

**City of Guelph**

**Water and Wastewater Servicing Master Plan**

**Volume II – Model Update, Field Testing, and  
Calibration Technical Memorandum**

February 2023





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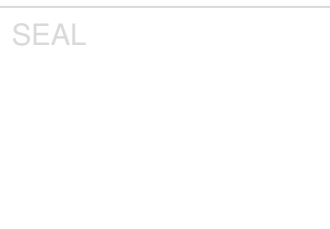
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DATE: February 9, 2023



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## 1.0 INTRODUCTION

The City of Guelph’s (City) water and wastewater models were updated in support of the Guelph Water and Wastewater Servicing Master Plan (WWSMP) and included updated model demands and system infrastructure to represent existing conditions. This technical memorandum (TM) summarizes the water and wastewater model update process and calibration results.

## 2.0 WATER MODEL UPDATE

Hydraulic water models consist of a network of pipes, junctions, pumps, valves, tanks, and reservoirs to represent a unique set-up of each drinking water distribution system. The model allows different scenarios to be created to represent operational control changes, demand fluctuations throughout the year, as well as population and water usage growth. This section details how each aspect of the City’s water model was updated.

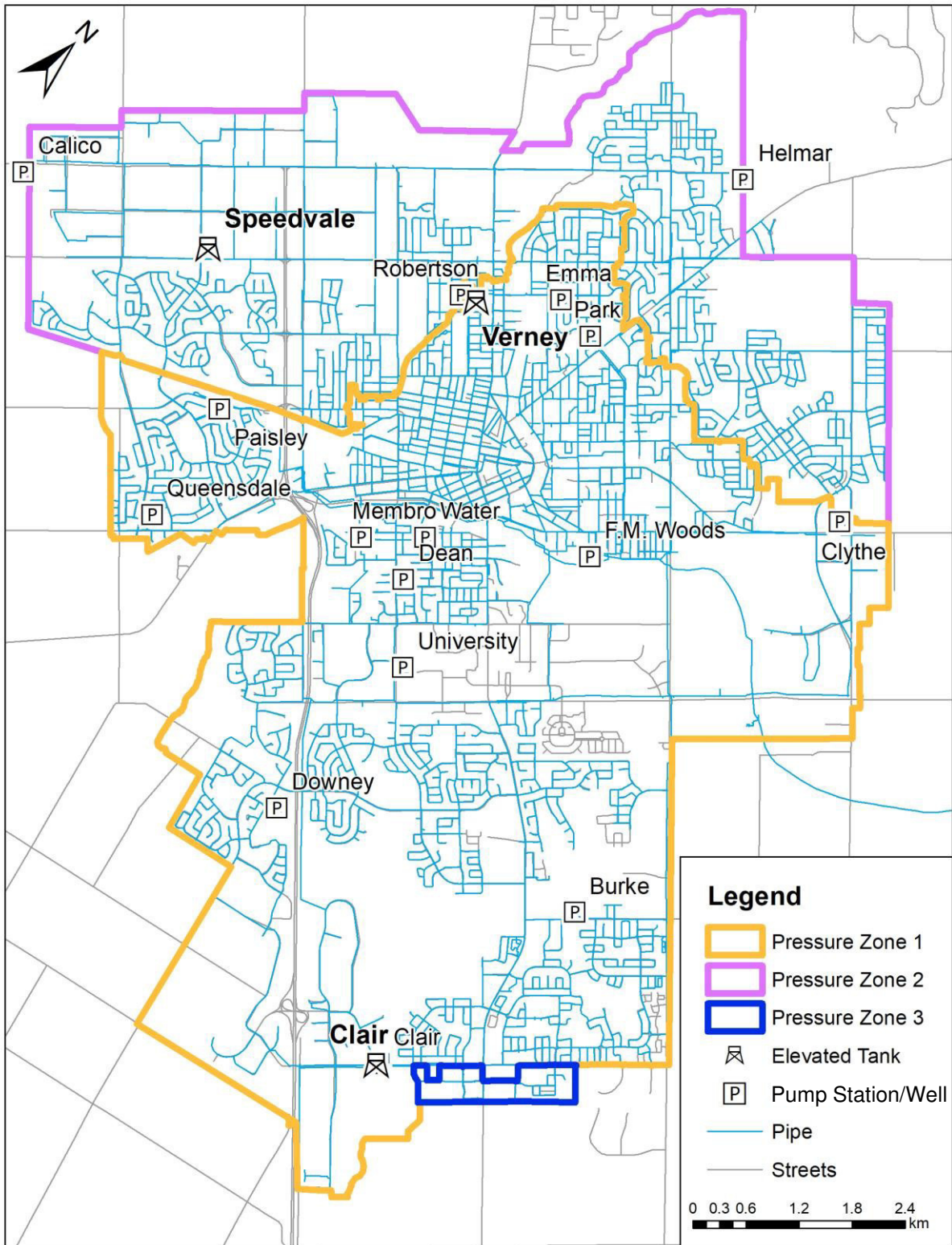
### 2.1 Background

#### 2.1.1 System Overview

The City’s water distribution system is split into three (3) pressure zones and is supplied by groundwater wells. Up to 80% of the City’s water supply can be provided by the F.M. Woods Water Treatment Plant (Woods WTP) which is supplied by the Arkell Wells, the Glen Collector, and the Carter Wells via the Arkell Aqueduct. Zone 1 is supplied by the Woods WTP, along with a number of other groundwater wells. Water is supplied from Zone 1 to Zone 2 via the Paisley, Robertson and Clythe pump stations (PS). Water is supplied from Zone 1 to Zone 3 via the Clair PS. The system has three (3) elevated tanks (ETs); Verney and Clair in Zone 1 and Speedvale in Zone 2. In-ground storage reservoirs are located at the Woods and University PSs in Zone 1 and the Paisley and Clythe PSs in Zone 2. A summary of the City’s Water system is presented in Table 2-1 and Figure 2-1 below.

**Table 2-1 Water System Summary**

Zone	Supply Wells	Pump Stations	Storage
1	Arkell Wells (Woods) Glen Collector (Woods) Carter Wells (Woods) Emma Park Water St. Dean Membro Queensdale Downey Burkes University	Woods University	Woods Reservoir University Reservoir Verney ET Clair ET
2	Paisley Calico Helmar	Paisley Clythe Robertson	Paisley Reservoir Clythe Reservoir Speedvale ET
3		Clair	



**Figure 2-1 System Overview**

### 2.1.2 Water Model History

The City currently uses InfoWater software for hydraulic modelling. The model is utilized for development applications, operational support, project planning and master planning as required.

The Team's understanding of the general history of the model and its development is summarized below:

- The City's first water model was developed in 2001 by KMK Consultants utilizing the existing GIS watermain records, pumping station records, billing data and SCADA Data. The original model was built in WaterCAD and included steady-state (SS) simulation capabilities only.
- In 2008 the model was transferred to InfoWater by Earth Tech Canada and updated to include extended period simulation (EPS). It was utilized in the development of the 2009 Water and Wastewater Servicing Master Plan (WWSMP). The model was also calibrated using available SCADA information.
- The City retained Stantec Consulting in 2010 to complete a model update and calibration that included a significant field-testing program including approximately 40 C-factor tests, 35 fire flow tests and 14 pressure data loggers. The model update also included the addition of raw water facilities, since the previous models had only included the treated water component of the water system, and the addition of complex controls mirroring the SCADA system.
- In 2012 the City was awarded a Showcasing Water Innovation (SWI) grant that included an energy optimization component. The model was updated by C3 Water with pump efficiency curves, energy rate curves and calibrated to match energy consumption in the field with the installation of power monitors on each water facility.
- In 2014 AECOM completed a Water/Wastewater Development Charges Update. As part of this project the model was updated with future infrastructure requirements to meet the projected demands.
- In 2014-15 C3 Water was retained to complete consecutive Zone 2 and Zone 1 Infrastructure Studies to prioritize capital projects recommended as part of the WWSMP and Development Charges update. As part of this project, the water model was updated to merge the planning model that AECOM had utilized with the Operational Model that C3 Water had updated as part of the SWI project. The model merge successfully brought together the operational model with the latest planned infrastructure and most recent demand projections.
- In 2016 C3 Water was retained to complete the Clair Booster Pumping Station & Zone 3 Commissioning Plan which included an update of the future and existing demands in Zone 3. Additionally, the layout of the Clair pump station was updated, and planned Zone 3 linear projects were added to the model.
- In 2016 – 2017 the model distribution network was updated by C3 Water based on the City's GIS records. Additionally, C-factor testing was completed on small diameter Cast Iron watermains to improve model accuracy in older areas of the City.
- In 2017 C3 Water and Cole Engineering were retained by the City to undertake the Downtown Servicing Study to determine the water servicing needs in support of existing and future development as the City works to implement the Downtown Secondary Plan and the associated draft Downtown Zoning By-law. Linear updates were applied to the City's model to confirm layout, diameter and material of water services in the downtown area to include recent construction.
- In 2018 C3 Water was retained to develop the City's InfoSurge model for transient analysis. This was developed from the existing InfoWater model and included input of parameters such as pipe wave speeds, pump inertia information and surge protection devices.
- 2016 – 2019: Periodic updates have been made to the model by C3 Water through the ongoing Model Support contract with the Water Services such as updates to controls and pump station layouts.

## 2.2 Water Model Update

The 2020 update of the model as part of the WWSMP was completed in InfoWater Pro software and included a full refresh of existing conditions demands and elements using 2020 GIS records, 2019/2020 production data, 2019 billing meter records and SCADA. System demands are always changing due to population growth, increased water efficiency, industrial process changes and many other factors. It is beneficial to update model demands periodically to improve accuracy when simulating “existing conditions”.

The updated model includes all valves and all hydrants, based on the City’s GIS records. In 2020, the City’s GIS department undertook an initiative to develop a more comprehensive GIS system which included splitting watermains at valves, hydrant laterals and tees. The updated GIS system has become a much closer representation of the geometric system that is required by the modeling software. By re-building the model based on GIS records, future updates to the system reflected in GIS can more easily be integrated in the model, helping to maintain an up-to-date network. Additionally, by basing the model network on GIS, all valves and all hydrants can be included. Including hydrants in the model improves the accuracy of fire flow analyses. Network valves can be used to simulate operational changes more precisely, such as closing a singular valve, rather than an entire pipe. Valves can also be used to improve calibration to reflect flow restrictions in the network.

### 2.2.1 Scenarios

The updated extended period simulation (EPS) existing conditions (2019) scenarios were built using 2019 billing meter records and production data.

Demand scenarios were included as follows:

- ADD, average day demand.
- MDD, maximum day demand, includes peak hour demand (PHD).

## 2.3 Infrastructure Update

The model’s distribution system pipes, valves, hydrants, and junctions were updated using the City’s latest GIS records. Facilities such as pump stations, wells and storage were imported from the existing model. Facilities have been periodically updated in the existing model through ongoing model support projects.

### 2.3.1 Distribution Network

The model distribution network was re-built using GIS data for pipes, valves, and hydrants. Now that the model matches closely with GIS (including IDs), the alignment with asset management, capital planning and other internal projects and departments is improved. The following steps were completed for developing the system network:

1. GIS data, provided on May 29, 2020 was imported into the Matrado Model Create Tool;
  - a. wMain
  - b. wValve
  - c. wLaterallLine
  - d. wHydrant
2. The Matrado tool was used to connect all pipes into a geometric network with junctions at the end of each pipe and split pipes at valves and hydrant laterals. Due to the ongoing GIS upgrade project at the City, most pipes were already split at valves and laterals. The tool then converted the network into an EPANET file that was imported into InfoWater.

3. The ID fields from GIS were used as InfoWater element IDs. The model element naming is summarized in Table 2-2 below. The junctions were given default IDs that can be modified if desired by the City.

**Table 2-2 Element Naming**

Element	Element Type	IW Element Name
Pipes	Pipe	P_'WMAINID' + proceeding -1, -2, etc. where pipe splits required
Valves	Valve	V-'WVALVEID'
Hydrants	Junction	H_'HYDRANTID'
Hydrant Laterals	Pipe	HL_'WLATERALID'

4. Pipe, node and valve data including material and year of installation were imported from GIS into model elements using tabular join and GIS Gateway tools based on the corresponding GIS IDs.
5. Elevations were assigned to model nodes (not including facilities) using available Lidar data and the InfoWater Elevation Extractor tool.
6. Connectivity checks were performed including searches for orphan nodes, orphan pipes, nodes in close proximity, parallel pipes, duplicate pipes, pipe split candidates, and crossing/intersecting pipes.
7. Pumps stations, well pump houses and storage facilities were imported from the existing model and connected to the distribution watermains from GIS.

### 2.3.2 Pipes

Pipe information including material and year of installation was transferred from GIS data. Pipes within the distribution system were labelled as “Distribution”. All pipes within pump stations and elevated tank inlet/outlets were not included in GIS and were transferred from the previous model and labelled as “Facility”. This categorization can be used for creating domains and filtering model results.

C-factors, or roughness values, were applied to all model pipes. C-factors are unitless numbers utilized by the Hazen-Williams hydraulic equation to calculate friction losses within the pipes. C-factors vary based on diameter, material and age of pipe. They can be referenced from literature and tested in the field. C-factors were imported from the existing model which were developed during calibration by Stantec in 2010 and additional field testing completed in 2017. Pipes that have been installed or replaced since the latest model calibration in 2017 were assigned C-factors based on literature values. Hydrant laterals were assigned a default C-factor of 120. Pipe materials in the model are summarized in Table 2-3 below.

Due to the significant C-factor testing that was previously completed on critical pipe sizes and material types, additional C-factor testing was not completed for the 2020 model update. Based on field testing data collected in 2020, the model was found to be suitably calibrated for master planning purposes. Model calibration is discussed further in Section 3.0.

**Table 2-3 Summary of Pipe Materials**

Material	Total Length (m)	% of Total Pipes by Length
Cast Iron	185,230	33%
Concrete	20,094	4%
Copper	2,690	0%
Cured In Place	2,851	1%
Ductile Iron	87,607	16%
Ductile Iron (Cement Lined)	61,419	11%
Polyethylene (High Density)	1,870	0%
Polyvinyl Chloride	202,766	36%

Roughness values in the model ranged from 46 (small diameter Cast Iron) to 140 (PVC) based on the existing model. In total, there are 20,105 existing pipes in the model, ranging from 25mm to 1050mm in diameter, as shown in Table 2-4 and Figure 2-2. Hydrant laterals account for 3,897 of the pipes.

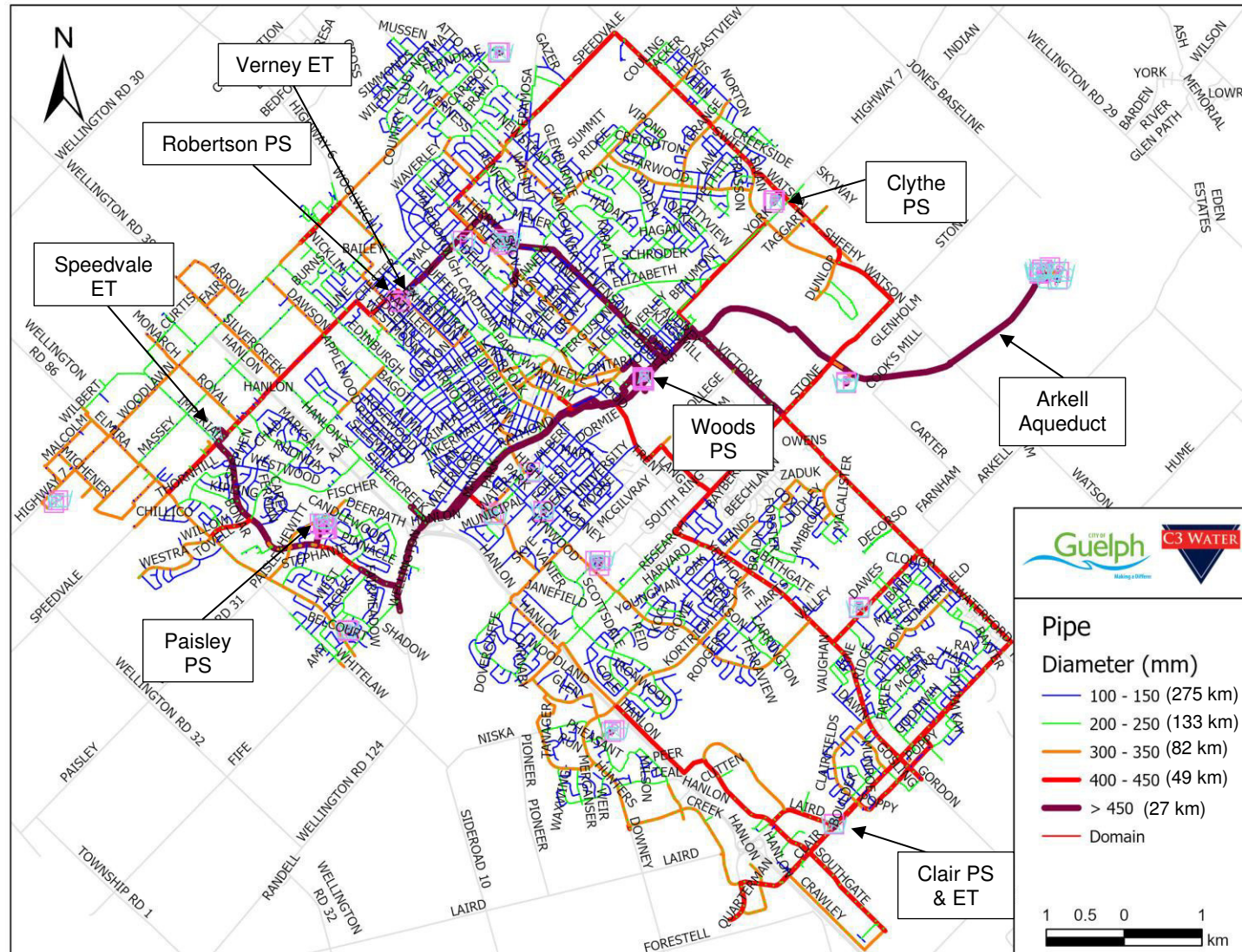
**Table 2-4 Summary of Pipe Diameters**

Diameter	Total Length (m)	% of Total Pipes by Length
100-150	274,696	49%
200-250	132,979	24%
300-350	81,882	14%
400-450	48,783	9%
>450	26,812	5%

### 2.3.3 Junctions

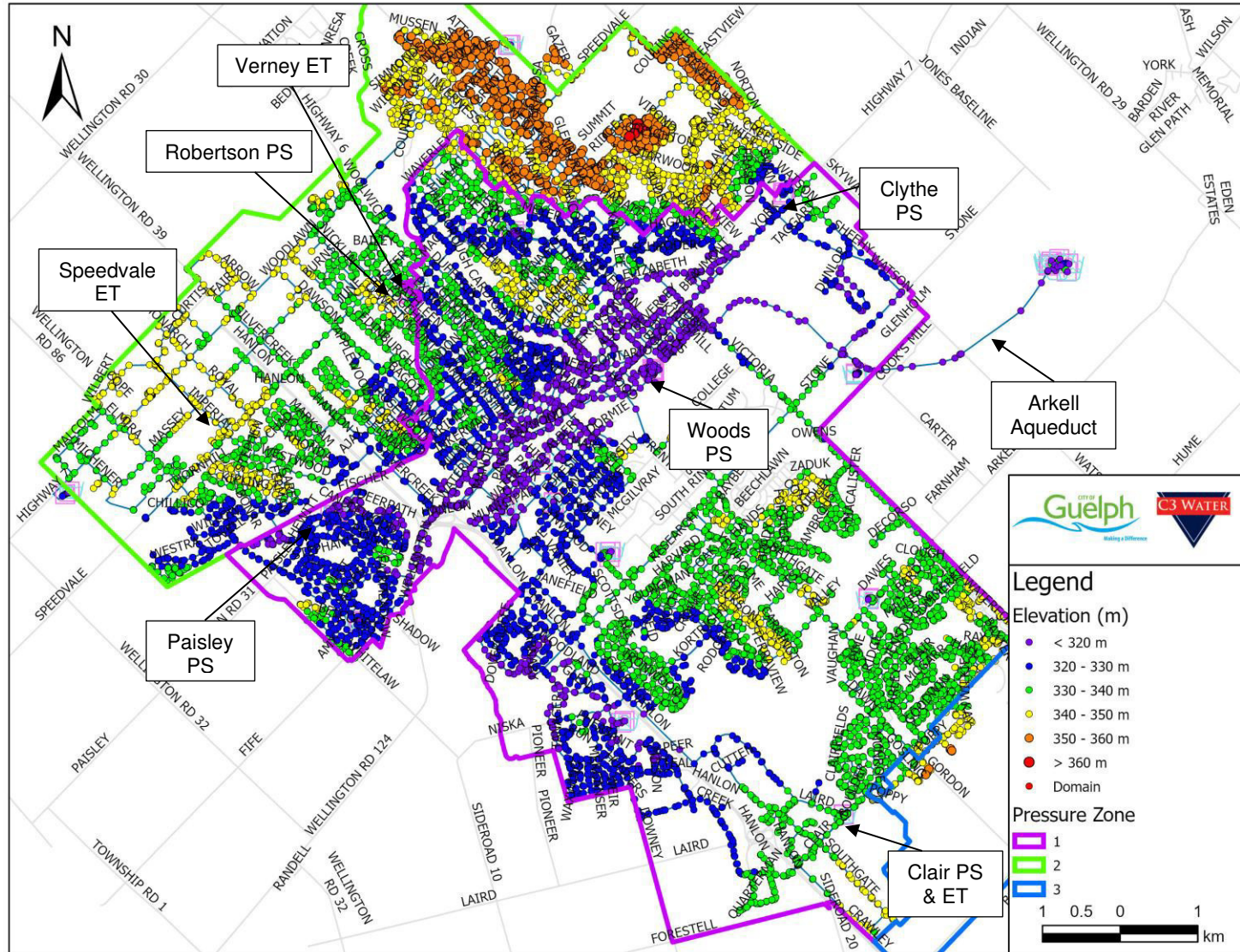
The Matrado Ltd. Model Create tool was used to integrate the City’s watermain, hydrant and valve layers and make it suitable for modeling purposes. Junction information such as year of installation was assigned based on connected pipe information. Junctions within pump stations or storage facilities were labelled as “Facility” for reporting purposes. Junctions were also used to represent hydrants from GIS. Hydrant node installation years were based on the hydrant shapefile.

Elevations were added to the model junctions using available Lidar data within the distribution system. Elevations within pump stations and other facilities were taken from the existing models which were based on finished floor elevations from as-built drawings. In total, there are 14,696 junctions in the model ranging from 276.5 to 362.4 mASL in elevation as shown in Figure 2-3.



**Figure 2-2 Pipe Diameters**





**Figure 2-3 Junction Elevations**

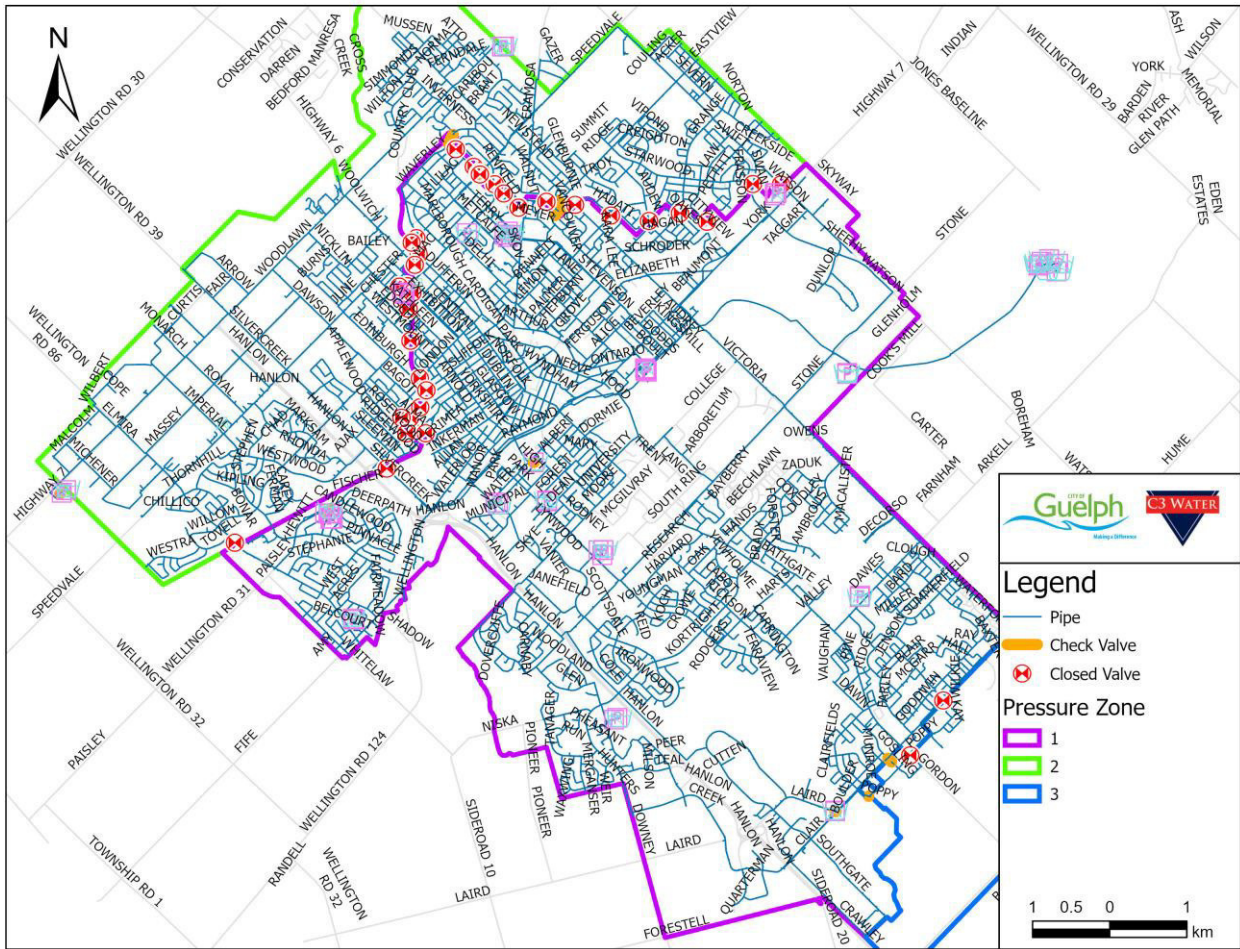
### 2.3.4 Valves

The model contains a total of 4,298 valves, 4,243 of which were based on GIS records. Currently, all valves from GIS are modelled as throttle control valves but are typical line valves. Valves that were noted as currently closed zone boundary valves in GIS were closed in the model. All other GIS valves were modelled as fully open. Valve settings may be adjusted in the future to calibrate the model based on field test results.

As this model includes all valves, the Pressure Zone boundaries were delineated by closing the appropriate valves. In the updated model, 41 valves are closed. Additionally, there are several check valves along the pressure zone boundaries which allow flow from Zone 1 into Zones 2 and 3 under emergency conditions. In the model, check valves are modelled as pipes. Network closed valves and check valves are shown in Figure 2-4 below. Check valve locations are as follows:

1. Zone 2:
  - a. Speedvale and Knightswood
  - b. Ottawa and Callander
  - c. Waverly and Windsor
  - d. Vancouver and Ottawa
2. Zone 3:
  - a. Clair and Gosling
  - b. Poppy, south of Clair
  - c. Clair BPS

Additionally, the model contains flow control valves (FCVs), Pressure Reducing Valves (PRVs) and Pressure Sustaining Valves (PSVs) to set the discharge flow conditions at wells and some PSs to match operational conditions. PS control valves and the Dodds Street valves were imported from the existing model. The Dodds valves were installed to direct more flow from Woods to the Clair ET rather than the Verney ET. At this time, it is understood that the Dodds valves are not operational and are fully open.



**Figure 2-4 Closed Valves and Check Valves**

**2.3.5 Facilities**

Model facilities were imported from the existing InfoWater model. Facility model elements are summarized in Table 2-5 below. Facility information such as pump curves, tank curves, flow control valves, etc. were imported from the existing model. Planned future upgrades to pump stations such as F.M. Woods, Paisley and Clythe will be reflected in future scenario facility sets. Facility sets define which model elements are active in each model scenario.

**Table 2-5 Facility Elements**

Model Element	Purpose
Pump	Well pumps and high-lift pumps
Reservoir	Wells
Tanks	Water towers and reservoirs
Valves	Controls valves to set pump discharge
Pipes	Facility piping

## 2.4 Model Controls

InfoWater utilizes control sets to store information about how the elements are operated throughout an extended period simulation. The initial status of pipes, pumps and valves can be set to control how each element is operating at time 0:00 (midnight). Using the controls, the status and setting of pumps and valves can then be altered throughout the simulation's time steps based on tank levels, pressure values, or clock time similar to how a SCADA system controls the water system operation.

All 2019 (existing) scenarios in the model used the same control set. This is ideal so that changes to the controls are reflected across all scenarios, and the model will have the flexibility to adjust to different demand conditions with the same logic. If operational controls are modified throughout the year, individual control sets can be setup for each scenario.

Pump controls are very important to the operation of the model since they determine how and when water is supplied to the system throughout the simulation. The controls in the model were imported from the existing model and updated to reflect 2020 SCADA data. A general summary of the model pump control logic is summarized in Table 2-6 for pump stations (PS) and high-lift pumps (HLPs) and Table 2-7 for system wells below.

**Table 2-6 Pump Station Controls**

Pressure Zone	Location	Controls Based On
Zone 1	Woods HLPs	Verney ET Level
	Park HLPs	Verney ET Level
	Dean HLP	On
	Membro HLP	Off
	Queensdale HLP	Off
	University HLPs	On
	Downey HLPs	Clair ET Level
	Burkes HLP	Clair ET Level
Zone 2	Paisley Zone 1 Inlet	Paisley Reservoir Level
	Paisley PS	Speedvale ET Level
	Robertson PS	Speedvale ET Level
	Clythe Zone 1 Inlet	Clythe Reservoir Level
	Clythe PS	Clythe Discharge Pressure and Speedvale ET Level
	Helmar HLP	On
	Calico HLP	On
Zone 3	Clair PS	Clair Discharge Pressure

**Table 2-7 Well Pump Controls**

Pressure Zone	Location	Controls Based On
Zone 1	Arkell Wells, Glen Collector & Carter Wells	Woods Reservoir Level
	Emma Well	Verney ET Level
	Park Wells	Park Reservoir Level
	Water St. Well	Verney ET Level
	Dean Well	Dean Reservoir Level
	Membro Well	Membro Reservoir Level
	Queensdale Well	Queensdale Reservoir Level
	University Well	University Reservoir Level
	Downey Well	Downey Reservoir Level
	Burkes Well	Burkes Reservoir
Zone 2	Paisley Well	Paisley Reservoir Level
	Helmar Well	Helmar Reservoir Level
	Calico Well	Calico Reservoir Level

## 2.5 Model Demands

Junctions in the model have assigned sets of demands that correspond to water usage in the system in units of L/s. Demands were spatially allocated based on geocoded 2019 billing records.

A summary of the 2019 model demand data is presented in Table 2-8 below. By comparing the 2019 revenue water (billed metered consumption and billed unmetered consumption) to the average daily production volume, the system has approximately 17% average non-revenue water (NRW) in 2019 or approximately 92 L/s. The MDD was developed using a total demand peaking factor of 1.34. This factor was established as part of the Water Supply Master Plan update based on the highest max day factor between 2010 and 2020. The max day factor is the ratio of water production on the highest single production day each year and the average annual day demand for the same year. The max day factor of 1.34 occurred in 2011.

**Table 2-8 Model Demand Summary (2019) (L/s)**

Scenario	Total Demand	Billed Consumed	NRW
<b>ADD 2019</b>	544	452	92
<b>MDD 2019</b>	729	637	92

The process for developing existing demands in the model was completed as follows:

1. InfoWater’s Demand Allocator tool was used to apply the meter records to model nodes using the “nearest pipe” method.
  - a. Each billing record was matched with the nearest distribution pipe in the system, and the total water consumption was summed for each pipe.
  - b. The program then splits the consumption between the two nodes of the pipe using a distance weighted method.

- c. The result is that all each junction contains the total ADD water consumption of nearby users.
2. Meter records were applied to Demand Type 1 on model nodes as existing demands.
3. Hydrant nodes, facility nodes and nodes on large watermains without service connections were not included in the demand allocation.
4. 2019 Water Production data was compared to total recorded consumption. The difference between the produced and consumed water was applied evenly across each pressure zone to Demand Type 2 as NRW to develop the 2019 ADD scenario.
5. The ADD total demand was multiplied by a factor of 1.34 to develop the 2019 MDD based on the max day peaking factor established as part of the Water Supply Master Plan update (Water Supply Master Plan TM2). NRW (Demand Type 2) demand was maintained consistent across the demand scenarios as leakage is expected to be relatively constant across varying demand conditions. The MDD consumption demand (Demand Type 1) was calculated from the difference between the total demand and the NRW.

### 2.5.1 Demand Patterns

Each demand junction is assigned a pattern which applies an hourly multiplication factor throughout the day to create a diurnal curve. Different patterns can be applied to simulate trends observed for specific customer types, pressure zones, or other factors. A unique diurnal curve was developed for each pressure zone for each demand scenario.

The demand patterns were developed for each pressure zone by completing a flow balance for the zone using 5-minute interval SCADA data. By comparing input, output and storage at each time step throughout the day, and subtracting the NRW usage, it was possible to determine the amount of water consumed at hourly intervals for ADD and MDD. Demand patterns for the top-5 water users were monitored in the field and subtracted from the overall zone demand patterns. Top water users are discussed further below.

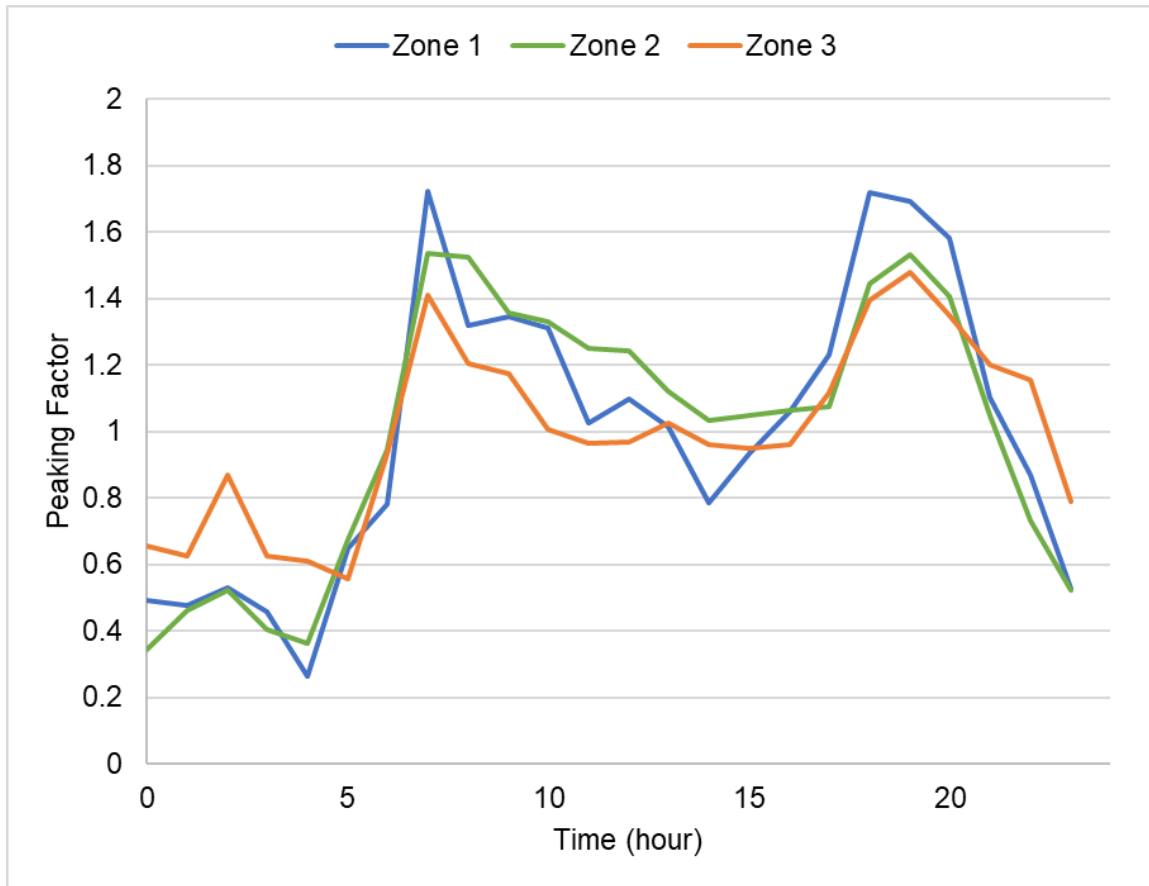
Demand patterns were applied to Demand Type 1 (Billed Consumption). At this time, a constant demand pattern was applied to Demand Type 2 (NRW) as system water loss is expected to be relatively constant throughout the day.

The dates used to develop demand patterns in the model are summarized in Table 2-9 below. Based on SCADA data and production data provided by the City, these dates were found to be representative of typical average and maximum demand days, respectively. The hourly water consumption peaking factor patterns for each pressure zone are presented in Figure 2-5 and Figure 2-6 below for ADD and MDD, respectively.

A 2020 date was used for the MDD pattern as this day was found to have higher water usage than any date in 2019. It should be noted that due to the provincial lockdowns in regards to Covid-19, water usage may have differed from historical years due to a number of factors including difference in business operation and hours and an increase in population working from home. It is not yet known how water usage patterns will continue to evolve in the future as a higher portion of the population may very likely continue to work from home compared to pre-2020 conditions. The July 8, 2020, zone balance was found to follow a trend which would be expected for a summer high water use day with a morning peak followed by a higher peak in the evening, likely as a result of lawn watering. Therefore, the July 8, 2020, diurnal pattern is considered to be representative of a typical MDD at this time.

**Table 2-9 Model Demand Patterns**

Pattern	SCADA Data Used
ADD_2019	Average of June 10 and Nov. 11, 2019
MDD_2019	July 8, 2020



**Figure 2-5 ADD 2019 Diurnal Patterns**



**Figure 2-6 MDD 2019 Diurnal Patterns**

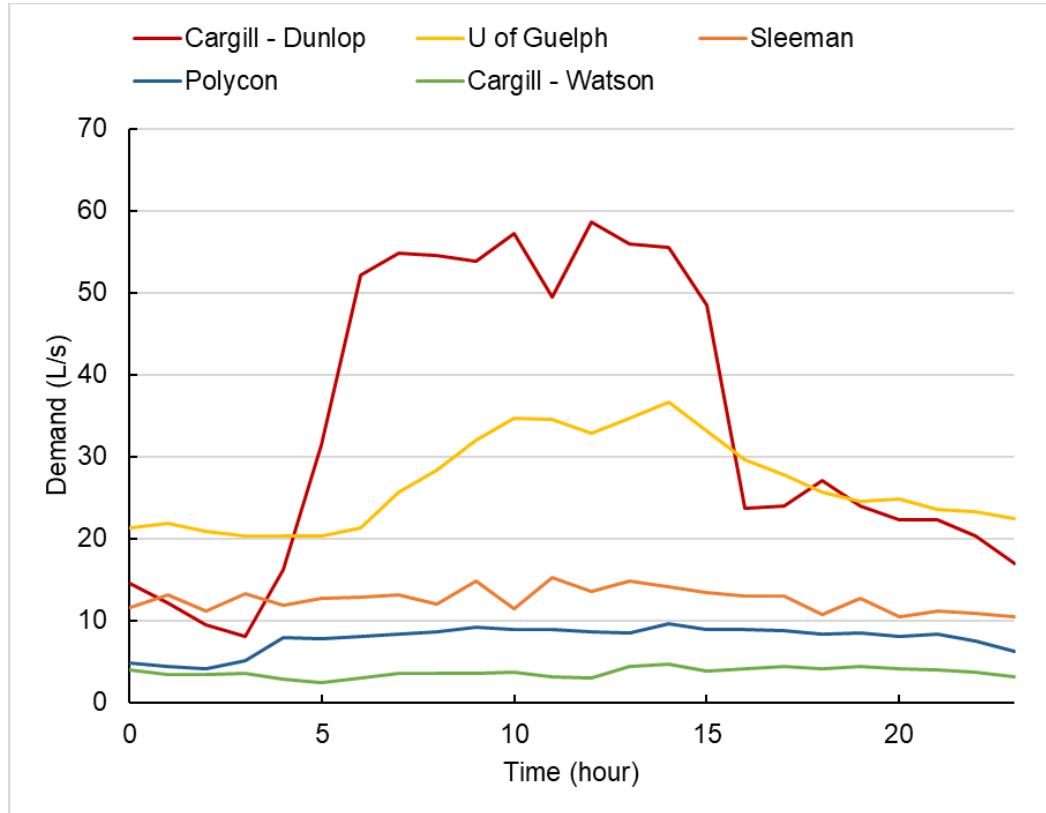
**2.5.1.1 Large Water Users**

Industrial, commercial and institutional (ICI) water use patterns can often vary from typical system usage and can have a significant impact on the system if they are a large water user. Based on the 2019 meter records provided, the top-5 customers accounted for 19% of the billed consumption and 16% of total production. Flow monitoring was completed in fall 2020 for a 2-week duration at each of the top-5 water users. Due to the project timing, large user monitoring was completed during the provincial lockdowns due to Covid-19. 2020 water usage may have varied from previous years. For large industrial users such as Cargill, Sleeman and Polycon, the total water usage and water usage pattern is expected to have remained similar to pre-2020 conditions, assuming that production has not drastically changed. At the University of Guelph, the total water consumption was significantly lower in 2020 compared to previous years due to the decrease in students and faculty on campus. However, the University water usage pattern still followed the expected trend of being highest from approximately 8:00am to 5:00pm during the hours that most people are on campus. The field data is compared to historical metered consumption in Section 3.1 below. As it is not yet known how water usage for large users will change in future years, the 2020 field data was used in the model for large user patterns at this time. The City should consider re-monitoring large users once the Province of Ontario has returned to normal societal function.

The 2020 field data was used to develop unique diurnal patterns for each of the 5 customers. The total demand used in the model at each large user was based on the 2019 billing meter records. The large

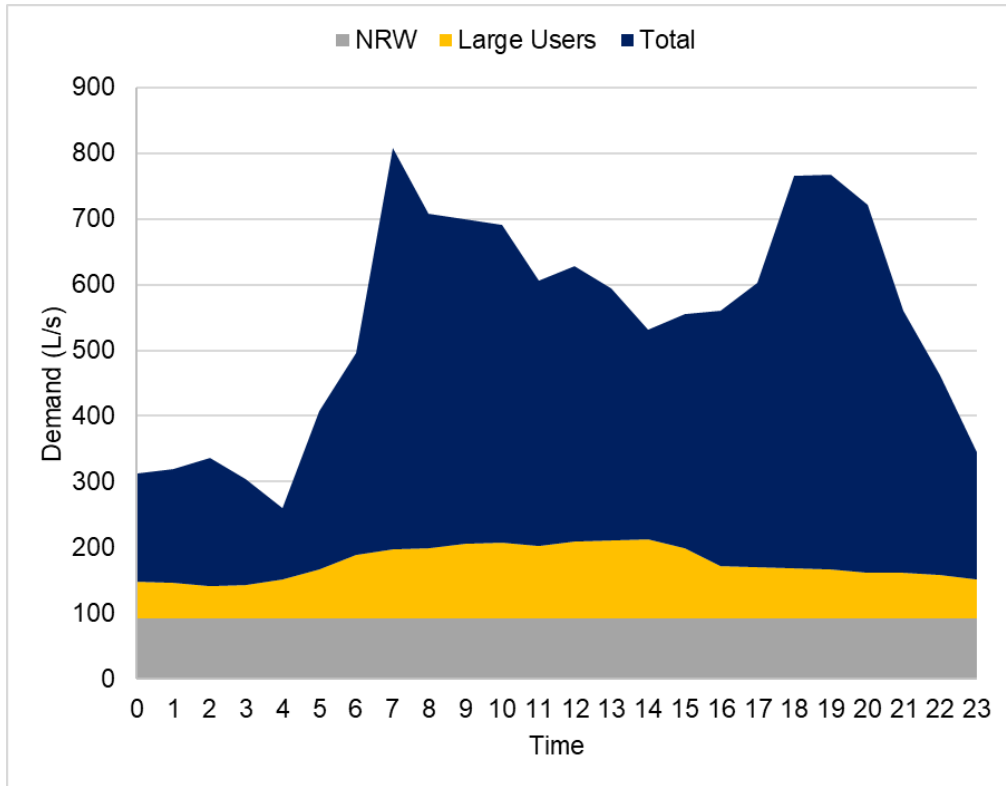


customer usage was then excluded from the overall pressure zone patterns applied to the remaining customers in the City. The top users are primarily located in Zone 1, except for Polycon which is in Zone 2. The model demand for the top users is presented in Figure 2-7 below. Large user data is discussed further in Section 3.1. Large user demands were expected to be relatively consistent between ADD and MDD.

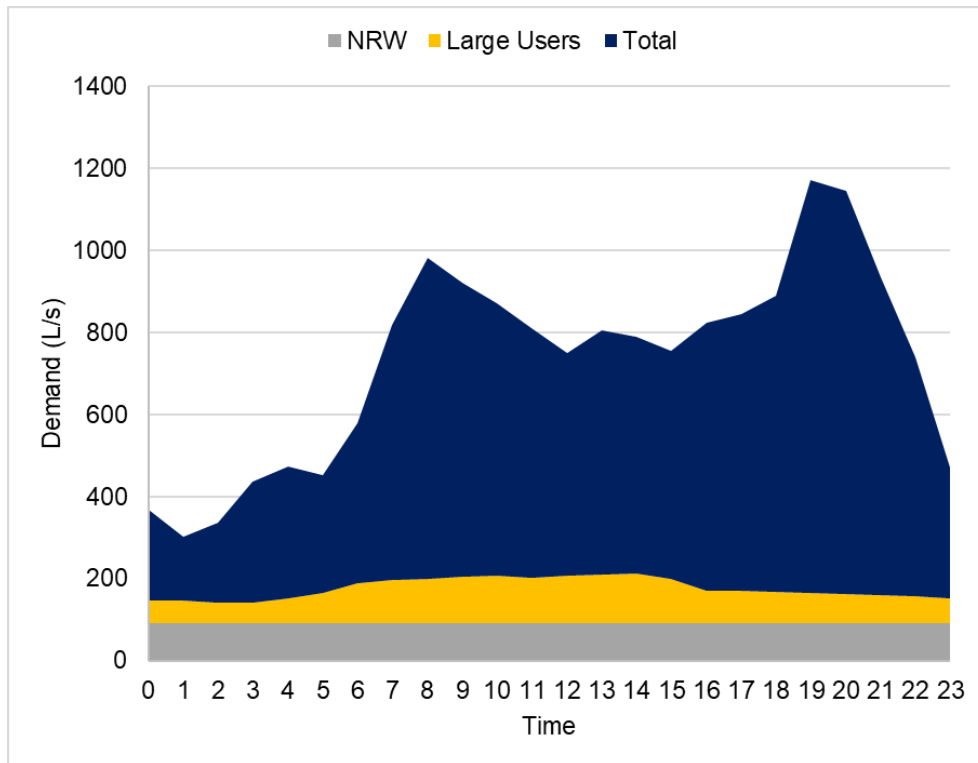


**Figure 2-7 Top 5 Users – Demand Patterns**

The overall system demand under ADD and MDD is presented in Figure 2-8 and Figure 2-9 below. The total demand and NRW are based on the values in Table 2-8 above.



**Figure 2-8 Total Demand – ADD 2019**



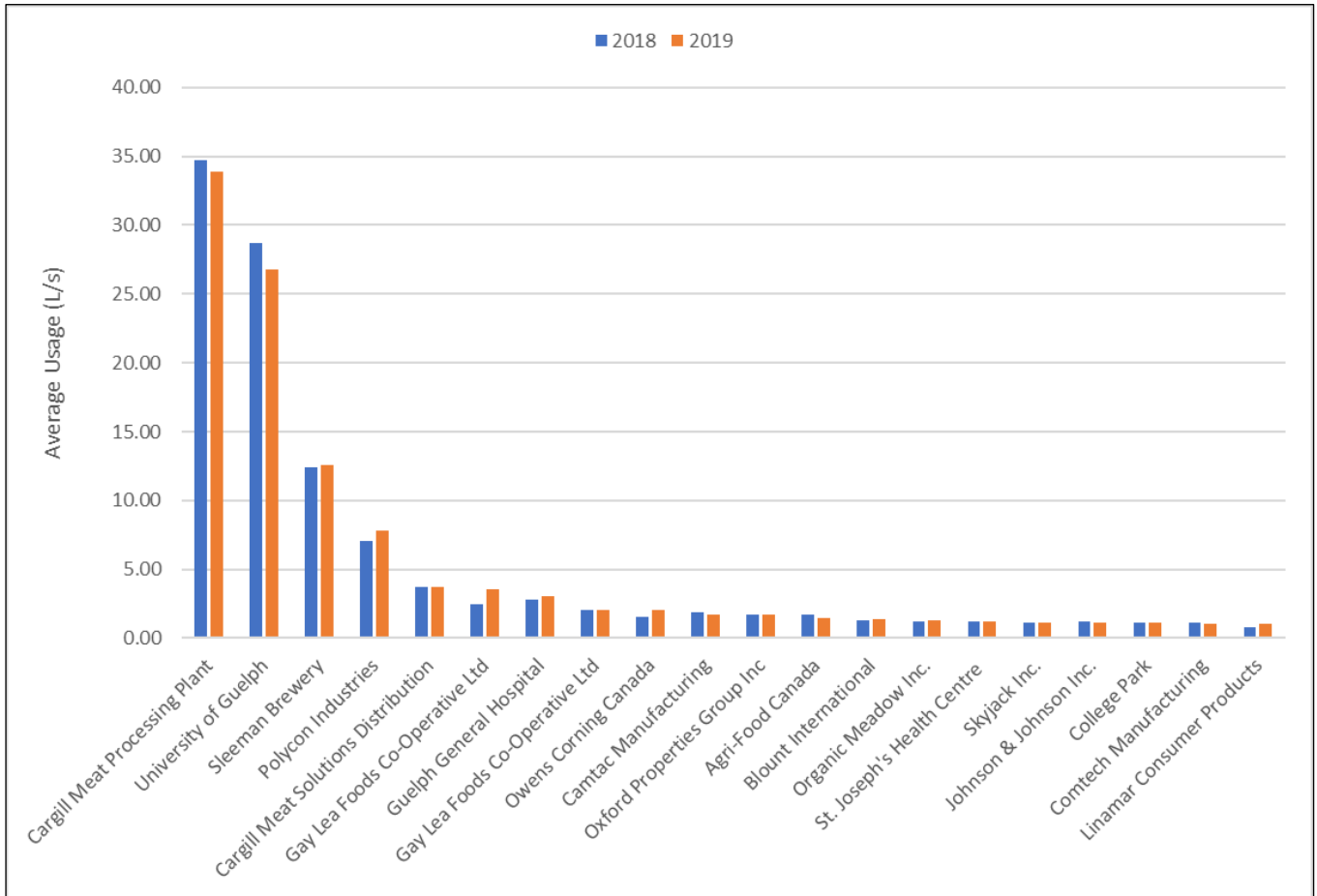
**Figure 2-9 Total Demand – MDD 2019**

### 3.0 WATER MODEL FIELD TESTING RESULTS AND CALIBRATION

#### 3.1 Field Testing

##### 3.1.1 Flow Monitoring of High Users

Large water users can have a significant impact on the distribution system. Customer billing meter records, provided by the City, were used to determine the largest users in the City. The annual usage for 2018 and 2019 for the top-20 customers is summarized in Figure 3-1 below. The top-4 users had significantly higher water usage compared to the remaining top-20.



**Figure 3-1 Top 20 Water Users – 2018 - 2019**

To improve the accuracy of the model demands, flow monitoring was completed at the top-5 users at the locations shown in Figure 3-2 below:

1. Cargill Meat Processing Plant (Cargill Dunlop)
2. University of Guelph (University)
3. Sleeman Brewery (Sleeman)
4. Polygon Industries (Polygon)
5. Cargill Meat Solutions Distribution (Cargill Watson)

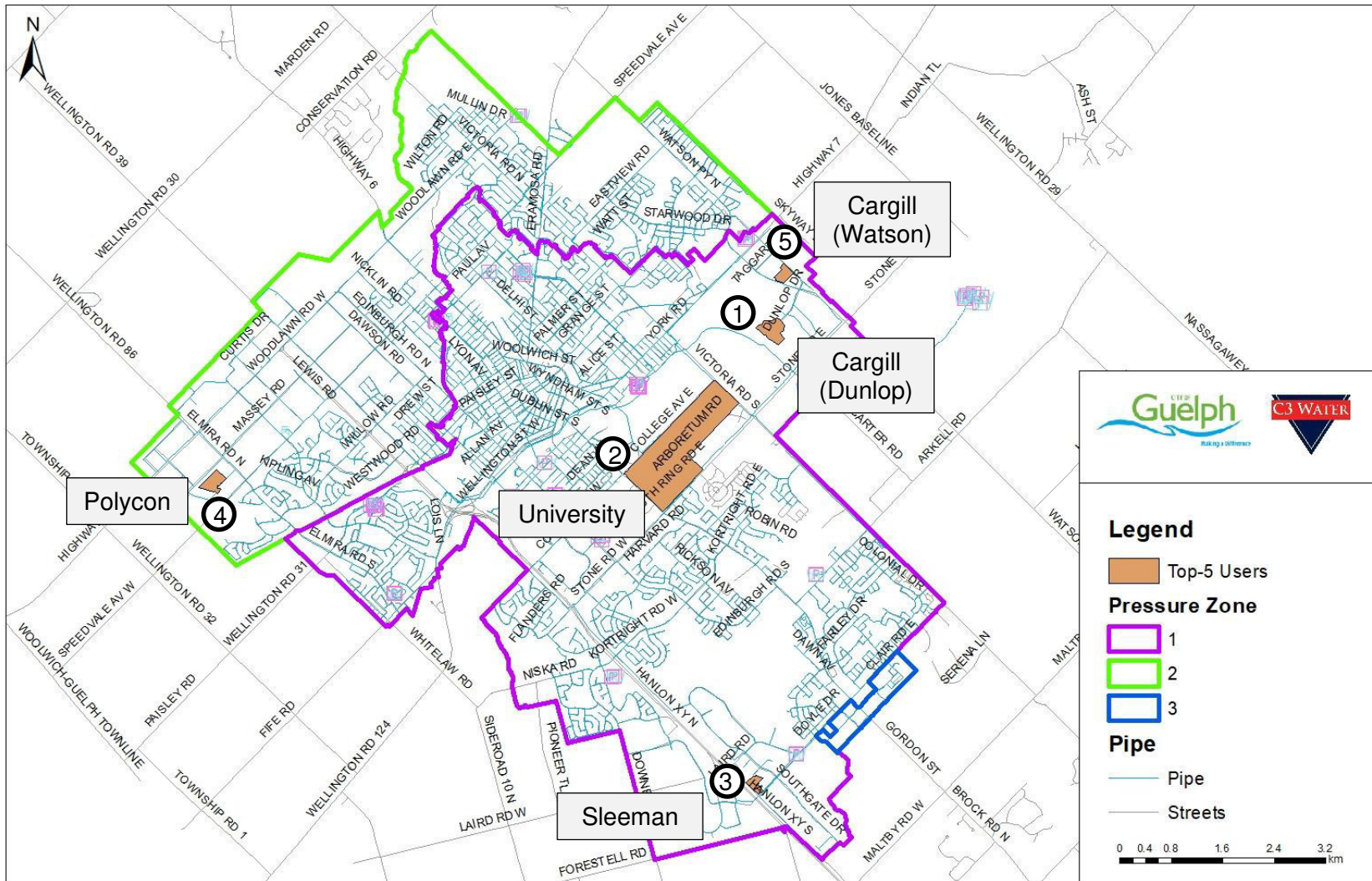


Figure 3-2 Flow Monitoring Locations

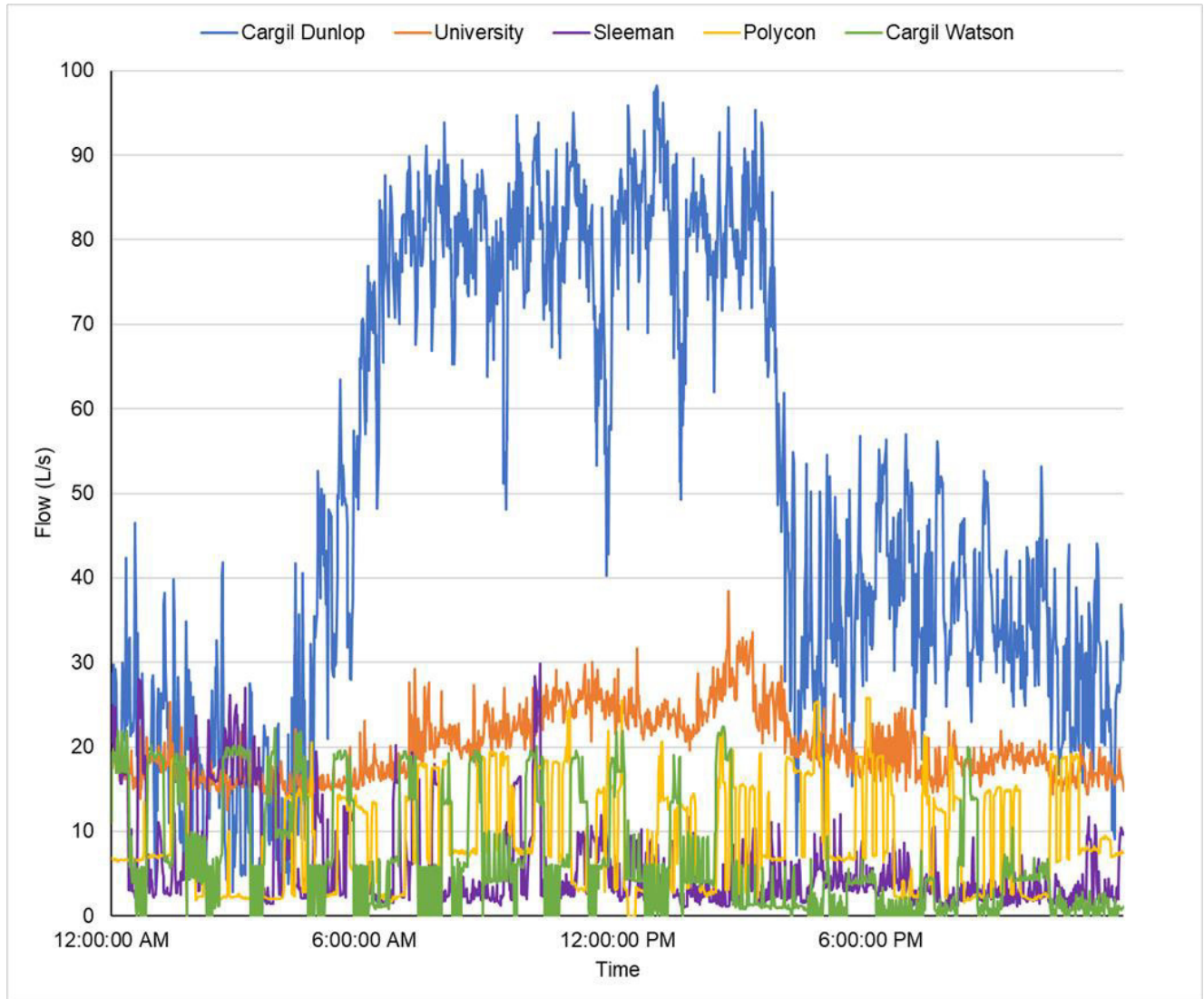
Flow monitoring was completed by Watermark using ultrasonic flowmeters from approximately September 10 – October 9, 2020. A summary of the flow testing and results is presented in Table 3-1 below. Flows were recorded at 1-minute intervals. Sleeman has multiple service connections, each with a significant water demand. As such, three (3) separate flowmeters were installed at Sleeman. Flow data at Cargill Watson was provided by the customer at a 15-second recording frequency.

**Table 3-1 Flow Monitoring Summary**

Customer		Start	End	Total Days	Average (L/s)	Maximum (L/s)
<b>Cargill</b>	Dunlop	11-Sep-20 11:04	09-Oct-20 11:31	28.0	37.2	100.9
<b>UofG</b>	n/a	16-Sep-20 10:50	08-Oct-20 11:55	22.0	18.3	44.9
<b>Sleeman</b>	Production	10-Sep-20 11:44	23-Sep-20 10:10	12.9	7.3	26.3
<b>Sleeman</b>	Meter 1 (upper)	10-Sep-20 11:00	09-Oct-20 09:47	28.9	1.8	20.1
<b>Sleeman</b>	Meter 2 (lower)	15-Sep-20 11:15	09-Oct-20 09:47	23.9	3.4	11.3
<b>Polycon</b>	n/a	10-Sep-20 14:33	07-Oct-20 11:53	26.9	7.7	26.1
<b>Cargill</b>	Watson	01-Aug-20 00:00	13-Oct-20 11:34	73.5	5.3	28.9

The average flow at each customer recorded during field testing was typically within 10% of the average 2019 total billed metered consumption. However, at the University, the average field recorded flow was 18 L/s compared to 27 L/s in the 2019 billing meter records. This difference is likely a result of a decrease in students and staff members on campus due to the Covid-19 pandemic. As it is not known at this time how water usage may continue to change as the City transitions out of the Covid-19 pandemic, the 2020 flow monitoring data was used in the model at this time. Changes in large-use consumption may continue to be monitored in future years to establish updated large user patterns.

Select field flow monitoring data is presented in Figure 3-3 below.



**Figure 3-3 Flow Monitoring Data**

The flow monitoring data was used to better understand the water consumption of large users and develop unique diurnal patterns. The large user patterns were then excluded from the overall pressure zone patterns during model development. As the top-5 user water demand accounts for approximately 16% of total water production, this field data improves the accuracy of the model demands. Although the large user diurnal patterns were based on 2020 field data, the total demand at each large user was based on the 2019 billing meter records.

### 3.1.2 Pressure Monitoring

Pressure monitoring was completed by Watermark and City staff. Pressure loggers were installed on hydrants throughout the distribution system. A total of 15 pressure monitoring locations were selected to achieve coverage of key areas of the system. The locations of existing District Metering Area (DMA) flowmeter and pressure logger chambers were also considered when selecting field testing locations. Pressure monitoring locations are summarized in Table 3-2 and Figure 3-4 below.

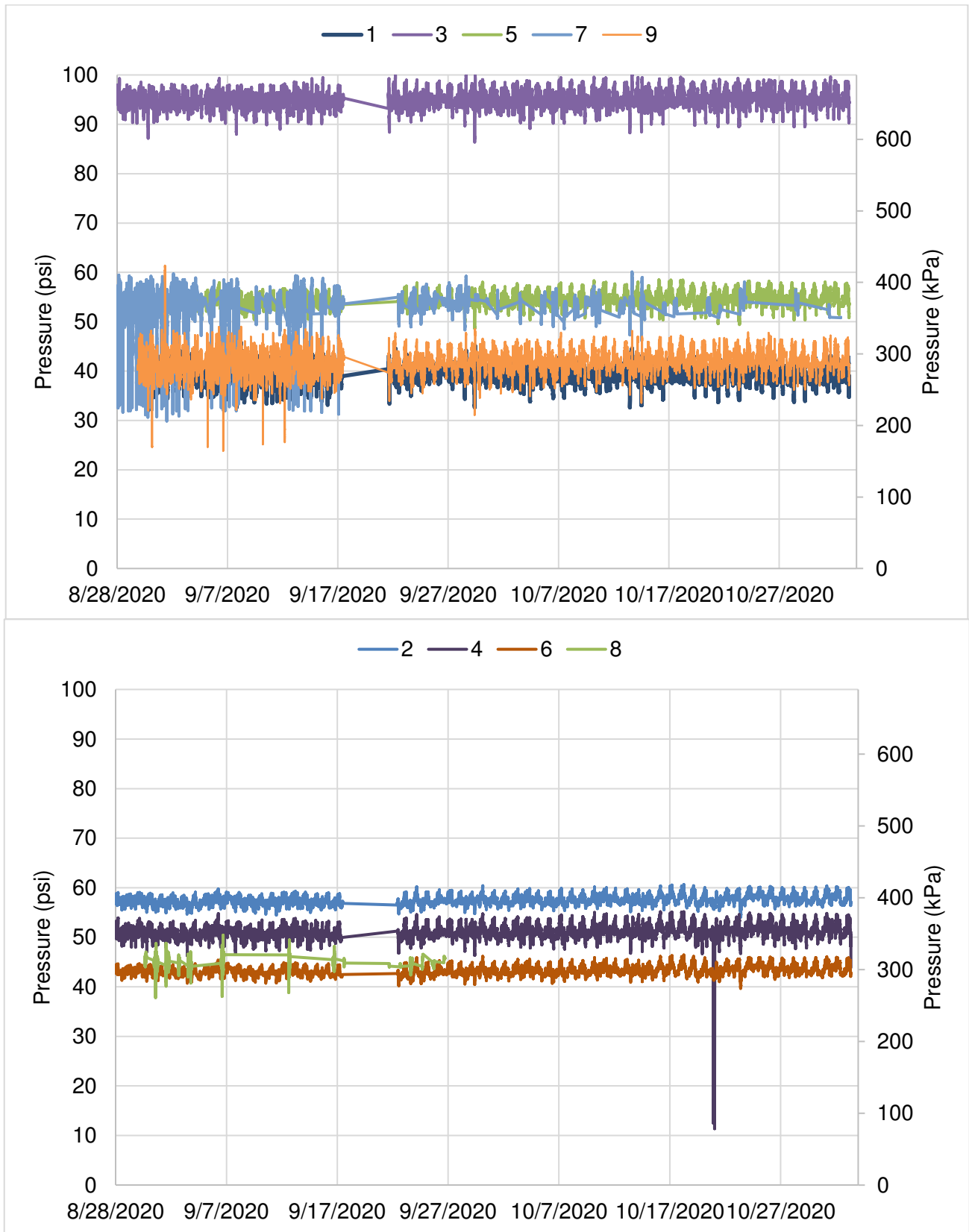
**Table 3-2 Pressure Monitoring Locations**

Number	Location	Zone	Hydrant	Installed By:
1	Stewart St & Palmer Rd.	1	H34-034	City
2	Southgate Dr & Admiral Pl.	1	H80-005	City
3	Bristol St. & Fountain St	1	H42-042	City
4	Amos Dr & Arkell Rd	1	H74-025	City
5	Robin Rd & Falcon Cl	1	H73-069	City
6	Southgate Drive	1	H83-004	City
7	University Ave & Lennox Ln	1	H51-005	City
8	Clair Rd & Victoria Rd	1	H82-023	City
9	Shoemaker Cr	1	H39-057	City
10	Gordon St & Clairfields Dr	1	H77-012	Watermark
11	Watson Parkway & Dunlop Drive	1	H62-005	Watermark
12	Eastview Rd & Summit Ridge Dr	2	H26-051	Watermark
13	Brant Ave & Muskoka Dr	2	H16-025	Watermark
14	Woodlawn Rd & Dawson Rd	2	H12-020	Watermark
15	Gosling St & Gordon St	3	H81-094	Watermark

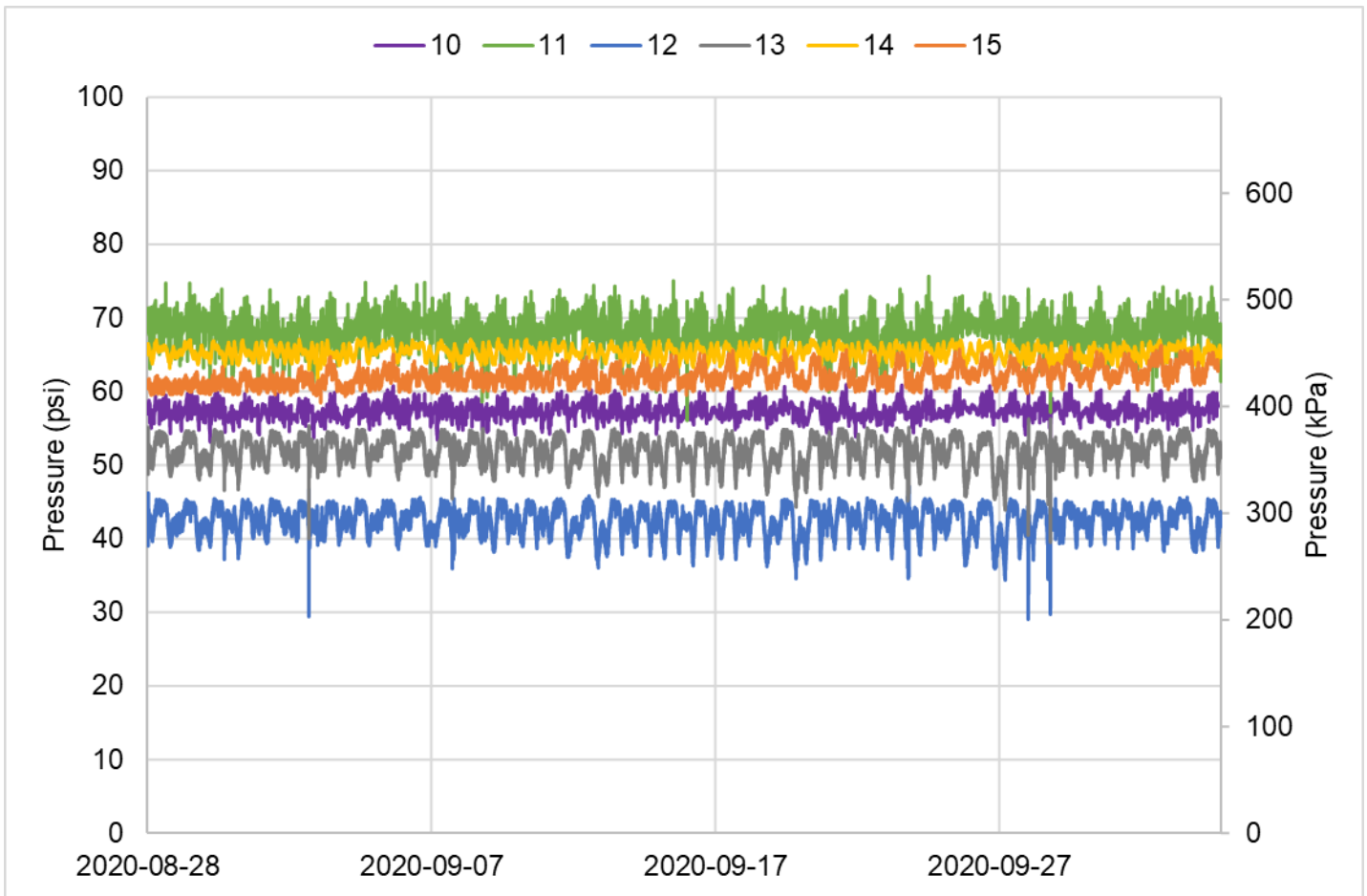




The pressure logging data collected by the City is summarized in Figure 3-5 and Figure 3-6 below, respectively. The data collected by the City included the minimum, maximum and average pressure recorded each minute. The City pressure loggers were re-programmed by City staff part-way through the field testing, which is reflected in the data gap from September 17-22. The plot below shows the average pressure at each reading. The range of pressure recorded at each logger is summarized in Table 3-3.



**Figure 3-5 Pressure Logger Data – City (average pressure)**

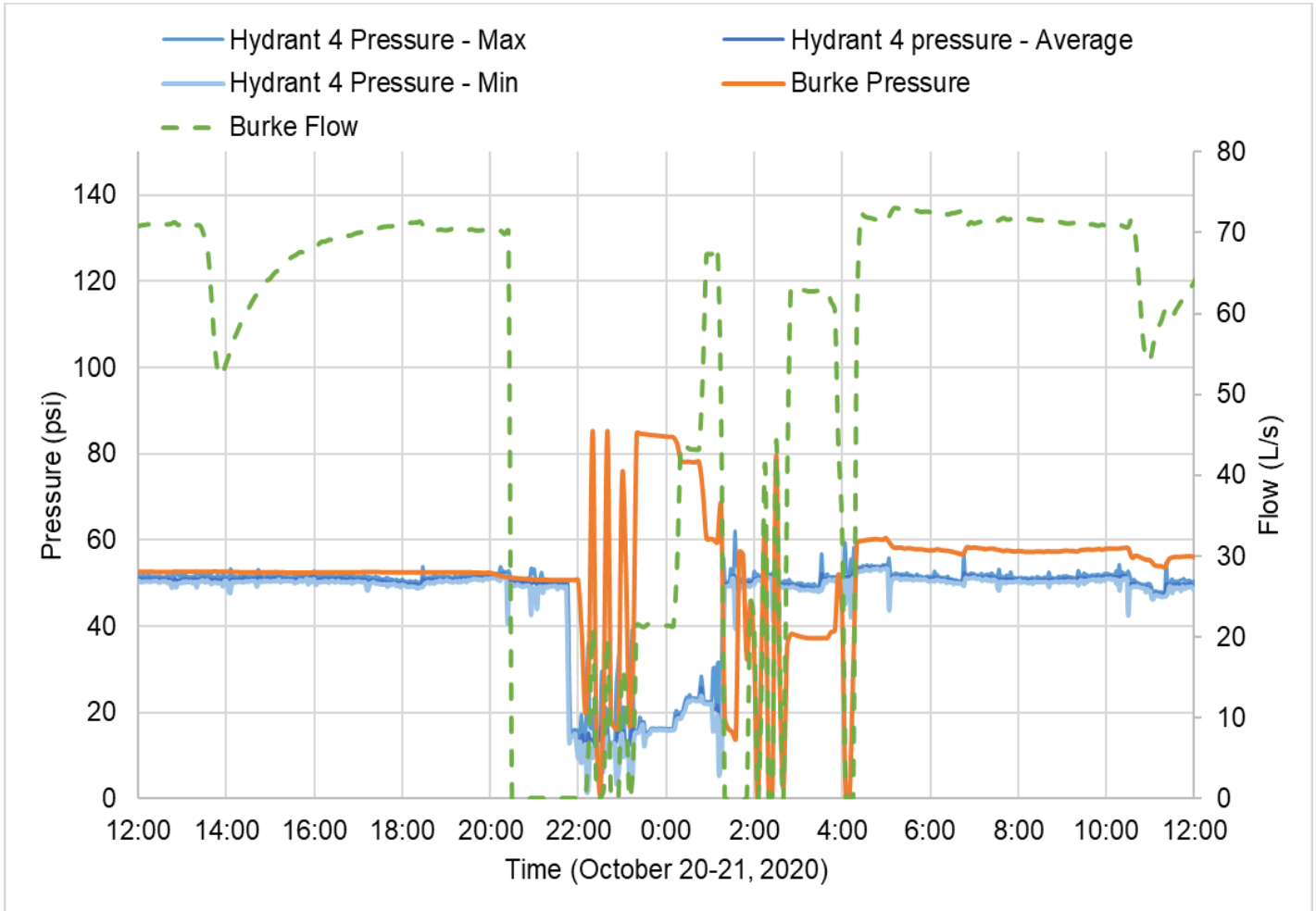


**Figure 3-6 Pressure Logger Data – Watermark**

**Table 3-3 Pressure Data Summary (kPa)**

Location	Hydrant	Field Data		
		Min	Average	Max
1	H34-034	152	272	345
2	H80-005	352	397	441
3	H42-042	538	655	689
4	H74-025	7	352	427
5	H73-069	241	374	441
6	H83-004	207	299	448
7	H51-005	179	354	786
8	H82-023	255	309	352
9	H39-057	110	288	531
10	H77-012	366	396	421
11	H62-005	387	471	522
12	H26-051	200	293	325
13	H16-025	273	358	388
14	H12-020	424	451	464
15	H81-094	403	428	457

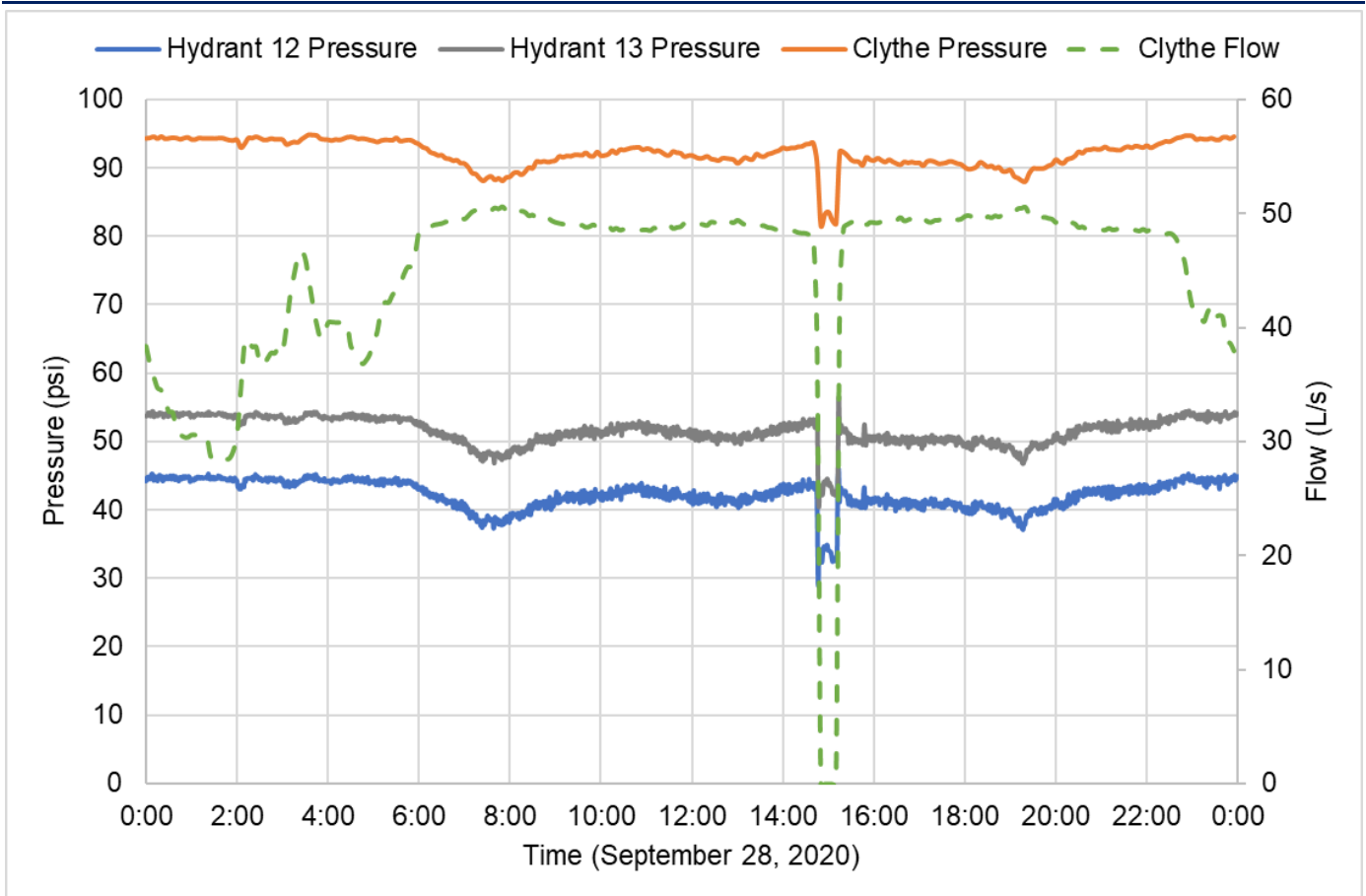
During the field testing period, brief but significant pressure drops were recorded at hydrant locations 4, 12 and 13. Hydrant location 4 is in close proximity to the Burke Well. The model results during the low pressure spike on October 20, 2020 are compared to the Burke discharge flow and pressure in Figure 3-7 below. The low pressure correlates with an abrupt stop in flow at Burke.



**Figure 3-7 Hydrant 4 Pressure and Burke Well Discharge**

Hydrant locations 12 and 13 are on the east side of Zone 2, supplied primarily by the Clythe PS. The pressure results are compared to the Clythe PS discharge on September 28, 2020 in Figure 3-8 below. The low pressure seen at the hydrants correlates with the Clythe PS shutoff. Based on discussion with City operators, these sudden shutdowns are likely caused by periodic generator testing at Clythe.

