[GEM, 2017:A1D1]

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Wairoa District Council Sewage Reticulation

Investigation of Options to Enhance Sewage Reticulation Capacity
September 2017

Wairoa District Council

Revision A

Client:	Wairoa District Council
Report Title:	Investigation of Options to Enhance Sewage Reticulation Capacity
File Reference:	10502\Investigation to Enhance Sewage Reticulation Capacity
Report Issue:	Revision A — Date: September 2017

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On behalf of: Good Earth Matters Consulting Limited

EXECUTIVE SUMMARY

Wairoa District Council (Council) have reported frequent discharges / overflows from sewage pumpstations to the Wairoa River during times of high flow; and Council seeks to reduce the overflows. Additionally, properties in two areas identified by Council, are reliant on septic tank systems. Council seeks to assess the available option(s) and the associated ballpark costs to extend their exiting sewage reticulation to service the existing developed properties in the two areas identified.

Therefore, Council appointed Good Earth Matters to undertake two assessments:

- an assessment of option(s) to enhance the capacity between Kopu, Fitzroy and the Pilot Hill Treatment Ponds, thereby reducing the frequency and associated quantity of overflows to the Wairoa River; and
- to assess option(s) to extend the existing sewage reticulation to service the existing developed properties within the two areas identified.

System Enhancement Options

The assessment used data derived from Council's SCADA system to develop a representative hydraulic model. The SCADA data record used for this assessment was between January 2016 and March 2017. Multiple overflows from pumpstations were recorded by Council during this timeframe, in addition to multiple rainfall events. The highest recorded rainfall event occurring during this timeframe was an event equivalent to a 1 in 20 year event based on a 12 hour rainfall duration. The hydraulic model was calibrated by comparing pumped volumes from the hydraulic model with the pumped volumes recorded in the SCADA data; and by comparing the spillages derived from first principles with the spillages predicted in the hydraulic model, for the selected assessment period.

The calibrated hydraulic model indicated that there is a strong correlation between large rainfall events and overflow discharges; and there is a less well defined and understood relationship between sustained low intensity events occurring in rapid succession, which result in overflows due to an apparent combination of infiltration due to high ground water conditions and direct inflow due to saturated ground conditions. With age and asset deterioration, Inflow and Infiltration has become a significant and catchment wide problem.

To understand the reticulation and system constraints, whilst examining measures to mitigate or eliminate the number of overflows, the following was assessed:

- Impact that North Clyde and Alexander Park pumpstations have on one another resulting from a shared rising main;
- Remaining capacity of the gravity trunk pipe to accommodate flows from Kopu pumpstation after servicing flows from North Clyde, Alexander Park and Fitzroy catchments;
- Kopu pumpstation's capacity and associated overflows based on existing system configuration; and
- Capacity of Fitzroy pumpstation's existing rising main.

Modifications / adjustments were made to the hydraulic model to identify potential areas of optimisation and enhancement, that may reduce overflows from pumpstations to the Wairoa River. A summary of the mitigation measures identified by incorporating adjustments to the calibrated hydraulic model are as follows:

a. Provide separate rising main for North Clyde and Alexander Park pumpstations. The separated rising main enhances each pumpstations' performance by eliminating the reduction of pumping capacity when both pumpstations are operational;

b. Replace Alexander Park pumpstation's pumps. The pump replacement will increase pumping capacity by approximately 50 percent which reduced the overflow quantity from the pumpstation by an estimated 27 percent (4,800 m^3) during the assessment period;

c. Upgrade pumps at Kopu pumpstation and install new rising main from Kopu pumpstation to Pilot Hill Treatment Ponds. According to the results from the hydraulic model, the upgrade / installation resulted in a reduced overflow (from Kopu pumpstation wetwell and associated rising main discharge chamber) of approximately 33 percent (78,000 m^3) during the assessment period; and

d. Duplicate Fitzroy pumpstations rising main. The duplication of Fitzroy pumpstation's rising main resulted in an increased pumping capacity of approximately 40 percent. Based on the results from the hydraulic model, the increased pumping capacity in conjunction with the previously mentioned mitigatory measures resulted in an elimination of overflows from Fitzroy pumpstation for the selected assessment period.

A high-level cost estimate associated to the proposed mitigation measures recommended above are summarised in the Table below.

Description	Total excl. GST	
	Low	High
Separate rising main for North Clyde and Alexander Park pumpstations	$174,250	$225,500
Replacement of pumps at Alexander Park pumpstation	$15,860	$19,450
Replace pumps at Kopu pumpstation and installation of new rising main to Pilot Hill Treatment Ponds	$1,344,760	$1,662,780
Installation of Duplicate rising main between Fitzroy and Pilot Hill Treatment Ponds	$291,000	$360,000
Total Indicative Cost Estimate	**$1,825,870**	**$2,267,730**

In addition to the proposed mitigatory measures, the following recommendations and conclusions can be drawn from this assessment:

- Investigate the pipe material of the existing rising main at Fitzroy pumpstation. Based on the available asset data available, the rising main is an asbestos cement pipe. Aging asbestos cement rising mains typically increase system frictional losses. Additionally, subtle changes in pressure may result in pipe failure. If the pipe is of asbestos cement, it is recommended to replace the pipe when installing the proposed duplicate rising main; and
- Based on outputs received from the calibrated hydraulic model, the mitigatory measures proposed, reduced the overall quantity of overflows from pumpstations by 33 percent during the assessment period. Additional reductions may be possible through the implementation of an Inflow and Infiltration management programme.

System Extension Option(s)
The two areas requiring reticulation extensions are located in the North Clyde catchment and along Kopu Road, on the boundary of the Fitzroy and Kopu catchments. Gravity and low pressure sewage conveyance systems were considered for each of the extension areas.

By abiding to Council's Engineering Code of Practice, it was found that the North Clyde extension area could not freely gravitate and discharge into the existing sewage reticulation, and therefore, only a low pressure sewage option was assessed for this area.

The Kopu Road extension area was able to freely gravitate using flatter slopes than recommended, however, the gravity conveyance system may require additional maintenance and scheduled system flushing to mitigate potential blockages. A high-level conceptual design was completed for each of the extension areas and the total indicative costs associated to each options evaluated is shown in the Table below:

Extension Option Description	Total excl. GST	
	Low	High
North Clyde Extension Area - Low Pressure Sewage System	$1,044,000	$1,227,500
Kopu Road Extension Area - Gravity Sewage System	$424,800	$508,500
Kopu Road Extension Area - Low Pressure Sewage System	$430,400	$510,100

The key recommendations and findings from reticulation extension assessment are as follows:

- The developed properties in the North Clyde extension area are sparse and widely scattered resulting in high costs associated with the required piping lengths;
- A low pressure sewage system is recommended for the Kopu Road extension area due to the relatively small price difference in comparison to the gravity system. It is anticipated that a low pressure system will require less maintenance and the system can easily be extended to accommodate additional flows arising from future development; and
- The anticipated average dry weather flows from the extension areas will be less than 1 ℓ/s. The existing reticulated sewage system will have adequate capacity to accommodate the additional flows from the proposed extension areas.

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Appendices

1 INTRODUCTION

Wairoa District Council (Council) have reported frequent discharges / overflows from sewage pumpstations to the Wairoa River during times of high flow. According to previous studies[1], the high flow events are typically a result of inflow and infiltration during times of high intensity and/or consecutive lower intensity rainfall events. Council are progressively implementing measures to reduce inflow and infiltration over the next three to five years. In combination with this work Council is working to understand the capacity of the reticulated sewage spine.

The spine of the reticulated sewage system is composed of four pumpstations, namely, North Clyde, Alexander Park, Kopu and Fitzroy; all interconnected by their associated rising mains and a gravity trunk. Each of these pumpstations are fitted with well-designed overflow outfalls that discharge into the Wairoa River. Discharges through these outfalls occur when there is insufficient pumping and/or system capacity to accommodate peak flow events.

Council seeks to reduce overflows from the pumpstations to the Wairoa River. Good Earth Matters has been engaged to undertake an assessment of option(s) to enhance the capacity between Kopu, Fitzroy and the Pilot Hill Treatment Ponds, thereby reducing the frequency and associated quantity of overflows to the Wairoa River. Due to the complexity and interconnected nature of the system, the scope was increased to include North Clyde and Alexander Park pumpstations in the assessment.

Additionally, a number of existing developed properties are reliant on septic tanks. The properties are located in two areas (referred to as the North Clyde and Kopu Road extension areas) that are outside the perimeter of the existing sewage reticulation. Council have received numerous property owner complaints with regards to how the old septic tanks operate. Therefore, Council engaged Good Earth Matters to identify and assess option(s) to extend the existing sewage reticulation to service the existing developed properties within the two areas identified.

Figure 1.1
Study Locality
(LINZ Data Service (a), 2011)

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[1] *Opus International Consultants, 2012*

A precursor to this study was a Rainfall Derived Inflow and Infiltration (RDII) study, which identified areas of improvement to reduce inflow peaks during times of high rainfall events. In addition to this, the study provided recommendations to increase the reticulated system's hydraulic performance which was founded on diurnal flow patterns and pump flow capacities based on pump curves.

During the RDII study in 2012, electromagnetic flowmeters were installed at the respective pumpstations. Amongst other monitoring (i.e. levels in wetwells, pump on/off switches, etc.), flow measurements from these flow meters were recorded by a Supervisory Control and Data Acquisition (SCADA) system.

Good Earth Matters' study, as detailed in this report, uses the SCADA data to develop a hydraulic model. The benefit of using historic data to conduct hydraulic modelling is the ease of calibration and through collaboration with Council, the ability to relate the outcomes derived from the model with the on-site system performance.

This report documents the study's outcomes, and the means upon which these outcomes were achieved. The report is divided into five main sections as follows:

The first section provides an **Introduction** of the study followed by the study approach. The **Existing Sewage Reticulation** section describes the system's key elements. The on-site performance of the system is compared with the compiled baseline hydraulic model to assess the need for calibration adjustments.

The **Study Approach** section summarises the methodology upon which this study was carried out. This includes the gathering of supporting data, processing thereof into meaningful information and finally, the use of this information to build a hydraulic model based on historic flow data.

Upon completion of the baseline hydraulic model, mitigatory scenario changes were made in an iterative manner. The results achieved from these changes guided the selection of appropriate, fit for purpose options to enhance the efficiency of the sewage reticulation. These options are discussed in the **System Improvement & Extension Options** section.

The results achieved through the system improvement options are discussed in the **Findings and Recommendations** section of this report. Indicative costs and the risks associated to each of these options are assessed together with a recommended staging for implementation purposes.

2 EXISTING SEWAGE RETICULATION OVERVIEW

Wairoa's sewage reticulation is made up of four predominant catchment areas, each of which discharge into their respective pumpstations. Through a combination of rising mains and gravity networks, three of the pumpstations redirect flows through a gravity trunk route towards Fitzroy pumpstation. Fitzroy pumpstation has its own catchment area, which is composed of a few properties. Fitzroy pumpstation pumps sewage up to the Pilot Hill Treatment Ponds.

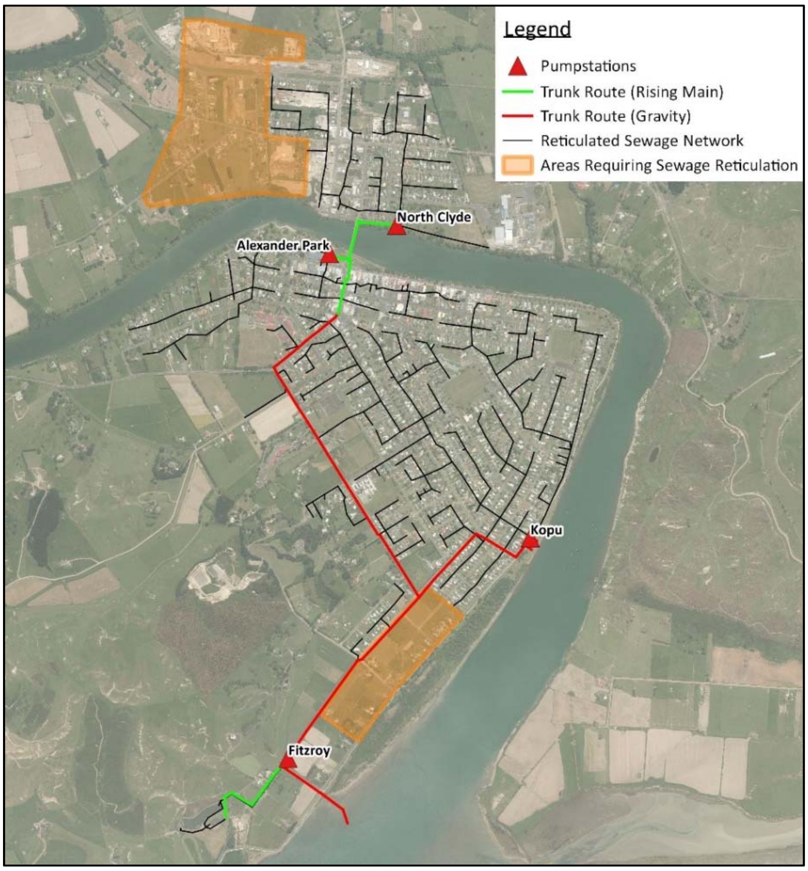
2.1 Sewage Reticulation Layout & Trunk Route

Based on the asset data received, Wairoa's reticulated sewer is made up of approximately 35.6 kilometres of interconnected pipes (diameters ranging between 100 mm to 450 mm) and 480 manholes. According to the asset data, the majority of the network was installed between 1948 - 1949. Subsequent to the original installation, some replacements and extensions were carried out, of which the most significant was an extension of 7 kilometres during 1980 - 1981.

According to the asset data, the reticulated network is made up of various pipe materials of which earthenware (38%), sulphide resistant cement concrete (31%) and asbestos cement (27%) are the most prevailing. The rising mains from both Kopu and Fitzroy pumpstations are of asbestos cement whereas Alexander Park and North Clyde pumpstations have steel rising mains.

Figure 2.1 shows the sewage reticulation layout (sourced from GIS data received from Council), trunk route being assessed in this study and the areas where the reticulation extensions are required.

Figure 2.1
Sewage Reticulation Layout, Trunk Route & Areas Requiring Reticulation
(LINZ Data Service (b), 2010-2011)



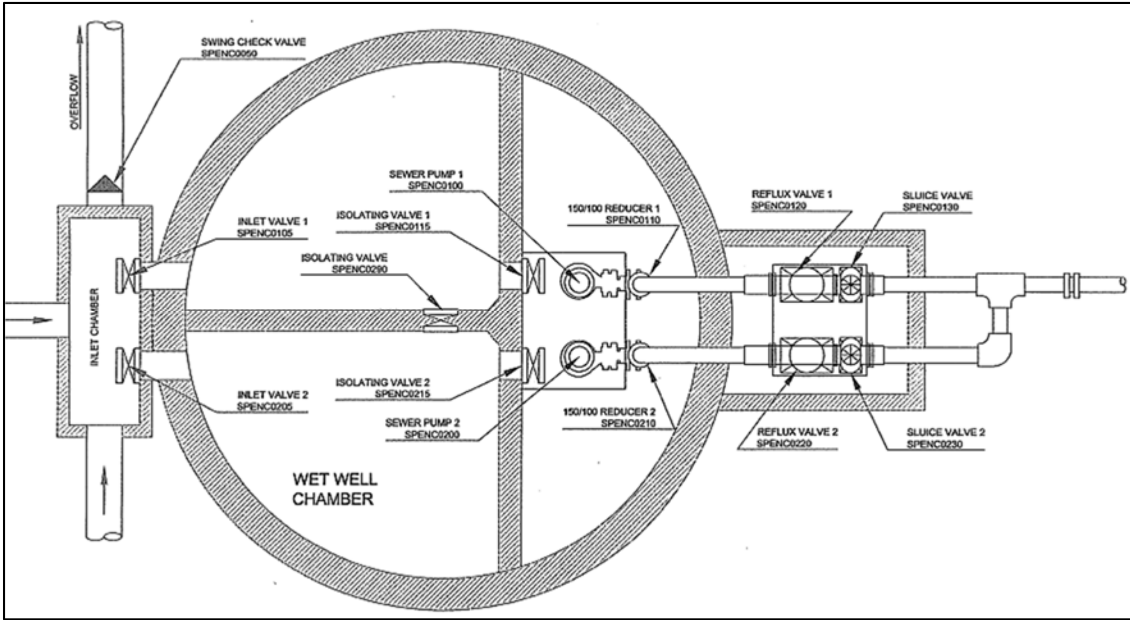
2.2 North Clyde & Alexander Park Pumpstations

North Clyde and Alexander Park pumpstations are situated in the northern portion of Wairoa. The pumpstations are positioned adjacent to one another, with North Clyde on the northern bank and Alexander Park pumpstation on the southern bank of the Wairoa River.

North Clyde pumpstation's rising main is fixed to the bottom of the Wairoa road (SH2) bridge. On the southern side of the bridge the rising main joins with the rising main of Alexander Park pumpstation. The 225 mm diameter shared rising main, is located along Paul Street and discharges into a manhole on the corner of Lucknow and Achilles Streets. The flows from these pumpstations are then gravitated through the trunk network to Fitzroy pumpstation.

North Clyde and Alexander Park pumpstations' civil structures are identical with the exception of their relative elevation and overflow levels. A typical layout of the pumpstations are shown in Figure 2.2.

Figure 2.2
North Clyde & Alexander Park Pumpstation Layout
(sourced from information received by Wairoa District Council)



The pumpstation chambers have an outer diameter of approximately 5.8 m. The maximum level of influent in the wet well is governed by the overflow pipe level, installed in the inlet chamber.

North Clyde pumpstation is fitted with two Flygt 3127 submersible pumps and pump stop / start control is managed by floats, whereas Alexander Park is fitted with two Flygt 3102 submersible pumps and controlled by a pressure level transducer.

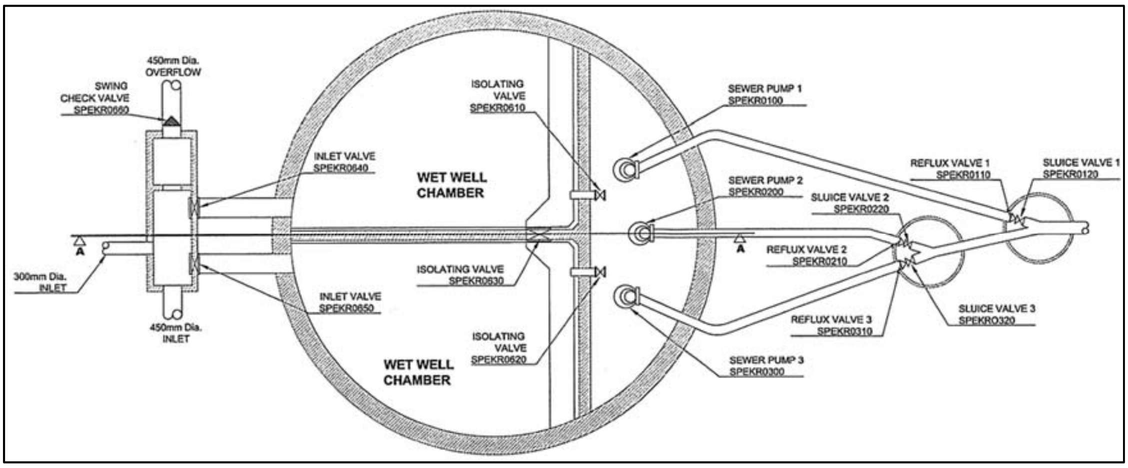
2.3 Kopu Pumpstation

Kopu pumpstation is located in the south-east of the Wairoa catchment. The pumpstation functions as a lift station, with a rising main length of approximately 50 metres, and effective lift of approximately 3.6 m (difference between wet well invert and downstream manhole invert).

The wet well chamber has an outer diameter of approximately 10.8 m. Kopu pumpstation's maximum wet well depth is controlled by the level of the overflow discharge pipe in the inlet chamber.

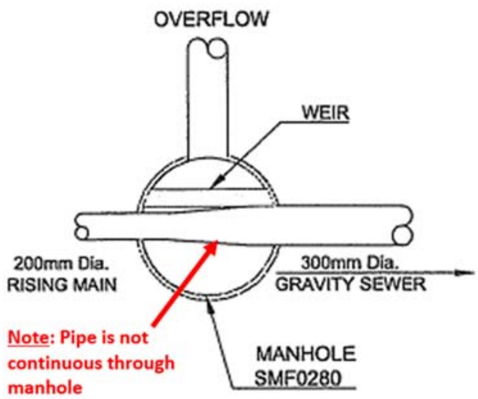
Kopu pumpstation is fitted with two Flygt 3102 submersible pumps and one Flygt 3152 submersible pump. The pumps' stop/start controls are currently managed by floats, however, Council are in the process of changing this to be controlled by the already installed pressure level transducer (which is more reliable).

Figure 2.3
Kopu Pumpstation Layout
(sourced from information received by Wairoa District Council)



The rising main discharges into a manhole (ID No. SMF0280) fitted with a flow control weir. The weir allows a maximum of approximately 52 ℓ/s to flow to the downstream trunk route. The calculation of which is based on unpressurised conditions and assuming that the downstream gravity system has capacity. Any pumped flows above 52 ℓ/s discharges through the manhole's overflow into the Wairoa River.

Figure 2.4
Kopu Rising Main Discharge Chamber
(sourced from information received by Wairoa District Council)

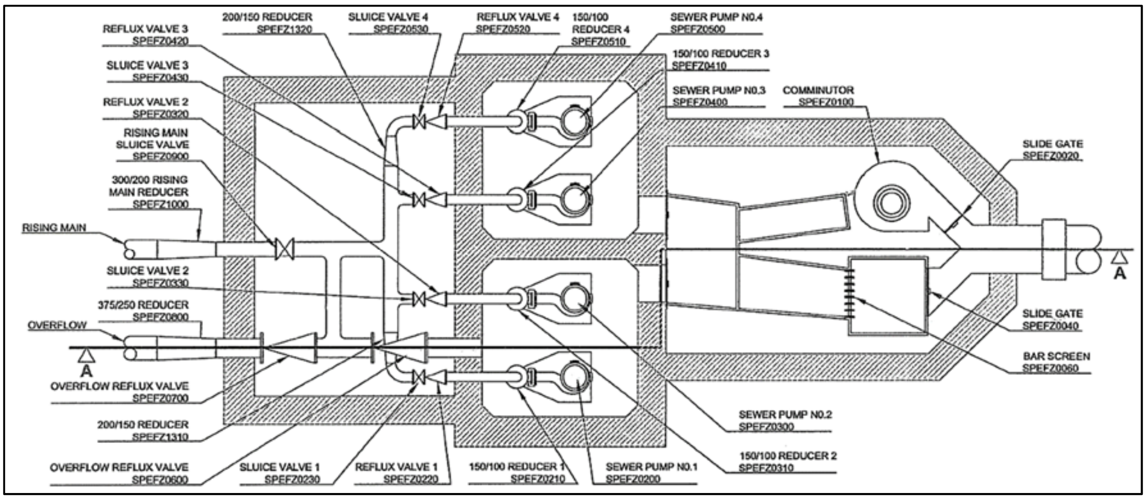


2.4 Fitzroy Pumpstation

The pumpstation is located on the southern boundary of the Wairoa catchment, approximately 280 m from the Wairoa River. Fitzroy pumpstation is furthest downstream in the Wairoa catchment. Based on the current reticulation's configuration, Fitzroy pumpstation is required to pump the entire catchment's sewage flow (dry weather flow and RDII) to the Pilot Hills Treatment Ponds.

Fitzroy pumpstation's maximum wet well level is controlled by an emergency overflow pipe installed inside the wet well. In the event that incoming flow exceeds the pumpstation's pump capacity, flow is discharged through the overflow pipe into the effluent gravity outfall used by the Pilot Hill Treatment Ponds.

Figure 2.5
Fitzroy Pumpstation Layout
(sourced from information received by Wairoa District Council)



Based on site observations, the comminutor / grinder and screen has been removed from the pumpstation. The pumpstation has been fitted with grinder type pumps and screening is done downstream at Pilot Hill Treatment Ponds.

The pumpstation is fitted with four Flygt 3152 submersible pumps. The stop / start control is managed by a pressure level transducer. The pumps are operated through variable frequency drives to reduce wear and tear by ramping on and ramping off over a time increment.

3 STUDY APPROACH

This study focuses on building a hydraulic model underpinned by the SCADA. The benefit of using historic data to conduct hydraulic modelling is the ease of calibration, and through collaboration with Council, the ability to relate the outcomes derived from the model with the on-site system performance.

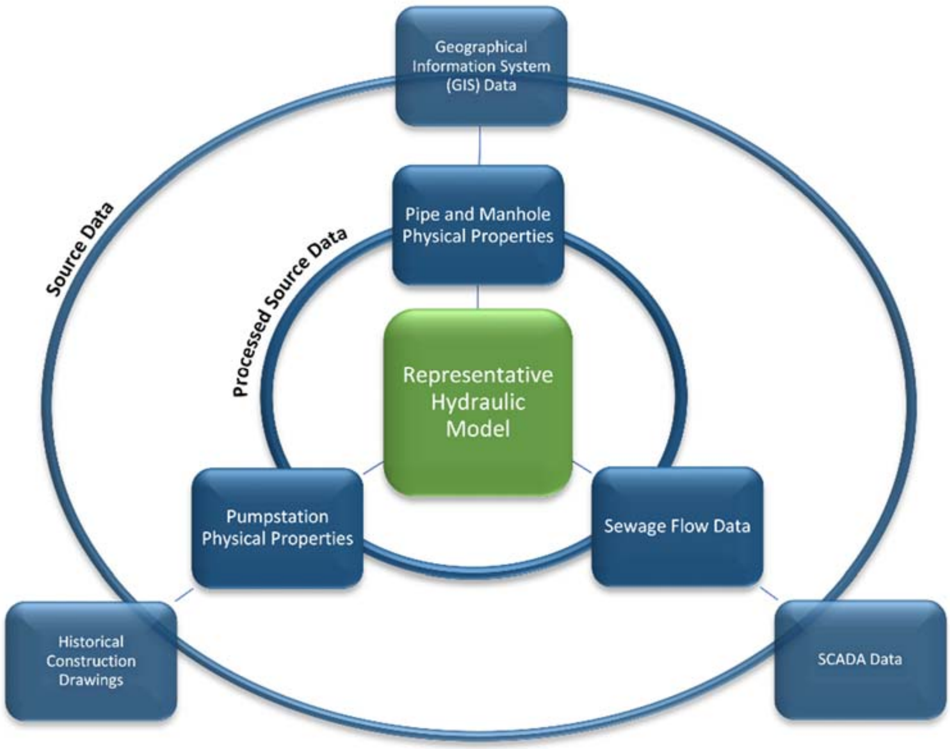
This section summarises the methodology upon which this study was carried out. This includes the gathering of supporting data, processing thereof into meaningful information and finally, the use of this information to build an intuitive representative baseline hydraulic model.

