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CLASS ENVIRONMENTAL ASSESSMENT FOR WATER AND WASTEWATER SERVICING IN THE COMMUNITY OF NOBLETON

EXISTING WASTEWATER SYSTEM HYDRAULIC ANALYSIS

Study 2

B&V PROJECT NO. 196238

PREPARED FOR

Regional Municipality of York

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Distribution List

BV FILE NO.	RevNo	Issued to	Date	Reason for Issue
	0	York Region – Afshin Naseri	Oct 2017	Draft for YR Review
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Black & Veatch Signatures

Prepared By

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Ed Gower
Infrastructure Planning Engineer

Reviewed By

____

Paul Martin
Infrastructure Planning Engineer

1 Introduction

1.1 PURPOSE OF STUDY

The purpose of the Existing Wastewater System Hydraulic Analysis is to:

- Confirm the existing capacity of the wastewater system (pipe capacity, pump capacity and flows arriving at the WRRF);
- Identify any hydraulic limitations (bottlenecks, etc.); and
- Assess the maximum population that the existing system can support before any major upgrades are required.

This report will be a supporting document for the Wastewater System Capacity & Optimization Study (Study 1B).

The next stage of the Hydraulic Analysis for this project will focus on the Future System Hydraulic Analysis, and will be documented in the Wastewater Needs Assessment and Justification Study.

1.2 BACKGROUND

Nobleton is a community in King Township, located in York Region. Currently, Nobleton is serviced by standalone water and wastewater systems to meet the needs of the current population. The York Region Water and Wastewater Master Plan (2016) indicated that both the water and wastewater systems would not have sufficient capacity to meet requirements to support growth to the 2041 Master Plan horizon. Therefore, the Master Plan recommended undertaking the current project, a Schedule C Class Environmental Assessment (EA), to identify preferred servicing solutions to accommodate growth.

1.3 EXISTING WASTEWATER SYSTEM INFRASTRUCTURE

The Regional Municipality of York (also referred to as the Region and York Region) is responsible for the wastewater conveyance and to maintain the WRRF of its local area municipalities, including the Community of Nobleton in the Township of King. The Nobleton wastewater system consists mainly of a gravity system. There are two pumping stations within Nobleton; Bluff Trail PS in the north east of the catchment and Janet Avenue PS towards the south of the catchment. The Janet Avenue PS pumps all the flows from the catchment to the Nobleton WRRF.

2 Model Review and Update

2.1 EXISTING HYDRAULIC MODEL

2.1.1 General Model Information

The existing hydraulic model of the Nobleton wastewater system was provided by the Township of King. The model was provided in the InfoSewer software format.

The model provided was a high level model which was used for planning purposes but was only set up for steady state model runs. This meant that the existing model was not fit for purpose of undertaking the Nobleton Environmental Assessment. In consultation with the Region a decision was taken to convert the model from InfoSewer to InfoWorks ICM to better represent the existing condition but to also undertake assessments on the effect of growth within the catchment. This required a large upgrade to the model, including:

- Modelling of both the Bluff Trail and Janet Avenue Pumping Station including start / stop levels, pump rates and available storage;
- Updating the distribution of the population to match the existing situation;
- Reviewing the per capita consumption and wastewater profiles; and
- Calibrating the model against observed data from monitors installed in the catchment and SCADA data available on the operation of the Janet Avenue PS.

2.1.2 Pumping Station Data

Within the model as it was provided the two pumping stations within the catchment were not represented. In order to understand the capacity of the existing system and the effect of the growth on the catchment the two pumping stations needed to be represented within the model. The model has been updated with the following information:

- As-built information on Janet Avenue PS including dimensions of the wet well and the pump curve information.
- Details of the size of the wet well and the capacity of the pumping station was obtained from the Ministry of the Environment for Ontario for Bluff Trail PS.

Within the model the size of the wet wells at the pumping stations have been represented as a level area relationship to accurately represent how the pumping stations will fill and empty.

2.1.3 Model Population

Within the existing model once it was converted to InfoWorks the population included in the model was over 8,000. This is significantly higher than the known population of Nobleton in 2016 of 5,547. To try and understand the flows that enter the system a review of the SCADA data for Janet Avenue PS and Nobleton WRRF has been undertaken. This has also been compared to the hydraulic analysis which has been carried out for the water distribution system. This approach will assist in determining the flows which are generated in the Township of Nobleton.

2.1.4 Model Network

Figure 1 displays the existing network in the provided InfoSewer model:

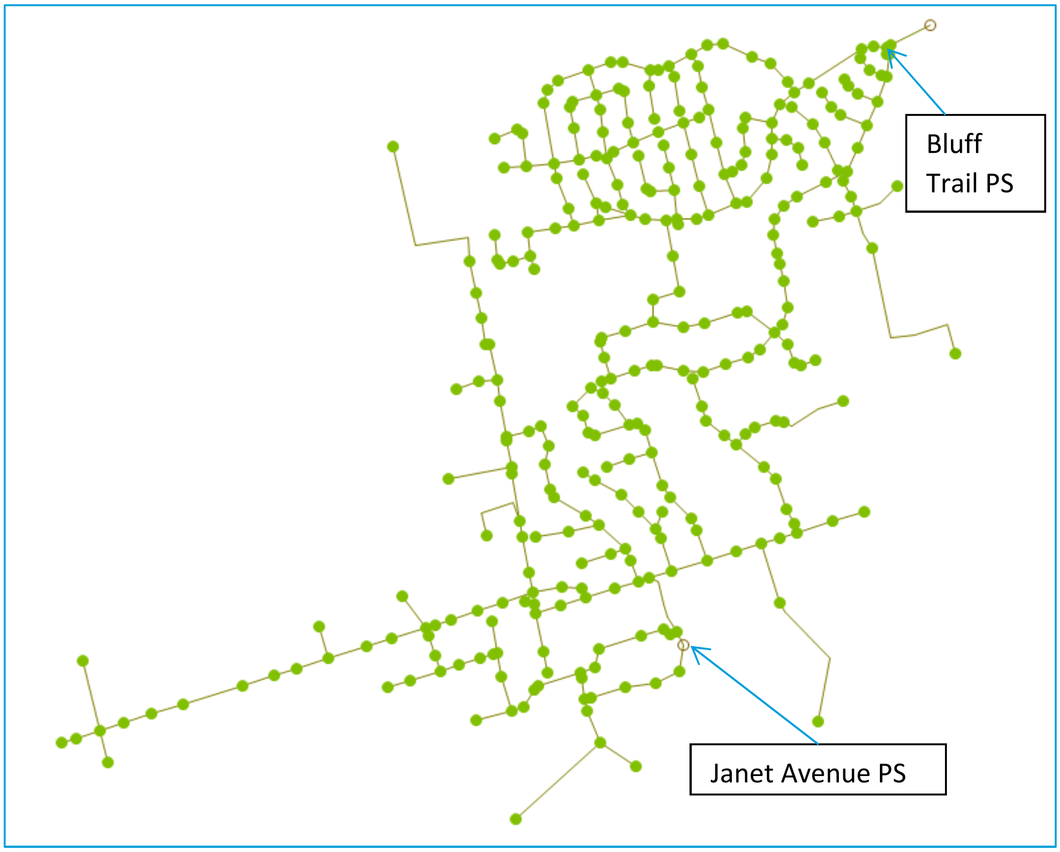


Figure 1 – Pipe network as provided

Generally, the provided model had an accurate representation of the existing sewer network. The provided model only had the existing network and did not include for any future upgrades. Within the converted model it was necessary to update some small parts of the network based on information which had been provided as part of the flow monitoring review which was undertaken in 2015. The updates to the network were undertaken using the information which was provided in the latest GIS data by the Region. This was compared to the information provided by the Region on the connectivity of the sanitary system. It was found that the revised network matched with the information from the Region. Where there was any missing data it was necessary to interpolate the levels and pipe sizes based on the known information which was included in the model already.

2.2 MODEL UPDATES

The following sections summarize the updates that were made to ensure that the hydraulic model is up-to-date and suitable for the analysis.

2.2.1 General Updates

Within the existing model although the locations where population related flows enter the system were represented these did not have any geographic boundary representation and therefore it was not possible to determine which areas of the catchment had been assumed to drain to which manhole. Using details on the extent of the network, new subcatchments, each allocated to a manhole, have been drawn around the housing plots of Nobleton to better define the distribution of inflows to the system.

In the initial model update it was assumed that all of Nobleton was connected to the sewer system although there were some areas where there were no sewers shown. However, when it came to the calibration of the model it was found that there was a significant area to the west of Highway 27 which did not drain to the system and also there were some areas to the south of the catchment which were not draining to the system. The location of these is shown in Figure 2.

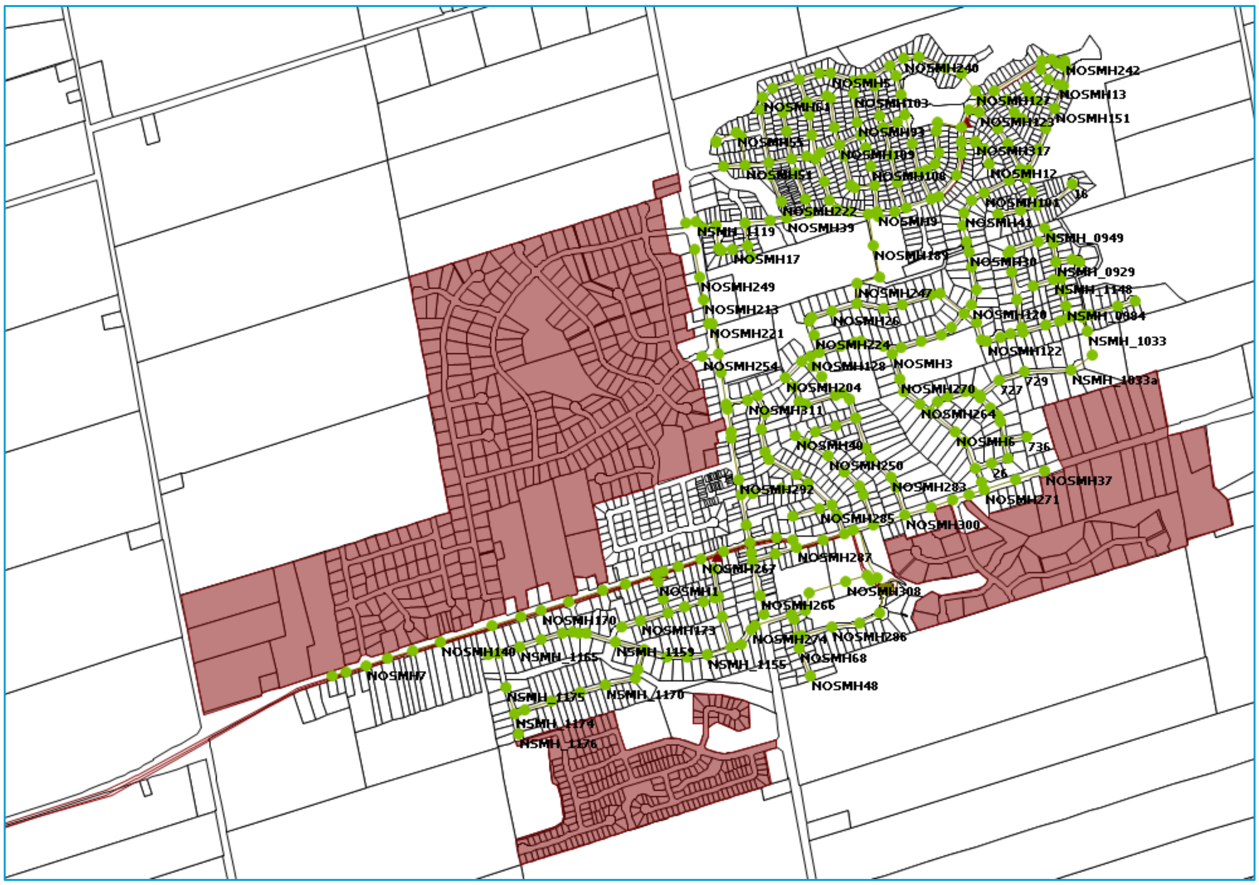


Figure 2 - Areas connected to Septic Tanks

As well as producing these subcatchments it was also necessary to generate a new wastewater profile which includes the details of the per capita consumption and the diurnal profile. During the calibration of the model base infiltration was added to the model as required to provide a model which represents the existing dry weather case and also the storm conditions.

2.2.2 Pumping Station Updates

As mentioned previously the existing model as provided did not include any representation of the two pumping stations which are within the Nobleton catchment. This has meant that these have had to be added to the model to provide a better understanding of the capacity of the existing network.

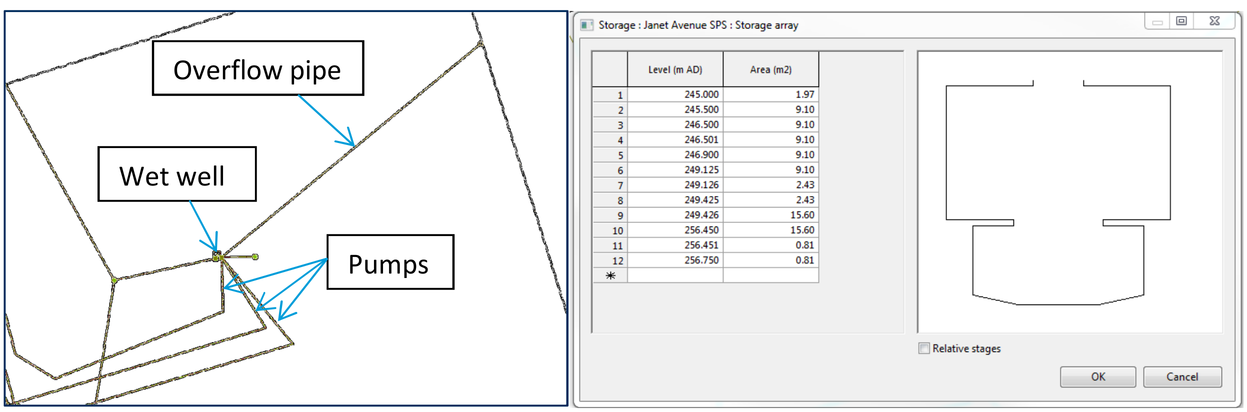
2.2.2.1 Bluff Trail PS

Information was obtained from the Ministry of the Environment for Ontario which included the size of the wet well and the capacity of the pumps. With the use of this information and the recorded flow information from the monitor downstream a representation of the pumping station has been added to the model. Where necessary this information has been updated to improve the calibration of the model.

2.2.2.2 Janet Avenue PS

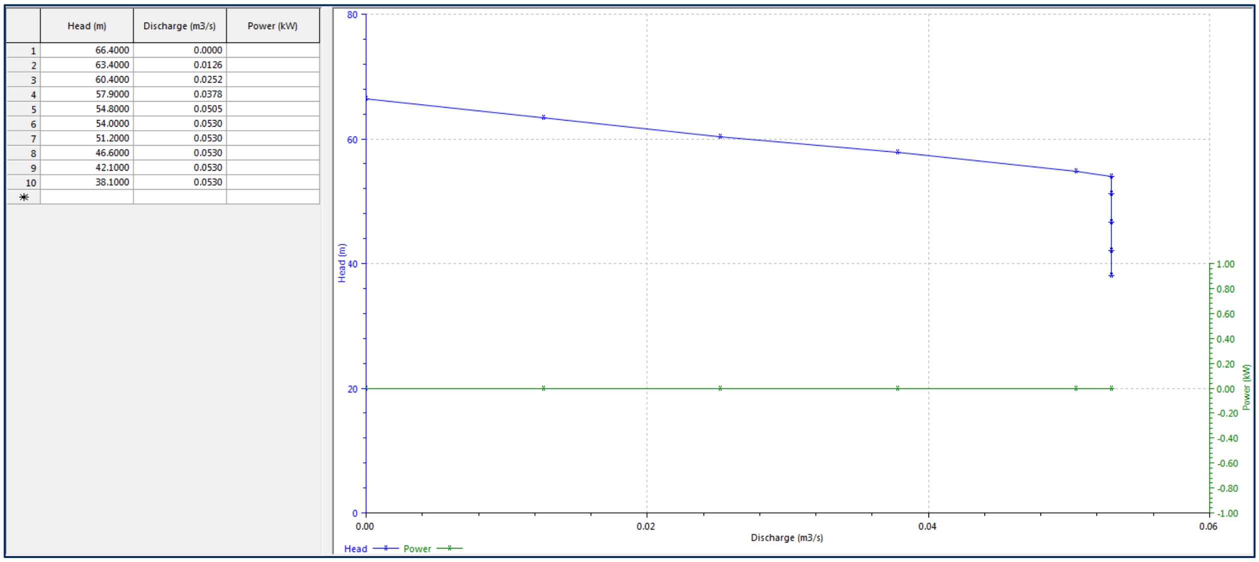
For Janet Avenue PS which is the main pumping station that pumps all of the catchment flows to the WRRF, as-built drawings were provided of the wet well and these have been used to derive a level area relationship which was used in the model. From the as-built information initial values for the pump start / stop levels and pump capacities were obtained.

However, in a similar manner to Bluff Trail PS these have been updated as required in order to improve the representation of the operation based on the SCADA data which was available. Figure 3 shows the final representation of the pumping station and the level area relationship of the wet well as it has been included in the model. The telemetry data has also been used to derive the existing start and stop levels of the pumps and the pump capacities. To represent the existing operation of the pumping station the pumps have been modelled as variable frequency drive pumps which can operate a maximum speed of 1650 rpm and deliver a maximum flow of 53 L/s each. Within the pumping station there are three pumps but currently only two are used at the same time to give a maximum capacity of 106 L/s. Figure 4 shows the head discharge which has been applied to the model. From a review of the telemetry data it was found that during dry weather flows the pump turns on and off based on the level within the wet well. This information has been added to the model and the control of the pumps has been undertaken with the use of Real Time Control (RTC).



	Level (m AD)	Area (m2)
1	245.000	1.97
2	245.500	9.10
3	246.500	9.10
4	246.501	9.10
5	246.900	9.10
6	249.125	9.10
7	249.126	2.43
8	249.425	2.43
9	249.426	15.60
10	256.450	15.60
11	256.451	0.81
12	256.750	0.81
*		

Figure 3 – Arrangement of PS in the model and level area relationship



	Head (m)	Discharge (m3/s)	Power (kW)
1	66.4000	0.0000	
2	63.4000	0.0126	
3	60.4000	0.0252	
4	57.9000	0.0378	
5	54.8000	0.0505	
6	54.0000	0.0530	
7	51.2000	0.0530	
8	46.6000	0.0530	
9	42.1000	0.0530	
10	38.1000	0.0530	
*			

Figure 4 – Head discharge curve used in the model

When reviewing the data there was an issue with the frequency of start and stops of the pumps in the telemetry data and the frequency that the model predicts. A thorough review of the as built drawings has been undertaken and it has not been possible to rectify the difference between the model and observed data. Possible reasons for the difference could be:

- Unknown volume within the wet well: this would need to be big enough to reduce the speed that the wet well fills to match the observed data.
- A restriction between the last flow monitor in the catchment and the pumping station: this is possible but to restrict the flow enough would have an impact on the level at the last flow monitor within the catchment.

Additional information is required to understand the exact operation of the pumping station. Currently a reasonable match has been achieved to the observed data but additional information is required.

2.2.3 Population information

From the water hydraulic analysis of Nobleton the residential population of the catchment had been calculated as 5,547, this has been used as the starting point for the wastewater modelling. For the sanitary system it was found that there are areas of Nobleton which are not yet connected which needed to be removed from the model as mentioned in section 2.2.1 of this report. This improved the match to the observed data. No changes were made to the per capita consumption rates.

The final model has a population of 3,643 with an occupancy ratio across the catchment of 3.1 per property. The exact distribution of the population across the catchment is not known as not all the properties which were previously connected to septic tanks have been connected to the sanitary system. The distribution of the population is based on trying to achieve a match to the flow monitors in the upstream catchment. The distribution was based on information provided which showed the different contract areas, however this did not include all of the catchment. Population was added upstream of Bluff Trail PS in order to match the observed flow data.

A detailed analysis of the historical water demands (average day and maximum day), as well as an analysis of the diurnal patterns is provided in Study 1A: Water System Capacity and Optimization Study.

As mentioned previously there are many properties which are not currently connected to the sanitary system due to them previously being connected to septic tanks. As mentioned the exact locations of those that are connected and those which are not is not known.

When assigning the population data to the model it has been assumed that groups of houses drain to particular manholes as per the drawn subcatchments. In producing the subcatchments some assumptions have been taken. However, when the model is compared to the observed data from the flow monitors in the catchment a good fit is achieved which would suggest that the allocation of population in the model is relatively accurate.

The per capita consumption values for the catchment have also been derived for the catchment. This has been done by matching the predicted volumes against the SCADA data for the Janet Avenue SPS. Within the model a value of 229.2 L/cap/day has been used. This value is based on 200 L/cap/day for the residential flows and an allowance of 63.8 L/cap/day in areas with non-residential flow. This value does not account for any infiltration within the system which has been applied separately as baseflow and is discussed in Section 2.2.4.

2.2.4 Base Infiltration

When looking at the observed data it was possible to see from the night time flows that there was an element of dry weather infiltration which was entering the system. This level of base infiltration has been applied to the model to improve the match between the observed data and the predicted data. Figure 5 shows the areas where it has been necessary to add this infiltration. For the whole catchment the average infiltration has been applied as 107 L/cap/day which equates to 4.5 L/s across the catchment.

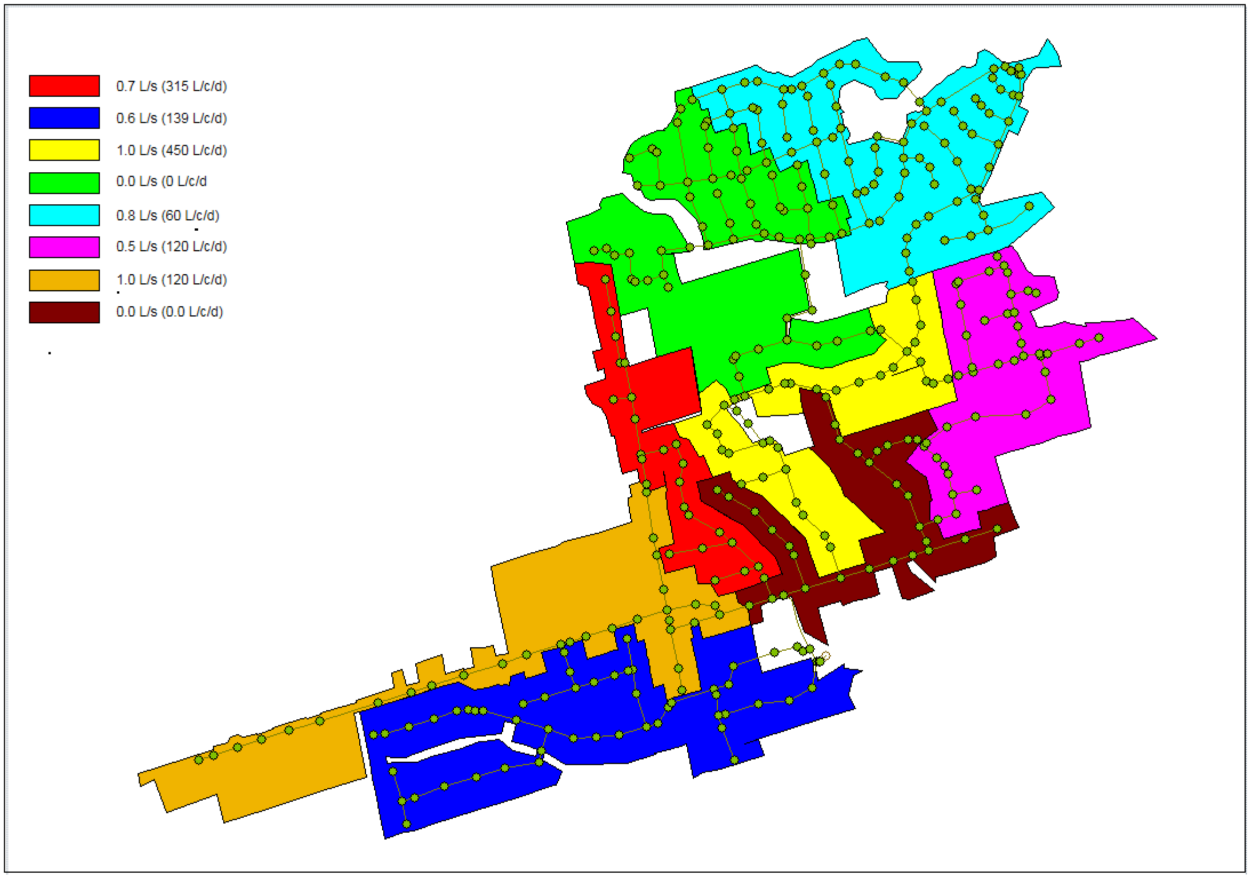


Figure 5 – Locations where base infiltration has been applied to the model

Base infiltration has been applied as per the numbers in Figure 5, Table 1 shows the amount of base infiltration for each flow monitor as a percentage. Also included is the proposed amount of growth upstream of each area. The planned growth population is in addition to the existing service population upstream of each monitor.

Table 1: Distribution of base infiltration and planned growth across the catchment

METER	AREA (HA)	RECORDED FLOW (L/S)	SERVICE POPULATION	RESIDENTIAL WW FLOW (L/S)	BASE INFILTRATION (L/S)	BASE INFILTRATION (%	PLANNED GROWTH POPULATION
KI004a_10	198.6	12.6	3271	8.3	4.3	34.1	5996
KI004a_20	45.63	1.5	372	0.9	0.6	40.0	1152
KI004b	114.28	5.7	1838	4.7	1.0	17.5	3274
KI005	45.62	5.1	1655	4.2	0.9	17.6	3106
KI006	30.77	1.4	359	0.9	0.5	35.7	112
KI008	20.62	1.1	172	0.4	0.7	63.6	550
KI009	43.73	2.9	714	1.8	1.1	37.9	1731
KI010	40.61	3.6	1144	2.9	0.7	19.4	957

An assessment to determine if there is any seasonal effect on the amount of base infiltration has been carried out. This has been done by looking at the daily pumped volumes from Janet Avenue Pumping Station to the WRRF. Figure 6 shows average, minimum and maximum daily pumped volumes per month from 2014 to 2018.

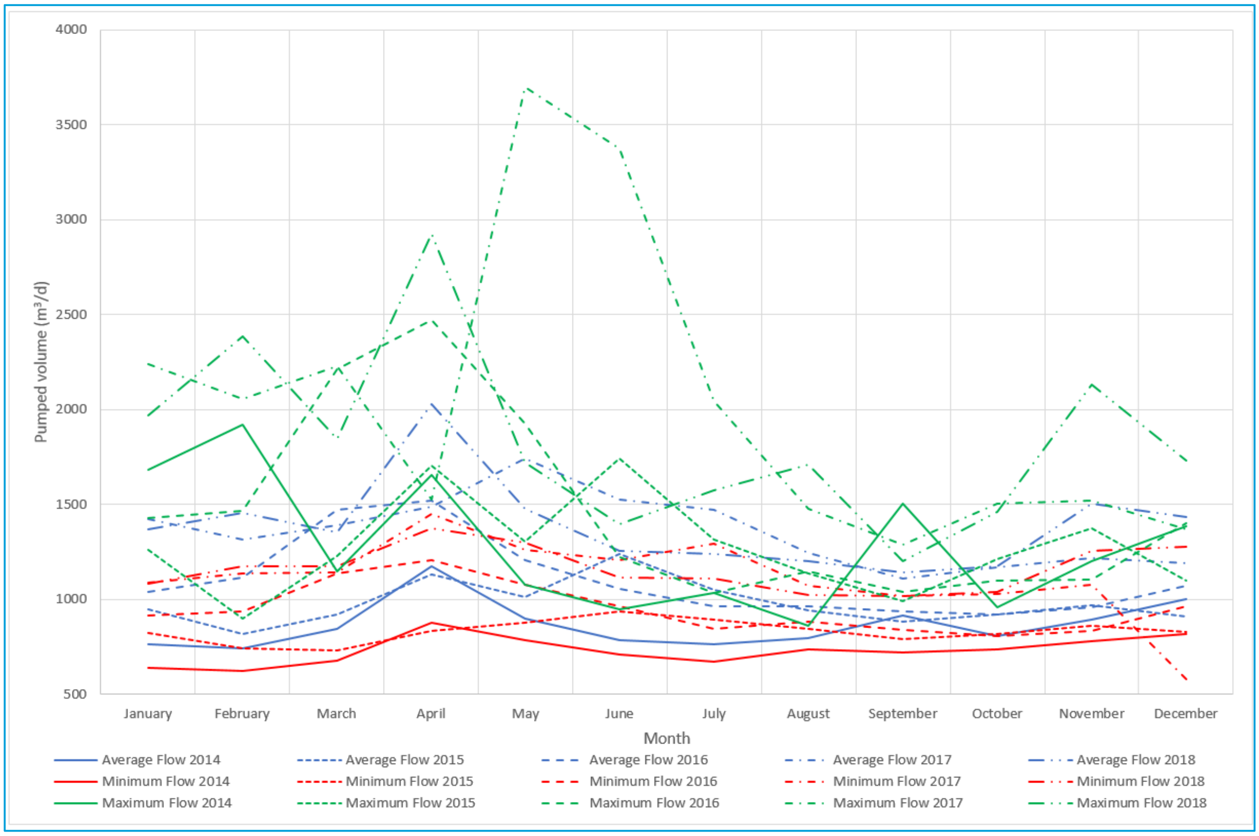


Figure 6 – Comparison of the average, minimum and maximum daily pumped volumes from Janet Avenue

From the minimum daily flows over the last 5 years, as shown by the red lines in Figure 6, it is possible to see that actually the volume pumped to the works does not vary that much across a year. There is a slight increase around April. It can also be seen that generally every year the minimum daily pumped volume has increased, this could be due to increased base infiltration and the increase in population which has taken place every year.

The average daily flows over the last 5 years, as shown by the blue line in Figure 6, show a similar pattern to the minimum daily flows. The maximum daily flows, as shown by the green lines, do not show a clear pattern as they are affected by the amount of rainfall which falls during any particular year.

Overall from this information it is not conclusive that there is a seasonal variation in the amount of base infiltration.

2.2.5 Network Updates

The model as it was provided includes most of the wastewater network when it was compared to the GIS data. There were a couple of areas where it was required to add some additional information to the model based on the GIS data which was available. Figure 7 shows the locations where changes have been made to the network.

2.2.6 Headloss Coefficients and Roughness Values

The model as it was provided did not include any entry and exit headloss values across the catchment. To represent the headlosses across the catchment it is important to include headloss values for the pipes where they enter and leave manholes. Within InfoWorks ICM this has been done by using the inference tool to automatically assign headlosses based on the angle of direction change. Table 2 shows the values which are applied by default. These values have been reviewed to ensure that they are acceptable. In some cases high headloss values have been adjusted to ensure correct values have been used.

Table 2: Headloss values assigned across the model

ANGLE	BEND VALUE
30	3.3
60	6.0
90	6.6
>90	8.0

The roughness values within the model were already assigned as Manning's 'n' with a value of 0.013 applied across the catchment. During the calibration stage there was no information to suggest this value needed to be adjusted.

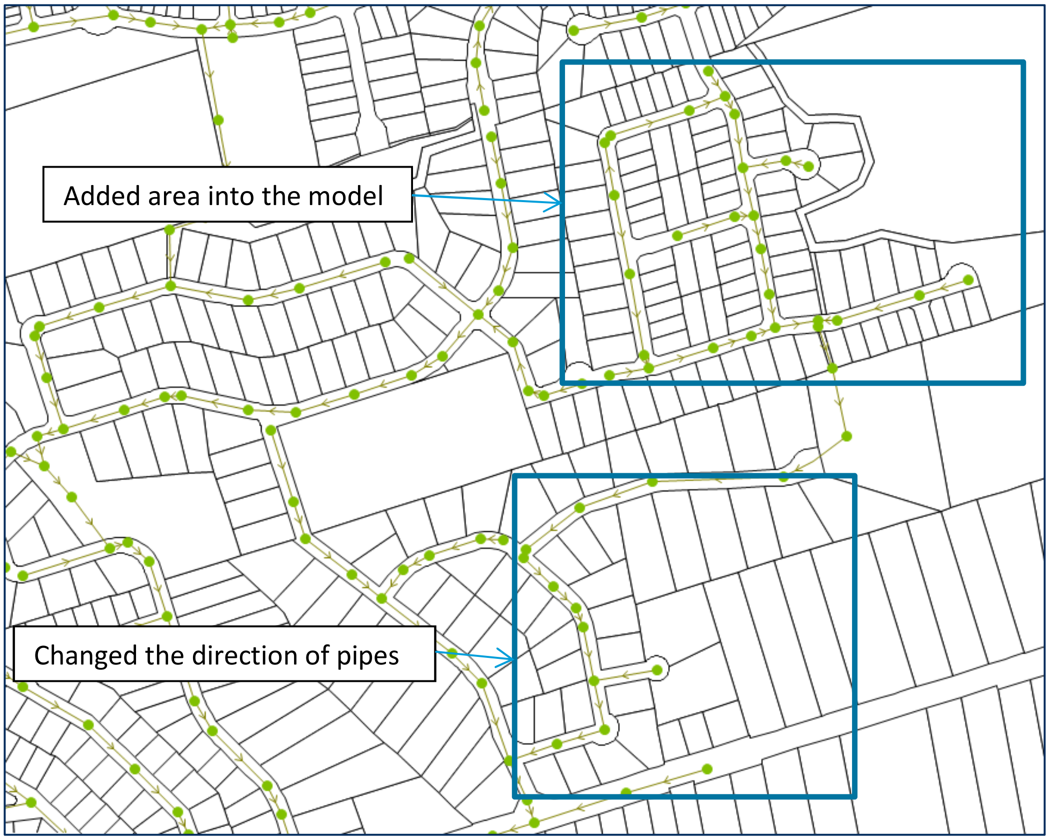


Figure 7 – Changes made to the network

2.2.7 Model Calibration

2.2.7.1 Overview

To calibrate the model flow monitor data has been used. This data has come from long term monitors which have been installed in the system. A decision was taken to use 2016 data to calibrate the model to dry weather flows and then for storm conditions the two largest storms recorded in 2017 have been used. These were the 17th June and the 23rd June 2017. If required for the storm calibration adjustments were made to the dry weather flows, either population or base infiltration. Figure 8 shows the locations of the eight flow monitors across the catchment. Figure 9 shows a schematic of the locations of the flow monitors.

Figure 8 – Location of flow monitors across the catchment

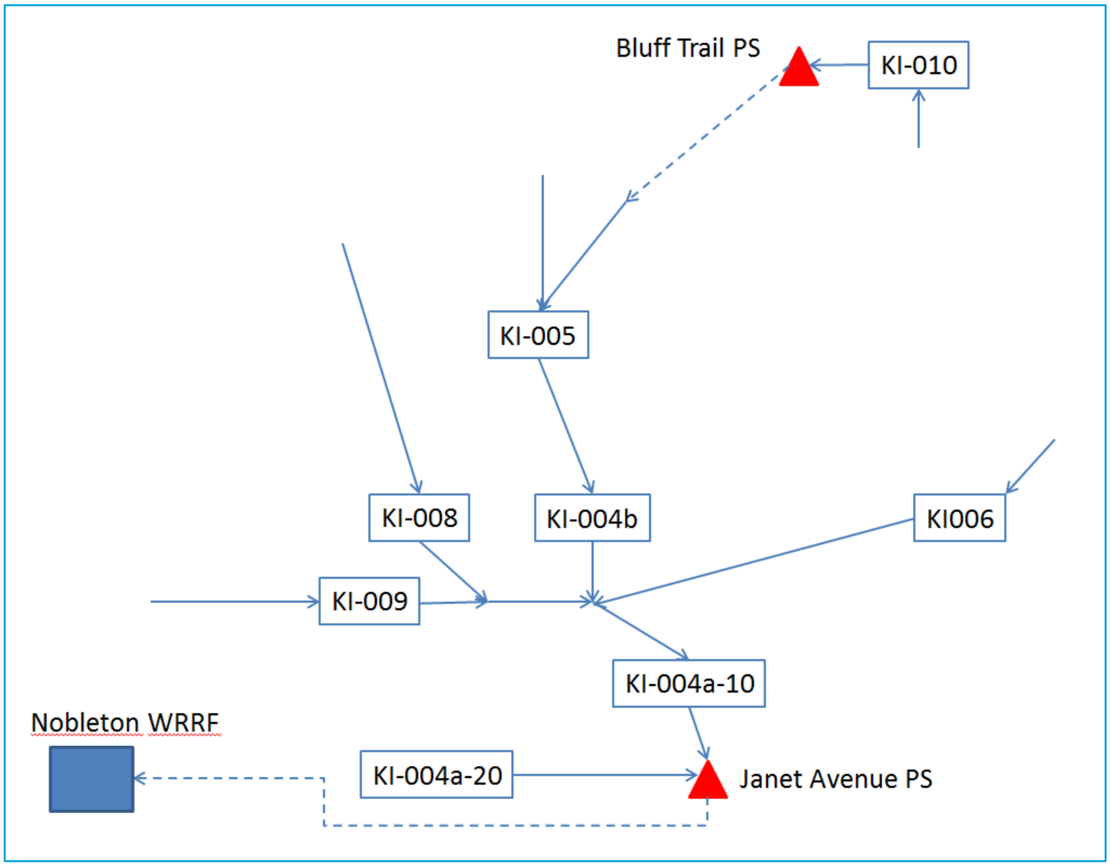


Figure 9 – Schematic showing the locations of the flow monitors

As well as the flow monitor data there were some SCADA data available. This included the depth data within Janet Avenue PS, the flow data downstream of Janet Avenue PS forcemain and flows received at the Nobleton WRRF.

Rainfall data from a rain gauge located at Janet Avenue PS was used to assist with the calibration.

When doing the comparison between the observed and the predicted data it was decided to use the UK Urban Drainage Group (UDG) guidelines. These are industry standards within the UK and globally to define acceptable differences between the observed and the predicted data.

The model calculates the flow from the information within the subcatchment:

- Population;
- Base infiltration;
- Area of impermeable surfaces (roads and roofs);
- Area of permeable surfaces.

From the flows, from the size, roughness and gradient of the pipe, and from any effects of downstream conditions, the model is then able to calculate the depth and velocity within the pipe. This can mean that in the model it is possible to achieve a good match on the flows but not on the depths or the velocities.

2.2.7.2 Dry Weather Flows

Dry weather flow calibration was initially done against two dry days, one weekday and one weekend. Additional dry days were also reviewed to understand if the model was representing the flows within the catchment. A dry day is one where there has been no precipitation and no precipitation in the three preceding days. The days used for the calibration were:

- Dry day 1: 21/07/2016 (Wednesday)
- Dry day 2: 03/09/2016 (Saturday)

The model was run for these two dry days initially with just the population connected to determine where changes were required to match the observed data. Following this initial run each flow monitor was looked at in turn working from the upstream to the downstream and baseflows were adjusted as required to improve the match between the observed and the predicted data.

From the observed data it was found that the data from KI005 appears to be over-recording when it is compared to KI004b and KI004a-10. As an additional comparison the volumes for these three monitors were compared to the SCADA data for Janet Avenue PS. It was found that the data for KI005 did not match the SCADA data but the other two monitors did. Therefore, a decision was taken to concentrate on KI004b and KI004a-10 and to use the data from KI005 only to confirm the depths at this location.

Other changes that were made were to Bluff Trail PS. As mentioned previously information was available from the Ministry of the Environment for Ontario. There was also information available for KI010 which is upstream of the pumping station and KI005 was downstream, although this data is not considered reliable. From this data it has been possible to model the pumping station wet well and a pump rate that matches the observed data. To further improve the accuracy of the model a survey of the pumping station would be required.

From a review of the observed data it was possible to identify an additional 20 weekdays and 7 weekend dry days. A review of the data for these days has been undertaken and the volumes passing each of the flow monitors has been calculated. This has been summarized in Table 3 in terms of the average, maximum, minimum volumes against the predicted volume from the model for a weekday and then Table 4 shows the comparison for a weekend.

Table 3: Summary of dry weather volumes for all the flow monitors for a weekday

FLOW MONITOR	AVERAGE VOLUME (MLD)	MAXIMUM VOLUME (MLD)	MINIMUM VOLUME (MLD)	MODELLED VOLUME (MLD)	DIFFERENCE TO AVERAGE (MLD)
KI-004a-10	0.988	1.199	0.847	1.090	+0.102
KI-004a-20	0.165	0.291	0.099	0.133	-0.032
KI-004b	0.563	0.682	0.497	0.489	-0.074
KI-005	0.790	0.947	0.721	0.449	-0.341
KI-006	0.139	0.219	0.065	0.125	-0.014
KI-008	0.118	0.158	0.091	0.102	-0.016
KI-009	0.318	0.399	0.255	0.252	-0.066
KI-010	0.335	0.402	0.309	0.315	-0.020
Janet Avenue PS	0.979	1.186	0.867	1.229	+0.250

Table 4: Summary of dry weather volumes for all the flow monitors for a weekend

FLOW MONITOR	AVERAGE VOLUME (MLD)	MAXIMUM VOLUME (MLD)	MINIMUM VOLUME (MLD)	MODELLED VOLUME (MLD)	DIFFERENCE TO AVERAGE (MLD)
KI-004a-10	0.990	1.084	0.892	1.098	+0.108
KI-004a-20	0.170	0.191	0.150	0.134	-0.036
KI-004b	0.604	0.649	0.545	0.500	-0.104
KI-005	0.835	0.897	0.747	0.460	-0.375
KI-006	0.156	0.210	0.081	0.125	-0.031
KI-008	0.118	0.142	0.087	0.102	-0.016
KI-009	0.311	0.365	0.254	0.253	-0.058
KI-010	0.343	0.378	0.322	0.316	-0.027
Janet Avenue PS	1.001	1.122	0.878	1.229	+0.228

From the data in these two tables it is possible to see that the dry weather volumes vary significantly in the observed data which makes it difficult to generate a model which matches this. However, the modelled dry weather flows are somewhere close to the average values which have been recorded and therefore can be considered to be calibrated.

Overall the match to the flow monitors in the upstream catchment is good. However, when the model predictions of the wet well depths and flows downstream of the forcemain for Janet Avenue were compared against the SCADA data it was found that the pump in the model was cycling approximately four times more frequently than the observed data suggested it should. Therefore, several changes have been made to the modelling of the operation of the pumping station to try and improve this representation. With the pumps modelled as a variable frequency drive pump the volume being pumped matched reasonably well. Work was undertaken to try and resolve this but it was not possible to reduce the number of start and stops during a day to match the observed data.

Table 5 shows the comparison of the observed to the predicted data at all the flow monitors and shows if the model is calibrated. This comparison is for the two dry days which have been looked at.

Table 5: Comparison of the observed versus predicted data for dry weather days (green text is within tolerances and red is outside of tolerances)

FLOW MONITOR	EVENT	OBS PEAK DEPTH (M)	MODEL PEAK DEPTH (M)	OBS PEAK FLOW (L/S)	MODEL PEAK FLOW (L/S)	OBS VOLUME (MLD)	MODEL VOLUME (M3)	PEAK DEPTH DIFF (M)	PEAK FLOW DIFF (%)	VOLUME DIFF (%)
K-I004a_10	DWF 1	0.167	0.197	27	34	0.963	1.090	0.030	26%	13%
	DWF 2	0.175	0.203	28	30	0.899	1.098	0.028	7%	22%
KI-004a_20	DWF 1	0.065	0.042	4	3	0.147	0.134	-0.023	-25%	-9%
	DWF 2	0.052	0.042	3	3	0.154	0.134	-0.010	0%	-13%
KI-004b	DWF 1	0.139	0.118	22	27	0.540	0.489	-0.021	23%	-9%
	DWF 2	0.139	0.106	22	22	0.583	0.500	-0.033	0%	-14%
KI-005	DWF 1	0.123	0.110	46	36	0.740	0.450	-0.013	-22%	-39%
	DWF 2	0.149	0.104	53	31	0.831	0.460	-0.045	-42%	-45%
KI-006	DWF 1	0.023	0.048	2	2	0.103	0.125	0.025	0%	21%
	DWF 2	0.034	0.048	4	4	0.210	0.125	0.014	0%	-41%
KI-008	DWF 1	0.026	0.057	2	2	0.112	0.102	0.031	0%	-9%
	DWF 2	0.028	0.057	2	2	0.094	0.102	0.029	0%	-8%
KI-009	DWF 1	0.038	0.051	5	5	0.339	0.252	0.013	0%	-25%
	DWF 2	0.037	0.051	5	5	0.266	0.253	0.014	0%	-5%
KI-010	DWF 1	0.056	0.067	7	7	0.330	0.315	0.011	0%	-4%
	DWF 2	0.050	0.067	6	7	0.339	0.361	0.017	17%	6%

Graphs showing the matches between the observed and predicted data at all locations including the depths and flows around Janet Avenue are included in Appendix A.

During 2018 an additional monitor was installed upstream of KI004a_10 named KI004a_11. Data from this monitor was investigated and an additional dry weather day from the period following its installation was investigated. The results of this dry weather day are included in Table 6 with the graphs showing the comparison included in Appendix A.

Table 6 – Comparison of dry weather day from 2018

FLOW MONITOR	EVENT	OBS PEAK DEPTH (M)	MODEL PEAK DEPTH (M)	OBS PEAK FLOW (L/S)	MODEL PEAK FLOW (L/S)	OBS VOLUME (MLD)	MODEL VOLUME (M3)	PEAK DEPTH DIFF (M)	PEAK FLOW DIFF (%)	VOLUME DIFF (%)
K-I004a_10	DWF 1	0.000	0.119	36	35	1.067	1.086	+0.119	-3%	+2%
	DWF 2	0.291	0.119	29	35	1.044	1.090	-0.172	+21%	+4%
KI-004a_11	DWF 1	0.095	0.119	31	35	1.247	1.086	+0.024	+13%	-13%
	DWF 2	0.096	0.120	33	35	1.175	1.089	+0.024	+6%	-7%
KI-004a_20	DWF 1	0.077	0.042	10	3	0.330	0.134	-0.035	-70%	-59%
	DWF 2	0.082	0.042	18	3	0.346	0.134	-0.040	-83%	-61%
KI-005	DWF 1	0.163	0.108	60	34	0.937	0.449	-0.055	-43%	-52%
	DWF 2	0.146	0.108	55	34	0.936	0.448	-0.038	-38%	-52%
KI-006	DWF 1	0.039	0.048	4	2	0.182	0.125	+0.009	-50%	-31%
	DWF 2	0.042	0.048	5	2	0.253	0.125	+0.006	-60%	-51%
KI-008	DWF 1	0.026	0.057	2	2	0.096	0.102	+0.031	0%	+6%
	DWF 2	0.025	0.057	2	2	0.092	0.101	+0.032	0%	10%
KI-009	DWF 1	0.036	0.051	5	5	0.223	0.252	+0.025	0%	+13%
	DWF 2	0.044	0.051	7	5	0.291	0.251	+0.007	-29%	-14%
KI-010	DWF 1	0.051	0.067	8	7	0.346	0.315	+0.016	-13%	-9%
	DWF 2	0.058	0.067	10	7	0.389	0.314	+0.009	-30%	-19%

2.2.7.3 Storm Calibration

Once the model was calibrated to the dry weather flows within the area it was necessary to calibrate the model for storm conditions. From a review of all the available data there were two storms which were identified as generating the largest flows being pumped from Janet Avenue. A review of the rainfall data has identified that these storms meet the requirements of a calibration storm. The definition of a calibration storm has come from the UDG guidelines. These state that storm needs to be:

- Greater than 5mm depth
- Have a peak intensity greater than 6mm/hr for more than 4 minutes

The other part to check was that there was observed data at all the monitors to be able to calibrate the model acceptably. For both large events there was data available for all the monitors except there is a period missing from one of the events at monitor KI-004a_10. The two storms which have been used are:

- Storm A: 17/06/2017

- Storm B: 23/06/2017 -

Table 7 shows some information about these two events. -

Table 7: Information about the two storm events used

STORM EVENT	START DATE AND TIME	FINISH DATE AND TIME	DURATION (MINS)	RAINFALL DEPTH (MM)	PEAK INTENSITY (MM/HR)
A	17/06/2017 12:55	17/06/2017 14:35	100	40	96
B	22/06/2017 23:10	23/06/2017 07:25	495	51.4	64.8

To represent the runoff from the catchment the model was set up so that the areas of contributing roads, roofs and permeable area could be represented. For the roads and roofs the fixed runoff method has been used and for the permeable area a decision was made to use the Groundwater Infiltration Module (GWI) which is a feature of InfoWorks that represents the effects of rainfall related infiltration that often persists for some time after rainfall.

Within the model each monitor has been looked at individually working from the upstream to the downstream to achieve a match to the observed data. The predicted values have also been compared to the observed values using UDG guidelines.

In order achieve a match with the rainfall response seen in the observed data it has been necessary to include an allowance for roof area per property to be connected. Within the catchment there was no information on which properties may have roof connections to the sanitary system so the quantity of roof area has been proportionally split based on the number of properties upstream of the monitors. Figure 10 shows the amount of roof area per zone that drains to the flow monitor that has been applied.

This allowance meant that the peak flow response from the catchment could be represented. However, from the observed data there is also a slow response through the catchment which needed to be represented. This has been done by adding in GWI area upstream of each monitor. The model assumes that rain falling on this area infiltrates into the ground and then into the sewers. Figure 11 shows the amount of GWI which has been applied upstream of each monitor.

A comparison of these runoff areas to the design standard of 0.26 L/s/ha is shown in Section 3 of this report.

The results of the calibration show a reasonable match to the observed data for each of the flow monitors within the catchment. Table 8 shows the comparison of the observed data to the predicted data for the three storms. The storm events have been run over two days in order to allow for the model to stabilize before the event and to then drain down afterwards. The results shown in Table 8 have been calculated for one day only around the peak of event. For Event A this is 06:00 17/06/2017 to 06:00 18/06/2017 and for Event B 21:00 22/06/2017 to 21:00 23/06/2017. The model appears to under predict the volumes but there is a lack of confidence in a number of the monitors upstream of Janet Avenue. Additional volume could be added but this would increase the differences predicted at Janet Avenue.

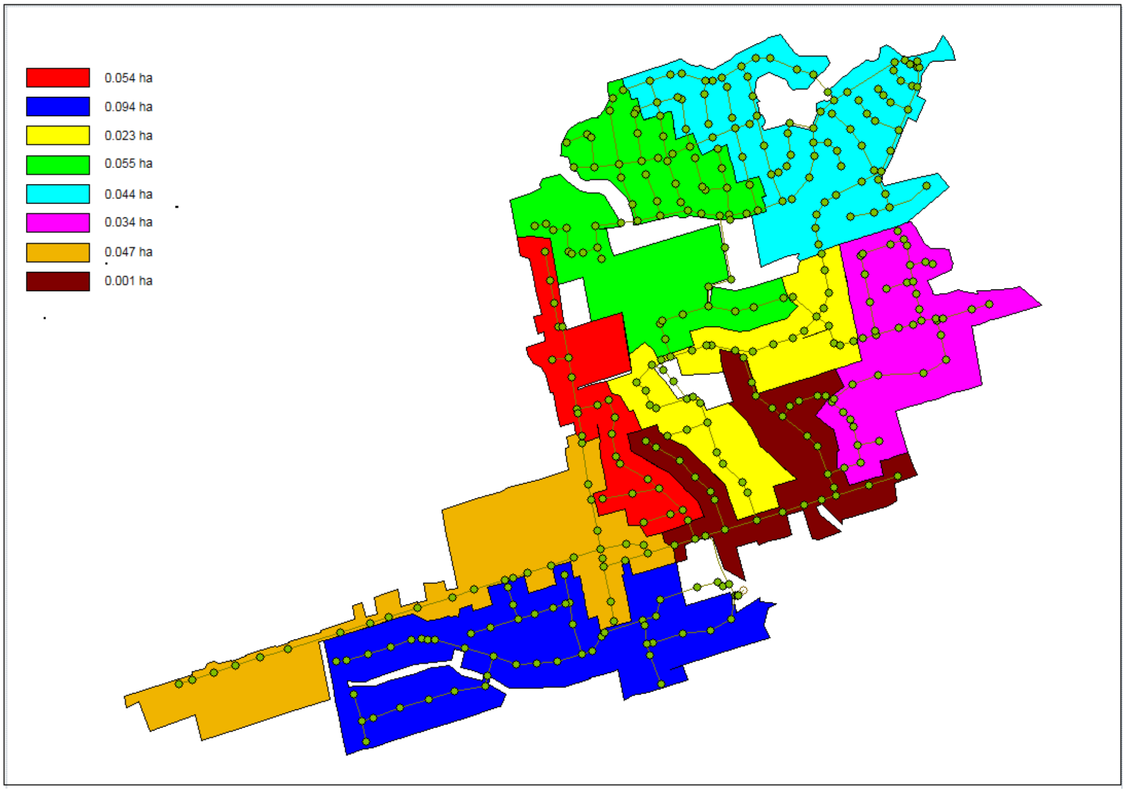


Figure 10 – Distribution of roof area applied within the model

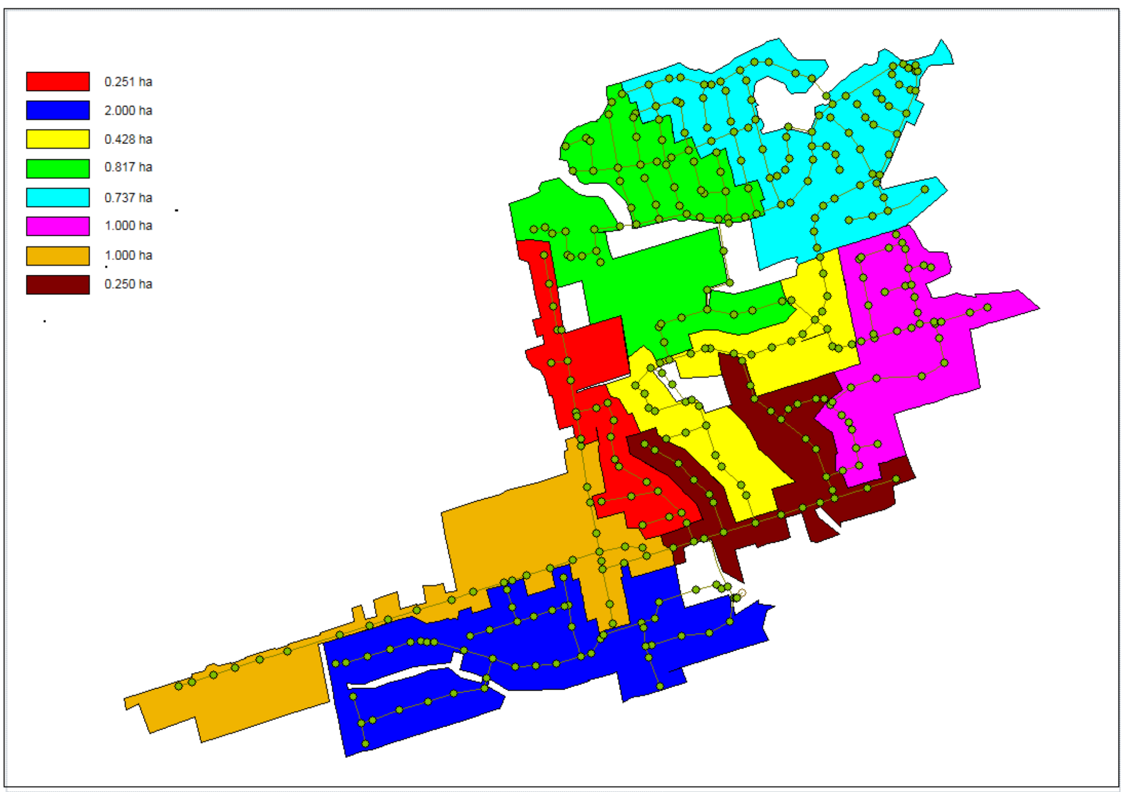


Figure 11 – Amount of GWI area upstream of each monitor

Table 8: Comparison of observed to predicted data for the three storms (green text is within tolerances and red text is outside of tolerances)

FLOW MONITOR	STORM EVENT	OBS PEAK DEPTH (M)	MODEL PEAK DEPTH (M)	OBS PEAK FLOW (L/S)	MODEL PEAK FLOW (L/S)	OBS VOLUME (MLD)	MODEL VOLUME (MLD)	PEAK DEPTH DIFF (M)*	PEAK FLOW DIFF (%)	VOLUME DIFF (%)
KI004a_10	EV A	0.600	1.314	42	45	1.508	1.814	0.714	7%	20%
	EV B	0.600	1.619	55	59	1.795	2.246	1.019	7%	25%
KI004a_20	EV A	0.111	0.114	22	22	0.445	0.444	0.003	0%	0%
	EV B	0.126	0.105	28	19	0.847	0.630	-0.021	-32%	-26%
KI004b	EV A	0.175	0.145	35	40	0.864	0.796	-0.030	14%	-8%
	EV B	0.187	0.146	55	40	1.271	0.984	-0.041	-27%	-23%
KI005	EV A	0.125	0.120	48	41	1.231	0.679	-0.005	14%	-45%
	EV B	0.129	0.121	53	41	1.533	0.825	-0.008	-23%	-46%
KI006	EV A	0.058	0.092	10	11	0.360	0.281	0.034	10%	-22%
	EV B	0.065	0.088	14	10	0.553	0.376	0.023	-29%	-32%
KI008	EV A	0.064	0.077	8	8	0.134	0.155	0.013	0%	16%
	EV B	0.060	0.070	7	6	0.247	0.186	0.010	-14%	-25%
KI009	EV A	0.060	0.079	14	14	0.488	0.415	0.019	0%	-15%
	EV B	0.072	0.073	19	12	0.812	0.513	0.001	-37%	-37%
KI010	EV A	0.074	0.092	12	13	0.402	0.428	0.018	8%	7%
	EV B	0.075	0.086	13	11	0.522	0.495	0.011	-15%	-5%

Overall a reasonable match to the observed data has been achieved with the model. As well as the flow monitors the model has been compared to the SCADA data at Janet Avenue. This shows a reasonable match to the information available. However, as the operation of the pumping station cannot be replicated exactly it has not been possible to achieve a good match when comparing the depths and the flows recorded at the pumping station. Within the model a throttle has been included in the model to try and slow down the peak the flows entering the wet well but there is no information to confirm that this exists. Graphs showing the match that has been achieved are included in Appendix A. Figure 12 shows the schematic with information about the level of calibration achieved at each monitor and at the pumping station.

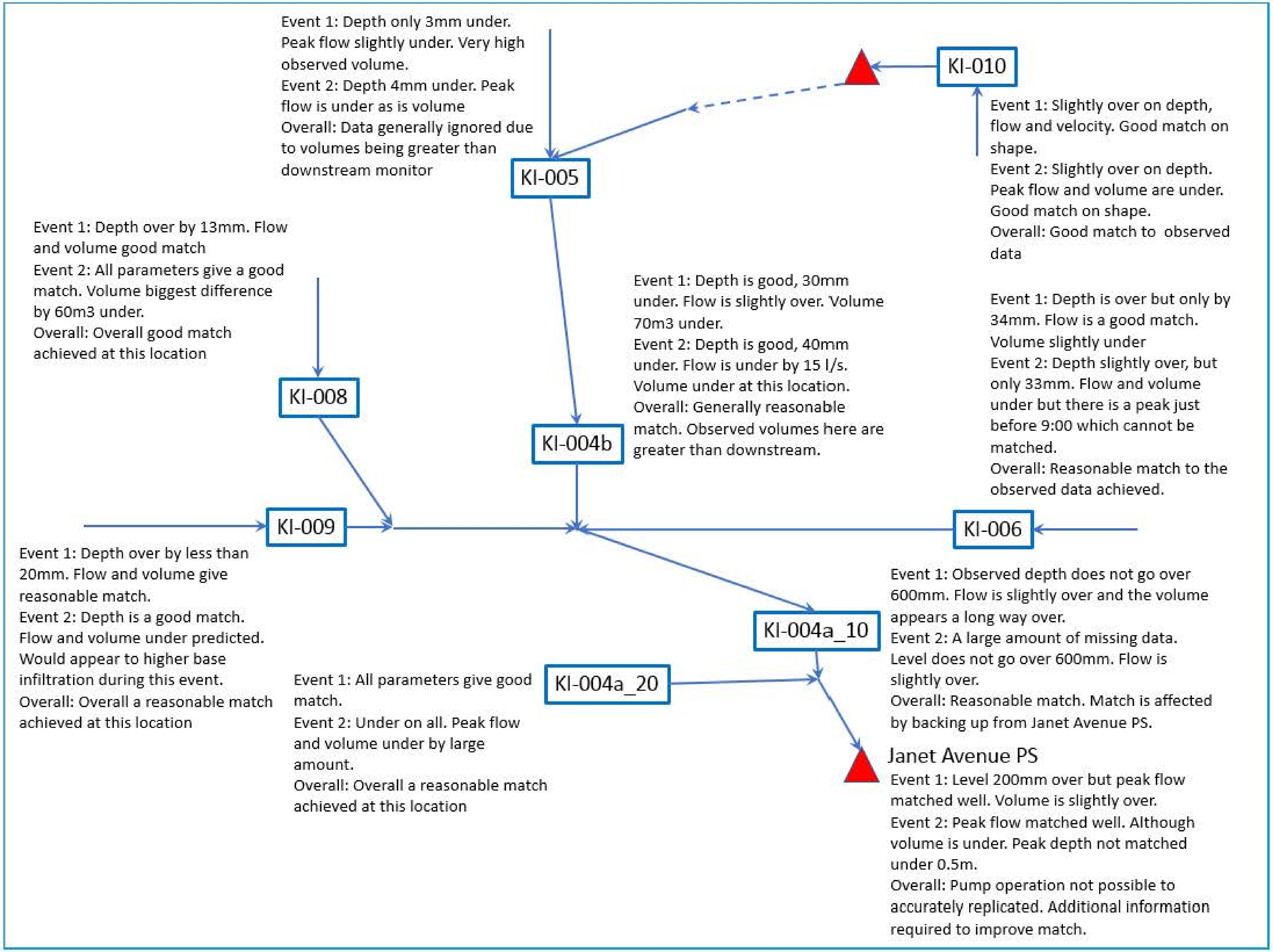


Figure 12: Overview of the level of calibration achieved across the catchment

3 Existing Wastewater System Capacity Review and Optimization

The Existing Wastewater System (Stage 1) Hydraulic Analysis evaluated what flow the existing Nobleton infrastructure is capable of servicing.

3.1 EXISTING SYSTEM CAPACITY REVIEW

The following section presents the results of running the model for a 1 in 25 year return period design storm from the Region's model. This is the standard that should be used to design new assets. The design storm used is a four hour duration event with a peak intensity of 86.4 mm/hr and a depth of 59.3mm. Figure 13 shows the hyetograph which has been used.

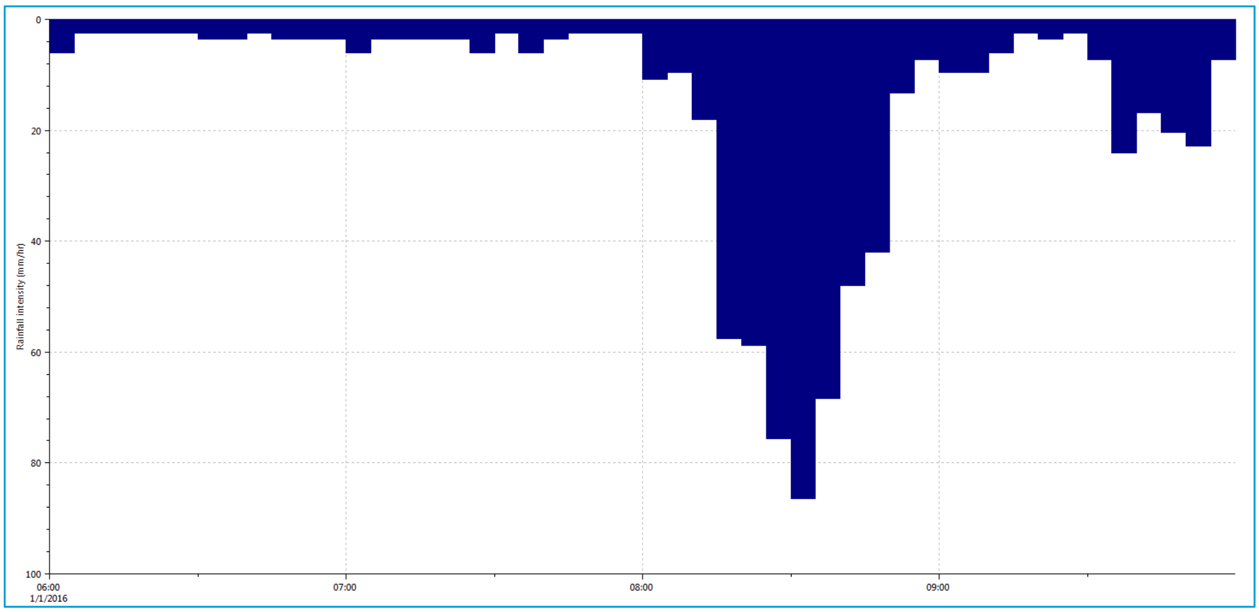


Figure 13 – Design Rainfall Applied to the Model

The modelling has looked at the following:

- Flooding
- Surcharging
- Velocities
- Capacity of Janet Avenue PS

Three different scenarios have initially been assessed when looking at the existing system:

- Existing population used to calibrate the model
- Population increased to account for the properties currently connected to septic tanks
- Population increased to account for all the future growth up to 10,800

The reason for looking at these three scenarios is that it is important to understand if the current system can cope with the current population which is connected but also whether it can cope when those properties currently on septic tanks are connected to the system and whether it can cope with the future population growth.

3.2 EXISTING POPULATION

The current population which the model was calibrated against is just under 4,000. This is residential population. To account for the non-residential population the per person consumption has been increased from 200 L/p/d to 229.2 L/p/d. This gives a good match to the observed data from the flow monitors which are located within the network and the SCADA data which was available on the operation of Janet Avenue SPS.

3.2.1 Infiltration and Inflow Rates

From the peak flows for each of the areas a comparison has been made to the design standard of 0.26 L/s/ha to understand whether the flows entering the system are above this standard. Table 9 shows this comparison. When calculating the flows in the table it has been assumed that the area is the total area which drains to the flow monitor. For flow monitor KI004a_10 there is backing up from Janet Avenue PS which will be affecting the peak flow which is generated. As an additional test the model has been run with a free discharge at Janet Avenue to determine if there is a difference in the amount of runoff generated. These differences are shown in brackets in Table 9.

Table 9: Comparison of modelled runoff to the design standard

FLOW MONITOR AREA	AREA (HA)	DESIGN STANDARD RUNOFF (L/S)	MODELLED RUNOFF (L/S)	PEAK I/I RATES (L/S/HA)	PEAK I/I RATES FROM CIVICA REPORT (L/S/HA)
KI004a_10	25.33	6.59	7.5 (11.3)	0.30 (0.45)	0.24
KI004a_20	45.63	11.86	29.2	0.64	0.98
KI004b	27.77	7.22	14.0	0.50	0.20
KI005	53.22	13.84	12.3	0.23	0.26
KI006	30.77	8.00	14.6	0.47	0.26
KI008	20.62	5.36	9.2	0.45	0.74
KI009	42.98	11.18	14.6	0.34	0.24
KI010	40.61	10.56	11.4	0.28	0.41

From these results it is possible to see that in the existing network there are several areas which are over the design standard of 0.26 L/s/ha. The largest of these are the area which drains to KI004a_20 which drains from the area to the south of the catchment directly into Janet Avenue.

Within Table 9 the numbers from the CIVICA report have been added and it is possible to see that there are a number of differences. The main reason for these differences could be related to the way that the number has been calculated. For the CIVICA report only the observed data has been used up to 2015 whereas the model has been calibrated against the largest storms which have been recorded in the catchment from 2017. Although there are some differences it is possible to see that there are some similarities in results. For both the area which drains to KI004a_20 has been identified as a priority, as has the area draining to KI008. The new results would suggest that the majority of the catchment should be a high priority due to the amount of runoff which is generated apart from the area draining to KI010 and then KI005.