As shown in Figure 3-6 and Figure 3-8, pressures at hydrant 12 at Eastview Road and Summit Ridge Drive typically range from 200-300 kPa (30-45 psi). Through the ongoing Zone 2 Storage EA, a recommendation has been made to increase in discharge pressure at the Clythe PS to increase pressures on the east side of Zone 2 which will be reflected in the future planning horizon model scenarios. It is recommended that this new pressure setting be field tested to observe the impacts to Zone 2 and Zone 1 during the preliminary design phase of the Clythe PS upgrades.



**Figure 3-8 Hydrant 12 and 13 Pressure and Clythe PS Discharge**

**3.2 Water Model Calibration**

In addition to the field data described above, the City consistently collects 5-minute interval SCADA data including pump station discharge flows and pressures and elevated tank levels. Additionally, there are 50 DMA flowmeters and pressure data loggers throughout the system.

The existing conditions scenarios were run to determine if the model simulated reasonable solutions. SCADA data was used to set-up the model boundary conditions, such as initial tank levels and demands, to represent specific dates. The model results were then compared to the field data from that date. October 23, 2019 was selected as to verify ADD model conditions as this date had demands similar to the overall 2019 daily average. October 23, 2019, represents a typical week day without unusually high or low water usage that might be seen on a hot summer day or a holiday. A 2020 date was used for MDD calibration as it had a higher recorded demand than any day in 2019. Overall water usage trends may vary in 2020 compared to previous years due to the Covid-19 pandemic. However, the July 8, 2020 date was used for model calibration due to the high demand which is ideal for stress-testing the model under MDD conditions.

The demand for each date was calculated using 5-minute interval SCADA data for each supply source into the system and balanced to account for the change in water level at the elevated tanks and reservoirs. The following dates were used to compare the water model results with field data:

**Table 3-4      Dates Used for Model Calibration**

Scenario	Date	Total Demand (L/s)*
ADD	23-Oct-19	550
MDD	8-Jul-20	721

*\*Calculated from SCADA data*

The following control scenarios were assessed for model calibration:

1. ADD 2019 – Dynamic Controls
2. MDD 2020 – Dynamic Controls
3. MDD 2020 – Time Controls

The model controls were set up based on the dynamic controls described in Table 2-6. This allows the model to automatically react to various demand and operational conditions.

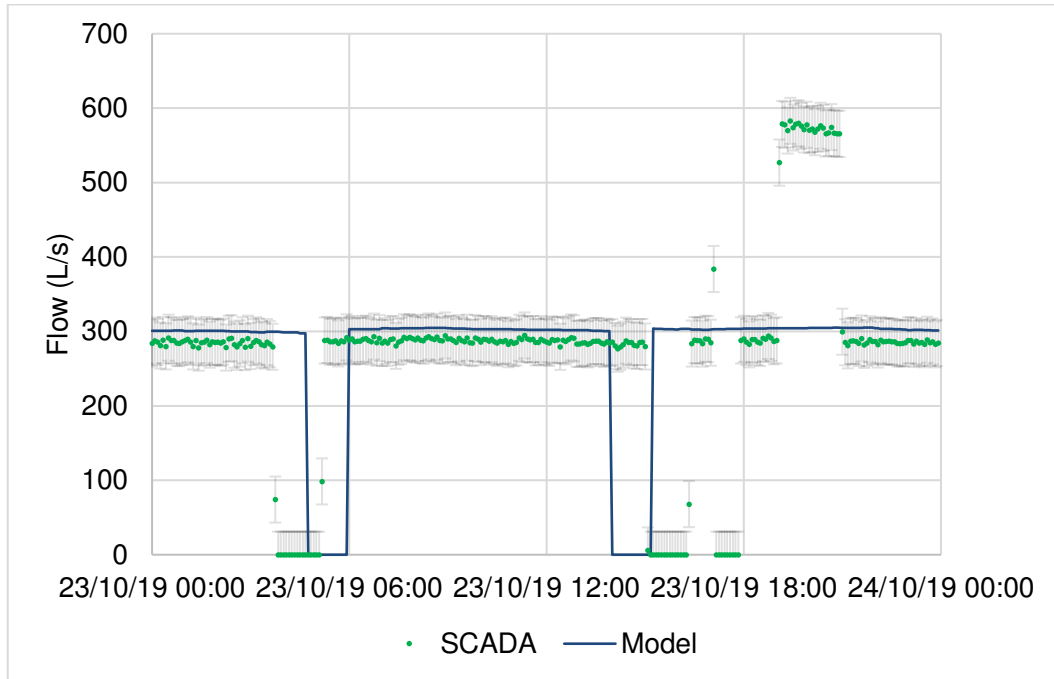
The MDD scenario was also run with the pumps set on time controls such that pump station discharge flows would closely match what was recorded on SCADA on that date. The time controls were modelled to assess how closely the results in the distribution system would match the SCADA data if the conditions at the pump stations were exactly the same.

Select calibration results are presented in the sections below.

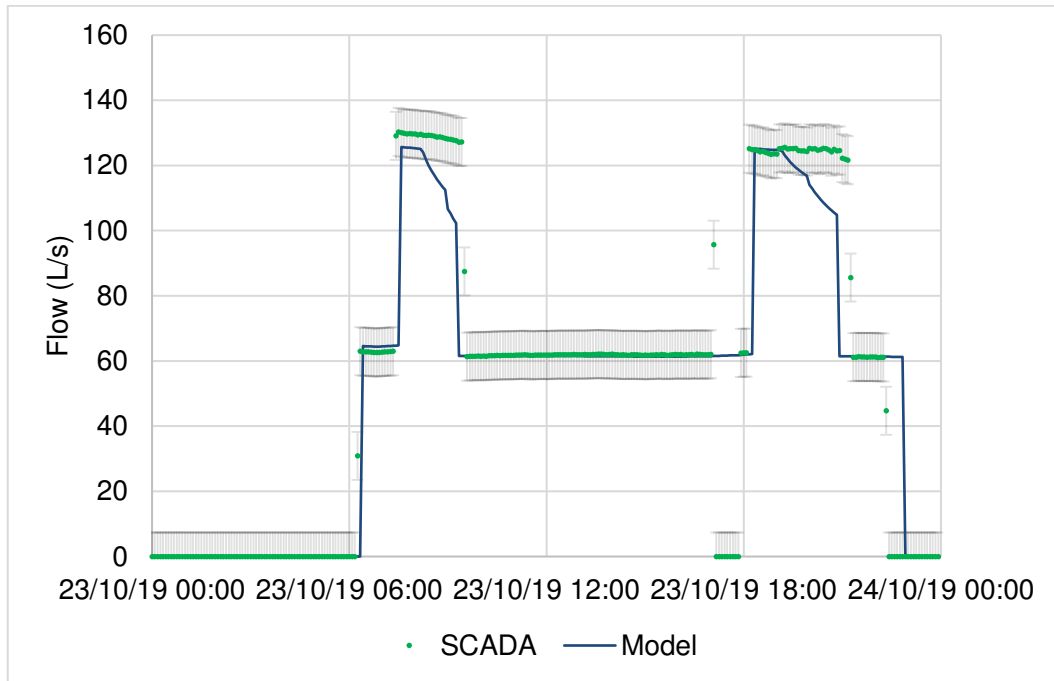
### 3.2.1 Pump Station Discharge Flow (Dynamic Controls)

The pump station discharge flows under ADD with dynamic controls at the major pump stations; Woods, Paisley, Robertson and Clythe are presented in Figure 3-9 to Figure 3-12 below. The model results generally matched SCADA, with the exception of a few brief time periods.

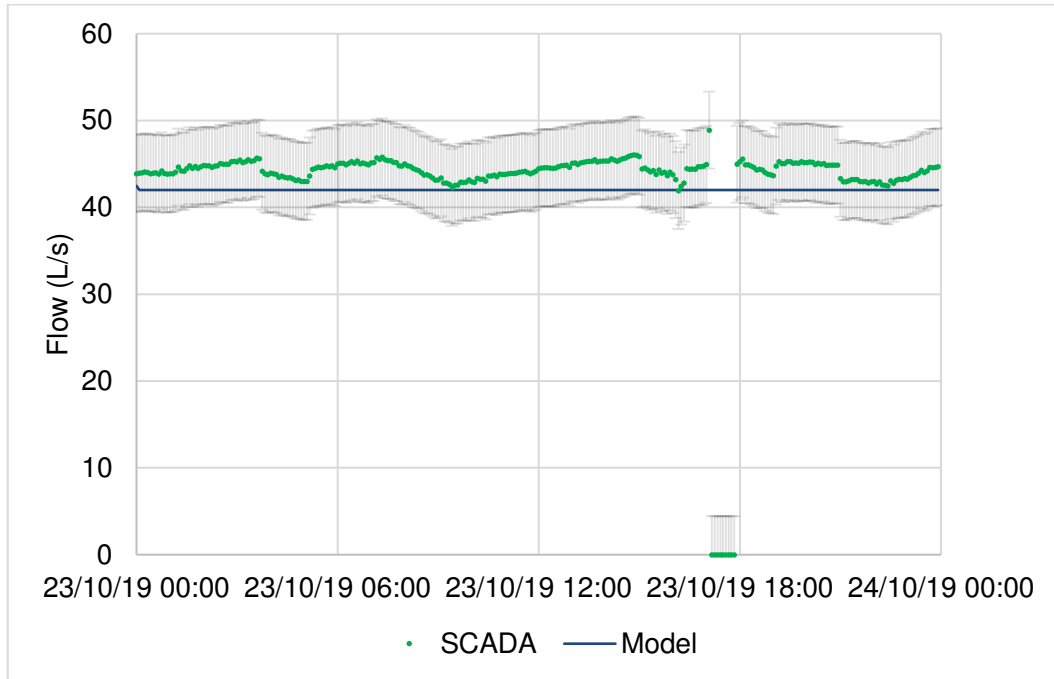
- Woods: For the majority of the day, one pump was running at approximately 300 L/s. All pumps turned off in the model at approximately 5:00am compared to 4:00am recorded in SCADA. In SCADA, a third pump turned on at about 7:00pm, which was not reflected in the model as a result of the dynamic controls used based on the Verney ET level. This indicates that the dynamic controls used in the model may vary slightly from the actual controls used on this date. Overall, average flow out of the Woods PS was aligned.
- Paisley: The model discharge flow generally matched SCADA closely.
- Robertson: The model matched SCADA closely with one pump running for the majority of the day. The discharge flow in the model is currently controlled with a FCV. The model was tested without a FCV and the booster pump in the model ran at approximately 65 L/s based on the pump manufacturer curve. The accuracy of the curve in the model may be improved by completing pump performance testing.
- Clythe: The model discharge flow generally followed the same trend as was recorded in SCADA. From approximately 2:00am to 6:00am and 9:00pm to midnight, the model flow was approximately 5 L/s lower than what was recorded in SCADA.



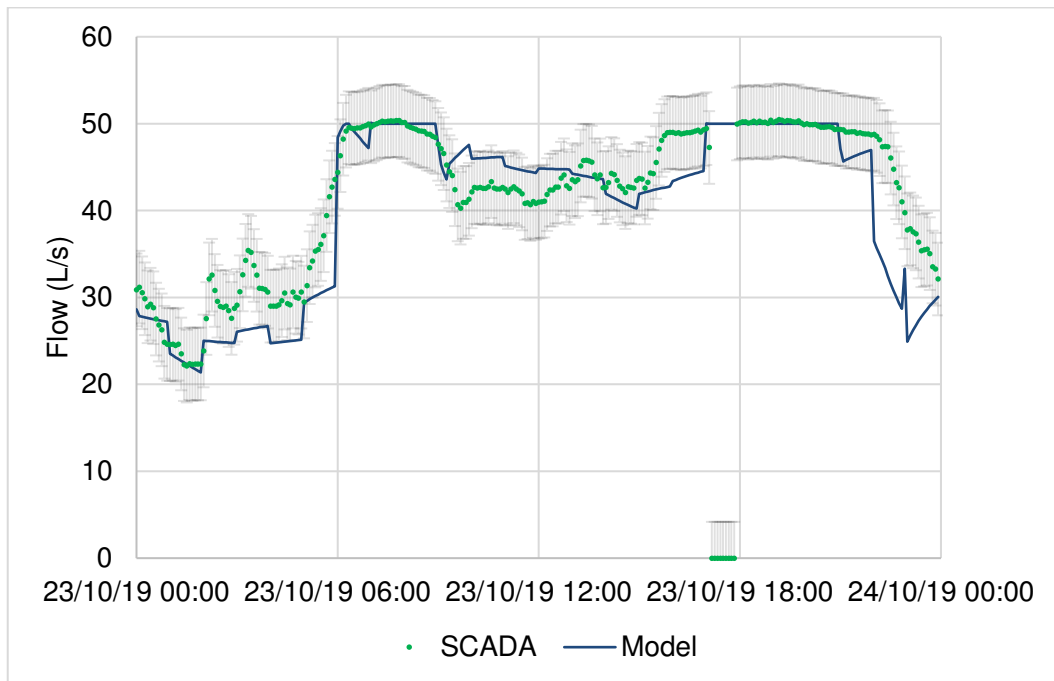
**Figure 3-9 FM Woods Discharge Flow – ADD 2019 – Dynamic Controls**



**Figure 3-10 Paisley Discharge Flow – ADD 2019 – Dynamic Controls**



**Figure 3-11 Robertson Discharge Flow – ADD 2019 – Dynamic Controls**



**Figure 3-12 Clythe Discharge Flow – ADD 2019 – Dynamic Controls**

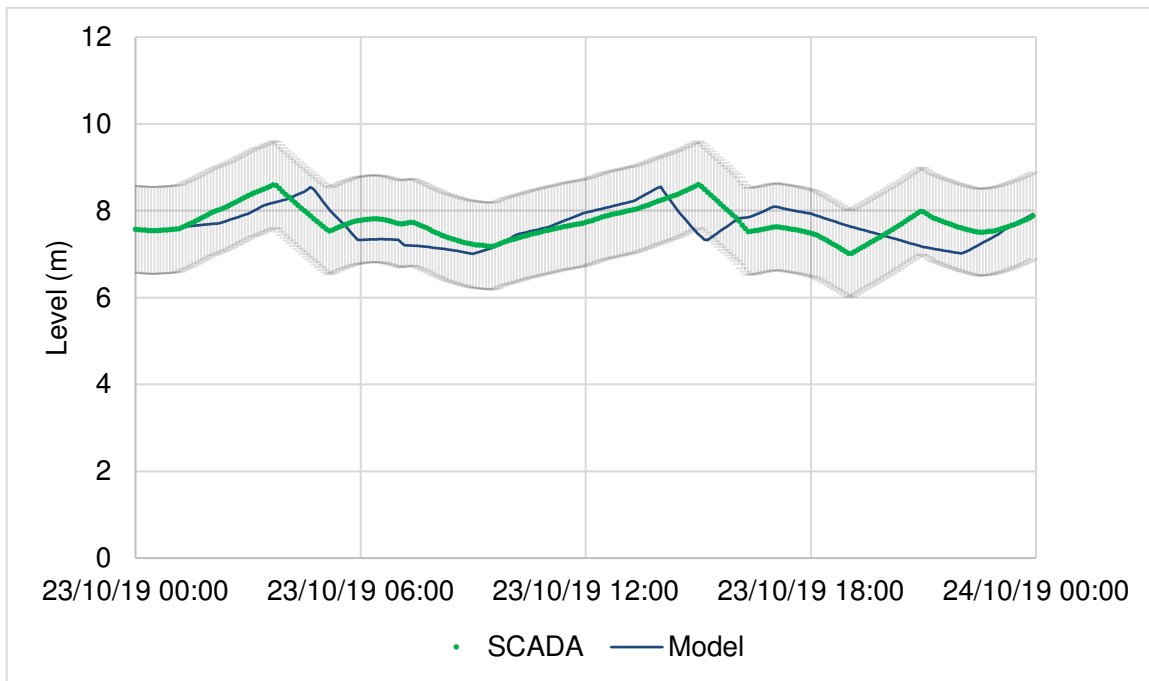
Overall, the discharge flows matched SCADA fairly closely using dynamic controls. Differences between the model and SCADA may be a result of operational adjustments to pump station control strategies.



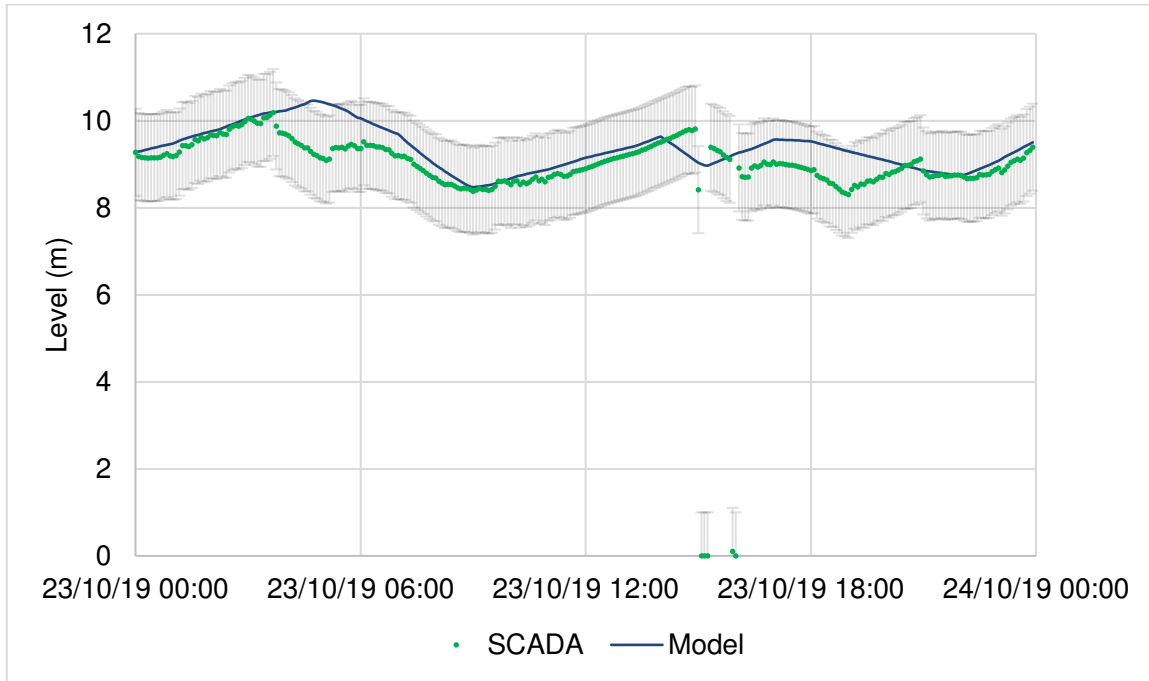
### 3.2.2 Tank Levels

The ET levels are a good indicator of reasonable model simulation since it should follow a predictable pattern throughout the day based on system demands and pump station controls. The American Water Works Association (AWWA) Manual of Water Supply Practices M32, provides guidelines for computer modelling of water distribution systems. It includes guidelines for Hydraulic Grade Line (HGL) and water level fluctuations, which suggest that modelled tank levels should be within 0.9 to 1.7 meters of those recorded in the field.

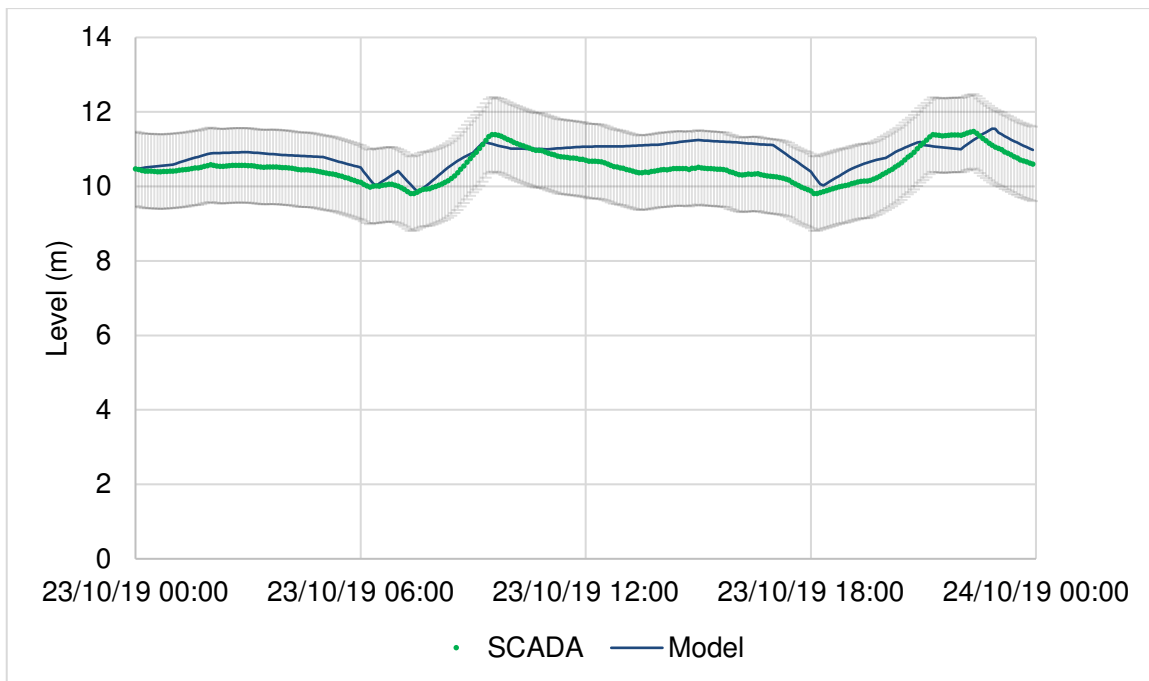
The Verney, Clair and Speedvale ET levels under ADD with dynamic controls are presented in Figure 3-13 to Figure 3-15 below. There were some differences between the model results and what was recorded in SCADA, likely due to slight differences in pump station controls. Overall, the ET levels remained within 1.7m of what was recorded in SCADA. It appears that the Clair ET level was not recorded in SCADA for a period from 12:00pm to 3:30pm. The Clair ET level was linearly interpolated for this time period.



**Figure 3-13 Verney ET Level – ADD 2019 – Dynamic Controls**



**Figure 3-14 Clair ET Level – ADD 2019 – Dynamic Controls**



**Figure 3-15 Speedvale ET Level – ADD 2019 – Dynamic Controls**

The tank levels under the MDD scenario with the pump stations modelled with time-based controls to match SCADA are summarized in Figure 3-16 to Figure 3-18 below. With the pump station discharge flows set to match what was recorded in SCADA, the tank level trends matched well.

- Verney: In the morning, the ET level drained slightly faster in the model than was recorded in SCADA. In The afternoon, the ET filled slightly faster in the model. Differences between the model and SCADA may be a result of slight differences in the distribution of demands between the model and what was occurring on this date. Additionally, one unknown of this area of Zone 1 is the flow from Zone 1 into the Clythe reservoir as this is not currently recorded in SCADA. At this time, for developing the diurnal pattern it is assumed that the flow from Zone 1 into the Clythe reservoir is equal to the Clythe discharge flow. In the morning, the Clythe inlet flow may have been higher in the model than it was on that date, causing the Verney ET to drain more quickly. Overall, the ET level in the model was consistently within 1.7m of the level recorded in SCADA.
- Clair: The model level was consistently slightly higher than SCADA but followed the trend very closely and was well within the range of 1.7m compared to SCADA.
- Speedvale: The model level generally followed the same trend as SCADA. In the morning the tank drained slightly more in the model than in SCADA. In the evening, the model level did not drop as low as was recorded in SCADA.

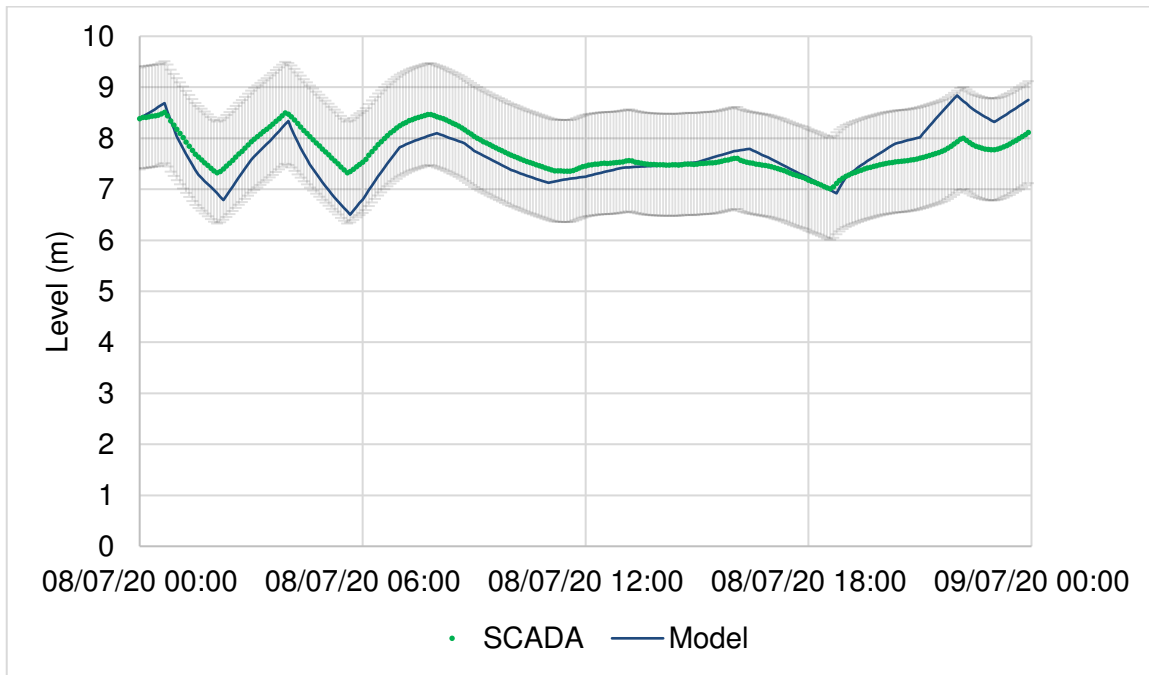
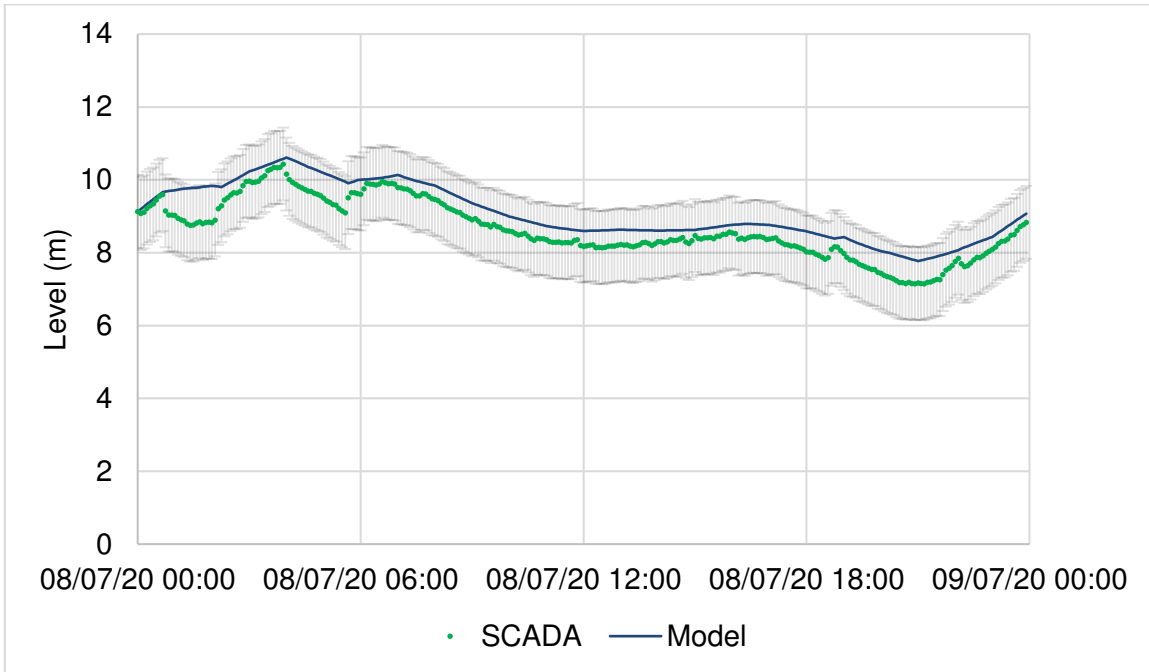
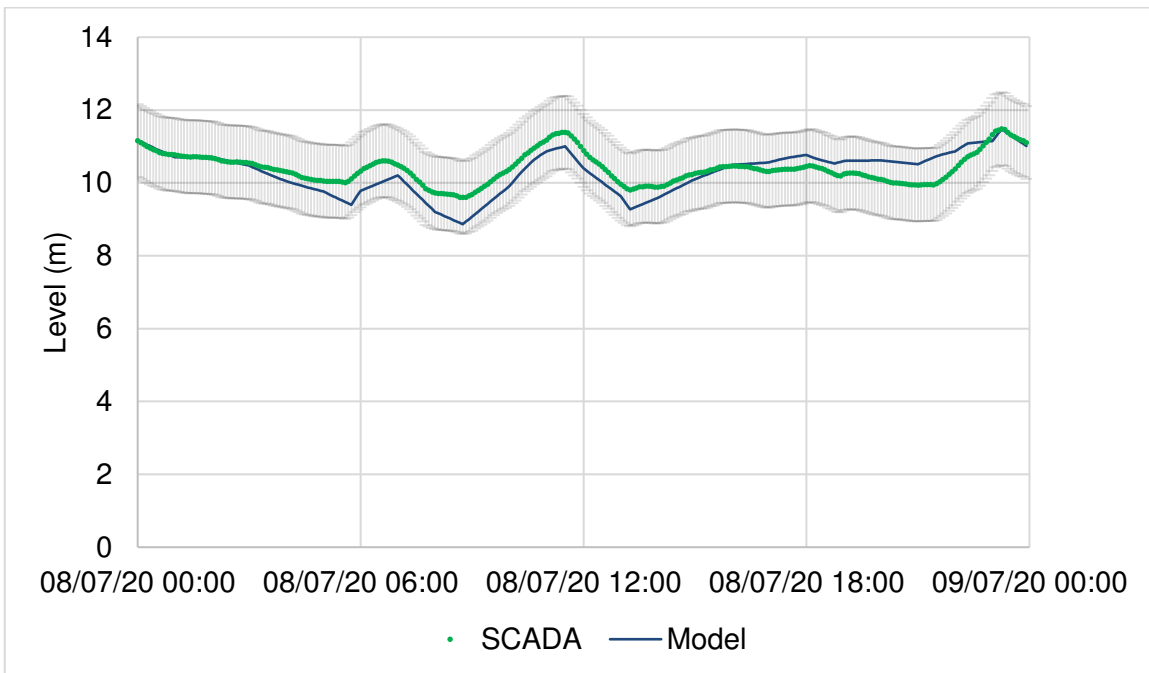


Figure 3-16 Verney ET Level – MDD 2020 – Time Controls



**Figure 3-17 Clair ET Level – MDD 2020 – Time Controls**



**Figure 3-18 Speedvale ET Level – MDD 2020 – Time Controls**

A comparison of the model results to SCADA for the three (3) scenarios that were modelled are summarized in Table 3-5 to Table 3-7 below. At each 5-minute time step, the difference in level was taken between the model results and the recorded SCADA data. The portion of the day that the difference in level fell within each category is summarized. The majority of the model results were within 0.5m of what was recorded in

SCADA. Under ADD with dynamic controls, the difference between SCADA and the model did not exceed 1m at Speedvale and only briefly exceeded 1m at Clair and Verney.

Under MDD, with dynamic controls the model results were primarily within 1m of SCADA. The difference between SCADA and the model briefly exceeded 1m at Verney and Clair. With time controls, all tanks were consistently within 1m of what was recorded in SCADA with the exception of Clair ET for a brief period of time. Using time controls was found to improve the ET level results in Zone 1. This is likely due to Woods pumps being operated with a slightly different control strategy on a day-to-day basis that is not captured in the common dynamic control set in the model.

Overall, the ET level results were all within the AWWA guideline of 1.7m, with the brief exception of Clair under the MDD scenario with Dynamic controls.

**Table 3-5 ET Level Model to SCADA Comparison – ADD 2019 – Dynamic Controls**

Level Difference	Verney	Clair	Speedvale
< 0.5m	81%	72%	77%
0.5 - 1m	17%	24%	23%
> 1 - 1.7m	2%	4%	0%
>1.7m	0%	0%	0%

**Table 3-6 ET Level Model to SCADA Comparison – MDD 2019 – Dynamic Controls**

Level Difference	Verney	Clair	Speedvale
< 0.5m	75%	34%	70%
0.5 - 1m	24%	58%	30%
> 1 - 1.7m	1%	8%	0%
>1.7m	0%	0%	0%

**Table 3-7 ET Level Model to SCADA Comparison – MDD 2019 – Time Controls**

Level Difference	Verney	Clair	Speedvale
< 0.5m	79%	64%	73%
0.5 - 1m	21%	34%	27%
> 1 - 1.7m	0%	2%	0%
>1.7m	0%	0%	0%

### 3.2.3 DMA Pressure

The DMA chamber pressure logger data was compared to the model results for the select dates. AWWA M32 states that the HGL prediction by the model should be within 1.5 to 3 meters (14 to 30 kPa) of those recorded in the field. Currently, the system DMAs are not isolated, and DMA boundary valves were not closed in the model.

---

Select results from the ADD dynamic controls scenario are presented in Figure 3-20 to Figure 3-24 below. The locations of these DMA chambers are shown in Figure 3-19 below. These locations were selected due to their geographic distribution in Zones 1 and 2. There are not currently any DMA chambers installed in Zone 3. Complete DMA results are provided in Appendix A.

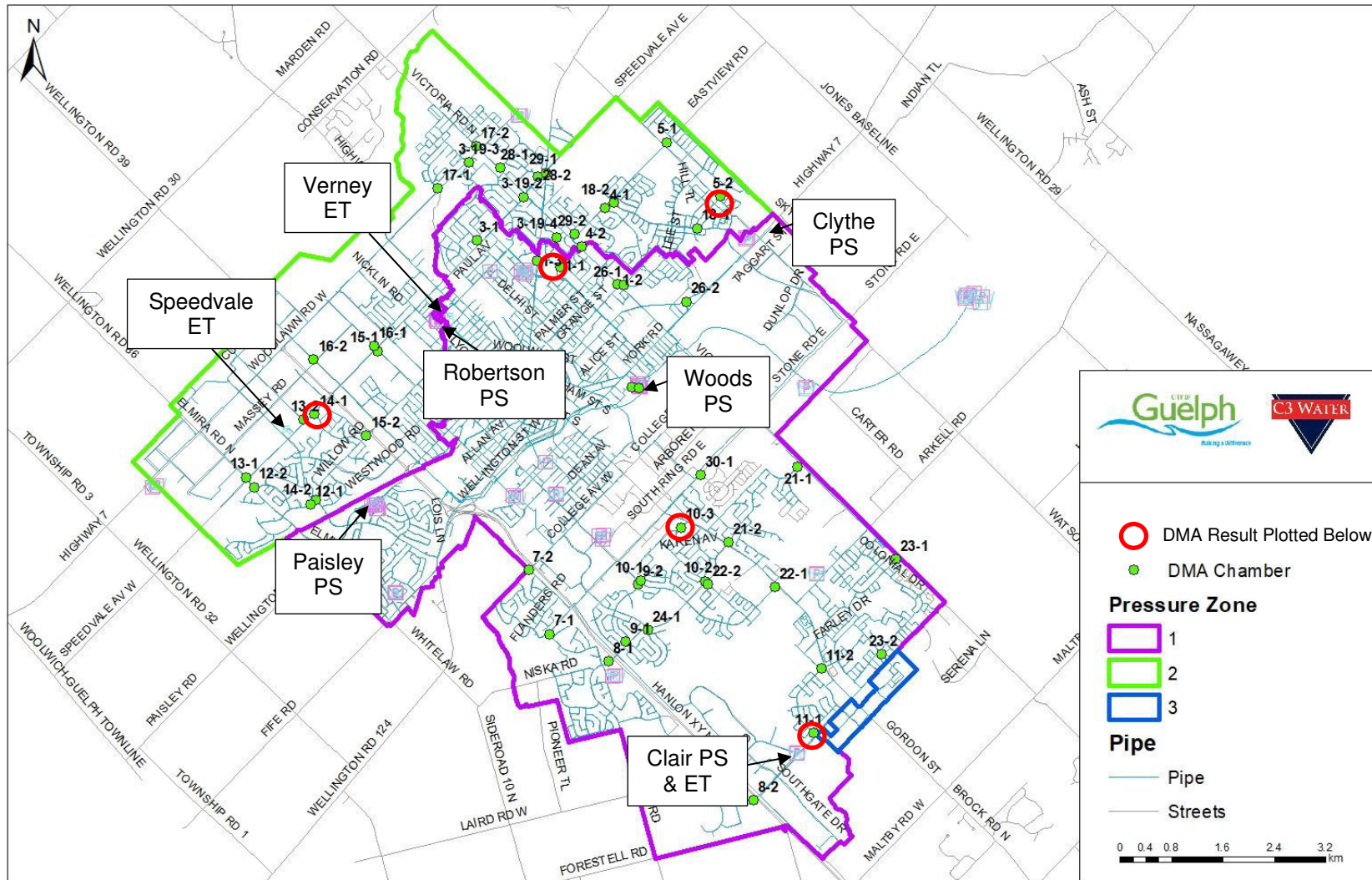
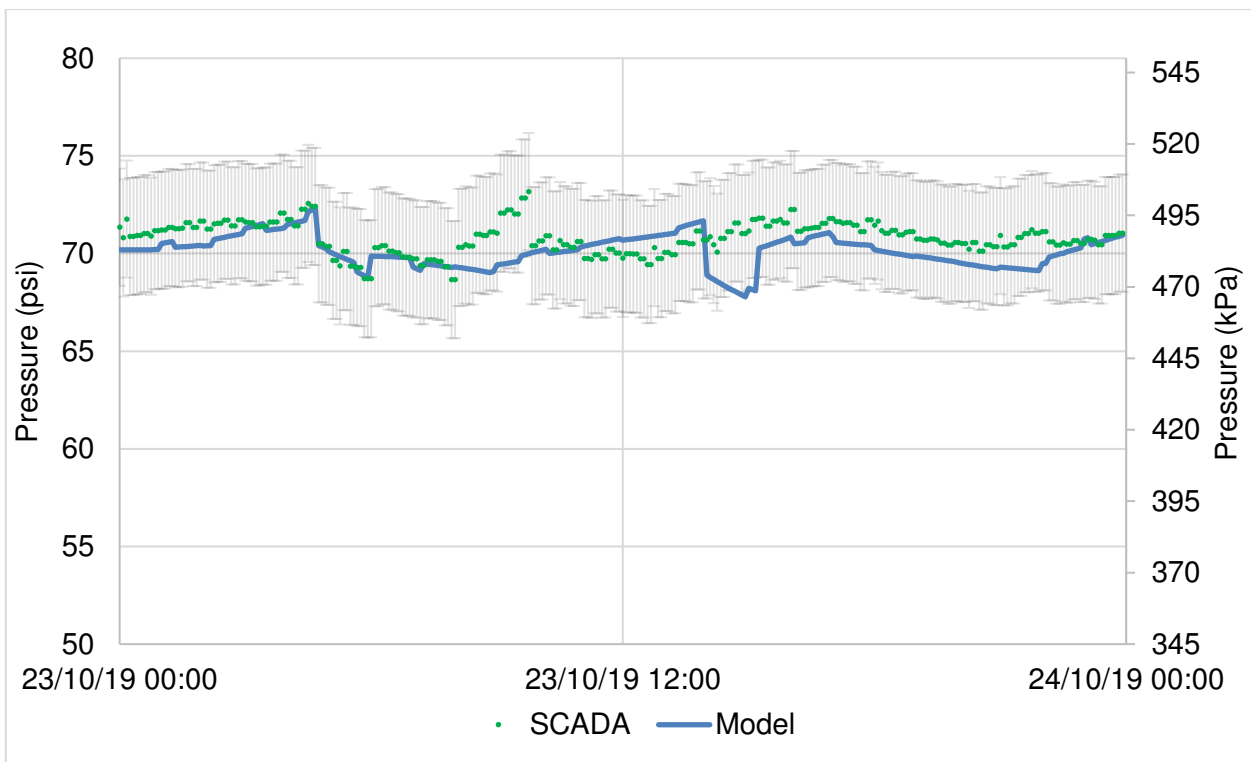


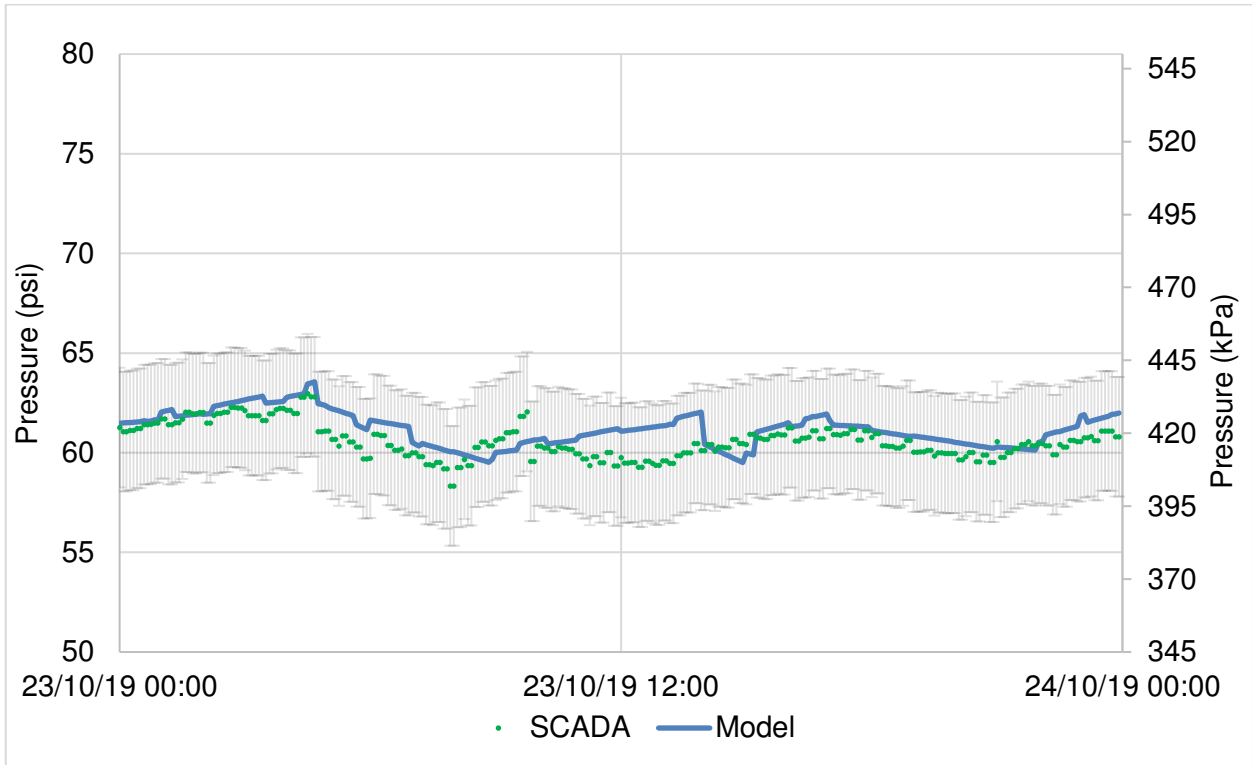
Figure 3-19 DMA Chamber Locations

- Chamber 1-1: The model results were generally followed the same trend as SCADA. In the model, the pressure dropped below SCADA from approximately 2:00pm to 3:00pm. This correlates with when all pump turned off at Woods in the afternoon, as shown in Figure 3-9 above.
- Chamber 10-3: The model results followed the SCADA data fairly closely. The model pressure exceeded SCADA by about 3 psi from approximately noon until the Woods pumps turned off in the afternoon.
- Chamber 11-1: The model results generally closely matched SCADA. The SCADA results showed that the pressure dropped slightly lower than was seen in the model at approximately 8:00am and 1:00pm. Similar to at chamber 10-3, the model pressure was about 3 psi higher in the model than SCADA from about noon to 3:00pm.
- Chamber 5-2: The model pressure matched SCADA closely. In the evening from 6:00pm to 8:00pm, the pressure the model pressure dropped about 1 psi lower than SCADA. This location is largely influenced by the Clythe PS discharge flow. The Clythe flow was slightly lower in the model than was recorded in SCADA from 6:00pm to 8:00pm.
- Chamber 14-1: The model pressures were generally within the same range as what was recorded in SCADA. At noon, the model pressure was higher than SCADA. This chamber location is likely influenced primarily by the Speedvale ET level. As shown in Figure 3-15, under ADD with dynamic controls, the model ET level was higher at noon than was recorded in SCADA.

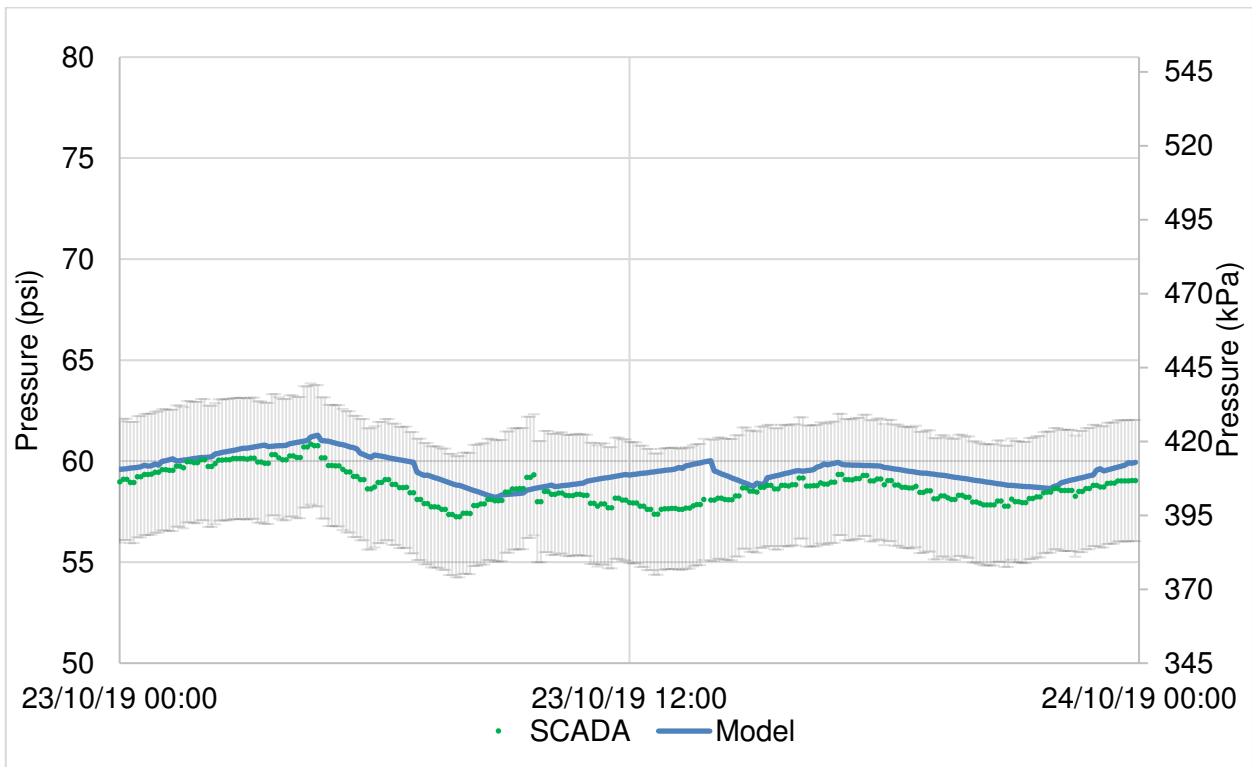


**Figure 3-20 DMA Chamber 1-1 (Zone 1: Meyer Dr) Pressure – ADD 2019 – Dynamic Controls**

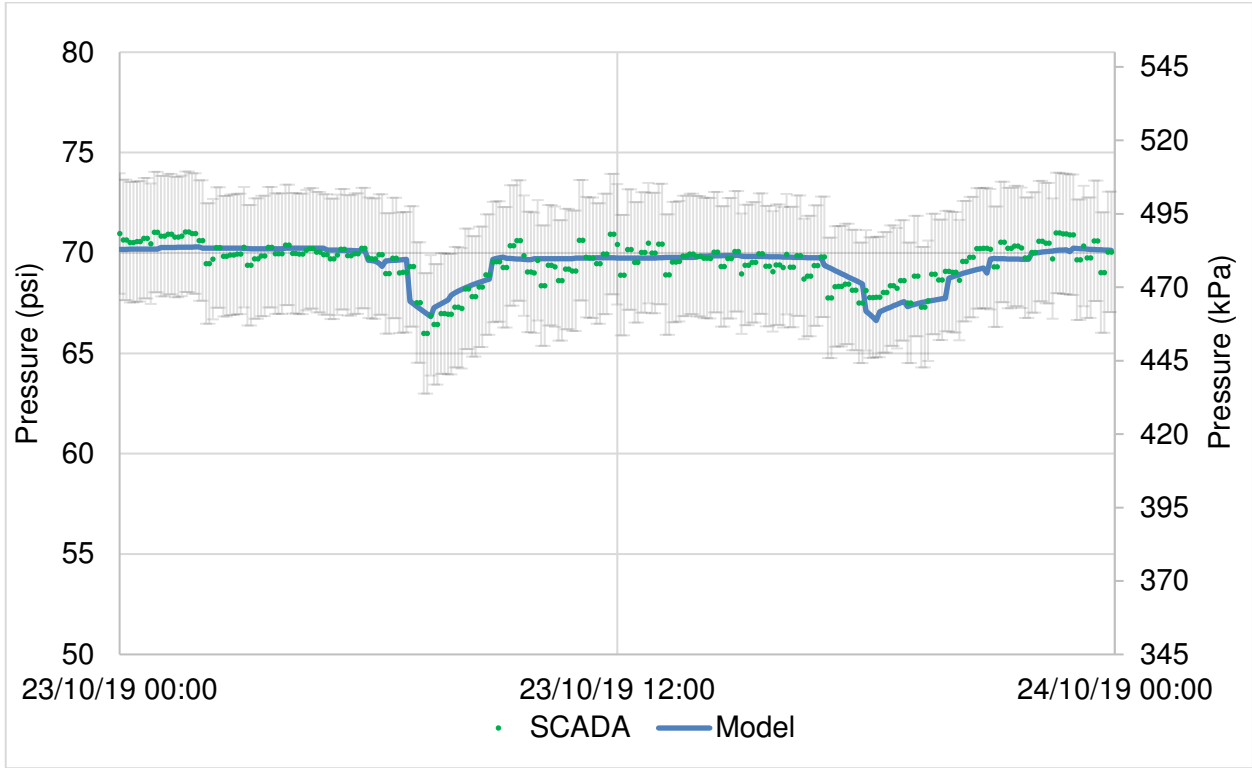




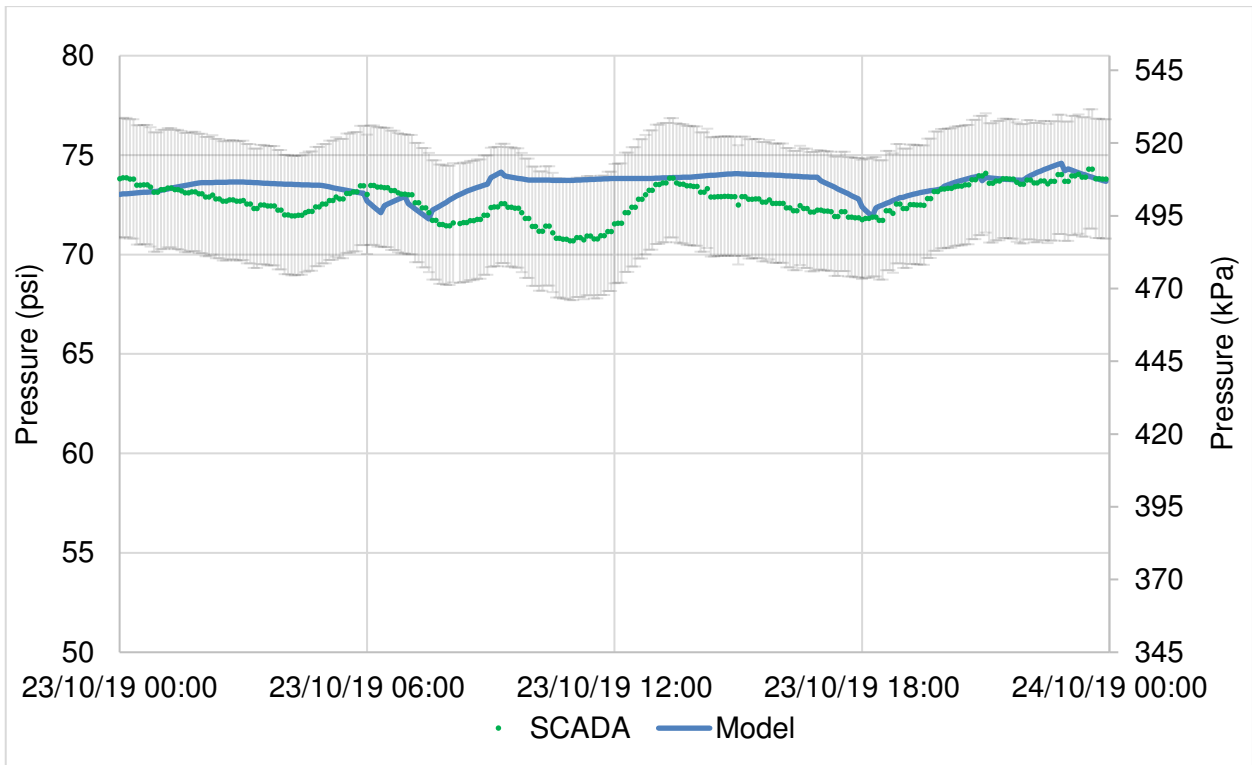
**Figure 3-21 DMA Chamber 10-3 (Zone 1: Harvard Rd) Pressure – ADD 2019 – Dynamic Controls**



**Figure 3-22 DMA Chamber 11-1 (Zone 1: Clairfields Rd) Pressure – ADD 2019 – Dynamic Controls**



**Figure 3-23 DMA Chamber 5-2 (Zone 2: Fleming Rd) Pressure – ADD 2019 – Dynamic Controls**



**Figure 3-24 DMA Chamber 14-1 (Zone 2: Willow Rd) Pressure – ADD 2019 – Dynamic Controls**

A comparison of the DMA chamber SCADA versus model pressure results at each time step for each of the three (3) scenarios modelled is provided in Table 3-8 to Table 3-10 below.

Under the ADD scenario with dynamic controls, the model pressures were primarily within 14 kPa of the SCADA data. The pressure difference only briefly exceeded the AWWA standard of 30 kPa. The only DMA chamber that had a difference of greater than 30 kPa for more than 5% of the day was 26-2 located in Zone 1 near York Road and Victoria Road.

**Table 3-8 DMA Pressure Model to SCADA Comparison – ADD 2019 – Dynamic Controls**

Pressure Difference	Zone 1	Zone 2
< 14 kPa	75%	84%
14 - 30 kPa	24%	16%
> 30 kPa	1%	0%

Under the MDD scenario with dynamic controls, the pressure difference between SCADA and the model exceeded 30 kPa 6% of the time in Zone 1 and 33% of the time in Zone 2. The results were improved when time controls were used with a difference of greater than 30 kPa 5% of the time in Zone 1 and 20% in Zone 2.

**Table 3-9 DMA Pressure Model to SCADA Comparison – MDD 2020 – Dynamic Controls**

Pressure Difference	Zone 1	Zone 2
< 14 kPa	71%	61%
14 - 30 kPa	23%	18%
> 30 kPa	6%	22%

**Table 3-10 DMA Pressure Model to SCADA Comparison – MDD 2020 – Time Controls**

Pressure Difference	Zone 1	Zone 2
< 14 kPa	74%	50%
14 - 30 kPa	21%	30%
> 30 kPa	5%	20%

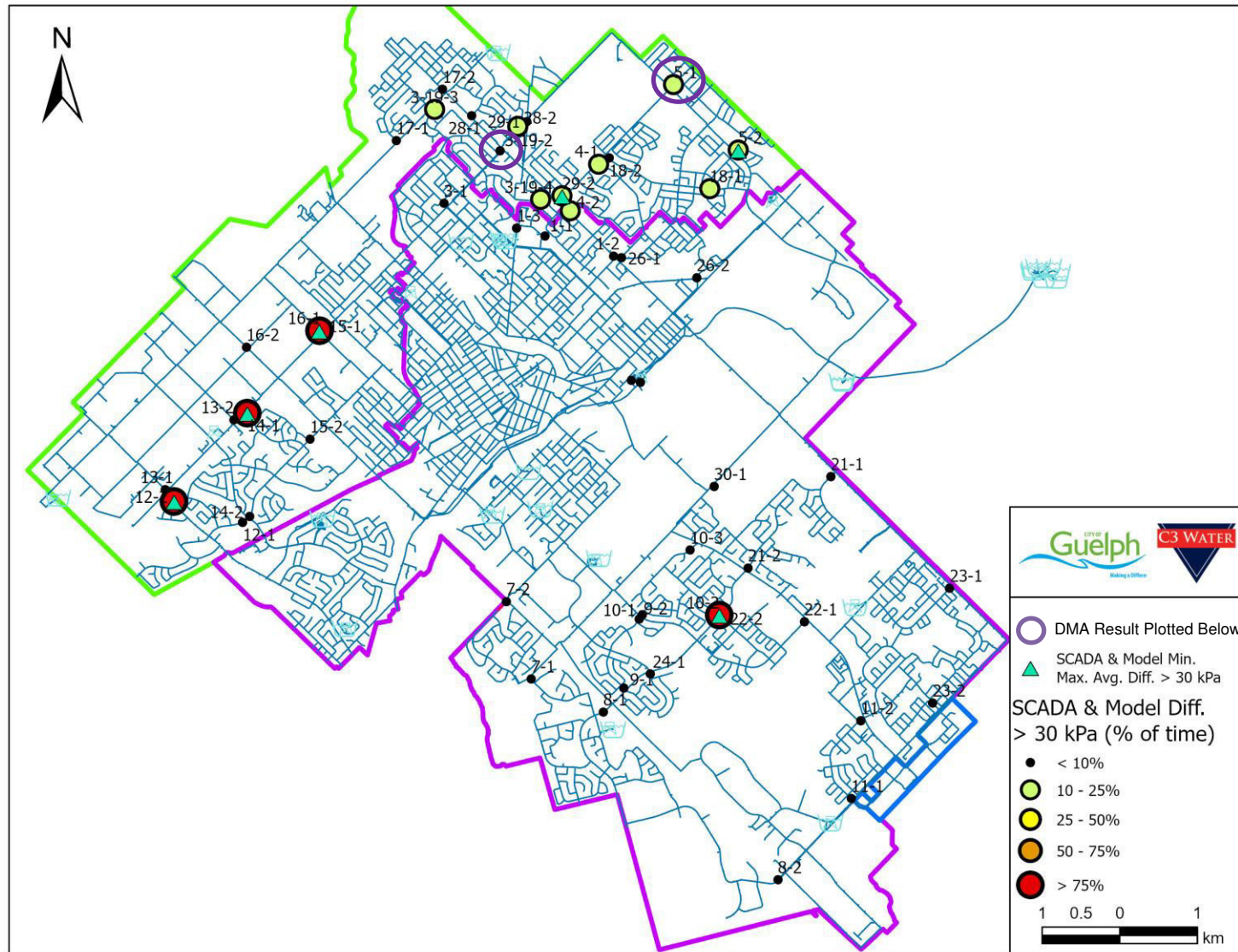
An overview of the DMA chambers where the difference between the model and SCADA exceeded 30 kPa more than 10% of the time under MDD with time controls is presented in Figure 3-25 below.

It can be seen that the chambers where the model results had a difference from SCADA of over 30 kPa more than 10% of the time were primarily located on the east side of Zone 2. There were a few points on the west side of Zone 2 and throughout Zone 1 that did not closely match SCADA for at least 75% of the time, although most are in close proximity to other DMA chambers which did match SCADA closely. This indicates an issue with the pressure monitor accuracy or an elevation discrepancy in the model. For example, at DMA chamber 14-1, the model pressure was consistently more than 30 kPa lower than what was recorded at SCADA. At DMA 13-2, located less than 200m away from 14-1, the model pressure was consistently within 14 kPa of the SCADA data.

For the MDD time controls scenario, a comparison was completed of the overall daily minimum, average and maximum pressure at each DMA chamber. The minimum, average or maximum model and SCADA pressure had a difference of greater than 30 kPa at the following DMA chambers which are shown as triangles in Figure 3-25 below:

- Zone 1:
  - 10-2
- Zone 2:
  - 5-2
  - 12-2
  - 14-1
  - 16-1
  - 29-2

In Figure 3-24 below, the triangles represent the overall daily difference between the model and SCADA at each chamber whereas the circles represent the difference between the model and SCADA at each 5-minute time-step through the simulation.



**Figure 3-25 DMA Chamber Pressure – Model & SCADA Difference > 30 kPa – MDD 2020 Time Controls**

As discussed, chambers where the model results had a difference from SCADA of over 30 kPa more than 10% of the time were primarily located on the east side of Zone 2. Select DMA chambers on the east side of Zone 2 under MDD with time controls are presented in Figure 3-26 and Figure 3-27 below. These locations are circled in purple in Figure 3-25 above. The model pressures followed SCADA fairly closely for the majority of the day. The model pressure was higher than SCADA for a period of about three hours in the evening. This pressure spike correlated with when a second pump at Clythe turned on. This indicates that the second pump turning on had more of an impact on the east Zone 2 pressures in the model than it did in reality. This may be a result of a high demand on the east side of Zone 2 in the evening, preventing the system pressure from increasing during the increase in discharge flow at Clythe. A better understanding of water usage trends may help this alignment of system pressure through DMA testing.

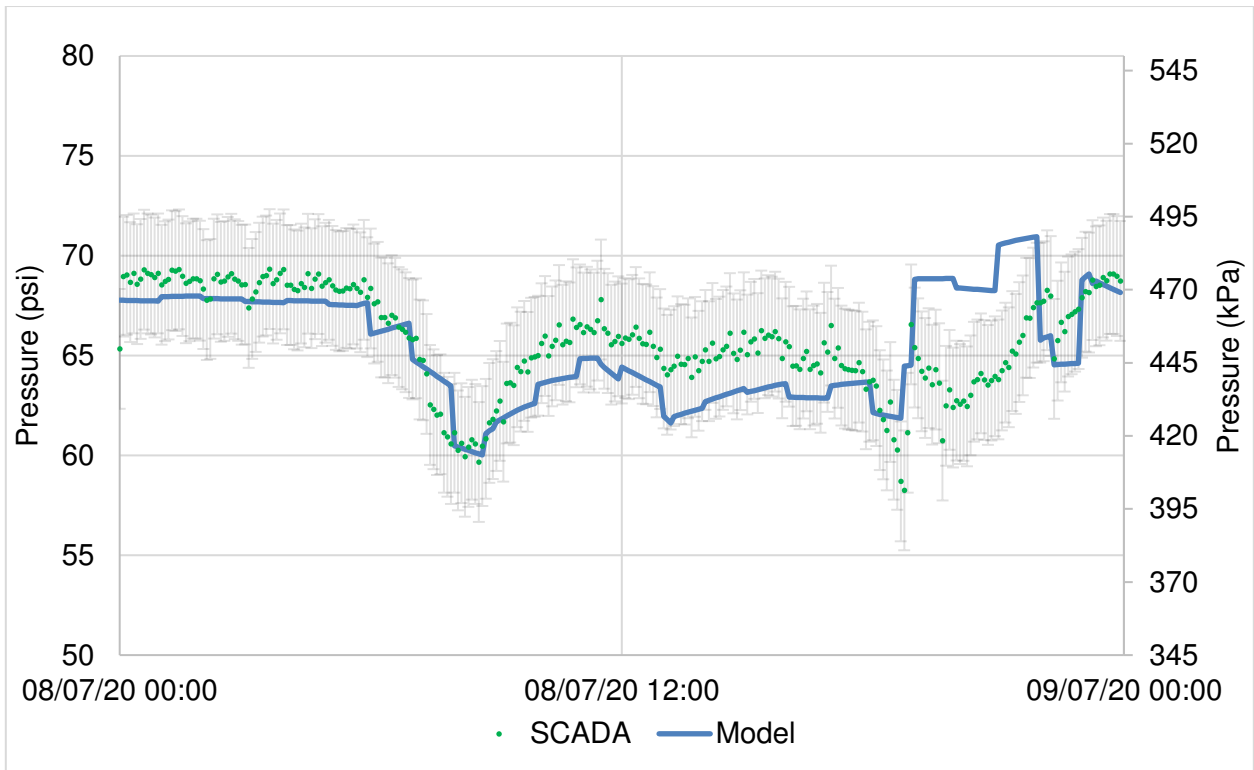
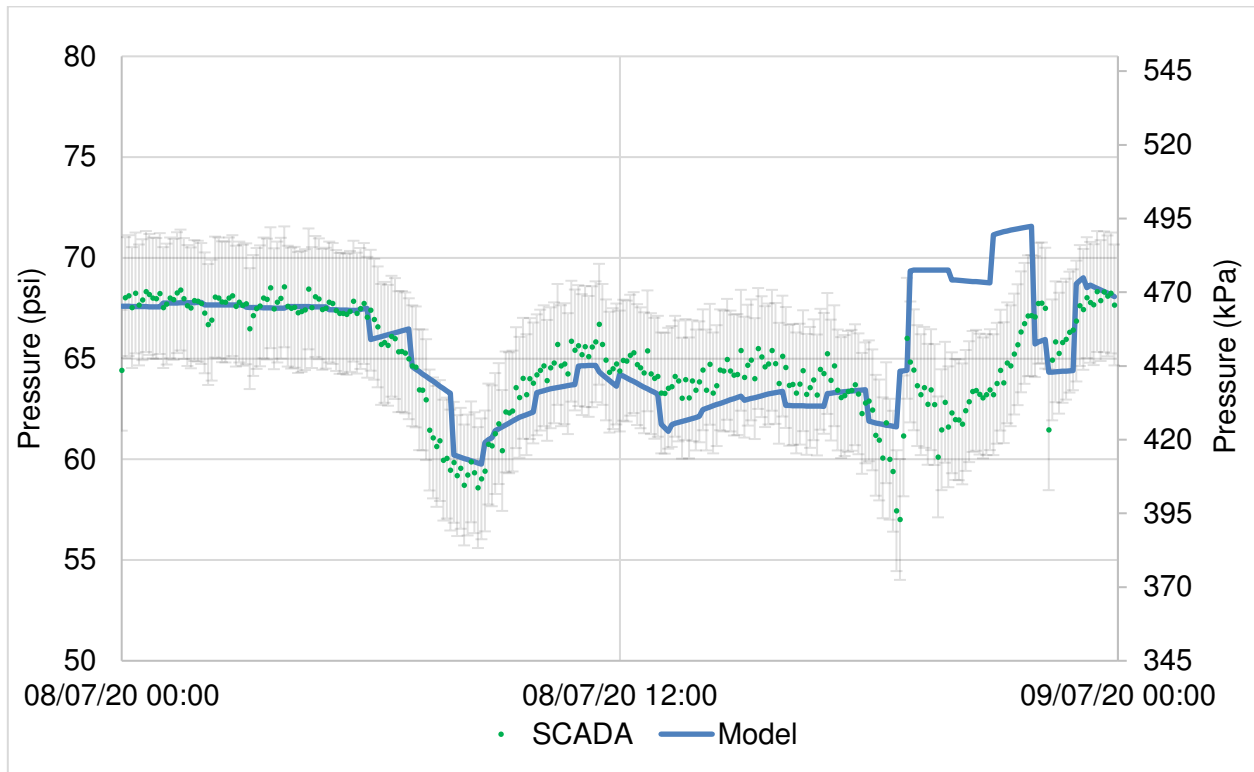


Figure 3-26 DMA Chamber 3-19-2 (Zone 2: Speedvale Ave & Victoria St) Pressure – MDD 2020 – Time Controls



**Figure 3-27 DMA Chamber 5-1 (Zone 2: Eastview Rd & Watson PW) Pressure – MDD 2020 – Time Controls**

Overall, the DMA pressure results from the model were found to match SCADA well at most locations. The DMA chambers in the model which did not closely match SCADA are assumed to be outliers, as a result of instrumentation error or elevation discrepancies. Instruments that did not match model well were found to be in close proximity to DMA chambers where the model closely match SCADA data. Additionally, the ET levels were found to match SCADA closely, indicating reasonable overall system accuracy in the model.

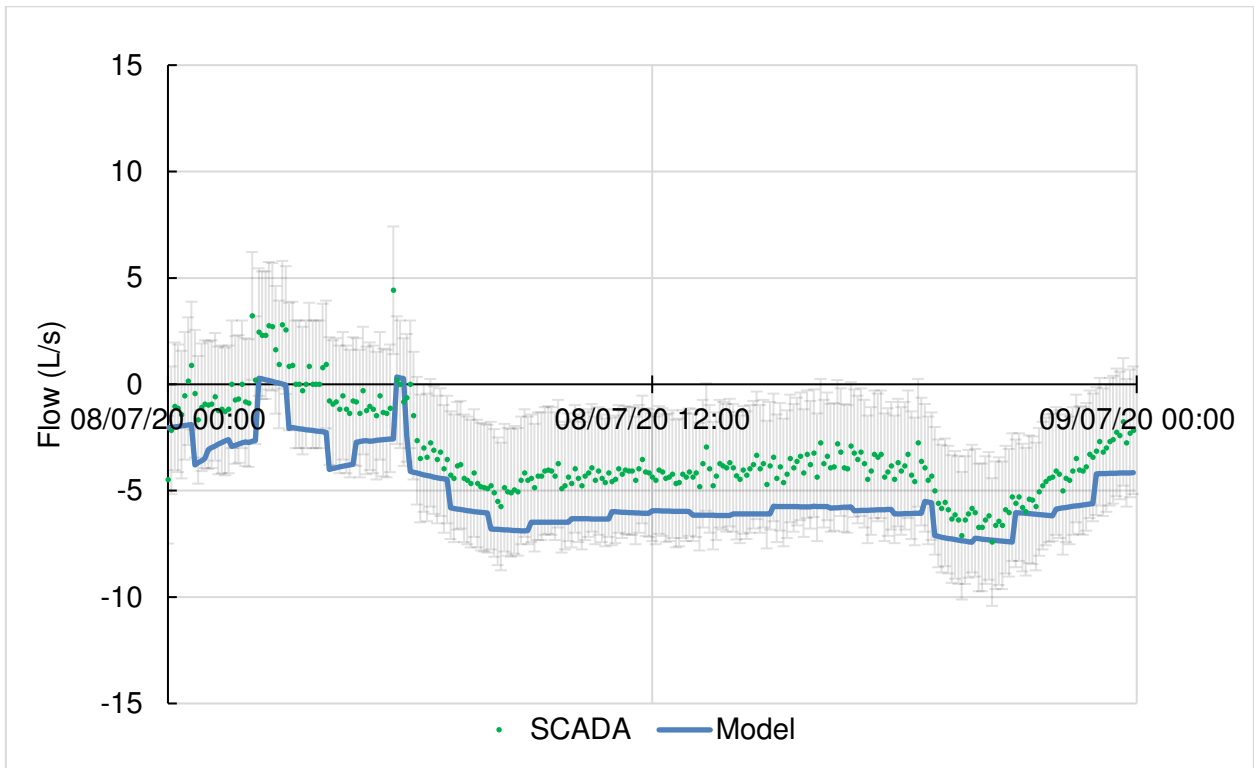
### 3.2.4 DMA Flow (Time Controls)

A comparison of the model and SCADA flow results at select DMA chambers is presented in Figure 3-28 to Figure 3-32 below under the MDD scenario with time controls.

- Chamber 1-1: The model flow generally followed the same trend as SCADA.
- Chamber 10-3: The model flow matched SCADA well. The alternating flow directions in the morning correlated with when Woods was turning on and off. The model flow slightly exceeded the flow recorded in SCADA in the evening when the Woods discharge flow was the highest.
- Chamber 11-1: The model flow followed SCADA closely. Similar to 10-3, the alternating flow in the morning at the DMA chamber corresponded with the Woods PS turning on and off.
- Chamber 5-2: At this location, the model and SCADA flow followed the same trend and were both relatively low, not exceeding 5 L/s.
- Chamber 14-1: The model flow was slightly higher than SCADA in the afternoon but generally followed the same trend.

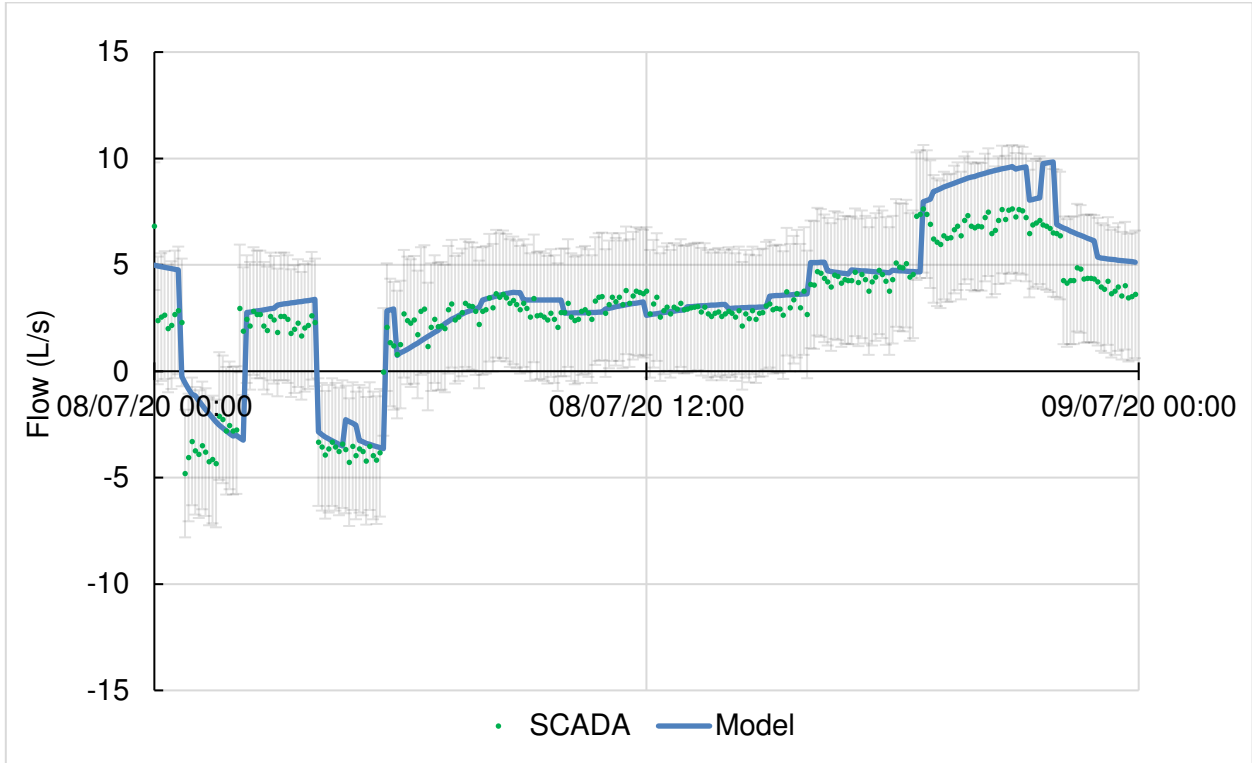
Minor differences between the model results and SCADA are likely a result of model demands. Demands are spatially allocated based on the 2019 total annual billing meter records, and diurnal patterns are

developed for each pressure Zone. The distribution of water usage throughout each Zone likely differs slightly on a day-to-day basis, causing differences in flow pathways throughout the system. Overall, the DMA flow results were found to match SCADA well.