3.1 Supporting Information

The first step associated with building the hydraulic model was to gather relevant system data. Once the data was collected, it was processed and assessed. The assessed data was then compared with on-site measurements and observations.

A model founded on sound reliable data generally requires less calibration and adjustment, which reduces the room for error which may arise during manual modification of the model's input data. The data that was gathered and processed is shown in Figure 3.1.

Figure 3.1
Required Supporting Information



3.1.1 Reticulated Pipe and Manhole Properties

Council provided Good Earth Matters with an electronic copy of the available GIS data representing the sewage reticulation. The GIS data includes a set of lines representing the pipes and points representing the manholes in the Wairoa network. Each line and point has a list of attributes defining some of the asset's properties. Some of these properties include information about the geographical location, installation date, unique asset identification number, material, length, manhole lid elevation and manhole depth, just to mention a few.

The physical properties assessed and of significance to the hydraulic model included the following:

- Diameter;
- Pipe length;
- Pipe invert elevations;
- Pipe material;

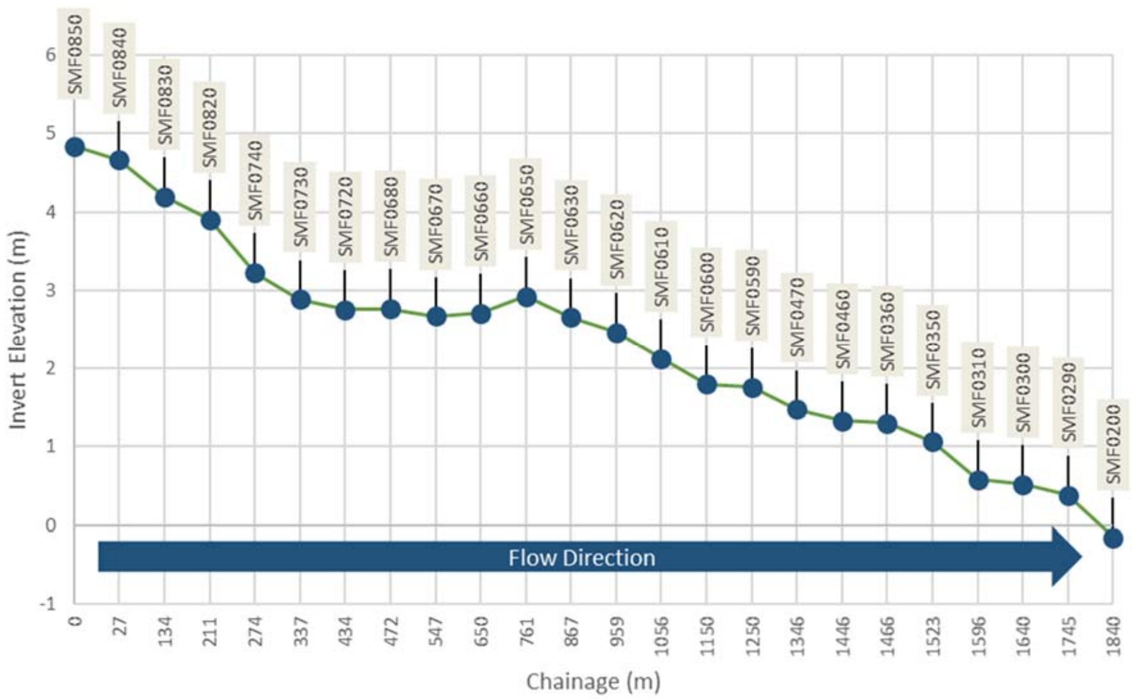
- Manhole invert elevations; and
- Manhole lid elevations.

Using the information above, long sections of the manhole invert elevations were generated and assessed.

Example of GIS data assessment
A sample long section of the trunk route's manhole inverts is shown in Figure 3.2. The trunk route between manhole SMF0850 and SMF0200 flows under gravity conditions. As can be seen, manhole SMF0660 and SMF0650 are at higher elevations than manholes located immediately upstream from them.

Based on the manhole lid elevations, the trunk route may be able to function under pressure conditions, however, based on experience, this is highly unlikely. Consequently, this was compared with invert elevation readings taken on-site and found to be erroneous.

Figure 3.2
Original Long Section Prior to Survey Correction



The erroneous invert elevations identified within the GIS data files established that an on-site survey of manhole elevations[2] was required. Manholes were identified for surveying along the trunk route. The updated list of manhole lid and invert elevations is available in Appendix A.

2 *Lid and invert elevations*

3.1.2 Pumpstation Properties

Some pumpstation dimensions and properties required as inputs to the hydraulic modelling were extracted from historic construction drawings. These properties included the following:

- Invert elevation of emergency overflow pipes;
- Elevation of pumpstation's wet well invert and roof elevations; and
- Wet well effective storage areas[3].

Many of the historic drawings were drawn using imperial units and from unknown datums. These drawings were therefore used in conjunction with elevations obtained from an on-site survey to process the data successfully.

Whilst processing the SCADA data[4], a difference was noticed between the emergency overflow invert levels, obtained from historic drawings, and the spillage levels defined in the SCADA. After closer investigation, it was found that the discrepancy was due to an offset in the SCADA's zero datum (i.e. transducer may be installed 150 mm above invert of wet well). For example, a SCADA measurement for the wet well sewage depth at a given time may be recorded as 600 mm, whereas in reality the actual depth is 750 mm.

This was taken into consideration and due to the flow data being derived from SCADA, the pumpstations' physical properties were adjusted to suit. Table 3.1 summarises the Inverts elevations and effective areas used for each of the pumpstations.

Table 3.1
Pumpstations' Physical Properties Used for Modelling[5]

Pumpstation	Emergency Spill Invert Elevation (m)	Wet Well Invert Elevation (m)	Wet Well Roof Elevation (m)	Effective Storage Area (m^2)
North Clyde	2.066	-0.359	5.366	17.2
Alexander Park	1.524	0.049	4.874	17.2
Kopu	0.366	-2.534	4.446	69.4
Fitzroy	1.243	-2.832	4.668	14

In addition to the required physical properties, the hydraulic model requires operation inputs. These operational inputs include rules that control when pumps are switched on and off. As discussed in section 2, the pumps are controlled by the level of sewage stored in the wet well. Table 3.2 summarises the inputs used to define the pump switching in the hydraulic model.

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3 *Required to create stage-storage curves*

4 *Discussed in section 3.1.3 and 3.2*

5 *Data presented based on New Zealand Vertical Datum 2016*

Table 3.2
Pump Switching in Hydraulic Model
(Sourced from SCADA Dashboard Screenshots)

Pumpstation	Depth at Which Pump Switches On (mm)				Depth at Which Pumps Switch Off (mm)
	Pump 1	Pump 2	Pump 3	Pump 4	
North Clyde	1745	1850	Not Applicable	Not Applicable	750
Alexander Park	1000	1250	Not Applicable	Not Applicable	500
Kopu	1100	1870	2150	Not Applicable	300
Fitzroy	1000	1350	1600	1900	500

3.1.3 Sewage Flow Data

Council has a Supervisory Control and Data Acquisition (SCADA) system installed, which enables monitoring of the reticulated sewage system. The SCADA allows real time monitoring and alerts Council with any problems / issues at the pumpstations (i.e. high level wet well measurements). On-site measurements are taken by sensors installed in the pumpstations and transmitted to the SCADA. This data is then recorded on a central server. Some of the measurements taken include levels in wetwells, pump on/off switches, instantaneous flow rate measurements and cumulative hourly pumped volumes.

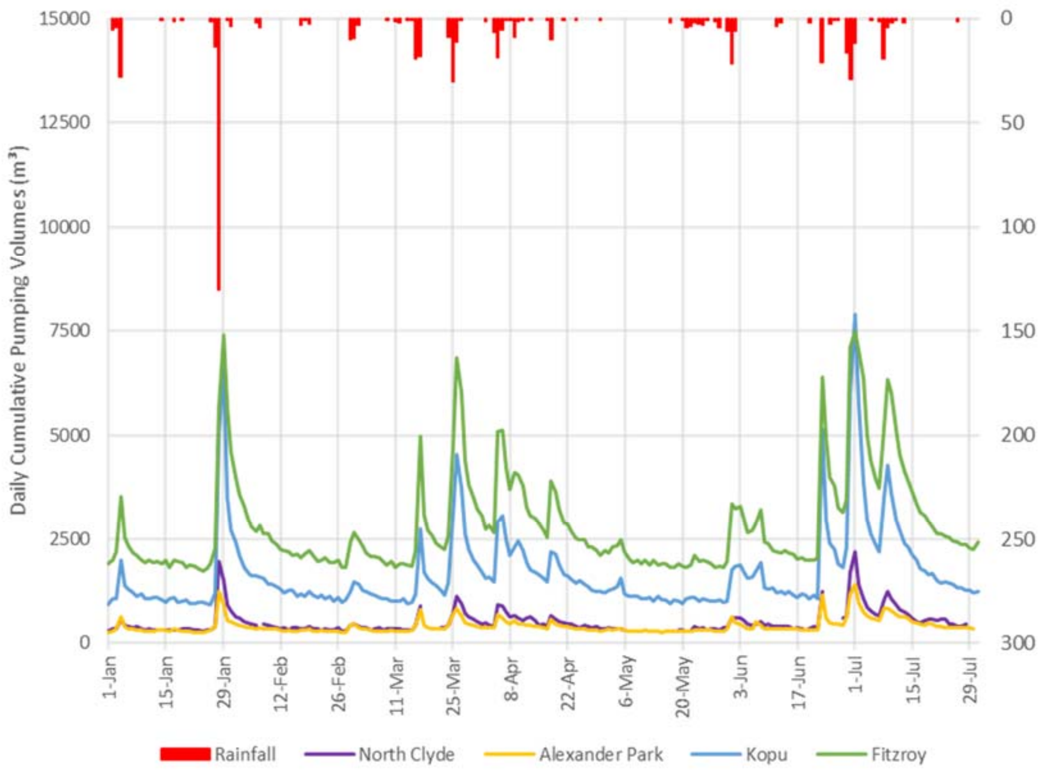
Based on a preliminary assessment of cumulative daily pump volumes received from Council, the period between January 2016 and March 2017 was identified as a representative data set during which multiple overflows were recorded by Council. Additionally, multiple high intensity rainfall events, and smaller recurring rainfall events were recorded within these dates. Therefore, the SCADA data set for this period was used for the hydraulic modelling.

A high level visual assessment of the cumulative daily pump volumes versus the recorded rainfall data confirmed a strong relation between pumping volumes and the effect thereon as a result of rainfall derived inflow and infiltration. This is shown for the period between January 2016 and July 2016 in Figure 3.3. Herein is demonstrated that the effect of one high intensity rainfall event may in certain circumstances be less than impacts from multiple recurring low intensity events[6].

It is important to note that the pumped volumes are not representative of the flows reporting to the pumpstations, as the effects of spillage have not been included in the volumes recorded in Figure 3.3. Nevertheless, it can be seen that rainfall directly influences the daily quantity of sewage that needs to be pumped.

6 *As demonstrated in Figure 3.3 - consecutive lower intensity rainfall events around 1 July, results in higher pumping rates than the isolated high intensity rainfall event of 130.5 mm on 28 January*

Figure 3.3
Daily Cumulative Pumping Volumes Compared to Rainfall



The SCADA data extracted and processed included wet well level recordings and cumulative hourly pumped volumes from each of the pumpstations. The wet well levels were typically recorded every 10 to 15 minutes, however measurements were also recorded every time a pump was switched on or switched off. This is equivalent to an average of 190 well level readings per pumpstation per day and 83,400 readings for each pumpstation during the selected assessment period. Additionally, the hourly cumulative volumes from each of the pumpstations consisted of 10 500 readings.

This data was assessed and arranged into a usable format for further processing. During the assessment processed, it was noticed that no pumping data was recorded for North Clyde pumpstation between 8 May 2016 to 17 May 2017. The impact of this data shortage is anticipated to be minimal since there was no significant rainfall recorded during this period.

Considering that the hydraulic modelling is highly reliant on the SCADA data, and the accuracy thereof, an on-site verification of the readings measured by the in-situ electromagnetic flowmeters was conducted. This verification was conducted using clamp-on type ultrasonic flow meters. Multiple flowrate readings were taken whilst varying the number of pumps switched on at the respective pumpstations.

Due to space constraints and insufficient access to a long enough portion of Kopu pumpstation's rising main, the flowrates from Kopu pumpstation could not be verified. However, the flowrates recorded from the other three pumpstations were successfully verified. As a result of the flowmeter's being of the same make, similar model and being installed during the same period, it was reasonable to assume that measurements recorded by Kopu pumpstation's flowmeter are also accurate.

Hence, the flowrate readings, recorded by the electromagnetic flow meters were consistent with flowrates recorded by the clamp-on flow meters. Difference that were encountered in the comparison of the flowrates were percentage errors and likely resulting from inaccurate input parameters such as the diameter and thickness of pipe. The verified flowrate readings were used as average flow rates, from the respective pumpstations, in the hydraulic model. The average flow rates recorded, varying the number of pumps switched on, is summarised in Table 3.3.

Table 3.3
Pumpstations' Average Flowrates

Pumpstation	Number of Pumps Switched On:			
	One Pump (ℓ/s)	Two Pumps (ℓ/s)	Three Pumps (ℓ/s)	Four Pumps (ℓ/s)
North Clyde	29.0	38.0	Not Applicable	Not Applicable
Alexander Park	14.4	19.4	Not Applicable	Not Applicable
Kopu	23.1	42.5	60	Not Applicable
Fitzroy	33	57.5	71	80

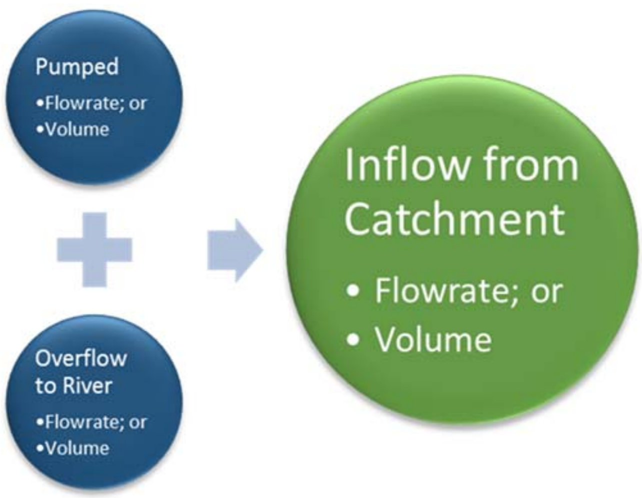
The effect of starting pumps in different sequences was investigated during the on-site assessment. The end result of having all the pumps switched on at a given pumpstation yielded the same results, irrespective of the order in which they were switched on.

3.2 Calculated Spillages from Pumpstations

One of the first steps required when designing a pumpstation is to assess the flows that will be reporting to it. The assessment requires the designer to calculate the average dry weather flow and the peak wet weather flow. These parameters are then used to select a suitable pump/s to accommodate the flows.

Similarly, a pumpstation's capacity can be assessed by evaluating its ability to accommodate the incoming flow. Furthermore, the incoming flow is required as an input parameter into the hydraulic model. The incoming flow reporting to the pumpstation can be calculated by applying a mass balance, which is demonstrated in Figure 3.4.

Figure 3.4
Application of Mass Balance



A pumpstation's inflow is equivalent to the summation of the flowrate pumped and the emergency overflow flowrate. Unfortunately, no overflow recordings are available for the system. Subsequently, the spillages were calculated from first principles, and based on the wet well levels recorded through the SCADA.

The emergency overflow discharge from each of the pumpstations operates under either pressure conditions or open channel / free flow conditions. The condition under which the discharge will occur is dependent on the level of sewage in the wet well. Pressure conditions will typically prevail when the level of sewage in the wet well is above the crown of the emergency discharge pipe, whereas, open channel / free flow conditions will occur when the level of sewage in the wet well is between the invert and crown of the emergency overflow pipe.

The prevailing discharge condition determined which formula was used to calculate the average rate of discharge for a given time increment. The Manning equation was used to determine flowrates during open channel / free flow conditions and the Bernoulli, Reynolds, Colebrook-White and Darcy-Weisbach equations were used to calculate the flow during pressure conditions. The formulas were applied to the wet well level SCADA datasets for each of the pumpstations and cumulative hourly spillage volumes were calculated accordingly. Appendix B contains the graphed results achieved during this analysis.

Upon completion of this assessment the inflow data for all the pumpstation were calculated. This was done simply by applying the methodology as defined in Figure 3.4. In order to synthesise inflow generated from Fitzroy pumpstation's associated catchment (which is a flow input required by the hydraulic model) another mass balance approach was required. This is discussed in greater detail in Section 3.3 below.

3.3 Generating Synthesised Flows from Fitzroy's Catchment

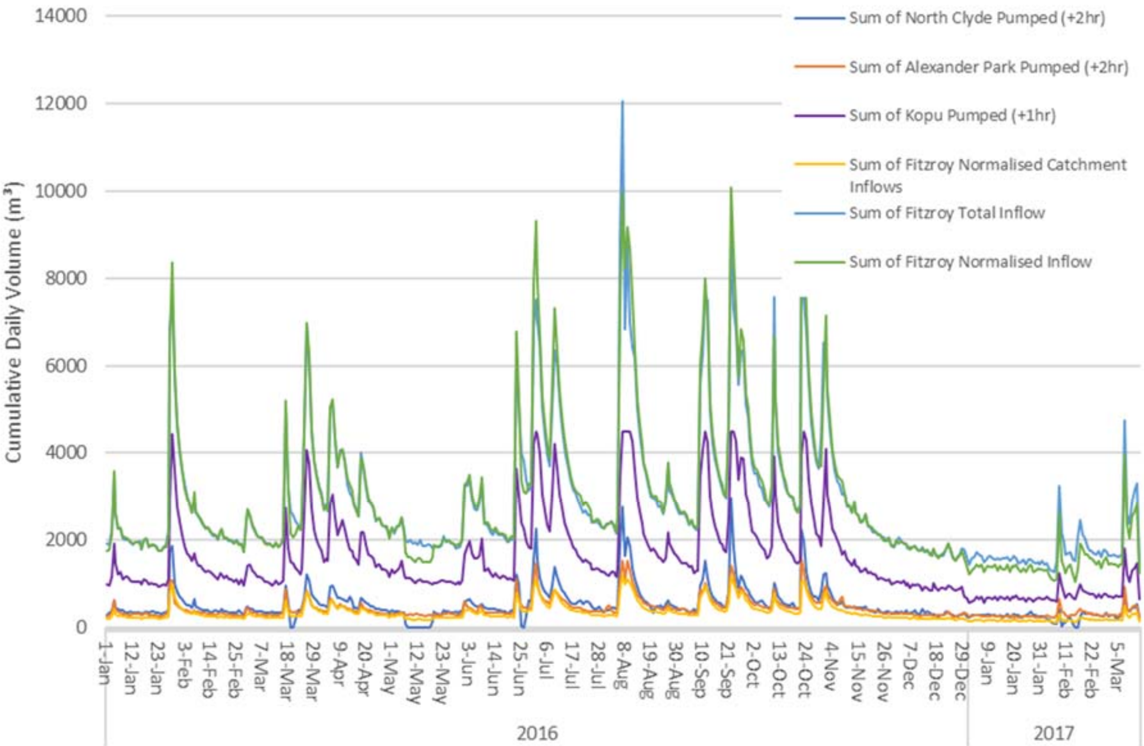
As defined in Figure 3.5, the flows reporting to Fitzroy pumpstation includes the pumped volumes from North Clyde, Alexander Park and Kopu pumpstation, and the flows generated from Fitzroy pumpstation's associated catchment. Fitzroy pumpstation's associated catchment is made up of a few houses located alongside the trunk route in addition to the flows from Rutherford pumpstation.

Considering that the total estimated inflow into Fitzroy pumpstation was calculated as defined in Section 3.2, the general principle used to calculate the flow generated from the catchment entailed subtracting the pumped volumes from North Clyde, Alexander Park and Kopu pumpstation from the calculated total inflow. For purposes of clarity this methodology is represented in Figure 3.5 below.

Figure 3.5
Calculation of Fitzroy Pumpstation's Associated Catchment Inflow

Due to the routing effects within the reticulation, each of the pumpstations pumping volumes were offset by the time it took for the associated flow to report to Fitzroy pumpstation. The flow data was offset in hourly increments, based on the highest resolution of flow data available. The time offsets used for each of the pumpstations were compared and adjusted to be consistent with the time difference found using the hydraulic model. The results obtained from this process is shown in Figure 3.6.

Figure 3.6
Synthesised Inflows from Fitzroy Catchment

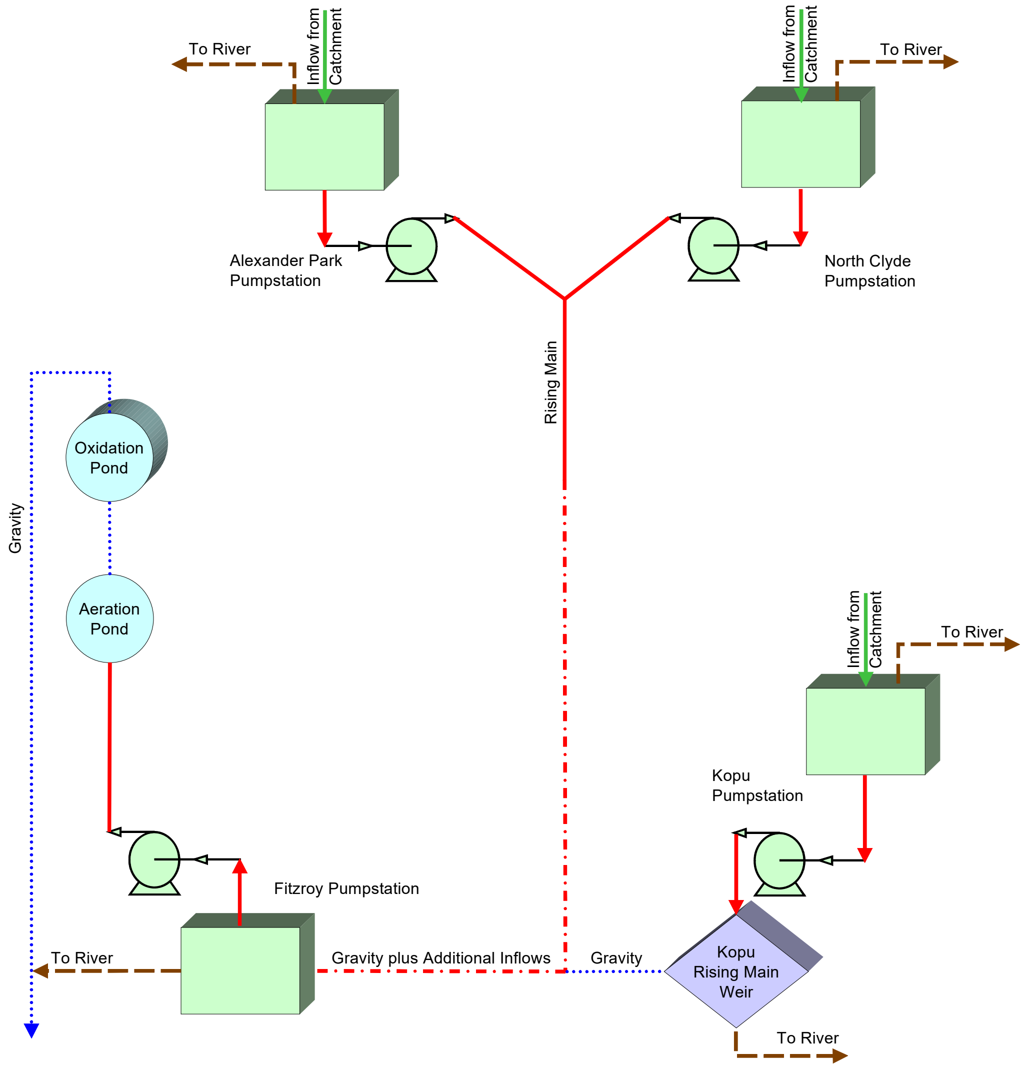


The plotted yellow line in Figure 3.6 represents the synthesised flow data generated for Fitzroy pumpstation's associated catchment area. Adding the pumped flows from each of the pumpstations with the synthesised catchment inflow, creates the data represented by the green line. Comparing the green line with the light blue line, which was calculated for Fitzroy pumpstation as described in Section 3.2, provides reasonable validation of the synthesised flow.

3.4 Compilation and Verification of Baseline Model

The data collected, assessed and processed, as explained previously, was used to compile the baseline hydraulic model in a software package called XPSWMM. XPSWMM was developed by XPSolutions and is used for planning, modelling and management of sustainable drainage solutions. A schematic of the baseline model, which includes the trunk route and associated pumpstations, is shown in Figure 3.7.

Figure 3.7
Baseline Model Schematic



The model was assembled in a step-wise manner. Each of the pumpstation modules were added and tested individually prior to being combined together as shown in Figure 3.7. As mentioned previously, a pumpstation's capacity can be assessed by evaluating its ability to accommodate the incoming flow. This assessment was conducted by monitoring the modelled discharges through the emergency overflow pipes from each of the pumpstations.

A method to assess the validity of the outputs from the hydraulic model was through comparing the discharge flows from the model with the spillage flows calculated, as described in Section 3.2. The graphed results used for this assessment is shown in Appendix C. The magnitude, duration, date and time of spillages for each data set for the corresponding pumpstations were compared and found to be alike.

In addition to this, another form of verification used was to compare the total pumped volumes in the hydraulic model with the pumped volumes recorded by the SCADA. This was conducted by adding and comparing the cumulative pumped volumes from each dataset. This information is summarised in Table 3.4.

Table 3.4
SCADA's Pumped Volume vs. Model's Pumped Volume

Pumpstation	SCADA's Total Pumped Volume (m^3)	Model's Total Pumped Volume (m^3)	Difference (%)
North Clyde	224,903	224,000	-0.4
Alexander Park	192,570	191,425	-0.59
Kopu	741,654	714,250	-3.69
Fitzroy	1,244,544	1,245,647	0.27

The total of 1.08 percentage difference between the SCADA and hydraulic model provides sufficient confidence that the baseline hydraulic model is sufficiently representative of the on-site sewage reticulation.

Upon successfully verifying the validity of the baseline model, mitigatory scenario changes were made in an iterative manner to identify feasible system improvement options. These system improvement options are discussed in Section 4.

3.5 Sewage Reticulation Extension Methodology

The two areas requiring extensions of the existing sewage reticulation are located in the North Clyde pumpstation's catchment and along Kopu Road on the boundary of the Fitzroy and Kopu catchments (refer to Figure 2.1 for locality). This section describes the methodology that was used to identify and assess option(s) to extend the existing sewage reticulation to service the areas identified by Council.

