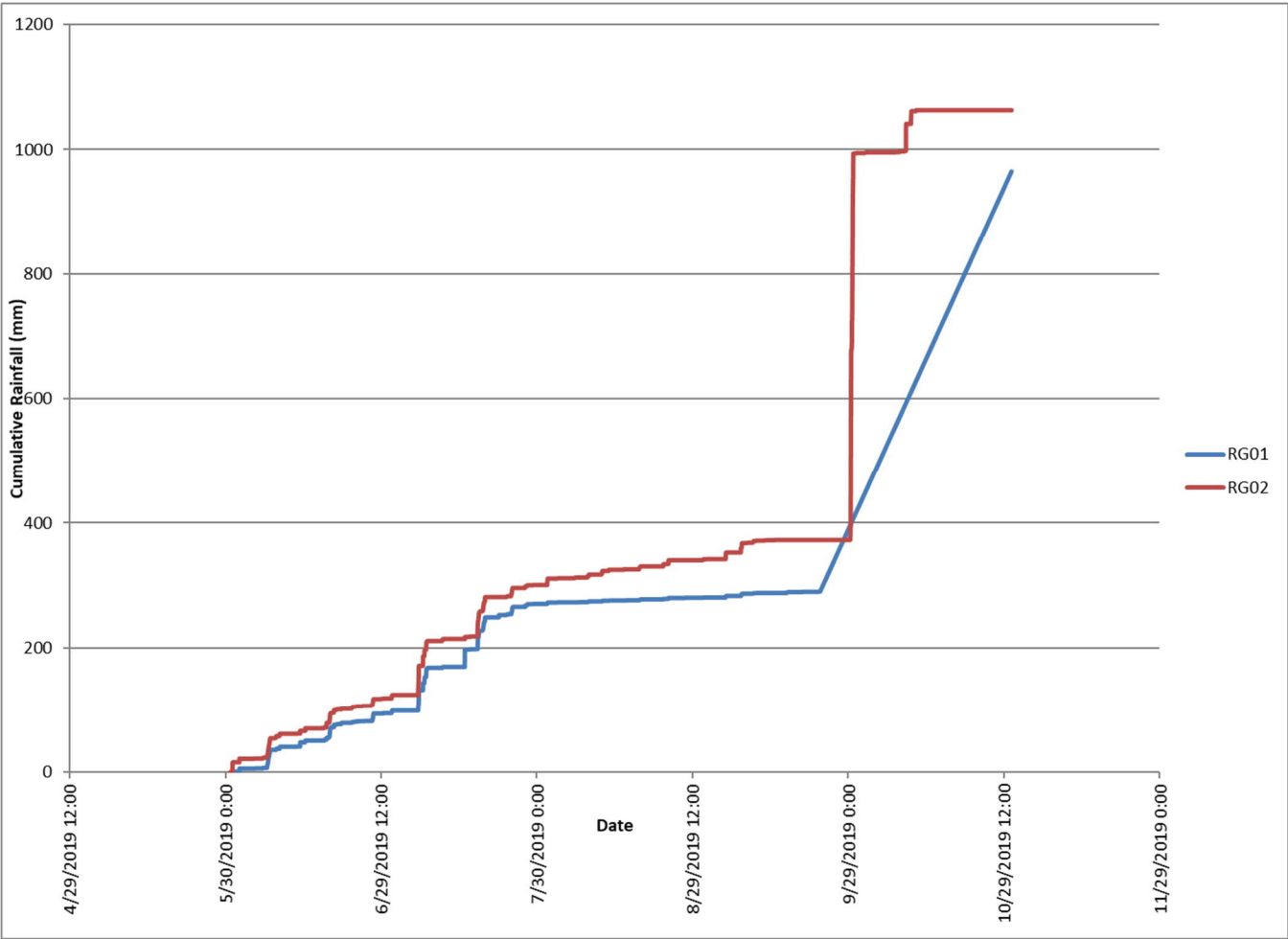


Figure 4-2
Wetaskiwin Cumulative Rainfall

5 MODEL CALIBRATION AND VALIDATION

Calibration and validation of a hydrologic and hydraulic model is imperative to provide results with confidence and reliability. The rainfall data collected during the 2019 Flow Monitoring Program was reviewed, as per [Section 4](#). Significant events were chosen for model calibration and validation purposes. [Table 5-1](#) provides a summary of the rainfall characteristics for the three largest storm events occurring over the season (June to October).

The coupled 1D-2D model was simulated for the three chosen storm events to provide calibration and validation of the storm sewer network. A comparison of the IDF profiles for the City of Edmonton (2018) and the observed storm events are presented in [Figure 5-1](#).

Table 5-1
Calibration/Validation Storm Events Summary

Date	Rainfall Depth (mm)	Peak intensity (mm/hr)	Duration (hours)	Approximate Design Storm Event Return Period (years)
June 7, 2019 ^V	31.0	6	16	< 1:2
July 6, 2019 ^C	47.2	105	4	1:10
July 18, 2019 ^V	39.7	60	10	1:2

C – Calibration Simulation

V – Validation Simulation

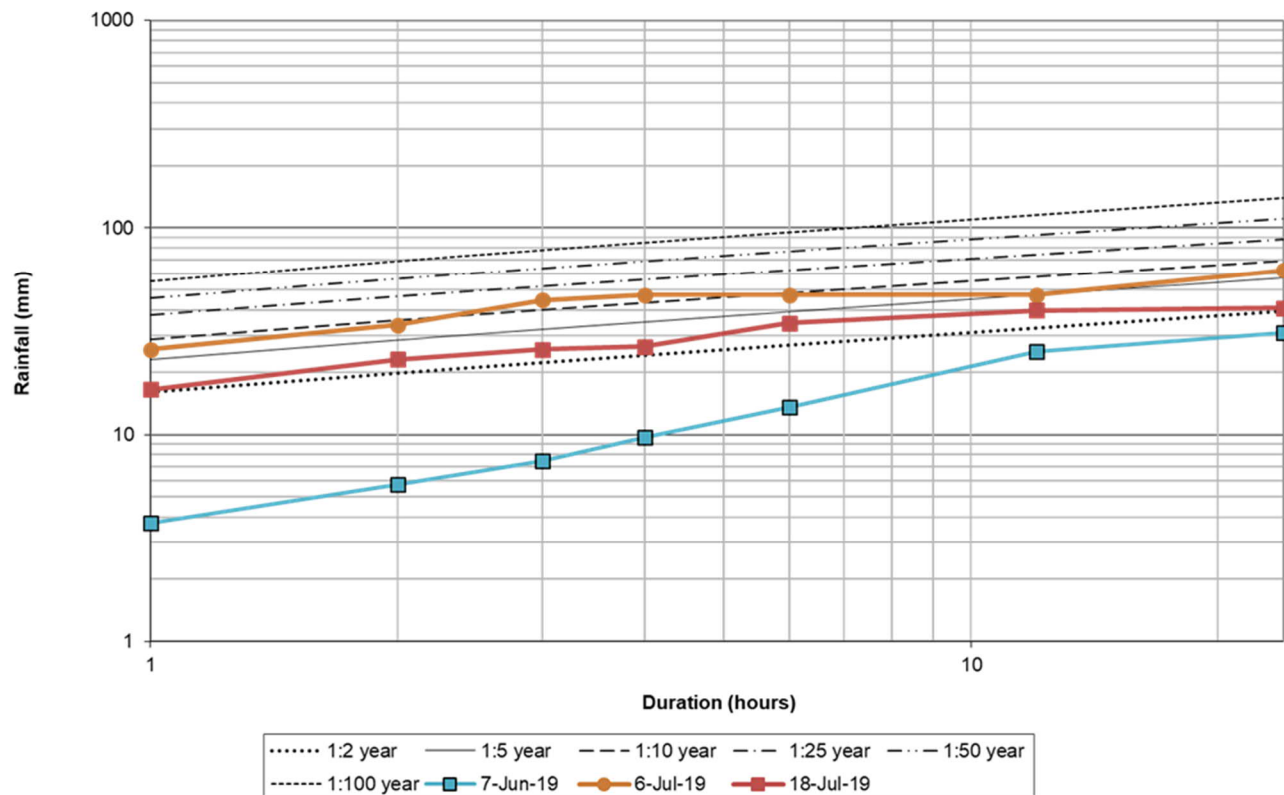


Figure 5-1
Storm IDF Profiles (RG02)

5.1 Model Calibration

The largest storm event during the flow monitoring period occurred on July 6, 2019. This event was chosen for the calibration of the developed model. This storm was approximately equivalent to a 1:10-year return period design storm event. This storm event was simulated and model parameters were adjusted through an iterative process to achieve the best match between simulated and observed results (flow and water level).

The parameters used for calibration included infiltration rates varying within the model domain according to the land use and maximum inlet flow at coupled locations between the 1D MIKE URBAN and 2D MIKE 21 FM models. The final infiltration rates used within the model are included in [Figure 3-6](#).

The infiltration rates were initially estimated using the SCS Curve Numbers for Urban Areas with an AMC II for Hydrologic Soil Group C. The curve numbers were modified within residential, urban (institutional and park areas), and industrial land use areas for calibration purposes.

The coupling locations were assumed to be catchbasin type F51 based on the *Capacities of Catch Basin Inlets Use in Alberta* (Wilson, D.). Initially, Associated Engineering assumed that coupling locations had a maximum inlet capacity of 0.366 m³/s (corresponding to 25.40 cm ponding depth). However, through the iterative process and model simulations, the maximum inlet capacity was reduced to be 0.255 m³/s, which corresponds to a depth of ponding of 15.24 cm or effectively curb height. This parameter provided a better fit between simulated and observed flows,

noting that these flows may be greater than inflows through different catchbasin types. It is recommended that the model be refined with catchbasin location and type as the information becomes available.

Appendix E provides comparison plots for the calibration results at all monitor locations.

The calibration results show that the modelled peak flows are overestimated by 30% on average. This can be attributed to the physically based modelling approach, estimated infiltration rates, lack of catchbasin locations, catchbasin type, and the coupling locations. This would tend to allow more stormwater to enter into the minor system than may occur in reality. As stated previously, Associated Engineering recommends that as more detailed information becomes available about the location and type of catchbasins within the City, the model be refined and coupled only at appropriate locations.

To confirm the hypothesis that coupling at all manhole locations increases system inflows, manholes within Monitor catchments 2 and 4 were disconnected where no simulated ponding was shown in model results. The difference this makes is illustrated in **Figure 5-2**. This figure shows that the simulated flows at Monitor 2 decreased by 30%.

Figures 5-3 and **5-4** provide an example of calibrated results at flow Monitors 2 and 5 during the July 6 storm event, respectively. The catchment of Monitor 2 consists of residential with some urban (institutional) land uses. Results show that the simulated peak flow at this location was approximately 30% greater than the observed flow specifically, at the third peak of the storm event. However, timing of the three simulated peaks achieved a good fit with the observed data. The catchment of Monitor 5 consists of predominantly highway commercial land use. The simulated third peak flow at this location was approximately 5% less than the observed flow. Similar to monitor 2, the timing of the three simulated peaks achieved a good fit.

The first peak during the calibration storm event is not matched well at Monitors 2, 3, and 4. This might be due to the inability to accurately represent initial soil conditions (antecedent soil moisture) within the model. This hypothesis is validated when the hydrograph at Monitor 5 is considered, where the first peak is matched well. This is due to the predominantly commercial land use within the catchment, resulting in less pervious areas subject to the initial soil conditions. Note, the timing and shape of all simulated hydrographs match reasonably well at all monitor sites between the observed and simulated data.

As discussed previously, Monitor 1 was relocated after the July 6 storm event; therefore, was not compared for model calibration.

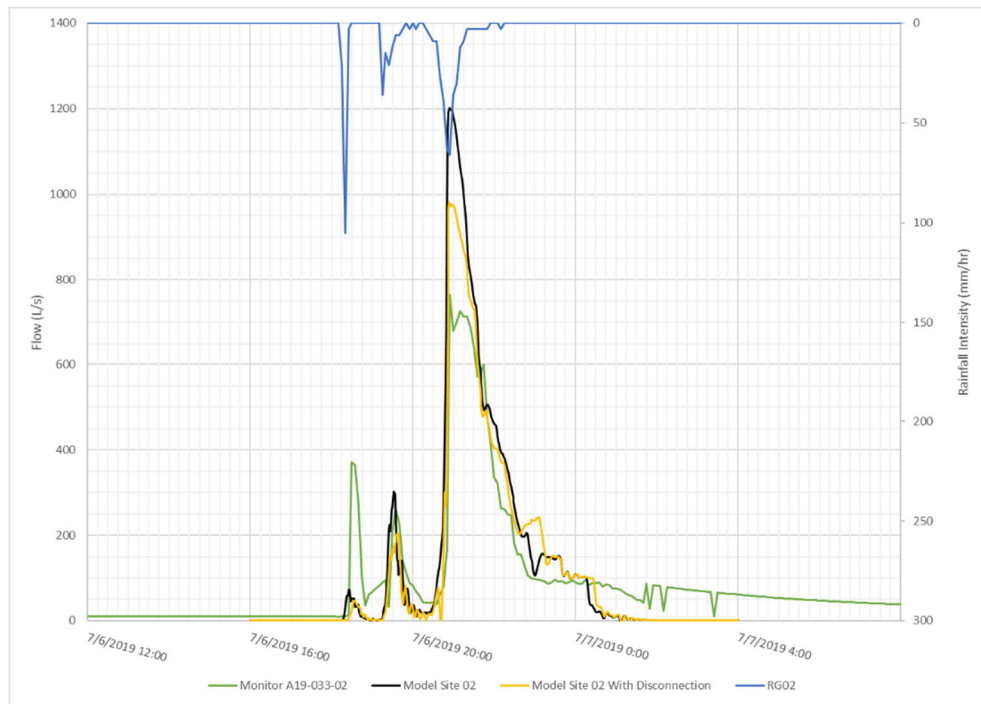


Figure 5-2
Monitor 2 Calibration Iteration Disconnecting Coupling Locations – July 6, 2019

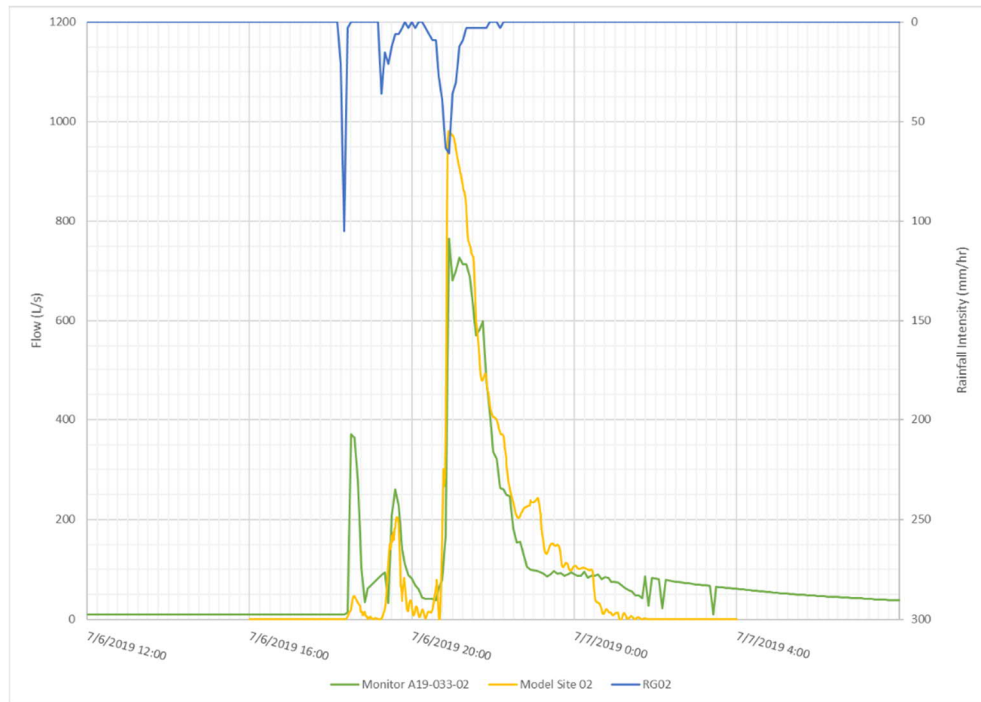


Figure 5-3
July 6, 2019 Calibration Flow Hydrograph for Flow Monitor 2

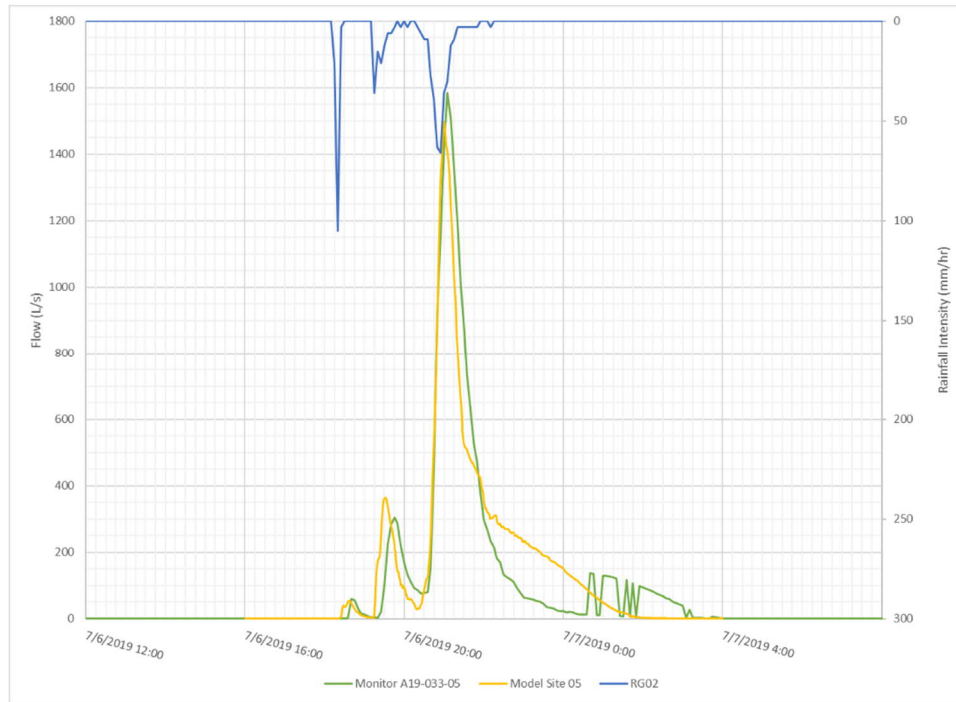


Figure 5-4
July 6, 2019 Calibration Flow Hydrograph for Flow Monitor 5

5.2 Model Validation

Validation runs were simulated for the July 18 and June 7, 2019 storm events. The July 18 event was approximately equivalent to a 1:2-year return period design storm event, while the June 7 event was less than a 1:2-year design storm event ([Figure 5-1](#)).

[Appendix E](#) provides comparison plots for the validation results at all monitor locations.

A reasonable match was achieved between simulated and observed flows at all monitor locations. An example of each validation event is illustrated in [Figure 5-5](#) and [5-6](#). Similar to the calibration results, validation flows overestimated on average by 30%. Additionally, the first peaks did not match well, and as previously stated, this was most likely due to the representation of the initial soil conditions (antecedent soil moisture) within the model. The events simulated for validation were smaller than the event chosen for model calibration. Therefore, as presented in [Figure 5-6](#), a slower response in the first simulated hydrograph peak can be attributed to the lower storm intensity and the land use in the Monitor 5 catchment. This is contrasted to the response time illustrated during the calibration simulation at this location. Note, the timing and shape of the simulated hydrographs remains consistent between simulated results and observed data.

In addition, the City confirmed that the storm event that occurred on July 6 caused significant flooding on most streets within the City. The City's review confirmed modelled flooding locations. Therefore, this provides additional validation of the model results.

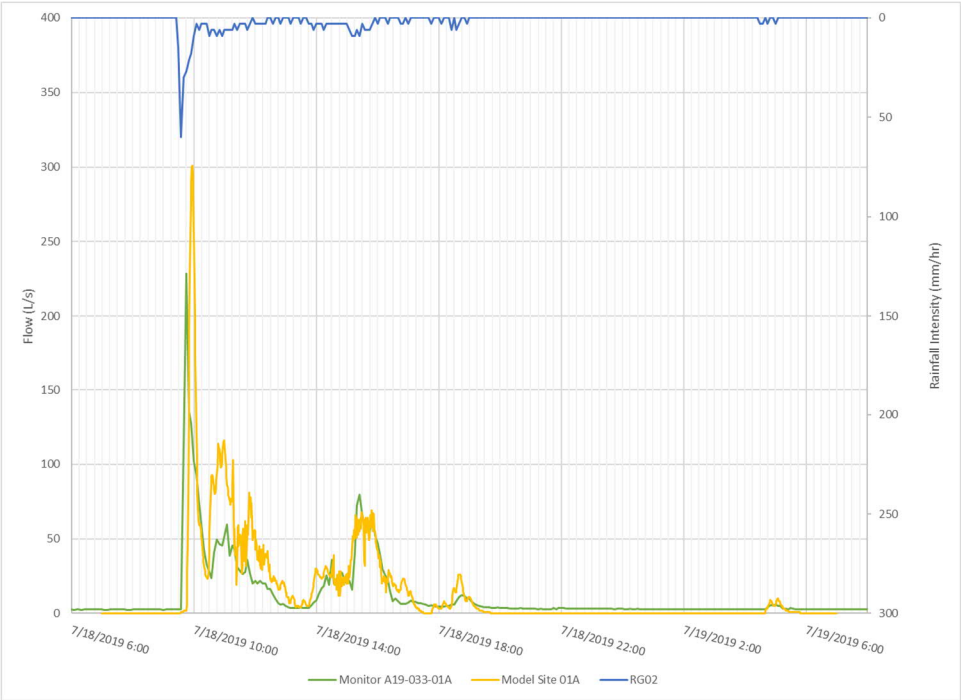


Figure 5-5
July 18, 2019 Validation Flow Hydrograph at Monitor 1

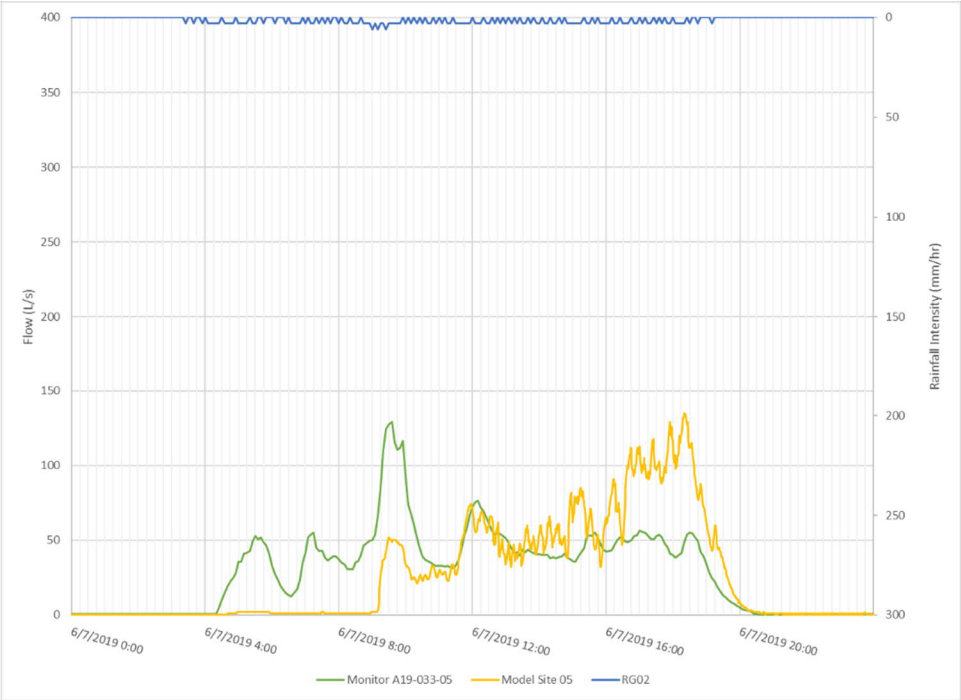


Figure 5-6
June 7, 2019 Validation Flow Hydrograph at Monitor 5

As discussed above, some backflow is suspected at Monitors 2, 3, and 4 throughout the monitoring period. This makes calibration and validation of the model challenging. This might be as a result of blockages within the pipe, inadequate downstream pipe capacity, or storm pipe deterioration. Associated Engineering recommends the City undertake a CCTV inspection program of their storm system to ensure no blockages or pipe deterioration exists.

This coupled 1D-2D model provides a conservative estimate of flows within the City's storm sewer system. Note, perfect agreement is not realistic at all locations for the various storm events. This is due to the spatial variability of rainfall intensities, model approximations, and limitations. Based on this, Associated Engineering recommends additional calibration and validation as more observed data becomes available. Therefore, it is recommended that the City implement a continuous flow monitoring program.

6 EXISTING SYSTEM PERFORMANCE

Following calibration and validation of the coupled 1D-2D model, the following design storm events were simulated using the 2018 EPCOR IDF curves for the City of Edmonton:

- 1:5-year 4 hour
- 1:100-year 4 hour

These storm event simulations were used to assess the existing storm system.

The model results for each node (manhole/catchbasin) were converted into Grade Line Factors (GLF) to illustrate surcharge levels. The GLF indicates the water level within each manhole, expressed in metres below ground. Areas where the system is surcharged to within 2.0 m of the ground elevation provide potential risk of basement flooding if storm services are directly connected to the storm sewer.

The model results for each storm pipe are presented as either flowing full or not flowing full based on the Hydraulic Grade Line (HGL) being above or below pipe crown. Storm pipes flowing full at either their upstream crown, downstream crown, or both are coloured red. Pipes with HGL below both the upstream and downstream crown elevation are coloured green.

Figures 6-1a and **6-2a** present the coupled 1D-2D storm surface ponding and manhole surcharge levels (GLF) for the 1:5-year and 1:100-year 4 hour design storm events, respectively. **Figures 6-1b** and **6-2b** present surface ponding and storm pipe results for the 1:5-year and 1:100-year 4 hour design storm events, respectively.

Results for both design storm events show overloading of the storm sewers within the core of the City. This is consistent with results presented in the Stormwater Management Master Plan Study (2004) which stated the original design for the storm system within the City corresponded to a 1:1-year or 1:2-year design storm event.

Surface ponding on streets agrees with the 1D MIKE URBAN model results. This indicates a good interaction between the coupled models. Note, the 2D surface ponding results presented are only for depths above 5 cm. It was assumed that ponding levels below 5 cm are not critical for this assessment. In addition, surface ponding within the 2D simulated results might not be realistic at low spots where adequate information (manhole/catchbasin) is not available. As more network detail becomes available, this model should be updated.

Table 6-1 summarizes the maximum simulated water level and discharge results for each culvert modelled within the City's ditch network. Water levels greater than the culvert crown are highlighted in red during the 1:5-year and 1:100-year 4 hour design storm events. This represents locations where culverts are flowing full and may create backup of flows within the ditch network. During the 1:100-year design storm event, the majority of the culverts are at their full capacity. During the 1:5-year design storm event, results indicate that the majority of the culverts have the capacity to convey flows generated during this design storm event.

As noted previously, the calibrated model overestimates flows within the minor storm system by 30% on average. To assess the impact of the overestimation within the existing minor system assessment, a simulation with a 30% reduction in the 1:5-year design storm event rainfall intensity was considered. The 1:2-year design storm event was also found to be approximately 30% less than the 1:5-year design storm event. Therefore, the 1:2-year 4 hour design storm event based on the 2018 EPCOR IDF curves was also simulated. Results from the 1:2-year 4 hour design storm

event are presented in [Figures 6-3a](#) and [6-3b](#). Results show that there are still minor surface ponding areas within the City during this event. Approximately half of the City's pipes are still running over capacity. In addition, there are isolated areas with manholes surcharged to ground. Therefore, Associated Engineering concluded that the City's storm system was not designed for the 1:5-year design storm event.

Pipe capacity results for the validation event simulation on June 7, 2019 are presented in [Figure 6-4](#). Based on these results and considering model overestimation, the majority of the City's storm networks (both minor and major) provide capacity up to approximately the 1:2-year 4 hour design storm event.

The majority of the City's culverts have capacity for up to the 1:5-year design storm event, assuming the ditches and culverts are kept clear of debris and excessive vegetation. Based on site visits conducted in 2019, the City's ditch and culvert network were found to have debris and vegetative growth which may be reducing the capacities. This network should be maintained to mitigate backup of flows into the minor system and to facilitate stormwater flows through the City. Note, the ditch network was not depressed in the coupled 1D-2D model as discussed in [Section 3](#).

There are a large number of shallow manholes within the City (less than 2 m depth). These provide concerns for both surface and basement flooding, as well as an opportunity for freeze-up during cold weather due to the lack of ground cover.