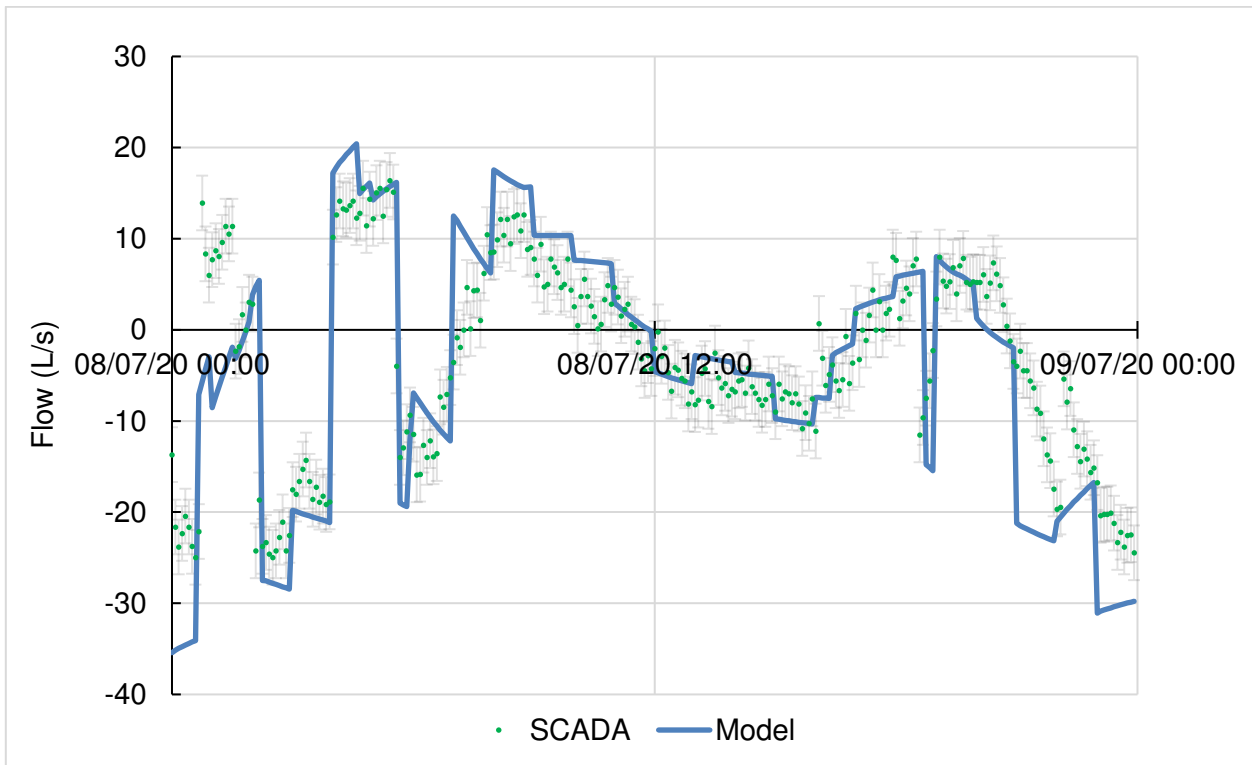


**Figure 3-28 DMA Chamber 1-1 (Zone 1: Meyer Dr) Flow – MDD 2019 – Time Controls**

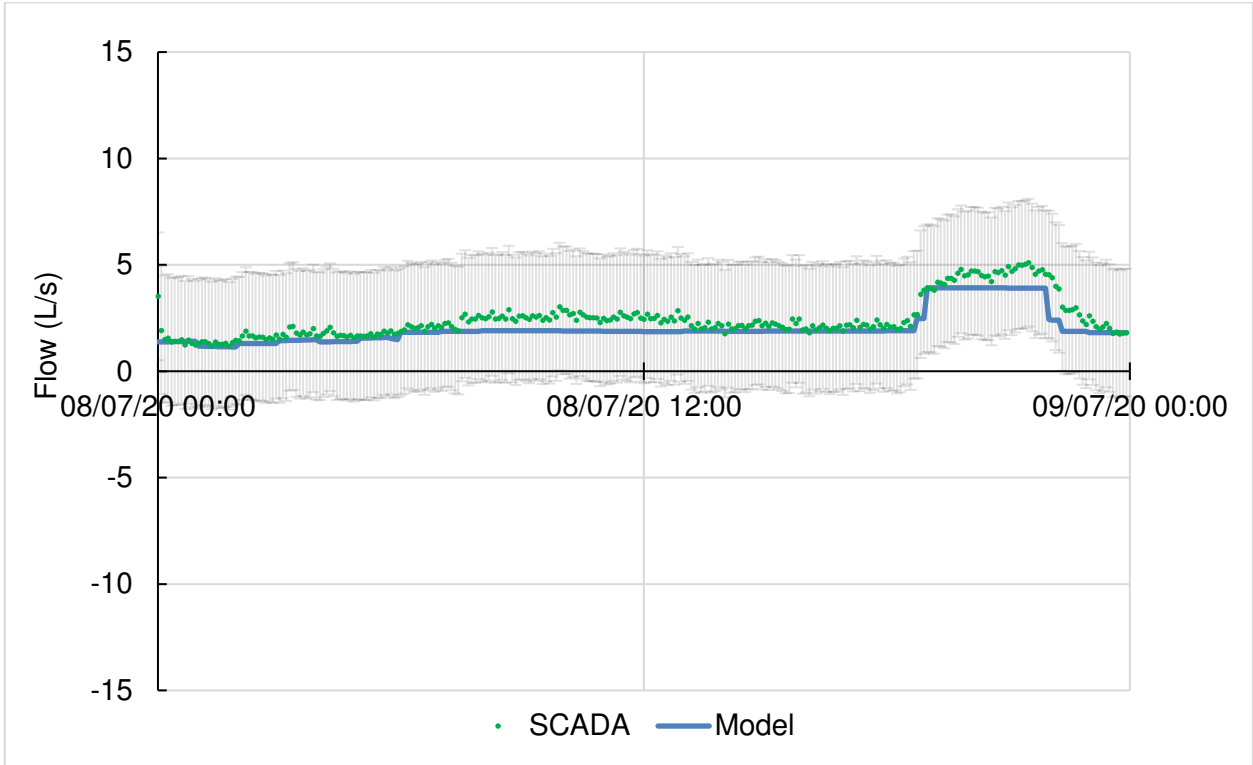




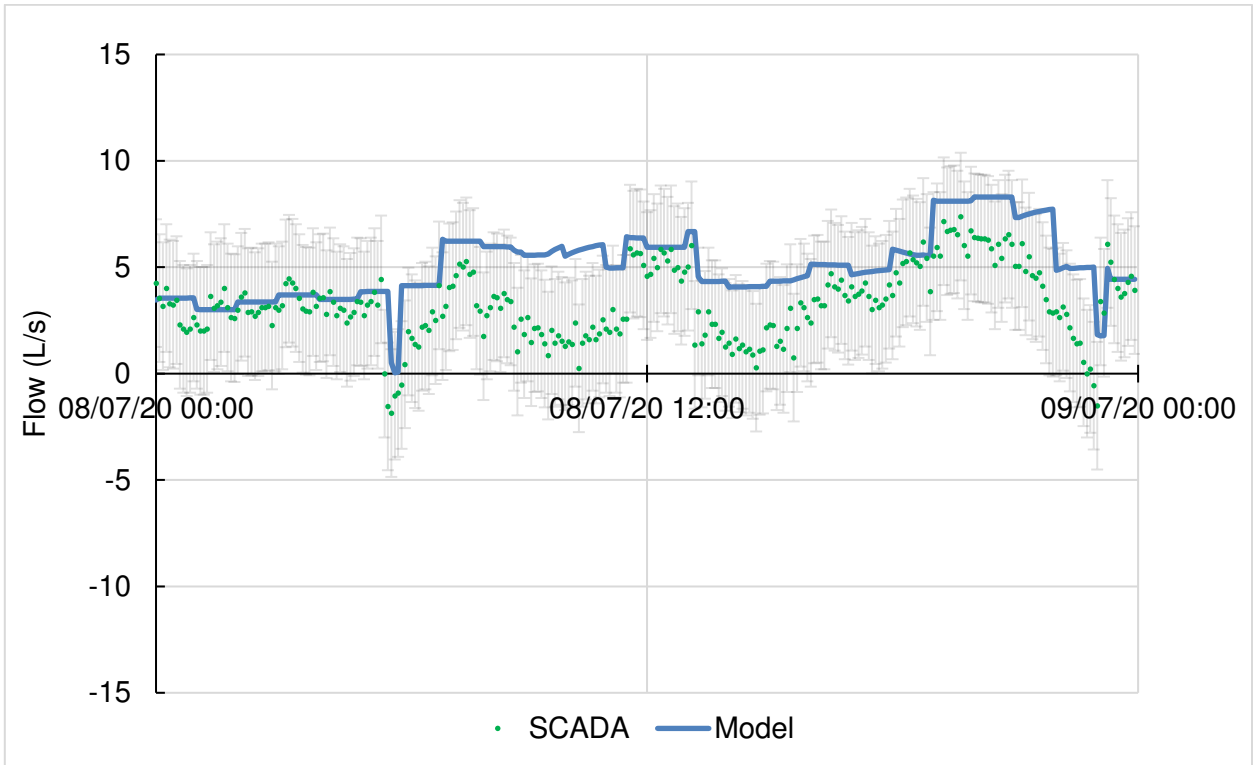
**Figure 3-29 DMA Chamber 10-3 (Zone 1: Harvard Rd) Flow – MDD 2019 – Time Controls**



**Figure 3-30 DMA Chamber 11-1 (Zone 1: Clairfields Rd) Flow – MDD 2019 – Time Controls**



**Figure 3-31 DMA Chamber 5-2 (Zone 2: Fleming Rd) Flow – MDD 2019 – Time Controls**



**Figure 3-32 DMA Chamber 14-1 (Zone 2: Willow Rd) Flow – MDD 2019 – Time Controls**

### 3.2.5 Pressure Data Loggers

The pressure logger data collected by Watermark and the City were compared to the model results at each hydrant location under the base 2019 ADD and MDD scenarios. The minimum, maximum and average pressure at each hydrant logger is summarized in Table 3-11 below. The average model pressure was within the 30 kPa of the field-testing data at all locations.

The difference between the model and field data was the highest at location 15 in Zone 3. The pressure in Zone 3 is influenced by the Clair PS. The SCADA discharge pressure at the Clair PS during the field testing period was compared to the model results and was found to match closely. The lower pressure at the hydrant in the model compared to the field data may be a result a difference in elevation.

The pressure difference was also relatively high at location 12, at Eastview and Summit Ridge on the east side of Zone 2. This is a known high elevation and low-pressure area in the system. The field data indicated lower pressure in the area than was simulated in the model results.

**Table 3-11 Model Comparison to Pressure Logger Data (kPa)**

Location	Hydrant	Field Data			Model Results			Difference (Avg.)
		Min	Average	Max	Min	Average	Max	
1	H34-034	152	272	345	260	278	312	6
2	H80-005	352	397	441	358	391	406	5
3	H42-042	538	655	689	636	658	690	3
4	H74-025	7	352	427	311	345	362	7
5	H73-069	241	374	441	336	368	386	6
6	H83-004	207	299	448	261	294	309	4
7	H51-005	179	354	786	337	360	392	6
8	H82-023	255	309	352	265	299	316	10
9	H39-057	110	288	531	261	296	324	8
10	H77-012	366	396	421	357	391	407	5
11	H62-005	387	471	522	441	470	503	0
12	H26-051	200	293	325	222	311	326	18
13	H16-025	273	358	388	276	363	379	5
14	H12-020	424	451	464	443	464	474	14
15	H81-094	403	428	457	392	403	408	25

## 4.0 WASTEWATER MODEL UPDATE

### 4.1 Model Introduction and Background

The City has been using hydraulic modelling for system analysis to support growth capacity assurance, flood risk reduction, operational assessment, and long-term capital planning. A general history of the City's wastewater models and their development is provided in Table 4-1.

**Table 4-1 Timeline of Existing Wastewater Models**

Scope	Consultant	Software	Year Completed
<b>Wastewater Master Plan</b> <ul style="list-style-type: none"> <li>• Complete Sewer Network</li> <li>• Trunk Level Calibration (8 FMs)</li> </ul>	Earth Tech	InfoSWMM	2008
<b>Hydraulic Modeling Update for the 2013 Development Charges (DC) Study</b> <ul style="list-style-type: none"> <li>• Update of 2008 Model</li> <li>• 1,487 new pipes added</li> <li>• Complete Sewer Network</li> <li>• No Calibration Performed</li> </ul>	Aecom	InfoSWMM	2013
<b>Guelph Innovation District (East End)</b> <ul style="list-style-type: none"> <li>• Secondary Plan</li> <li>• Local Sewer Network</li> </ul>	AMEC	PCSWMM	2014
<b>Nima Trails (North End)</b> <ul style="list-style-type: none"> <li>• Local Sewer Network Subdivision</li> <li>• Localized Calibration (8 FMs)</li> </ul>	GM BluePlan	InfoSWMM	June 2017
<b>Clair-Maltby (South End)</b> <ul style="list-style-type: none"> <li>• Secondary Plan</li> <li>• Local Sewer Network</li> <li>• No Calibration Performed</li> </ul>	Wood (AMEC)	InfoWorks ICM	January 2019
<b>Downtown</b> <ul style="list-style-type: none"> <li>• Secondary Plan</li> <li>• Local Sewer Network</li> <li>• Localized Calibration (5 FMs)</li> </ul>	Cole	PCSWMM	January 2020
<b>Clair-Gordon (South End)</b> <ul style="list-style-type: none"> <li>• Local Sewer Network</li> <li>• Localized Calibration (6 FMs)</li> </ul>	Civica	VH-SWMM	June 2020

The City's initial wastewater model was developed in XPSWMM, migrating to InfoSWMM in 2008 as part of the original Water and Wastewater Master Plan by Earth Tech. In 2013, AECOM completed a Water/Wastewater Development Charges Update, including model update with future infrastructure requirements to meet the projected growth. This was not recalibrated and reuses flow parameters from the original 2008 model. Since 2013, various sub-models were built at a local scale in support of development projects, each with a different modelling methodologies, inputs, degrees of calibration, and use of software. While there is some recalibration as part of the sub-models, the events selected are not consistent between these models.

It is important to have confidence in the performance of the entire collection system when completing a Master Planning level assessment where decisions are made on major infrastructure and capital budgets. As such, a review, consolidation, asset update and recalibration to recent field monitoring data is necessary to support the objectives and outcomes of this WWSMP update.

## **4.2 Required Model Updates**

The following section provides details on the approach used to update the wastewater hydraulic model.

### 4.2.1 Infrastructure Validation

The City's wastewater sewer system is comprised of maintenance holes, gravity sewer pipes, in-line storage pipes, forcemains, siphons, and pump stations and their wet wells. Information regarding these were reviewed and validated for input into the model.

#### 4.2.1.1 GIS Asset Database

The City provided a GIS asset database of its wastewater infrastructure which is the basis of the new hydraulic model pipe network. The GIS layers of interest for the wastewater sewer system include:

- wwGravityMain
- wwMaintenanceHole
- wwNetworkStructure
- wwPressureMain

The layers contain some data gaps and connectivity issues when brought into the model environment. The data gaps were identified in TM 1 and a gap-filling exercise completed based on inference and select drawing review. An updated version of the GIS database was received on May 29, 2020, as part of the Asset Management Division's initiative to restructure and enhance the geodatabase. The base attribute data (i.e., pipe diameter or inverts) was not part of the update, therefore the original gaps remain. However, the update did include the addition of recent capital projects (both built and planned). The model gap-filling exercise was updated within the hydraulic model, provided in tabular format in Appendix B.

#### 4.2.1.2 Model Build

The wastewater sewer system was established using the updated GIS assets (May 29, 2020). This ensures that the modeled infrastructure is properly identified and consistent with the City's GIS unique ID. This helps maintain a direct link between the hydraulic model and the asset database, and allows for tabular joins using GIS tools. A Model Assessment and Software Recommendation (TM 2) was completed, with the software package PCSWMM recommended, which is as a versatile, user-friendly, local platform that aligns with the stormwater Master Plan software. The City's decision to accept this recommendation remains, however the model build task has proceeded using the recommended software. Should a different software be preferred, the model would need to be transferred to this other software.

Only the necessary fields for each type of infrastructure were imported into PCSWMM, as presented in Table 4-2. Since the original GIS IDs are being used in the model, each model element can be traced back to the GIS asset database allowing a direct link for future model updates and communicating model results.

**Table 4-2 Imported GIS Layers for Model Build**

Wastewater Infrastructure	GIS Layer Source	PCSWMM Layer	Imported GIS field	Equivalent PCSWMM field
Maintenance Holes	wwMaintenanceHole	Junctions	WWMHID	NAME
			STATUS	DESCRIPTION
			RIMELEV	RIM ELEV. (m)
Wet Wells & Pump Stations Locations	wwNetworkStructure	Storages	NAME	NAME
			INVERT	INVERT
			DEPTH	DEPTH
Gravity Sewer Pipes	wwGravityMain	Conduits	WWGMAINID	NAME
			STATUS	DESCRIPTION
			DIAMETER	GEOM 1 (m)
			UPINVERT	INLET ELEV. (m)
			DOWNINVERT	OUTLET ELEV. (m)
			FROMMH	INLET NODE
			TOMH	OUTLET NODE
Forcemains and Siphons	wwPressureMain	Conduits	WWGMAINID	NAME
			STATUS	DESCRIPTION
			PIPETYPE	TAG
			DIAMETER	GEOM 1 (m)
			UPINVERT	INLET ELEV. (m)
			DOWNINVERT	OUTLET ELEV. (m)
			FROMMH	INLET NODE
			TOMH	OUTLET NODE

Once imported into PCSWMM, the following engineering validation tasks were performed:

- Connectivity Tracing: Identified connectivity issues through tracing tools. Adjusted erroneous ID references and created dummy nodes where required to resolve connectivity issues
  - The naming procedure for the dummy nodes is as follows:
    - DUMMY-upstream pipe ID@ downstream pipe ID.
- Profile Confirmation: Check and correct invert/rim elevations, diameters, negative slopes, pipe lengths, etc.
  - Changes are identified in the DATA\_SOURCE field in the model
- Check and update pump curves, wet well dimensions, pump operation levels etc.
- Remove abandoned infrastructure as identified in the STATUS field

#### 4.2.1.3 Importing Existing Models

To capitalize on the advancements of previous modelling efforts, the existing calibrated sub-models and their flow parameters were reviewed and imported into the new PCSWMM Master Plan model as follows:

- South End VH-SWMM Model (Civica, 2020):
  - Clair-Gordon area, calibrated with 6 flow monitors (See Section 4.3.1.2).
- Downtown Secondary Plan Model (Cole Engineering, 2020):
  - Downtown core area, calibrated with 5 flow monitors (See Section 4.3.1.2)

The flow generation parameters for dry and wet weather were maintained.

- North End Nima Trails (GM BluePlan, 2017)
  - 8 flow meters were used to assess this area in the north part of the City.

The flow monitoring data from this project is older than that used for the current calibration effort. Data from FM20 which is in the same area was used for calibration. A validation of the model performance to the Nima Trails project is provided in Section 5.1.12.

#### 4.2.1.4 Sanitary Pump Stations

The sanitary pump stations were modeled in detail when sufficient information was available; otherwise, they were modeled as an ideal pump, whereby pump outflow is equal to inflow. Available approval documents, drawings and design reports were reviewed to populate the modeled sanitary pump station characteristics, as summarized in Table 4-3.

**Table 4-3 Sanitary Pump Stations**

Sanitary Pump Station	Address	Number of Pumps	Capacity (L/s)	Model Approach
Barton Estates	49 Robin Road	1+1	8.9 L/s	Pump Curve
Gazer Mooney	672 Speedvale Avenue East	3+1	14.9 L/s	Ideal Pump
Gordon Street	1020 Gordon Street (decommissioned)	2+1 (decommissioned)	30.8 L/s (decommissioned)	Ideal Pump. (Removed in existing conditions model but included for calibration.)
Kortright Heights	1005 Victoria Road South	2+1	130.6 L/s	Pump Curve
Landfill Site on Eastview	186 Eastview Road	3 stations comprised of 2 (1+1) pumps each	19.6 L/s from annual pump data	Ideal Pump. Constant flow loaded at 19.6 L/s. See <b>Section 4.4.3.2</b> for more details.
NiMa Trails	Shakespeare Drive	Existing: Temporary SPS  Future: 2+1	Existing: Temporary SPS of unknown capacity  Future: 26 L/s	Existing conditions: Ideal Pump

Sanitary Pump Station	Address	Number of Pumps	Capacity (L/s)	Model Approach
				Future: Pump Curve per the provided model
Northern Heights	68 Ingram Drive	1+1	33.0 L/s	Pump Curve
Terraview	51 Terraview Crescent	1+1	13.0 L/s	Ideal Pump
Rockwood	Valley Road	2	33.0L/s	Conservatively included as constant 33L/s flow.

#### 4.2.2 Sewershed Delineation

The sanitary sewershed delineation was performed using the City’s parcel layer as a base. The parcels were then assigned appropriate outlet nodes based on the lateral connections, when available. Otherwise, the closest node was assigned. The street parcels were divided using the Thiessen polygon method (or “Voronoi decomposition” in PCSWMM). This method consists in dividing polygons based on their proximity to given points, assigning areas to the nearest point. Once the outlets assigned, PCSWMM’s upstream selection tool made it possible to identify the tributary parcels to each flow monitoring site, and assign a flow monitoring tag to each parcel, thus identifying the sewershed.

Instead of using the parcel areas, a 50m buffer around the sewer was applied to prevent overestimating tributary sewershed areas that are used to generate rainfall derived inflow and infiltration (RDII) and validate GWI infiltration flow rates. Each node representing a maintenance hole was assigned the sewershed area resulting from the 50m buffer. (See Figure 4-1).

The non-buffered area (based on the parcels) was kept in the sewershed characteristics to assess ICI full parcel areas.

The sewershed delineation is maintained in the model within PCSWMM’s “Subcatchments” layer.



**Figure 4-1 Sewershed Delineation and Buffer**

Additional characteristics were added to the parcel-based sewershed, such as land use and water consumption records. These characteristics serve as the basis to calculate the representative flow generation rates. Table 4-4 provides the subcatchment characteristics as tracked in the model.



**Table 4-4 PCSWMM Subcatchment Characteristics**

PCSWMM field name	Assigned property
<b>Default Characteristics in PCSWMM</b>	
NAME	Parcel number (GPID)
DESCRIPTION	Land use
TAG	Flow monitor it is tributary to
OUTLET	Outlet node
AREA	Non-buffered area (ha)
<b>Added Characteristics</b>	
BUFF_AREA	Buffered Area (ha)
POPULATION_2016	Population in 2016
POPULATION_2019	Population in 2019 obtained by multiplying 2016's population by a factor
STUDENTS	Number of off-campus students
STUDENTRES	Number of on-campus student-residents
H2O_2018	Water consumption of 2018 (m <sup>3</sup> )
H2O_2019	Water consumption of 2019 (m <sup>3</sup> )
TOT_GWI	Total groundwater infiltration for the whole flow monitoring site, excluding upstream monitors (L/s)
TOT_RES	Total residential flow for the whole flow monitoring site, excluding upstream monitors (L/s)
TOT_ICI	Total ICI flow for the whole flow monitoring site, excluding upstream monitors (L/s)
TOT_ADSF	Total average dry sewage flow for the whole flow monitoring site, excluding upstream monitors (L/s)
GW	Groundwater infiltration distributed to the individual catchment (L/s)
RES	Residential flow distributed to the individual catchment (L/s)
ICI	ICI flow distributed to the individual catchment (L/s)
STUDENTFLO	Sewage flow generated by students at the individual catchment (L/s)
ADSF	Average dry sewage flow distributed to the individual catchment. ADSF = RES + ICI + STUDENTFLO

#### 4.2.3 Population

Guelph's 2016 population and its distribution were obtained from Statistics Canada's census data at a dissemination block (DB) level. The population was then distributed to the residential parcels proportionately to the water consumption records. The same population distribution was maintained but multiplied by a factor to obtain the 2019 population provided by the City in September of 2020 (Table 4-5).

**Table 4-5 City of Guelph Population**

Year	Population	Source
Population 2016	131,794	Census Data
Population 2019	143,500	Email from City (September 2020)

The student population provided by the University of Guelph’s website was compared with the census data. Based on the difference, it is understood that the census does not account for the student population. To populate the campus with its students, information from the University’s website was used and is detailed in Section 4.4.3.4.

**4.2.4 Implementing Recent Capital Work Upgrades**

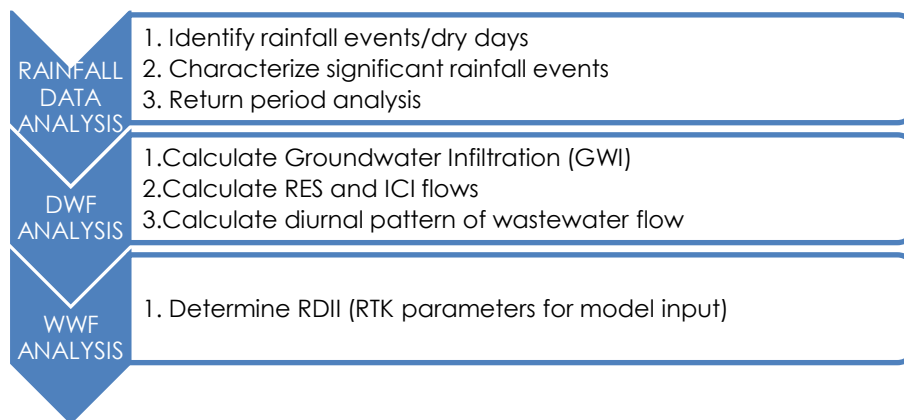
A list of recent capital work upgrades and the associated drawings were provided by the City (see Appendix C) during the model calibration process. Those upgrades that would have influenced the calibration were included. The upgrades that have been or will be introduced outside of the calibration period will be considered in the existing and future conditions assessment tasks.

The drawings were reviewed and cross-referenced with the GIS asset. The completion date of each capital work was also validated to assess the conditions of the collection system during the flow monitoring period and the differences with its existing conditions. The list also provides future upgrades that need to be considered when running future scenarios.

**4.3 Rainfall and Flow Monitoring Analysis**

The City has been operating a rainfall and flow monitoring program for several years, to support development capacity assessments, operations, and the ongoing infiltration and inflow initiatives. The most recent flow monitors were installed between December 2016 to February 2020. These monitors provide data that gives insight into the actual flow generation (dry and wet weather conditions) and the distribution in the collection system. Understanding the dry and wet weather flow generation characteristics based on an assessment of actual recorded depth, velocity and calculated flow data greatly enhances the confidence in the model results.

The general process for monitoring data analytics is presented in Figure 4-2.



**Figure 4-2 Rain & Flow Analysis Process**

### 4.3.1 Rainfall Data Analysis

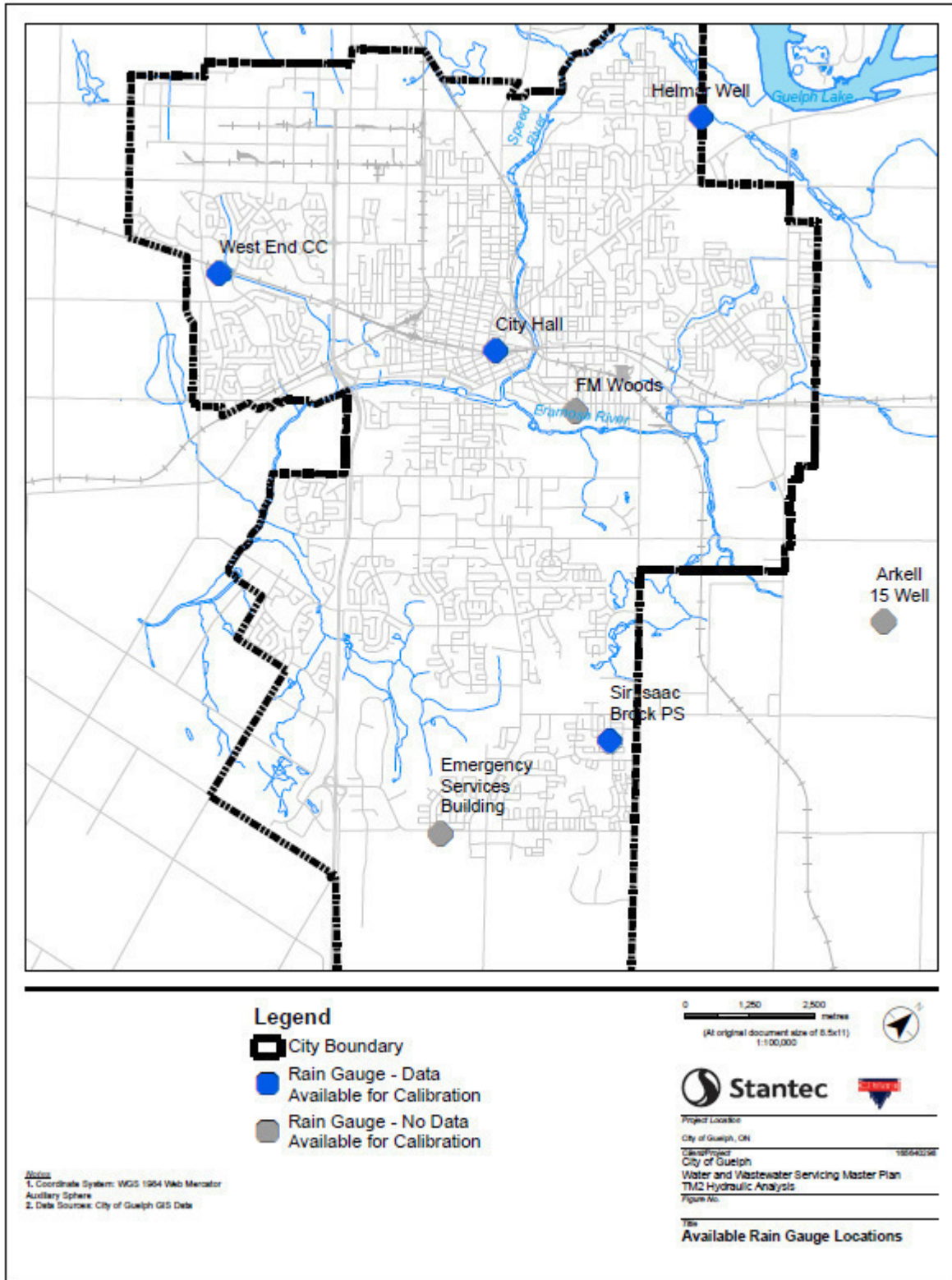
#### 4.3.1.1 Available Rainfall data

Rainfall data was provided from four rain gauges with data ranging from 2017 to 2020, as shown in Table 4-6. The location and data coverage of these rain gauges were analyzed to provide as much coverage as possible and to account for spatial distribution of rainfall.

**Table 4-6 Available Rainfall Data**

Rain Gauge	Location	Available Data
RG01	Sir Isaac Brock Public School	2017/01/19 – 2020
RG02	Guelph City Hall	2017/01/19 – 2020
Helmar Well	673, Woodlawn Road E	2019/11/01 – 2020
West End CC	West End Community Center	2019/11/01 – 2020

Figure 4-5 shows the location of existing rain gauges, highlighting the ones for which data was provided, which were used for the model recalibration.



**Figure 4-3 Available Rain Gauge Locations**

4.3.1.2 Available Flow Monitoring Data

4.3.1.2.1 Flow Monitor Selection

A total of 24 flow monitoring sites were available for the Master Plan. Among them, several have already been used for calibration in the existing sub-models. Since these sub-models have been imported into the new city-wide model, monitors covering areas that have not yet been calibrated were prioritized and selected for use in the Master Plan recalibration exercise (FM10 to FM21). The previously calibrated monitors were primarily used for validation.

The list of available flow monitors is presented at Table 4-7.

**Table 4-7 List of Flow Monitors**

Flow Monitor	Maintenance Hole ID	Sewer Pipe ID	Sewer Pipe Diameter (mm)	Description
FM01	6577	7239	250ø	2017 Clair-Gordon Calibration
FM02	5955	6535	450ø	
FM03	5955	6679	450ø	
FM04	6407	7053	375ø	
FM05	5459	5986	600ø	
FM06	8055	8852	450ø	2020 Downtown Calibration
FM06a	7537	8391	250ø	
FM07	7417	8262	375ø	
FM07a	1337	1947	300ø	
FM08	8737	9468	450ø	Selected for Master Plan
FM09	5254	5731	825ø	
FM10	7486	8335	750ø	
FM11	37	9421	675ø	
FM12	4816	5235	900ø	
FM13	3188	3463	750ø	
FM14a	3156	3442	500ø	
FM14b	3155	3429	750ø	
FM15	5419	5932	600ø	
FM16	1622	2985	600ø	
FM17	7322	8148	750ø	
FM18	1439	1687	900ø	
FM19	3203	3493	450ø	
FM20	3603	3828	825ø	
FM21	3897	4163	375ø	

The flow monitoring schematic in Figure 4-4 shows which sites are installed in series or in parallel, and which ones that have already been used for calibration by other consultants.

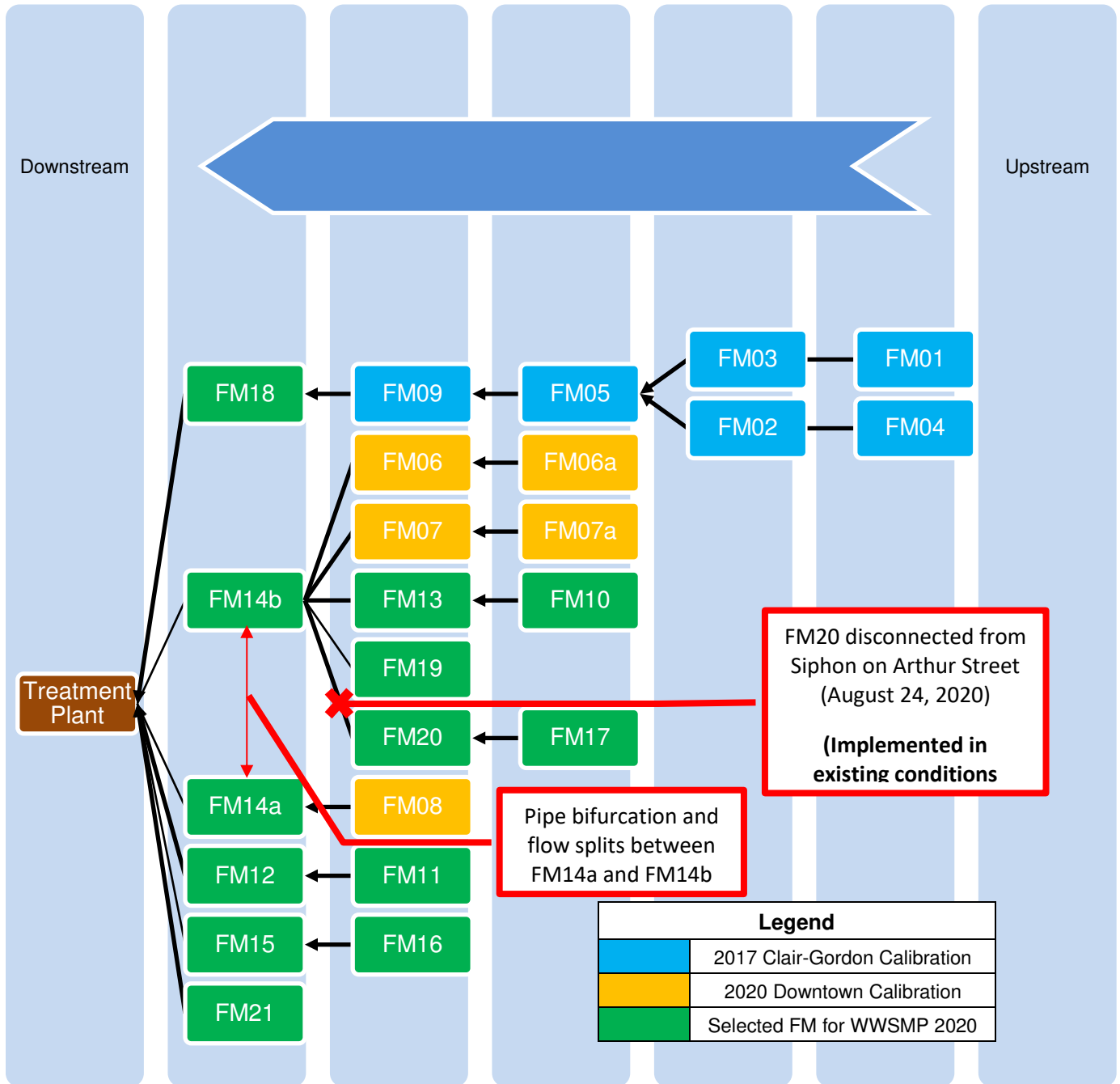


Figure 4-4 Flow Monitoring Schematic

#### 4.3.1.2.2 *Flow Monitoring Coverage*

The flow monitoring program operated by the City has made it possible to obtain data covering the majority (70%) of the area serviced by the City's wastewater collection system. The remaining areas not covered by the monitoring program are mainly of residential land use, the University of Guelph campus, and the Guelph innovation district. For the purpose of master planning, the flow monitoring coverage is considered sufficient. Future monitoring efforts could prioritize characterization of the areas that have not been monitored to date, and/or look to confirm the flows in any priority locations identified in the collection system assessment phase of this project. The spatial distribution of the flow meter coverage is provided in Figure 4-5.

#### 4.3.1.2.3 *Flow Monitoring Data Quality*

The quality of data for each site was verified to determine if any significant issues were identifiable and to understand the overall appropriateness of the use of the data for model calibration. To help visualize the data quality, velocity-depth scattergraphs were plotted and are presented in Appendix D. The velocity/depth distribution of the data (its shape, tendency, suggested roughness, etc.) can be used to identify irregularities.

One way to use the scattergraph is to compare the data distribution with theoretical values based on Manning's equation which describes the relationship between velocity, depth, slope, and pipe roughness. The monitoring data may or may not agree with the theoretical manning curves depending on the site conditions. Comments for each monitoring site are presented in Table 4-8.

Of importance, the distribution of the collected data aligning in accordance or outside of the theoretical roughness range is a consideration that may or may not be significant. The roughness of a sewer can be an indication of a temporary condition that can be flushed through seasonal rainfall or might warrant field maintenance to clean. The accuracy of velocity/depth measurement also impact the trend of the data distribution and can result in the data appearing to reside outside of expected ranges. These and other factors require consideration when assessing data and interpreting the consequential model calibration process.

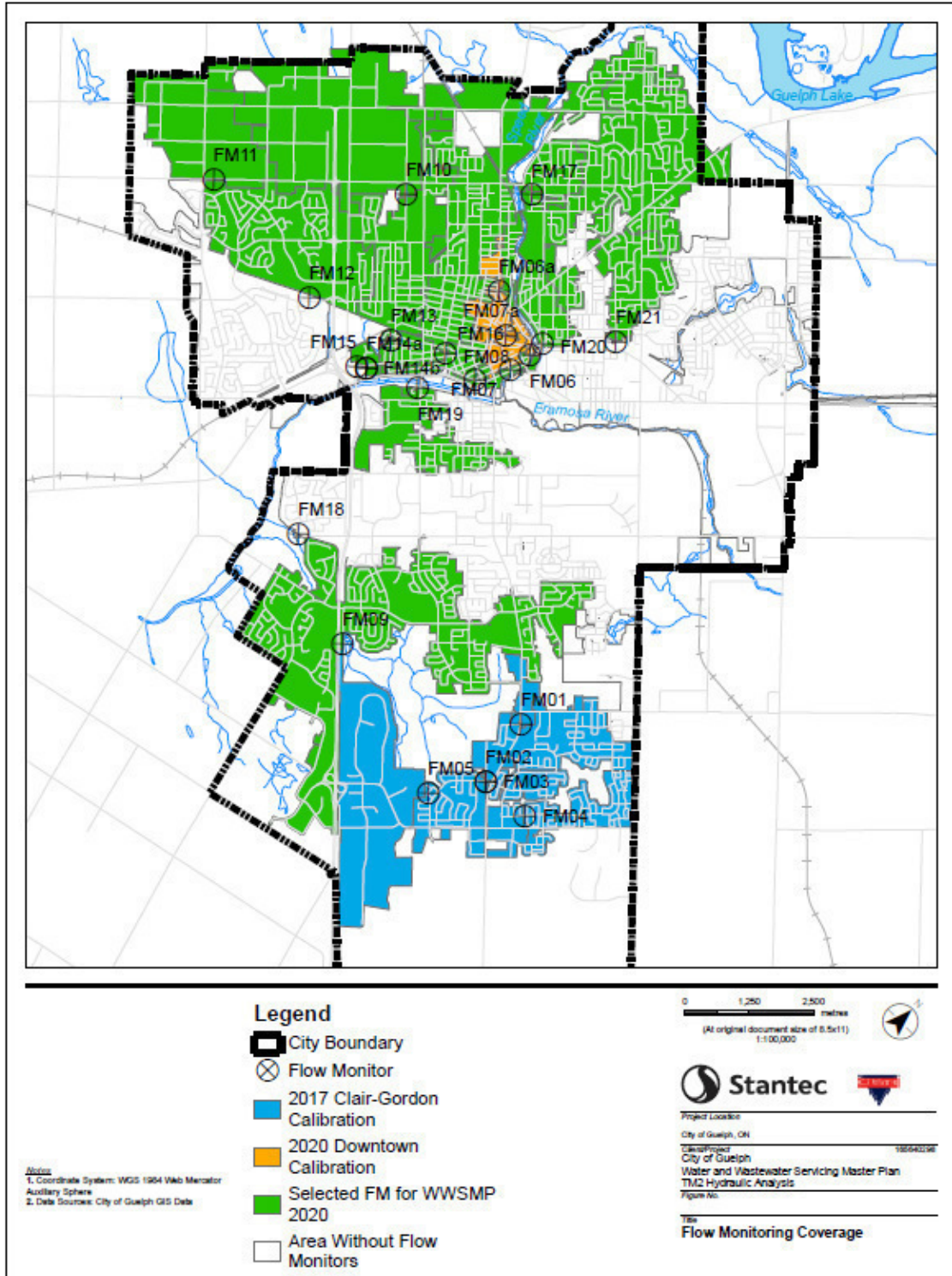


Figure 4-5 Flow Monitoring Coverage



**Table 4-8      Flow Monitoring Data Quality**

Flow Monitor	Scattergraph Observations
FM10	Fairly narrow data distribution with an apparent tendency but falls outside of expected range of Manning's equation and suggests a roughness over 0.019.
FM11	Multiple dispersed data distributions. Possibly running in a transitional flow regime.
FM12	Nice and narrow data distribution following an apparent tendency described by a Manning roughness between 0.016 and 0.019. Installed slope possibly gentler than what's provided by the GIS.
FM13	Fairly narrow data distribution with an apparent tendency but falls outside of expected range of Manning's equation and suggests an abnormally high Manning roughness (> 0.070). Possibility of sensor malfunction.
FM14a	Dispersed data distribution falling outside of expected range of Manning's equation and suggesting a roughness over 0.019.
FM14b	Nice and narrow data distribution following an apparent tendency but falls outside of expected range of Manning's equation and suggests a roughness over 0.019.
FM15	Falls outside of expected range of Manning's equation and suggests a roughness under 0.011. Possibly running in a transitional flow regime.
FM16	Nice and narrow data distribution following an apparent tendency within expected range of Manning's equation.
FM17	Two data distributions with nice and narrow shapes suggesting a Manning roughness between 0.016 and 0.019.
FM18	Falls outside of expected range of Manning's equation and suggests a roughness under 0.011. Possibly running in supercritical flow.
FM19	Falls outside of expected range of Manning's equation and suggests a roughness under 0.011. Possibly running in supercritical flow.
FM20	Fairly narrow data distribution with an apparent tendency and suggests a Manning roughness between 0.013 and 0.019. Shows evidence of backwater and surcharge.
FM21	Fairly narrow data distribution with an apparent tendency and suggests a Manning roughness between 0.011 and 0.019.

4.3.1.3 Selection of Dry Periods and Wet Weather Events

Both the rainfall and flow monitoring data were analyzed to identify potential dry weather periods and wet weather events that could be used for calibration. Figure 4-6 shows the overlap between rainfall and flow monitoring data coverage.

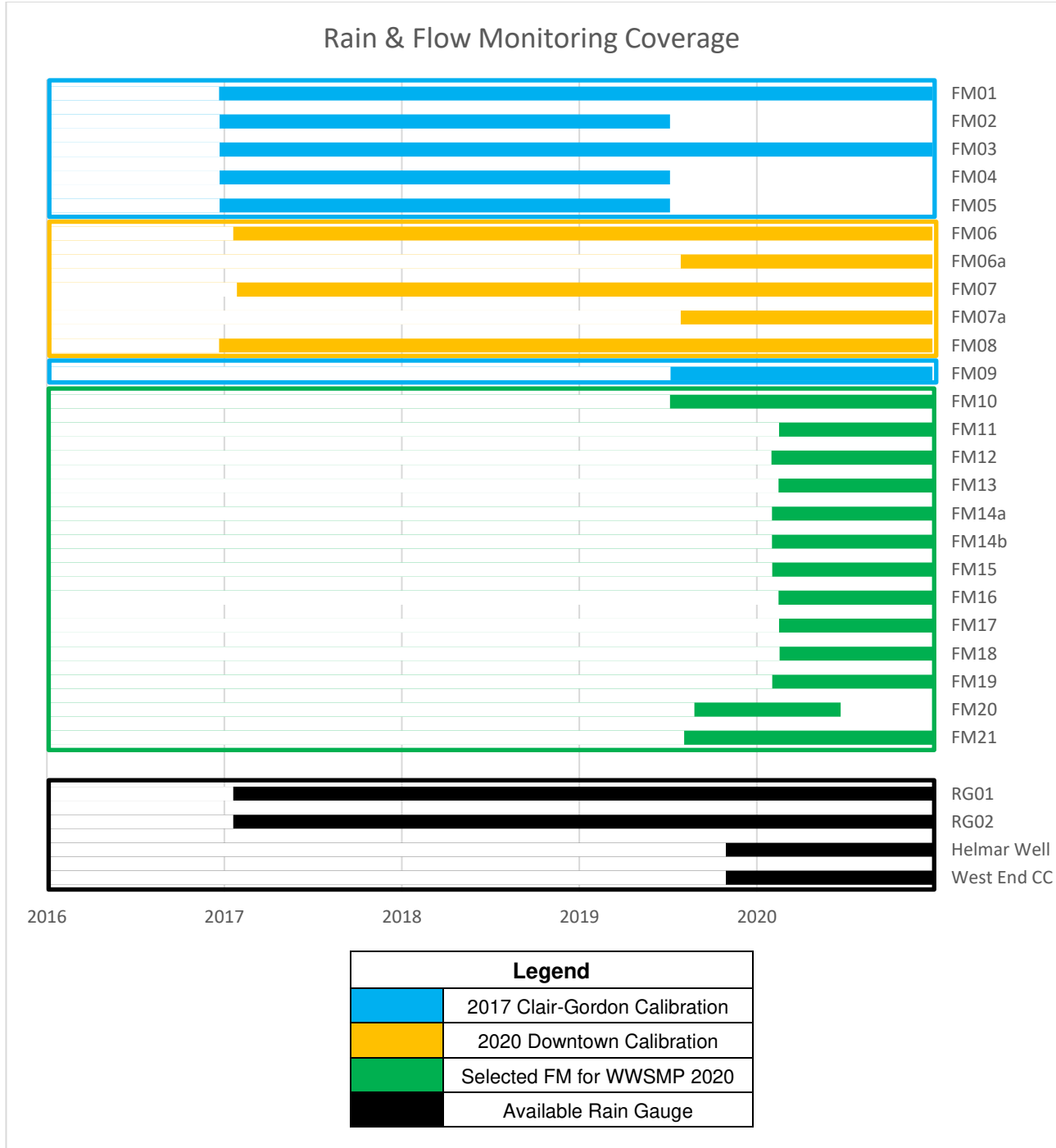


Figure 4-6 Rain & Flow Monitoring Coverage

To optimize the overlap between the rainfall data and flow monitoring data, dry weather and wet weather events between January 2020 and June 2020 were ideally chosen for calibration. Table 4-9 provides a summary of the selected rainfall event characteristics.

**Table 4-9 Identified Rainfall Events**

Event	Rain Gauge	Start	End	Duration (hrs)	Total Rainfall (mm)	Peak Intensity (mm/hr)	Return Period
January 10, 2020	RG01	Jan 10, 2020 12:05	Jan 12, 2020 04:54	40.83	88.75	21	Greater than 25 yrs
	RG02	Jan 10, 2020 16:20	Jan 12, 2020 05:41	37.33	75.75	15	Greater than 10 yrs
	Helmar Well	Jan 10, 2020 16:15	Jan 12, 2020 06:15	38.00	108.50	45	Greater than 100 yrs
	West End CC	Jan 10, 2020 23:00	Jan 12, 2020 06:10	31.17	97.75	18	Greater than 50 yrs
May 29, 2020	RG01	May 29, 2020 13:40	May 29, 2020 13:59	0.33	2.00	9	< 3 months
	RG02	May 29, 2020 13:25	May 29, 2020 13:59	0.58	13.25	81	Greater than 9 months
	Helmar Well	May 29, 2020 13:25	May 29, 2020 14:05	0.67	15.25	75	Greater than 1 yr
	West End CC	May 29, 2020 13:25	May 29, 2020 14:29	1.08	13.50	75	Greater than 9 months
June 10, 2020	RG01	Jun 10, 2020 19:10	Jun 11, 2020 05:10	10.00	34.75	69	Greater than 1 yr
	RG02	Jun 10, 2020 19:05	Jun 10, 2020 22:45	3.67	28.50	48	Greater than 1 yr
	Helmar Well	Jun 10, 2020 18:45	Jun 11, 2020 06:15	11.50	32.00	42	Greater than 1 yr
	West End CC	Jun 10, 2020 18:50	Jun 11, 2020 04:45	9.92	28.00	51	Greater than 9 months

Event	Rain Gauge	Start	End	Duration (hrs)	Total Rainfall (mm)	Peak Intensity (mm/hr)	Return Period
July 10, 2020	RG01	Jul 10, 2020 18:50	Jul 11, 2020 12:39	17.83	15.50	24	Greater than 3 months
	RG02	Jul 10, 2020 12:20	Jul 10, 2020 11:45	23.42	42.75	108	Greater than 1 yr
	Helmar Well	Jul 10, 2020 12:05	Jul 11, 2020 12:15	24.17	48.50	102	Greater than 2 yrs
	West End CC	July 10, 2020 11:55	Jul 11, 2020 11:25	23.50	34.50	126	Greater than 9 months

A dry weather period which overlaps with the selected monitoring sites' available data was identified. With over a week of no rainfall, the period between June 11, 2020 and June 19, 2020 is ideal for determining dry weather flow (DWF) characteristics. Based on the *Wastewater Planning Users Group Code of Practice for Hydraulic Modelling of Sewer Systems* (WAPUG, 2002) the dry weather flow calibration was performed with a 2-day period furthest from the influence of a preceding rainfall event. As such, the selected dry weather period is from June 17, 2020 to June 19, 2020.

#### 4.3.2 Influence of 2020 Global Pandemic

The year of 2020 was marked by an unprecedented global pandemic which had many people working from home. Given that the available data is primarily from 2020, it is unclear how representative the established DWF and diurnal patterns are when comparing them to a normal period. The possibility of the work from home practices being maintained post-pandemic also makes it difficult to predict how the City's dry weather wastewater generation might vary in the upcoming years. A reassessment and update of DWF may be of interest as a result. The year 2020 was nevertheless retained for the DWF analysis, as it provided the greatest flow monitoring data coverage, as previously shown in Figure 4-6.

#### 4.3.3 DWF Analysis

Having selected a dry weather period, the flow monitoring data was analyzed to determine DWF characteristics for each site. DWF is comprised of two main components including groundwater infiltration (GWI) and average dry weather sewage flow (ADSF) generated by residents (RES), and industrial, commercial, and institutional (ICI) land use.

$$DWF = GWI + ADSF$$

The purpose of the DWF analysis is to establish these components, determine if they are within expected ranges, and to establish the diurnal pattern characteristics for each monitor.

##### 4.3.3.1 Groundwater Infiltration

Groundwater infiltration represents the flow resulting from groundwater leaking into the system through pipe joints, broken pipes, etc, during dry weather (i.e., not rainfall induced). This component can be evaluated using the Stevens-Schutzbach's empirical equation as follows:

$$GWI = \frac{0.4(MDF)}{1 - 0.6 \left( \frac{MDF}{ADF} \right)^{ADF^{0.7}}}$$

Where MDF is the minimum daily flow and ADF is average daily flow (base equation in million gallons per day units).

The procedure to calculate the GWI for each monitoring site is as follows:

1. Establish MDF and ADF from flow monitoring data.
2. Calculate GWI with Stevens-Schutzbach's equation, for each monitoring site.
3. Determine resulting GWI L/s/ha rates based on the 50m buffer sewershed area.
4. Validate rates against typical values and land use.
5. Adjust GWI, if required.

4.3.3.2 Average Dry Weather Sewage Flow

Having evaluated GWI, ADSF can be determined by subtracting GWI from the observed DWF hydrograph.

$$ADSF = DWF - GWI$$

The next step is then to distribute the resulting sewage flow to either a residential or ICI generation. This is achieved by using both water consumption records and land use information.

$$ADSF = RES + ICI$$

The general steps to determining ADSF, RES and ICI for each monitoring site are as follows:

1. Calculate ADSF by subtracting GWI from DWF.
2. Distribute ADSF to RES and ICI based on the proportions from the water consumption records.
3. Determine resulting RES per capita rate.
4. Determine ICI rate based on total ICI parcel area.
5. Validate rates with typical values.
6. Adjust RES and ICI by redistributing ADSF, if required.

4.3.3.3 Establishing Diurnal Patterns

A diurnal pattern represents the variation of peak sewage flow during the day and is associated with the City’s water usage habits, which varies by land use and neighbourhood. Figure 4-7 shows the diurnal pattern for each of the monitoring sites, and the overall City average. The patterns specific to each monitoring site were applied accordingly while the non-monitored areas were assigned the City’s overall average.

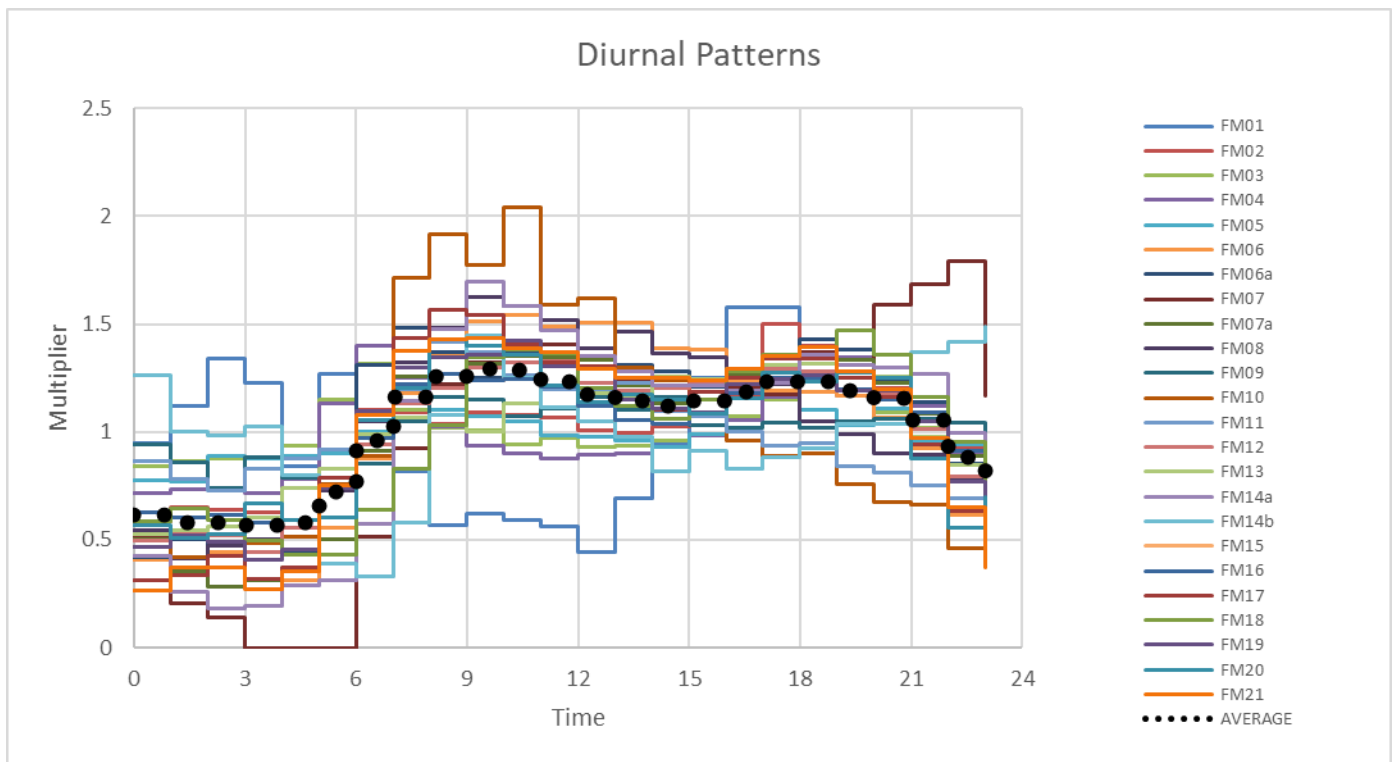
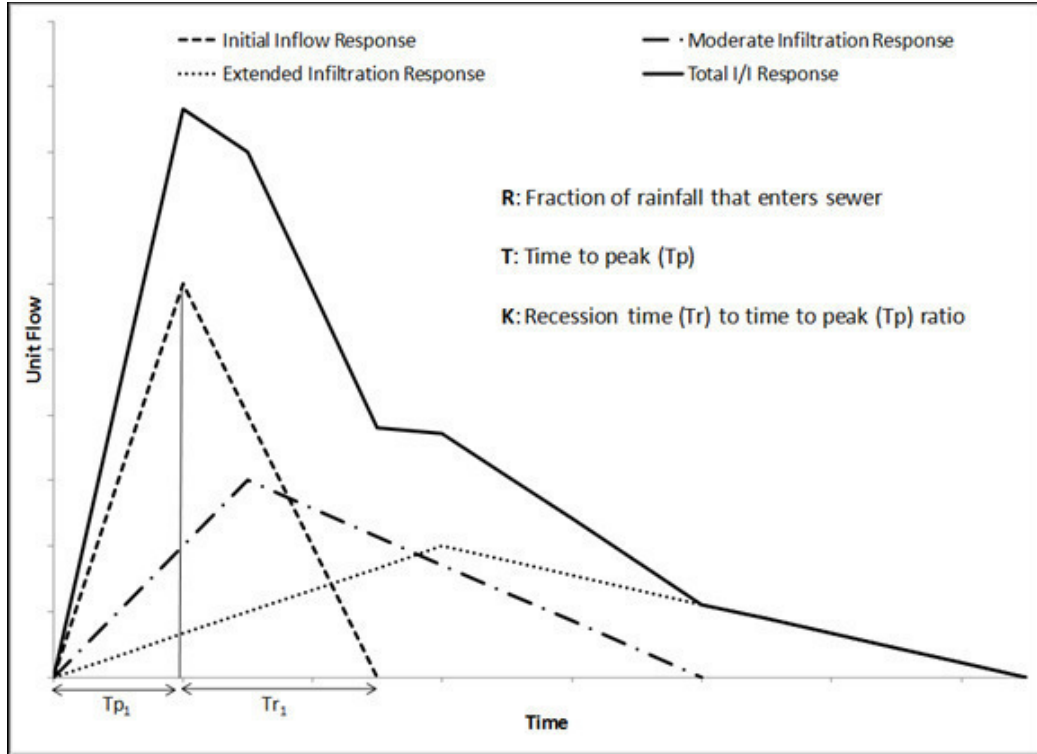


Figure 4-7 Diurnal Patterns

#### 4.3.4 WWF Analysis

The purpose of WWF analysis is to determine the quantity of rainfall entering the sewer system and to establish the rainfall derived infiltration and inflow (RDII) parameters. The RDII methodology used to determine wet weather flow for this Master Plan is the commonly applied RTK method.



**Figure 4-8 RTK Unit Hydrograph**

The RTK method generates wet weather flow entering the sewer system by assigning a unit hydrograph obtained from combining three unit-hydrographs representing different types of characteristic responses:

- Short term response (initial inflow)
- Medium term response (moderate infiltration)
- Long term response (slow infiltration)

Each one of these unit hydrographs is comprised of three parameters, where:

- R = Portion of rainfall that enters the sewer (percentage, unitless)
- T = Time to peak of hydrograph (hours)
- K = Ratio of the recession time to time to peak (unitless).

RTK parameters can be established from the flow monitoring data using a curve-fitting approach. See Section 4.4.2.

## 4.4 Flow Generation and Loadings

Flow generation in a hydraulic model is primarily based on tributary population (existing and projected), groundwater infiltration also known as baseflow, non-residential large users (usually ICI land users), and wet weather flow resulting from RDII.

The following sections detail the methodology used to load the hydraulic model with flow and other characteristics. Overviews of the PCSWMM interface, functionalities and layer properties are provided in Figure 4-9 and Figure 4-10, for subcatchments and junctions, respectively.

### 4.4.1 Dry Weather Flow

#### 4.4.1.1 Groundwater Infiltration

GWl was first loaded into the model's "Subcatchments" layer (see Figure 4-9). It was then distributed to each flow monitoring site proportionately to its 50m buffer sewershed area. In the model, this parameter is loaded into the "Junctions" layer under the "BaseFlow" property (see Figure 4-10). This was achieved using PCSWMM's LLOOKUP function in the attribute editor.

#### 4.4.1.2 Average Dry Sewage Flow

ADSF was divided into RES and ICI generation based on the water consumption records and land use information. Since PCSWMM performs computations based on ADSF rather than RES and ICI populations, these parameters were first loaded into the "Subcatchments" for documentation purposes (see Figure 4-9).

RES flow was distributed to each flow monitoring site proportionately to its population, while ICI flow was distributed proportionately to the ICI water consumption. ADSF for each individual catchment was then obtained by summing both RES and ICI. In PCSWMM, ADSF resides in the "Junctions" layer under the "AvgValue" property (see Figure 4-10). The average dry sewage flow is then multiplied by a 24-hrs diurnal pattern to reflect the variation of peak flow throughout the day. The patterns were established for each monitoring site and assigned accordingly. For areas without flow monitoring coverage, see Section 4.4.3.5.

### 4.4.2 Wet Weather Flow

The RTK method was used to generate WWF which consists of assigning each flow monitoring site with a unit hydrograph and a sewershed area. A set of typical RTK values were initially assigned to each monitoring site for iterative calibration (see Section 5.2). The tributary sewershed area per monitor was based on the 50m buffer around the sewer as explained in Section 4.2.2.

In PCSWMM, these parameters reside in the "Junctions" layer under the "Hydrograph" and "Sewershed Area" characteristics (see Figure 4-10).



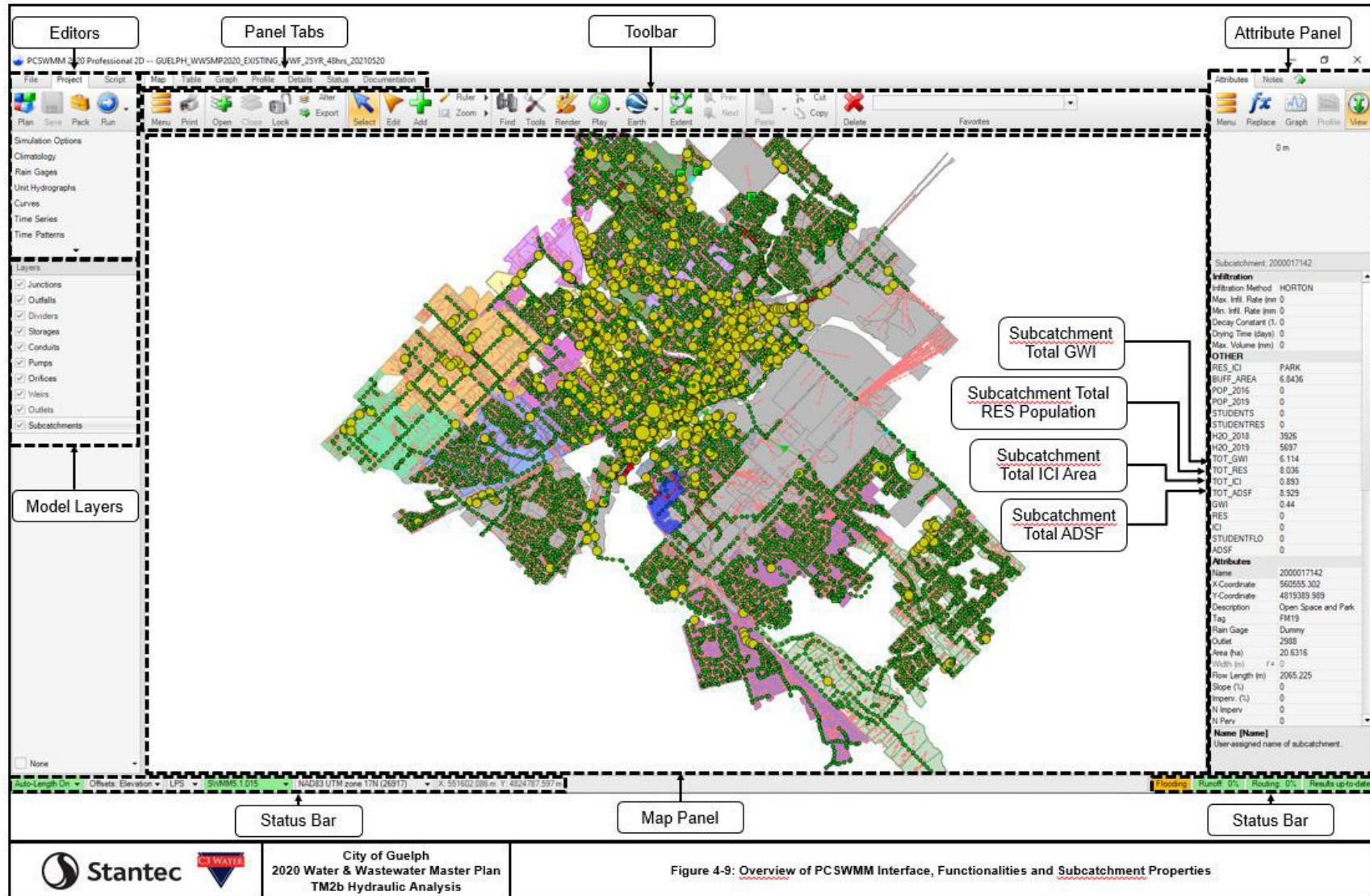


Figure 4-9 Overview of PCSWMM Interface, Functionalities and Subcatchment Properties

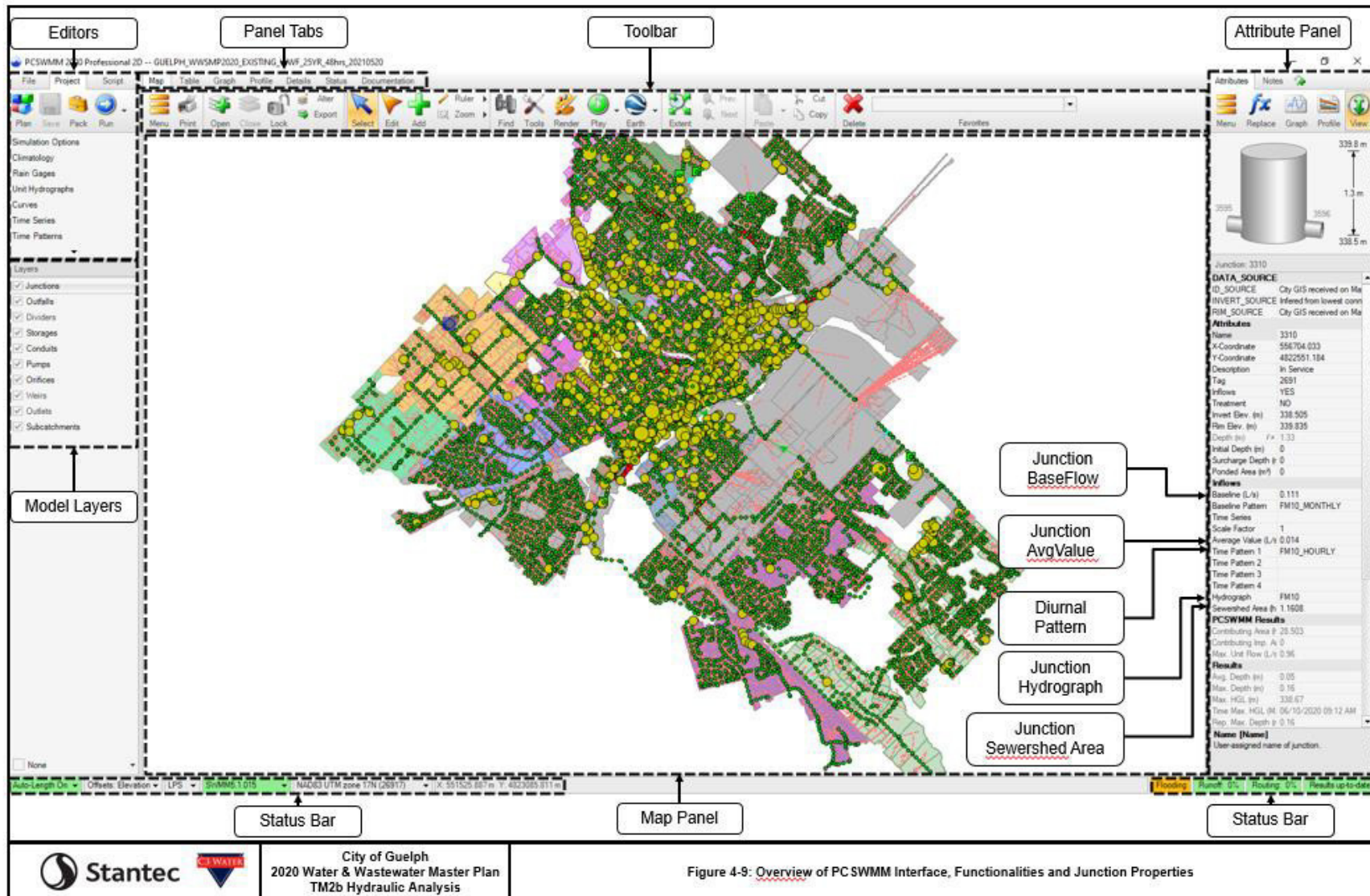


Figure 4-10 Overview of PCSWMM Interface, Functionalities and Junction Properties

#### 4.4.3 Flow from Particular Sites

The following sections detail the methodology used to load flow at particular sites in the study area.

##### 4.4.3.1 Gazer Mooney

Right outside the City boundary is a residential area composed of 75 units and is serviced by the Gazer Mooney sanitary pump station. A density of 4 people per unit was maintained per previous works and loaded to the model. This area is tributary to the flow monitoring site FM20. As such, DWF and WWF was loaded with the results obtained from the FM20 analysis and later calibrated.

##### 4.4.3.2 Landfill Site on Eastview

The landfill site includes three pump stations: Main, West, and South as discussed in Section 4.2.1.4. Each station contains two pumps. Weekly pumped volume data (provided by the City) was used to determine the average weekly flow between 2017 and 2019. It is assumed that the pumps work in an alternating sequence. The weekly average flow was calculated by dividing the recorded volumes with the pump’s total runtime. The results show that the highest recorded weekly flow was 19.6 L/s. This was conservatively loaded to the model as a constant flow. The calibration process is not influenced by this source of flow because it is located outside of the monitoring sites.

##### 4.4.3.3 Rockwood Community

Also located outside the City boundary, the Rockwood community is serviced by Guelph’s wastewater collection system. In the original WWSMP from 2008, a peak flow of 26.7 L/s was established from daily flow records. This flow was loaded into the new model as a constant flow in a dummy node named “Rockwood”. The node is located on highway 7, right outside the City boundary. Considering that the established flow comes from a 12-year-old Master Plan, it would be relevant to reassess this value.

Based on updated information provided by the City, the Rockwood pumps were upgraded to variable speed pumps, which can provide a maximum flow of 33.0 L/s. Like the landfill site, the flow from Rockwood is not tributary to any monitoring site, which means the flow generation can be updated without affecting the calibration results. This update will be reflected in the existing conditions modelling.

##### 4.4.3.4 University of Guelph

The student population for the University of Guelph was determined from the available information on their website. The mean calibrated residential flow rate obtained from the DWF calibration of 227 L/cap/d (see Table 5-1) was used for on-campus students. A flow rate of 140 L/cap/d was used for the off-campus students as suggested by the MECP guidelines (Table 4-10).

**Table 4-10 University of Guelph Population**

Type of Students	Number of Students	Flow Rate Used	Generated Flow
On-Campus Students	5,194	227 L/cap/d	13.6 L/s
Off-Campus Students	24,313	140 L/cap/d	39.4 L/s
Total	29,507	-	53.0 L/s

For WWF, a sewershed delineation was performed using a 50m buffer around the streets layer since the GIS asset provided by the City did not have the sewer network within the University area. The mean calibrated RTK parameters was used for this area. It is noted that the University did provide details of their sewer network and that our assumptions were deemed acceptable.

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#### 4.4.3.5 *Non-monitored areas*

All areas without flow monitoring were assigned the average flow rates (RES and ICI generation applied according to land use), average diurnal pattern, and average RTK parameters that resulted from the calibration results.

## 5.0 WASTEWATER MODEL CALIBRATION

The process of adjusting model parameters to correlate results with observed data is referred to as model calibration. This calibration process includes an iterative approach to bring key model results within targeted ranges.

### 5.1 Dry Weather Calibration

#### 5.1.1 Summary

DWF was iteratively adjusted in the model until a target margin of error between observed and modelled results was met. Once calibrated, the resulting flow rates were used to determine mean RES and ICI flow rates and apply them to non-monitored areas. Note the per capita rates (L/cap/d) are within expected ranges. FM15 and FM19 are considered lower and higher within that range, however. These rates are dependent on the upstream population and ICI distribution and as such are sensitive to these. Further investigation into the distribution of the upstream population and ICI distributions could be completed to understand if these are representative, however no impact on the model calibration results would be expected.

The final calibrated flow rates are presented in Table 5-1.

**Table 5-1 DWF Calibrated Flow Rates**

Flow Monitor	Metershed Characteristics				Dry Weather Flow (DWF)			DWF Rates		
	Area (ha) <sup>1</sup>	Buffered Area (ha) <sup>2</sup>	Population <sup>3</sup>	ICI Area (ha) <sup>4</sup>	GWI Flow (L/s)	RES Flow (L/s)	ICI Flow (L/s)	GWI Rate (L/s/ha) <sup>5</sup>	RES Rate (L/cap/d)	ICI Rate (L/d/ha) <sup>6</sup>
FM10	421	140	44	284	13.4	0.1	13.3	0.10	228	4,052
FM11	229	87	-	167	3.2	-	7.5	0.04	-	3,897
FM12	298	216	11,965	10	15.4	32.5	1.7	0.07	235	14,222
FM13	223	158	8,058	53	6.5	23.9	5.9	0.04	256	9,709
FM14a	32	31	1,539	-	0.9	4.0	-	0.03	223	-
FM14b	35	31	1,686	5	1.4	4.3	1.9	0.04	222	30,020
FM15	37	30	1,079	-	1.0	2.2	-	0.03	173	-
FM16	246	202	8,875	33	10.7	21.2	10.0	0.05	207	26,438
FM17	224	148	5,821	35	5.5	14.1	1.6	0.04	210	3,960
FM18	660	458	14,403	37	11.3	34.6	3.9	0.02	207	8,974
FM19	130	95	2,029	5	6.1	8.1	0.9	0.06	345	14,171
FM20	458	385	13,762	13	19.2	33.0	3.7	0.05	207	24,122
FM21	154	130	4,132	14	2.7	9.9	4.3	0.02	207	26,103
Average								0.05	227	15,114

Note 1: Excludes upstream areas covered by upstream monitors.

Note 2: Area from a 50m buffer around sewer. Excludes upstream area covered by upstream monitors.  
 Note 3: Population in 2019. Excludes upstream population covered by upstream monitors.  
 Note 4: Total area of ICI parcels. Excludes upstream area covered by upstream monitors.  
 Note 5: GWI flow rates were calculated using the buffered area.  
 Note 6: ICI rates were calculated using the ICI full parcel area.

Based on the WAPUG’s code of practice, the DWF calibration was carried out for two full dry weather days and the modeled average and peak flows were compared to the observed values. In addition to tracking the overall general shape, the flow hydrographs should meet the following criteria:

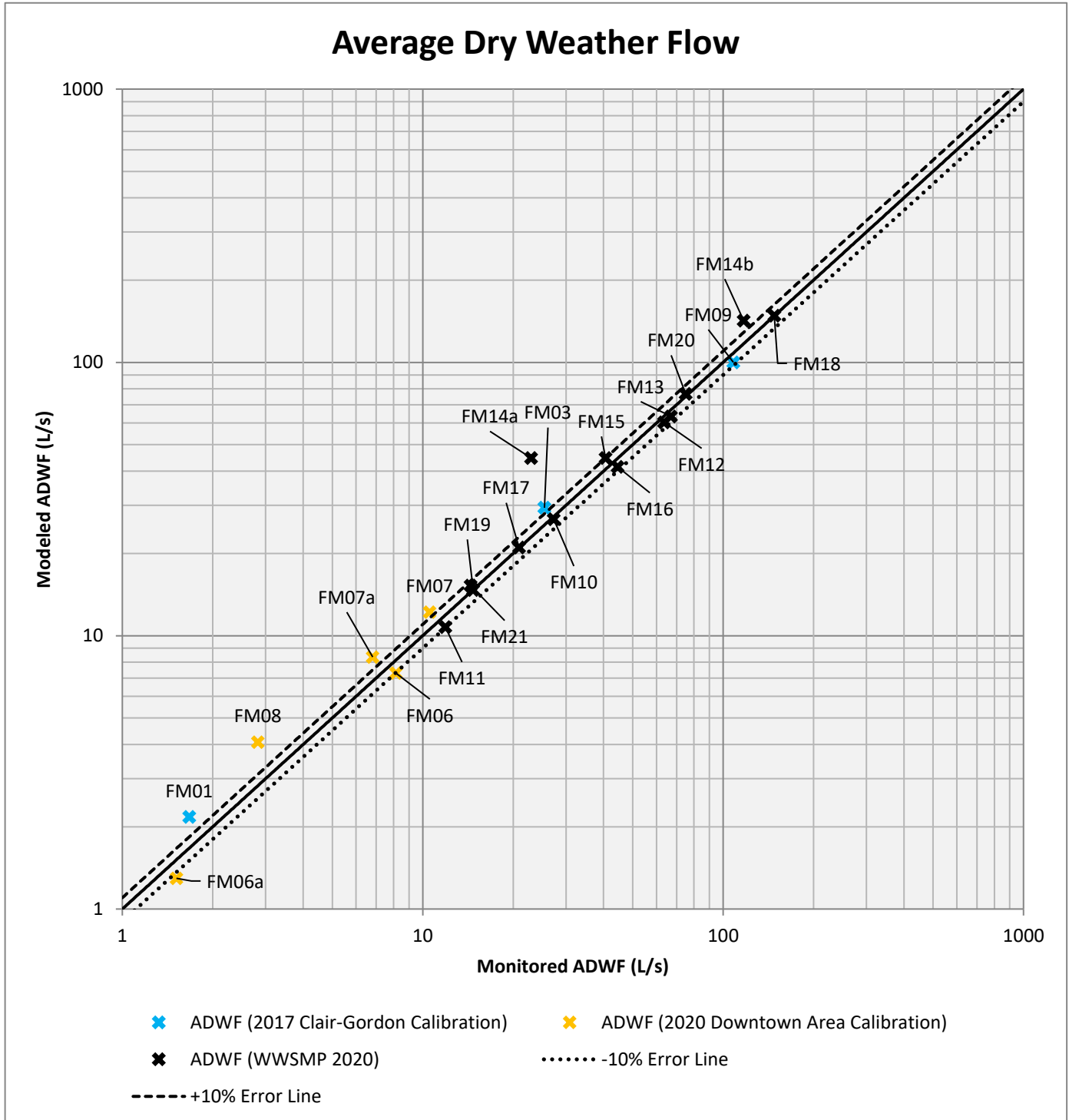
1. The alignment of the peaks and valleys of the time series should be within 1 hour.
2. The peak flows should be within  $\pm 10\%$  of each other.
3. The 48-hour volume should be within  $\pm 10\%$ . Care should be taken to exclude periods of missing or inaccurate data.

The DWF calibration results are presented in Table 5-2, Figure 5-1, and Figure 5-2 for a 48-hrs simulation run (June 17-19, 2020; see Section 4.3.1.3 for justification and implication of selected calibrated period). A logarithmic scale is used to facilitate the comparison of results with order of magnitude differences. The plot comparison of monitored and modeled flow is available in Appendix E. The results are further discussed herein for each flow monitoring site.

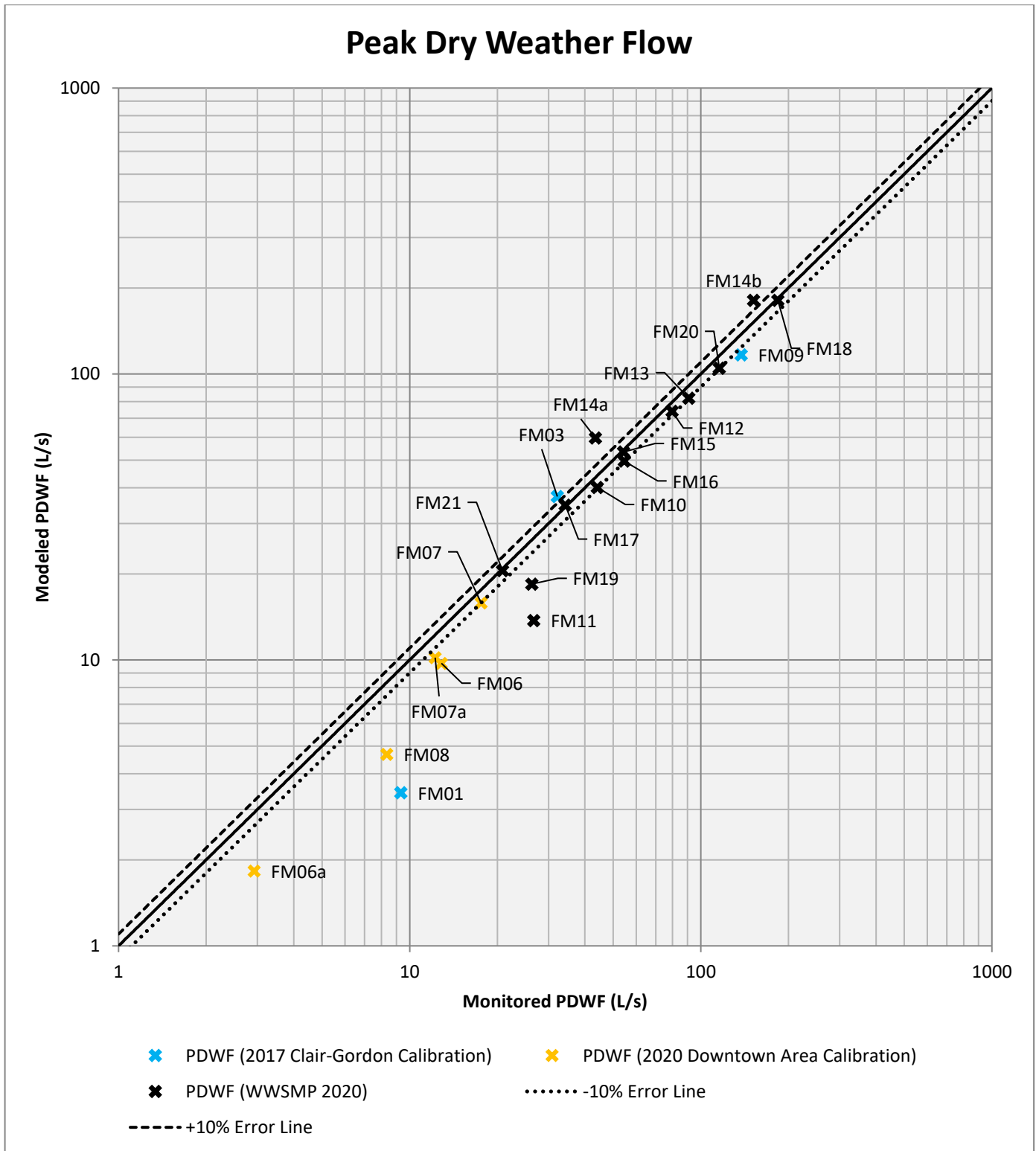
**Table 5-2 DWF Calibration Results**

Flow Monitor	Monitored (June 17-19, 2020)		Modeled (June 17-19, 2020)		% Error	
	ADWF (L/s)	PDWF (L/s)	ADWF (L/s)	PDWF (L/s)	ADWF (%)	PDWF (%)
FM10	27.3	44.2	26.7	40.1	-2%	-9%
FM11	11.9	26.7	10.8	13.7	-10%	-49%
FM12	63.6	79.7	60.4	74.1	-5%	-7%
FM13	67.0	90.9	63.5	82.3	-5%	-9%
FM14a	22.9	43.4	44.7	59.8	95%	38%
FM14b	117.1	151.8	142.1	181.0	21%	19%
FM15	40.6	54.3	44.7	53.4	10%	-2%
FM16	44.5	54.6	41.5	49.6	-7%	-9%
FM17	20.9	34.2	21.0	34.8	1%	2%
FM18	148.0	184.0	148.2	180.9	0%	-2%
FM19	14.7	26.3	14.7	18.4	0%	-30%
FM20	75.1	115.8	76.7	104.8	2%	-9%

FM21	14.4	20.8	15.3	20.6	6%	-1%
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**Figure 5-1: Average DWF Calibration Results**



**Figure 5-2: Peak DWF Calibration Results**



### 5.1.2 Monitor FM10

The DWF calibration results for monitor FM10 are within acceptable range when comparing to monitored flow. Both average and peak flow for the two-day calibration period (June 17-19, 2020) meet the criteria of a 10% margin of error.