Firstly, the anticipated sewage flows from each of the catchments were calculated based on Council's Engineering Code of Practice (Wairoa District Council, 1999). Only developed properties (i.e. properties with buildings on them) were considered in this assessment. Once the sewage flows were calculated, a location to connect into the existing sewage reticulation was identified.

The accessibility (i.e. not in private properties) and the envisaged downstream system capacity (refer to Section 4) were the criteria used to identify the location of the proposed connection to the existing sewage reticulation. After the connection location for each catchment was selected, two system conveyance options were considered, namely gravity and low pressure sewage systems.

The design criteria, in Council's Engineering Code of Practice, was used to assess whether the gravity conveyance was suitable for the areas in question. The confining design parameters used to assess the gravity systems included minimum pipe cover requirements, minimum pipe sizes, pipe roughness, secondary losses, pipe slopes and the associated flow velocities. By conforming to the design parameters, it was possible to assess whether the downstream invert elevation of the proposed system extension is able to gravitate into the existing sewage reticulation. Whether or not the gravity conveyance was considered to be a feasible option, a low pressure system was considered as an alternative option for both areas requiring reticulation.

A low pressure system makes use of small, low-powered, grinder pumps in each property, which are each connected to a central discharge network. Each of these units include a storage tank with a typical capacity of approximately 900-litres, semi-positive displacement grinder pump and an automatic electronic controller. An indicative low pressure system was drawn for each extension area and costed accordingly.

4 SYSTEM IMPROVEMENT & EXTENSION OPTIONS

According to the New Zealand Infiltration & Inflow Control Manual 2015, there are numerous approaches / solutions that could be used to improve the performance of wastewater systems to address the lack of capacity that may be due to excess Inflow and Infiltration. These Include the following:

- conveyance system augmentation;
- operational optimisation;
- peak flow attenuation through detention storage;
- flow reduction through Inflow & Infiltration management; and
- use of controlled overflow structures.

Each of these approaches have a unique role in the development of a sustainable wastewater system. The mitigatory options identified in this assessment focuses on conveyance system augmentation and operational optimisation. Peak flow attenuation was not considered as a viable option in this assessment due to the high rainfall derived system inflows and/or infiltration (RDII). It is Good Earth Matter's understanding that Council is in the process of implementing a RDII management strategy.

As mentioned previously, the current wastewater system is fitted with well-designed controlled overflow structures which have successfully alleviated / eliminated pumpstation surcharging. Subsequently this has been excluded from the list of potential mitigatory options adopted in this evaluation.

A similar approach was adopted in applying mitigatory options, as was applied when compiling the baseline hydraulic model. Feasible mitigatory options were applied to the baseline hydraulic model individually; assessed; and then combined in a consecutive manner. Therefore, the proposed improvements in Option 1 are included in Option 2, together with its associated adjustments. This process allows the system to be assessed holistically, which is required when proposing changes to an integrated sewage network. System improvement options were selected that would make a distinguishable improvement to the efficiency of the system. In addition to the baseline model, the following improvement options were considered:

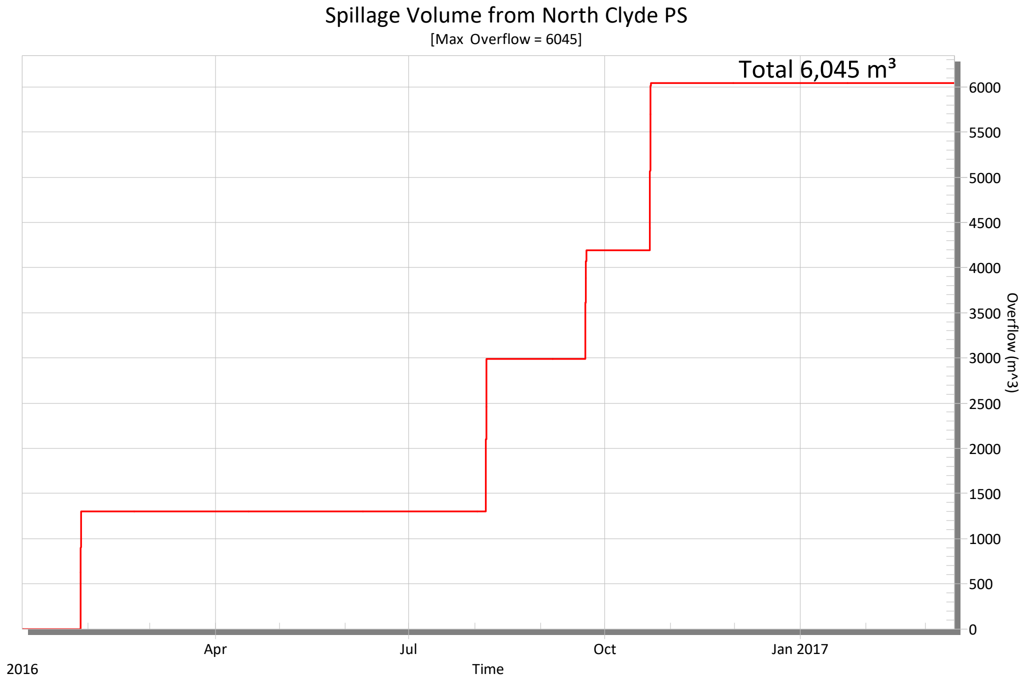
- Option 1a & 1b - Separate rising main for North Clyde and Alexander Park pumpstations;
- Option 2 - Duplicate rising main at Fitzroy pumpstation;
- Option 3a - Installation of rising main from Kopu pumpstation to Fitzroy pumpstation (whilst maintaining pump capacities similar to the existing; and
- Option 3b - Installation of rising main from Kopu pumpstation to Pilot Hill Treatment Ponds with an increased pump capacity of 50 percent.

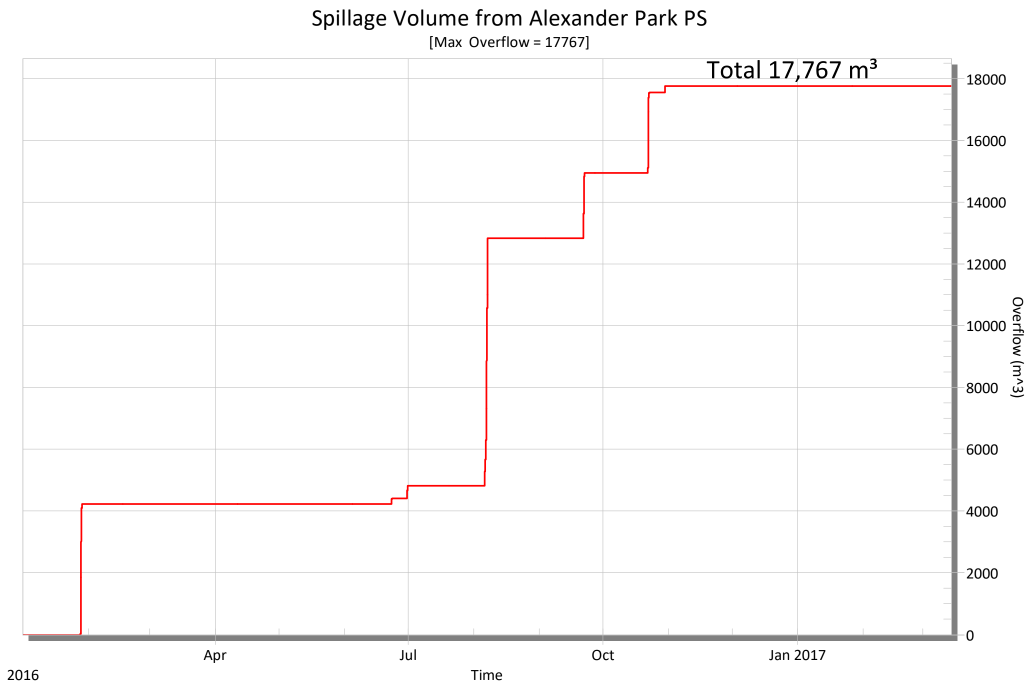
The results from the baseline hydraulic model and system improvement options are discussed in detail below.

4.1 Option 1 - System Improvement

A performance indicator used to assess the effectiveness of the proposed mitigatory options was based on reduced spillages from each of the pumpstations. As can be seen in Figure 4.1, the baseline analysis indicated a cumulative spillage of 6,000 m³ for North Clyde and 17,800 m³ for Alexander Park pumpstation during the period of 1 January 2016 and 14 March 2017.

Figure 4.1
Baseline Model - Spillages from North Clyde and Alexander Park Pumpstation





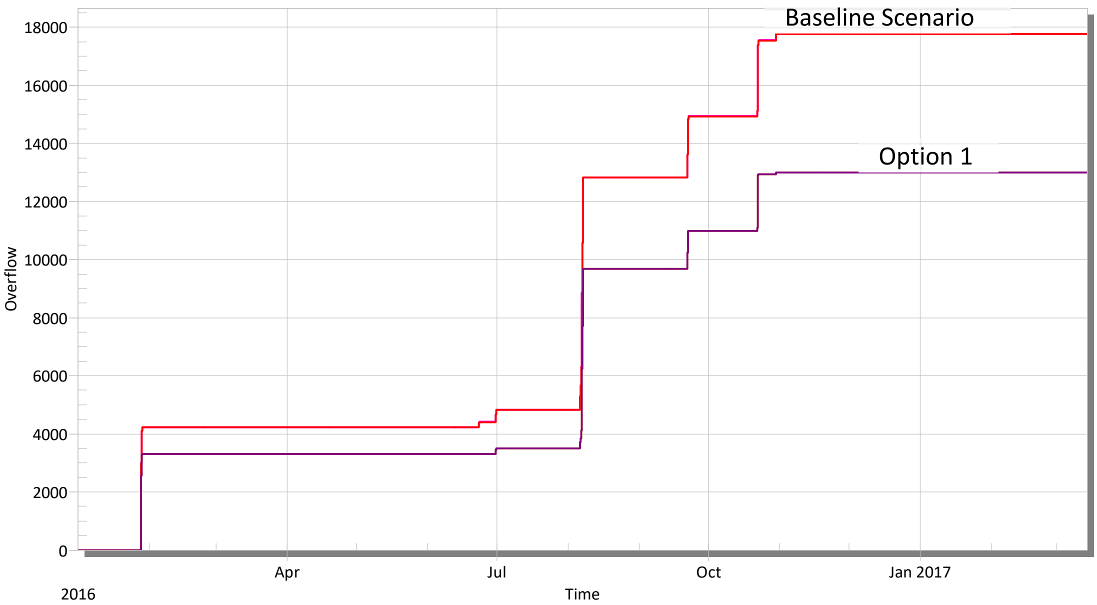
Additionally, it was noticed that simultaneous pumping of North Clyde and Alexander Park had an adverse effect on the pumping flow rates (in the order of a 5 ℓ/s reduced pump capacity for Alexander Park pumpstation). This is likely, considering that the pumpstations share a rising main, thus increasing the head loss that the pumps are required to overcome, whilst both pumps are operational. Subsequently, the first option considered was to separate the rising mains (i.e. install new rising main for Alexander Park pumpstation). This is shown schematically in Figure 4.3.

Kopu pumpstation was excluded from this assessment. This was done to evaluate the available capacity in the trunk route gravity pipeline, shown as a dotted red line in the figure.

An immediate system optimisation will be achieved through the implementation of a separated rising main. The outcome will ensure that North Clyde and Alexander Park pumpstations will be able to operate independently without negatively impacting one another's flow capacities.

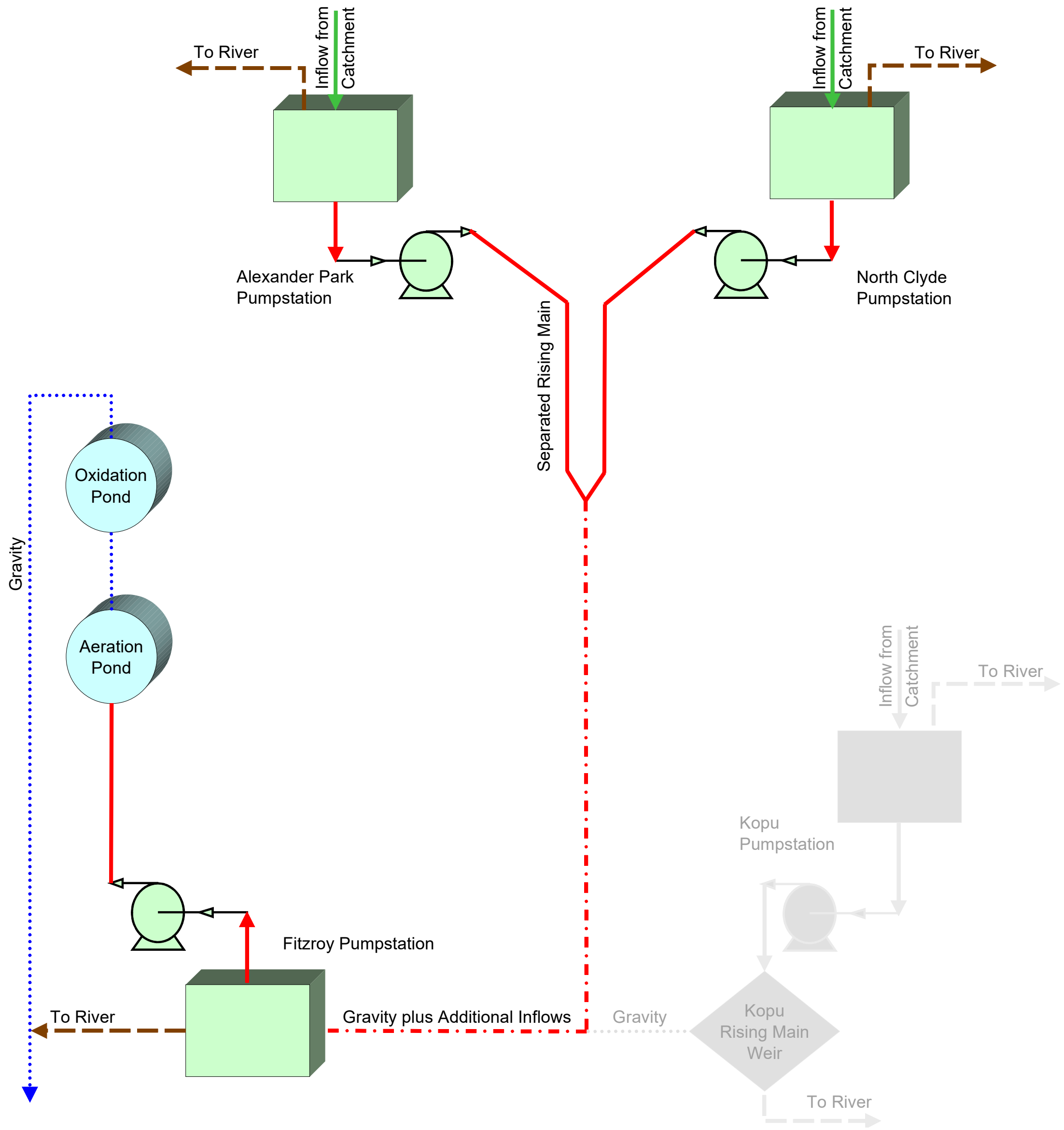
To reduce spillage through the emergency overflow, the pump capacity of Alexander Park pumpstation was increased by 50 percent, which can be achieved through the installation of two new pumps, similar to those installed in North Clyde pumpstation. The results achieved from this is shown in Figure 4.2.

Figure 4.2
Reduced Spillage from Alexander Park Pumpstation



By increasing the flow capacity of Alexander Park pumpstation by 50 percent (equivalent to an increased pumping flowrate of 19 ℓ/s), reduced the spillage by approximately 4,800 m³ during the assessment period. A similar adjustment was made for North Clyde pumpstation (increased total flowrate by 19 ℓ/s), however the spillage reduction was only in the order of 2300 m³. Additionally, the pumps installed in North Clyde pumpstation are relatively new, which made the proposed adjustment less viable.

Figure 4.3
Option 1 - Separate Rising Main for North Clyde and Alexander Park Pumpstations



The effects of the proposed increased flow rate at Alexander Park pumpstation was assessed in the downstream trunk route. This was achieved by monitoring the dynamic long section of the trunk route (still disregarding flows from Kopu pumpstation). Figure 4.4 shows the maximum water level lines in the long section during baseline flows and Figure 4.5 shows the maximum water level line for the increased flowrate. The maximum water level lines are represented by the magenta line in the following figures.

Figure 4.4
Baseline Dynamic Long Section of Trunk Route

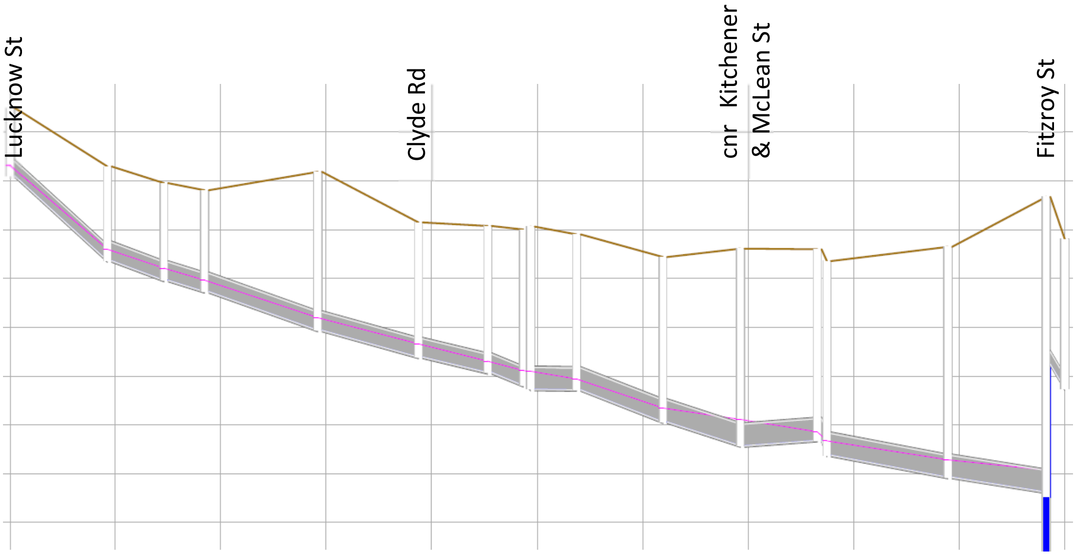


Figure 4.5
Option 1 Increased Flowrate Dynamic Long Section

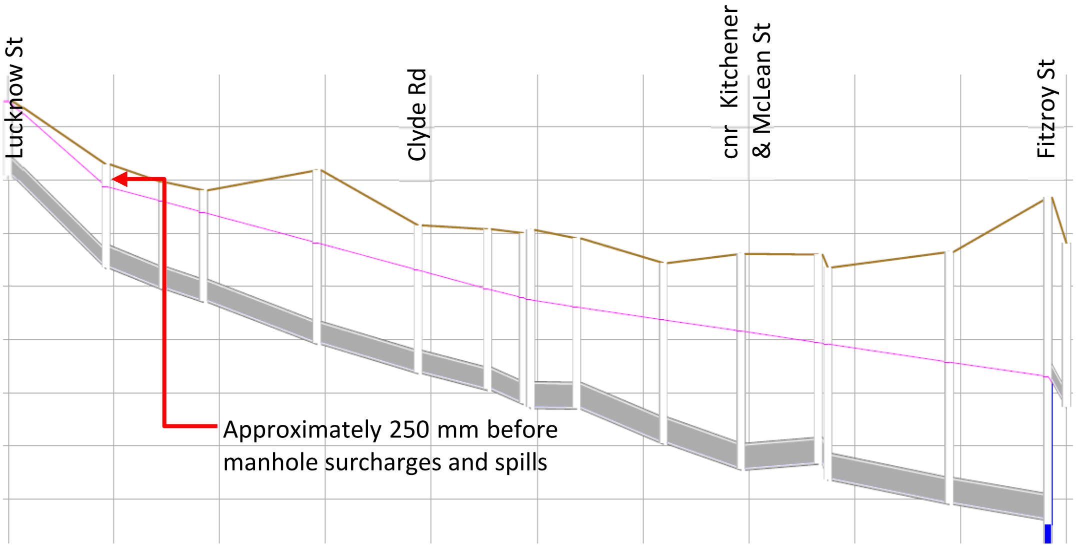


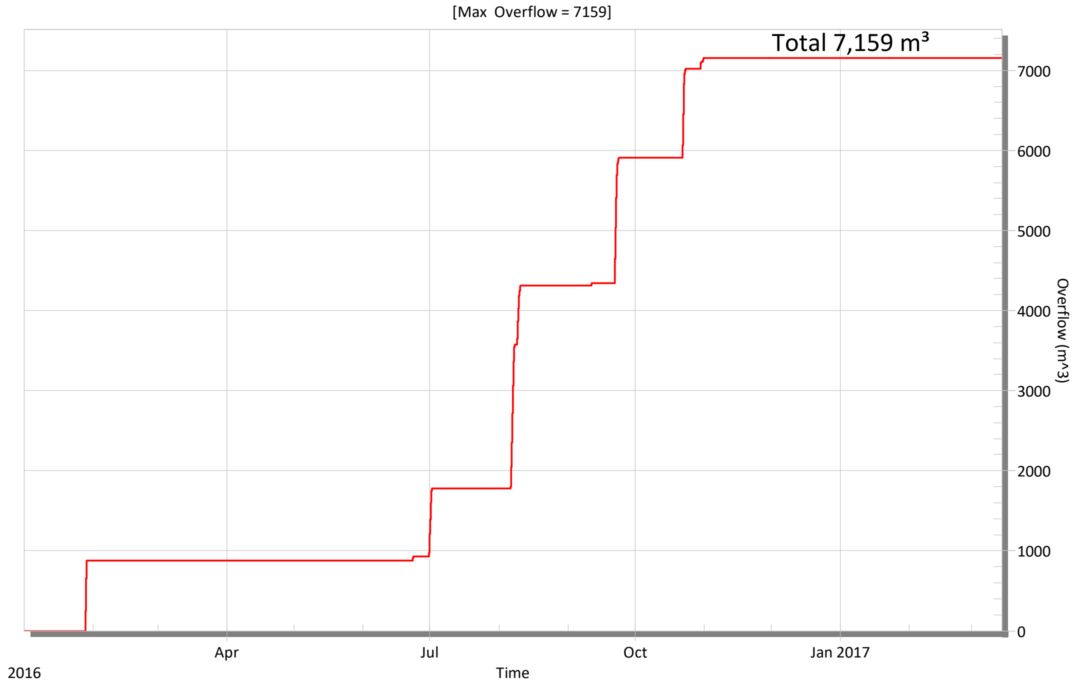
Figure 4.5 shows that the increased flowrate of 19 ℓ/s causes static head build-up in the manholes along the trunk route. The reason for the surcharging effect is due to Fitzroy pumpstation's inadequate capacity which causes backing up of sewage in the system.

Considering that flows from Kopu pumpstation were disregarded / switched-off during this analysis, identifies that Fitzroy pumpstation does not have adequate capacity to accommodate the baseline flows from North Clyde, Alexander Park and Kopu pumpstations. This is discussed in further detail and addressed in Option 2 and Option 3.

4.2 Option 2 - System Improvement

Fitzroy pumpstation is fitted with four Flygt 3152 submersible pumps, which are required to service Wairoa's entire sewage catchment area. The emergency overflows from the pumpstation derived from the baseline hydraulic model is shown in Figure 4.6.

Figure 4.6
Baseline Model - Spillages from Fitzroy Pumpstation

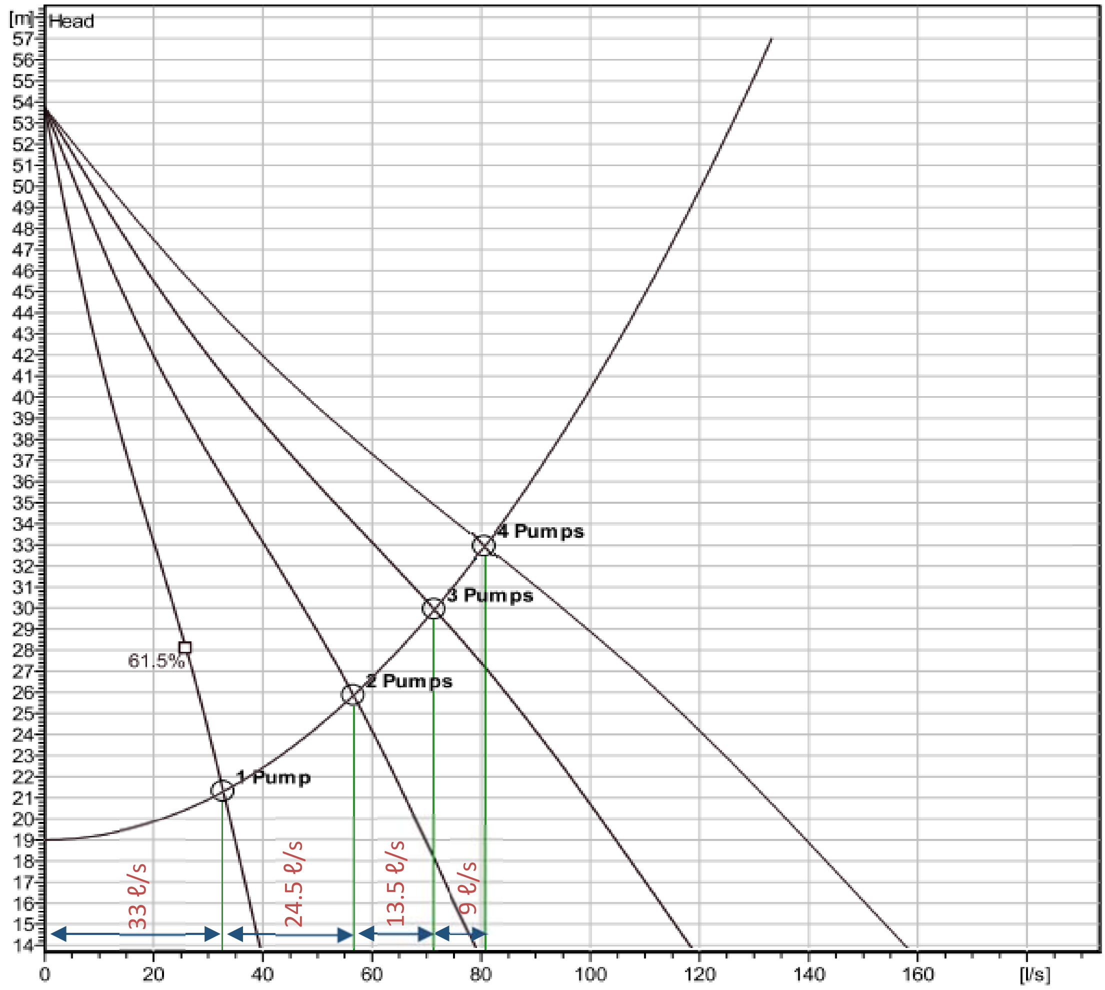


The 7,159 m³ spillage volume from Fitzroy pumpstation during the baseline hydraulic model reinforces the findings observed during the mitigatory changes proposed in Option 1. It is important to note that these spillages are limited to periods of high intensity or the resultant of cumulative low intensity rainfall events. The system has adequate capacity to accommodate average dry weather flows.