3.2.2 Flooding

When the model was run for the 1 in 25 year design storm there was no predicted flooding within the catchment. This suggests that the current system can cope with the existing population. As mentioned in Section 2 of this report as part of the calibration of the model it was necessary to add some roof area connected directly to the sanitary system to match the observed data and an element of RDII. Although this means that there is an increase in the flows in the system during a rainfall event it is not enough to cause any flooding during this design event for the existing situation.

3.2.3 Surcharging

As mentioned there is no flooding predicted within the catchment for this design event. However, there are some pipes which become surcharged. The locations of these are generally just upstream of Janet Avenue SPS and there are also a couple of other pipes upstream of Bluff Trail SPS. As well as these locations there are also a couple of other pipes within the catchment which show surcharging. The pipes where the surcharging occurs are shown in Figure 14 (pipes highlighted in red are surcharged).

Figure 14 – Pipes which are surcharged during the design event for the existing system

The reasons for the surcharging are due to backing up of the flows from Janet Avenue SPS. As mentioned as part of the calibration of the model to achieve a match to the SCADA data it was necessary to add a restriction to the inlet to the pumping station. The other bits of surcharging are due to local incapacities in the area.

3.2.4 Velocities

A review of the velocities across the catchment has been undertaken. The reason for doing this is that is that if the velocities are too low in the catchment then there would be a risk of the solids dropping out and potentially causing blockages within the system.

In the dry weather situation there are many pipes where the maximum velocity is below 0.6 m/s which is the minimum as stated in the Ministry of the Environment and Climate Change (MOECC) design criteria.

In the design event more than half of the pipes in the network are predicted to have a peak velocity less than 0.6 m/s. This means that there is currently a risk of potential blockages within the system.

3.2.5 Storage Capacity at Janet Avenue SPS

The current arrangement at Janet Avenue SPS is that the pumps operate on a duty / assist / standby arrangement. This arrangement has been calculated based on the SCADA data which was available. With the current population the model shows that both the duty and the assist pumps are operational during the peak of the 25 year return period event. This means that there is a peak flow arriving at the WRRF of 106 l/s. This modelled flow ties into the peak flows which have been recorded on the SCADA system.

In terms of the peak water levels within the pumping station the level currently gets high enough to back up into the system upstream. The peak water level is at a level of 248.18 mAOD which means that it has filled the lower part of the wet well and it has started to the fill the area of the wet well above. This peak water level is a long way below the overflow level at the pumping station. The overflow is at a level of 254.135 mAOD which is almost 6m above the peak water levels.

3.3 CAPACITY OF THE EXISTING SYSTEM

Before looking at the impact of any growth within the area it is important to understand what the capacity of the existing system is. This has been carried out to ensure that there is no flooding within the existing system or any spilling from the emergency overflow at Janet Avenue pumping station. The additional population has been added with base infiltration of 90 L/cap/day and a peak storm response of 0.26 L/s/ha.

The modelling which has been carried out currently shows that the network is able to cope with the existing population of 3,643. The maximum population which the catchment can cope with is 5,318 assuming that no optimization strategies are carried out. This is a long way short of the required capacity with all of the growth being built up to a population of 10,800.

3.4 EXISTING POPULATION PLUS THE PROPERTIES ON SEPTIC TANKS AND DEVELOPMENTS COMPLETED SINCE 2016

There is currently a project which is underway to connect the remaining properties of Nobleton which are currently on septic tanks to the existing wastewater system. This will increase the population from the current modelled residential population of 3,643 to a population 5,547. Within the model these areas have been added to the model using the same occupancy ratio as the rest of the catchment. Base infiltration has been added to these areas using a consumption of 90

L/cap/day. Storm contribution has been added as 0.26 L/s/ha where the area has been taken as the total area.

3.4.1 Flooding

The model with this additional population still does not show any predicted flooding across the catchment. There is a fairly large increase in the peak flow as the population has increased by approximately 2,500 which is an increase in baseflow of 2.6 L/s. There is a large increase in the storm runoff as well.

3.4.2 Surcharging

With the increase in population the areas where there is surcharging predicted are increased. This is especially the case around Janet Avenue where a much large area of the network is now surcharged. Figure 15 shows the locations across the catchment where surcharging is now predicted.

Figure 15 – Predicted pipes which are surcharged with the septic tank population included

3.4.3 Velocities

Due to the increase in flows during the storm conditions there are large areas of the network where the velocity is now above 0.8 m/s and therefore the risk of blockages are reduced.

3.4.4 Storage Capacity at Janet Avenue

Due to the population upstream of Janet Avenue SPS and the increase in the storm flows the pumps at Janet Avenue are beaten and the wet well fills to a level whereby flow is predicted to be spilt through the emergency overflow. In this scenario 80 m^3 is predicted to be spilt. It should be noted that this has been run with only two pumps operating which gives a maximum capacity of 106 L/s of the pumping station. Potentially if the third pump is turned on then this volume would not be spilt through the overflow but it would significantly increase the peak flow which is seen at the WRRF.

3.5 POPULATION INCREASED TO INCLUDE ALL FUTURE DEVELOPMENTS

The catchment of Nobleton is planned to have significant growth which will increase the population to 10,800. As part of the water study a plan was developed where this growth would occur across the catchment of Nobleton. Figure 16 shows the planned growth across the catchment.

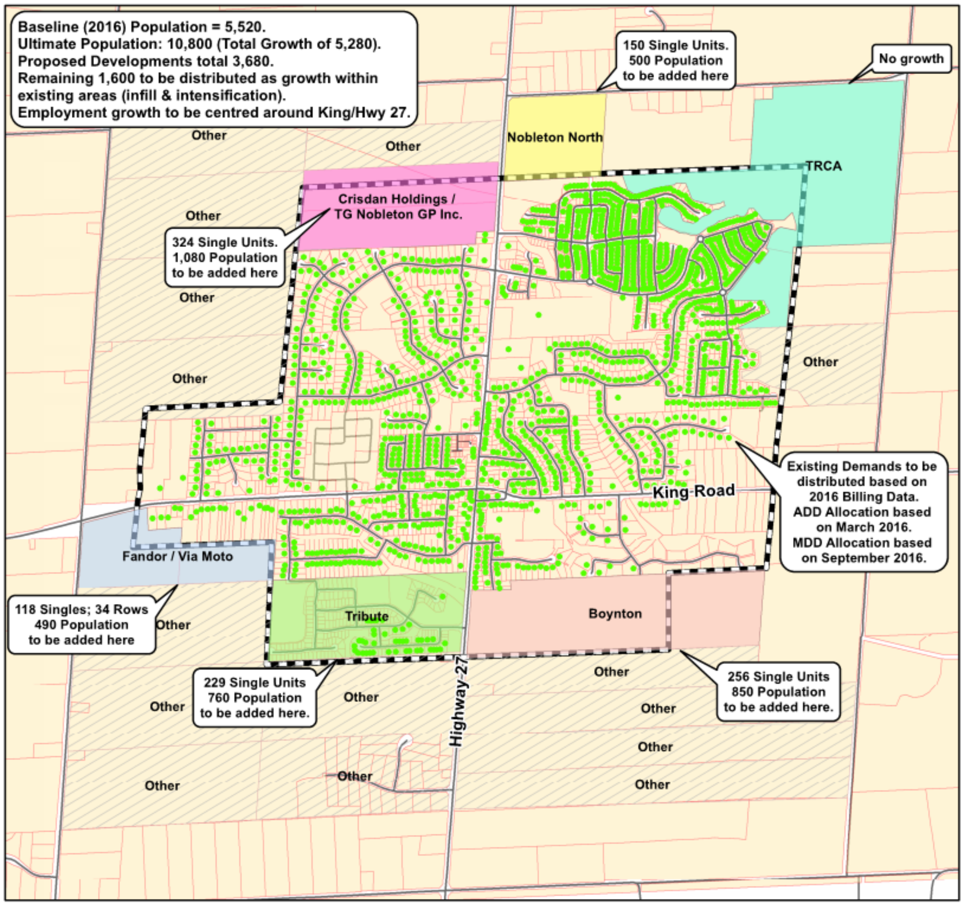


Figure 16 – Plan of proposed growth across the catchment

This growth has been added to the model using the same approach as those which are currently connected to septic tanks. This means that the base infiltration has been added as 90 L/cap/day and the storm response has been added as 0.26 L/s/ha.

3.5.1 Flooding

When the model has been run there are two manholes which are predicted to flood. These are located on King Road to the east of Janet Avenue. This flooding is caused by the backing up of the sewers from Janet Avenue SPS. They are also the two lowest manholes in the area upstream of the pumping station.

3.5.2 Surcharging

As there is such a large increase in the population, the amount of surcharging across the catchment is increased significantly. This is especially the case in the area upstream of Janet Avenue pumping station where the flow is backed up from the pumps. There is also an area in Paradise Valley Trail and Parkheights Trail where surcharging is predicted. Figure 17 shows the surcharging with all the growth included.

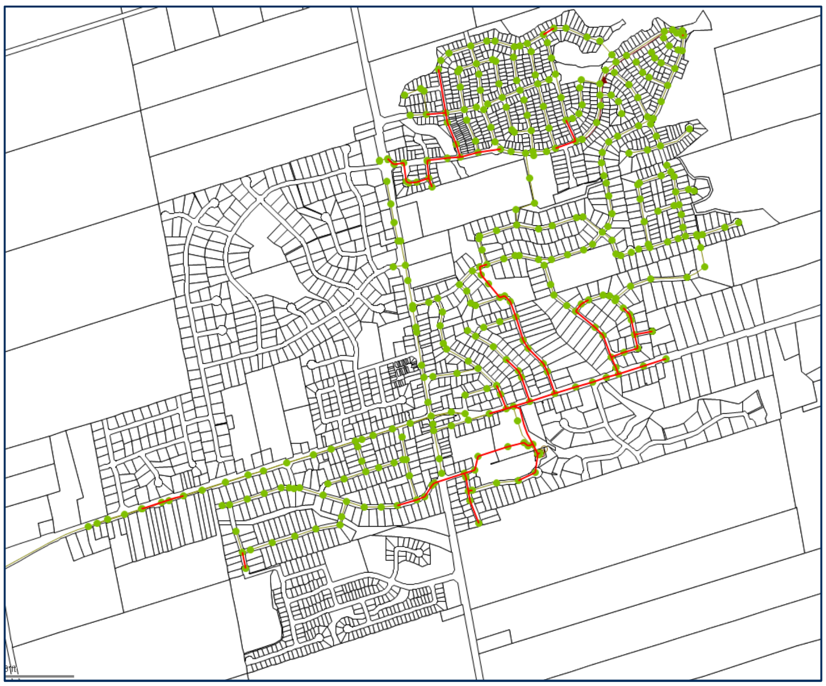


Figure 17 – Surcharging within the catchment with all the proposed growth included

3.5.3 Storage Capacity at Janet Avenue

Due to the large increase in the flows arriving at the pumping station the capacity of the pumping station is exceeded and this causes 870 m^3 of volume to be spilt through the emergency overflow at the pumping station. It should be noted that this has been run with two pumps in operation and if the third pump was used then this volume would reduce but there is still likely to be volume spilt through the overflow. This would also increase the peak flows that arrive at the WRRF.

3.6 EXISTING SYSTEM OPTIMIZATION OPPORTUNITIES

The next part of the Existing System Analysis is to determine if there is anything that can be done to optimize the existing system in order free up head room to allow for the increase in population.

3.6.1 Per Capita Consumption

In order to free up space within the system there is potential that per capita consumption of the existing residents could be reduced. However, in the case of Nobleton the results of the modelling have shown that the current per capita consumption is at 229.2 L/person/day. This is made up of 200 L/person/day for residential flows and 63 L/person/day for non residential flows. These numbers are close to the region average numbers and there is limited opportunity to reduce this further. To reduce to any further would require customer engagement to change people's habits in the use of water within the house. This would suggest that in this catchment this may not be possible.

3.6.2 Base Infiltration

When the model was calibrated it was found that it was necessary to include an element of base infiltration to match the night time flows which were seen by the flow monitors in the catchment. Figure 5 shows the areas where this has been included in the model. The values added to the model are in fact low and potentially the sources of these would be difficult to identify.

However, with the use of CCTV it may be possible to find out if there are certain sections of pipes where this infiltration could be entering the system. A review would need to be carried out after any work had been undertaken on the system to identify if there had been a reduction in this flow.

3.6.3 Removal of Roof Area

Within the modelling to achieve the calibration of the model the impermeable area contributing flows to the system has been assumed to be roof area. It could however, be other connected area but additional surveys would be required. The sewer system in Nobleton is meant to be sanitary only and therefore there should be no roof area connected to the system. A model run has been undertaken with the roof area removed to determine what impact this would have on the surcharge levels within the catchment and also the operation of the pumping station.

This has no impact on the level of surcharge within the system but it does slightly reduce the peak flows which arrive at the pumping station. Within the model run the peak flow reduces from 124.6 L/s to 105 L/s. This small reduction in the peak flow would suggest that it is not worth carrying out the work to remove this contribution to the system. However, for all the future developments it is important that the roofs are not allowed to connect so that the flow is not increased.

3.6.4 Removal of the Ground Water Infiltration (RDI)

Within the model an allowance was made for some rainfall related ground water infiltration to match the observed flows from the catchment. The amount of area which has been included can be seen in Figure 11. A model run has been carried out to remove this from the system although it is noted that this will be difficult to achieve due to the nature of how this flow enters the system. By removing this flow from the system the peak flow reduces from 124.6 L/s to 48.6 L/s. This is a large decrease in the peak flow which would free up a large capacity within the system.

As part of the flow monitoring study which was undertaken in 2015 it was highlighted that there was potentially groundwater entering the system and this has also been identified as part of the modelling which has now been undertaken. The issue with this type of inflow is trying to identify where it is entering the system and how much can be removed by either undertaking relining or replacement of pipes.

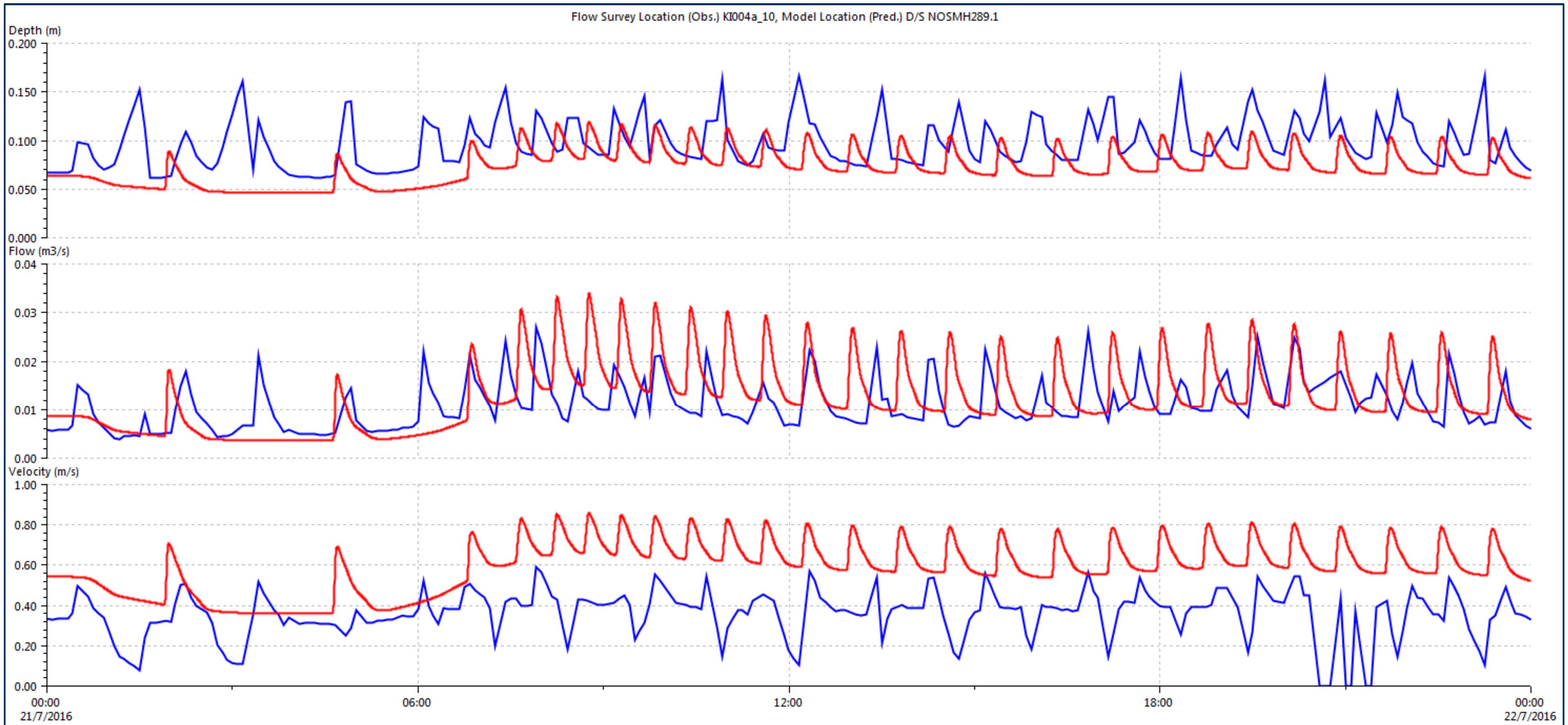
4 Summary and Conclusions

The following conclusions can be made based on the results of the existing system analysis:

- No flooding is predicted within Nobleton for the 1 in 25 year design storm for existing condition. Flooding is only predicted when the full growth is added to model and this is limited to only a couple of manholes.
- Surcharging is predicted for the 1 in 25 year design storm in some locations. This is mainly related to the flows backing up from Janet Avenue SPS. As the population is increased the amount of surcharge within the system is also increased.
- There is enough capacity within Janet Avenue SPS for the existing connected population and the population up to 5,318. Above this number there is no flooding predicted but flows will start being spilt through the emergency overflow. Additional capacity within the system could be achieved by using the third pump but this would increase the peak flows arriving at the WRRF.
- The current dry weather flows within the catchment do not cause any issues within the existing network.
- The current operation of Janet Avenue SPS is the main reason for the surcharging around the pumping station. Within the model these have been setup as variable speed pumps to try and match the observed data, however, the data suggests that there may be some sort of restriction within the system which needs to be further investigated.
- The removal of base infiltration from the system will only increase the available capacity slightly.
- The removal of the inflow the from roofs has a small beneficial impact on the network. The removal of groundwater infiltration is predicted to have more of a beneficial impact on the network but may be difficult to achieve in practice.
- By adding in the population connected to the septic tanks the surcharge within the system is increased and there is a spill through the emergency overflow.
- The calibrated model provides a suitable tool for assessing the capacity of the system for accommodating future growth.
- The future growth causes a lot more surcharging within the system and also causes a larger volume to be spilt through the emergency overflow.

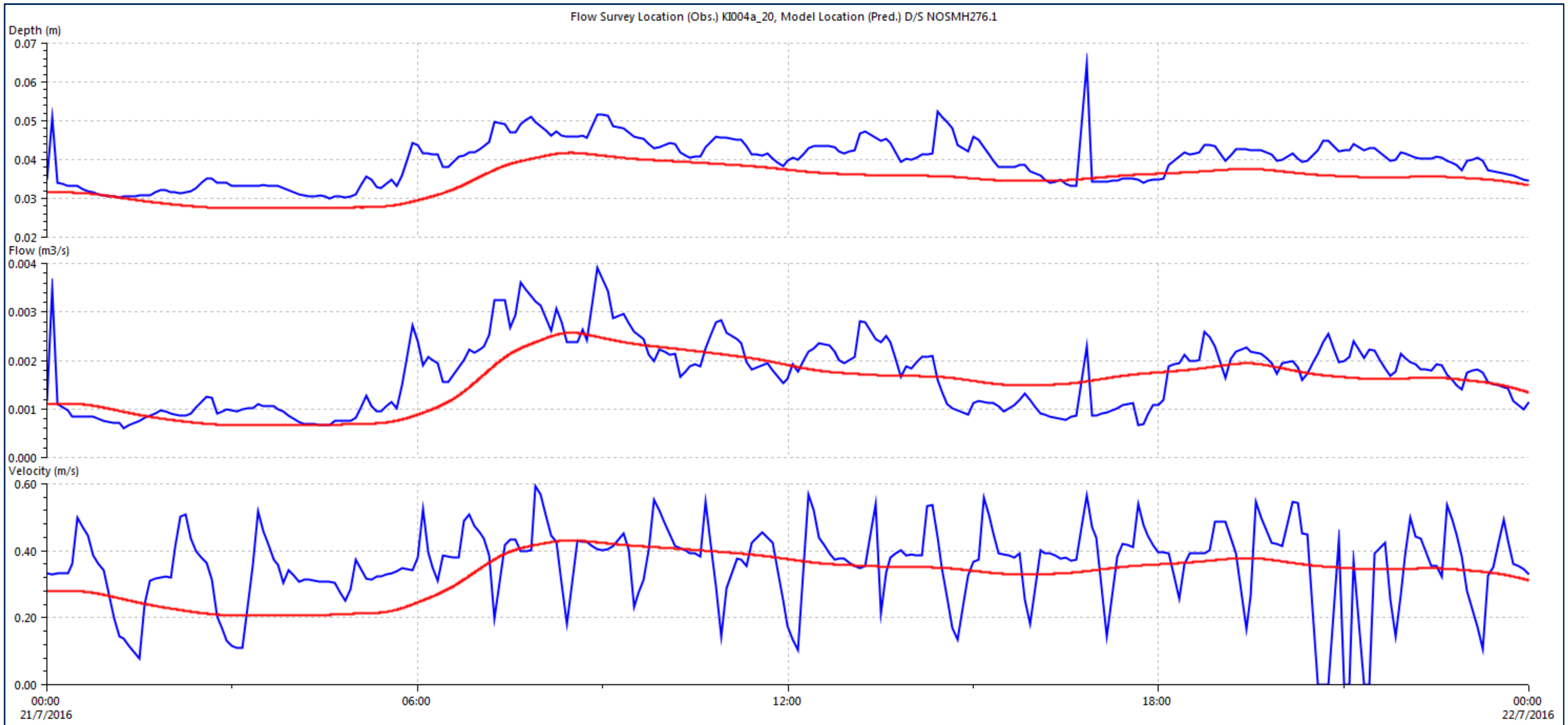
APPENDIX A: CALIBRATION GRAPHS

Figure A1: Dry weather graph for KI-004a_10 on 21/07/2016



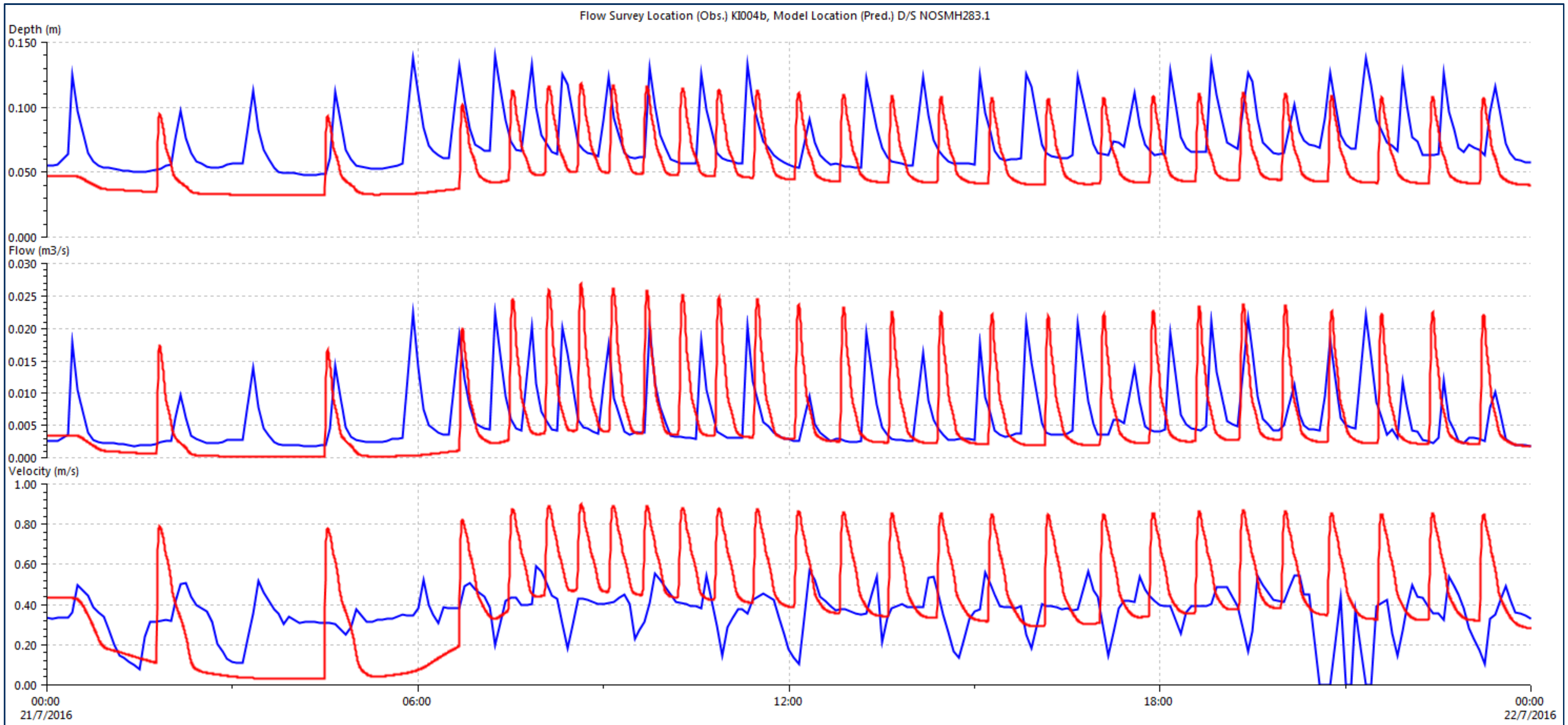
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.062	0.167	0.004	0.027	963.426	0.000	0.594
...ion 3.1 occ with sluice!>DWF	0.046	0.119	0.004	0.034	1087.839	0.360	0.857

Figure A2: Dry weather graph for KI-004a_20 on 21/07/2016



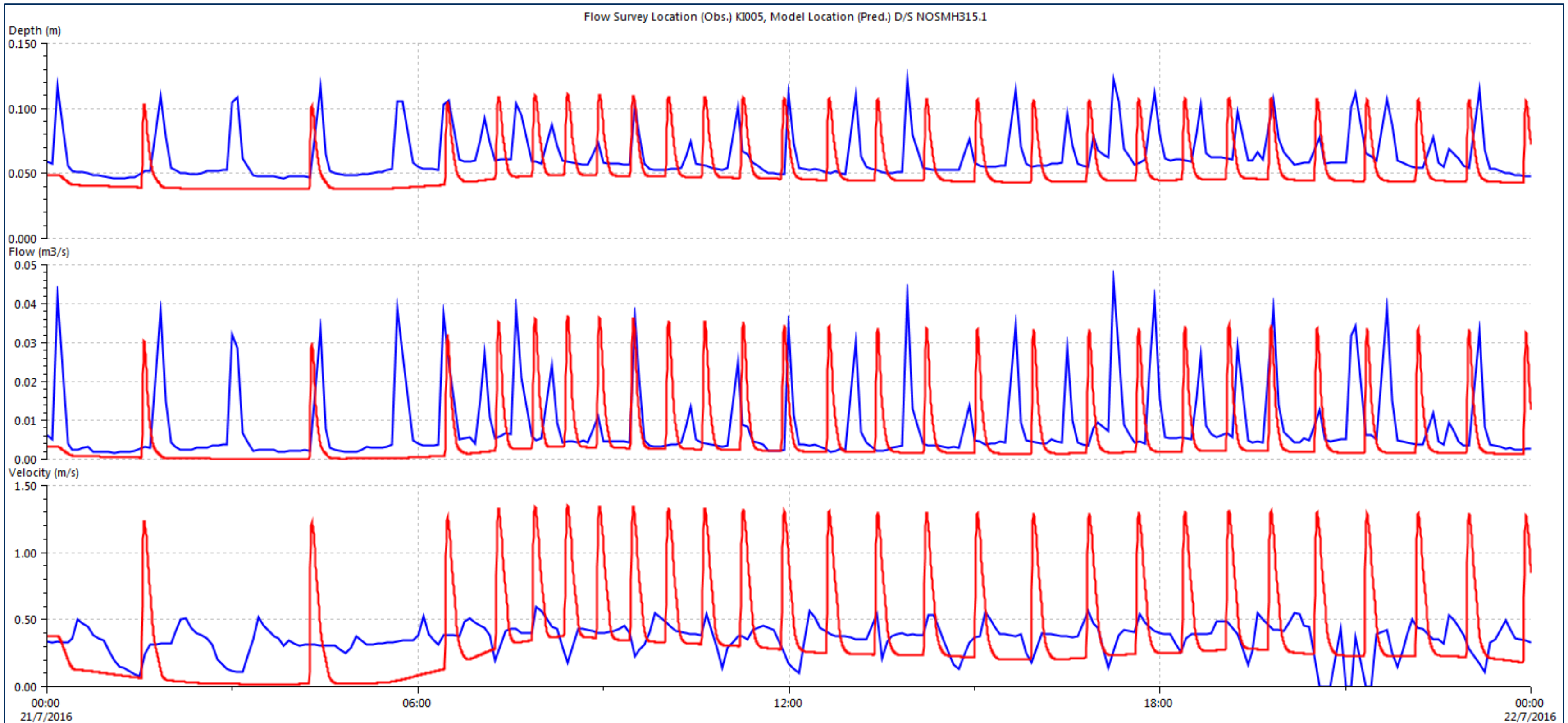
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.030	0.065	0.001	0.004	147.366	0.000	0.594
...ion 3.1 occ with sluice!> DWF	0.027	0.042	0.001	0.003	133.936	0.207	0.431

Figure A3: Dry weather graph for KI-004b on 21/07/2016



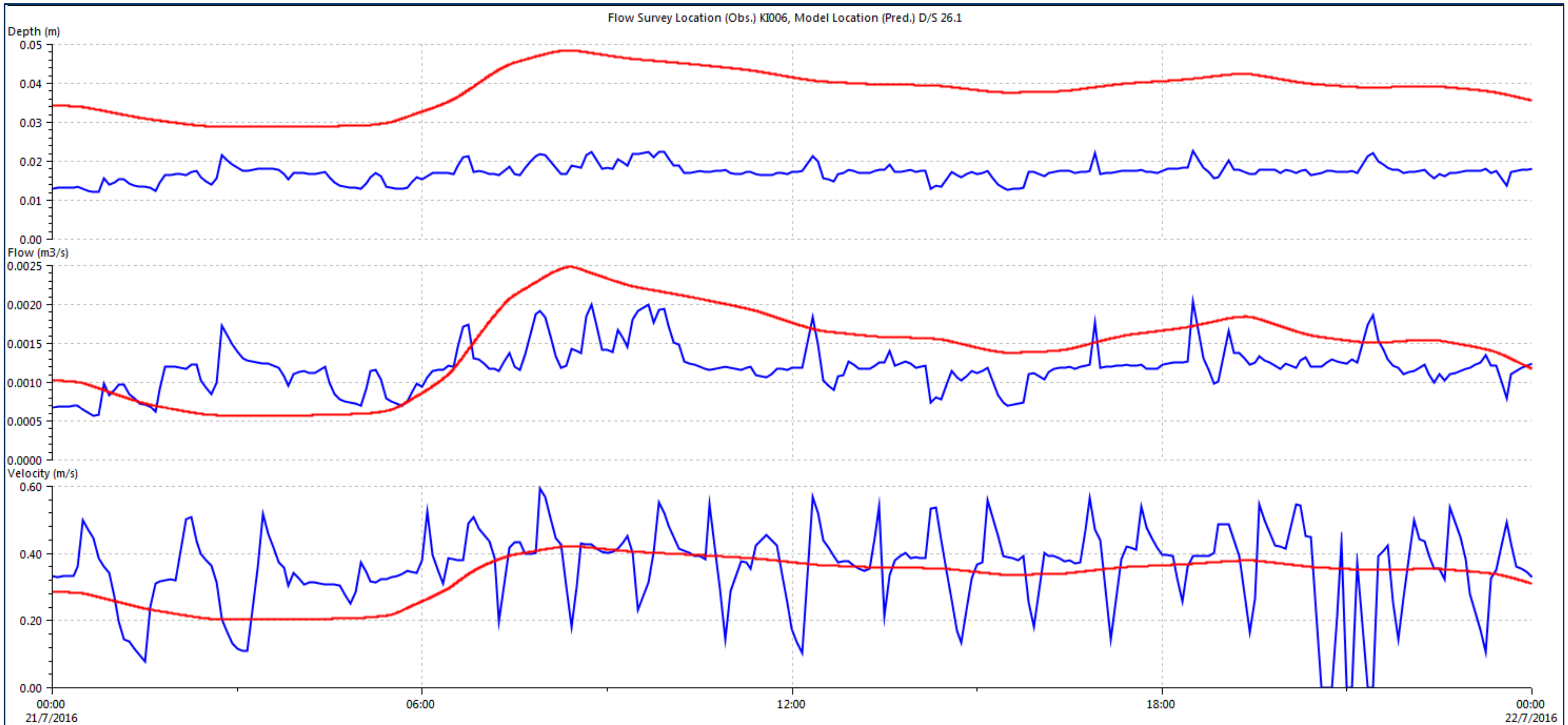
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.047	0.139	0.002	0.022	539.575	0.000	0.594
...ion 3.1 occ with sluice!>DWF	0.033	0.118	0.000	0.027	485.829	0.032	0.900

Figure A4: Dry weather graph for KI-005 on 21/07/2016



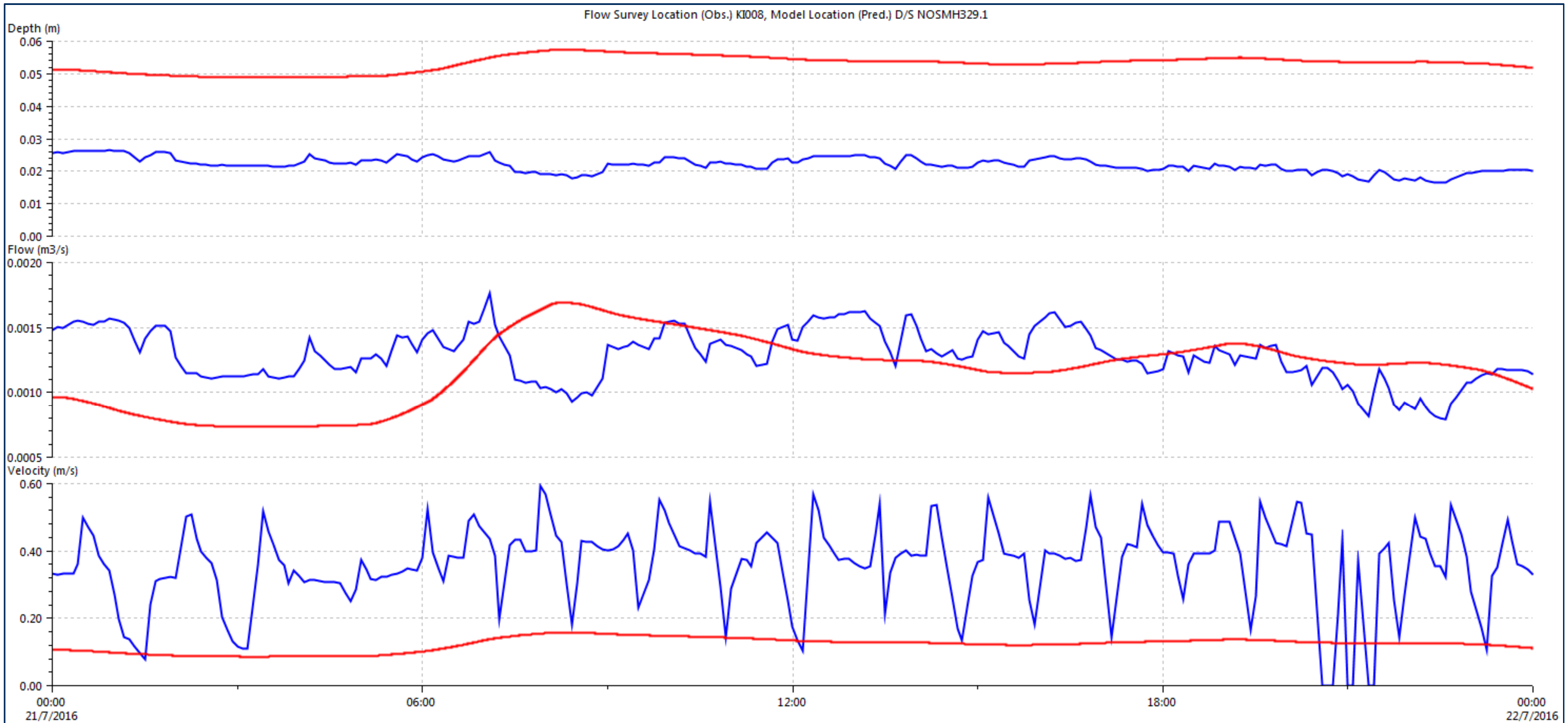
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.046	0.123	0.002	0.046	739.942	0.000	0.594
...ion 3.1 occ with sluice!> DWF	0.038	0.111	0.000	0.037	447.712	0.019	1.353

Figure A5: Dry weather graph for KI-006 on 21/07/2016



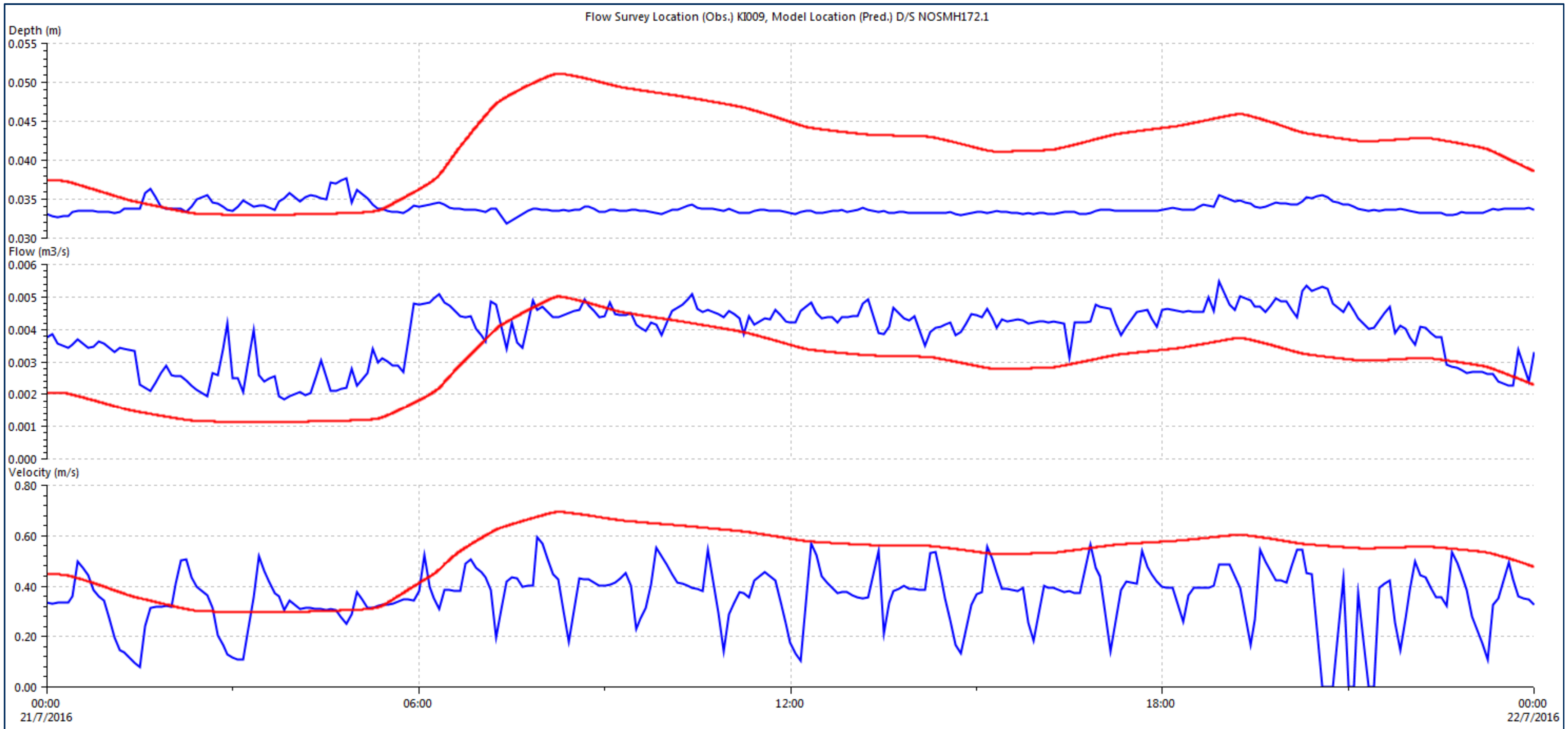
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.012	0.023	0.001	0.002	103.254	0.000	0.594
...ion 3.1 occ with sluice!> DWF	0.029	0.048	0.001	0.002	125.269	0.203	0.422

Figure A6: Dry weather graph for KI-008 on 21/07/2016



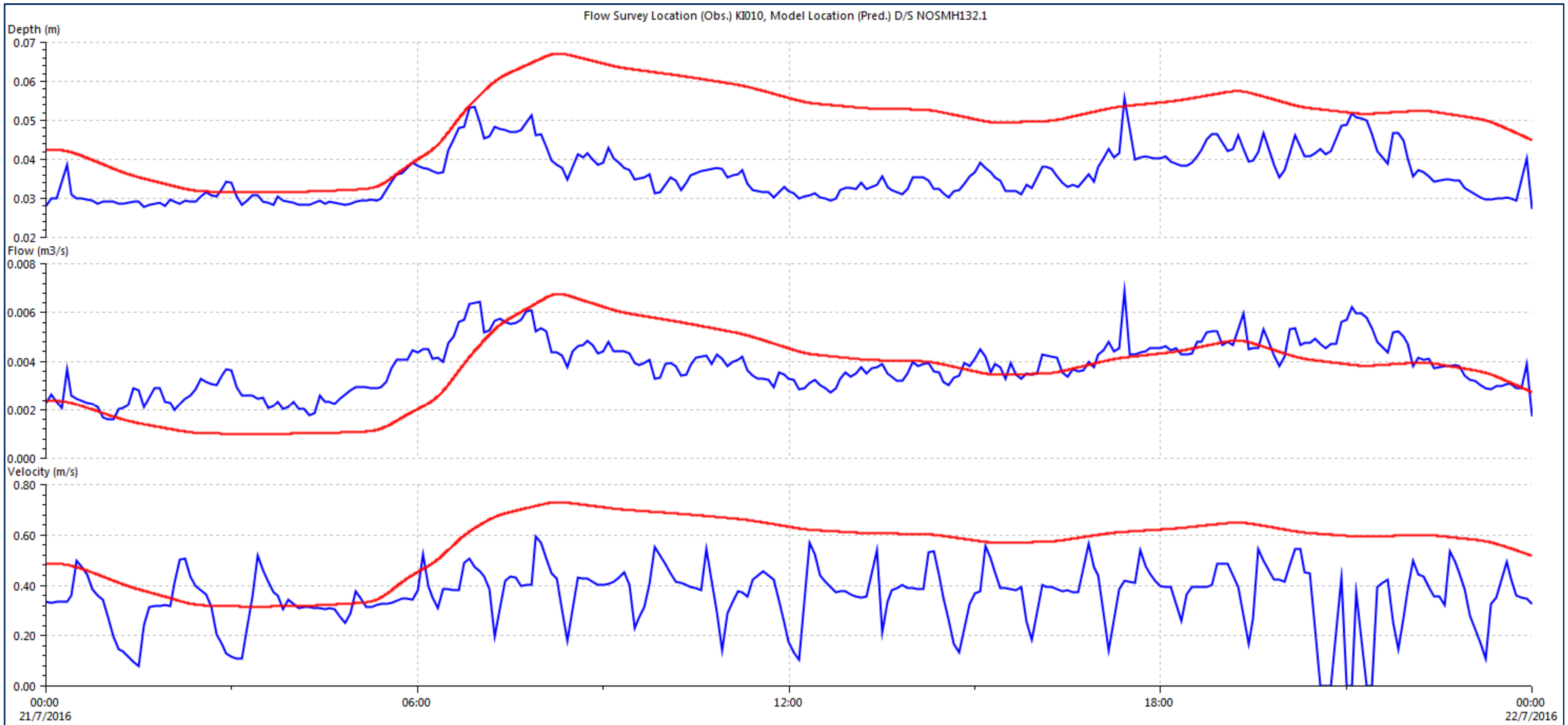
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.016	0.026	0.001	0.002	111.511	0.000	0.594
...ion 3.1 occ with sluice!> DWF	0.049	0.057	0.001	0.002	102.118	0.086	0.158

Figure A7: Dry weather graph for KI-009 on 21/07/2016



	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.032	0.038	0.002	0.005	338.519	0.000	0.594
...ion 3.1 occ with sluice!>DWF	0.033	0.051	0.001	0.005	252.427	0.297	0.695

Figure A8: Dry weather graph for KI-010 on 21/07/2016



		Depth		Flow			Velocity	
		Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed		0.027	0.056	0.002	0.007	330.093	0.000	0.594
...ion 3.1 occ with sluice!> DWF		0.032	0.067	0.001	0.007	315.326	0.316	0.731

Figure A9: Dry weather graph showing the comparison of the depth in Janet Avenue PS on 21/07/2016

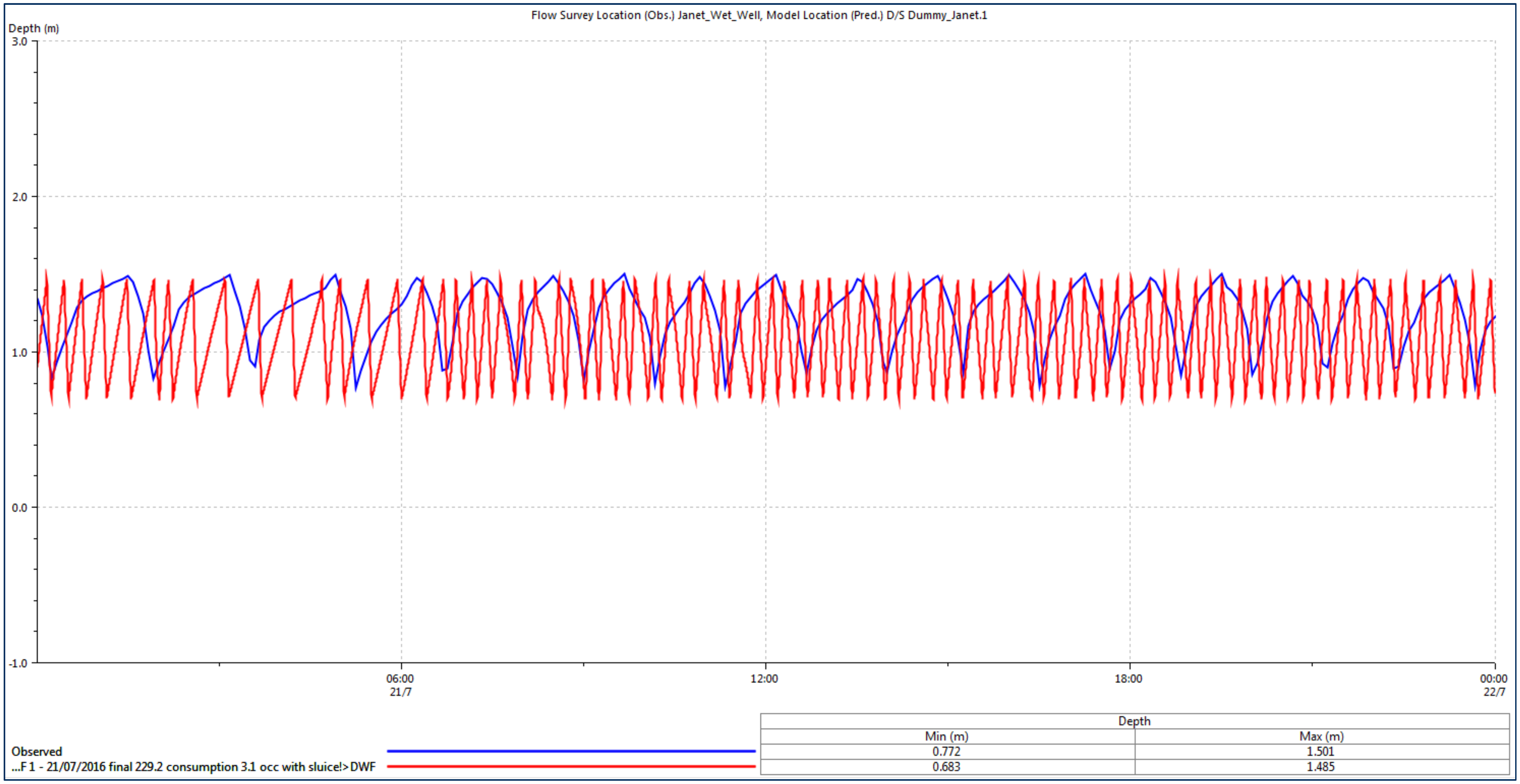
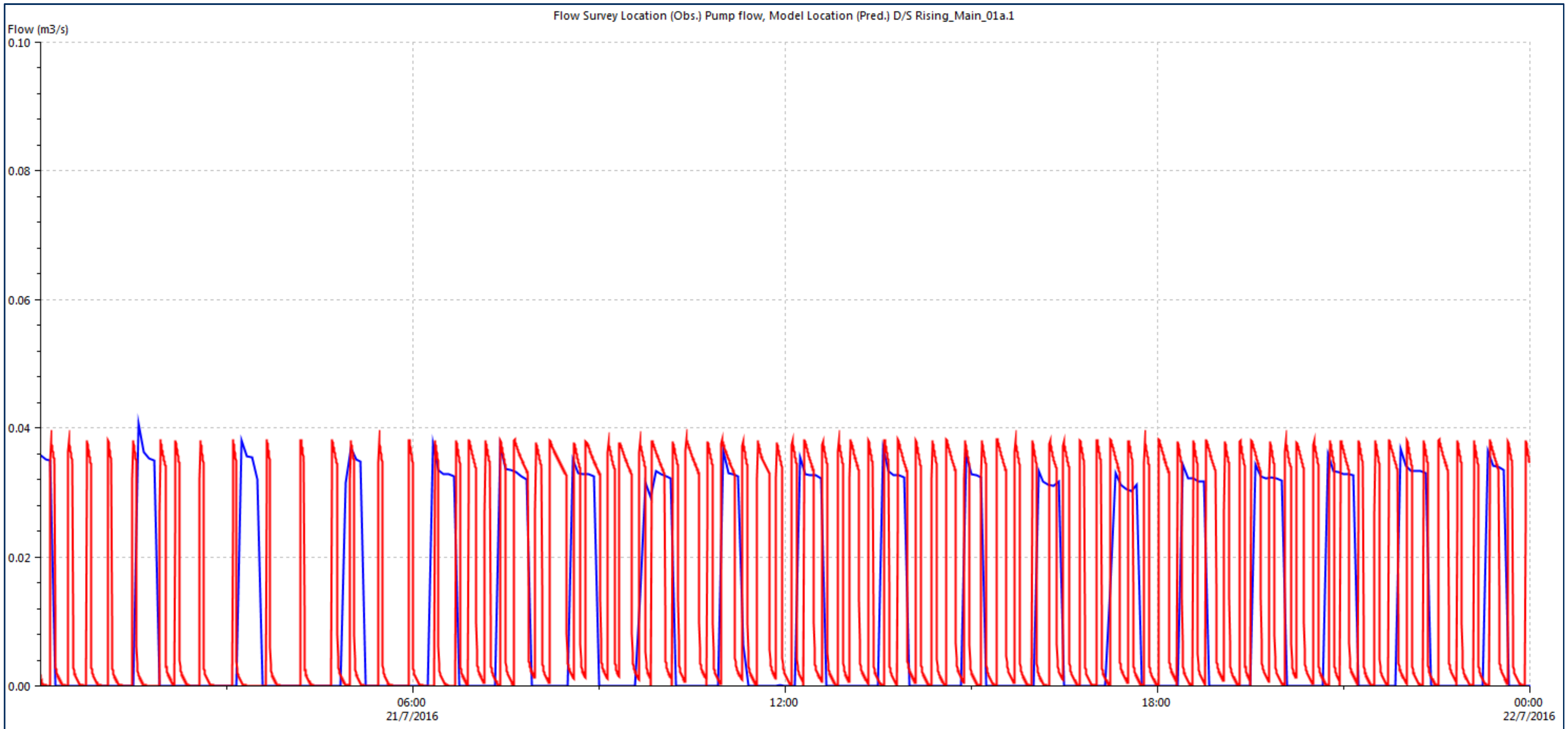
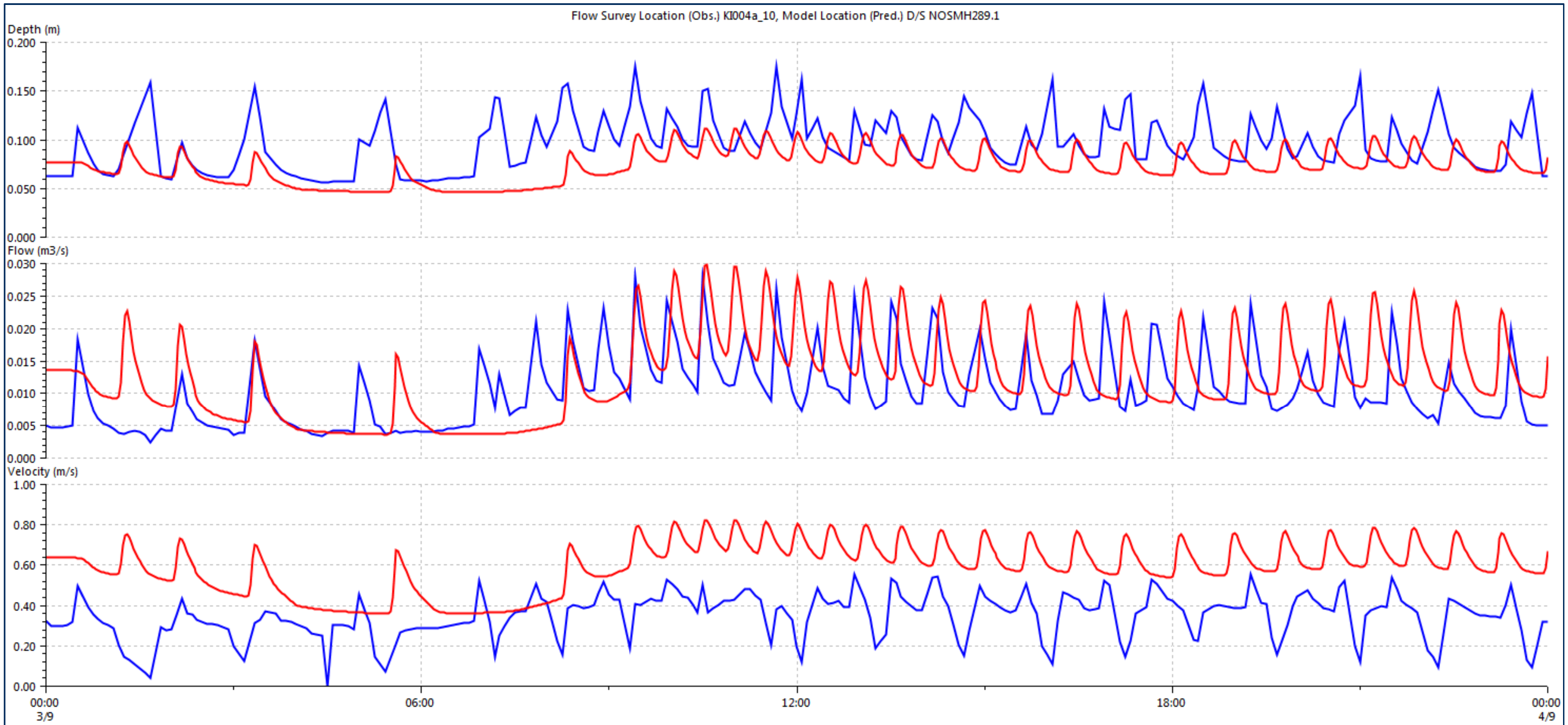


Figure A10: Dry weather graph showing the comparison of the flow along Janet Avenue PS forcemain on 21/07/2016



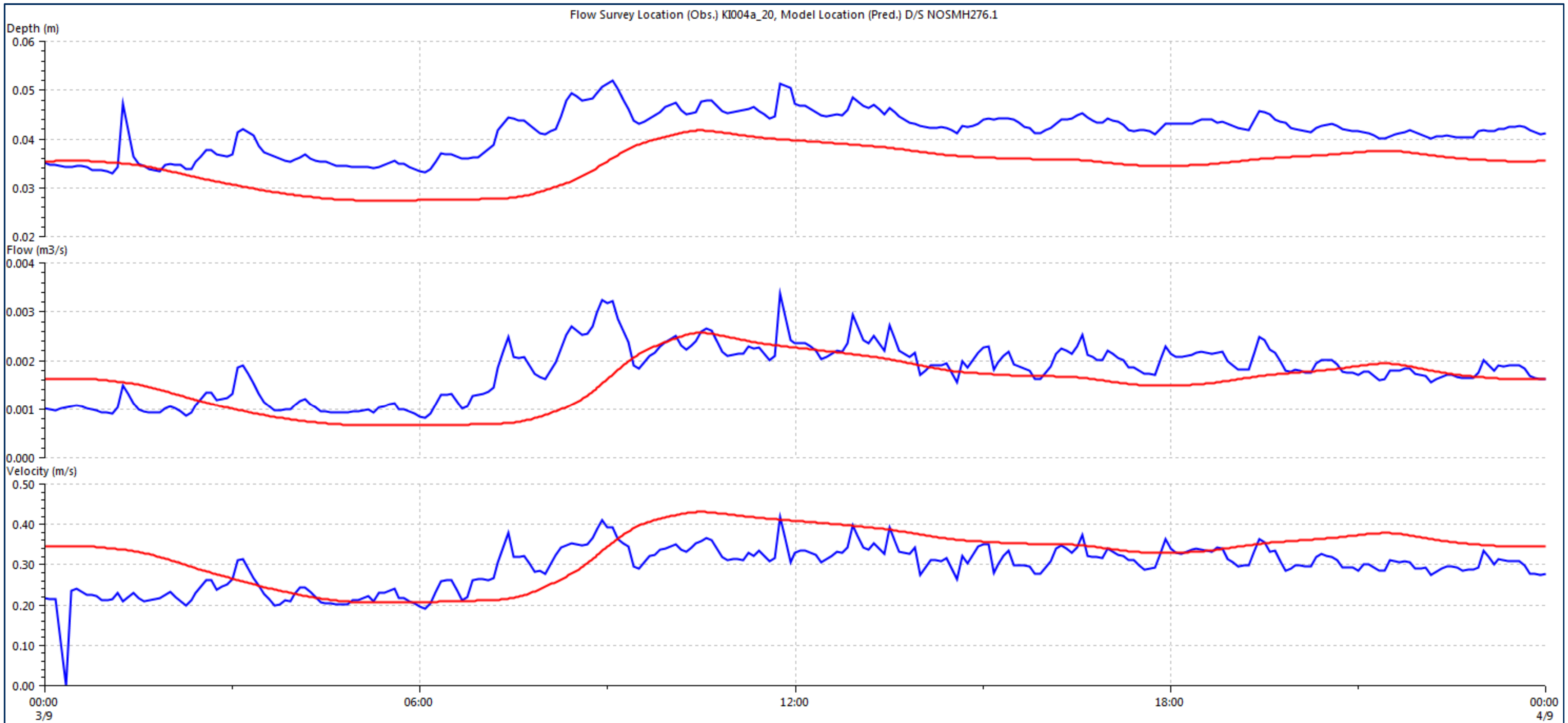
	Flow		
	Min (m3/s)	Max (m3/s)	Volume (m3)
Observed	0.000	0.041	929.739
...016 final 229.2 consumption 3.1 occ with sluice!>DWF	0.000	0.038	1229.131

Figure A11: Dry weather graph for KI-004a_10 on 03/09/2016



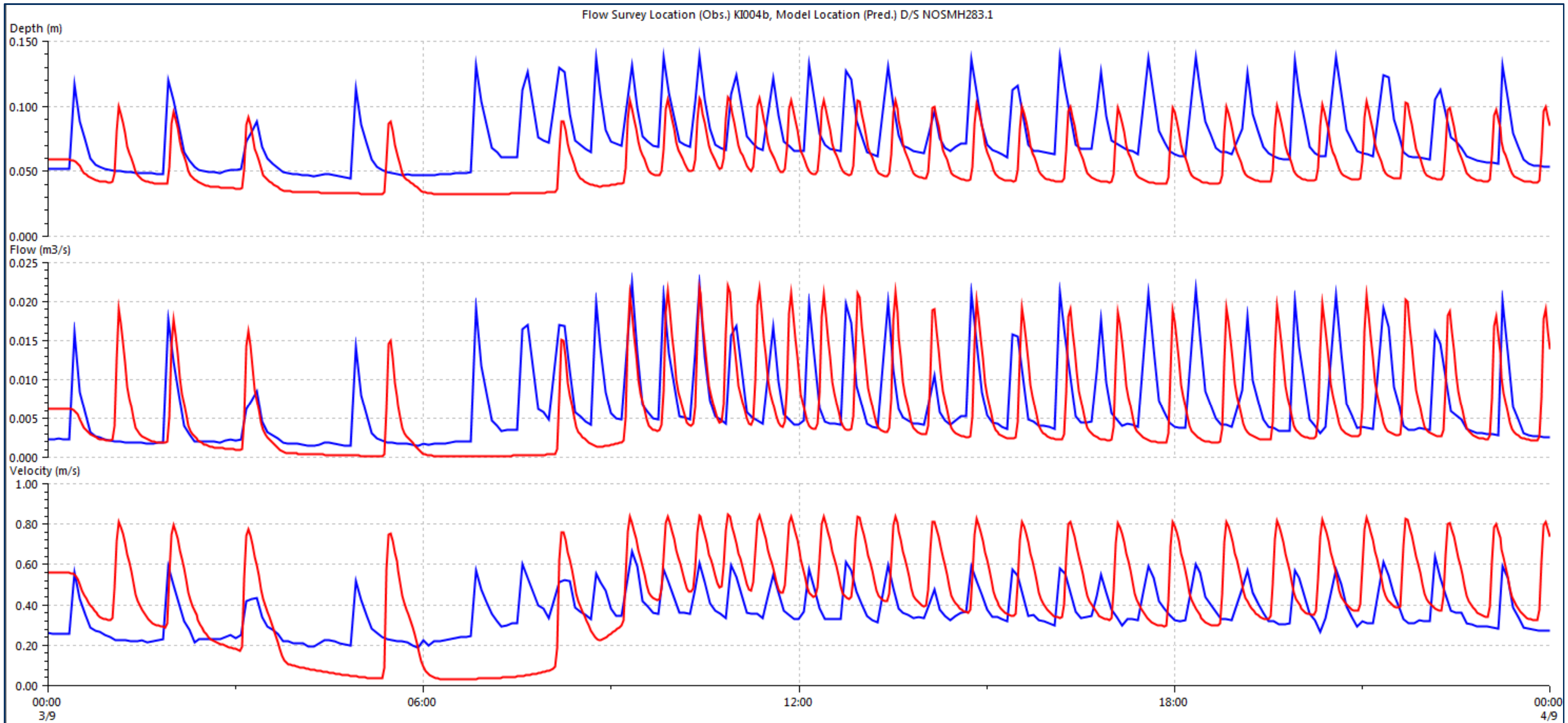
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.056	0.175	0.002	0.028	898.759	0.000	0.554
... consumption 3.1 occ>DWF	0.046	0.112	0.004	0.030	1095.328	0.360	0.824

Figure A12: Dry weather graph for KI-004a_20 on 03/09/2016



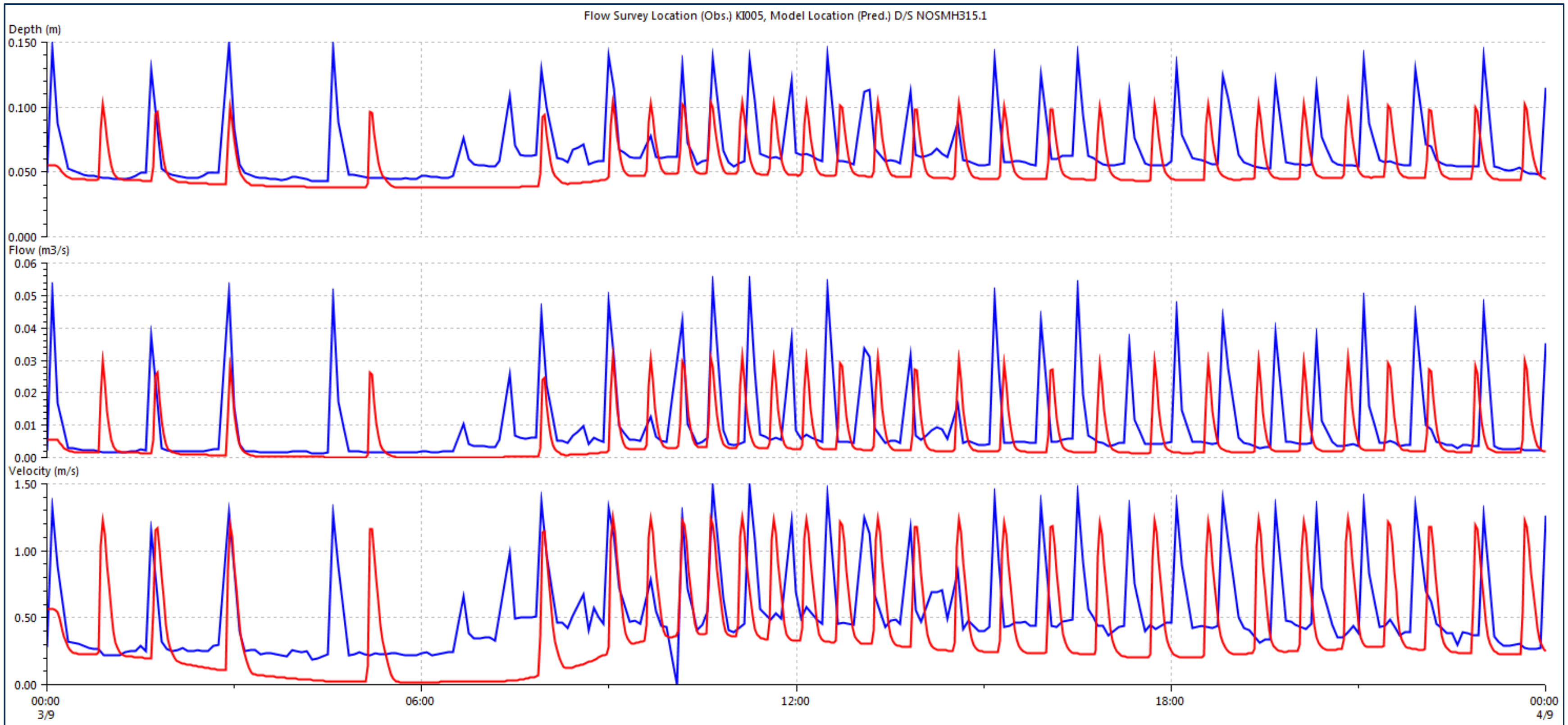
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.033	0.052	0.001	0.003	154.450	0.000	0.419
... consumption 3.1 occ>DWF	0.027	0.042	0.001	0.003	134.191	0.207	0.431