### 5.1.3 Monitor FM11

For monitor FM11, only the average modeled flow was within acceptable range. The peak flow is underestimated by 49% and is caused by a sudden jump in observed flow during the second day of the dry period. Attempts were made to calibrate using a different two-day period, but the inconsistent flow monitoring data for FM11 proved challenging. As such, it was chosen to keep the obtained results with the average modeled flow meeting the criteria of a 10% margin of error. Further considerations are made as a result during the WWF calibration process.

### 5.1.4 Monitor FM12

The DWF calibration results for monitor FM12 are within acceptable range when comparing to monitored flow. Both average and peak flow for the two-day calibration period (June 17-19, 2020) meet the criteria of a 10% margin of error.

### 5.1.5 Monitor FM13

The DWF calibration results for monitor FM13 are within acceptable range when comparing to monitored flow. Both average and peak flow for the two-day calibration period (June 17-19, 2020) meet the criteria of a 10% margin of error.

### 5.1.6 Monitors FM14a and FM14b

For both monitors FM14a and FM14b, the calibration results have not met target margins of error. It was initially thought to be caused by manholes with pipe bifurcation, which allow flow to split between the two monitoring sites. This would have been the case if one had its modeled flow underestimated while the other overestimated. But in this case, both have their average and peak flow overestimated.

The majority of flow at monitors FM14a and FM14b are generated by parameters calibrated from upstream monitors. Given that these upstream sites were calibrated first, it could be that the flow monitoring data might be erroneous. As such, it is possible that the flow monitoring data for FM14a, FM14b or their tributary sites include some inaccuracies.

Upstream sites include FM06, FM07, FM13, FM19 and FM20. The monitored flow at FM14b should approximately be the sum of all the measured flow from these upstream sites. But when looking at the monitored flow, it is largely below the expected theoretical flow.

To verify the flow monitoring data, velocity-depth scattergraphs were plotted (See Appendix D) and compared to expected values based on Manning's equation. The scattergraphs for FM14a and FM14b show that the measured velocity is lower than anticipated. The typical Manning roughness for a sewer system is often assumed to be 0.013. The monitoring data suggests that the sewers at FM14a and FM14b have a roughness of over 0.019, which is unlikely.

It is believed that the monitors FM14a and FM14b are measuring lower velocities, which in turn leads to lower measured flow. This would explain why the modeled flow is overestimated and why the calibration results do not fall within acceptable error margins. Alternatively, if the roughness values are as high as the measured data suggests, then further consideration might be made to represent this. If this is the case, then cleaning is likely warranted. For model calibration purposes, it was recommended to proceed with an appreciation for these findings and assuming a typical roughness value of  $n = 0.013$ . Since flow is

overestimated, these meters were kept in the calibration, to be conservative. Further investigation may be of interest.

#### 5.1.7 Monitor FM15

The DWF calibration results for monitor FM15 are within acceptable range when comparing to monitored flow. Both average and peak flow for the two-day calibration period (June 17-19, 2020) meet the criteria of a 10% margin of error.

#### 5.1.8 Monitor FM16

The DWF calibration results for monitor FM16 are within acceptable range when comparing to monitored flow. Both average and peak flow for the two-day calibration period (June 17-19, 2020) meet the criteria of a 10% margin of error.

It is worth mentioning that only velocity and depth data were available for this monitor. Flow was computed from the velocity and depth using Manning's equation with a roughness of 0.016, a diameter of 750mm and a slope of 0.5%. A roughness value of 0.016 was used as it provided a good flow continuity with the measured flow from site FM15 located downstream.

#### 5.1.9 Monitor FM17

The DWF calibration results for monitor FM17 are within acceptable range when comparing to monitored flow. Both average and peak flow for the two-day calibration period (June 17-19, 2020) meet the criteria of a 10% margin of error.

#### 5.1.10 Monitor FM18

Monitor FM18 is located downstream of the Clair-Gordon area (South End), which was modeled and calibrated by others with FM01 through FM 05 and FM09. The existing sub-model was imported and all its calibrated parameters were kept as they were. The calibration accuracy of these upstream sites directly affects the accuracy of site FM18. Once the DWF calibration for FM18 was completed, the peak and average flow met the 10% margin of error.

#### 5.1.11 Monitor FM19

Only the average modeled flow was within acceptable range for monitor FM19. The peak flow is underestimated by 31% and is explained by the presence of an unknown source of regular inflow. The observed data seem to indicate the presence of a pump regularly injecting 5-10 L/s into the collection system. Drawings were reviewed to try and identify the source of this inflow, but the source could not be determined and was outside the purview of the Master Plan to review any further. Further to the City's review of this document, it has been indicated that this inflow may be related to either the Membro well's treatment process discharge (located at 290 Water St) or to the private pumping station servicing a private townhouse complex at 295 Water St. The significance of including this private pump station will be further considered in the existing conditions assessment task.

### 5.1.12 Monitor FM20

The DWF calibration results for monitor FM20 are within acceptable range when comparing to monitored flow. Both average and peak flow for the two-day calibration period (June 17-19, 2020) meet the criteria of a 10% margin of error.

Table 5-3 provides a comparison of the DWF calibration from the 2017 Nima Trails modelling to the 2023 MP update. The modelled flows between both assignments are considered comparable. An exception may be considered at location NT06 where ~8L/s difference is found.

**Table 5-3: Nima Trails Validation.**

Flow Monitoring Location	Nima Trails Model (2017 GM BluePlan)			2023 MP Update Model
	Spring (L/s)	Summer (L/s)	Average (L/s)	(L/s)
NT01 (not used for calibration)				
NT02	3.8	3.0	3.4	3.9
NT03	4.6	3.5	4.1	5.3
NT04 (not used for calibration)				
NT05 (not used for calibration)				
NT06	13.3	11.8	12.6	20.5
NT07	40.4	32.3	36.4	42.2
NT08	11.8	12.2	12.0	8.3

### 5.1.13 Monitor FM21

The DWF calibration results for monitor FM21 are within acceptable range when comparing to monitored flow. Both average and peak flow for the two-day calibration period (June 17-19, 2020) meet the criteria of a 10% margin of error.

## 5.2 Wet Weather Calibration

### 5.2.1 Summary

The wet weather response was modeled using the RTK method. Each flow monitoring site is assigned a unit hydrograph composed of 3 sets of 3 parameters. Each set of parameters represents a type of wet weather response: short-term, medium-term, and long-term response. The calibrated parameters for each flow monitoring site are shown at Table 5-4.

The results indicate that the collection system is not susceptible to high rainfall derived inflow and infiltration (RDII), with most R-values being below 1%. This is consistent with the findings from previous consultants. Additional flow monitoring programs are recommended in an attempt to capture larger events with the intent of confirming how the RDII might change under these conditions. These RDII values may be increased during the assessment tasks of this master plan to understand the extent of the implications of higher R-values on the City’s wastewater collection system.

The results also indicate that the use of the short-term RDII response parameter (R1, T1, K1) is mostly adequate for calibrating to the flow monitor recorded response. Being limited by the magnitude of available rainfall events, the medium- and long-term RDII responses for some sites was not necessary. This does not equate to there not being medium- or longer-term responses at these locations. It does suggest however that additional calibration to larger events should consider all the response parameters (i.e., short, medium, long term) and not conclude that the findings from this calibration exercise can be extended to larger events.

**Table 5-4 WWF Calibrated RTK parameters**

Flow Monitor	Total R	Short Term			Medium Term			Long Term		
		R	T	K	R	T	K	R	T	K
FM10	0.020 = 2.0%	0.007	0.05	0.4	0.006	0.50	1.5	0.007	3.00	2.5
FM11	0.005 = 0.5%	0.005	0.10	0.4	-	-	-	-	-	-
FM12	0.006 = 0.6%	0.003	0.05	1.5	0.003	5.00	2.5	-	-	-
FM13	0.002 = 0.2%	0.001	0.10	0.4	0.001	1.25	1.5	-	-	-
FM14a	0.033 = 3.3%	0.033	1.25	1.5	-	-	-	-	-	-
FM14b	0.004 = 0.4%	0.004	1.25	1.5	-	-	-	-	-	-
FM15	0.002 = 0.2%	0.002	1.25	1.5	-	-	-	-	-	-
FM16	0.017 = 1.7%	0.004	0.20	1.5	0.006	4.00	2.5	0.007	12.5	5.0
FM17	0.004 = 0.4%	0.004	0.50	1.5	-	-	-	-	-	-
FM18	0.001 = 0.1%	0.001	1.25	1.5	-	-	-	-	-	-
FM19	0.007 = 0.7%	0.004	0.50	1.5	0.002	4.00	2.5	0.001	12.50	5.0
FM20	0.004 = 0.4%	0.004	0.50	1.5	-	-	-	-	-	-

Flow Monitor	Total R	Short Term			Medium Term			Long Term		
		R	T	K	R	T	K	R	T	K
FM21	0.003 = 0.3%	0.003	0.50	1.5	-	-	-	-	-	-

Also based on the WAPUG’s code of practice, the wet weather flow (WWF) calibration was carried out using the flow monitoring data. The selected 3 events were based on the magnitude and quality of events obtained and reviewed with the City. Selection considered depth, intensity, and volume. Smaller events are sometimes selected for wet weather flow calibration as these are subject to different inflow characteristics when compared to the larger events. In addition, the January 10, 2020 rain and snowmelt event were considered as a validation event for FM10, FM20, and FM21. Data from this larger event was not available for the other sites used for calibration.

The modeled flow was compared to the observed values from the corresponding rainfall event. The hydrographs should closely follow each other both in shape and in magnitude, until they substantially return to DWF conditions. In addition to the shape, the observed and modeled hydrographs should meet the following criteria for most of the events considered.

1. The timing of the peaks and valleys should be similar for the duration of the event.
2. The peak flow rates at each significant peak should be in the range of -15% to +25%.
3. The volume of flow should be within -10% to +20%.

The WWF calibration results are presented at Table 5-5, Figure 5-5, Figure 5-3, and Figure 5-4 for a 48-hrs simulation run. The plot comparison of monitored and modeled flow is available in Appendix E. The results are further discussed below for each flow monitoring site.

**Table 5-5 WWF Calibration Results**

Flow Meter	WWF Event	Assigned Rain Gauge	Return Period	Monitored		Modeled		% Error	
				Volume (m3)	Peak (L/s)	Volume (m3)	Peak (L/s)	Volume (m3)	Peak (L/s)
FM10	Jan 11, 2020 (Validation)	West End	>50 yrs	16,290	175.4	7,337	85.3	-55%	-51%
	May 29, 2020		>9 mths	4,847	96.1	5,233	90.5	8%	-6%
	Jun 10, 2020		>9 mths	6,058	101.3	5,571	97.7	-8%	-4%
	Jul 10, 2020		>9 mths	4,156	97.6	4,919	120.4	18%	23%
FM11	May 29, 2020	West End	>9 mths	1,690	52.1	1,612	31.7	-5%	-39%

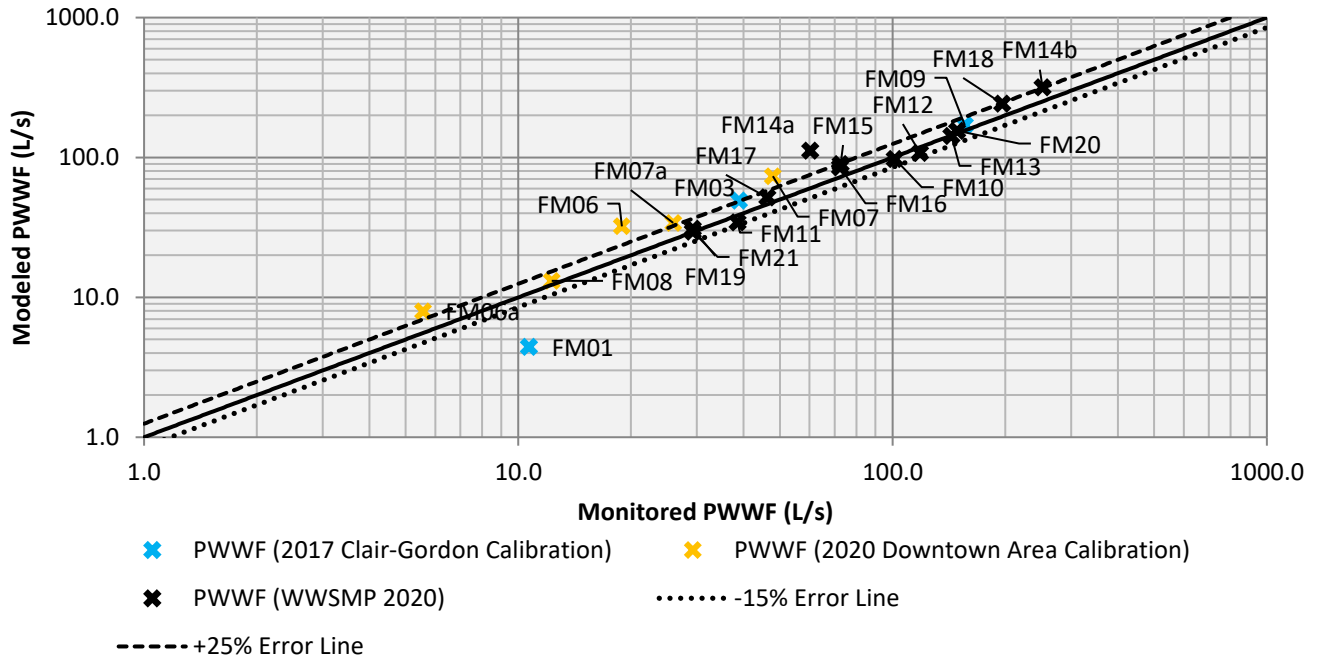
Flow Meter	WWF Event	Assigned Rain Gauge	Return Period	Monitored		Modeled		% Error	
				Volume (m3)	Peak (L/s)	Volume (m3)	Peak (L/s)	Volume (m3)	Peak (L/s)
	Jun 10, 2020		>9 mths	2,935	38.5	2,659	34.3	-9%	-11%
	Jul 10, 2020		>9 mths	1,666	31.1	1,691	38.4	2%	23%
FM12	May 29, 2020	West End	>9 mths	8,677	96.4	8,630	91.4	-1%	-5%
	Jun 10, 2020		>9 mths	10,987	118.3	11,614	108.0	6%	-9%
	Jul 10, 2020		>9 mths	7,142	102.1	7,297	113.8	2%	11%
FM13	May 29, 2020	RG02	>9 mths	11,303	132.6	11,631	136.8	3%	3%
	Jun 10, 2020		>1 yr	12,513	143.6	12,039	142.2	-4%	-1%
	Jul 10, 2020		>1 yr	10,432	136.9	11,419	169.7	9%	24%
FM14a	May 29, 2020	RG02	>9 mths	4,106	54.2	7,892	80.8	92%	49%
	Jun 10, 2020		>1 yr	4,479	60.2	8,476	111.6	89%	85%
	Jul 10, 2020		>1 yr	4,910	78.7	8,576	115.2	75%	46%
FM14b	May 29, 2020	RG02	>9 mths	21,922	218.7	25,480	256.0	16%	17%
	Jun 10, 2020		>1 yr	23,119	251.6	26,789	317.4	16%	26%
	Jul 10, 2020		>1 yr	21,540	267.5	26,324	336.7	22%	26%
FM15	May 29, 2020	RG02	>9 mths	7,019	74.9	8,005	71.4	14%	-5%

Flow Meter	WWF Event	Assigned Rain Gauge	Return Period	Monitored		Modeled		% Error	
				Volume (m3)	Peak (L/s)	Volume (m3)	Peak (L/s)	Volume (m3)	Peak (L/s)
	Jun 10, 2020		>1 yr	7,373	72.6	8,795	89.5	19%	23%
	Jul 10, 2020		>1 yr	7,978	119.2	8,962	122.2	12%	3%
FM16	May 29, 2020	RG02	>9 mths	8,270	76.8	7,445	67.8	-10%	-12%
	Jun 10, 2020		>1 yr	7,952	72.3	8,221	85.0	3%	17%
	Jul 10, 2020		>1 yr	7,801	96.5	8,385	117.9	7%	22%
FM17	May 29, 2020	Helmar Well	>1 yr	3,797	44.9	3,720	45.5	-2%	1%
	Jun 10, 2020		>1 yr	3,788	46.3	3,875	51.8	2%	12%
	Jul 10, 2020		>2 yrs	3,838	67.6	3,947	63.9	3%	-5%
FM18	May 29, 2020	RG01	<3 mths	24,093	183.9	25,634	181.0	6%	-2%
	Jun 10, 2020		>1 yr	25,533	196.2	26,608	242.9	4%	24%
	Jul 10, 2020		>3 mths	22,146	161.8	25,884	194.2	17%	20%
FM19	May 29, 2020	RG02	>9 mths	2,868	27.8	2,595	26.2	-10%	-6%
	Jun 10, 2020		>1 yr	2,705	29.3	2,747	30.9	2%	6%
	Jul 10, 2020		>1 yr	2,830	44.6	2,789	44.3	-1%	-1%
FM20	Jan 11, 2020 (Validation)	Helmar Well	>100 yrs	44,927	390.2	15,523	178.7	-65%	-54%

Flow Meter	WWF Event	Assigned Rain Gauge	Return Period	Monitored		Modeled		% Error	
				Volume (m3)	Peak (L/s)	Volume (m3)	Peak (L/s)	Volume (m3)	Peak (L/s)
	May 29, 2020		>1 yr	14,217	131.8	13,526	139.5	-5%	6%
	Jun 10, 2020		>1 yr	14,197	148.7	14,056	155.2	-1%	4%
	Jul 10, 2020		>2 yrs	N/A	N/A	N/A	N/A	N/A	N/A
FM21	Jan 11, 2020 (Validation)	RG02	>100 yrs	8,230	69.8	2,875	25.3	-65%	-64%
	May 29, 2020		>9 mths	2,660	27.2	2,673	28.4	0%	4%
	Jun 10, 2020		>1 yr	2,815	29.3	2,762	29.5	-2%	1%
	Jul 10, 2020		>1 yr	2,770	35.3	2,776	43.1	0%	22%



### June 10, 2020: Peak Wet Weather Flow



### June 10, 2020: Wet Weather Volume

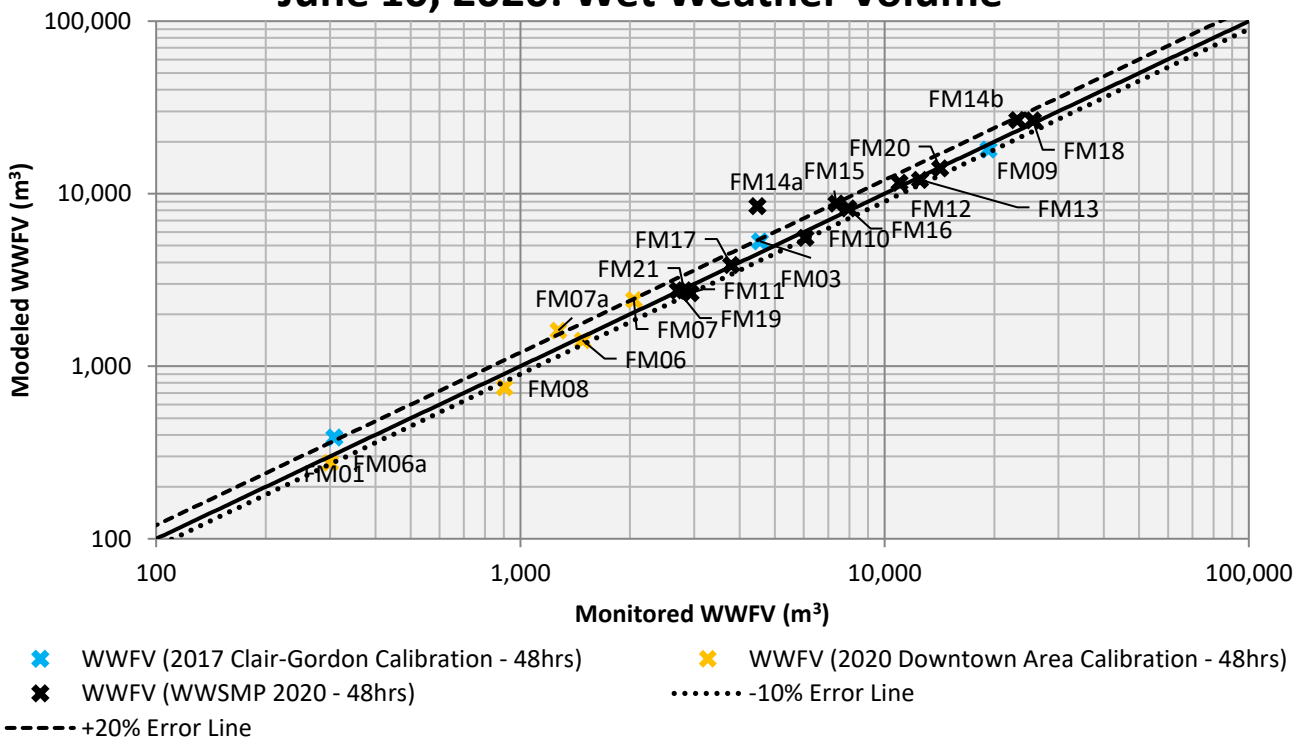
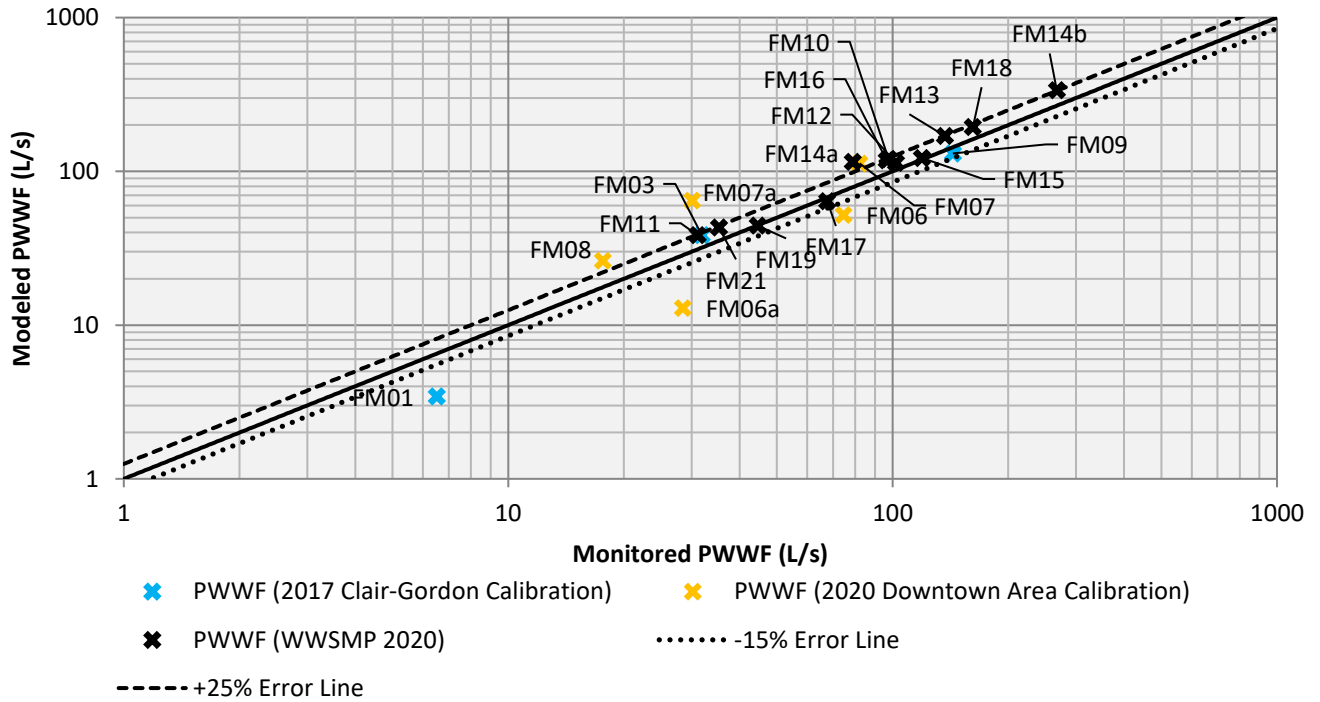


Figure 5-3: WWF Calibration Results (June 10, 2020)

### July 10, 2020: Peak Wet Weather Flow



### July 10, 2020: Wet Weather Volume

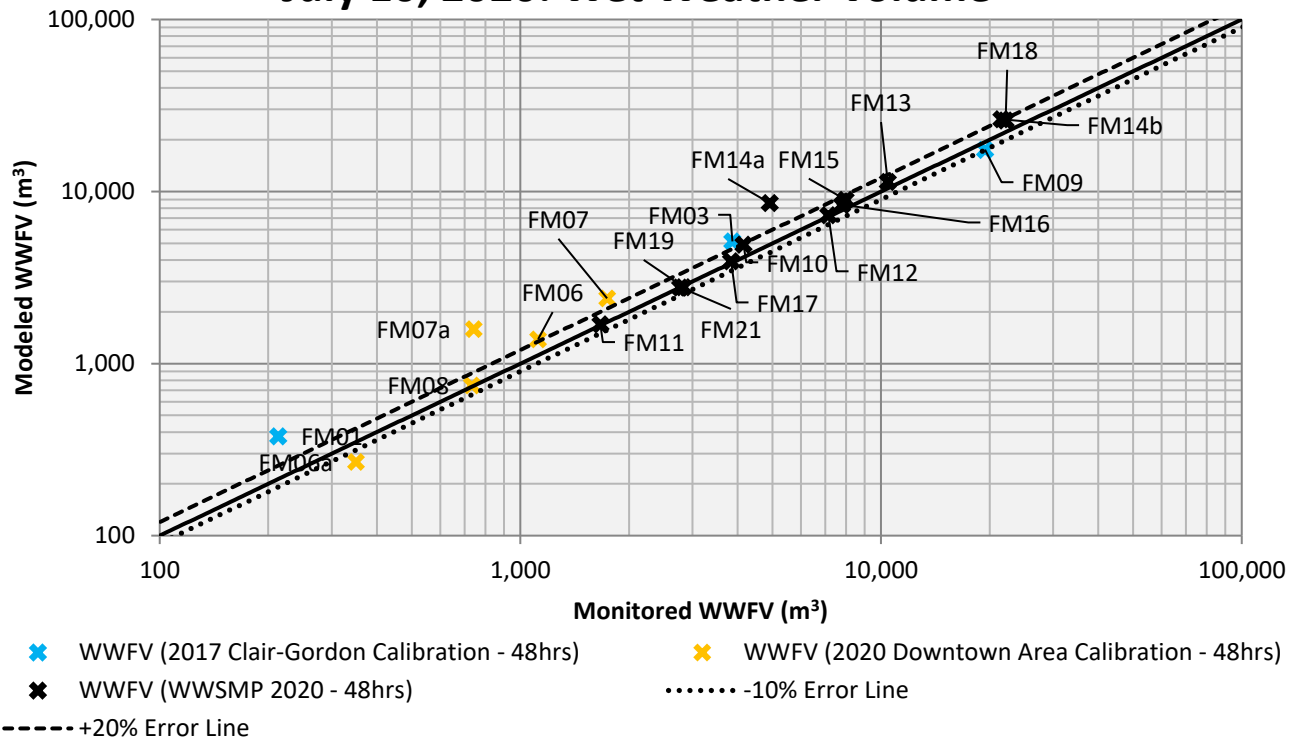
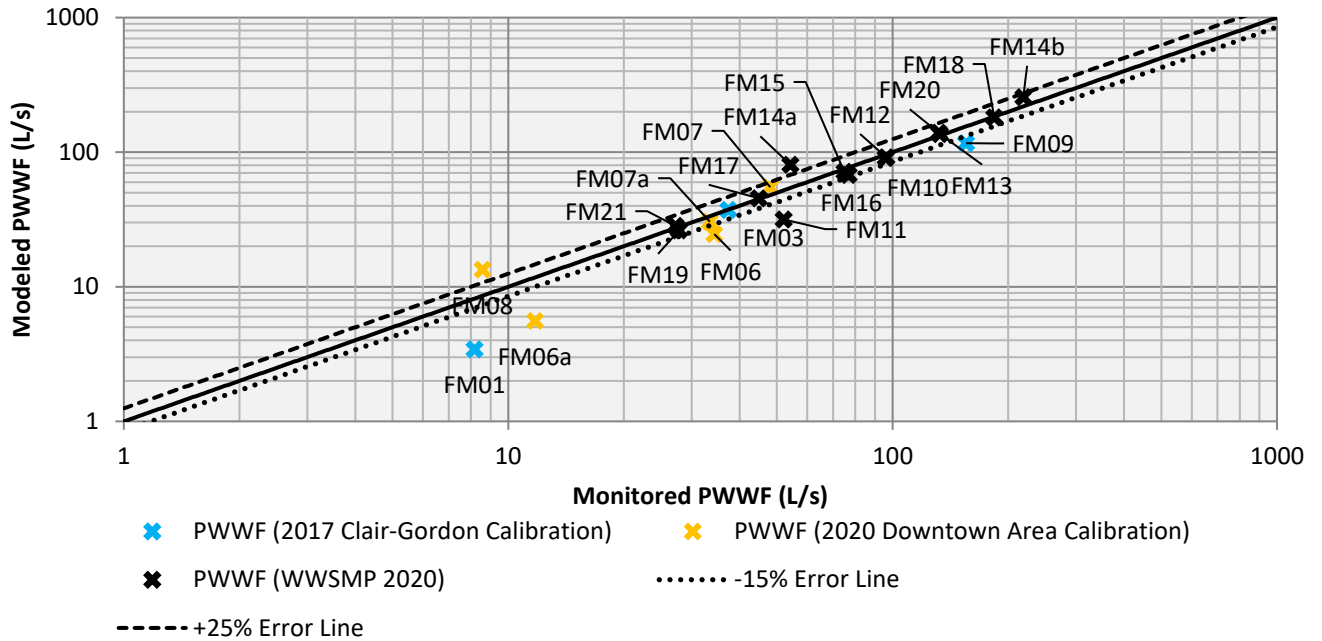


Figure 5-4: WWF Calibration Results (July 10, 2020)

### May 29, 2020: Peak Wet Weather Flow



### May 29, 2020: Peak Wet Weather Volume

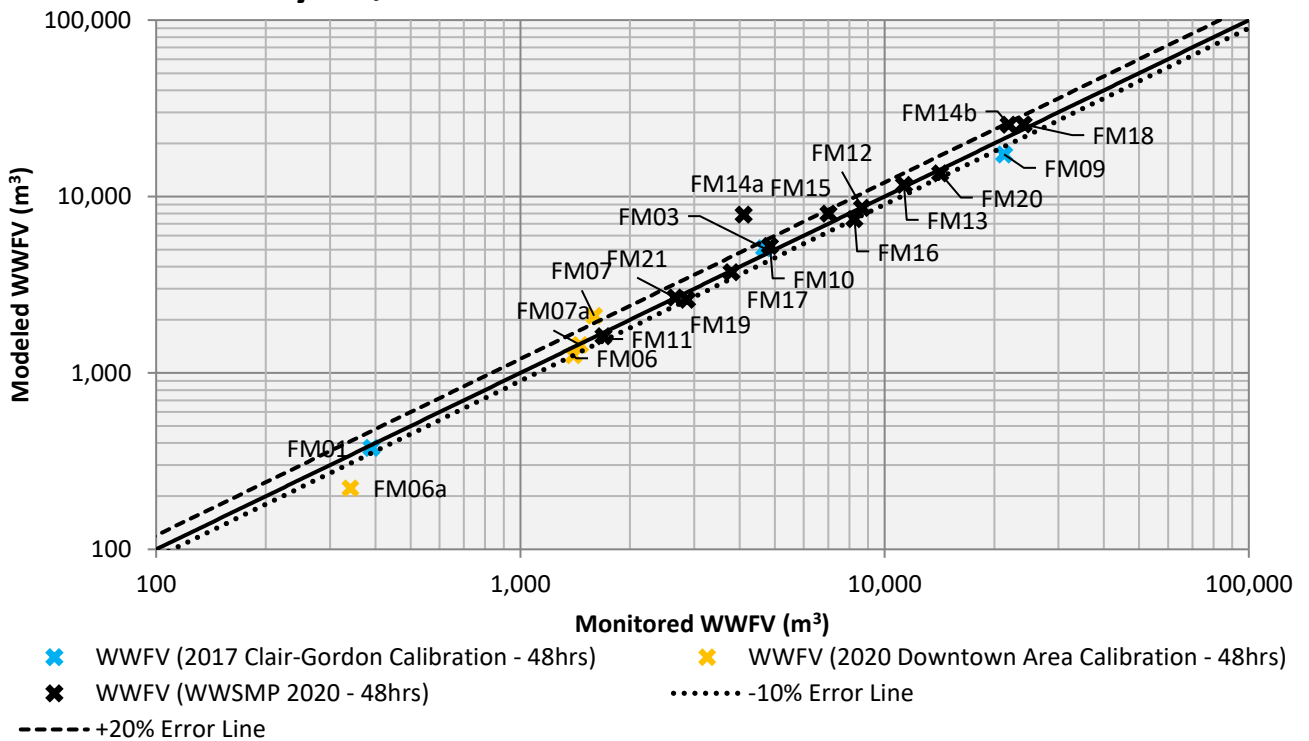
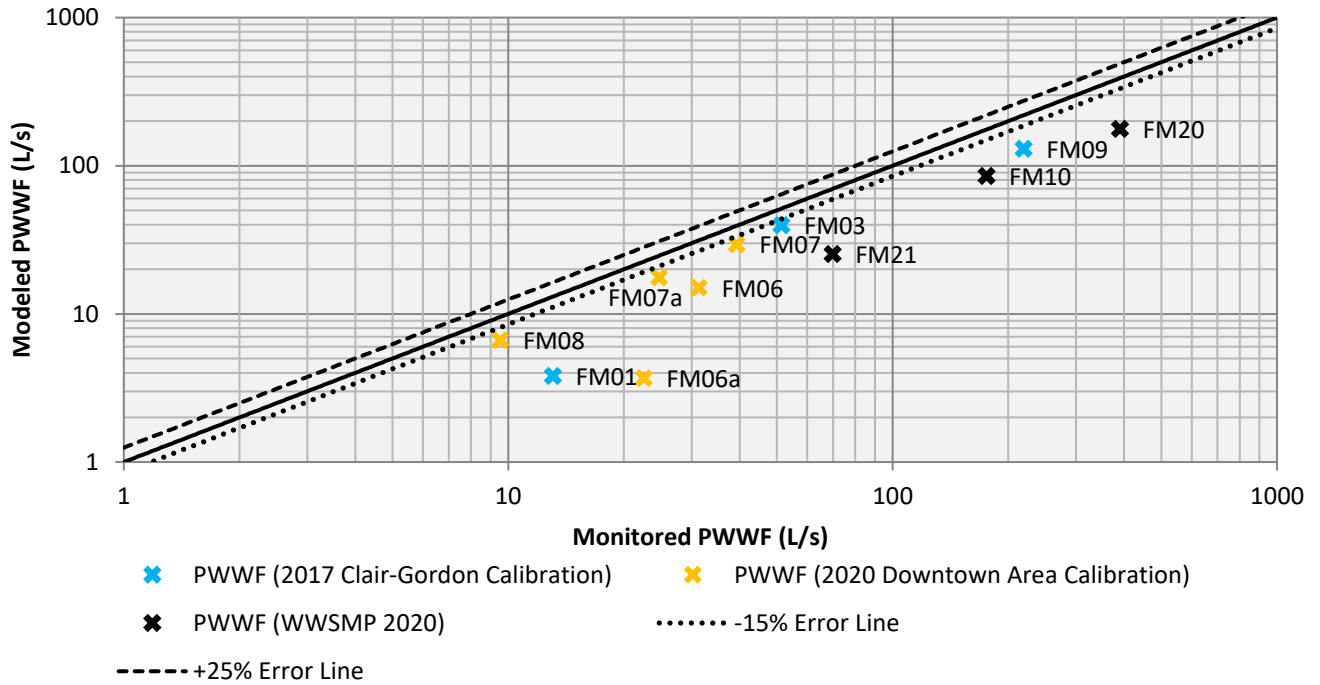


Figure 5-5: WWF Validation Results (May 29, 2020)

### January 11, 2020: Peak Wet Weather Flow



### January 11, 2020: Peak Wet Weather Volume

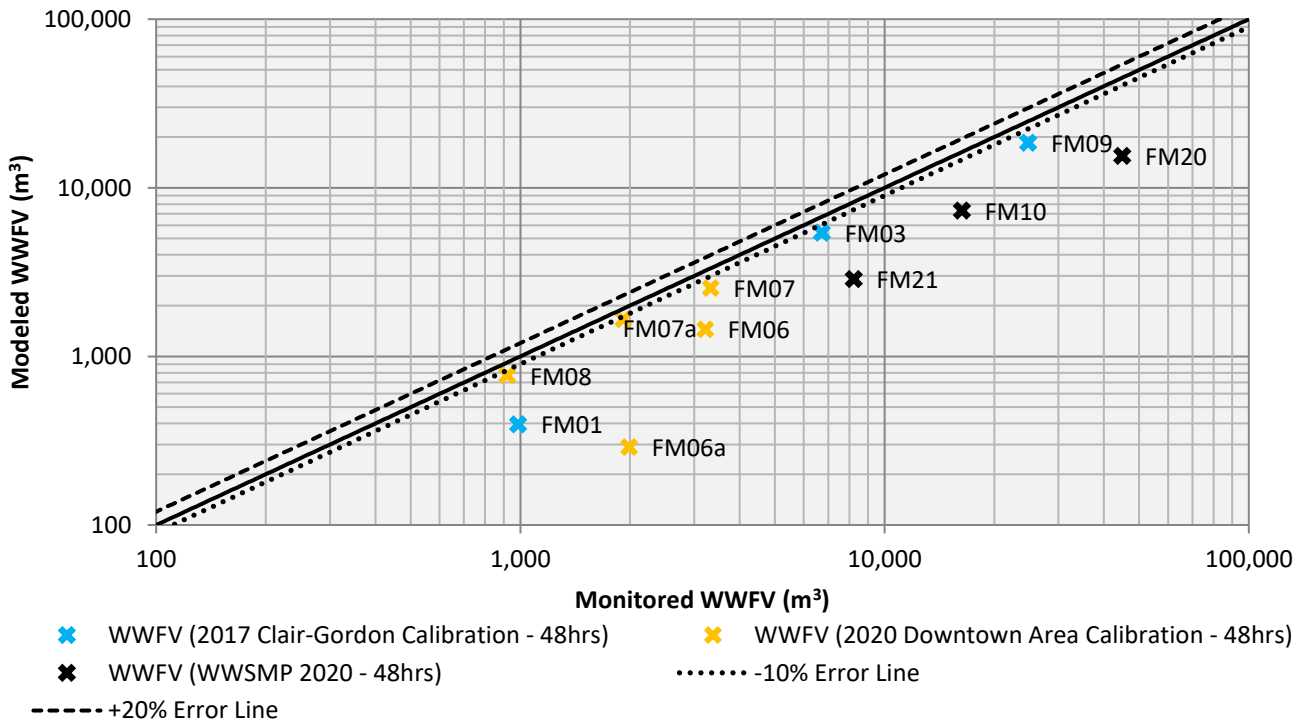


Figure 5-6: WWF Validation Results (January 11, 2020)

### 5.2.2 Monitor FM10

Wet weather flow for monitor FM10 was calibrated using the June and July events. The RTK parameters were calibrated so that the modeled flow falls within acceptable range for both events. Target margin of errors were achieved but it did require some adjustments in modeled dry weather flow (DWF). GWI had to be reduced by 30% in the July event to achieve a peak flow error below 25% and the volume below 20%.

The calibrated RTKs were then validated using the May 29 event. Once again, GWI needed to be adjusted to reach target margins of error. This time, GWI was increased by 10%. Peak flow for the May event is slightly underestimated by 6%. The results for the three wet weather events (May, June, and July) show that less intense events slightly underestimate, while more intense events slightly overestimate the peak flow.

For further validation, the January 10 event, equivalent to a 50-year event, was run as well. The results show that the peak flow and volume are greatly underestimated by over 50%. This is partly explained by the unknown contribution from snowmelt that is not accounted for in the model. Given the lack of data concerning the snow cover, it is impossible to evaluate the exact amount of flow contributed by snowmelt.

Given that both peak and volume are greatly underestimated for the January 50-year event, an increase of the calibrated R-values may be warranted to stress the system.

### 5.2.3 Monitor FM11

Wet weather flow for monitor FM11 was calibrated using the June and July events. The RTK parameters were calibrated so that the modeled flow falls within acceptable range for both events. Target margin of errors were achieved but it did require some adjustments in modeled dry weather flow (DWF). As previously mentioned, monitor FM11 shows high volatility in its measured flow. Dry weather flow can vary from one week to another and even from two consecutive days at this location.

For the June wet weather event, ADSF needed to be increased by 50% to meet target range. On the other hand, the July event required ADSF to be decreased by 50% on the second simulated day (Saturday).

Once calibrated, the RTK parameters were validated with the May 29 event. Like the July event, ADSF was decreased on the second simulated day (Saturday). Modeled peak flow did not meet the observed flow within acceptable range, but the total volume and the general shape of the hydrograph are similar. Given the low quality of data from monitor FM11, it is possible that the measured peak flow might be an aberration. Given that the peak flow was met for both the June and July events, the wet weather calibration is considered to be adequate.

### 5.2.4 Monitor FM12

Wet weather flow for monitor FM12 was calibrated using the June and July events. The RTK parameters were calibrated so that the modeled flow falls within acceptable range for both events. Target margin of errors were achieved but it did require some adjustments in modeled dry weather flow (DWF). Total DWF had to be reduced by 40% in the July event to achieve a peak flow error below 25% and the volume below 20%.

The calibrated RTKs were then validated using the May 29 event. Once again, DWF needed to be adjusted to reach target margins of error. This time, DWF was decreased by 20%. Peak flow for the May event is slightly underestimated by 5%. The results for the three wet weather events (May, June, and July) show that less intense events slightly underestimate, while more intense events slightly overestimate the peak flow.

### 5.2.5 Monitor FM13

Wet weather flow for monitor FM13 was calibrated using the June and July events. The RTK parameters were calibrated so that the modeled flow falls within acceptable range for both events. The calibrated RTKs were then validated using the May 29 event. Target margin of errors were achieved for all three wet weather events (May, June, and July).

### 5.2.6 Monitors FM14a and FM14b

For monitors FM14a and FM14b, both peak flow and volume are overestimated. As explained in the dry weather calibration section, it could be that there are some inaccuracies within the flow monitoring data for FM14a, FM14b or the meters in their upstream tributary sites. Considering that these upstream sites have been calibrated first, it is suspected that the flow monitoring data at FM14a and 14b might be erroneous.

The expected theoretical flow at FM14b should approximately be the sum of all the measured flow coming from upstream sites, which include FM06, FM07, FM13, FM19 and FM20. The monitored flow at FM14b shows lower flow than what is anticipated. For this reason, it is believed that the flow monitoring data is inaccurate for one or several of these sites. As such, the calibration results for FM14a and FM14b do not fall within acceptable error margins. Since flow is overestimated, the model calibration is maintained as is and is considered conservative. Further investigation of these specific locations may be of interest.

### 5.2.7 Monitor FM15

Wet weather flow for monitor FM15 was calibrated using the June and July events. The RTK parameters were calibrated so that the modeled flow falls within acceptable range for both events. The calibrated RTKs were then validated using the May 29 event. Target margin of errors were achieved for all three wet weather events (May, June, and July).

### 5.2.8 Monitor FM16

Wet weather flow for monitor FM16 was calibrated using the June and July events. The RTK parameters were calibrated so that the modeled flow falls within acceptable range for both events. The calibrated RTKs were then validated using the May 29 event. Target margin of errors were achieved for all three wet weather events (May, June, and July).

### 5.2.9 Monitor FM17

Wet weather flow for monitor FM17 was calibrated using the June and July events. The RTK parameters were calibrated so that the modeled flow falls within acceptable range for both events. The calibrated RTKs were then validated using the May 29 event. Target margin of errors were achieved for all three wet weather events (May, June, and July).

### 5.2.10 Monitor FM18

Wet weather flow for monitor FM18 was calibrated using the June and July events. The RTK parameters were calibrated so that the modeled flow falls within acceptable range for both events. The calibrated RTKs were then validated using the May 29 event. Target margin of errors were achieved for all three wet weather events (May, June, and July).

Monitor FM18 is located downstream of the Clair-Gordon area (South End), which was modeled and calibrated by Civica. Their existing model was imported into the new model and all its calibrated parameters were kept as they were. The monitors used in Civica's calibration were FM01 through FM05 and FM09. The calibration accuracy of these upstream sites directly affects the accuracy of site FM18.

### 5.2.11 Monitor FM19

Wet weather flow for monitor FM19 was calibrated using the June and July events. The RTK parameters were calibrated so that the modeled flow falls within acceptable range for both events. The calibrated RTKs were then validated using the May 29 event. Target margin of errors were achieved for all three wet weather events (May, June, and July).

The observed data seem to indicate the presence of a pump regularly injecting 5-10 L/s into the collection system. Drawings were reviewed to try and identify the source of this inflow, but none was found. Ulterior information from the City indicated that this inflow may be related to either the Membro well's treatment process discharge (located at 290 Water St) or to the private pumping station servicing a private townhouse complex at 295 Water St. Additional details to confirm the source will be requested and possibly included in the existing and future condition assessments if the contribution is deemed significant to the assessment.

### 5.2.12 Monitor FM20

Wet weather flow for monitor FM20 was calibrated using only the June event because data was not available for July. The RTK parameters were calibrated so that the modeled flow falls within acceptable range. The calibrated RTKs were then validated using the May 29 event. Target margin of errors were achieved for both wet weather events (May and June).

For further validation, the January 10 event, equivalent to a 50-year event, was run as well. The results show that the peak flow and volume are greatly underestimated by over 50%. This is partly explained by the unknown contribution from snowmelt that is not accounted for in the model. Given the lack of data concerning the snow cover, it is hard to evaluate the exact amount of flow contributed by snowmelt. Additional flow monitoring may be of interest to further understand the response to snow melt in the collection system.

Given that both peak and volume are greatly underestimated for the January 50-year event, an increase of the calibrated R-values may be warranted to stress the system during the existing and future condition assessment tasks.

### 5.2.13 Monitor FM21

Wet weather flow for monitor FM21 was calibrated using the June and July events. The RTK parameters were calibrated so that the modeled flow falls within acceptable range for both events. The calibrated RTKs were then validated using the May 29 event. Target margin of errors were achieved for all three wet weather events (May, June, and July).

For further validation, the January 10 event, equivalent to a 10-year event, was run as well. The results show that the peak flow and volume are greatly underestimated by over 50%. This is partly explained by the unknown contribution from snowmelt that is not accounted for in the model. Given the lack of data concerning the snow cover, it is hard to evaluate the exact amount of flow contributed by snowmelt. Additional flow monitoring may be of interest to further understand the response to snow melt in the collection system.

Given that both peak and volume are greatly underestimated for the January 50-year event, an increase of the calibrated R-values may be warranted to stress the system during the existing and future condition assessment tasks.

## 6.0 SUMMARY AND RECOMMENDATIONS

### 6.1 *Water Model Update*

The update of the City's hydraulic model included 2020 GIS infrastructure data, 2019 geocoded water billing records, and SCADA data. The resulting model simulated existing conditions within reasonable accuracy for purposes of Master Planning.

Additional field testing and calibration is not required for the purpose of the Master Plan. As the model is also used for operational and planning purposes, it is recommended that a second phase of field testing occur to fine tune the model, when possible. For accurate operational level modeling to investigate in greater detail at specific locations, it is recommended that detailed and localized information be gathered. Since the largest unknown is the water consumption usage patterns, it is recommended that the DMAs be isolated, and testing occur within the isolated DMAs.

To further improve the model accuracy, it is recommended that additional field testing and calibration is completed in select DMAs. A Phase 2 field-testing plan will be developed for review by the City.

### 6.2 *Wastewater Model Update*

The model was calibrated to a range of rainfall events with the available data provided. One of the validation events considered was an ~50year event concurrent to a snow melt occurrence. The system response during this event was significant and greater than any recorded from the events used for the model calibration completed. As a result, it is worth exploring if the existing and future condition assessments should consider the use of increased RDII producing "R" parameters to ensure the collection system can accommodate these larger events. Additionally, increased snow cover and snow melt monitoring may be beneficial for future studies.

The flow monitoring coverage was generally sufficient for the purposes of this Master Plan, nevertheless increasing data coverage can be beneficial for future Master Planning initiatives. This could be achieved by continuing with regular sewer flow monitoring and rainfall monitoring programs to collect data for this purpose.

### 6.3 *Water & Wastewater Common Recommendations*

In addition to ongoing improvements, it is recommended that the model data and calibration be updated every 1-2 years to keep the models current.

For the water model, this includes maintaining annual data for water billing records, water production, GIS pipe data, control strategies and pressure zone boundary changes.

For the wastewater model, this includes considering recalibrating to more recent and appropriate flow monitoring and collecting rainfall data, updates to the GIS database, operational updates, etc.

### 6.4 *Next Steps*

The next steps for the upcoming existing and future conditions assessment tasks include:

1. Existing Conditions Assessments
  - a. Establish if details for the Membro well and treatment process or the private pumping station at 295 Water Street are available. Review and agree to a consideration for the master plan going forward.

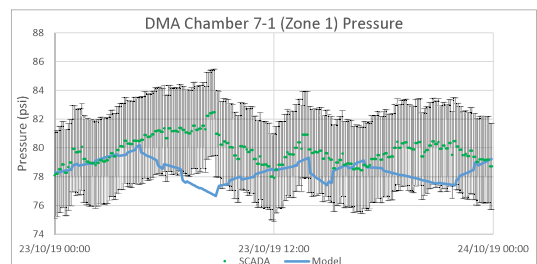
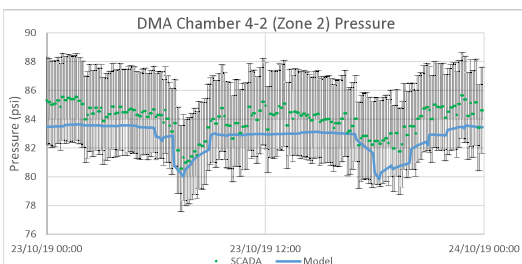
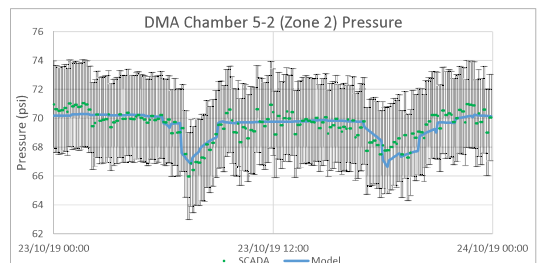
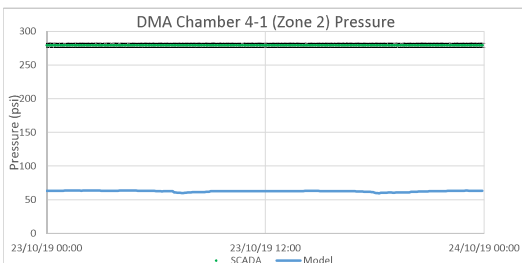
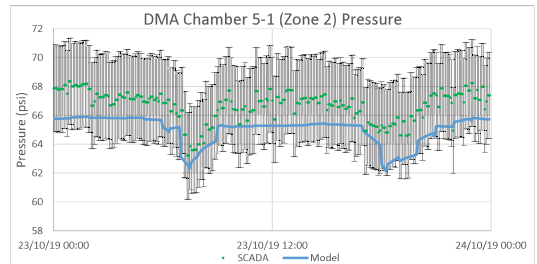
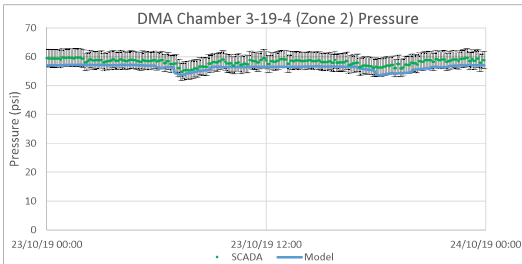
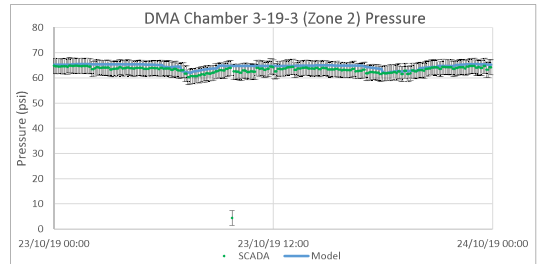
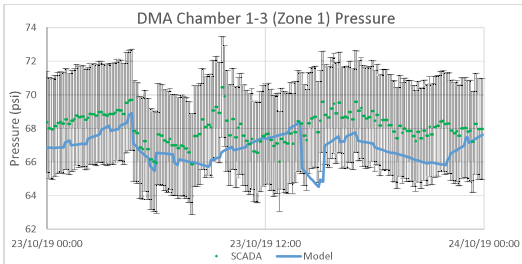
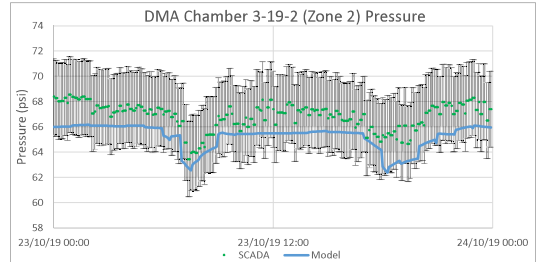
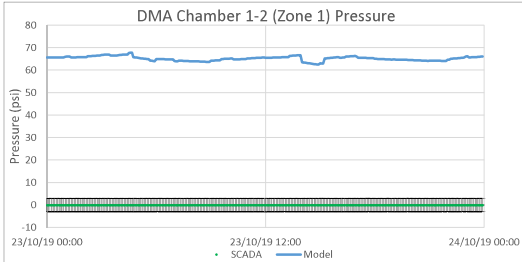
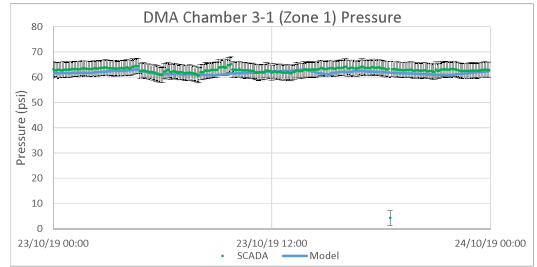
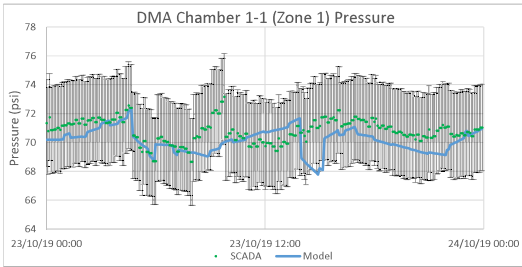


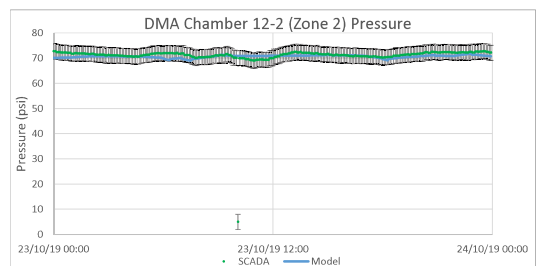
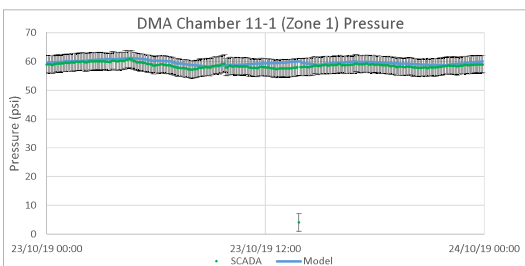
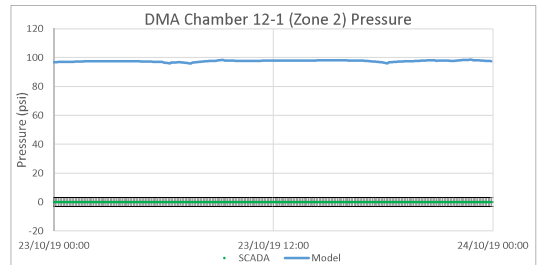
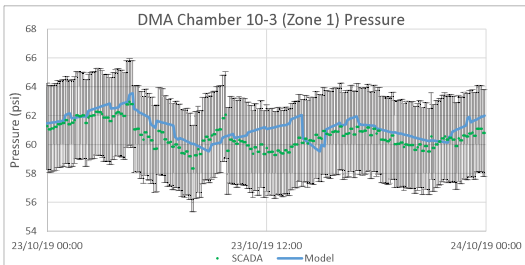
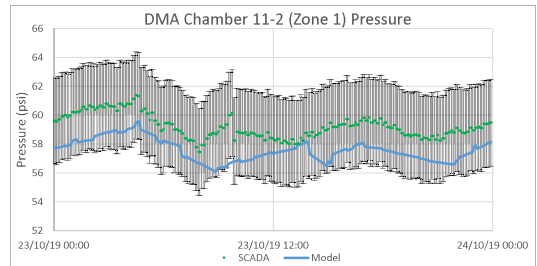
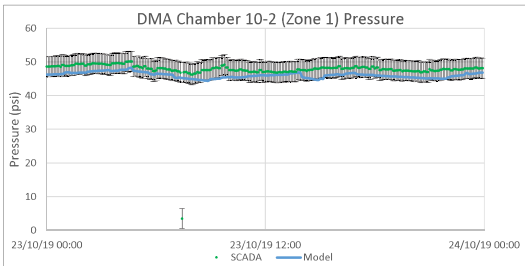
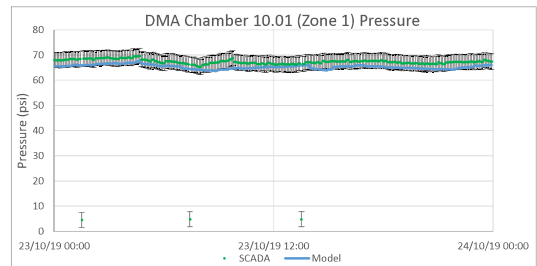
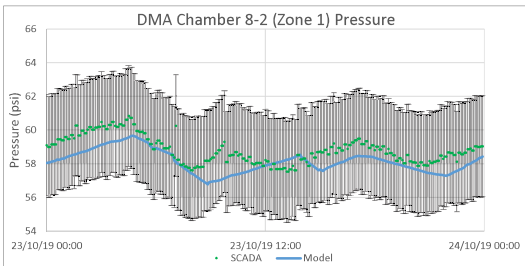
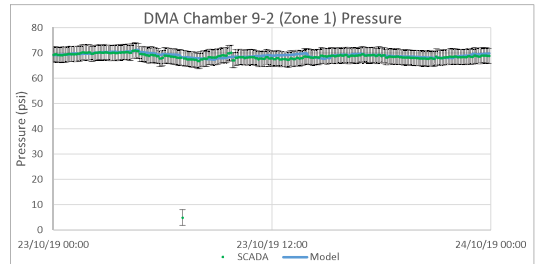
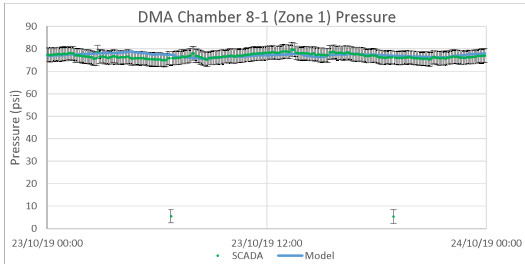
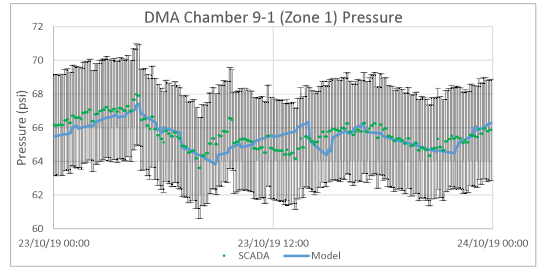
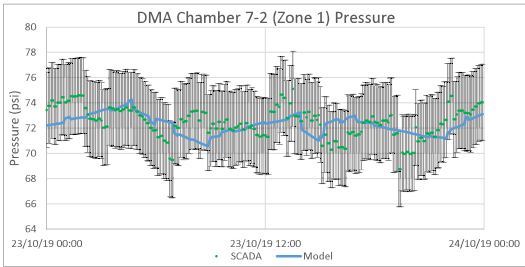
- 
- b. Assess the existing conditions model for the targeted level of service and identify opportunities and constraints.
    - i. Wastewater – review the parameters and explore if increased RDII “R” parameters should be used. (i.e., in response to the higher flows seen during the January 2020 validation event than predicted by the calibrated model)
  - 2. Future Conditions Assessments
    - a. Develop future conditions scenarios based on population and growth area projections provided by the City’s planning department.
    - b. Implement planned capital improvements in the future conditions’ models.
    - c. Assess the future conditions models for the targeted level of service and identify opportunities and constraints.

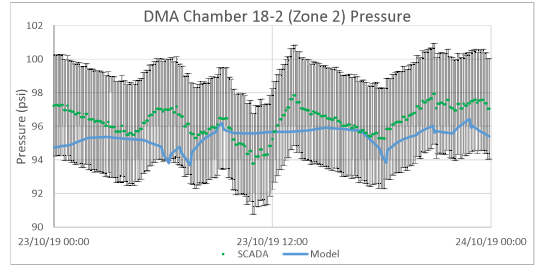
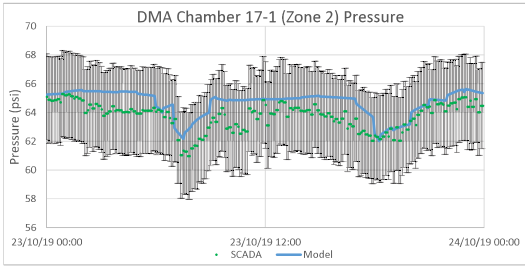
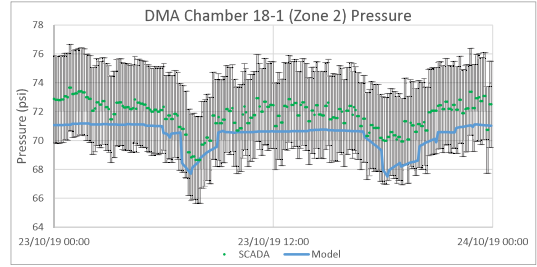
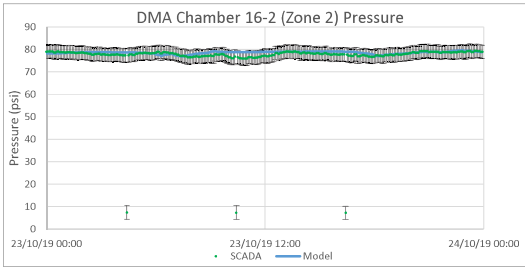
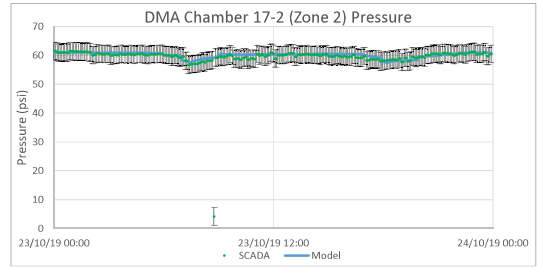
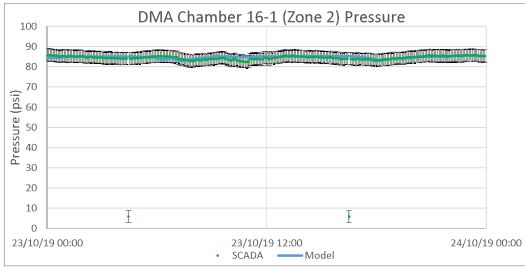
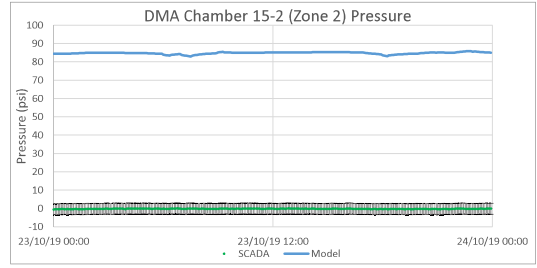
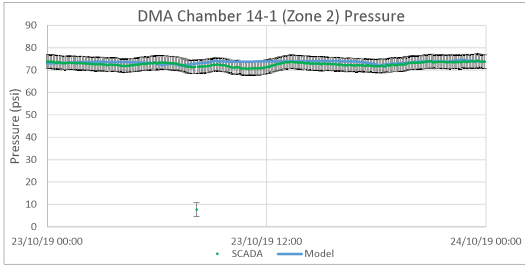
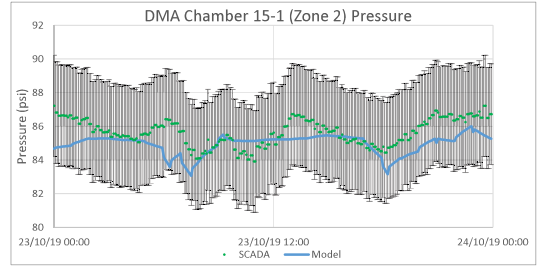
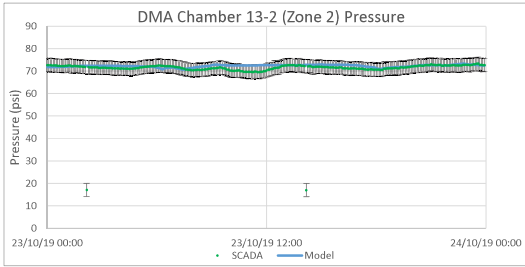
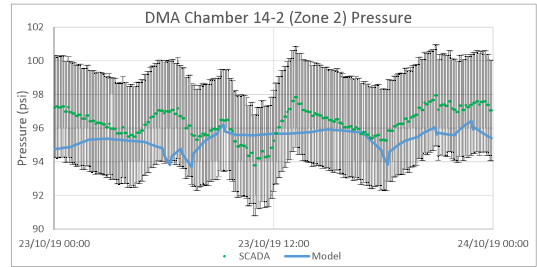
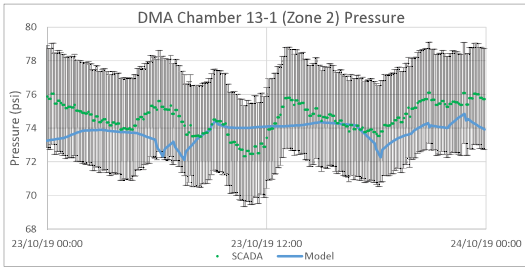
# APPENDIX A

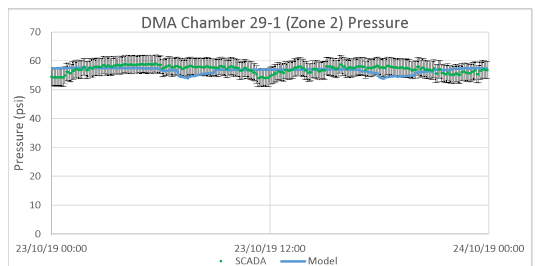
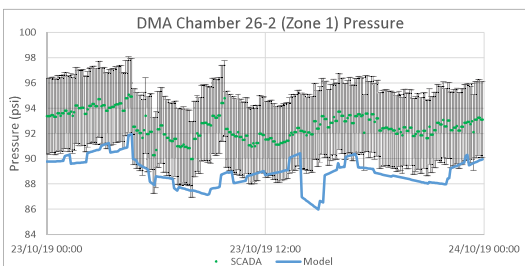
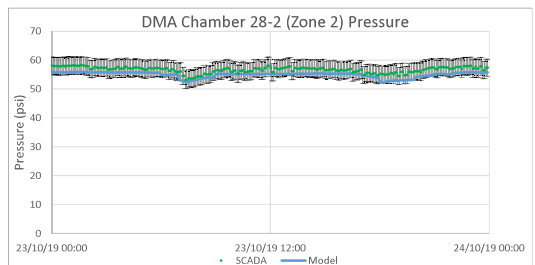
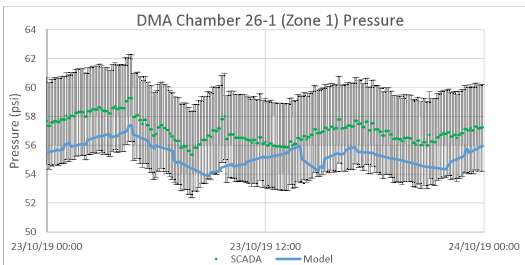
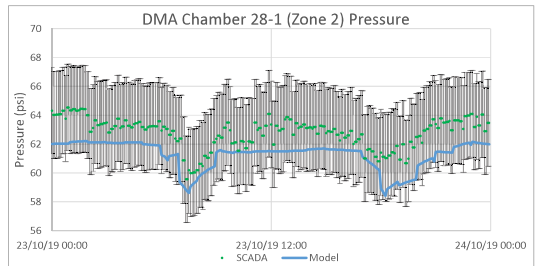
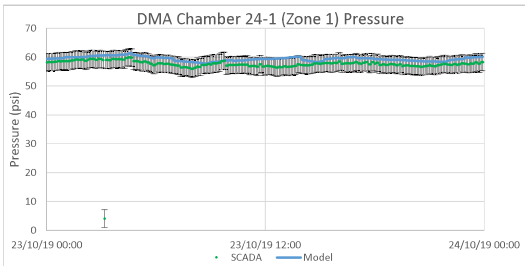
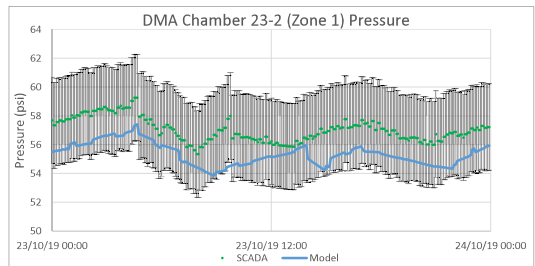
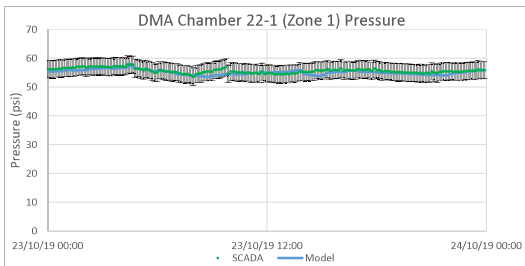
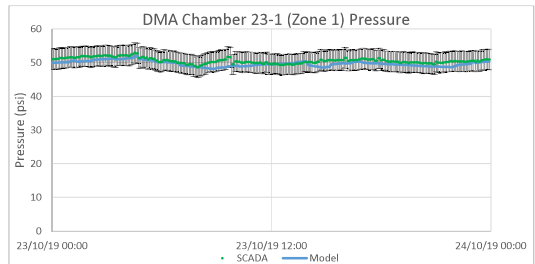
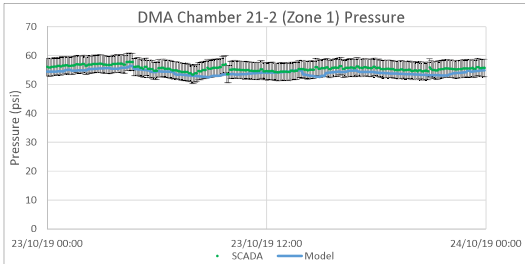
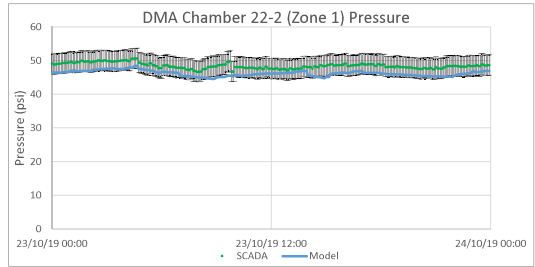
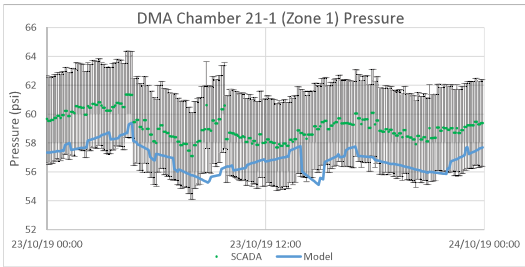
## Water Model Calibration Results

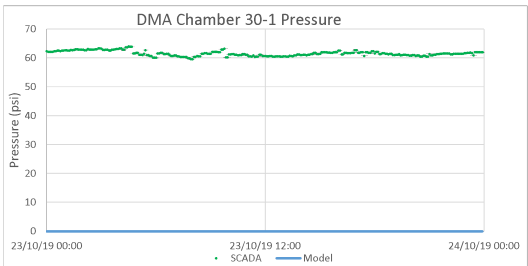
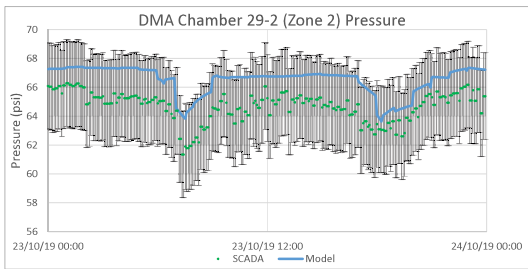
## **ADD – Dynamic Controls - Pressure**





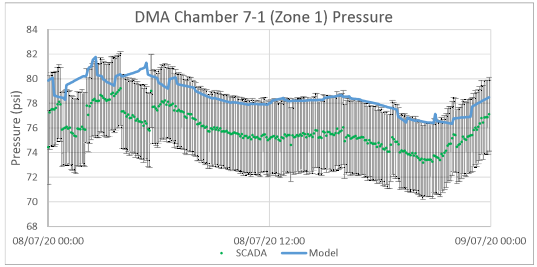
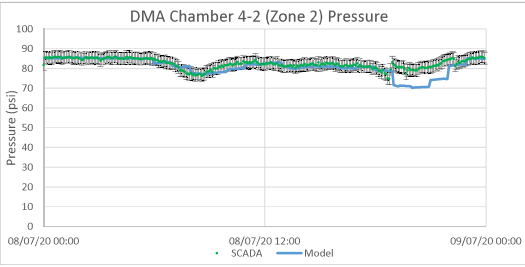
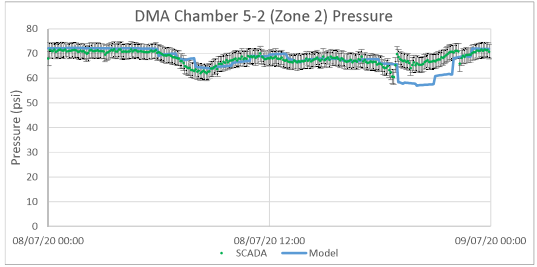
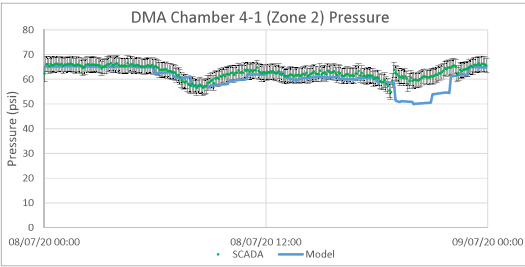
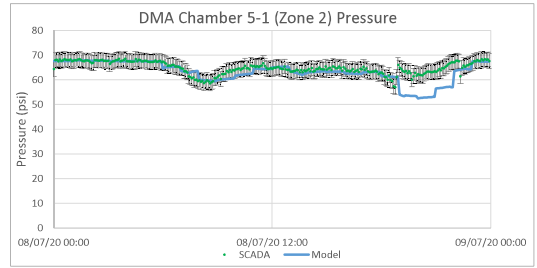
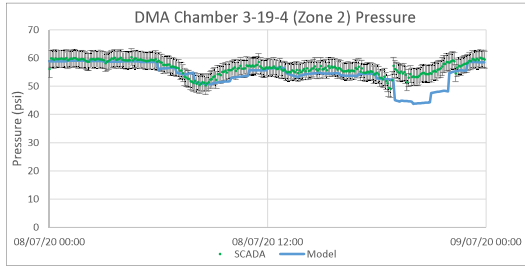
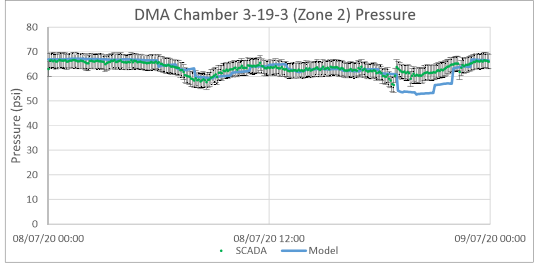
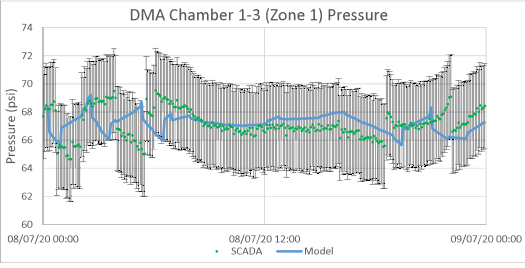
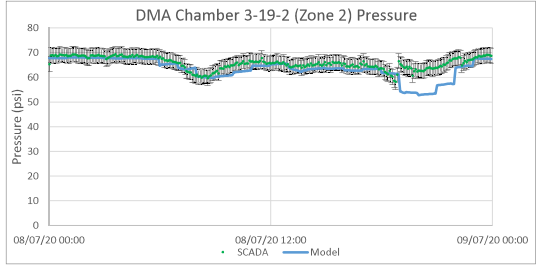
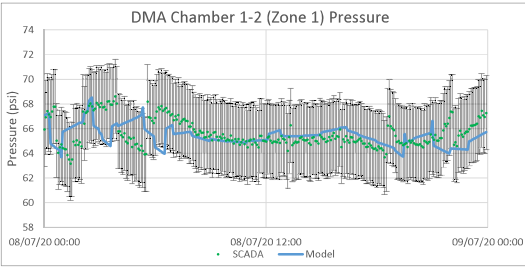
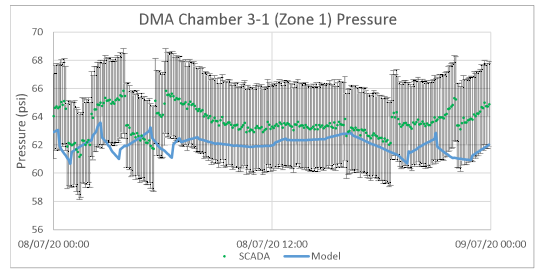
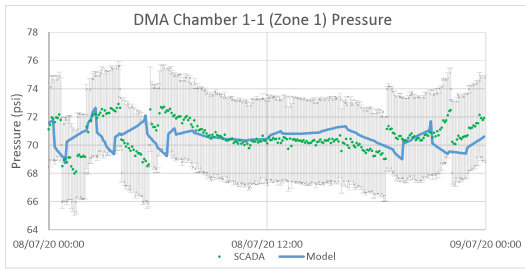