Based on the verified on-site flowrate measurements, each of the pumps at Fitzroy pumpstation have an individual flow capacity (based on the system configuration) of approximately 33 ℓ/s. Multiplying the flow capacity by four results in a total of 132 ℓ/s. However as shown in Table 3.3, the combined pumping capacity is only 80 ℓ/s when all four pumps are running simultaneously. This is equivalent to only 61 percent of the individual isolated capacities. The reduced pumping capacity typically occurs when operating multiple pumps in parallel. The cause of the reduced flow is a result of the increased frictional losses (frictional headloss) as the velocity and associated flow increases in the rising main. This effect is graphed and demonstrated in Figure 4.7.

The effective flowrate capacity gained is reduced based on the number of pumps switched on. The 300 mm diameter rising main pipe is therefore limiting the maximum potential pumping capacity. Therefore, in addition to the changes incorporated to the baseline hydraulic model in Option 1, Option 2 proposes to duplicate the rising main between Fitzroy pumpstation and the Pilot Hill Treatment Ponds. This is shown schematically in Figure 4.8.

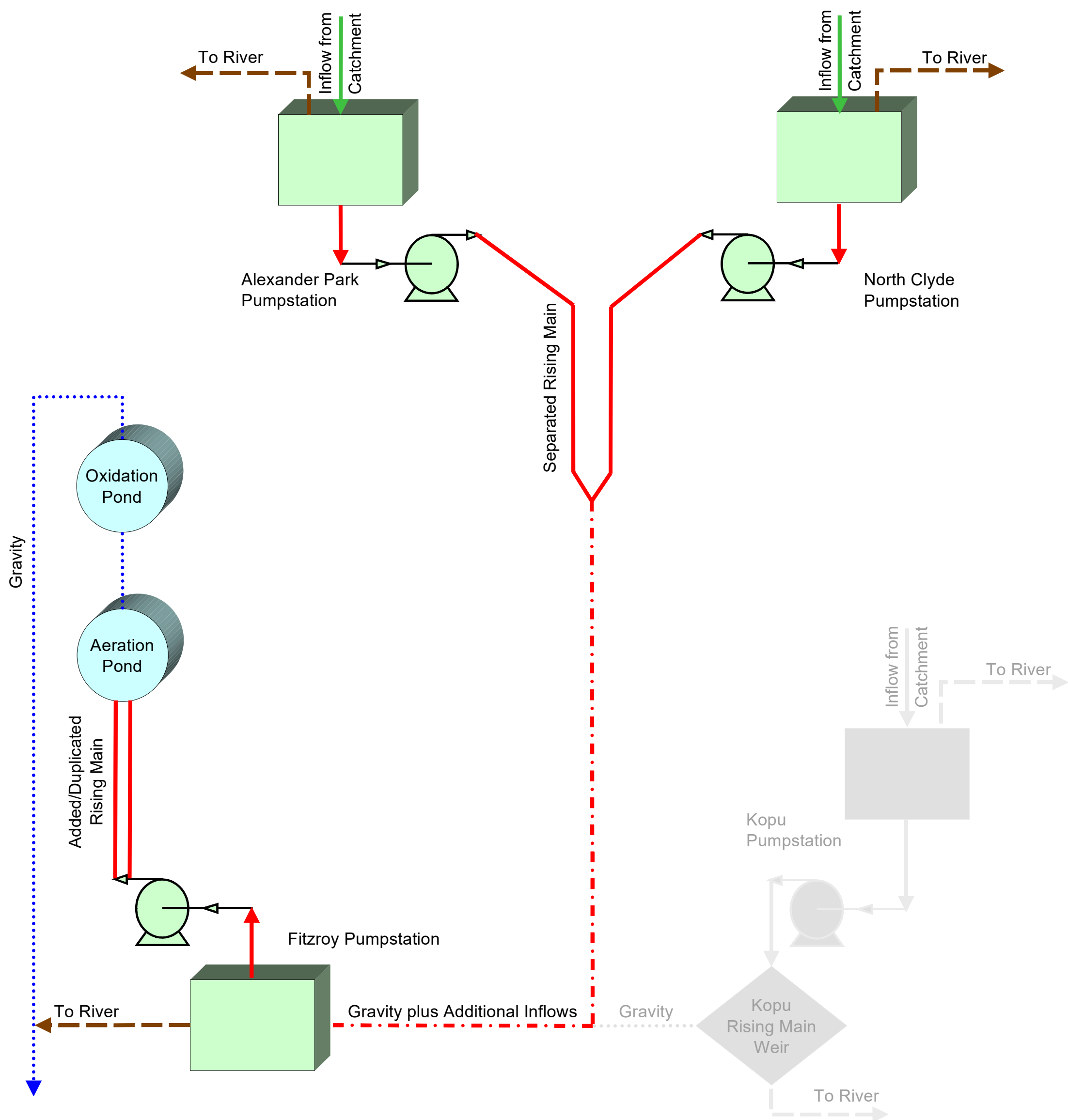
Figure 4.7
Fitzroy Pumpstation's Pump Capacity
(Xylem, 2017)



The anticipated effects of the duplicated rising main will increase Fitzroy pumpstation's pumping capacity from 80 ℓ/s to approximately 115 ℓ/s, which is equivalent to 87 percent of the individual isolated capacities. The changes were incorporated into the hydraulic model as indicated in Figure 4.8 and spillages were eliminated from Fitzroy pumpstation's emergency overflow during the analysis period (with flows from Kopu pumpstation still excluded from the model). Additionally, the backing-up effect and surcharging of manholes as shown in Figure 4.5 was also resolved.

Up to this point of the modelling, with the exception of the baseline hydraulic model, flows from Kopu pumpstation have not been considered in the analysis. Option 3 incorporates the flows from Kopu pumpstation and proposes mitigatory system enhancements to reduce overflows.

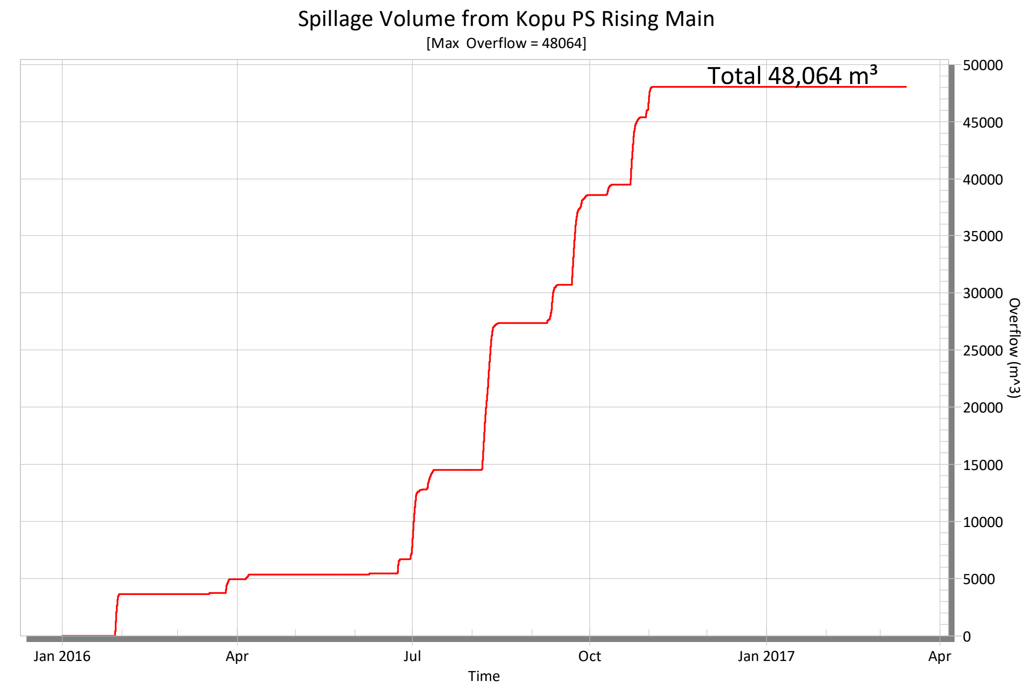
Figure 4.8
Option 2 - Duplicate Rising Main at Fitzroy Pumpstation

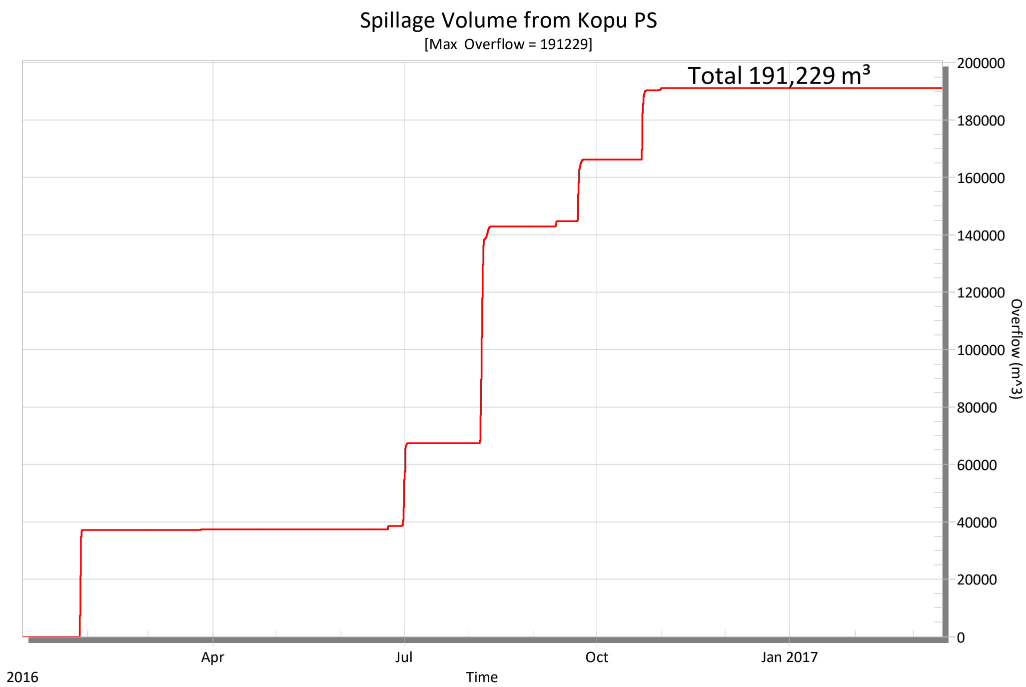


4.3 Option 3 - System Improvement

Kopu pumpstation is currently able to perform controlled emergency overflows from both the inlet side of the wet well and at the discharge end of the rising main. As mentioned previously, any flow from the rising main exceeding 52 ℓ/s, is discharged into the Wairoa River. Both emergency overflows were monitored in the baseline hydraulic model, during the selected assessment period, and the overflow volumes therefrom were graphed and shown in Figure 4.9.

Figure 4.9
Kopu Pumpstation Emergency Overflows





The net overflow during the assessment period was in excess of 48,000 m³ at the rising main end and approximately 190,000 m³ from the wet well's emergency overflow pipe. The first option evaluated to optimise the system was to try and eliminate spillages from the rising main discharge chamber.

4.3.1 Option 3a

The gravity trunk route downstream from Kopu pumpstation operates at/near maximum design capacity during rain events. Consequently, bypassing the overflow weir and discharging Kopu pumpstation's flow into the gravity trunk route would simply cause surcharging of manholes downstream. Therefore, Option 3a proposes the installation of a new 300 mm diameter rising main from Kopu pumpstation to Fitzroy pumpstation. This is shown schematically in Figure 4.11.

Kopu pumpstation's existing rising main is only approximately 50 m long and the associated frictional losses are relatively small. The installation of a new rising main between Kopu pumpstation and Fitzroy pumpstation will substantially increase the frictional losses due to the increased length. In order for the pumping capacity to remain consistent with the status quo, the pumps in Kopu pumpstation will need to be replaced, regardless of increasing the flow capacity or not. Consequently, a scenario of increasing Kopu pumpstation's flow capacity was assessed in addition to the existing / status quo pump flow capacity. The spillage from the wet well was recorded for each option and shown in Figure 4.10.

Figure 4.10
Option 3a & 3b - Spillage from Kopu Pumpstation

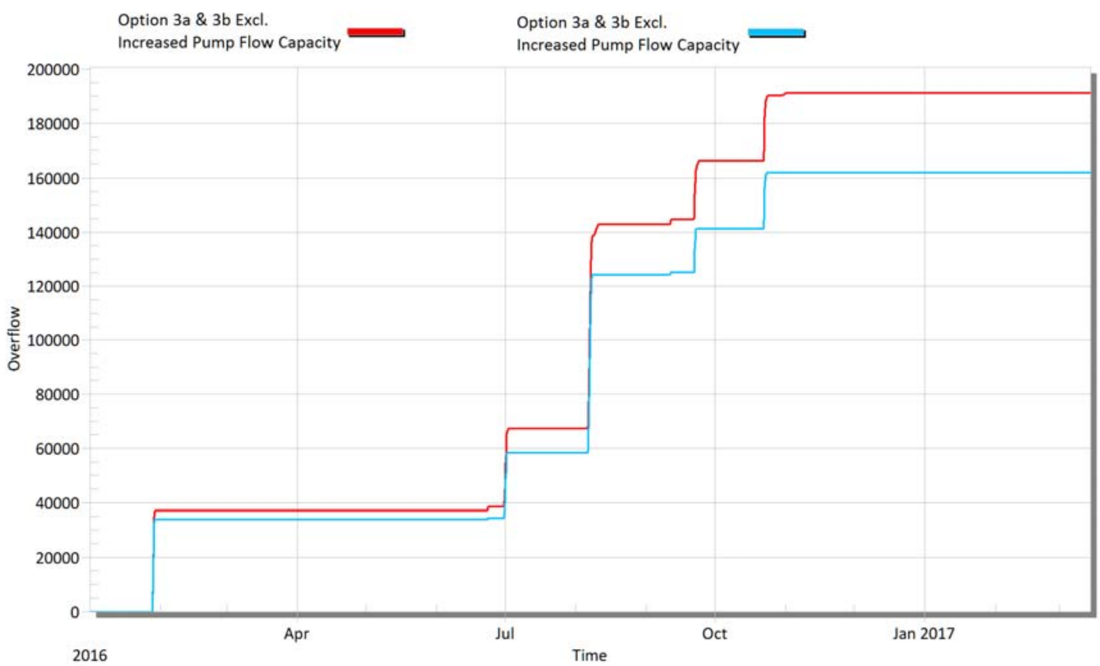
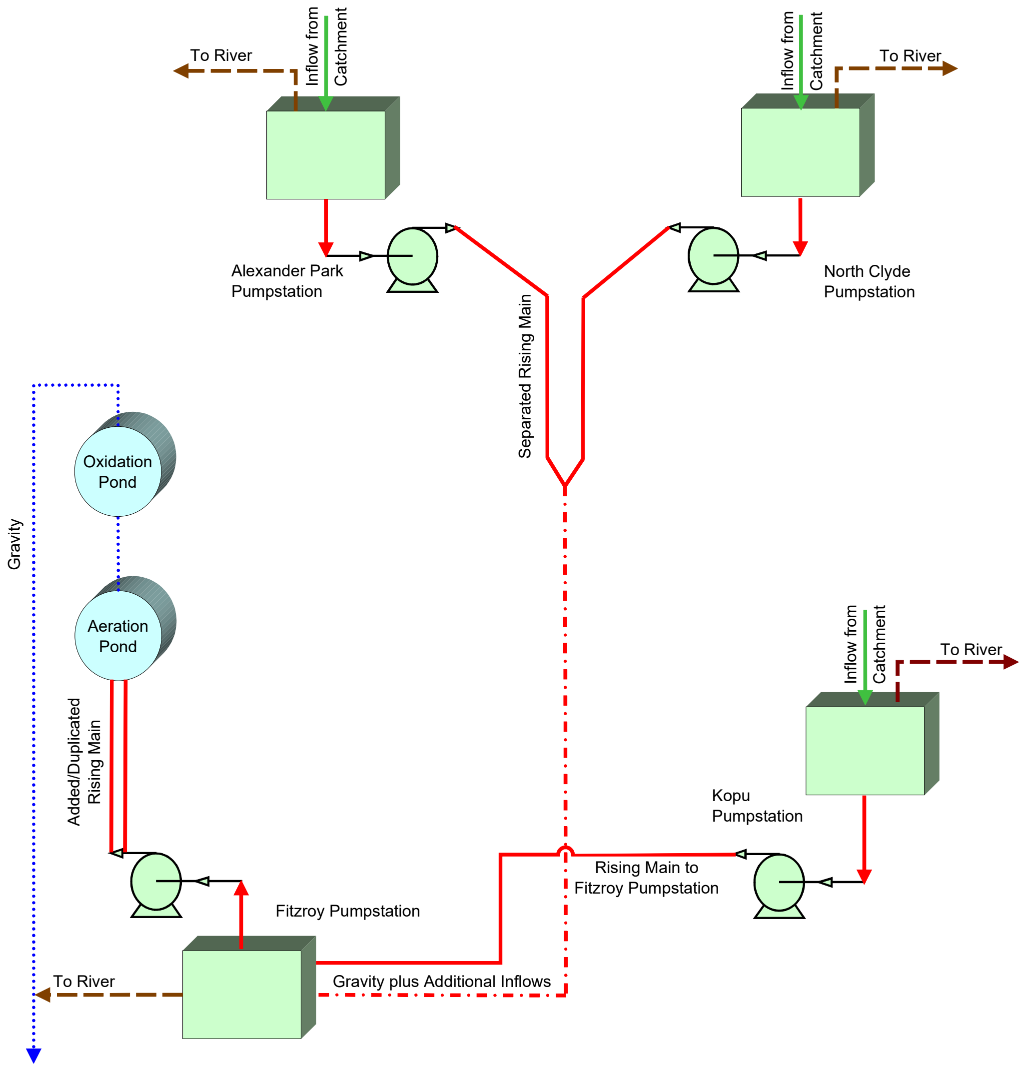


Figure 4.11
Option 3a - Installation of Rising Main from Kopu Pumpstation to Fitzroy Pumpstation



A 50 percent increase in pumping capacity was considered economically feasible[7].The effect of the increased pumping capacity reduced overflows from Kopu pumpstation's wet well by approximately 30,000 m³ in the hydraulic model. This was in addition to the eliminated spillages of 48,000 m³ that was occurring from the rising main's discharge chamber.

The proposed increase in pumping capacity from Kopu to Fitzroy pumpstation may have decreased spillages from Kopu pumpstation's emergency overflow, however, the hydraulic model showed spillages from Fitzroy pumpstation as a result of the increased flow. The spillages from Fitzroy pumpstation's emergency overflow were taking place with the suggested improvements recommended in Option 2 (i.e. duplicated rising main from Fitzroy to Pilot Hills Treatment Ponds).

To address the spillages caused by pumping from Kopu to Fitzroy pumpstation, Option 3b proposes to pump directly from Kopu pumpstation to the Pilot Hill Treatment Ponds.

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7 *It is anticipated that an increased pumping capacity beyond 50 percent may require additional costly upgrades to the pumpstation*

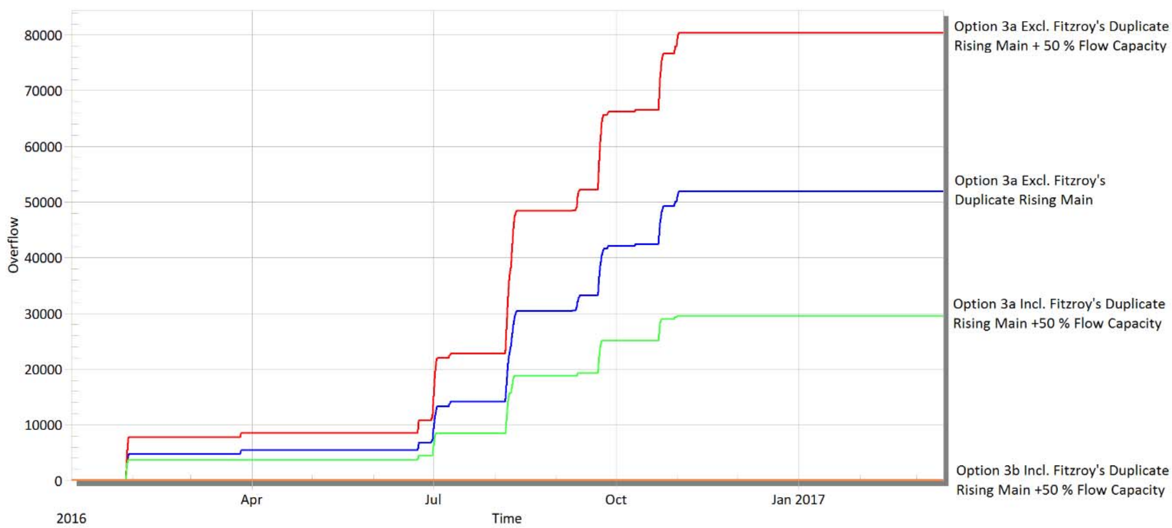
4.3.2 Option 3b

To eliminate spillages from Fitzroy pumpstation, resulting from changes recommended in Option 3a, the rising main between Kopu and Fitzroy pumpstation was removed from the hydraulic model and replaced with a rising main between Kopu pumpstation and the Pilot Hills Treatment Ponds. The proposed change was incorporated and the hydraulic model was run for the assessment period. The incorporated changes are shown schematically in Figure 4.13. The proposed rising main between Kopu and the Pilot Hills Treatment Ponds will require the existing pumps to be replaced with larger pumps capable of meeting the increased head requirements (both due to an increased static differential and additional frictional losses associated to the increased pipe length).

The spillages from Kopu pumpstation's wet well are as shown in Figure 4.10. As mentioned previously, the spillage reductions at Kopu pumpstation are dependent on the selected increase in flow capacity, however it is anticipated that a 50 percent increase is reasonable and achievable without significant additions / changes to the existing pumpstation.

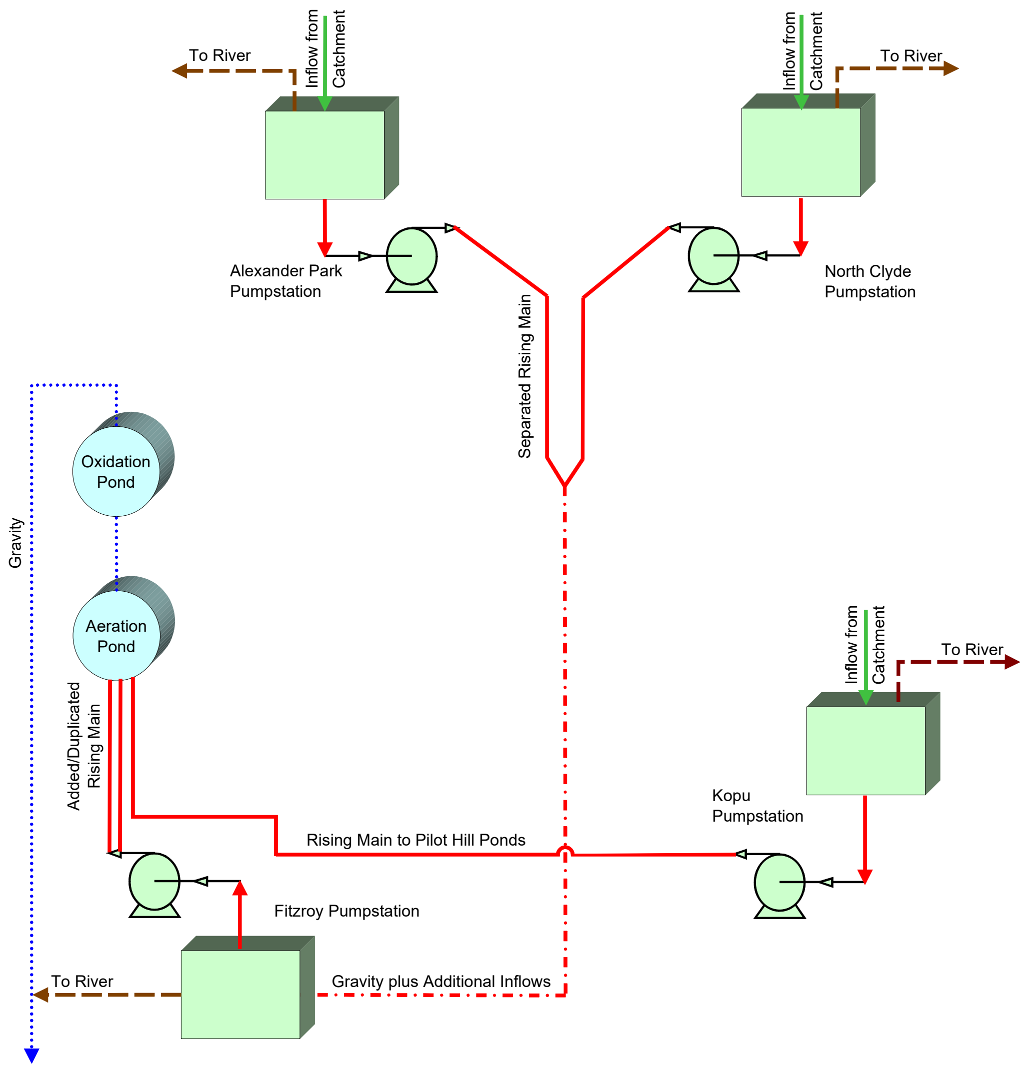
The variance in spillage from Fitzroy pumpstation, associated to the adjustments made in Option 3a and 3b, was graphed and shown in Figure 4.12.

Figure 4.12
Option 3a & 3b - Spillage from Fitzroy Pumpstation



According to the results from the hydraulic model, spillages from Fitzroy pumpstation were eliminated based on the adjustments proposed in Option 3b.

Figure 4.13
Option 3b - Installation of Rising Main from Kopu Pumpstation to Pilot Hill Treatment Ponds



4.4 North Clyde Reticulated Extension Area

4.4.1 Gravity Conveyance

Based on a pipe size of 150 mm and a slope of 1 vertical to 150 horizontal (Grade of 0.5 percent), it was found that the extension area was unable to freely gravitate and discharge into the existing sewage reticulation. Flatter pipe slopes were also considered with no avail. The connection point selected for the assessment is as shown in Figure 4.14. Gravity conveyance was therefore ruled out as a feasible conveyance option for the North Clyde extension area.