Figure A13: Dry weather graph for KI-004b on 03/09/2016



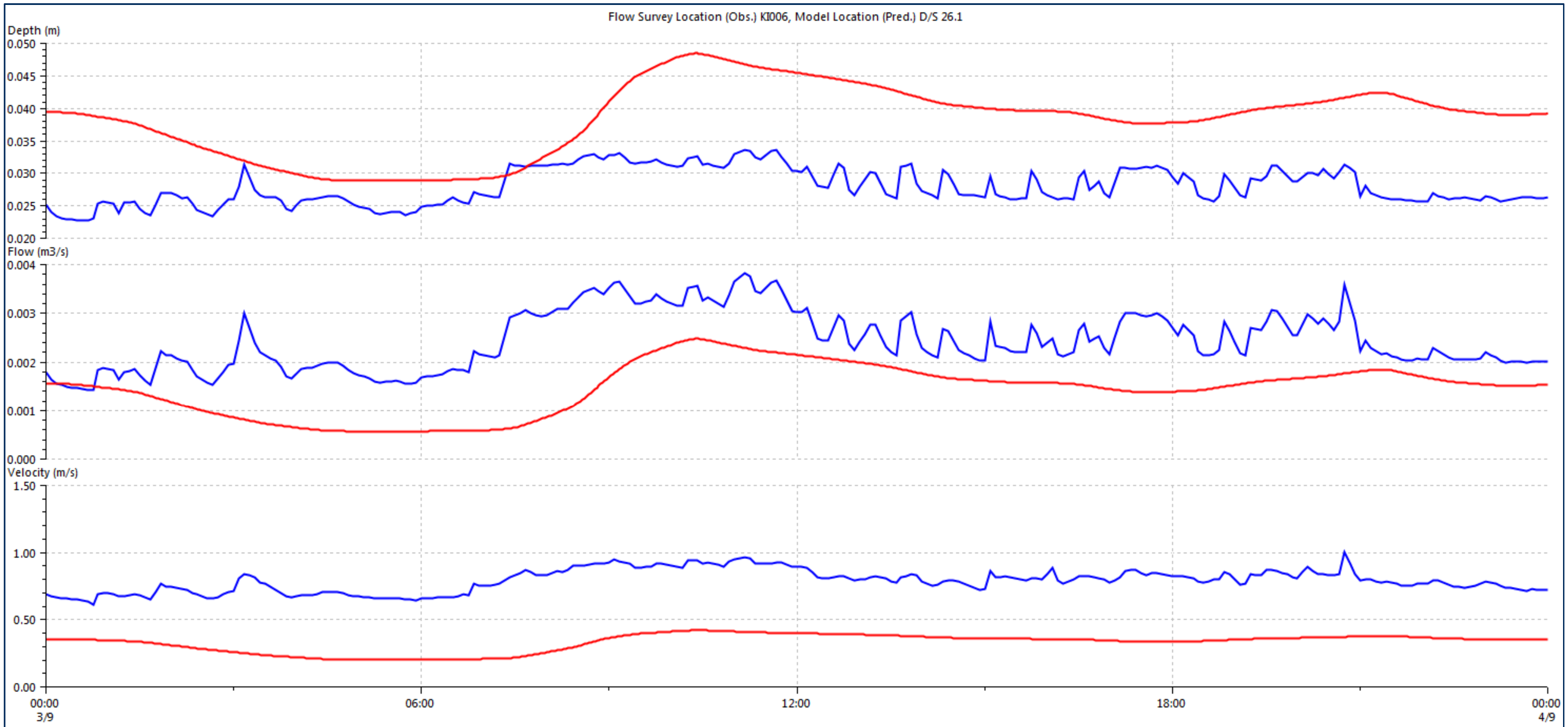
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.045	0.139	0.001	0.022	582.693	0.189	0.662
... consumption 3.1 occ>DWF	0.033	0.107	0.000	0.022	498.044	0.031	0.847

Figure A14: Dry weather graph for KI-005 on 03/09/2016



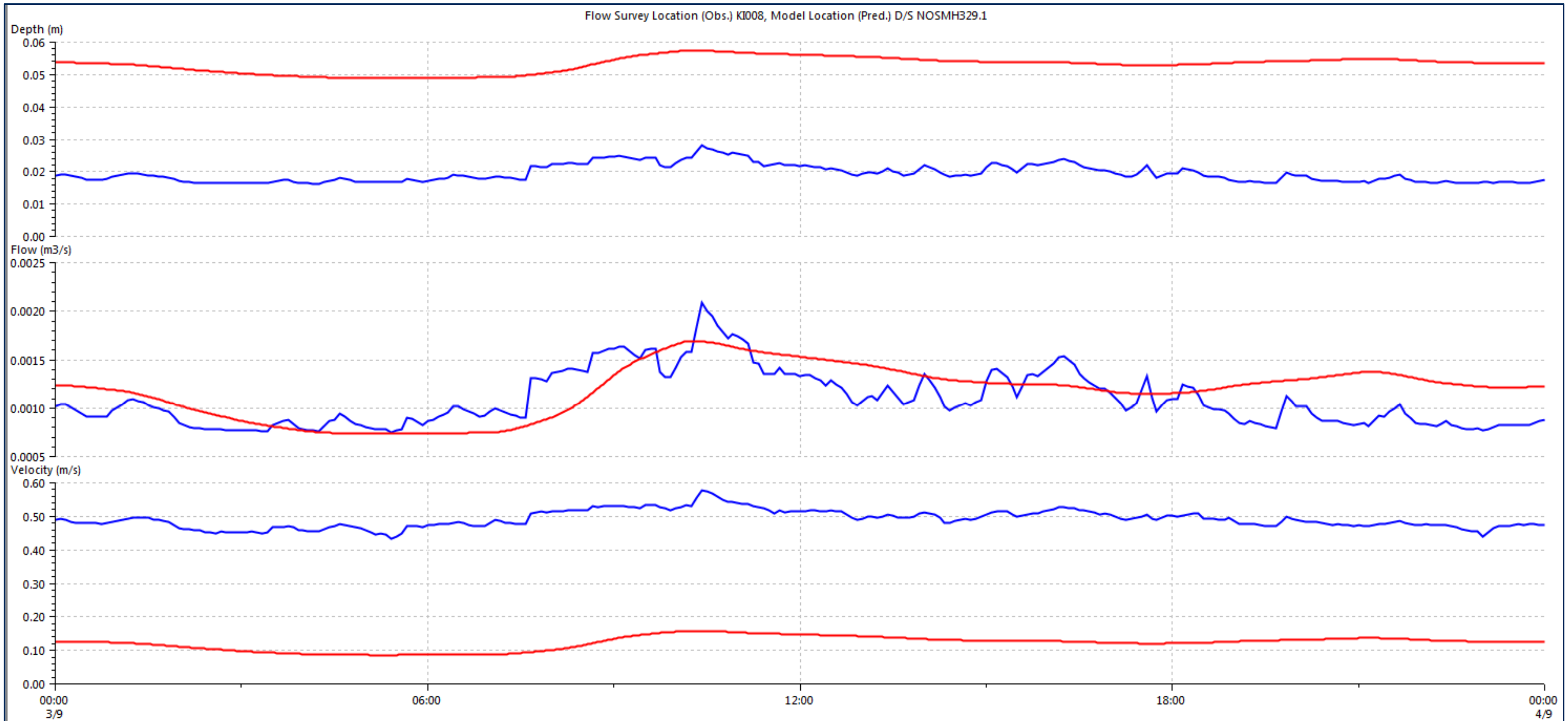
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.043	0.149	0.001	0.053	830.565	0.000	1.480
... consumption 3.1 occ>DWF	0.038	0.104	0.000	0.031	457.950	0.018	1.248

Figure A15: Dry weather graph for KI-006 on 03/09/2016



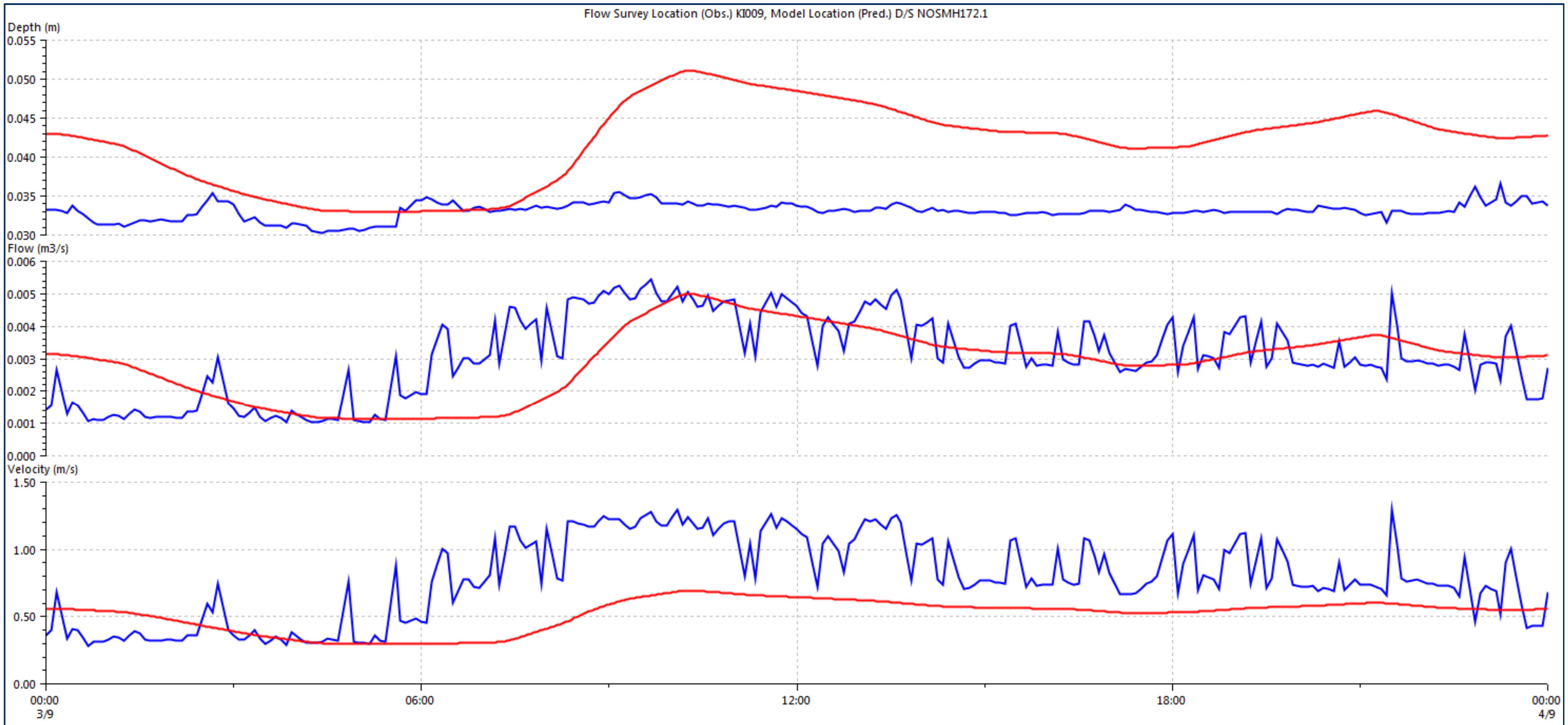
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.023	0.034	0.001	0.004	210.482	0.616	1.008
... consumption 3.1 occ>DWF	0.029	0.048	0.001	0.002	125.414	0.203	0.422

Figure A16: Dry weather graph for KI-008 on 03/09/2016



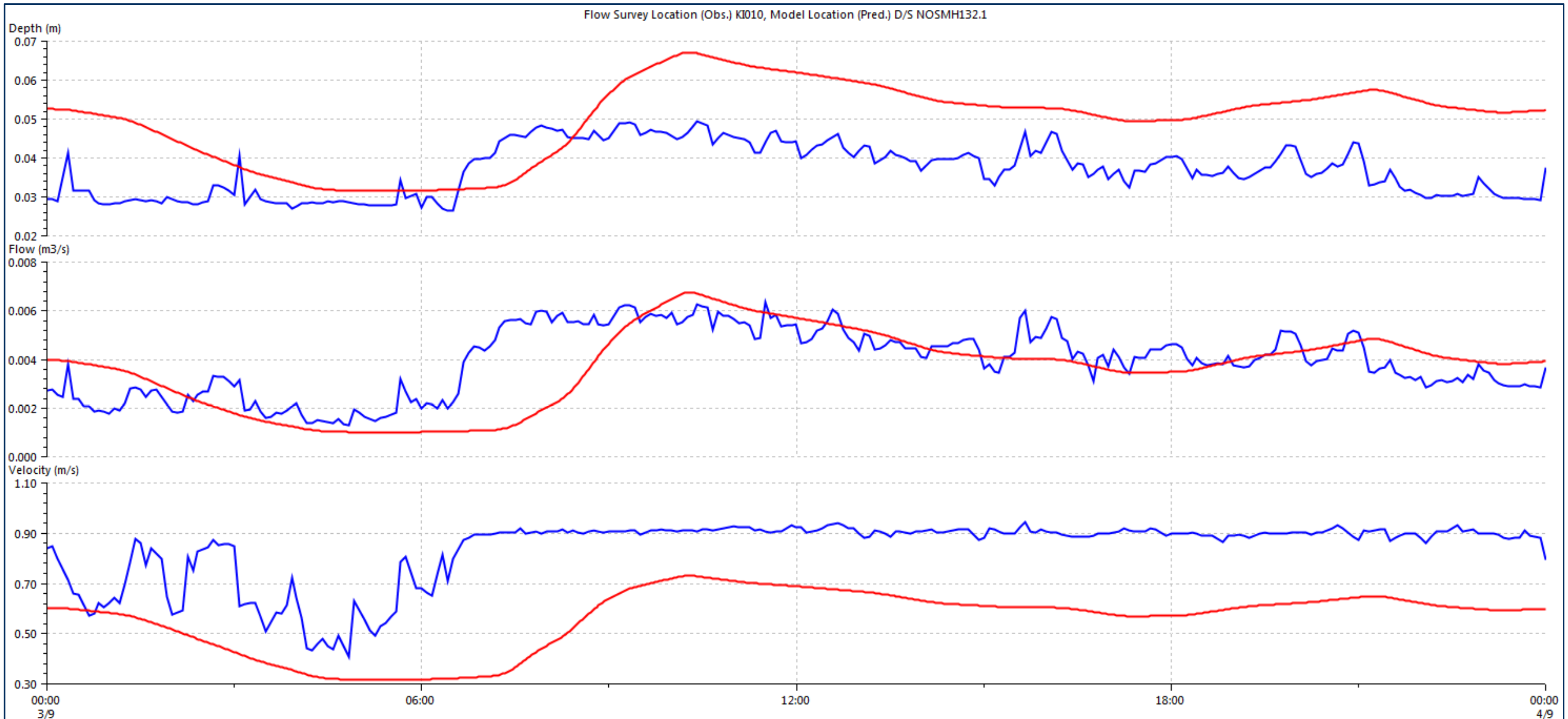
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.016	0.028	0.001	0.002	93.946	0.432	0.579
... consumption 3.1 occ>DWF	0.049	0.057	0.001	0.002	102.178	0.086	0.158

Figure A17: Dry weather graph for KI-009 on 03/09/2016



	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.030	0.037	0.001	0.005	265.798	0.286	1.299
... consumption 3.1 occ>DWF	0.033	0.051	0.001	0.005	252.615	0.297	0.695

Figure A18: Dry weather graph for KI-010 on 03/09/2016



	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.026	0.050	0.001	0.006	339.210	0.409	0.944
... consumption 3.1 occ>DWF	0.032	0.067	0.001	0.007	315.596	0.316	0.730

Figure A19: Dry weather graph showing the comparison of the depth in Janet Avenue PS on 03/09/2016

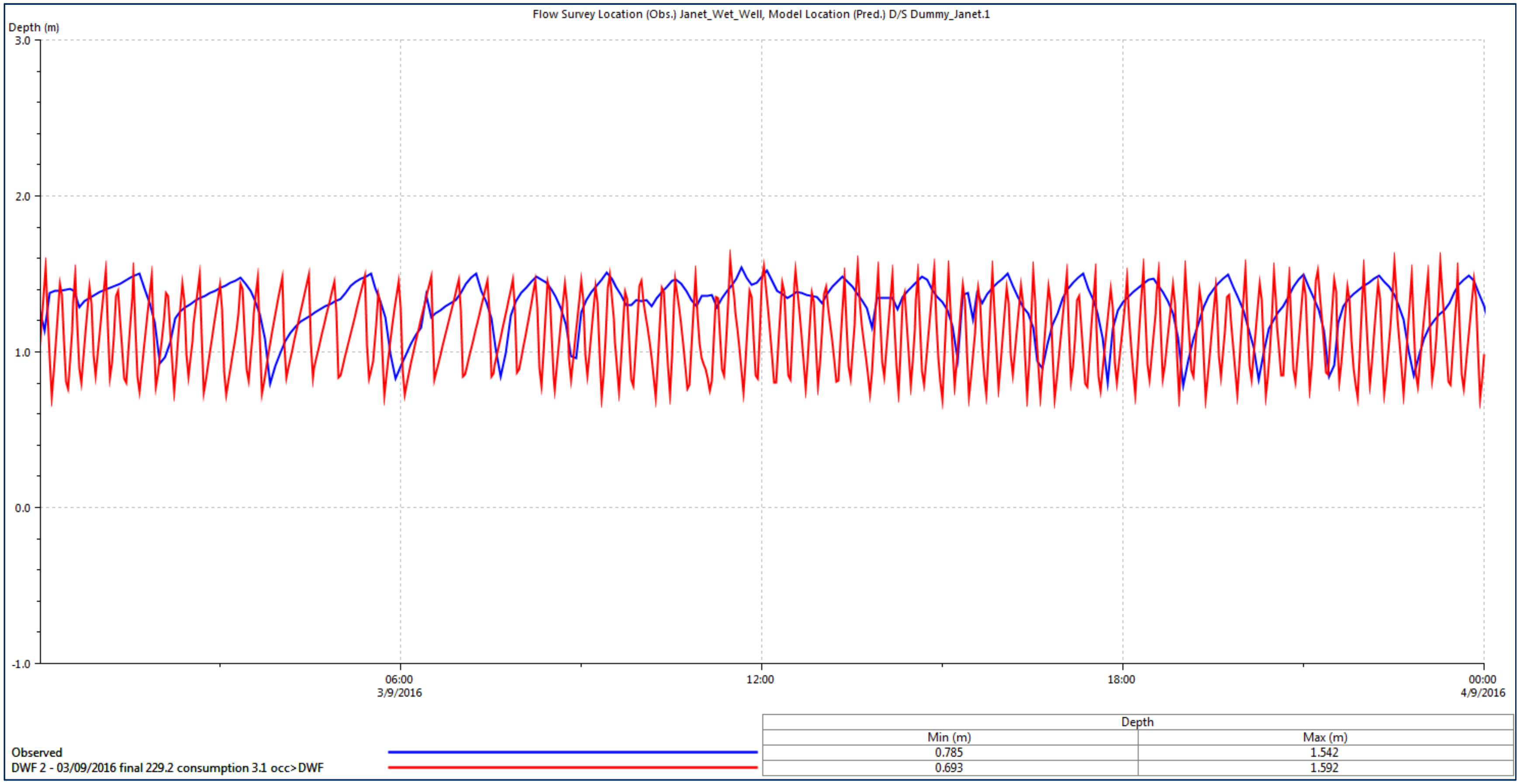


Figure A20: Dry weather graph showing the comparison of the flow along Janet Avenue PS forcemain on 21/07/2016

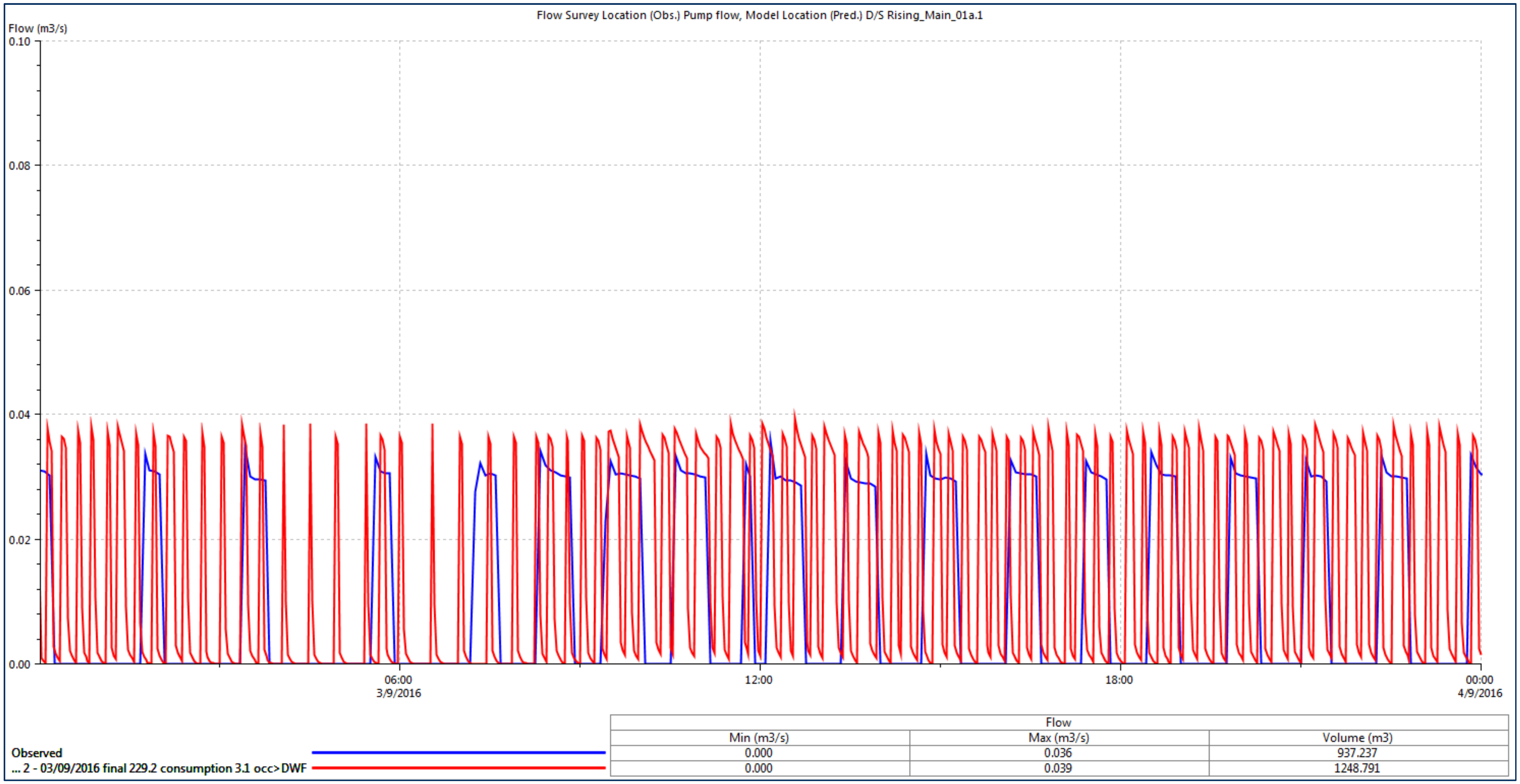
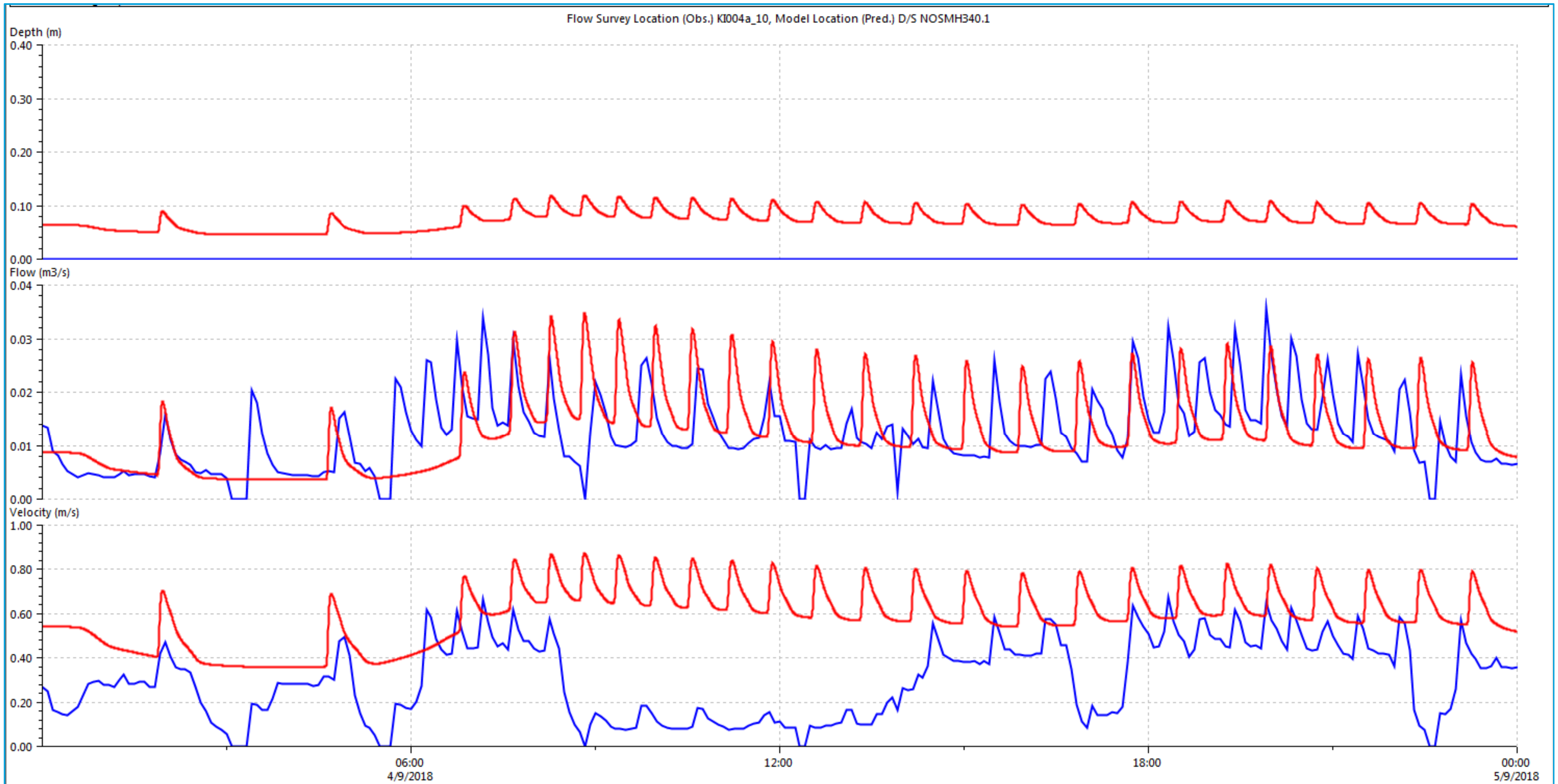
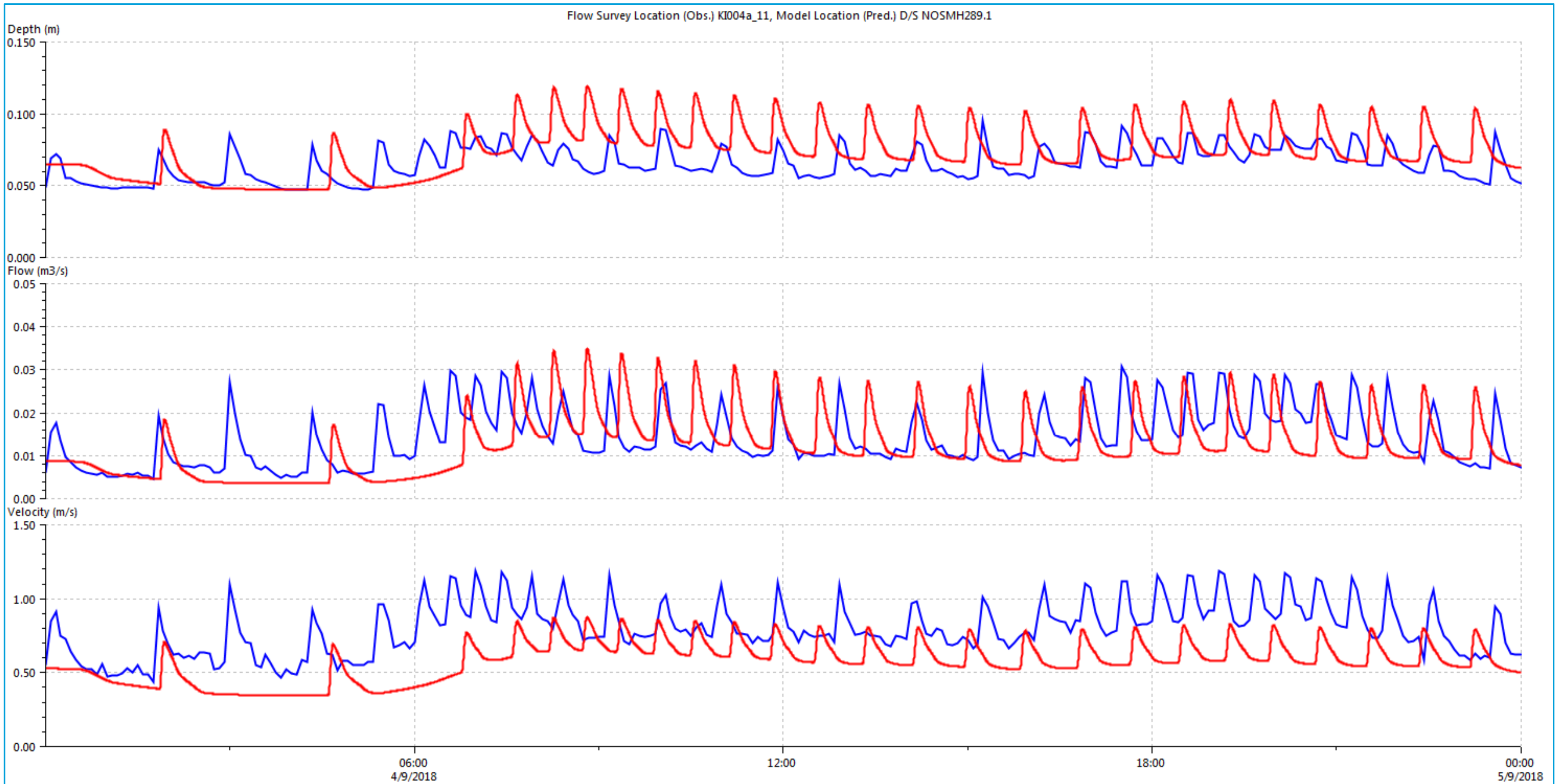


Figure A21: Dry weather graph for KI-004a_10 on 04/09/2018



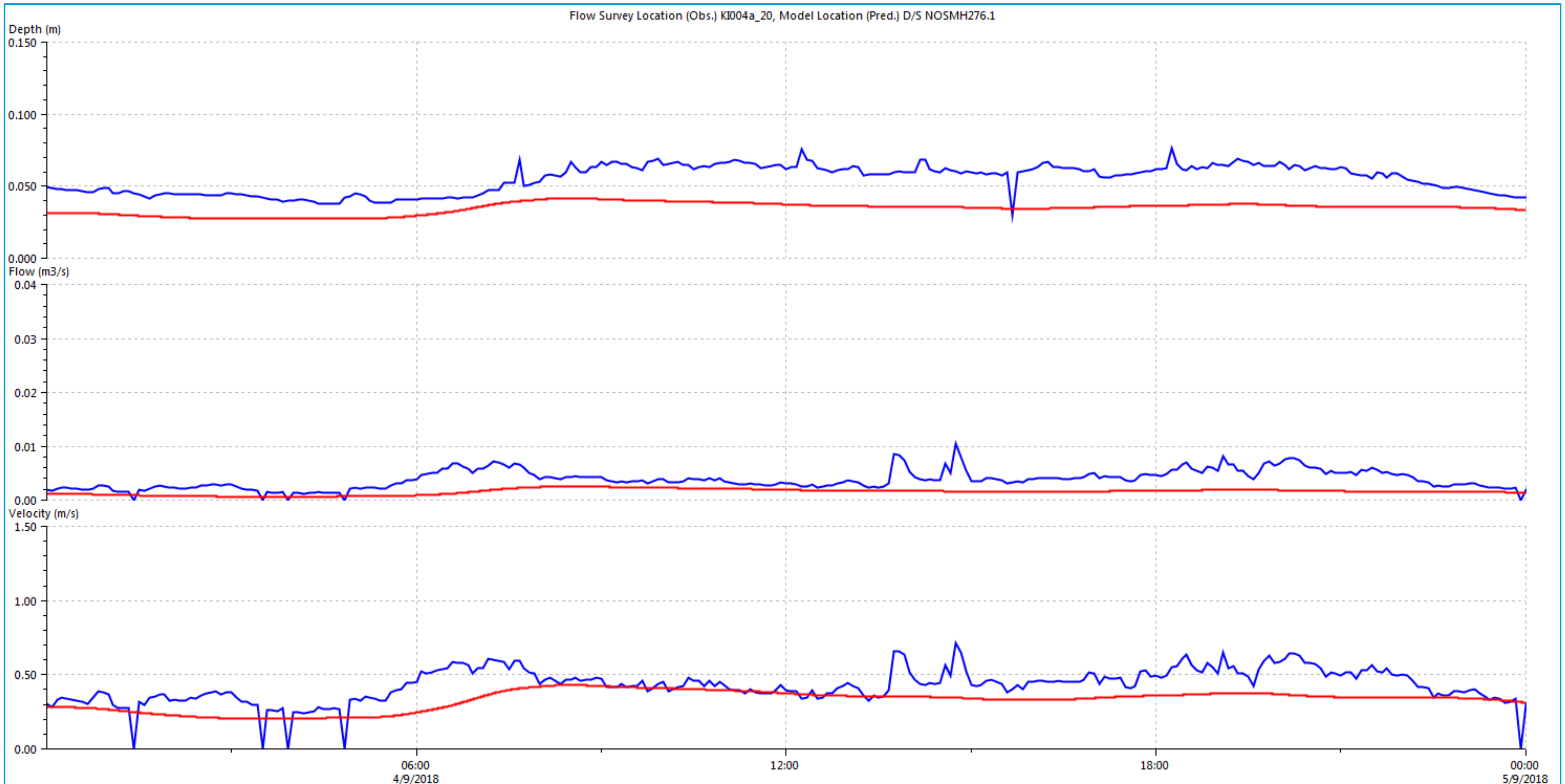
		Depth		Flow			Velocity	
		Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed		0.000	0.000	0.000	0.036	1062.873	0.000	0.674
DWF 1 - 04/09/2018>DWF		0.046	0.119	0.004	0.035	1086.005	0.359	0.874

Figure A22: Dry weather graph for KI-004a_11 on 04/09/2018



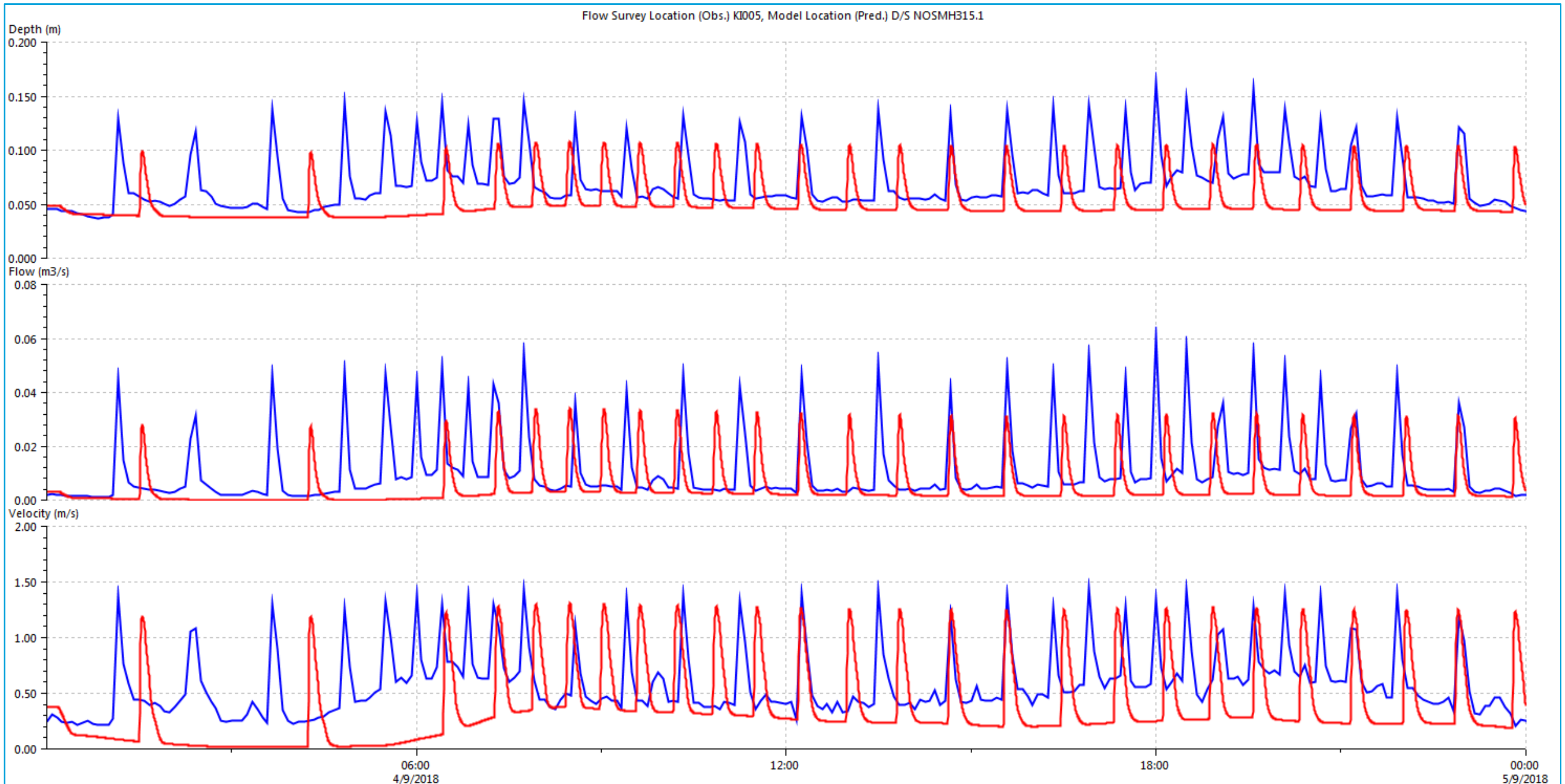
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.047	0.095	0.005	0.031	1244.043	0.436	1.190
DWF 1 - 04/09/2018> DWF	0.048	0.119	0.004	0.035	1085.916	0.347	0.876

Figure A23: Dry weather graph for KI-004a_20 on 04/09/2018



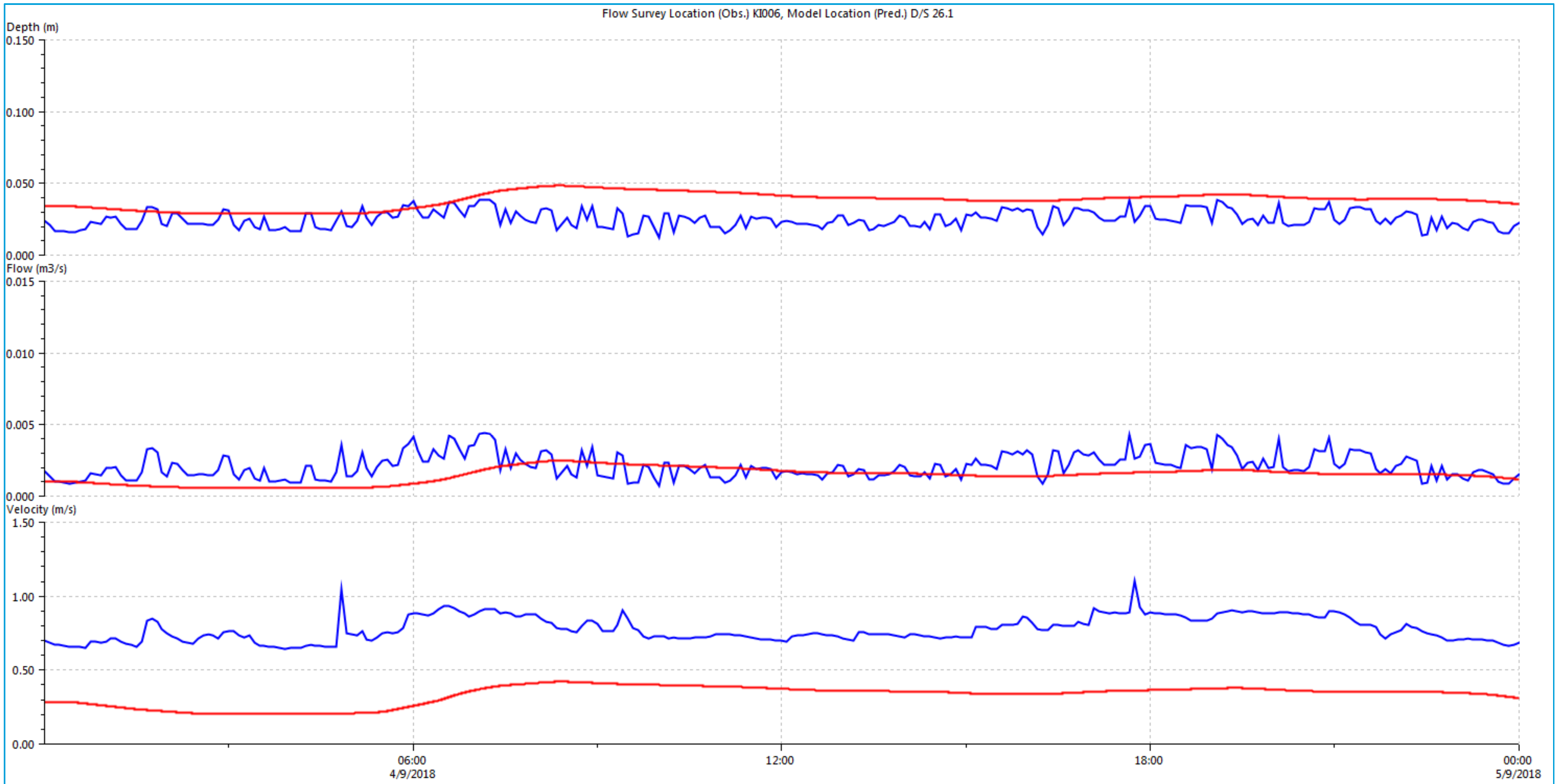
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.030	0.077	0.000	0.010	329.493	0.000	0.712
DWF 1 - 04/09/2018>DWF	0.027	0.042	0.001	0.003	133.920	0.207	0.431

Figure A24: Dry weather graph for KI-005 on 04/09/2018



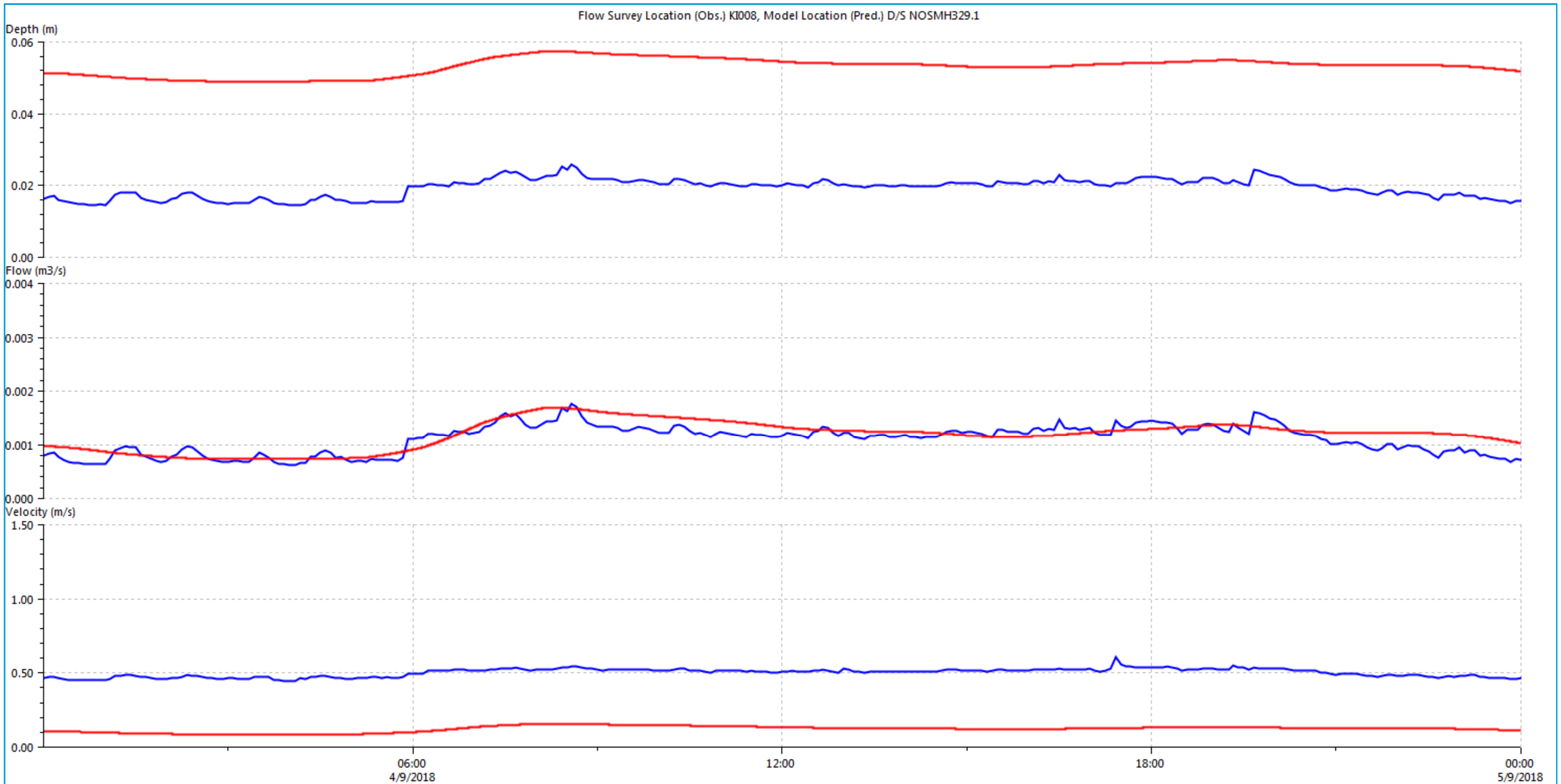
		Depth		Flow			Velocity	
		Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	blue line	0.037	0.163	0.001	0.060	935.948	0.204	1.445
DWF 1 - 04/09/2018>DWF	red line	0.038	0.108	0.000	0.034	448.620	0.019	1.308

Figure A25: Dry weather graph for KI-006 on 04/09/2018



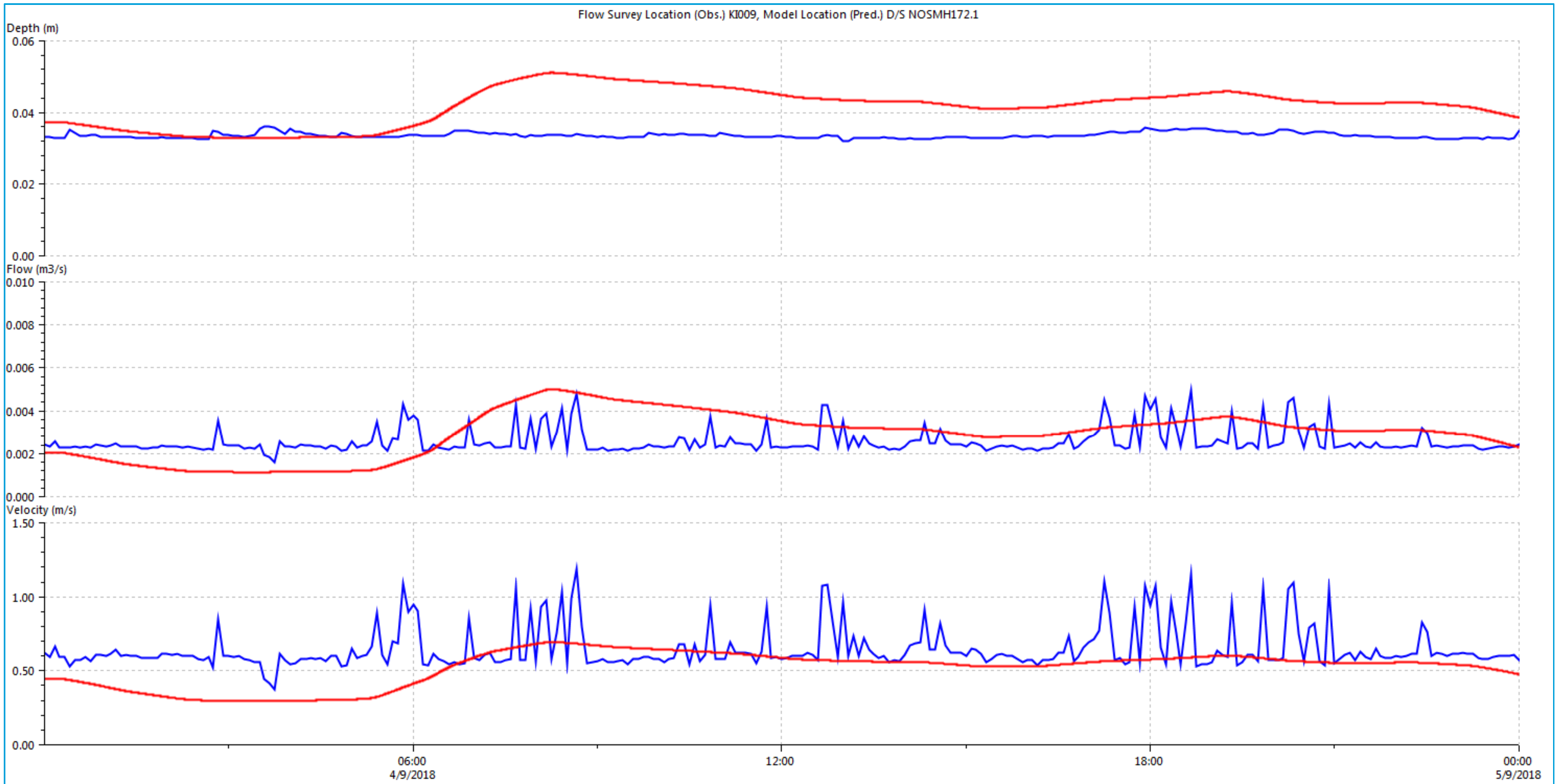
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.013	0.039	0.001	0.004	180.925	0.645	1.107
DWF 1 - 04/09/2018> DWF	0.029	0.048	0.001	0.002	125.255	0.203	0.422

Figure A26: Dry weather graph for KI-008 on 04/09/2018



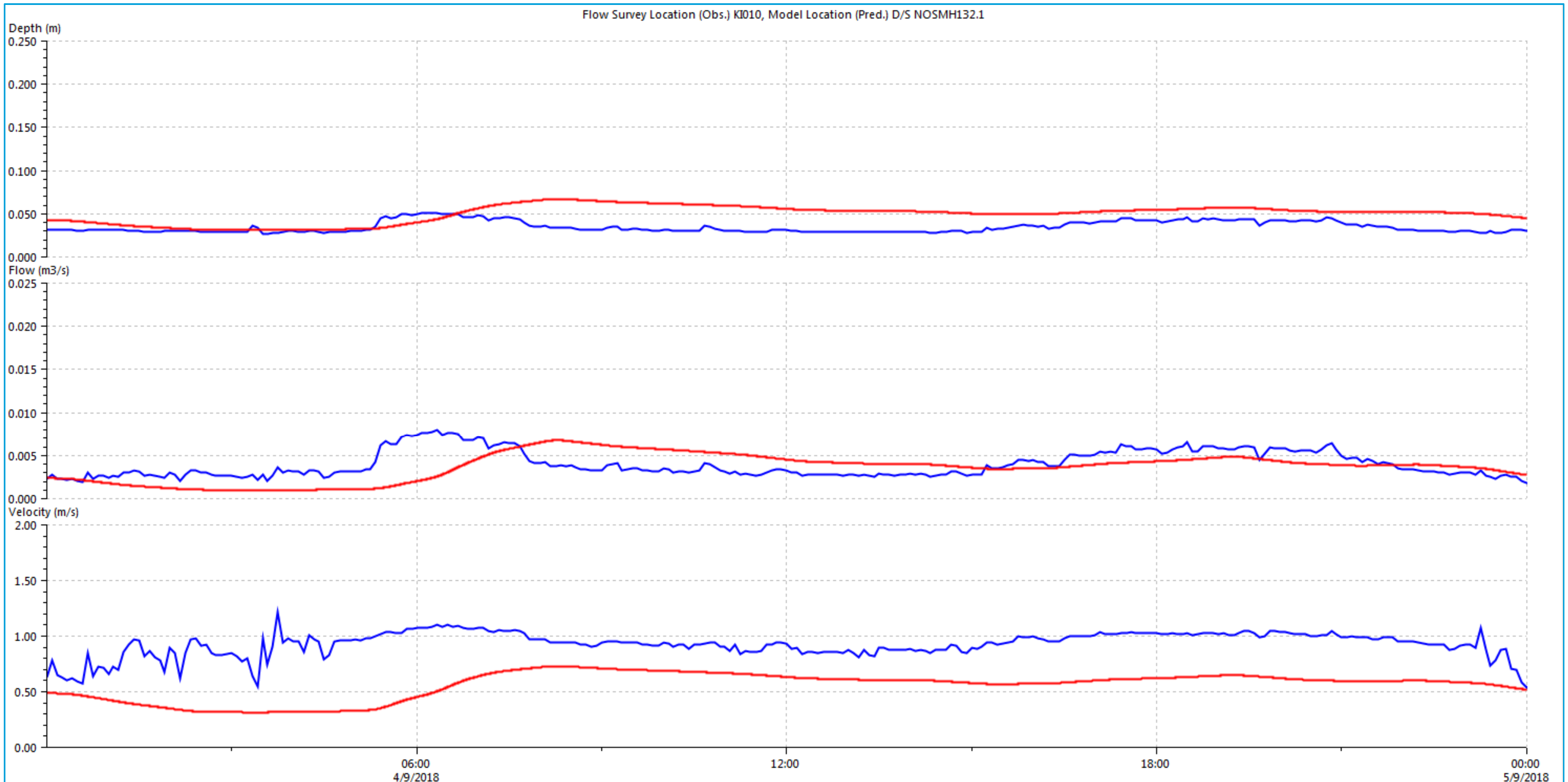
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.015	0.026	0.001	0.002	95.276	0.446	0.606
DWF 1 - 04/09/2018>DWF	0.049	0.057	0.001	0.002	102.104	0.086	0.158

Figure A27: Dry weather graph for KI-009 on 04/09/2018



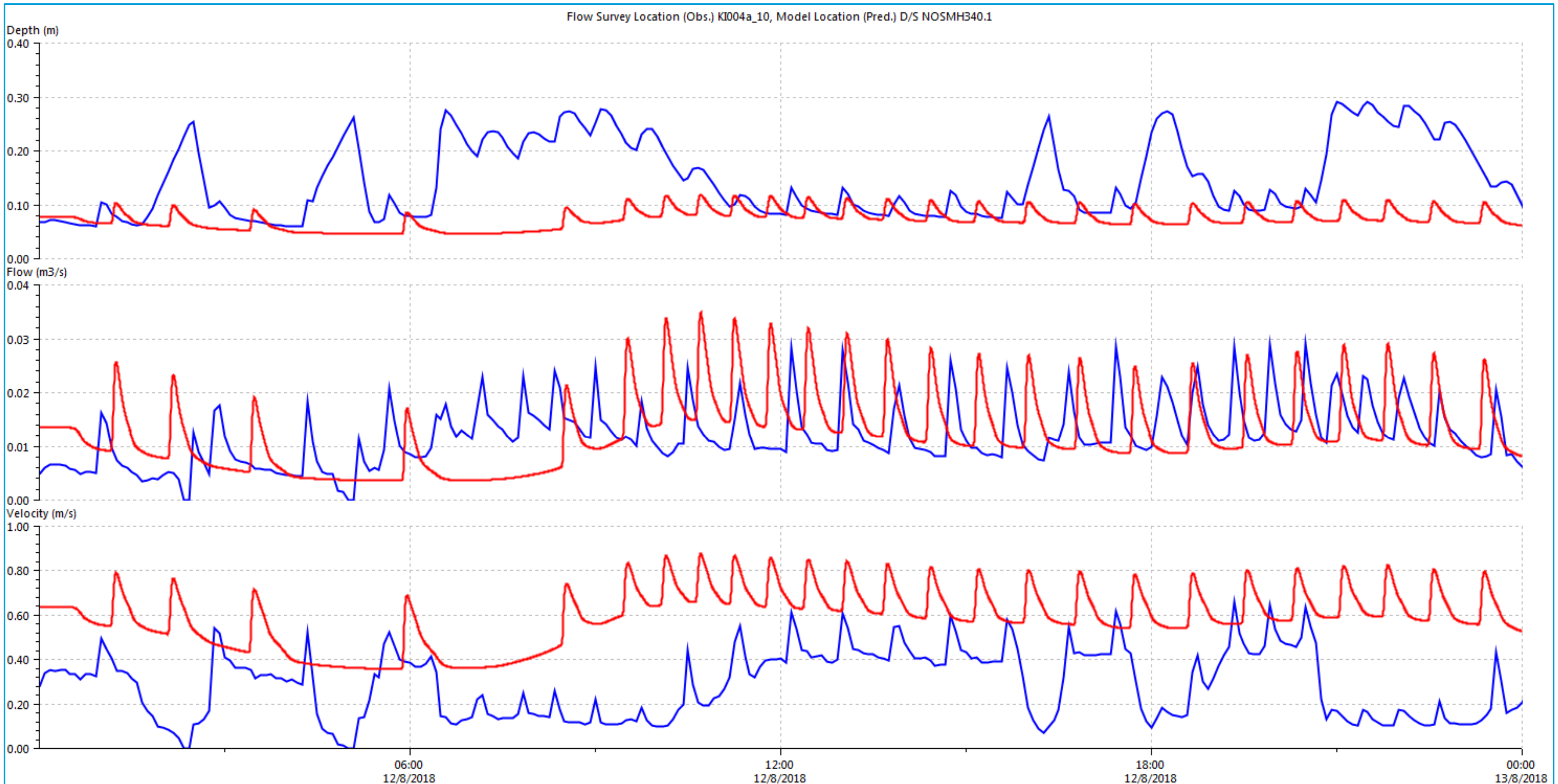
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.032	0.036	0.002	0.005	222.236	0.373	1.187
DWF 1 - 04/09/2018>DWF	0.033	0.051	0.001	0.005	252.398	0.297	0.695

Figure A28: Dry weather graph for KI-010 on 04/09/2018



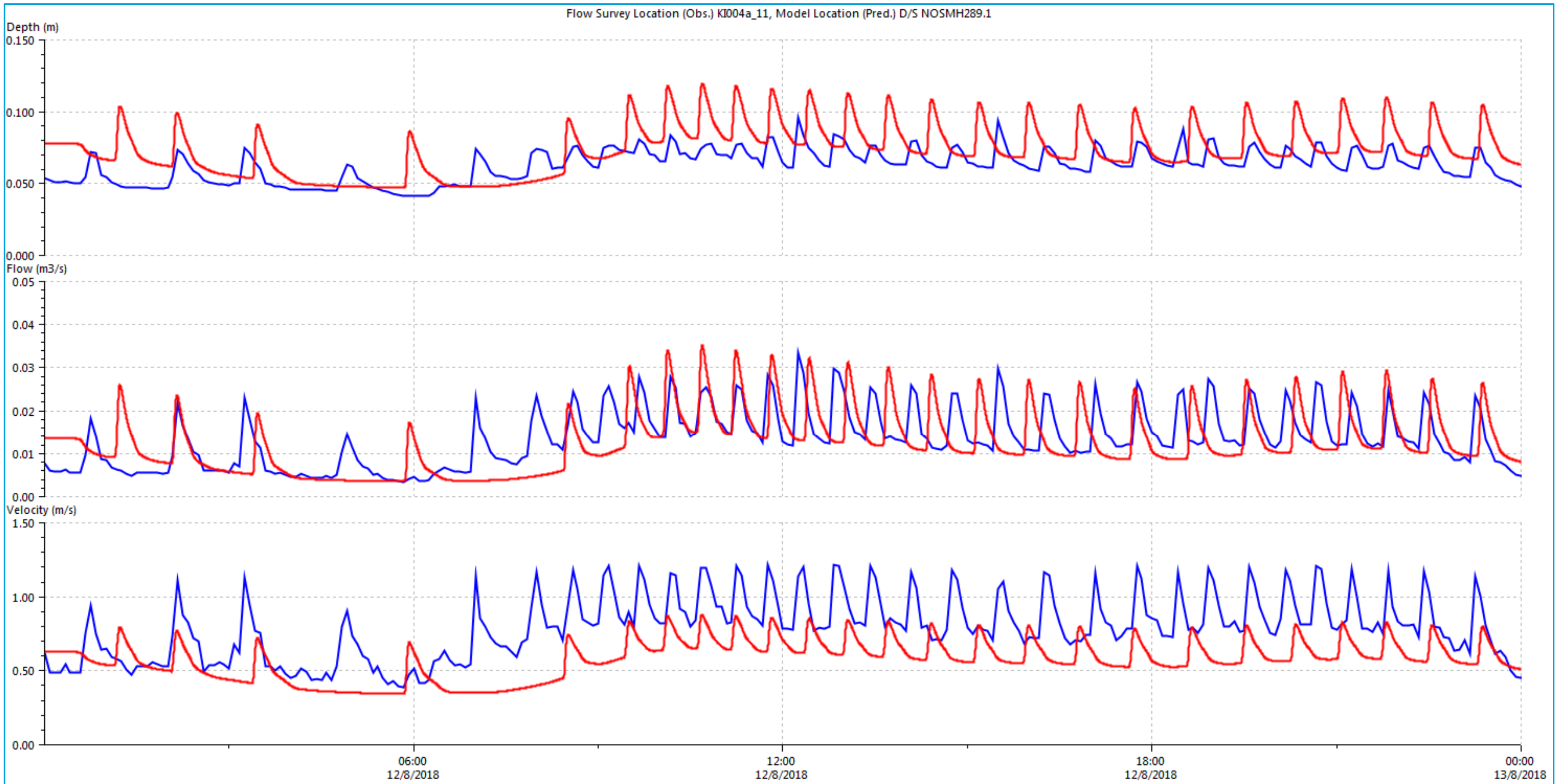
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.026	0.051	0.002	0.008	345.196	0.542	1.218
DWF 1 - 04/09/2018>DWF	0.032	0.067	0.001	0.007	315.292	0.316	0.731

Figure A29: Dry weather graph for KI-004a_10 on 12/08/2018



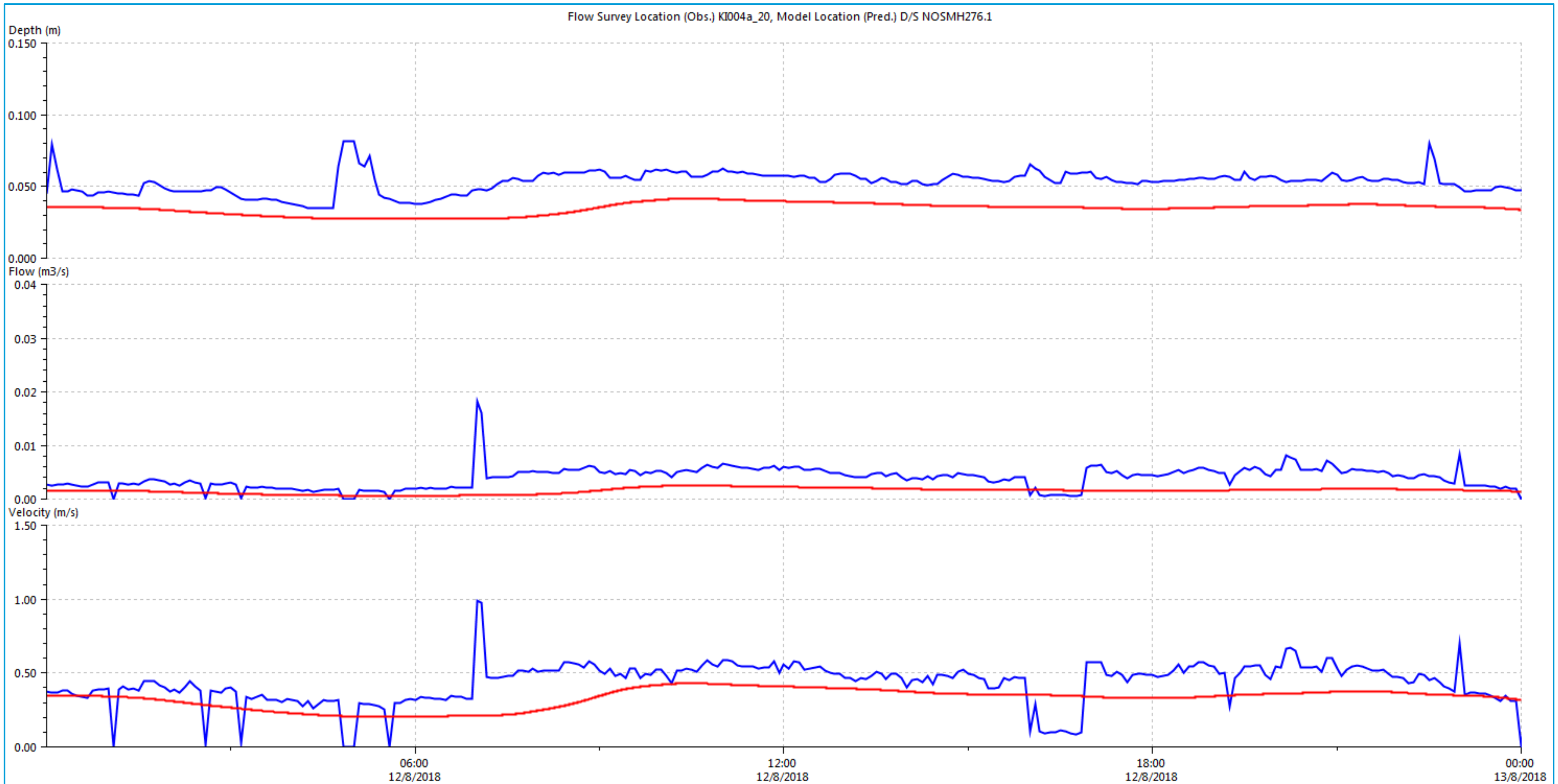
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.060	0.291	0.000	0.029	1043.732	0.000	0.661
DWF 1 - 12/08/2018>DWF	0.046	0.119	0.004	0.035	1090.568	0.359	0.875