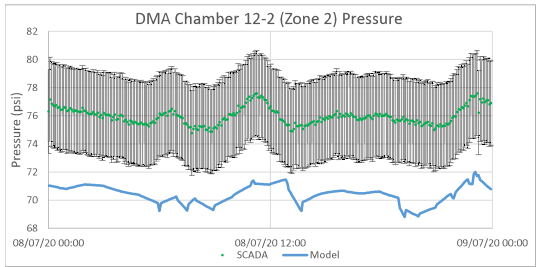
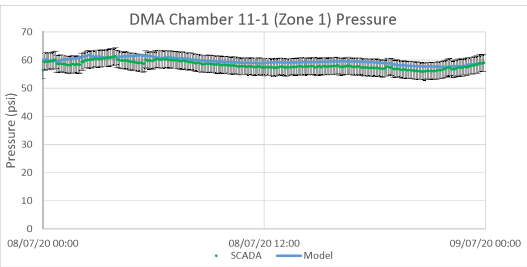
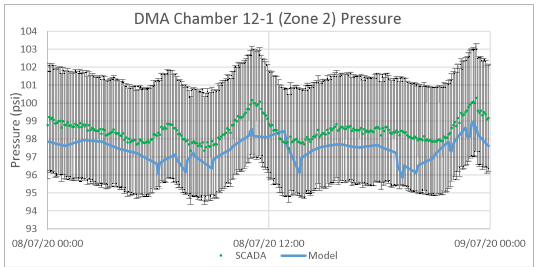
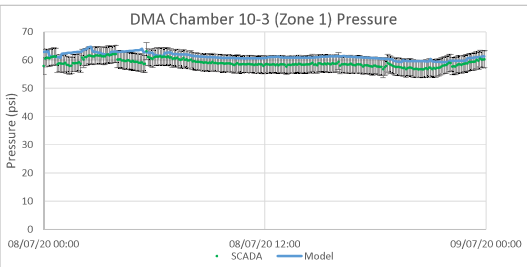
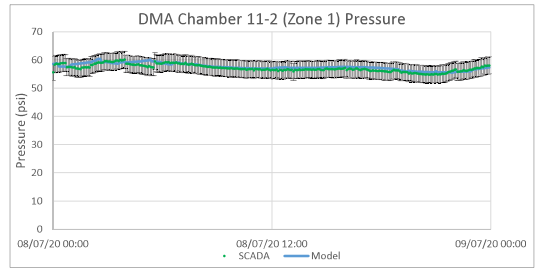
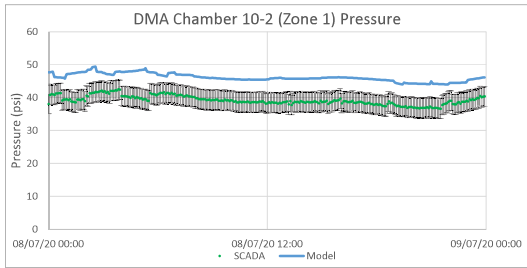
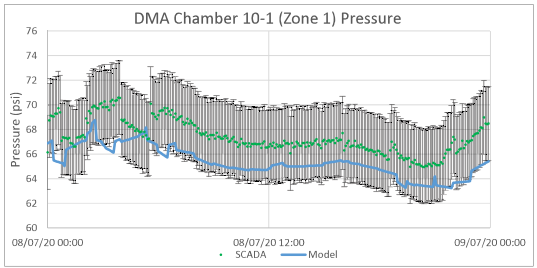
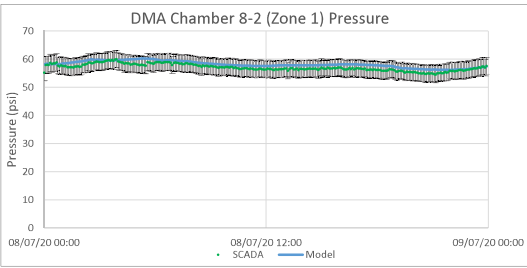
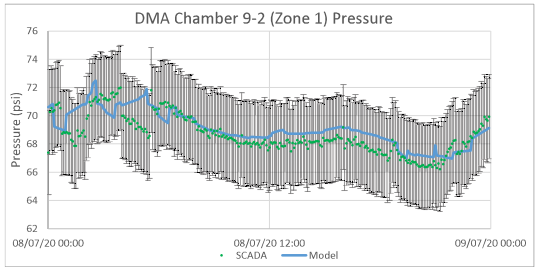
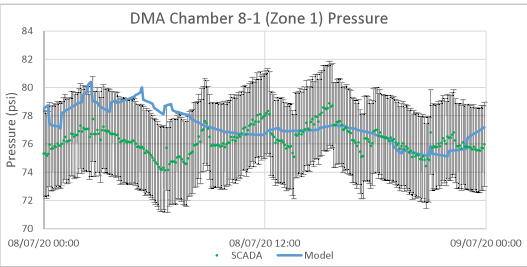
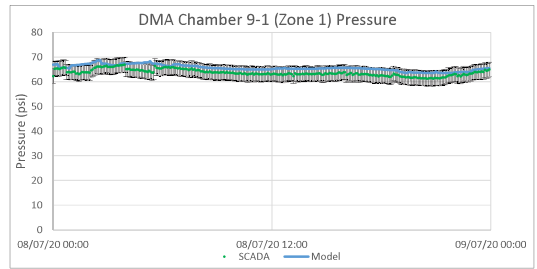
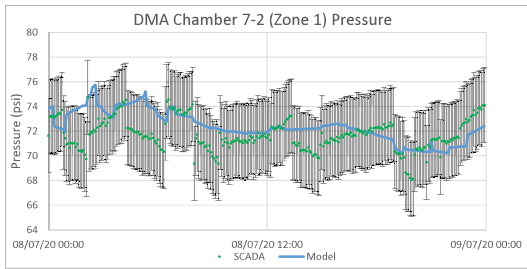


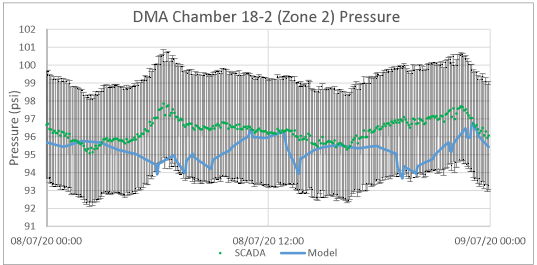
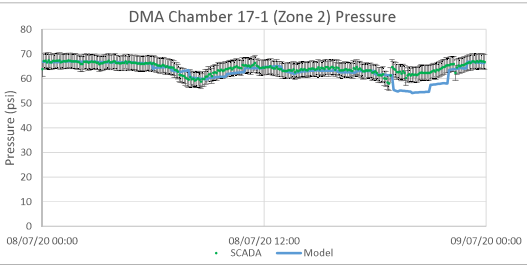
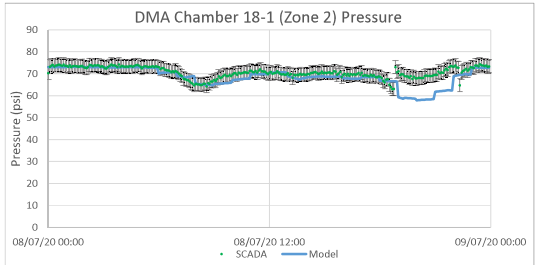
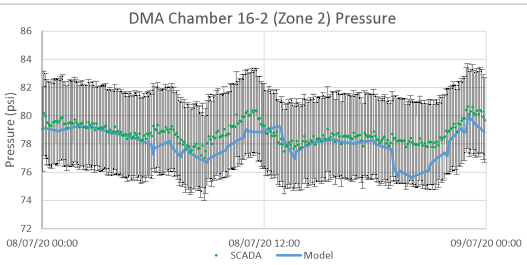
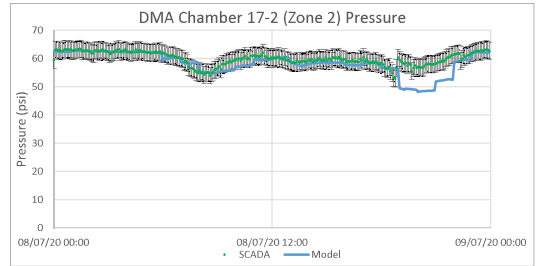
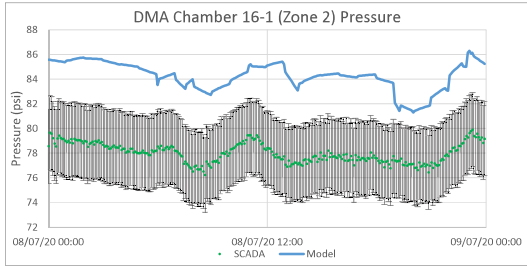
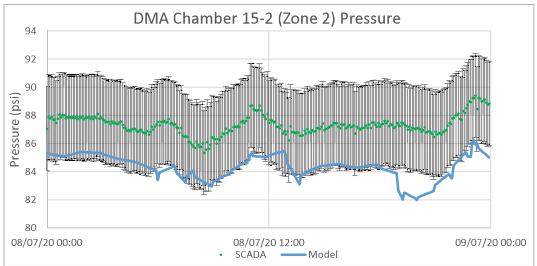
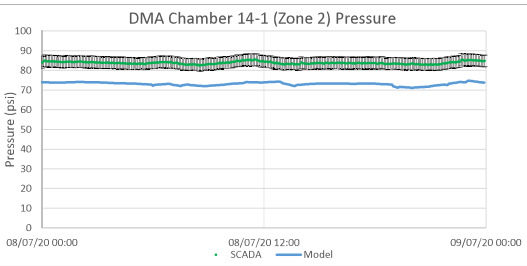
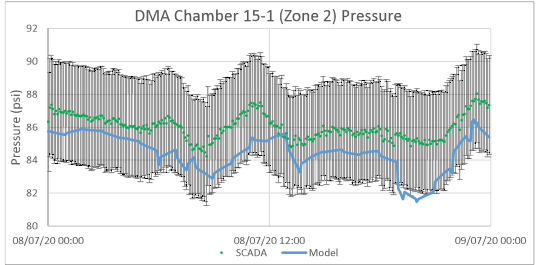
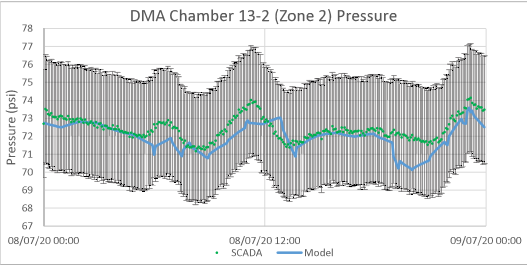
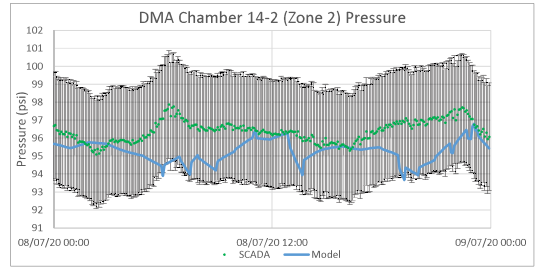
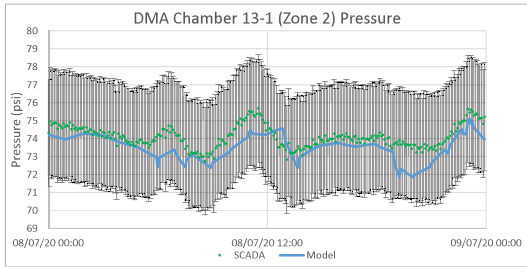




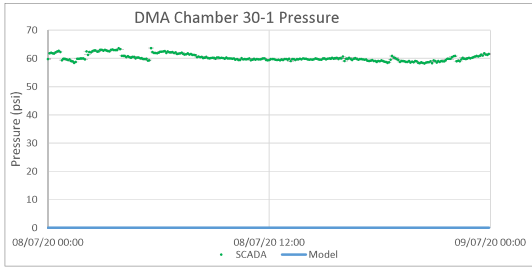
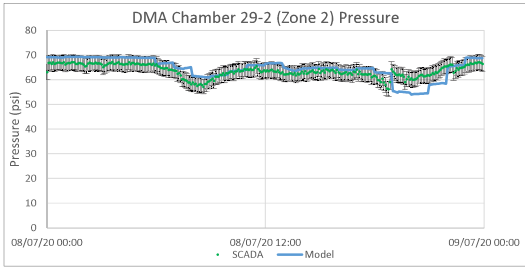
## **MDD – Dynamic Controls - Pressure**



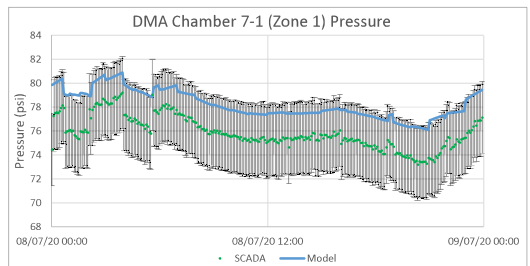
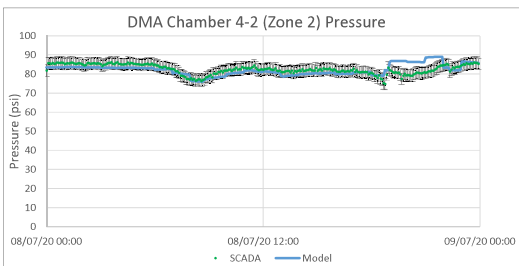
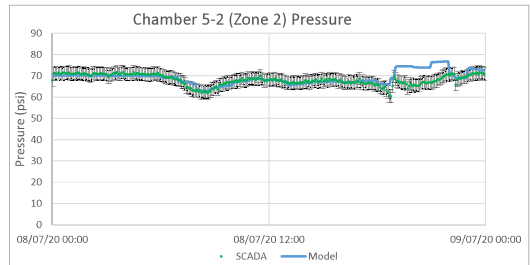
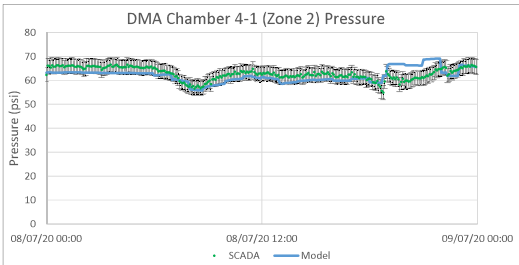
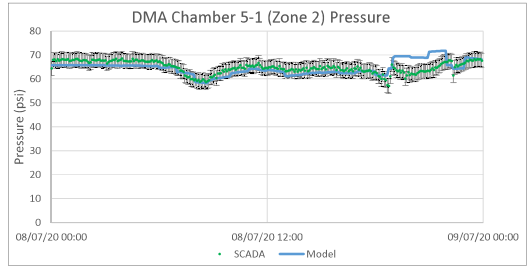
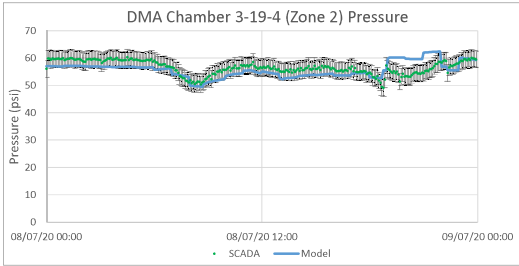
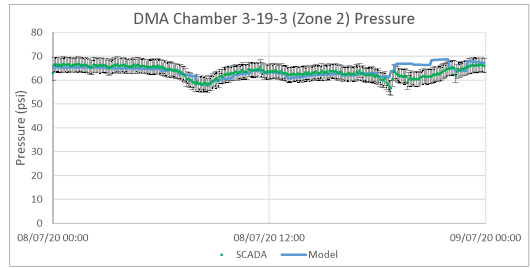
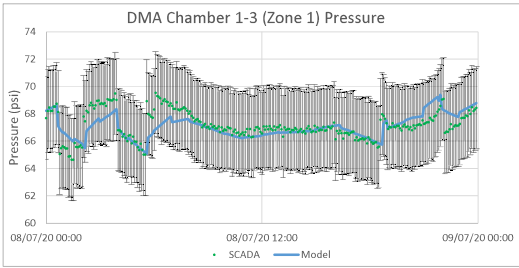
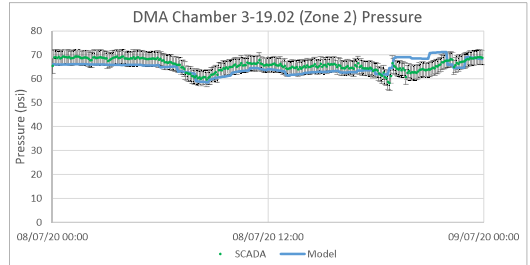
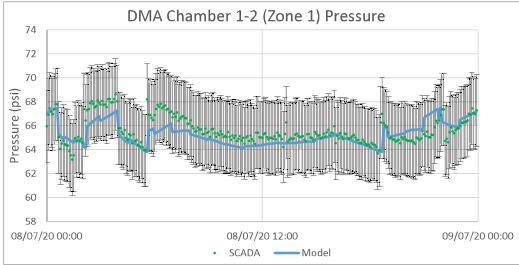
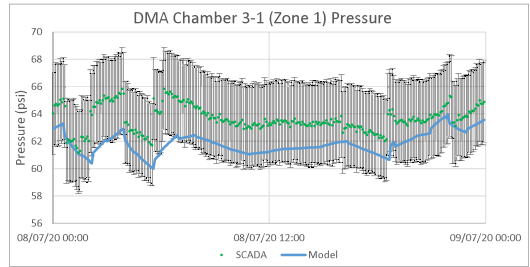
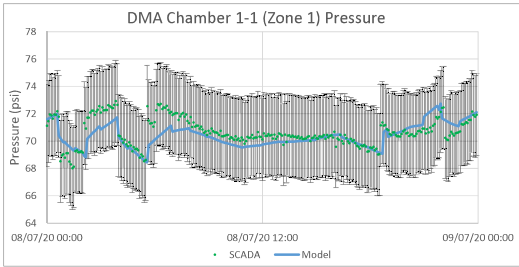




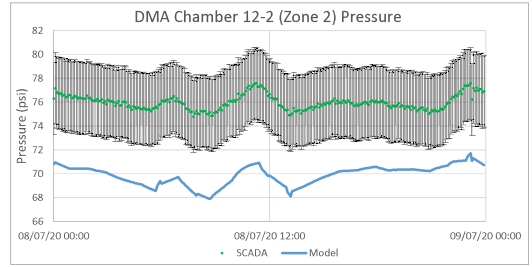
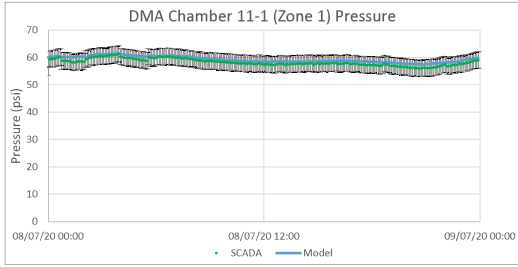
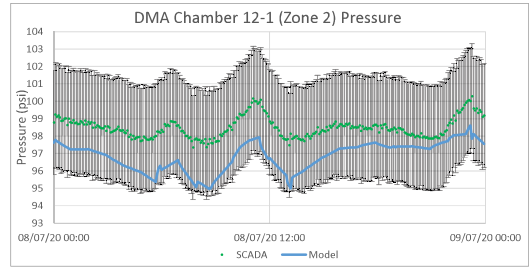
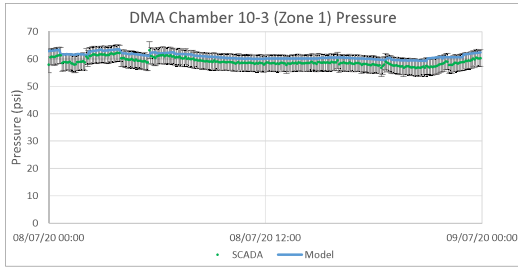
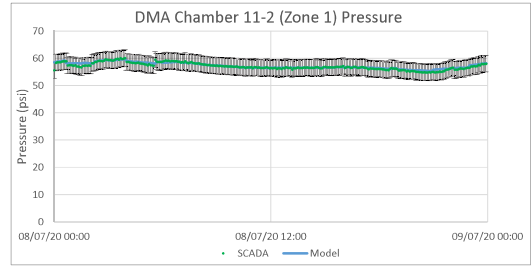
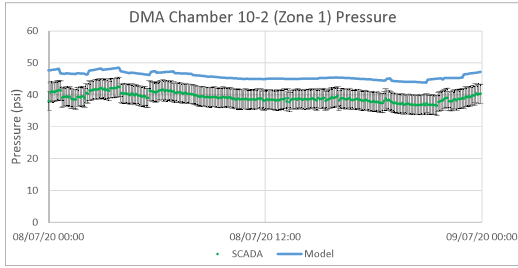
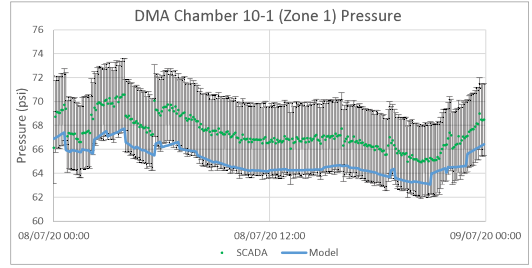
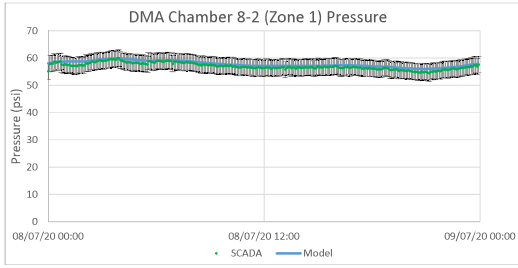
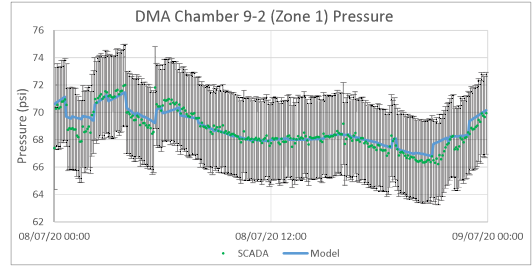
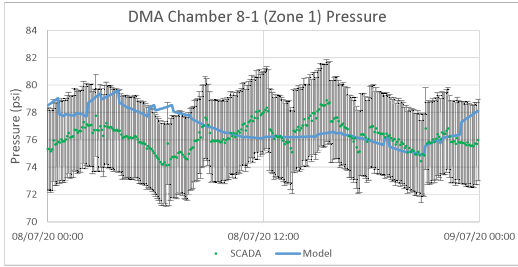
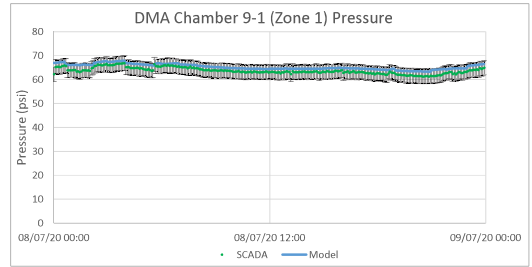
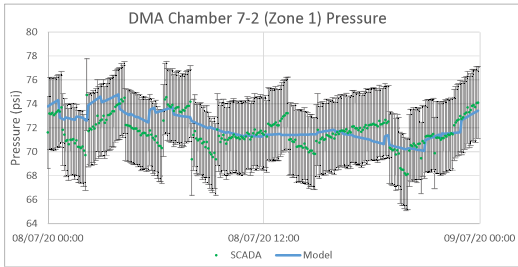


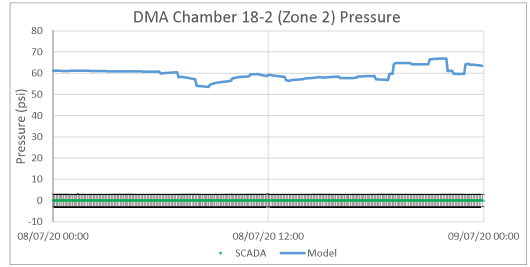
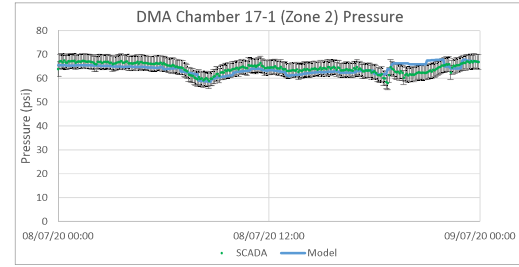
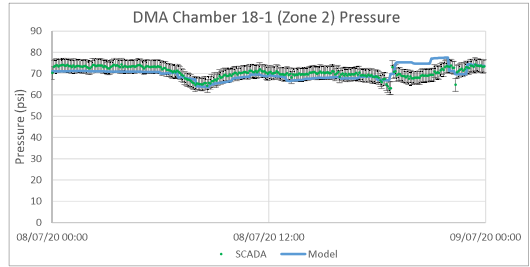
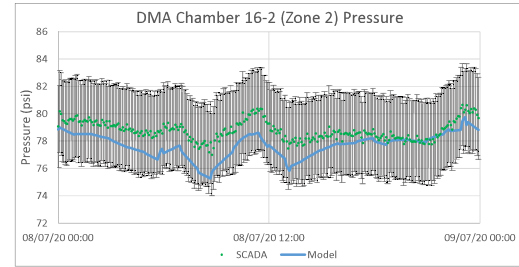
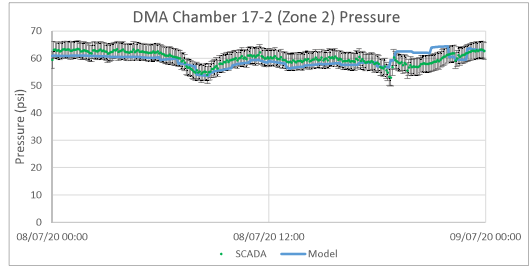
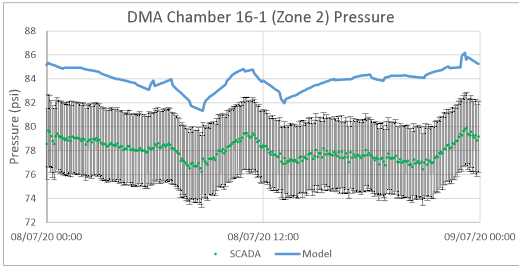
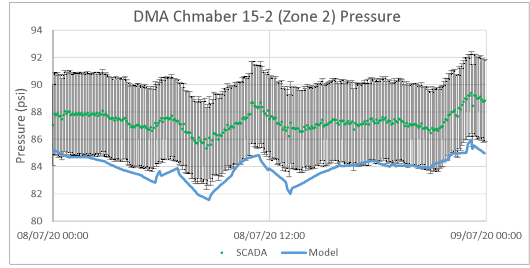
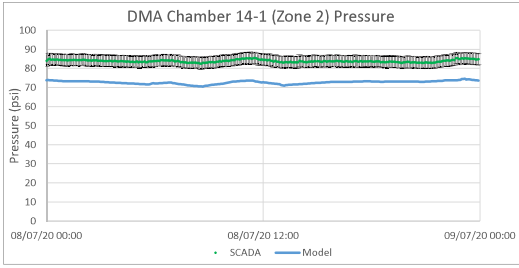
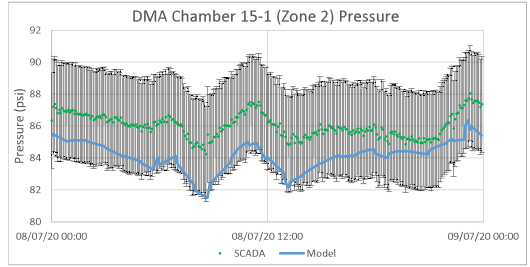
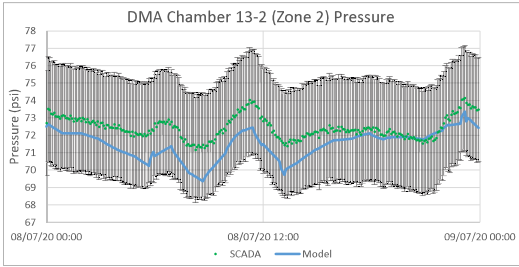
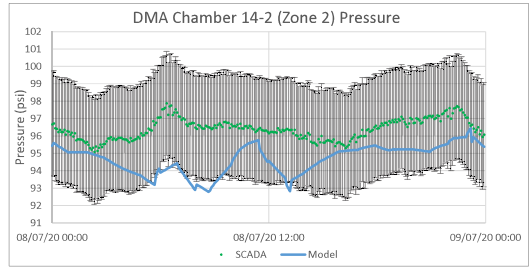
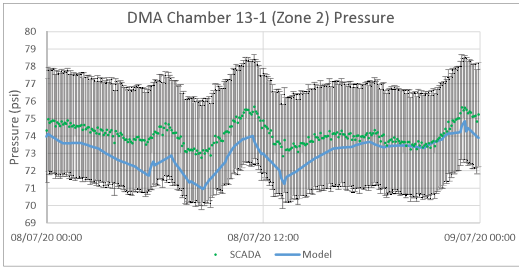


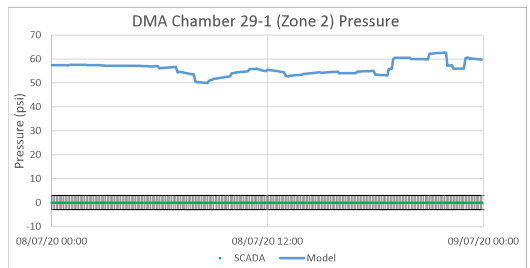
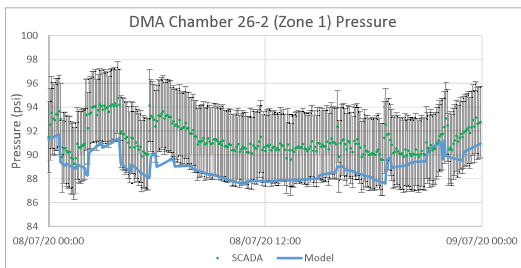
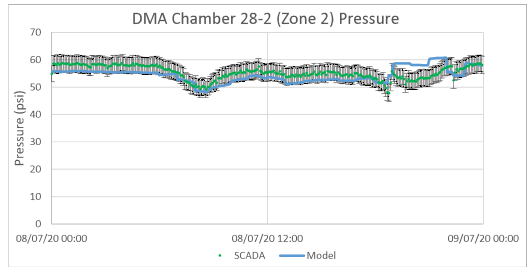
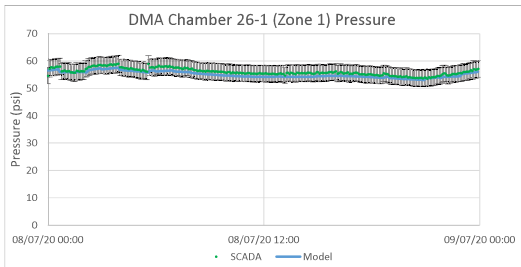
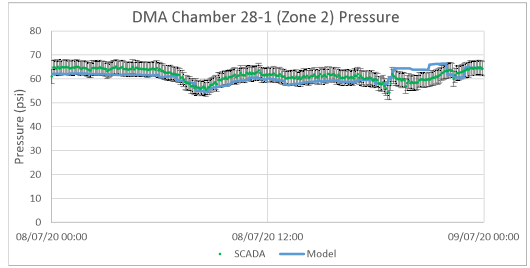
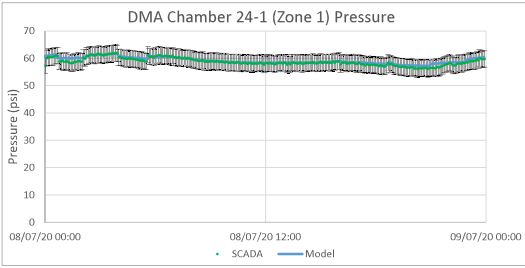
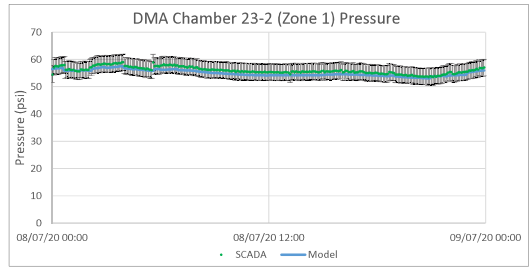
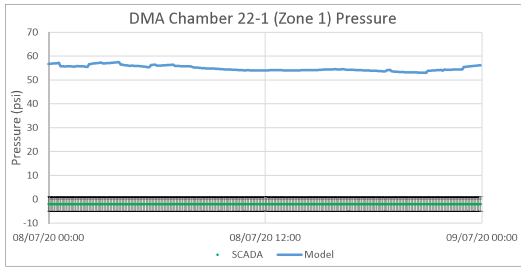
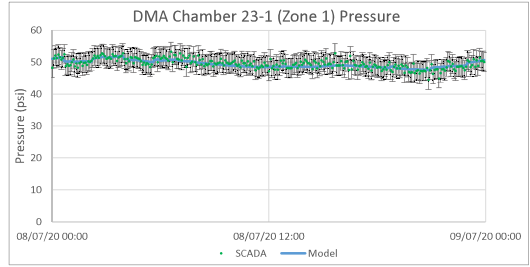
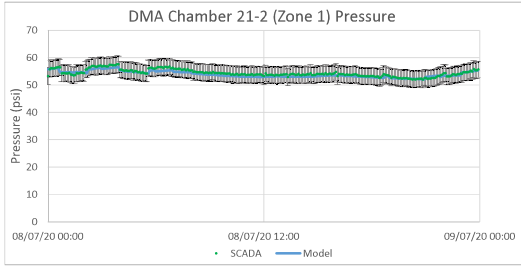
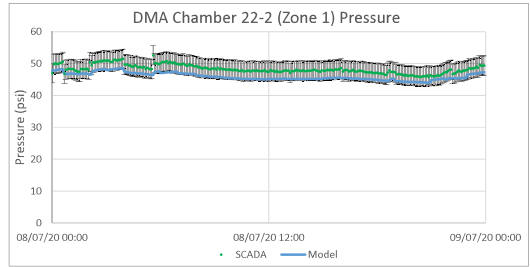
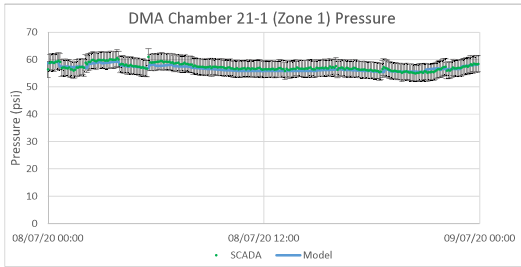
## MDD – Time Controls - Pressure





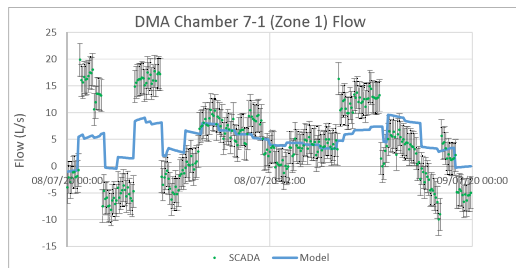
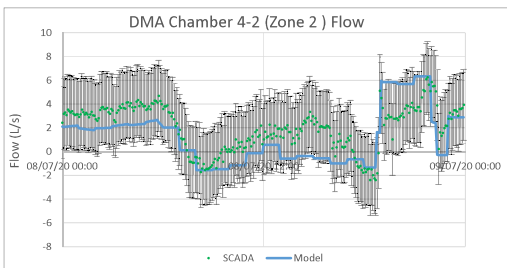
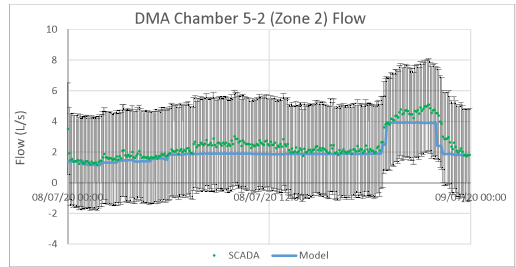
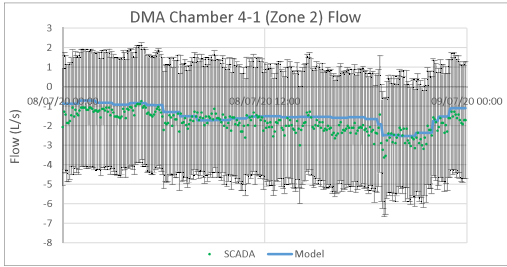
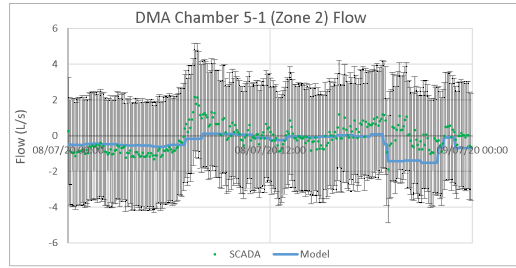
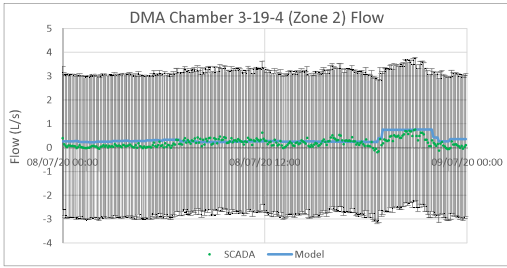
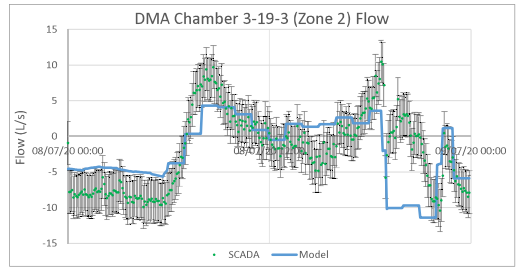
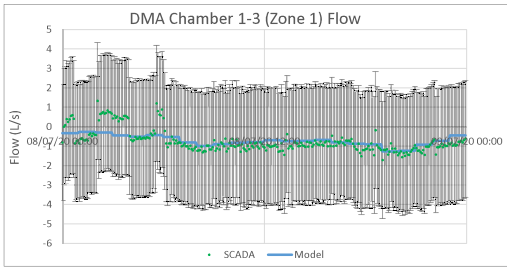
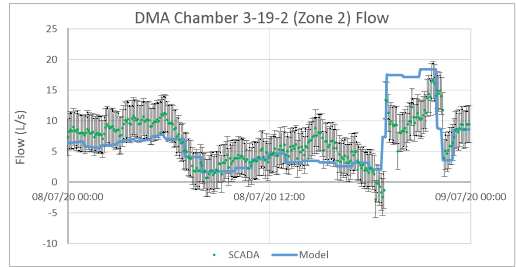
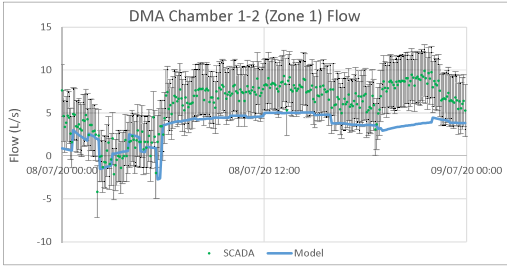
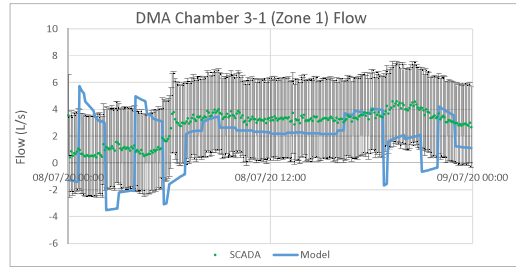
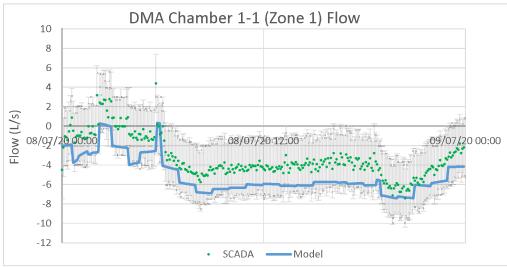


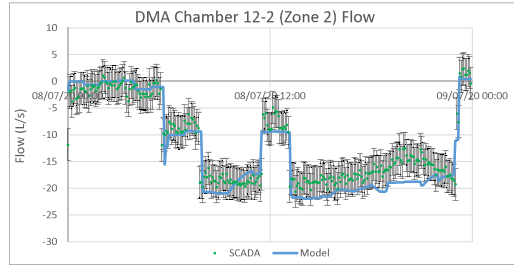
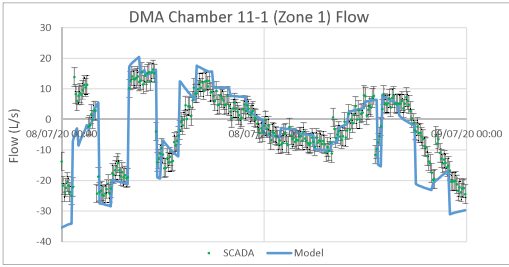
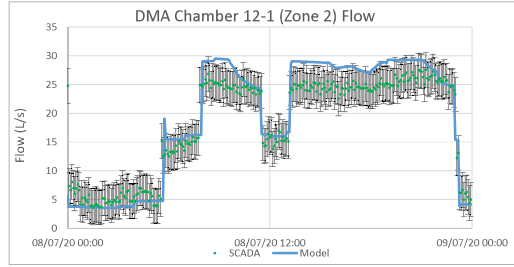
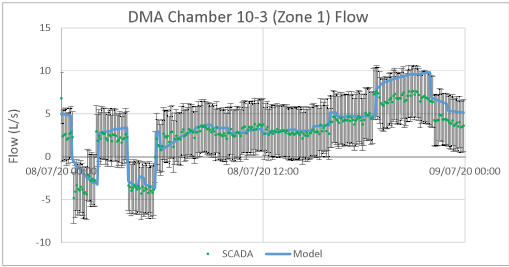
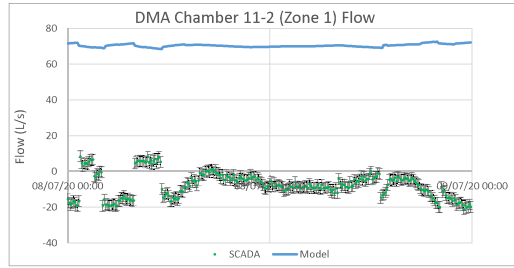
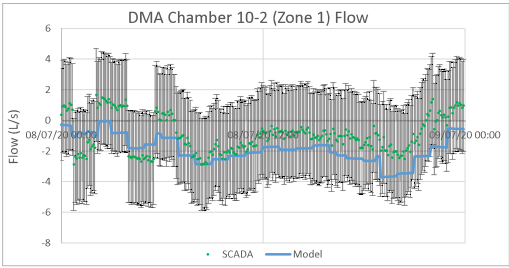
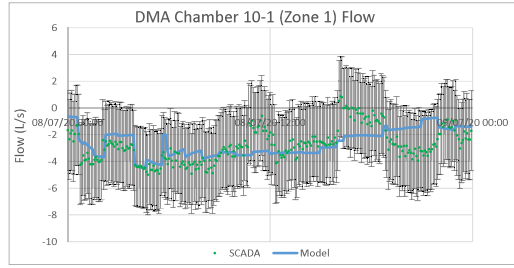
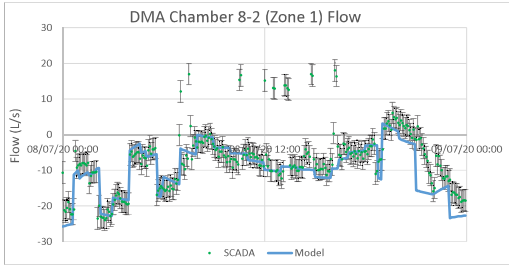
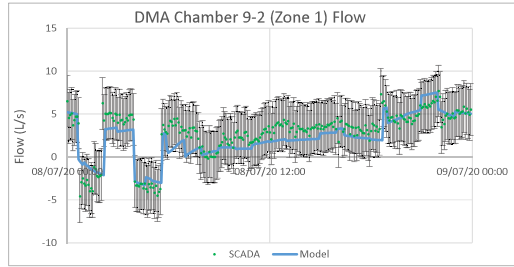
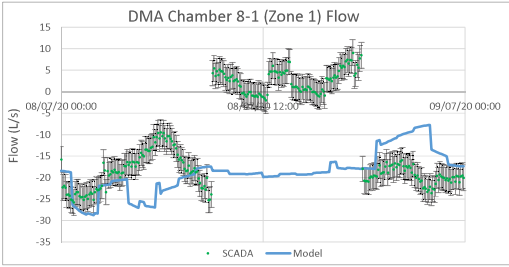
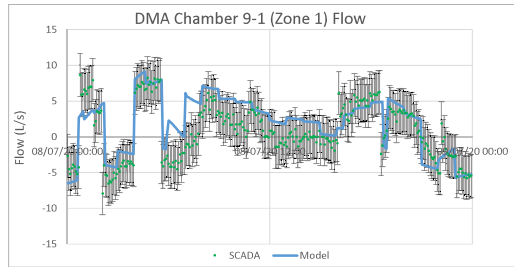
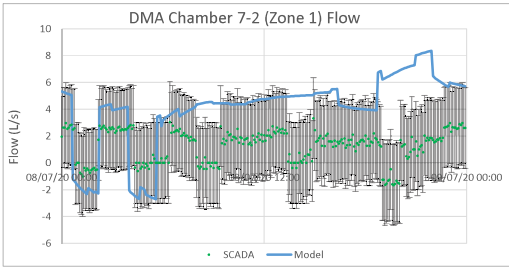


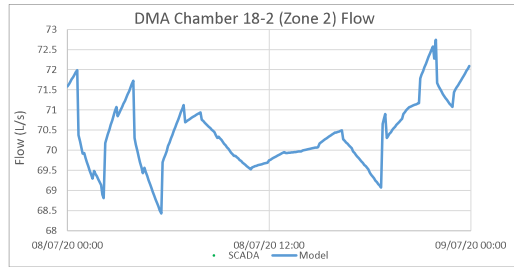
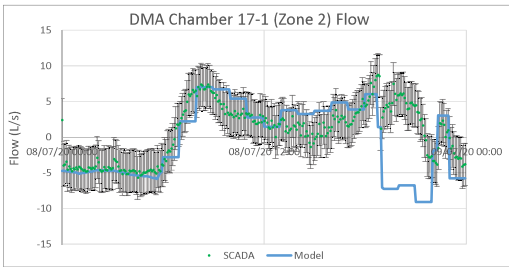
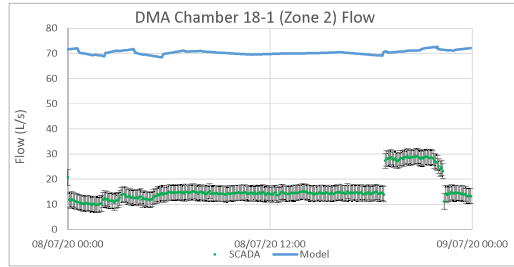
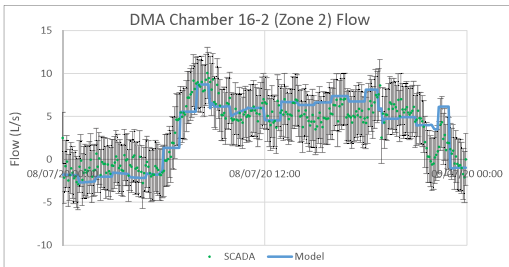
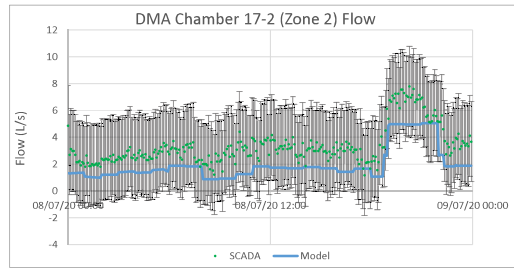
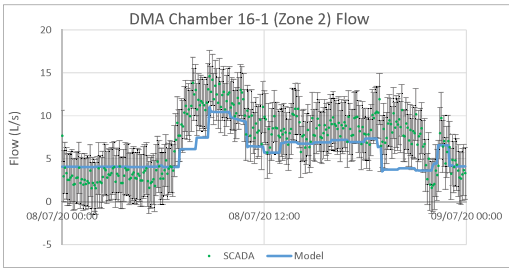
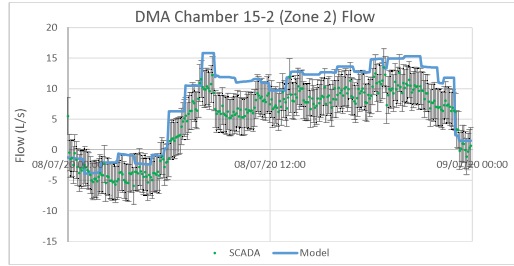
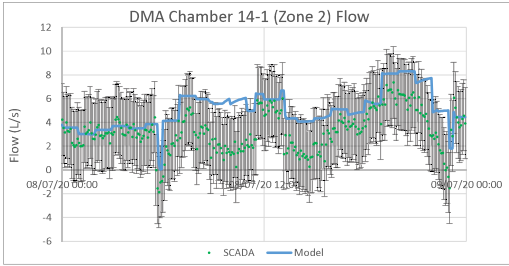
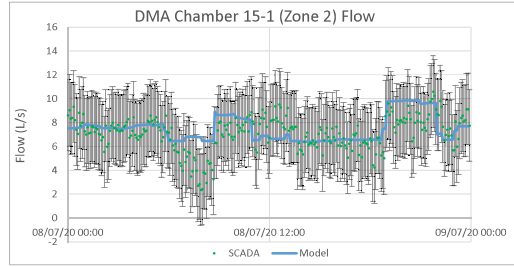
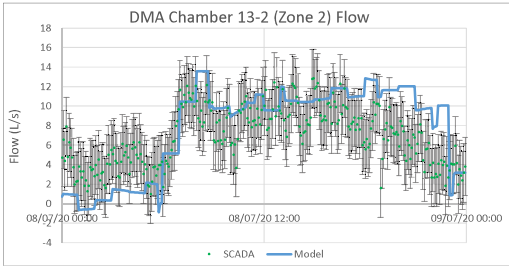
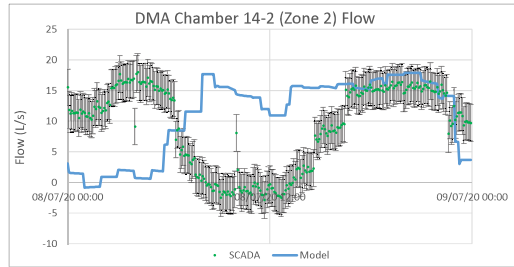
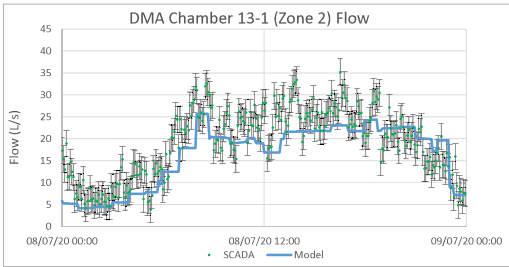




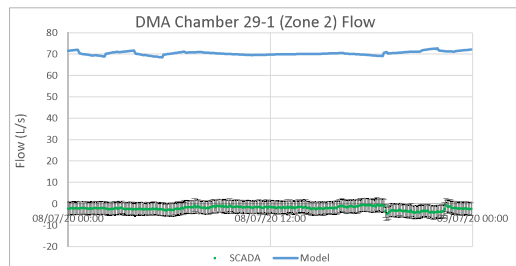
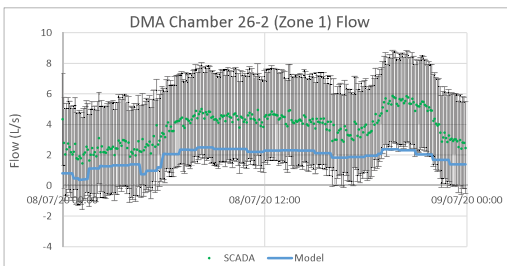
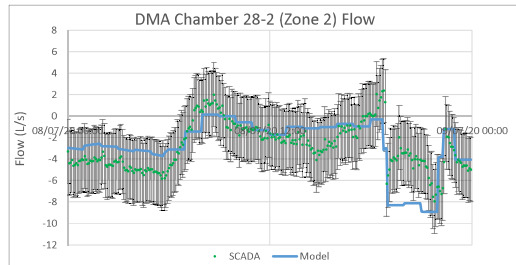
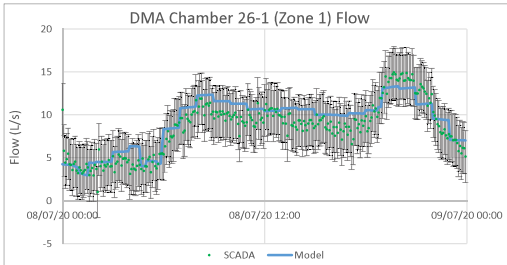
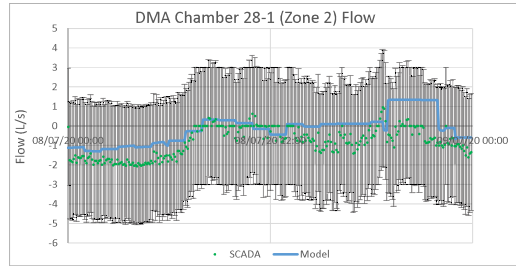
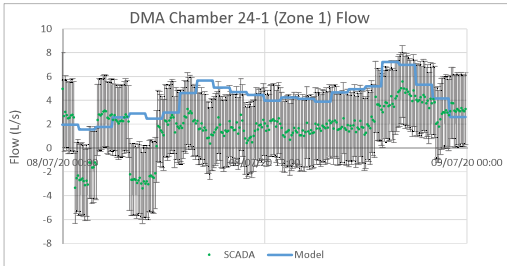
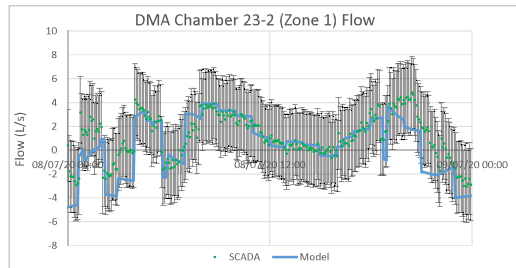
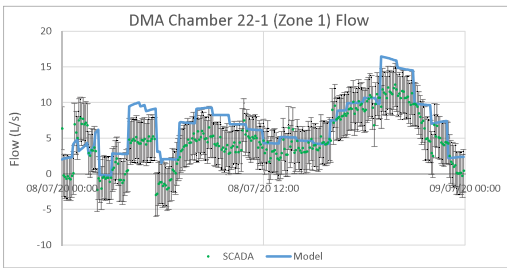
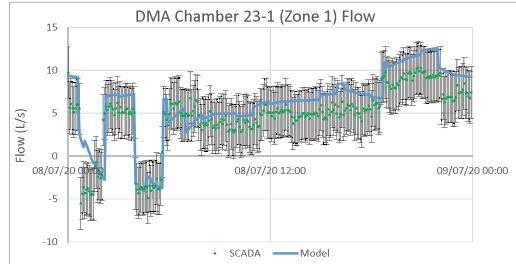
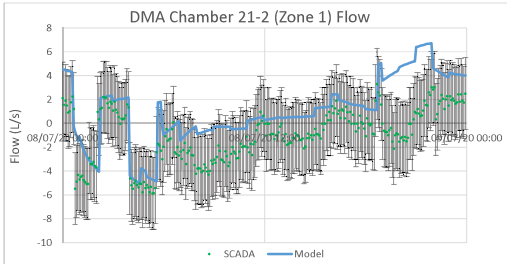
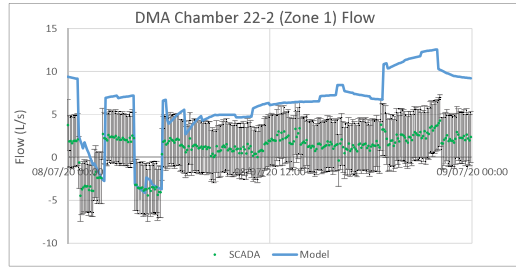
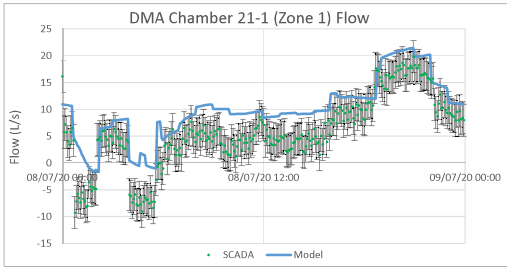
## **MDD – Time Controls - Flow**













## **APPENDIX B**

# **Wastewater Model Gap Analysis**



































































































































JUNCTION ID	ID_SOURCE	INVERT_SOURCE	RIM_SOURCE
DUMMY-US-START@9319	Dummy node created for missing pipe connectivity for WWSMP 2020	Infered from lowest connecting pipe invert	DEM received from City on May 19, 2020
DUMMY-US-START@9325	Dummy node created for missing pipe connectivity for WWSMP 2020	Infered from lowest connecting pipe invert	DEM received from City on May 19, 2020
DUMMY-US-START@9348	Dummy node created for missing pipe connectivity for WWSMP 2020	Infered from lowest connecting pipe invert	DEM received from City on May 19, 2020
DUMMY-US-START@9349	Dummy node created for missing pipe connectivity for WWSMP 2020	Infered from lowest connecting pipe invert	DEM received from City on May 19, 2020
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DUMMY-US-START@9356	Dummy node created for missing pipe connectivity for WWSMP 2020	Infered from lowest connecting pipe invert	DEM received from City on May 19, 2020
DUMMY-US-START@9385	Dummy node created for missing pipe connectivity for WWSMP 2020	Infered from lowest connecting pipe invert	DEM received from City on May 19, 2020
DUMMY-US-START@9419	Dummy node created for missing pipe connectivity for WWSMP 2020	Infered from lowest connecting pipe invert	DEM received from City on May 19, 2020
DUMMY-US-START@9515	Dummy node created for missing pipe connectivity for WWSMP 2020	Infered from lowest connecting pipe invert	DEM received from City on May 19, 2020
DUMMY-US-START@9545	Dummy node created for missing pipe connectivity for WWSMP 2020	Infered from lowest connecting pipe invert	DEM received from City on May 19, 2020
DUMMY-US-START@968	Dummy node created for missing pipe connectivity for WWSMP 2020	Infered from lowest connecting pipe invert	DEM received from City on May 19, 2020
NiMa_Trails_Temporary_SPS_Outlet	Dummy node created for missing pipe connectivity for WWSMP 2020	Infered from lowest connecting pipe invert	Engineering judgment for WWSMP 2020
Rockwood	Dummy node created for missing pipe connectivity for WWSMP 2020	Infered from lowest connecting pipe invert	#N/A
Terraview_SPS	Dummy node created for missing pipe connectivity for WWSMP 2020	Infered from lowest connecting pipe invert	Terraview SPS Drawings

## **APPENDIX C**

### **Capital Works Updates**

Project No	Project Description	Contract No	Contract Description	Date of Installation	Record Drawings Status	Model Pipe ID Start	Model Pipe ID End	Implemented in Model?
WW-I-2	Replace Stevenson Trunk	2-1005	Stevenson York Rd to Elizabeth	2010	Completed	8405	8423	Y
WW-I-1	Twinning and replacement of existing York Trunk from east of Hanlon to Victoria	2-1412	York Trunk Sewer and Paisley Clythe Watermain: From Waterloo Ave to across the Speed River	2015	Completed	9100	9078	Y
WW-I-2	Replace Stevenson Trunk	2-1515	Stevenson: Grange to Elizabeth	2015	Completed	9071	9281	Y
WW-I-21	Arthur Trunk from Elizabeth Street to York Trunk	2-1514	Stage 1: Wyndham St. Reconstruction from York Rd. to the bridge at Speed River	2016	Completed	9056	9060	Y
WW-I-2	Replace Stevenson Trunk	2-1609	Stevenson: Grange to Bennett	December 2016	Completed	9267	9278	Y
WW-I-21	Arthur Trunk from Elizabeth Street to York Trunk	2-1611	Howitt St. Reconstruction: Wyndham St. to Neeve St.	2017	Completed	9289	9283	Y
WW-I-3	Replace Speed Trunk from East of Hanlon to Yorkshire St S	2-1614	Bristol St. Reconstruction Ph. 1 from Edinburgh Rd. S. to East of Holliday St.	June 28, 2017	Completed	9241	9256	Y
WW-I-1	Twinning and replacement of existing York Trunk from east of Hanlon to Victoria	2-1606	York Trunk and Paisley-Clythe Watermain Phase 2A	January 2, 2018	Not Finished	9525	9543	Y
WW-I-19	Add connection to York Trunk from 1050 mm, along Waterworks Pl. from York Rd. to Royal Recreation Trail (part of contract 2-1606)	2-1606	York Trunk and Paisley-Clythe Watermain Phase 2A	January 2, 2018	Not Finished	4427	9545	N Drawings not available
WW-I-21	Arthur Trunk from Elizabeth Street to York Trunk	2-1708	Neeve St. and Cross St. Reconstruction Phase III: Howitt St. to Cross St. to Arthur St.	2018	Completed	9367	9360	Y Implemented in existing conditions model (siphon and river crossing maintained, but disconnected from upstream areas)
WW-I-3	Replace Speed Trunk from East of Hanlon to Yorkshire St S	2-1723	Bristol Street Trunk Sewer - Phase 2 - Yorkshire / Wellington	November 30, 2018	Not Finished	9455	9469	Y
WW-I-4	Paisley Feedermain Project: Sanitary sewers on Silvercreek Pkwy. S. converge with Waterloo Trunk sewer and drain to the same outlet [Hanlon Crossing]	2-1812	Paisley Feedermain: From Waterloo Avenue to Paisley Road	March 22, 2019	Not Finished	-	-	N Drawings not available
WW-I-4	Paisley Feedermain Project: Sanitary sewers on Silvercreek Pkwy. S. converge with Waterloo Trunk sewer and drain to the same outlet [Hanlon Crossing]	2-1905	Paisley Feedermain Phase II Silvercreek to Reservoir	April 12, 2019	Not Finished	-	-	N Drawings not available
WW-I-1	Twinning and replacement of existing York Trunk from east of Hanlon to Victoria	2-1717	York Trunk Sewer Phase 2B: Waterworks Pl to Victoria Rd S	January 2020	Not Finished	4477	4302	Y Implemented in existing conditions model
WW-I-21	Arthur Trunk from Elizabeth Street to York Trunk	2-1908	Stage 5: Arthur St. Reconstruction from MacDonell St. to 170m South on Arthur St.	September 2020	Not Finished	3837	3828	Y Implemented in existing conditions model (siphon and river crossing maintained, but disconnected from upstream areas)
WW-I-4	Paisley Feedermain Project: Sanitary sewers on Silvercreek Pkwy. S. converge with Waterloo Trunk sewer and drain to the same outlet [Hanlon Crossing]	-	Paisley Phase III- Hanlon to Reservoir	2021	Not Finished	-	-	N Drawings not available



Project No	Project Description	Contract No	Contract Description	Date of Installation	Record Drawings Status	Model Pipe ID Start	Model Pipe ID End	Implemented in Model?
WW-I-7	Speedvale Collector from Arthur Trunk to Metcalf	PN0097	Speedvale Ph1 Glenwood to Marlborough	2021	-	-	-	N Drawings not available
WW-I-2	Replace Stevenson Trunk	2-2006	Stevenson: Eramosa to Bennett	June 2021	Not Finished	4060	4051	N Drawings not available
WW-I-7	Speedvale Collector from Arthur Trunk to Metcalf	PN0097	Speedvale Ph2 Marlborough to beyond Delhi	2022	-	-	-	N Drawings not available
WW-I-7	Speedvale Collector from Arthur Trunk to Metcalf	PN0097	Speedvale Ph3 Delhi to Manhattan Ct	2023	-	-	-	N Drawings not available
WW-I-8	Replace Water St Collector	PN0102	Water- Maple / Gordon (Ww-I-8) Water St from Maple St to Gordon St	2023	-	-	-	N Drawings not available
W-I-1A	Add parallel pipe on Wellington St W	PN0107	Parallel Pipe East of Hanlon to Wastewater Treatment Plant	2025	-	-	-	N Drawings not available
WW-I-5	Replace Yorkshire Trunk (including Trunk Relief Lines)	PN0090	-	2025	-	-	-	N Drawings not available
WW-I-5	Replace Yorkshire Trunk (including Trunk Relief Lines)	PN0091	-	2027	-	-	-	N Drawings not available
WW-I-5	Replace Yorkshire Trunk (including Trunk Relief Lines)	PN0092	-	2028	-	-	-	N Drawings not available
WW-I-12	Siphon improvements (2 siphons 450 mm in diam.)- Edinburgh Rd. S. to Royal recreation Trail (Crossing wellington St. W. and Speed River) to Bristol	-	-	2028	-	-	-	N Drawings not available
WW-I-5	Replace Yorkshire Trunk (including Trunk Relief Lines)	PN0093	-	2029	-	-	-	N Drawings not available
WW-I-5	Replace Yorkshire Trunk (including Trunk Relief Lines)	PN0094	-	2030	-	-	-	N Drawings not available
WW-I-5	Replace Yorkshire Trunk (including Trunk Relief Lines)	PN0095	-	2031	-	-	-	N Drawings not available
WW-I-4	Replace Waterloo Trunk from Yorkshire St S to East of Hanlon	PN0103	Waterloo Street- Silvercreek / Yorkshire	2034	Not Finished	-	-	N Drawings not available
WW-I-20	Monticello Cr. From north of Stone Rd. E to Dimson Av.	PN0089	Montcll:Stevenson / Dmsn Avenue (Ww-I-20)	2038	-	-	-	N Drawings not available
WW-I-3	Replace Speed Trunk from East of Hanlon to Yorkshire St S	PN0073	Bristol Street Trunk Sewer Upgrades	On Hold	Not Finished	-	-	N Drawings not available
WW-I-18	Upsize pipe along Yorkshire St. N from Bristol St. to Waterloo Ave	2-1723	No road reconstruction done, only some pipe work was done on Holliday St. as part of 2-1723 Bristol Ph2	-	-	-	-	N Drawings not available
WW-I-21	Arthur Trunk from Elizabeth Street to York Trunk	2-1804	Arthur St. Reconstruction from Elizabeth St. to Cross St. (Watermain only)	-	Draft	9502	9511	Y

## WW-I-1 Twinning and replacement of existing York Trunk from east of Hanlon to Victoria

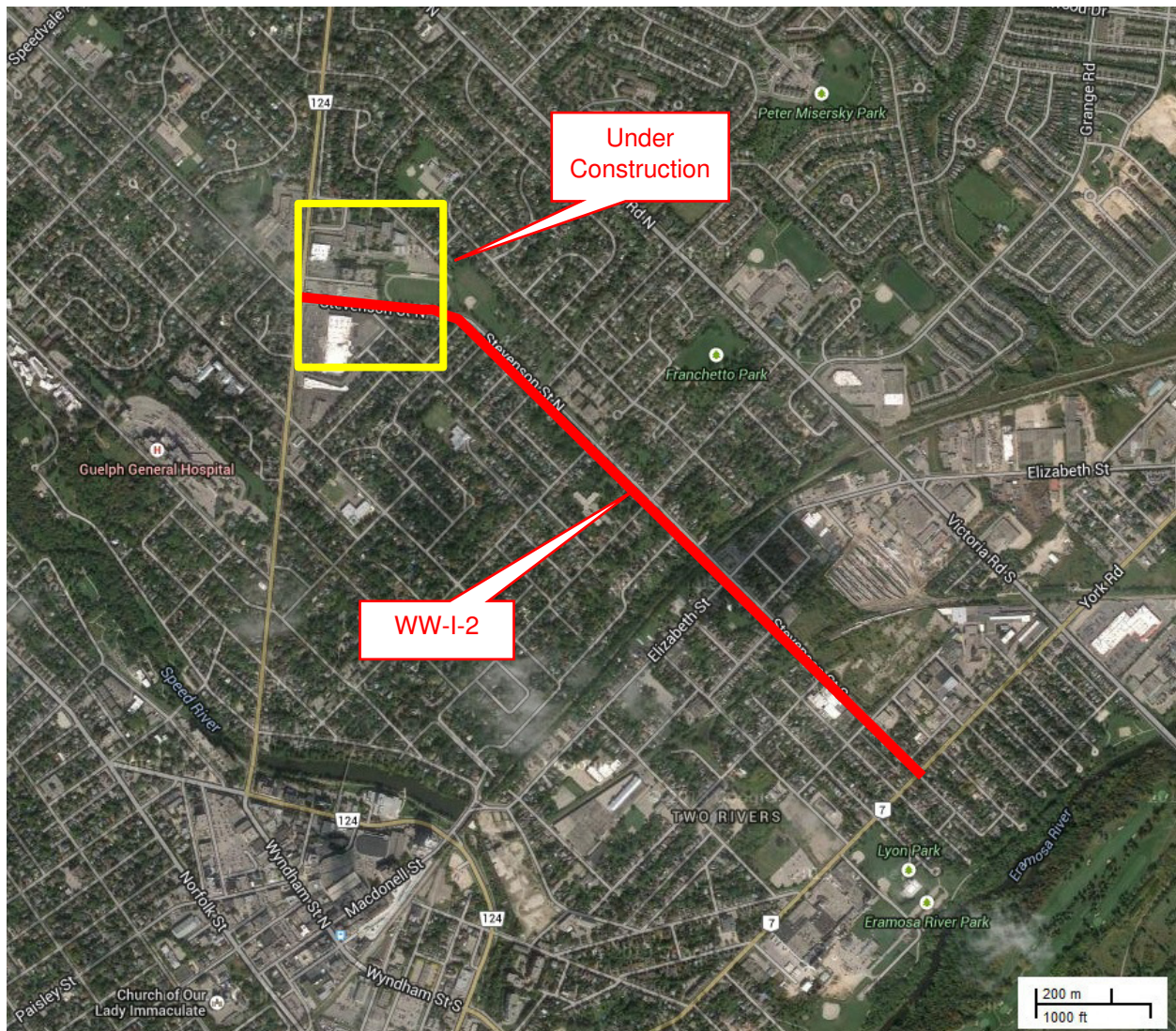


- 2-1412 York Trunk Sewer and Paisley Clyde Watermain: From Waterloo Ave to across the Speed River (Installed in 2015)
- 2-1606 York Trunk and Paisley-Clythe Watermain Phase 2A ('Live' since January 2<sup>nd</sup>, 2018) Record Drawings not finished
- 2-1717 York Trunk Sewer Phase 2B: Waterworks PI to Victoria Rd S ('Live' since January 2020) Record Drawings not finished

## WW-I-19 Add connection to York Trunk from 1050 mm, along Waterworks Pl. from York Rd. to Royal Recreation Trail (part of contract 2-1606)

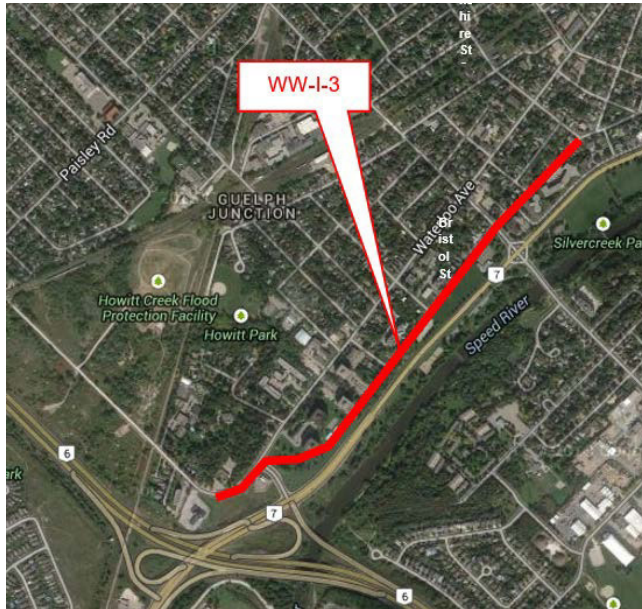


## WW-I-2 Replace Stevenson Trunk



- 2-2006 Stevenson: Eramosa to Bennett- **Approximate completion: June 2021**
- 2-1609: Stevenson: Grange to Bennett, **(installed in Dec 2016)**
- 2-1515: Stevenson: Grange to Elizabeth **(installed in 2015)**
- 2-1005: Stevenson York Rd to Elizabeth **(installed in 2010)**

### WW-I-3 Replace Speed Trunk from East of Hanlon to Yorkshire St S



**Western Portion:** Bristol Street Trunk Sewer Upgrades (WW-I-3) PN0073: On hold until Master Plan is updated- Alignment as shown in right of way is not feasible with current road profile [not enough fall on pipe to provide adequate cover]

#### **Eastern Portion:**

2-1614 Bristol St. Reconstruction Ph. 1 from Edinburgh Rd. S. to East of Holliday St. ('Live' in June 28<sup>th</sup>, 2017)  
2-1723 Bristol Street Trunk Sewer - Phase 2 - Yorkshire / Wellington ('Live' in November 30<sup>th</sup>, 2018) Record Drawings not finished

### WW-I-4 Replace Waterloo Trunk from Yorkshire St S to East of Hanlon

PN0103 Waterloo Street- Silvercreek / Yorkshire Planned 2034

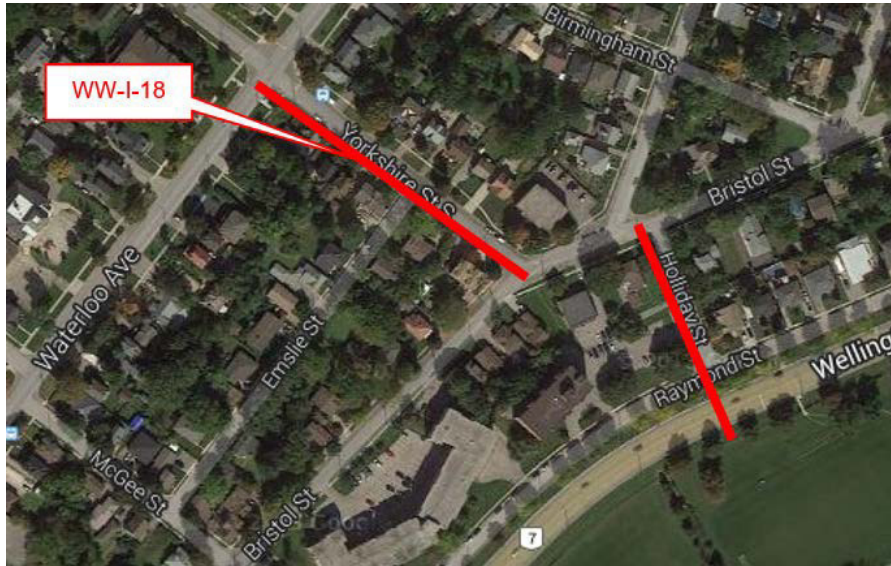


**Paisley Feedermain Project:** Sanitary sewers on Silvercreek Pkwy. S. converge with Waterloo Trunk sewer and drain to the same outlet [Hanlon Crossing]

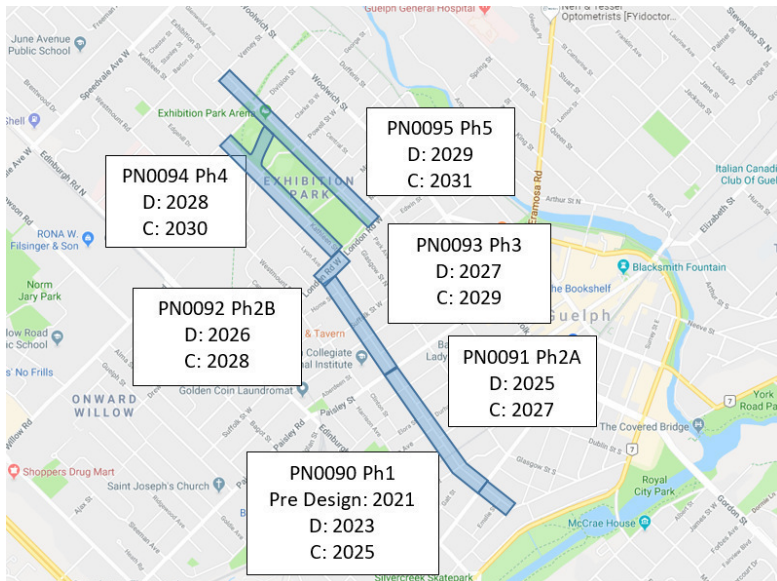
- Paisley Phase III- Hanlon to Reservoir (Construction planned for 2021)
- 2-1905 Paisley Feedermain Phase II Silvercreek to Reservoir (Completion April 12<sup>th</sup>, 2019) Record Drawings not finished
- 2-1812 Paisley Feedermain: From Waterloo Avenue to Paisley Road ('Live' since March 22<sup>nd</sup>, 2019) Record Drawings not finished

## WW-I-18 Upsize pipe along Yorkshire St. N from Bristol St. to Waterloo Ave

No road reconstruction done, only some pipe work was done on Holliday St. as part of 2-1723 Bristol Ph2

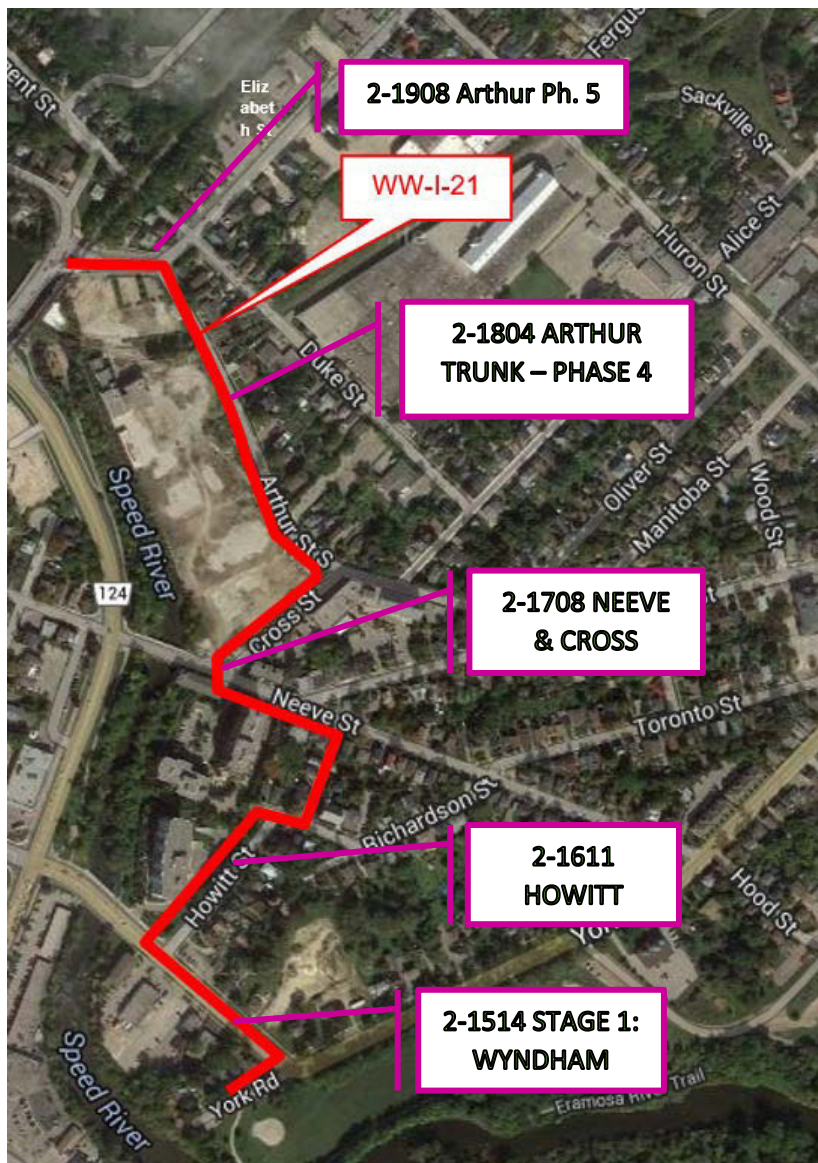


## WW-I-5 Replace Yorkshire Trunk (including Trunk Relief Lines) – **Planned**



## WW-I-21 Arthur Trunk from Elizabeth Street to York Trunk

- 2-1908 Stage 5: Arthur St. Reconstruction from MacDonell St. to 170m South on Arthur St. ('Live' September 2020) Record drawings not finished
- 2-1804 Stage 4: Arthur St. Reconstruction from Elizabeth St. to Cross St. (Watermain only) Draft As-built
- 2-1708 Neeve St. and Cross St. Reconstruction Phase III: Howitt St. to Cross St. to Arthur St. (Installed in 2018)
- 2-1611 Howitt St. Reconstruction: Wyndham St. to Neeve St. (Installed in 2017)
- 2-1514 Stage 1: Wyndham St. Reconstruction from York Rd. to the bridge at Speed River (Installed in 2016)