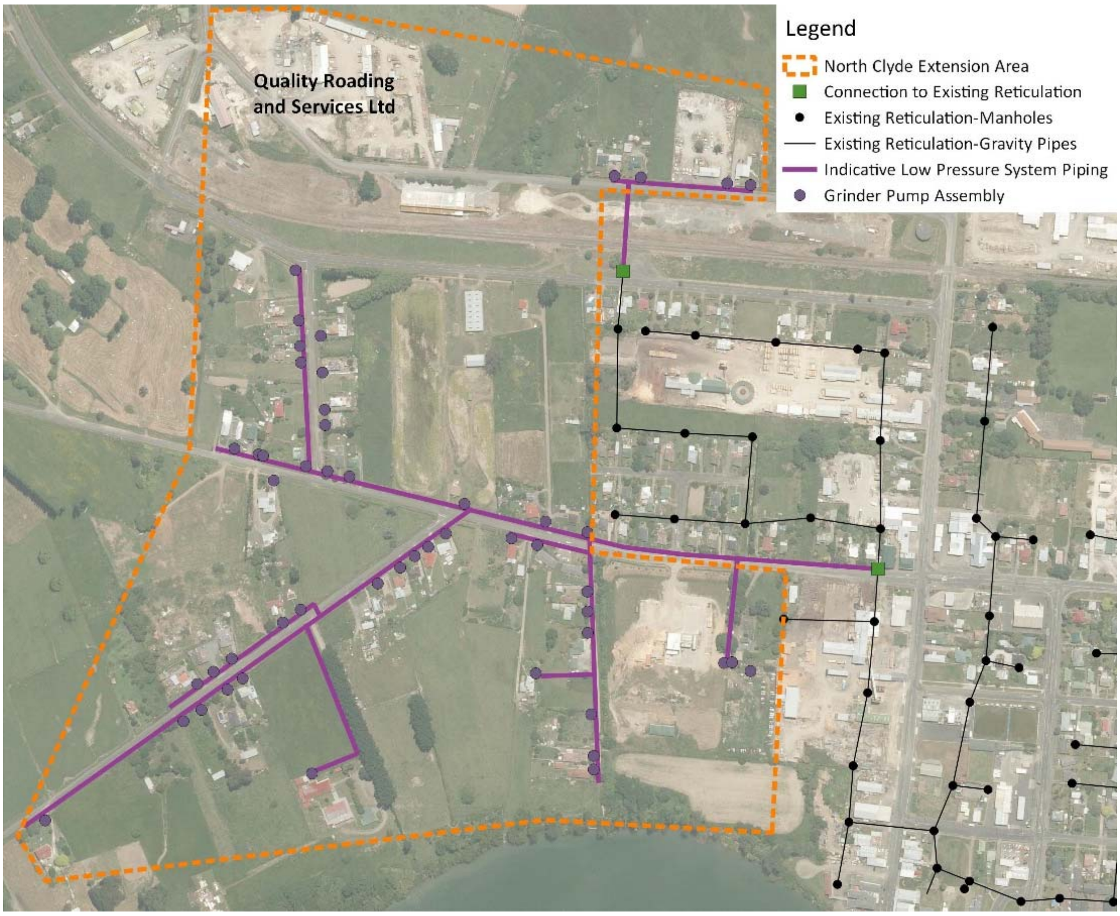
4.4.2 Low Pressure Sewer Conveyance

An indicative low pressure sewage system conceptual design was completed for the North Clyde extension area. A schematic of the proposed system is shown in Figure 4.14. The low pressure sewage system is divided into two networks located either side of the Kiwi Rail siding.

The proposed network to the north of the rail siding is composed of four developed properties. Quality Roading and Services Ltd's property was not included in the assessment as they recently constructed a sewage pumpstation and rising main that connects to Council's existing sewage reticulation. Pipe drilling will be required beneath the Kiwi Rail siding to enable the properties to the north of the rail siding to connect to the existing sewage reticulation.

The area to the south of the rail siding is composed of 46 developed properties. The proposed low pressure sewage network consists of approximately 2.8 km of pipe and 46 grinder pump assemblies. The developed areas in the North Clyde extension area are sparse and widely scattered resulting in the high length of pipe required. Additionally, the conceptual design focussed on reducing the number of road crossings required. Therefore, the total pipe length may be reduced by increasing the number of road crossings, however the cost benefits gained from this will require further investigation and falls beyond the scope of this assessment.

Figure 4.14
North Clyde Extension Area - Low Pressure Sewage System



4.5 Kopu Road Reticulated Extension Area

4.5.1 Gravity Conveyance

In lieu of the capacity constraints at Kopu pumpstation (refer to Section 4.3), it is recommended to connect to the existing sewage reticulation along the trunk gravity pipeline that discharges into Fitzroy pumpstation. Three connection points to the existing sewage reticulation is recommended for the gravity conveyance option at the Kopu Road extension area. The multiple connection points to the existing gravity trunk pipe reduces the maximum length of proposed pipes. This inherently reduces the total height differential required between the connection point and manhole furthest upstream from the connection. The relationship between the maximum pipe length and height differential is a result of the required pipe slope / grade.

The flattest pipe grade, recommended in Council's Engineering Code of Practice, for a 150 mm internal diameter pipe is a grade of approximately 0.7 percent. Based on the flat terrain, length of pipe required to service the Kopu Road extension area and the minimum allowable pipe slopes, the area will be unable to gravitate freely and discharge into the existing sewage reticulation. However, the extension area is able to gravitate based on a pipe grade of 0.5 percent. The flatter pipe slope will result in reduced flow velocities which may cause settlement and blockages. To mitigate these effects, regular maintenance and scheduled pipe flushing will be required for the proposed gravity system in the Kopu Road extension area. An indicative conceptual layout of the extended gravity sewage reticulation is shown in Figure 4.15.

Figure 4.15
Kopu Road Extension Area - Gravity System

4.5.2 Low Pressure Sewer Conveyance

An alternative to the gravity conveyance option is available in the form of a low pressure sewage system. As no gravity is required to operate the system, low pressure sewage systems are not as susceptible to settlement and blockages. Low pressure sewage systems are reliable and have the potential to be expanded as the demand increases.

An indicative low pressure sewage system conceptual design was completed for the Kopu Road extension area. A schematic of the proposed system is shown in Figure 4.16. Two connections have been proposed to the existing sewage reticulation, however, the layout can be reconfigured to only allow for one connection point.

Figure 4.16
Kopu Road Extension Area - Low Pressure Sewage System

5 FINDINGS AND RECOMMENDATIONS

5.1 Summary of Enhancement Options and the Net Effect

This section summarises the system optimisation achieved, in terms of spillage reduction, through the mitigatory options proposed in this assessment. As mentioned previously, to facilitate system-wide feasible solutions, mitigatory adjustments were incorporated in a step-wise manner. Therefore, proposed precursory optimisation adjustments to the hydraulic model were maintained, whilst investigating additional system improvement options.

The system optimisation gained through the adjustments proposed and made to the hydraulic model (for the assessment period), up to and including Option 3b, are summarised in Table 5.1.

Table 5.1
Summary of System Improvement Options

System Improvement	Reduced Overflow Spillage (m^3)*	Comments
• Installation of new rising main from Alexander Park pumpstation to discharge manhole • Alexander Park pumpstation's pumping capacity increased by 50 percent	4,800	The spillage reduction through incorporation of rising main alone was not quantified, however, North Clyde and Alexander Park pumpstation will no longer affect one another's maximum pumping capacity. Pumps at Alexander pumpstation will need to be replaced with two Flygt 3127 submersible pumps to achieve an increased pumping capacity of 50 percent
• Installation of duplicate rising main at Fitzroy pumpstation	7,160	Based on the asset data received, the existing rising main from Fitzroy to the treatment ponds is an Asbestos Cement pipe. If this is the case two new rising mains of 300 mm will need to be installed **
• Installation of new rising main from Kopu pumpstation to Pilot Hills Treatment Pond	48,000	Spillage eliminated from Kopu pumpstation's existing discharge chamber
• Kopu pumpstation's pumping capacity increased by 50 percent	30,000	Will require installation of three new pumps
Total	**89,960**	Equivalent to **spillage reduction of approximately 33 percent** based on total spillages from baseline hydraulic model

** During selected assessment period*

*** Discussed in Section 5.2*

The 33 percent reduction, as indicated in Table 5.1, is based on the hydraulic model for the assessment period between 1 January 2016 and 14 March 2017. During this period, there was one rainfall event (on 28 January 2016) equivalent to a 1 in 20 year rainfall event based on a 12 hour duration / time of concertation and multiple smaller concurrent rainfall events. The smaller concurrent events appeared to be more straining on the sewage reticulation than the isolated higher intensity rainfall event. The spillage reduction percentage is dependent on the rainfall and higher reductions are possible during future events.

As mentioned in Section 4, this assessment focused on increasing system efficiency through conveyance system augmentation and operational optimisation. Further viable reductions are possible through implementation of Council's Inflow & Infiltration management strategies followed by potential opportunities to investigate installation of attenuation facilities.

5.2 Options Costing and Risk Assessment

5.2.1 System Enhancement Options

Although not always the case, the risk is often inversely proportional to the cost associated to implement a proposed enhancement to a system. This holds true for some of the mitigatory options proposed in this assessment.

In the case of North Clyde, Alexander Park and Fitzroy pumpstations, through the implementation of relatively inexpensive solutions in conjunction with effective inflow and infiltration management, the risks associated with spillages can be significantly reduced. Whereas significant spillage reductions from Kopu pumpstation could be achieved, through the implementation of the proposed system enhancements in conjunction with effective inflow and infiltration management, the risks associated with spillage may be further reduced by increasing the pump capacity for instance. However, further increasing the pumpstation's capacity (beyond the proposed increase of 50 percent) is considered premature at this stage, and this should only be considered once the inflow and infiltration management strategy has successfully been implemented. A successfully implemented inflow and infiltration strategy may reduce peak flows (maximum wet weather flow) to be within plus/minus eight times larger than average flow (average dry weather flow).

As summarised in Table 5.1, the proposed mitigatory enhancement options include upgrading the pumps in Alexander Park and Kopu pumpstations. In order to estimate indicative ballpark costs associated with the mitigatory options, a preliminary pump sizing assessment was conducted. This consisted of calculation of new system duty points which required the calculation of the system curves and more specifically the static and frictional head losses at the required flowrates.

Upon calculating each system's duty points, Xylem's online pump selection software called Xylect was used to select appropriately sized pumps. A export datasheet for each of the pumps selected is available in Appendix D. The duty points and pumps selected for each pumpstation is shown in Table 5.2.

Table 5.2
Pump Selection

Pumpstation	Duty Point		Selected Pumps
	Flow Rate (ℓ/s)	Dynamic Head (m)	
Alexander Park	38	10.1	Flygt NP3127 HT 489
Kopu	90	36.0	Flygt NP3202 HT 456

Based on experience, PVC-O pipes are recommended as the suitable pipe material for the proposed installation of rising mains. In comparison to polyethylene (PE) and other pipe materials, polyvinyl chloride (PVC) is less susceptible to internal biofilm growth and fat build-up, which results in loss of capacity.

To establish indicative costs, rates were adopted from similar work, particularly those where similar groundwater conditions were encountered. Based on the rates adopted, the estimated costs associated to implementing each of the mitigatory options proposed, is summarised in Table 5.3.

Table 5.3
Proposed Mitigatory Options' Indicative Ballpark Costs

Item Description	Rate Range*		Unit	Quantity	Total excl. GST	
	Low	High			Low	High
DN200 PVC-O, new rising main between Alexander Park pumpstation and discharge manhole	$425	$550	m	410	$174,250	$225,500
Supply and Installation of Flygt NP3127 HT 489 at Alexander Park pumpstation	$7,930	$9,725	No	2	$15,860	$19,450
DN300 PVC-O, replace and duplicate rising main between Fitzroy pumpstation and Pilot Hill Treatment Ponds	$485	$600	m	2 x 600	$582,000	$720,000
DN300 PVC-O new rising main between Kopu pumpstation and Pilot Hill Treatment Ponds	$485	$600	m	2600	$1,261,000	$1,560,000
Supply and Installation of Flygt NP3202 HT 456 at Kopu pumpstation	$27,920	$34,260	No	3	$83,760	$102,780
Total					**$2,116,870**	**$2,627,730**

** Indicative rates include the supply and installation of items*

5.2.2 System Extension Options

In a similar manner as discussed in Section 5.2.1, the rates used for the proposed gravity system in the Kopu Road extension area were adopted from national databases of similar work. The rates include allowances for dewatering and trenching as a high water table is expected in close vicinity to the Wairoa River. In a similar fashion rates for the proposed low pressure sewage systems were adopted from the rates used in a previous project.

The rates proposed for pipes are conservative and have in certain instances been escalated to make provision for miscellaneous items such as bends, isolation valves, air valves, etc. Additionally, the rates used include the supply of materials and costs associated to install the proposed reticulation systems. Quantities were extracted from the conceptual designs (refer to Section 4.4 and 4.5) and the indicative ballpark costs associated with supply and installation of the proposed reticulation extensions are summarised in Table 5.4.

Table 5.4
Proposed Extension Option(s) Indicative Ballpark Costs

Item Description	Rate Range*		Unit	Quantity	Total excl. GST	
	Low	High			Low	High
North Clyde Extension Area - Low Pressure Sewage System						
Directional Drill Supply and Install PE 100 SDR 11 DN50 Pipe, complete with required valves and bends	$200	$250	m	2,770	$554,000	$692,500
Supply and Install Grinder Pump Assembly, complete with connection to existing septic system and connection to main branch	$9,800	$10,700	No.	50	$490,000	$535,000
North Clyde Extension Area - Low Pressure Sewage System Total					**$1,044,000**	**$1,227,500**
Kopu Road Extension Area - Gravity Sewage System						
Installed DN150 PVC pipe complete, including piping, sheet piling, trenching, dewatering, laying and bedding, bedding material, and backfill	$250	$300	m	1,380	$345,000	$414,000
Manholes complete with all installation requirements	$3,800	$4,500	No.	21	$79,800	$94,500
Kopu Road Extension Area - Gravity Sewage System Total*					**$424,800**	**$508,500**

Item Description	Rate Range*		Unit	Quantity	Total excl. GST	
	Low	High			Low	High
Kopu Road Extension Area - Low Pressure Sewage System						
Directional Drill Supply and Install PE 100 SDR 11 DN50 Pipe, complete with required valves and bends	$200	$250	m	1,270	$254,000	$317,500
Supply and Install Grinder Pump Assembly, complete with connection to existing septic system and connection to main branch	$9,800	$10,700	No.	18	$176,400	$192,600
Kopu Road Extension Area - Low Pressure Sewage System Total					**$430,400**	**$510,100**

* *The indicative costs for the gravity system does not account for the additional maintenance and flushing required*

5.3 Recommendations and Conclusions

The following recommendations and conclusions can be drawn from this assessment:

1. Based on outputs received from the calibrated hydraulic model, the mitigatory options proposed were able to reduce spillages by 33 percent during the assessment period. Additional reductions may be possible through the implementation of an Inflow and Infiltration management programme.
2. After conducting a detailed design for each of the proposed mitigatory options, it is recommended to implement the mitigatory options in the following order:
   i. Installation of Alexander Park pumpstation's new rising main;
   ii. Installation of new and replacement of Fitzroy pumpstation's old asbestos cement rising main, if required (i.e. installation of two PVC-O rising mains of DN300);
   iii. Replacement of pumps at Alexander Park pumpstation with two Flygt NP3127 HT 489;
   iv. Installation of Kopu pumpstation's new rising main to Pilot Hill Treatment Ponds; and
   v. Replacement of pumps at Kopu pumpstation with three Flygt NP3202 HT 456.
3. Investigate the effects of the additional flows reporting to the Pilot Hills Treatment Ponds.
4. If gravity conveyance is the preferred conveyance option for the Kopu Road extension area, preliminary designs need to take the undeveloped properties into account to allow for potential future growth and further expansions of the reticulation.
5. The existing reticulated system will have adequate capacity to accommodate the additional flows from the extension areas. Based on previous studies (Truebridge Callender Beach Ltd, 1996), the anticipated average flows from all the extensions combined will be less than 1 ℓ/s.
6. A low pressure sewage system is recommended for the Kopu Road extension area due to the relatively small price difference in comparison to the gravity system. It is anticipated that a low pressure system will require less maintenance and the system can easily be extended to accommodate additional flows arising from future developments.

6 REFERENCES

LINZ Data Service (a). (2011). *Land Information New Zealand*. Retrieved from LINZ Data Service: https://data.linz.govt.nz/layer/2324-nz-topo250-gridless-maps/ and licensed by LINZ for re-use under the Creative Commons Attribution 3.0 New Zealand licence.

LINZ Data Service (b). (2010-2011). *Land Information New Zealand*. Retrieved from LINZ Data Service: https://data.linz.govt.nz/layer/1778-hawkes-bay-04m-rural-aerial-photos-2010-2011/ and licensed by LINZ for re-use under the Creative Commons Attribution 3.0 New Zealand licence.

Opus International Consultants. (2012). Wairoa Waste Water Modelling. *Stage 3 - Detailed Wastewater Network Model*.

Statistics New Zealand. (2013). *2013 Census QuickStats about a place: Wairoa District*. Retrieved from Stats NZ: http://www.stats.govt.nz/Census/2013-census/profile-and-summary-reports/quickstats-about-a-place.aspx?request_value=14021&tabname=

Truebridge Callender Beach Ltd. (1996). *Wairoa Distict Council - Sewage Extension Assessment.*

Wairoa District Council. (1999). *Engineering Code of Practice.* WDC.

Xylem. (2017, August 26). *Product Configuration*. Retrieved from Xylect: http://www.xylect.com/bin/Xylect.dll?RQID=pu_prodsel-5271130055090163

Appendix A

MANHOLE LID AND INVERT ELEVATIONS

Asset_ID	Invert_Lev	Lid_Level
SNK2055	3.840	5.510
SNK0005	0.046	0
SNF0284	Manhole not found on site	
SLK1250	0.471	4.021
SMA0030	2.308	5.878
SMA0050	3.751	5.351
SMA0060	2.956	5.656
SMA0080	Manhole not found on site	
SMA0110	5.024	6.354
SMA0200	2.857	5.487
SMA0310	3.578	4.948
SMA0330	4.125	5.925
SMA0420	4.739	6.039
SMA0480	2.104	4.554
SMA0510	2.332	4.892
SMA0580	2.669	5.739
SMA0600	4.053	6.283
SMA0690	5.680	6.880
SMA0710	3.357	5.647
SMA0740	4.189	4.939
SMA0840	5.657	7.207
SMA0900	Manhole not found on site	
SMF0030	-1.076	3.654
SMF0060	-0.627	3.353
SMF0100	-0.315	3.605
SMF0120	-0.430	3.610
SMF0200	0.055	3.435
SMF0210	0.564	3.634
SMF0220	1.016	3.836
SMF0230	1.433	3.713
SMF0240	1.718	4.031
SMF0250	Welded	3.740
SMF0260	2.368	3.598
SMF0270	2.682	4.142
SMF0280	2.978	4.528
SMF0310	0.712	3.902

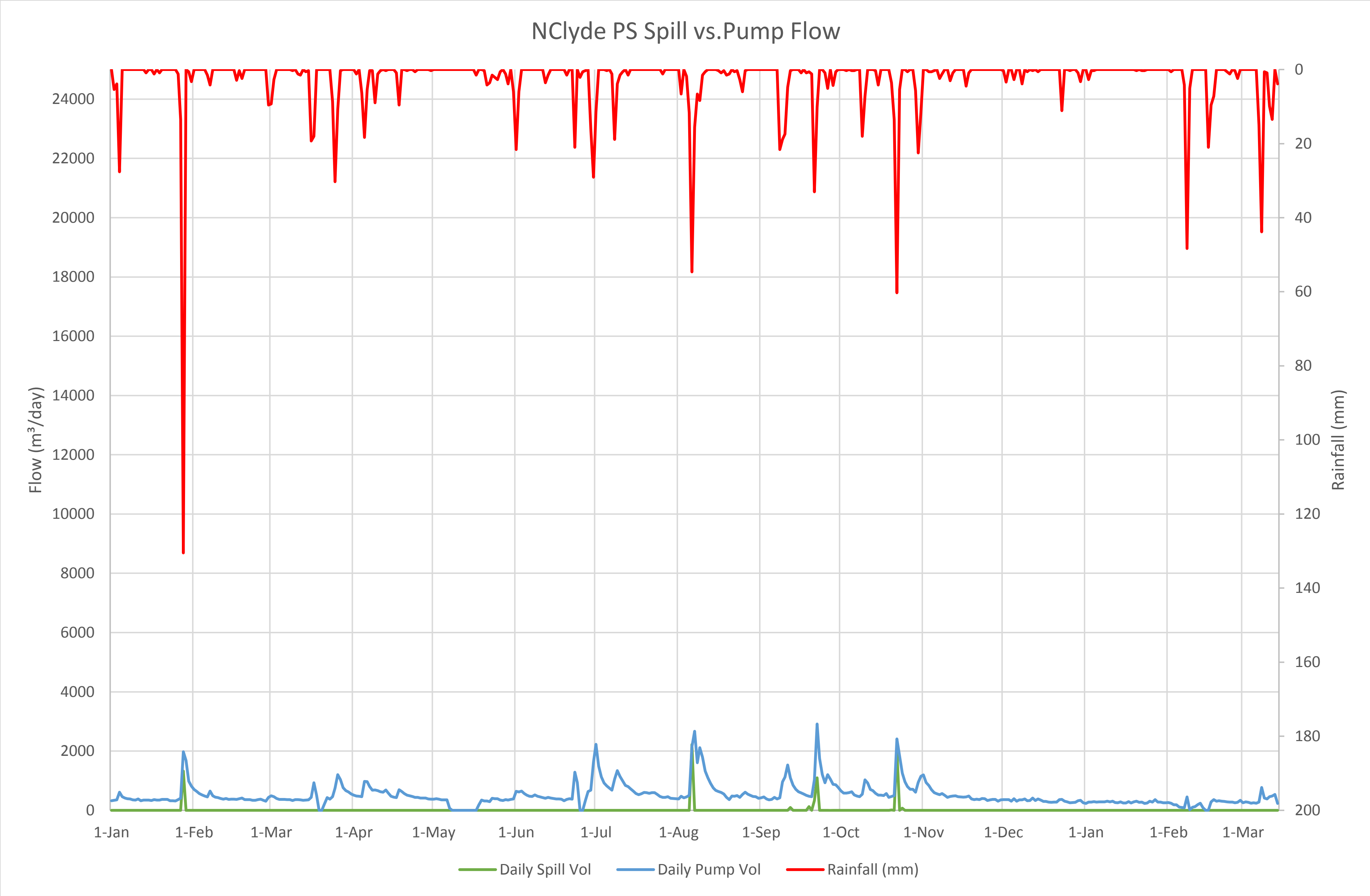
Asset_ID	Invert_Lev	Lid_Level
SMF0360	0.726	4.066
SMF0370	Manhole not found on site	
SMF0460	0.801	4.001
SMF0470	1.071	4.071
SMF0480	1.612	4.612
SMF0520	2.644	4.884
SMF0570	3.25	5.110
SMF0600	1.388	4.148
SMF0630	1.942	5.192
SMF0670	2.728	4.808
SMF0720	2.959	4.959
SMF0740	3.365	5.295
SMF0850	5.117	6.507
SMF0860	16.436	0
SMK0020	0.397	3.547
SMK0030	0.390	3.550
SMK0130	1.012	3.272
SMK0190	1.809	3.909
SMK0220	2.889	4.389
SMK0290	0.69	3.280
SMK0310	1.471	4.541
SMK0360	3.395	5.045
SMK0380	1.709	4.469
SMK0390	1.832	4.632
SMK0430	1.984	4.574
SMK0470	3.833	4.953
SMK0490	2.332	4.792
SMK0510	2.841	4.761
SMK0590	3.277	5.257
SMK0670	3.059	5.009
SMK0700	3.476	5.326
SMK0780	0.131	4.451
SMK0800	0.658	4.198
SMK0810	1.005	3.865
SMK0860	2.828	3.618
SMK0870	0.386	4.106

Asset_ID	Invert_Lev	Lid_Level
SMK0910	1.089	4.489
SMK0940	1.864	4.199
SMK0990	2.952	4.552
SMK1020	2.379	4.439
SMK1060	Private Property	
SMK1080	1.034	3.684
SMK1120	2.659	3.729
SMK1130	1.320	3.620
SMK1140	1.576	3.796
SMK1160	Manhole not found on site	
SMK1200	2.080	3.710
SMK1220	2.501	3.651
SMK1260	0.601	4.151
SMK1280	Manhole not found on site	
SMK1330	2.732	4.402
SMK1370	0.931	4.281
SMK1390	Manhole not found on site	
SMK1550	1.163	3.963
SMK1620	2.062	4.362
SMK1640	1.363	4.143
SMK1660	2.737	4.477
SMK1720	2.062	4.402
SMK1790	3.032	4.602
SMK1870	2.570	4.730
SMK1930	Manhole not found on site	
SMK1950	1.304	3.964
SMK2000	1.892	4.252
SMK2110	2.265	4.945
SMK2170	4.016	5.016
SMK2230	2.848	5.898
SMK2300	3.764	5.044
SMK2310	3.646	6.166
SMK2350	5.354	6.534
SMN0031	Manhole not found on site	
SMN0040	1.912	5.912
SMN0170	4.504	6.604