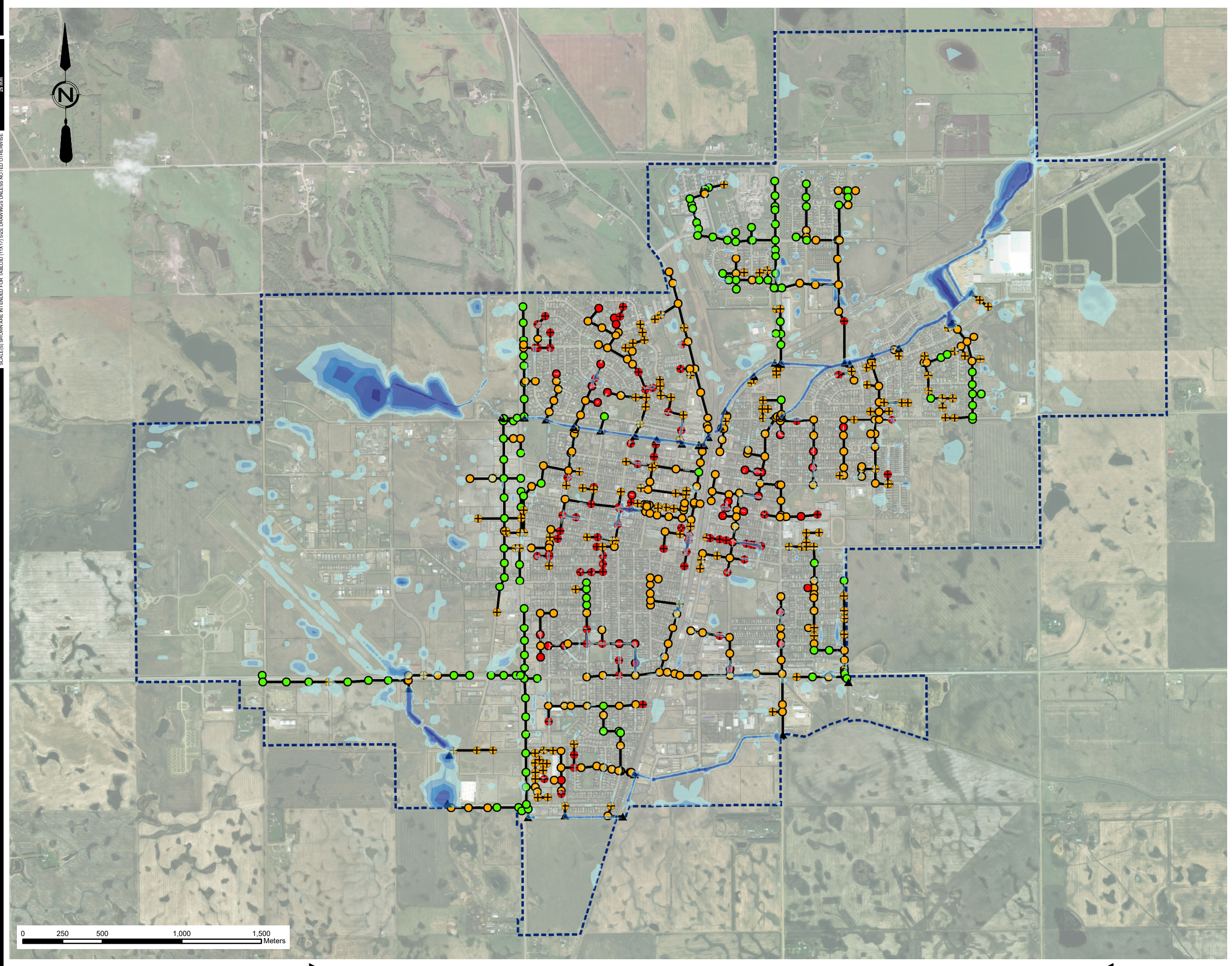
Table 6-1
Existing System Performance – City of Wetaskiwin Culverts

		1:100-Year 4 Hour		1:5-Year 4 Hour	
Culvert ID	Culvert Crown Elevation (Upstream)	Maximum Simulated Discharge	Maximum Simulated Water Level (Upstream)	Maximum Simulated Discharge	Maximum Simulated Water Level (Upstream)
	m	m ³ /s	m	m ³ /s	m
NDitch01	761.49	0.00	760.59	0.00	760.59
NDitch02	761.77	1.00	761.42	0.00	760.55
NDitch03** (storm inlet)	761.77	0.46	760.31	0.05	760.08
NDitch04	758.83	4.68	759.17	2.04	758.60
NDitch05	758.57	4.26	758.86	1.82	758.06
NDitch06	758.34	2.77	758.51	1.54	757.94
NDitch07a	757.70	4.96	758.49	4.87	757.88
NDitch08**	757.67	3.16	758.38	0.97	757.63
NDitch09a	757.17	8.67	757.97	0.68	757.08
NDitch10a	755.67	9.09	756.57	3.65	755.24
NDitch11	754.45	8.76	754.94	4.54	754.03
NDitch12	752.95	0.03	753.54	0.00	753.11
NDitch13	752.58	10.83	753.42	5.50	752.46
NDitchInlet01	753.50	0.16	753.34	0.00	752.60
NDiversion01	763.84	7.39	762.65	0.10	761.30
NDiversion02a	761.61	0.00	760.82	0.00	759.93
SDitch01a	757.80	3.26	759.28	2.06	758.48
SDitch02.5	756.12	1.66	757.23	0.77	756.22
SDitch03	756.34	4.40	756.23	1.79	755.55
SDitch04	756.04	6.54	755.78	2.05	755.12
SDitch05a	755.00	0.00	755.44	3.20	755.13
SDitch06	753.51	5.44	755.04	1.13	752.88

**Modelled in MIKE URBAN

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IF NOT 25 mm ASBESTUS SCALES
SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE



Legend

Surcharge Level (m below ground)

- < 0
- 0 - 2
- > 2
- ⊕ Shallow (<2m Depth)
- ▲ Storm Outfall/Inlet
- Storm Pipe
- City of Wetaskiwin Boundary

Surface Ponding (Depth, m)

- 0.05 - 0.15
- > 0.15 - 0.30
- > 0.30 - 0.50
- > 0.50 - 1.00
- < 1.00



FIGURE 6-1A

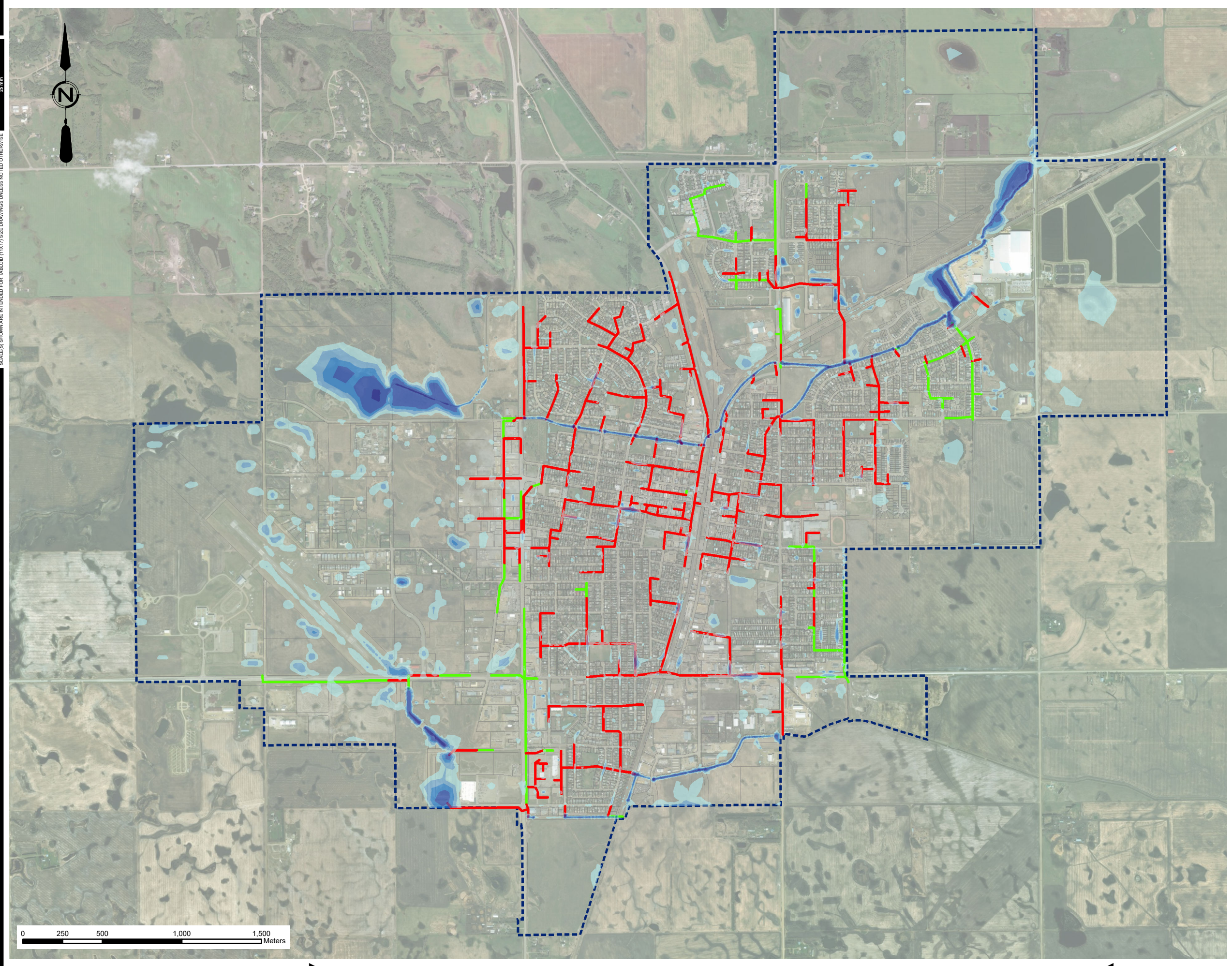
CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE

COUPLED 1D - 2D MODEL
RESULTS: 1:5 YEAR 4 HOUR

AE PROJECT No.	2019-3447
SCALE	1:23,000
APPROVED	
DATE	2020APR09
REV	
DESCRIPTION	ISSUED FOR DRAFT

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IF NOT 25 mm ASBESTUS SCALES
SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE



Legend

Pipe Identification

- Full
- Not Full
- City of Wetaskiwin Boundary

Surface Ponding (Depth, m)

- 0.05 - 0.15
- > 0.15 - 0.30
- > 0.30 - 0.50
- > 0.50 - 1.00
- < 1.00



FIGURE 6-1B

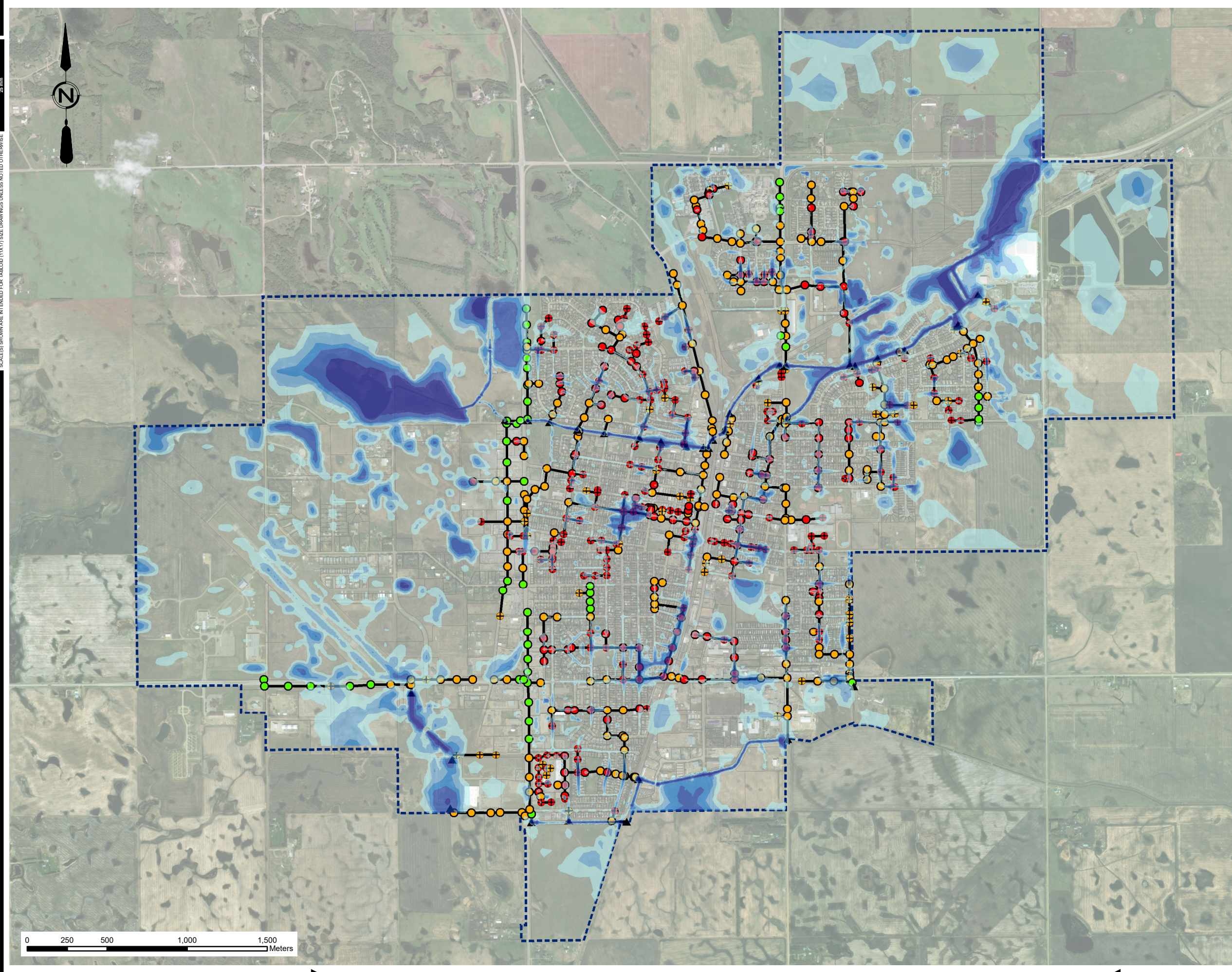
CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE

COUPLED 1D - 2D MODEL
RESULTS: 1:5 YEAR 4 HOUR

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APPROVED	
DATE	2020APR09
REV	
DESCRIPTION	ISSUED FOR DRAFT

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IF NOT 25 mm ASBESTUS SCALES
SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE



- Legend**
- Surcharge Level (m below ground)**
- < 0
 - 0 - 2
 - > 2
 - ⊕ Shallow (<2m Depth)
 - ▲ Storm Outfall/Inlet
 - Storm Pipe
 - ⬡ City of Wetaskiwin Boundary
- Surface Ponding (Depth, m)**
- 0.05 - 0.15
 - > 0.15 - 0.30
 - > 0.30 - 0.50
 - > 0.50 - 1.00
 - < 1.00



FIGURE 6-2A

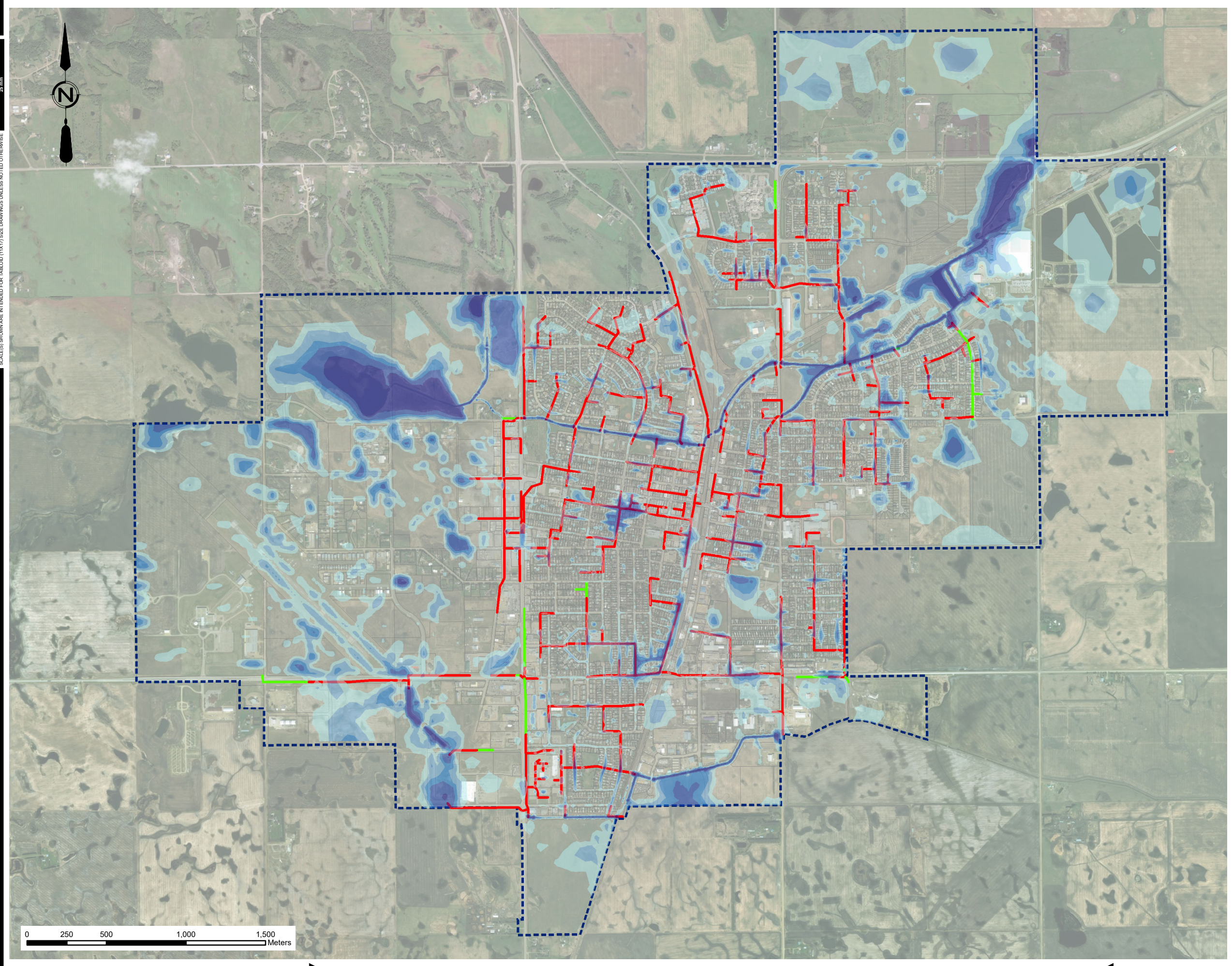
CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE

COUPLED 1D - 2D MODEL
RESULTS: 1:100 YEAR 4 HOUR

AE PROJECT No.	2019-3447
SCALE	1:23,000
APPROVED	
DATE	2020APR09
REV	
DESCRIPTION	ISSUED FOR DRAFT

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IF NOT 25 mm ASBESTUS SCALES
SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE



Legend

Pipe Identification

- Full
- Not Full
- City of Wetaskiwin Boundary

Surface Ponding (Depth, m)

- 0.05 - 0.15
- > 0.15 - 0.30
- > 0.30 - 0.50
- > 0.50 - 1.00
- < 1.00



FIGURE 6-2B

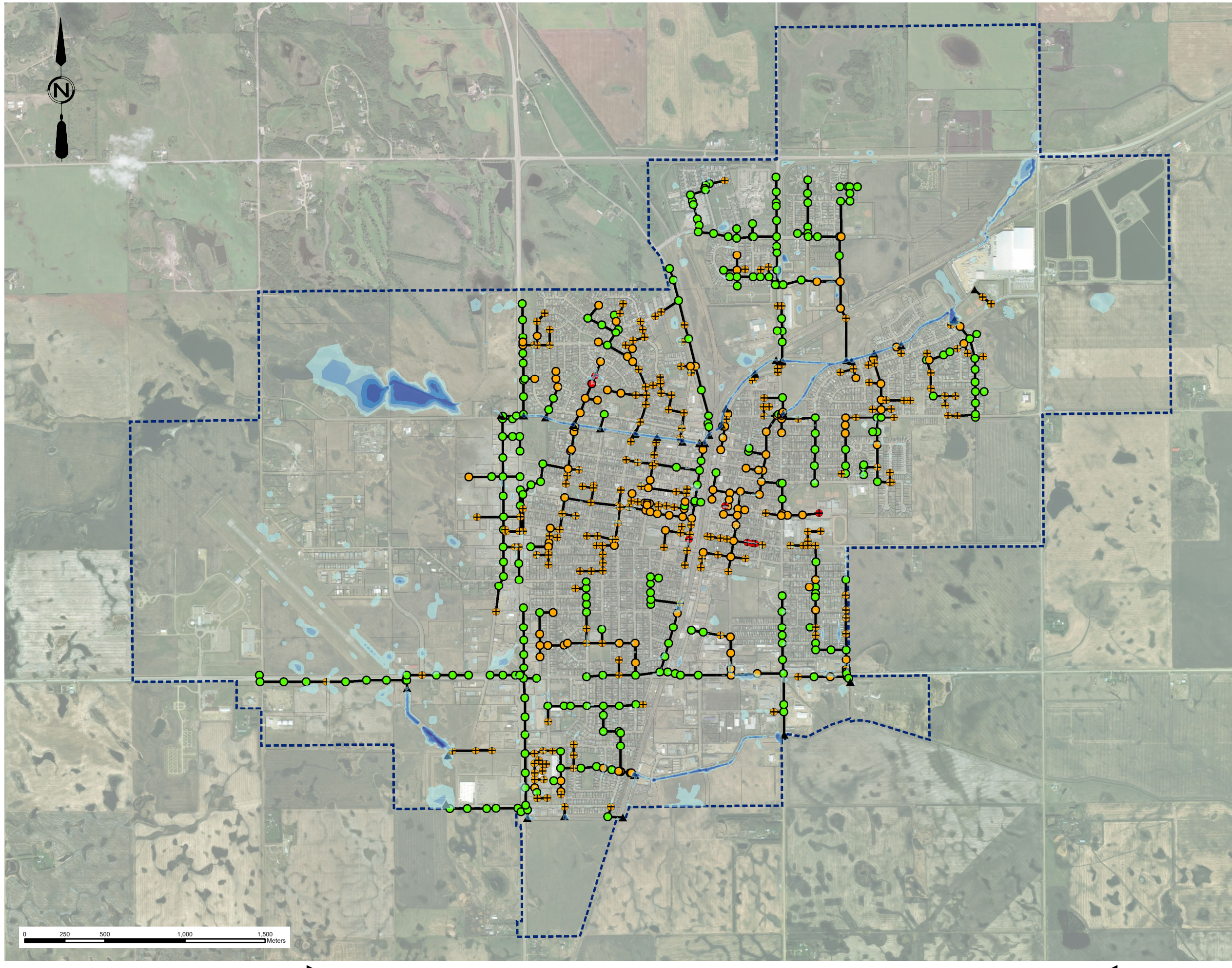
CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE

COUPLED 1D - 2D MODEL
RESULTS: 1:100 YEAR 4 HOUR

AE PROJECT No.	2019-3447
SCALE	1:23,000
APPROVED	
DATE	2020APR09
REV	
DESCRIPTION	ISSUED FOR DRAFT

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IF NOT 25 mm ASBESTUS SCALES
SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE



Legend

Surcharge Level (m below ground)

- < 0
- 0 - 2
- > 2
- ⊕ Shallow (<2m Depth)
- ▲ Storm Outfall/Inlet
- Storm Pipe
- ▭ City of Wetaskiwin Boundary

Surface Ponding (Depth, m)

- 0.05 - 0.15
- > 0.15 - 0.30
- > 0.30 - 0.50
- > 0.50 - 1.00
- > 1.00



FIGURE 6-3A

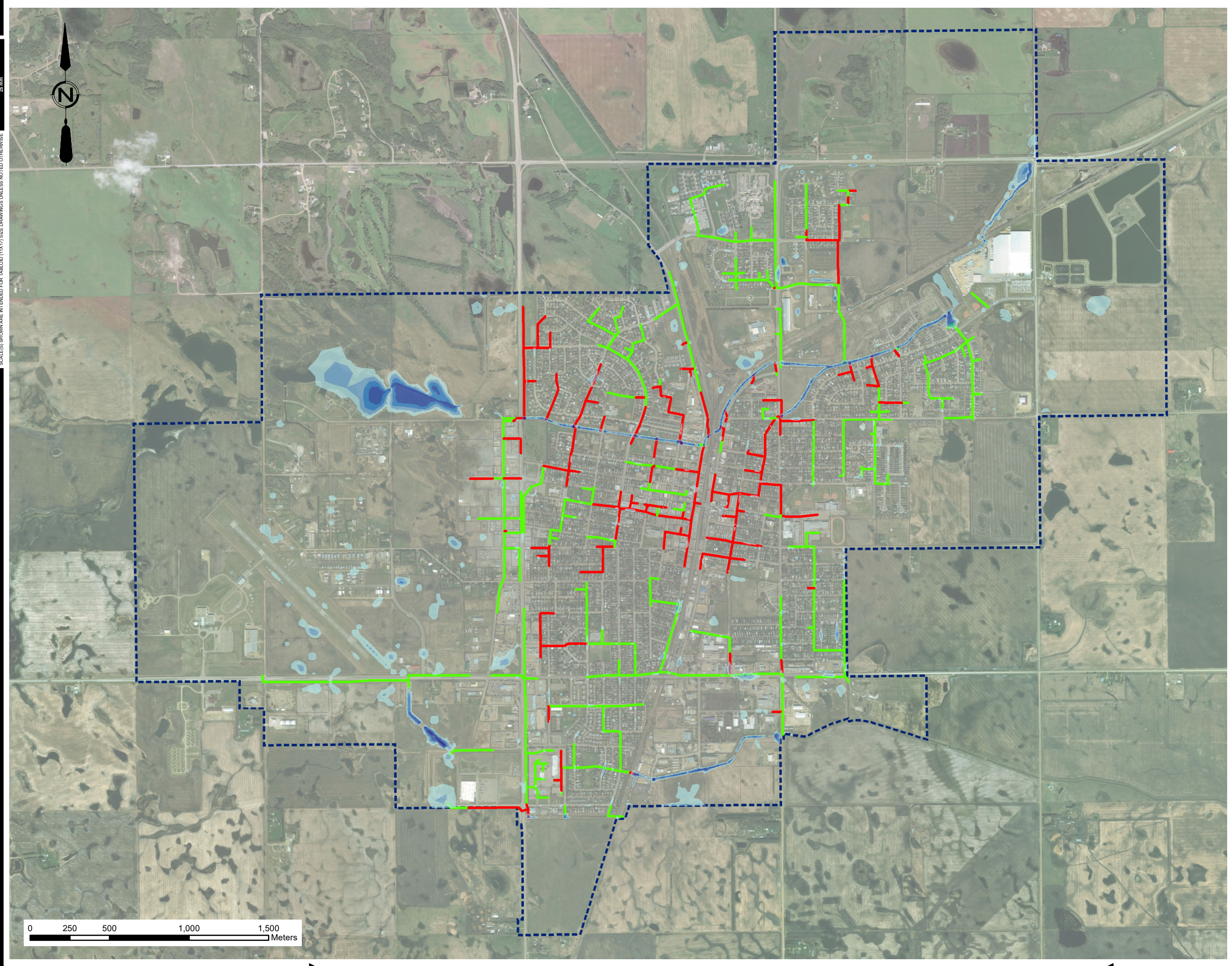
CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE

COUPLED 1D - 2D MODEL
RESULTS: 1:2 YEAR 4 HOUR

AE PROJECT No.	2019-3447
SCALE	1:23,000
APPROVED	
DATE	2020APR09
REV	
DESCRIPTION	ISSUED FOR DRAFT

PLOT DATE: 4/9/2020 2:05:32 PM
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IF NOT 25 mm ASBEST SCALE
SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE



Legend

Pipe Identification

- Full
- Not Full
- City of Wetaskiwin Boundary

Surface Ponding (Depth, m)

- 0.05 - 0.15
- > 0.15 - 0.30
- > 0.30 - 0.50
- > 0.50 - 1.00
- > 1.00



FIGURE 6-3B

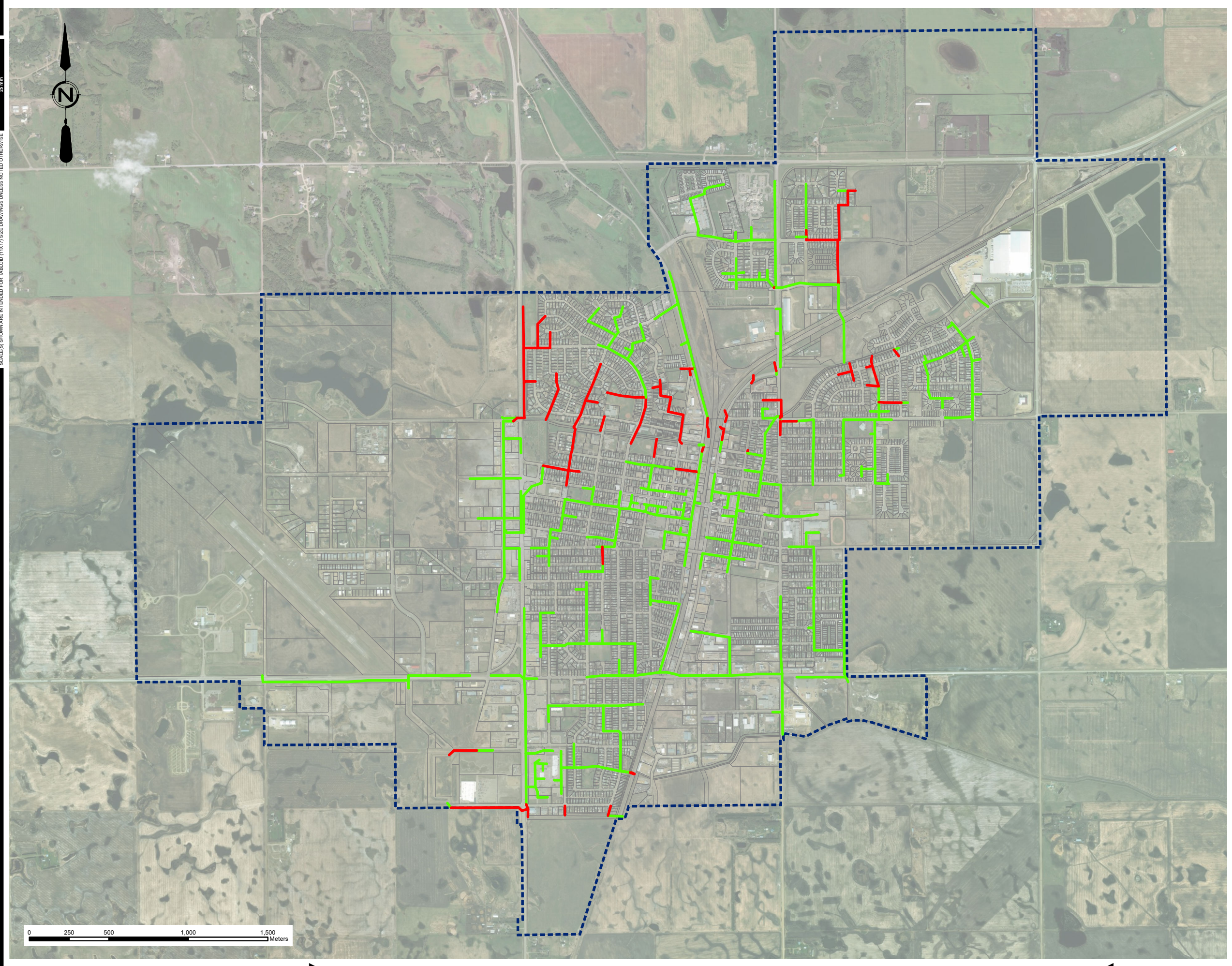
CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE

COUPLED 1D - 2D MODEL
RESULTS: 1:2 YEAR 4 HOUR

AE PROJECT No.	2019-3447
SCALE	1:23,000
APPROVED	
DATE	2020APR09
REV	
DESCRIPTION	ISSUED FOR DRAFT

PLOT DATE: 4/9/2020 2:04:10 PM
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SAVED BY: Kent Richardson
DWG PATH: \\s-e-dm-fs-01\projects\2019\3447\00_UWP_Update\Engineering\03_00_Conceptual_Feasibility_Design_Master_Plans\010_GIS\ump_fg_64t_pipeCapacityDuringJune72019ValidationModelSimulation.mxd

IF NOT 25 mm AS SHOWN SCALES
SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE



Legend

Pipe Identification

- Full
- Not Full
- City of Wetaskiwin Boundary



FIGURE 6-4

CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE

PIPE CAPACITY DURING JUNE 7, 2019 VALIDATION
MODEL SIMULATION

AE PROJECT No.	2019-3447
SCALE	1:23,000
APPROVED	
DATE	2020APR09
REV	
DESCRIPTION	ISSUED FOR DRAFT

7 EXISTING SYSTEM UPGRADES

The current Municipal standards require that the minor system be designed to accommodate runoff generated by a 1:5-year design storm event. However, the City's core minor system currently does not provide a 1:5-year level of service. Flooding of private property can be limited by mitigating surcharge in the storm system during this design storm event. To upgrade the City's entire minor storm system to provide a 1:5-year level of service would require a capital budget of approximately **\$ 132 Million**. The City stated that this is not a feasible solution. Alternatively, the City could complete upgrades to provide a 1:2-year level of service. A capital budget of **\$ 71 Million** is estimated for this option. The City also stated that this is not a feasible option. These estimates are based on the overall length of pipe that is currently undersized, multiplied by an average unit rate to upsize the pipe.