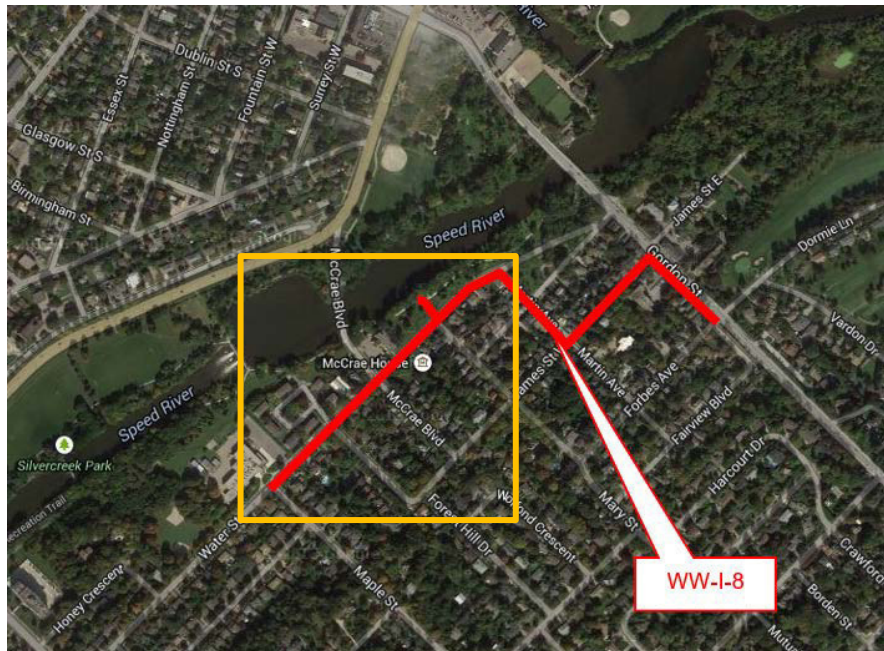


## WW-I-7 Speedvale Collector from Arthur Trunk to Metcalf (Planned)



## WW-I-8 Replace Water St Collector

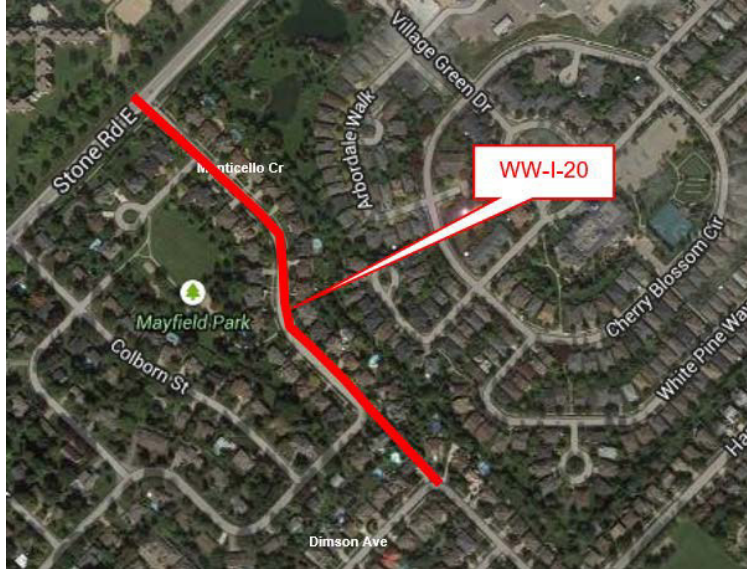
PN0102 Water- Maple / Gordon (Ww-I-8) Water St from Maple St to Gordon St  
Western portion planned for 2023



**WW-I-20 Monticello Cr. From north of Stone Rd. E to Dimson Av.**

PN0089 Montcll:Stevenson / Dmsn Avenue (Ww-I-20)

Construction planned for 2038



**W-I-1A Add parallel pipe on Wellington St W**

PN0107- Parallel Pipe East of Hanlon to Wastewater Treatment Plant (Design: 2023 Construction: 2025)

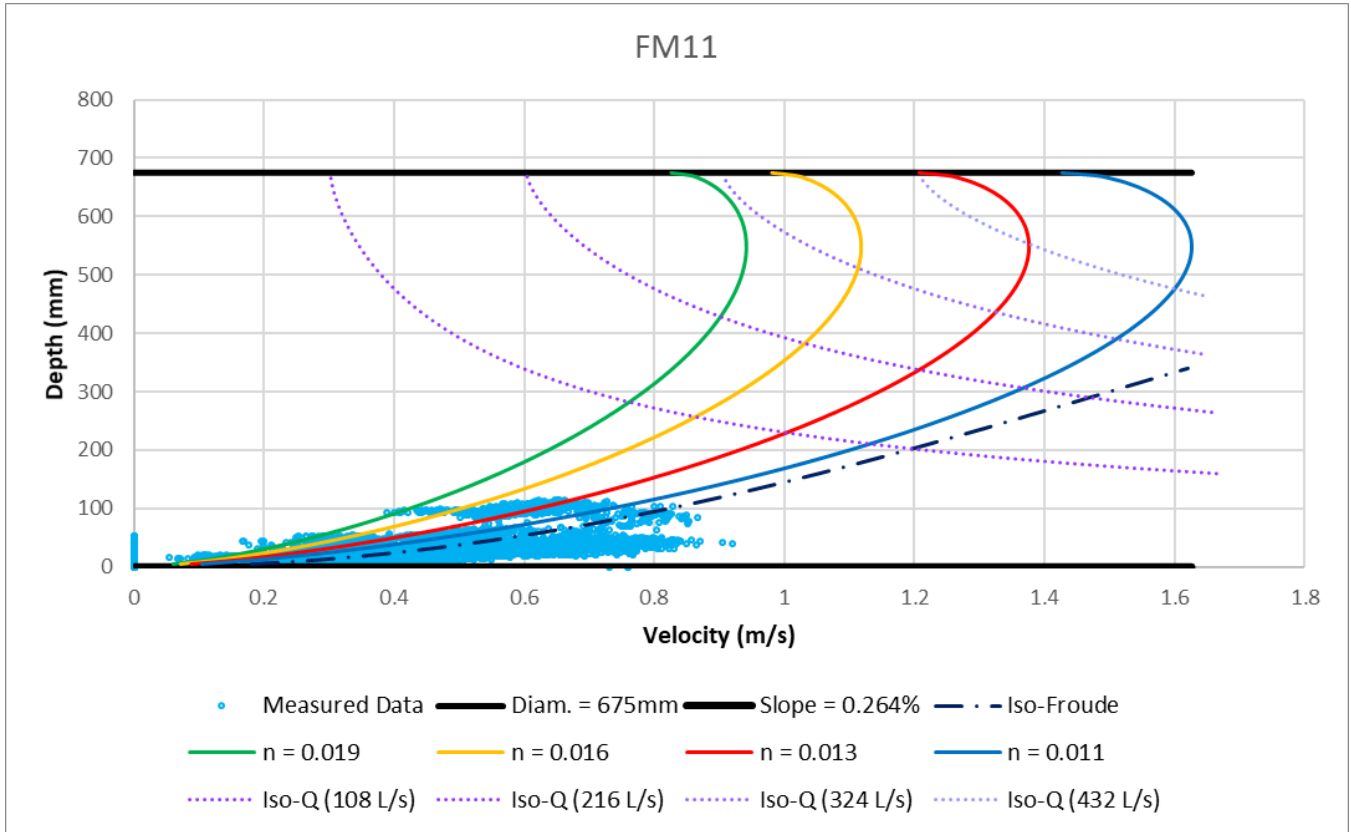
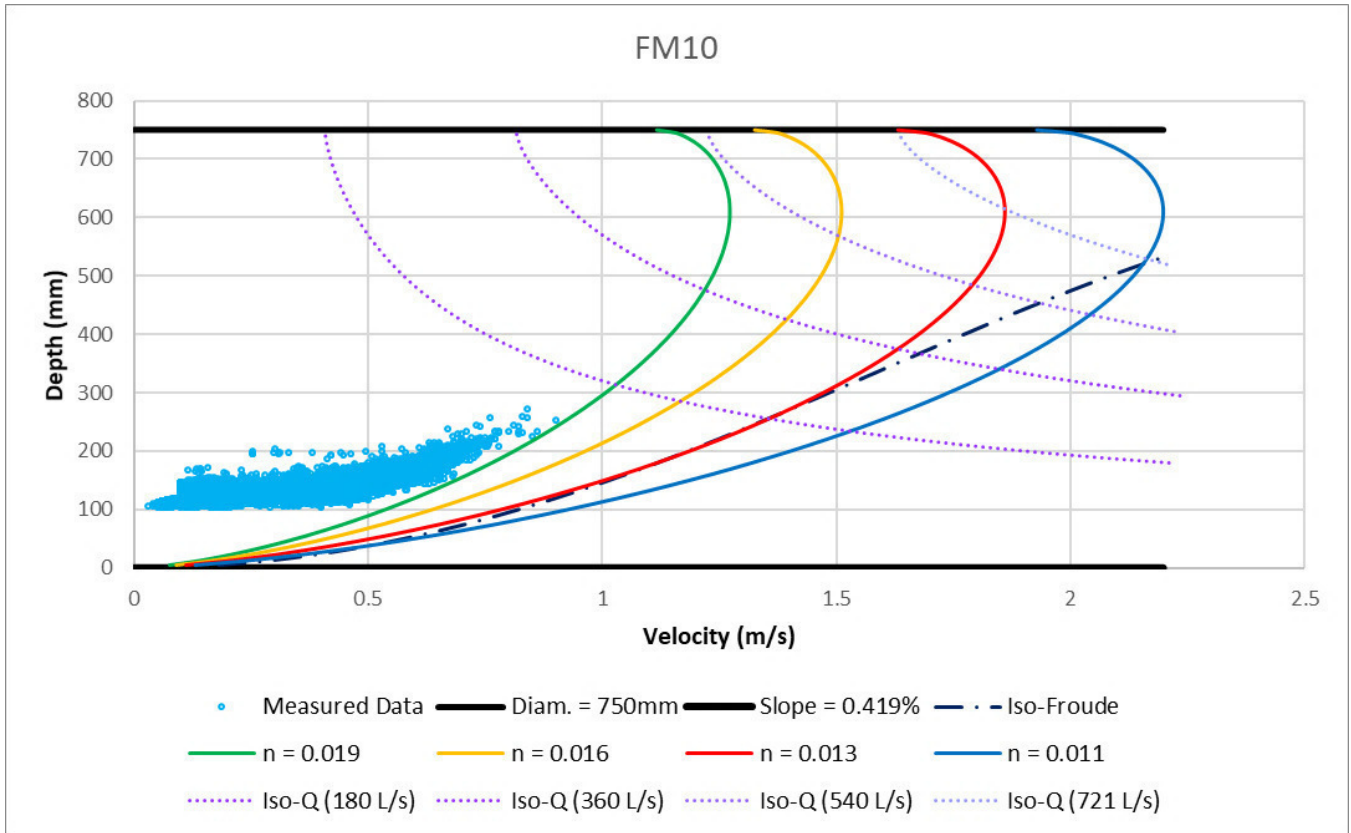
**WW-I-12 Siphon improvements (2 siphons 450 mm in diam.)- Edinburgh Rd. S. to Royal recreation Trail (Crossing wellington St. W. and Speed River) to Bristol Construction planned for 2028**



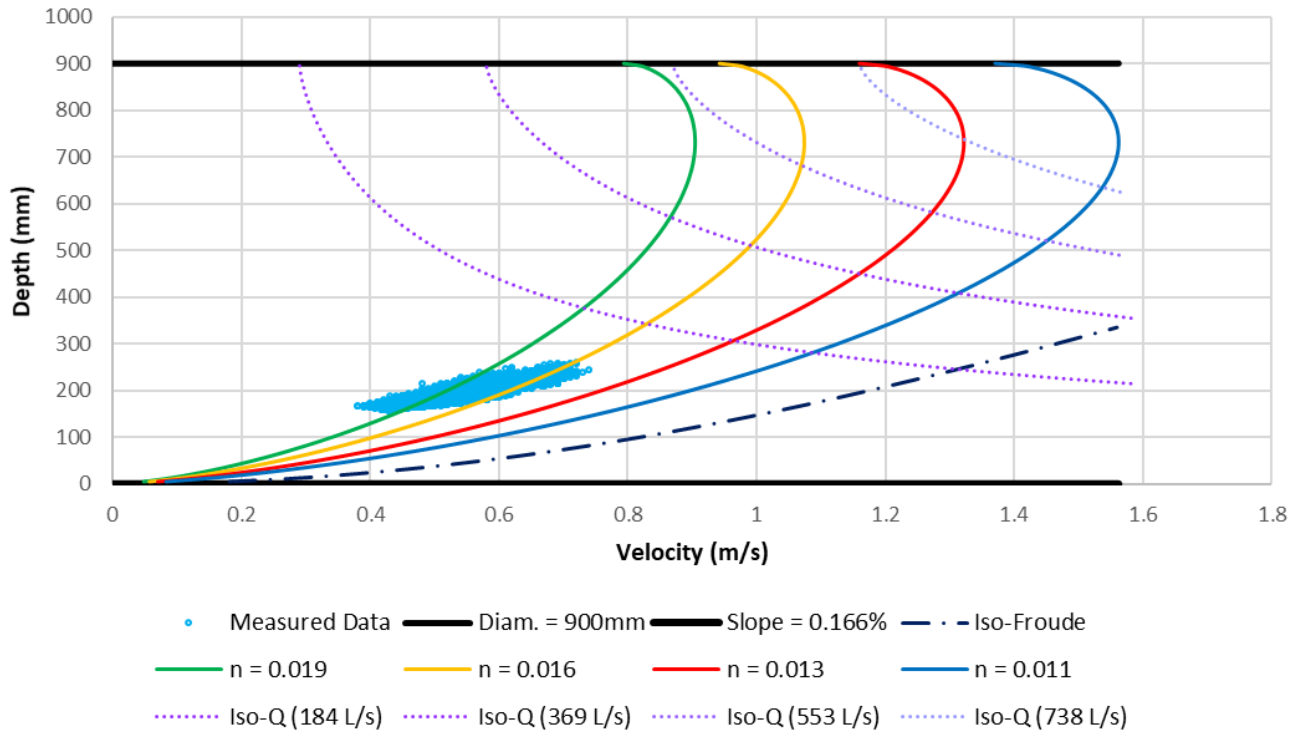


## **APPENDIX D**

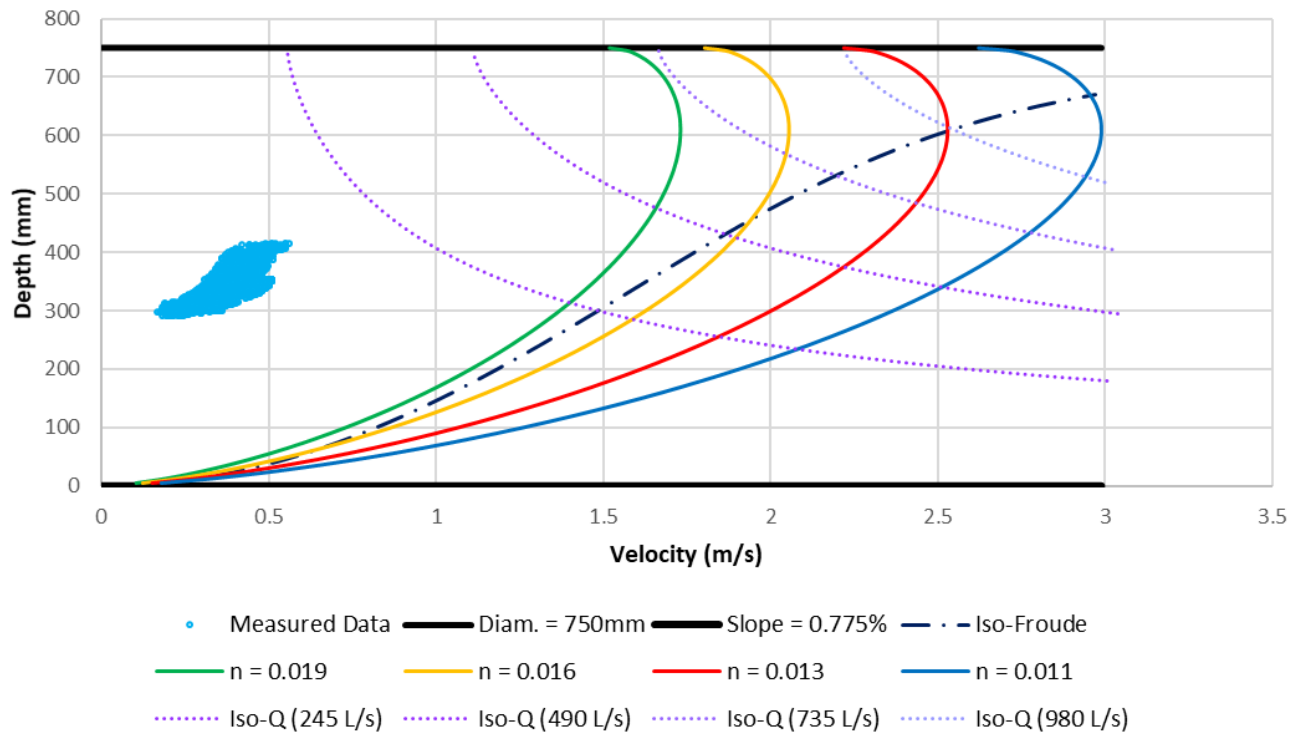
### **Flow Monitoring Scattergraphs**

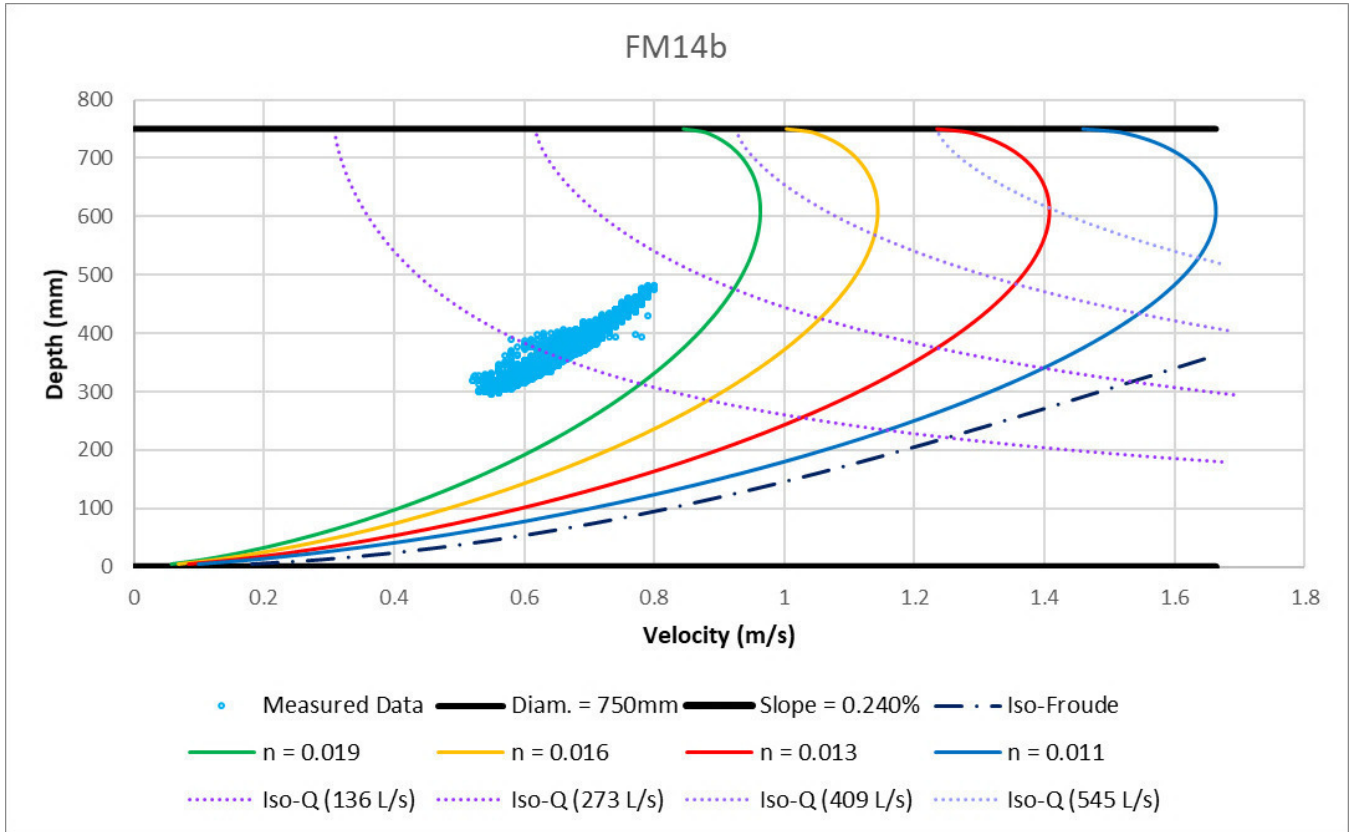
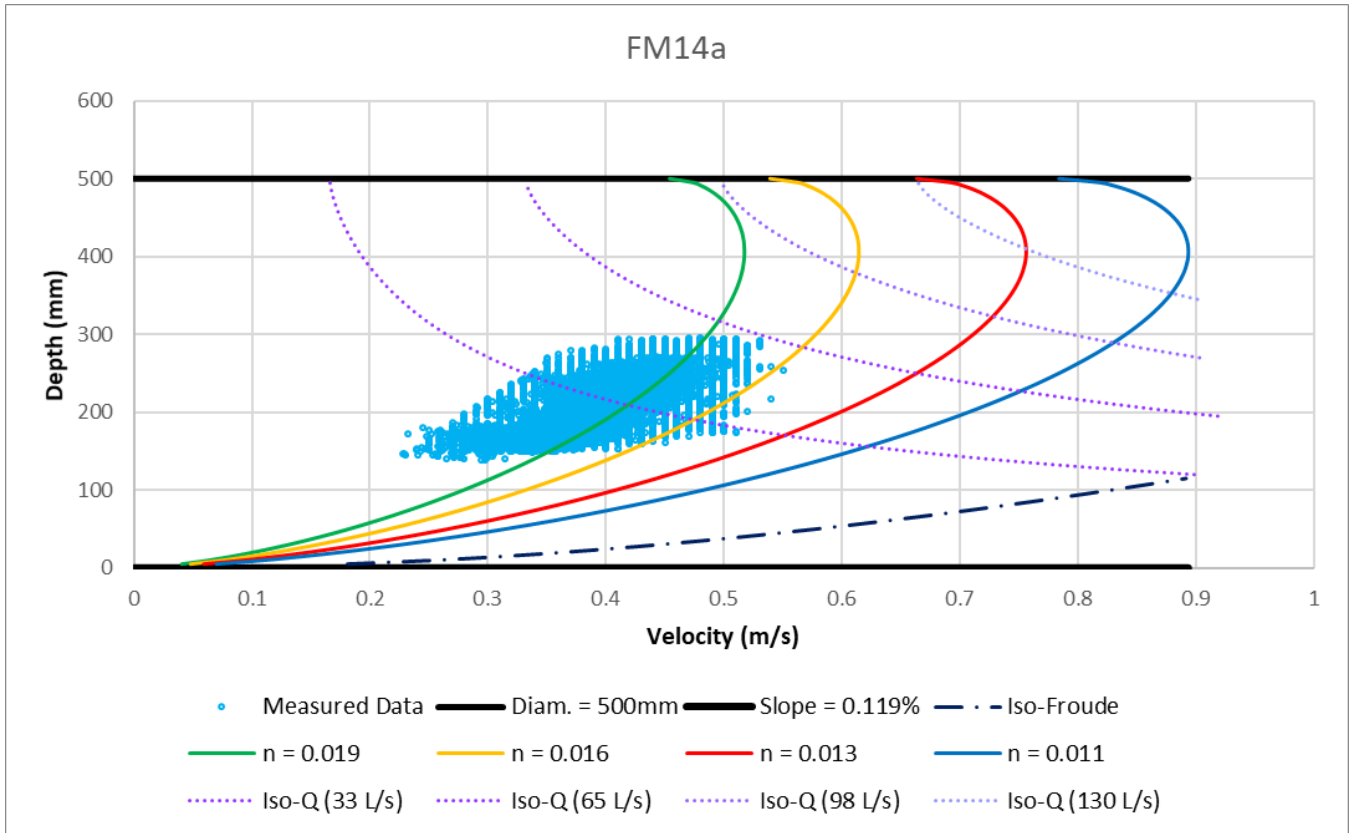


FM12

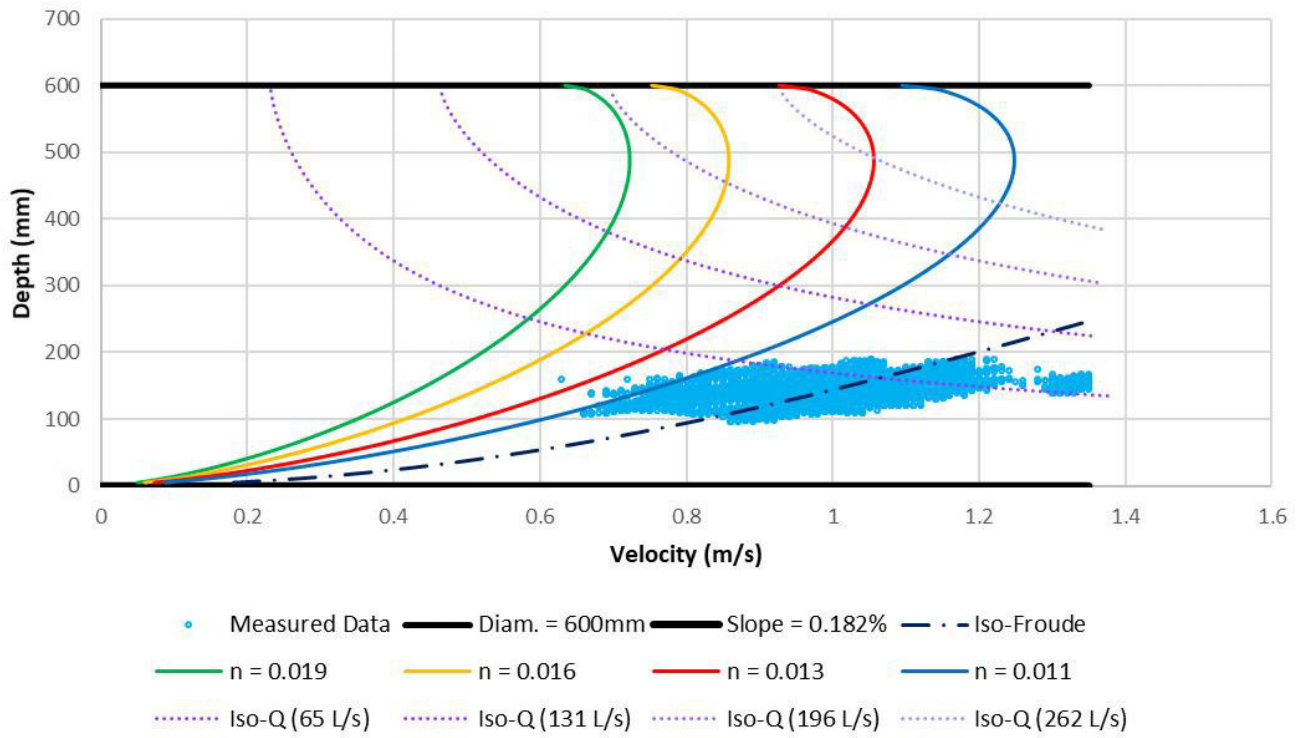


FM13

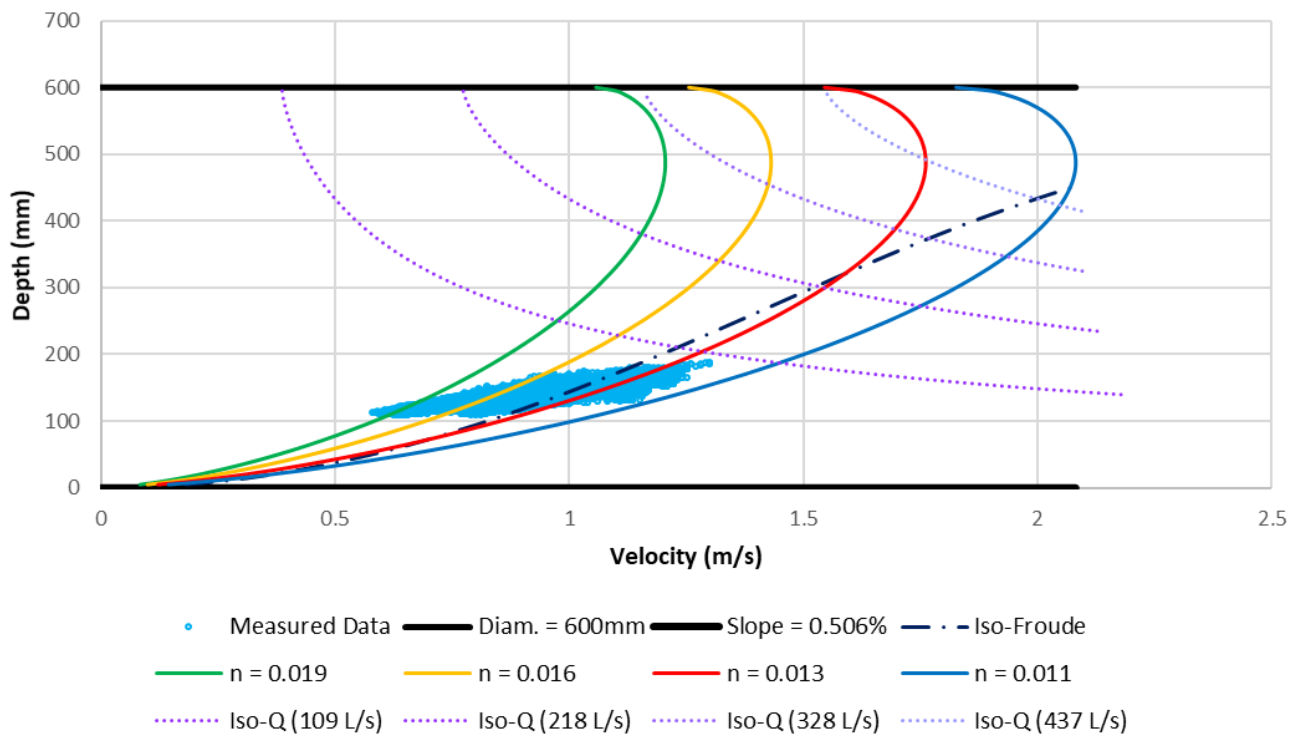




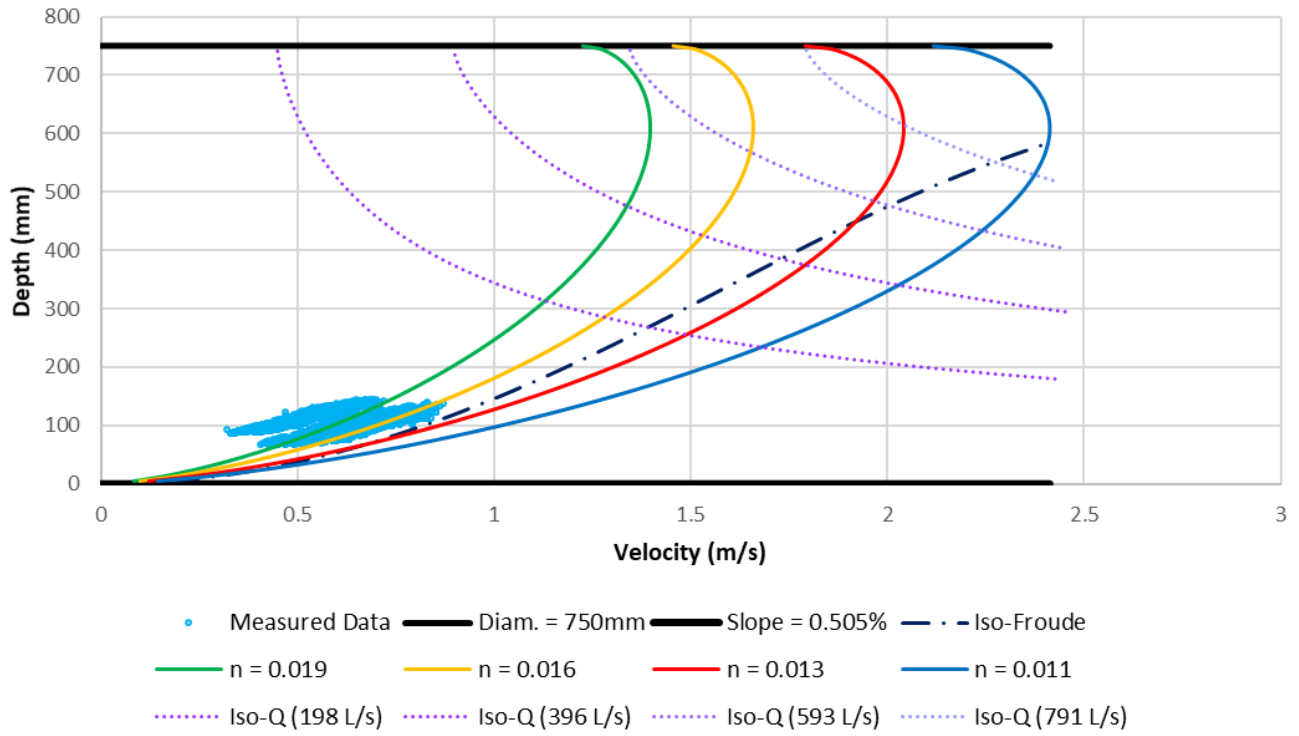
FM15



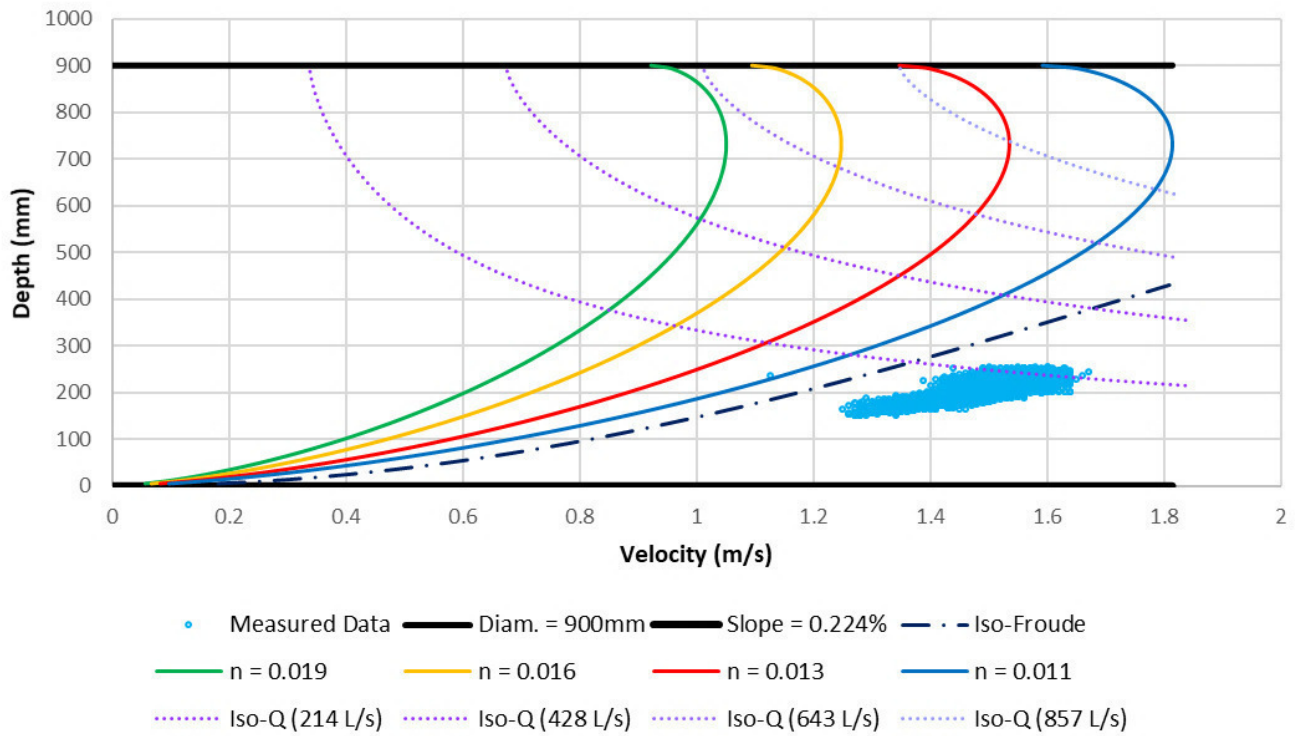
FM16



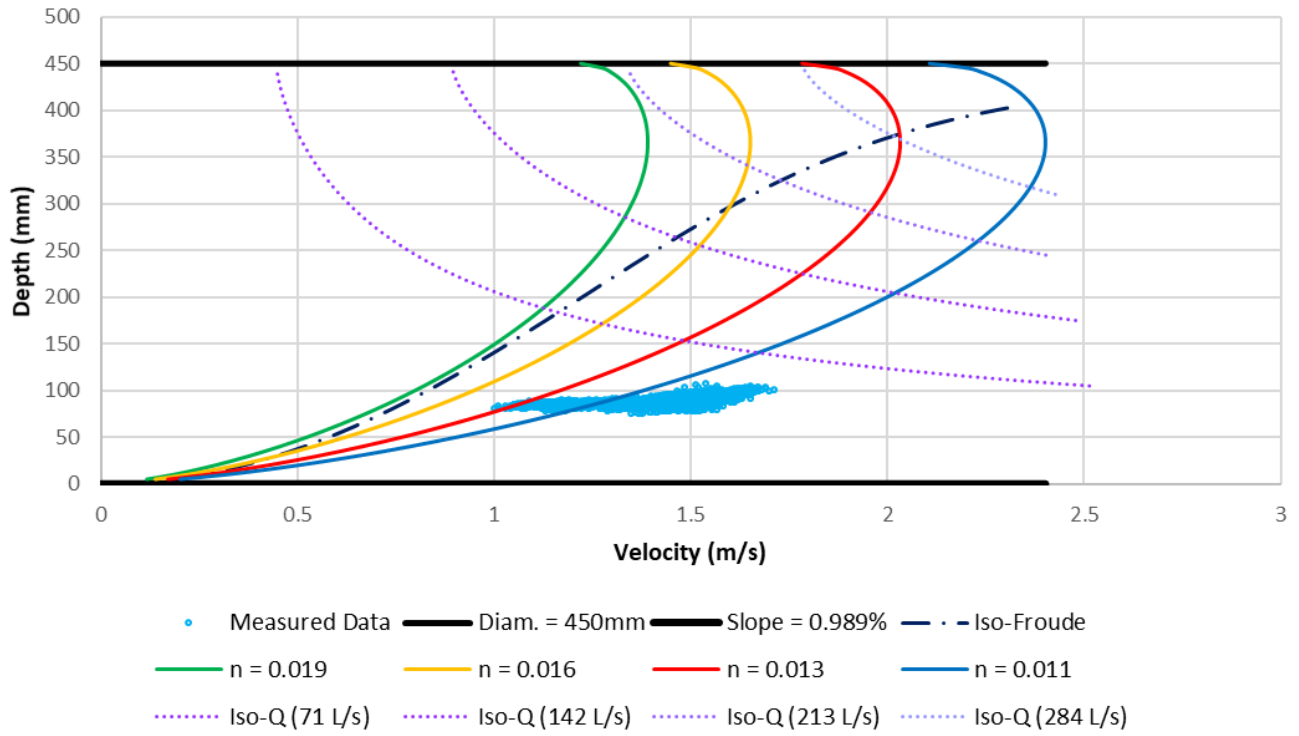
FM17



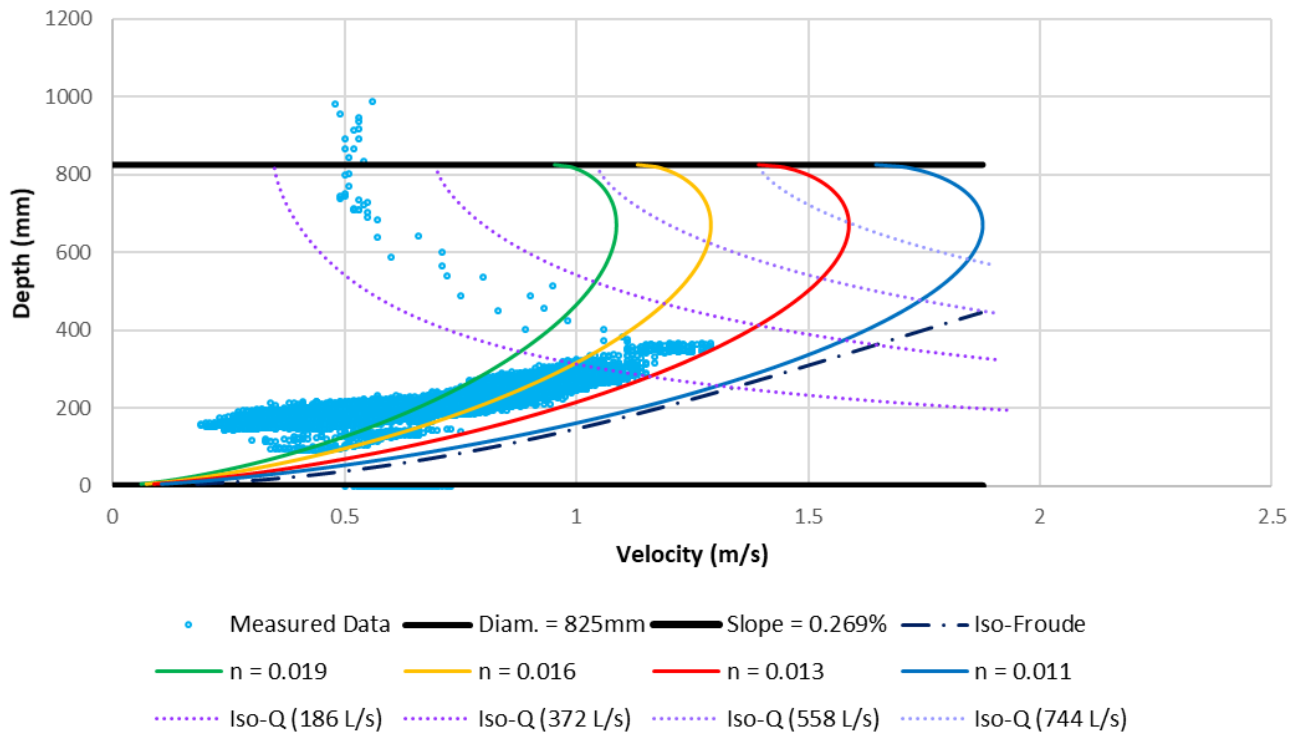
FM18



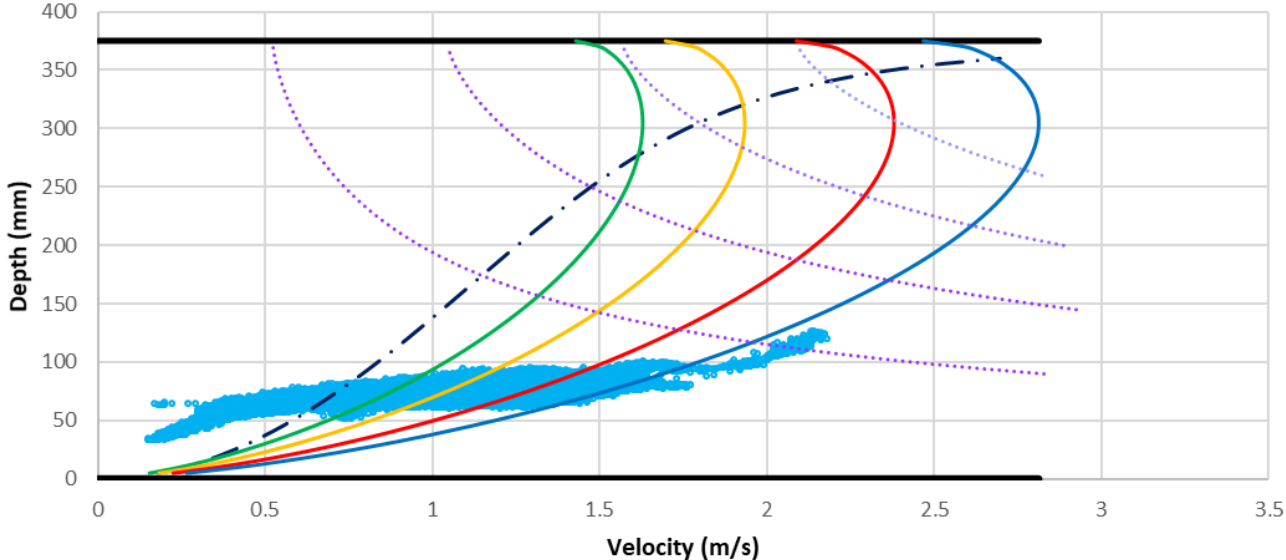
FM19



FM20



FM21



- Measured Data
- Diam. = 375mm
- Slope = 1.729%
- · - Iso-Froude
- n = 0.019
- n = 0.016
- n = 0.013
- n = 0.011
- ..... Iso-Q (58 L/s)
- ..... Iso-Q (115 L/s)
- ..... Iso-Q (173 L/s)
- ..... Iso-Q (231 L/s)