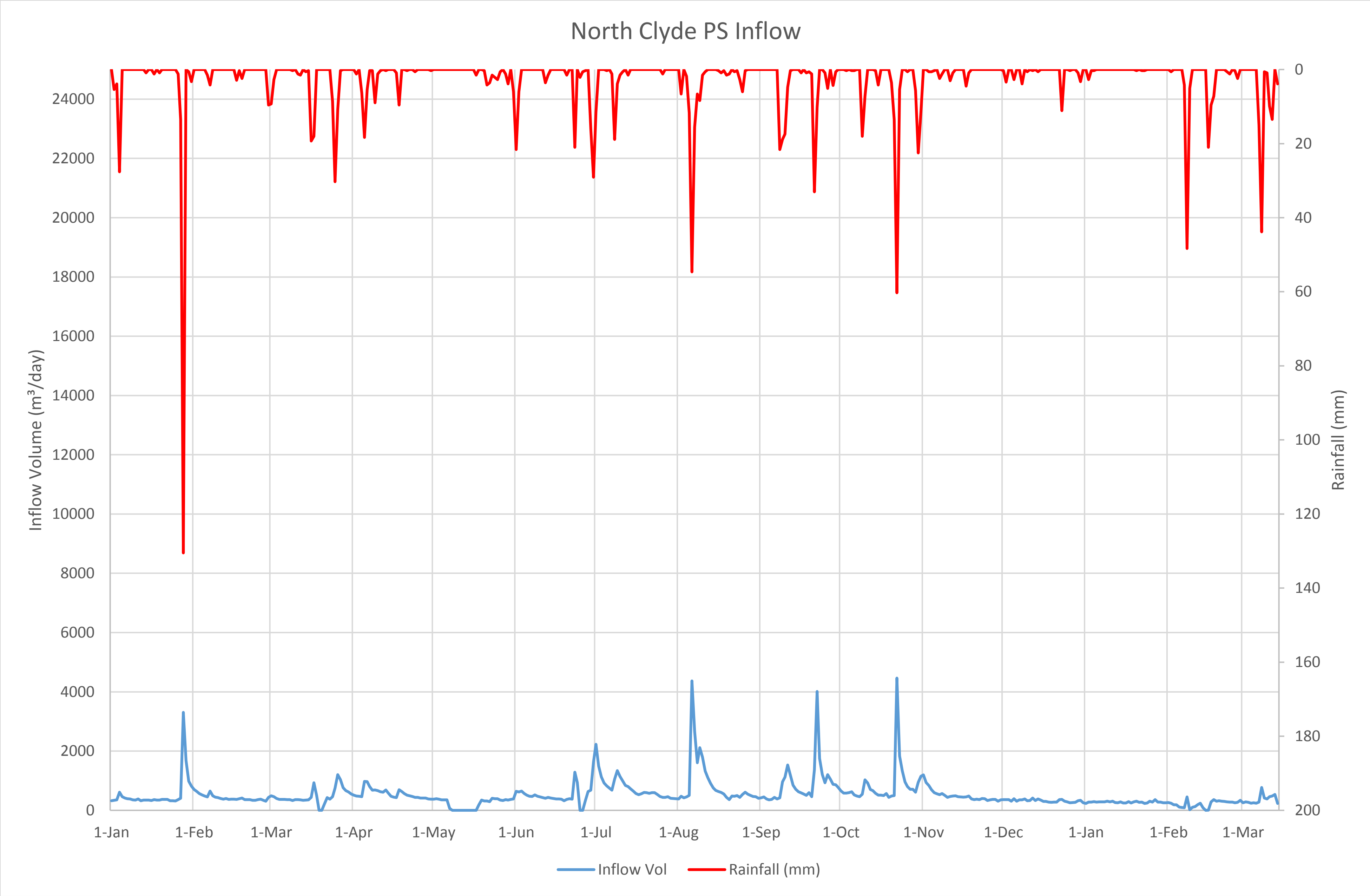
Asset_ID	Invert_Lev	Lid_Level
SMN0340	2.099	6.259
SMN0350	Manhole not found on site	
SMN0400	5.092	6.442
SMN0420	Manhole not found on site	
SMN0440	2.610	5.990
SMN0460	2.710	5.990
SMN0490	Manhole not found on site	
SMN0520	Manhole not found on site	
SMN0540	4.768	7.228
SMN0570	Manhole not found on site	
SMN0580	5.213	6.983
SMN0590	3.008	6.268
SMN0620	Manhole not found on site	
SMN0630	Manhole not found on site	
SMN0660	4.276	7.116
SMN0670	Manhole not found on site	
SMN0680	Manhole not found on site	
SMN0690	Manhole not found on site	
SMN0700	Manhole not found on site	
SMN0710	Manhole not found on site	
SMN0730	Manhole not found on site	
SMN0750	Manhole not found on site	
SMN0760	5.548	7.018
SMN0780	Manhole not found on site	
SMN0800	Manhole not found on site	
SMO0030	-0.184	3.806
SMO0060	-0.211	3.679
SMO0070	0.741	3.811
SMO0100	Manhole not found on site	
SMO0120	Manhole not found on site	
SMR0030	Manhole not found on site	
SNN0010	0.456	0
SMA0910	0.814	3.874
SMO0124	6.694	10.664
SIP0011	Manhole not found on site	
SBJ0011	Manhole not found on site	

Appendix B

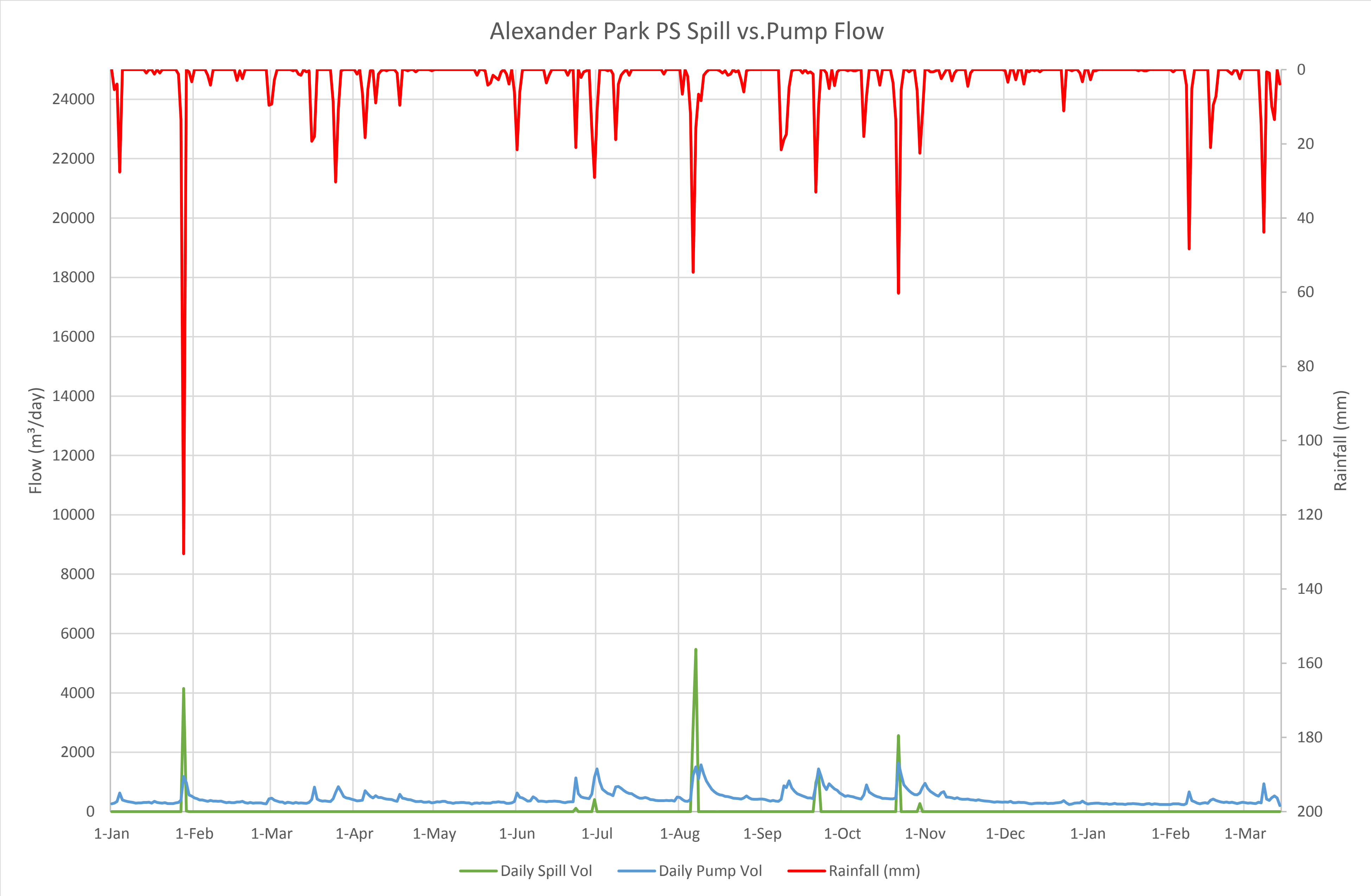
CALCULATED SPILLAGE VOLUMES FROM PUMPSTATIONS



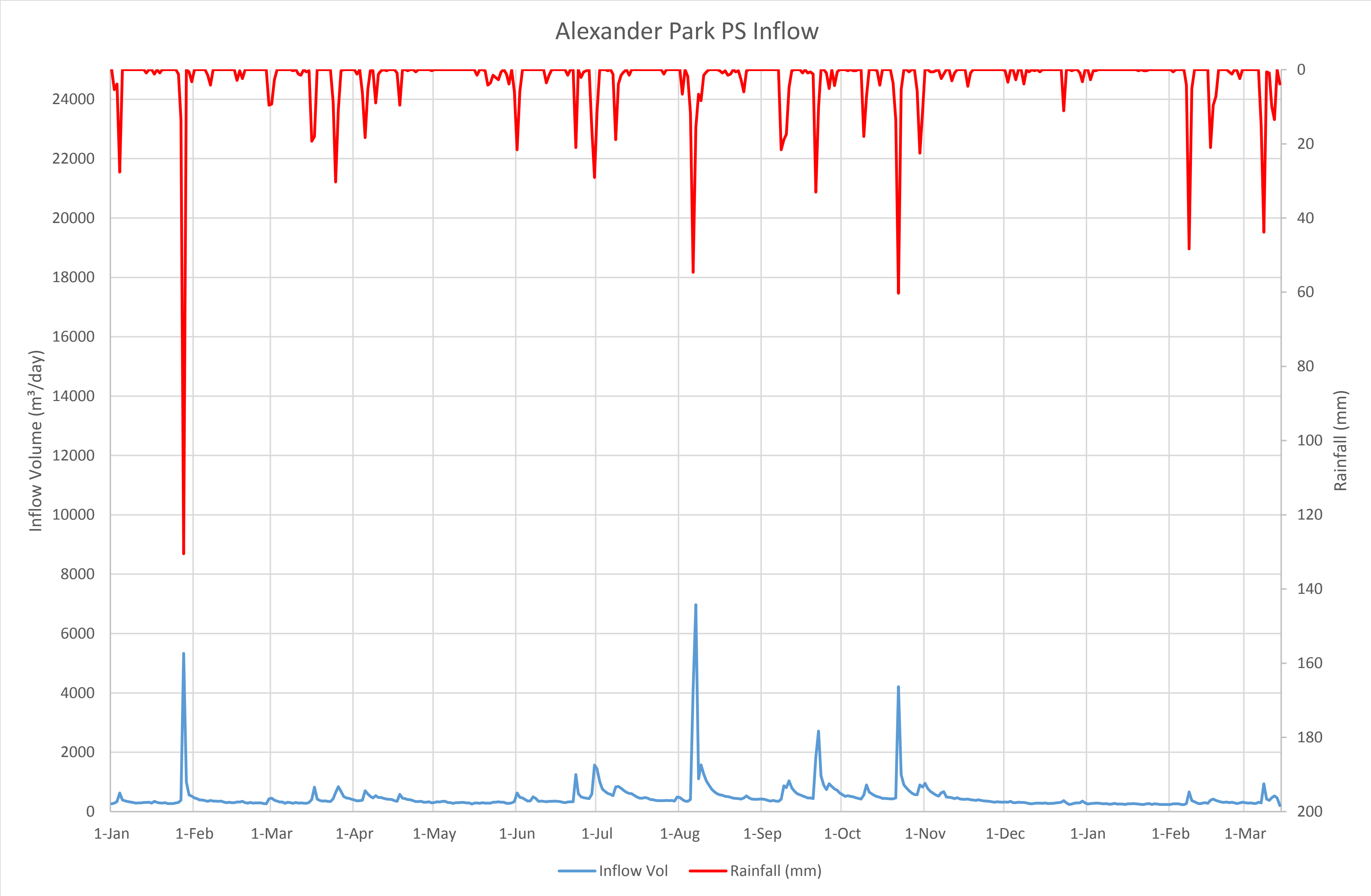
NClyde PS Spill vs.Pump Flow
Flow (m³/day)
Rainfall (mm)
24000
22000
20000
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
0
20
40
60
80
100
120
140
160
180
200
1-Jan
1-Feb
1-Mar
1-Apr
1-May
1-Jun
1-Jul
1-Aug
1-Sep
1-Oct
1-Nov
1-Dec
1-Jan
1-Feb
1-Mar
Daily Spill Vol
Daily Pump Vol
Rainfall (mm)



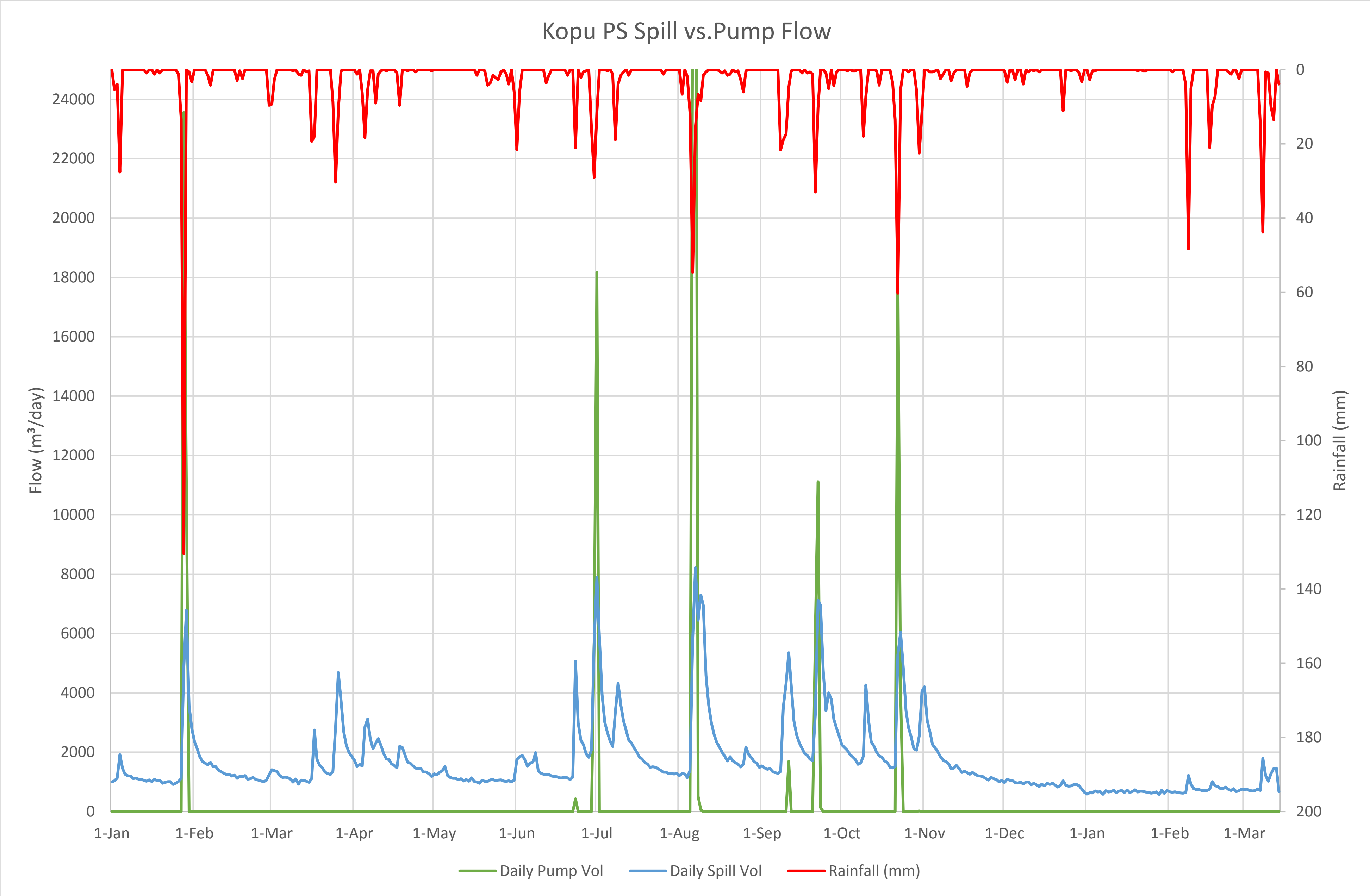
North Clyde PS Inflow
Inflow Volume (m³/day)
Rainfall (mm)
24000
22000
20000
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
20
40
60
80
100
120
140
160
180
200
1-Jan
1-Feb
1-Mar
1-Apr
1-May
1-Jun
1-Jul
1-Aug
1-Sep
1-Oct
1-Nov
1-Dec
1-Jan
1-Feb
1-Mar
Inflow Vol
Rainfall (mm)

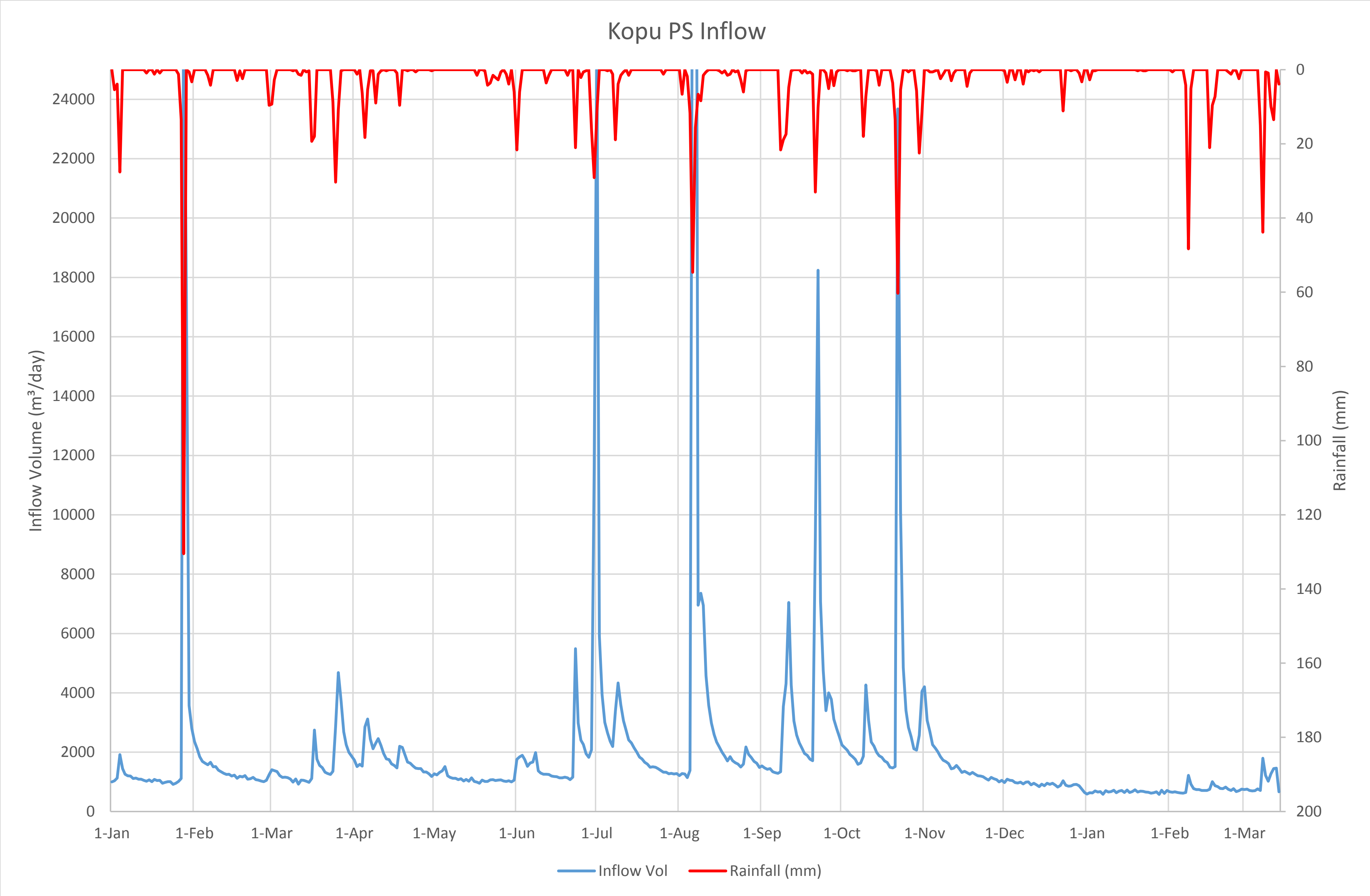


Alexander Park PS Spill vs.Pump Flow
Flow (m³/day)
Rainfall (mm)
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000
22000
24000
0
20
40
60
80
100
120
140
160
180
200
1-Jan
1-Feb
1-Mar
1-Apr
1-May
1-Jun
1-Jul
1-Aug
1-Sep
1-Oct
1-Nov
1-Dec
1-Jan
1-Feb
1-Mar
Daily Spill Vol
Daily Pump Vol
Rainfall (mm)

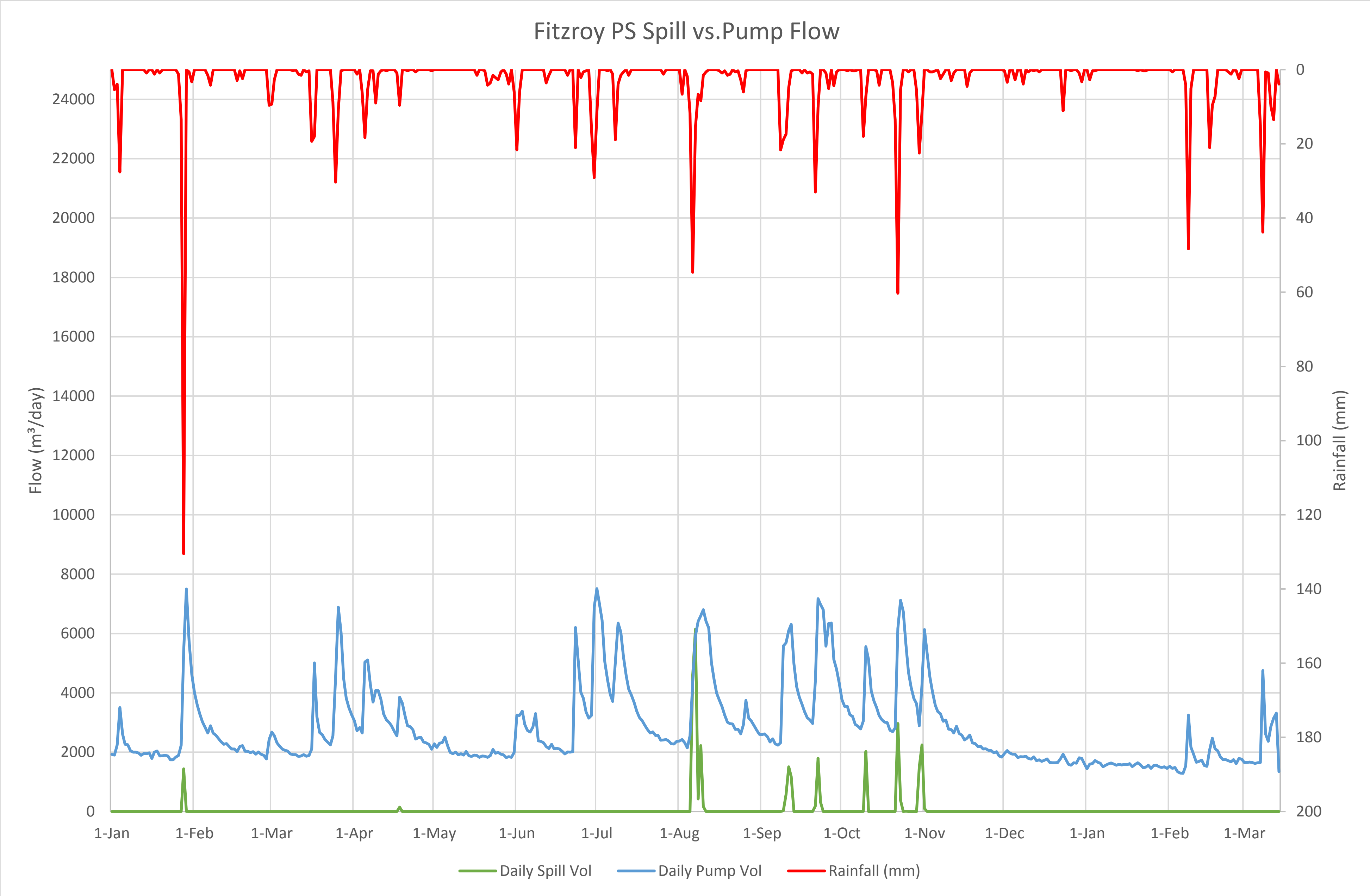


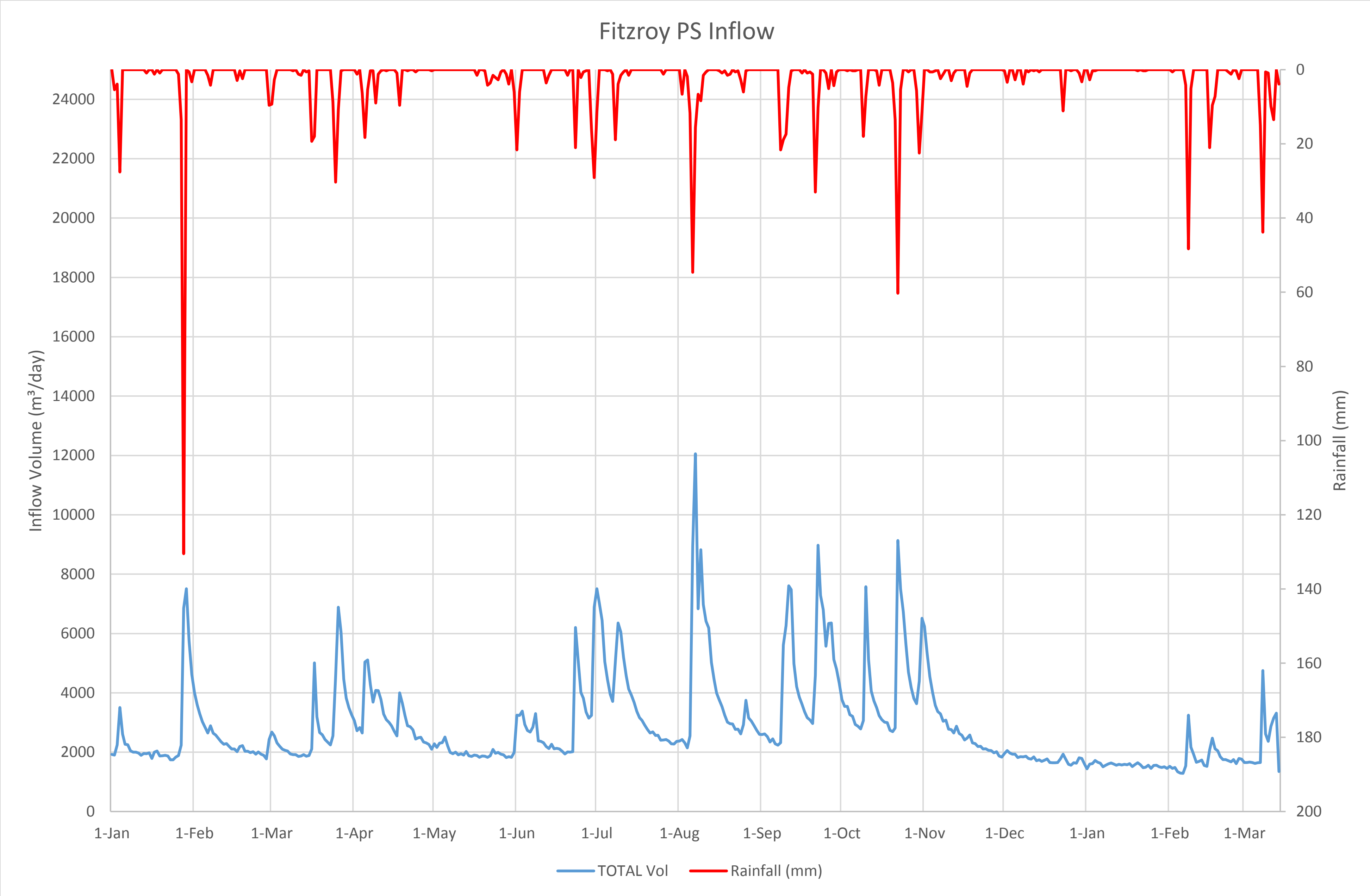
Alexander Park PS Inflow
Inflow Volume (m³/day)
Rainfall (mm)
24000
22000
20000
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
0
20
40
60
80
100
120
140
160
180
200
1-Jan
1-Feb
1-Mar
1-Apr
1-May
1-Jun
1-Jul
1-Aug
1-Sep
1-Oct
1-Nov
1-Dec
1-Jan
1-Feb
1-Mar
Inflow Vol
Rainfall (mm)