In contrast, a "Do Nothing" option always exists that includes the development of an Emergency Response Plan (ERP) which would outline the measures put in place to mitigate flooding of property during a significant rainfall event.

Associated Engineering discussed the above options with the City. It was agreed that a "hybrid" solution would be put in place. Therefore, results from the 1:2-year and 1:5-year design storm events were combined and locations where surface ponding was occurring were highlighted. Ponding contained within roadways was deemed low risk. Ponding not contained within roadways was investigated for mitigation options. Four key locations within the City were targeted for surface ponding mitigation:

- 48 Avenue between 47A and 48 Streets;
- 50 Street between 47 and 48 Avenues;
- 49 Avenue/53 Street; and
- 52 Street at 40A Avenue/Parkdale School Yard.

Model results suggest that the City's ditch network is a significant limitation of the minor system capacity. During a site visit conducted in April 2019, Associated Engineering personnel noted that there was standing water within the ditches and culverts. In addition, minor system outlet pipes were not visible for the most part and were most likely under water. To determine the impacts of this constraint, Associated Engineering simulated a model eliminating backup from the ditch network. Results were significantly improved during the 1:2-year design storm event and improved in localized areas (near ditch outfalls) during the 1:5-year design storm event simulation. This indicates the need for some modifications to the ditch network. Associated Engineering recommends ditch modifications to mitigate backup into the City's minor storm system. In addition, the installation of flap gates at storm outfalls is recommended. Note, no ditch modifications were modelled as part of the **Stormwater Master Plan Update**. A high level cost of approximately **\$ 6.5 Million** was estimated for ditch modification in the North Central Ditch, a portion of the South Ditch, and installation of flap gates at all storm outfalls. Storm outfall diameters within the City range from 250 mm to 2100 mm.

Associated Engineering assessed several upgrades to address excessive flooding not confined to streets during a 1:5-year 4 hour design storm event. Upgrades were proposed using the following considerations:

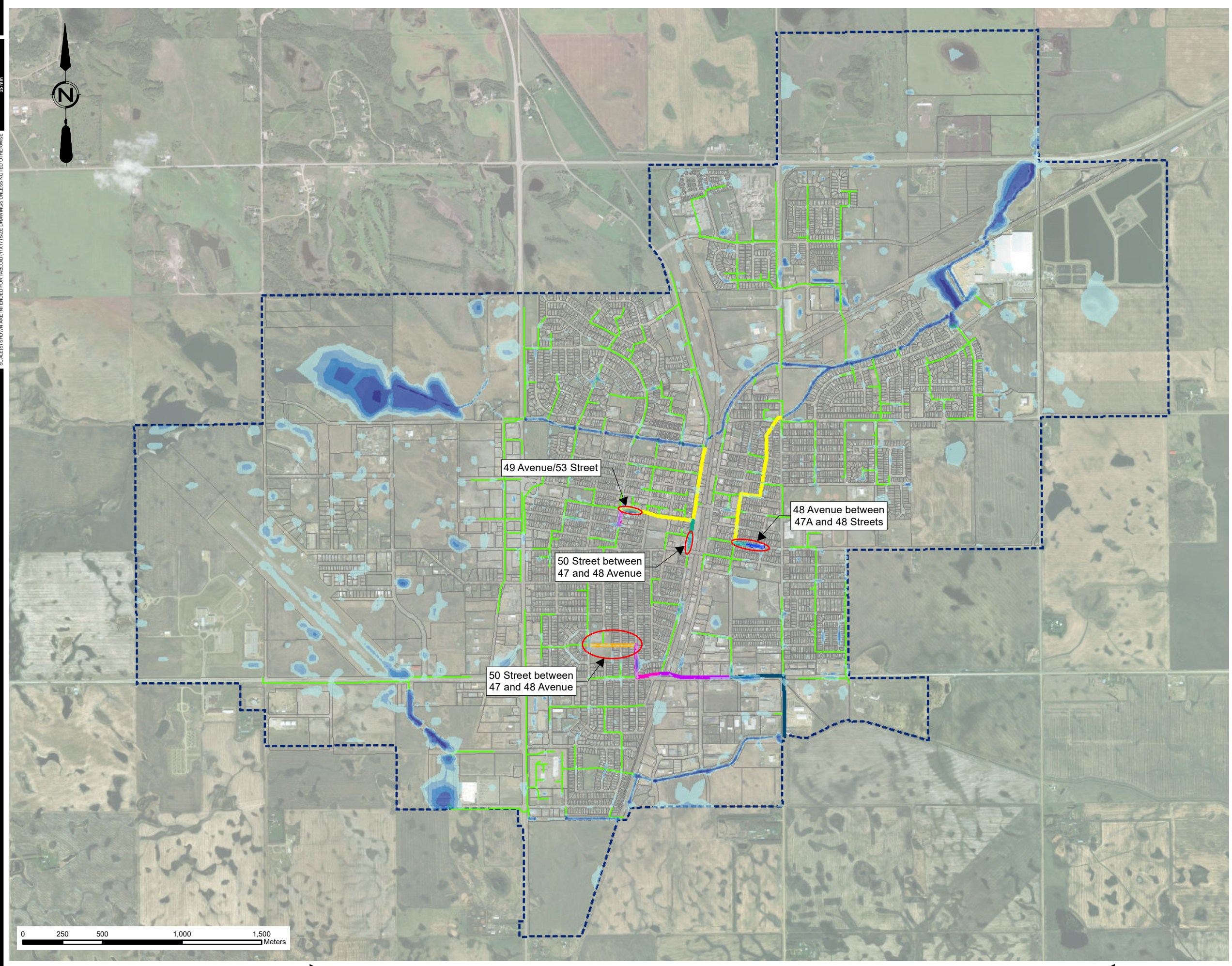
1. Model simulations did not include backup from the City's ditch network into the minor storm system. It is assumed that channel modifications will be carried out before any other upgrades are completed within the storm system.

2. As shown in [Table 6-1](#), most culverts are not sized to accommodate 1:100-year 4 hour design storm event flows from the City. Note, if culverts are upsized, their capacity should correspond to that of the ditch network. In addition, downstream impacts should be evaluated.
3. Proposed pipe upgrades were based on existing invert elevations and alignments.

The recommended upgrade plan is illustrated in [Figure 7-1](#). The coupled 1D-2D model was simulated for the 1:5-year 4 hour design storm event with the recommended pipe upgrades. Results are presented in [Figure 7-2a](#) and [7-2b](#). The HGLs comparing existing and upgraded conditions and are presented in [Appendix F](#). A comparison in surface ponding at critical locations is shown in [Table 7-1](#). Results show that surface ponding is not eliminated; however, it is now confined within streets or, where possible, the HGL was lowered below ground.

Further refinement of each recommended upgrade should be completed during site assessment and design. The solution for each location should be based upon site constraints, construction method (open-cut, trenchless), budget, etc. In addition, upgrades may incorporate a combination of solutions such as storage, diversion, Low Impact Development (LID), raingardens, soil cells, etc.

Associated Engineering also recommends that the City of Wetaskiwin develop an ERP. This document should incorporate “lessons learned” during and after major storm events. Specifically, the 2019 season when a 1:10-year equivalent design storm event was experienced in the City. It is recommended that the plan also include input from other City stakeholders, required actions, written procedures, and the resources available, should the City experience an extreme or major storm event. The ERP should be reviewed and updated after each significant storm event.



- Legend**
- Proposed Pipe Upgrade**
- Size
- 450mm
 - 750mm
 - 900mm
 - 1050mm
 - 1200mm
 - 1350mm
 - 1500mm
 - Existing Storm Pipe
- Surface Ponding (Depth, m)**
- 0.05 - 0.15
 - > 0.15 - 0.30
 - > 0.30 - 0.50
 - > 0.50 - 1.00
 - < 1.00
 - City of Wetaskiwin Boundary

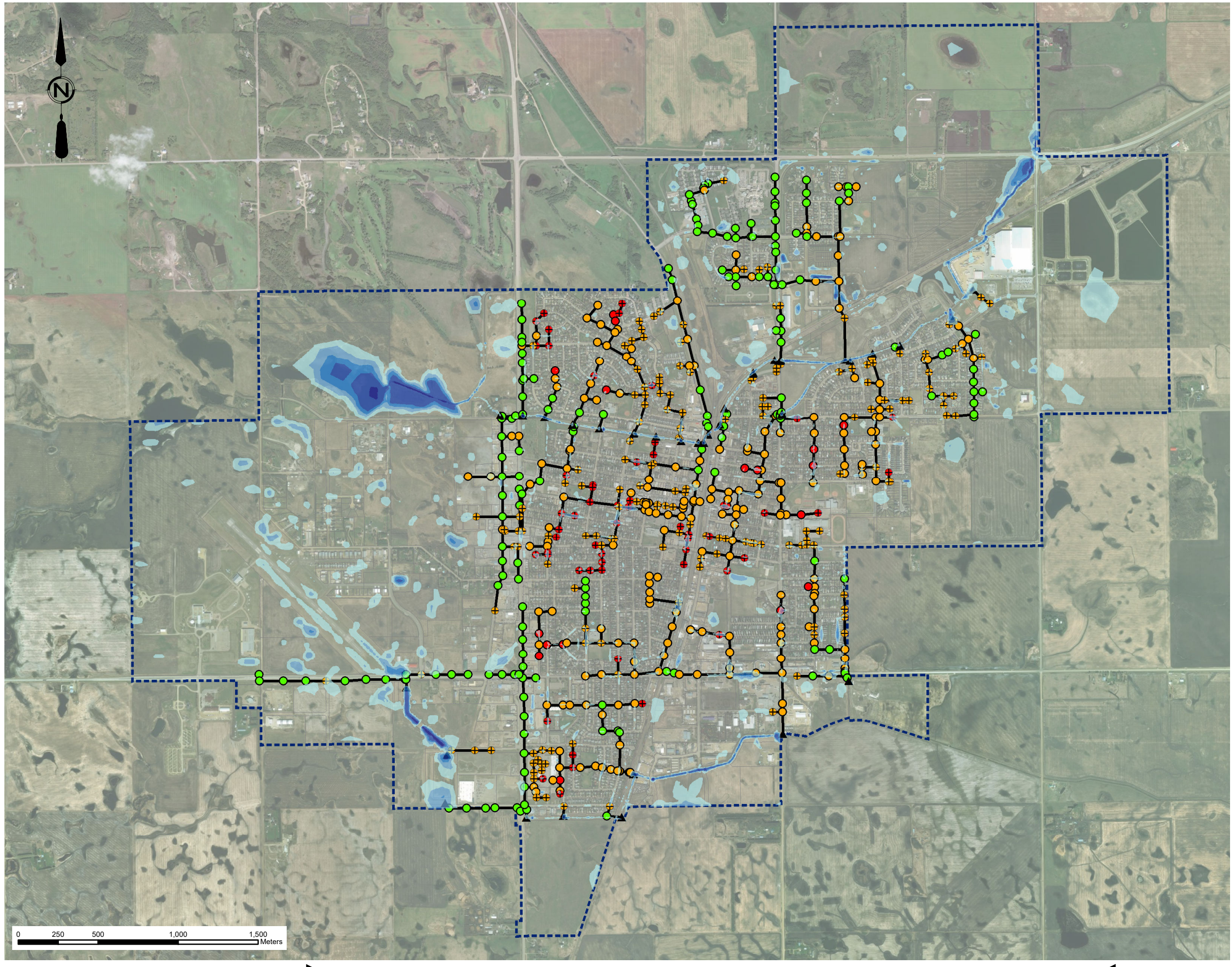


FIGURE 7-1
CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE

RECOMMENDED UPGRADE PLAN	
AE PROJECT No.	2019-3447
SCALE	1:23,000
APPROVED	
DATE	2020APR20
REV	
DESCRIPTION	ISSUED FOR DRAFT

PLOT DATE: 3/30/2020 3:25:32 PM
SAVE DATE: 3/28/2020 3:25:32 PM
DWG PATH: \\s-01\projects\2019\344700_UWP_Update\Engineering\03_00_Conceptual_Feasibility_Design_Master_Plans\010_GIS\ump_fig_7_2a_Coupled1D_2DModelResults.mxd

IF NOT 25 mm AS JUST SCALES
SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE



Legend

Surcharge Level (m below ground)

- < 0
- 0 - 2
- > 2

⊕ Shallow (<2m Depth)

▲ Storm Outfall/Inlet

— Storm Pipe

▭ City of Wetaskiwin Boundary

Surface Ponding (Depth, m)

- 0.05 - 0.15
- > 0.15 - 0.30
- > 0.30 - 0.50
- > 0.50 - 1.00
- < 1.00



FIGURE 7-2A

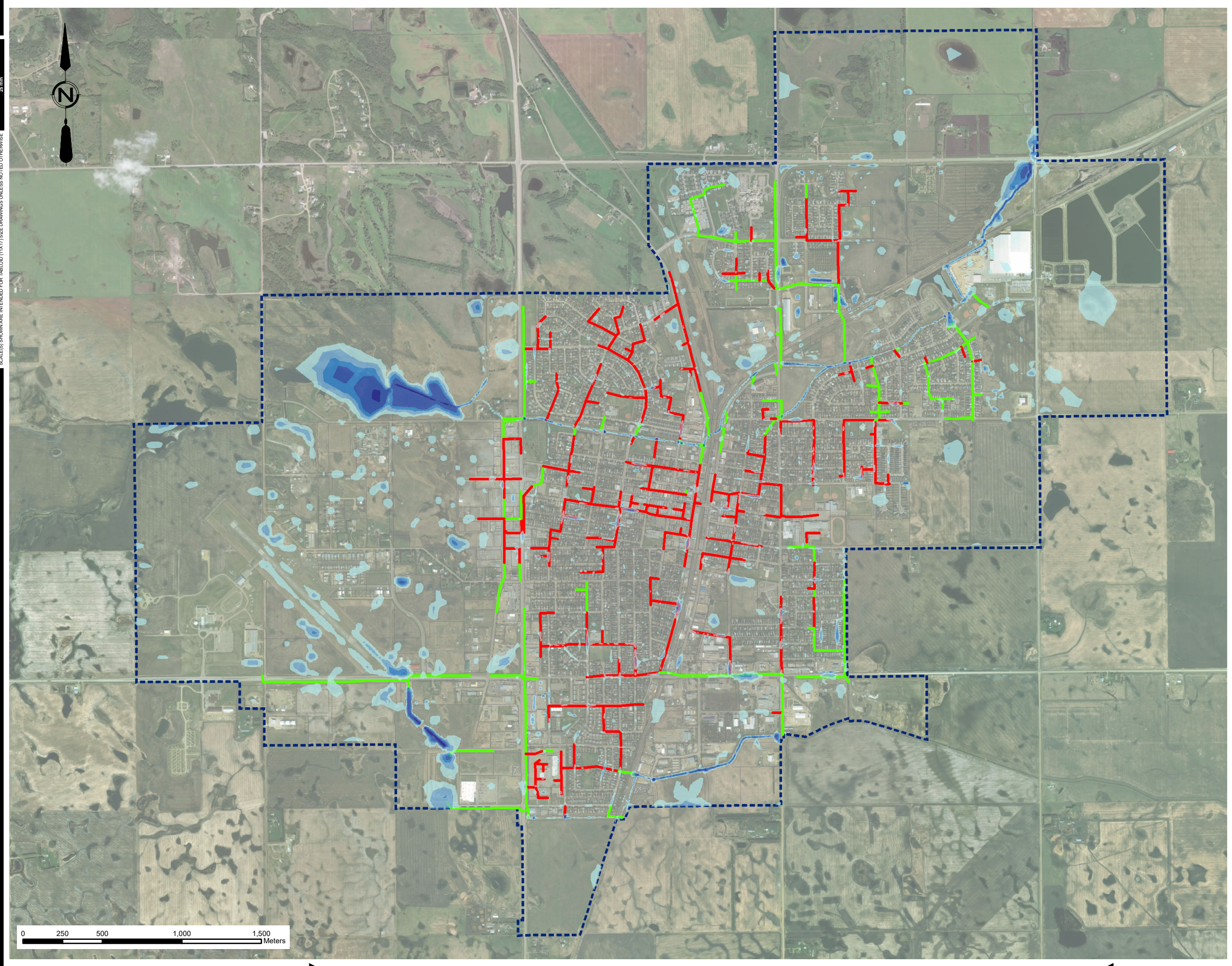
CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE

COUPLED 1D - 2D MODEL RESULTS:
RECOMMENDED UPGRADES TO
EXISTING 1:5 YEAR 4 HOUR

AE PROJECT No.	2019-3447
SCALE	1:23,000
APPROVED	
DATE	2020MAR30
REV	
DESCRIPTION	ISSUED FOR DRAFT

PLOT DATE: 3/30/2020 3:24:24 PM
SAVE DATE: 3/28/2020 3:24:24 PM
DWG PATH: \\s-edi-fs-01\projects\2019\344700_UWP_Update\Engineering\03_00_Conceptual_Feasibility_Design_Master_Plans\010_GIS\ump_fig_7_2b_pipesCoupled1D_2DmodelResults.mxd

IF NOT 25 mm AS JUST SCALES
SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE



- Legend**
- Pipe Identification**
- Full
 - Not Full
 - City of Wetaskiwin Boundary



FIGURE 7-2B

CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE

COUPLED 1D - 2D MODEL RESULTS:
RECOMMENDED UPGRADES TO
EXISTING 1:5 YEAR 4 HOUR

AE PROJECT No.	2019-3447
SCALE	1:23,000
APPROVED	
DATE	2020MAR30
REV	
DESCRIPTION	ISSUED FOR DRAFT

Table 7-1
Surface Ponding Comparisons – 1:5-year 4 Hour

Location	Existing	Upgrade
48 Avenue between 47A and 48 Streets		
50 Street between 47 and 48 Avenues		

Location	Existing	Upgrade
49 Avenue/53 Street		

8 FUTURE SYSTEM EXPANSION

Figure 8-1 presents the proposed future staged growth areas for the City of Wetaskiwin. These areas were determined through consultation with the City. The future storm system concept is presented in four phases: Stages 1 through 3 and Ultimate Development. Future developments have not been connected to the existing storm networks where possible to avoid additional overloading of the existing systems. The recommended piped storm systems shall be designed to convey runoff from a 1:5-year design storm event. Conceptual storm trunk locations are presented. The estimated runoff flow for each future development area and stage has been completed using the Rational Method with inlet times corresponding to the City's Municipal Engineering Standards. Area Structure Plans (ASPs) were reviewed where available and are reflected in the future storm concept where appropriate.

Figure 8-2 illustrates the recommended Ultimate Future System Expansion Storm Servicing Concept. Where possible, ponds or bioswales for water quality treatment and control are recommended before runoff is discharged outside of the City boundary and makes its way into Pipestone Creek. A South Regional Pond is recommended at the southeast City boundary extent to provide final water quality treatment to all stormwater runoff flowing through the South Ditch.

The Walker Farm ASP included two stormwater management facilities located at low spots within the development area. However, they stated that all storm sewers will discharge to the existing storm main along 56 Avenue. This plan was altered as presented in **Figure 8-2** based on the coupled 1D-2D model results showing limited existing pipe capacity. In addition, a portion of non-residential development within the Highway North neighbourhood is proposed to flow through the Walker Farm Pond 2 in the ultimate development.

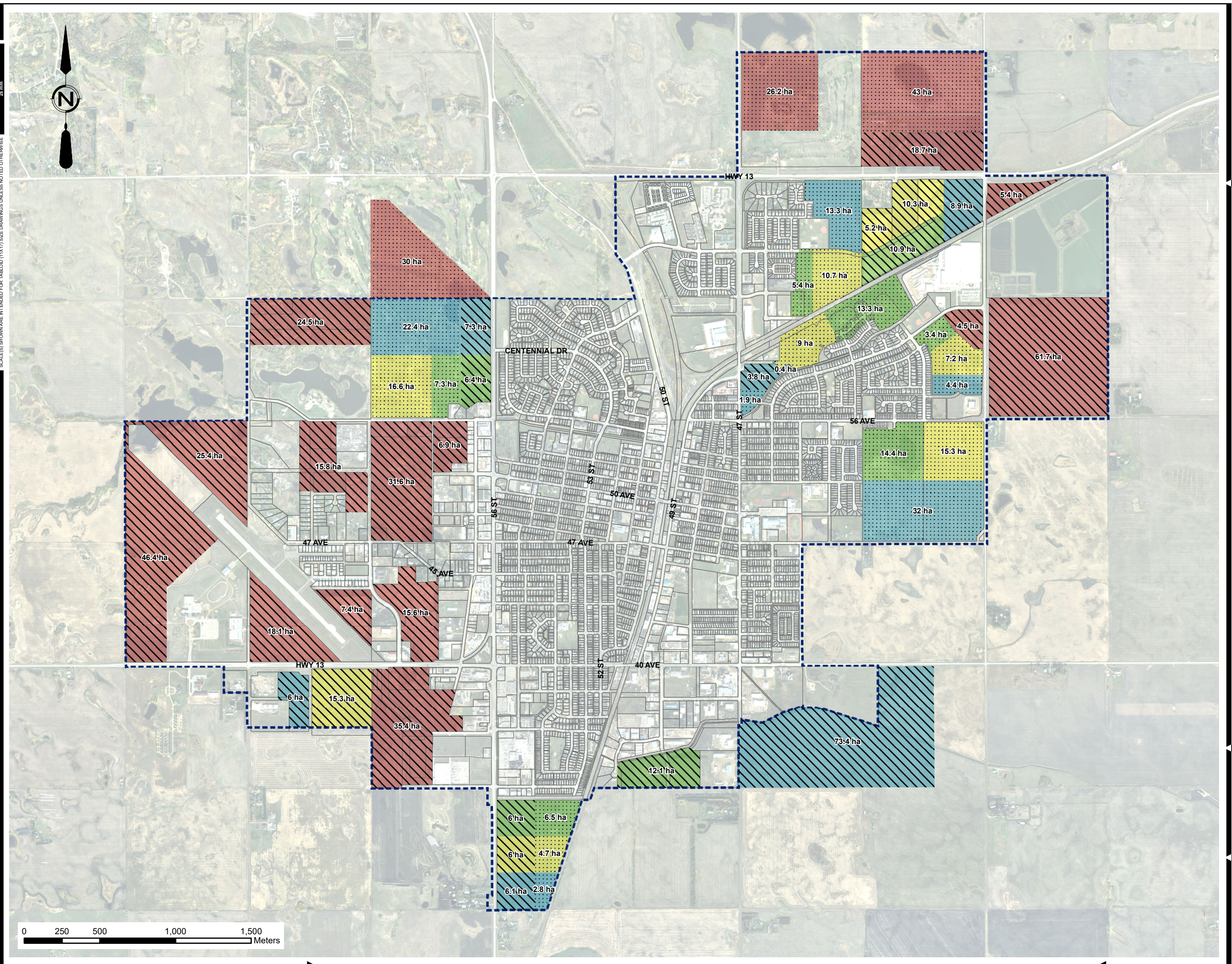
Future development within the Nashville and Parkside neighbourhoods is proposed to flow through the existing Parkside minor storm system which has capacity for additional flows based on the coupled 1D-2D model results. However, some local upgrades may be required and detailed analyses should be conducted. This also applies to any other locations where future developments are proposed to connect to existing storm sewers such as in the Highway North and Highway south neighbourhoods.

The Creeks Crossing ASP has been incorporated into the Stormwater Master Plan Future System Expansion. This includes the Aspen Ridge Pond, an extension of the Regional Storm Pond, and modification to the North Central Ditch through the Garden Meadows neighbourhood.

It is recommended that the Ultimate Future System Expansion Storm Servicing Concept be followed as development within the City progresses. Development should progress in an orderly manner with discouragement of "leap-frogging". In addition, it is recommended that the City coordinate stormwater runoff works with Wetaskiwin County to avoid adverse impacts occurring from offsite drainage areas.

PLOT DATE: 4/13/2020 8:36:31 AM
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IF NOT 25 mm ADJUST SCALES
SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE



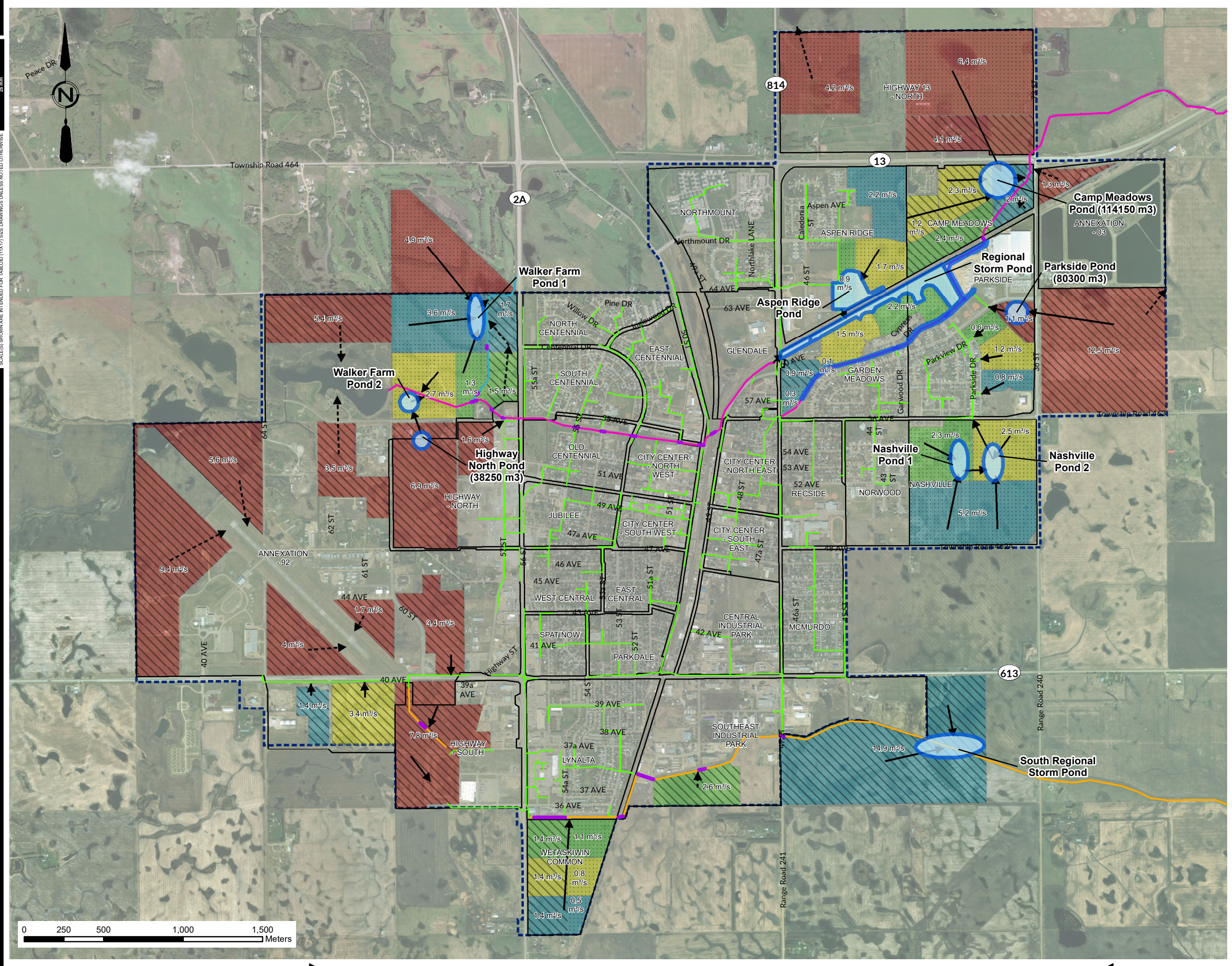
- Legend**
- City of Wetaskiwin Boundary
 - Staged Growth Areas**
 - Stage 1
 - Stage 2
 - Stage 3
 - Ultimate
 - Future Land Use**
 - Residential
 - Non-Residential

FIGURE 8-1
CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE
PROJECTED STAGED GROWTH AREAS

AE PROJECT No.	2019-3447
SCALE	1:24,000
APPROVED	
DATE	2020APR13
REV	
DESCRIPTION	ISSUED FOR DRAFT

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IF NOT 25 mm ADJUST SCALES
SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE



- Legend**
- Proposed Storm Trunk
 - Proposed Bioswale
 - Modelled Culvert
 - Storm Pipe
 - North Central Ditch
 - North Division Ditch
 - South Ditch
 - Stormwater Management Facility
 - Neighbourhood Name
 - City of Wetaskiwin Boundary
 - 1.4 m³/s Estimated Runoff
- Future Land Use**
- Residential
 - Non-Residential
- Staged Growth Areas**
- Stage 1
 - Stage 2
 - Stage 3
 - Ultimate



FIGURE 8-2
CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE
ULTIMATE FUTURE SYSTEM EXPANSION
STORM SERVICING CONCEPT

AE PROJECT No.	2019-3447
SCALE	1:23,000
APPROVED	
DATE	2020APR09
REV	
DESCRIPTION	ISSUED FOR DRAFT

9 COST ESTIMATES

A summary of capital cost estimates is provided in [Table 9-1](#) for the proposed upgrades to the existing storm system. The estimates include an allowance for engineering (15%) and contingency (15%), but do not include GST. All costs are presented in 2020 dollars. A breakdown of unit costs is included in [Appendix G](#).