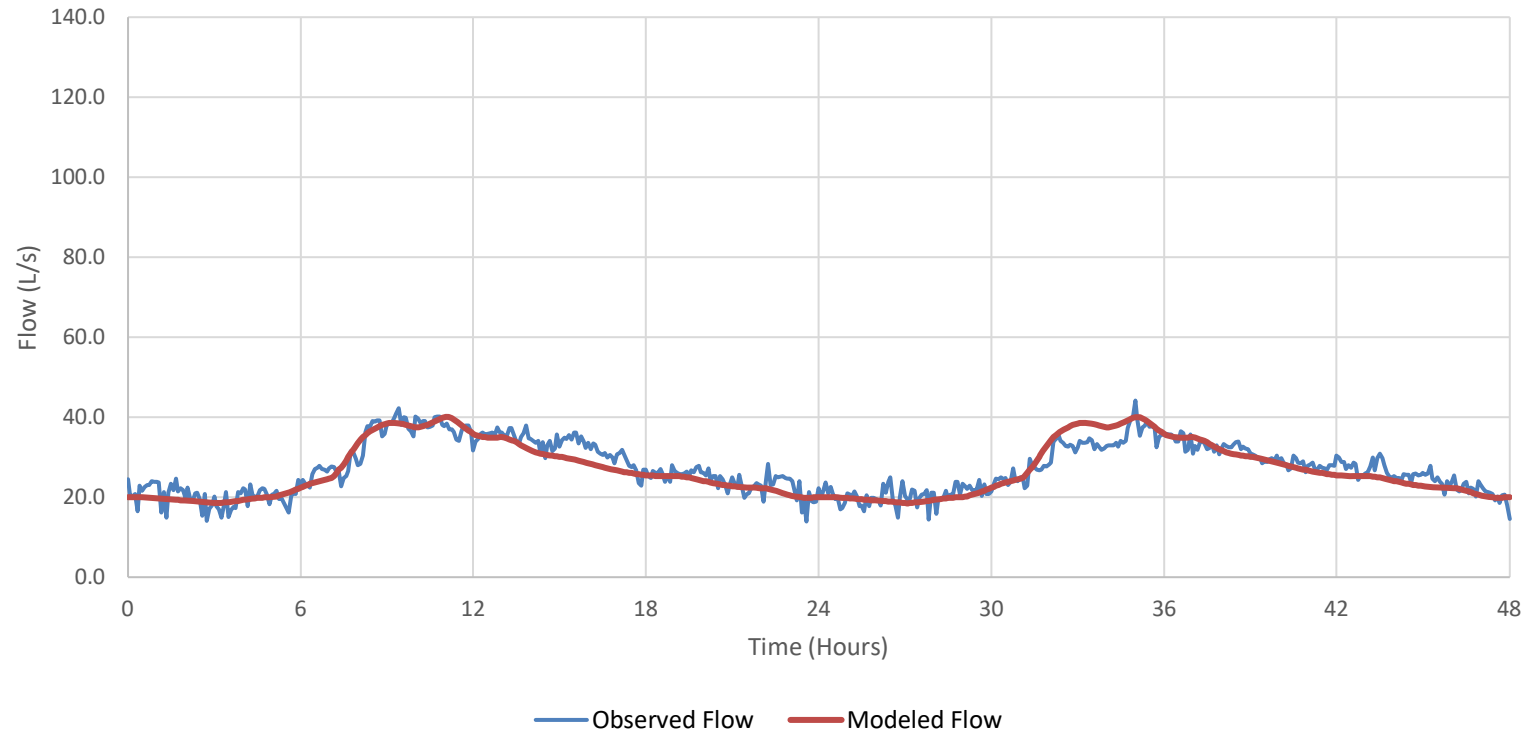


# **APPENDIX E**

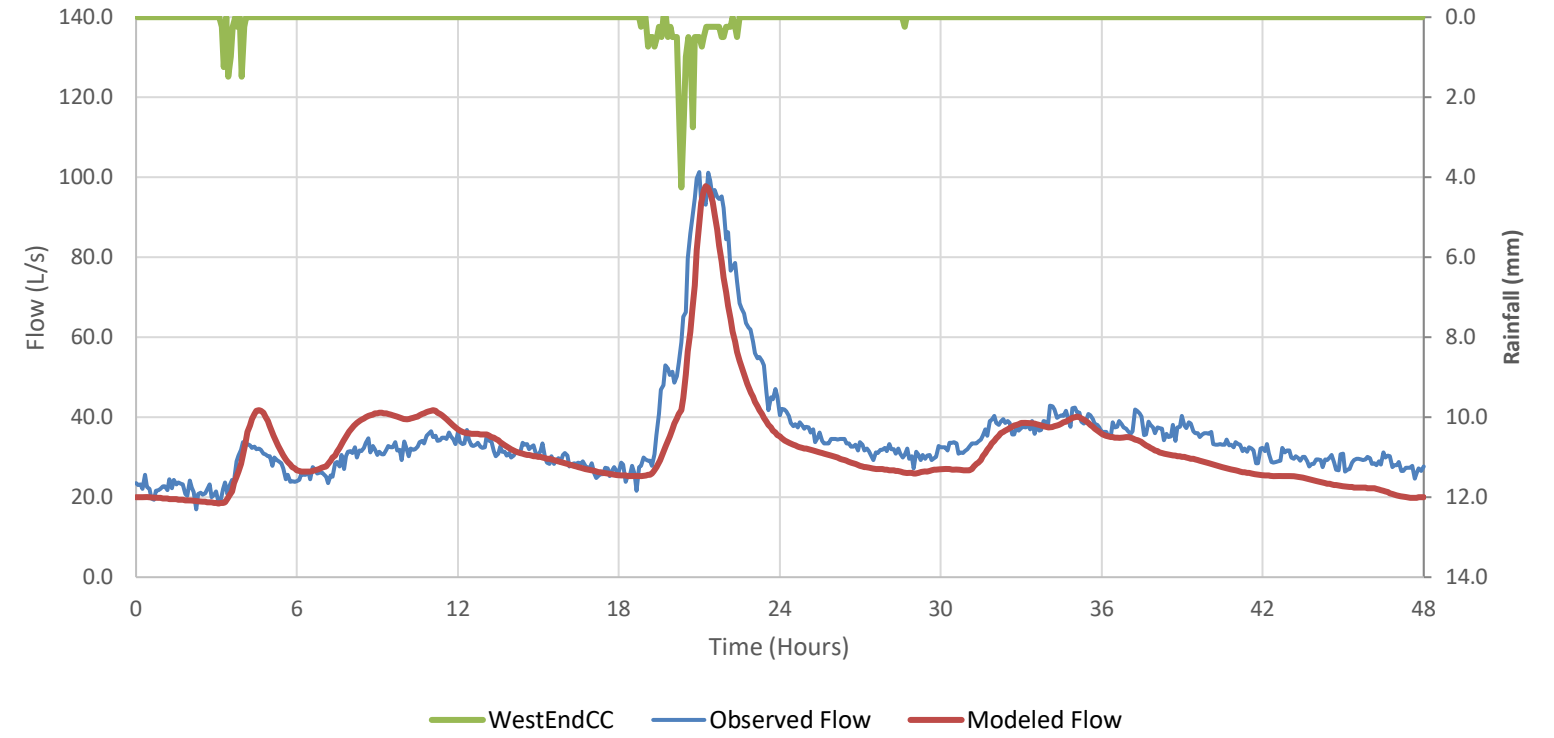
## **Wastewater Calibration Results**

# FM10 CALIBRATION AND VALIDATION

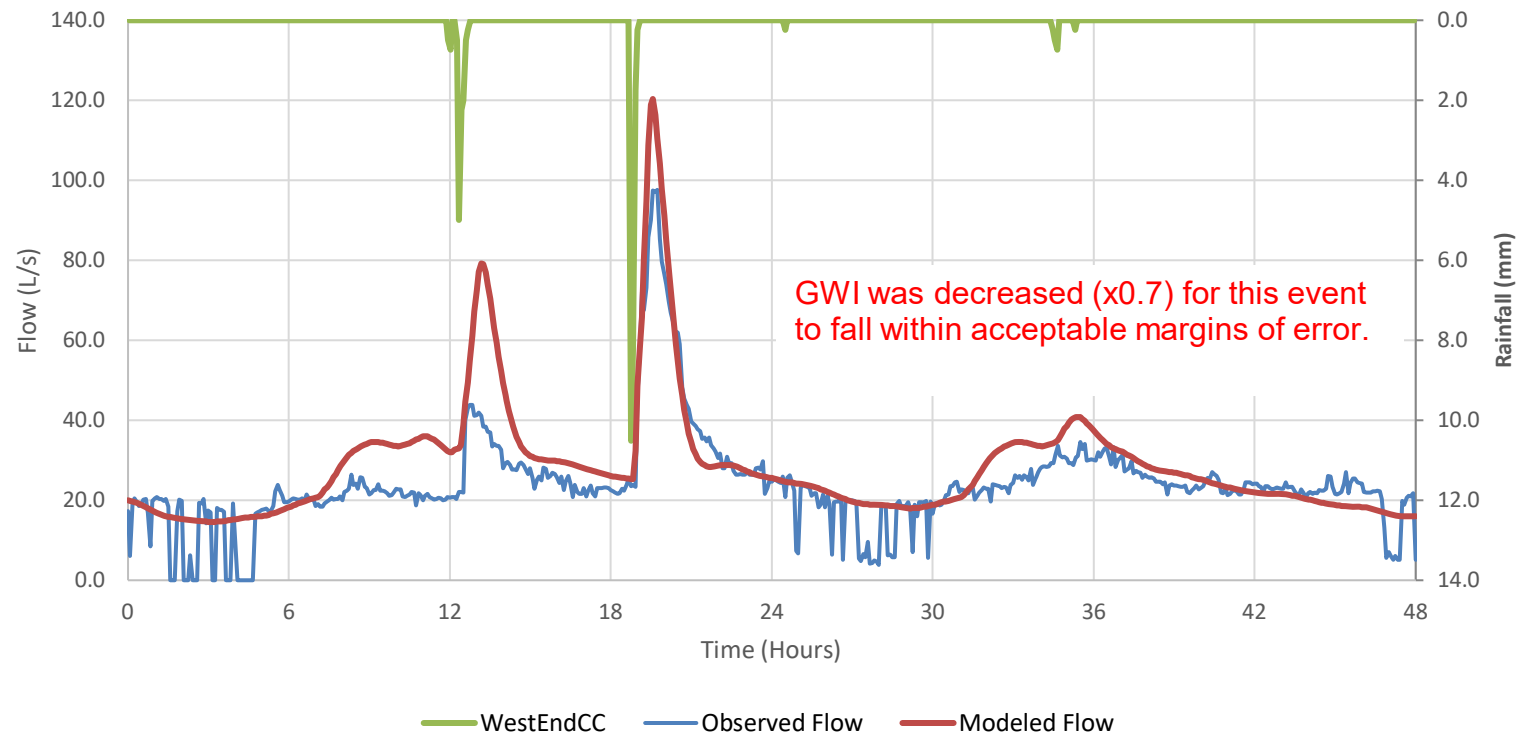
FM10: DWF CALIBRATION - June 17, 2020



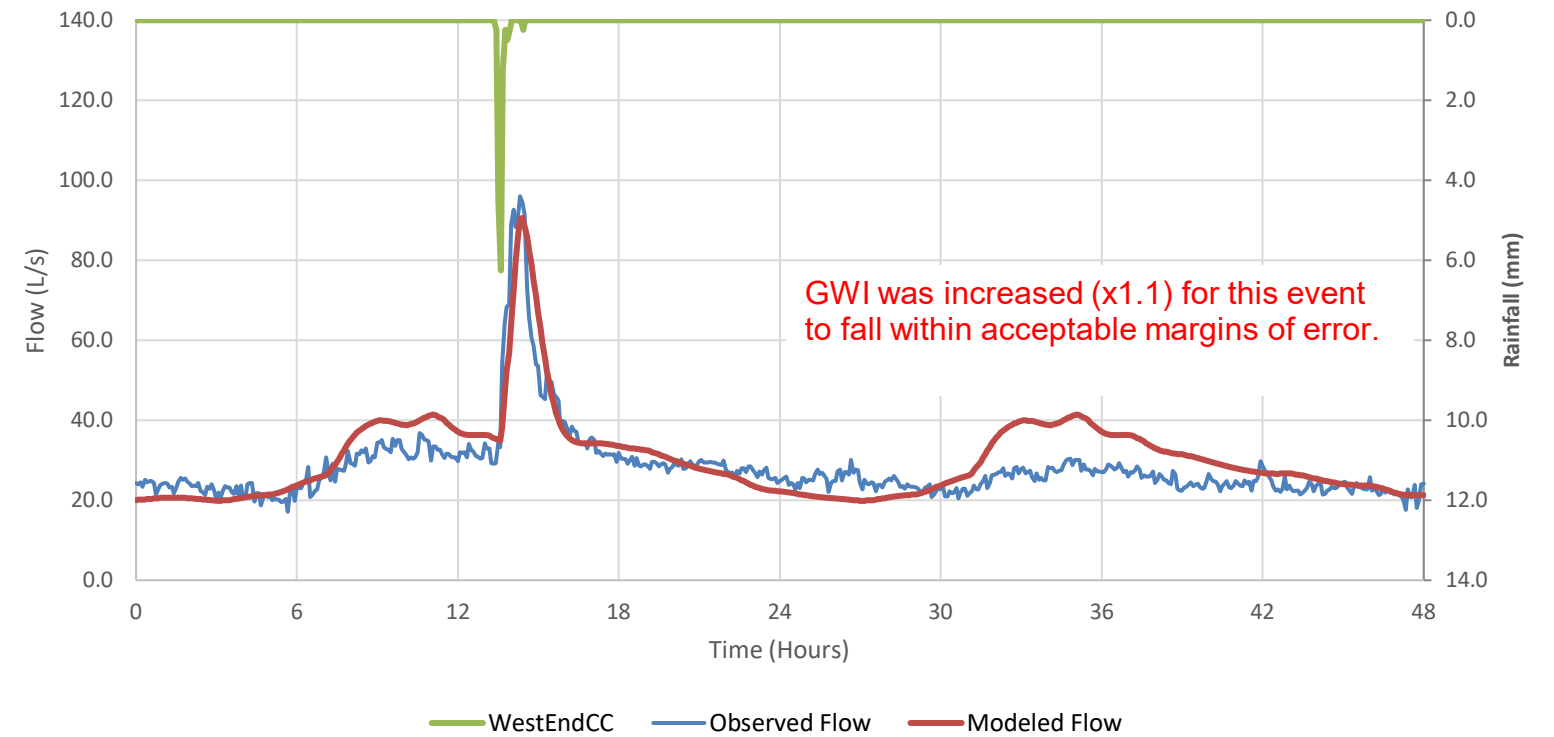
FM10: WWF CALIBRATION - June 10, 2020



FM10: WWF CALIBRATION - July 10, 2020

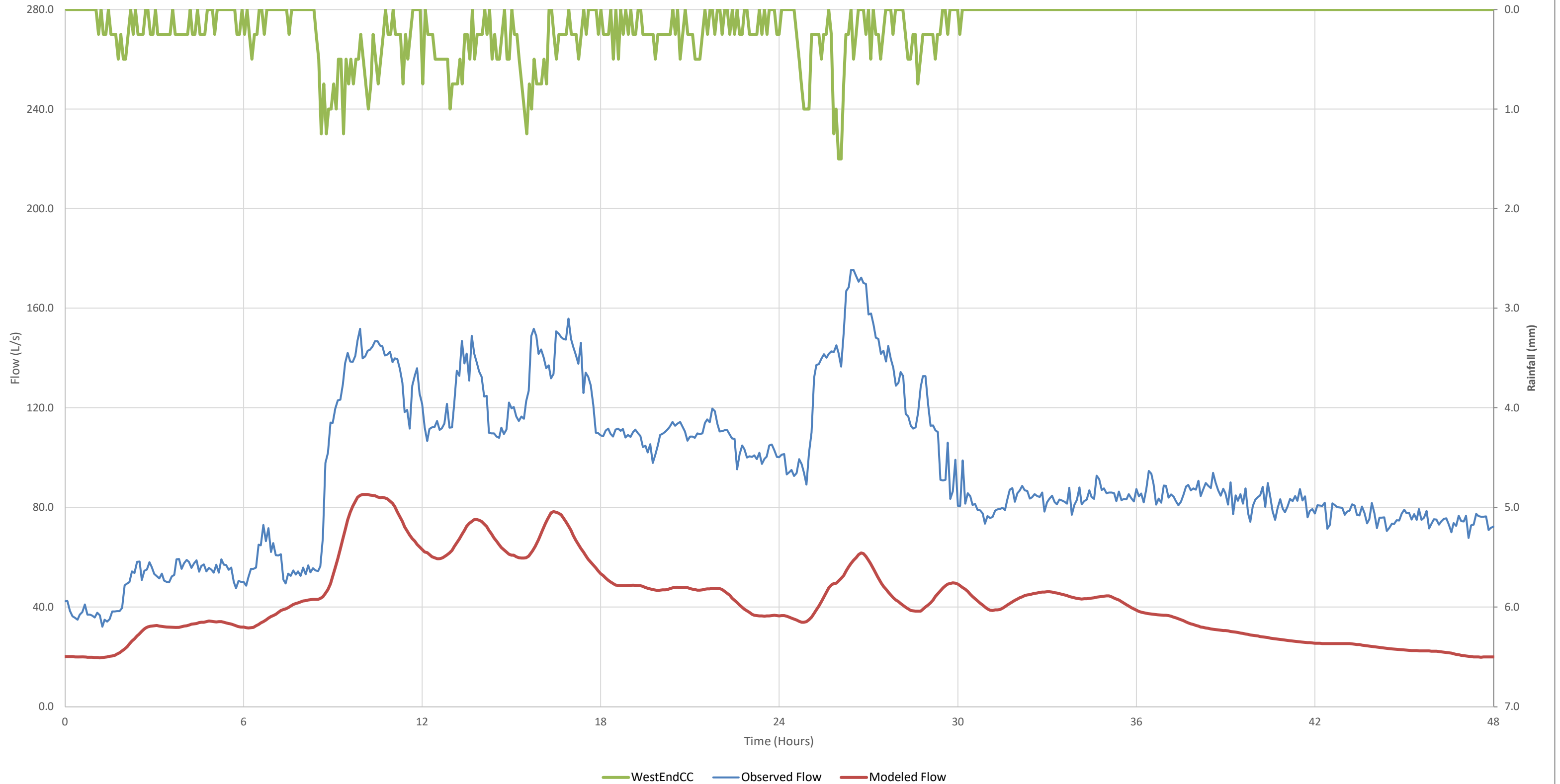


FM10: WWF VALIDATION - May 29, 2020



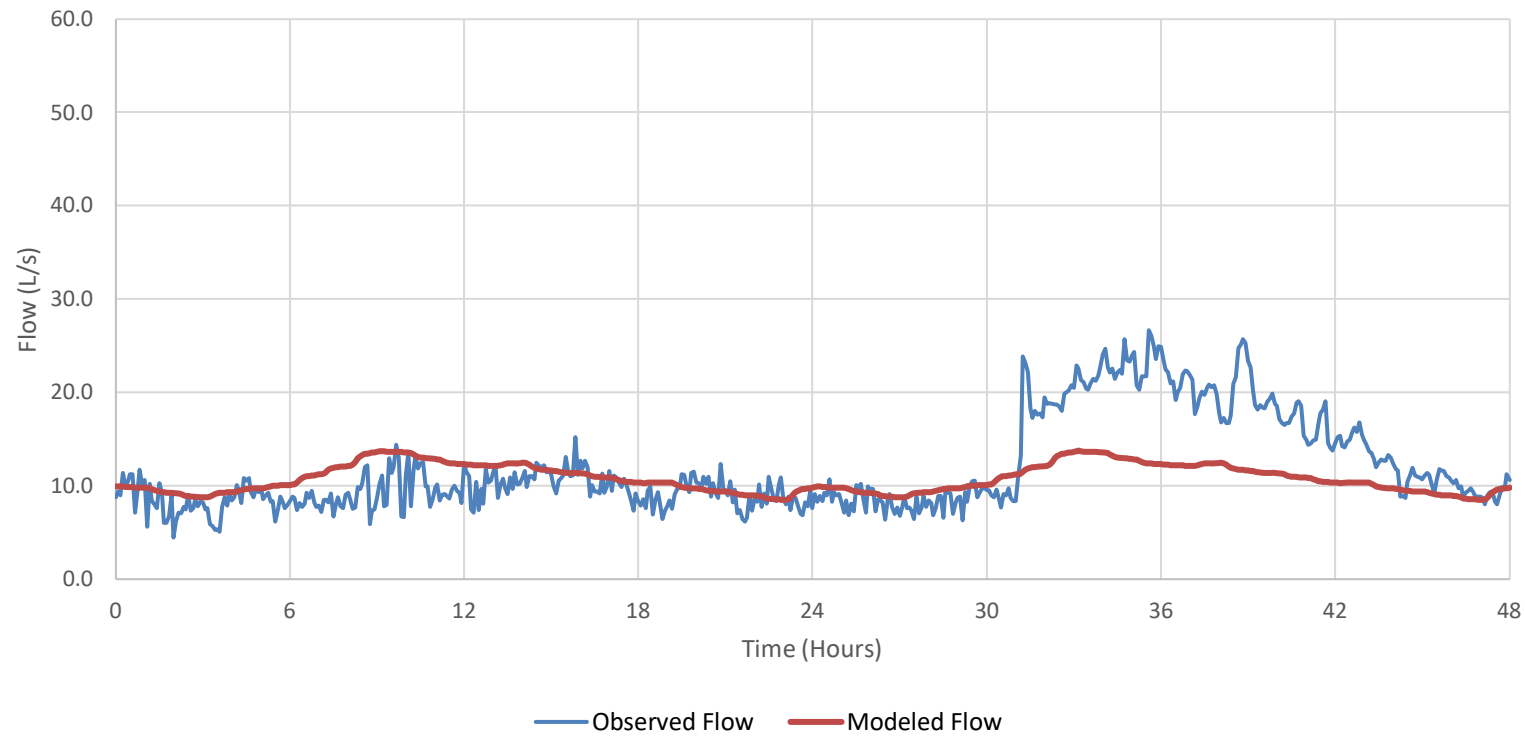
# FM10 CALIBRATION AND VALIDATION

FM10: WWF VALIDATION - January 11, 2020

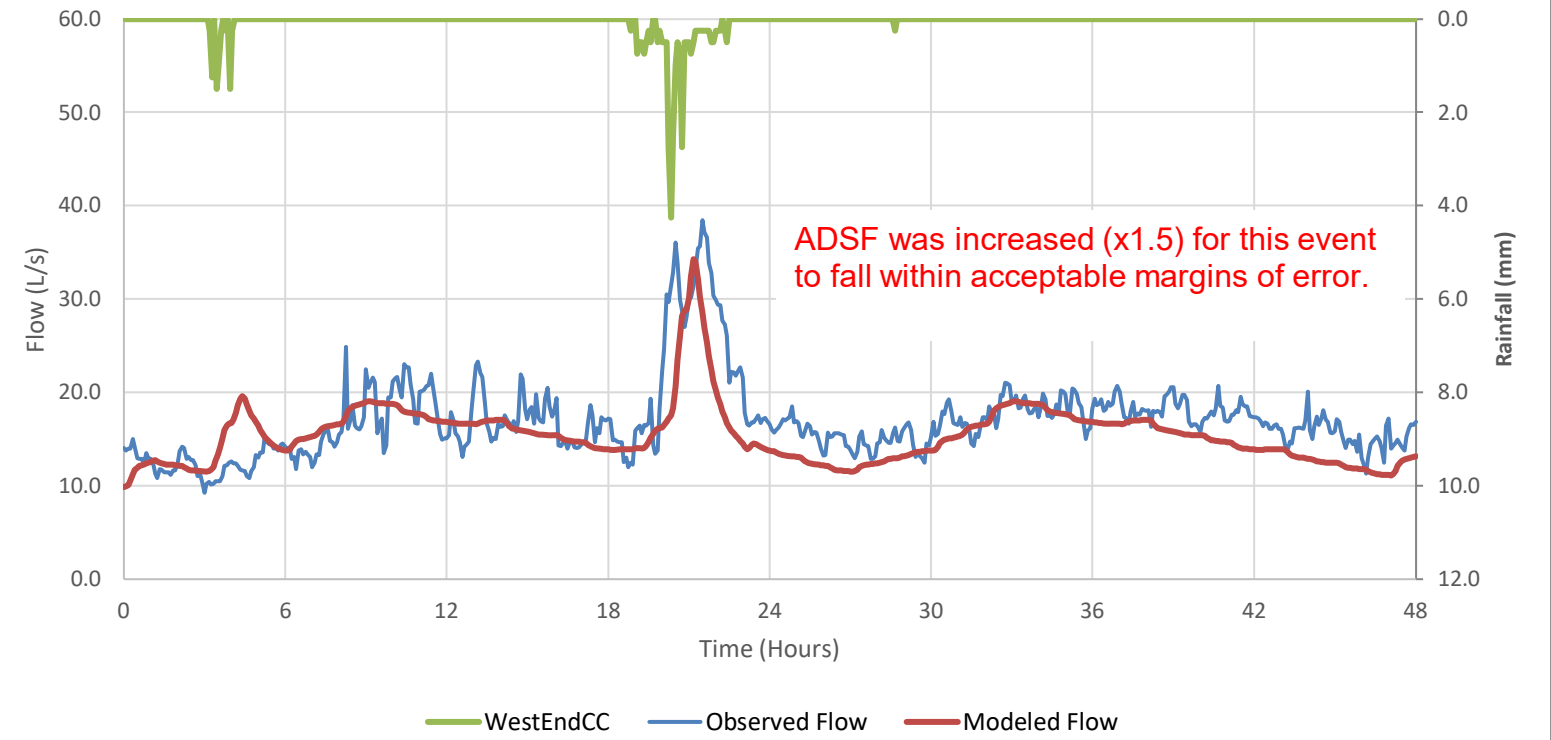


# FM11 CALIBRATION AND VALIDATION

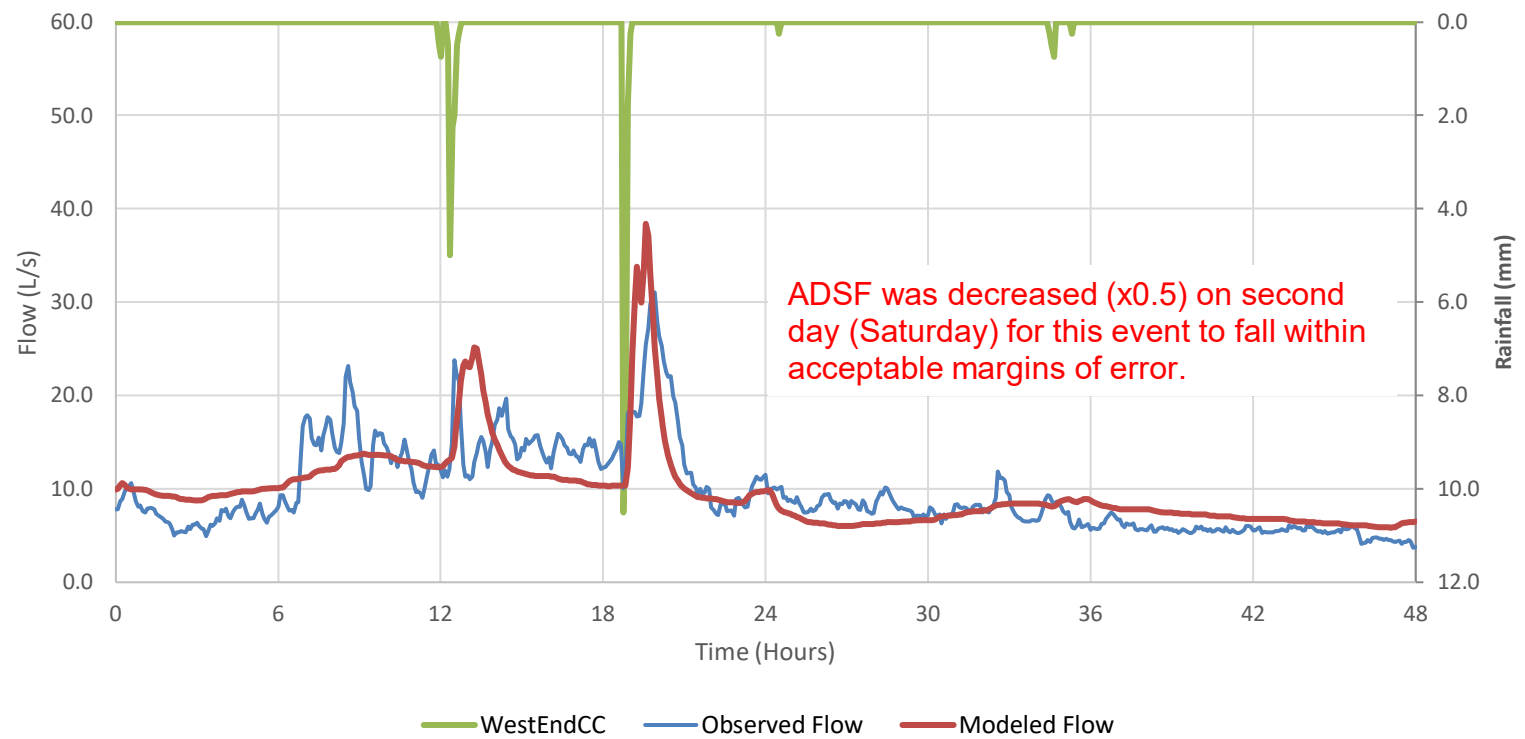
FM11: DWF CALIBRATION - June 17, 2020



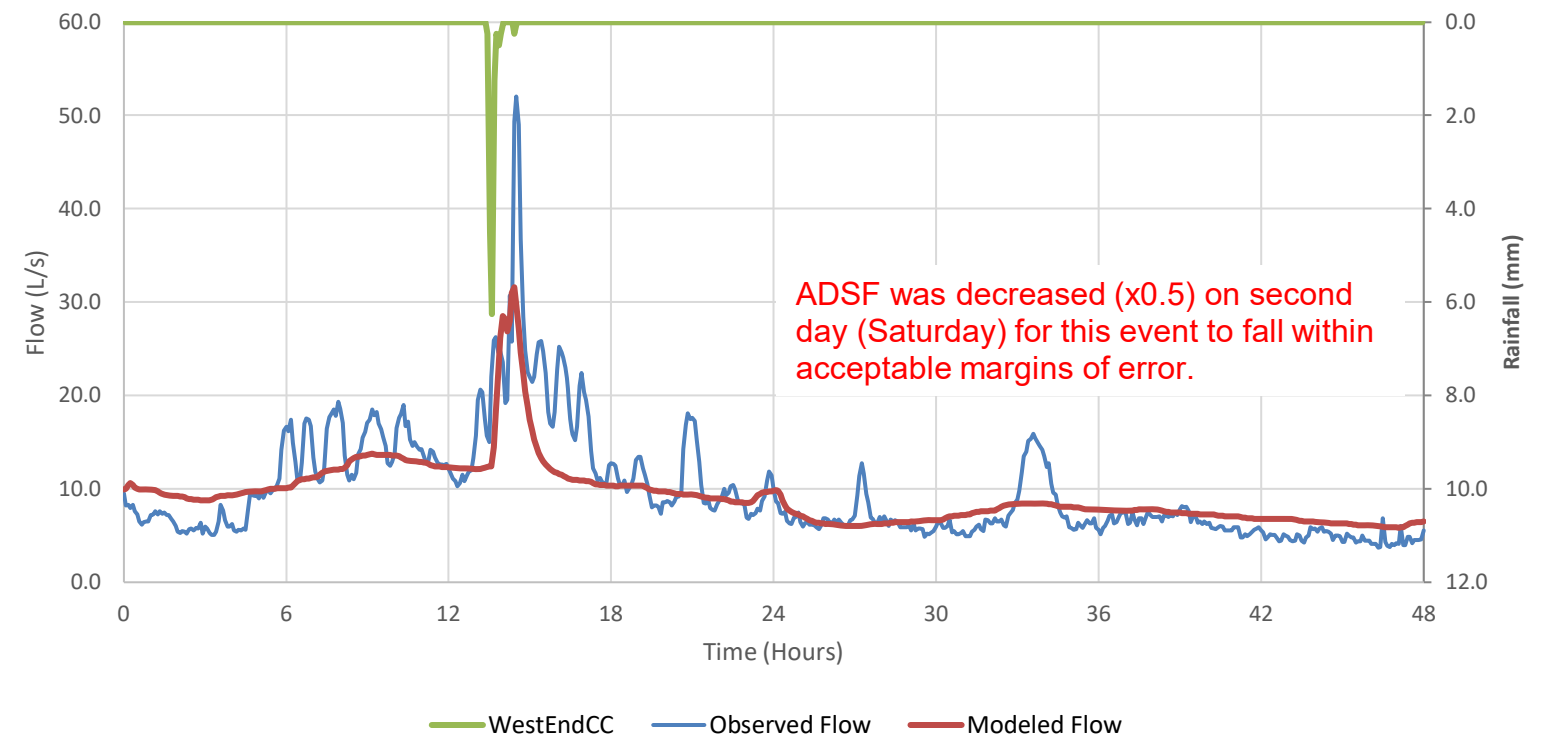
FM11: WWF CALIBRATION - June 10, 2020



FM11: WWF CALIBRATION - July 10, 2020

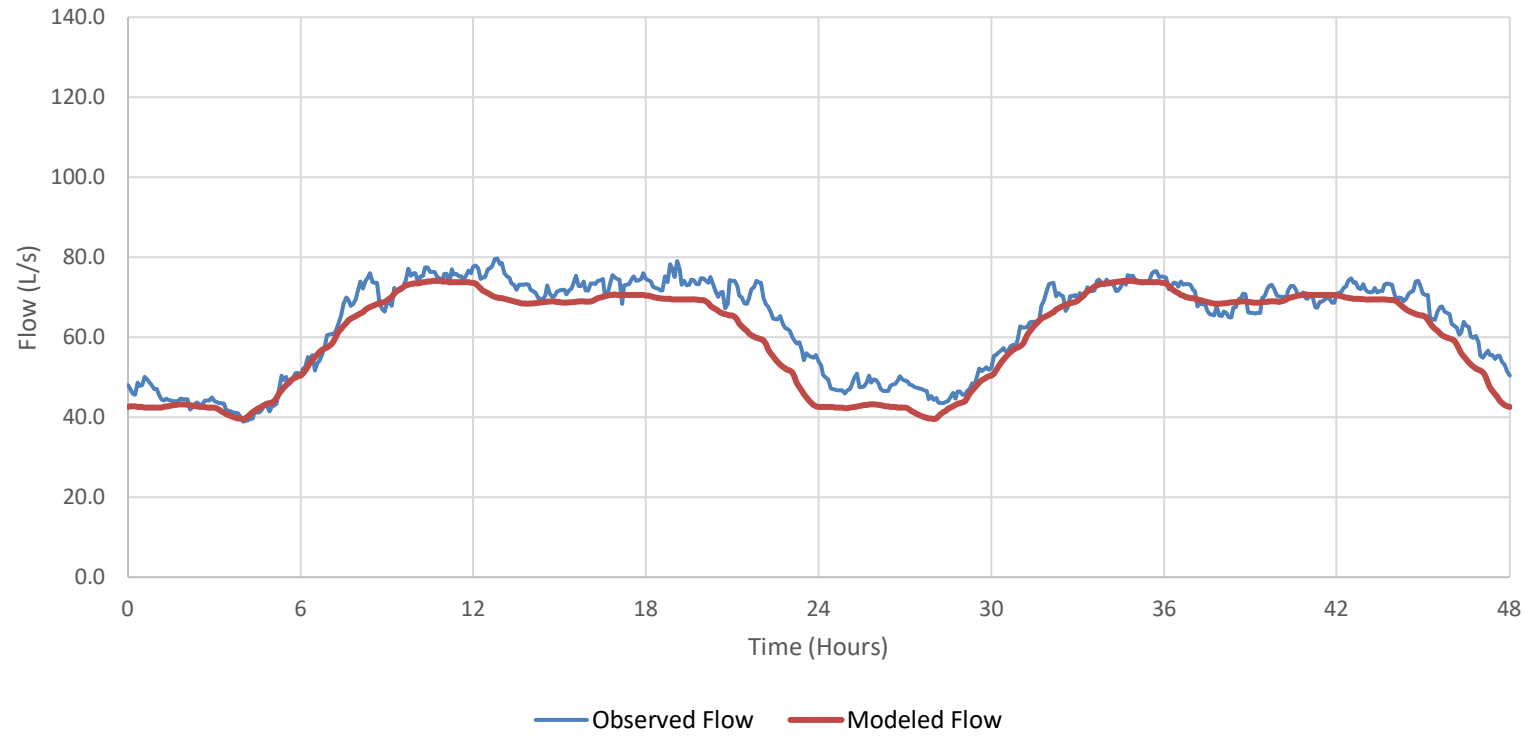


FM11: WWF VALIDATION - May 29, 2020

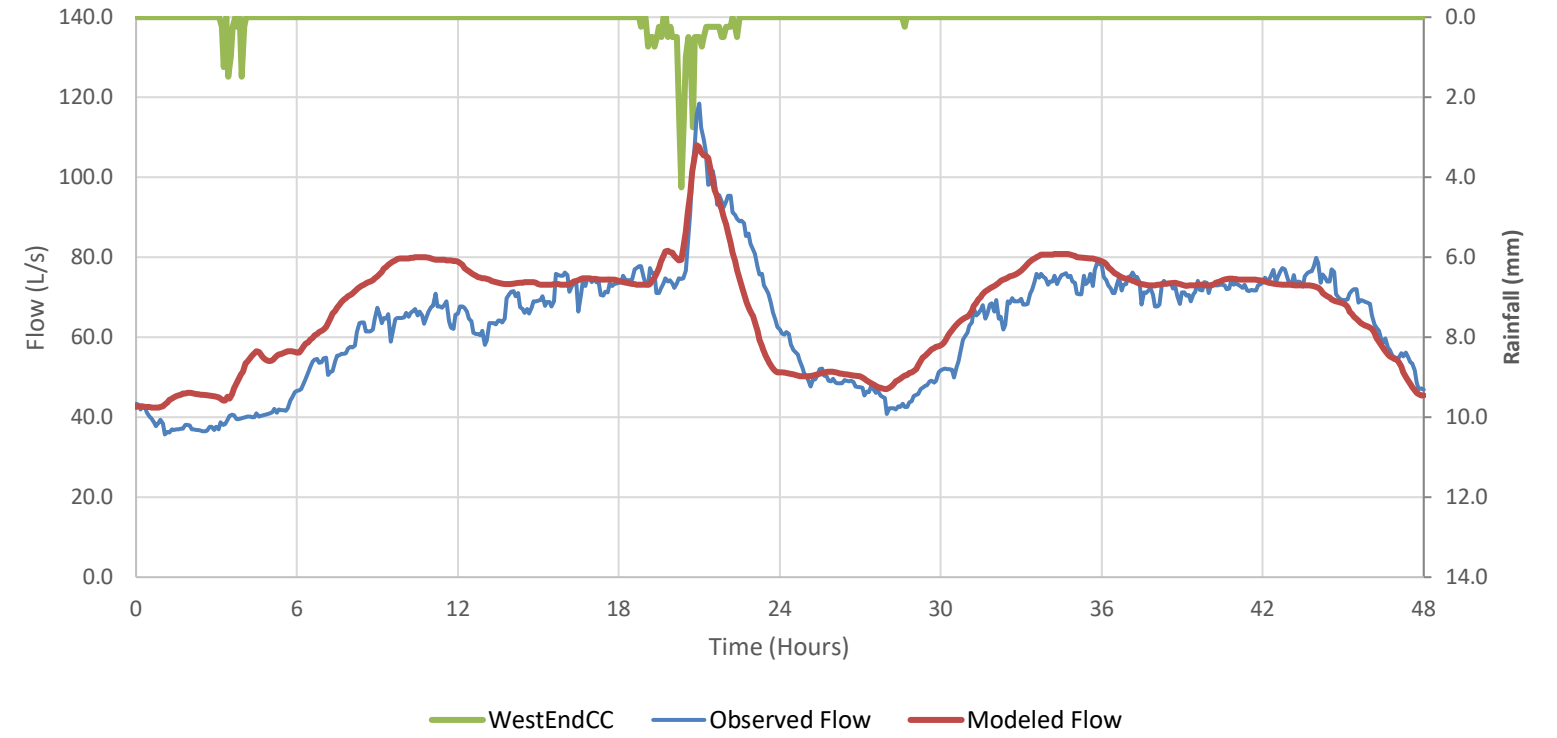


# FM12 CALIBRATION AND VALIDATION

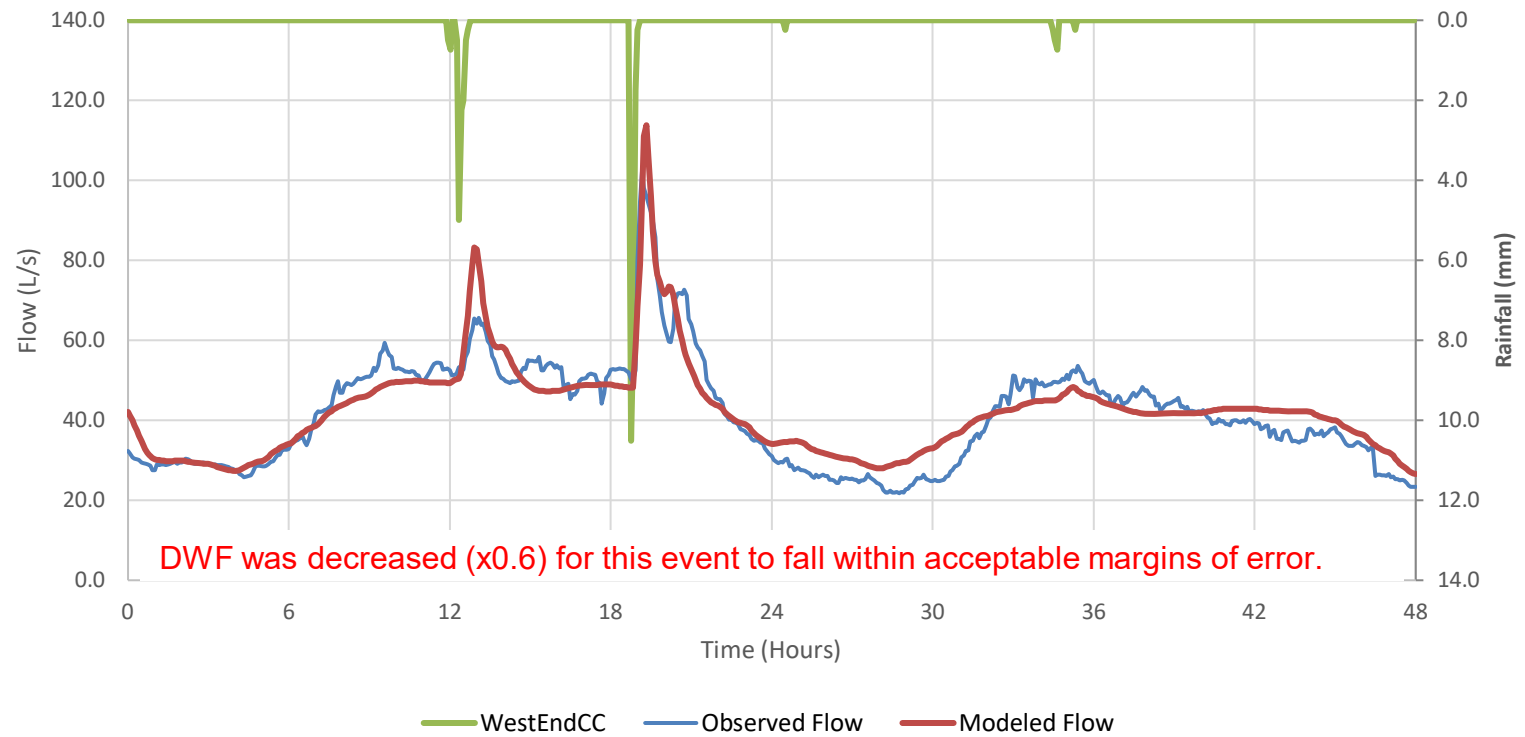
FM12: DWF CALIBRATION - June 17, 2020



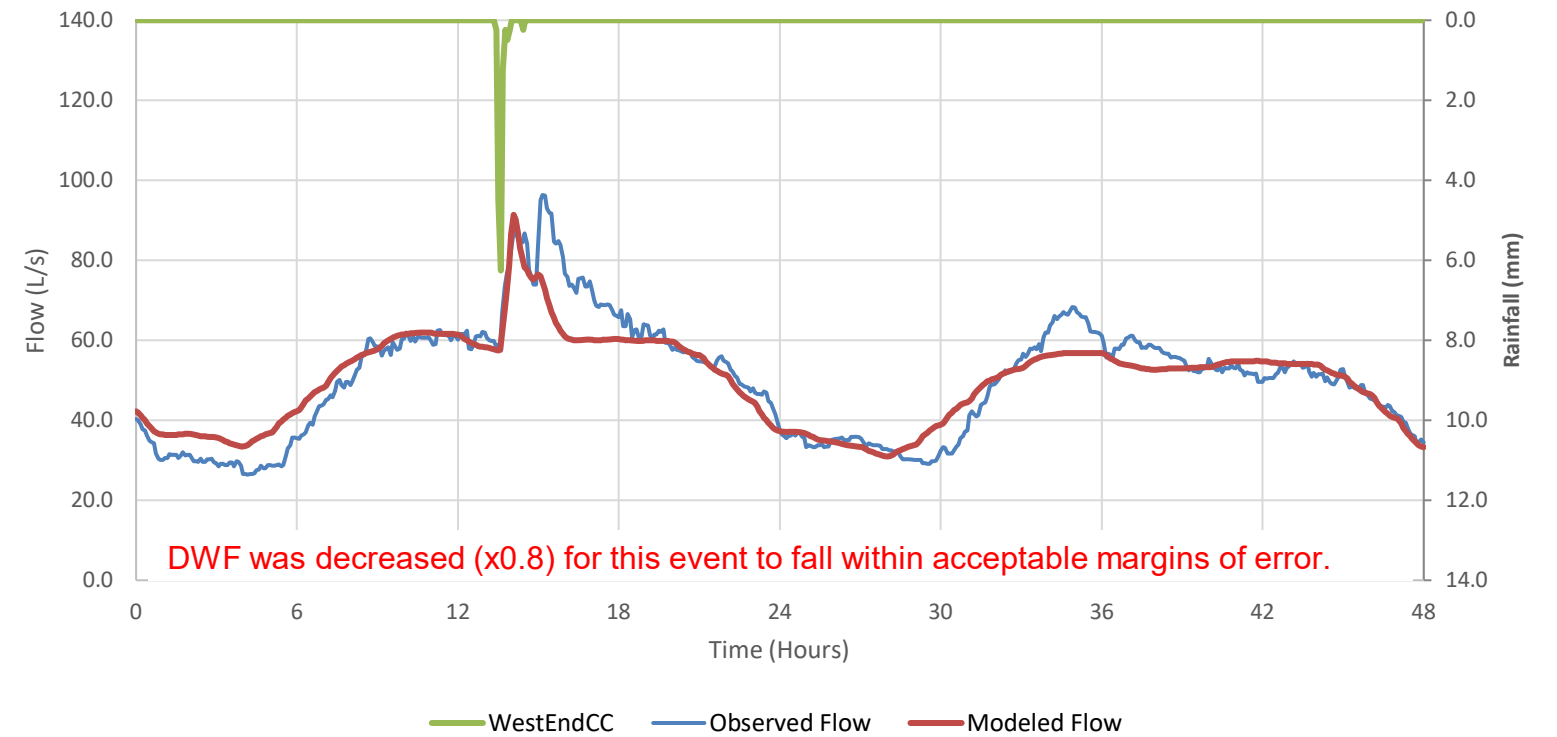
FM12: WWF CALIBRATION - June 10, 2020



FM12: WWF CALIBRATION - July 10, 2020

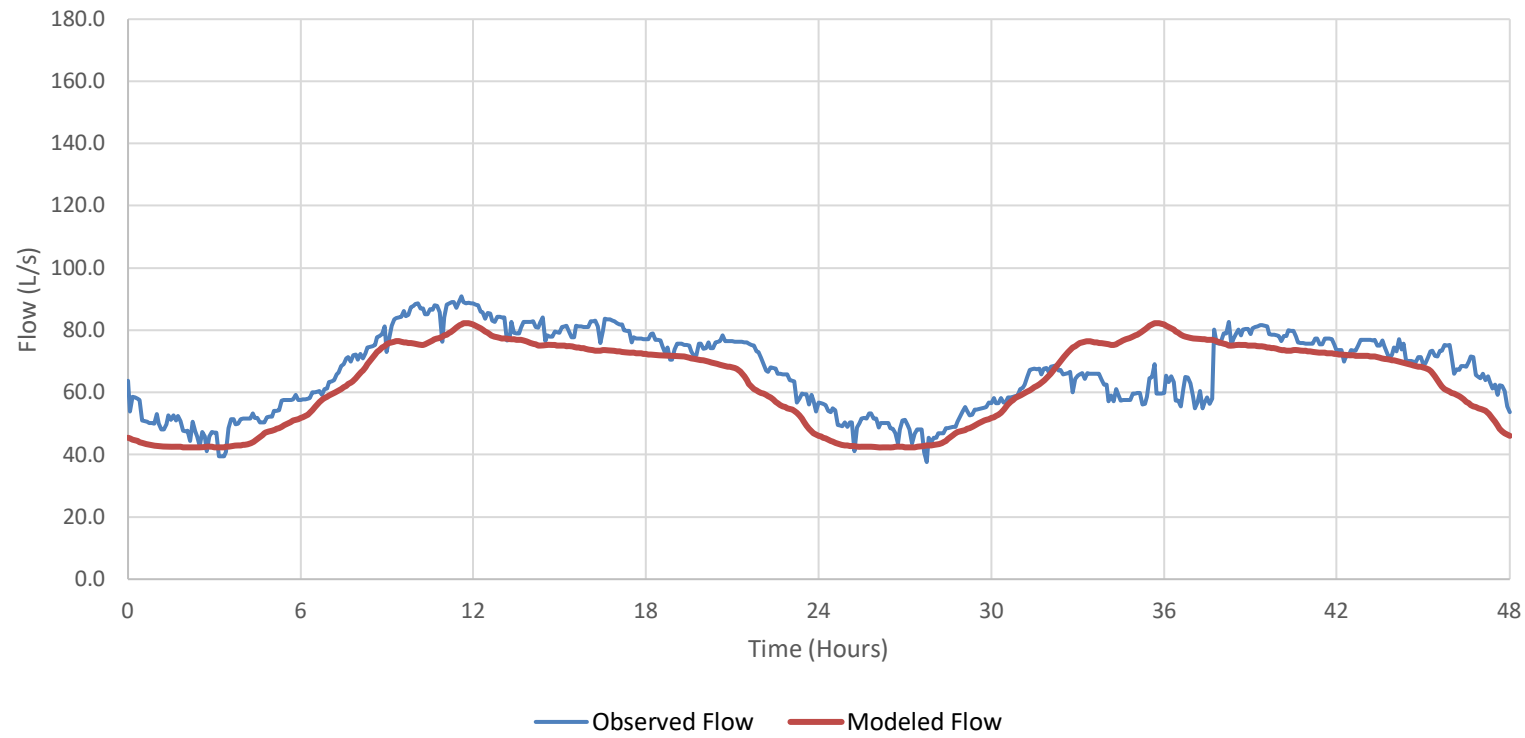


FM12: WWF VALIDATION - May 29, 2020

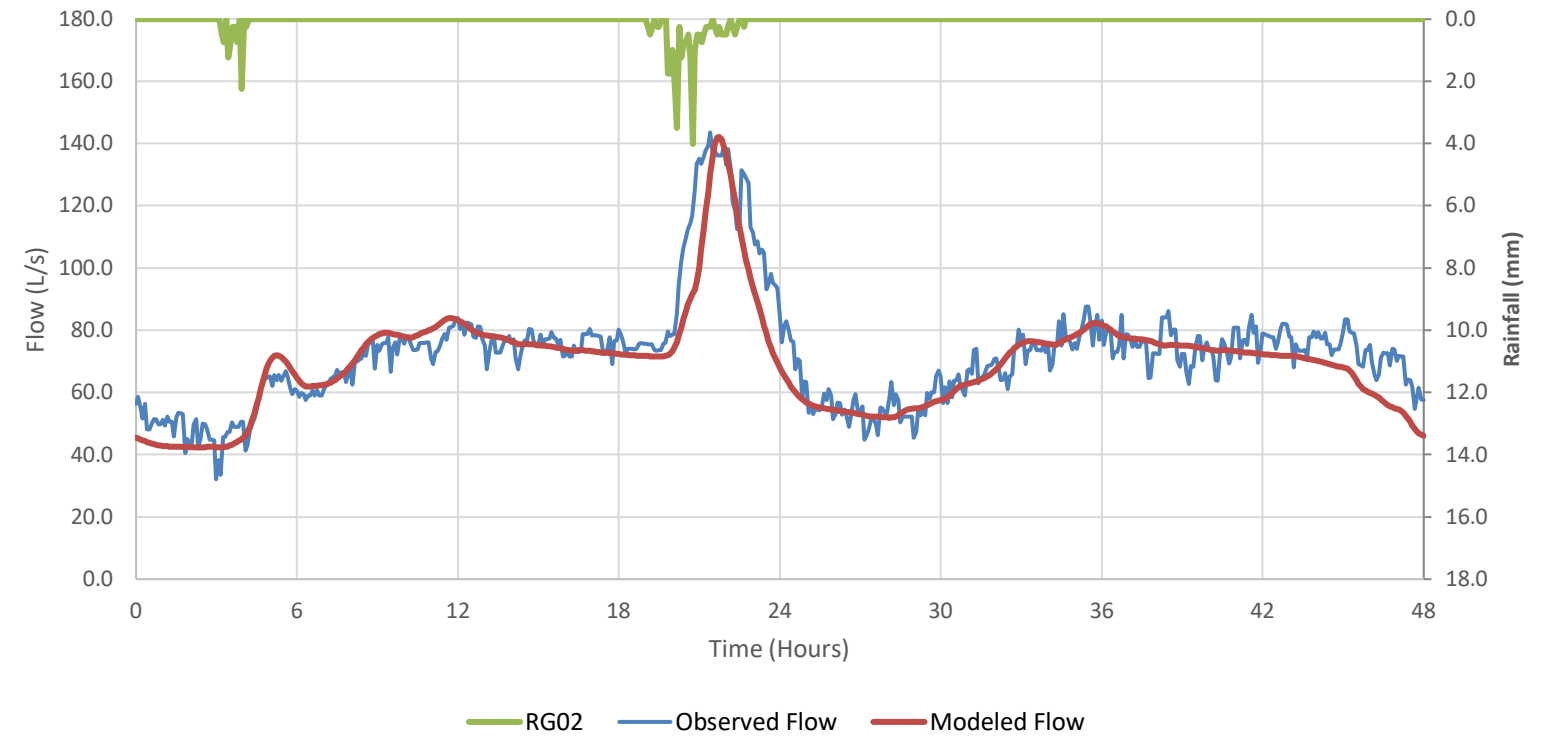


# FM13 CALIBRATION AND VALIDATION

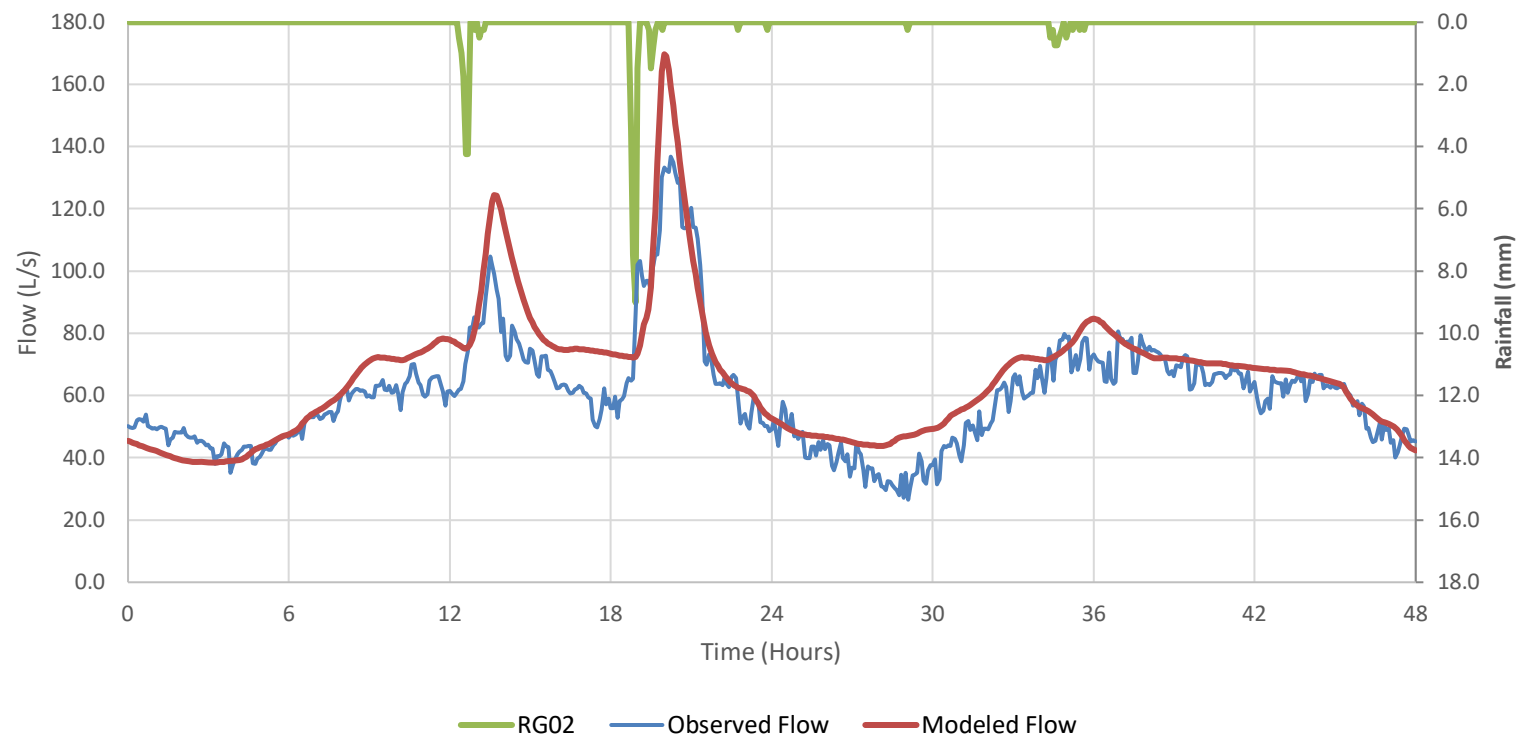
FM13: DWF CALIBRATION - June 17, 2020



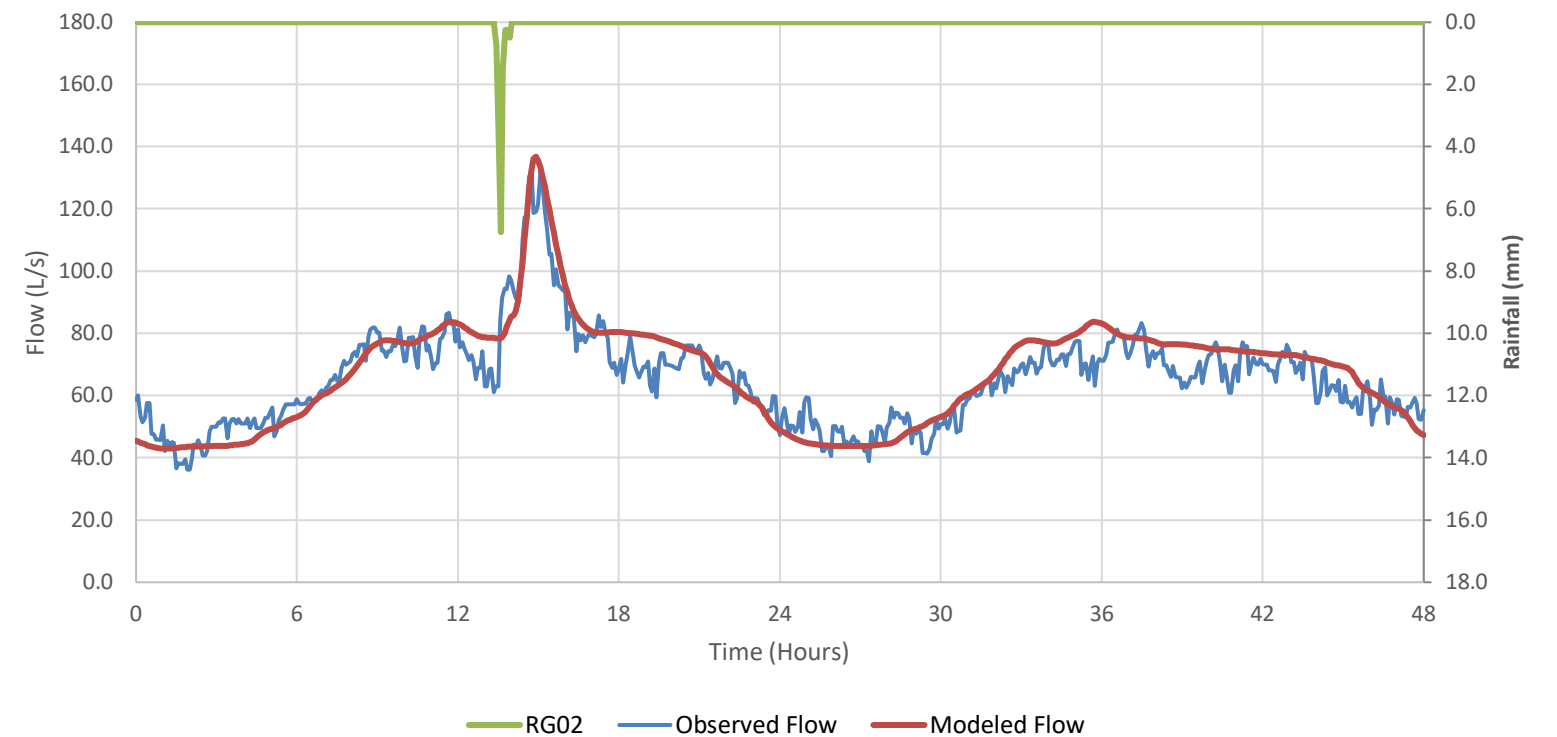
FM13: WWF CALIBRATION - June 10, 2020



FM13: WWF CALIBRATION - July 10, 2020

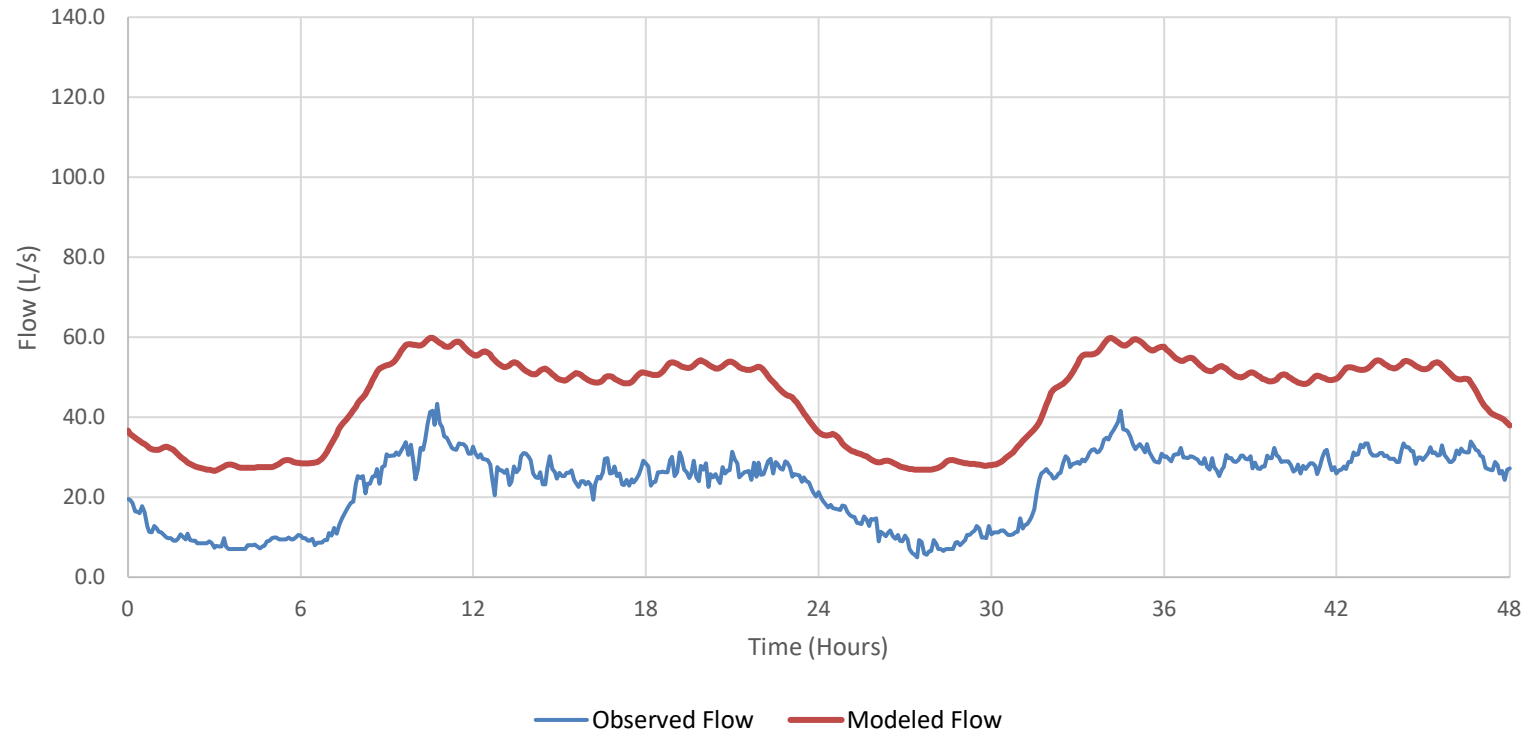


FM13: WWF VALIDATION - May 29, 2020

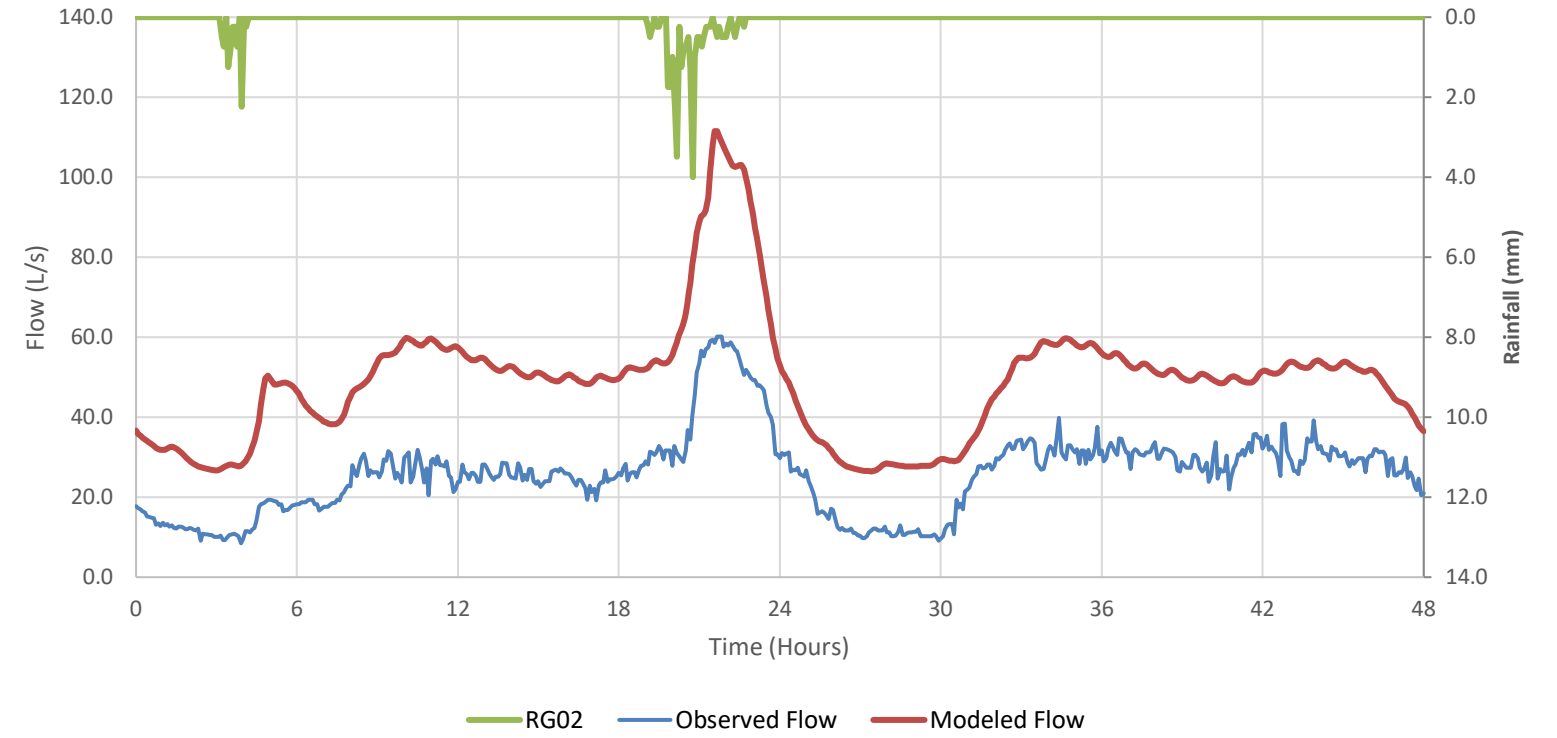


# FM14a CALIBRATION AND VALIDATION

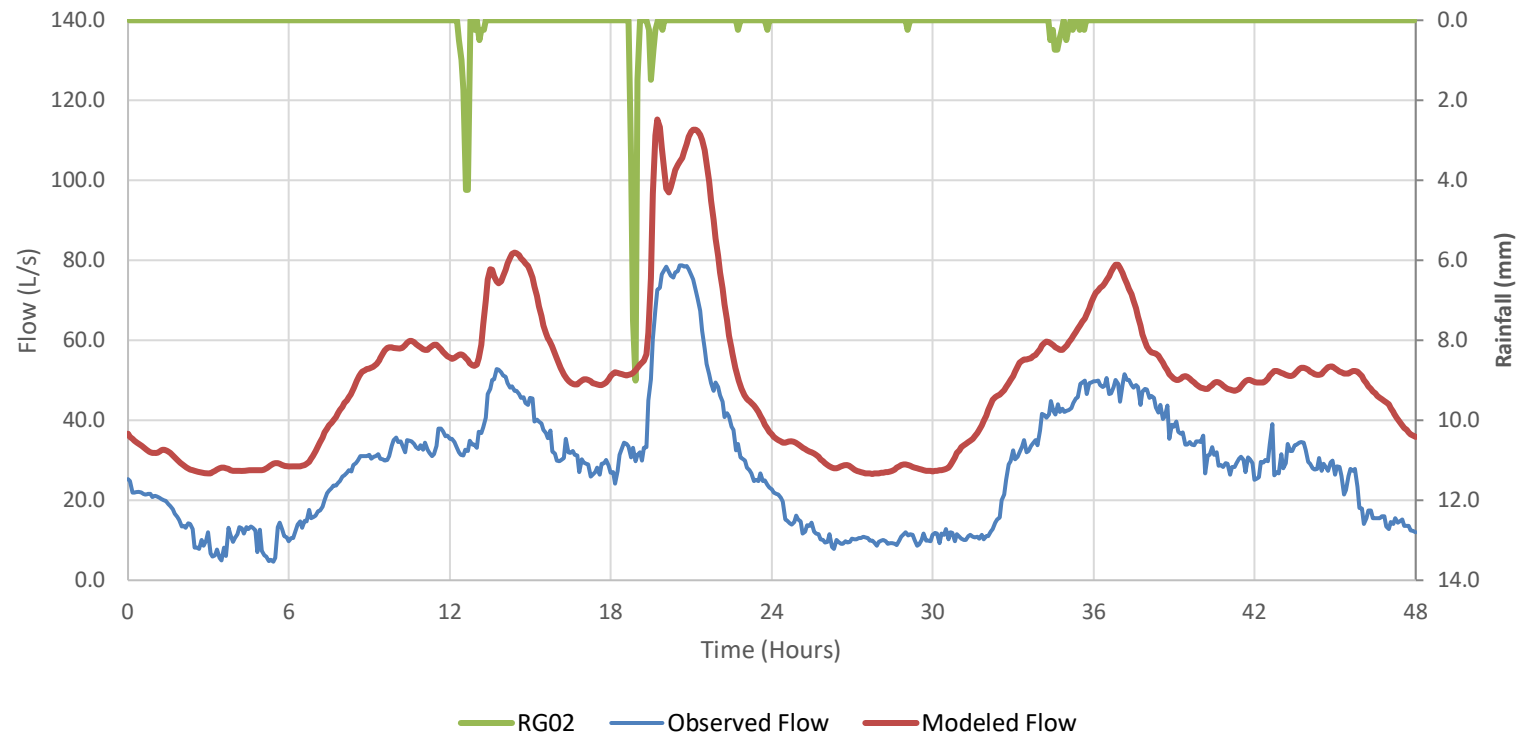
FM14a: DWF CALIBRATION - June 17, 2020



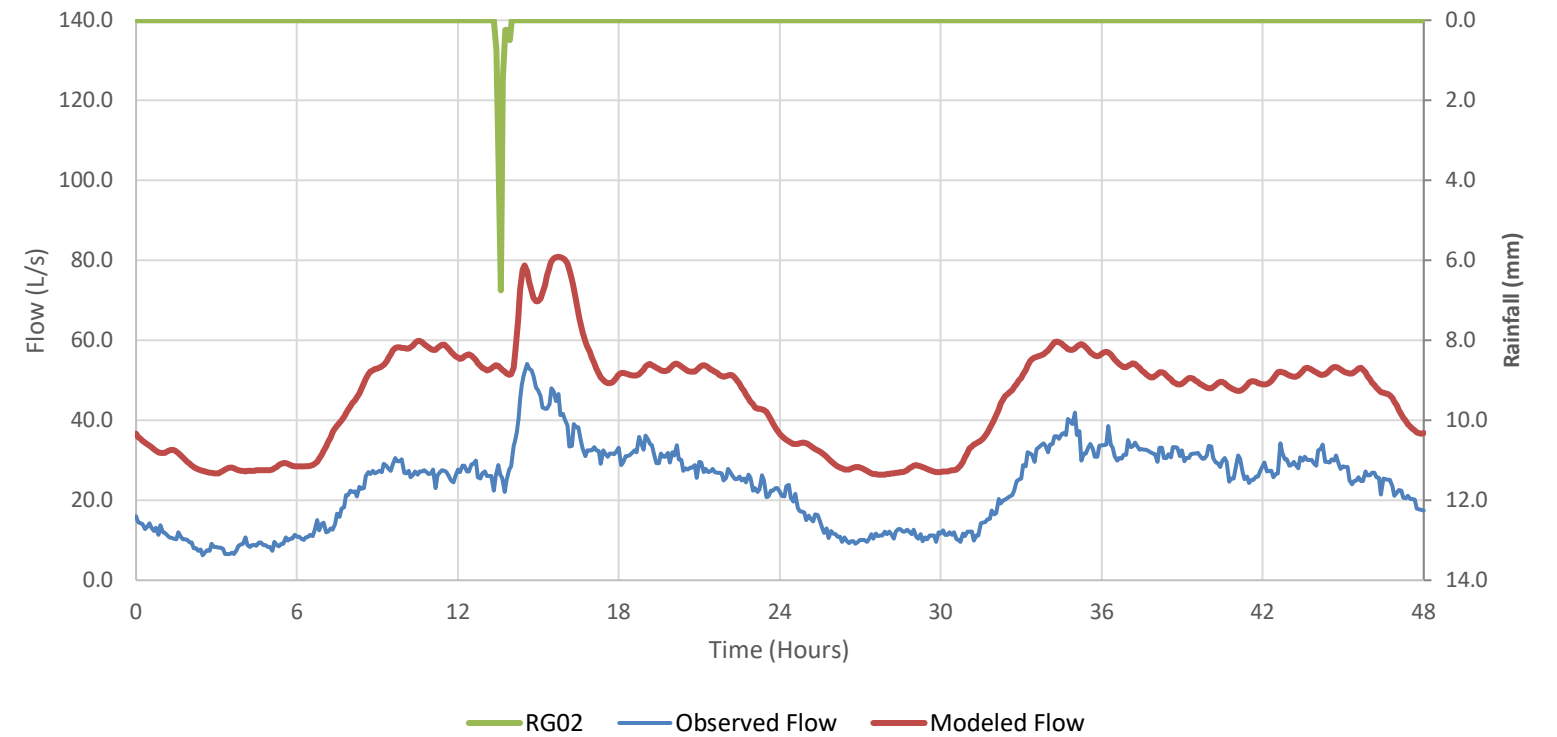
FM14a: WWF CALIBRATION - June 10, 2020



FM14a: WWF CALIBRATION - July 10, 2020

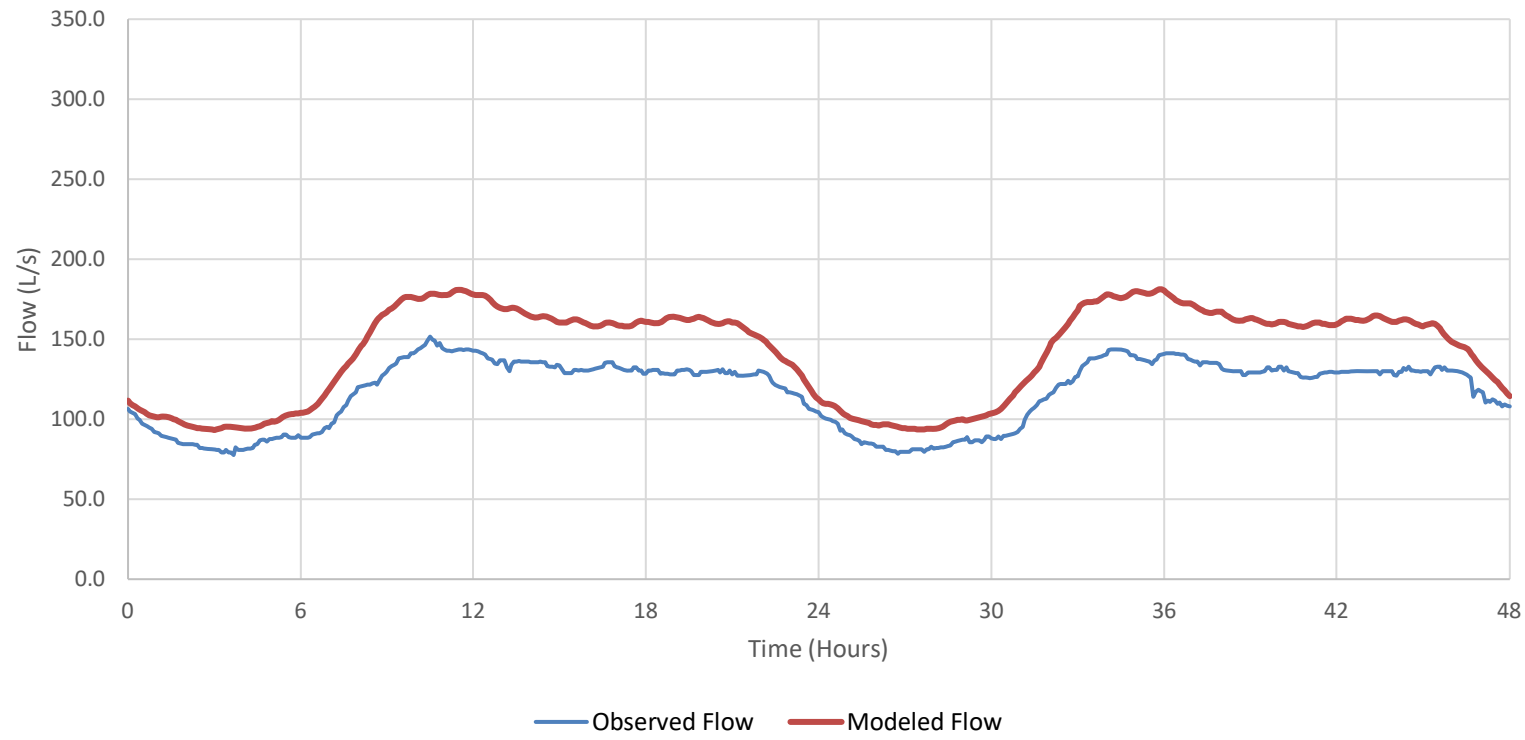


FM14a: WWF VALIDATION - May 29, 2020

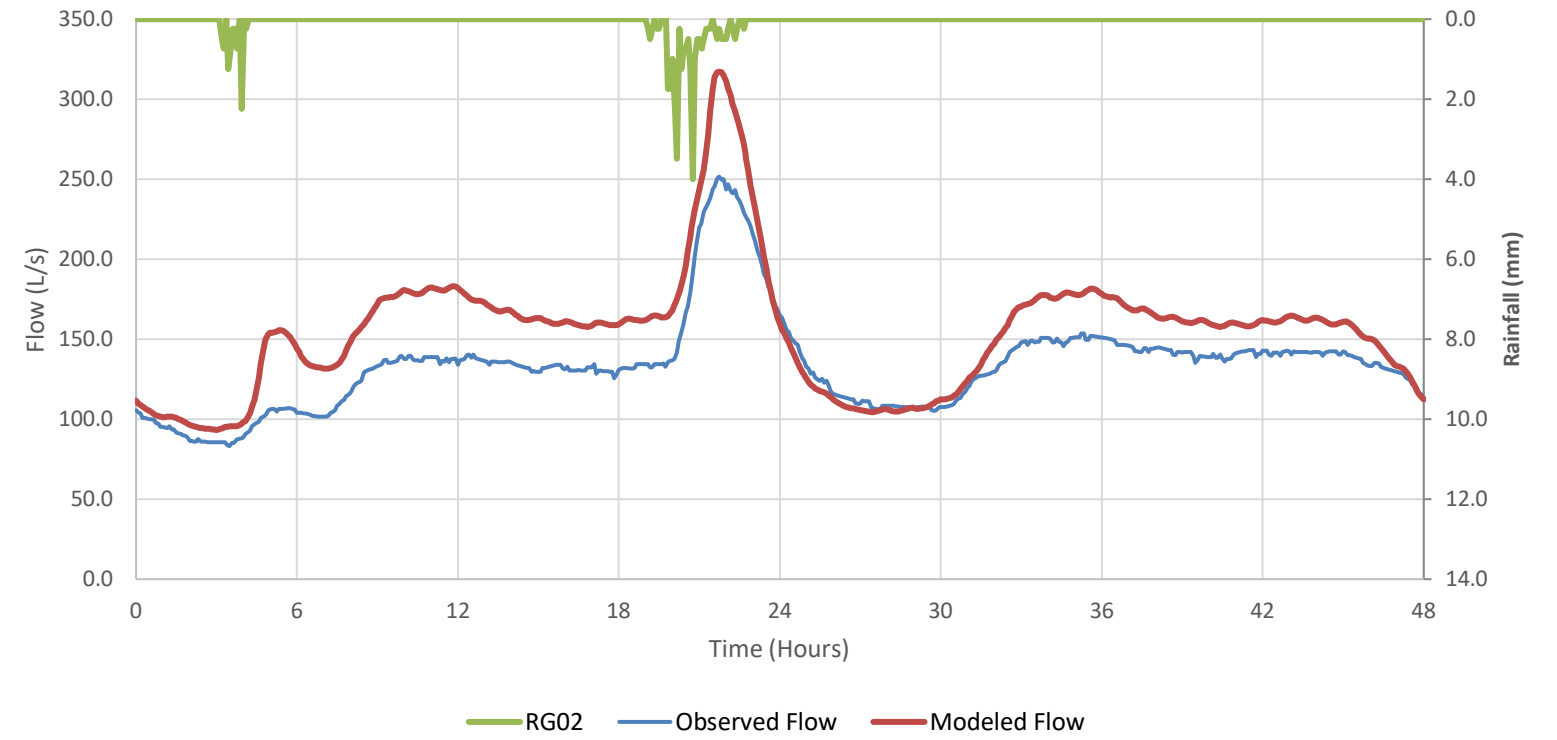


# FM14b CALIBRATION AND VALIDATION

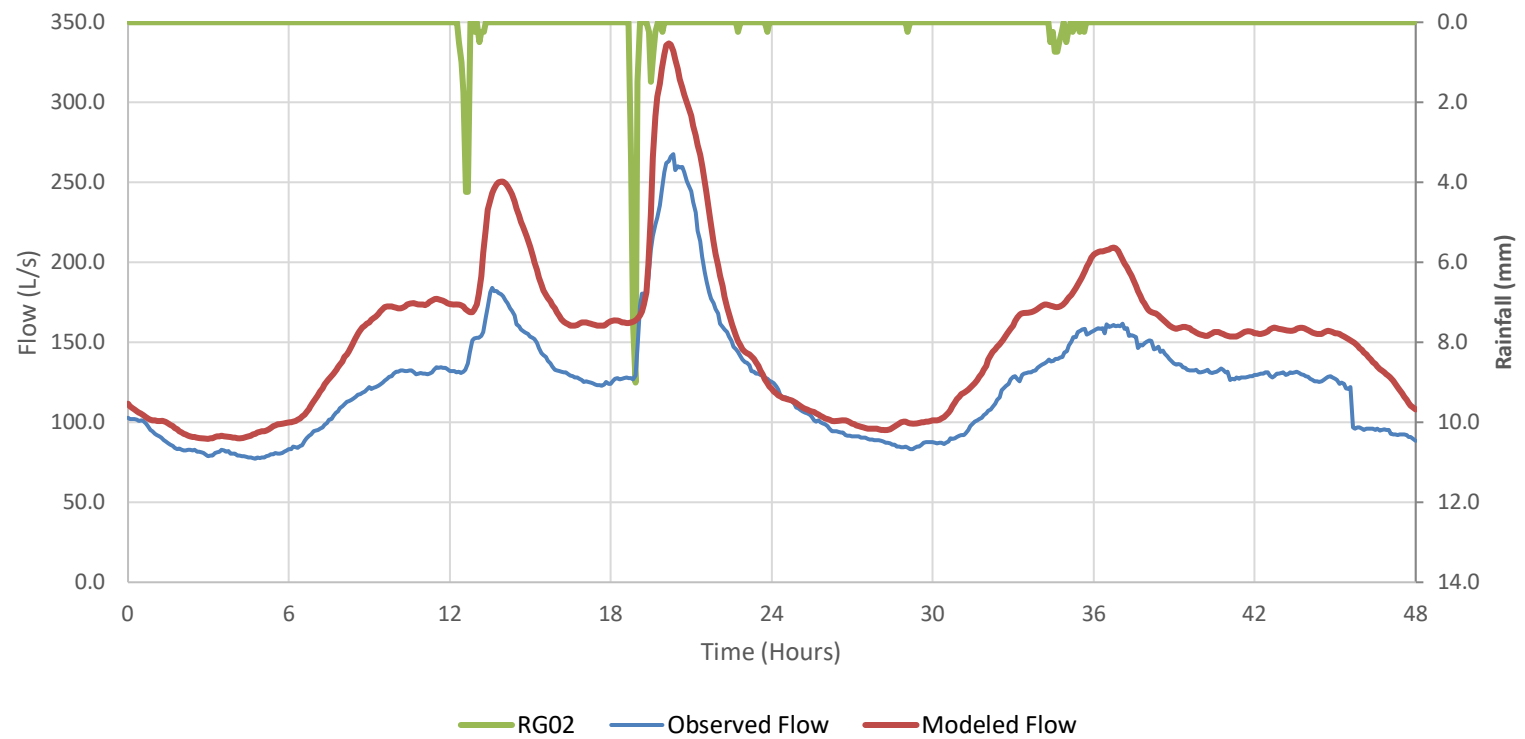
FM14b: DWF CALIBRATION - June 17, 2020



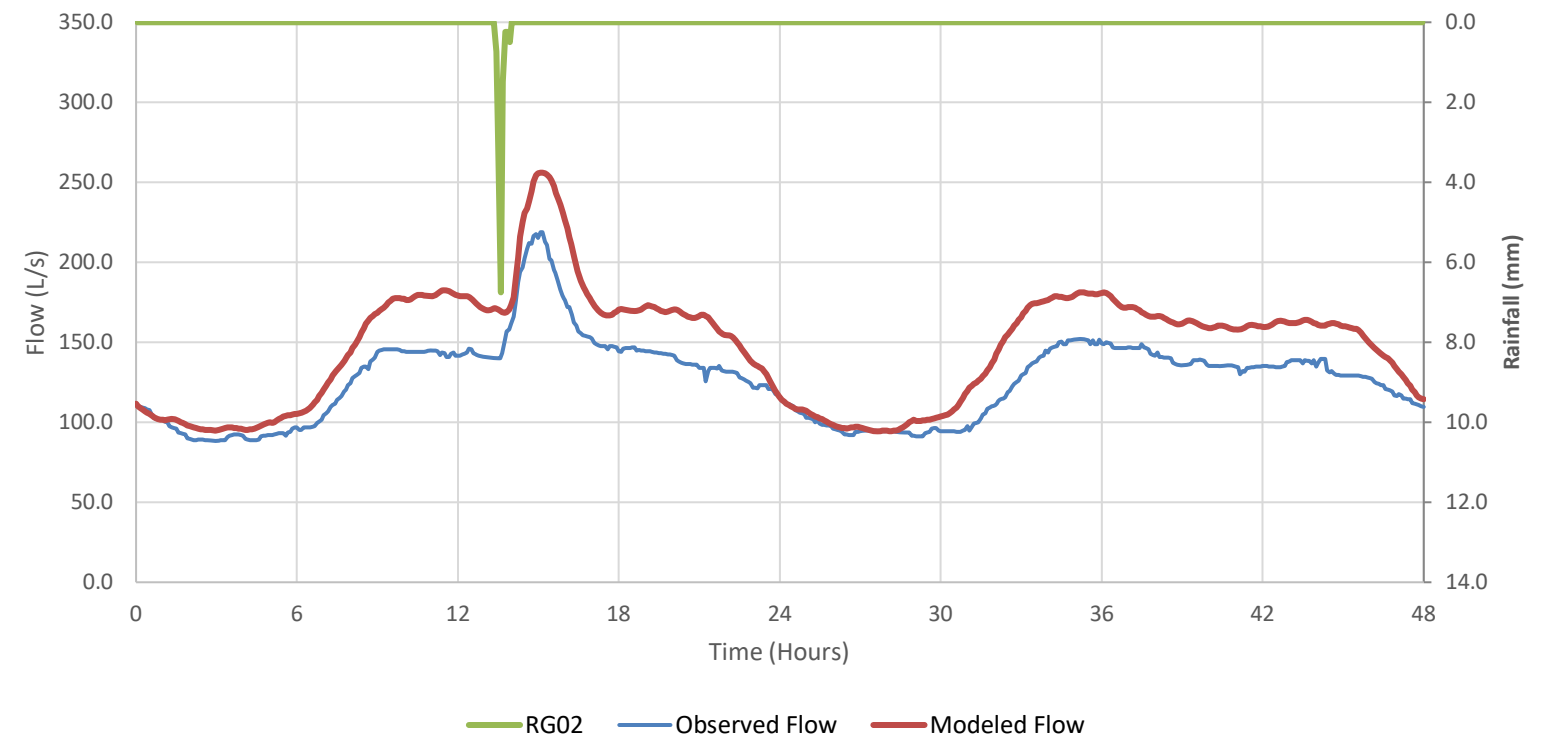
FM14b: WWF CALIBRATION - June 10, 2020



FM14b: WWF CALIBRATION - July 10, 2020



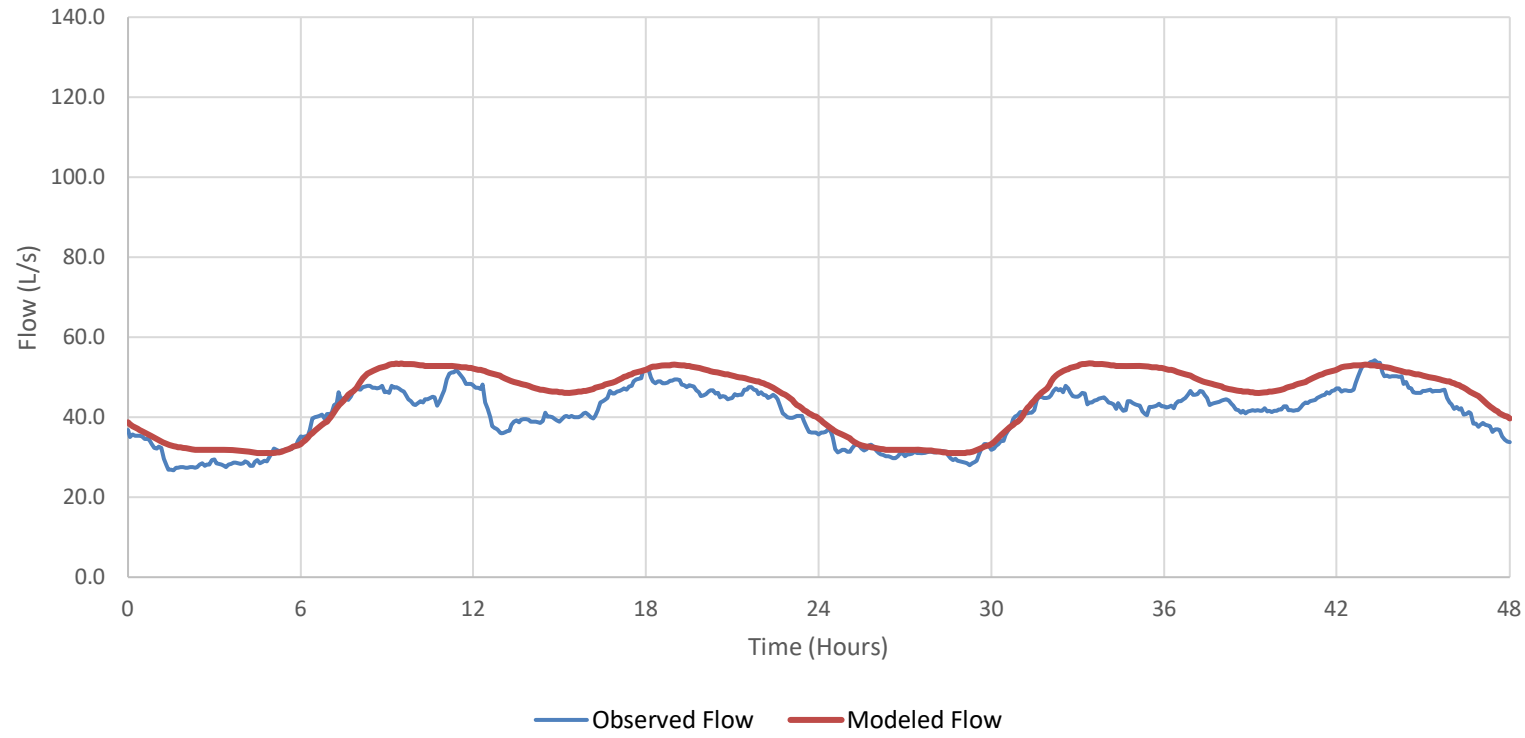
FM14b: WWF VALIDATION - May 29, 2020



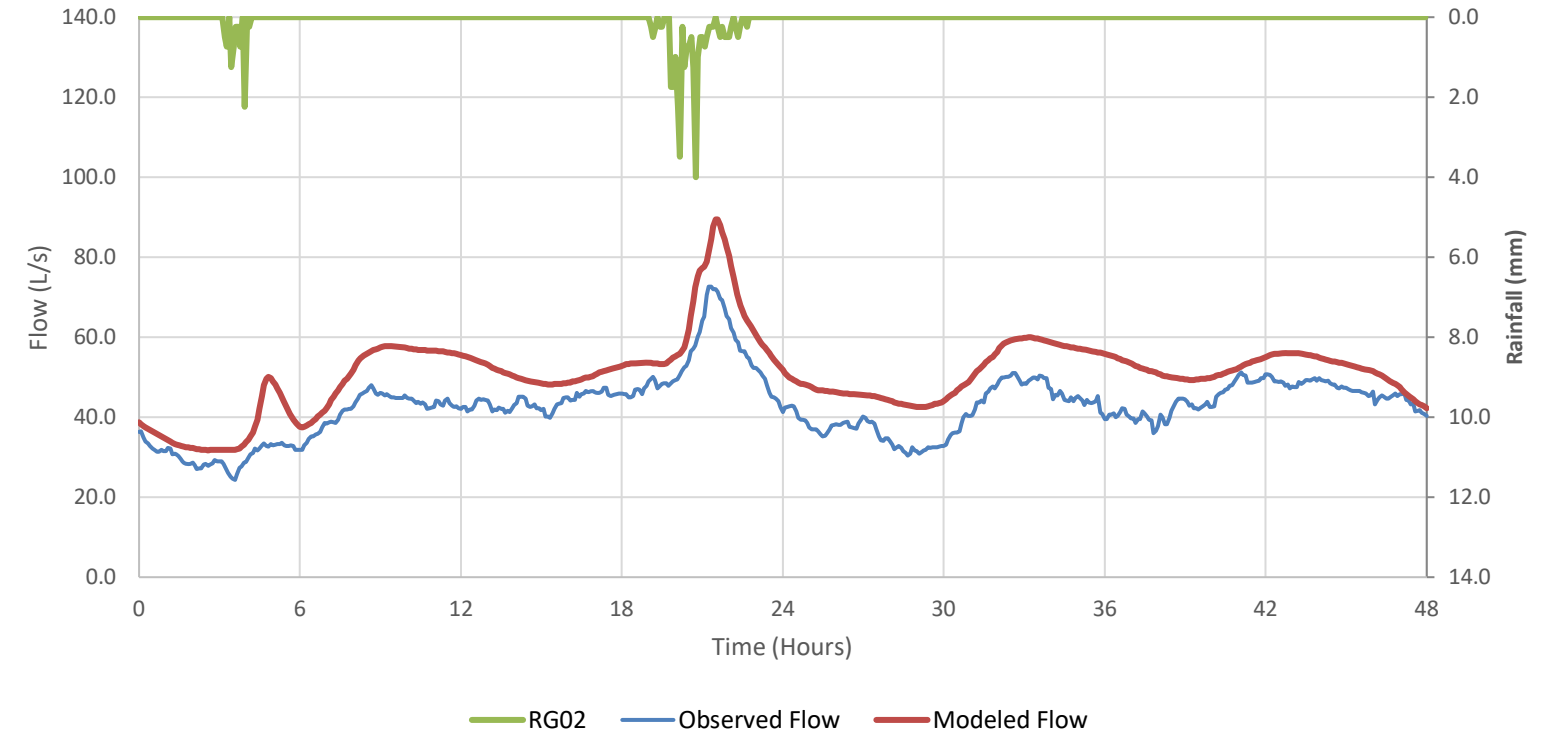


# FM15 CALIBRATION AND VALIDATION

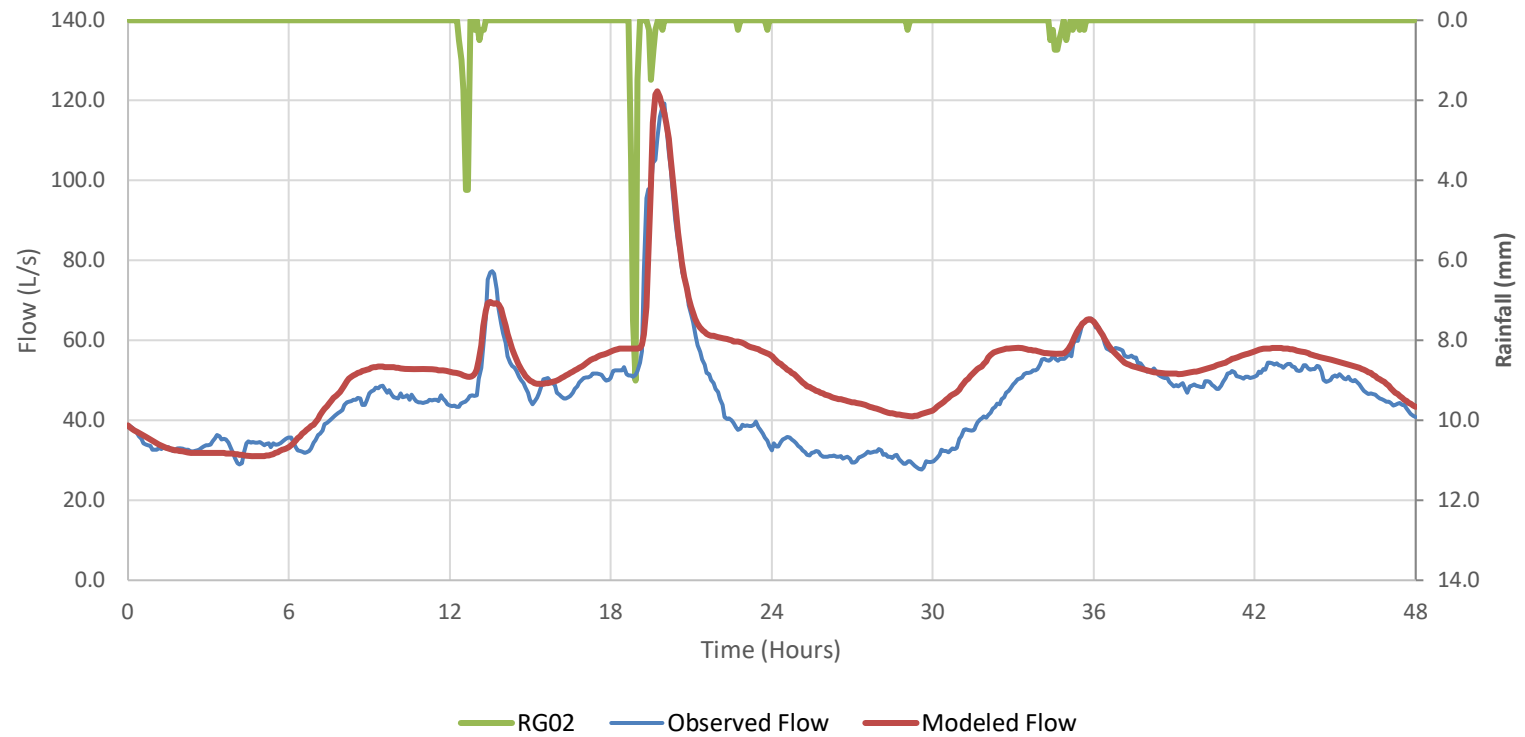