Figure A30: Dry weather graph for KI-004a_11 on 12/08/2018



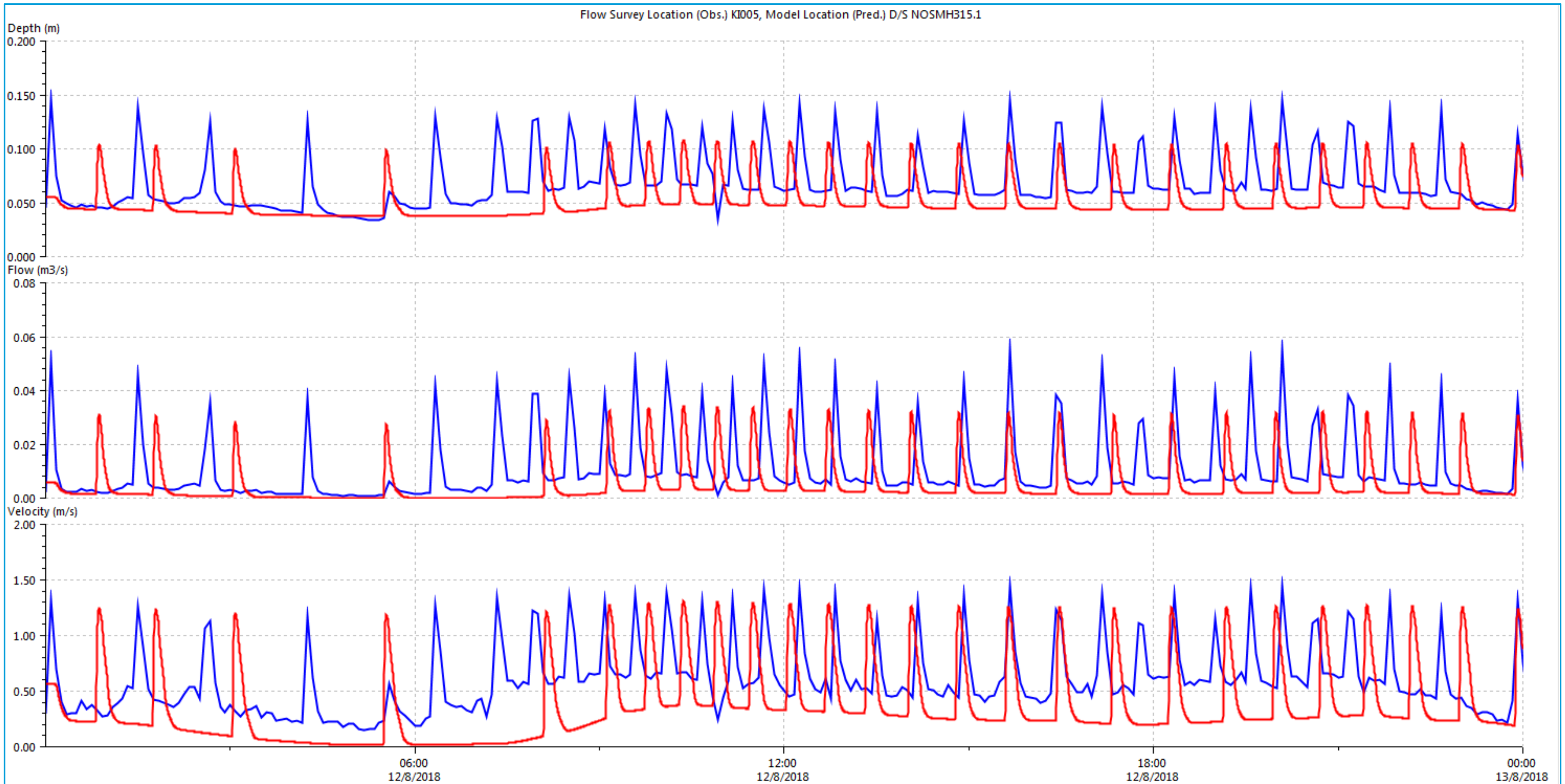
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.041	0.096	0.003	0.033	1175.312	0.386	1.220
DWF 1 - 12/08/2018>DWF	0.048	0.120	0.004	0.035	1090.192	0.347	0.878

Figure A31: Dry weather graph for KI-004a_20 on 12/08/2018



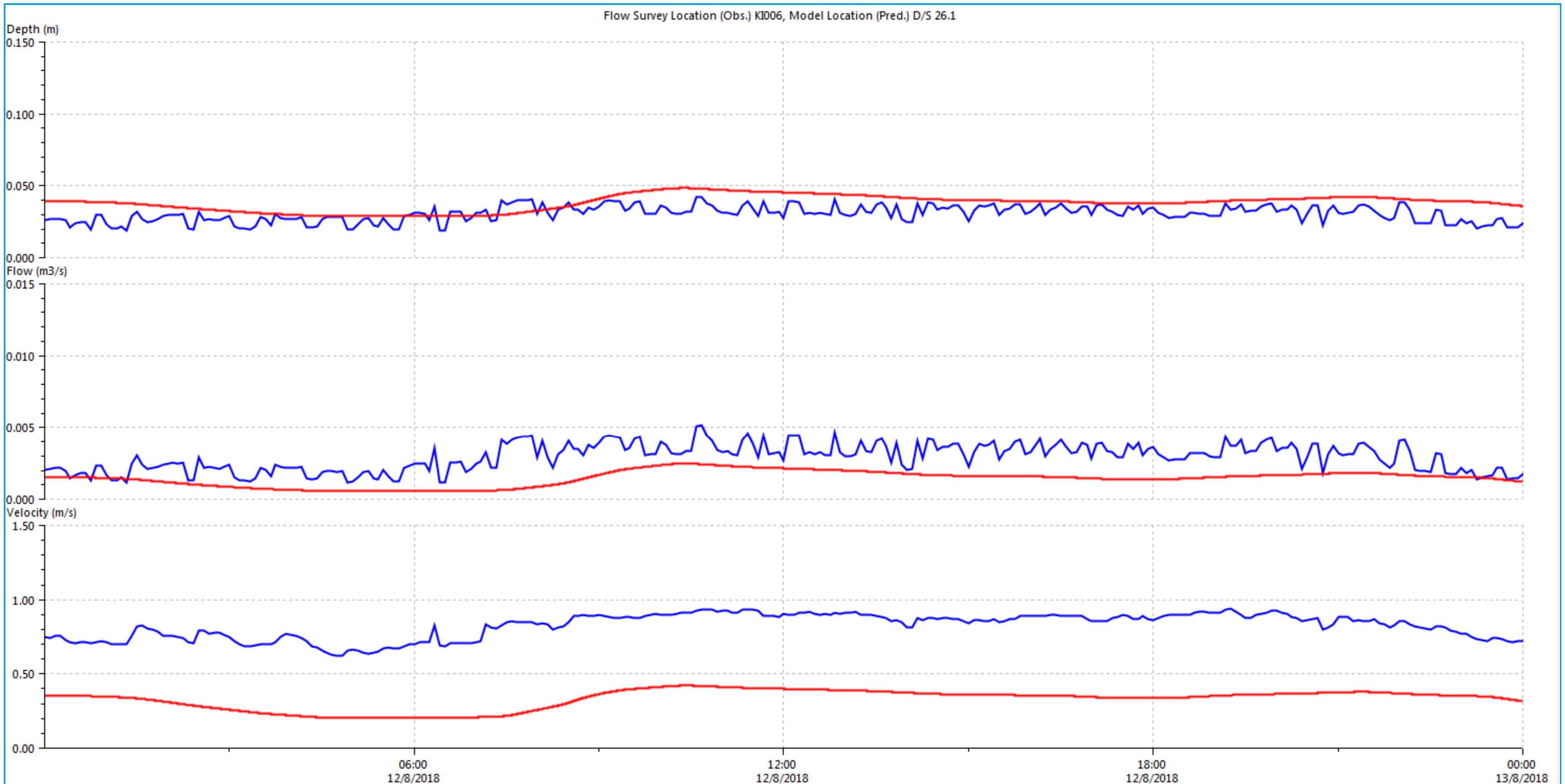
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.035	0.082	0.000	0.018	345.631	0.000	0.990
DWF 1 - 12/08/2018>DWF	0.027	0.042	0.001	0.003	133.939	0.207	0.431

Figure A32: Dry weather graph for KI-005 on 12/08/2018



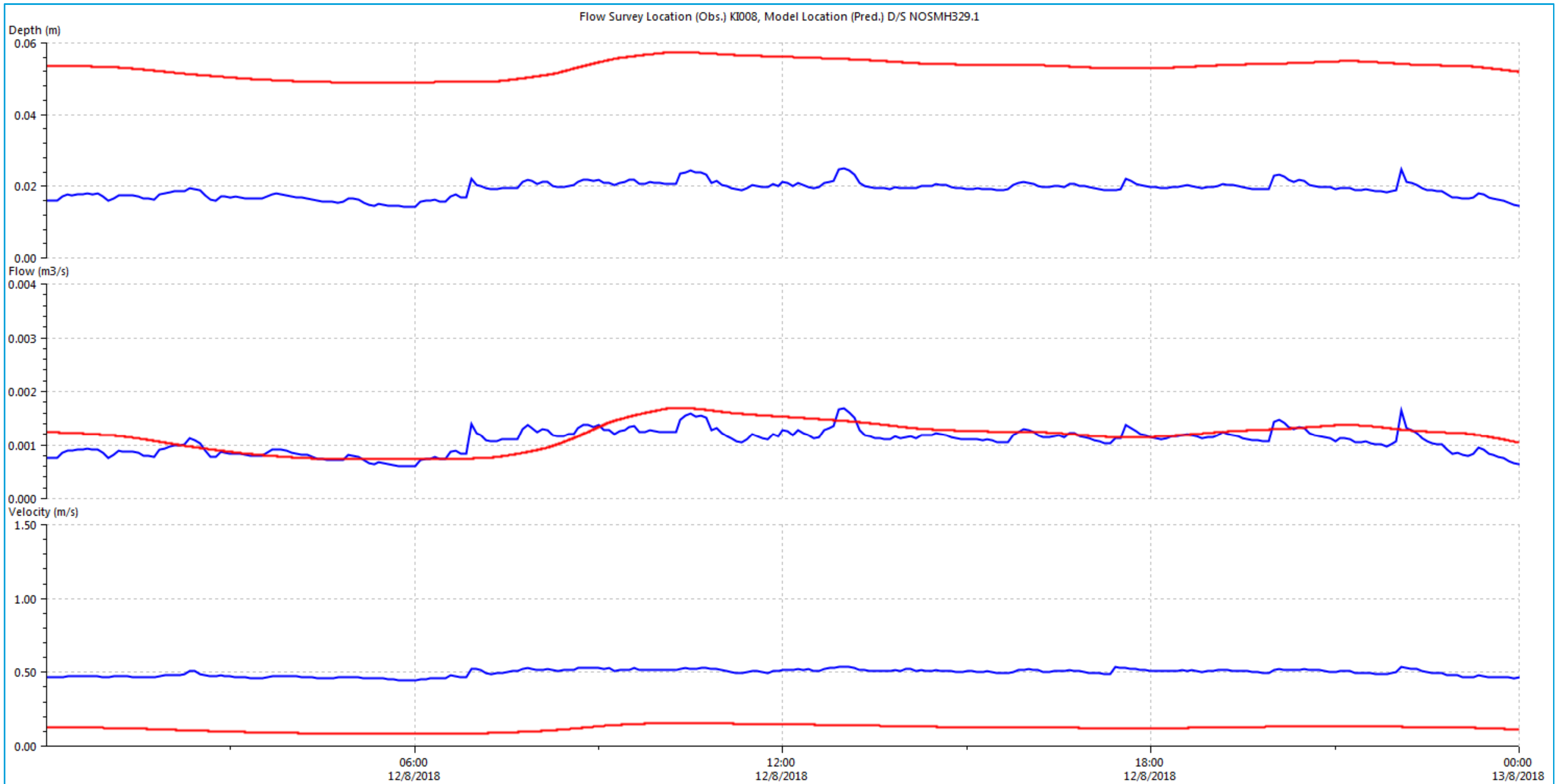
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.034	0.146	0.001	0.055	935.367	0.155	1.444
DWF 1 - 12/08/2018>DWF	0.038	0.108	0.000	0.034	448.010	0.019	1.309

Figure A33: Dry weather graph for KI-006 on 12/08/2018



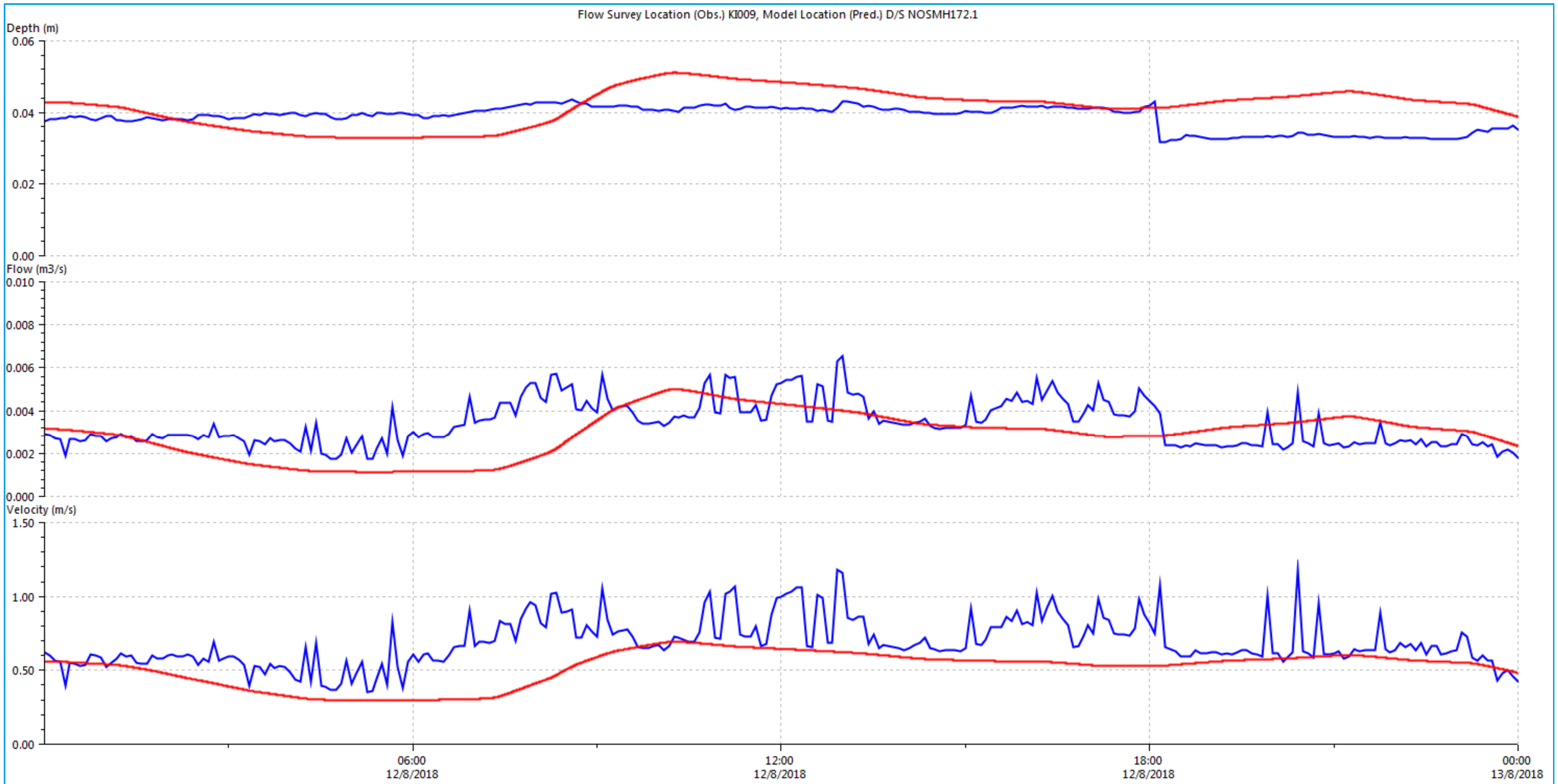
		Depth		Flow			Velocity	
		Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed		0.019	0.042	0.001	0.005	252.556	0.622	0.941
DWF 1 - 12/08/2018>DWF		0.029	0.048	0.001	0.002	125.016	0.203	0.422

Figure A34: Dry weather graph for KI-008 on 12/08/2018



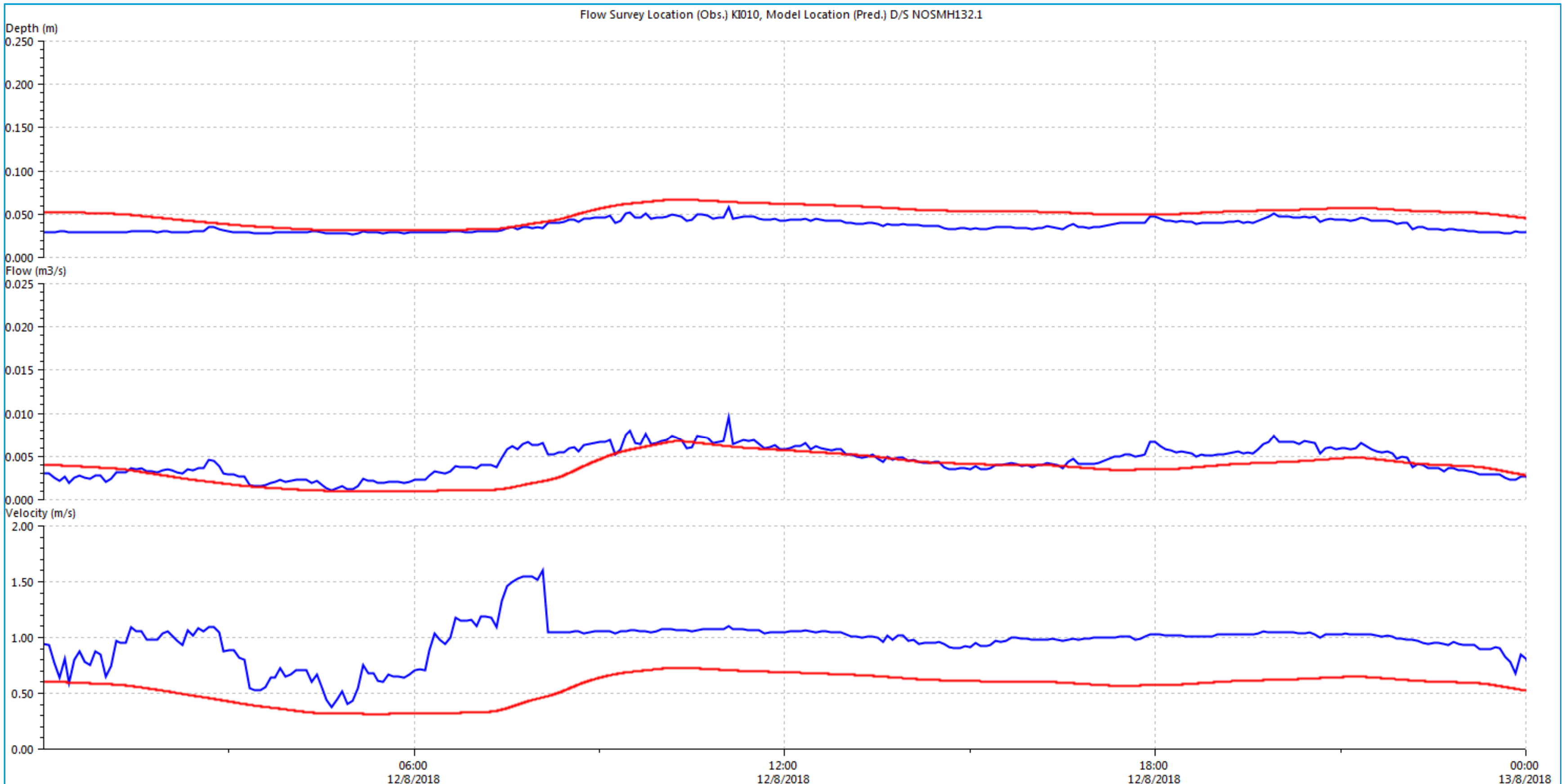
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.014	0.025	0.001	0.002	92.805	0.446	0.541
DWF 1 - 12/08/2018>DWF	0.049	0.057	0.001	0.002	101.916	0.086	0.158

Figure A35: Dry weather graph for KI-009 on 12/08/2018



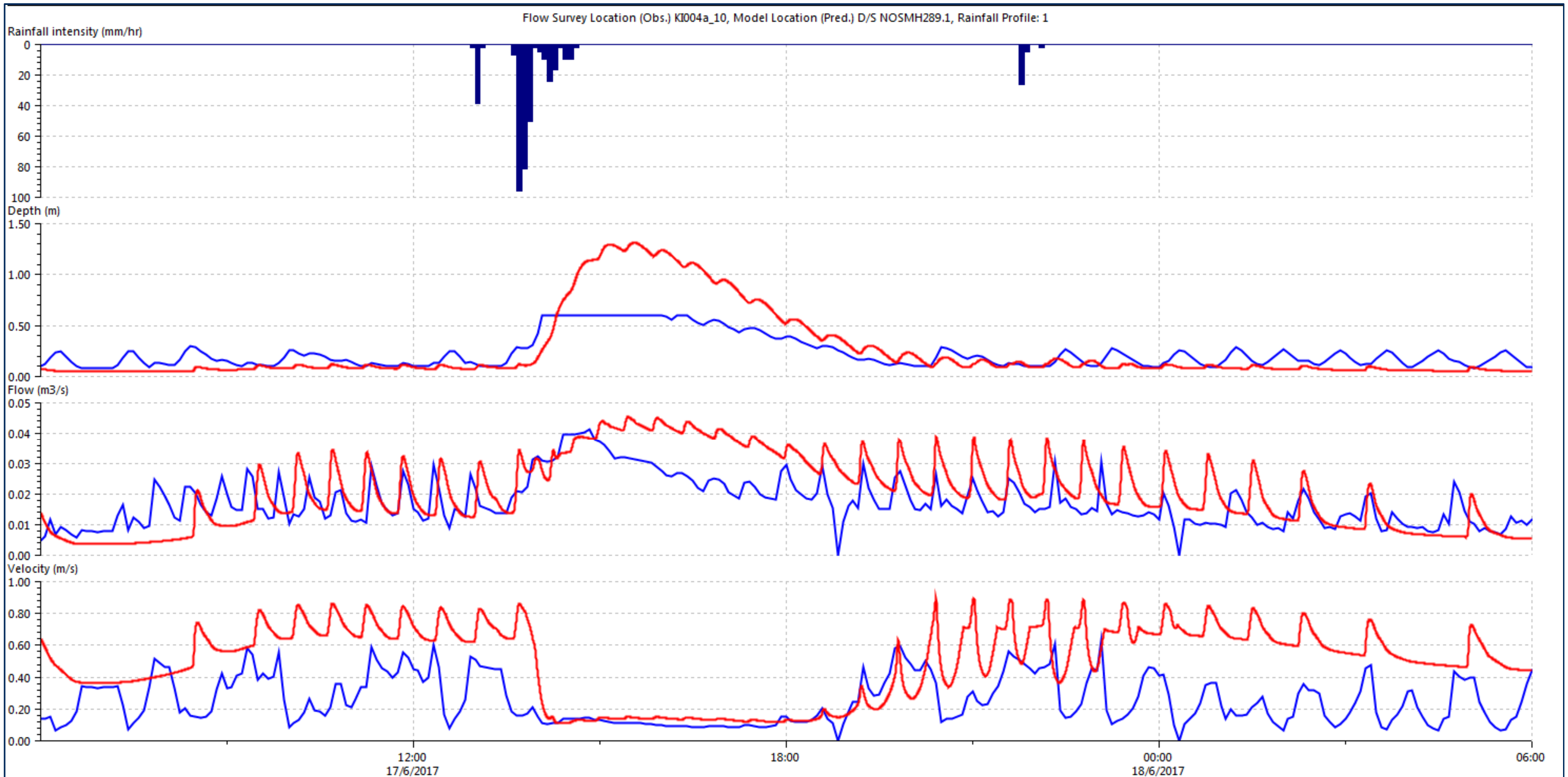
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.032	0.044	0.002	0.007	291.049	0.355	1.182
DWF 1 - 12/08/2018>DWF	0.033	0.051	0.001	0.005	251.569	0.297	0.695

Figure A36: Dry weather graph for KI-010 on 12/08/2018



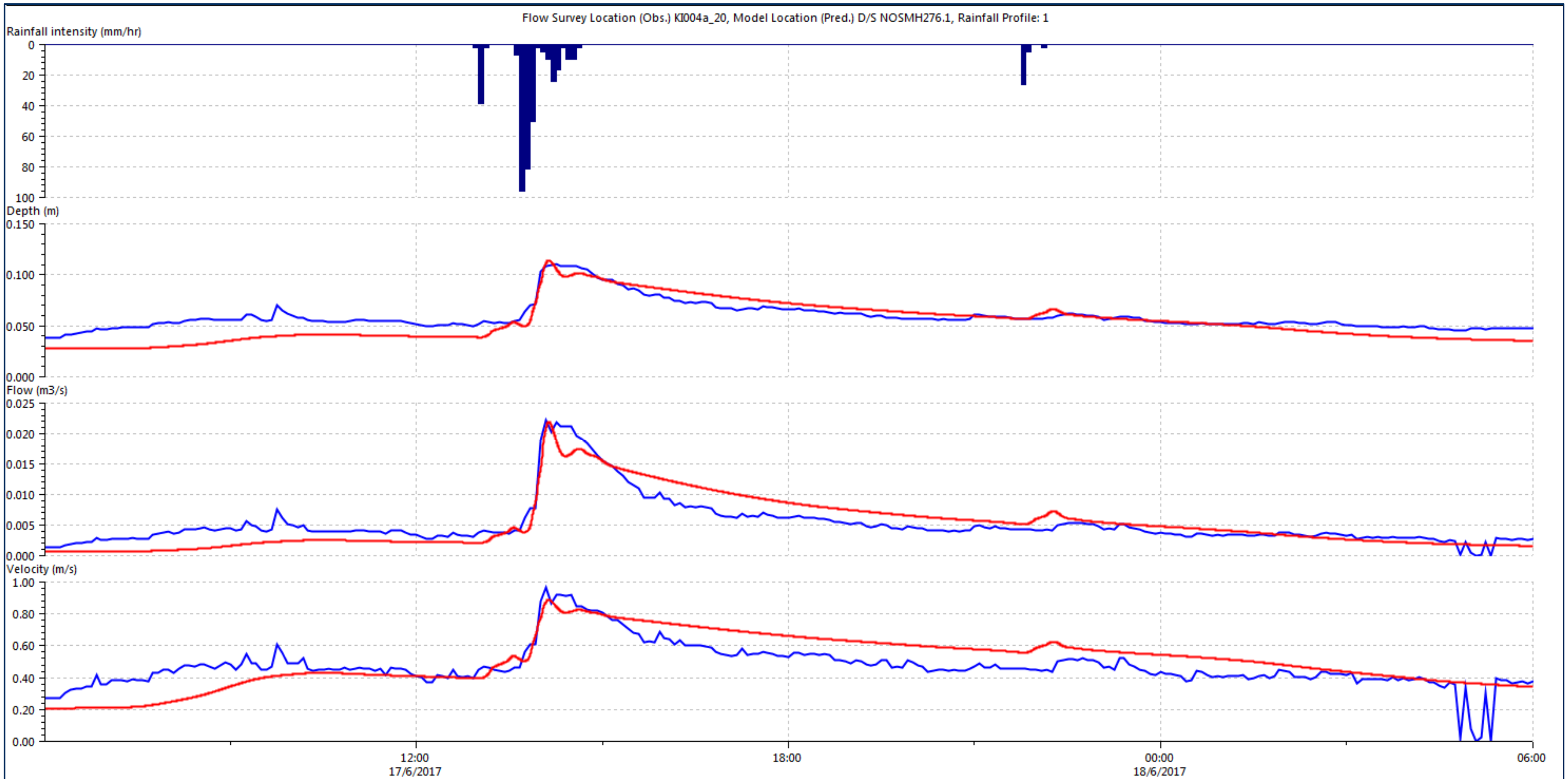
	Depth		Flow			Velocity	
	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Observed	0.027	0.058	0.001	0.010	388.737	0.381	1.603
DWF 1 - 12/08/2018>DWF	0.032	0.067	0.001	0.007	314.087	0.316	0.731

Figure A37: Storm event for KI-004a_10 on 17/06/2017



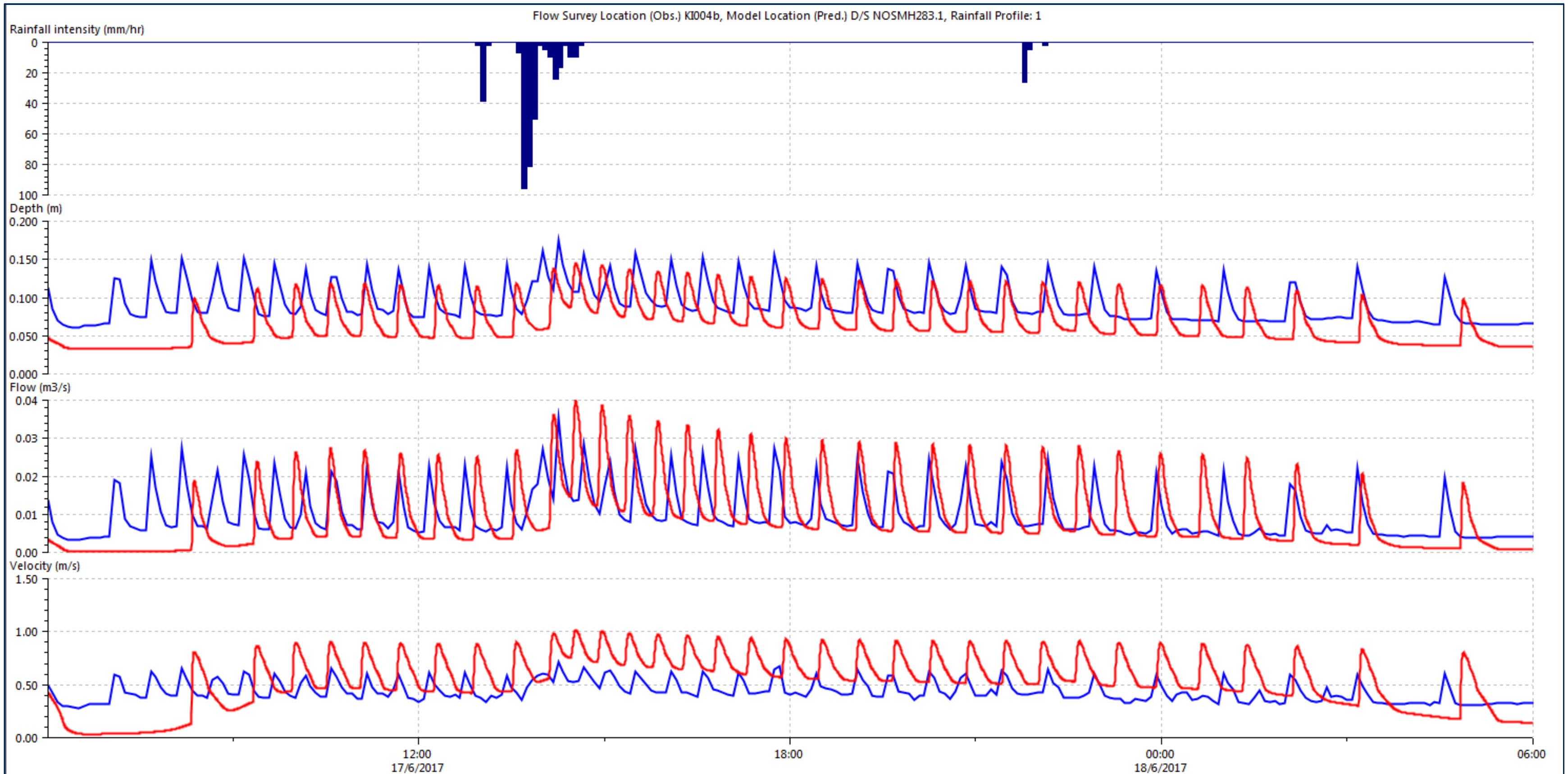
	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	32.800	96.000	1.367							
Observed				0.082	0.600	0.000	0.042	1508.096	0.000	0.633
...e>Storm 17/06/2017				0.047	1.314	0.004	0.045	1814.390	0.110	0.894

Figure A38: Storm event for KI-004a_20 on 17/06/2017



	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	32.800	96.000	1.367							
Observed				0.038	0.111	0.000	0.022	444.762	0.000	0.964
...e> Storm 17/06/2017				0.027	0.114	0.001	0.022	444.293	0.208	0.888

Figure A39: Storm event for KI-004b on 17/06/2017

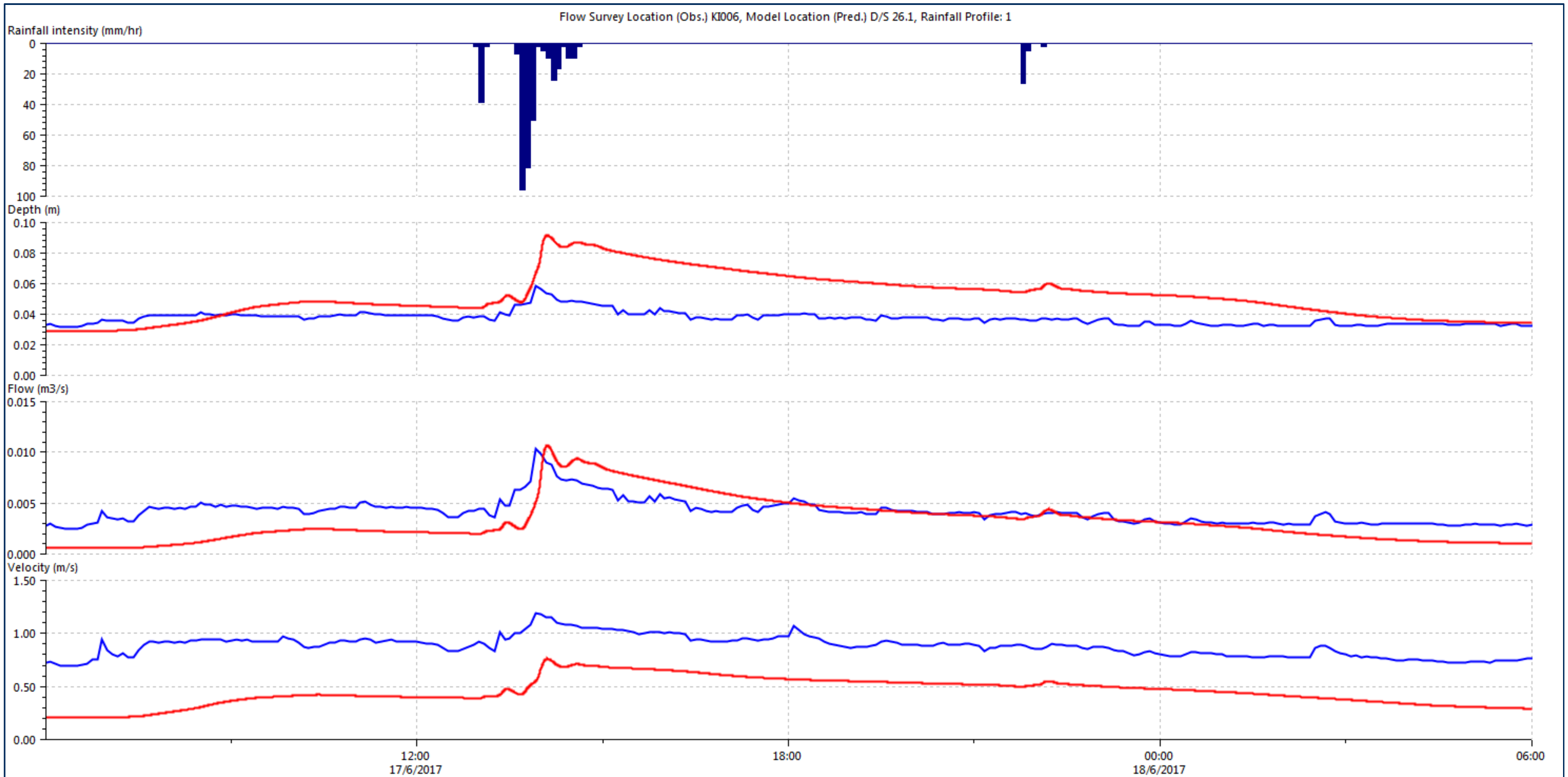


	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	32.800	96.000	1.367							
Observed				0.061	0.175	0.003	0.035	864.163	0.281	0.715
...e>Storm 17/06/2017				0.033	0.145	0.000	0.040	796.435	0.034	1.015

Figure A40: Storm event for KI-005 on 17/06/2017

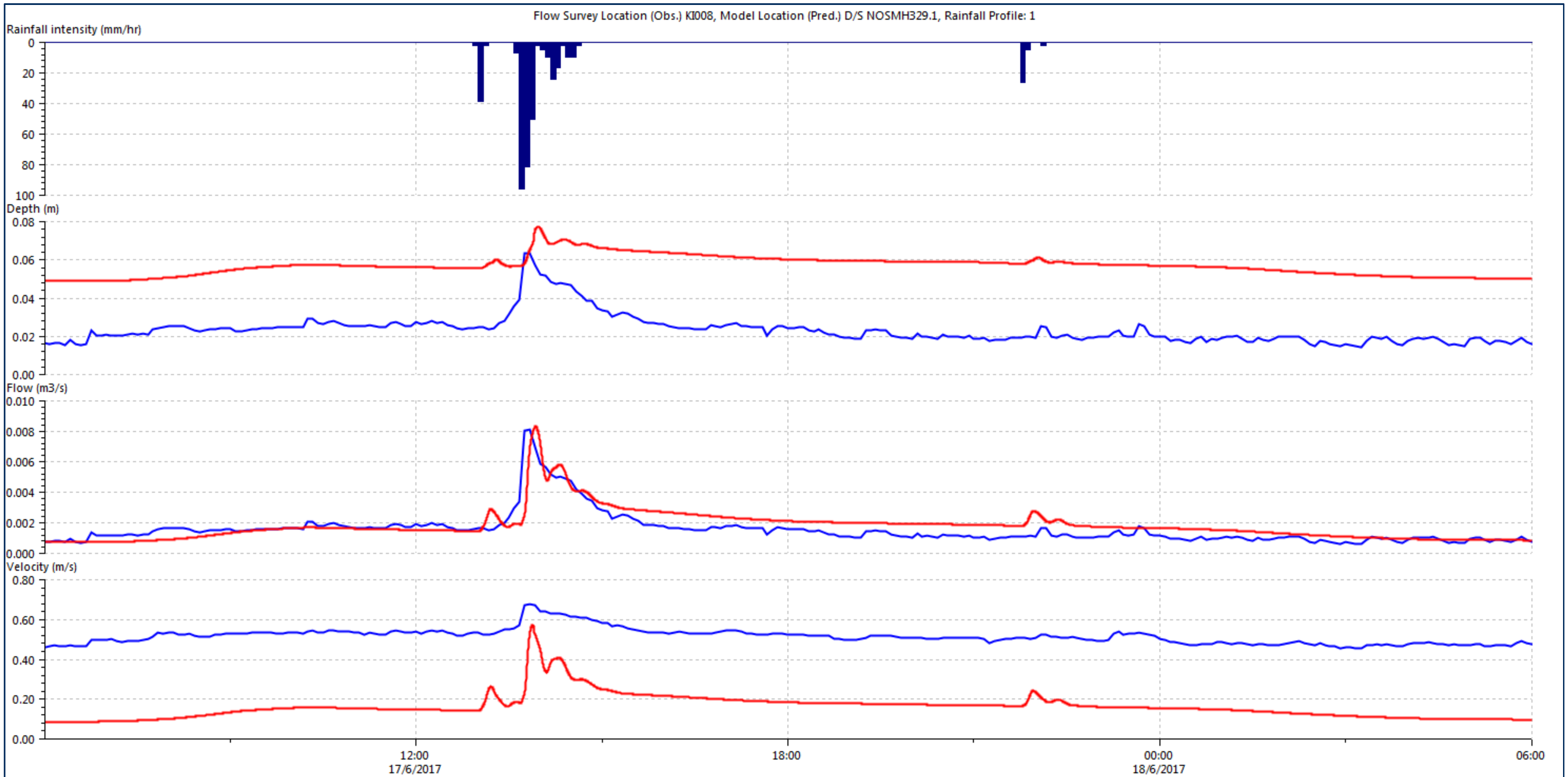
	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	32.800	96.000	1.367							
Observed				0.028	0.123	0.001	0.048	1230.768	0.219	1.685
...e>Storm 17/06/2017				0.038	0.120	0.000	0.041	678.732	0.020	1.377

Figure A41: Storm event for KI-006 on 17/06/2017



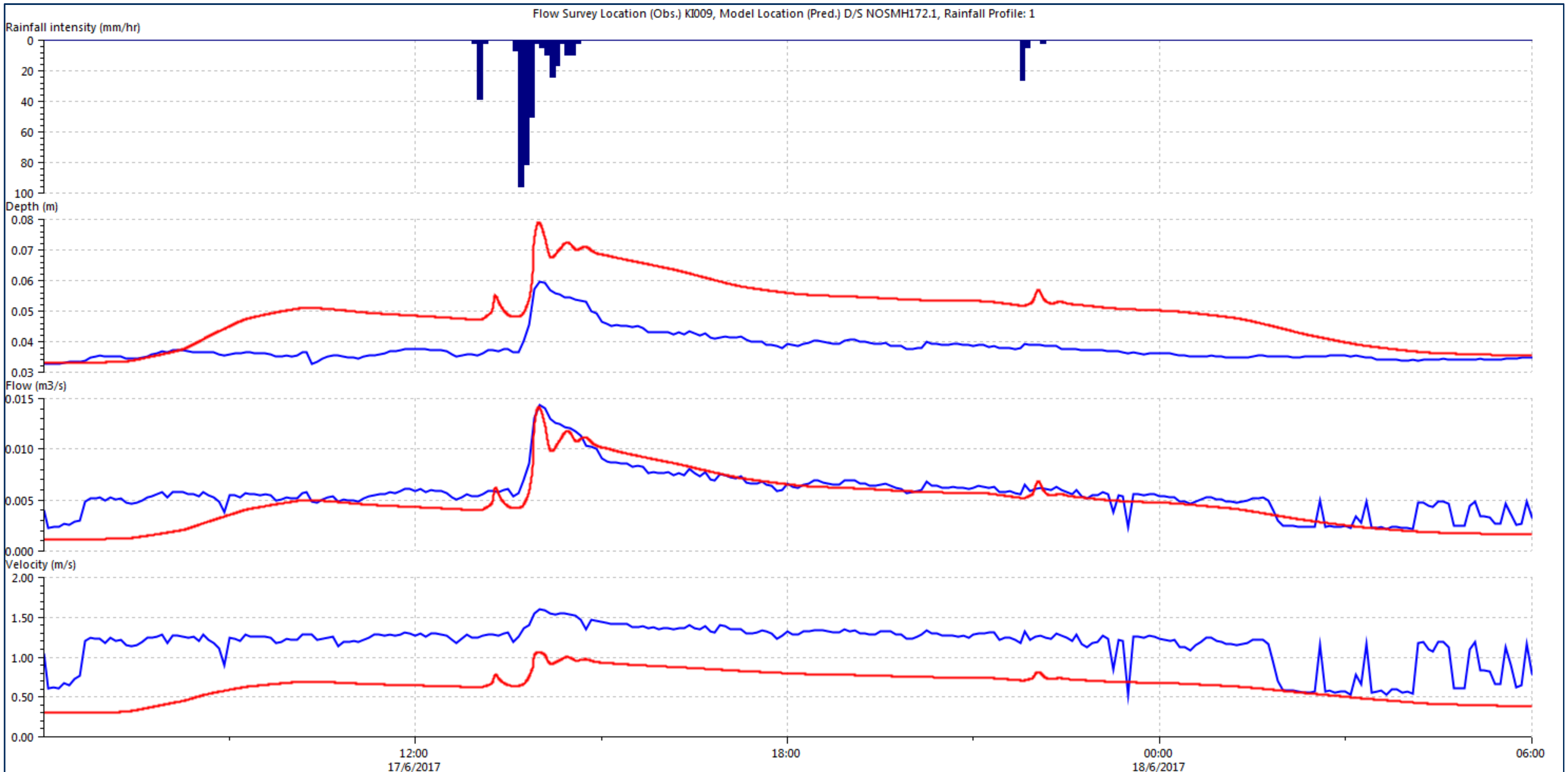
	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	32.800	96.000	1.367							
Observed				0.031	0.058	0.002	0.010	359.874	0.691	1.187
...e> Storm 17/06/2017				0.029	0.092	0.001	0.011	281.052	0.204	0.760

Figure A42: Storm event for KI-008 on 17/06/2017



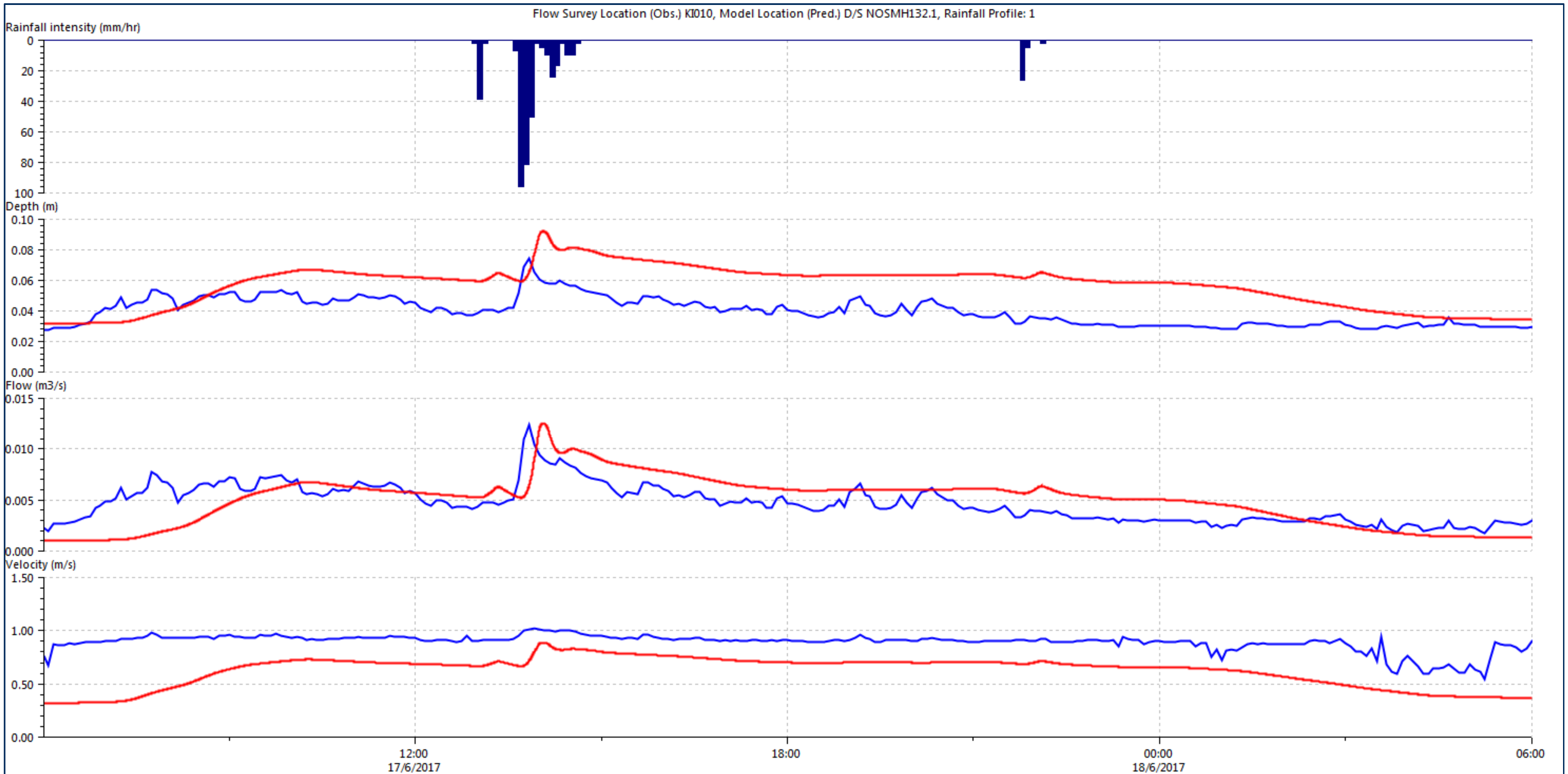
	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	32.800	96.000	1.367							
Observed				0.015	0.064	0.001	0.008	133.630	0.456	0.679
...e> Storm 17/06/2017				0.049	0.077	0.001	0.008	154.910	0.087	0.572

Figure A43: Storm event for KI-009 on 17/06/2017



	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	32.800	96.000	1.367							
Observed				0.033	0.060	0.002	0.014	489.703	0.530	1.603
...e> Storm 17/06/2017				0.033	0.079	0.001	0.014	415.257	0.299	1.062

Figure A44: Storm event for KI-010 on 17/06/2017



	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	32.800	96.000	1.367							
Observed				0.027	0.074	0.002	0.012	401.846	0.550	1.019
...e> Storm 17/06/2017				0.032	0.092	0.001	0.013	427.985	0.318	0.889

Figure A45: Storm event graph showing the comparison of the depth in Janet Avenue PS on 17/06/2017

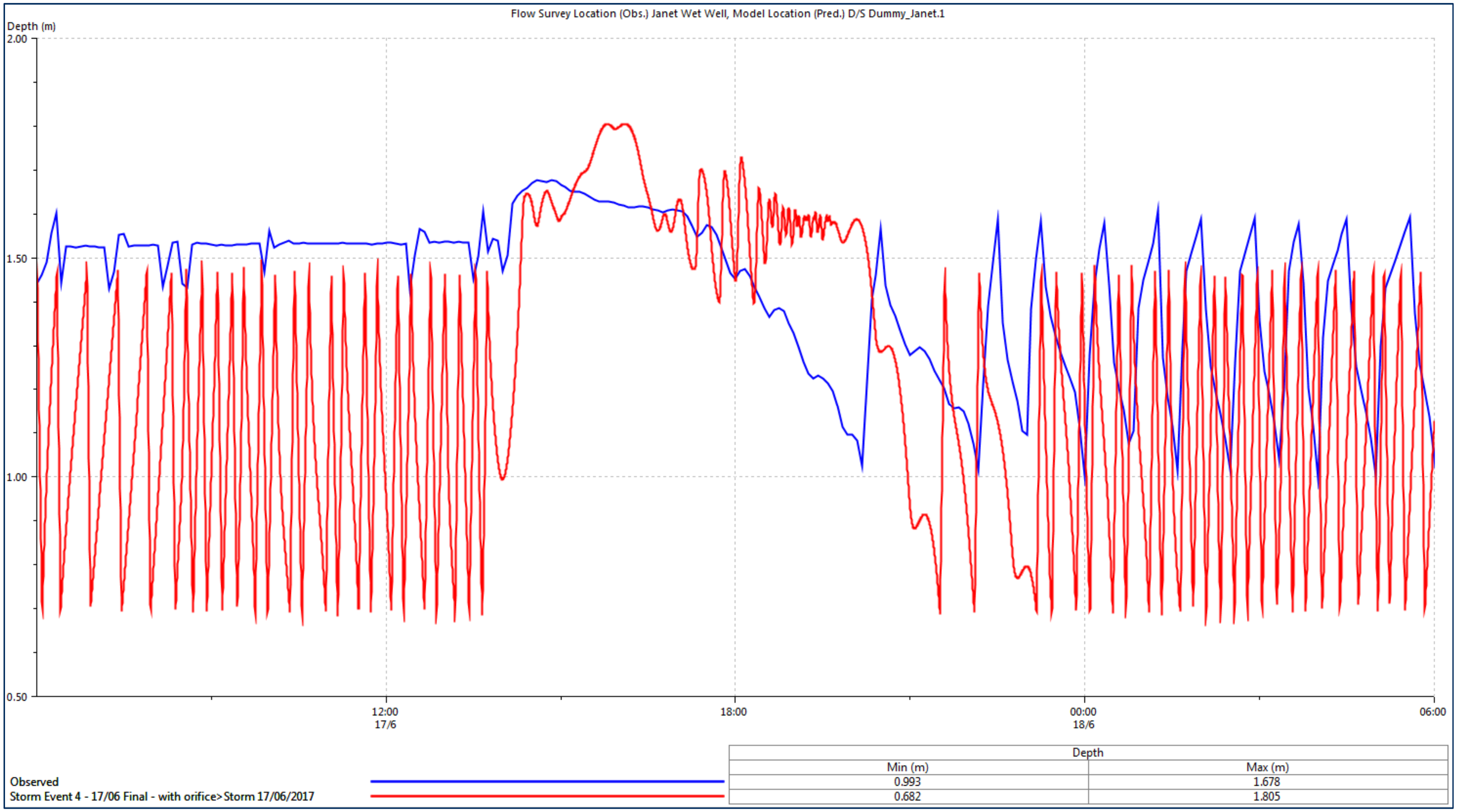


Figure A46: Storm Event graph showing the comparison of the flow along Janet Avenue PS forcemain on 17/06/2017

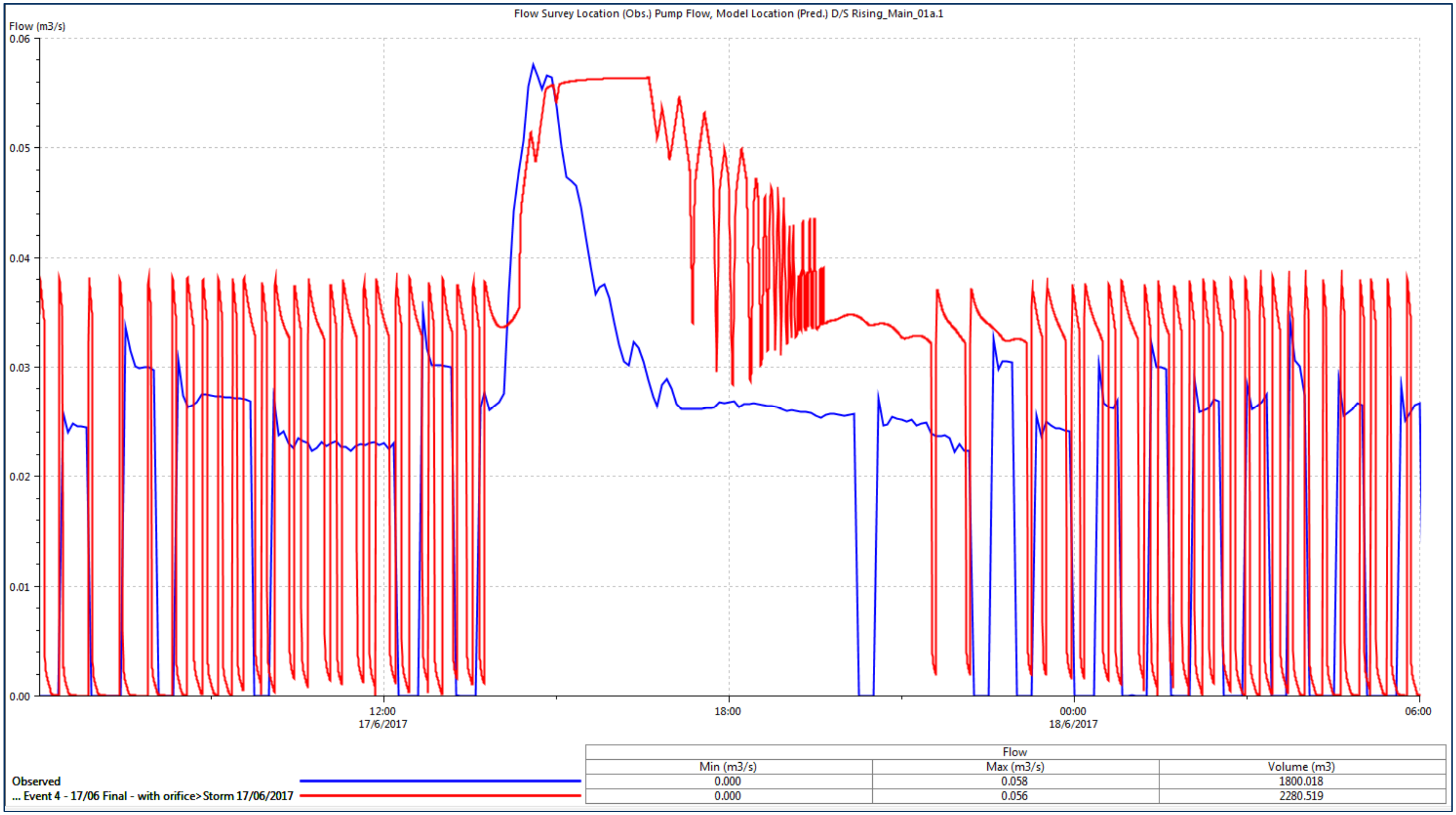
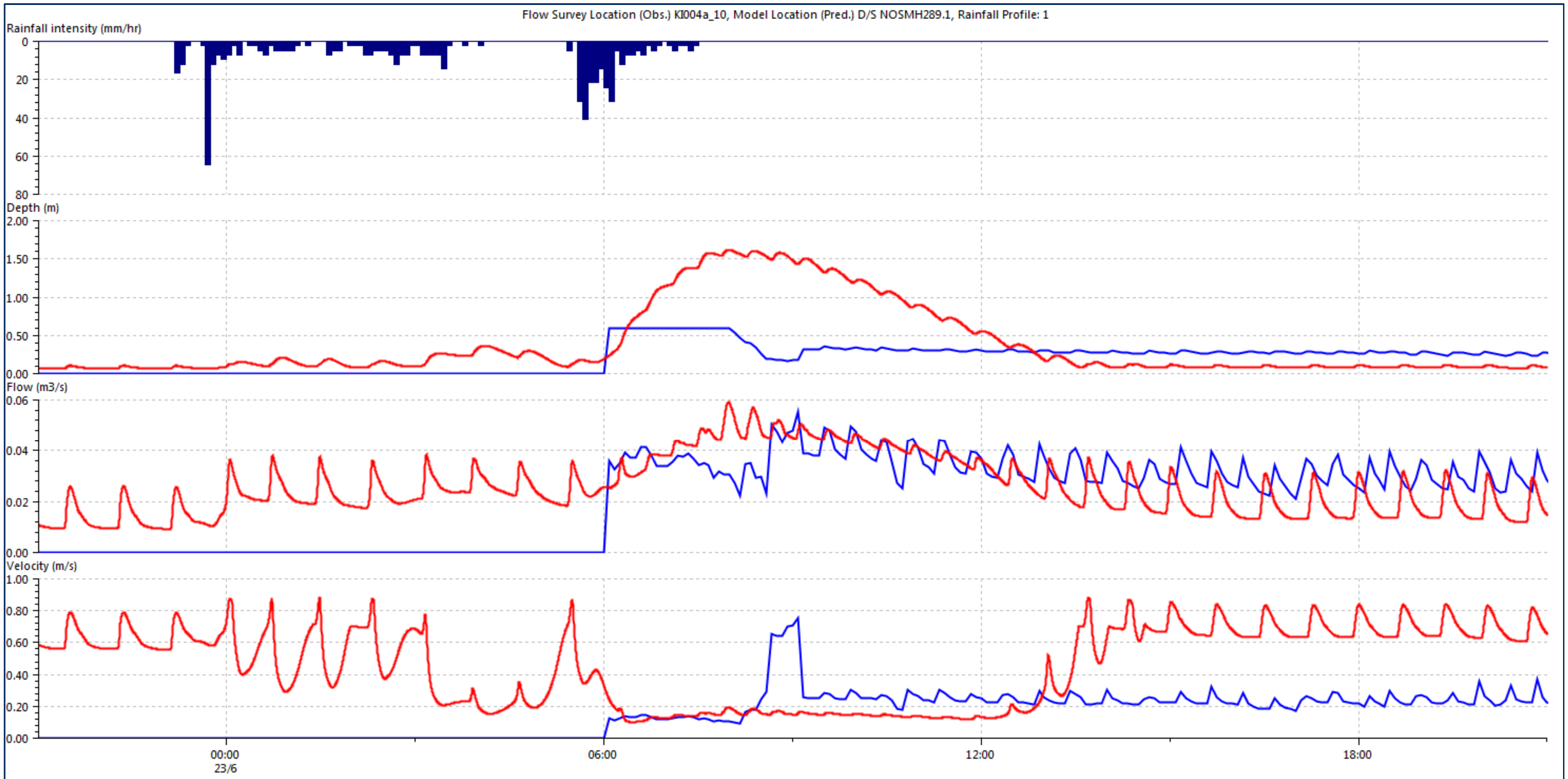
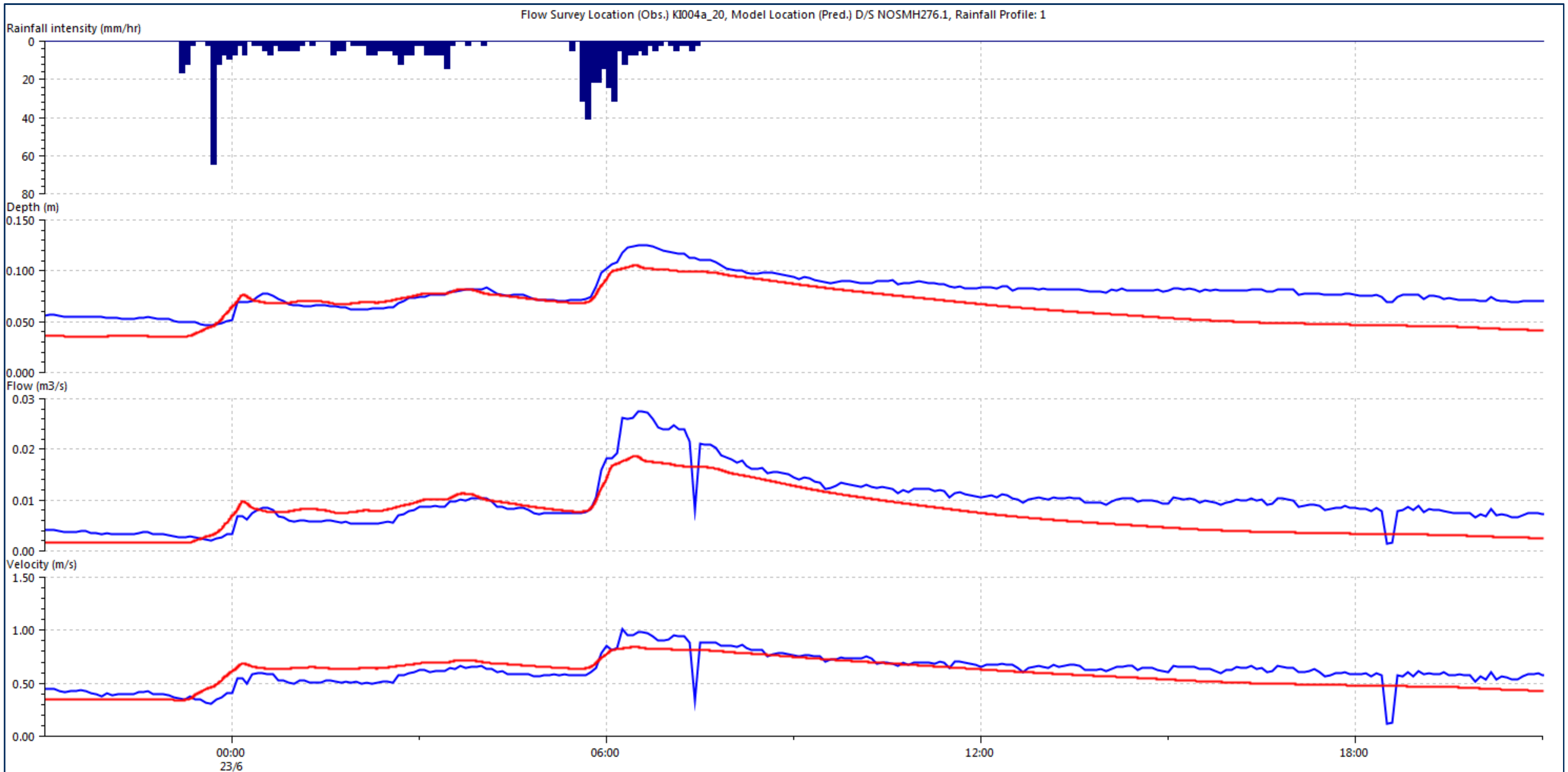


Figure A47: Storm event for KI-004a_10 on 23/06/2017



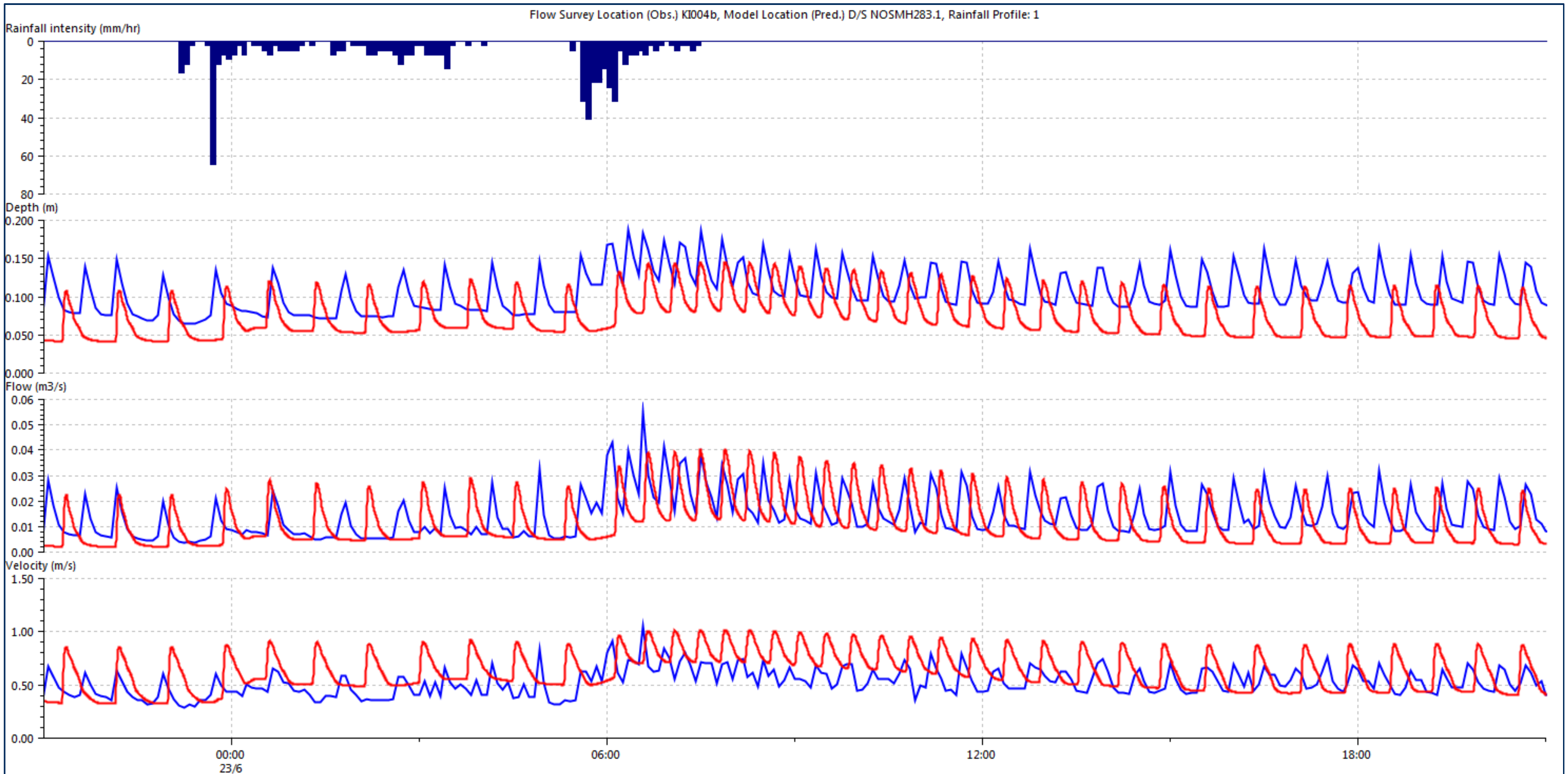
	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	49.200	64.800	2.050							
Observed				0.000	0.600	0.000	0.055	1795.405	0.000	0.754
...e> Storm 22/06/2017				0.065	1.619	0.009	0.059	2245.895	0.101	0.883