Table 9-1
City of Wetaskiwin Existing System Upgrades Capital Cost Estimate

LOCATION	PIPE_ID	FROM	TO	EXISTING PIPE DIAMETER (m)	PROPOSED PIPE DIAMETER (m)	LENGTH (m)	UNIT COST (\$/m)	SUBTOTAL	ENGINEERING & CONTINGENCY (30%)	TOTAL
Ditch Network Modification										
North Central Ditch						3000	\$ 1,200.00	\$ 3,600,000.00	\$ 1,080,000.00	\$ 4,680,000.00
South Ditch						500	\$ 1,200.00	\$ 600,000.00	\$ 180,000.00	\$ 780,000.00
Backflow Prevention Valves						35	varies	\$ 760,050.00	\$ 228,015.00	\$ 988,065.00
SUBTOTAL										\$ 6,448,065.00
48 Avenue between 47A and 48 Streets										
47 STREET DITCH OUTLET	L480	N234	N1050	0.9	1.2	10	\$ 4,000.00	\$ 40,000.00	\$ 12,000.00	\$ 52,000.00
47 STREET	L223	N233	N234	0.9	1.2	20	\$ 4,000.00	\$ 80,000.00	\$ 24,000.00	\$ 104,000.00
56 AVENUE/47 STREET	L222	N232	N233	0.9	1.2	20	\$ 4,000.00	\$ 80,000.00	\$ 24,000.00	\$ 104,000.00
47A STREET/56 AVENUE	L221	N231	N232	0.9	1.2	120	\$ 4,000.00	\$ 480,000.00	\$ 144,000.00	\$ 624,000.00
47A STREET	L220	N230	N231	0.9	1.2	130	\$ 4,000.00	\$ 520,000.00	\$ 156,000.00	\$ 676,000.00
47A STREET	L219	N229	N230	0.75	1.2	110	\$ 4,000.00	\$ 440,000.00	\$ 132,000.00	\$ 572,000.00
52/53 AVENUE LANE	L218	N228	N229	0.75	1.2	60	\$ 4,000.00	\$ 240,000.00	\$ 72,000.00	\$ 312,000.00
51/52 AVENUE LANE	L217	N227	N228	0.75	1.2	90	\$ 4,000.00	\$ 360,000.00	\$ 108,000.00	\$ 468,000.00
51 AVENUE	L215	N226	N227	0.75	1.2	120	\$ 4,000.00	\$ 480,000.00	\$ 144,000.00	\$ 624,000.00
48 STREET	L214	N225	N226	0.75	1.2	110	\$ 4,000.00	\$ 440,000.00	\$ 132,000.00	\$ 572,000.00
48 STREET	L213	N224	N225	0.75	1.2	40	\$ 4,000.00	\$ 160,000.00	\$ 48,000.00	\$ 208,000.00
48 STREET	L212	N223	N224	0.675	1.2	50	\$ 4,000.00	\$ 200,000.00	\$ 60,000.00	\$ 260,000.00
48 STREET	L211	N222	N223	0.525	1.2	100	\$ 4,000.00	\$ 400,000.00	\$ 120,000.00	\$ 520,000.00
48 AVENUE	L577	N584M	N222	0.375	0.75	90	\$ 1,900.00	\$ 171,000.00	\$ 51,300.00	\$ 222,300.00
48 AVENUE	L6230	N6234	N584M	0.3	0.45	30	\$ 1,500.00	\$ 45,000.00	\$ 13,500.00	\$ 58,500.00
48 AVENUE	L6255	N6235	N6234	0.3	0.45	10	\$ 1,500.00	\$ 15,000.00	\$ 4,500.00	\$ 19,500.00
48 AVENUE	L6228	N6236	N6235	0.3	0.45	50	\$ 1,500.00	\$ 75,000.00	\$ 22,500.00	\$ 97,500.00
SUBTOTAL										\$ 5,493,800.00
50 Street between 47 and 48 Avenues/49 Avenue/53 Street										
50 STREET	L209	N219	N1038	1.05	1.2	10	\$ 4,000.00	\$ 40,000.00	\$ 12,000.00	\$ 52,000.00
50 STREET	L208	N217	N219	1.05	1.2	100	\$ 4,000.00	\$ 400,000.00	\$ 120,000.00	\$ 520,000.00
50 STREET	L207	N213	N217	1.05	1.2	70	\$ 4,000.00	\$ 280,000.00	\$ 84,000.00	\$ 364,000.00
50 STREET	L203	N212	N213	1.05	1.2	90	\$ 4,000.00	\$ 360,000.00	\$ 108,000.00	\$ 468,000.00
50 STREET	L202	N211	N212	1.05	1.2	100	\$ 4,000.00	\$ 400,000.00	\$ 120,000.00	\$ 520,000.00
50 STREET	L201	N210	N211	1.05	1.2	10	\$ 4,000.00	\$ 40,000.00	\$ 12,000.00	\$ 52,000.00
50 STREET	610.1	N208	N210	1.05	1.2	100	\$ 4,000.00	\$ 400,000.00	\$ 120,000.00	\$ 520,000.00
50 STREET	1386.1	N596M	N208	0.45	0.75	50	\$ 1,900.00	\$ 95,000.00	\$ 28,500.00	\$ 123,500.00
50 STREET	1385.1	N209	N596M	0.45	0.75	50	\$ 1,900.00	\$ 95,000.00	\$ 28,500.00	\$ 123,500.00
50 STREET	1374.1	N590M	N209	0.375	0.75	50	\$ 1,900.00	\$ 95,000.00	\$ 28,500.00	\$ 123,500.00
49 AVENUE	606.1	N207	N208	0.9	1.2	60	\$ 4,000.00	\$ 240,000.00	\$ 72,000.00	\$ 312,000.00
49 AVENUE	604.1	N206	N207	0.9	1.2	80	\$ 4,000.00	\$ 320,000.00	\$ 96,000.00	\$ 416,000.00
49 AVENUE	602.1	N205	N206	0.9	1.2	80	\$ 4,000.00	\$ 320,000.00	\$ 96,000.00	\$ 416,000.00
49 AVENUE	600.1	N204	N205	0.9	1.2	70	\$ 4,000.00	\$ 280,000.00	\$ 84,000.00	\$ 364,000.00
49 AVENUE	598.1	N203	N204	0.9	1.2	30	\$ 4,000.00	\$ 120,000.00	\$ 36,000.00	\$ 156,000.00
49 AVENUE	596.1	N202	N203	0.9	1.2	100	\$ 4,000.00	\$ 400,000.00	\$ 120,000.00	\$ 520,000.00
49 AVENUE	594.1	N198	N202	0.9	1.2	60	\$ 4,000.00	\$ 240,000.00	\$ 72,000.00	\$ 312,000.00
53 STREET	592.1	N201	N198	0.75	1.05	100	\$ 3,500.00	\$ 350,000.00	\$ 105,000.00	\$ 455,000.00
SUBTOTAL										\$ 5,817,500.00
52 Street at 40A Avenue/Parkdale School Yard										
47 STREET	L601	N330	N1123	1.2	1.5	190	\$ 5,000.00	\$ 950,000.00	\$ 285,000.00	\$ 1,235,000.00
47 STREET	L305	N329	N330	1.2	1.5	40	\$ 5,000.00	\$ 200,000.00	\$ 60,000.00	\$ 260,000.00
47 STREET	L304	N323	N329	1.2	1.5	190	\$ 5,000.00	\$ 950,000.00	\$ 285,000.00	\$ 1,235,000.00
40 AVENUE	L297	N321	N323	1.2	1.5	160	\$ 5,000.00	\$ 800,000.00	\$ 240,000.00	\$ 1,040,000.00
40 AVENUE	L296	N314	N321	1.2	1.5	160	\$ 5,000.00	\$ 800,000.00	\$ 240,000.00	\$ 1,040,000.00
40 AVENUE	L289	N313	N314	1.05	1.35	200	\$ 4,500.00	\$ 900,000.00	\$ 270,000.00	\$ 1,170,000.00
40 AVENUE	L288	N312	N313	1.05	1.35	110	\$ 4,500.00	\$ 495,000.00	\$ 148,500.00	\$ 643,500.00
40 AVENUE	L287	N311	N312	1.05	1.35	40	\$ 4,500.00	\$ 180,000.00	\$ 54,000.00	\$ 234,000.00
40 AVENUE	L286	N310	N311	1.05	1.35	50	\$ 4,500.00	\$ 225,000.00	\$ 67,500.00	\$ 292,500.00
40 AVENUE	L285	N299	N310	1.05	1.35	50	\$ 4,500.00	\$ 225,000.00	\$ 67,500.00	\$ 292,500.00
40 AVENUE	L284	N306	N299	0.9	1.05	150	\$ 3,500.00	\$ 525,000.00	\$ 157,500.00	\$ 682,500.00
52 STREET	L280	N305	N306	0.75	1.05	80	\$ 3,500.00	\$ 280,000.00	\$ 84,000.00	\$ 364,000.00
52 STREET	L279	N304	N305	0.75	1.05	130	\$ 3,500.00	\$ 455,000.00	\$ 136,500.00	\$ 591,500.00
41 AVENUE	L278	N303	N304	0.675	0.9	100	\$ 2,900.00	\$ 290,000.00	\$ 87,000.00	\$ 377,000.00
41 AVENUE	L277	N302	N303	0.6	0.9	100	\$ 2,900.00	\$ 290,000.00	\$ 87,000.00	\$ 377,000.00
PARKDALE SCHOOL PARK	L276	N301	N302	0.6	0.9	100	\$ 2,900.00	\$ 290,000.00	\$ 87,000.00	\$ 377,000.00
SUBTOTAL										\$ 10,211,500.00
TOTAL										\$ 27,970,865.00

10 CONCLUSIONS

Following are the key findings of the Stormwater Master Plan Update:

- The City of Wetaskiwin includes a mix of residential, commercial, and industrial areas. The topography of the area generally falls toward the east with the highest elevations occurring in the west portion of the City, and the lowest elevations located in the northeast.
- The minor system within the City includes pipes ranging in size from 200 to 2100 mm in diameter, manholes, catchbasins, and outfall structures. Meanwhile, the major system consists of streets, roads, a regional Stormwater Management Facility (SWMF), parkland, and overland runoff routes. There are three major ditches flowing through the City which include a number of culverts.
- Previous Master Plans used different release rates for future development. A more detailed analysis is required prior to stormwater management facility designs to confirm this rate.
- An area of approximately 90 km² flows west from Wetaskiwin County towards the City of Wetaskiwin. However, the By the Lake Park and North Diversion Ditch were constructed in 1988 as part of the West Wetaskiwin Flood Diversion Project to attenuate this offsite drainage area and divert flows around the City into Bigstone Creek.
- Detailed information on catchbasin locations was not available.
- A site visit of the City's ditch and culvert network was conducted by Associated Engineering personnel in April 2019. It was noted that the ditches had standing water in them, as well as vegetation growth and debris which reduces the capacity of the ditches and culverts.
- Associated Engineering completed topographic survey of the ditches and culverts within the City in October 2019. During the survey, a control gate was found at the North Diversion/North Central Ditch confluence just east of By the Lake Park which controls flows into the North Central ditch. The gate appeared to be manually operated in order to direct flows from the west (Wetaskiwin County) into the North Diversion Ditch and around the City. For this study, this gate was assumed to be closed for all modelled simulations.
- In general, the flow monitoring data was found to be of good quality. The data required little to no modification during quality control reviews. No data adjustments were required for the rainfall data; however, data inconsistencies were noted from the end of September to October. Therefore, data collected during this period was discounted.
- Standing water was observed by SFE at Monitors 2, 3, and 4 during the monitoring season. The flow monitoring data included some negative flows and suspect hydrographs indicating a potential backflow from time to time.
- Storm events occurred on June 7, July 6, and July 18 during the 2019 Flow Monitoring Program and were chosen for model calibration and validation events. The developed coupled 1D-2D model was simulated for the three chosen storm events. The largest event occurred on July 6, 2019 and was approximately equivalent to a 1:10-year design storm event. The event on July 18 was approximately equal to a 1:2-year design storm event, while the July 7 event was less than a 1:2-year design storm event.

- The largest storm event on July 6, 2019 was chosen for calibration of the developed model. The calibration results showed that peak flows were overestimated on average by 30%. This can be attributed to the physically based modelling approach, estimated infiltration rates, lack of catchbasin locations, catchbasin type, and the coupling locations. However, the timing and shape of all simulated hydrographs match reasonably well at all monitor sites between the observed and simulated data resulting in a reasonable calibration of the developed model.
- Validation runs were simulated for the July 18 and June 7, 2019 storm events. Similar to the calibration results, validation flows overestimated on average by 30%; however, timing and shape of all simulated hydrographs match reasonably well between observed and simulated data.
- The majority of the City's storm networks (both minor and major) provide capacity up to approximately the 1:2-year 4 hour design storm event. In addition, the majority of the City's culverts have capacity for up to the 1:5-year design storm event. Significant surface ponding was simulated during the 1:5-year and 1:100-year 4 hour design storm events. Currently, the City's ditch network does not provide capacity for the 1:100-year 4 hour design storm event.
- Several upgrades were proposed to address flooding not confined to streets within the 1:5-year 4 hour design storm event. These included ditch modifications and upsizing of pipes. Results showed that surface ponding is not eliminated; however, it is now confined within the streets or, where possible, the HGL was lowered below ground.
- High level estimates to upgrade the City's entire minor storm system to provide a 1:5-year or 1:2-year level of service would require capital budgets of approximately \$ 132 Million or \$ 71 Million, respectively. The City stated that these are not feasible options for the City.
- The City's existing ditch network is currently causing backup of stormwater flows within the minor storm network.
- Proposed future staged growth areas for the City of Wetaskiwin were determined through consultation with the City during the Water Master Plan Update. The future storm system concept is presented in four phases: Stages 1 through 3 and Ultimate Development. Future developments have not been connected to the existing storm networks where possible to avoid overloading the existing systems.

11 RECOMMENDATIONS

Associated Engineering recommends the following:

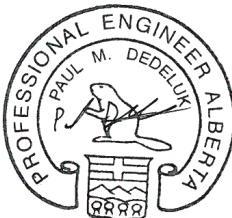
- The storm hyetographs developed by EPCOR in 2018 for the 4-hour Chicago and the 24-hour Huff design storms be adopted to provide consistency for modelling purposes and are included in the City's Municipal Engineering Standards.
- A detailed analysis be completed prior to stormwater management facility designs to confirm acceptable pre-development release rates.
- As more detailed information becomes available about the location and type of catchbasins within the City, the model be refined and coupled only at appropriate locations.
- The City undertake a CCTV inspection program of their entire minor storm system to ensure no blockages or pipe deterioration exists.
- Additional calibration and validation of the developed coupled 1D-2D model as more observed data becomes available.
- The City implement a continuous storm sewer flow monitoring program.
- The City develop and implement a ditch network maintenance program to mitigate backup of flows into the minor system and to facilitate stormwater flows through the City.
- The control gate at the North Diversion/North Central Ditch confluence remain closed at all times to limit stormwater flows from outside of the City flowing through the North Central Ditch system.
- Development of modifications to the existing ditch network be completed to mitigate backup into the City's minor storm system. Downstream impacts should also be assessed.
- Further refinement of each proposed existing minor system upgrade should be completed during design. The solution in each location should be based upon site constraints, construction method (open-cut, trenchless), budget, etc. In addition, upgrades may incorporate a combination of solutions such as storage, diversion, Low Impact Development (LID), raingardens, soil cells, etc.
- An Emergency Response Plan be developed by the City of Wetaskiwin. This document should include "lessons learned" during and after major storm events, input from other City stakeholders, required actions, written procedures, and the resources available, should the City experience a major storm event.
- The Ultimate Future System Expansion Storm Servicing Concept be followed as development within the City progresses. Development should progress in an orderly manner with discouragement of "leap-frogging".
- The City coordinate stormwater runoff works with Wetaskiwin County to avoid adverse impacts occurring from offsite drainage areas.

CLOSURE

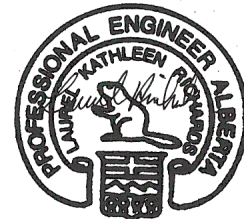
This report was prepared for the City of Wetaskiwin to complete an update of the City's Stormwater Master Plan.

The services provided by Associated Engineering Alberta Ltd. in the preparation of this report were conducted in a manner consistent with the level of skill ordinarily exercised by members of the profession currently practicing under similar conditions. No other warranty expressed or implied is made.

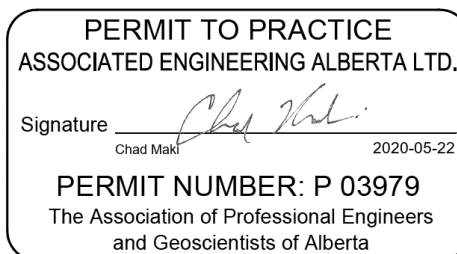
Respectfully submitted,
Associated Engineering Alberta Ltd.



Paul Dedeluk, P.Eng.
Project Manager



Laurel Richards, P.Eng.
Project Engineer



APPENDIX A - EPCOR IDF CURVES

2018 EPCOR IDF CURVES

Intensity Duration Frequency (IDF) curves are a result of a Climate Change study conducted by the University of Alberta and EPCOR encompassing 32 years of records (1984 – 2015) at 11 different monitoring stations across the city of Edmonton.

Table A7 - IDF Curves - Intensity Table (Part 1 of 3)
Edmonton 11 Rain Gauges Upper Bound - IDF Period: 1984-2015
Maximum Years of Record = 32
IDF Intensity (mm/hr)

Time		Return Frequency						
Minutes	Hours	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	200-yr
5	0.25	66.0	89.0	108.6	138.0	162.2	188.4	217.8
6		61.3	83.2	101.9	129.9	152.9	178.1	206.8
7		57.2	78.3	96.1	122.9	144.7	169.0	197.1
8		53.8	73.9	91.1	116.7	137.5	160.9	188.4
9		50.8	70.1	86.6	111.1	131.1	153.7	180.6
10		48.2	66.7	82.6	106.2	125.3	147.2	173.5
11		45.8	63.7	79.0	101.7	120.1	141.3	167.0
12		43.8	61.0	75.7	97.7	115.4	136.0	161.1
13		41.9	58.6	72.7	93.9	111.1	131.1	155.6
14		40.2	56.3	70.0	90.6	107.1	126.6	150.5
15		38.7	54.3	67.6	87.4	103.5	122.4	145.9
16		37.2	52.4	65.3	84.6	100.2	118.6	141.5
17		36.0	50.7	63.2	81.9	97.0	115.0	137.5
18		34.8	49.1	61.2	79.4	94.1	111.7	133.7
19		33.7	47.6	59.4	77.1	91.4	108.6	130.1
20		32.6	46.2	57.7	74.9	88.9	105.7	126.8
21		31.7	44.9	56.1	72.9	86.6	102.9	123.6
22		30.8	43.7	54.6	71.0	84.3	100.4	120.7
23		30.0	42.6	53.2	69.2	82.2	97.9	117.9
24		29.2	41.5	51.9	67.5	80.3	95.6	115.2
25		28.5	40.5	50.7	65.9	78.4	93.5	112.7
26		27.8	39.5	49.5	64.4	76.6	91.4	110.3
27		27.1	38.6	48.4	63.0	74.9	89.5	108.0
28		26.5	37.8	47.3	61.6	73.4	87.6	105.9
29	0.5	25.9	37.0	46.3	60.3	71.8	85.8	103.8
30		25.4	36.2	45.3	59.1	70.4	84.2	101.8
31		24.8	35.5	44.4	57.9	69.0	82.5	100.0
32		24.3	34.8	43.6	56.8	67.7	81.0	98.1
33		23.9	34.1	42.7	55.7	66.4	79.5	96.4
34		23.4	33.5	41.9	54.7	65.2	78.1	94.8
35		23.0	32.9	41.2	53.7	64.1	76.8	93.2
36		22.5	32.3	40.4	52.7	63.0	75.5	91.6
37		22.1	31.7	39.7	51.8	61.9	74.2	90.2
38		21.8	31.2	39.1	50.9	60.9	73.0	88.7
39		21.4	30.7	38.4	50.1	59.9	71.9	87.4
40		21.0	30.2	37.8	49.3	58.9	70.8	86.0
41		20.7	29.7	37.2	48.5	58.0	69.7	84.8

Table A7 - IDF Curves - Intensity Table (Part 2 of 3)
Edmonton 11 Rain Gauges Upper Bound - IDF Period: 1984-2015
Maximum Years of Record = 32
IDF Intensity (mm/hr)

Time		Return Frequency						
Minutes	Hours	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	200-yr
42	0.75	20.4	29.2	36.6	47.8	57.1	68.7	83.5
43		20.0	28.8	36.1	47.0	56.3	67.7	82.4
44		19.7	28.3	35.5	46.3	55.5	66.7	81.2
45		19.4	27.9	35.0	45.7	54.7	65.8	80.1
46		19.2	27.5	34.5	45.0	53.9	64.8	79.0
47		18.9	27.1	34.0	44.4	53.2	64.0	78.0
48		18.6	26.8	33.5	43.8	52.5	63.1	77.0
49		18.4	26.4	33.1	43.2	51.8	62.3	76.0
50		18.1	26.1	32.6	42.6	51.1	61.5	75.0
51		17.9	25.7	32.2	42.1	50.4	60.7	74.1
52		17.6	25.4	31.8	41.5	49.8	60.0	73.2
53		17.4	25.1	31.4	41.0	49.2	59.3	72.3
54		17.2	24.8	31.0	40.5	48.6	58.5	71.5
55		17.0	24.5	30.6	40.0	48.0	57.9	70.7
56		16.8	24.2	30.3	39.5	47.4	57.2	69.9
57	1.0	16.6	23.9	29.9	39.0	46.9	56.5	69.1
58		16.4	23.6	29.6	38.6	46.3	55.9	68.3
59		16.2	23.3	29.2	38.1	45.8	55.3	67.6
60		16.0	23.1	28.9	37.7	45.3	54.7	66.8
61		15.8	22.8	28.6	37.3	44.8	54.1	66.1
62		15.7	22.6	28.3	36.9	44.3	53.5	65.5
63		15.5	22.3	27.9	36.5	43.9	53.0	64.8
64		15.3	22.1	27.6	36.1	43.4	52.4	64.1
65		15.2	21.8	27.4	35.7	42.9	51.9	63.5
66		15.0	21.6	27.1	35.3	42.5	51.4	62.9
67		14.8	21.4	26.8	35.0	42.1	50.9	62.3
68		14.7	21.2	26.5	34.6	41.7	50.4	61.7
69		14.5	21.0	26.3	34.3	41.3	49.9	61.1
70		14.4	20.8	26.0	33.9	40.9	49.4	60.5
71		14.3	20.6	25.8	33.6	40.5	49.0	59.9
72	1.25	14.1	20.4	25.5	33.3	40.1	48.5	59.4
73		14.0	20.2	25.3	33.0	39.7	48.1	58.9
74		13.9	20.0	25.0	32.7	39.4	47.6	58.3
75		13.7	19.8	24.8	32.4	39.0	47.2	57.8
76		13.6	19.6	24.6	32.1	38.6	46.8	57.3
77		13.5	19.5	24.4	31.8	38.3	46.4	56.8
78		13.4	19.3	24.1	31.5	38.0	46.0	56.4

Table A7 - IDF Curves - Intensity Table (Part 3 of 3)
Edmonton 11 Rain Gauges Upper Bound - IDF Period: 1984-2015
Maximum Years of Record = 32
IDF Intensity (mm/hr)

Time		Return Frequency						
Minutes	Hours	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	200-yr
79		13.2	19.1	23.9	31.2	37.6	45.6	55.9
80		13.1	19.0	23.7	31.0	37.3	45.2	55.4
81		13.0	18.8	23.5	30.7	37.0	44.8	55.0
82		12.9	18.6	23.3	30.4	36.7	44.5	54.5
83		12.8	18.5	23.1	30.2	36.4	44.1	54.1
84		12.7	18.3	22.9	29.9	36.1	43.7	53.6
85		12.6	18.2	22.7	29.7	35.8	43.4	53.2
86		12.5	18.0	22.6	29.4	35.5	43.1	52.8
87		12.4	17.9	22.4	29.2	35.2	42.7	52.4
88		12.3	17.8	22.2	29.0	35.0	42.4	52.0
89		12.2	17.6	22.0	28.7	34.7	42.1	51.6
90	1.5	12.1	17.5	21.8	28.5	34.4	41.8	51.2
120	2	9.9	14.3	17.8	23.3	28.2	34.3	42.2
180	3	7.4	10.7	13.3	17.4	21.1	25.8	31.8
240	4	6.0	8.7	10.8	14.1	17.2	21.1	26.0
300	5	5.1	7.4	9.2	11.9	14.6	18.0	22.2
360	6	4.5	6.5	8.0	10.4	12.8	15.8	19.5
420	7	4.0	5.8	7.2	9.3	11.5	14.1	17.4
480	8	3.6	5.3	6.5	8.4	10.4	12.8	15.8
540	9	3.4	4.9	6.0	7.7	9.5	11.8	14.5
600	10	3.1	4.5	5.5	7.1	8.8	10.9	13.5
660	11	2.9	4.2	5.1	6.6	8.2	10.2	12.6
720	12	2.7	3.9	4.8	6.2	7.7	9.6	11.8
780	13	2.6	3.7	4.5	5.8	7.3	9.0	11.1
840	14	2.4	3.5	4.3	5.5	6.9	8.6	10.5
900	15	2.3	3.3	4.1	5.3	6.5	8.1	10.0
960	16	2.2	3.2	3.9	5.0	6.2	7.8	9.6
1020	17	2.1	3.1	3.7	4.8	6.0	7.4	9.2
1080	18	2.0	2.9	3.6	4.6	5.7	7.1	8.8
1140	19	1.9	2.8	3.4	4.4	5.5	6.8	8.4
1200	20	1.9	2.7	3.3	4.2	5.3	6.6	8.1
1260	21	1.8	2.6	3.2	4.1	5.1	6.4	7.8
1320	22	1.7	2.5	3.1	3.9	4.9	6.2	7.6
1380	23	1.7	2.4	3.0	3.8	4.8	6.0	7.3
1440	24	1.6	2.4	2.9	3.7	4.6	5.8	7.1

Table A8 - IDF Curves - Intensity Table-Summary
Edmonton 13 Rain Gauges Upper Bound - IDF Period: 1984-2015
Maximum Years of Record = 32
IDF Intensity (mm/hr)

Time		Return Frequency						
Minutes	Hours	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	200-yr
5	0.083	66.0	89.0	108.6	138.0	162.2	188.4	217.8
10	0.167	48.2	66.7	82.6	106.2	125.3	147.2	173.5
15	0.250	38.7	54.3	67.6	87.4	103.5	122.4	145.9
20	0.333	32.6	46.2	57.7	74.9	88.9	105.7	126.8
25	0.417	28.5	40.5	50.7	65.9	78.4	93.5	112.7
30	0.500	25.4	36.2	45.3	59.1	70.4	84.2	101.8
35	0.583	23.0	32.9	41.2	53.7	64.1	76.8	93.2
40	0.667	21.0	30.2	37.8	49.3	58.9	70.8	86.0
45	0.750	19.4	27.9	35.0	45.7	54.7	65.8	80.1
50	0.833	18.1	26.1	32.6	42.6	51.1	61.5	75.0
55	0.917	17.0	24.5	30.6	40.0	48.0	57.9	70.7
60	1	16.0	23.1	28.9	37.7	45.3	54.7	66.8
120	2	9.9	14.3	17.8	23.3	28.2	34.3	42.2
180	3	7.4	10.7	13.3	17.4	21.1	25.8	31.8
240	4	6.0	8.7	10.8	14.1	17.2	21.1	26.0
360	6	4.5	6.5	8.0	10.4	12.8	15.8	19.5
720	12	2.7	3.9	4.8	6.2	7.7	9.6	11.8
1440	24	1.6	2.4	2.9	3.7	4.6	5.8	7.1