Kopu PS Spill vs.Pump Flow
Flow (m³/day)
Rainfall (mm)
24000
22000
20000
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
0
20
40
60
80
100
120
140
160
180
200
1-Jan
1-Feb
1-Mar
1-Apr
1-May
1-Jun
1-Jul
1-Aug
1-Sep
1-Oct
1-Nov
1-Dec
1-Jan
1-Feb
1-Mar
Daily Pump Vol
Daily Spill Vol
Rainfall (mm)



Kopu PS Inflow
Inflow Volume (m³/day)
Rainfall (mm)
24000
22000
20000
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
0
20
40
60
80
100
120
140
160
180
200
1-Jan
1-Feb
1-Mar
1-Apr
1-May
1-Jun
1-Jul
1-Aug
1-Sep
1-Oct
1-Nov
1-Dec
1-Jan
1-Feb
1-Mar
Inflow Vol
Rainfall (mm)

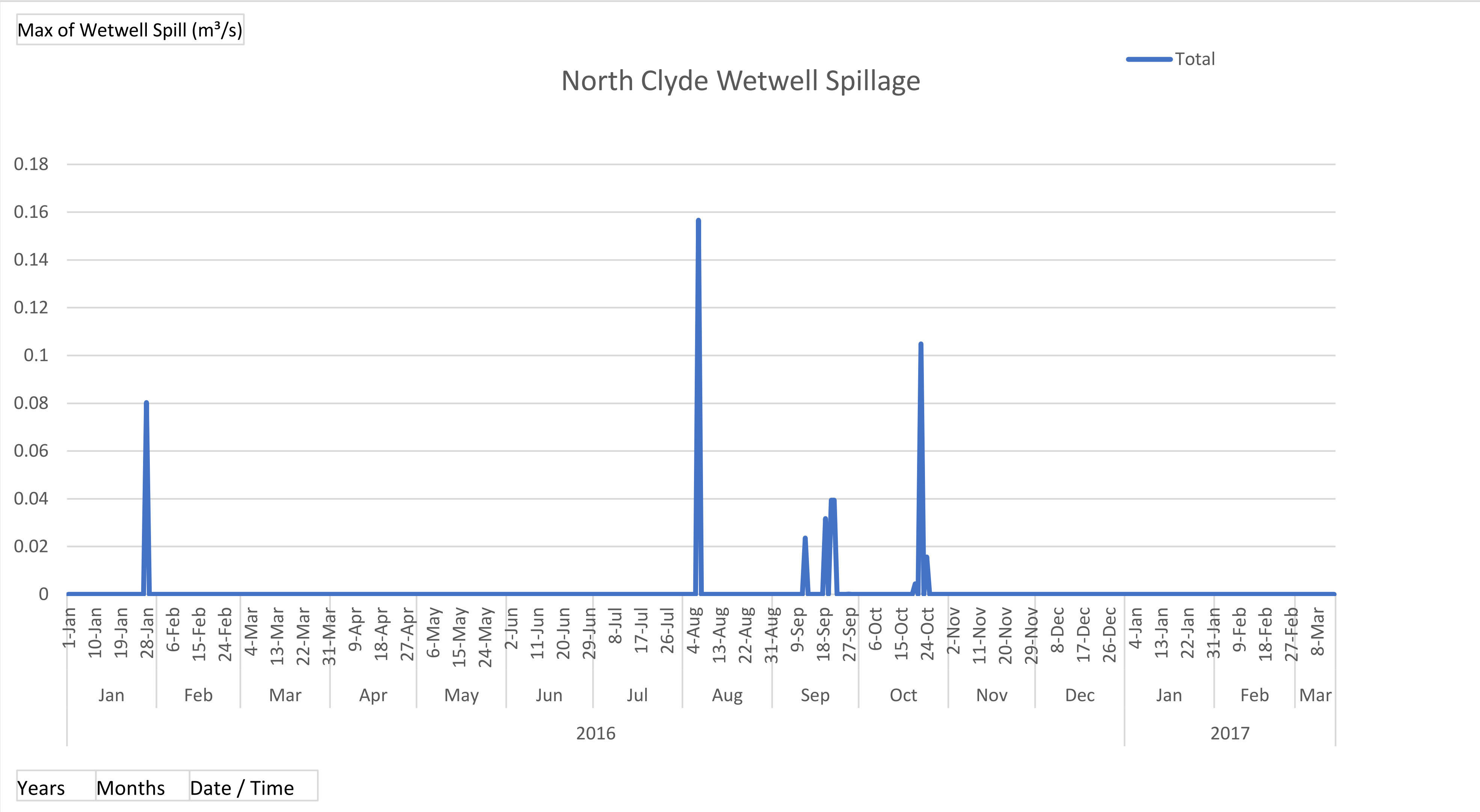

Fitzroy PS Spill vs.Pump Flow
Flow (m³/day)
Rainfall (mm)
24000
22000
20000
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
0
20
40
60
80
100
120
140
160
180
200
1-Jan
1-Feb
1-Mar
1-Apr
1-May
1-Jun
1-Jul
1-Aug
1-Sep
1-Oct
1-Nov
1-Dec
1-Jan
1-Feb
1-Mar
Daily Spill Vol
Daily Pump Vol
Rainfall (mm)



Fitzroy PS Inflow
Inflow Volume (m³/day)
Rainfall (mm)
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000
22000
24000
0
20
40
60
80
100
120
140
160
180
200
1-Jan
1-Feb
1-Mar
1-Apr
1-May
1-Jun
1-Jul
1-Aug
1-Sep
1-Oct
1-Nov
1-Dec
1-Jan
1-Feb
1-Mar
TOTAL Vol
Rainfall (mm)

Appendix C

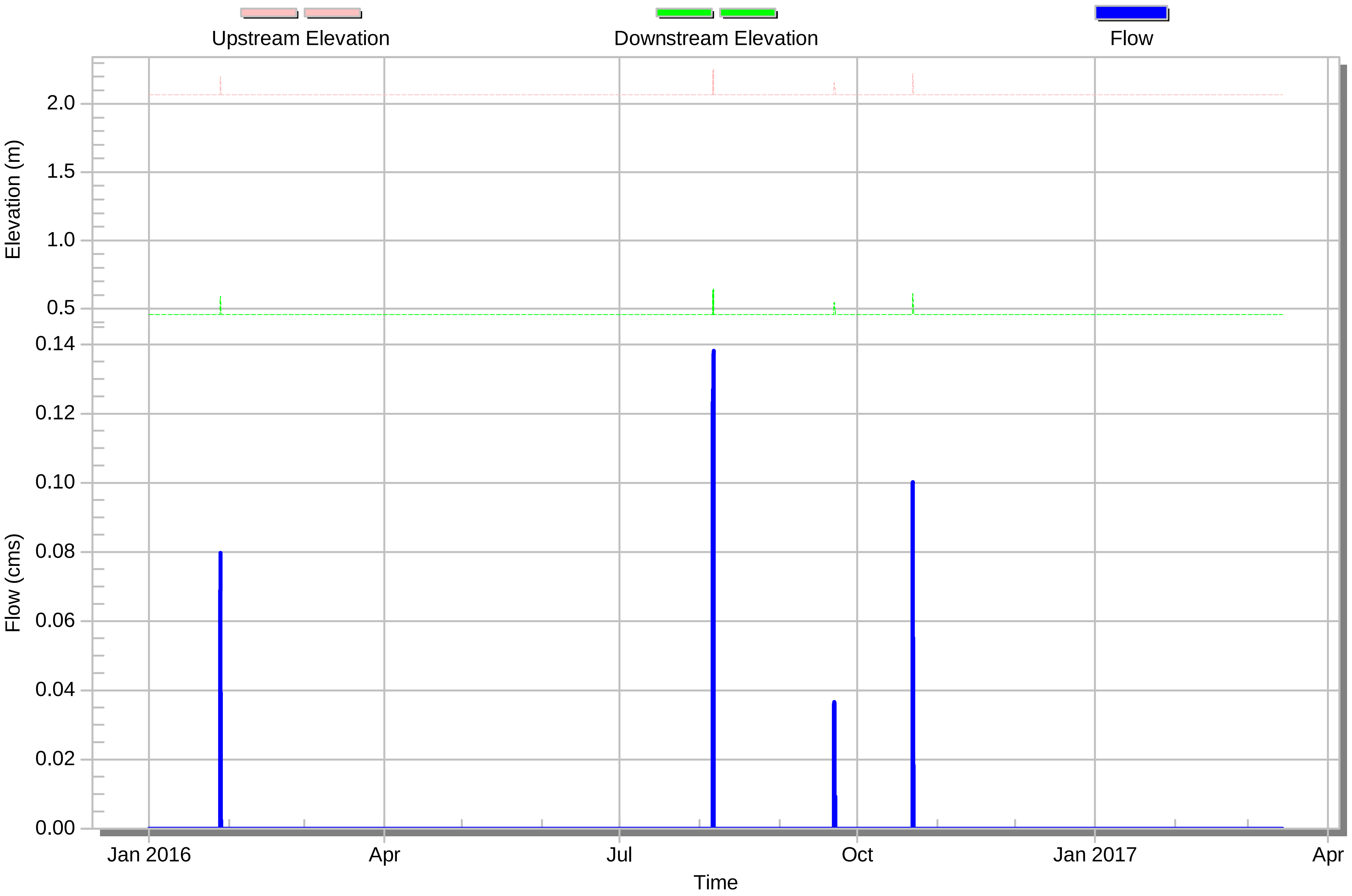
MODELLED SPILLAGES VS. CALCULATED SPILLAGES

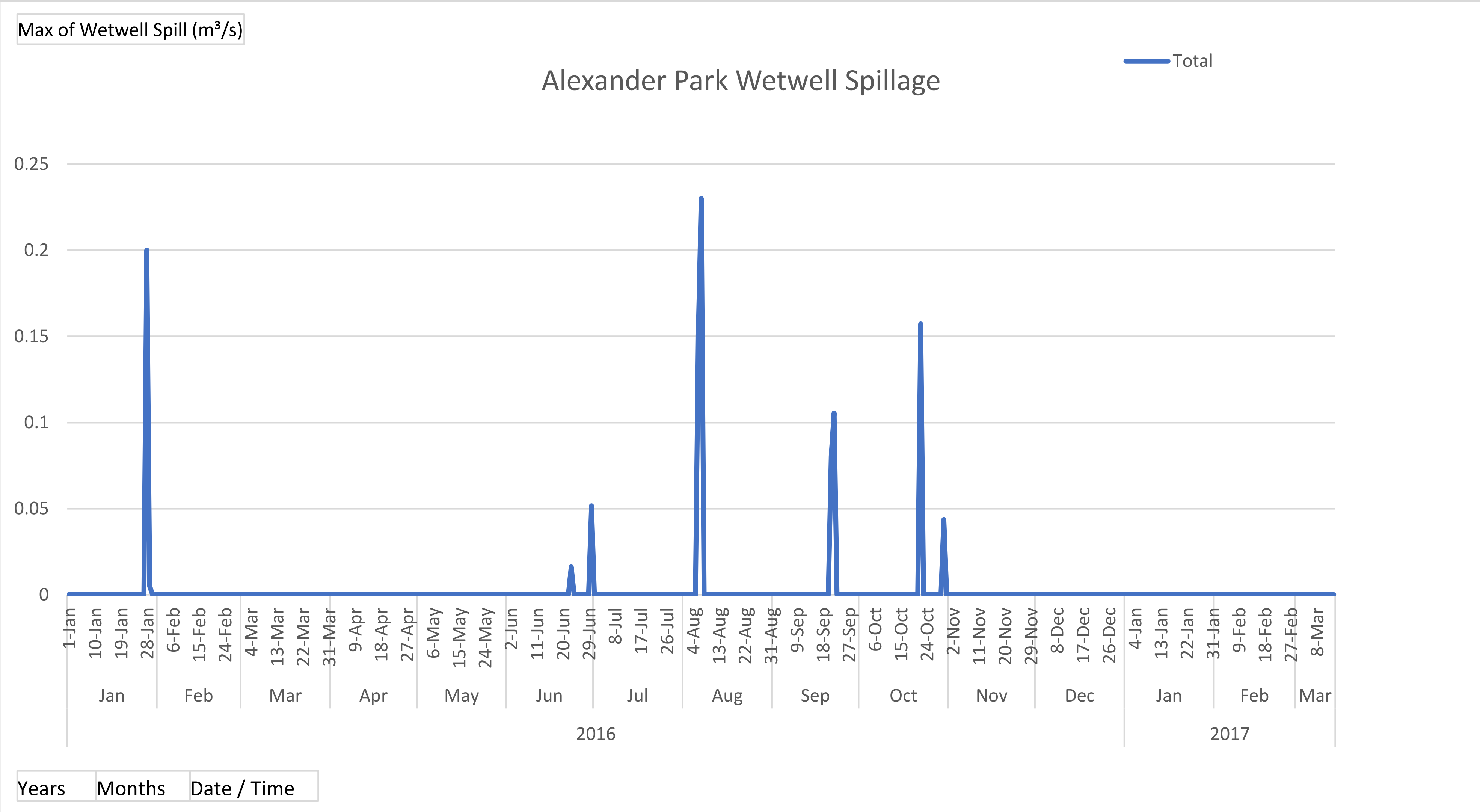


Max of Wetwell Spill (m³/s)
North Clyde Wetwell Spillage
Total
0.18
0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
1-Jan
10-Jan
19-Jan
28-Jan
6-Feb
15-Feb
24-Feb
4-Mar
13-Mar
22-Mar
31-Mar
9-Apr
18-Apr
27-Apr
6-May
15-May
24-May
2-Jun
11-Jun
20-Jun
29-Jun
8-Jul
17-Jul
26-Jul
4-Aug
13-Aug
22-Aug
31-Aug
9-Sep
18-Sep
27-Sep
6-Oct
15-Oct
24-Oct
2-Nov
11-Nov
20-Nov
29-Nov
8-Dec
17-Dec
26-Dec
4-Jan
13-Jan
22-Jan
31-Jan
9-Feb
18-Feb
27-Feb
8-Mar
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Jan
Feb
Mar
2016
2017
Years
Months
Date / Time

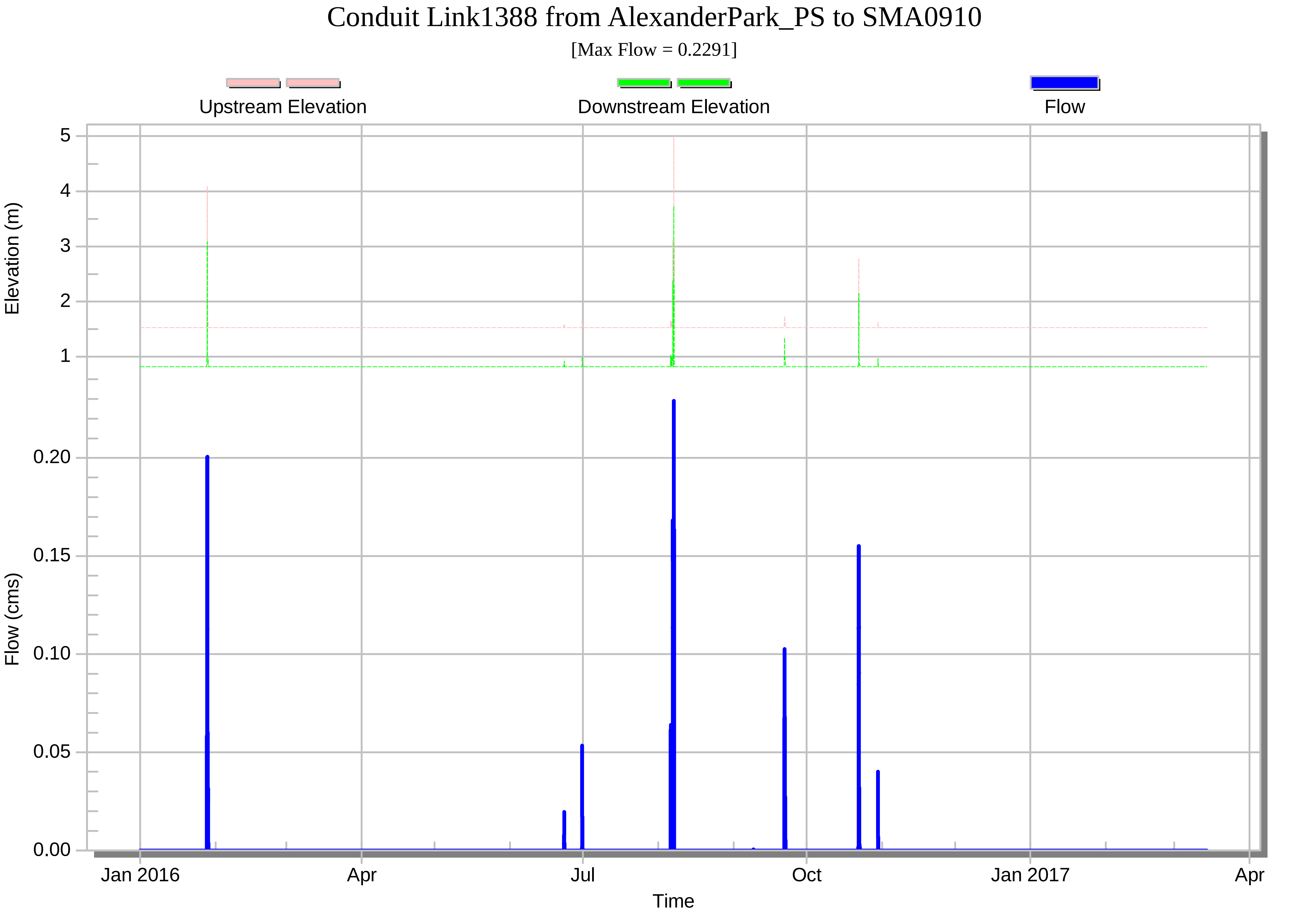
Conduit Link1383 from NorthClyde_PS to SNN0010

[Max Flow = 0.1381]

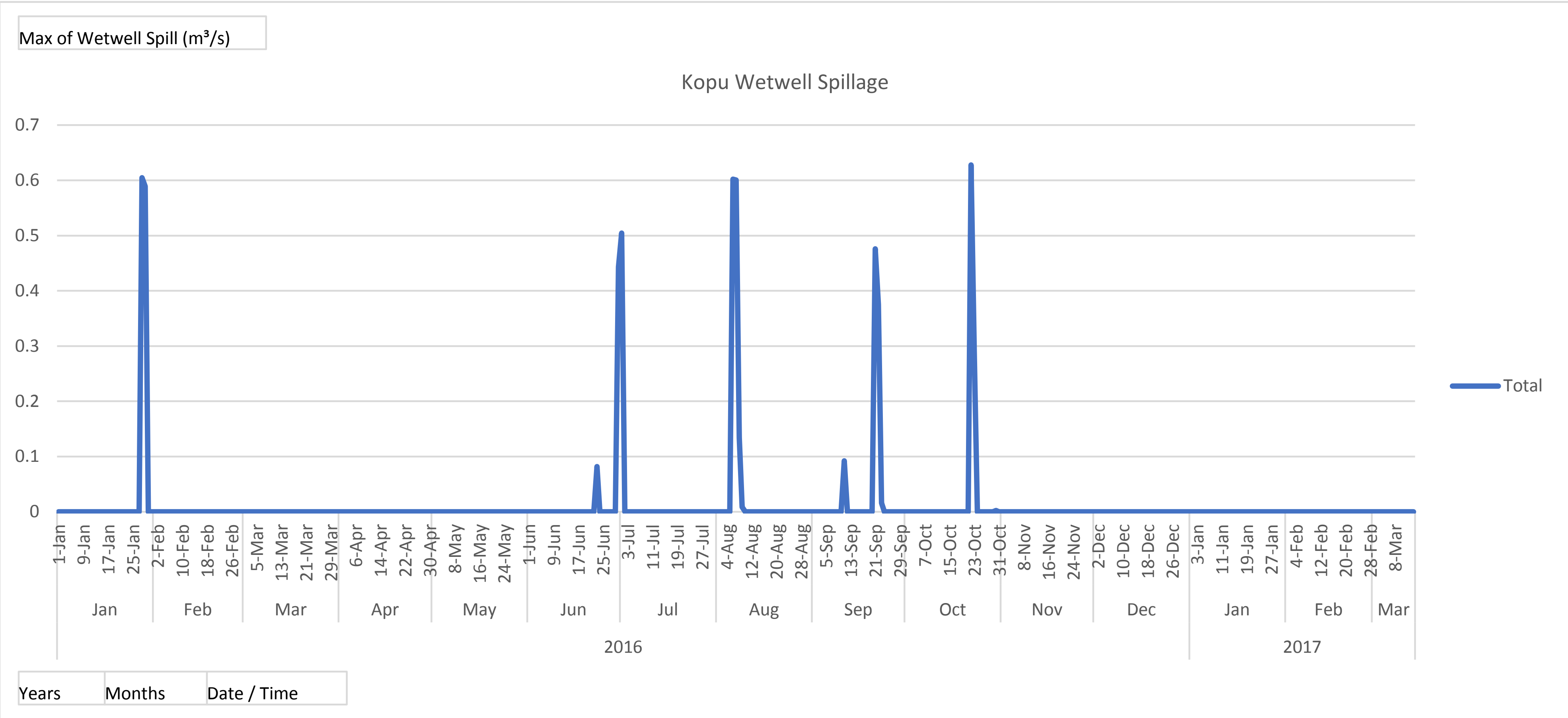


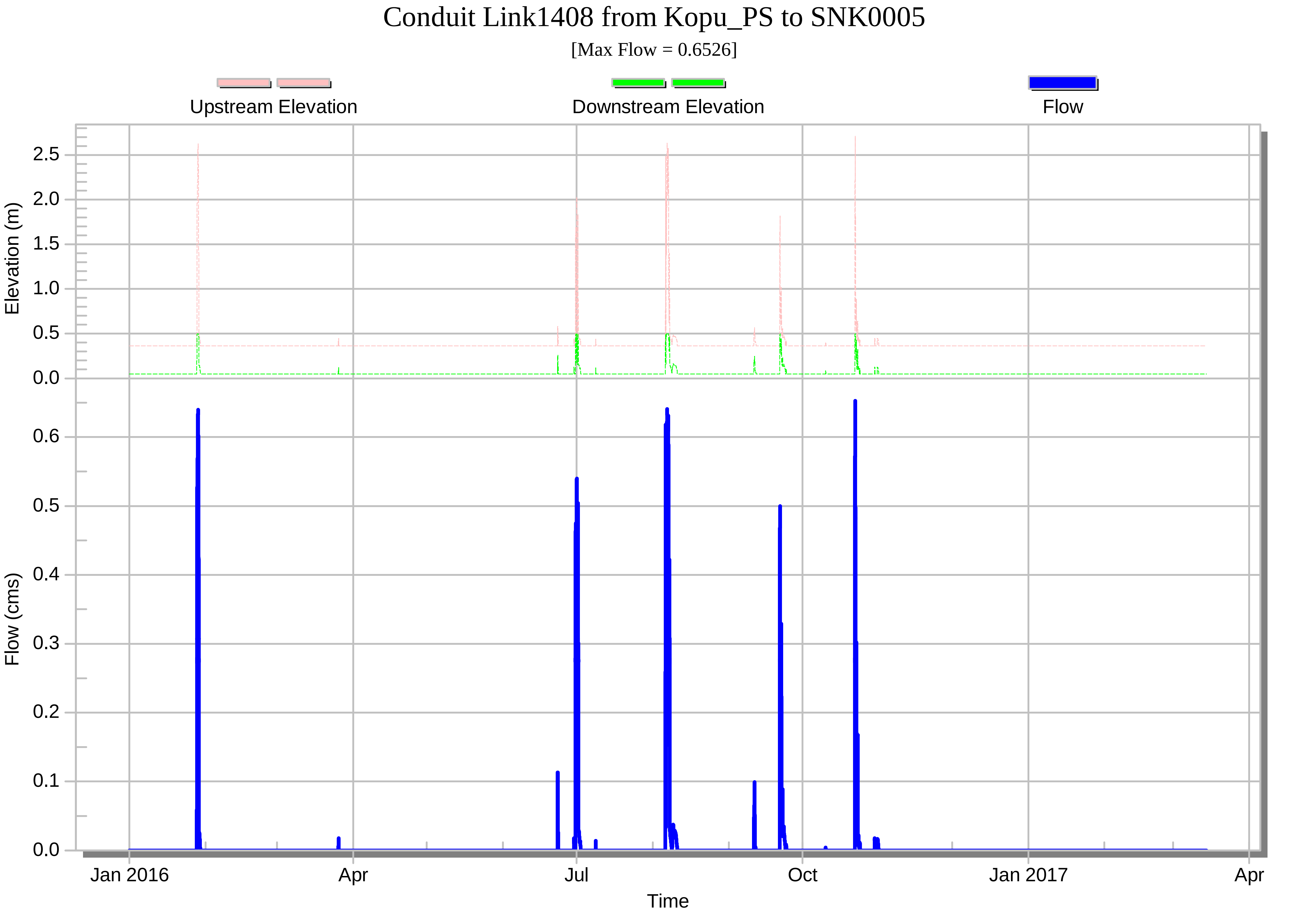


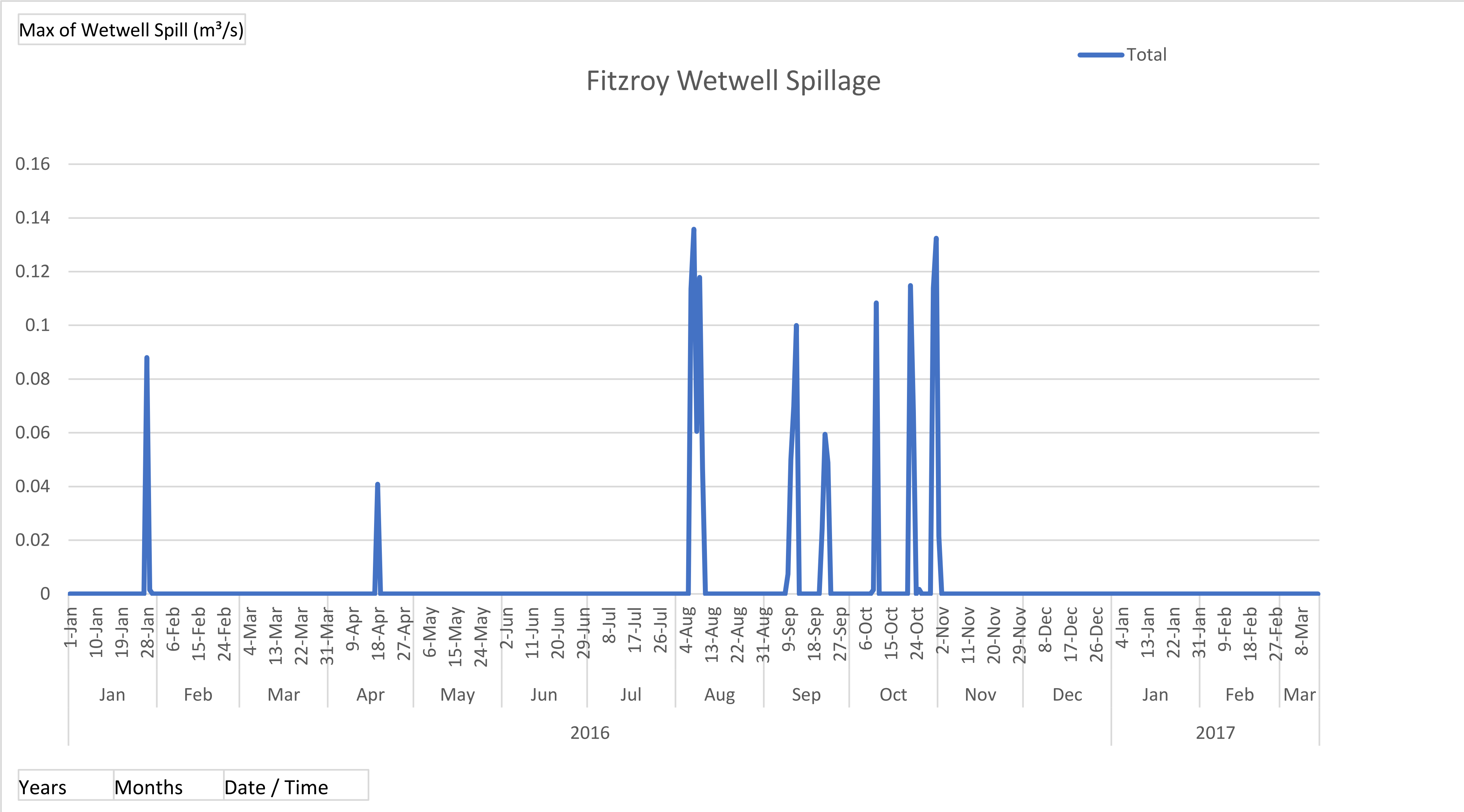
Max of Wetwell Spill (m³/s)
Alexander Park Wetwell Spillage
Total
0.25
0.2
0.15
0.1
0.05
0
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Jan
Feb
Mar
2016
2017
Years
Months
Date / Time