Figure A48: Storm event for KI-004a_20 on 23/06/2017



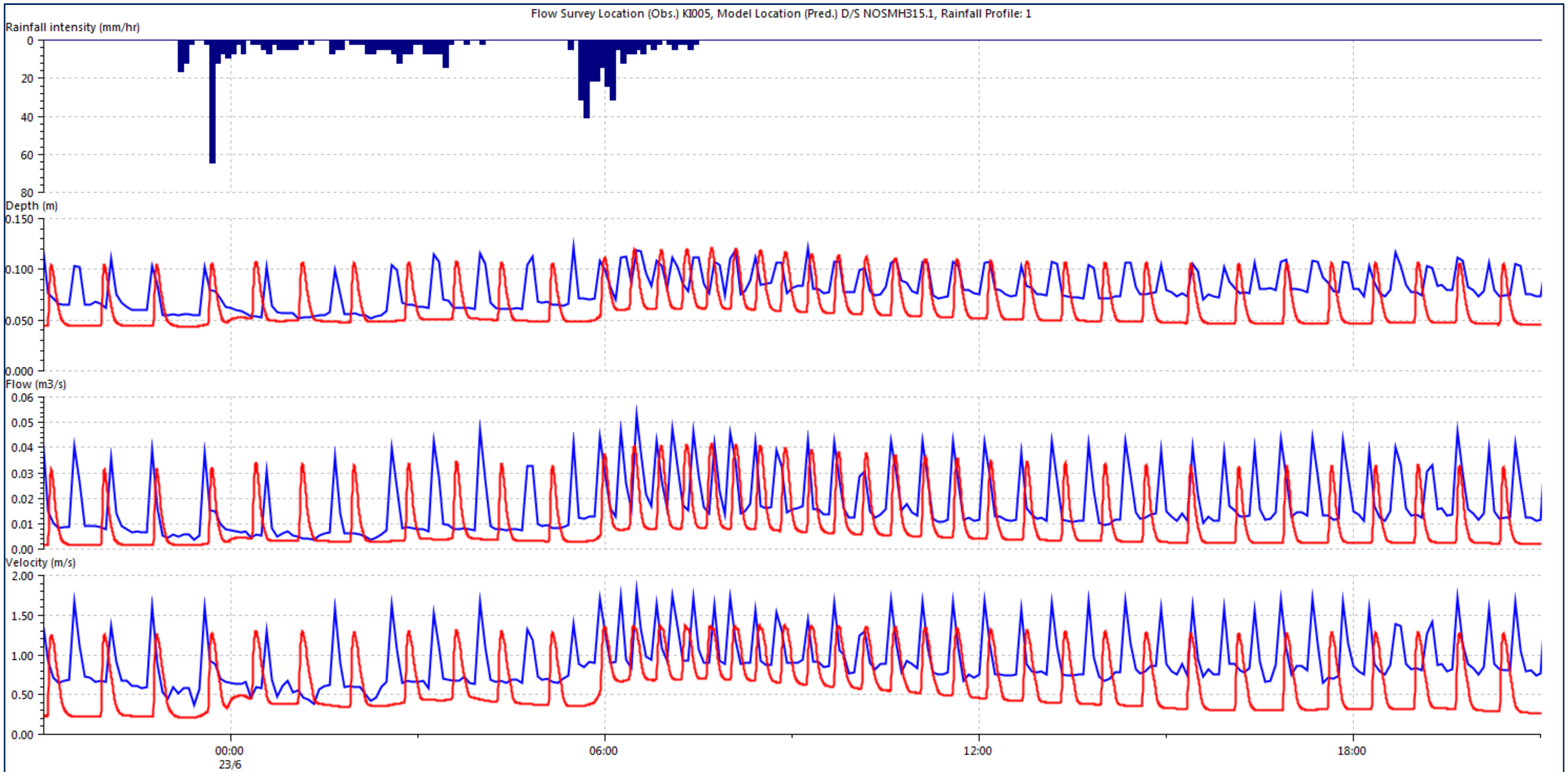
	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	49.200	64.800	2.050							
Observed				0.047	0.126	0.001	0.028	846.645	0.116	1.011
...e> Storm 22/06/2017				0.035	0.105	0.002	0.019	630.112	0.340	0.844

Figure A49: Storm event for KI-004b on 23/06/2017



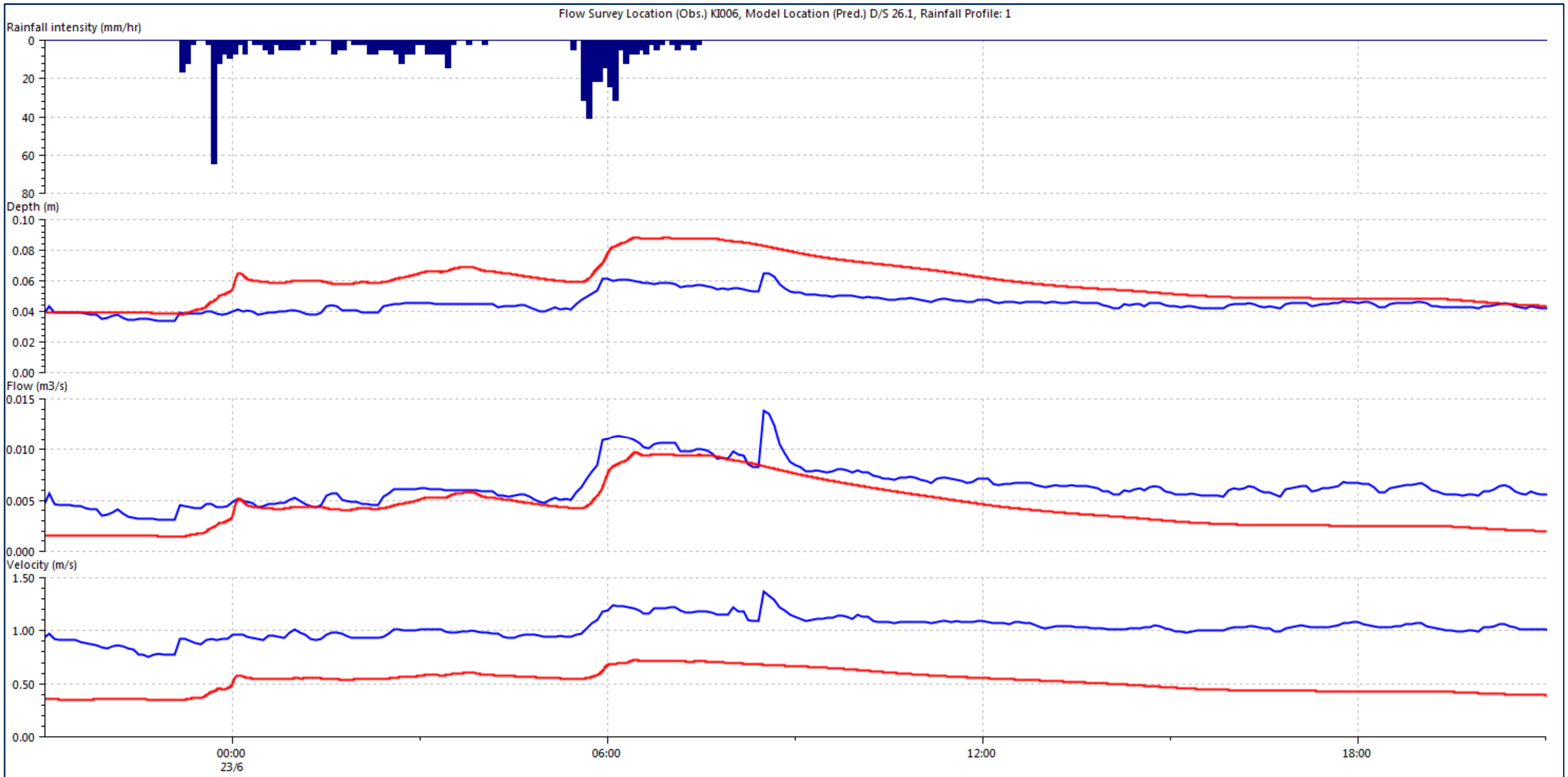
	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	49.200	64.800	2.050							
Observed				0.064	0.187	0.004	0.055	1270.729	0.284	1.049
...e> Storm 22/06/2017				0.042	0.146	0.002	0.040	983.839	0.324	1.019

Figure A50: Storm event for KI-005 on 23/06/2017



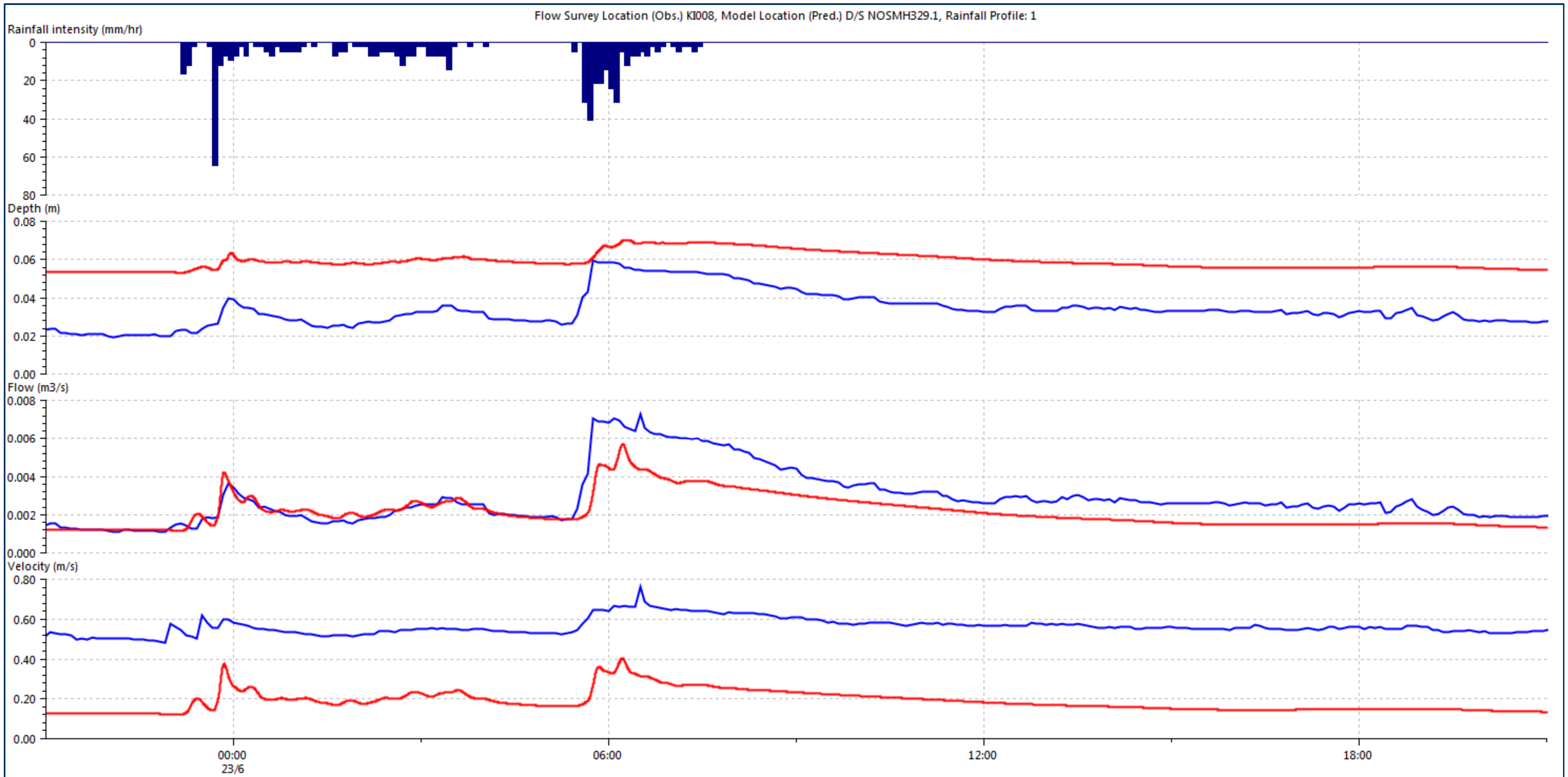
	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	49.200	64.800	2.050							
Observed				0.052	0.122	0.004	0.053	1532.517	0.371	1.810
...e> Storm 22/06/2017				0.044	0.121	0.002	0.041	824.694	0.215	1.378

Figure A51: Storm event for KI-006 on 23/06/2017



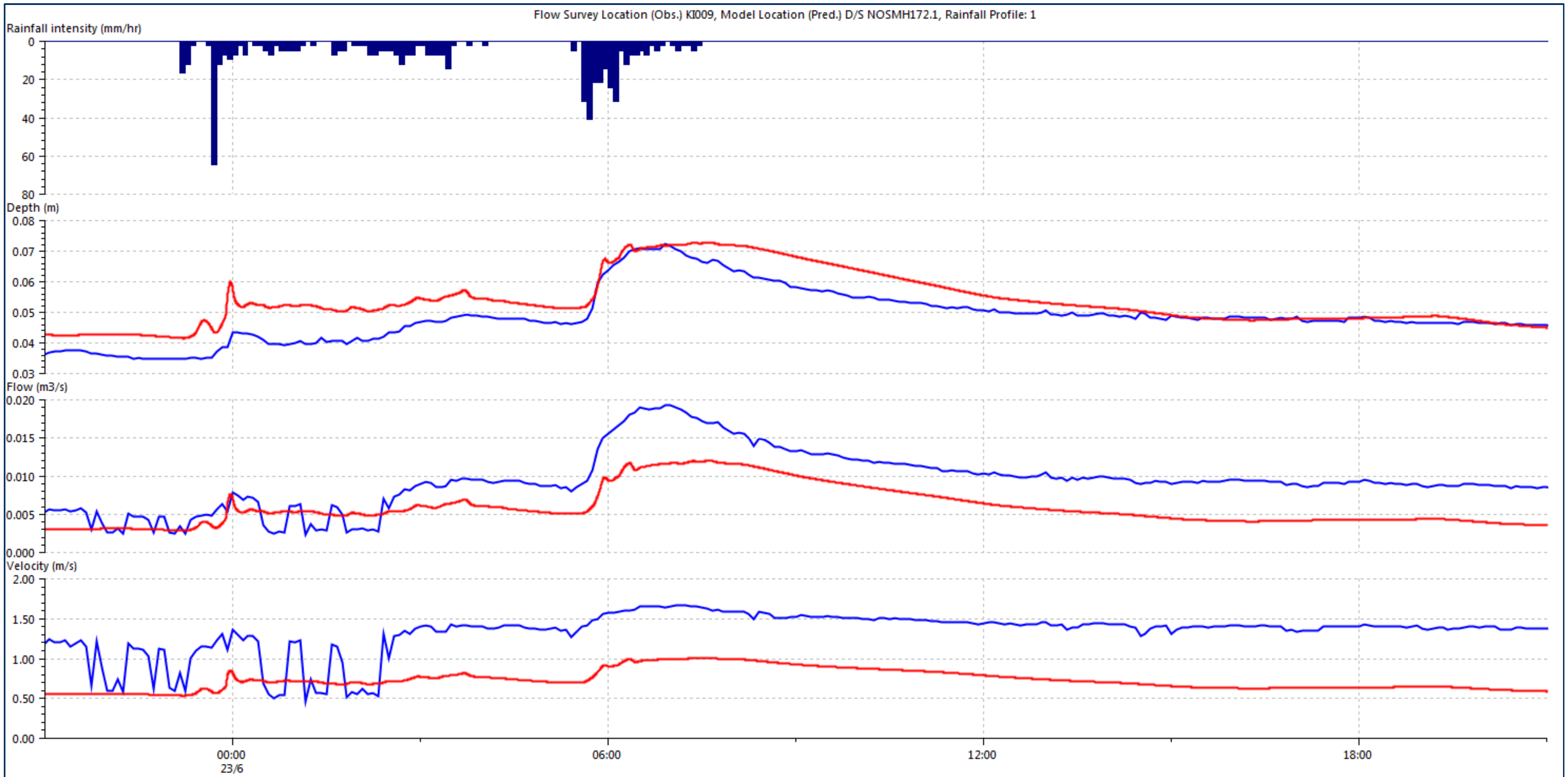
	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	49.200	64.800	2.050							
Observed				0.033	0.065	0.003	0.014	553.447	0.759	1.367
...e> Storm 22/06/2017				0.038	0.088	0.001	0.010	375.792	0.344	0.724

Figure A52: Storm event for KI-008 on 23/06/2017



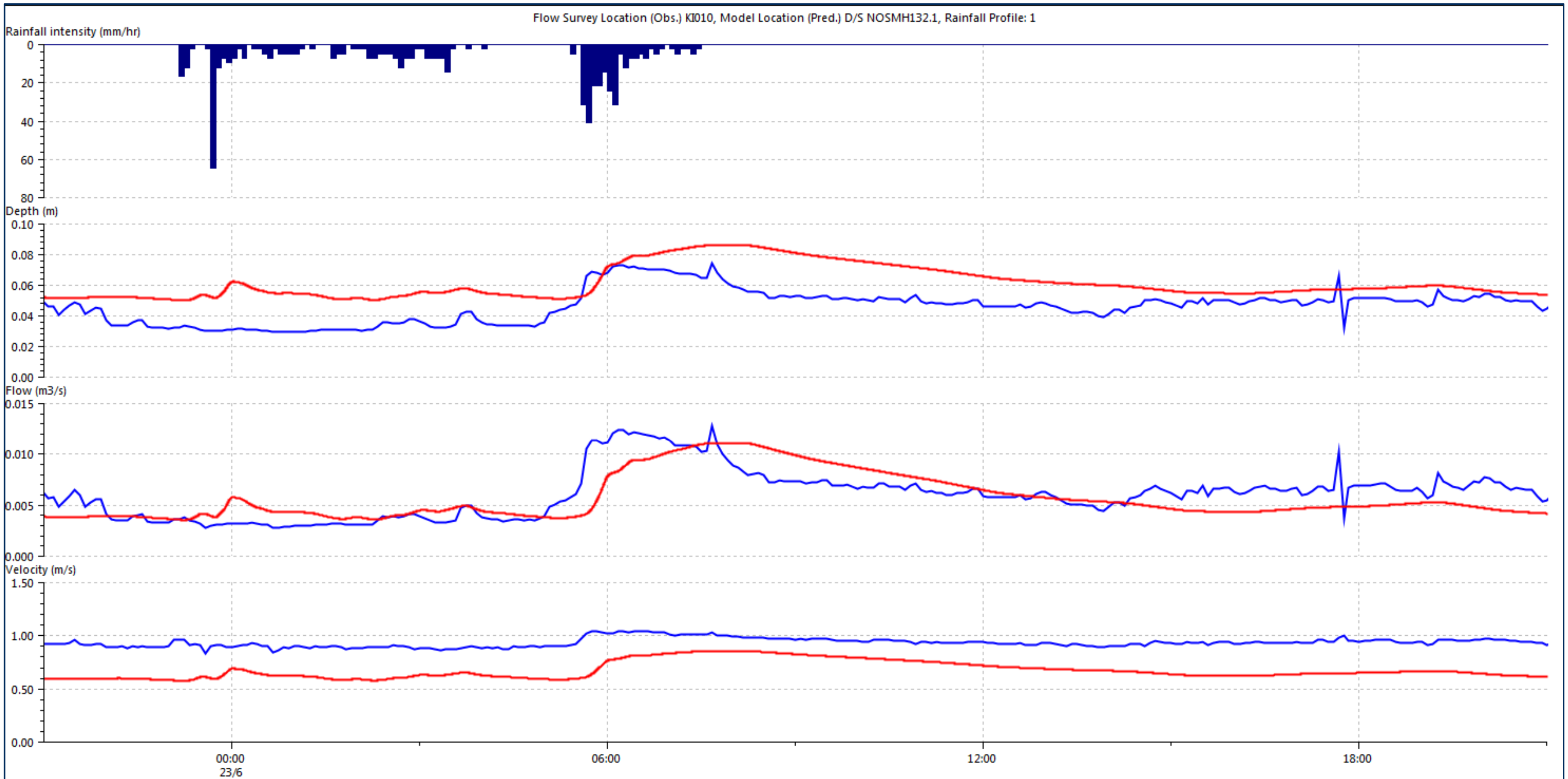
	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	49.200	64.800	2.050							
Observed				0.020	0.060	0.001	0.007	246.855	0.481	0.761
...e>Storm 22/06/2017				0.053	0.070	0.001	0.006	185.520	0.122	0.402

Figure A53: Storm event for KI-009 on 23/06/2017



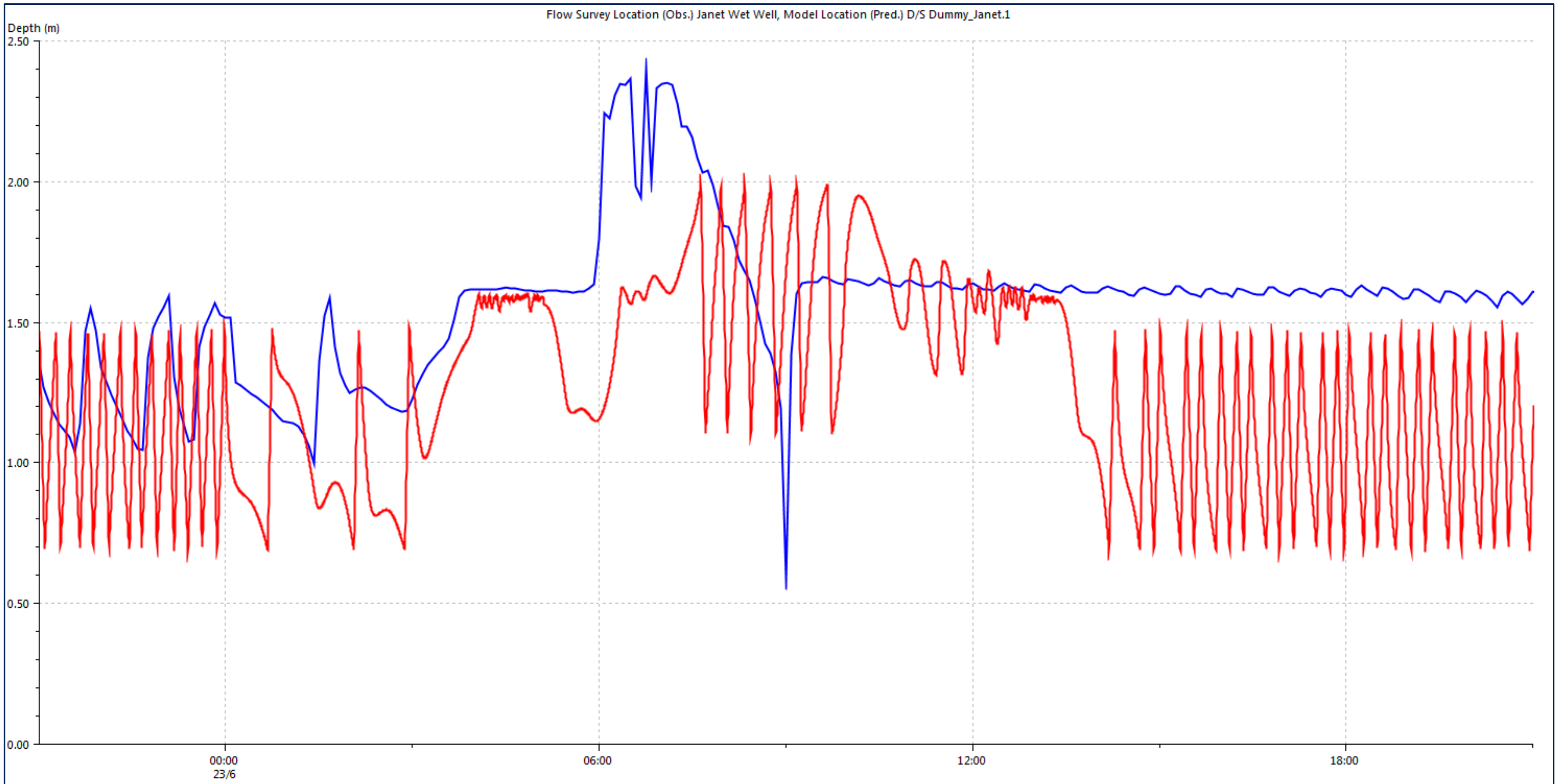
	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	49.200	64.800	2.050							
Observed				0.035	0.072	0.002	0.019	811.593	0.470	1.670
...e> Storm 22/06/2017				0.042	0.073	0.003	0.012	513.124	0.535	1.009

Figure A54: Storm event for KI-010 on 23/06/2017



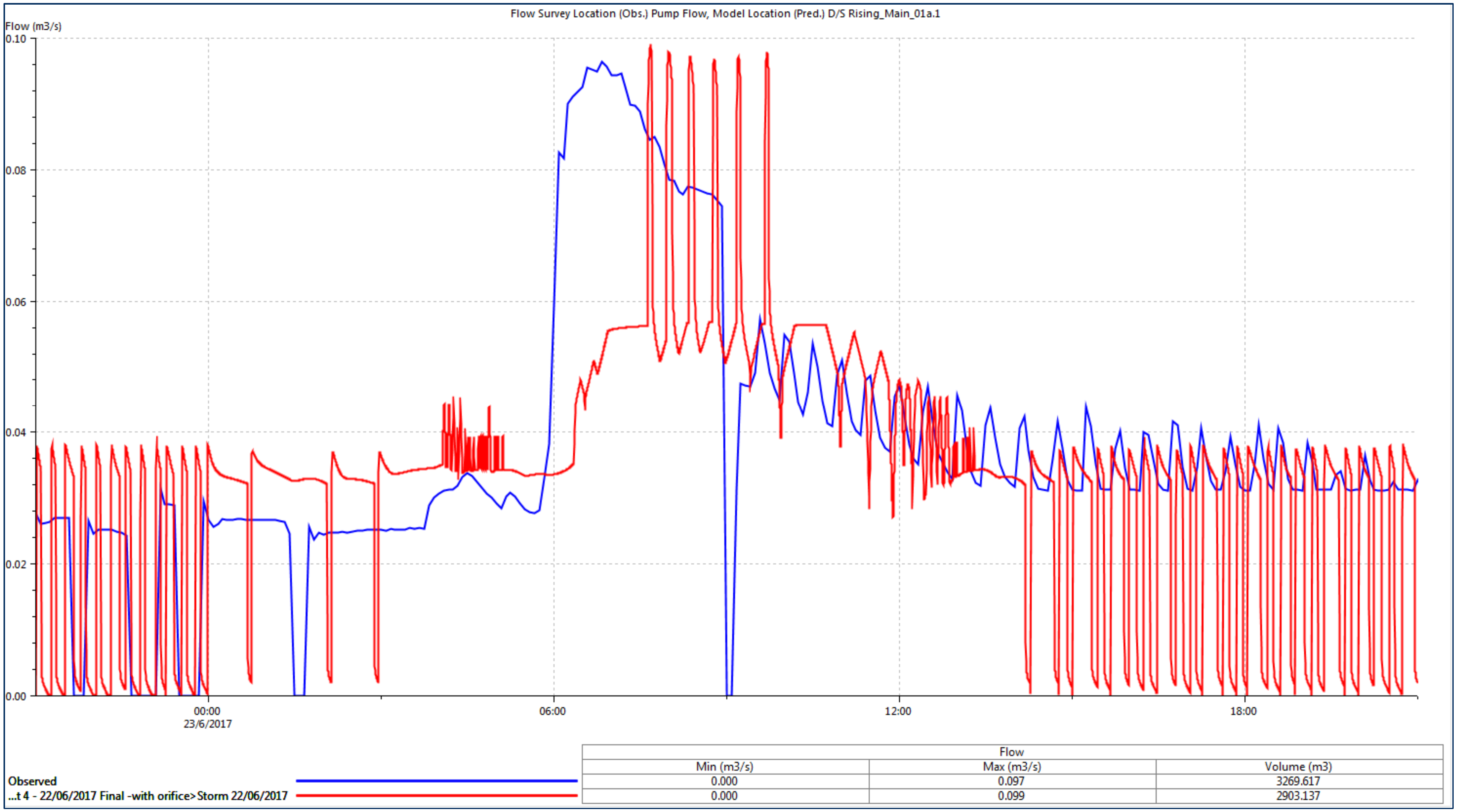
	Rainfall			Depth		Flow			Velocity	
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)	Min (m/s)	Max (m/s)
Rain	49.200	64.800	2.050							
Observed				0.030	0.075	0.003	0.013	522.251	0.830	1.046
...e> Storm 22/06/2017				0.050	0.086	0.004	0.011	495.366	0.576	0.857

Figure A55: Storm event graph showing the comparison of the depth in Janet Avenue PS on 23/06/2017



Depth	
Min (m)	Max (m)
0.586	2.404
0.681	1.994

Figure A56: Storm Event graph showing the comparison of the flow along Janet Avenue PS forcemain on 23/06/2017



FINAL

CLASS ENVIRONMENTAL ASSESSMENT FOR WATER AND WASTEWATER SERVICING IN THE COMMUNITY OF NOBLETON

WATER NEEDS ASSESSMENT AND JUSTIFICATION STUDY

Study 3A

B&V PROJECT NO. 196238

PREPARED FOR

Regional Municipality of York

4 JUNE 2019

BLACK & VEATCH

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LIST OF FIGURES

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Distribution List

BV FILE NO.	RevNo	Issued to	Date	Reason for Issue
	0	York Region – Afshin Naseri	12-18-2018	First Draft for Review
	1	York Region – Afshin Naseri	06-04-2019	Revised based on comments received

Revision Log

BV FILE NO.	RevNo	Prepared By	Reviewed By	Authorized By	Issue/Revision Description
42.3120	0	Steve Jobson	Abra Ens / Oya Koc	John Bourrie	Draft Submission
42.3120	1	Steve Jobson	Oya Koc	John Bourrie	Final Submission

Black & Veatch Signatures

Prepared By

____

Steve Jobson, P.Eng.
Infrastructure Planning Engineer

Reviewed By

____

Oya Koc, P. Eng.
Project Advisor

1 Introduction

1.1 PURPOSE OF STUDY

The purpose of the Water Needs Assessment and Justification Study is to:

- Identify needs or gaps in water servicing (well supply, storage, and distribution) to support growth in Nobleton to 10,800 people
- Identify any required system improvements to meet future growth
- Determine feasibility of servicing to future growth targets
- Using the Needs Assessment and Justification analyses, develop draft Opportunity Statement to feed into the Class EA

1.2 BACKGROUND

Nobleton is a community in King Township, located in York Region. Currently, Nobleton is serviced by standalone water and wastewater systems to meet the needs of the current population. The York Region Water and Wastewater Master Plan (2016) indicated that both the water and wastewater systems would not have sufficient capacity to meet requirements to support growth to the 2041 Master Plan horizon. Therefore, the Master Plan recommended undertaking the current project, a Schedule C Class Environmental Assessment (EA), to identify preferred servicing solutions to accommodate growth.

1.3 EXISTING WATER SYSTEM INFRASTRUCTURE

The Regional Municipality of York (also referred to as the Region and York Region) is responsible for the water production, treatment, storage and transmission to its local area municipalities, including the Community of Nobleton in the Township of King. The Nobleton water supply system consists of three groundwater wells and two elevated storage tanks that provide service to the Nobleton Pressure District. There is also a booster pumping station (BPS) that services a higher elevation area in the northwest portion of the distribution system. The wells operate based on level at either of the elevated tanks. The booster pumping station operates independently from the rest of the water system controls.

1.4 RELEVANT PLANS AND POLICIES

1.4.1 Regional Official Plan

The purpose of the Region's Official Plan is to, "guide economic, environmental and community building decisions to manage growth".

One of the Region's major goals is, "To provide the services required to support the Region's residents and businesses to 2031 and beyond, in a sustainable manner".

Based on this goal, the Region's objective for water and wastewater servicing is, "To deliver safe, clean drinking water and provide long term water and wastewater services to York Region's communities, that are safe, well-managed, and sustainable".

To meet this objective, the following Policies are outlined in the Region's Official Plan:

- 7.3.12 - To supply the Urban Area and Towns and Villages with water from the Great Lakes or from Lake Simcoe, subject to the restrictions of the Greenbelt Plan, Lake Simcoe Protection Plan, or other Provincial plans and statutes. A limited amount of groundwater resources will be used and managed in a way that sustains healthy flow into creeks, streams and rivers.
- 7.3.15 - That development within and expansions to the urban uses within Towns and Villages will occur on the basis of full municipal water and wastewater treatment services where such facilities currently exist. For existing or previously approved development in Towns and Villages, water and wastewater treatment services will be continued where feasible and in keeping with the provisions of local official plans and this Plan.
- 7.3.16 - That within the Oak Ridges Moraine, Greenbelt, and Lake Simcoe watershed, all improvements or new water and wastewater infrastructure systems shall conform with the Oak Ridges Moraine Conservation Plan, the Greenbelt Plan or the Lake Simcoe Protection Plan.
- 7.3.17 - That the construction or expansion of partial services is prohibited in the Oak Ridges Moraine unless it has been deemed necessary to address a serious health or environmental concern identified by the Medical Officer of Health or other designated authority.
- 7.3.18 - To provide reliable water and wastewater services to residents and businesses to ensure continuing community well-being and the economic vitality of the Region.
- 7.3.25 - To ensure that wastewater effluent is managed to minimize impacts on the quality of the receiving water.
- 7.3.30 - That the planning and design of water and wastewater infrastructure will consider potential impacts from climate change.
- 7.3.31 - To ensure secure and resilient Regional water and wastewater systems to maintain continual service.
- 7.3.32 - That water and wastewater services will be planned, constructed and operated in a manner that protects, enhances, and provides net benefit to the Region's natural and cultural heritage.
- 7.3.34 - That the water and wastewater systems be sized to consider the potential for expansion of the service area, intensification and increased allocation where permitted by York Region Master Plans and Provincial Plans.

The Official Plan is relevant to the Class EA study since it outlines the policies that guide the economic, environmental and community building decisions to manage growth. It emphasizes the need to develop water and wastewater services that support the economic growth of the Region while protecting the Region's natural and cultural heritage.

1.4.2 York Region Corporate Strategic Plan

The 2015-2019 York Region Strategic Plan is a roadmap that guides toward the vision of the future. It serves as a plan to get the Region from where they are to where they want to be in 2051 and focuses on Economic Vitality, Healthy Communities, Sustainable Environment and Good Government.

The key Regional Performance Measures listed in the Strategic Plan that relate to the Nobleton Water and Wastewater Servicing Class EA are the following:

- Maintain percentage of treated water returned to environment within regulated standards;
- Reduce quantity of inflow and infiltration in Regional and local wastewater systems;
- Decrease average residential water demand.

The Region's Corporate Strategic Plan is relevant to the Class EA because it emphasizes key performance measures for water and wastewater systems that should be used as a vision for the future including an emphasis on reducing inflow and infiltration and reducing residential water demands.

1.4.3 King Township Draft Official Plan

The purpose of the King Township Official Plan is to provide direction and a policy framework for managing growth, land use and infrastructure decisions over the planning period to 2031.

The Draft Official Plan notes the following specifics regarding Nobleton:

- The population forecast for Nobleton reflects limitations posed by the municipal sanitary sewer services that can accommodate a total population in Nobleton of 6,750 to 7,000 to 2031.
- Notwithstanding the above, the potential exists for additional development and population growth to occur on lands that are within the Village of Nobleton settlement area boundary. The total population of the Village of Nobleton could reach between 9,600 and 10,900 persons based on the amount of land designated for residential development / redevelopment.
- This additional development and population growth will require an amendment to this plan and can be considered when the Township completes its next municipal comprehensive review to the planning horizon of 2041. In addition to an amendment to this plan, the additional development described above will also require a servicing solution to the satisfaction of the Township of King and Region of York.

The King Township Official Plan is relevant to the Class EA because it specifies the limitations and framework for Nobleton's population growth.

1.4.4 Water and Wastewater Master Plan

This document reports on the update of the Water and Wastewater Master Plan for The Regional Municipality of York. The updated Master Plan will guide investments in water and wastewater systems to support the Region's projected growth to 2041. The Master Plan had the following major objectives that relate to the Class EA:

- Develop a cost-effective, resilient water and wastewater infrastructure plan to service future growth to 2041 and beyond
- Develop an integrated, long-term strategy to provide sustainable water and wastewater services

The Master Plan also noted the following regarding stand-alone communities:

- Communities currently serviced by stand-alone water and/or wastewater systems will continue to be serviced by stand-alone systems. These include Keswick and Sutton (Town of Georgina), Mount Albert (Town of East Gwillimbury), Ballantrae (Town of Whitchurch-Stouffville), Ansnorveldt, Nobleton and Schomberg (Township of King). Kleinburg Water Resource Recovery Facility will continue to service new developments up to its permitted capacity, after which all new developments will be serviced by the York Durham Sewage System.

Further to the Master Plan, York also developed the "One Water Action Plan" which includes the following action areas:

1. Implement the Long-Term Water Conservation Strategy and Water Reuse
2. Implement Inflow and Infiltration Reduction
3. Enhance Integration of Asset Renewal with Growth Projects
4. Develop Climate Change Adaptation and Mitigation Strategies
5. Continue Energy Optimization and Renewable Energy Initiatives; and
6. Ensure Financial Sustainability.

The Region's Water and Wastewater Master Plan is relevant to the Class EA because it serves as the guiding document on water and wastewater system investments to 2041. It specifically mentions the desire to continue servicing stand-alone systems as stand-alone systems.

1.4.5 Provincial Policy Statement

The Provincial Policy Statement provides policy direction on matters of provincial interest related to land use planning and development. As a key part of Ontario's policy-led planning system, the Provincial Policy Statement sets the policy foundation for regulating the development and use of land. It also supports the provincial goal to enhance the quality of life for all Ontarians.

The following key policies from the 2014 Provincial Policy Statement are summarized below:

- 1.6.6.1 - Planning for sewage and water services shall:
    - direct and accommodate expected growth or development in a manner that promotes the efficient use and optimization of existing: 1. municipal sewage services and municipal water services; and 2. private communal sewage services

and private communal water services, where municipal sewage services and municipal water services are not available;

- ensure that these systems are provided in a manner that: 1. can be sustained by the water resources upon which such services rely; 2. is feasible, financially viable and complies with all regulatory requirements; and 3. protects human health and the natural environment;
- promote water conservation and water use efficiency.

The Provincial Policy Statement is relevant to the Class EA because it again emphasizes the need to develop water and wastewater services to meet the expected growth, while sustaining our water resources and protecting the natural and cultural environment.

1.4.6 Greenbelt Plan

The province's Oak Ridges Moraine Conservation Act, 2001 and Greenbelt Act, 2005 are intended to reduce pressure on natural and agricultural lands in the Greater Golden Horseshoe Area. As a large portion of the Region's lands are located within the Oak Ridges Moraine and Greenbelt, these Acts have significant implications on development and water and wastewater infrastructure planning. Specifically, the Oak Ridges Moraine Conservation Plan prohibits "partial servicing" of water or wastewater (except in very limited circumstances) and the Greenbelt Plan restricts the extension of lake-based water and wastewater servicing.

The Community of Nobleton is denoted as a Town/Village in the Protected Countryside of the Greenbelt Area. It is surrounded on all sides by Protected Countryside Areas, therefore any proposed infrastructure must satisfy the policies set forth in the Greenbelt Plan (particularly Section 4.2).

The Greenbelt Plan is relevant to the Class EA because, unless there are changes to the Greenbelt Plan, servicing the Town of Nobleton by connecting to the Lake Based Water System is only considered a suitable option if no other suitable options exist to safely service the communities' needs from within. In other words, well supply would need to be definitively proven to be insufficient, either in quality or quantity, to prefer an extension of lake-based water and wastewater servicing.

1.4.7 Oak Ridges Moraine Conservation Plan

The purpose of the Oak Ridges Moraine Conservation Plan is to provide land use and resource management planning direction to provincial ministers, ministries, and agencies, municipalities, landowners and other stakeholders on how to protect the Moraine's ecological and hydrological features and functions.

The north portion of Nobleton is designated a settlement area under the Oak Ridges Moraine Conservation Plan, and areas North-East of the community are designated natural areas under the plan.

In the Oak Ridges Moraine, new infrastructure corridors or facilities shall only be allowed in the Natural Core Areas and Natural Linkage Areas if they are shown to be necessary and there is no reasonable alternative. They shall also have to meet stringent review and approval standards.

The Oak Ridges Moraine Conservation Plan is relevant to the Class EA because the Oak Ridges Moraine is located at the northeast portion of Nobleton. Within these areas, certain restrictions exist both in terms of land use and infrastructure which need to be considered.

1.4.8 Watershed Management Plans

The Humber River Watershed Plan – Pathways to a Healthy Humber (2008), was prepared by the Toronto and Region Conservation Authority (TRCA), in partnership with municipal, provincial and federal government representatives and other stakeholders including the Humber Watershed Alliance. The Watershed Plan provides guidance to local, regional and provincial governments and TRCA as they update their policies and programs for environmental protection, conservation, and restoration within the contexts of land and water use, and the planning of future development. It also provides direction to local non-governmental organizations and private landowners with regard to best management practices and opportunities for environmental stewardship. The Watershed Plan is based on a strong understanding of current conditions developed through analysis of environmental monitoring information, combined with leading edge approaches to predicting potential future conditions that involved modelling and expert input.

The Humber River Watershed Plan is relevant to the Class EA because the current water reclamation facility discharges to the Humber River. Therefore, any changes in discharge quantity or quality needs to be analyzed and discussed in collaboration with the TRCA.

1.4.9 Great Lakes – St. Lawrence River Basin Sustainable Water Resources Agreement (Intra-Basin Transfer of Water)

The Ontario Water Resources Act, 1990 as amended by the Safeguarding and Sustaining Ontario's Water Act, 2007, bans transfers of water from one Great Lakes watershed to another except under strictly regulated conditions. This is a challenge for the Region, because it straddles the Lake Huron (Simcoe) and Lake Ontario watersheds. The Region has received permission to transfer no more than 105 million litres a day of water and must meet ongoing conditions for this transfer.

Currently, all water originating in Nobleton is maintained within the Lake Ontario (Humber River) Watershed, therefore it does not impact the intra-basin transfer limit.

The Intra-Basin Transfer Agreement is relevant to the Class EA because it emphasizes the need to maintain a balance between the Lake Ontario and Lake Huron watersheds. Currently, all water originating in Nobleton is maintained within the Lake Ontario watershed. As long as it stays this way, then this agreement does not impact the Nobleton Class EA.

2 Needs Assessment and Justification

2.1 FUTURE SYSTEM NEEDS

2.1.1 Water Demand Projections

Based on a review of historical data and subsequent discussions with York Region staff, the following Nobleton Water System design criteria was established. Details of the historical review are provided in Study 1A: Water System Capacity Optimization Study.

Table 1: Water Demand Design Criteria

DESIGN CRITERIA	2016	FUTURE
Residential Population	5,520	10,800
Employment Population	772	1,800
Residential Per Capita Demand (L/cap/d)	220	220
Employment Per Capita Demand (L/cap/d)	64	182 *
Non-Revenue Water %	26.5%	26.5%
ADD:MDD Peaking Factor	2.1	2.1
*Since the current Nobleton employment per capita demand is significantly lower than the remainder of York Region, it is recommended that for future employment projections the higher per capita demand rate of 182 L/cap/d be used. The type of future employment in Nobleton is currently unknown, so this will allow for slightly larger consuming employment users than those that currently exist. The selected 182 L/cap/d is based on the York Region Master Plan 2016 Employment per capita rate.		

Using the above criteria, the average and maximum day demands can be calculated and are presented in Table 2:

Table 2: Projected Future Water Demands

CATEGORY	FUTURE DEMAND (L/S)
Average Day Demand (L/s)	42.6
Maximum Day Demand (L/s)	89.5

The demands shown in Table 2 are established as the design basis for alternative solutions that do not include any water conservation. However, understanding that water conservation improvements could be considered as alternatives (or as a component of an alternative), the above demands may be lower in other alternative solutions.

2.2 WELL (SUPPLY) NEEDS

Based on the well capacity and storage capacity in the Nobleton Water System (presented in detail in Study 1A: Water System Capacity Optimization Study), the following summarizes the current water system capacity limitations in Nobleton:

Table 3: Existing Water System Capacity Summary

CATEGORY	CAPACITY LIMIT (L/S)
Nobleton Well #2	22.7 L/s
Nobleton Well #3	28.9 L/s
Nobleton Well #5	28.9 L/s
Existing Permit to Take Water Limit (Firm Capacity: Well #2 plus #3 or #5)	51.6 L/s
Three Existing Nobleton Wells (Total Capacity, not Firm Capacity)	80.5 L/s

Furthermore, according to York Region's desktop assessment of the potential maximum sustainable capacity of the existing Nobleton Production Wells, it is expected that Nobleton Well 2 could have a potential capacity up to 67 L/s. with various facility upgrades (pump, treatment, etc.). Additionally, it is believed that the Nobleton Well #5 site also has potential for additional capacity. The current limiting factor at Nobleton Well #5 is the screen transmitting capacity which may not allow for any additional sustainable production. Therefore, an added well at the same site or at a new site may be more feasible.

Based on the existing well capacities and the projected maximum day demand of 86.5 L/s, additional well capacity is required for the Nobleton Water System. Furthermore, for the purposes of having increased system redundancy, it is critical to the Region that the well supply system be designed such that the largest well can be taken out of service during maximum day demands and still have sufficient supply capacity.

So, for alternative solutions that do not involve any water conservation measures, the combined existing and future Nobleton wells will require a firm capacity of at least 89.5 L/s. This could be achieved in a number of different ways, including increasing the capacity of the existing wells, adding new production wells or connecting to another water supply. Table 4 provides one of the many alternative solutions that could be considered as part of the Environmental Assessment.

Table 4: Alternative Solution - New Well Plus Expansion of Well #2 with No Additional Water Conservation

CATEGORY	CURRENT CAPACITY (L/S)	FUTURE CAPACITY (L/S)
Nobleton Well #2	22.7 L/s	32 L/s (Expansion of Existing Facility)
Nobleton Well #3	28.9 L/s	28.9 L/s
Nobleton Well #5	28.9 L/s	28.9 L/s
Potential New Nobleton Well	n/a	32 L/s
Permit to Take Water Limit / Nobleton Wells Firm Capacity	51.6 L/s	89.8 L/s
Nobleton Wells Total Capacity	80.5 L/s	121.8 L/s

However, as part of the EA, it is understood that certain alternatives could also include water conservation measures that reduce the water design criteria (per capita consumption rate, non-revenue water %, peaking factor, etc.).

Various alternatives that balance increased supply and reduced water demands will be considered as part of the Class EA.

2.3 STORAGE NEEDS

As detailed in Study 1A: Water System Capacity Optimization Study, the existing storage capacity of the Nobleton system is sufficient to meet the fire, emergency and equalization storage requirements that correspond to an MDD in Nobleton of up to 86.85 L/s. Since the projected maximum day demand is slightly higher (89.5L/s), a marginal amount of additional storage would ultimately be required. However, it is unlikely that a new storage facility would be added to make up such a small deficit. Therefore, water conservation measures (to reduce the maximum day demand to below 86.85L/s) will be considered. Alternatively, additional supply capacity could be used to offset any minor storage deficits by pumping some of the equalization storage.

2.4 DISTRIBUTION / TRANSMISSION NEEDS

Based on the hydraulic analysis of the system, there are no system bottlenecks or limitations that are preventing the Region's well supply and storage volume to be distributed to the Township of King owned infrastructure in Nobleton.

The only Regional watermains that may need to be added are related to the ultimate location of a new Nobleton well and the potential expansion of the Nobleton Well #2. When evaluating alternate well locations, the required connecting watermain will need to be established and documented.

2.5 OPTIMIZATION OPPORTUNITIES

Using spare storage capacity was evaluated in the Water System Capacity Optimization Study Report. However, since maximum day demands often occur for multiple days in a row during drought conditions, using surplus storage to meet supply deficits is not recommended.

3 Opportunity Statement

The Opportunity Statement is required for the future Class EA for Water/Wastewater Servicing in the Community of Nobleton. The Opportunity Statement is common for water and wastewater infrastructure, and therefore, the below sections are reflective of that. Additional information on the wastewater system is included in the Wastewater Needs Assessment and Justification Study (Black & Veatch, 2018).

3.1 DEVELOPMENT OF OPPORTUNITY STATEMENT

Factors considered in the development of the problem statement include:

- Between 1996 and 2001, Nobleton was subject to an EA process that resulted in shifting the population from existing septic systems to servicing by the Nobleton Water Resource Recovery Facility. Though the Nobleton system is designed to accommodate approved growth in the community to a population of 6,500, the planning context has changed substantially since that time. Growth and development in York Region and King Township are now subject to the Greenbelt Plan and Growth Plan for the Greater Golden Horseshoe and planning to the 2041 growth horizon now envisions a population of 10,800 in Nobleton;
- In drafting its current Official Plan, King directed the majority of new growth to the communities of Schomberg and King City. Allowing more growth in Nobleton will require an Official Plan Amendment that has not yet been presented to the public;
- The holistic approach of the EA will require informing residents about Alternative Servicing Solutions that are technical in nature, requiring effective and user-friendly communications;
- The existing water supply facilities are inadequate to support the population increase and so new infrastructure and/or innovative water practices are required. Existing storage facilities are adequate to support the population increase. The existing water supply and storage facilities are adequate to support the existing population.

3.2 DRAFT OPPORTUNITY STATEMENT

"Identify innovative, safe and reliable water and wastewater servicing solutions for the community of Nobleton in King Township, to support approved population growth from 6,500 to 10,800, while optimizing the use of existing systems. The preferred solution must be socially, environmentally and financially sustainable."

DRAFT

WASTEWATER NEEDS ASSESSMENT & JUSTIFICATION STUDY

Study 3B

B&V PROJECT NO. 196238

PREPARED FOR

Regional Municipality of York

30 MAY 2019

BLACK & VEATCH

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LIST OF FIGURES

Distribution List

BV FILE NO.	RevNo	Issued to	Date	Reason for Issue
42.3220	0	York Region – Afshin Naseri	January 17, 2019	Draft for review
42.3230	1	York Region – Afshin Naseri	May 30, 2019	Final

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Black & Veatch Signatures

Prepared By

____

Andisheh Yazdanpanah
Civil EIT

Reviewed By ____

Zhifei Hu
Process Engineer

1 Introduction

1.1 PURPOSE OF STUDY

The purpose of the Wastewater Needs Assessment & Justification Study is to:

- Identify needs or gaps in wastewater servicing (collection and treatment) to support growth in Nobleton to 10,800 people
- Identify any required system improvements to meet future growth
- Determine feasibility of servicing to future growth targets
- Using the Needs Assessment and Justification analyses, develop draft Opportunity Statement to feed into the Class EA upon initiation

1.2 BACKGROUND

Nobleton is a community in King Township in the Regional Municipality of York (Region). Currently, Nobleton is serviced by stand-alone water and wastewater systems. The Regional Water and Wastewater Master Plan (2016) indicated that both the water and wastewater systems would not have sufficient capacity to support growth to the 2041 Master Plan horizon. Therefore, the Master Plan recommended undertaking the current project, a Schedule C Class Environmental Assessment (EA), to identify preferred servicing solutions to accommodate growth (York Region, 2016).

1.3 EXISTING WASTWEATER SYSTEM INFRASTRUCTURE

1.3.1 Wastewater Collection System

The Nobleton wastewater collection system is comprised of a gravity sewage system which includes two pumping stations: Bluff Trail PS (located in the northeast of the catchment) and Janet Avenue PS (placed toward the south of the catchment). The Janet Avenue PS pumps all of the flows from the catchment to the Nobleton WRRF.

1.3.2 Wastewater Treatment Plant

The Nobleton WRRF is an extended aeration plant with tertiary filtration. The rated capacity defined by ECA is 2,925 m^3/day with a peak design flow of 9,177 m^3/day. The plant was originally designed to service 6,500 people. Based on a capacity assessment, the Region later granted the increase of the service population to 6,590 people.

1.4 RELEVANT PLANS AND POLICIES

1.4.1 Regional Official Plan

The Region continues to experience rapid population and employment growth. In accordance with the York Region Official Plan 2010, significant population growth is expected within the next 25 years, to the planning horizon of 2031 (York Region).

The York Region Official Plan has forecasted a population growth within King Township from 20,300 people in 2006 to 34,900 people in 2031. This represents an increase of 14,600 people.

To meet this objective, the following Policies are outlined in the Region's Official Plan:

- 7.3.15 - That development within and expansions to the urban uses within Towns and Villages will occur on the basis of full municipal wastewater treatment services where such facilities currently exist. For existing or previously approved development in Towns and Villages, wastewater treatment services will be continued where feasible and in keeping with the provisions of local official plans and this Plan.

- 7.3.16 - That within the Oak Ridges Moraine, Greenbelt, and Lake Simcoe watershed, all improvements or new wastewater infrastructure systems shall conform with the Oak Ridges Moraine Conservation Plan, the Greenbelt Plan or the Lake Simcoe Protection Plan.

- 7.3.17 - That the construction or expansion of partial services is prohibited in the Oak Ridges Moraine unless it has been deemed necessary to address a serious health or environmental concern identified by the Medical Officer of Health or other designated authority.

- 7.3.18 - To provide reliable wastewater services to residents and businesses to ensure continuing community well-being and the economic vitality of the Region.

- 7.3.25 - To ensure that wastewater effluent is managed to minimize impacts on the quality of the receiving water.

- 7.3.30 - That the planning and design of wastewater infrastructure will consider potential impacts from climate change.

- 7.3.31 - To ensure secure and resilient Regional wastewater systems to maintain continual service.

- 7.3.32 - That wastewater services will be planned, constructed and operated in a manner that protects, enhances, and provides net benefit to the Region's natural and cultural heritage.

- 7.3.34 - That the wastewater systems be sized to consider the potential for expansion of the service area, intensification and increased allocation where permitted by York Region Master Plans and Provincial Plans.

The Official Plan is relevant to the Class EA study since it outlines the policies that guide the economic, environmental and community building decisions to manage growth. It emphasizes the need to develop water and wastewater services that support the economic growth of the Region while protecting the Region's natural and cultural heritage.

1.4.2 York Region Corporate Strategic Plan

The 2015-2019 York Region Strategic Plan is a roadmap that guides toward the vision of the future. It serves as a plan to get the Region from where they are to where they want to be in 2051 and focuses on Economic Vitality, Healthy Communities, Sustainable Environment and Good Government.

The key Regional Performance Measures listed in the Strategic Plan that relate to the Nobleton Wastewater Servicing Class EA is to reduce quantity of inflow and infiltration in Regional and local wastewater systems.

The Region's Corporate Strategic Plan is relevant to the Class EA because it emphasizes key performance measures for water and wastewater systems that should be used as a vision for the future including an emphasis on reducing inflow and infiltration and reducing residential water demands.

1.4.3 King Township Draft Official Plan

The "Parent Official Plan" (Township of King, 1970) is the current King Township Official Plan which was approved in 1970. This document establishes land use, transportation, and development policies for King Township.

In 1997, the Nobleton Community Plan was added to the King Township Official Plan through Official Plan Amendment 57 and adopted by the Regional Council; the latest Office Consolidation was in 2005.

The King Township Official Plan is relevant to the Class EA because it specifies the limitations and framework for Nobleton's population growth.

1.4.4 Water and Wastewater Master Plan

The Region updated the Regional Water and Wastewater Master Plan in November 2016. The objectives of this update are:

- to determine the water and wastewater infrastructure requirements needed to support provincially mandated growth forecasts and proposed community expansion; and
- to develop a long-term strategy to ensure that the Region continues to serve its residents in an environmentally and economically sustainable manner (York Region, 2016).

The Master Plan recommended conducting a Schedule C Class EA project to provide alternatives to increase the water supply capacity to support proposed community expansion to about 9,500 people by 2041 through either addition of new wells and/or revision of existing Ministry of the Environment, Conservation and Parks (MECP) Permit to Take Water (PTTW). Similarly, a Schedule C Class EA project was also recommended for wastewater servicing.

The Region's Water and Wastewater Master Plan is relevant to the Class EA because it serves as the guiding document on water and wastewater system investments to 2041. It specifically mentions the desire to continue servicing stand-alone systems as stand-alone systems.

1.4.5 Provincial Policy Statement

The Provincial Policy Statement provides policy direction on matters of provincial interest related to land use planning and development. As a key part of Ontario's policy-led planning system, the Provincial Policy Statement sets the policy foundation for regulating the development and use of land. It also supports the provincial goal to enhance the quality of life for all Ontarians.

The following key policies from the 2014 Provincial Policy Statement are summarized below:

- 1.6.6.1 - Planning for sewage and water services shall:
    - direct and accommodate expected growth or development in a manner that promotes the efficient use and optimization of existing: 1. municipal sewage services and municipal water services; and 2. private communal sewage services and private communal water services, where municipal sewage services and municipal water services are not available;
    - ensure that these systems are provided in a manner that: 1. can be sustained by the water resources upon which such services rely; 2. is feasible, financially viable and complies with all regulatory requirements; and 3. protects human health and the natural environment.

The Provincial Policy Statement is relevant to the Class EA because it again emphasizes the need to develop water and wastewater services to meet the expected growth, while sustaining our water resources and protecting the natural and cultural environment.

2 Needs Assessment and Justification

2.1 EXISTING SYSTEM DESCRIPTION

2.1.1 Wastewater collection system

The Nobleton wastewater collection system consists of a gravity sewage system which includes two pumping stations: Bluff Trail PS (in the northeast of the catchment) and Janet Avenue (toward the south of the catchment).

Table 2-1 shows the historical data (years 2014 to 2017) for the total volume that was pumped to the Nobleton WRRF. The number of properties connected to the sewer system has increased every year which has resulted in the average daily flows to increase on a yearly basis. The peak daily volume, however, is depended on the size of the largest rainfall event happening that year and thus, it varies for each year (Figure 2-1).

Table 2-1 Historical Daily Pumped Volumes to Nobleton WRRF

YEAR	2014	2015	2016	2017
Average Daily Pumped Volume (m^3)	864	978	1,100	1,380
Maximum Daily Pumped Volume (m^3)	1.950	1,780	2,550	3,890
Minimum Daily Pumped Volume (m^3)	620	732	806	1,086

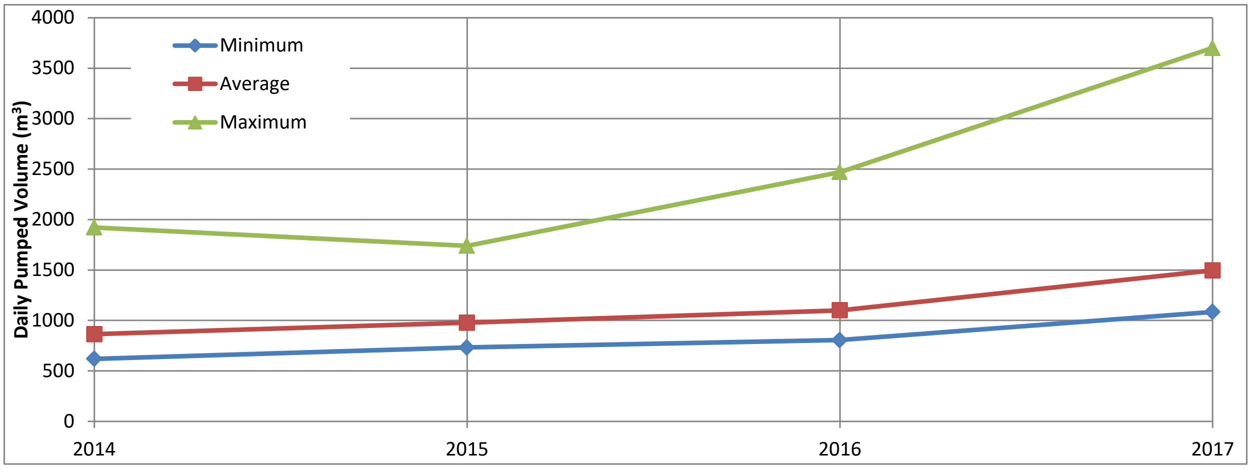

Figure 2-1: Historical Nobleton Minimum, Average, and Maximum Pumped Volumes

2.1.2 Janet Avenue Pumping Station

The role of Janet Avenue PS is to pump the flows from the community of Nobleton to the WRRF. The flows enter an inlet chamber before draining through one of three orifices into a wet well with an storage volume of 20 m^3. There is a larger area above the wet well where the flow can fill during wet weather. There is also a dry well with three dry pit submersible non-clog pumps operating on a two-duty and one-stanby regime, with an existing capacity of 53 L/s at 54 TDH for each pump, resulting in a firm capacity of 106 L/s or 9,158 m^3/d. Flooding is prevented by an emergency overflow in case there are issues with the pumps.

The existing forcemain is a 300mm polyvinyl chloride (PVC) pipe and delivers the flow from the Janet Avenue PS to the Nobleton WRRF. It rises from the PS at Janet Avenue to a peak level of 284.02 mAD along King Road before dropping down to a level of 242.25 mAD at the Nobleton WRRF.

2.1.3 Wastewater Treatment Plant

The Nobleton WRRF consists of an extended aeration plant with tertiary filtration. The rated average day flow capacity is 2,925 m^3/day with a peak design flow of 9,177 m^3/day. The plant was originally designed to service 6,500 people and the Region granted to increase to 6,590 people based on capacity assessment.

2.1.4 Historical Wastewater Flows and Generation Rates

The average day flow (ADF) and average dry weather flow (ADWF) along with the average wastewater generation rates for 2014 to 2017 are summarized in Table 2-2. The detailed discussion on flow rates and population in service are included in Study 1B: Wastewater System Capacity Optimization Study Report.

Table 2-2: Summary of Historical Wastewater Generation Rates

YEAR	POPULATION IN SERVICE	AVERAGE DRY WEATHER FLOW (ADWF)		ANNUAL AVERAGE DAY FLOW (ADF)	
		Flow	Generation Rate	Flow	Generation Rate
2014	2,923	0.83 MLD	284 L/c/d	0.88 MLD	300 L/c/d
2015	3,119	0.95 MLD	304 L/c/d	0.99 MLD	318 L/c/d
2016	3,643	1.03 MLD	283 L/c/d	1.14 MLD	313 L/c/d
2017	3,891	1.32 MLD	340 L/c/d	1.45 MLD	374 L/c/d
	Average:		303 L/c/d		326 L/c/d

The data suggest that Year 2017 has the highest annual average day flow (ADF) of 374 L/c/d and highest annual average dry weather flow (ADWF) of 340 L/c/d.

The historical flows into the Nobleton WRRF from January 2014 to December 2017 are used to determine the following flow variations which are summarized in Figure 2-3.

Table 2-3: Summary of Historical Raw Sewage Flows and Peaking Factors into the Nobleton WRRF

YEAR	ADF	MMF(1) (PEAKING FACTOR)	PDF (PEAKING FACTOR)	PIF (PEAKING FACTOR)	PHF(2) (PEAKING FACTOR)
2014	0.88 MLD	1.20 MLD (1.4)	1.95 MLD (2.2)	5.26 MLD (6.0)	4.10 MLD (4.7)
2015	0.99 MLD	1.30 MLD (1.3)	1.78 MLD (1.8)	7.32 MLD (7.4)	4.10 MLD (4.1)
2016	1.14 MLD	1.77 MLD (1.6)	2.55 MLD (2.2)	6.60 ML D (5.8)	4.77 MLD (4.2)
2017	1.45 MLD	1.99 MLD (1.4)	3.89 MLD (2.7)	8.83 MLD (6.1)	8.60 MLD (5.9)
Average Peaking Factor		**1.4**	**2.2**	**6.3**	**4.7**

Notes;
Sources: SCADA Data: RSHW_FIT1
(1) Maximum Monthly Flow was determined using a 30-day moving average.
(2) Peak Hourly Flow based off the hourly average of the Peak Instantaneous Flow (5-min Flow), using a moving average of 12

2.2 COLLECTION SYSTEM NEEDS

Based on the hydraulic model of the sewer system, it is concluded that most of the existing system has sufficient capacity to drain the current flows and the future projected flows to the Janet Avenue PS. The analysis shows that there are some locations within the trunk sewer where surcharging is predicted to occur, but no flooding is predicted as the water levels in the surcharged trunk sewer will still be below the ground level.

At an observed peaking factor of 6.3 for the peak instantaneous flow, the Janet Avenue PS has an equivalent ADF capacity of 1,430 m^3/d and an equivalent serviceable population of 3,865 persons.

This is based on the assumption that the peak instantaneous flow would last until the wet well operating level reaches the high operating level. The detailed assessment is included in Study 1B: Wastewater System Capacity Optimization Study Report.

The existing forcemain from the Janet Avenue PS has insufficient capacity to accommodate the future peak flows from the collection system.

2.3 NOBLETON WRRF NEEDS

2.3.1 Future Wastewater Flow Projections

Nobleton WRRF is expected to service a population growth of up to 10,800 people in 2041. According to the historical data, average day flow and peaking factors to project future flows are summarized in Table 2-4. The detailed flow projections are included in Study 1B: Wastewater System Capacity Optimization Study Report.

Table 2-4: Wastewater Flow Projection

DESIGN FLOW CRITERIA	BASELINE (2017)	FUTURE
Residential Population	3,891	10,800
Wastewater Generation Rate	370 L/c/d	370 L/c/d
Average Day Flow Capacity	Rated capacity in ECA: 2,925 m^3/day	3,996 m^3/day
Peaking Factors Maximum Month Flow (MMF) Peak Day Flow (PDF) Peak Hour Flow (PHF) Peak Instantaneous Flow (PIF)	 1.4 2.2 4.7 6.3	 1.4 2.2 4.7 6.3

A value of 370 L/c/d is recommended for both the existing population and future growth. Based on this value, the future average wastewater flow for a future population of 10,800 (provided by the Region) is calculated to be 3,996 m^3/d.

2.3.2 Existing Nobleton WRRF Capacity Assessment Summary

The Nobleton WRRF experiences high PHF and PIF, with an average peaking factor of 4.7 and 6.3, respectively, which are significantly higher than peaking factor of 3.14 used in 2007 design. As a result, the capacities of some process units are less than the currently rated capacity of 2,925 m^3/d (Figure 2-2: Unit Process Equivalent ADF Capacities and Serviceable Population). The detailed capacity assessment is included in Study 1B: Wastewater System Capacity Optimization Study Report.

Figure 2-2: Unit Process Equivalent ADF Capacities and Serviceable Population summarizes the existing and future capacity of various unit processes of Nobleton WRRF, calculated based on the peaking factors identified with the historical data. Based on Figure 2-2, the ADF capacity the existing Nobleton WRRF is approximately 1,457 m^3/d.

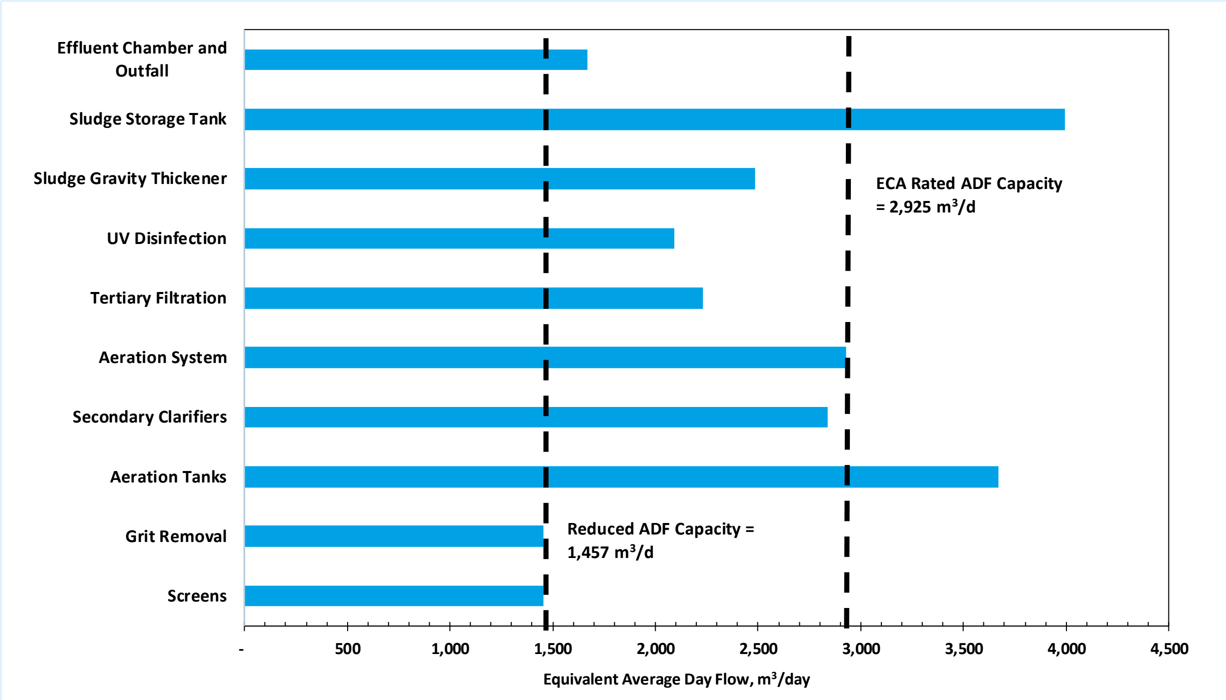

Figure 2-2: Unit Process Equivalent ADF Capacities and Serviceable Population

2.4 FUTURE CAPACITY NEEDS SUMMARY

The existing Nobleton collection system and the WRRF experience high peak hourly (PHF) and instantaneous flows (PIF), with an average peaking factor of 4.3 and 6.3, respectively. These peaking factors are significantly higher than peaking factor of 3.14 used in 2007 design. As a result, the capacities of some process units are less than the currently rated capacity of 2,925 m^3/d, including:

- The Janet Avenue PS has an equivalent ADF capacity of 1,430 m^3/d and an equivalent serviceable population of 3,865 persons.
- The existing Nobleton WRRF has an ADF capacity of approximately 1,457 m^3/d limited based on the screening capacity and grit removal tanks. This capacity is an equivalent serviceable population of 3,938 persons.