IDF Parameters

Rate= $a \cdot (t+c)^b$	Return Frequency						
Parameters	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	200-yr
a (t in min)	337.0	498.1	665.2	909.9	1027.7	1200.5	1498.1
b	-0.732	-0.735	-0.748	-0.757	-0.742	-0.733	-0.735
c (min)	4.3	5.4	6.3	7.1	7.0	7.5	8.8
R ²	0.994	0.997	0.998	0.998	0.998	0.997	0.994

Table A9.1 - Chicago Distribution (Modified¹): 4-Hr Design Storm Data (mm/hr) (Part 1 of 2)

Edmonton 11 Rain Gauges Upper Bound, IDF-Period: 1984-2015

Maximum Years of Record = 32

Chicago Type Distribution - Design Storm (5-Minute Increment)

Time (min)	Return Frequency						
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	200-yr
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5	1.73	2.52	3.01	3.82	4.92	6.25	7.76
10	1.82	2.65	3.18	4.04	5.19	6.58	8.18
15	1.93	2.80	3.36	4.28	5.49	6.97	8.67
20	2.05	2.98	3.58	4.56	5.84	7.41	9.22
25	2.18	3.18	3.83	4.89	6.25	7.92	9.87
30	2.34	3.42	4.13	5.27	6.73	8.52	10.63
35	2.53	3.70	4.48	5.74	7.31	9.24	11.54
40	2.76	4.04	4.91	6.31	8.01	10.12	12.65
45	3.05	4.47	5.45	7.02	8.89	11.22	14.04
50	3.42	5.03	6.15	7.95	10.03	12.63	15.85
55	3.91	5.77	7.10	9.20	11.57	14.54	18.28
60	4.61	6.83	8.45	11.00	13.76	17.26	21.74
65	5.69	8.47	10.55	13.82	17.18	21.48	27.11
70	7.61	11.37	14.32	18.89	23.26	28.94	36.59
75	12.07	18.13	23.11	30.69	37.26	45.90	57.82
80	47.05	65.15	80.55	103.56	122.26	143.62	169.27
85	47.05	65.15	80.55	103.56	122.26	143.62	169.27
90	21.85	32.56	41.62	55.17	66.01	80.00	99.06
95	13.85	20.82	26.60	35.36	42.79	52.54	66.05
100	10.28	15.44	19.62	26.01	31.74	39.26	49.58
105	8.26	12.37	15.62	20.63	25.34	31.48	39.81
110	6.95	10.38	13.03	17.15	21.18	26.40	33.37
115	6.03	8.98	11.22	14.72	18.26	22.81	28.81
120	5.35	7.95	9.89	12.93	16.10	20.14	25.42
125	4.83	7.15	8.86	11.55	14.43	18.08	22.79
130	4.40	6.51	8.04	10.46	13.10	16.44	20.70
135	4.06	5.99	7.37	9.57	12.02	15.10	18.99
140	3.77	5.55	6.82	8.83	11.12	13.98	17.57
145	3.53	5.19	6.35	8.21	10.36	13.04	16.36
150	3.31	4.87	5.95	7.68	9.71	12.23	15.33
155	3.13	4.60	5.60	7.22	9.14	11.52	14.44
160	2.97	4.35	5.30	6.82	8.65	10.91	13.65
165	2.83	4.14	5.03	6.46	8.21	10.36	12.96
170	2.70	3.95	4.79	6.15	7.82	9.87	12.34
175	2.58	3.78	4.58	5.87	7.47	9.44	11.79
180	2.48	3.62	4.38	5.61	7.15	9.04	11.29

¹ Instantaneous Peak Averaged out

Table A9.1 - Chicago Distribution (Modified): 4-Hr Design Storm Data (mm/hr) (Part 2 of 2)

Edmonton 11 Rain Gauges Upper Bound, IDF-Period: 1984-2015

Maximum Years of Record = 32

Chicago Type Distribution - Design Storm (5-Minute Increment)

Time (min)	Return Frequency						
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	200-yr
185	2.38	3.48	4.21	5.38	6.86	8.68	10.83
190	2.30	3.35	4.04	5.17	6.60	8.36	10.42
195	2.22	3.23	3.90	4.98	6.36	8.06	10.04
200	2.14	3.12	3.76	4.80	6.14	7.78	9.69
205	2.08	3.02	3.64	4.64	5.94	7.53	9.37
210	2.01	2.93	3.52	4.49	5.75	7.29	9.07
215	1.95	2.84	3.41	4.35	5.57	7.07	8.80
220	1.90	2.76	3.31	4.21	5.41	6.86	8.54
225	1.85	2.69	3.22	4.09	5.26	6.67	8.30
230	1.80	2.62	3.13	3.98	5.12	6.49	8.07
235	1.75	2.55	3.05	3.87	4.98	6.33	7.86
240	1.71	2.49	2.97	3.77	4.86	6.17	7.66

Table A9.2 - Huff Distribution: Design Storm for Lake Drawdown Analysis Only (Part 1 of 3)

Edmonton 11 Rain Gauges Upper Bound, IDF-Period: 1984-2015

Maximum Years of Record = 32

Storm Duration = 24 hours

Huff Distribution (First-Quartile 50% Probability), mm/hr

Time		Return Frequency						
Mins.	Hrs.	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	200-yr
0	0	0	0	0	0	0	0	0
15		0.29	0.42	0.52	0.66	0.83	1.03	1.28
30		0.59	0.85	1.03	1.32	1.66	2.07	2.55
45		0.88	1.27	1.55	1.98	2.49	3.10	3.83
60	1	1.18	1.70	2.06	2.64	3.32	4.14	5.10
75		1.57	2.26	2.75	3.52	4.42	5.51	6.80
90		2.35	3.39	4.12	5.27	6.62	8.26	10.18
105		3.12	4.52	5.49	7.03	8.82	11.00	13.56
120	2	3.90	5.64	6.85	8.78	11.02	13.74	16.94
135		4.68	6.77	8.22	10.53	13.22	16.48	20.33
150		5.21	7.53	9.14	11.71	14.70	18.33	22.60
165		5.35	7.74	9.39	12.03	15.11	18.84	23.23
180	3	5.49	7.94	9.65	12.36	15.51	19.34	23.85
195		5.64	8.15	9.90	12.68	15.92	19.85	24.47
210		5.78	8.36	10.15	13.00	16.32	20.35	25.10
225		5.69	8.23	9.99	12.80	16.07	20.04	24.70
240	4	5.45	7.87	9.56	12.25	15.37	19.17	23.64
255		5.20	7.52	9.13	11.69	14.68	18.30	22.57
270		4.95	7.16	8.70	11.14	13.98	17.44	21.50
285		4.71	6.81	8.26	10.59	13.29	16.57	20.43
300	5	4.47	6.46	7.84	10.05	12.61	15.72	19.39
315		4.23	6.11	7.42	9.51	11.93	14.88	18.35
330		3.99	5.77	7.00	8.97	11.26	14.04	17.31
345		3.75	5.42	6.58	8.43	10.58	13.20	16.27
360	6	3.51	5.07	6.16	7.89	9.91	12.36	15.24
375		3.29	4.76	5.78	7.40	9.29	11.59	14.29
390		3.07	4.44	5.39	6.91	8.67	10.82	13.34
405		2.85	4.13	5.01	6.42	8.06	10.05	12.39
420	7	2.64	3.81	4.63	5.93	7.44	9.28	11.44
435		2.43	3.51	4.27	5.47	6.86	8.56	10.55
450		2.28	3.30	4.00	5.13	6.44	8.03	9.90
465		2.13	3.08	3.74	4.79	6.01	7.50	9.24
480	8	1.98	2.86	3.47	4.45	5.59	6.97	8.59
495		1.83	2.64	3.21	4.11	5.16	6.44	7.94

Table A9.2 - Huff Distribution: Design Storm for Lake Drawdown Analysis Only (Part 2 of 3)

Edmonton 11 Rain Gauges Upper Bound, IDF-Period: 1984-2015

Maximum Years of Record = 32

Storm Duration = 24 hours

Huff Distribution (First-Quartile 50% Probability), mm/hr

Time		Return Frequency						
Mins.	Hrs.	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	200-yr
510		1.71	2.48	3.01	3.85	4.84	6.03	7.44
525		1.65	2.39	2.90	3.72	4.67	5.82	7.17
540	9	1.59	2.30	2.79	3.58	4.49	5.60	6.91
555		1.53	2.21	2.69	3.44	4.32	5.38	6.64
570		1.47	2.12	2.58	3.30	4.14	5.17	6.37
585		1.41	2.03	2.47	3.16	3.97	4.95	6.10
600	10	1.34	1.94	2.36	3.02	3.80	4.73	5.84
615		1.28	1.86	2.25	2.89	3.62	4.52	5.57
630		1.22	1.77	2.15	2.75	3.45	4.30	5.30
645		1.16	1.68	2.04	2.61	3.28	4.09	5.04
660	11	1.10	1.60	1.94	2.48	3.12	3.89	4.79
675		1.05	1.52	1.84	2.36	2.96	3.70	4.56
690		1.00	1.44	1.75	2.24	2.81	3.50	4.32
705		0.94	1.36	1.65	2.11	2.65	3.31	4.08
720	12	0.89	1.28	1.56	1.99	2.50	3.12	3.84
735		0.86	1.24	1.51	1.93	2.42	3.02	3.73
750		0.83	1.20	1.46	1.87	2.35	2.93	3.61
765		0.80	1.16	1.41	1.81	2.27	2.83	3.49
780	13	0.78	1.12	1.36	1.75	2.19	2.73	3.37
795		0.75	1.08	1.31	1.68	2.11	2.63	3.25
810		0.71	1.03	1.25	1.60	2.01	2.51	3.10
825		0.68	0.98	1.19	1.53	1.92	2.39	2.95
840	14	0.65	0.93	1.13	1.45	1.82	2.27	2.80
855		0.61	0.88	1.07	1.37	1.72	2.15	2.65
870		0.59	0.85	1.03	1.32	1.66	2.07	2.55
885		0.58	0.84	1.02	1.31	1.64	2.04	2.52
900	15	0.57	0.83	1.01	1.29	1.62	2.02	2.49
915		0.57	0.82	1.00	1.28	1.60	2.00	2.46
930		0.56	0.81	0.98	1.26	1.58	1.97	2.43
945		0.55	0.80	0.97	1.24	1.56	1.95	2.40
960	16	0.55	0.79	0.96	1.23	1.54	1.92	2.37
975		0.54	0.78	0.95	1.21	1.52	1.90	2.34
990		0.53	0.77	0.94	1.20	1.50	1.88	2.31
1005		0.53	0.76	0.92	1.18	1.49	1.85	2.28

Table A9.2 - Huff Distribution: Design Storm for Lake Drawdown Analysis Only (Part 3 of 3)

Edmonton 11 Rain Gauges Upper Bound, IDF-Period: 1984-2015

Maximum Years of Record = 32

Storm Duration = 24 hours

Huff Distribution (First-Quartile 50% Probability), mm/hr

Time		Return Frequency						
Mins.	Hrs.	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	200-yr
1020	17	0.52	0.75	0.91	1.17	1.47	1.83	2.25
1035		0.51	0.74	0.90	1.15	1.45	1.80	2.22
1050		0.51	0.73	0.89	1.14	1.43	1.78	2.20
1065		0.50	0.72	0.88	1.12	1.41	1.76	2.17
1080	18	0.49	0.71	0.86	1.11	1.39	1.73	2.14
1095		0.49	0.70	0.85	1.09	1.37	1.71	2.11
1110		0.48	0.69	0.84	1.08	1.35	1.68	2.08
1125		0.47	0.68	0.83	1.06	1.33	1.66	2.05
1140	19	0.46	0.67	0.82	1.05	1.31	1.64	2.02
1155		0.46	0.66	0.80	1.03	1.29	1.61	1.98
1170		0.44	0.64	0.78	1.00	1.25	1.56	1.92
1185		0.43	0.62	0.75	0.97	1.21	1.51	1.86
1200	20	0.42	0.60	0.73	0.93	1.17	1.46	1.80
1215		0.40	0.58	0.71	0.90	1.13	1.41	1.74
1230		0.39	0.56	0.68	0.87	1.10	1.37	1.68
1245		0.37	0.54	0.66	0.84	1.06	1.32	1.63
1260	21	0.36	0.52	0.63	0.81	1.02	1.27	1.57
1275		0.35	0.50	0.61	0.78	0.98	1.22	1.51
1290		0.33	0.48	0.59	0.75	0.94	1.17	1.45
1305		0.32	0.46	0.56	0.72	0.90	1.13	1.39
1320	22	0.31	0.44	0.54	0.69	0.86	1.08	1.33
1335		0.29	0.42	0.51	0.66	0.83	1.03	1.27
1350		0.28	0.40	0.49	0.63	0.79	0.98	1.21
1365		0.27	0.38	0.47	0.60	0.75	0.93	1.15
1380	23	0.25	0.36	0.44	0.57	0.71	0.89	1.09
1395		0.24	0.34	0.42	0.53	0.67	0.84	1.03
1410		0.22	0.32	0.39	0.50	0.63	0.79	0.97
1425		0.21	0.30	0.37	0.47	0.59	0.74	0.91
1440	24	0.20	0.28	0.35	0.44	0.56	0.69	0.85

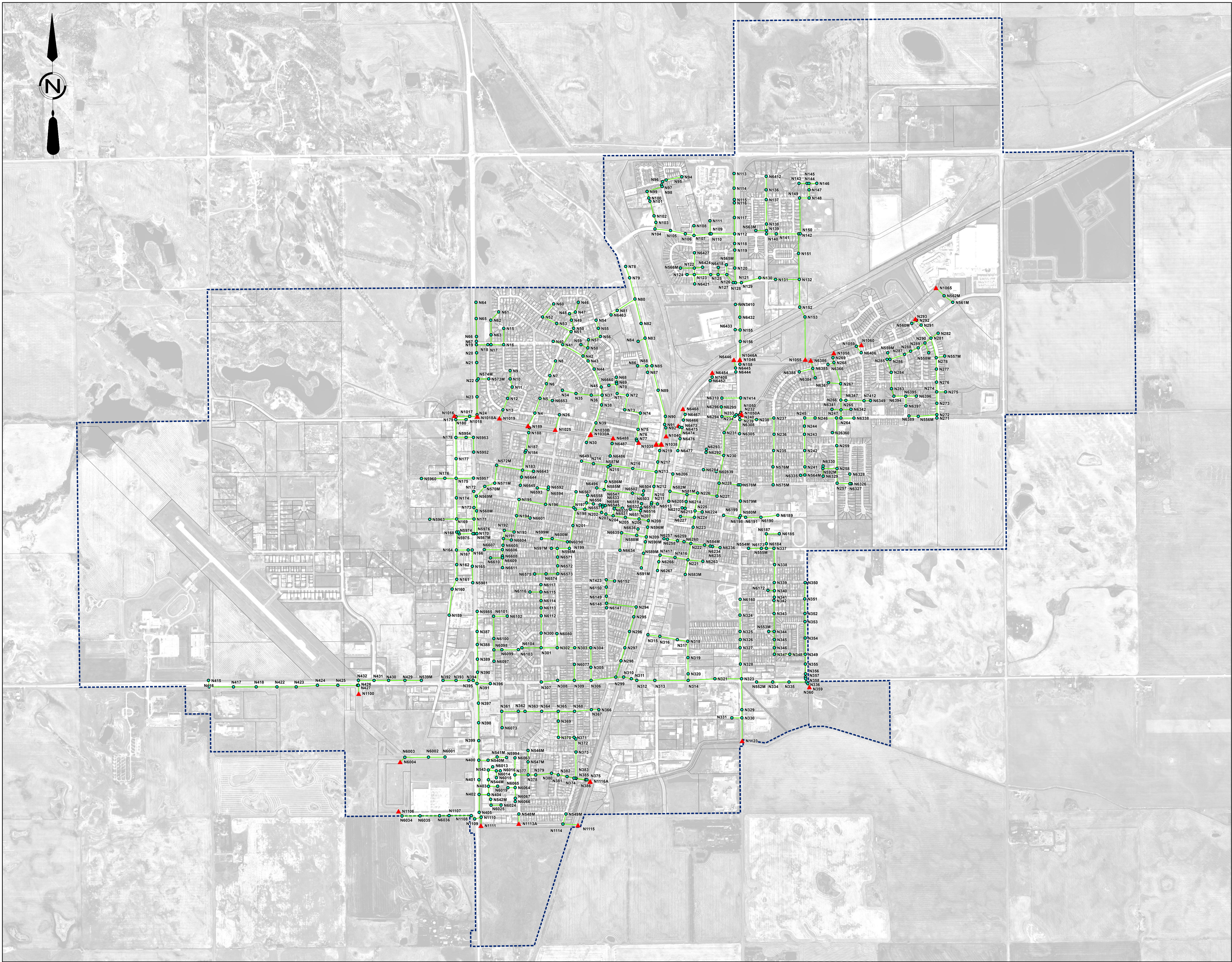
APPENDIX B - OFFSITE DRAINAGE MODEL SETUP PARAMETERS

\\s-edm-fs-01\projects\2019\3447\00_UMP_Update\Engineering\03.00_Conceptual_Feasibility_Design_Master_Plans\Storm\Report\Report\2019\3447_Storm\WtrMstrPlan_final_draft.docx



PCSWMM Parameter	Value
Percent of Impervious Area	10 %
Manning's n for Impervious Area	0.016
Manning's n for Pervious Area	0.5
Depth of Depression Storage - Impervious Area	7 mm
Depth of Depression Storage - Pervious Area	7 mm
Percent of Pervious Area without Depression Storage	25 %
Percent Routed	100 %
Horton Infiltration Parameters	
Maximum Infiltration Rate	75 mm/hr
Minimum Infiltration Rate	5 mm/hr
Decay Constant	4 /hr
Drying Time	7 days

APPENDIX C - COUPLED LINK CONNECTIONS



Legend

- Storm Manhole/CB
- Storm Outfall
- Storm Pipe
- City of Wetaskiwin Boundary



FIGURE C-1
CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE

EXISTING STORM SYSTEM MANHOLE
NUMBERING PLAN

AE PROJECT No.	2019-3447
SCALE	1:11,500
APPROVED	
DATE	2020MAR25
REV	
DESCRIPTION	ISSUED FOR DRAFT

Coupled 1D-2D Model
River/Urban Link Options

Link Type	River Name	Urban ID	<u>River Chainage</u> From	Type
River/Urban		N1115		MIKE URBAN Outlet to MIKE 11

Coupled 1D-2D Model
Standard/Structure Link Options

Link Type	River Name	Chainage	Mom Fact	Ext Fact	Add/Replace Flow	Depth Adjust	Activation Depth (min)	Activation Depth (max)	Exponential Smoothing Factor
Standard	NDiversion01	0	0			No			1
Standard	NDiversion01	14.671	0			No			1
Standard	NDiversion02a	0	0			No			1
Standard	NDiversion02a	17.598	0			No			1
Standard	NDitch02	0	0			No			1
Standard	NDitch02	9.636	0			No			1
Standard	NDitch04	0	0			No			1
Standard	NDitch04	26.882	0			No			1
Standard	NDitch05	0	0			No			1
Standard	NDitch05	26.791	0			No			1
Standard	NDitch06	0	0			No			1
Standard	NDitch06	32.228	0			No			1
Standard	NDitch07a	0	0			No			1
Standard	NDitch07a	18.783	0			No			1
Standard	NDitch09a	0	0			No			1
Standard	NDitch09a	30.383	0			No			1
Standard	NDitch10a	0	0			No			1
Standard	NDitch10a	55.541	0			No			1
Standard	NDitch11	0	0			No			1
Standard	NDitch11	30.73	0			No			1
Standard	NDitch12	0	0			No			1
Standard	NDitch12	24.208	0			No			1
Standard	NDitchInlet01	0	0			No			1
Standard	NDitchInlet01	7.016	0			No			1
Standard	SDitch01a	0	0			No			1
Standard	SDitch01a	44.809	0			No			1
Standard	SDitch02.5	0	0			No			1
Standard	SDitch02.5	196.114	0			No			1
Standard	SDitch03	0	0			No			1
Standard	SDitch03	31.069	0			No			1
Standard	SDitch04	0	0			No			1
Standard	SDitch04	105.133	0			No			1
Standard	SDitch05a	0	0			No			1
Standard	SDitch05a	40.958	0			No			1
Standard	SDitch06	0	0			No			1
Standard	NDitch13	0	0			No			1
Standard	NDitch13	10.603	0			No			1

Coupled 1D-3D Model

Urban Link Options

Link Type	Urban ID	Type	Max flow	Inlet area	Inlet method	Crest width	Freeboard	Discharge coef.	Scaling	Exp.	Qdh factor	DQ table
Urban	N124	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N125	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N126	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N127	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N128	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N129	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N219	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N22	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N221	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N222	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N223	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N224	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N367	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N368	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N369	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N37	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N370	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N371	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N372	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N373	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N374	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N375	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N377	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N378	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N379	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N38	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N380	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N381	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N382	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N383	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N385	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N386	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N387	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N388	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N389	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N39	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N390	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N391	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N392	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N393	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N394	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N395	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N396	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N397	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N398	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N399	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N4	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N40	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N13	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N130	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N131	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N132	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N136	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N137	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N138	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N139	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N140	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N141	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N142	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N143	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N144	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N145	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N146	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N147	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N148	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N149	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N15	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N150	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N151	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N152	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N153	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N155	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N156	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N158	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N159	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N160	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N161	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N162	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N164	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N165	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N166	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N167	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N168	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N169	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N17	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N170	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N171	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N172	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N173	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N174	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N175	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N176	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N177	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N178	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N179	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N18	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N180	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	

Urban Link Options

Link Type	Urban ID	Type	Max flow	Inlet area	Inlet method	Crest width	Freeboard	Discharge coef.	Scaling	Exp.	Qdh factor	DQ table
Urban	N183	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N184	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N187	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N188	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N189	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N19	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N191	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N192	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N193	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N194	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N195	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N196	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N197	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N198	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N199	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N20	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N200	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N201	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N202	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N203	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N204	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N205	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N206	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N207	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N208	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N209	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N21	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N210	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N211	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N212	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N213	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N214	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N215	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N216	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N217	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N230	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N231	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N232	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N233	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N234	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N235	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N225	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N226	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N227	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N228	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N229	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N23	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N327	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N329	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N330	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N331	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N334	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N335	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N336	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N337	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N338	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N339	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N34	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N340	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N341	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N342	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N343	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N344	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N345	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N346	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N347	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N348	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N349	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N35	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N350	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N351	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N352	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N353	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N354	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N355	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N356	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N357	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N358	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N359	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N36	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N361	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N362	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N363	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N364	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N365	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N366	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N236	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N237	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N238	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N239	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N24	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N240	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N244	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N245	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N246	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N258	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N259	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	

Coupled 1D-3D Model

Urban Link Options

Link Type	Urban ID	Type	Max flow	Inlet area	Inlet method	Crest width	Freeboard	Discharge coef.	Scaling	Exp.	Qdh factor	DQ table
Urban	N26	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N268	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N269	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N271	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N272	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N273	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N274	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N275	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N276	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N277	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N278	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N281	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N282	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N283	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N284	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N285	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N286	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N287	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N288	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N289	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N290	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N291	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N292	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N295	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N297	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N298	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N299	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N30	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N300	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N301	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N302	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N303	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N304	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N305	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N306	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N309	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N314	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N315	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N317	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N319	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N324	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N325	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N10	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N100	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N101	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N1016	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N1017	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N102	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N103	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N104	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N105	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N1059	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N106	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N107	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N108	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N109	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N11	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N110	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N1108	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N1109	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N111	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N1110	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N1114	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N112	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N113	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N114	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N115	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N116	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N117	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N118	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N119	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N12	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N120	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N121	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N122	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N123	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N400	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N401	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N402	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N403	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N404	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N405	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N41	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N415	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N416	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N417	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N418	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N42	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N422	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N423	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N424	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N425	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N426	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N427	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N429	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N43	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N430	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	

Coupled 1D-3D Models

Urban Link Options

Link Type	Urban ID	Type	Max flow	Inlet area	Inlet method	Crest width	Freeboard	Discharge coef.	Scaling	Exp.	Qdh factor	DQ table
Urban	N431	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N432	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N44	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N46	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N47	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N48	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N49	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N5	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N50	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N51	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N52	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N53	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N539M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N54	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N540M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N541M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N542M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N543	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N544M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N546M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N547M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N548M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N549M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N55	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N552M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N553M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N554M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N555M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N556M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N557M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N558M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N559M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N56	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N560M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N561M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N562M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N563M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N564M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N565M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N566M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N567M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N568M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N569M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N57	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N570M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N571M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N572M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N573M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N574M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N575M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N576M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N577M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N578M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N579M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N58	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N580M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N581M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N582M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N583M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N584M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N585M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N586M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N587M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N588M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N589M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N59	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N590M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N591M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N596M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N597M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N598M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N599M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N60	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N600M	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N601	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N61	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N62	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N63	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N64	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N65	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N66	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N67	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N68	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N69	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N70	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N71	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N72	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N73	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N74	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N75	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N76	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N77	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N78	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N79	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N8	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N80	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	

Coupled 1D-2D Mode

Urban Link Option

Link Type	Urban ID	Type	Max flow	Inlet area	Inlet method	Crest width	Freeboard	Discharge coef.	Scaling	Exp.	Qdh factor	DQ table
Urban	N81	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N82	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N83	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N84	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N85	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N86	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N87	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N88	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N89	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N9	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N90	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N91	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N92	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N94	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N95	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N96	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N97	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N98	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N99	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N1107	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N16	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N45	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N7	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N5952	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N5953	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N5954	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N5957	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N5960	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N5963	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N5974	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N5975	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N5976	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N5981	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N5985	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N5994	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6001	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6002	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6003	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6013	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6014	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6015	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6016	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6019	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6024	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6025	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6034	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6035	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6036	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6063	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6064	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6065	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6066	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6067	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6073	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6077	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6080	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6097	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6100	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6103	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6150	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6172	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6173	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6184	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6185	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6187	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6189	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6190	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6191	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6198	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6199	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6205	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6206	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6212	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6213	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6214	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6224	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6227	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6234	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6235	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6236	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6257	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6258	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6259	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6260	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6263	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6266	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6267	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6283	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6284	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6292	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6293	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6294	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6295	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6296	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6305	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6308	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6310	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	

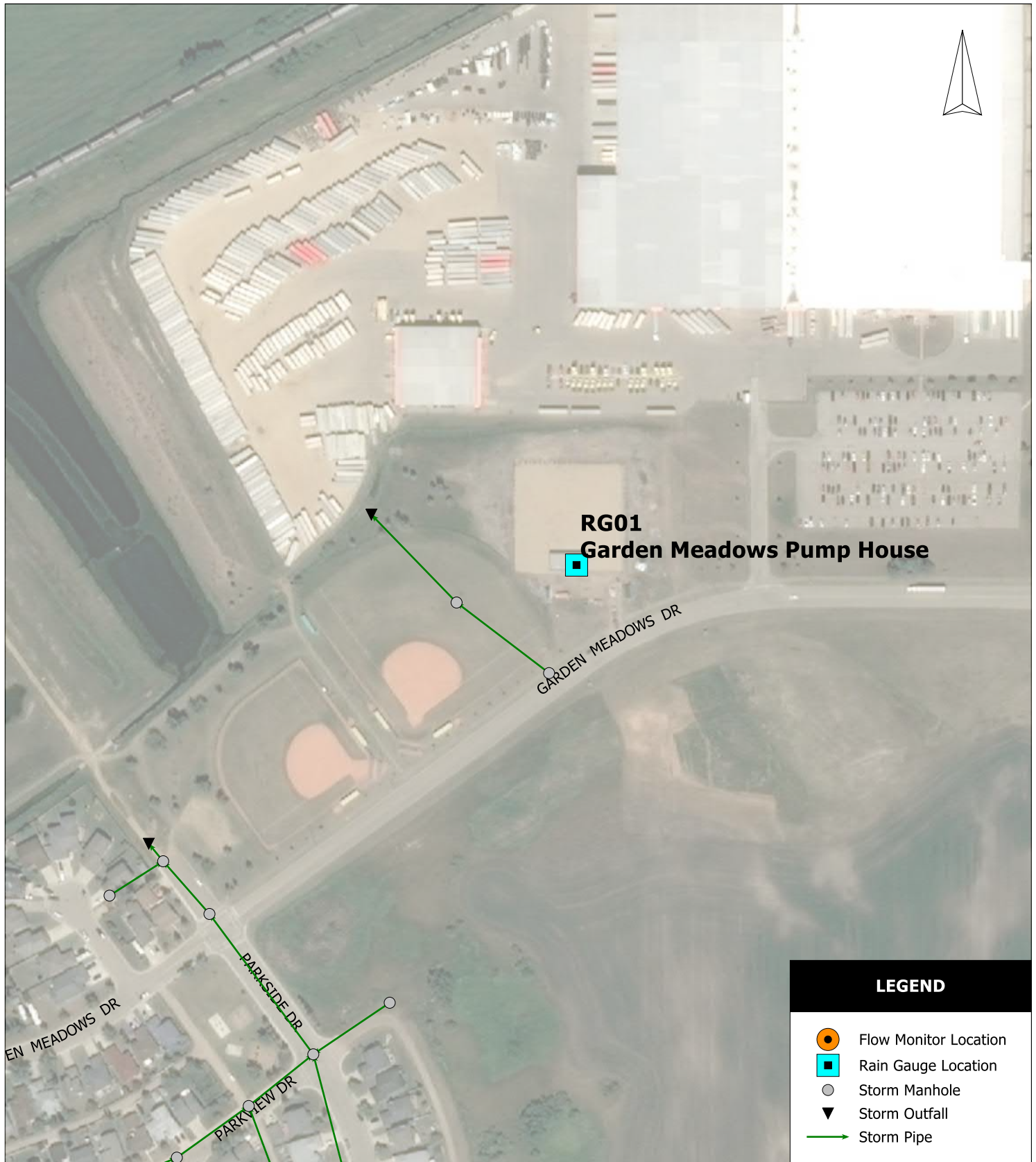
Coupled 1D-2D Mode

Urban Link Options

Link Type	Urban ID	Type	Max flow	Inlet area	Inlet method	Crest width	Freeboard	Discharge coef.	Scaling	Exp.	Qdh factor	DQ table
Urban	N6327	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6328	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6338	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6347	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6366	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6384	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6385	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6388	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6389	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6394	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6395	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6396	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6397	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6406	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6412	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6418	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6421	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6424	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6427	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6430	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6432	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6433	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6444	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6445	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6452	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N7416	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N7417	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6463	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6466	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6467	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6474	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6475	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6476	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6477	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6486	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6487	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6493	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6496	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6502	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6503	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6504	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6513	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6516	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6517	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	
Urban	N6518	M21 to inlet	0.2552	0.3	Orifice eq.			0.6			0	</