FM15: DWF CALIBRATION - June 17, 2020



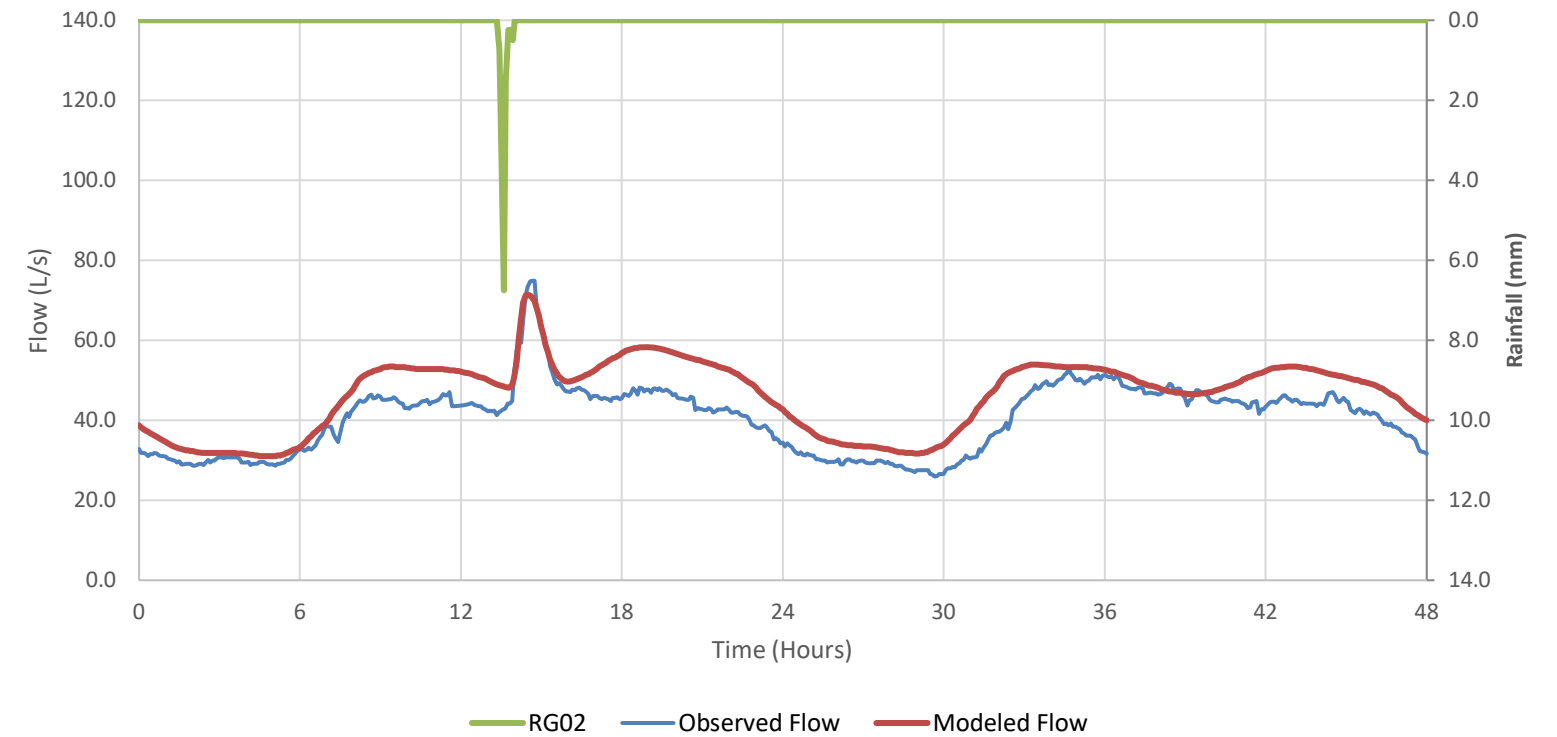
FM15: WWF CALIBRATION - June 10, 2020



FM15: WWF CALIBRATION - July 10, 2020

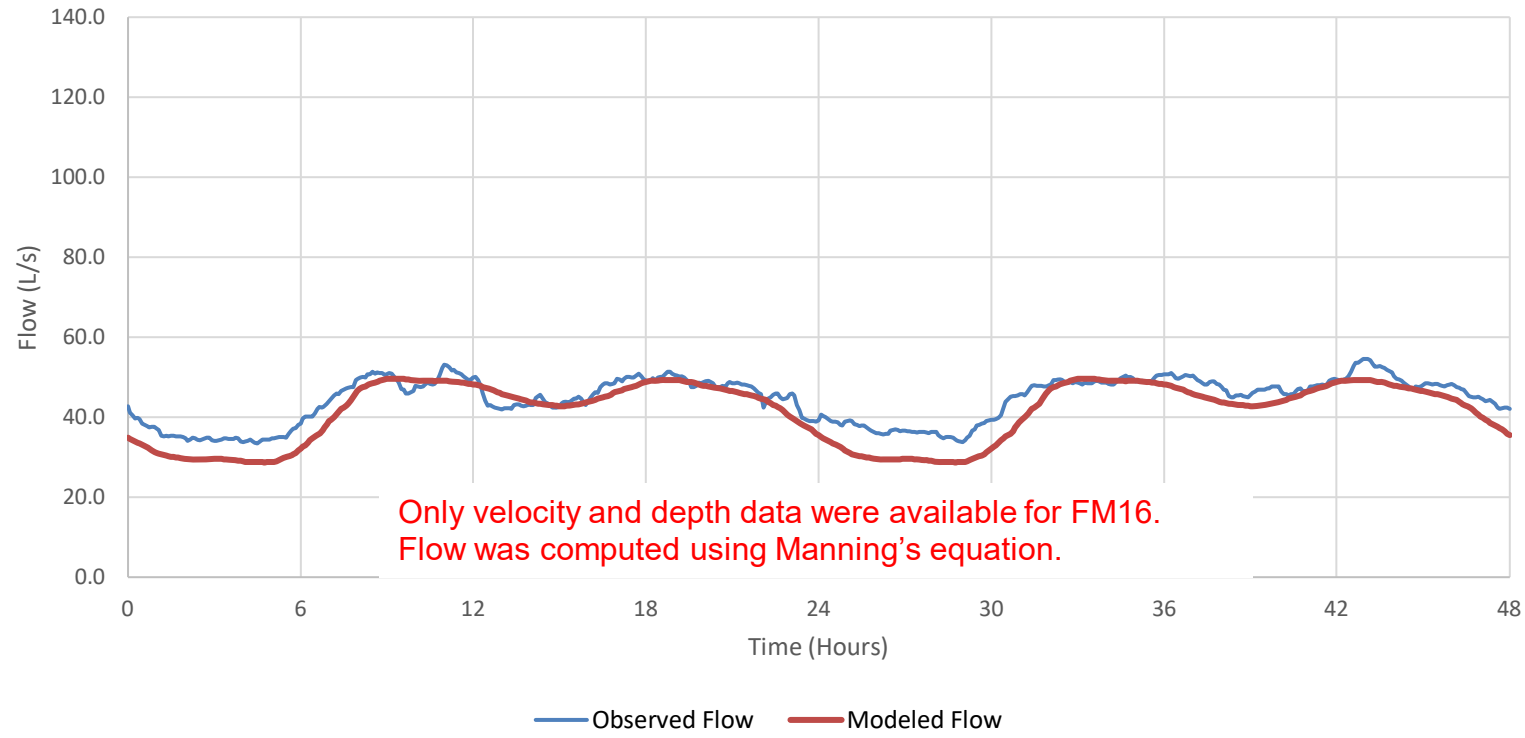


FM15: WWF VALIDATION - May 29, 2020

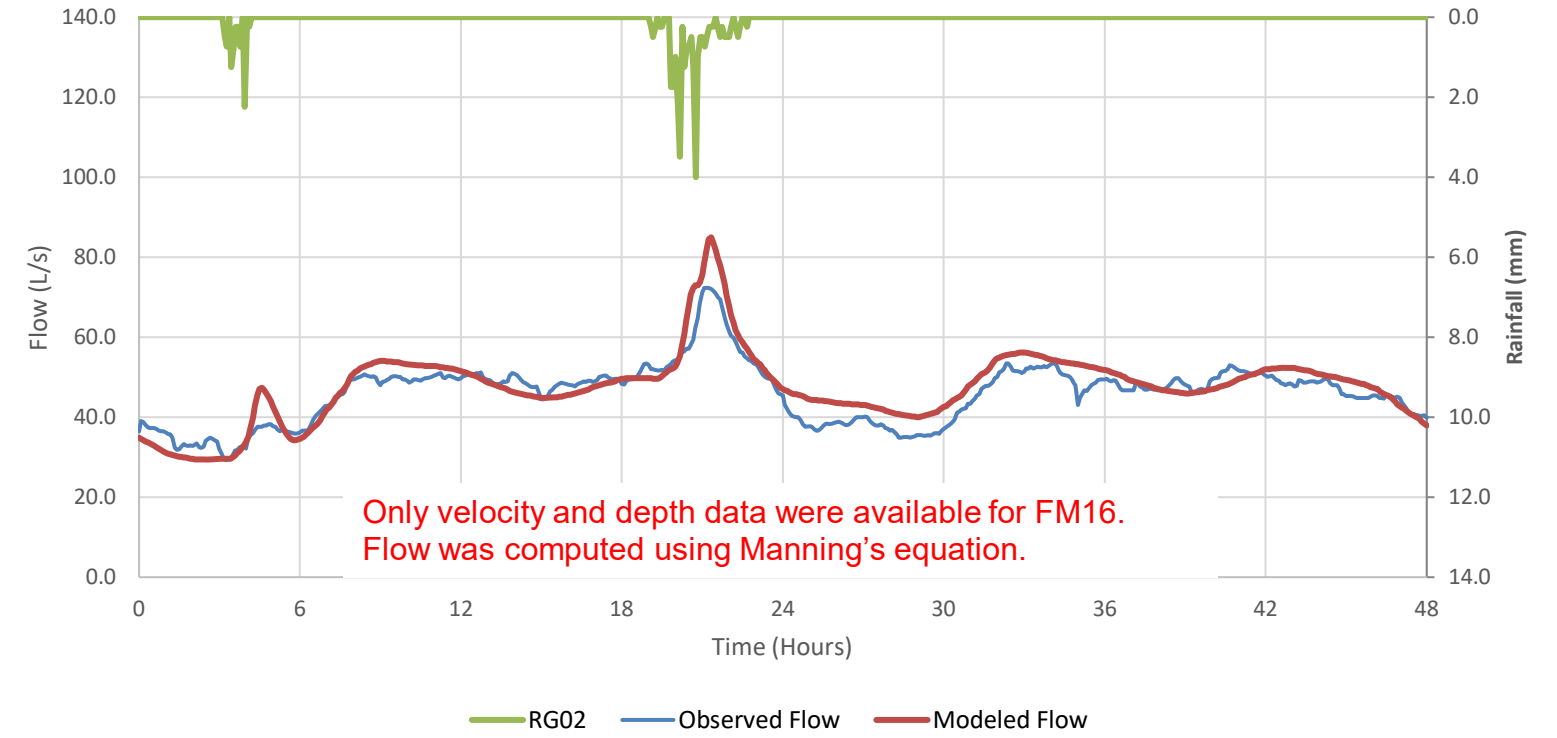


# FM16 CALIBRATION AND VALIDATION

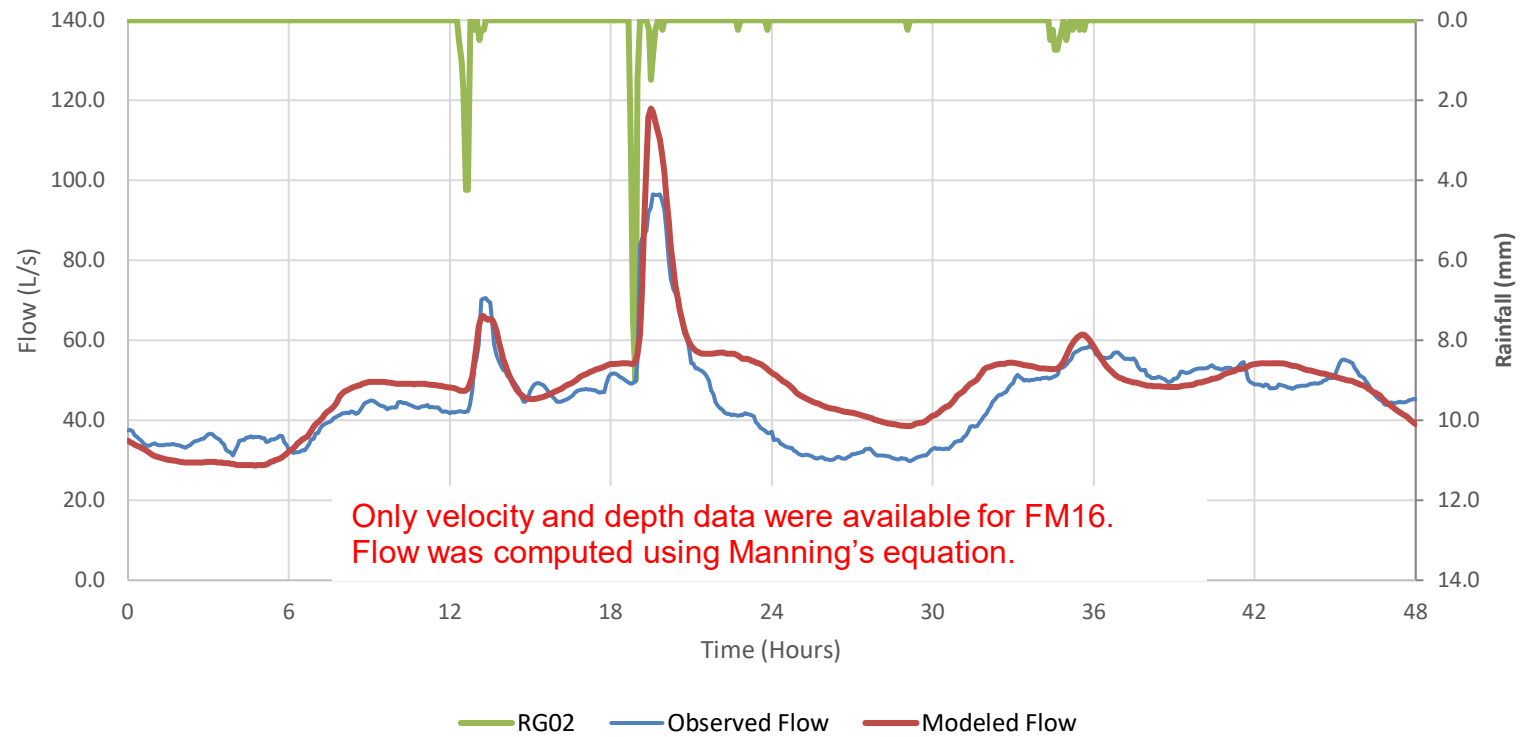
FM16: DWF CALIBRATION - June 17, 2020



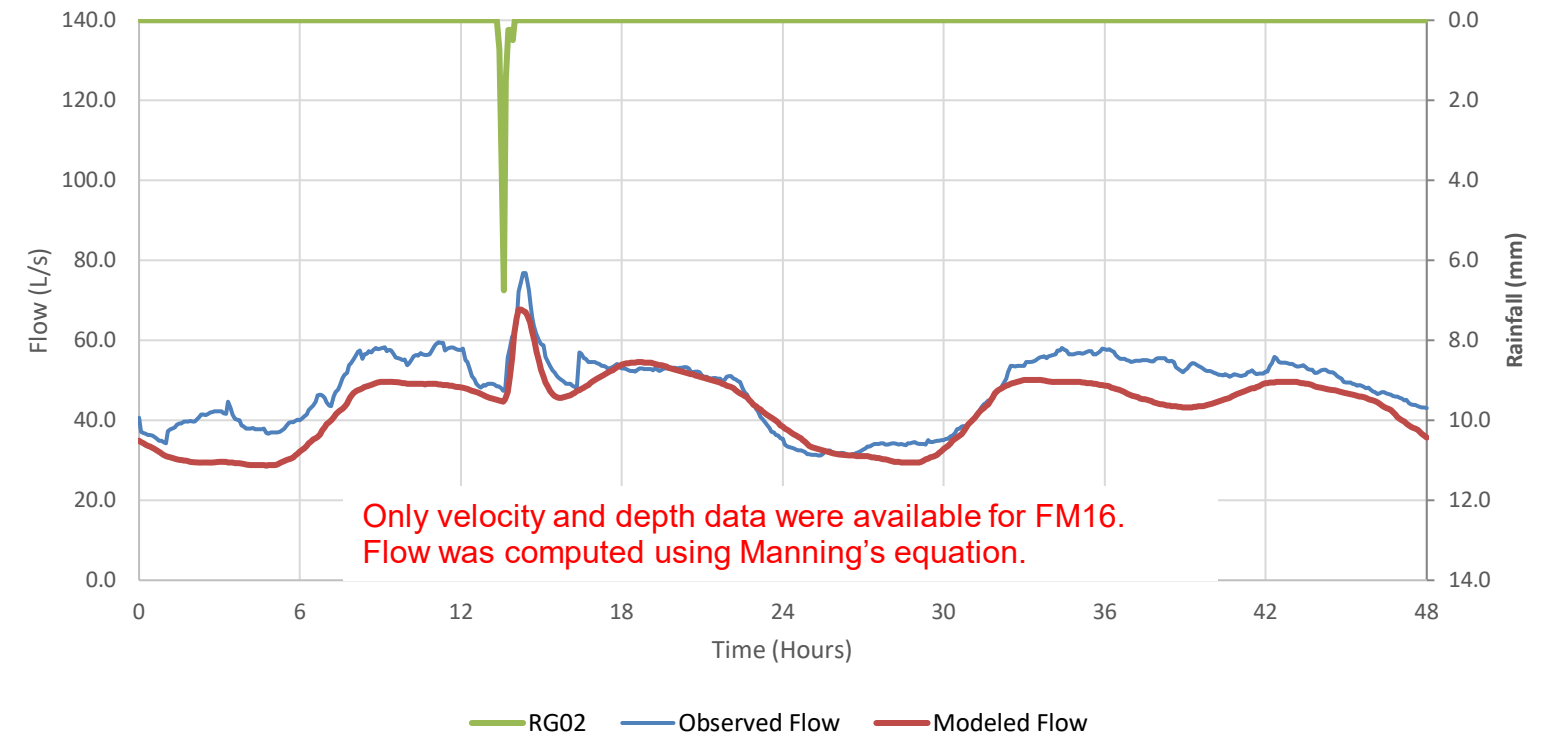
FM16: WWF CALIBRATION - June 10, 2020



FM16: WWF CALIBRATION - July 10, 2020

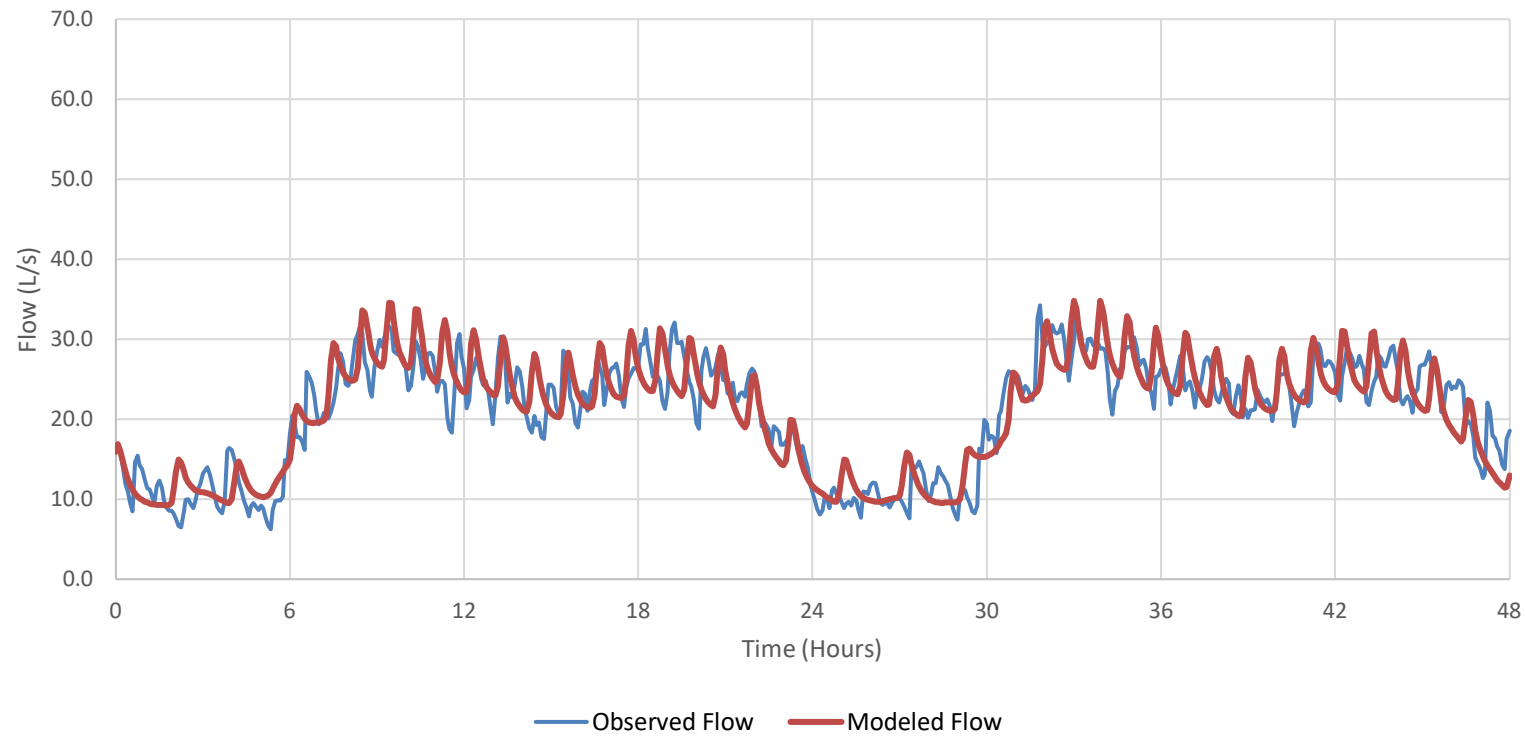


FM16: WWF VALIDATION - May 29, 2020

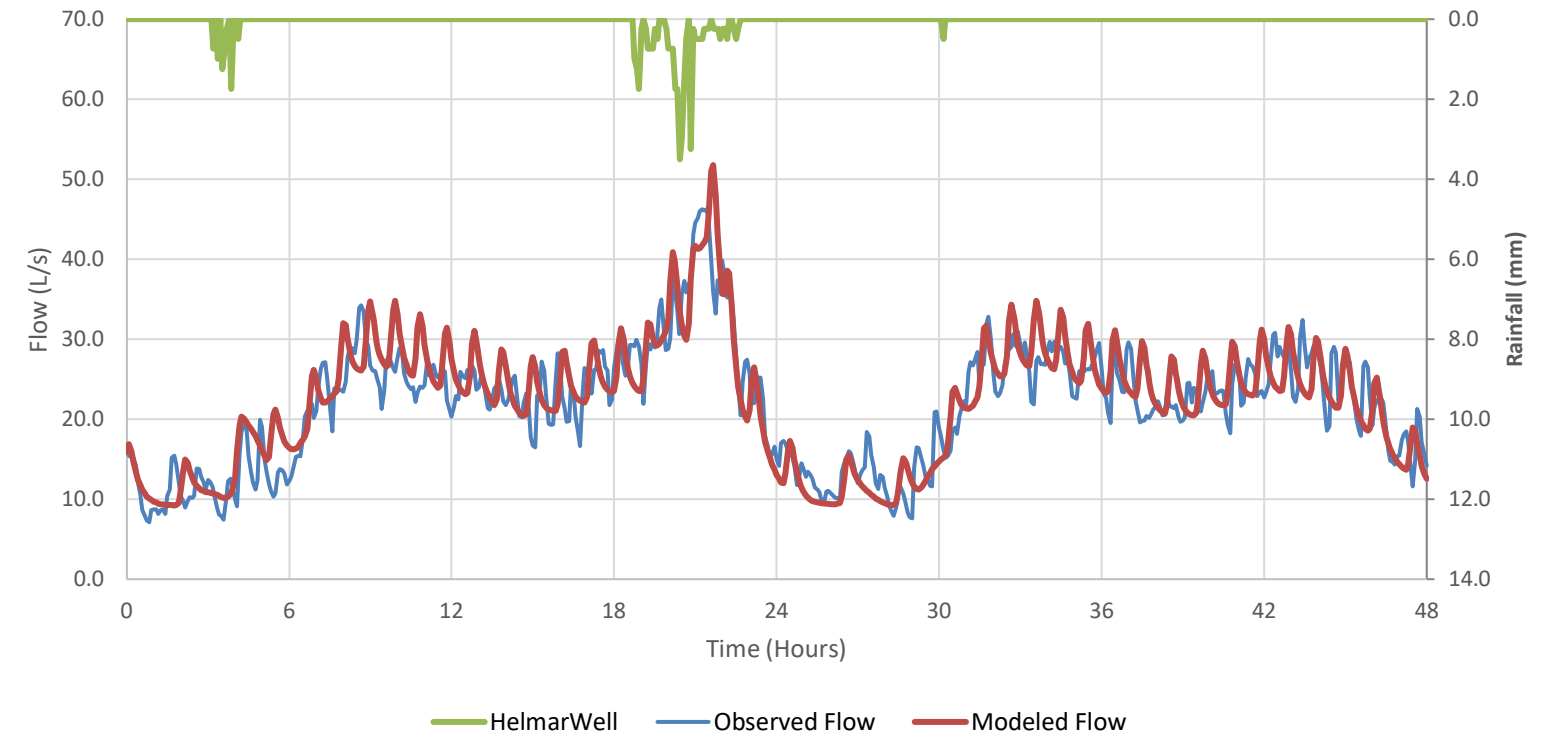


# FM17 CALIBRATION AND VALIDATION

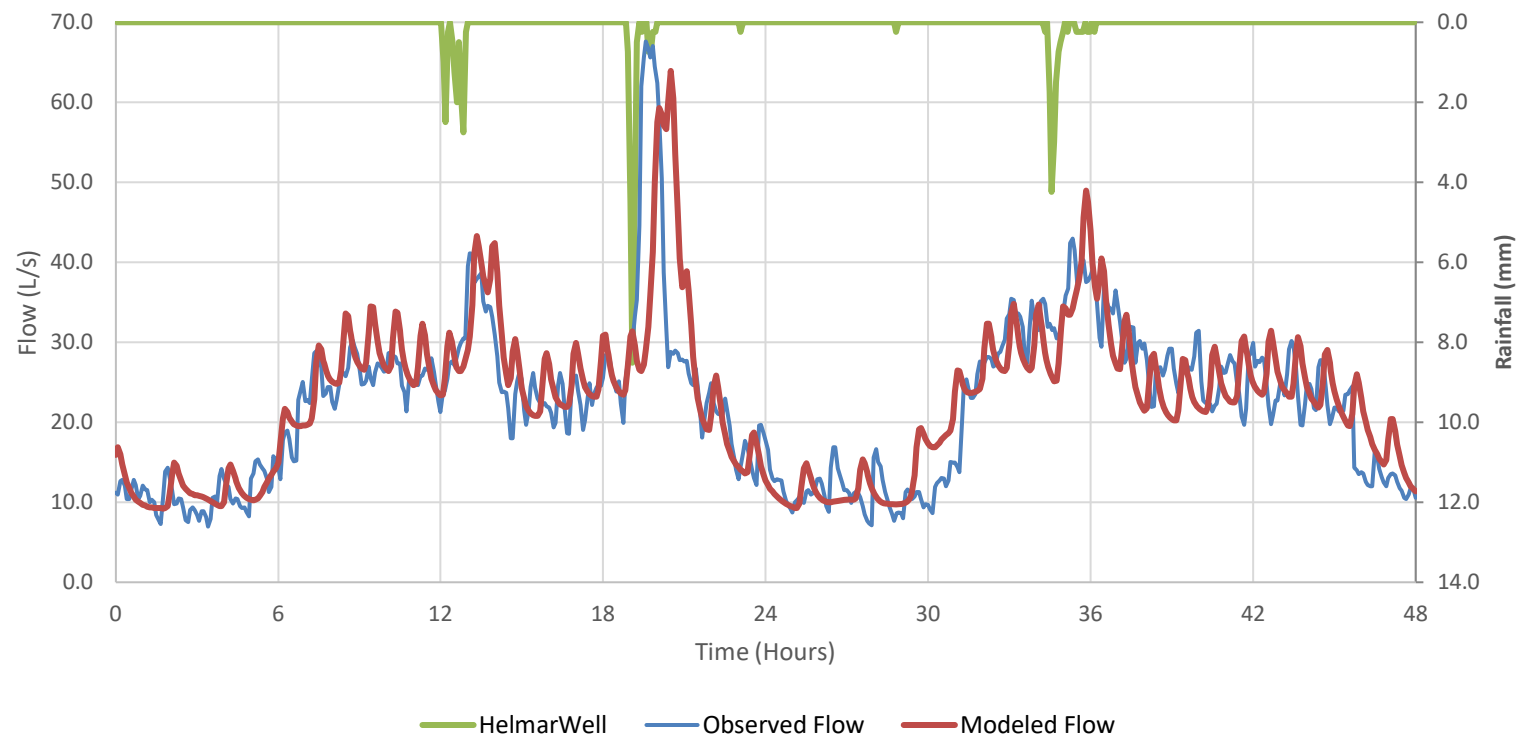
FM17: DWF CALIBRATION - June 17, 2020



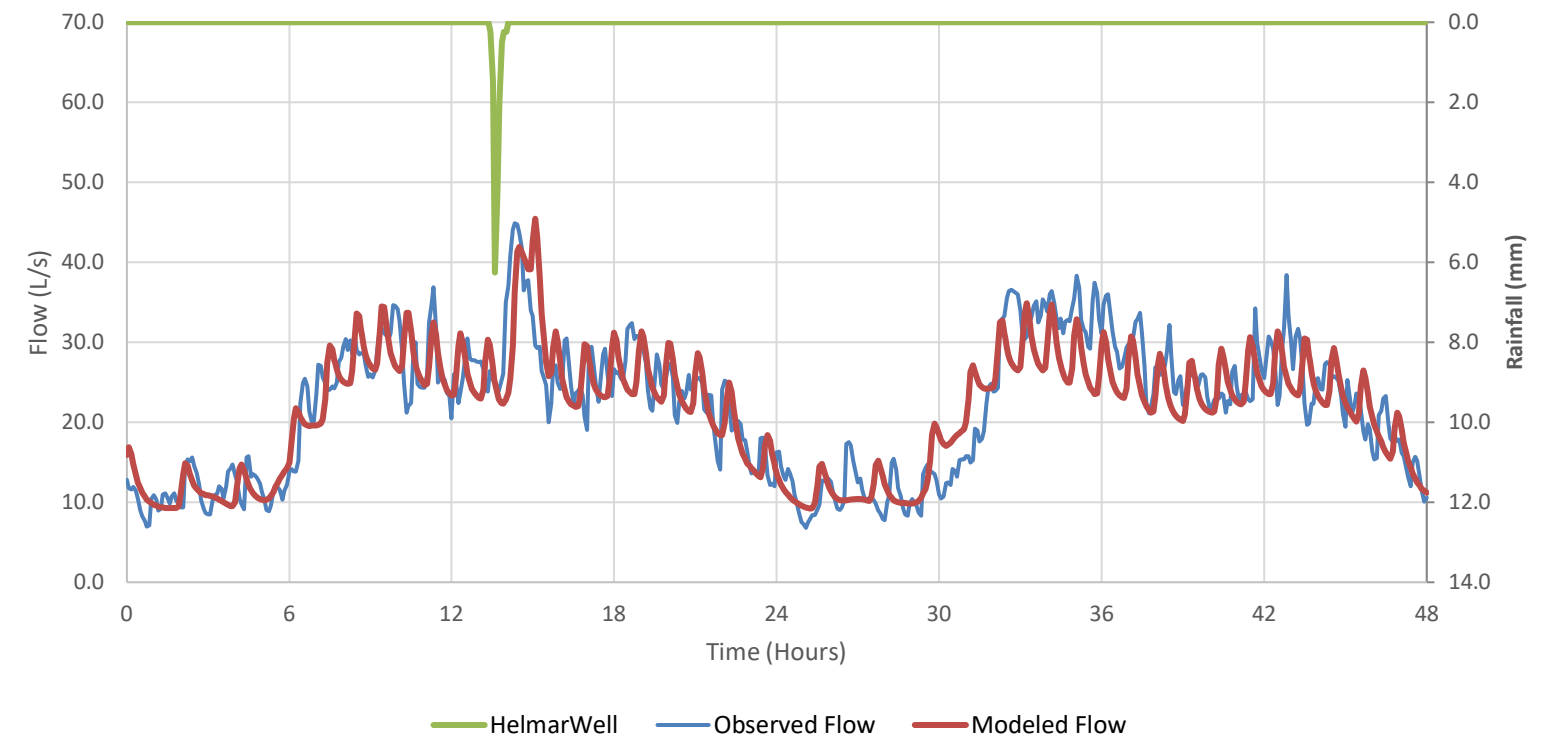
FM17: WWF CALIBRATION - June 10, 2020



FM17: WWF CALIBRATION - July 10, 2020

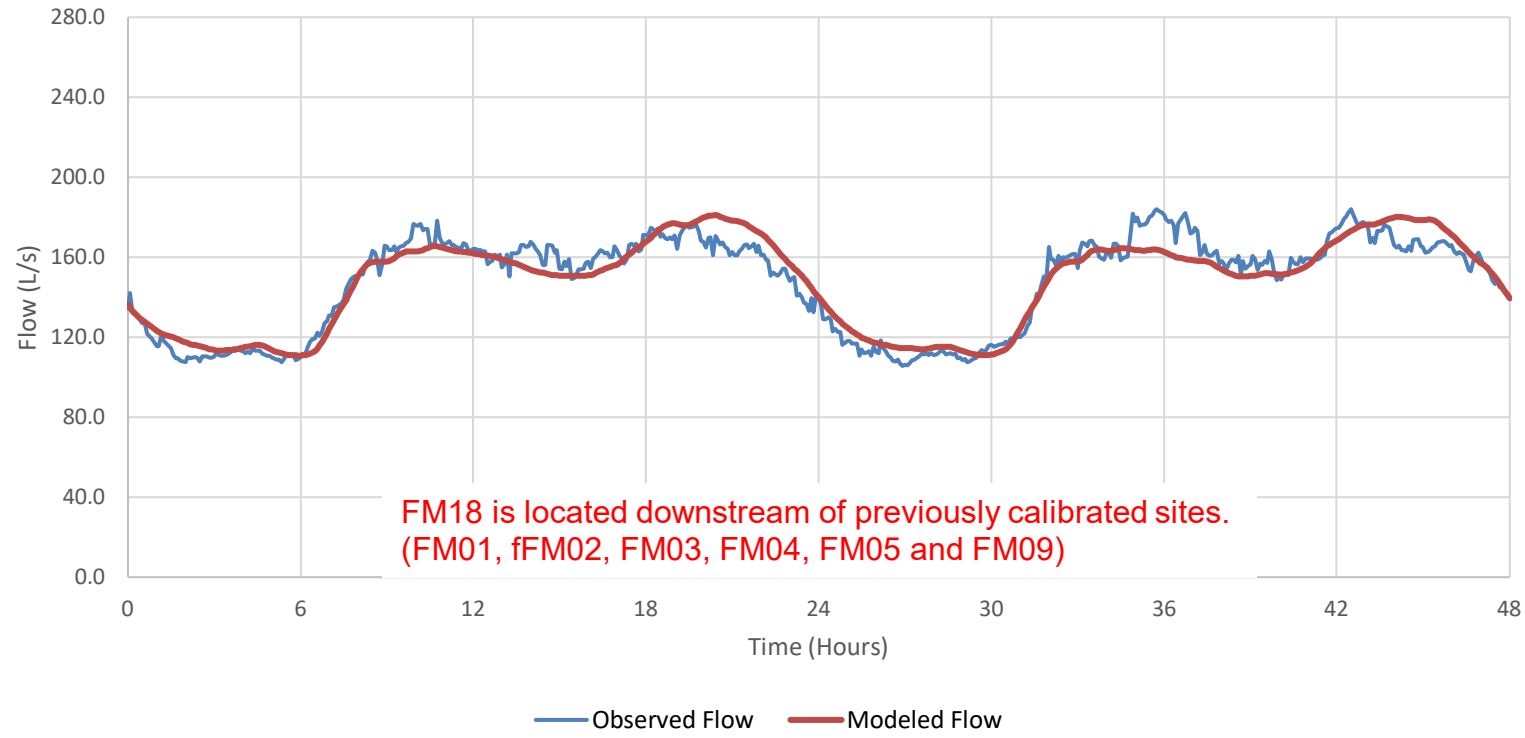


FM17: WWF VALIDATION - May 29, 2020

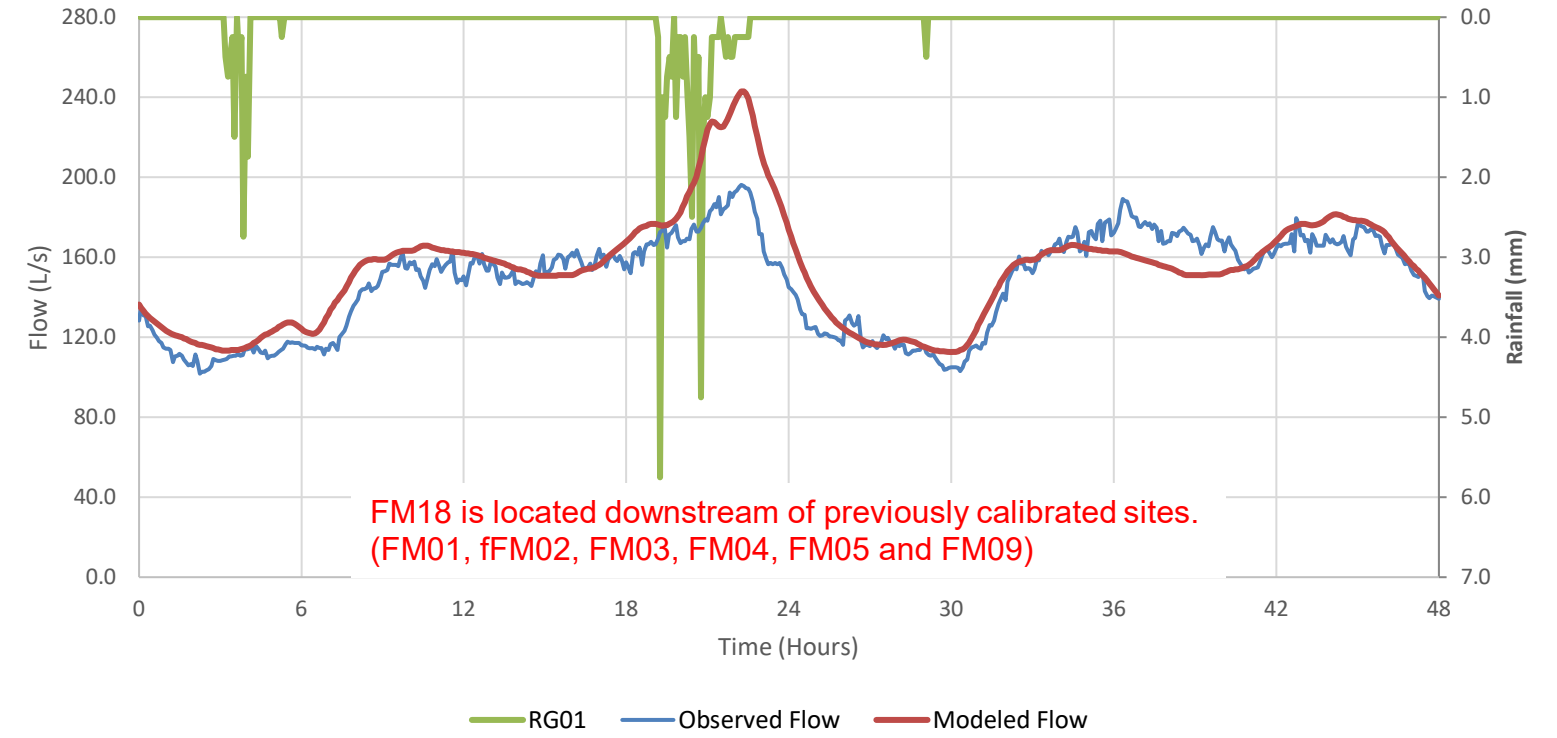


# FM18 CALIBRATION AND VALIDATION

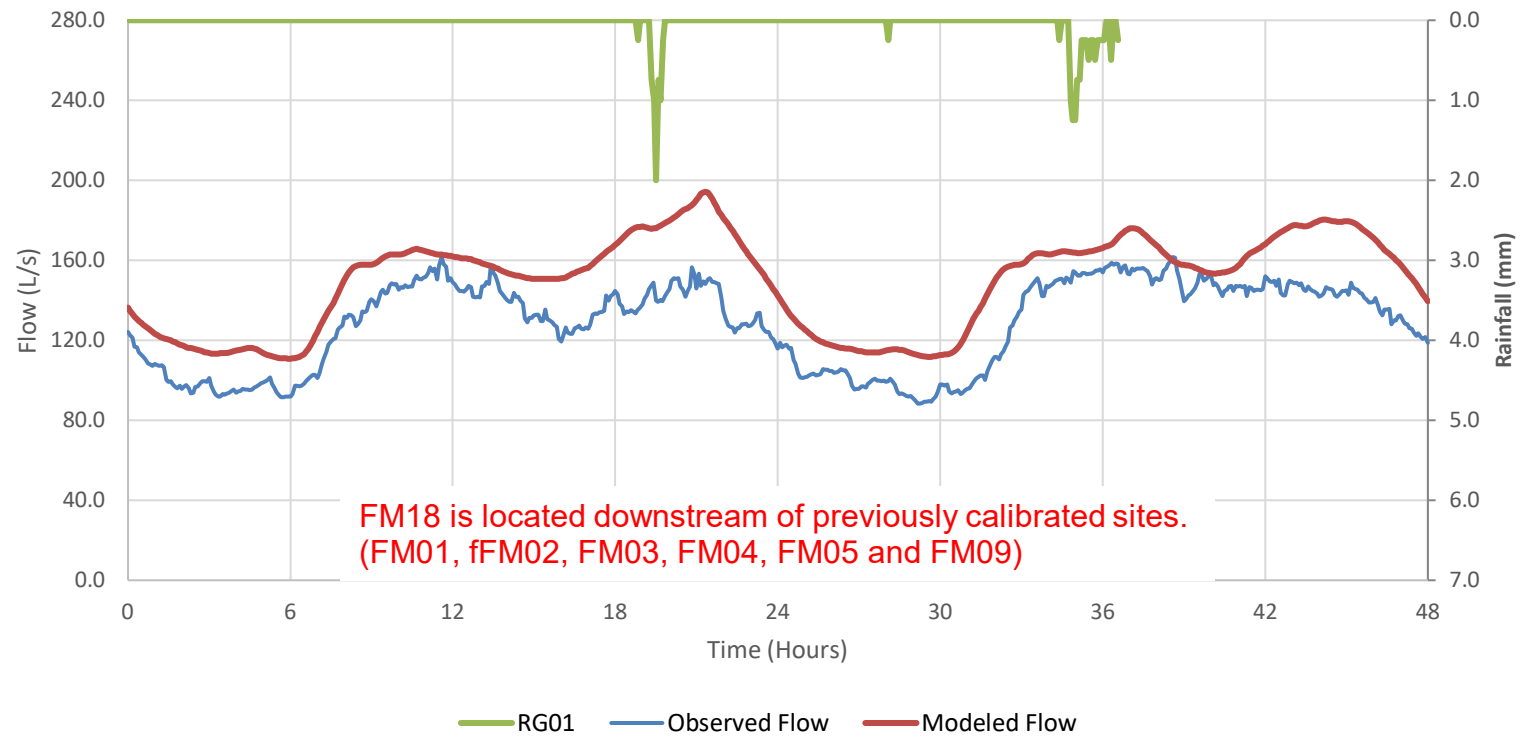
FM18: DWF CALIBRATION - June 17, 2020



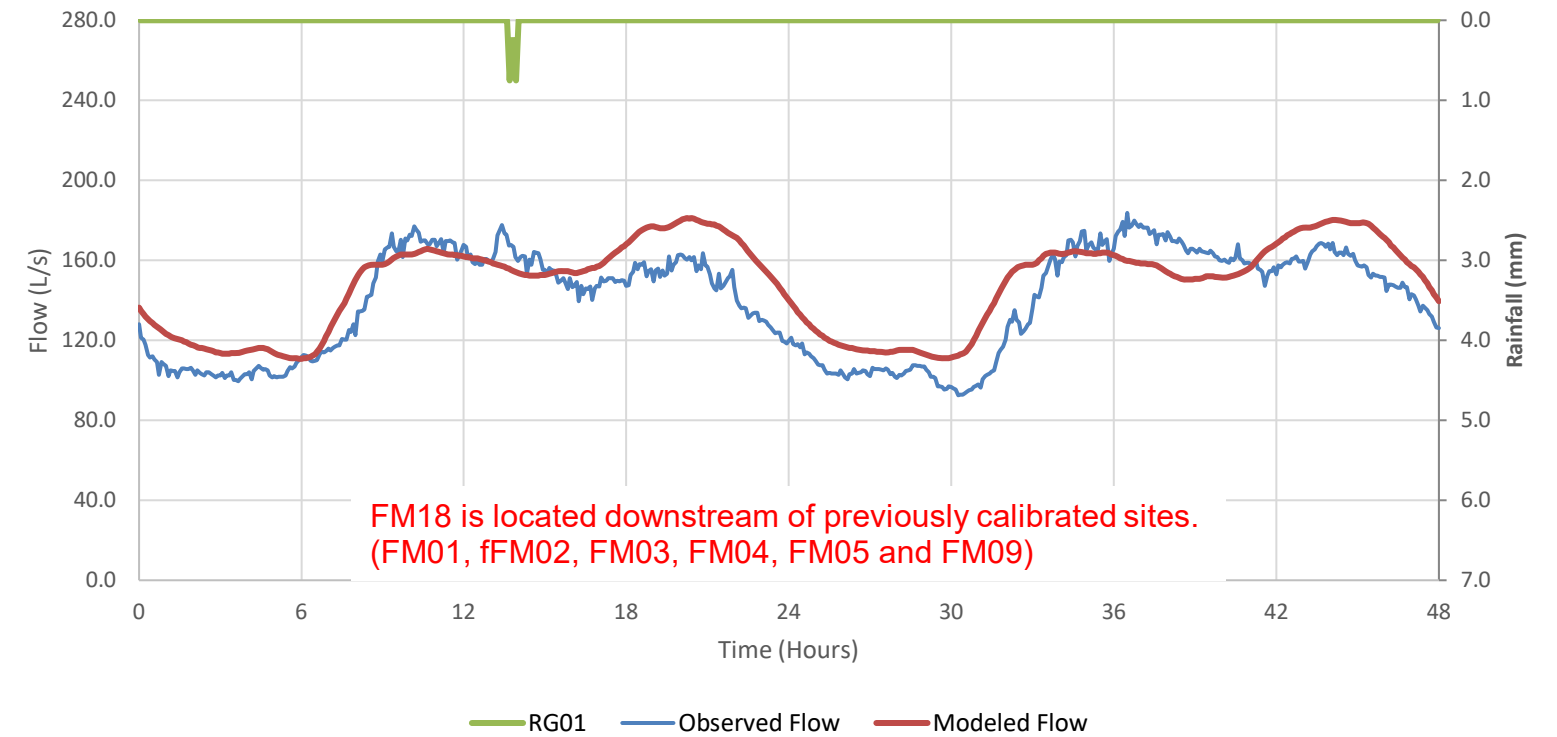
FM18: WWF CALIBRATION - June 10, 2020



FM18: WWF CALIBRATION - July 10, 2020

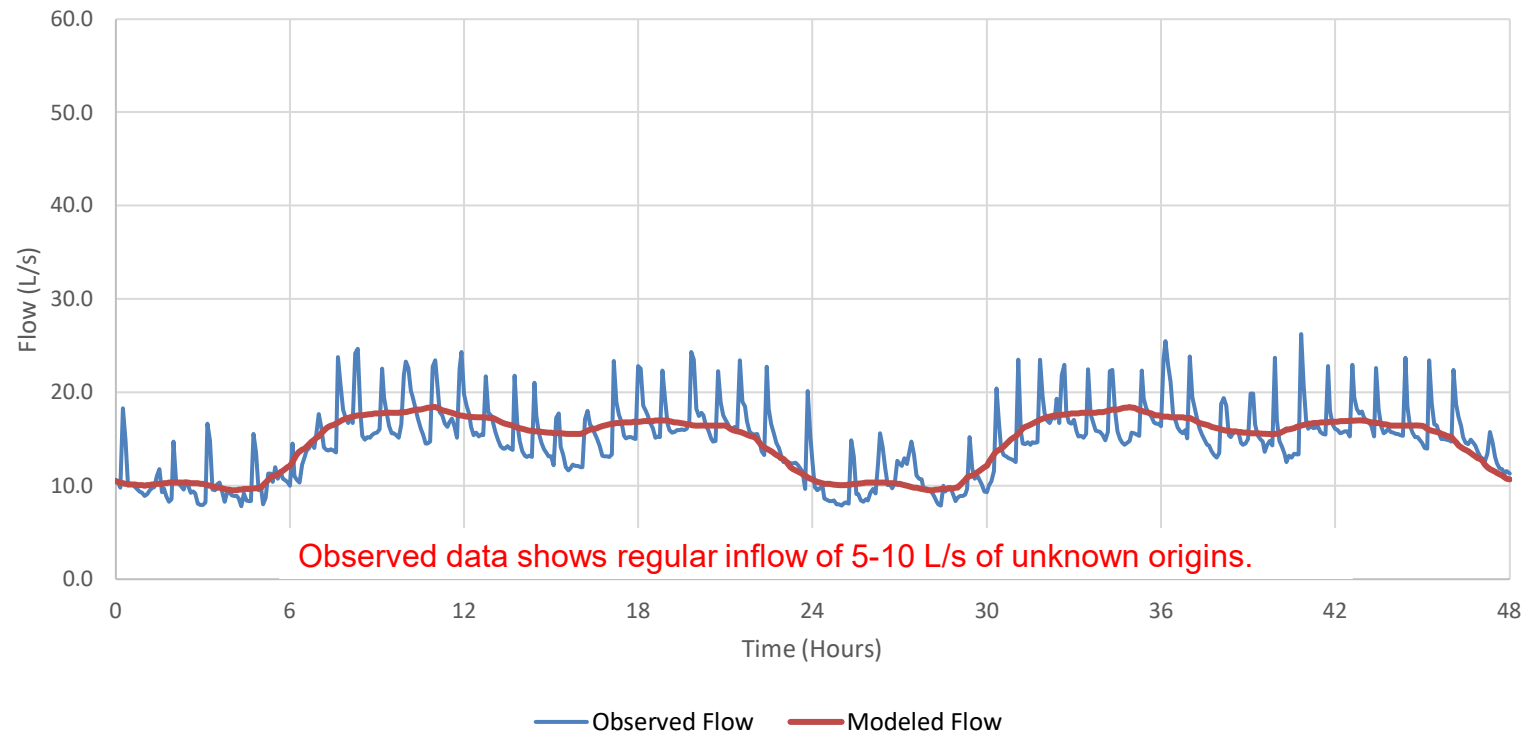


FM18: WWF VALIDATION - May 29, 2020

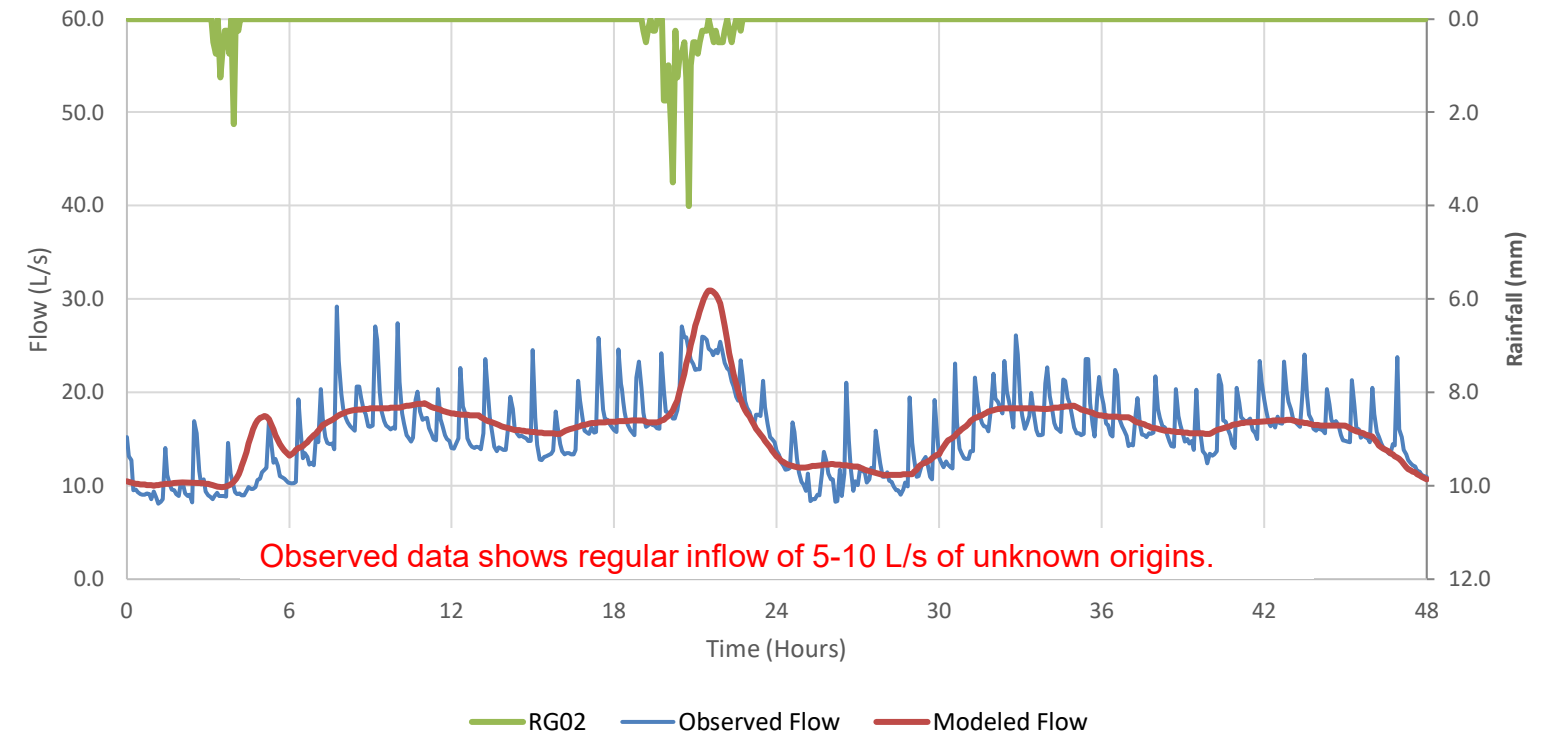


# FM19 CALIBRATION AND VALIDATION

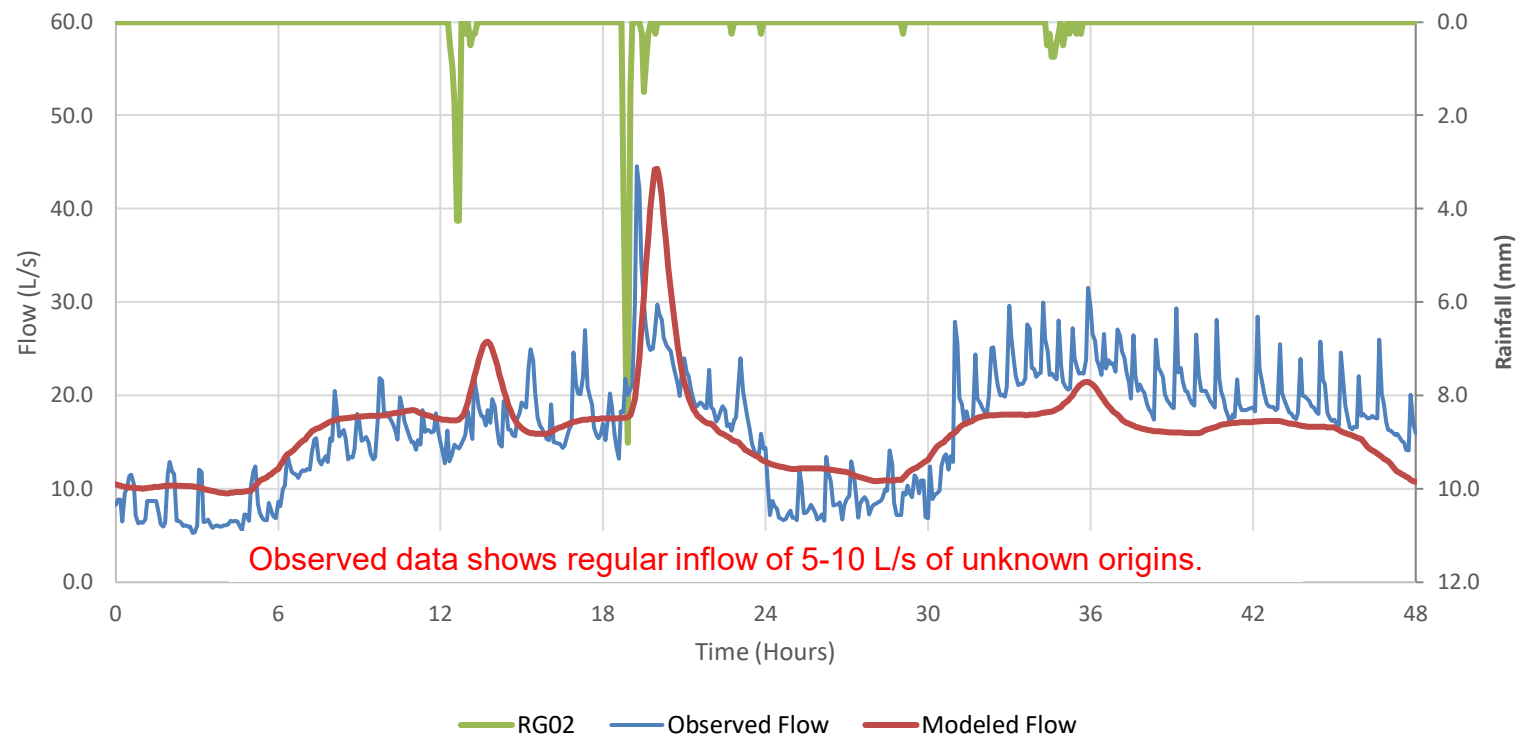
FM19: DWF CALIBRATION - June 17, 2020



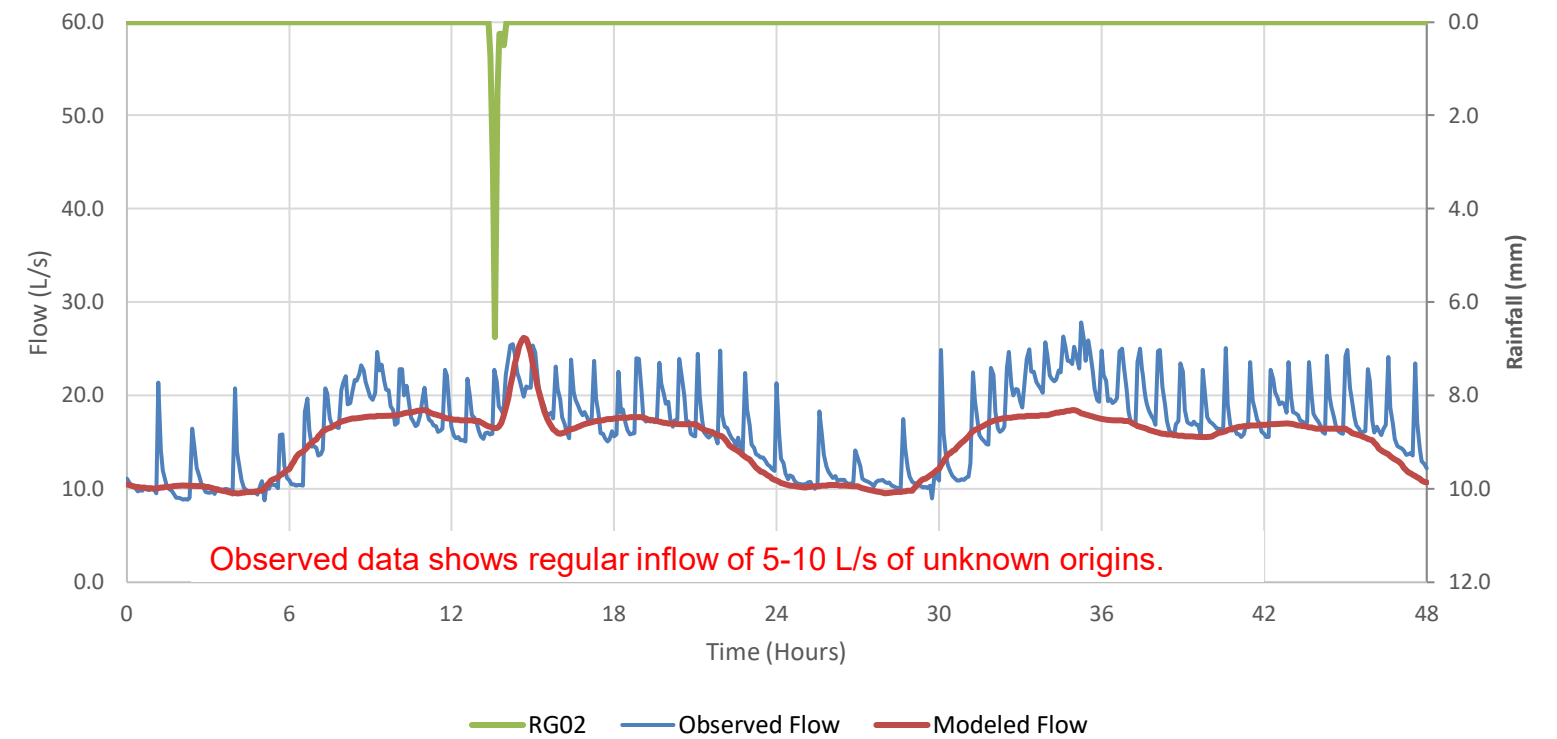
FM19: WWF CALIBRATION - June 10, 2020



FM19: WWF CALIBRATION - July 10, 2020

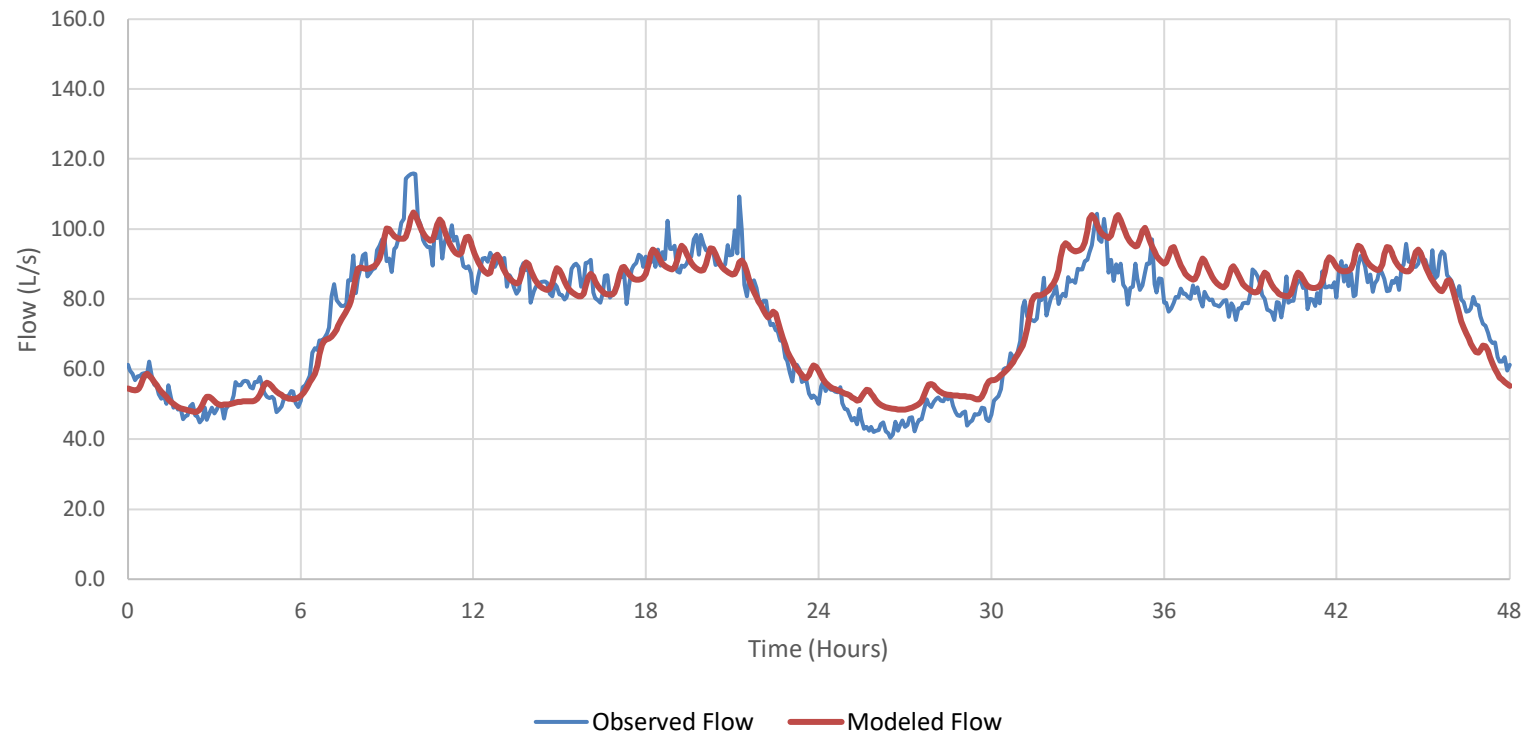


FM19: WWF VALIDATION - May 29, 2020

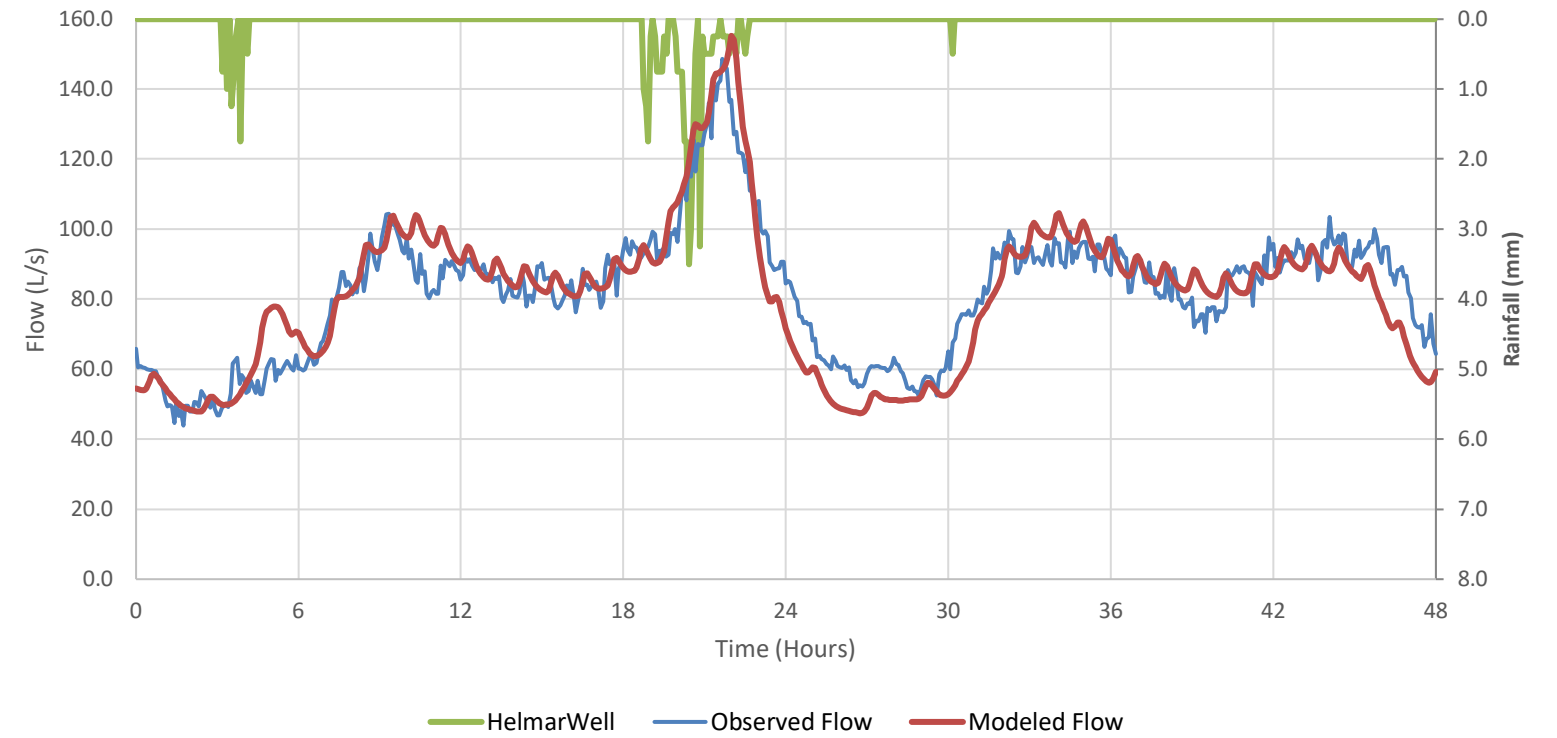


# FM20 CALIBRATION AND VALIDATION

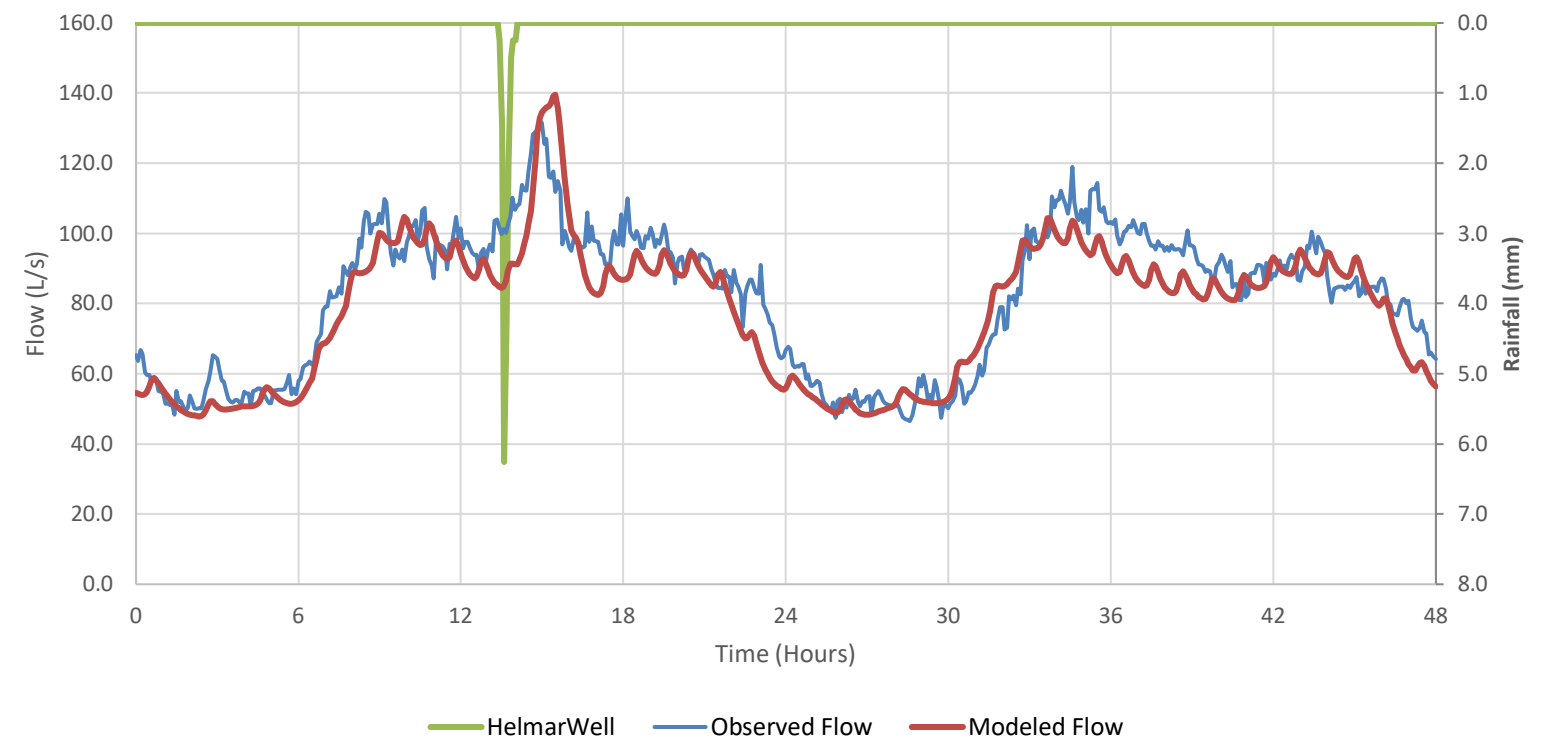
FM20: DWF CALIBRATION - June 17, 2020



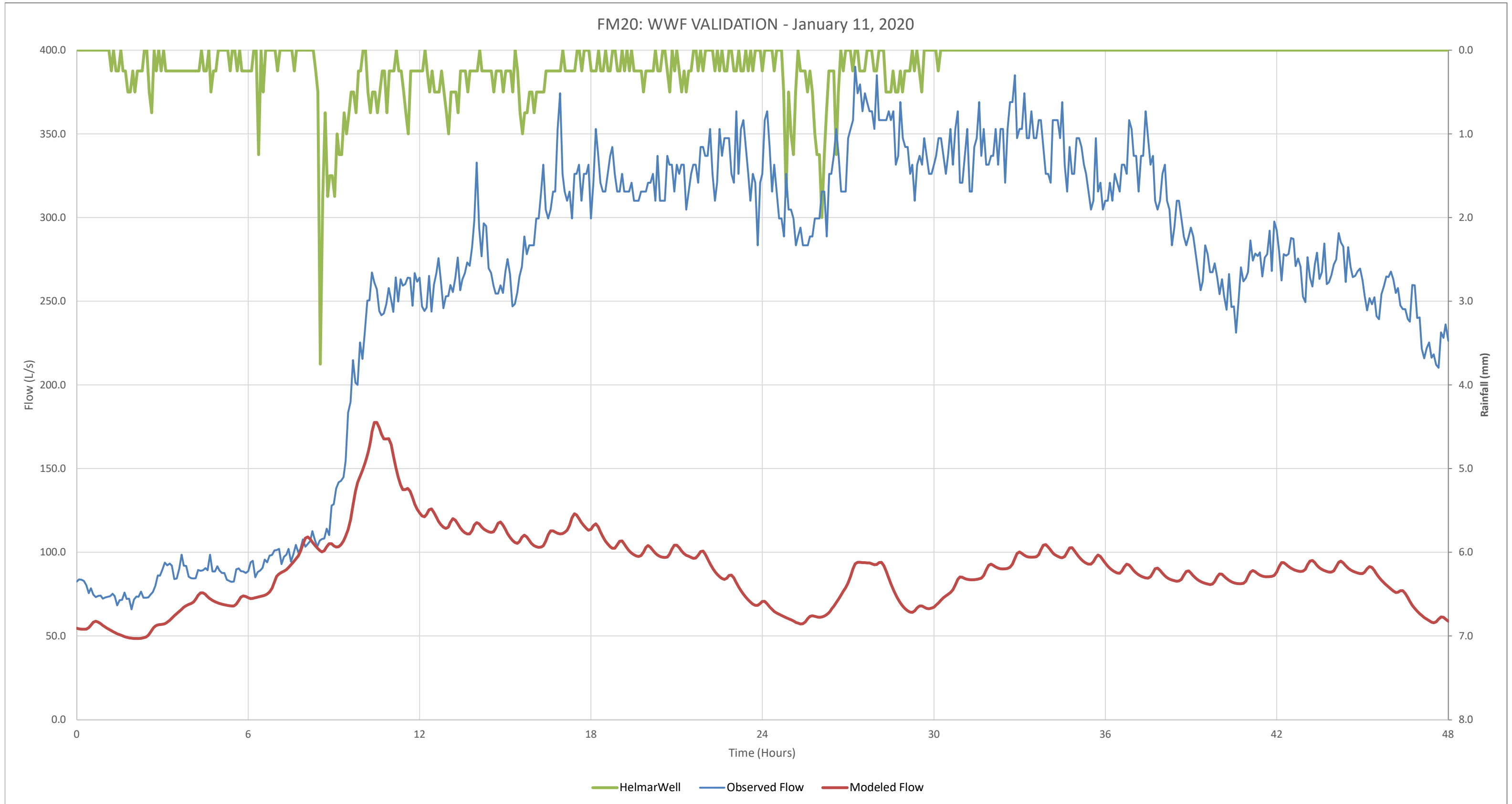
FM20: WWF CALIBRATION - June 10, 2020



FM20: WWF VALIDATION - May 29, 2020

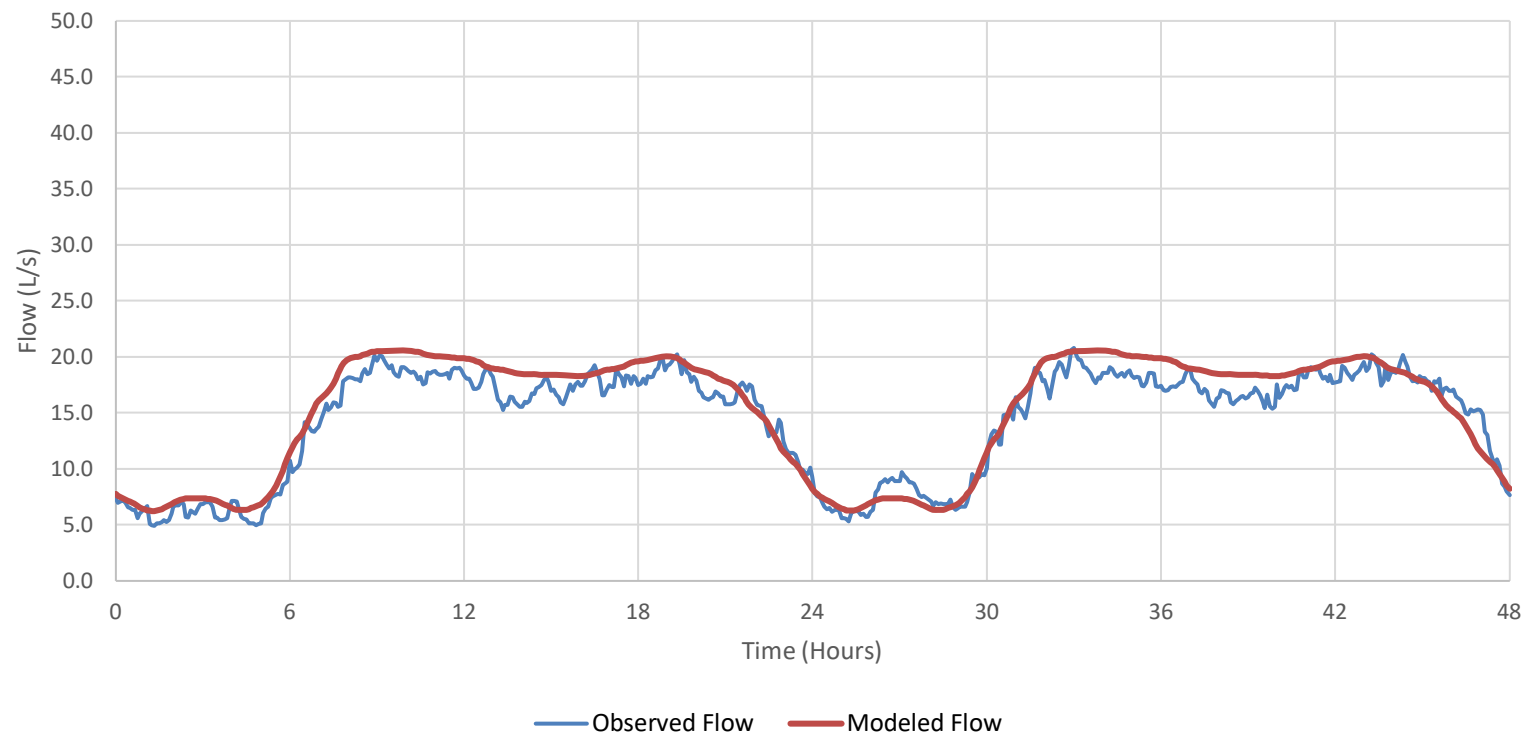


# FM20 CALIBRATION AND VALIDATION

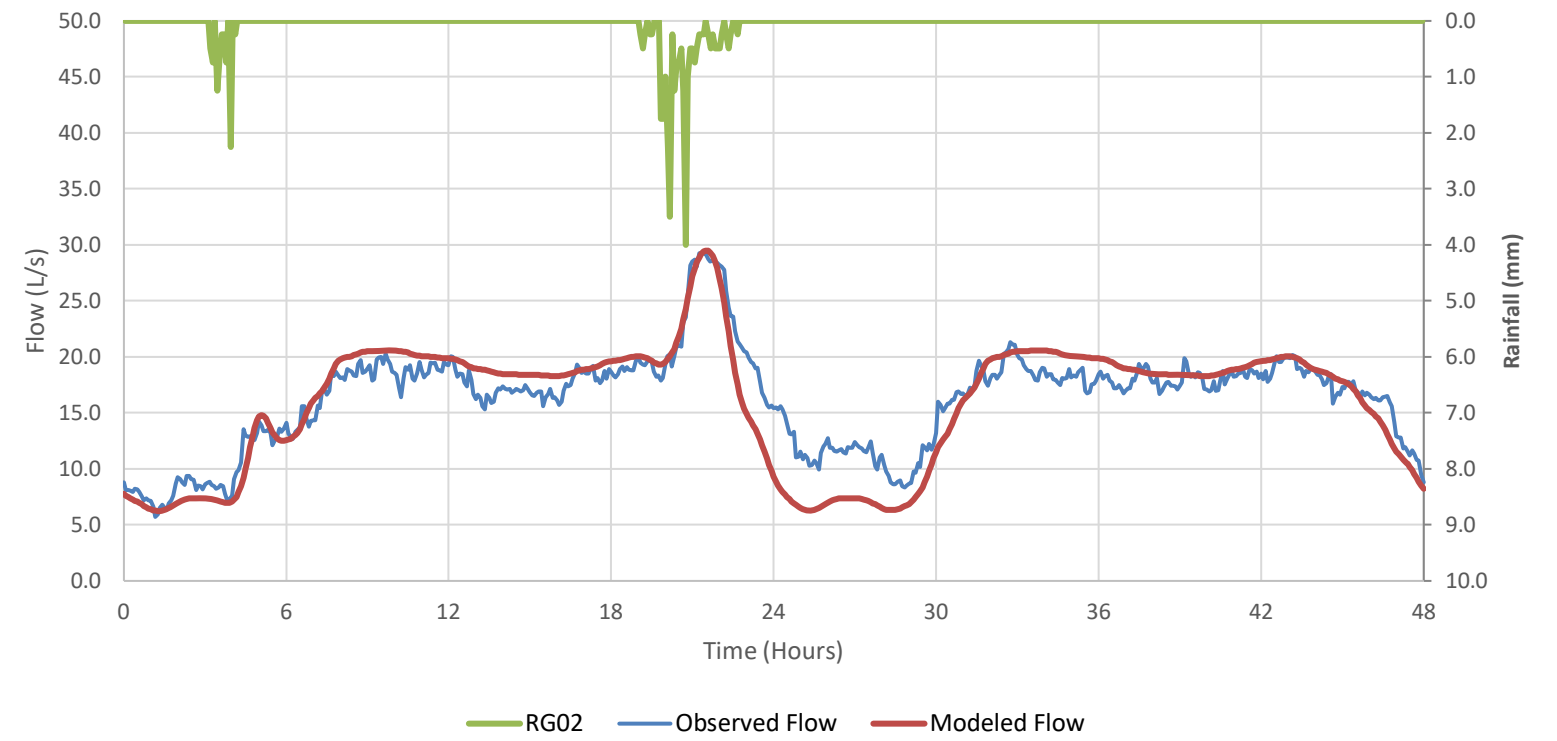


# FM21 CALIBRATION AND VALIDATION

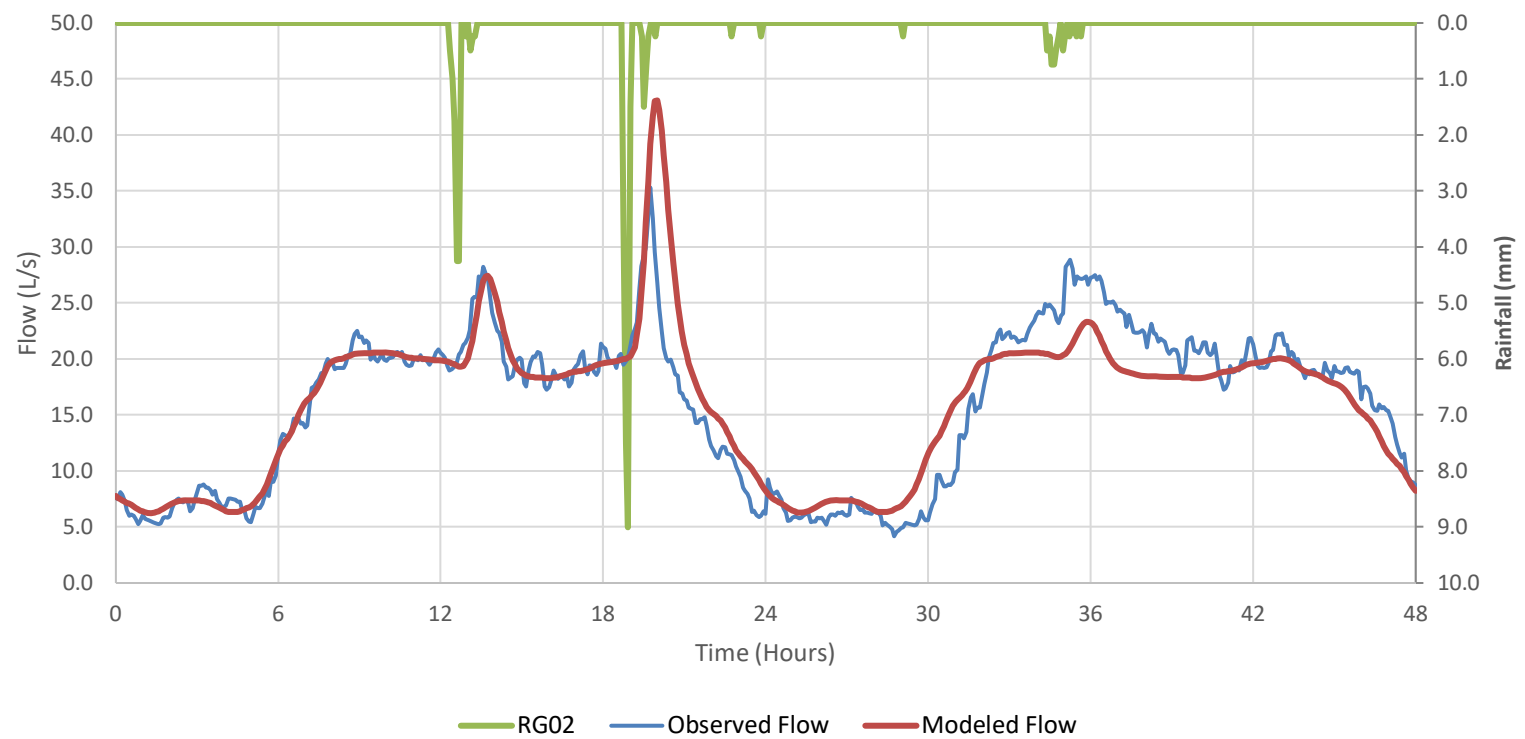
FM21: DWF CALIBRATION - June 17, 2020



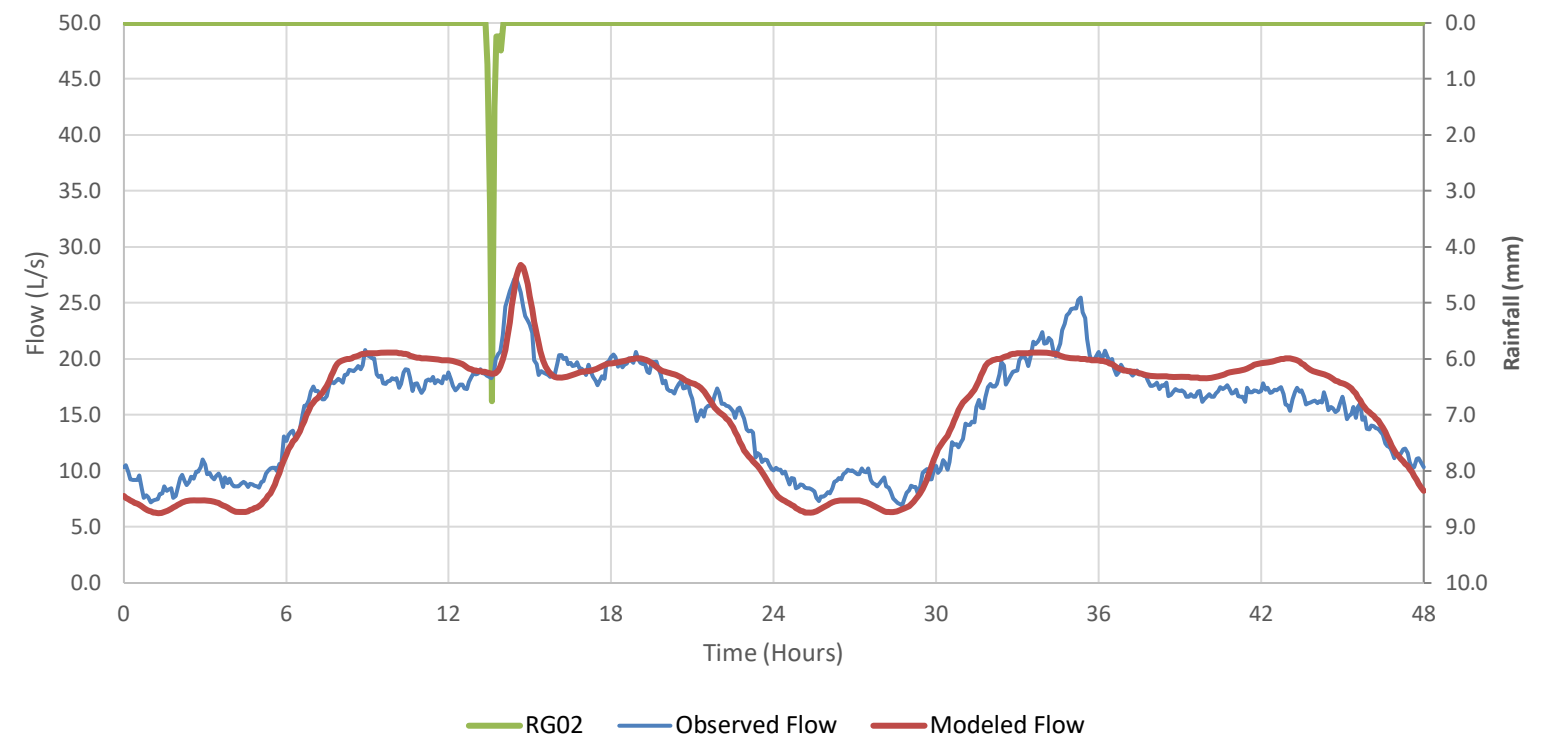
FM21: WWF CALIBRATION - June 10, 2020



FM21: WWF CALIBRATION - July 10, 2020



FM21: WWF VALIDATION - May 29, 2020





# FM21 CALIBRATION AND VALIDATION

FM21: WWF VALIDATION - January 11, 2020