Therefore, there is a need to provide additional wastewater service capacity for the Janet Avenue PS and the Nobleton WRRF.

2.5 OPTIMIZATION OPPORTUNITIES

The Nobleton wastewater collection system and the WRRF experience peak flows higher than the design values. The collection and WRRF could be optimized through rain derived inflow/infiltration (RDII) reduction and peak flow management.

3 Opportunity Statement

The Opportunity Statement is required for the future Class EA for Water/Wastewater Servicing in the Community of Nobleton. The Opportunity Statement is common for water and wastewater infrastructure, and therefore, the below sections are reflective of that. Additional information on the wastewater system is included in the Wastewater Needs Assessment and Justification Study (Black & Veatch, 2018).

3.1 DEVELOPMENT OF OPPORTUNITY STATEMENT

Factors considered in the development of the problem statement include:

- Between 1996 and 2001, Nobleton was subject to an EA process that resulted in shifting the population from existing septic systems to servicing by the Nobleton Water Resource Recovery Facility. Though the Nobleton system is designed to accommodate approved growth in the community to a population of 6,500, the planning context has changed substantially since that time. Growth and development in York Region and King Township are now subject to the Greenbelt Plan and Growth Plan for the Greater Golden Horseshoe and planning to the 2041 growth horizon now envisions a population of 10,800 in Nobleton;
- In drafting its current Official Plan, King directed the majority of new growth to the communities of Schomberg and King City. Allowing more growth in Nobleton will require an Official Plan Amendment that has not yet been presented to the public;
- The holistic approach of the EA will require informing residents about Alternative Servicing Solutions that are technical in nature, requiring effective and user-friendly communications.

3.2 DRAFT OPPORTUNITY STATEMENT

“Identify innovative, safe and reliable water and wastewater servicing solutions for the community of Nobleton in King Township, to support the tentative population growth from 6,500 to 10,800, while optimizing the use of existing systems. The preferred solution must be socially, environmentally and financially sustainable.”

DRAFT

TECHNOLOGY OPTIONS TO MEET RECEIVING WATER QUALITY STUDY

Study 4

B&V PROJECT NO. 196238

PREPARED FOR

Regional Municipality of York

29 APRIL 2021

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List of Abbreviations

ADD	Average Day Demand
ADF	Average Day Flow (Annual)
EA	Environmental Assessment
I/I	Inflow and Infiltration
km	Kilometer
L/s	Liters per second
MECP	Ministry of Environment, Conservation and Parks
m^3/day	cubic meters per day
MDD	Max Day Demand
ML	Million Litres
MLD	million liters per day
PDF	Peak Day Flow
PF	Peak Factor
PHF	Peak Hourly Flow
PIF	Peak Instantaneous Flow
pp	Persons
PS	Pumping Station
PTTW	Permit to Take Water
RDII	Rainfall Derived Infiltration and Inflow
TM	Technical Memorandum
WWF	Wet Weather Flow
WRRF	Water Resource Recovery Facility

1.0 Introduction

Wastewater treatment consists of multiple processes in sequence to transform raw sewage into a treated effluent that satisfies all requirements of the ECA. The most critical process for achieving the desired effluent quality is the secondary biological treatment process. It is largely responsible for the quality of treated effluent discharged. Upstream processes remove debris and particulate matter through straining or sedimentation. Downstream processes remove particulate matter remaining after secondary treatment and eliminate pathogens.

The existing Nobleton WRRF consists of the following processes:

- Preliminary Treatment Screening – Coarse screens
- Preliminary Treatment Grit Removal – Induced vortex
- Secondary Biological Treatment – Extended Aeration
- Nutrient Removal – Chemical with alum
- Tertiary Treatment – Deep bed sand filtration
- Disinfection – UV disinfection
- Sludge Thickening – Gravity thickening
- Sludge Storage – Aeration sludge storage

Treated effluent is discharged to the Humber River. Residual solids are hauled to Aurora.

The existing wastewater treatment processes have performed well and produce an effluent in compliance with the requirements of the ECA. Furthermore, the equipment is functional and still within the expected service life. The main reason for the project is to service the projected population growth. Nonetheless, it is worthwhile to identify feasible alternatives to the existing technologies that will satisfy treatment requirements with the lowest overall cost.

1.1 Purpose of the Study

The purpose of this study is to screen the long list of technology alternatives for each wastewater treatment process. Screening and evaluation is performed according to the method described in Section 3 of TM3.

Each process is covered in sequence in the sections that follow. The long list of technology alternatives is described, and the alternatives are screened according to the method described in Section 3 of TM3. Technologies that pass the screening are evaluated in Section 5 of TM3.

2.0 Preliminary Treatment - Screening

The purpose of screening is to remove bulk materials from the wastewater to prevent interference with downstream equipment and to improve aesthetics of hauled residual materials.

2.1 Long List of Alternative Design Screening Technologies

Coarse screen technology is currently used at Nobleton WRRF. Fine screen technology is not used but may be required for some secondary biological treatment technologies.

2.1.1 Coarse Screening Equipment

The primary purpose of coarse screening is to remove objects and debris larger than ½ inch (12 mm) in size from the wastewater stream to protect the downstream influent pumps. Coarse screening options are largely dependent on the depth and configuration of downstream process as this dictates the depth and available space from which coarse screenings must be captured and lifted to the surface for handling and disposal. As such, shallower conveyance alternatives around 50 ft in depth or less, like the force main and gravity micro-tunnel alternatives, are better suited for conventional mechanically raked bar screens. Alternatives of greater depth, like those with a large diameter tunnel, are better suited for a deep tunnel bar screen with a specialized rake design. Each of these coarse screen technologies is described in more detail below.

Application depends on the downstream treatment processes. Coarse screens are adequate for conventional secondary biological treatment processes. Fine screens may be required for some secondary biological treatment technologies.

2.1.1.1 Climber/Crawler Bar Screens

A single 600-mm climber screen is installed in existing inlet works area of the Nobelton WRRF Process Building.

A climber/crawler bar screen is a conventional mechanically raked bar screen that uses a single mechanical raking mechanism (climber/crawler) to clean the screen. Climber/crawler bar screens for coarse screening applications can be provided with ½ to 3-inch (12 mm to 75 mm) spacing and have no mechanical components permanently located under water. In lieu of chains and a lower sprocket, these screens have wheels that move along a heavy-duty pin rack. As the rake assembly rotates around the lower end of the heavy-duty pin rack, the teeth on the raker heads engage the bar rack and collect debris as the rake assembly ascends back up the screen to the point of discharge. Once at the point of discharge, the wiper blade cleans the rake head and discharges screenings into a conveyor, compactor, or dumpster.

When compared to a multiple rake bar screen as described in the next section, a climber/crawler bar screen takes longer to clean because it only includes one rake; therefore, the travel time needs to be considered when utilizing this type of screen to ensure the screen doesn't become blinded before the rake returns from its cleaning pass. Combined sewer overflow applications typically are more prone to a rapid influx of coarse debris (e.g. leaves) which could blind a climber/crawler screen in the time it takes for the raking mechanism to pass through an entire cleaning cycle.

Wastewater treatment applications not tied to a combined sewer system, while prone to traditional inflow and infiltration during wet weather, would likely be less susceptible to a rapid influx of coarse debris.

There are several manufacturers that offer climber/crawler bar screens including Infilco Degremont and Vulcan Industries, examples of which are shown on Figure 2-1, along with WSG & Solutions, and WTP Equipment Corporation, the supplier of the existing screen, and others.

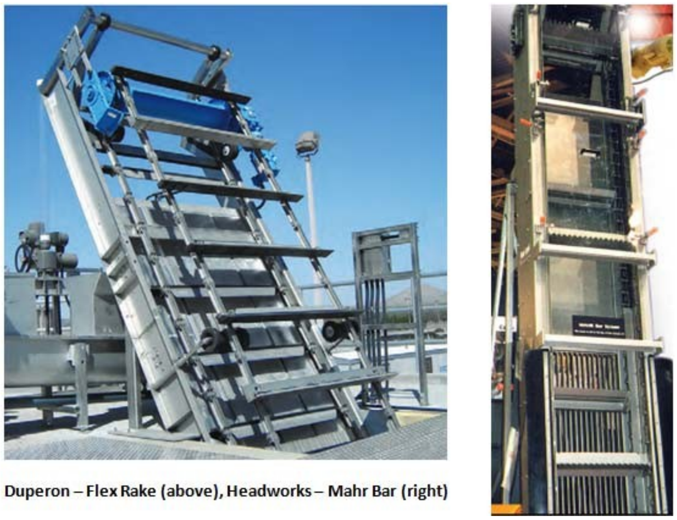
Infilco Degremont – Climber Screen   Vulcan Industries – Mensch Crawler

Figure 2-1 Climber/Crawler Bar Screens

2.1.1.2 Multi-Rake Bar Screens

A multi-rake bar screen is a conventional mechanically raked bar screen that uses a series of rakes to clean the screen. Multi-rake bar screens for coarse screening applications can be provided with ½ to 6-inch spacing. These types of screens are chain driven and include a lower submerged sprocket, with the exception of the Duperon Flex Rake as shown on Figure 2-2, which does not include a submerged sprocket. Multi-rake bar screens are less prone to blinding given their higher frequency of cleaning, with rakes engaging the screen as often as every 5 to 10 seconds. The rakes travel in a continuous circuit from the bottom of the channel, up the bar rack, and past the debris plate. The screenings are scraped off the rake into the discharge chute and dropped into a conveyor, compactor, or dumpster.

There are a number of manufacturers that offer this equipment including Duperon and Headworks International, which are shown on Figure 2-2, along with JWC Environmental (like those currently installed at DRPTP), Huber Technology Inc., HydroDyne, Vulcan Industries, and Wastetech.



Duperon – Flex Rake (above), Headworks – Mahr Bar (right)

Figure 2-2 Multi-Rake Bar Screens

2.1.1.3 Deep Raker Screen

A deep raker screen is a specialized mechanically raked bar screen designed for deep applications up to depths of 250 feet or greater. Deep raker screens can be provided with ½ to 6-inch bar spacing and range from 10 to 30 feet in height in single or double rack systems. The cleaning mechanism is operated by an overhead hoist and trolley system and consists of a gripper that engages with the bars and descends to the bottom of the screen while collecting debris in its jaws during the descent. When the gripper reaches the bottom of the screen, it closes and the hoist raises it back up to the trolley at grade. The trolley and gripper then travel to the discharge area where the gripper opens, releasing the debris directly into a dumpster.

There are a limited number of manufacturers that provide these types of specialized screens. Fairfield Service Company, Ovivo, and Kuenz are the known manufacturers operating in the U.S. Figure 2-3 depicts the Bosker Deep Raker screen by Ovivo.

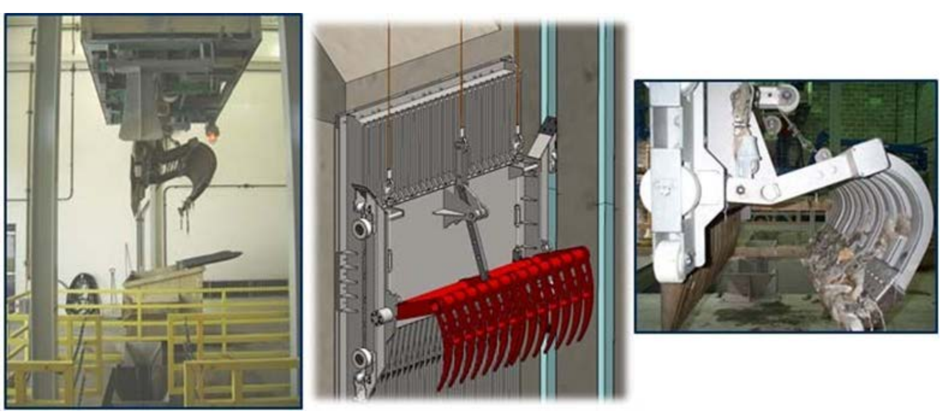


Figure 2-3 Deep Raker (Ovivo - Bosker)

2.1.1.4 Coarse Screening Equipment Advantages and Disadvantages

Table 2-1 summarizes the advantages and disadvantages of the coarse screening technologies described in this section.

Table 2-1 Advantages and Disadvantages of Coarse Screening Technologies

Technology	Status	Advantages	Disadvantages
Force Main or Micro Tunnel			
Climber/Crawler Bar Screen	Conventional: This is a mature technology that is widely used.	• No major submerged mechanical components • Rugged construction • Minimal operator attention required • Multiple manufacturers	• Requires higher overhead clearances • Can clog or be damaged by large and heavy debris • Single rake more prone to blinding during high solids loadings
Multi-Rake Bar Screen	Conventional: This is a mature technology that is widely used.	• Multiple manufacturers • Less prone to blinding during high solids loading • Rugged construction • Less headroom required • Minimal operator attention typically required • Duperon Flex Rake does not have a lower sprocket and can flex around large debris to prevent jamming	• Lower submerged sprocket (except Duperon Flex Rake) may require in-channel maintenance
Deep Micro Tunnel or Large Diameter Tunnel			
Deep Raker Screen	Conventional: This is a mature technology that is widely used.	• Material handling system included (raking, conveyance, and debris-loading) • Minimal operator attention • Rugged construction	• Limited number of manufacturers • Single gripper/rake more prone to blinding during high solids loadings

2.1.2 Fine Screening Equipment

Fine screens are required for many of the secondary biological treatment intensification technologies. The existing coarse screens would be replaced in the event the selected Wastewater Design Concept includes biological treatment intensification.

2.1.2.1 Perforated Plate Screen

A perforated plate screen is a type of self-cleaning, in-channel screening device utilizing perforated plate media with 1/16-inch to ¼-inch spacing and no submerged bearings. All of the perforated plate screens are moving screens that trap media and transfer it up to the discharge point, with the exception of the Duperon FlexRake Perforated Fixed-Element screen. This screen operates similarly to a multi-rake bar screen, in which the actual screen is stationary and plate panels rotate to collect and transport the screenings to the discharge point. At the discharge point for perforated plate screens, the screenings are either discharged by gravity or cleaned with a brush assembly and water spray. The movement of the screen (or plate panels) can be continuous or intermittent, depending on the manufacturer. Some manufacturers have a continuous screening belt and some recommend intermittent movement of the belt or plate panels so solids are able to build up on the screen to increase capture rate. Perforated plate screens are more widely used than step screens, but typically introduce higher headloss.

There are several manufacturers of perforated plate screens including Duperon, Headworks Inc., Huber Technology, John Meunier, JWC Environmental, Parkson (shown on Figure 2-4), WSG & Solutions, WesTech, and others.

Parkson – "Aquaguard PF" Perforated Plate

Figure 2-4 Perforated Plate Screen

2.1.2.2 Step Screen

A stair/step screen is a type of self-cleaning, in-channel screening device that operates on a system of alternating fixed and movable stair-shaped screening elements with 1/32-inch to ¼-inch spacing and no submerged bearings. Debris is collected on the "steps" and forms a mat which acts as a filter to remove particles that would otherwise pass between the screens. When the headloss reaches a predetermined value, the movable steps are activated to rotate upward to lift the debris to the next highest step level. This slow progress from channel to discharge point allows the debris to shed water while suspended on the stair. Eventually the debris reaches the discharge point where it is mechanically forced off the screen by the movable screen without the need for brushes or spray systems. Screenings are then discharged to a conveyor, compactor, or dumpster. Step screens are not as widely used as perforated screens, but typically introduce lower headloss.

There are several manufacturers of step screens including John Meunier, Parkson, Vulcan, WesTech (shown on Figure 2-5), and others.

WesTech – Mono Screen Step Screen

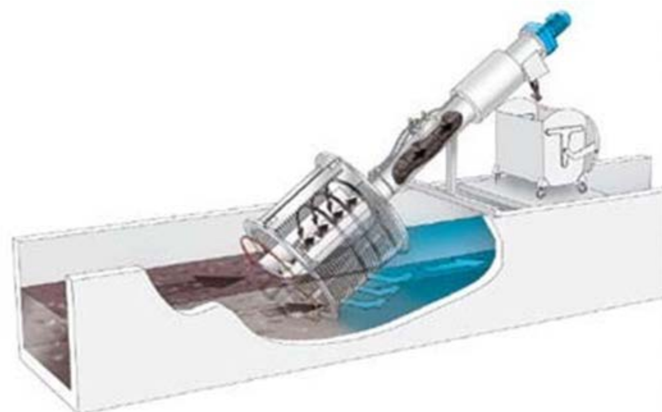
Figure 2-5 Step Screen

2.1.2.3 Rotary Drum Screens

A rotary drum screen is a type of self-cleaning fine screen in a drum arrangement with a perforated plate screen having 1/16-inch to 3/8-inch openings. Most manufacturers also offer a wedge wire type rotary drum screen with smaller openings down to 1/32-inch. Rotary drum screens are typically internally fed units similar to the JWC unit shown on Figure 2-6, where influent enters a headbox or distribution tray and then directed into the rotating drum screen. As the influent hits the rotating screen, the solids are caught inside the drum cylinder and the liquid passes through to the outside. Diverters on the drum screen move the solids along the length of the screen to the discharge end of the drum where they are discharged into a conveyor, compactor, or dumpster. Units are equipped with spray bars for cleaning.

Huber offers a unit that can be installed either directly in a channel, as shown on Figure 2-6, or in a separate tank. Wastewater influent flows into the open end of the inclined screen basket where screenings are captured and screened wastewater passes through. When the headloss reaches a predetermined value, the rake arm situated on the center axle starts to rotate. While rotating, its tines, which are extended completely through the screen bars, clean the basket to remove all the screenings from the drum. Screenings are collected into the center trough housing a screw conveyor and then transported out of the trough into an inclined pipe. As the screenings are pushed through the inclined pipe, they are dewatered and compacted prior to discharging into a conveyor or dumpster.

There are several manufacturers that supply rotary drum screens including Andritz, Huber Technology Inc., JWC Environmental, Parkson, and WesTech. The Huber and JWC Environmental units are shown on Figure 2-6.



Huber – Fine Screen ROTAMAT Ro1

JWC Environmental – IFO Internally Fed Rotary Screen

Figure 2-6 Rotary Drum Screens

2.1.2.4 Catenary Screens

A catenary screen (shown in Figure 2-7) is a variation of the traditional front-cleaned, front-return chain and rake screen. The catenary screen has the advantage of having no submerged sprockets that could be damaged or blocked by large solids that are common during high flow events. The headroom requirements for the catenary screen are also typically less than that for other screen types. The bar rake is held against the rack by weight of a heavy chain. If a large object does become lodged in the bars, the rakes pass over the objects instead of jamming. The downside is that catenary screens require a larger installation footprint compared to many other types of screens. Additionally, catenary screens are typically lighter duty compared to chain and rake screens.

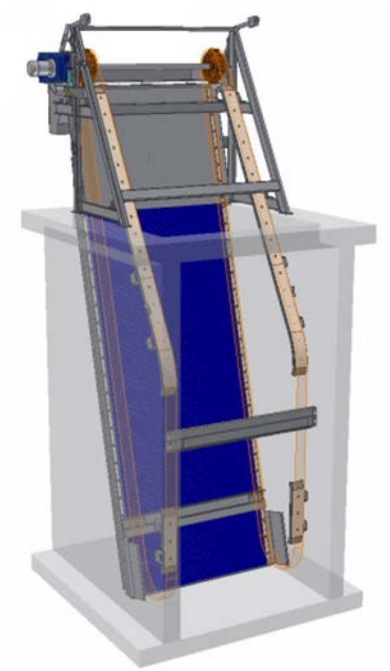


Figure 2-7 Catenary Screen

2.1.2.5 Continuous Belt Screens

The continuous belt screen (shown in Figure 2-8) is a relatively new type of screen used in the United States. Continuous belt screens are self-cleaning belts that can remove coarse and/or fine screenings. A large number of rakes are attached to the belt that clean the screen faster than single rake climber screens. The frequent cleanings also lowers the headloss through the screen. Most continuous belt screens have no major maintenance items located below the water level, which improves the ease of maintenance. The rake has multiple plastic pieces that can wear, especially in the presence of grit. Depending on the characteristics of the wastewater, these screens might not be suitable if there is a high concentration of grit. The rake may also be limited in handling large or heavy debris.

Figure 2-8 Continuous Belt Screen

2.1.2.6 Fine Screening Equipment Advantages and Disadvantages

Table 2-2 summarizes the advantages and disadvantages of the fine screening technologies.

Table 2-2 Advantages and Disadvantages of Fine Screening Technologies

Technology	Status	Advantages	Disadvantages
Combination Coarse and Fine Screens			
Climber/Crawler Bar Screen	Conventional: This is a mature technology that is widely used.	• No major submerged mechanical components • Rugged construction • Minimal operator attention required • Multiple manufacturers	• Requires higher overhead clearances • Can clog or be damaged by large and heavy debris • Single rake more prone to blinding during high solids loadings
Multi-Rake Bar Screen	Conventional: This is a mature technology that is widely used.	• Multiple manufacturers • Less prone to blinding during high solids loading • Rugged construction • Less headroom required • Minimal operator attention typically required • Duperon Flex Rake does not have a lower sprocket and can flex around large debris to prevent jamming	• Lower submerged sprocket (except Duperon Flex Rake) may require in-channel maintenance
Stand Alone Fine Screens			
Perforated Plate Screens	Conventional: This is a mature technology that is widely used.	• Major maintenance items located above water surface • Captures fine screenings and grit with opening sizes down to 1/16-inch • More widely used than step screens with a number of manufacturers • Can be installed in existing channel	• Frequent maintenance can be required for plate cleaning • More prone to blinding during high solids loading given fine solids capture • Less rugged construction than combination coarse and fine screens • Higher headloss than step screens • Compactor required due to wash water

Technology	Status	Advantages	Disadvantages
Step Screen	Conventional: This is a mature technology that is widely used.	• Less headloss than perforated plate screens • Captures fine screenings and grit with opening sizes down to 1/32-inch • Typically does not require separate wash water system • Can be installed in existing channel	• Frequent maintenance can be required for cleaning • More prone to blinding during high solids loading given fine solids capture • Less rugged construction than combination coarse and fine screens • Less widely used than perforated plate screens with a limited number of manufacturers
Rotary Drum Screen	Conventional: This is a mature technology that is widely used.	• Captures fine screenings and grit with opening sizes down to 1/32-inch • Some units provide additional dewatering and compaction • Huber version can be installed in existing channel • Lower required headroom	• Frequent maintenance can be required for cleaning • More prone to blinding during high solids loading given fine solids capture • Less rugged construction than combination coarse and fine screens
Catenary Screen	Conventional: This is a mature technology that is widely used.	• No submerged sprockets that could be blocked or damaged by large solids • Lower required headroom • Rakes pass over lodged large objects instead of jamming	• Larger installation footprint • Typically lighter duty compared to chain and rake screens
Continuous Belt Screens	Emerging: Relatively new type of screen used in the U.S.	• Self-cleaning belts that can remove coarse and/or fine screenings • Cleans belt faster than single rake climber screens • Lower headloss through the screen • No major submerged mechanical components	• Relatively new • Multiple plastic pieces that might wear, especially in the presence of grit • Rake may be limited in handling large or heavy debris

2.1.2.7 Combination Coarse and Fine Screening Options

The climber/crawler bar screen for fine screening applications are also identical to those for coarse screening applications with bar spacing of 1/4 to 5/8-inch.

The multi-rake bar screen for fine screening applications are also identical to those for coarse screening applications with bar spacing of 1/4 to 5/8-inch.

2.2 Screening of Long List of Alternative Screening Technologies

The screening of the long list alternatives of coarse and fine screening options is shown in Table 2-3 on the following page.

Table 2-3 Screening of the Long List of Alternative Screening Technologies

Long List of Alternative Screening Concepts	Screening Criteria						Notes
	Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	
Coarse Screening Equipment							
1. Climber/Crawler Bar Screens	✓	✓	✓	✓	✓	✓	Proceed to detailed evaluation. This is currently what Nobleton WRRF has installed and is still effective as a coarse screening technology. Technology is compatible with existing WRRF, a proven technology, performs robustly, satisfies regulatory stakeholders, with acceptable associated construction impacts and capital/operating costs.
2. Multi-Rake Bar Screens	✓	✓	✓	×	×	✓	Eliminated due to stakeholder acceptance and to reduce construction impacts.
3. Deep Raker Screen	×	✓	✓	✓	×	✓	Eliminated due to changes that would be required to the current channel and construction impacts.
Fine Screening Equipment							
4. Perforated Plate Screen	✓	✓	✓	✓	✓	✓	Proceed to detailed evaluation. Technology is compatible with existing WRRF, a proven technology, performs robustly, satisfies regulatory stakeholders, with acceptable associated construction impacts and capital/operating costs.
5. Step Screen	✓	✓	✓	×	✓	✓	Eliminated due to stakeholder acceptance.
6. Rotary Drum Screens	×	✓	✓	✓	×	✓	Eliminated due to incompatibility and construction impacts to the channel.
7. Catenary Screens	✓	✓	✓	×	✓	✓	Eliminated due to stakeholder acceptance.
8. Continuous Belt Screens	✓	✓	✓	×	✓	✓	Eliminated due to stakeholder acceptance.

2.3 Short-List of Screening Technologies

Coarse screening is recommended for conventional secondary biological treatment design concepts. Fine screening is required for secondary biological treatment intensification design concepts.

The following screening treatment technologies will be carried over for the final evaluation as an alternative design concept for the WRRF:

- Coarse screening:
  - Climber/Crawler Bar Screen
- Fine screening:
  - Perforated Plate Screen

3.0 Preliminary Treatment - Grit Removal

The purpose of grit removal is to remove finer, dense solid material to reduce wear on downstream solids handling equipment. Grit removal systems can remove up to 95 percent of grit with a number of available technologies including channel type, detritor, aerated grit, forced vortex, and hydraulic vortex; each of these technologies are described below. In general, each of these technologies work on the principle of flow velocity control, whereby there is sufficient velocity to keep organic solids in suspension but is low enough to allow the denser, inorganic grit material to settle out. Once settled, the resulting grit slurry can then be pumped to a grit classifier for washing, dewatering, and disposal.

Two grit removal tanks are installed at the Nobelton WRRF.

3.1 Long List of Alternative Grit Removal Technologies

3.1.1 Channel

A grit removal channel is a configuration based on generating a desired velocity profile required to settle grit and keep organic solids in suspension. Along the top of the channel a series of grit pumps or a moving bridge with a single grit pump have a suction line that extend into the base of the sloped channel to lift the grit and directs it to a separate grit slurry channel.

Figure 3-1 shows a travelling bridge style grit and grease removal channel by Schreiber. For this unit, wastewater flows along a deep, narrow channel. Air is released into the bottom edge of the channel to create rolling water turbulence in an effort to wash the organics from the grit. The washed grit then settles to the bottom of the grit channel. A traveling bridge supported above the channel moves a grit pump the length of the channel to periodically pump the grit slurry from the channel bottom to a grit slurry trough for dewatering and disposal. The grease removal portion of this system consists of a grease channel parallel to the grit removal channel that is designed to allow grease to float to the top. The grit removal channel and the grease channel are separated by a baffle curtain wall to separate the rolling turbulence in the grit channel from the quiet pool needed for grease removal in the adjacent channel. Grit channels are not widely used and there are a limited number of manufacturers.

Figure 3-1 Grit Removal Channel

3.1.2 Detritor

A grit removal detritor is an older technology similar to channel grit removal in which flow is introduced to a velocity profile intended to keep organics in suspension and allow grit to settle to the bottom. In the case of a detritor, flow is distributed across a wide, shallow basin, similar to a clarifier, in a single direction to the outlet side. Flow enters the shallow basin/chamber via a series of inlet baffles designed to promote even flow distribution and uniform velocity across the entire width of the basin and promote grit settling. The outlet side is equipped with a sharp edged weir. As flow travels across the tank, grit settles on the bottom in a recessed, circular sump and is collected and transported into a collection hopper on the periphery of the tank by a slowly rotating scraper mechanism supported from above. From the collection hopper, a grit pump is typically used to transport settled grit slurry for dewatering and disposal.

There are a limited number of manufacturers of detritors, including Ovivo and Voltas Limited. The Ovivo J+A Crossflow unit is shown on Figure 3-2. New detritor systems are uncommon given the age of the technology, and a number of the original detritor systems have since been replaced with newer technologies; one example is for the Metropolitan Sewer District of Greater Cincinnati (MSDGC) Mill Creek WWTP, which recently replaced its detritors with vortex grit removal units.

Figure 3-2 Grit Removal Detritor

3.1.3 Aerated Grit Chamber

A grit removal aerated grit chamber is a technology in which air is introduced at the bottom of the chamber to keep organics in suspension and allows grit to settle to a sloped bottom. A dedicated blower introduces air flow into a tube which is located near the bottom of the chamber. The continuous rising air flow is intended to allow the grit to settle to the bottom of the chamber while keeping lighter organic material in suspension. Either a recessed-impeller grit pump or, more commonly, an air lift pump is used to lift settled grit slurry from the chamber bottom for dewatering and disposal. An aerated grit chamber is installed at the Eastern WRF, and as recently indicated by MCES staff, is not achieving the desired grit removal performance. This type of performance issue is not uncommon with this technology given the challenge of establishing and sustaining the right air and wastewater velocity and flow profile to effectively settle the grit. Similar to the detritor technology, MSDGC also replaced aerated grit at its Little Miami WWTP with vortex grit removal units.

There are several aerated grit chamber manufacturers including Fluidyne, Walker Process, WesTech (as shown on Figure 3-3), and others.

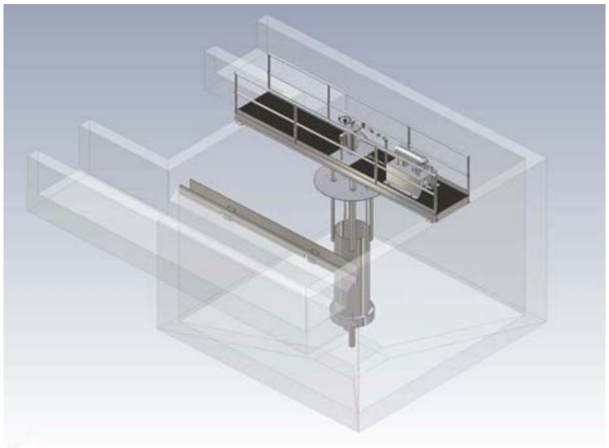


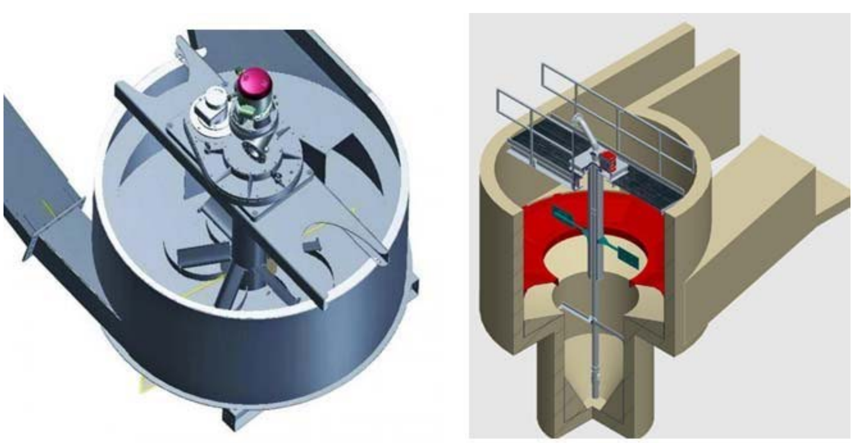
Figure 3-3 Aerated Grit Chamber

3.1.4 Vortex

3.1.4.1 Forced Vortex

Two forced vortex grit chambers manufactured by WTP Equipment Corporation are installed in the inlet works area of the Nobleton WRRF Process Building. Forced vortex grit removal chambers also work on the principle of establishing a desired velocity profile to settle grit to a collection point. Forced vortex introduces flow at a tangentially around a circular chamber with or without baffling and/or a rotating paddle to promote vortex flow. Effluent leaves the chamber tangentially in a separate channel and grit settles to the center of the chamber. Grit slurry is either lifted from a top-mounted grit pump or is pumped from a grit pump located in an adjacent dry pit to direct grit slurry to dewatering and disposal.

There are a number of vortex grit removal manufacturers including John Meunier, Ovivo, Smith and Loveless, Wastetech, WesTech , WTP Equipment Corporation, and others. The John Meunier and Smith and Loveless units are shown on Figure 3-4.



Smith and Loveless – Pista Grit Chamber

John Meunier – MECTAN V Grit Chamber

Figure 3-4 Forced Vortex

3.1.4.2 Hydraulic Vortex

A hydraulic vortex unit is similar to the more common forced vortex units, but is a proprietary technology manufactured by Hydro International, as shown on Figure 3-5. These units consist of stacked grit separator trays with no rotating parts. While these units are advertised to remove slightly finer grit than forced vortex (95 percent of grit greater than 75 microns versus 100 microns), they do introduce more headloss. Given the larger surface area provided by a stacked tray arrangement, a smaller footprint than forced vortex is required. A flow distribution header is provided to more evenly distributes influent flow tangentially over multiple conical trays and establish a vortex flow pattern where solids settled on each tray and are swept down into a center underflow collection chamber. A grit pump is installed at the underside of the unit, similar to some of the forced vortex units, to direct grit slurry to dewatering and disposal.

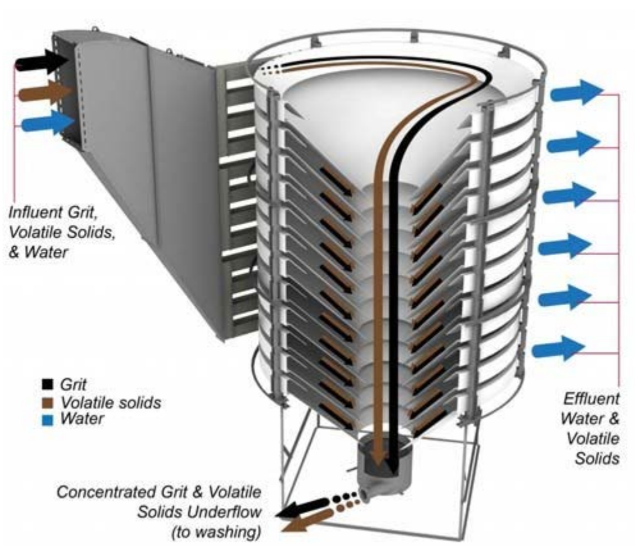


Figure 3-5 Hydraulic Vortex (Hydro International - Headcell)

3.1.5 Combined Rake and Clamshell System

Given the depths of a deep tunnel conveyance system, the above grit removal technologies are impractical for installation on the influent side of the tunnel pumps. As a means to remove coarse and some fine grit ahead of the deep tunnel pumps, a specialized rake and clamshell system can be used in this type of application to settle grit in a pit just upstream of the pump inlet header and then periodically lift it to the surface with a clamshell for disposal. Clamshell operation is typically manual and is initiated infrequently when the pumps aren't running or are running at low flow.

There are a limited number of manufacturers that provide these types of specialized rake and clamshell systems. Fairfield Service Company, Ovivo, and Kuenz are the known manufacturers operating in the U.S. Figure 3-6 depicts the Fairfield deep tunnel rake and clamshell system.

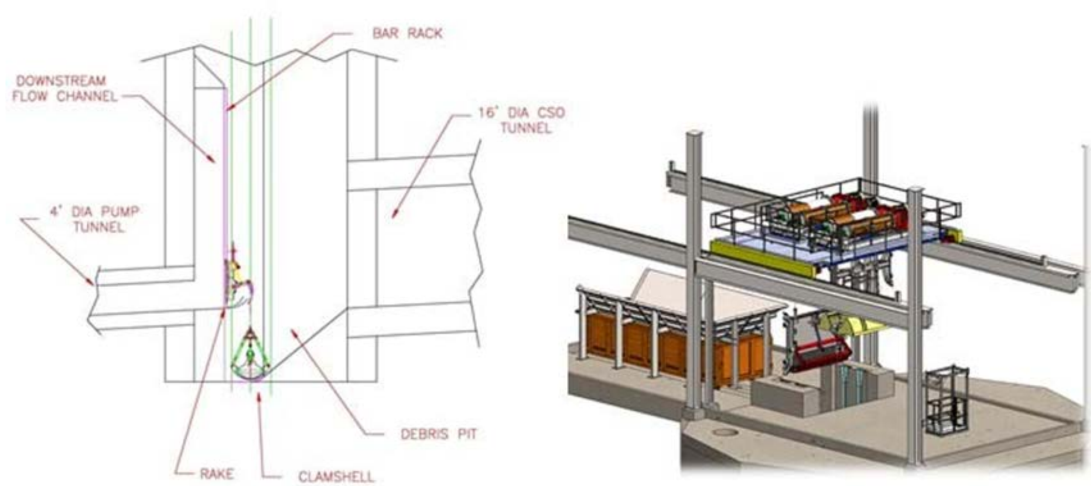


Figure 3-6 Combined Rake and Clamshell System

3.1.6 Grit Removal System Advantage and Disadvantages

Table 3-1 summarizes the advantages and disadvantages of the grit removal technologies.

Table 3-1 Advantages and Disadvantages of Grit Removal Systems

Technology	Status	Advantages	Disadvantages
Grit Removal Technologies			
Channel	Conventional: This is a mature technology that is widely used.	• No major mechanical components under water • Grease removal	• Limited system manufacturers
Detritor	Conventional: This is a mature technology that is widely used.	• Several facilities with long time detritor installations	• Lower grit removal performance when compared with newer vortex grit removal technology • Largest footprint to acoommodate wide, shallow detritor chambers • Limited system manufacturers
Aerated Grit	Conventional: This is a mature technology that is widely used.	• No moving parts below the water surface	• Requires dedicated blower system • Challenging air and wastewater flow arrangement
Forced Vortex	Conventional: This is a mature technology that is widely used.	• Widely used newer technology • Numerous system manufacturers • Designed to handle wide range of flows • Removal of ~95 percent of fine grit	• May require installations of dry pit to house grit pumps
Hydraulic Vortex	Conventional: This is a mature technology that is widely used.	• Designed to handle wide range of flows • Removal of ~95 percent of fine grit • No moving parts or external power needs	• Introduces more headloss than forced vortex units • Requires installation of dry pit to house grit pumps • Proprietary technology
Grit Removal Technologies for Deep Tunnels			
Combined Rake and Clamshell	Conventional: This is a mature technology that is widely used.	• Simple, infrequent clamshell operation • Offers coarse and some fine grit removal ahead of deep tunnel pumps for protection	• Additional grit removal system needed downstream of deep tunnel pumps if fine grit removal is desired

3.2 Screening of Long List of Alternative Grit Removal Technologies

The screening of the long list alternatives of grit removal technologies is shown on Table 3-2.

3.3 Short-List of Alternative Grit Removal Technologies

The following grit removal treatment technologies will be carried over for the final evaluation as an alternative design concept for the WRRF:

- Forced Vortex

Table 3-2 Screening of the Long List of Alternative Grit Removal Technologies

Long List of Alternative Grit Removal Concepts	Screening Criteria						Notes
	Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	
1. Channel	✓	✓	✓	✓	×	✓	Eliminated due to construction impacts. A new channel/basin would be required to be build and would require more footprint than existing technology.
2. Detritor	✓	✓	×	✓	×	×	Eliminated due to construction impacts and performance robustness. A new channel/basin would be required to be build and would require more footprint than existing technology.
3. Aerated Grit	✓	✓	✓	✓	×	✓	Eliminated due to construction impacts. A new channel/basin would be required to be build and would require more footprint than existing technology.
4. Forced Vortex	✓	✓	✓	✓	✓	✓	Proceed to detailed evaluation. This is the current technology installed at Nobleton WRRF. While the existing technology is not currently in use, it is still an acceptable option and can be rehabbed.
5. Hydraulic Vortex	×	✓	✓	✓	✓	✓	Eliminated due to hydraulic headloss imposed on WRRF's hydraulic profile between preliminary treatment and secondary treatment and could require pumping
6. Combined Rake and Clamshell	×	✓	×	✓	✓	✓	Eliminated due to performance robustness and the intermittent staffing at Nobleton WRRF. Combined Rake and Clamshell requires manual operation and would require more operator attention on preliminary treatment than currently provided.

4.0 Primary Treatment

The purpose of primary treatment is to remove settleable organic solids thereby decreasing the load on the secondary biological treatment process.

Primary treatment is not currently installed at Nobleton WRRF. It will be considered for design concepts to increase secondary biological treatment capacity.

4.1 Long List of Alternative Primary Treatment Technologies

4.1.1 Conventional Primary Sedimentation

Conventional primary treatment by sedimentation is to physically remove readily settleable solids and floating material found in the influent raw wastewater and reduce the suspended solids content. Primary sedimentation is typically the first step in further processing the wastewater following coarse/fine solids and grit removal in the preliminary treatment stage. Efficiently designed and operated treatment plants can achieve TSS removal from 50 to 70 percent and BOD removal from 25 to 40 percent in primary sedimentation tanks.

Almost all treatment plants that have primary sedimentation use mechanically cleaned sedimentation tanks that are of standard circular or rectangular design. The selection of type of sedimentation tank for a given application is typically governed by size of installation, local regulations, site conditions, stakeholder desires, and the experience and judgement of the design engineer.

4.1.2 Chemically Enhanced Primary Treatment

Chemically Enhanced Primary Treatment (CEPT) is often used to enhance settling of primary solids and subsequently increase the capacity of primary clarifiers. CEPT involves dosing chemicals, metal salts and a polymer, into the primary clarifiers to improve coagulation, flocculation and settling characteristics, thereby enhancing the removal of suspended solids and colloidal material in the primary clarifier. CEPT allows the primary clarifiers to be operated at higher overflow rates compared to conventional primary clarifiers. Importantly, through CEPT implementation the removal efficiency of total suspended solids (TSS) and biochemical oxygen demand (BOD) is enhanced by as much as 30%. Typical clarifiers, operating without CEPT achieve 50 to 60% removal of TSS and 20 to 35% removal of BOD.

CEPT also facilitates phosphorus removal. Metal ions in the dosed coagulant react with soluble ortho-phosphate present in the wastewater to form metal phosphates, which are then removed in the primary sludge. The two metal salts most commonly used in the CEPT process are ferric chloride (ferric) and aluminum sulfate (alum) although there are a number of other coagulants that are readily available on the market. The stoichiometric equations for the chemical precipitation of phosphorus using the previously highlighted metal salts are as shown in Table 4-1.

Table 4-1 Stoichiometric Equations for the Removal of TP

<table>
<tr><th>Metal Salt</th><th>Equation</th><th colspan="2">Comments</th></tr>
<tr><td>Ferric Chloride</td><td>$FeCl_3 + H_3PO_4 = FePO_4 + 3HCl_3$</td><td>• 1 mole of Iron III (Fe^{3+}) is theoretically required to remove 1 mole of P.
• In practice however, more Fe (the molar ratio is typically in the range of 2:1 to 4:1 Fe to TP) is required due to the likelihood of competing reactions.</td><td rowspan="2">• For both metal salts and due to the acidic byproducts produced, alkalinity is consumed in the process.</td></tr>
<tr><td>Aluminum Sulfate</td><td>$Al_2(SO_4)_3.14H_2O + 2H_3PO_4 = 2AlPO_4 + 3H_2SO_4 + 18H_2O$</td><td>• The stoichiometric ratio for the removal of P is the same as that for Ferric.
• As is the case with Ferric, the applicable dosage rate should exceed this stoichiometric ratio for effective TP removal</td></tr>
</table>

4.1.3 Ballasted Flocculation

Ballasted flocculation, also known as high rate clarification, is a physical-chemical treatment process that uses continuously recycled media and a variety of additives to improve the settling properties of suspended solids through improved floc bridging. Typical ballasted flocculation removal efficiency is 85-95% TSS removal, and 50-80% BOD removal. The objective of this process is to form micro-floc particles with a specific gravity of greater than 2.0. Faster floc formation and decreased particle settling time allows the settlement process to proceed up to ten times faster than with conventional clarification, allowing treatment of flows at a significantly higher rate than possible with traditional unit processes. There are two types of ballasted flocculation systems on the market: (1) those that recycle sludge as a ballast (e.g., DensaDeg) and (2) those that add an exogenous material (e.g., ACTIFLO, CoMag). Possible ballasted flocculation technologies include:

- The Co-Mag process is a ballasted settlement technology that uses magnetite to weigh down solids and enhance solids capture in a settler at a much higher overflow rate. The ballast is recovered by shearing the floc and then separating the magnetite using a magnetic recovery drum.
- The Actiflo process combines ballasted settling using micro-sand with lamella settlers to provide high-rate settling. A hydro-cyclone separates out the micro-sand, which is re-injected into the maturation tank. This process has been used successfully for both water treatment and for wet weather excess flow treatment. It has not been used commonly for primary treatment.
- The DensaDeg process creates a floc using a coagulant and a polymer. The floc is settled by gravity using lamellas. A portion of this sludge is recycled to the flocculation step.
- The Rapisand process is similar to the Actiflo and Densadeg processes. A ballasted floc is created by mixing influent wastewater with a coagulant, polymer and microsand.

These technologies are depicted in Figure 4-1.

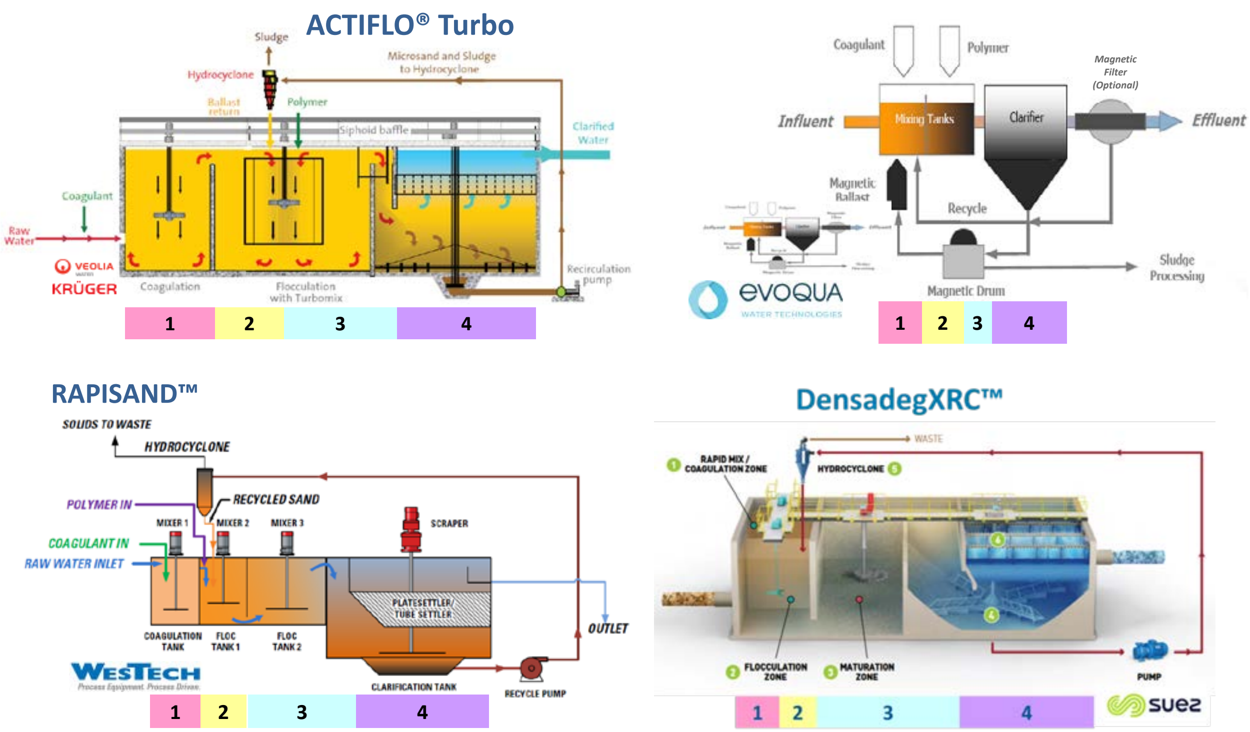


Figure 4-1 Examples of Ballasted Flocculation/Sedimentation Technologies

4.1.4 Primary Filtration Technologies

Direct filtration of raw wastewater is commonly not practiced in North America, but gaining ground with more and more full-scale installations. As an advanced primary treatment technology, primary filtration, specifically cloth media filtration, increases the removal of primary solids (approximately a 20% increase of removal efficiencies) in comparison to conventional primary sedimentation. Compressible media filters schematics are given in Figure 4-2.

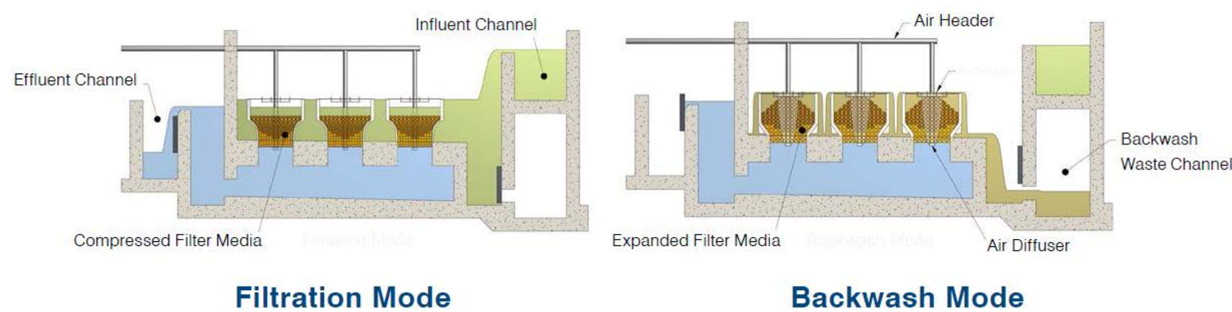


Figure 4-2 WWETCO Compressible Media Filter

Cloth Media Filtration

Cloth media filtration can be used for advanced primary treatment. There are several full-scale installations in place and soon to be installed across North America. When used in advanced primary treatment applications, cloth media filtration can achieve approximately 80% TSS removal and 50% total BOD removal. This kind of treatment application can help reduce the carbon load to the downstream secondary treatment process, which can also lead to aeration energy savings, increases in existing secondary treatment capacity or reduced basin size for the secondary treatment process. Primary filtration can also have a dramatically reduced footprint as compared to conventional primary sedimentation.

4.1.5 Primary Treatment Advantages and Disadvantages

Table 4-2 is a comparison of the primary treatment options evaluated for this project.

Table 4-2 Comparison of Primary Treatment Enhancement Technologies

Technology	Status	Advantages	Disadvantages
Conventional Primary Sedimentation	Conventional: Primary sedimentation is the standard for primary treatment in municipal wastewater facilities across North America.	• TSS and BOD removal prior to secondary treatment • Conventional removal efficiencies • Simple construction • Simple and easy operation	• Larger footprint • Increased headloss between preliminary and secondary treatment • Odour control technology required
CEPT	Conventional: Several facilities use CEPT year-round including San Diego, CA, Sydney - Australia, and Bloomington, NY.	• Improved TSS and BOD removal compared to conventional primary clarifiers (as high as 85% TSS removal, 65% BOD removal) • Consistent performance • Easy to retrofit into existing primary clarifiers • Simple and easy operation	• High chemical use resulting in high operating costs • Health and safety considerations for chemical handling • Required jar testing to determine proper water testing for correct chemicals • May remove too much carbon, requiring external source of carbon for BNR plants • Increased production of primary solids • Odour control technology required
Ballasted Flocculation	Emerging: Has been used successfully for both water treatment and for wet weather excess flow treatment. It has not been used commonly for primary treatment in North America. There are some primary treatment Actiflo installations in Europe.	• Improved TSS and BOD removal compared to conventional primary clarifiers (85-95% TSS removal, 50-80% BOD removal) • Small footprint	• May have higher construction cost than conventional primary clarifiers • Ballast may be expensive • More complex and mechanically intensive than conventional primary treatment • Proprietary technology • Increased production of primary solids • Odour control technology required
Primary Filtration (e.g., Compressed media filters, Salsnes Filters, Clear Cove, AquaPrime	Emerging: There have been several North American installations in recent years. These installations have been either used in place of primary treatment or used after primary treatment to further remove BOD and TSS before the secondary process.	• Improved TSS and BOD removal compared to conventional primary clarifiers • Can target a specific TSS removal, depending on particle size by selecting the type of media or mesh size • Smallest footprint	• More complex and mechanically intensive than conventional primary treatment. • Proprietary technology. • Headloss through filters may require additional pumping • Odour control technology required

4.2 Screening of Long List of Alternative Primary Treatment Technologies

The screening of the long list alternatives of primary treatment technologies is shown in Table 4-3.

4.3 Short-List of Alternative Primary Treatment Technologies

The following primary treatment technologies will be carried over for the final evaluation as an alternative design concept for the WRRF:

- Primary Filtration

Table 4-3 Screening of the Long List of Alternative Primary Treatment Technologies

Long List of Alternative Primary Treatment Concepts	Screening Criteria						Notes
	Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	
1. Conventional Primary Sedimentation	✗	✓	✓	✗	✓	✗	Eliminated due to stakeholder acceptance and compatibility with the existing WRRF. Conventional primary sedimentation would also require the building of primary sedimentation basins and cost more than other alternatives. Primary equipment would require the construction of odour control technology as well.
2. CEPT	✗	✓	✓	✗	✓	✗	Eliminated due to stakeholder acceptance and compatibility with the existing WRRF. CEPT would also require the building of primary sedimentation basins and cost more than other alternatives. Primary equipment would require the construction of odour control technology as well.
3. Ballasted Flocculation	✗	✓	✓	✗	✓	✗	Eliminated due to stakeholder acceptance and compatibility with the existing WRRF. Primary equipment would require the construction of odour control technology as well.
4. Primary Filtration	✓	✓	✓	✓	✓	✓	Eliminated due to stakeholder acceptance and compatibility with the existing WRRF. Primary equipment would require the construction of odour control technology as well.

5.0 Secondary Treatment

The purpose of secondary treatment is to remove carbonaceous and nitrogenous oxygen demanding substances from wastewater.

Extended aeration is the current secondary biological treatment process at Nobleton WRRF.

5.1 Long List of Alternative Secondary Treatment Technologies

5.1.1 Conventional Nitrifying Activated Sludge Process

For a nitrifying conventional activated sludge (CAS) process at the Nobleton WRRF, primary clarifiers would be required between the headworks and the aeration tanks to reduce loadings onto the secondary treatment system. Based on the minimum month temperature of 12 °C, the solids retention time (SRT) for the CAS would be approximately 12 days to achieve the required level of nitrification. Typical design values are food to micro-organisms ratio (F/M) of 0.05-0.25 kgBOD/kgMLVSS.day, volumetric loading of 0.31-0.72 kgBOD/m3.d, MLSS concentration of 3,000-5,000 mg/L, and hydraulic retention time (HRT) of minimum 6 hours. Aeration should be 1 kg O2 per each kg of BOD in the influent, as well as an additional 4.6 kg O2 per kg of TKN influent for nitrification.

5.1.2 Extended Aeration

The extended aeration process is a modification of the CAS process which provides biological treatment for the removal of biodegradable organics under aerobic conditions. EA design solids retention time (SRT) is very high (20 to 30 d) and the hydraulic retention time (HRT) is typically 18 to 24 hours. Typical design values for extended aeration systems which provide nitrification are F/M of 0.05-0.15 kgBOD/kgMLVSS.day, organic loading of 0.17-0.24 kgBOD/m^3.day, MLSS concentration of 3,000-5,000 mg/L, and hydraulic retention time (HRT) of minimum 15 hours (if nitrification is required year-round, a longer detention time may be required). Because of the long solids retention time, aeration requirements should account for endogenous respiration, meaning that instead of 1 kg O_2 per kg BOD in the influent, 1.5 kg O_2 per daily average BOD should be considered for carbonaceous oxygen demand. If nitrification is provided, 4.6 kg O_2 per kg influent TKN is added as nitrogenous oxygen demand.

Because of the large tankage volume needed and relatively low volumetric oxygen demand rate, the aeration equipment design is used extensively for pre-engineered plants for small communities. Mechanical or diffused aeration provide the oxygen required to sustain the aerobic biological process. Mixing must be provided by aeration or mechanical means to maintain the microbial organisms in contact with the dissolved organics. The pH must also be controlled to optimize the biological process and essential nutrients must be present to facilitate biological growth and the continuation of biological degradation. Generally primary clarification is not used for EAs. Secondary clarifiers are designed at lower hydraulic loading rates than CAS clarifiers to better handle large flowrate variations. A flow equalization tank may be necessary at the WRRF prior to the EA tanks to prevent overloading of the system from inconsistent flow rates in the morning and evening.

The existing Nobleton WRRF extended aeration treatment system includes two aeration tanks, two clarifiers, and associated pumps, blowers, and air distribution equipment.

5.1.3 Sequencing Batch Reactor (SBR)

The sequencing batch reactor (SBR) is a variation of the activated sludge process. They act as a fill-and-draw type reactor system involving a single complete-mix reactor in which all steps of activated sludge processes occur. Mixed liquor remains in the reactor during all cycles and thus, eliminating the need for separate sedimentation tanks or clarifiers. For the Nobleton WRRF, at least 2 tanks are required so that one tank is in the fill mode while the other goes through react, solids settling, and effluent withdrawal. Decanting of effluent is accomplished by either fixed or floating decanter mechanisms. Based on influent flowrate and tank volume used, SBR hydraulic retention times generally range from 18 to 30 hours. An SBR goes through a number of cycles per day; a typical cycle may consist of 3-h fill, 2-h aeration, 0.5-h settle, and 0.5-h for withdrawal of supernatant. An idle step may also be included to accommodate peak flows. The aeration tank volumetric loading should not exceed 0.24 kg $BOD_5/(m^3 \cdot day)$, and design F/M ratios should be within the range of 0.05 to 0.1 kgBOD/(kgMLVSS.day).

Aeration may be provided by jet aerators or coarse/fine diffusers with submerged mixers. Dissolved oxygen (DO) should be monitored during this phase to ensure it is maintained above 2 mg/L so that nitrification can occur. For denitrification, DO level should be lowered to less than 0.5 mg/L. The treatment cycle can be adjusted to undergo aerobic, anaerobic, and anoxic conditions in order to achieve biological nutrient removal, including nitrification, denitrification, and some phosphorus removal. With SBRs, effluent BOD levels of less than 5 mg/L and NO_3-N concentrations of less than 5 mg/L are achievable. If the SBR provides denitrification, total nitrogen can reach to less than 5 mg/L. Low phosphorus limits of less than 2 mg/L can also be achieved by using a combination of biological treatment (anaerobic phosphorus absorbing organisms) and chemical addition (aluminum or iron salts) within the tank.

5.1.4 Rotating Biological Contactor

RBC is a fixed film biological treatment device in which microorganisms are grown on circular plastic disks mounted on a horizontal shaft that rotates slowly while partially immersed in wastewater. The rotating disks (known as the media) are contained in a tank or trough and rotate at between 2 to 5 revolutions per minute. The rotation helps to slough off excess solids. Commonly used plastics for the media are polyethylene, PVC and expanded polystyrene. The shaft is aligned with the flow of wastewater so that the discs rotate at right angles to the flow, with several packs usually combined to make up a treatment train. About 40% of the disc area is immersed in the wastewater. The disc system can be staged in series to obtain nearly any detention time or degree of removal required. Since the systems are staged, the culture of the later stages can be acclimated to the slowly degraded materials. Hydraulic loading to the RBCs should range between 75 to 155 $L/(m^2 \cdot d)$ of media surface area without nitrification and 30 to 80 $L/(m^2 \cdot d)$ with nitrification. Organic loading to the first stage of an RBC train should not exceed 0.03 to 0.04 kg $BOD_5/(m^2 \cdot d)$ or 0.012 to 0.02 kg $BOD_5/(m^2 \cdot d)$. Loadings in the higher end of these ranges will increase the likelihood of developing problems such as heavier than normal biofilm thickness, depletion of DO, nuisance organisms and deterioration of overall process performance. The optimum tank volume determined when treating municipal sewage of up to 300 mg/L BOD_5 is 0.042 L/m^2, which considers sewage displaced by the media and attached biomass. Based on a tank volume of 0.042 L/m^2, the detention time in each RBC stage should range between 40 to 120 minutes without nitrification and 90 to 250 minutes with nitrification.

The temperature of sewage entering any RBC should not drop below 5 °C unless there is sufficient flexibility to decrease the hydraulic loading rate. Otherwise, insulation or additional heating should be provided to the plant. Year-round operation requires that the RBC be covered to protect the biological growth from cold temperatures and the excessive loss of heat from the sewage with the resulting loss of performance.

RBCs need to be preceded by effective primary sedimentation tanks equipped with scum and grease removal devices or pretreatment devices which provide for effective removal of grit, debris and excessive oil and grease prior to the RBC units. Solids separation is an important part of the RBC process; accordingly, downstream secondary clarification is required.

5.1.5 Process Intensification Technologies

5.1.5.1 Moving Bed Bioreactor (MBBR)

MBBR is an integrated fixed film activated sludge (IFAS) or hybrid process. IFAS consists of an activated sludge system in which a material to support attached biomass growth has been added in addition to the suspended biomass growth in an activated sludge reactor. The MBBR process is similar to the IFAS process with mixed, suspended media contained within the reactor by effluent sieves, with the exception that there is no return activated sludge. The media fill volume is generally higher (up to 70 percent), and the suspended solids concentration in the flow to the secondary clarifier may be in the range of 100 to 250 mg/L versus 2,500 to 3,500 mg/L in an IFAS. Process design for MBBR can also include the suspended media in anoxic zones for fixed film biological denitrification. MBBR reactor effluent, filtration processes including granular media and membrane filtration, and dissolved air floatation can be used in lieu of gravity settling.

5.1.5.2 Biologically Active Filters (BAF)

The term biological aerated filter refers to the fact that the attached growth process is aerated to provide oxygen for BOD removal and nitrification. Biological aerated filter fall within a broader category called biological active filter (BAF). Biological active filter has the biological aerated filter design but working in anoxic conditions to provide denitrification for nitrogen removal.

Veolia is one of the vendors that provide this technology. Veolia's BIOSTYR® system is a very compact process combining fixed film biological treatment and filtration in a single unit operation with relatively high pollutant loads depending on the carbon and nitrogen requirements. BAF processes are very well suited when space is an important site constraint. During the last 25 years, more than 150 BIOSTYR® facilities have been built and operated to treat municipal wastewater around the world, thereby also demonstrating the wide-range of treatment applications in the marketplace. Figure 5-2 shows a schematic of a conventional BIOSTYR® cell.

The design and cost of BAF is impacted directly by hydraulic flow rate and flow equalizations should be considered for high hydraulic peak flows from wet weather events. Also, solids filtration may be implemented to produce a high-quality effluent.

As a case study, in 2014 a BAF unit was installed in a WWTP in New York, NY with a capacity of 94 MLD (280 MLD peak flow). This system was able to successfully reduce the effluent Total Nitrogen loading from 907 kg/day to 90.7 kg/day, and reach the tighter restriction of 4.0 mg/L TN regulated by the State Pollutant Discharge Elimination System.

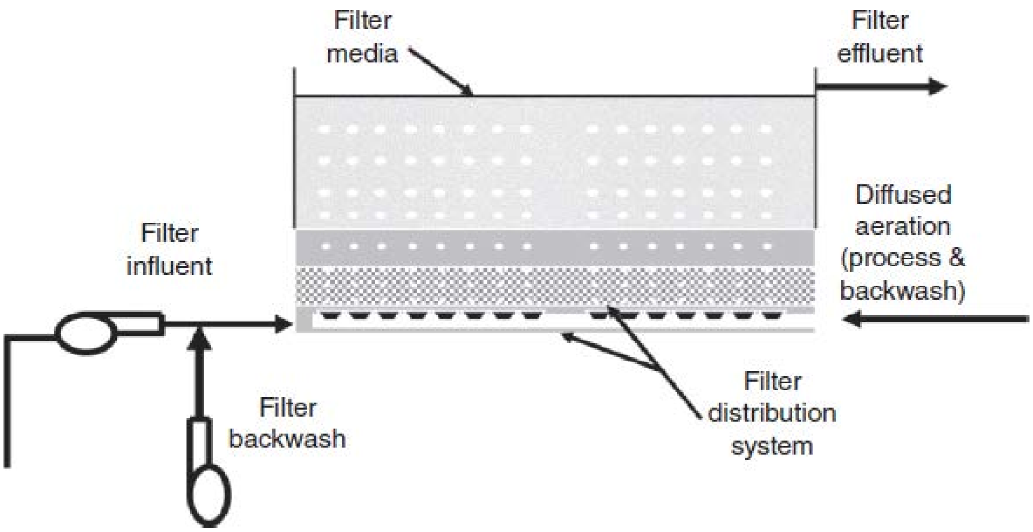


Figure 5-1 Biological Aerated Upflow Filter

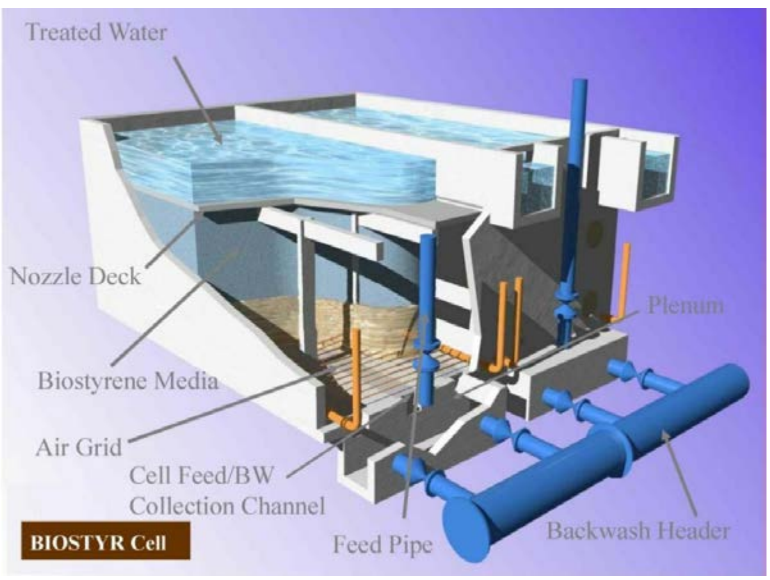


Figure 5-2 BIOSTYR® System Cell General Arrangement

5.1.5.3 Integrated Fixed-Film Activated Sludge (IFAS)

The IFAS process includes a RAS stream to provide for activated sludge as well as fixed film biomass for biological treatment.

Organic loading rates for these reactors are typically in the order of 3.5 to 7.0 g BOD_5/m^2 of media surface area/day for $CBOD_5$ removal and less than 3.5 g BOD_5/m^2 of media surface area/day for nitrification. For nitrification with the IFS process, the required media surface area will usually be dictated by TKN loading, TAN removal requirements and biological growth conditions in the reactor (e.g. temperature, pH, DO). Vendor should be consulted for design details.