Coupled 1D-2D Model												
Urban Link Options												
Link Type	Urban ID	Type	Max flow	Inlet area	Inlet method	Crest width	Freeboard	Discharge coef.	Scaling	Exp.	Qdh factor	DQ table
Urban	N1058	M21 to outlet			Orifice eq.						0	
Urban	N1060	M21 to outlet			Orifice eq.						0	
Urban	N1065	M21 to outlet			Orifice eq.						0	
Urban	N1100	M21 to outlet			Orifice eq.						0	
Urban	N1106	M21 to outlet			Orifice eq.						0	
Urban	N1111	M21 to outlet			Orifice eq.						0	
Urban	N1116	M21 to outlet			Orifice eq.						0	
Urban	N6004	M21 to outlet			Orifice eq.						0	
Urban	N1116A	M21 to outlet			Orifice eq.						0	
Urban	N1050A	M21 to outlet			Orifice eq.						0	
Urban	N6386	M21 to outlet			Orifice eq.						0	
Urban	N6446	M21 to outlet			Orifice eq.						0	
Urban	N1018A	M21 to outlet			Orifice eq.						0	
Urban	N1030A	M21 to outlet			Orifice eq.						0	
Urban	N1030B	M21 to outlet			Orifice eq.						0	
Urban	N1046A	M21 to outlet			Orifice eq.						0	
Urban	N6454	M21 to outlet			Orifice eq.						0	
Urban	N6468	M21 to outlet			Orifice eq.						0	
Urban	N6473	M21 to outlet			Orifice eq.						0	
Urban	N6488	M21 to outlet			Orifice eq.						0	
Urban	Node_3	M21 to outlet			Orifice eq.						0	
Urban	Node_4	M21 to outlet			Orifice eq.						0	
Urban	Node_6	M21 to outlet			Orifice eq.						0	
Urban	Node_7	M21 to outlet			Orifice eq.						0	
Urban	Node_8	M21 to outlet			Orifice eq.						0	
Urban	Node_9	M21 to outlet			Orifice eq.						0	

APPENDIX D - FLOW MONITORING SITE MAPS



STORM SEWER
2019 FLOW MONITORING PROGRAM

Rain Gauge 01

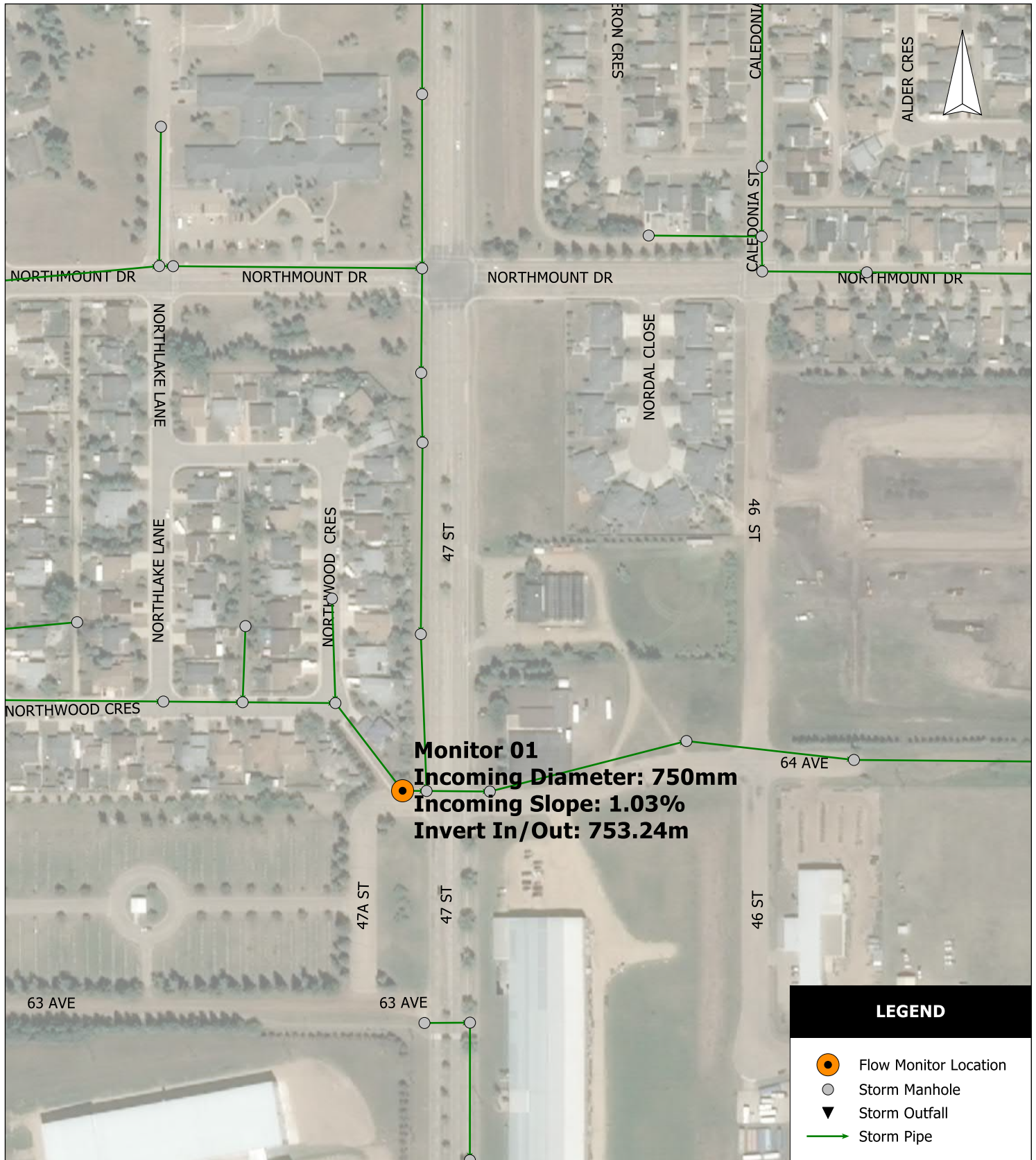
Scale 1:5,000



STORM SEWER 2019 FLOW MONITORING PROGRAM

Rain Gauge 02

Scale 1:5,000



**STORM SEWER
2019 FLOW MONITORING PROGRAM**

Flow Meter Site 01

Scale 1:5,000



STORM SEWER 2019 FLOW MONITORING PROGRAM

Flow Meter Site 02

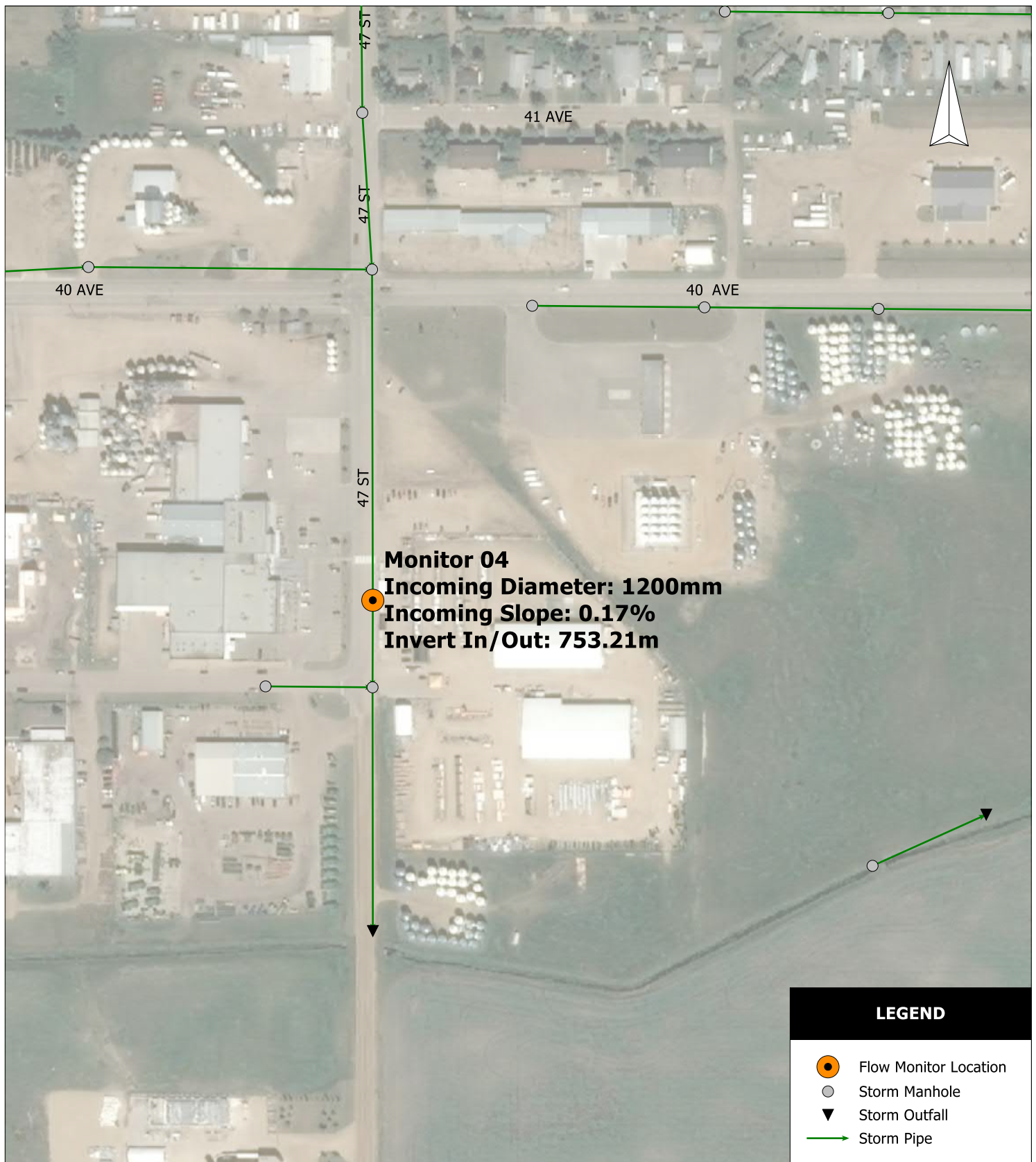
Scale 1:5,000



STORM SEWER 2019 FLOW MONITORING PROGRAM

Flow Meter Site 03

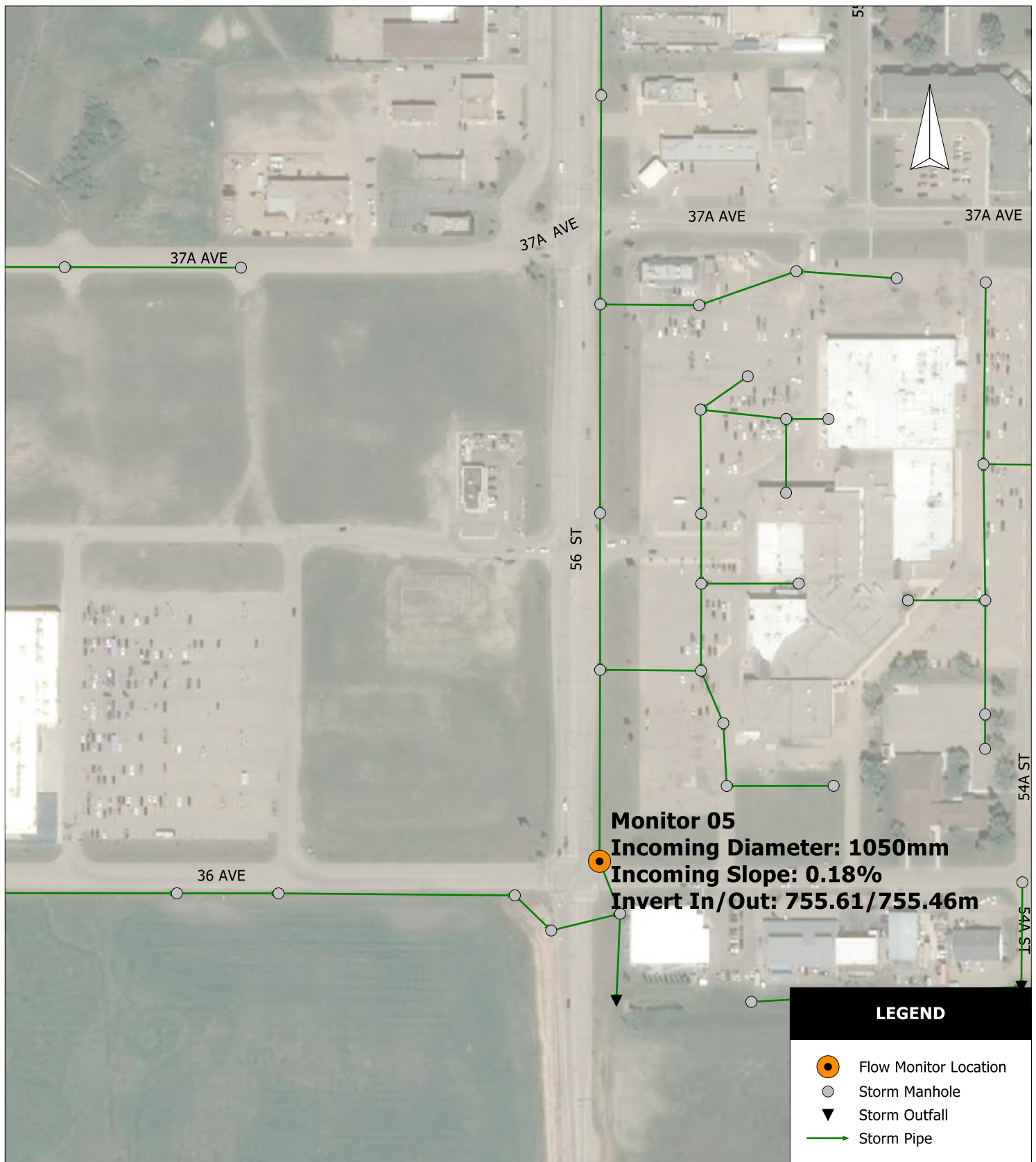
Scale 1:5,000



STORM SEWER
2019 FLOW MONITORING PROGRAM

Flow Meter Site 04

Scale 1:5,000



STORM SEWER 2019 FLOW MONITORING PROGRAM

Flow Meter Site 05

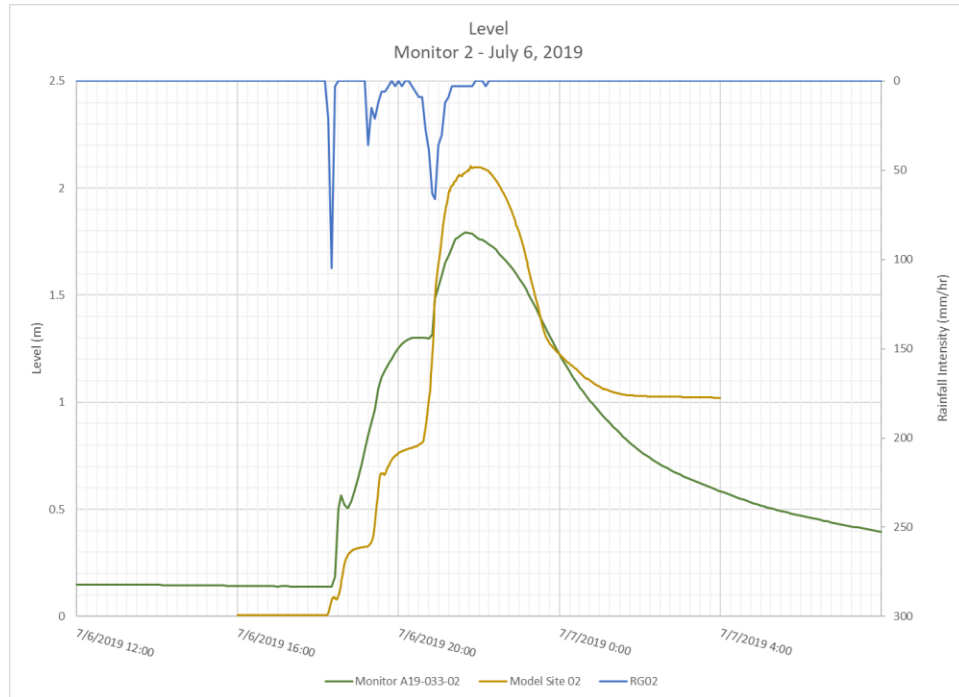
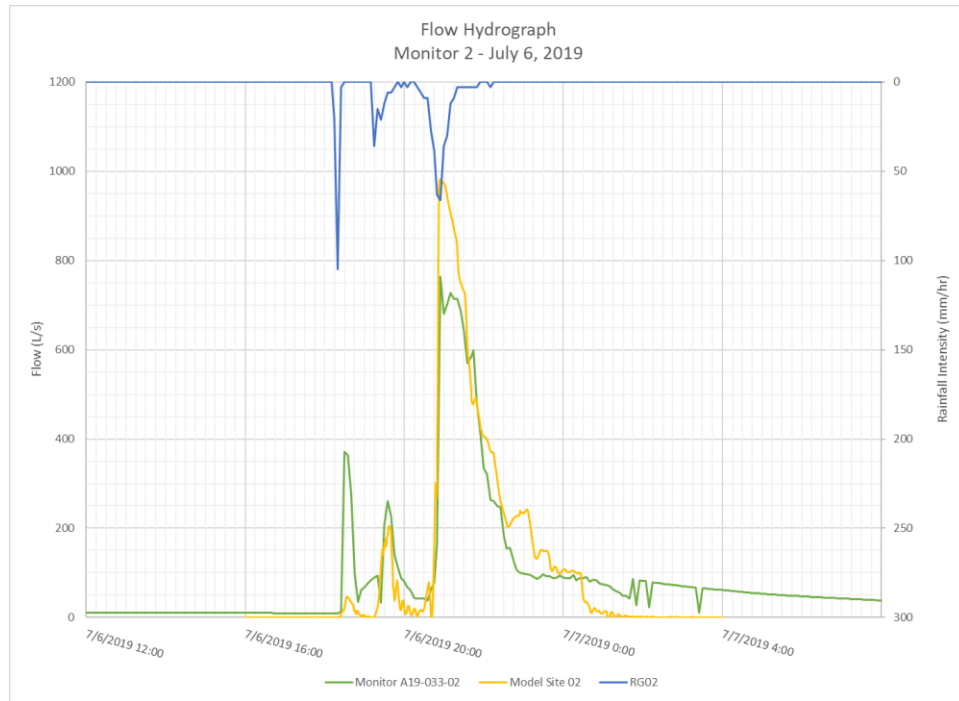
Scale 1:5,000

APPENDIX E - CALIBRATION/VALIDATION PLOTS

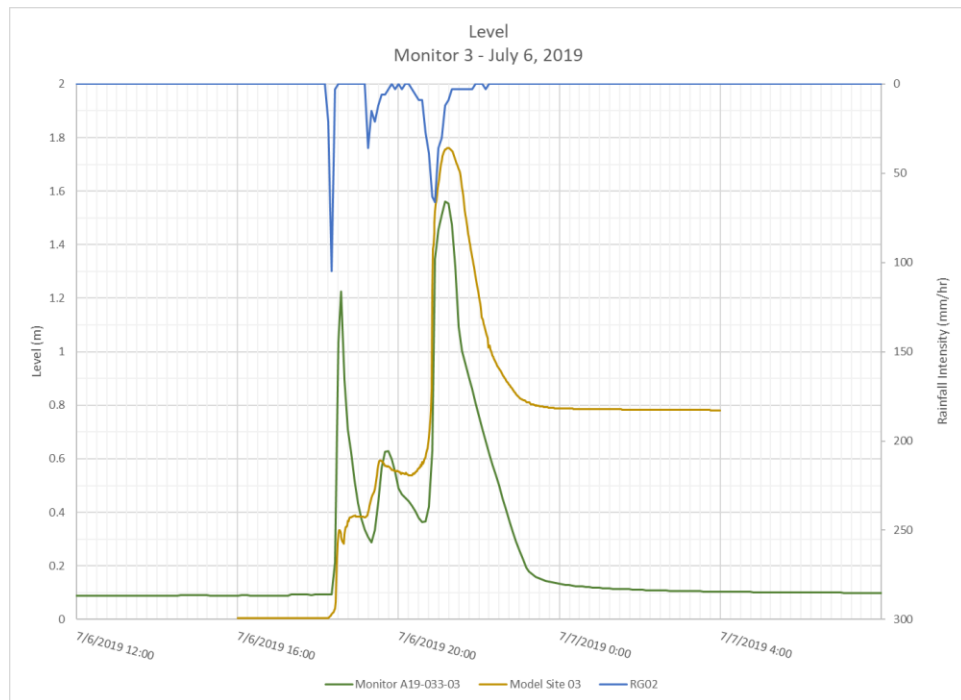
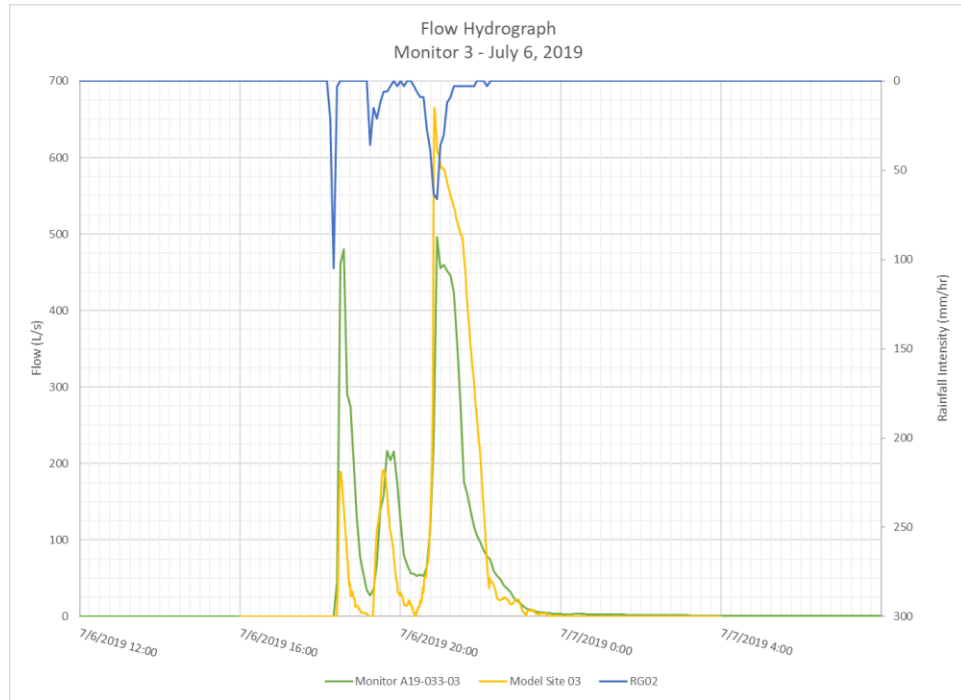
\\s-edm-fs-01\projects\2019\3447\00_UMP_Update\Engineering\03.00_Conceptual_Feasibility_Design_Master_Plans\Storm\Report\Report\2019\3447_Storm\WtrMstrPlan_final_draft.docx



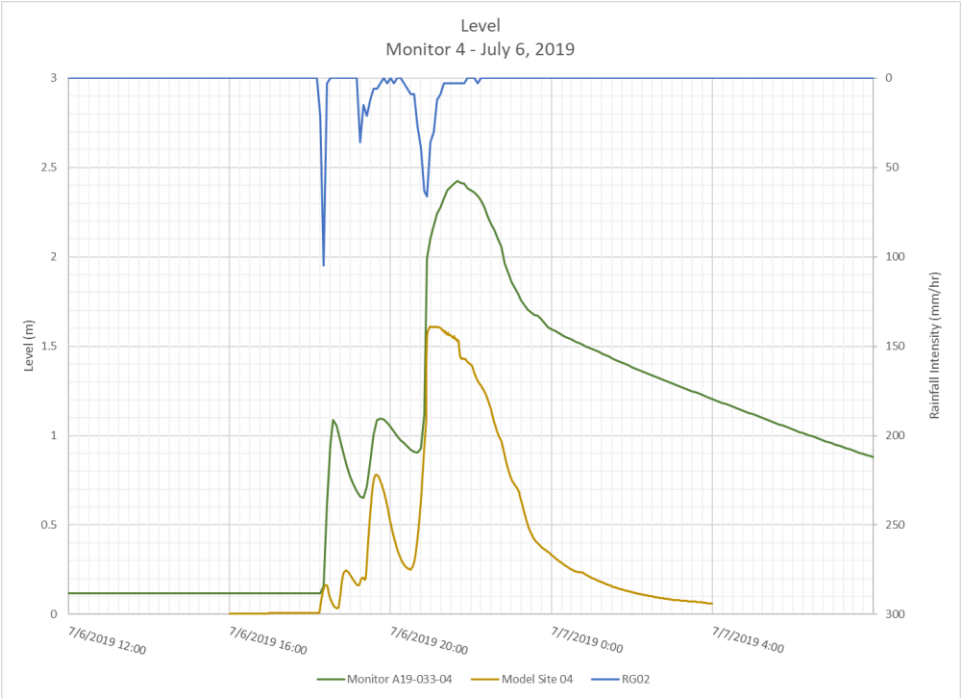
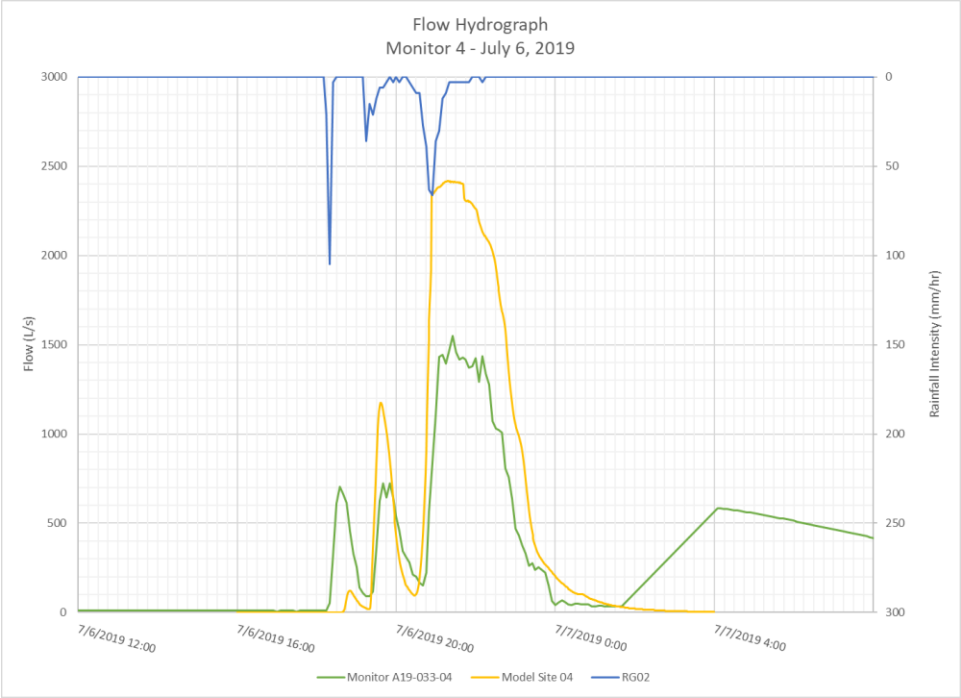
Calibration



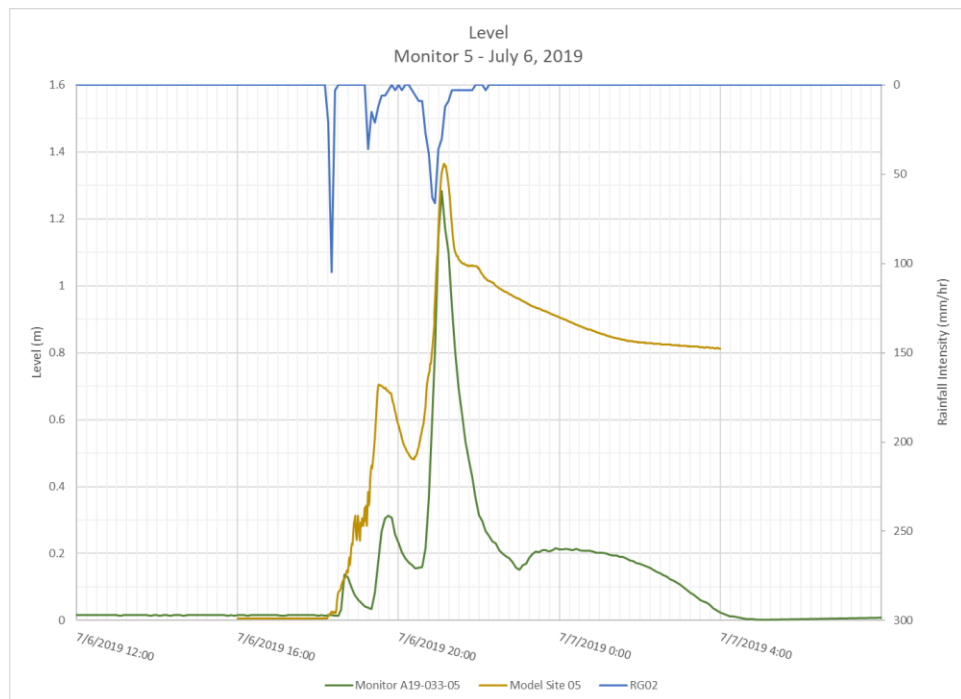
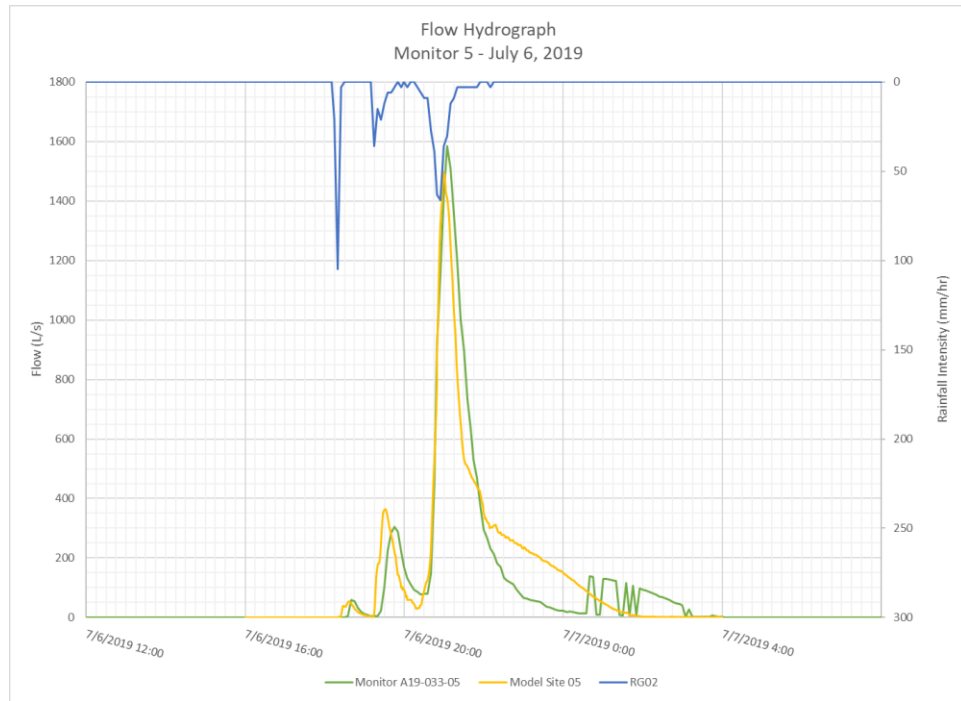
Calibration



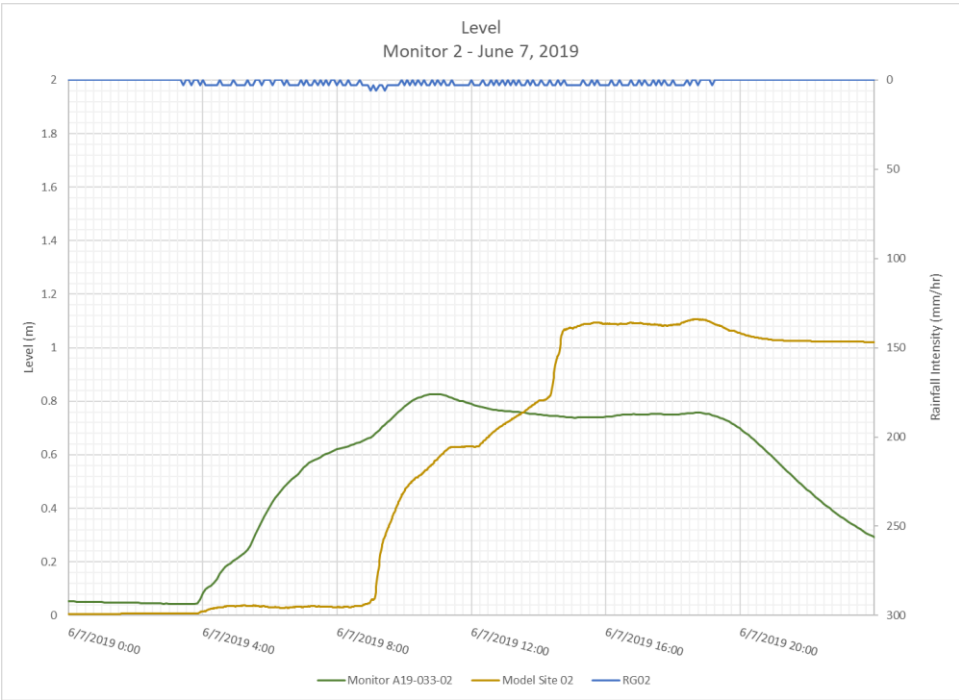
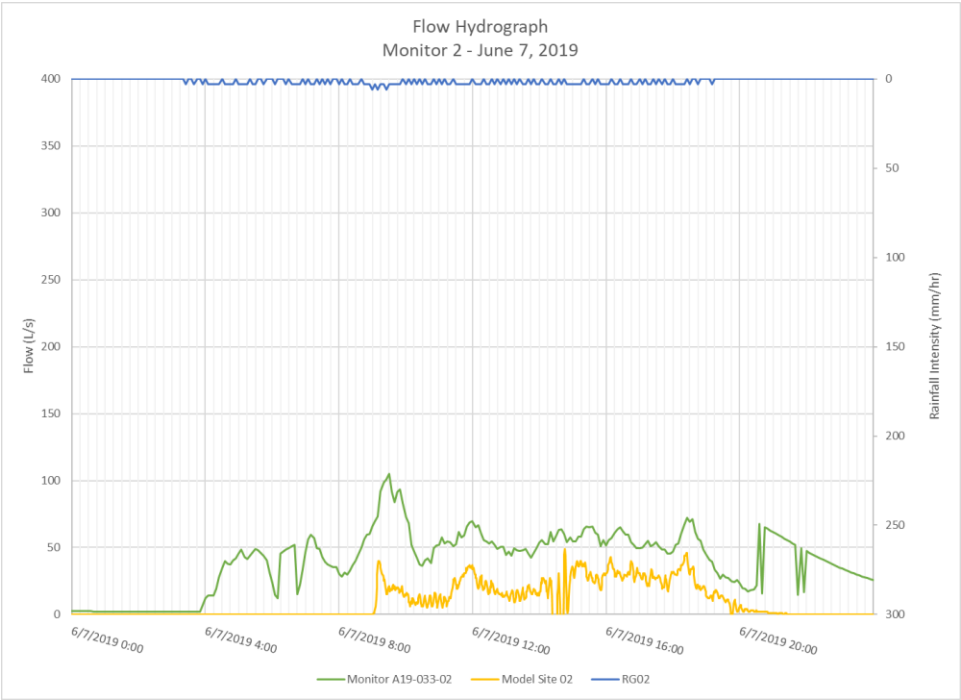
Calibration



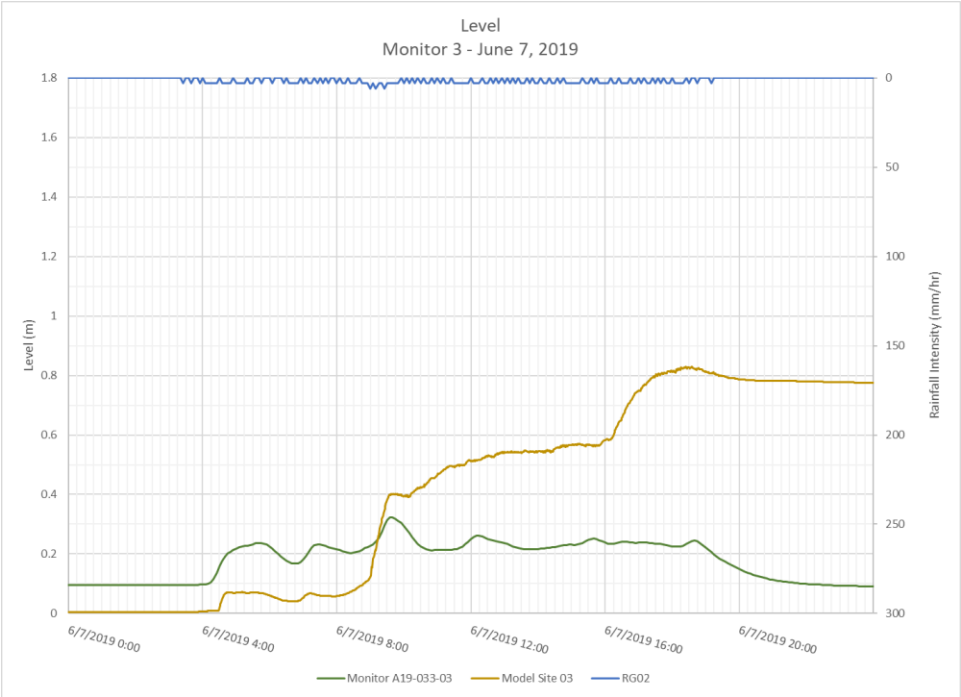
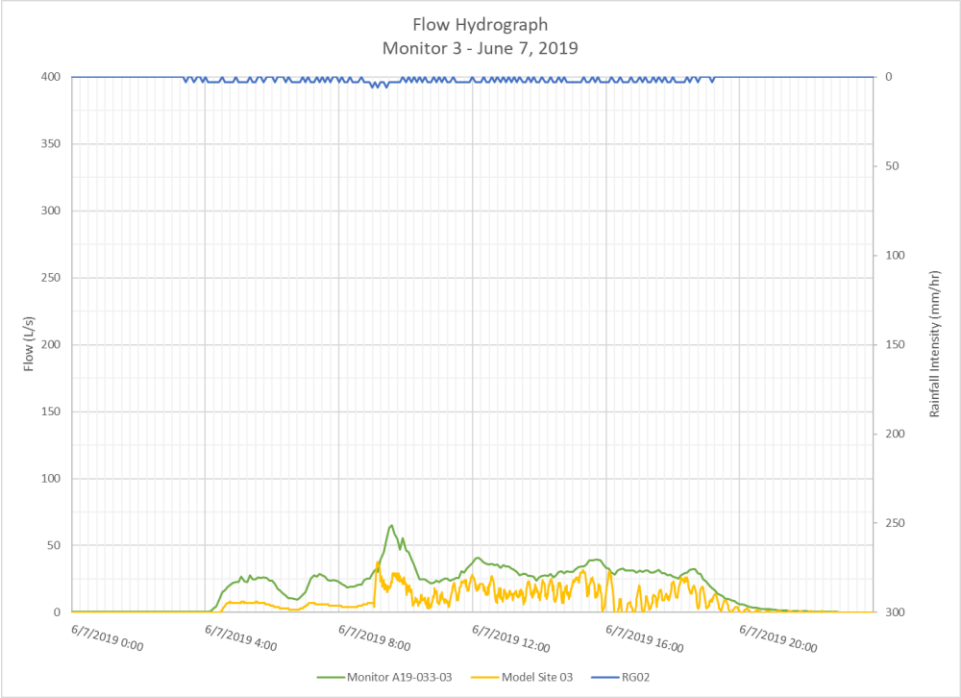
Calibration



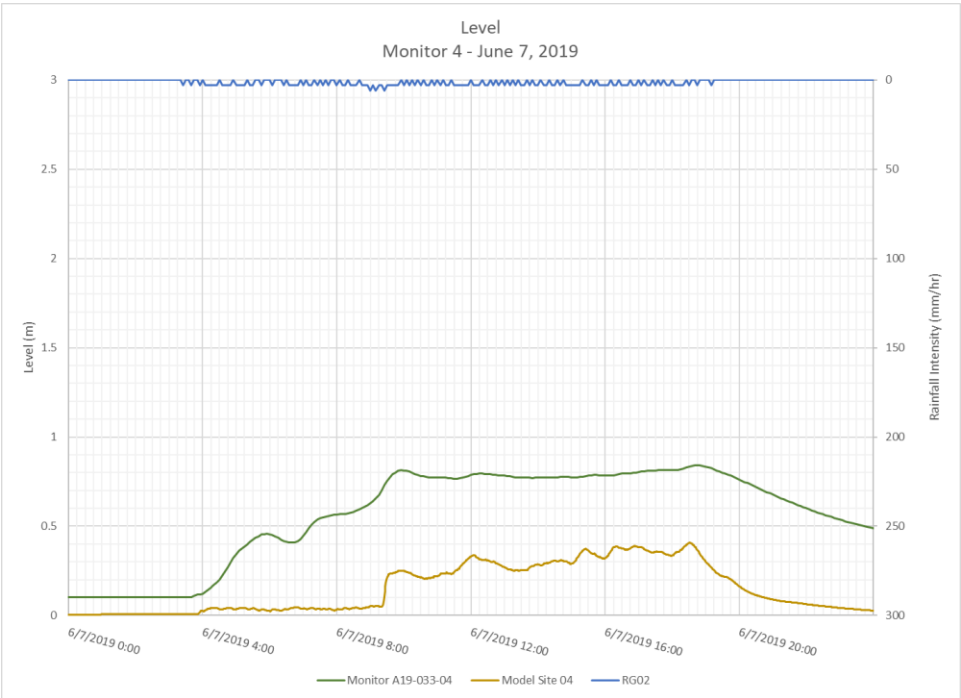
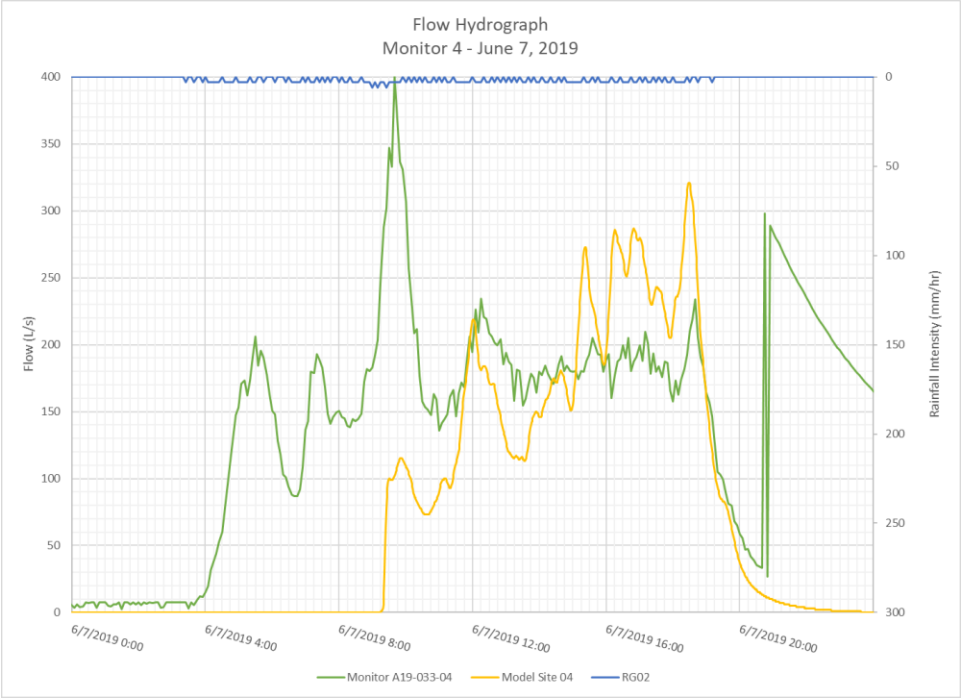
Validation



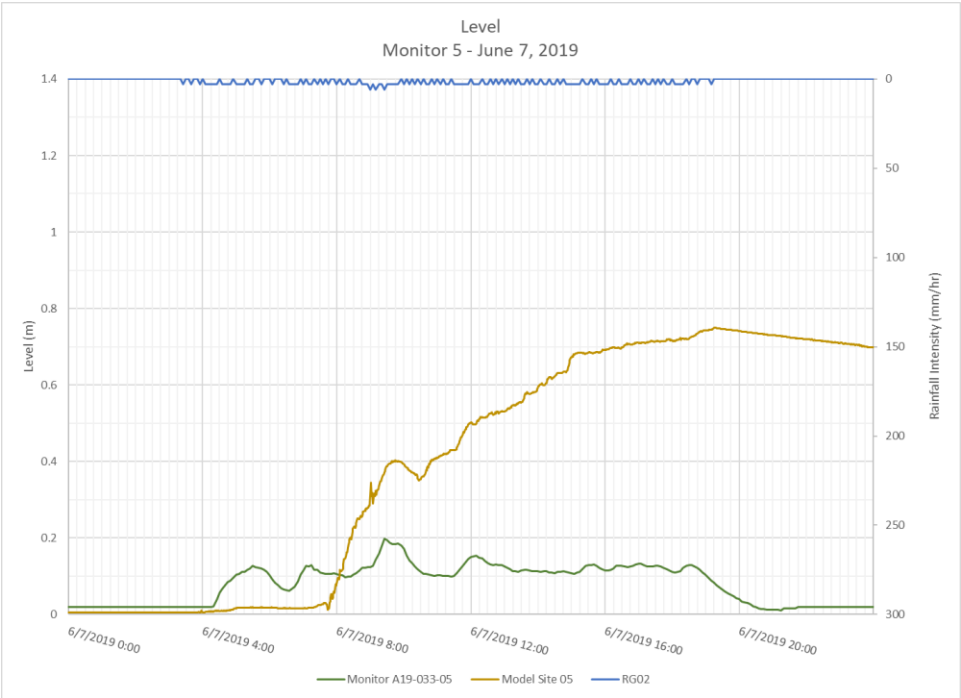
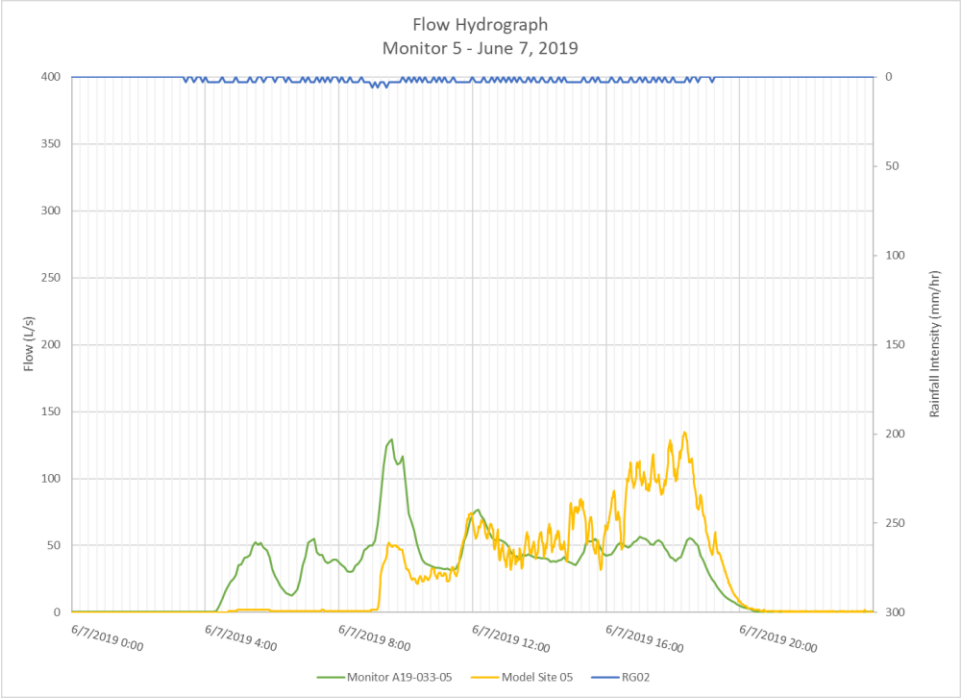
Validation



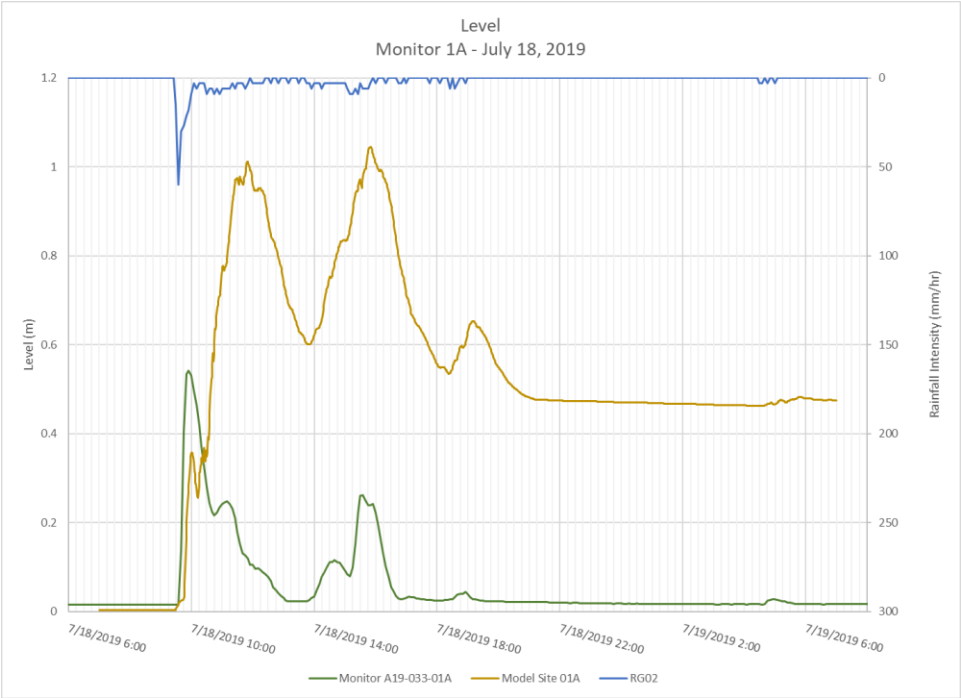
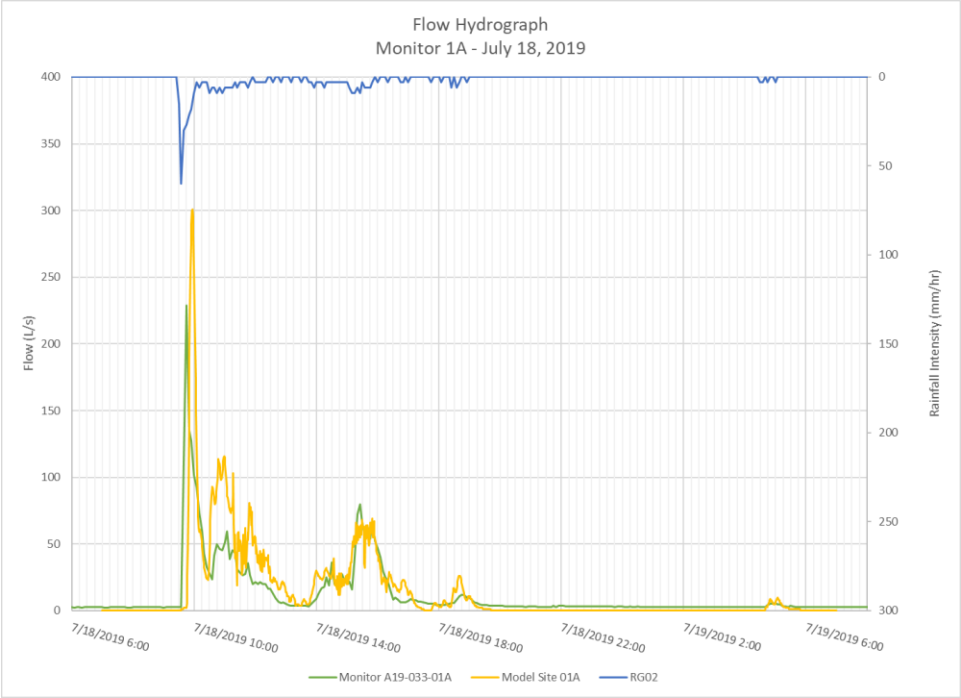
Validation



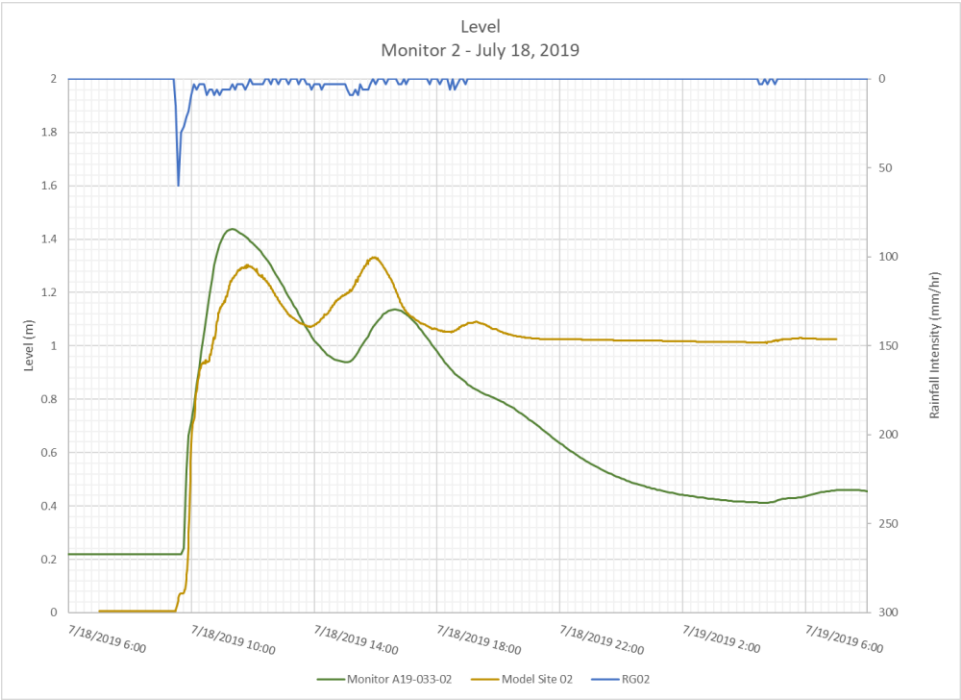
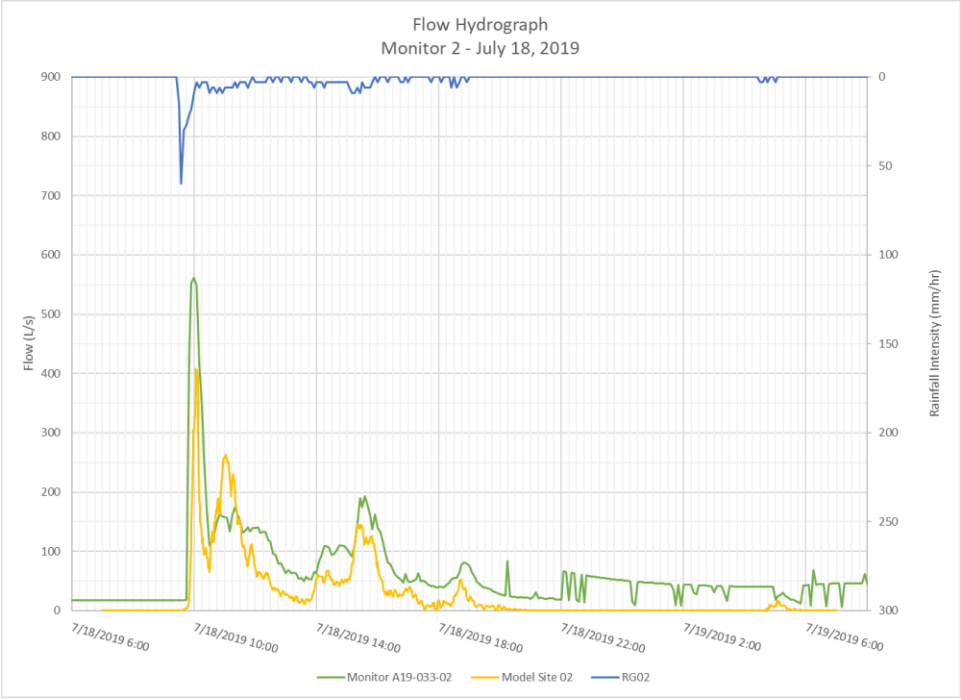
Validation