A single-pass IFAS have continuously operating, non-cloggable fixed-film reactors with no need for backwashing or return sludge flows, low head-loss and high specific biofilm surface area. This is achieved by having the biomass grow on small carrier elements that move along with the sewage in the reactor or the attached growth support media may be immobile within the reactor for some designs. In the case of free-moving carrier elements, movement is normally induced by coarse bubble aeration in the aerated zone, although fine bubble aeration systems have also been used, while mechanical mixing is utilized in an anoxic/anaerobic zone. For small plants, mechanical mixers are omitted for simplicity reasons and pulse aeration for a few seconds a few times per day can be used to move the biofilm carriers in anoxic reactors.

Free-moving biofilm carrier elements are generally made of polyethylene or polypropylene. A screen is placed at the outlet of the reactor to keep the biofilm elements in the reactor. Agitation constantly moves the carrier elements over the surface of the screen and the scrubbing action prevents clogging. Upstream fine screening of raw sewage should also be considered for such designs. Also, downstream secondary clarification is required for IFAS systems

5.1.5.4 Membrane Bioreactor (MBR)

A membrane bioreactor (MBR) is an activated sludge system with membranes located at the end of the activated sludge tank(s) for liquid-solid separation instead of using secondary clarifiers. Low-pressure membranes (either microfiltration [0.07 to 2.0 µm] or ultrafiltration [0.008 to 0.2 µm]) are typically used in MBRs. The membranes are mounted in modules that can be lowered into the bioreactor. The modules are comprised of the membranes, support structure for the membranes, feed inlet and outlet connections, and an overall support structure. The membranes are subjected to a vacuum (less than 50 kPa) that draws water (permeate) through the membrane while retaining solids in the reactor. To minimize the accumulation of solids and fouling on the exterior side of the membranes, compressed air is introduced through a distribution manifold at the base of the membrane module. As the air bubbles rise to the surface, scouring of the membrane surface occurs; the air also provides oxygen to maintain aerobic conditions and solids suspension within the reactor.

There are two configurations for MBR systems: external (or submerged) and integrated. In the external system, membranes are a separate unit process requiring an intermediate pumping step. In the integrated MBR system, the key component is the microfiltration membrane that is immersed directly into the activated sludge reactor. The submerged configuration relies on coarse bubble aeration to produce mixing and limit fouling. Aeration also maintains solids in suspension, scours the membrane surface and provides oxygen to the biomass, leading to a better biodegradability and cell synthesis. The energy demand of the submerged system can be up to 2 orders of magnitude lower than that of the side stream systems and submerged systems operate at a lower flux, demanding more membrane area.

The principal operational problems with MBR systems are foaming and fouling. Similar to activated sludge and secondary clarifier systems, Nocardioform foaming can occur in MBR systems operated with fine pore diffused aeration. MBR systems must be operated in a preventative maintenance mode to avoid operating problems from fouled membranes. The WRRF capacity can be compromised due to the lower flux associated with fouled membrane. Membrane fouling is prevented by employing the cleaning and operating procedures provided by the membrane supplier, maintaining the upstream fine screening equipment, and operating the system within acceptable SRT and MLSS concentration limits. Improper screening would allow the accumulation of hair and fibrous material in the membranes, which cannot be removed by the normal membrane cleaning program. A lower SRT of about 0.8 d is normally recommended to prevent excessive fouling due to the release of microbial substances from a younger activated sludge. Excessively high SRTs may result in higher amount of free bacteria and floc fines to increase fouling rates.

Concentrations of MLSS in the range of 8,000 to 14,000 mg/L are normally within acceptable operating ranges. Very high MLSS concentrations require a much lower flux to maintain a balance between the amount of solids directed to the membrane surface versus the solids removal rate by the air scour. If excessive MLSS concentrations (>18,000 mg/L) exist under operation of normal design flux values, the membranes can become what is termed "sludged up" and special cleaning methods may be needed to regain the expected operation flux.

Certain wastewater substances must be prevented from entering the treatment facility or MBR system to maintain proper membrane operation. Cooking oils and grease can collect on membrane surfaces and lead to excessive fouling that can only be removed by special membrane cleaning methods.

The process performance of an MBR system is often regulated by effluent concentrations of BOD, COD, ammonia, TN, phosphorus, TSS, and turbidity. Membrane equipment can only control the concentration of the TSS and turbidity. The remaining criteria are governed by biological process design and area affected by SRT, dissolved-oxygen concentrations, recirculation rates within the process, volatile acid concentrations, and other design parameters.

5.1.5.5 Membrane Aerated Bioreactor (MABR)

The membrane aerated biofilm reactor (MABR) is a disruptive municipal wastewater treatment technology that reduces energy requirements for aeration by up to 40 percent, decreases tank requirements for nitrification and increases the level of simultaneous nitrification and denitrification (SND) occurring in the activated sludge process. The MABR relies on gas transferring membranes to deliver oxygen at the base of a nitrifying biofilm. This oxygen transfer is based on diffusion to the biofilm and not transfer from a gas bubble, resulting in transfer efficiencies up to 90%. This also results in a liquid around the membranes maintaining anoxic conditions, which results in nitrification in the biofilm and denitrification in the bulk liquid.

This technology has been in development since the 1980s, with significant bench-scale and pilot-scale work being completed in the 2000s. Initial attempts to incorporate membrane aeration into biological processes focused on using the membranes solely for gas transfer and not as a support structure for biofilms. However, gas transfer efficiency decreased rapidly due to biofouling of the membranes. Timberlake et al (1988) were the first to design a system to take advantage of the aeration membranes as a support for bacteria. By pressurizing hollow fiber membranes with air, Timberlake et al. found a significant amount of TN removal was achievable. Additional studies focused on achieving nitrification and denitrification in a stratified biofilm for TN removal. The thickness and density of the biofilm led to mass transfer and biofilm management concerns. Research began to examine a hybrid system, where a nitrifying biofilm was supported by the MABR, but suspended growth was maintained under anoxic conditions. Pilot-scale studies indicated that this hybrid system could achieve a high TN removal while maintaining a thinner biofilm. Even with all of the research investment since the 1980s, MABR technology has only been commercially available on the market in the past 8 years.

MABR technology is a suitable option for Nobleton WRRF due to limitations in the ability to build a new treatment train. While it can be done, there are hydraulic limitations to take into account with an additional treatment train. This would require additional pumping and piping, along with redundant equipment for the third treatment train and could make the capital costs comparable to MABR technology.

5.1.5.6 Granular Activated Sludge

5.1.6 Secondary Treatment Advantages and Disadvantages

Table 5-1 is a comparison of the secondary treatment options evaluated for this project.

Table 5-1 Comparison of Secondary Treatment Enhancement Technologies

Technology	Status	Advantages	Disadvantages
Conventional Nitrifying Activated Sludge Process (CAS)	Conventional: This technology has been applied in many wastewater treatment facilities in North America and around the world.	• Common and proven • Ability to treat BOD and ammonia in a single stage • Relatively uncomplicated design • Suitable for all kinds of aeration equipment	• Larger footprint required because of the need for primary clarifiers in this application • Larger footprint of aeration basins needed due to colder weather in this application • Stability linked to operation of secondary clarifier for biomass return (RAS)
Extended Aeration (EA)	Conventional: This technology has been applied in many wastewater treatment facilities in North America and around the world and is a modification of the CAS process.	• Relatively uncomplicated design and operation • Easy installation • Smaller footprint • Handles variability of organic loads and flow • High quality effluent • Low biosolids production	• Require large aeration tanks with long aeration periods • Does not achieve denitrification or phosphorus removal • Limited adaptability to changing effluent requirements • Possibility for filamentous sludge bulking and settling issues
Sequencing Batch Reactor (SBR)	Conventional: This is a mature technology that is widely used.	• Simple layout with littler operation and maintenance • Does not require final clarifiers/RAS pumping • Smaller footprint compared to EA • Easy installation • No need to optimize aeration and decanting to comply with power requirement and lower decant discharge rates • Consistently perform nitrification, denitrification, and phosphorus removal • Operational flexibility • Automatic and positive control of MLSS concentration and SRT • MLSS cannot be washed out by high flows because of flow equalization	• Process design and control complicated • Greater level of maintenance • High specific energy consumption and volumetric tankage requirements • Batch discharge may require equalization and secondary clarifiers primary to tertiary treatment and disinfection • High risk flows can disrupt operation • Sludge must be disposed of frequently • Effluent quality depends on operational reliability of decanting facility

Technology	Status	Advantages	Disadvantages
Rotating Biological Contactor (RBC)	Conventional: This is a mature technology that is widely used.	• Short retention time due to large active surface • Capability of handling wide range of flows • Good biomass settleability and easy solids separation • Ease of operation and excellent process control • Low power requirements	• Necessary to cover units to protect against freezing cold weather • Frequent maintenance of shaft bearings and mechanical drive units
Process Intensification			
Moving Bed Bioreactor (MBBR)	Conventional: Over 700 wastewater systems (both municipal and industrial) installed in over 50 countries that are operating.	• Similar BOD and nitrogen removal treatment performance as CAS • Small footprint • Simplicity of operation – no need for manual sludge wasting, SRT control, and sludge recycle • No sludge bulking • Can handle peak wet weather flow variations • Well suited for retrofit application with reduced time and little if any tank construction • More versatile and adaptable for BNR • Continuous operation that does not require special operation or interruption of treatment for biofilm thickness control or flushing out excess solids	• Higher energy demand • Potential issues caused by media removal for diffuser maintenance • High hydraulic profile headloss due to flow through the media screening devices • Limitations for phosphorus removal only by chemical addition
Biologically Active Filters (BAF)	Conventional: This is a mature technology that is widely used.	• Relatively small footprint • Ability to effectively treat dilute wastewaters • No issues with regard to sludge settling characteristics • Simplicity of operation	• More complex in terms of operations and maintenance of instrumentation and controls • Limitations of economies of scale for application to larger facilities • Higher capital cost unless land is at a premium or not available • Vulnerable to high headloss from high solids loadings

Technology	Status	Advantages	Disadvantages
Integrated Fixed-Film Activated Sludge (IFAS)	Conventional: This is a mature technology that is widely used.	• Retrofit flexibility – almost any size or shape of tank can be retrofitted • Carrier elements in the reactor may be decided for each case based on degree of treatment desired, BOD_5, TKN, hydraulic loadings, temperature, and oxygen transfer capability • Reactor volume completely mixed – no "dead" or unused space in reactor • Improved nitrification compared to simple suspended growth systems	• High energy requirements due to aeration • High costs for construction and operation • Challenges in finding mechanical spare parts locally
Membrane Bioreactor (MBR)		• Effluent qualities less dependent on MLSS concentration and sludge properties • Can be operated at higher MLSS concentrations (8,000 to 12,000 mg/L) • Reduction in reactor volume necessary to treat same loading rate • Enhanced ammonia removal • Can potentially reduce or eliminate need for secondary clarification and effluent filters – reduced footprint • Can be retrofitted into existing tankage • Higher SRTs – reduced sludge production • Capital cost can be offset by a lack of needing tertiary filtration • Ease of installation • Ease of flexibility and expansion potential for the future	• High capital costs – although have gotten less expensive • Hydraulic limitations – overloading can lead to fouling of membrane • Redundancy needs to due hydraulic limitations and availability of spare parts can limit flexibility of operations and maintenance staff in working on units or taking units out of service • Limited peaking availability • Optimization needed for chemical usage for membrane cleaning to limit effect of purchasing chemicals on operating costs • Membrane replacement cost affects life-cycle costs • Membrane equipment systems are unique, having different configurations and shapes depending on the manufacturer

Technology	Status	Advantages	Disadvantages
Membrane Aerated Bioreactor (MABR)	Emerging: MABR technology has gone through a lot of research, bench-scale, and pilot-scale testing, but has only been commercially available on the market for about 8 years. While there are many pilot-scale facilities, 1 full-scale facility is in operation since 2017 (Yorkville Bristol Sanitary District, US) and a full-scale facility in construction at Waterloo (expected completion 2021 and driving distance from Nobleton WRRF).	• Reduction in aeration energy by up to 40% • Increased nitrification reliability due to the retention time of attached biomass in the MABR biofilm • Ability to more readily control nitrite shunt for mainstream short cut nitrogen removal • Potential to reduce the SRT seasonally or year-round to increase wet weather treatment capacity • Adoption in the North America accelerating	• Limited manufacturers • Can have a higher capital cost when land is not at a premium, or when there is flexibility to build redundant train • Emerging technology, with more common pilot-scale demonstrations, and one full-scale operating facility in North America.
Granular Activated Sludge	Emerging: Background		

5.2 Screening of Long List of Alternative

The screening of the long list alternatives of secondary treatment technologies is shown in Table 5-2 on the following page. Supplemental to secondary treatment technologies, the various technologies that encompass process intensification are also screened in Table 5-3.

Secondary Treatment Technologies

Conventional: This is a mature technology that is widely used.

5.3 Short-List of Alternative Secondary Treatment Technologies

The following secondary treatment technologies will be carried over for the final evaluation as an alternative design concept for the WRRF:

- Extended Aeration
- Process Intensification: Membrane Aerated Bioreactor (MABR)

Table 5-2 Screening of the Long List of Alternative Secondary Treatment Technologies

Long List of Alternative Secondary Treatment Concepts	Screening Criteria						Notes
	Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	
1. Conventional Nitrifying Activated Sludge Process (CAS)	✗	✓	✓	✓	✓	✓	Eliminated due to incompatibility with existing WRRF. More complex operation and therefore, generally applicable to large WRRFs that are continuously staffed. Higher sludge generation. This technology is generally applied to settled wastewater so a primary clarifier would be constructed.
2. Extended Aeration (EA)	✓	✓	✓	✓	✓	✓	Proceed to detailed evaluation. Technology is compatible with existing WRRF, a proven technology, performs robustly, satisfies regulatory stakeholders, with acceptable associated construction impacts and capital/operating costs.
3. Sequencing Batch Reactor (SBR)	✓	✓	✓	✗	✓	✓	Eliminated due to stakeholder acceptance.
4. Rotating Biological Contactor (RBC)	✗	✓	✓	✓	✓	✓	Eliminated due to incompatibility with existing WRRF. RBC units have large footprints; therefore, they are not suitable when there is limited space availability. Moreover, these systems require effective primary sedimentation tanks equipped with scum and grease removal devices. This will add to space availability issue mentioned above.
5. Process Intensification	✓	✓	✓	✓	✓	✓	Proceed to detailed evaluation. Technology is compatible with existing WRRF, a proven technology, performs robustly, satisfies regulatory stakeholders, with acceptable associated construction impacts and capital/operating costs.

Table 5-3 Screening of the Long List of Alternative Process Intensification Technologies

Long List of Alternative Process Intensification Concepts	Screening Criria: Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	Notes
1. Moving Bed Bioreactor (MBBR)	×	✓	✓	✓	✓	✓	Eliminated due to potential for sieve used for catching media to induce more headlosses into the system
2. Biologically Active Filters (BAF)	×	✓	✓	✓	✓	✓	Proceed to detailed evaluation. Technology is compatible with existing WRRF, a proven technology, performs robustly, satisfies regulatory stakeholders, with acceptable associated construction impacts and capital/operating costs.
3. Integrated Fixed-Film Activated Sludge (IFAS)	×	✓	✓	✓	✓	✓	Eliminated due to potential for sieve used for catching media to induce more headlosses into the system
4. Membrane Bioreactor (MBR)	✓	✓	✓	×	✓	×	Eliminated due to high capital and lifecycle costs. Membrane replacement cost affects life-cycle cost analysis. Also, stakeholder acceptance.
5. Membrane Aerated Bioreactor (MABR)	✓	✓	✓	✓	✓	✓	Proceed to detailed evaluation. Technology is compatible with existing WRRF, a proven technology, performs robustly, satisfies regulatory stakeholders, with acceptable associated construction impacts and capital/operating costs.
6. Granular Activated Sludge	×	×	✓	✓	✓	✓	Eliminated due to lack of full-scale application in North America. It is a batch process that would operate very different from the existing flow-through biological treatment process.

6.0 Tertiary Treatment

The main objective of secondary filtration is to reduce TSS and turbidity levels to comply with more stringent effluent requirements (compared to secondary effluent limitations). Filtration also further removes total (and in some technologies, even soluble) phosphorous remaining in secondary effluent.

Tertiary filtration is currently used at Nobleton WRRF.

6.1 Long List of Alternative Tertiary Treatment Technologies

6.1.1 Deep Bed Sand Filtration

Four deep bed Parkson Dynasand filters are installed in the Process Building at the Nobleton WRRF. Figure 6-1 shows Parkson Dynasand filter system shcematic.

This is a common filtration technology. Chemicals are added upstream to coagulate and flocculate solids containing phosphorus which are then removed by filtration in the sand matrix.

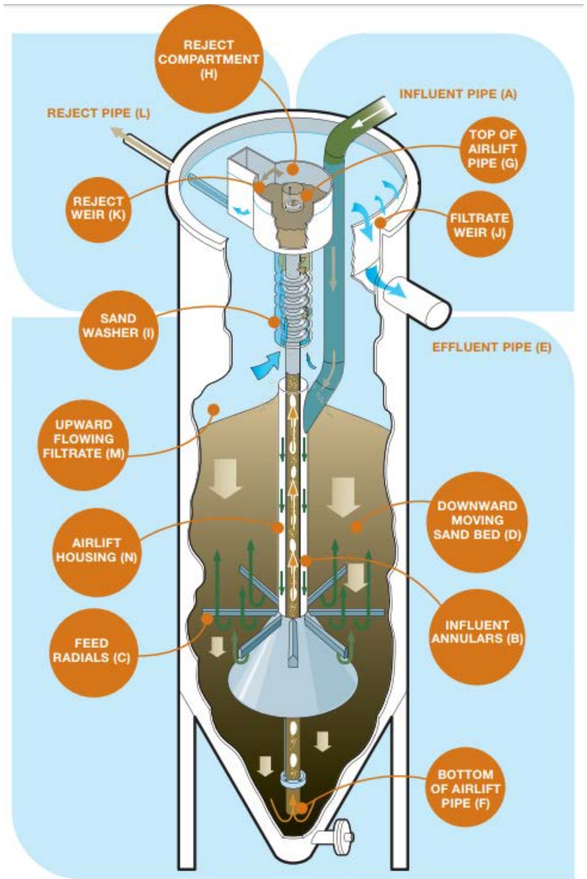


Figure 6-1 Parkson Dynasand Filter Process Schematic

6.1.2 Cloth Disk Filtration (CDF)

Cloth media filters are made of cloth woven or fiber pile (manufacturer dependent) with pores to filter TSS from the wastewater coming from the secondary system. Manufacturers may also offer different cloth media in order to address site-specific conditions (e.g., chemical resistance, different pore size characteristics). The use of woven pile cloth materials has emerged as the most common type of CDF due to improvements in backwash efficiency. Nominal pore size ranges between 5 and 10 µm for different type of cloth materials, but significant removals can be realized in smaller particle size ranges. The most common geometry for these filters is the disk configuration. Cloth disk filters are used as a pretreatment step prior to the membrane filtration system or for effluent TSS polishing, water reuse, and phosphorous removal.

According to the Water Environment Federation (WEF) manual of practice No. 8, typical maximum design filtration rates are between 240 to 280 L/(m^2·min). Although testing has shown that these filters can operate at hydraulic loading rates up to 800 L/(m^2·min) for short periods. The maximum hydraulic loading rate can also be limited by the influent TSS when the solids loading rate exceeds the manufacturer's recommendation.

During the filtration cycle, the wastewater flow is from the outside to the inside of the disks. Several cloth disks covered by cloth media are mounted vertically to a common hollow tube, which conveys filtered effluent from the filter. Wastewater passes through the cloth media by gravity and enters inside filter disks that are connected to the effluent line by the hollow tube. A total hydraulic head between 0.75 and 1.2 m is required for the operation of the disk filters.

Backwash cycle starts when the terminal headloss or a certain run time is reached. The disk filters backwash more frequently (e.g., compared to sand filters) because of the low head operational characteristics and low terminal headloss design values. Clean medium headloss ranges between 5 and 10 cm.

CFD technology was implemented in March 2014 by Nexom for a small municipal wastewater treatment plant for the community of Sundridge, ON (with the design flow of 0.45 MLD). After installing and having the two-tank infini-D system in operation for 18 months, effluent TP concentrations reduced from 8.3 mg/L to less than 0.1 mg/L.

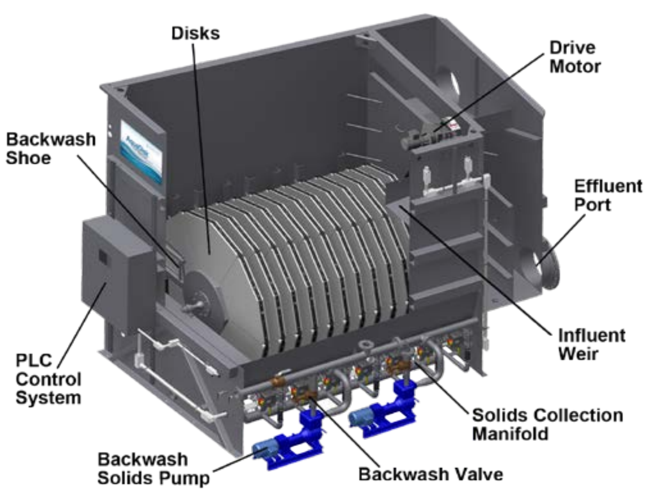


Figure 6-2 Cloth Media Filter with OptiFiber® Configuration

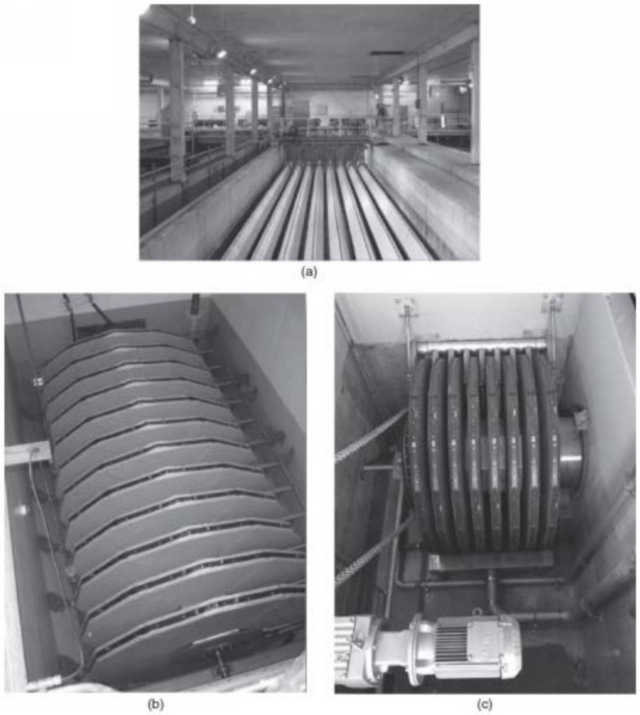


Figure 6-3 Different Cloth Depth Filter Configurations: (a) Aquadiamond Configuration; (b) Aquadisk

6.1.3 Blue PRO Filter System

The Blue PRO technology combines co-precipitation and sorption to remove both particulate and soluble phosphorus. It is similar to the deep bed sand filtration technology except that the media are coated with a chemical that adsorbs soluble phosphorus. Through these processes, some phosphorus is precipitated and removed from water as it moves upward though the sand media. At the same time, some phosphorus is adsorbed onto the hydrous ferric oxide coated sand. This adsorption mechanism allows the process to achieve very low concentrations of phosphorus in the effluent. The phosphorus is then removed from the sand through abrasion and separated in the sand washer at the top of the filter.

The Blue PRO process schematic is shown in Figure 6-4. An iron-based chemical is added to the wastewater before it passes into the rapid conditioning zone. The rapid conditioning zone allows the proper contact time for the mixture to optimize the adsorption process. The mixture enters the moving bed sand filter through distribution arms at the bottom of the sand bed, flowing upwards through the sand bed. The Blue PRO process uses ferric chloride or ferric sulphate for continuous regeneration of hydrous ferric oxide coated media for adsorption of phosphorus.

After filtration, treated water discharges from the top of the filter. Internally, the sand moves slowly from top to bottom, then returns to the top of the filter via an airlift located in the central assembly.

After adsorption, the iron and phosphorus are subsequently abraded off the sand both in the sand bed and in the airlift. A wash-box at the top of the filter separates sand from iron and phosphorus waste particulates. The sand is retained within the filter and falls back to the top of the bed; the residuals, including the iron and phosphorus or other contaminants, exit in a reject line.

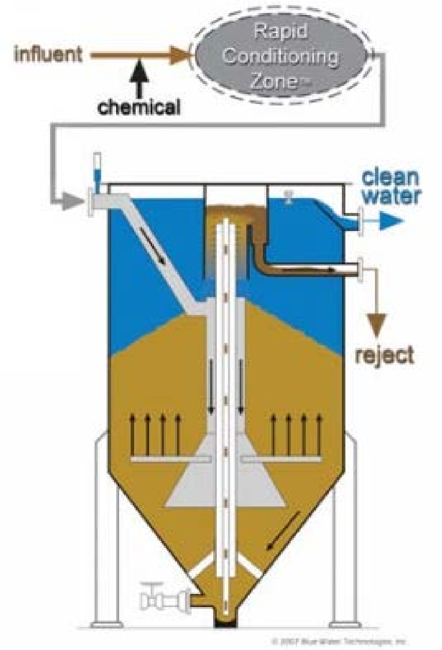


Figure 6-4 BluePRO Reactive Filtration System Process Schematic

6.1.4 Tertiary Low-Pressure Membrane Filtration (MF)

Membrane filtration is used to produce high quality effluent and serves as a pretreatment process for the reverse osmosis (RO) system. Membrane Filtration (MF) is a physical separation process sized based on the peak daily flow (PDF) and remove suspended/colloidal solids from the feed stream through a porous membrane. Figure 6-5 is a typical flow schematic that shows how membrane units and support systems are interrelated in an effluent filtration application.

Low-pressure membrane effluent filtration systems typically consist of the MF or Ultra-Filtration (UF) membrane system and various pretreatment and post-treatment systems. At a nominal size of 0.01μm, the UF membrane pores are approximately 1/10th the size of typical MF membrane pores. An MF membrane will reject particulates, including bacteria and suspended solids while the UF membranes can reject these solids as well as some macromolecules including emulsified oils. Compared with pressurized membrane systems, immersed membrane processes have significantly lower operating costs. For instance, the pumping energy needed for a 4,000 m^3/day immersed UF membrane system operating at 0.5 bar TMP and 65% pump efficiency is only 3.5 kW/h.

There are two types of membrane configurations: pressurized and immersed. Pressurized membrane configurations consist of membranes located within individual pressure vessels, with groupings of these pressure vessels housed in frames within buildings or on concrete pads. Immersed membrane configurations consist of membranes assembled into filter cells (also known as racks or cassettes) located within one or more tanks containing the wastewater to be treated. Ancillary systems for both configurations are typically located adjacent to the tanks or pressure vessels. Although the configurations are very different, the performance and filtrate water quality of the membranes are effectively the same.

Microfilter membranes operate by a surface removal mechanism and are similar to a fine screen or sieve. The pore size at the surface of most membranes is highly uniform and has a narrow pore size distribution. Particles larger than the size of the largest pore are rejected by the membrane surface and remain on the feed or concentrate side. The bulk carrier fluid, and any particles finer than the largest pore, can pass through the membrane to the filtrate side.

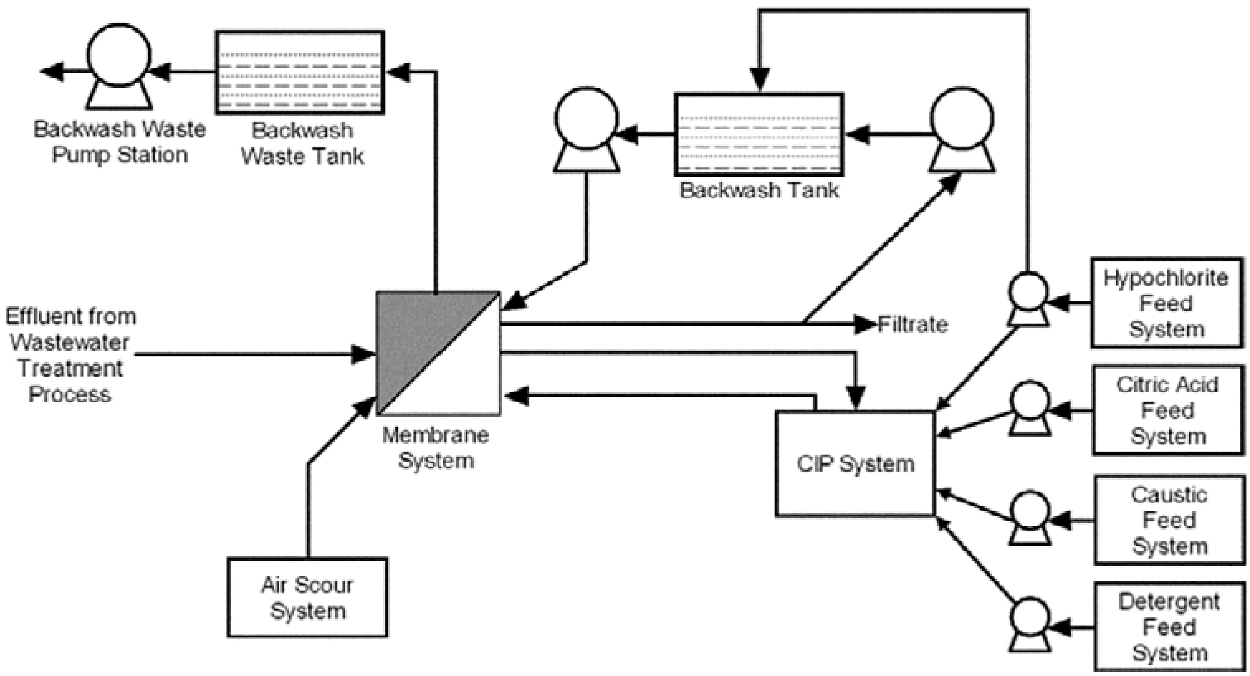


Figure 6-5 Diagram of a Typical Effluent Membrane Filtration System

6.1.5 Reverse Osmosis (High-Pressure Membrane Filtration)

Reverse Osmosis (RO) is a widely accepted unit operation for water purification. It is a high-pressure membrane filtration process with much smaller pores. This system consists of multiple components: 1) RO transfer pumps to pump the MF permeate through the Cartridge filters; 2) Cartridge filters for protection of the RO membranes; 3) RO high-pressure feed pumps to pump the water through the RO modules; 4) RO skids which hold the RO modules; and 5) Decarbonation system to raise the pH of the product water. The feedwater is treated by reverse osmosis after pretreatment and boosted to the required pressure by the high-pressure pump. The modules produce two process streams: (1) permeate, which is the product water, and (2) concentrate or reject, which is a waste stream. Figure 6-6 shows typical single-array and two-array reverse osmosis facility layouts. A significant advantage of the two-array configuration is that the product recovery is increased compared to single-stage operations.

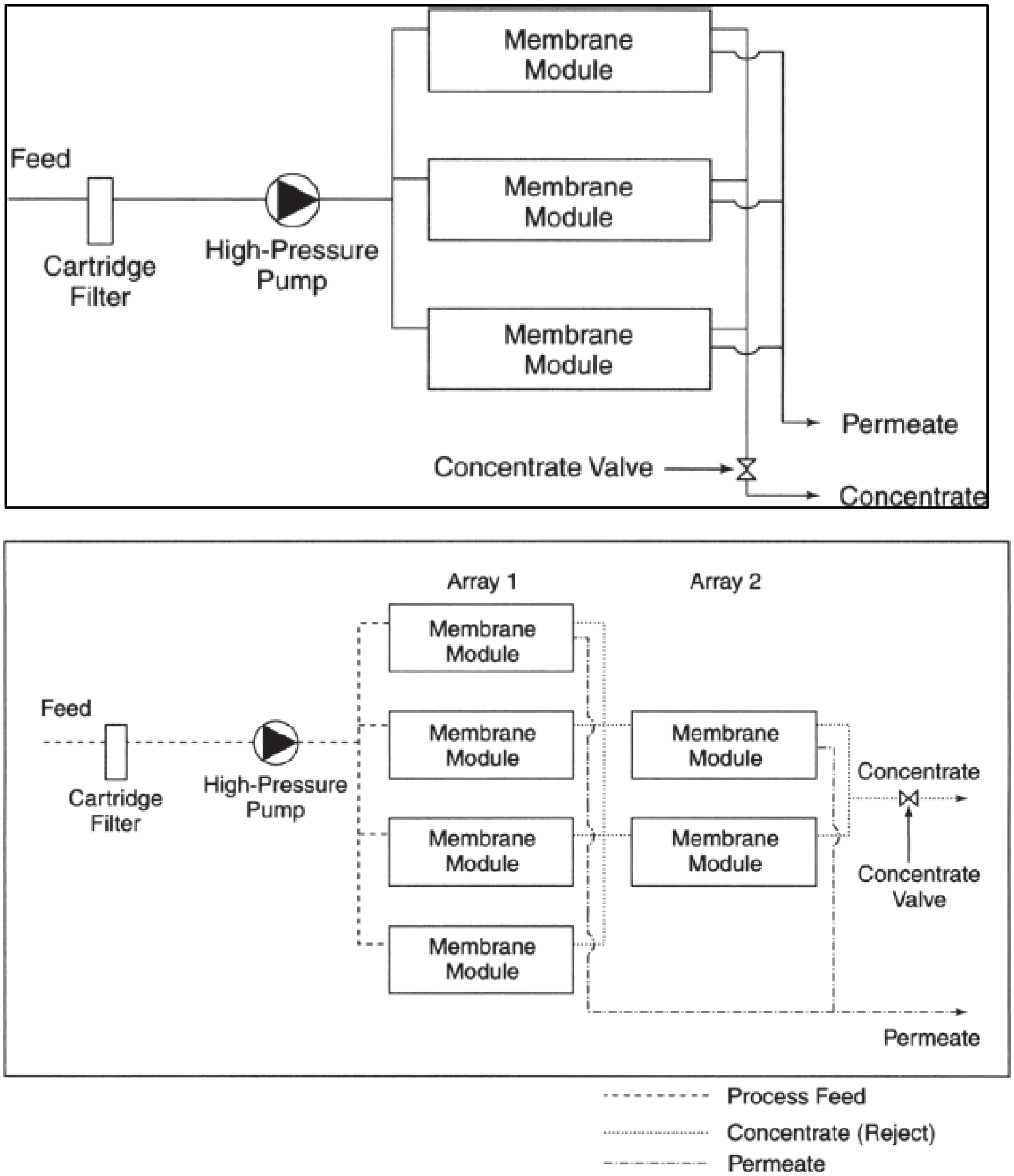


Figure 6-6 Simplified Schematic Diagram of a Single-Array (Top) and a Two-Array (Bottom) Reverse Osmosis Process

6.1.6 Tertiary Treatment Advantages and Disadvantages

Table 6-1 is a comparison of the secondary treatment options evaluated for this project.

Table 6-1 Comparison of Tertiary Treatment Enhancement Technologies

Technology	Status	Advantages	Disadvantages
Deep Bed Sand Filtration	Conventional: Well-established technology with numerous installations across North America	• Relatively common and able to meet tight effluent limits • Effective solids removal	• Relatively large footprint • Capital costs • Need for intermediate pumping
Cloth Disk Filtration (CDF)	Conventional: This is a mature technology that is widely used.	• Can reduce TSS concentrations down to 5 mg/L while removing TP down to less than 0.1 mg/L • Removal performance can be increased with chemical addition • Flexible in handling peal flows • Smaller footprint • Filtration operation is continuous due to small portion of media out of service during backwash – no need for backwash reject water storage basin • Filtered water used for backwash – no need for separate backwash water supply	• Chemical addition can prevent medium blinding if careful consideration not taken into account • Solids can sometimes pass through the pile media during high-pressure cleanings • Complicated system • Biological matter can grow on the filtrate side of the cloth • Filtration process must be taken offline to initiate high-pressure backwash cycle
Blue PRO Filter System	Emerging: 4 full-scale operations of Blue PRO Filter System.	• High efficiency and can remove 99+% of TP from municipal wastewater • Low chemical dose • No need for backwashing • Low capital, operating, and maintenance costs • Can reduce sludge handling costs • Works without pH adjustment • Highly tolerant of interfering water chemistry • Significantly lower turbidity and BOD.	• Large footprint • Large and tall building required over filters to allow for removal of air lift equipment • Proprietary equipment.

Technology	Status	Advantages	Disadvantages
Tertiary Low-Pressure Membrane Filtration (MF)	Conventional: This is a mature technology that is widely used.	• Smaller footprint • Automatically operated • Lower chemical usage • For Nobleton, a pressurized system will most likely be more cost effective • Membrane modules easily accessed	• Fouling • Membrane material properties, module hydrodynamic conditions, and feed water characteristics dictate the degree to which a membrane will foul
Reverse Osmosis (High-Pressure Membrane Filtration)	Conventional: This is a mature technology that is widely used.	• Removes nearly all contaminant ions and most dissolved non-ions • Capable of low effluent concentrations (especially TP) • Simplicity of operation • Automation allows for less operator attention • Demonstrated lowest of effluent phosphorous concentrations of current technologies	• High capital and operating costs • Permeate remineralization and brine disposal • Rejects charged species such as orthophosphate as well as large organic compounds • Consideration for reject brine disposal, permeate remineralization, and high energy cost in comparison to other alternatives

6.2 Screening of Long List of Alternative Wastewater Servicing Design Concepts

The screening of the long list alternatives of tertiary treatment technologies is shown in Table 6-2.

6.3 Short-List of Design Concepts

The following tertiary treatment technologies will be carried over for the final evaluation as an alternative design concept for the WRRF:

- Deep Bed Sand Filtration

Table 6-2 Screening of the Long List of Alternative Tertiary Treatment Technologies

Long List of Alternative tertiary Treatment Concepts	Screening Criteria: Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	Notes
1. Deep Bed Sand Filtration	✓	✓	✓	✓	✓	✓	Proceed to detailed evaluation. This is the current technology at Nobleton WRRF and is effective at obtaining Nobleton's effluent goals, and would require only a modest expansion for the future design.
2. Cloth Disk Filtration (CDF)	×	✓	✓	✓	✓	×	Eliminated due to cost of retrofitting or building a new filtration facility.
3. Blue PRO Filter System	✓	✓	✓	✓	✓	×	Eliminated due to the relative higher cost of retrofitting the technology.
4. Tertiary Low-Pressure Membrane Filtration (MF)	×	✓	✓	×	✓	×	Eliminated due to cost of retrofitting or building a new filtration facility and high operating costs. Membrane filtration is a higher level of treatment than required.
5. Reverse Osmosis (High-Pressure Membrane Filtration)	×	✓	✓	×	✓	×	Eliminated due to cost of retrofitting or building a new filtration facility and high operating costs. Reverse osmosis is a higher level of treatment than required.

7.0 Disinfection

The purpose of disinfection is to eliminate pathogens from treated wastewater.

UV disinfection technology is currently used at the Nobleton WRRF.

7.1 Long List of Alternative Design Concepts

7.1.1 Chlorine Based Methods

Chlorine is one of the most widely used disinfectants for municipal wastewater. It destroys target organisms by oxidizing cell wall material, causing leakage of cellular constituents outside of the cell. Overall, chlorine disinfection is reliable and effective against a wide spectrum of pathogenic organisms.

However, due to the toxicity of chlorine residuals at extremely low concentrations (11 to 19 μg/l) it is difficult to control chlorine-induced toxicity to aquatic life in the receiving waters. This is not as critical as an issue at the plant as the current chlorine residual ranges from 0.5 to 0.6 mg/L. With this effluent chlorine residual concentration, the plant has been able to eliminate the use of the use of a dechlorinating agent. Chlorination can also produce undesirable by-products such as trihalomethanes (THMs) and haloacetic acids (HAAs).

Additionally, some parasitic species have shown resistance to low doses of chlorine, including oocysts, of Crptosporidium parvum, cysts of Endamoeba histolytica and Giardia lamblia, and eggs of parasitic worms.

Two of the main forms of using Chlorine for disinfection are presented below.

7.1.1.1 Chlorine Gas

Chlorine gas (Cl_2) is the most common means of disinfection in the United States. Since chlorine gas is frequently used, the design parameters and dosing requirements are well established. The equipment is fairly reliable and easy to operate. Typical gaseous chlorine facilities are comprised of a chlorine cylinder storage area equipped with storage cradles, scales, chlorine gas detectors, and an overhead crane or hoist. Chlorine feeders transfer the chlorine from the cylinders and disperse a dose of chemical into a stream of water.

The largest drawback to chlorine gas is the significant health hazard that an accidental release would incur on the surrounding community while in transport or at the plant. An emergency scrubber is commonly installed to capture and neutralize any chlorine gas leaks, but this is not full-proof.

7.1.1.2 Bulk Sodium Hypochlorite

Bulk Sodium Hypochlorite, commonly known as “liquid bleach”, is another common form of chlorine for disinfection. It is generally produced as a 12.5% w/v NaOCl diluted aqueous solution, and is increasing in water and wastewater treatment applications due to safety concerns associated with the use, storage and transport of chlorine gas. Caution has to be exercised in the handling and storage of sodium hypochlorite to prevent exposure and minimize degradation of the chemical. Due to the toxicity of chlorine residuals, bisulfite is used to quench the residual chlorine levels. Figure 7-1 shows a hypochlorite storage facility located in California.

Figure 7-1 Hypochlorite Storage Facility

7.1.2 Peracetic Acid

Peracetic Acid, or PAA, has been regularly used as a wastewater disinfectant in Europe and Canada for the past 30 years. It is a clear, colorless liquid that forms an equilibrium mixture with hydrogen peroxide and acetic acid. It is reported to be an inherently stronger oxidant and more rapid disinfectant than chlorine-based disinfectants. Additionally, it dissipates rapidly and does not generate harmful disinfectant byproducts even if overdosed. The largest drawback of PAA use in the plant is the absence of U.S. operation standards as it is still under investigation and testing by the EPA. Figure 7-2 shows a PPA storage tank facility.

Figure 7-2 PPA Storage Tank Facility

7.1.3 Ultraviolet Irradiation

Over the past several years, UV disinfection technology has grown in popularity, resulting in growth of new technology and more sophisticated and reliable systems that operate more cost effectively. It is a physical disinfecting agent, separating it from the chemical disinfectant options, using ~254 nm wavelength to penetrate cell walls and break apart the cellular DNA and RNA. UV light is effective as both a bactericide and virucide. Since UV light is not a chemical agent, no toxic residuals are produced. An example of an UV system is shown in Figure 7-3.

Figure 7-3 UV Disinfection System

The main water quality parameter used to specify UV disinfection systems and with which the performance is determined is UV transmittance (UVT). It is important to understand seasonal, wet-weather, and diurnal UVT trends. The importance of UVT is borne out of the fact that, for each 0.05 drop in UVT (on a zero to one scale), only half the volume of water can be disinfected using the same predetermined dosage rate.

Many UV disinfection systems have been installed in municipal wastewater treatment plants as effluent chlorine residual limits become tighter. There are multiple UV technology systems on the market today, and new advances are emerging as the market responds to user demands.

Two banks of low-pressure, low output bulbs are installed in a channel downstream from tertiary filtration in the Process Building

7.1.4 Disinfection Advantages and Disadvantages

Table 7-1 is a comparison of the disinfection technology options evaluated for this project.

Table 7-1 Comparison of Disinfection Treatment Enhancement Technologies

Technology	Status	Advantages	Disadvantages
Chemical Based Disinfection Technologies			
Chlorine Gas	Conventional: One of the most widely used disinfectants for municipal wastewater.	• Widely used • Reliable and effective against wide spectrum of pathogenic organisms • Dosing flexibility to handle peak flows • Ease of implementation • Chlorine scrubbing towers can mitigate the risk of chlorine gas	• Toxicity of chlorine residuals at extremely low concentrations – chlorine induced toxicity to aquatic life • Needs dichlorination agent if effluent chlorine residual concentrations are too high • Can produce undesirable by-products such as trihalomethanes and haloacetic acids • Some parasitic species have shown resistance to low doses of chlorine • Significant health hazard should an accidental release occur
Bulk Sodium Hypochlorite	Conventional: Another common form of chlorine for disinfection.	• Widely used • Reliable and effective against wide spectrum of pathogenic organisms • While more expensive per unit weight of chlorine than chlorine gas, aqueous form poses less health hazards – incur lower costs • Ease of implementation	• Toxicity of chlorine residuals at extremely low concentrations – chlorine induced toxicity to aquatic life • Needs dichlorination agent if effluent chlorine residual concentrations are too high • Can produce undesirable by-products such as trihalomethanes and haloacetic acids • Some parasitic species have shown resistance to low doses of chlorine • Can handle peak flows so long as chemicals are available
Peracetic Acid	Conventional: Regularly used as a wastewater disinfectant in Europe and Canada for the past 30 years.	• Widely used • Stronger oxidant and more rapid disinfectant the chlorine-based disinfectants • Dissipates rapidly and does not generate harmful disinfectant byproducts even if overdosed • Potential to be expanded for future growth/regulatory requirements • Ease of implementation	• Reliably proven for smaller facilities only (which is fine in this application as Nobleton is a smaller facility) • Operating cost highly dependent on market price for PAA

Technology	Status	Advantages	Disadvantages
Physical Based Disinfection Technologies			
Ultraviolet Irradiation	Conventional: Grown rapidly in the past several years, and is widely used across North America.	• Not a chemical agent – no toxic residuals are produced • Reliable • Operate more cost effectively • Potential to be expanded for future growth/regulatory requirements • Existing facility already has UV disinfection – capital cost would not be much compared to other alternatives having to replace/rehab existing infrastructure • Ease of implementation	• Capital cost – requires significant capital investment • Operating costs include electricity as a significant portion

7.2 Screening of Long List of Alternative Wastewater Servicing Design Concepts

The screening of the long list alternatives of disinfection treatment technologies is shown in Table 7-2.

7.3 Short-List of Design Concepts

The following disinfection treatment technologies will be carried over for the final evaluation as an alternative design concept for the WRRF:

- UV Irradiation

Table 7-2 Screening of the Long List of Alternative Disinfection Treatment Technologies

Long List of Alternative Disinfection Treatment Concepts	Screening Criteria						Notes
	Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	
1. Chlorine Gas	✗	✓	✓	✓	✓	✗	Eliminated due to UV disinfection already existing at Nobleton WRRF. By swapping to chemical-based disinfection, Nobleton WRRF would incur capital costs in order to change the existing channel to a contact basin and would add in operating costs.
2. Bulk Sodium Hypochlorite	✗	✓	✓	✓	✓	✗	Eliminated due to UV disinfection already existing at Nobleton WRRF. By swapping to chemical-based disinfection, Nobleton WRRF would incur capital costs in order to change the existing channel to a contact basin and would add in operating costs.
3. Peracetic Acid	✗	✓	✓	✓	✓	✗	Eliminated due to UV disinfection already existing at Nobleton WRRF. By swapping to chemical-based disinfection, Nobleton WRRF would incur capital costs in order to change the existing channel to a contact basin and would add in operating costs.
4. Ultraviolet Irradiation	✓	✓	✓	✓	✓	✓	Proceed to detailed evaluation. This is the current technology existing at the Nobleton WRRF. Technology is compatible with existing WRRF, a proven technology, performs robustly, satisfies regulatory stakeholders, with acceptable associated construction impacts and capital/operating costs.

8.0 Sludge Thickening and Dewatering

The purpose of sludge thickening and dewatering is to reduce the volume and weight of sludge for hauling or downstream handling. The product of sludge thickeners is liquid, the product of sludge dewatering is cake.

A sludge thickener is installed in the Nobleton WRRF Process Building. Sludge dewatering

8.1 Long List of Alternative Sludge Thickening Technologies

8.1.1 Sludge Thickening - Gravity

8.1.1.1 Gravity Thickeners

Gravity thickening is one of the most common methods used for solids thickening and is accomplished in a tank similar in design to a conventional sedimentation tank. Feed sludge is allowed to settle and compact, and the thickened sludge is withdrawn from the bottom of the tank.

Gravity thickening is primarily used for primary sludge and mixtures of primary and waste activated sludge. Due to better performance of other thickening methods for WAS, gravity thickening has limited application for such sludges. Gravity thickening on untreated primary sludge, or primary sludge mixed with waste active sludge, is often used as it can achieve resulting sludge concentrations in the range of 4 to 6 percent.

A non-mechanical gravity thickener is currently used to thicken waste activated sludge prior to storage and hauling.

8.1.1.2 Dissolved Air Flotation (DAF)

Dissolved air flotation (DAF) thickening concentrates solids by attaching microscopic air bubbles to the suspended solids, increasing the buoyancy of the solids and causing them to float to the surface. A recycle stream from the DAF subnatant is super-saturated with air and discharge into the DAF influent. When this combined stream (whitewater) is released in the DAF, the dissolved air comes out of solution forming fine bubbles. A pressure tank (saturator) and compressor system has been typically used to make the whitewater; however, air handling recycle pumps are available that combine the pumping and air injection steps, eliminating the need for saturators and compressors. A DAF thickener is shown in Figure 8-1.

Figure 8-1 DAF Thickener (Courtesy of Envirex)

Dissolved air flotation thickeners are typically sized based on the solids loading rates and can be operated with or without polymer conditioning. Variables that can affect the performance of a DAF thickener include hydraulic loading, recycle flow, air-to-solids ratio, dissolution ratio, and the rate of removal of the float solids. The thickened solids concentrations range from 3 to 4 percent at greater than 90 percent capture efficiency. At this concentration, polymer is unlikely to be required, but the facility should be provided as a backup. DAF thickening technology is available from a number of manufacturers, including Evoqua/Envirex, Suez, and Ovivo.

8.1.2 Sludge Thickening - Mechanical

8.1.2.1 Centrifugation

Centrifuge thickening is commonly used for WAS thickening in medium- to large-capacity facilities. It is a self-contained process that uses high speed centrifugal forces to separate suspended solids from the liquid. The solids are forced to the perimeter of the bowl, conveyed by a scroll to one end of the unit and discharged. The liquid flows through ports at the opposite end of the unit and is typically returned to the headworks. The principle of operation is presented in Figure 8-2. An installed unit is shown in Figure 8-3. Centrifuge equipment is available from a number of manufacturers, including Westfalia, Andritz, and Alfa Laval.

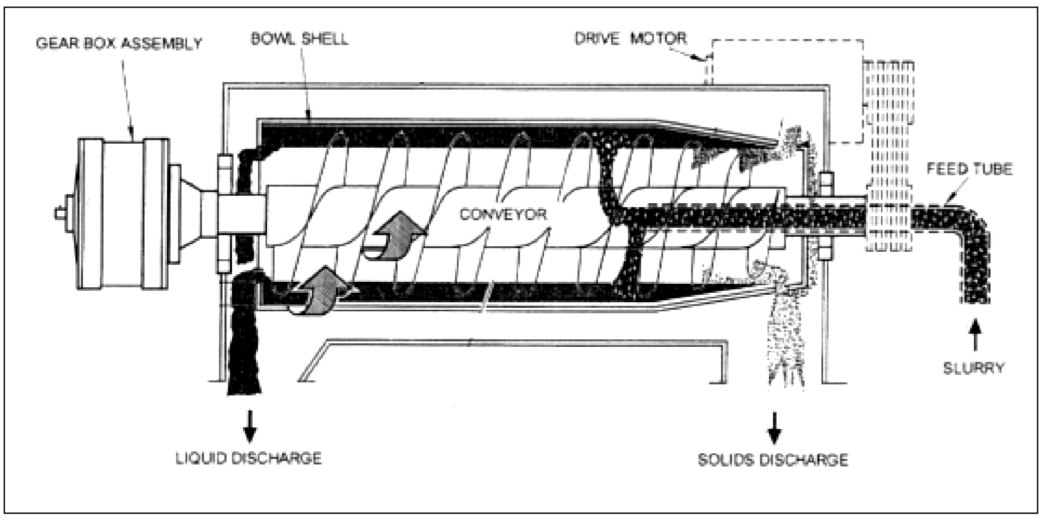


Figure 8-2 Centrifuge Principle of Operation (Courtesy of Alfa Laval)

Figure 8-3 Installed Centrifuge

In WAS thickening applications, centrifuge typically achieve solids concentrations ranging from 5 to 6 percent at solids capture efficiencies of 90 to 95 percent. Higher solids concentrations up to 8 percent TS are possible in co-thickening applications. Polymer addition can increase solids capture to approximately 95 percent, but generally does not increase the thickened solids concentration. Typically, facilities using centrifuges for WAS thickening feed up to 10 pounds of polymer per dry ton of solids; however, some installations have been able to operate thickening centrifuges with little or no polymer. Operational control of the process is possible through variation of hydraulic throughput, adjustment of scroll speed, pool depth, and polymer feed.

Centrifuges have higher power consumption than the other thickening technologies. Routine maintenance of centrifuges can be performed by the plant staff, but periodically the scroll/bowl assembly may have to be shipped to a maintenance facility. This can result in extended downtime for the equipment. Some centrifuge suppliers have started providing replacement scroll/bowl assemblies for use at the time the existing one is pulled to minimize downtime.

8.1.2.2 Gravity Belt Thickener (GBT)

Gravity belt thickeners have widespread use for WAS thickening applications. Gravity belt thickeners separate free water from the solids by gravity drainage through a porous belt. Dilute solids are introduced at the head end of a horizontal filter belt. As the solids move along the belt, free water drains through the porous belt into a collection tray and is returned to the headworks. Plows in the gravity zone break up the solids and aid the release of water. Thickened solids are discharged at the end of the horizontal filter belt. Gravity belt thickeners are available in belt widths ranging from 1 to 3 meters. Figure 8-4 and Figure 8-5 show the operation principle of a GBT and an installed unit, respectively.

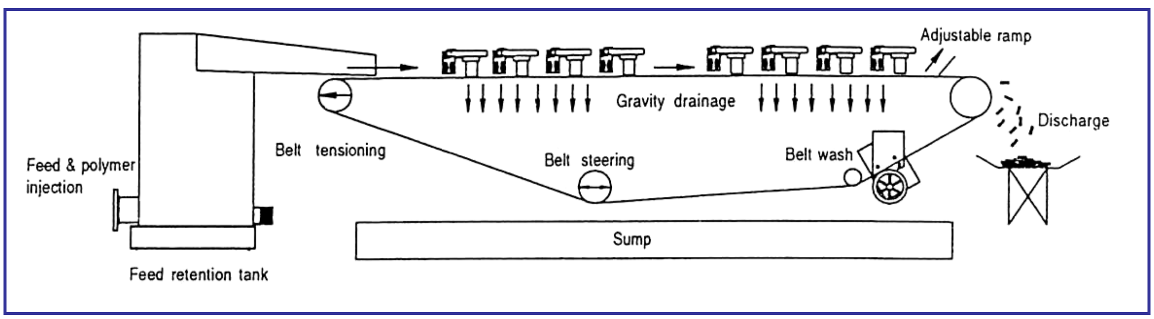


Figure 8-4 Gravity Belt Thickener Principle of Operations (Courtesy of Ashbrook)

Figure 8-5 Installed Gravity Belt Thickeners at the Bissell WWTP

The feed solids are conditioned with a polymer to form a stable floc before introduction to the belt. With the use of a polymer, GBTs can achieve solids captures of 95 percent. Operation of a GBT can be controlled by adjusting solids feed rate, polymer feed rate, belt speed to control solids retention time on the belt, and position of the solids plow.

Gravity belt thickeners have an open equipment design and can be difficult to capture odorous emissions for treatment, requiring odour control for the whole airspace. The belt has to be washed continuously to avoid blinding. They also require 1/2 hour operator attendance on startup and shutdown. Gravity belt thickeners are available from several manufacturers, including Bellmer, Komline-Sanderson, Ashbrook, and Siemens.

8.1.2.3 Rotary Drum Thickener/Rotary Screw Thickener

Rotary drum thickeners (RDT) and rotary screw thickeners (RST) are parallel technologies based on a similar premise. Both technologies use gravity to drain the solids as they pass through a mesh or perforated basket. Besides the need for polymer addition, a flocculation tank upstream, and a system of spray nozzles to keep the media clean, the main differences between the technologies are:

- RDTs:
  - Rotating shell made of wire or polyethylene mesh or perforated steel
  - Drum is differentiated into zones based on mesh size, with a finer mesh at the inlet where the feed solids contain more water and mesh size increases towards the drum outlet to facilitate drainage of the more concentrated solids
  - Feed solids are pumped into the drum, where drum rotation helps drive the filtrate through the perforations into a collection trough
  - Rings of varying heights inside the drum control the solids retention time in each zone

    - RDTs can produce 4-6 percent solids with 95 percent solids recovery with the use of polymer
    - Typically enclosed to contain odours
- RSTs:
    - Uses rotating screws with stationary drums
    - Flocculated solids overflow into the lower portion of the inclined drum with a static perforated basket
    - Equipped with a slowly rotating screw that conveys solids upward to the drum discharge while allowing water to drain through the basket
    - Basket is continuously cleaned with brushes to prevent solids accumulation and periodically cleaned with an automatic spray wash
    - RSTs can produce 4-8 percent solids with 95 percent solids capture

Figure 8-6 and Figure 8-7 show a rotary drum thickener and a rotary screw thickener, respectively.

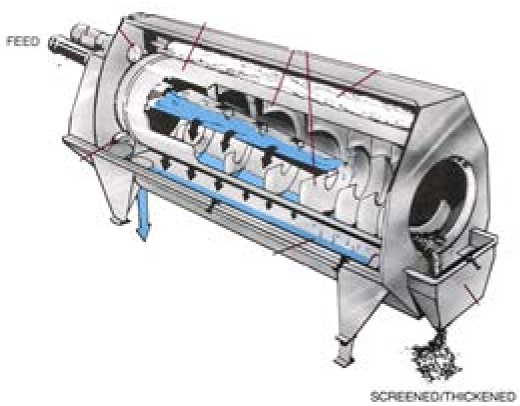


Figure 8-6 Rotary Drum Thickener Principle of Operation (Courtesy of Parkson)

Figure 8-7 Rotary Screw Thickener (Courtesy of Huber)

8.1.3 Solids Thickening Technologies Advantages and Disadvantages

Table 8-1 is a comparison of the solids thickening treatment options evaluated for this project.

Table 8-1 Comparison of Solids Thickening Technologies

Technology	Status	Advantages	Disadvantages
Non Mechanical Thickening			
Gravity Thickeners	Conventional: This is a mature technology that is widely used.	• Proven technology • Currently existing at facility	• WAS only sludge – performance only 2-3% solids
Dissolved Air Flotation (DAF)	Conventional: This is a mature technology that is widely used.	• Provides "wide spot" in line, minimizing need for WAS storage • Little operator attention • Can be designed for low or no polymer consumption • Relatively insensitive to hydraulic loading rate changes • Technology available from several manufacturers	• Relatively high power use – varies depending on saturation technology • Open tank, requiring odour control for the whole building airspace • Can achieve lower thickened solids concentration than other thickening technologies (WAS only DAFs) • Can have large footprint requirement • Higher capital costs compared to some of the other thickening technologies
Mechanical Thickening			
Centrifuge	Conventional: This is a mature technology that is widely used.	• High capacity equipment – well suited for larger plants • Higher solids concentrations (5-8% TS), depending on feed solids characteristics • Minimum space requirements • Little operator attention when operations are stable • Enclosed technology – good odour containment and housekeeping • Technology available from several manufacturers	• Higher capital costs compared to some of the other thickening technologies • Higher energy use • Major maintenance must be performed by the manufacturer • Polymer required • Closer operator attention is required to achieve thickened concentrations less than 5%

Technology	Status	Advantages	Disadvantages
Gravity Belt Thickener (GBT)	Conventional: This is a mature technology that is widely used.	• Moderate operational complexity; relatively low requirement for operator attention • Relatively high unit capacity • Relatively low initial capital cost • Low power requirements	• Open equipment design – potential for odours and high humidity • Require frequent belt washing to avoid blinding – high wash water flows • Requires operator intervention at startup • Closer operator attention is required to achieve thickened concentrations less than 5% • Polymer required
Rotary Drum Thickener/ Rotary Screw Thickener	Conventional: This is a mature technology that is widely used.	• Moderate operational complexity • Low initial capital cost • Low power usage • Good odour containment • Technology available from several manufacturers	• Higher polymer consumption – varies by manufacturer • High wash water requirements • Relatively low unit capacities • Closer operator attention is required to achieve thickened concentrations less than 5% • Requires operator intervention at startup

8.1.4 Sludge Dewatering

8.1.4.1 Centrifuges

Centrifugation is used widely in the industry as a means to separate liquids of different density, thickening slurries, or removing solids. Centrifuge types for dewatering applications include solid bowl, basket, and disc centrifuges. The most frequently used of these is the continuous countercurrent solids bowl centrifuge. In this type of centrifuge, sludge is fed at a constant flowrate into a rotating bowl, where the sludge separates into either a dense cake containing solids or a dilute liquid stream called "centrate."

Solid-bowl centrifuges are suitable for a number of dewatering applications and chemicals can be used to aid in conditioning to achieve the desired dewatering performance.

8.1.4.2 Belt Filter Presses

A belt filter press consists of two continuous, separate belts. One belt is a press belt and the other is a filter belt. The sludge is confined between the two belts with the press belt exerting pressure on the filter belt, therefore continuously dewatering the sludge.

For belt filter presses, there are generally three distinct dewatering zones. The first zone is the gravity drainage zone, the second is the pressure zone, and the third is the shear zone. Pressure is exerted by the rollers, conveying belts, or other external devices. The shear zone allows the cake to be further dewatered by deforming the sludge cake by passing the belts around rolls and/or between vertically offset rollers causing a serpentine-like configuration in the sludge cake movement.

8.1.4.3 Filter Presses

Filter presses are a conventional means of dewatering that were on the decline; however, recent changes in the design of filter presses, including the elimination of leakage problems, more automation, improved filter media, greater unit capacities, and the development of high molecular weight polymers and compatible polymer feed systems has resulted in a renewed interest in this sludge dewatering technology.

8.1.5 Solids Dewateromg Technologies Advantages and Disadvantages

Table 8-2 is a comparison of the solids dewatering treatment options evaluated for this project.

Table 8-2 Comparison of Solids Dewatering Technologies

Technology	Status	Advantages	Disadvantages
Centrifuges	Conventional: This is a mature technology that is widely used.	• Clean appearance • Minimal odour problems • Fast startup and shut down capabilities • Easy to install • Produces relatively dry sludge cake • Low capital cost-to-capacity ratio	• Scroll wear potentially a high maintenance problem • Requires grit removal and possibly sludge grinder in the feed stream • Skilled maintenance personnel required • Moderately high suspended solids content in centrate • Cannon observe dewatering zone to optimize/adjust performance
Belt Filter Presses	Conventional: This is a mature technology that is widely used.	• Low energy requirements • Relatively low capital and operating costs • Less complex mechanically and easier to maintain • High pressure machines are capable of producing very dry cake • Minimal effort required for a system shut down	• Hydraulically limited in throughput • Requires sludge grinder in feed stream • Very sensitive to incoming sludge feed characteristics • Short media life as compared to other devices using cloth media • Automatic operation generally not advised
Filter Presses	Conventional: This is a mature technology that is widely used.	• Highest cake solids concentration • Low suspended solids in filtrate • Simple operation • High solids capture rate	• Batch operation • High equipment cost • High labor cost • Special support structure requirements • Large floor area required for equipment • Skilled maintenance personnel required • Additional solids due to large chemical addition require disposal.

8.2 Screening of Long List of Alternative Sludge Thickening and Dewatering Technologies

The screening of the long list alternatives of solids treatment technologies is shown in Table 8-3 on the following page. Based on the screening completed in the following table, the only options that carry over are solids thickening by gravity thickening or by mechanical thickening. Based on the variety of solids thickening technologies, further screening is completed in Table 8-4 on various thickening technologies.

8.3 Short-List of Design Concepts

The following solids thickening treatment technologies will be carried over for the final evaluation as an alternative design concept for the WRRF:

- Gravity Thickening
- Mechanical Thickening
  - Gravity Belt Thickener
  - Rotary Drum Thickener/Rotary Screw Thickener

Table 8-3 Screening of the Long List of Alternative Solids Alternatives

Long List of Alternative Solids Treatment Concepts	Screening Criteria						Notes
	Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	
1. Gravity Thickening	✓	✓	✓	✓	✓	✓	Proceed to detailed evaluation.
2. Mechanical Thickening	✓	✓	✓	✓	✓	✓	Proceed to detailed evaluation.
3. Dewatering	✗	✓	✓	✓	✗	✗	Eliminated due to incompatibility with the WRRF, construction impacts, and cost. In order to add in solids dewatering, the WRRF will be also required to upgrade solids thickening capacity prior to dewatering which will incur construction impacts and higher costs.

Table 8-4 Screening of the Long List of Alternative Solids Thickening Technologies

Long List of Alternative Solids Thickening Treatment Concepts	Screening Criteria						Notes
	Compatibility	Proven Technology	Performance Robustness	Stakeholder Acceptance	Construction Impacts	Cost	
1. Gravity Thickeners	✓	✓	✓	✓	✓	✓	Eliminated due to ....
2. Dissolved Air Flotation (DAF)	×	✓	✓	✓	×	×	Proceed to detailed evaluation.
3. Centrifuge	✓	✓	✓	×	✓	✓	
4. Gravity Belt Thickener (GBT)	✓	✓	✓	✓	✓	✓	
5. Rotary Drum Thickener/Rotary Screw Thickener	✓	✓	✓	✓	✓	✓	

9.0 Summary

The short lists for each of these stages of treatment will be carried over into the Technical Memo #3 to go through Stage 2 of the technology evaluation for the alternative design concepts for the Nobleton WRRF. The short lists for each stage are as follows in Table 9-1:

Table 9-1 Short-Listed Technology Alternatives for Each WRRF Treatment Process

WRRF Treatment Process	Short Listed Technology Alternative(s)	Notes
Coarse Screening	A. Climber Screen	Existing technology. This option would be used with conventional secondary treatment processes
Fine Screening	A. Perforated plate	This option would be used with secondary treatment in intensified secondary treatment processes
Grit Removal	A. Induced vortex	Existing technology
Primary Treatment	A. Primary Filtration	Primary treatment applies only to alternative wastewater design concepts that include primary treatment
Secondary Treatment Conventional	A. Extended Aeration	Existing technology
Secondary Treatment Intensification	A. Membrane-Aerated Biofilm Reactor	
Tertiary Treatment	A. Two-Stage sand filtration	Existing technology
Effluent Disinfection	A. Ultraviolet disinfection	Existing technology
Sludge Thickening	A. Gravity Thickener B. Mechanical Thickening	The short list is evaluated in this Section.