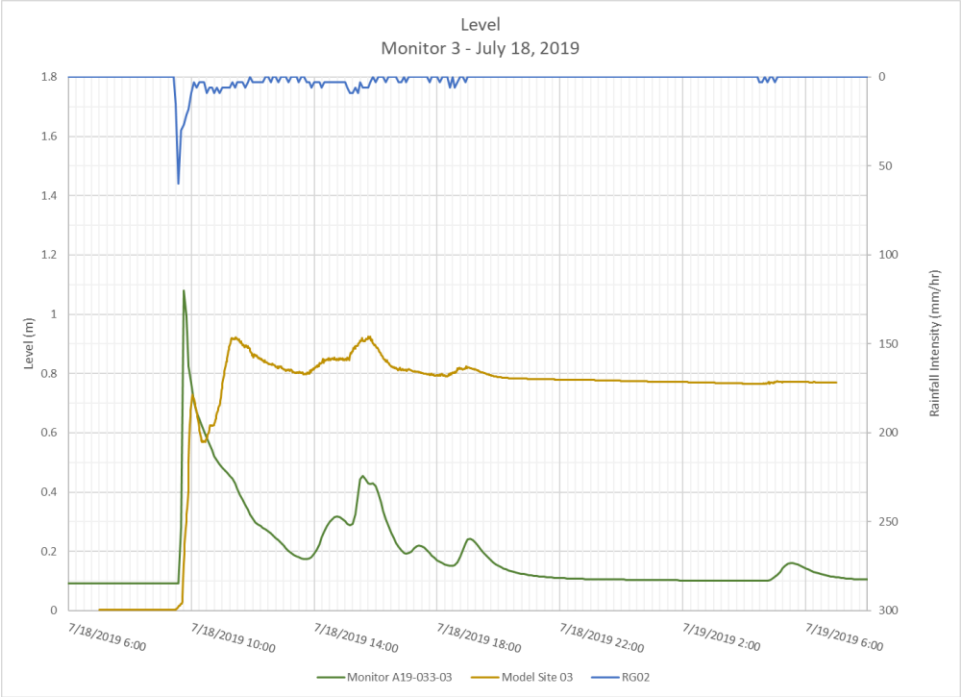
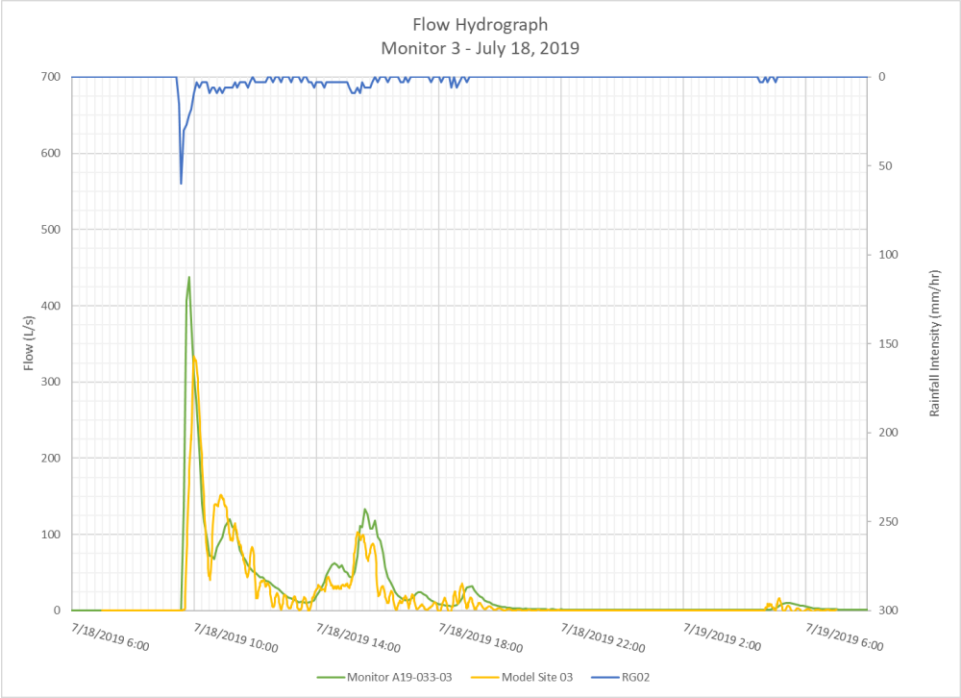
Validation



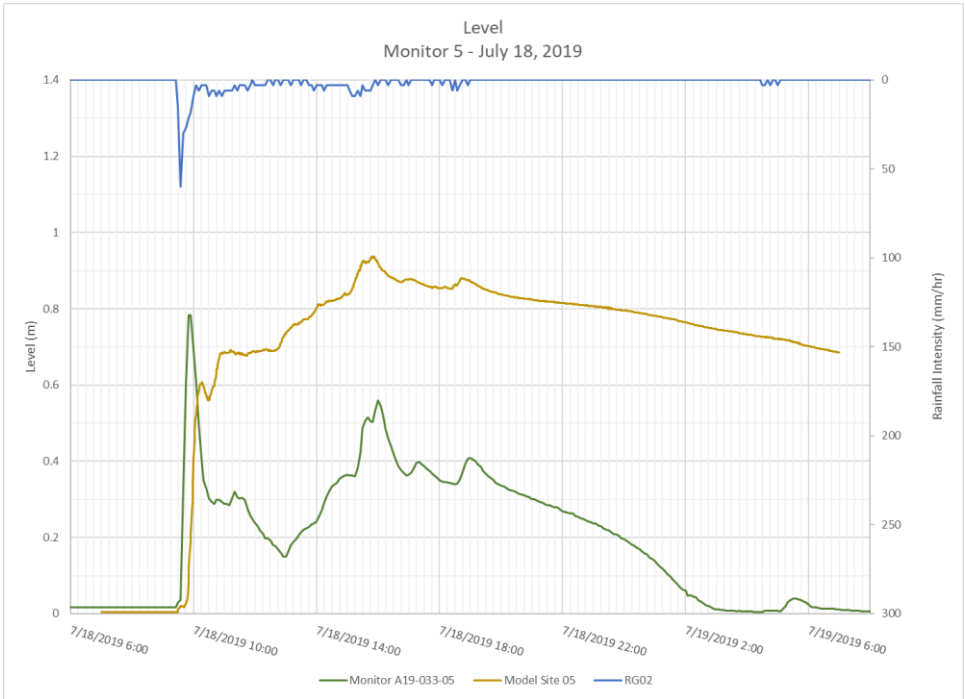
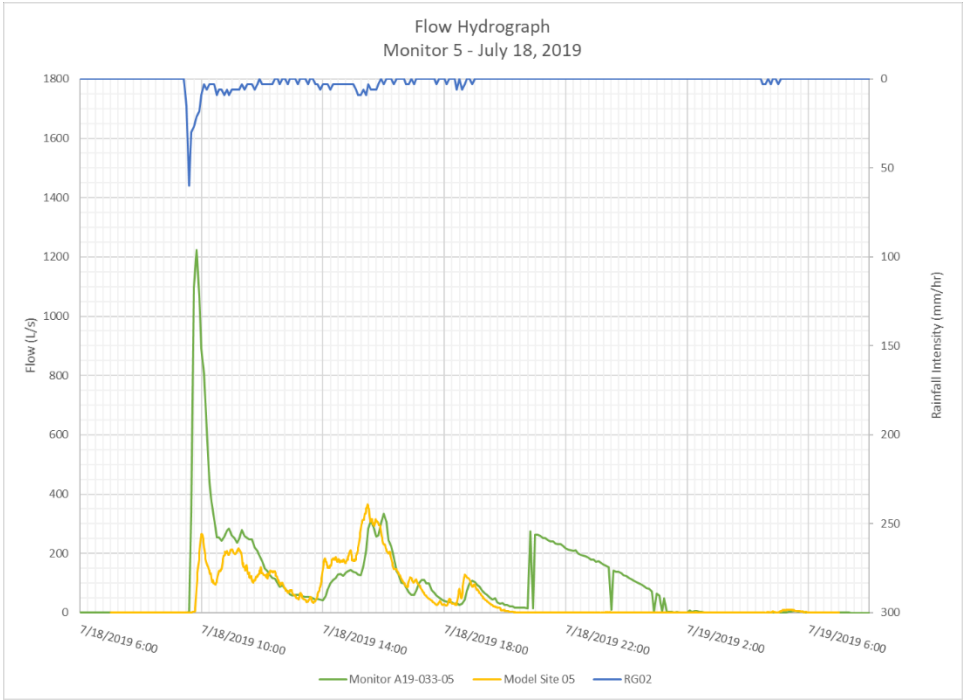
Validation



Validation



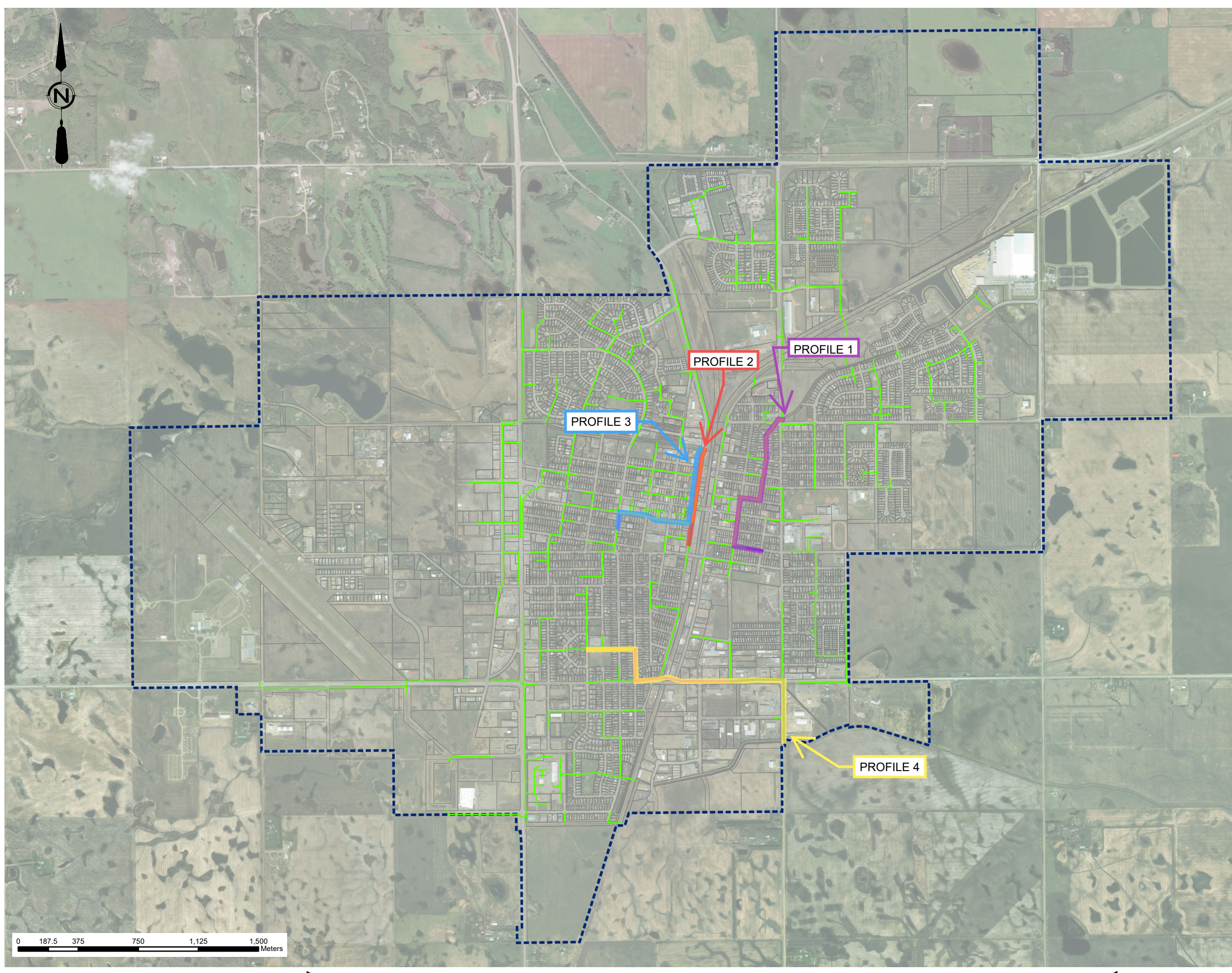
Validation



APPENDIX F - UPGRADES TO EXISTING HGLS

PLOT DATE: 3/27/2020 2:42:35 PM
SAVE DATE: 3/10/2020 2:42:35 PM
DWG PATH: \\s-01\projects\2019\3447\00_UWP_Update\Engineering\03_00_Conceptual_Feasibility_Design_Master_Plans\010_GIS\map_fig-1_ProposedUpgradePlan.mxd

IF NOT 25 mm ASJUST SCALES
SCALE(S) SHOWN ARE INTENDED FOR TABLOID (11X17) SIZE DRAWINGS UNLESS NOTED OTHERWISE



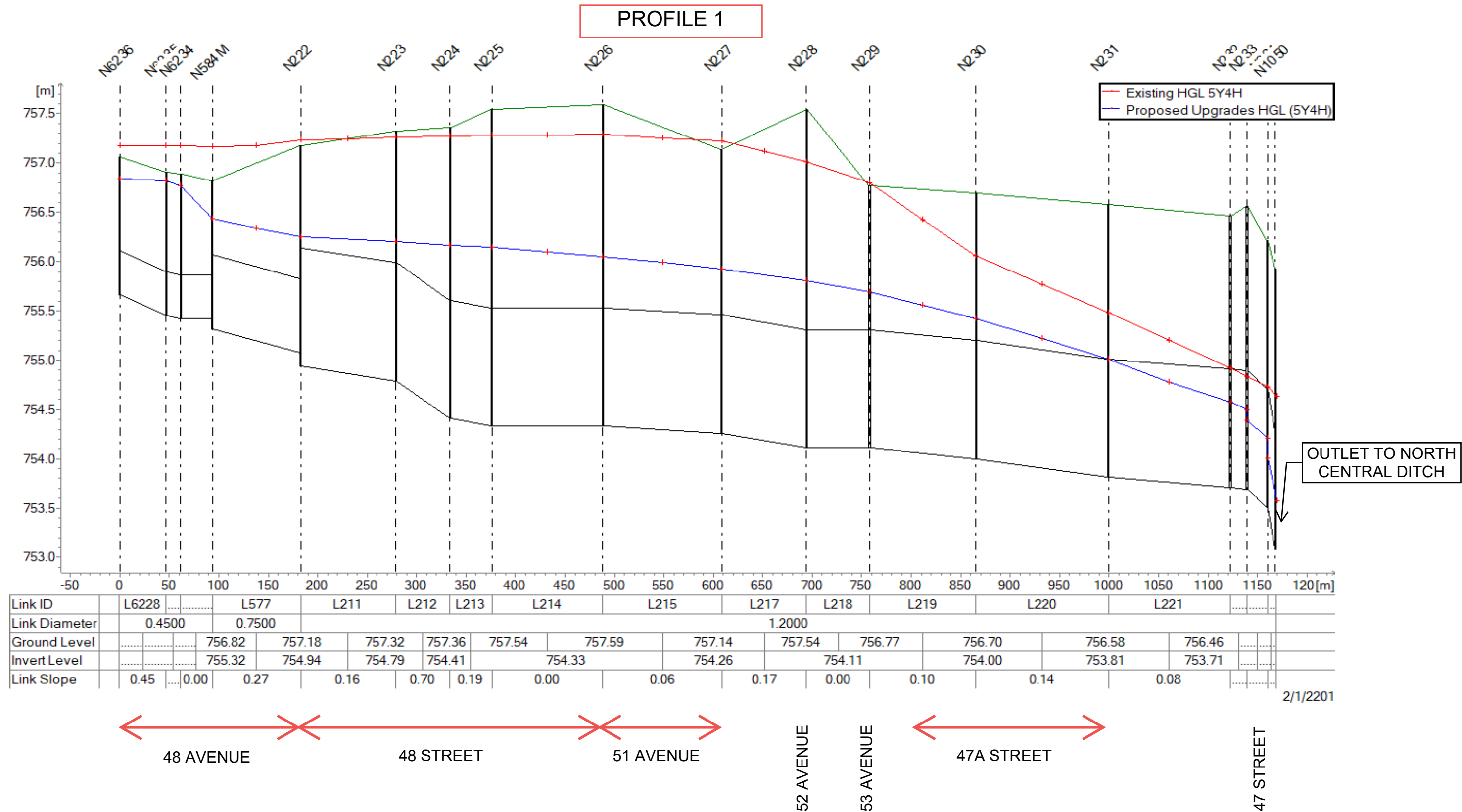
- Legend**
- Proposed Pipe Upgrade**
- Size
- 0.45
 - 0.75
 - 0.90
 - 1.05
 - 1.20
 - 1.35
 - 1.50
- Existing Storm Pipe
- City of Wetaskiwin Boundary



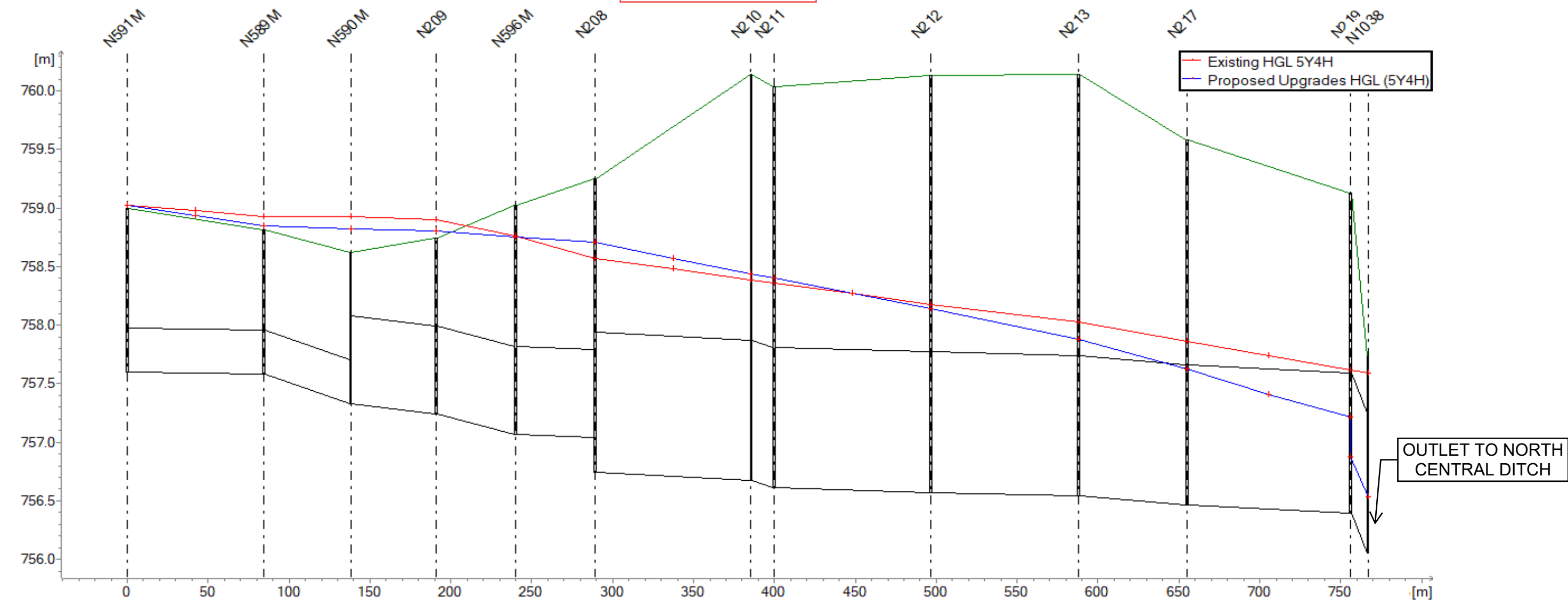
FIGURE F-1
CITY OF WETASKIWIN
STORM MASTER PLAN UPDATE

PROPOSED UPGRADE PLAN

AE PROJECT No.	2019-3447
SCALE	1:23,000
APPROVED	
DATE	2020MAR27
REV	
DESCRIPTION	ISSUED FOR DRAFT



PROFILE 2



Link ID	1376.1	1373.1	1374.1	1385.1	1386.1	610.1		L202	L203	L207	L208	
Link Diameter	0.3750		0.7500		1.2000							
Ground Level	759.00	758.81	758.62	758.74	759.02	759.25	760.14	760.04	760.13	760.14	759.58	759.13
Invert Level	757.60	757.58	757.33	757.24	757.07	756.74	756.67	756.61	756.57	756.54	756.46	756.39
Link Slope	0.02	0.47	0.17	0.35	0.06	0.07		0.04	0.03	0.12	0.07	

2/1/2201



50 STREET

49 AVENUE

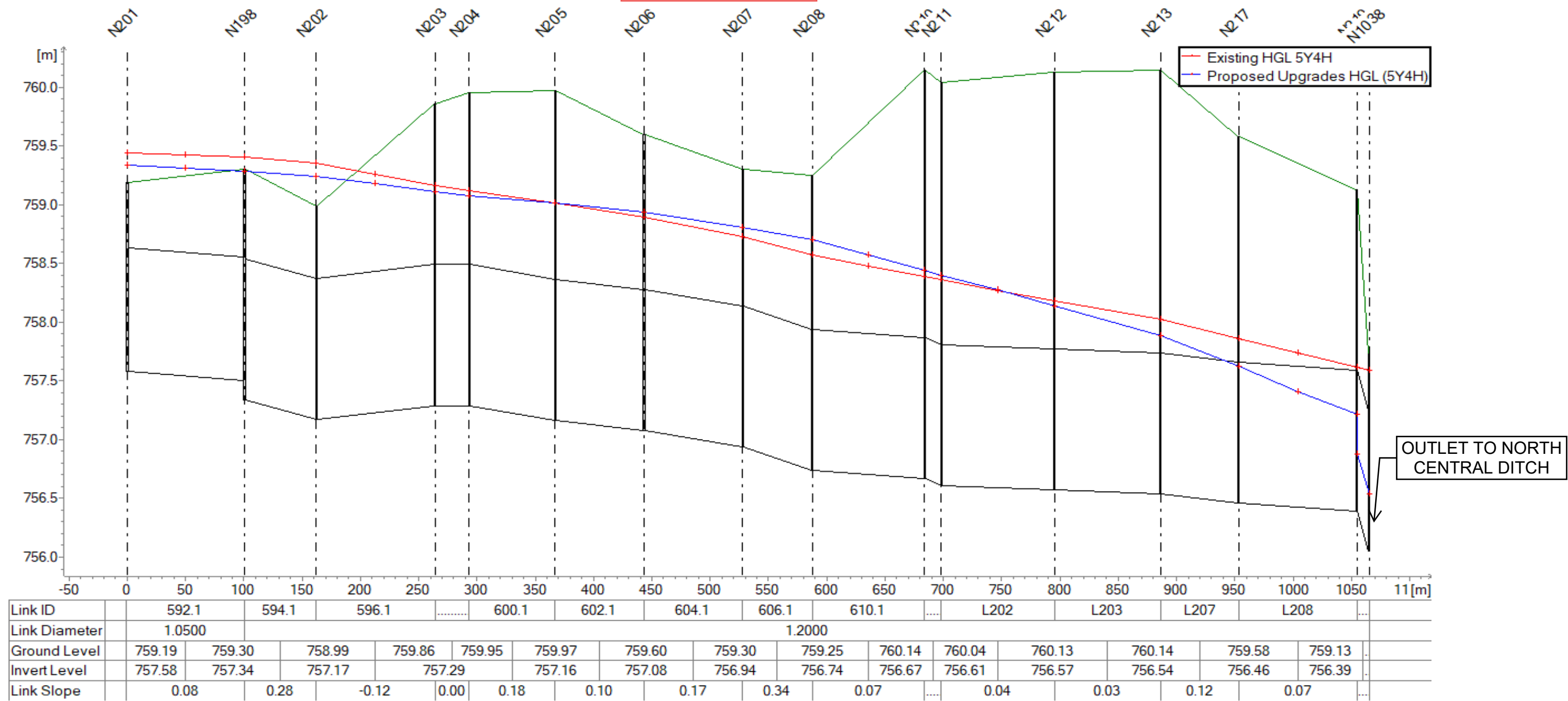
50 AVENUE

51 AVENUE

52 AVENUE

53 AVENUE

PROFILE 3



2/1/2201



49 STREET

50 STREET

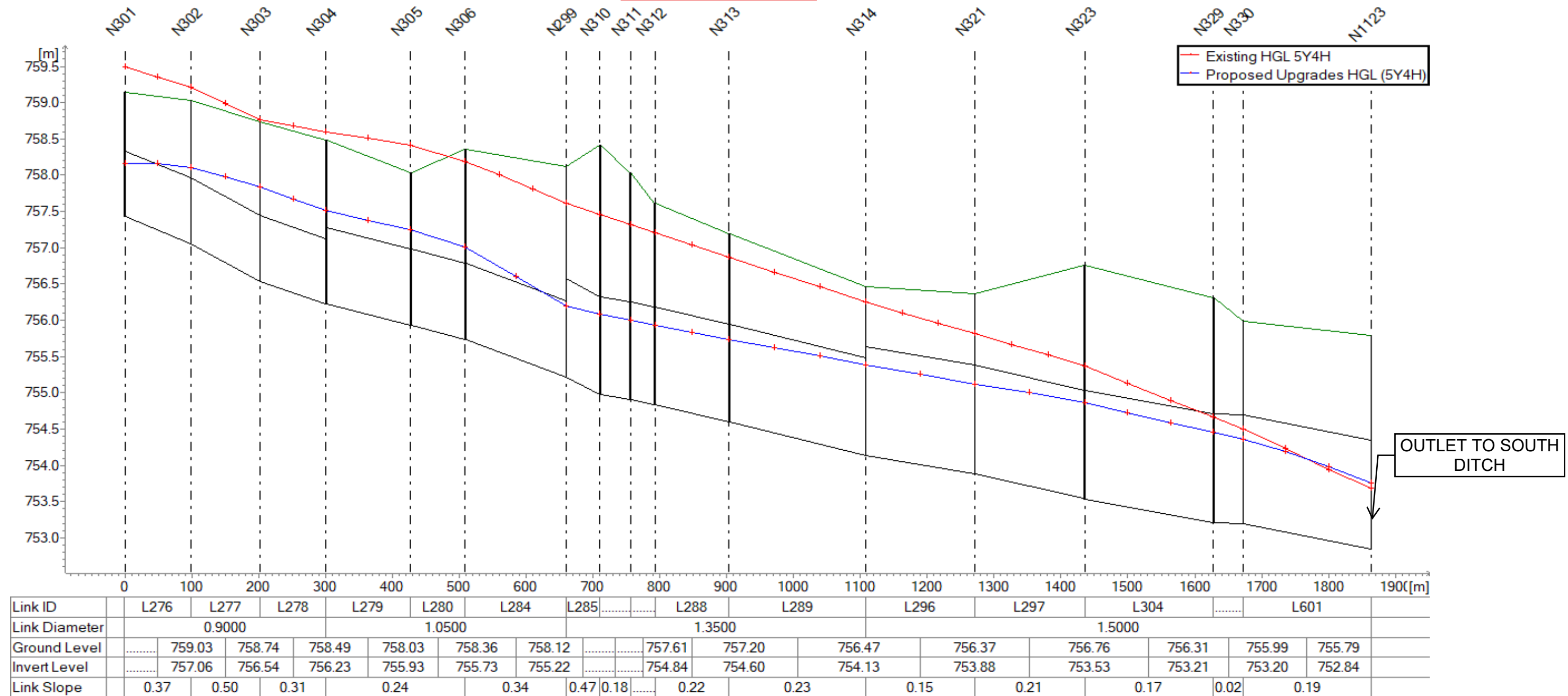
50 AVENUE

51 AVENUE

52 AVENUE

53 AVENUE

PROFILE 4



2/1/2201



APPENDIX G - UNIT COST BREAKDOWN

\\s-edm-fs-01\projects\2019 3447\00_UMP_Update\Engineering\03.00_Conceptual_Feasibility_Design_Master_Plans\Storm\Report\rpt_20193447_Storm\WtrMstrPlan_final_draft.docx



Flap Gate Check Valve Cost Estimate

Pipe Size	Material Cost	Install Multipiler	Total Supply and Install Cost
250	\$ 2,000.00	3	\$ 6,000.00
300	\$ 2,200.00	3	\$ 6,600.00
375	\$ 2,600.00	3	\$ 7,800.00
450	\$ 2,900.00	3	\$ 8,700.00
600	\$ 5,100.00	2.5	\$ 12,750.00
675	\$ 7,000.00	2.5	\$ 17,500.00
750	\$ 7,700.00	2.5	\$ 19,250.00
900	\$ 8,300.00	2.5	\$ 20,750.00
1050	\$ 9,000.00	2.5	\$ 22,500.00
1200	\$ 9,900.00	2.5	\$ 24,750.00
1350	\$ 15,000.00	2	\$ 30,000.00
1500	\$ 22,000.00	2	\$ 44,000.00
1800	\$ 28,000.00	2	\$ 56,000.00
2000	\$ 32,000.00	2	\$ 64,000.00
2100	\$ 33,600.00	2	\$ 67,200.00

Storm Sewer (Concrete) Replacement Cost Breakdown

Pipe size	Remove and Replace Asphalt \$/m	Remove Ex. Pipe	Excavation \$/m	Backfill \$/m	Bedding \$/m	Misc	Pipe \$/m	\$/m
450	\$ 198.00	\$ 50.00	\$ 455.75	\$ 420.00	\$ 60.00	\$ 200.00	\$ 120.00	\$ 1,500.00
750	\$ 198.00	\$ 50.00	\$ 488.75	\$ 420.00	\$ 120.00	\$ 200.00	\$ 350.00	\$ 1,900.00
900	\$ 297.00	\$ 50.00	\$ 670.25	\$ 630.00	\$ 150.00	\$ 600.00	\$ 500.00	\$ 2,900.00
1050	\$ 346.50	\$ 50.00	\$ 769.25	\$ 735.00	\$ 150.00	\$ 600.00	\$ 770.00	\$ 3,500.00
1200	\$ 396.00	\$ 50.00	\$ 868.25	\$ 840.00	\$ 150.00	\$ 600.00	\$ 1,060.00	\$ 4,000.00
1350	\$ 445.50	\$ 50.00	\$ 967.25	\$ 945.00	\$ 200.00	\$ 600.00	\$ 1,230.00	\$ 4,500.00
1500	\$ 495.00	\$ 50.00	\$ 1,066.25	\$ 1,050.00	\$ 200.00	\$ 600.00	\$ 1,540.00	\$ 5,000.00

Assumptions:

1. Curb and gutter removal/replacement not included
2. Remove existing pipe assumed at \$50/m
3. Misc includes manhole - \$20,000/100m for 1200mm dia. Manholes, and \$60,000/100m of pipe for 1800mm dia. Manholes