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Appendix B. Calculations for Storage Volume of the Flow Attenuation Tank at the Janet Avenue Pumping Station

INLET HYDROGRAPH AT THE JANET AVENUE PUMPING STATION FOR A 1 IN 25 YEAR STORM, AND CALCULATIONS FOR STORAGE VOLUME

MAXIMUM PUMPING RATE FOR JANET AVENUE PUMPING STATION (m^3/s) **0.145**

Date	Time	DS Flow (m^3/s)	US Flow (m^3/s)	Rainfall (Rainfall intensity (mm/hr))	PUMPING RATE (ASSUMED CONSTANT) m^3/s	PUMPED VOLUME OVER THE 5 MINUTE TIMESTEP m^3	INFLOW VOLUME OVER THE 5 MINUTE TIMESTEP (m^3)	DEPARTURE (INFLOW-OUTFLOW) m^3/ 5 Minutes	CUMULATIVE EXCESS OUTFLOW (m^3)
1/1/2016	0:00:00	0.033045	0.033045	0	0.145	43.5	9.9135	-33.5865	
1/1/2016	0:05:00	0.033047	0.033047	0	0.145	43.5	9.9141	-33.5859	
1/1/2016	0:10:00	0.033031	0.03303	0	0.145	43.5	9.9093	-33.5907	
1/1/2016	0:15:00	0.032932	0.032929	0	0.145	43.5	9.8796	-33.6204	
1/1/2016	0:20:00	0.032679	0.032673	0	0.145	43.5	9.8037	-33.6963	
1/1/2016	0:25:00	0.032278	0.032268	0	0.145	43.5	9.6834	-33.8166	
1/1/2016	0:30:00	0.031225	0.031191	0	0.145	43.5	9.3675	-34.1325	
1/1/2016	0:35:00	0.02885	0.028797	0	0.145	43.5	8.655	-34.845	
1/1/2016	0:40:00	0.026621	0.026587	0	0.145	43.5	7.9863	-35.5137	
1/1/2016	0:45:00	0.0253	0.025278	0	0.145	43.5	7.59	-35.91	
1/1/2016	0:50:00	0.024415	0.0244	0	0.145	43.5	7.3245	-36.1755	
1/1/2016	0:55:00	0.025567	0.025685	0	0.145	43.5	7.6701	-35.8299	
1/1/2016	1:00:00	0.038756	0.038911	0	0.145	43.5	11.6268	-31.8732	
1/1/2016	1:05:00	0.03575	0.035609	0	0.145	43.5	10.725	-32.775	
1/1/2016	1:10:00	0.028486	0.028355	0	0.145	43.5	8.5458	-34.9542	
1/1/2016	1:15:00	0.023539	0.023466	0	0.145	43.5	7.0617	-36.4383	
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1/1/2016	1:35:00	0.017623	0.017607	0	0.145	43.5	5.2869	-38.2131	
1/1/2016	1:40:00	0.016914	0.0169	0	0.145	43.5	5.0742	-38.4258	
1/1/2016	1:45:00	0.018114	0.018241	0	0.145	43.5	5.4342	-38.0658	
1/1/2016	1:50:00	0.029965	0.030093	0	0.145	43.5	8.9895	-34.5105	
1/1/2016	1:55:00	0.027068	0.026936	0	0.145	43.5	8.1204	-35.3796	
1/1/2016	2:00:00	0.021362	0.021265	0	0.145	43.5	6.4086	-37.0914	
1/1/2016	2:05:00	0.017729	0.017665	0	0.145	43.5	5.3187	-38.1813	
1/1/2016	2:10:00	0.015076	0.015021	0	0.145	43.5	4.5228	-38.9772	
1/1/2016	2:15:00	0.013461	0.013439	0	0.145	43.5	4.0383	-39.4617	
1/1/2016	2:20:00	0.012902	0.012894	0	0.145	43.5	3.8706	-39.6294	
1/1/2016	2:25:00	0.01259	0.012583	0	0.145	43.5	3.777	-39.723	
1/1/2016	2:30:00	0.012319	0.012312	0	0.145	43.5	3.6957	-39.8043	
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1/1/2016	2:40:00	0.011839	0.011834	0	0.145	43.5	3.5517	-39.9483	
1/1/2016	2:45:00	0.011642	0.011637	0	0.145	43.5	3.4926	-40.0074	
1/1/2016	2:50:00	0.011471	0.011467	0	0.145	43.5	3.4413	-40.0587	
1/1/2016	2:55:00	0.011325	0.011321	0	0.145	43.5	3.3975	-40.1025	
1/1/2016	3:00:00	0.011613	0.011651	0	0.145	43.5	3.4839	-40.0161	
1/1/2016	3:05:00	0.020803	0.021082	0	0.145	43.5	6.2409	-37.2591	
1/1/2016	3:10:00	0.023196	0.023108	0	0.145	43.5	6.9588	-36.5412	
1/1/2016	3:15:00	0.01861	0.018518	0	0.145	43.5	5.583	-37.917	
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1/1/2016	3:40:00	0.011018	0.011016	0	0.145	43.5	3.3054	-40.1946	
1/1/2016	3:45:00	0.010979	0.010979	0	0.145	43.5	3.2937	-40.2063	
1/1/2016	3:50:00	0.010979	0.010979	0	0.145	43.5	3.2937	-40.2063	
1/1/2016	3:55:00	0.010988	0.010989	0	0.145	43.5	3.2964	-40.2036	
1/1/2016	4:00:00	0.011003	0.011003	0	0.145	43.5	3.3009	-40.1991	
1/1/2016	4:05:00	0.011021	0.011022	0	0.145	43.5	3.3063	-40.1937	
1/1/2016	4:10:00	0.011046	0.011046	0	0.145	43.5	3.3138	-40.1862	
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1/1/2016	4:20:00	0.011113	0.011114	0	0.145	43.5	3.3339	-40.1661	
1/1/2016	4:25:00	0.011161	0.011162	0	0.145	43.5	3.3483	-40.1517	
1/1/2016	4:30:00	0.011227	0.011231	0	0.145	43.5	3.3681	-40.1319	
1/1/2016	4:35:00	0.014016	0.014235	0	0.145	43.5	4.2048	-39.2952	
1/1/2016	4:40:00	0.025514	0.02557	0	0.145	43.5	7.6542	-35.8458	
1/1/2016	4:45:00	0.022046	0.021941	0	0.145	43.5	6.6138	-36.8862	
1/1/2016	4:50:00	0.017715	0.017636	0	0.145	43.5	5.3145	-38.1855	
1/1/2016	4:55:00	0.014649	0.014586	0	0.145	43.5	4.3947	-39.1053	
1/1/2016	5:00:00	0.012712	0.012685	0	0.145	43.5	3.8136	-39.6864	
1/1/2016	5:05:00	0.012069	0.01206	0	0.145	43.5	3.6207	-39.8793	
1/1/2016	5:10:00	0.01189	0.011888	0	0.145	43.5	3.567	-39.933	
1/1/2016	5:15:00	0.011956	0.011961	0	0.145	43.5	3.5868	-39.9132	
1/1/2016	5:20:00	0.012287	0.012298	0	0.145	43.5	3.6861	-39.8139	
1/1/2016	5:25:00	0.012841	0.012855	0	0.145	43.5	3.8523	-39.6477	
1/1/2016	5:30:00	0.013473	0.013488	0	0.145	43.5	4.0419	-39.4581	
1/1/2016	5:35:00	0.014166	0.014183	0	0.145	43.5	4.2498	-39.2502	
1/1/2016	5:40:00	0.014939	0.014958	0	0.145	43.5	4.4817	-39.0183	

INLET HYDROGRAPH AT THE JANET AVENUE PUMPING STATION FOR A 1 IN 25 YEAR STORM, AND CALCULATIONS FOR STORAGE VOLUME

MAXIMUM PUMPING RATE FOR JANET AVENUE PUMPING STATION (m^3/s) **0.145**

Date	Time	DS Flow (m^3/s)	US Flow (m^3/s)	Rainfall (Rainfall intensity (mm/hr))	PUMPING RATE (ASSUMED CONSTANT) m^3/s	PUMPED VOLUME OVER THE 5 MINUTE TIMESTEP m^3	INFLOW VOLUME OVER THE 5 MINUTE TIMESTEP (m^3)	DEPARTURE (INFLOW-OUTFLOW) m^3/ 5 Minutes	CUMULATIVE EXCESS OUTFLOW (m^3)
1/1/2016	5:45:00	0.015805	0.015825	0	0.145	43.5	4.7415	-38.7585	
1/1/2016	5:50:00	0.01678	0.016803	0	0.145	43.5	5.034	-38.466	
1/1/2016	5:55:00	0.017911	0.017938	0	0.145	43.5	5.3733	-38.1267	
1/1/2016	6:00:00	0.019411	0.019455	5.829207	0.145	43.5	5.8233	-37.6767	
1/1/2016	6:05:00	0.028077	0.028448	2.502476	0.145	43.5	8.4231	-35.0769	
1/1/2016	6:10:00	0.037025	0.036985	2.4	0.145	43.5	11.1075	-32.3925	
1/1/2016	6:15:00	0.032644	0.032558	2.4	0.145	43.5	9.7932	-33.7068	
1/1/2016	6:20:00	0.029888	0.029866	2.4	0.145	43.5	8.9664	-34.5336	
1/1/2016	6:25:00	0.02999	0.030007	2.4	0.145	43.5	8.997	-34.503	
1/1/2016	6:30:00	0.031405	0.031442	3.56584	0.145	43.5	9.4215	-34.0785	
1/1/2016	6:35:00	0.033708	0.03376	3.600001	0.145	43.5	10.1124	-33.3876	
1/1/2016	6:40:00	0.038341	0.038504	2.434158	0.145	43.5	11.5023	-31.9977	
1/1/2016	6:45:00	0.057145	0.057473	3.56584	0.145	43.5	17.1435	-26.3565	
1/1/2016	6:50:00	0.059913	0.059831	3.600002	0.145	43.5	17.9739	-25.5261	
1/1/2016	6:55:00	0.054697	0.054648	3.599999	0.145	43.5	16.4091	-27.0909	
1/1/2016	7:00:00	0.053764	0.053777	5.931683	0.145	43.5	16.1292	-27.3708	
1/1/2016	7:05:00	0.057546	0.057686	3.668318	0.145	43.5	17.2638	-26.2362	
1/1/2016	7:10:00	0.078701	0.07896	3.599999	0.145	43.5	23.6103	-19.8897	
1/1/2016	7:15:00	0.083711	0.083661	3.599999	0.145	43.5	25.1133	-18.3867	
1/1/2016	7:20:00	0.076333	0.076262	3.599999	0.145	43.5	22.8999	-20.6001	
1/1/2016	7:25:00	0.073616	0.07364	5.931683	0.145	43.5	22.0848	-21.4152	
1/1/2016	7:30:00	0.086631	0.086768	2.502479	0.145	43.5	25.9893	-17.5107	
1/1/2016	7:35:00	0.100934	0.10092	5.897524	0.145	43.5	30.2802	-13.2198	
1/1/2016	7:40:00	0.093712	0.093552	3.668318	0.145	43.5	28.1136	-15.3864	
1/1/2016	7:45:00	0.087751	0.087754	2.434155	0.145	43.5	26.3253	-17.1747	
1/1/2016	7:50:00	0.102382	0.102609	2.400002	0.145	43.5	30.7146	-12.7854	
1/1/2016	7:55:00	0.114737	0.114732	2.400002	0.145	43.5	34.4211	-9.0789	
1/1/2016	8:00:00	0.107817	0.107749	10.560885	0.145	43.5	32.3451	-11.1549	
1/1/2016	8:05:00	0.098543	0.098525	9.634159	0.145	43.5	29.5629	-13.9371	
1/1/2016	8:10:00	0.108761	0.10891	17.760895	0.145	43.5	32.6283	-10.8717	
1/1/2016	8:15:00	0.123455	0.123464	56.472759	0.145	43.5	37.0365	-6.4635	
1/1/2016	8:20:00	0.120114	0.12008	58.765846	0.145	43.5	36.0342	-7.4658	
1/1/2016	8:25:00	0.115557	0.115568	75.121765	0.145	43.5	34.6671	-8.8329	
1/1/2016	8:30:00	0.134457	0.134499	86.092583	0.145	43.5	40.3371	-3.1629	
1/1/2016	8:35:00	0.162244	0.162251	68.912384	0.145	43.5	48.6732	5.1732	5.1732
1/1/2016	8:40:00	0.17309	0.173094	48.580708	0.145	43.5	51.927	8.427	13.6002
1/1/2016	8:45:00	0.182253	0.182256	42.170765	0.145	43.5	54.6759	11.1759	24.7761
1/1/2016	8:50:00	0.197177	0.197182	14.019817	0.145	43.5	59.1531	15.6531	40.4292
1/1/2016	8:55:00	0.217577	0.217582	7.3708	0.145	43.5	65.2731	21.7731	62.2023
1/1/2016	9:00:00	0.231932	0.231936	9.531677	0.145	43.5	69.5796	26.0796	88.2819
1/1/2016	9:05:00	0.240329	0.240331	9.599983	0.145	43.5	72.0987	28.5987	116.8806
1/1/2016	9:10:00	0.243231	0.243231	6.102473	0.145	43.5	72.9693	29.4693	146.3499
1/1/2016	9:15:00	0.239796	0.239794	2.502501	0.145	43.5	71.9388	28.4388	174.7887
1/1/2016	9:20:00	0.236582	0.236582	3.565818	0.145	43.5	70.9746	27.4746	202.2633
1/1/2016	9:25:00	0.238446	0.238447	2.434194	0.145	43.5	71.5338	28.0338	230.2971
1/1/2016	9:30:00	0.240574	0.240574	7.063329	0.145	43.5	72.1722	28.6722	258.9693
1/1/2016	9:35:00	0.241204	0.241204	23.521811	0.145	43.5	72.3612	28.8612	287.8305
1/1/2016	9:40:00	0.241073	0.241073	17.004936	0.145	43.5	72.3219	28.8219	316.6524
1/1/2016	9:45:00	0.241175	0.241175	20.297527	0.145	43.5	72.3525	28.8525	345.5049
1/1/2016	9:50:00	0.241999	0.241999	22.731676	0.145	43.5	72.5997	29.0997	374.6046
1/1/2016	9:55:00	0.241768	0.241767	7.644072	0.145	43.5	72.5304	29.0304	403.635
1/1/2016	10:00:00	0.237668	0.237667	0.204965	0.145	43.5	71.3004	27.8004	431.4354
1/1/2016	10:05:00	0.237147	0.237148	0	0.145	43.5	71.1441	27.6441	459.0795
1/1/2016	10:10:00	0.241782	0.241783	0	0.145	43.5	72.5346	29.0346	488.1141
1/1/2016	10:15:00	0.24557	0.245571	0	0.145	43.5	73.671	30.171	518.2851
1/1/2016	10:20:00	0.246727	0.246727	0	0.145	43.5	74.0181	30.5181	548.8032
1/1/2016	10:25:00	0.246032	0.246031	0	0.145	43.5	73.8096	30.3096	579.1128
1/1/2016	10:30:00	0.244153	0.244152	0	0.145	43.5	73.2459	29.7459	608.8587
1/1/2016	10:35:00	0.240824	0.240823	0	0.145	43.5	72.2472	28.7472	637.6059
1/1/2016	10:40:00	0.233123	0.233119	0	0.145	43.5	69.9369	26.4369	664.0428
1/1/2016	10:45:00	0.224503	0.224502	0	0.145	43.5	67.3509	23.8509	687.8937
1/1/2016	10:50:00	0.223622	0.223622	0	0.145	43.5	67.0866	23.5866	711.4803
1/1/2016	10:55:00	0.2251	0.2251	0	0.145	43.5	67.53	24.03	735.5103
1/1/2016	11:00:00	0.225619	0.225619	0	0.145	43.5	67.6857	24.1857	759.696
1/1/2016	11:05:00	0.224595	0.224594	0	0.145	43.5	67.3785	23.8785	783.5745
1/1/2016	11:10:00	0.218845	0.218842	0	0.145	43.5	65.6535	22.1535	805.728
1/1/2016	11:15:00	0.209593	0.20959	0	0.145	43.5	62.8779	19.3779	825.1059
1/1/2016	11:20:00	0.207718	0.207719	0	0.145	43.5	62.3154	18.8154	843.9213
1/1/2016	11:25:00	0.210372	0.210373	0	0.145	43.5	63.1116	19.6116	863.5329

INLET HYDROGRAPH AT THE JANET AVENUE PUMPING STATION FOR A 1 IN 25 YEAR STORM, AND CALCULATIONS FOR STORAGE VOLUME

MAXIMUM PUMPING RATE FOR JANET AVENUE PUMPING STATION (m^3/s) **0.145**

Date	Time	DS Flow (m^3/s)	US Flow (m^3/s)	Rainfall (Rainfall intensity (mm/hr))	PUMPING RATE (ASSUMED CONSTANT) m^3/s	PUMPED VOLUME OVER THE 5 MINUTE TIMESTEP m^3	INFLOW VOLUME OVER THE 5 MINUTE TIMESTEP (m^3)	DEPARTURE (INFLOW-OUTFLOW) m^3/ 5 Minutes	CUMULATIVE EXCESS OUTFLOW (m^3)
1/1/2016	11:30:00	0.210928	0.210927	0	0.145	43.5	63.2784	19.7784	883.3113
1/1/2016	11:35:00	0.205854	0.205852	0	0.145	43.5	61.7562	18.2562	901.5675
1/1/2016	11:40:00	0.196566	0.196563	0	0.145	43.5	58.9698	15.4698	917.0373
1/1/2016	11:45:00	0.192808	0.192808	0	0.145	43.5	57.8424	14.3424	931.3797
1/1/2016	11:50:00	0.196879	0.19688	0	0.145	43.5	59.0637	15.5637	946.9434
1/1/2016	11:55:00	0.198257	0.198256	0	0.145	43.5	59.4771	15.9771	962.9205
1/1/2016	12:00:00	0.194056	0.194054	0	0.145	43.5	58.2168	14.7168	977.6373
1/1/2016	12:05:00	0.183703	0.183698	0	0.145	43.5	55.1109	11.6109	989.2482
1/1/2016	12:10:00	0.17792	0.177921	0	0.145	43.5	53.376	9.876	999.1242
1/1/2016	12:15:00	0.181841	0.181843	0	0.145	43.5	54.5523	11.0523	1010.1765
1/1/2016	12:20:00	0.184256	0.184256	0	0.145	43.5	55.2768	11.7768	1021.9533
1/1/2016	12:25:00	0.178775	0.17877	0	0.145	43.5	53.6325	10.1325	1032.0858
1/1/2016	12:30:00	0.167148	0.167144	0	0.145	43.5	50.1444	6.6444	1038.7302
1/1/2016	12:35:00	0.167776	0.167779	0	0.145	43.5	50.3328	6.8328	1045.563
1/1/2016	12:40:00	0.17325	0.173252	0	0.145	43.5	51.975	8.475	1054.038
1/1/2016	12:45:00	0.172551	0.172548	0	0.145	43.5	51.7653	8.2653	1062.3033
1/1/2016	12:50:00	0.159583	0.159572	0	0.145	43.5	47.8749	4.3749	1066.6782
1/1/2016	12:55:00	0.15201	0.152014	0	0.145	43.5	45.603	2.103	1068.7812
1/1/2016	13:00:00	0.163218	0.16322	0	0.145	43.5	48.9654	5.4654	1074.2466
1/1/2016	13:05:00	0.165506	0.165505	0	0.145	43.5	49.6518	6.1518	1080.3984
1/1/2016	13:10:00	0.15381	0.1538	0	0.145	43.5	46.143	2.643	1083.0414
1/1/2016	13:15:00	0.141819	0.141815	0	0.145	43.5	42.5457	-0.9543	1082.0871
1/1/2016	13:20:00	0.149443	0.149453	0	0.145	43.5	44.8329	1.3329	1083.42
1/1/2016	13:25:00	0.157511	0.15751	0	0.145	43.5	47.2533	3.7533	1087.1733
1/1/2016	13:30:00	0.148852	0.148844	0	0.145	43.5	44.6556	1.1556	1088.3289
1/1/2016	13:35:00	0.136139	0.136132	0	0.145	43.5	40.8417	-2.6583	
1/1/2016	13:40:00	0.138605	0.138616	0	0.145	43.5	41.5815	-1.9185	
1/1/2016	13:45:00	0.149422	0.149423	0	0.145	43.5	44.8266	1.3266	1.3266
1/1/2016	13:50:00	0.143481	0.143472	0	0.145	43.5	43.0443	-0.4557	
1/1/2016	13:55:00	0.130468	0.130459	0	0.145	43.5	39.1404	-4.3596	
1/1/2016	14:00:00	0.129848	0.129861	0	0.145	43.5	38.9544	-4.5456	
1/1/2016	14:05:00	0.142273	0.142318	0	0.145	43.5	42.6819	-0.8181	
1/1/2016	14:10:00	0.140738	0.140731	0	0.145	43.5	42.2214	-1.2786	
1/1/2016	14:15:00	0.127211	0.127224	0	0.145	43.5	38.1633	-5.3367	
1/1/2016	14:20:00	0.121126	0.121143	0	0.145	43.5	36.3378	-7.1622	
1/1/2016	14:25:00	0.133053	0.133066	0	0.145	43.5	39.9159	-3.5841	
1/1/2016	14:30:00	0.136168	0.136162	0	0.145	43.5	40.8504	-2.6496	
1/1/2016	14:35:00	0.123121	0.123072	0	0.145	43.5	36.9363	-6.5637	
1/1/2016	14:40:00	0.11333	0.113306	0	0.145	43.5	33.999	-9.501	
1/1/2016	14:45:00	0.122718	0.122774	0	0.145	43.5	36.8154	-6.6846	
1/1/2016	14:50:00	0.128574	0.128566	0	0.145	43.5	38.5722	-4.9278	
1/1/2016	14:55:00	0.117161	0.117087	0	0.145	43.5	35.1483	-8.3517	
1/1/2016	15:00:00	0.1058	0.105745	0	0.145	43.5	31.74	-11.76	
1/1/2016	15:05:00	0.112292	0.112414	0	0.145	43.5	33.6876	-9.8124	
1/1/2016	15:10:00	0.122749	0.122755	0	0.145	43.5	36.8247	-6.6753	
1/1/2016	15:15:00	0.113787	0.11373	0	0.145	43.5	34.1361	-9.3639	
1/1/2016	15:20:00	0.100818	0.100727	0	0.145	43.5	30.2454	-13.2546	
1/1/2016	15:25:00	0.100742	0.100841	0	0.145	43.5	30.2226	-13.2774	
1/1/2016	15:30:00	0.115688	0.115764	0	0.145	43.5	34.7064	-8.7936	
1/1/2016	15:35:00	0.113636	0.11358	0	0.145	43.5	34.0908	-9.4092	
1/1/2016	15:40:00	0.10091	0.100793	0	0.145	43.5	30.273	-13.227	
1/1/2016	15:45:00	0.092942	0.092923	0	0.145	43.5	27.8826	-15.6174	
1/1/2016	15:50:00	0.100861	0.101297	0	0.145	43.5	30.2583	-13.2417	
1/1/2016	15:55:00	0.11214	0.112132	0	0.145	43.5	33.642	-9.858	
1/1/2016	16:00:00	0.103051	0.10294	0	0.145	43.5	30.9153	-12.5847	
1/1/2016	16:05:00	0.091778	0.091717	0	0.145	43.5	27.5334	-15.9666	
1/1/2016	16:10:00	0.095035	0.095161	0	0.145	43.5	28.5105	-14.9895	
1/1/2016	16:15:00	0.108807	0.108834	0	0.145	43.5	32.6421	-10.8579	
1/1/2016	16:20:00	0.103115	0.103017	0	0.145	43.5	30.9345	-12.5655	
1/1/2016	16:25:00	0.091232	0.091136	0	0.145	43.5	27.3696	-16.1304	
1/1/2016	16:30:00	0.087836	0.087859	0	0.145	43.5	26.3508	-17.1492	
1/1/2016	16:35:00	0.102183	0.102526	0	0.145	43.5	30.6549	-12.8451	
1/1/2016	16:40:00	0.106115	0.106062	0	0.145	43.5	31.8345	-11.6655	
1/1/2016	16:45:00	0.095706	0.095603	0	0.145	43.5	28.7118	-14.7882	
1/1/2016	16:50:00	0.087332	0.087312	0	0.145	43.5	26.1996	-17.3004	
1/1/2016	16:55:00	0.094723	0.094907	0	0.145	43.5	28.4169	-15.0831	
1/1/2016	17:00:00	0.107205	0.107212	0	0.145	43.5	32.1615	-11.3385	
1/1/2016	17:05:00	0.099057	0.098944	0	0.145	43.5	29.7171	-13.7829	
1/1/2016	17:10:00	0.088453	0.088378	0	0.145	43.5	26.5359	-16.9641	

INLET HYDROGRAPH AT THE JANET AVENUE PUMPING STATION FOR A 1 IN 25 YEAR STORM, AND CALCULATIONS FOR STORAGE VOLUME

MAXIMUM PUMPING RATE FOR JANET AVENUE PUMPING STATION (m^3/s) **0.145**

Date	Time	DS Flow (m^3/s)	US Flow (m^3/s)	Rainfall (Rainfall intensity (mm/hr))	PUMPING RATE (ASSUMED CONSTANT) m^3/s	PUMPED VOLUME OVER THE 5 MINUTE TIMESTEP m^3	INFLOW VOLUME OVER THE 5 MINUTE TIMESTEP (m^3)	DEPARTURE (INFLOW-OUTFLOW) m^3/ 5 Minutes	CUMULATIVE EXCESS OUTFLOW (m^3)
1/1/2016	17:15:00	0.088002	0.088071	0	0.145	43.5	26.4006	-17.0994	
1/1/2016	17:20:00	0.104861	0.104952	0	0.145	43.5	31.4583	-12.0417	
1/1/2016	17:25:00	0.103451	0.103368	0	0.145	43.5	31.0353	-12.4647	
1/1/2016	17:30:00	0.091518	0.091387	0	0.145	43.5	27.4554	-16.0446	
1/1/2016	17:35:00	0.08692	0.086936	0	0.145	43.5	26.076	-17.424	
1/1/2016	17:40:00	0.100178	0.100298	0	0.145	43.5	30.0534	-13.4466	
1/1/2016	17:45:00	0.104255	0.104195	0	0.145	43.5	31.2765	-12.2235	
1/1/2016	17:50:00	0.093246	0.093059	0	0.145	43.5	27.9738	-15.5262	
1/1/2016	17:55:00	0.086222	0.086209	0	0.145	43.5	25.8666	-17.6334	
1/1/2016	18:00:00	0.09681	0.096967	0	0.145	43.5	29.043	-14.457	
1/1/2016	18:05:00	0.106381	0.106356	0	0.145	43.5	31.9143	-11.5857	
1/1/2016	18:10:00	0.097409	0.097357	0	0.145	43.5	29.2227	-14.2773	
1/1/2016	18:15:00	0.086565	0.086518	0	0.145	43.5	25.9695	-17.5305	
1/1/2016	18:20:00	0.085637	0.085712	0	0.145	43.5	25.6911	-17.8089	
1/1/2016	18:25:00	0.103276	0.10329	0	0.145	43.5	30.9828	-12.5172	
1/1/2016	18:30:00	0.103219	0.10315	0	0.145	43.5	30.9657	-12.5343	
1/1/2016	18:35:00	0.091525	0.091374	0	0.145	43.5	27.4575	-16.0425	
1/1/2016	18:40:00	0.084479	0.084443	0	0.145	43.5	25.3437	-18.1563	
1/1/2016	18:45:00	0.091588	0.091845	0	0.145	43.5	27.4764	-16.0236	
1/1/2016	18:50:00	0.106153	0.106189	0	0.145	43.5	31.8459	-11.6541	
1/1/2016	18:55:00	0.098859	0.098751	0	0.145	43.5	29.6577	-13.8423	
1/1/2016	19:00:00	0.087635	0.087568	0	0.145	43.5	26.2905	-17.2095	
1/1/2016	19:05:00	0.086328	0.086396	0	0.145	43.5	25.8984	-17.6016	
1/1/2016	19:10:00	0.10397	0.104022	0	0.145	43.5	31.191	-12.309	
1/1/2016	19:15:00	0.103292	0.103213	0	0.145	43.5	30.9876	-12.5124	
1/1/2016	19:20:00	0.090962	0.090815	0	0.145	43.5	27.2886	-16.2114	
1/1/2016	19:25:00	0.084225	0.084201	0	0.145	43.5	25.2675	-18.2325	
1/1/2016	19:30:00	0.094849	0.095034	0	0.145	43.5	28.4547	-15.0453	
1/1/2016	19:35:00	0.103697	0.103653	0	0.145	43.5	31.1091	-12.3909	
1/1/2016	19:40:00	0.093836	0.093659	0	0.145	43.5	28.1508	-15.3492	
1/1/2016	19:45:00	0.082249	0.082143	0	0.145	43.5	24.6747	-18.8253	
1/1/2016	19:50:00	0.083262	0.083363	0	0.145	43.5	24.9786	-18.5214	
1/1/2016	19:55:00	0.097333	0.097313	0	0.145	43.5	29.1999	-14.3001	
1/1/2016	20:00:00	0.095042	0.09492	0	0.145	43.5	28.5126	-14.9874	
1/1/2016	20:05:00	0.08122	0.081042	0	0.145	43.5	24.366	-19.134	
1/1/2016	20:10:00	0.07404	0.074018	0	0.145	43.5	22.212	-21.288	
1/1/2016	20:15:00	0.083953	0.084112	0	0.145	43.5	25.1859	-18.3141	
1/1/2016	20:20:00	0.092421	0.092406	0	0.145	43.5	27.7263	-15.7737	
1/1/2016	20:25:00	0.082941	0.082785	0	0.145	43.5	24.8823	-18.6177	
1/1/2016	20:30:00	0.071839	0.071752	0	0.145	43.5	21.5517	-21.9483	
1/1/2016	20:35:00	0.072433	0.072576	0	0.145	43.5	21.7299	-21.7701	
1/1/2016	20:40:00	0.087058	0.087145	0	0.145	43.5	26.1174	-17.3826	
1/1/2016	20:45:00	0.085037	0.084912	0	0.145	43.5	25.5111	-17.9889	
1/1/2016	20:50:00	0.072052	0.071941	0	0.145	43.5	21.6156	-21.8844	
1/1/2016	20:55:00	0.065929	0.065904	0	0.145	43.5	19.7787	-23.7213	
1/1/2016	21:00:00	0.075876	0.076145	0	0.145	43.5	22.7628	-20.7372	
1/1/2016	21:05:00	0.084416	0.084388	0	0.145	43.5	25.3248	-18.1752	
1/1/2016	21:10:00	0.07374	0.073616	0	0.145	43.5	22.122	-21.378	
1/1/2016	21:15:00	0.065583	0.065506	0	0.145	43.5	19.6749	-23.8251	
1/1/2016	21:20:00	0.066117	0.066241	0	0.145	43.5	19.8351	-23.6649	
1/1/2016	21:25:00	0.081648	0.081736	0	0.145	43.5	24.4944	-19.0056	
1/1/2016	21:30:00	0.076566	0.076442	0	0.145	43.5	22.9698	-20.5302	
1/1/2016	21:35:00	0.066866	0.066755	0	0.145	43.5	20.0598	-23.4402	
1/1/2016	21:40:00	0.062439	0.062457	0	0.145	43.5	18.7317	-24.7683	
1/1/2016	21:45:00	0.074755	0.075004	0	0.145	43.5	22.4265	-21.0735	
1/1/2016	21:50:00	0.079834	0.079745	0	0.145	43.5	23.9502	-19.5498	
1/1/2016	21:55:00	0.070081	0.069958	0	0.145	43.5	21.0243	-22.4757	
1/1/2016	22:00:00	0.062366	0.062297	0	0.145	43.5	18.7098	-24.7902	
1/1/2016	22:05:00	0.064022	0.064159	0	0.145	43.5	19.2066	-24.2934	
1/1/2016	22:10:00	0.080624	0.080721	0	0.145	43.5	24.1872	-19.3128	
1/1/2016	22:15:00	0.075722	0.075598	0	0.145	43.5	22.7166	-20.7834	
1/1/2016	22:20:00	0.06582	0.065703	0	0.145	43.5	19.746	-23.754	
1/1/2016	22:25:00	0.061196	0.061212	0	0.145	43.5	18.3588	-25.1412	
1/1/2016	22:30:00	0.073071	0.073311	0	0.145	43.5	21.9213	-21.5787	
1/1/2016	22:35:00	0.077954	0.077863	0	0.145	43.5	23.3862	-20.1138	
1/1/2016	22:40:00	0.06799	0.067858	0	0.145	43.5	20.397	-23.103	
1/1/2016	22:45:00	0.059699	0.05962	0	0.145	43.5	17.9097	-25.5903	
1/1/2016	22:50:00	0.060194	0.060311	0	0.145	43.5	18.0582	-25.4418	
1/1/2016	22:55:00	0.075818	0.075943	0	0.145	43.5	22.7454	-20.7546	

INLET HYDROGRAPH AT THE JANET AVENUE PUMPING STATION FOR A 1 IN 25 YEAR STORM, AND CALCULATIONS FOR STORAGE VOLUME

MAXIMUM PUMPING RATE FOR JANET AVENUE PUMPING STATION (m^3/s) **0.145**

Date	Time	DS Flow (m^3/s)	US Flow (m^3/s)	Rainfall (Rainfall intensity (mm/hr))	PUMPING RATE (ASSUMED CONSTANT) m^3/s	PUMPED VOLUME OVER THE 5 MINUTE TIMESTEP m^3	INFLOW VOLUME OVER THE 5 MINUTE TIMESTEP (m^3)	DEPARTURE (INFLOW-OUTFLOW) m^3/ 5 Minutes	CUMULATIVE EXCESS OUTFLOW (m^3)
1/1/2016	23:00:00	0.071739	0.071615	0	0.145	43.5	21.5217	-21.9783	
1/1/2016	23:05:00	0.061503	0.06137	0	0.145	43.5	18.4509	-25.0491	
1/1/2016	23:10:00	0.055623	0.055595	0	0.145	43.5	16.6869	-26.8131	
1/1/2016	23:15:00	0.062853	0.063089	0	0.145	43.5	18.8559	-24.6441	
1/1/2016	23:20:00	0.074521	0.074482	0	0.145	43.5	22.3563	-21.1437	
1/1/2016	23:25:00	0.066091	0.065957	0	0.145	43.5	19.8273	-23.6727	
1/1/2016	23:30:00	0.056998	0.056902	0	0.145	43.5	17.0994	-26.4006	
1/1/2016	23:35:00	0.053737	0.053731	0	0.145	43.5	16.1211	-27.3789	
1/1/2016	23:40:00	0.061704	0.061954	0	0.145	43.5	18.5112	-24.9888	
1/1/2016	23:45:00	0.074561	0.074536	0	0.145	43.5	22.3683	-21.1317	
1/1/2016	23:50:00	0.066653	0.066522	0	0.145	43.5	19.9959	-23.5041	
1/1/2016	23:55:00	0.057824	0.057732	0	0.145	43.5	17.3472	-26.1528	
2/1/2016	0:00:00	0.054821	0.054821	0	0.145	43.5	16.4463	-27.0537	
2/1/2016	0:05:00	0.063525	0.063779	0	0.145	43.5	19.0575	-24.4425	
2/1/2016	0:10:00	0.075785	0.075759	0	0.145	43.5	22.7355	-20.7645	
2/1/2016	0:15:00	0.067578	0.067446	0	0.145	43.5	20.2734	-23.2266	
2/1/2016	0:20:00	0.058598	0.058503	0	0.145	43.5	17.5794	-25.9206	
2/1/2016	0:25:00	0.055847	0.05587	0	0.145	43.5	16.7541	-26.7459	
2/1/2016	0:30:00	0.067031	0.0673	0	0.145	43.5	20.1093	-23.3907	
2/1/2016	0:35:00	0.074132	0.074051	0	0.145	43.5	22.2396	-21.2604	
2/1/2016	0:40:00	0.064542	0.064401	0	0.145	43.5	19.3626	-24.1374	
2/1/2016	0:45:00	0.056095	0.056015	0	0.145	43.5	16.8285	-26.6715	
2/1/2016	0:50:00	0.056485	0.056607	0	0.145	43.5	16.9455	-26.5545	
2/1/2016	0:55:00	0.072221	0.07236	0	0.145	43.5	21.6663	-21.8337	
2/1/2016	1:00:00	0.068624	0.068496	0	0.145	43.5	20.5872	-22.9128	
2/1/2016	1:05:00	0.058163	0.058025	0	0.145	43.5	17.4489	-26.0511	
2/1/2016	1:10:00	0.051982	0.051927	0	0.145	43.5	15.5946	-27.9054	
2/1/2016	1:15:00	0.054999	0.055173	0	0.145	43.5	16.4997	-27.0003	
2/1/2016	1:20:00	0.069962	0.070022	0	0.145	43.5	20.9886	-22.5114	
2/1/2016	1:25:00	0.063717	0.063574	0	0.145	43.5	19.1151	-24.3849	
2/1/2016	1:30:00	0.053155	0.05303	0	0.145	43.5	15.9465	-27.5535	
2/1/2016	1:35:00	0.047673	0.04762	0	0.145	43.5	14.3019	-29.1981	
2/1/2016	1:40:00	0.047143	0.04722	0	0.145	43.5	14.1429	-29.3571	
2/1/2016	1:45:00	0.061687	0.061892	0	0.145	43.5	18.5061	-24.9939	
2/1/2016	1:50:00	0.06137	0.061234	0	0.145	43.5	18.411	-25.089	
2/1/2016	1:55:00	0.050067	0.049899	0	0.145	43.5	15.0201	-28.4799	
2/1/2016	2:00:00	0.043557	0.043485	0	0.145	43.5	13.0671	-30.4329	
2/1/2016	2:05:00	0.040167	0.040122	0	0.145	43.5	12.0501	-31.4499	
2/1/2016	2:10:00	0.04217	0.042355	0	0.145	43.5	12.651	-30.849	
2/1/2016	2:15:00	0.055144	0.05521	0	0.145	43.5	16.5432	-26.9568	
2/1/2016	2:20:00	0.048882	0.048721	0	0.145	43.5	14.6646	-28.8354	
2/1/2016	2:25:00	0.040482	0.04037	0	0.145	43.5	12.1446	-31.3554	
2/1/2016	2:30:00	0.035131	0.035047	0	0.145	43.5	10.5393	-32.9607	
2/1/2016	2:35:00	0.032164	0.032131	0	0.145	43.5	9.6492	-33.8508	
2/1/2016	2:40:00	0.0315	0.031519	0	0.145	43.5	9.45	-34.05	
2/1/2016	2:45:00	0.040643	0.040943	0	0.145	43.5	12.1929	-31.3071	
2/1/2016	2:50:00	0.045653	0.045554	0	0.145	43.5	13.6959	-29.8041	
2/1/2016	2:55:00	0.038011	0.037866	0	0.145	43.5	11.4033	-32.0967	
2/1/2016	3:00:00	0.031415	0.031304	0	0.145	43.5	9.4245	-34.0755	
2/1/2016	3:05:00	0.027465	0.027416	0	0.145	43.5	8.2395	-35.2605	
2/1/2016	3:10:00	0.025807	0.025782	0	0.145	43.5	7.7421	-35.7579	
2/1/2016	3:15:00	0.024786	0.024768	0	0.145	43.5	7.4358	-36.0642	
2/1/2016	3:20:00	0.024049	0.02404	0	0.145	43.5	7.2147	-36.2853	
2/1/2016	3:25:00	0.028844	0.02912	0	0.145	43.5	8.6532	-34.8468	
2/1/2016	3:30:00	0.039348	0.039324	0	0.145	43.5	11.8044	-31.6956	
2/1/2016	3:35:00	0.033163	0.033021	0	0.145	43.5	9.9489	-33.5511	
2/1/2016	3:40:00	0.026562	0.026454	0	0.145	43.5	7.9686	-35.5314	
2/1/2016	3:45:00	0.022787	0.022732	0	0.145	43.5	6.8361	-36.6639	
2/1/2016	3:50:00	0.020784	0.020748	0	0.145	43.5	6.2352	-37.2648	
2/1/2016	3:55:00	0.019391	0.019366	0	0.145	43.5	5.8173	-37.6827	
2/1/2016	4:00:00	0.018527	0.018512	0	0.145	43.5	5.5581	-37.9419	
2/1/2016	4:05:00	0.017913	0.0179	0	0.145	43.5	5.3739	-38.1261	
2/1/2016	4:10:00	0.01735	0.017338	0	0.145	43.5	5.205	-38.295	
2/1/2016	4:15:00	0.016825	0.016815	0	0.145	43.5	5.0475	-38.4525	
2/1/2016	4:20:00	0.016655	0.01667	0	0.145	43.5	4.9965	-38.5035	
2/1/2016	4:25:00	0.024448	0.024762	0	0.145	43.5	7.3344	-36.1656	
2/1/2016	4:30:00	0.029203	0.02912	0	0.145	43.5	8.7609	-34.7391	
2/1/2016	4:35:00	0.023656	0.023549	0	0.145	43.5	7.0968	-36.4032	
2/1/2016	4:40:00	0.019632	0.019567	0	0.145	43.5	5.8896	-37.6104	

INLET HYDROGRAPH AT THE JANET AVENUE PUMPING STATION FOR A 1 IN 25 YEAR STORM, AND CALCULATIONS FOR STORAGE VOLUME

MAXIMUM PUMPING RATE FOR JANET AVENUE PUMPING STATION (m^3/s) **0.145**

Date	Time	DS Flow (m^3/s)	US Flow (m^3/s)	Rainfall (Rainfall intensity (mm/hr))	PUMPING RATE (ASSUMED CONSTANT) m^3/s	PUMPED VOLUME OVER THE 5 MINUTE TIMESTEP m^3	INFLOW VOLUME OVER THE 5 MINUTE TIMESTEP (m^3)	DEPARTURE (INFLOW-OUTFLOW) m^3/ 5 Minutes	CUMULATIVE EXCESS OUTFLOW (m^3)
2/1/2016	4:45:00	0.017011	0.016956	0	0.145	43.5	5.1033	-38.3967	
2/1/2016	4:50:00	0.015102	0.015076	0	0.145	43.5	4.5306	-38.9694	
2/1/2016	4:55:00	0.014518	0.014511	0	0.145	43.5	4.3554	-39.1446	
2/1/2016	5:00:00	0.014322	0.014319	0	0.145	43.5	4.2966	-39.2034	
2/1/2016	5:05:00	0.014202	0.014199	0	0.145	43.5	4.2606	-39.2394	
2/1/2016	5:10:00	0.014107	0.014105	0	0.145	43.5	4.2321	-39.2679	
2/1/2016	5:15:00	0.014026	0.014024	0	0.145	43.5	4.2078	-39.2922	
2/1/2016	5:20:00	0.013957	0.013955	0	0.145	43.5	4.1871	-39.3129	
2/1/2016	5:25:00	0.013897	0.013895	0	0.145	43.5	4.1691	-39.3309	
2/1/2016	5:30:00	0.013842	0.01384	0	0.145	43.5	4.1526	-39.3474	
2/1/2016	5:35:00	0.013941	0.013954	0	0.145	43.5	4.1823	-39.3177	
2/1/2016	5:40:00	0.019871	0.020161	0	0.145	43.5	5.9613	-37.5387	
2/1/2016	5:45:00	0.027206	0.027159	0	0.145	43.5	8.1618	-35.3382	
2/1/2016	5:50:00	0.022519	0.022418	0	0.145	43.5	6.7557	-36.7443	
2/1/2016	5:55:00	0.018565	0.018497	0	0.145	43.5	5.5695	-37.9305	
2/1/2016	6:00:00	0.01575	0.015692	0	0.145	43.5	4.725	-38.775	
2/1/2016	6:05:00	0.014106	0.014087	0	0.145	43.5	4.2318	-39.2682	
2/1/2016	6:10:00	0.013678	0.013673	0	0.145	43.5	4.1034	-39.3966	
2/1/2016	6:15:00	0.013573	0.013572	0	0.145	43.5	4.0719	-39.4281	
2/1/2016	6:20:00	0.013556	0.013556	0	0.145	43.5	4.0668	-39.4332	
2/1/2016	6:25:00	0.013562	0.013562	0	0.145	43.5	4.0686	-39.4314	
2/1/2016	6:30:00	0.013573	0.013573	0	0.145	43.5	4.0719	-39.4281	
2/1/2016	6:35:00	0.013588	0.013588	0	0.145	43.5	4.0764	-39.4236	
2/1/2016	6:40:00	0.013607	0.013608	0	0.145	43.5	4.0821	-39.4179	
2/1/2016	6:45:00	0.013632	0.013633	0	0.145	43.5	4.0896	-39.4104	
2/1/2016	6:50:00	0.013662	0.013663	0	0.145	43.5	4.0986	-39.4014	
2/1/2016	6:55:00	0.013714	0.013717	0	0.145	43.5	4.1142	-39.3858	
2/1/2016	7:00:00	0.015903	0.016065	0	0.145	43.5	4.7709	-38.7291	
2/1/2016	7:05:00	0.027849	0.027963	0	0.145	43.5	8.3547	-35.1453	
2/1/2016	7:10:00	0.025152	0.02504	0	0.145	43.5	7.5456	-35.9544	
2/1/2016	7:15:00	0.020499	0.02042	0	0.145	43.5	6.1497	-37.3503	
2/1/2016	7:20:00	0.017722	0.017673	0	0.145	43.5	5.3166	-38.1834	
2/1/2016	7:25:00	0.015895	0.01587	0	0.145	43.5	4.7685	-38.7315	
2/1/2016	7:30:00	0.015642	0.015649	0	0.145	43.5	4.6926	-38.8074	
2/1/2016	7:35:00	0.016129	0.016143	0	0.145	43.5	4.8387	-38.6613	
2/1/2016	7:40:00	0.016841	0.016858	0	0.145	43.5	5.0523	-38.4477	
2/1/2016	7:45:00	0.017699	0.01772	0	0.145	43.5	5.3097	-38.1903	
2/1/2016	7:50:00	0.018695	0.018718	0	0.145	43.5	5.6085	-37.8915	
2/1/2016	7:55:00	0.019841	0.019867	0	0.145	43.5	5.9523	-37.5477	
2/1/2016	8:00:00	0.021146	0.021174	0	0.145	43.5	6.3438	-37.1562	
2/1/2016	8:05:00	0.022546	0.022575	0	0.145	43.5	6.7638	-36.7362	
2/1/2016	8:10:00	0.02409	0.02413	0	0.145	43.5	7.227	-36.273	
2/1/2016	8:15:00	0.033024	0.033389	0	0.145	43.5	9.9072	-33.5928	
2/1/2016	8:20:00	0.04297	0.042951	0	0.145	43.5	12.891	-30.609	
2/1/2016	8:25:00	0.03882	0.038737	0	0.145	43.5	11.646	-31.854	
2/1/2016	8:30:00	0.035299	0.035261	0	0.145	43.5	10.5897	-32.9103	
2/1/2016	8:35:00	0.035235	0.035259	0	0.145	43.5	10.5705	-32.9295	
2/1/2016	8:40:00	0.037316	0.037368	0	0.145	43.5	11.1948	-32.3052	
2/1/2016	8:45:00	0.042287	0.042447	0	0.145	43.5	12.6861	-30.8139	
2/1/2016	8:50:00	0.061616	0.061835	0	0.145	43.5	18.4848	-25.0152	
2/1/2016	8:55:00	0.062249	0.062153	0	0.145	43.5	18.6747	-24.8253	
2/1/2016	9:00:00	0.056153	0.056096	0	0.145	43.5	16.8459	-26.6541	
2/1/2016	9:05:00	0.054869	0.05488	0	0.145	43.5	16.4607	-27.0393	
2/1/2016	9:10:00	0.060622	0.060837	0	0.145	43.5	18.1866	-25.3134	
2/1/2016	9:15:00	0.081568	0.081712	0	0.145	43.5	24.4704	-19.0296	
2/1/2016	9:20:00	0.079237	0.079147	0	0.145	43.5	23.7711	-19.7289	
2/1/2016	9:25:00	0.07164	0.071566	0	0.145	43.5	21.492	-22.008	
2/1/2016	9:30:00	0.071819	0.071931	0	0.145	43.5	21.5457	-21.9543	
2/1/2016	9:35:00	0.088338	0.088514	0	0.145	43.5	26.5014	-16.9986	
2/1/2016	9:40:00	0.091861	0.091775	0	0.145	43.5	27.5583	-15.9417	
2/1/2016	9:45:00	0.082006	0.081882	0	0.145	43.5	24.6018	-18.8982	
2/1/2016	9:50:00	0.0802	0.080311	0	0.145	43.5	24.06	-19.44	
2/1/2016	9:55:00	0.097258	0.097339	0	0.145	43.5	29.1774	-14.3226	
2/1/2016	10:00:00	0.099085	0.099016	0	0.145	43.5	29.7255	-13.7745	
2/1/2016	10:05:00	0.088101	0.087989	0	0.145	43.5	26.4303	-17.0697	
2/1/2016	10:10:00	0.084249	0.084282	0	0.145	43.5	25.2747	-18.2253	
2/1/2016	10:15:00	0.101237	0.101577	0	0.145	43.5	30.3711	-13.1289	
2/1/2016	10:20:00	0.107357	0.107306	0	0.145	43.5	32.2071	-11.2929	
2/1/2016	10:25:00	0.096745	0.096689	0	0.145	43.5	29.0235	-14.4765	

INLET HYDROGRAPH AT THE JANET AVENUE PUMPING STATION FOR A 1 IN 25 YEAR STORM, AND CALCULATIONS FOR STORAGE VOLUME

MAXIMUM PUMPING RATE FOR JANET AVENUE PUMPING STATION (m^3/s) **0.145**

Date	Time	DS Flow (m^3/s)	US Flow (m^3/s)	Rainfall (Rainfall intensity (mm/hr))	PUMPING RATE (ASSUMED CONSTANT) m^3/s	PUMPED VOLUME OVER THE 5 MINUTE TIMESTEP m^3	INFLOW VOLUME OVER THE 5 MINUTE TIMESTEP (m^3)	DEPARTURE (INFLOW-OUTFLOW) m^3/ 5 Minutes	CUMULATIVE EXCESS OUTFLOW (m^3)
2/1/2016	10:30:00	0.088423	0.088462	0	0.145	43.5	26.5269	-16.9731	
2/1/2016	10:35:00	0.103479	0.103719	0	0.145	43.5	31.0437	-12.4563	
2/1/2016	10:40:00	0.106595	0.106536	0	0.145	43.5	31.9785	-11.5215	
2/1/2016	10:45:00	0.095452	0.095338	0	0.145	43.5	28.6356	-14.8644	
2/1/2016	10:50:00	0.086671	0.086672	0	0.145	43.5	26.0013	-17.4987	
2/1/2016	10:55:00	0.098535	0.09862	0	0.145	43.5	29.5605	-13.9395	
2/1/2016	11:00:00	0.103146	0.103091	0	0.145	43.5	30.9438	-12.5562	
2/1/2016	11:05:00	0.091642	0.091465	0	0.145	43.5	27.4926	-16.0074	
2/1/2016	11:10:00	0.081409	0.081312	0	0.145	43.5	24.4227	-19.0773	
2/1/2016	11:15:00	0.090961	0.091285	0	0.145	43.5	27.2883	-16.2117	
2/1/2016	11:20:00	0.101387	0.101385	0	0.145	43.5	30.4161	-13.0839	
2/1/2016	11:25:00	0.09184	0.091659	0	0.145	43.5	27.552	-15.948	
2/1/2016	11:30:00	0.079471	0.079337	0	0.145	43.5	23.8413	-19.6587	
2/1/2016	11:35:00	0.081794	0.081961	0	0.145	43.5	24.5382	-18.9618	
2/1/2016	11:40:00	0.097003	0.097077	0	0.145	43.5	29.1009	-14.3991	
2/1/2016	11:45:00	0.094984	0.094855	0	0.145	43.5	28.4952	-15.0048	
2/1/2016	11:50:00	0.082243	0.082083	0	0.145	43.5	24.6729	-18.8271	
2/1/2016	11:55:00	0.075179	0.075202	0	0.145	43.5	22.5537	-20.9463	
2/1/2016	12:00:00	0.087611	0.087764	0	0.145	43.5	26.2833	-17.2167	
2/1/2016	12:05:00	0.094358	0.094298	0	0.145	43.5	28.3074	-15.1926	
2/1/2016	12:10:00	0.083327	0.083168	0	0.145	43.5	24.9981	-18.5019	
2/1/2016	12:15:00	0.073617	0.073583	0	0.145	43.5	22.0851	-21.4149	
2/1/2016	12:20:00	0.083354	0.083534	0	0.145	43.5	25.0062	-18.4938	
2/1/2016	12:25:00	0.092846	0.092855	0	0.145	43.5	27.8538	-15.6462	
2/1/2016	12:30:00	0.084336	0.084187	0	0.145	43.5	25.3008	-18.1992	
2/1/2016	12:35:00	0.072384	0.072302	0	0.145	43.5	21.7152	-21.7848	
2/1/2016	12:40:00	0.073745	0.073909	0	0.145	43.5	22.1235	-21.3765	
2/1/2016	12:45:00	0.088082	0.088154	0	0.145	43.5	26.4246	-17.0754	
2/1/2016	12:50:00	0.087208	0.087142	0	0.145	43.5	26.1624	-17.3376	
2/1/2016	12:55:00	0.073623	0.073486	0	0.145	43.5	22.0869	-21.4131	
2/1/2016	13:00:00	0.06809	0.068098	0	0.145	43.5	20.427	-23.073	
2/1/2016	13:05:00	0.081679	0.081879	0	0.145	43.5	24.5037	-18.9963	
2/1/2016	13:10:00	0.086295	0.08625	0	0.145	43.5	25.8885	-17.6115	
2/1/2016	13:15:00	0.073731	0.073579	0	0.145	43.5	22.1193	-21.3807	
2/1/2016	13:20:00	0.066007	0.065965	0	0.145	43.5	19.8021	-23.6979	
2/1/2016	13:25:00	0.074814	0.075055	0	0.145	43.5	22.4442	-21.0558	
2/1/2016	13:30:00	0.084188	0.084178	0	0.145	43.5	25.2564	-18.2436	
2/1/2016	13:35:00	0.073587	0.073444	0	0.145	43.5	22.0761	-21.4239	
2/1/2016	13:40:00	0.064254	0.064168	0	0.145	43.5	19.2762	-24.2238	
2/1/2016	13:45:00	0.067118	0.067329	0	0.145	43.5	20.1354	-23.3646	
2/1/2016	13:50:00	0.081372	0.081393	0	0.145	43.5	24.4116	-19.0884	
2/1/2016	13:55:00	0.072969	0.072824	0	0.145	43.5	21.8907	-21.6093	
2/1/2016	14:00:00	0.062495	0.062372	0	0.145	43.5	18.7485	-24.7515	
2/1/2016	14:05:00	0.058989	0.059048	0	0.145	43.5	17.6967	-25.8033	
2/1/2016	14:10:00	0.074062	0.074244	0	0.145	43.5	22.2186	-21.2814	
2/1/2016	14:15:00	0.072103	0.071979	0	0.145	43.5	21.6309	-21.8691	
2/1/2016	14:20:00	0.061715	0.061572	0	0.145	43.5	18.5145	-24.9855	
2/1/2016	14:25:00	0.05495	0.054911	0	0.145	43.5	16.485	-27.015	
2/1/2016	14:30:00	0.062428	0.062688	0	0.145	43.5	18.7284	-24.7716	
2/1/2016	14:35:00	0.074994	0.074959	0	0.145	43.5	22.4982	-21.0018	
2/1/2016	14:40:00	0.066354	0.066219	0	0.145	43.5	19.9062	-23.5938	
2/1/2016	14:45:00	0.056349	0.056234	0	0.145	43.5	16.9047	-26.5953	
2/1/2016	14:50:00	0.051783	0.051764	0	0.145	43.5	15.5349	-27.9651	
2/1/2016	14:55:00	0.058731	0.058981	0	0.145	43.5	17.6193	-25.8807	
2/1/2016	15:00:00	0.071292	0.071268	0	0.145	43.5	21.3876	-22.1124	
2/1/2016	15:05:00	0.063113	0.062968	0	0.145	43.5	18.9339	-24.5661	
2/1/2016	15:10:00	0.053896	0.053801	0	0.145	43.5	16.1688	-27.3312	
2/1/2016	15:15:00	0.05183	0.051889	0	0.145	43.5	15.549	-27.951	
2/1/2016	15:20:00	0.065316	0.065553	0	0.145	43.5	19.5948	-23.9052	
2/1/2016	15:25:00	0.067983	0.067885	0	0.145	43.5	20.3949	-23.1051	
2/1/2016	15:30:00	0.058194	0.058048	0	0.145	43.5	17.4582	-26.0418	
2/1/2016	15:35:00	0.051213	0.05115	0	0.145	43.5	15.3639	-28.1361	
2/1/2016	15:40:00	0.051573	0.051683	0	0.145	43.5	15.4719	-28.0281	
2/1/2016	15:45:00	0.066513	0.0667	0	0.145	43.5	19.9539	-23.5461	
2/1/2016	15:50:00	0.065946	0.065832	0	0.145	43.5	19.7838	-23.7162	
2/1/2016	15:55:00	0.055967	0.055835	0	0.145	43.5	16.7901	-26.7099	
2/1/2016	16:00:00	0.050231	0.050183	0	0.145	43.5	15.0693	-28.4307	
2/1/2016	16:05:00	0.053115	0.05329	0	0.145	43.5	15.9345	-27.5655	
2/1/2016	16:10:00	0.068428	0.068521	0	0.145	43.5	20.5284	-22.9716	

INLET HYDROGRAPH AT THE JANET AVENUE PUMPING STATION FOR A 1 IN 25 YEAR STORM, AND CALCULATIONS FOR STORAGE VOLUME

MAXIMUM PUMPING RATE FOR JANET AVENUE PUMPING STATION (m^3/s) **0.145**

Date	Time	DS Flow (m^3/s)	US Flow (m^3/s)	Rainfall (Rainfall intensity (mm/hr))	PUMPING RATE (ASSUMED CONSTANT) m^3/s	PUMPED VOLUME OVER THE 5 MINUTE TIMESTEP m^3	INFLOW VOLUME OVER THE 5 MINUTE TIMESTEP (m^3)	DEPARTURE (INFLOW-OUTFLOW) m^3/ 5 Minutes	CUMULATIVE EXCESS OUTFLOW (m^3)
2/1/2016	16:15:00	0.063717	0.063581	0	0.145	43.5	19.1151	-24.3849	
2/1/2016	16:20:00	0.053689	0.053573	0	0.145	43.5	16.1067	-27.3933	
2/1/2016	16:25:00	0.048858	0.048822	0	0.145	43.5	14.6574	-28.8426	
2/1/2016	16:30:00	0.052268	0.052461	0	0.145	43.5	15.6804	-27.8196	
2/1/2016	16:35:00	0.067301	0.067389	0	0.145	43.5	20.1903	-23.3097	
2/1/2016	16:40:00	0.06224	0.0621	0	0.145	43.5	18.672	-24.828	
2/1/2016	16:45:00	0.051971	0.051844	0	0.145	43.5	15.5913	-27.9087	
2/1/2016	16:50:00	0.046972	0.046931	0	0.145	43.5	14.0916	-29.4084	
2/1/2016	16:55:00	0.051514	0.051739	0	0.145	43.5	15.4542	-28.0458	
2/1/2016	17:00:00	0.064644	0.064682	0	0.145	43.5	19.3932	-24.1068	
2/1/2016	17:05:00	0.057698	0.057529	0	0.145	43.5	17.3094	-26.1906	
2/1/2016	17:10:00	0.048203	0.048101	0	0.145	43.5	14.4609	-29.0391	
2/1/2016	17:15:00	0.04405	0.044009	0	0.145	43.5	13.215	-30.285	
2/1/2016	17:20:00	0.043797	0.043866	0	0.145	43.5	13.1391	-30.3609	
2/1/2016	17:25:00	0.058524	0.058762	0	0.145	43.5	17.5572	-25.9428	
2/1/2016	17:30:00	0.060325	0.060206	0	0.145	43.5	18.0975	-25.4025	
2/1/2016	17:35:00	0.05004	0.049919	0	0.145	43.5	15.012	-28.488	
2/1/2016	17:40:00	0.044341	0.044281	0	0.145	43.5	13.3023	-30.1977	
2/1/2016	17:45:00	0.041978	0.041954	0	0.145	43.5	12.5934	-30.9066	
2/1/2016	17:50:00	0.043873	0.044019	0	0.145	43.5	13.1619	-30.3381	
2/1/2016	17:55:00	0.060214	0.06039	0	0.145	43.5	18.0642	-25.4358	
2/1/2016	18:00:00	0.058455	0.058317	0	0.145	43.5	17.5365	-25.9635	
2/1/2016	18:05:00	0.048837	0.048728	0	0.145	43.5	14.6511	-28.8489	
2/1/2016	18:10:00	0.04413	0.044082	0	0.145	43.5	13.239	-30.261	
2/1/2016	18:15:00	0.04249	0.042486	0	0.145	43.5	12.747	-30.753	
2/1/2016	18:20:00	0.05027	0.050564	0	0.145	43.5	15.081	-28.419	
2/1/2016	18:25:00	0.062774	0.062781	0	0.145	43.5	18.8322	-24.6678	
2/1/2016	18:30:00	0.055312	0.05516	0	0.145	43.5	16.5936	-26.9064	
2/1/2016	18:35:00	0.047843	0.047773	0	0.145	43.5	14.3529	-29.1471	
2/1/2016	18:40:00	0.044956	0.04493	0	0.145	43.5	13.4868	-30.0132	
2/1/2016	18:45:00	0.045556	0.045636	0	0.145	43.5	13.6668	-29.8332	
2/1/2016	18:50:00	0.061088	0.061333	0	0.145	43.5	18.3264	-25.1736	
2/1/2016	18:55:00	0.063754	0.063647	0	0.145	43.5	19.1262	-24.3738	
2/1/2016	19:00:00	0.054203	0.05408	0	0.145	43.5	16.2609	-27.2391	
2/1/2016	19:05:00	0.048907	0.048863	0	0.145	43.5	14.6721	-28.8279	
2/1/2016	19:10:00	0.047891	0.04791	0	0.145	43.5	14.3673	-29.1327	
2/1/2016	19:15:00	0.05896	0.059259	0	0.145	43.5	17.688	-25.812	
2/1/2016	19:20:00	0.06898	0.068933	0	0.145	43.5	20.694	-22.806	
2/1/2016	19:25:00	0.06051	0.060366	0	0.145	43.5	18.153	-25.347	
2/1/2016	19:30:00	0.052798	0.052724	0	0.145	43.5	15.8394	-27.6606	
2/1/2016	19:35:00	0.051533	0.05159	0	0.145	43.5	15.4599	-28.0401	
2/1/2016	19:40:00	0.065495	0.065759	0	0.145	43.5	19.6485	-23.8515	
2/1/2016	19:45:00	0.069344	0.069253	0	0.145	43.5	20.8032	-22.6968	
2/1/2016	19:50:00	0.059898	0.059758	0	0.145	43.5	17.9694	-25.5306	
2/1/2016	19:55:00	0.053258	0.053203	0	0.145	43.5	15.9774	-27.5226	
2/1/2016	20:00:00	0.054709	0.054845	0	0.145	43.5	16.4127	-27.0873	
2/1/2016	20:05:00	0.071383	0.071538	0	0.145	43.5	21.4149	-22.0851	
2/1/2016	20:10:00	0.068981	0.068867	0	0.145	43.5	20.6943	-22.8057	
2/1/2016	20:15:00	0.059304	0.05918	0	0.145	43.5	17.7912	-25.7088	
2/1/2016	20:20:00	0.054273	0.054255	0	0.145	43.5	16.2819	-27.2181	
2/1/2016	20:25:00	0.062579	0.06285	0	0.145	43.5	18.7737	-24.7263	
2/1/2016	20:30:00	0.074874	0.074833	0	0.145	43.5	22.4622	-21.0378	
2/1/2016	20:35:00	0.066602	0.066472	0	0.145	43.5	19.9806	-23.5194	
2/1/2016	20:40:00	0.05816	0.05808	0	0.145	43.5	17.448	-26.052	
2/1/2016	20:45:00	0.059427	0.05958	0	0.145	43.5	17.8281	-25.6719	
2/1/2016	20:50:00	0.076713	0.076822	0	0.145	43.5	23.0139	-20.4861	
2/1/2016	20:55:00	0.072336	0.072221	0	0.145	43.5	21.7008	-21.7992	
2/1/2016	21:00:00	0.063009	0.062896	0	0.145	43.5	18.9027	-24.5973	
2/1/2016	21:05:00	0.059393	0.059428	0	0.145	43.5	17.8179	-25.6821	
2/1/2016	21:10:00	0.074228	0.074508	0	0.145	43.5	22.2684	-21.2316	
2/1/2016	21:15:00	0.080187	0.080102	0	0.145	43.5	24.0561	-19.4439	
2/1/2016	21:20:00	0.069684	0.069551	0	0.145	43.5	20.9052	-22.5948	
2/1/2016	21:25:00	0.061127	0.061046	0	0.145	43.5	18.3381	-25.1619	
2/1/2016	21:30:00	0.062692	0.062849	0	0.145	43.5	18.8076	-24.6924	
2/1/2016	21:35:00	0.080237	0.080326	0	0.145	43.5	24.0711	-19.4289	
2/1/2016	21:40:00	0.074711	0.074566	0	0.145	43.5	22.4133	-21.0867	
2/1/2016	21:45:00	0.06379	0.063655	0	0.145	43.5	19.137	-24.363	
2/1/2016	21:50:00	0.057258	0.057228	0	0.145	43.5	17.1774	-26.3226	
2/1/2016	21:55:00	0.066084	0.066356	0	0.145	43.5	19.8252	-23.6748	

INLET HYDROGRAPH AT THE JANET AVENUE PUMPING STATION FOR A 1 IN 25 YEAR STORM, AND CALCULATIONS FOR STORAGE VOLUME

MAXIMUM PUMPING RATE FOR JANET AVENUE PUMPING STATION (m^3/s) **0.145**

Date	Time	DS Flow (m^3/s)	US Flow (m^3/s)	Rainfall (Rainfall intensity (mm/hr))	PUMPING RATE (ASSUMED CONSTANT) m^3/s	PUMPED VOLUME OVER THE 5 MINUTE TIMESTEP m^3	INFLOW VOLUME OVER THE 5 MINUTE TIMESTEP (m^3)	DEPARTURE (INFLOW-OUTFLOW) m^3/ 5 Minutes	CUMULATIVE EXCESS OUTFLOW (m^3)
2/1/2016	22:00:00	0.076859	0.076811	0	0.145	43.5	23.0577	-20.4423	
2/1/2016	22:05:00	0.067217	0.067077	0	0.145	43.5	20.1651	-23.3349	
2/1/2016	22:10:00	0.057089	0.056969	0	0.145	43.5	17.1267	-26.3733	
2/1/2016	22:15:00	0.052789	0.052788	0	0.145	43.5	15.8367	-27.6633	
2/1/2016	22:20:00	0.062647	0.062896	0	0.145	43.5	18.7941	-24.7059	
2/1/2016	22:25:00	0.069843	0.06977	0	0.145	43.5	20.9529	-22.5471	
2/1/2016	22:30:00	0.060558	0.060406	0	0.145	43.5	18.1674	-25.3326	
2/1/2016	22:35:00	0.052139	0.052052	0	0.145	43.5	15.6417	-27.8583	
2/1/2016	22:40:00	0.051562	0.051662	0	0.145	43.5	15.4686	-28.0314	
2/1/2016	22:45:00	0.066014	0.0662	0	0.145	43.5	19.8042	-23.6958	
2/1/2016	22:50:00	0.065195	0.065078	0	0.145	43.5	19.5585	-23.9415	
2/1/2016	22:55:00	0.054964	0.054828	0	0.145	43.5	16.4892	-27.0108	
2/1/2016	23:00:00	0.049038	0.048988	0	0.145	43.5	14.7114	-28.7886	
2/1/2016	23:05:00	0.051791	0.051975	0	0.145	43.5	15.5373	-27.9627	
2/1/2016	23:10:00	0.067153	0.067242	0	0.145	43.5	20.1459	-23.3541	
2/1/2016	23:15:00	0.062204	0.062064	0	0.145	43.5	18.6612	-24.8388	
2/1/2016	23:20:00	0.052242	0.052125	0	0.145	43.5	15.6726	-27.8274	
2/1/2016	23:25:00	0.047581	0.047542	0	0.145	43.5	14.2743	-29.2257	
2/1/2016	23:30:00	0.050254	0.050434	0	0.145	43.5	15.0762	-28.4238	
2/1/2016	23:35:00	0.066182	0.066299	0	0.145	43.5	19.8546	-23.6454	
2/1/2016	23:40:00	0.062112	0.061976	0	0.145	43.5	18.6336	-24.8664	
2/1/2016	23:45:00	0.052378	0.052265	0	0.145	43.5	15.7134	-27.7866	
2/1/2016	23:50:00	0.047822	0.047783	0	0.145	43.5	14.3466	-29.1534	
2/1/2016	23:55:00	0.048727	0.048839	0	0.145	43.5	14.6181	-28.8819	
3/1/2016	0:00:00	0.064851	0.06506	0	0.145	43.5	19.4553	-24.0447	
TOTAL STORAGE VOLUME REQUIRED (m^3)									**1088**
ADD 20% CONTINGENCY (m^3)									**218**
TOTAL OPERATIONAL VOLUME TO BE PROVIDED AT THE JANET AVENUE PUMPING STATION (m^3)									**1306**