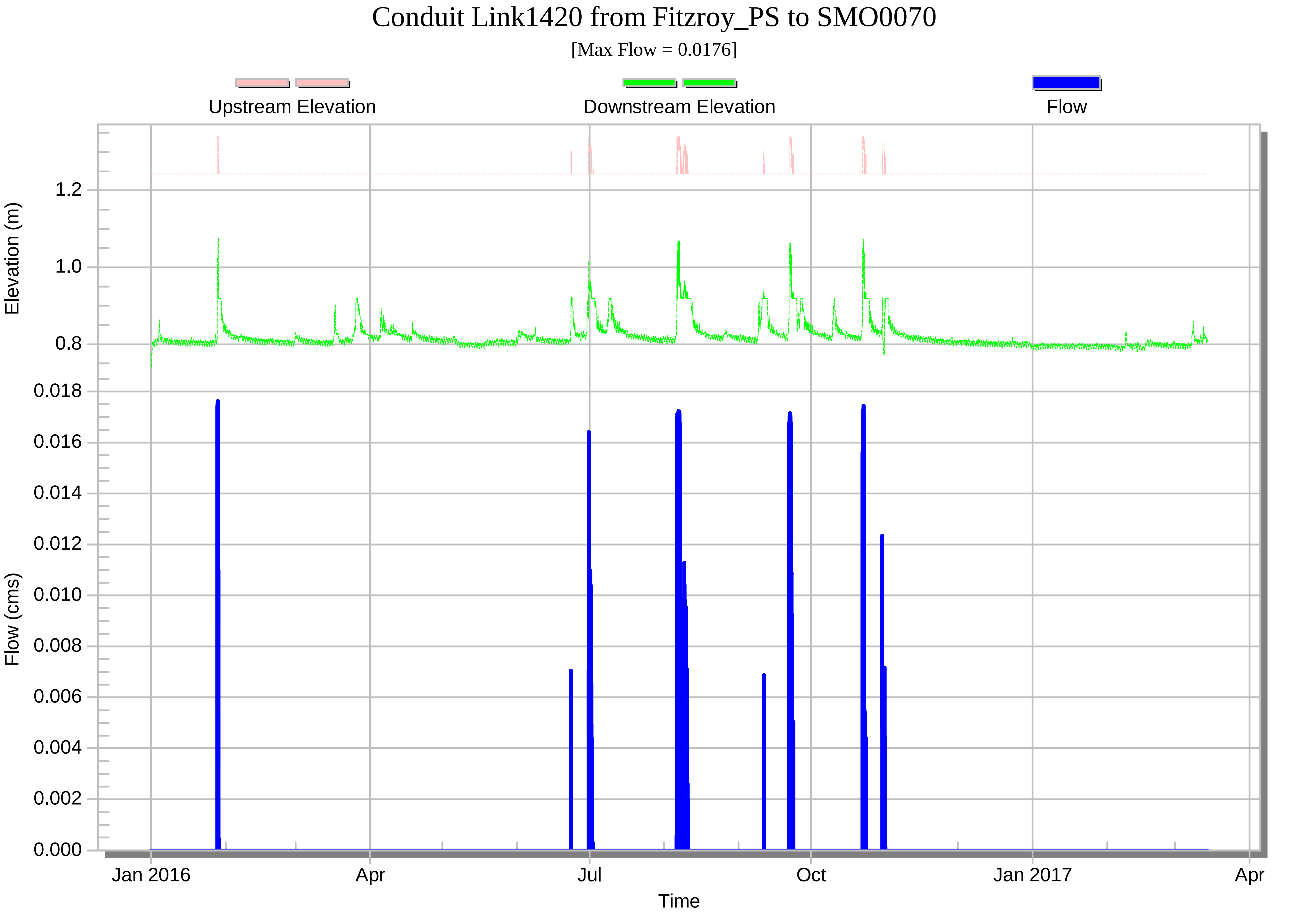
Conduit Link1388 from AlexanderPark_PS to SMA0910
[Max Flow = 0.2291]
Upstream Elevation
Downstream Elevation
Flow
Elevation (m)
5
4
3
2
1
Flow (cms)
0.20
0.15
0.10
0.05
0.00
Jan 2016
Apr
Jul
Oct
Jan 2017
Apr
Time


Max of Wetwell Spill (m^3/s)
Kopu Wetwell Spillage
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
Total
1-Jan
9-Jan
17-Jan
25-Jan
2-Feb
10-Feb
18-Feb
26-Feb
5-Mar
13-Mar
21-Mar
29-Mar
6-Apr
14-Apr
22-Apr
30-Apr
8-May
16-May
24-May
1-Jun
9-Jun
17-Jun
25-Jun
3-Jul
11-Jul
19-Jul
27-Jul
4-Aug
12-Aug
20-Aug
28-Aug
5-Sep
13-Sep
21-Sep
29-Sep
7-Oct
15-Oct
23-Oct
31-Oct
8-Nov
16-Nov
24-Nov
2-Dec
10-Dec
18-Dec
26-Dec
3-Jan
11-Jan
19-Jan
27-Jan
4-Feb
12-Feb
20-Feb
28-Feb
8-Mar
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Jan
Feb
Mar
2016
2017
Years
Months
Date / Time


Conduit Link1408 from Kopu_PS to SNK0005
[Max Flow = 0.6526]
Upstream Elevation
Downstream Elevation
Flow
Elevation (m)
2.5
2.0
1.5
1.0
0.5
0.0
Flow (cms)
0.6
0.5
0.4
0.3
0.2
0.1
0.0
Jan 2016
Apr
Jul
Oct
Jan 2017
Apr
Time



Max of Wetwell Spill (m³/s)
Total
Fitzroy Wetwell Spillage
0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
1-Jan
10-Jan
19-Jan
28-Jan
6-Feb
15-Feb
24-Feb
4-Mar
13-Mar
22-Mar
31-Mar
9-Apr
18-Apr
27-Apr
6-May
15-May
24-May
2-Jun
11-Jun
20-Jun
29-Jun
8-Jul
17-Jul
26-Jul
4-Aug
13-Aug
22-Aug
31-Aug
9-Sep
18-Sep
27-Sep
6-Oct
15-Oct
24-Oct
2-Nov
11-Nov
20-Nov
29-Nov
8-Dec
17-Dec
26-Dec
4-Jan
13-Jan
22-Jan
31-Jan
9-Feb
18-Feb
27-Feb
8-Mar
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Jan
Feb
Mar
2016
2017
Years
Months
Date / Time

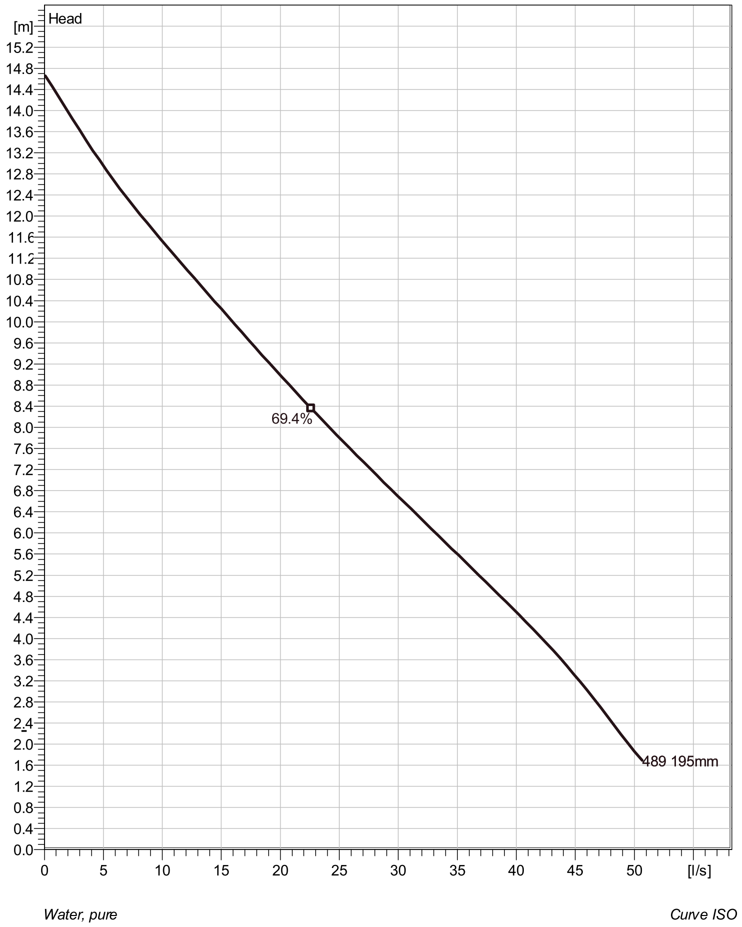

Conduit Link1420 from Fitzroy_PS to SMO0070
[Max Flow = 0.0176]
Upstream Elevation
Downstream Elevation
Flow
Elevation (m)
1.2
1.0
0.8
Flow (cms)
0.018
0.016
0.014
0.012
0.010
0.008
0.006
0.004
0.002
0.000
Jan 2016
Apr
Jul
Oct
Jan 2017
Apr
Time

Appendix D

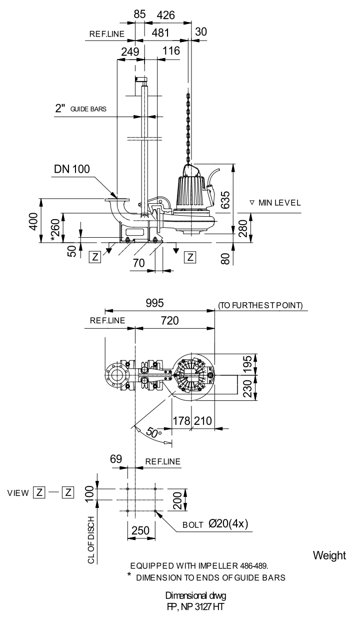
PUMP DATASHEETS

NP 3127 HT 3~ Adaptive 489

Technical specification



Installation: P - Semi permanent, Wet



Weight

Note: Picture might not correspond to the current configuration.

General

Patented self cleaning semi-open channel impeller, ideal for pumping in most waste water applications. Possible to be upgraded with Guide-pin® for even better clogging resistance. Modular based design with high adaptation grade.

Impeller

Impeller material	Grey cast iron
Discharge Flange Diameter	100 mm
Suction Flange Diameter	100 mm
Impeller diameter	195 mm
Number of blades	2

Motor

Motor #	N3127.161 21-10-4AL-W 4.7KW Standard
Stator variant	2
Frequency	50 Hz
Rated voltage	400 V
Number of poles	4
Phases	3~
Rated power	4.7 kW
Rated current	9.6 A
Starting current	58 A
Rated speed	1445 rpm
Power factor	
1/1 Load	0.85
3/4 Load	0.81
1/2 Load	0.71
Motor efficiency	
1/1 Load	83.0 %
3/4 Load	83.8 %
1/2 Load	82.4 %

Configuration

Project	Project ID	Created by	Created on	Last update
			8/28/2017	

NP 3127 HT 3~ Adaptive 489

Performance curve

Pump

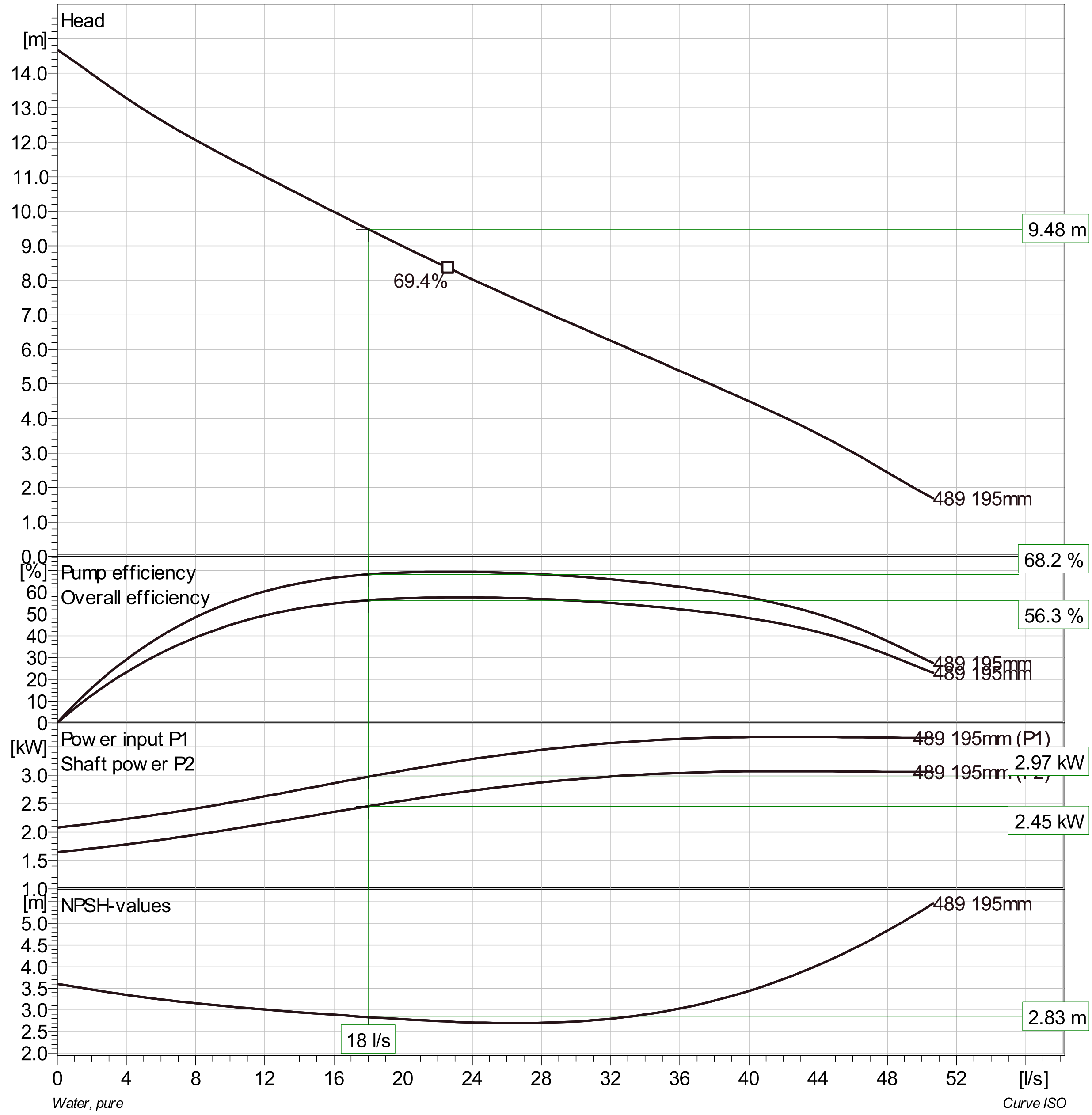
Discharge Flange Diameter	100 mm
Suction Flange Diameter	100 mm
Impeller diameter	195 mm
Number of blades	2

Motor

Motor #	N3127.161 21-10-4AL-W 4.7KW
Stator variant	2
Frequency	50 Hz
Rated voltage	400 V
Number of poles	4
Phases	3~
Rated power	4.7 kW
Rated current	9.6 A
Starting current	58 A
Rated speed	1445 rpm

Power factor	
1/1 Load	0.85
3/4 Load	0.81
1/2 Load	0.71

Motor efficiency	
1/1 Load	83.0 %
3/4 Load	83.8 %
1/2 Load	82.4 %

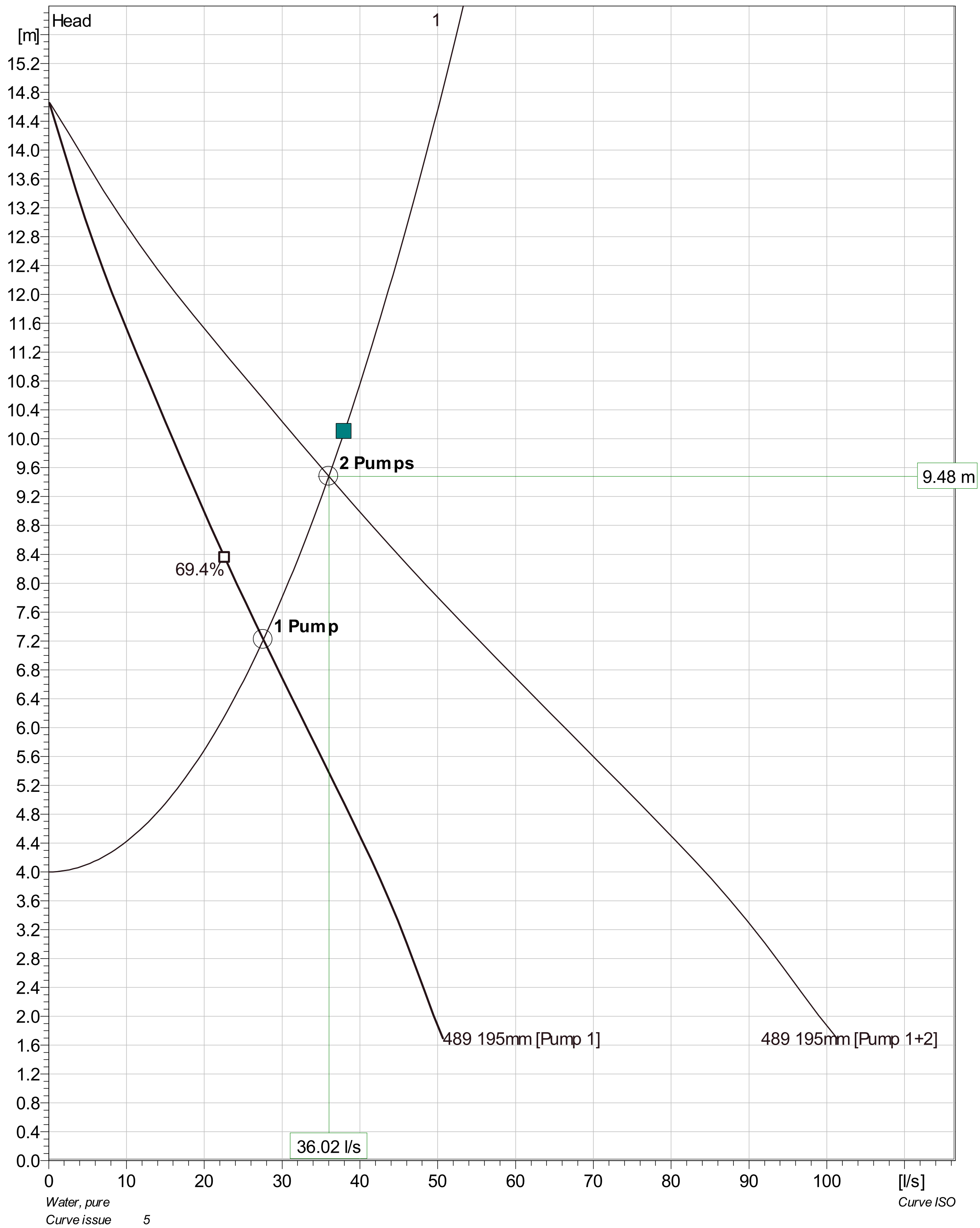


Duty point		Guarantee	
Flow	**Head**	**ISO:1999**	**Grade**
19 l/s	10.1 m	No	

Project	Project ID	Created by	Created on	Last update
			8/28/2017	

NP 3127 HT 3~ Adaptive 489

Duty Analysis



Pumps running /System	Individual pump Flow	Head	Shaft power	Total Flow	Head	Shaft power	Pump eff.	Specific energy	NPSHre
2 / 1	18 l/s	9.48 m	2.45 kW	36 l/s	9.48 m	4.91 kW	68.2 %	0.0459 kWh/m³	2.83 m
1 / 1	27.6 l/s	7.22 m	2.86 kW	27.6 l/s	7.22 m	2.86 kW	68.4 %	0.0351 kWh/m³	2.7 m

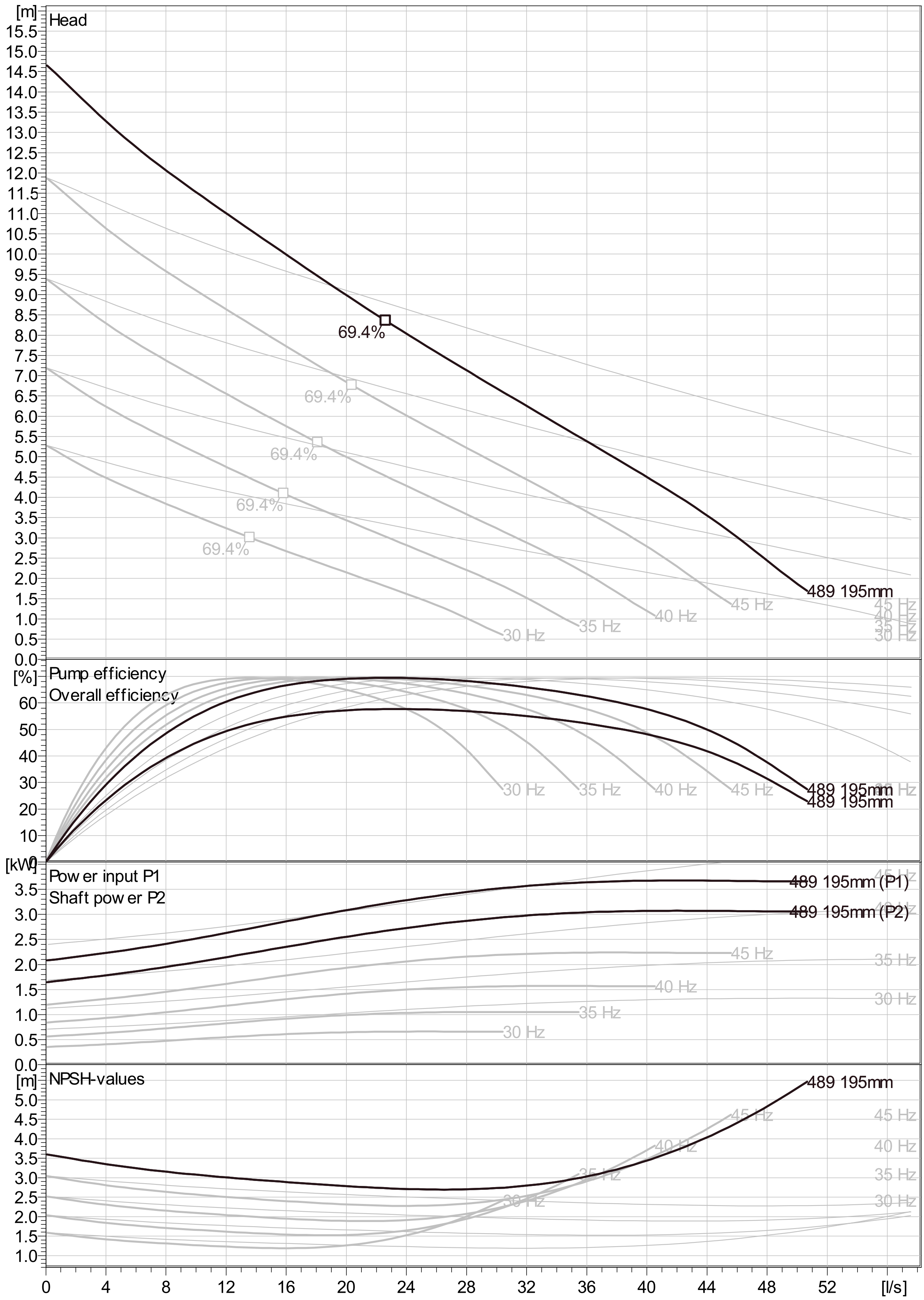
Project	Project ID	Created by	Created on	Last update
			8/28/2017	

xylem

NP 3127 HT 3~ Adaptive 489

VFD Curve

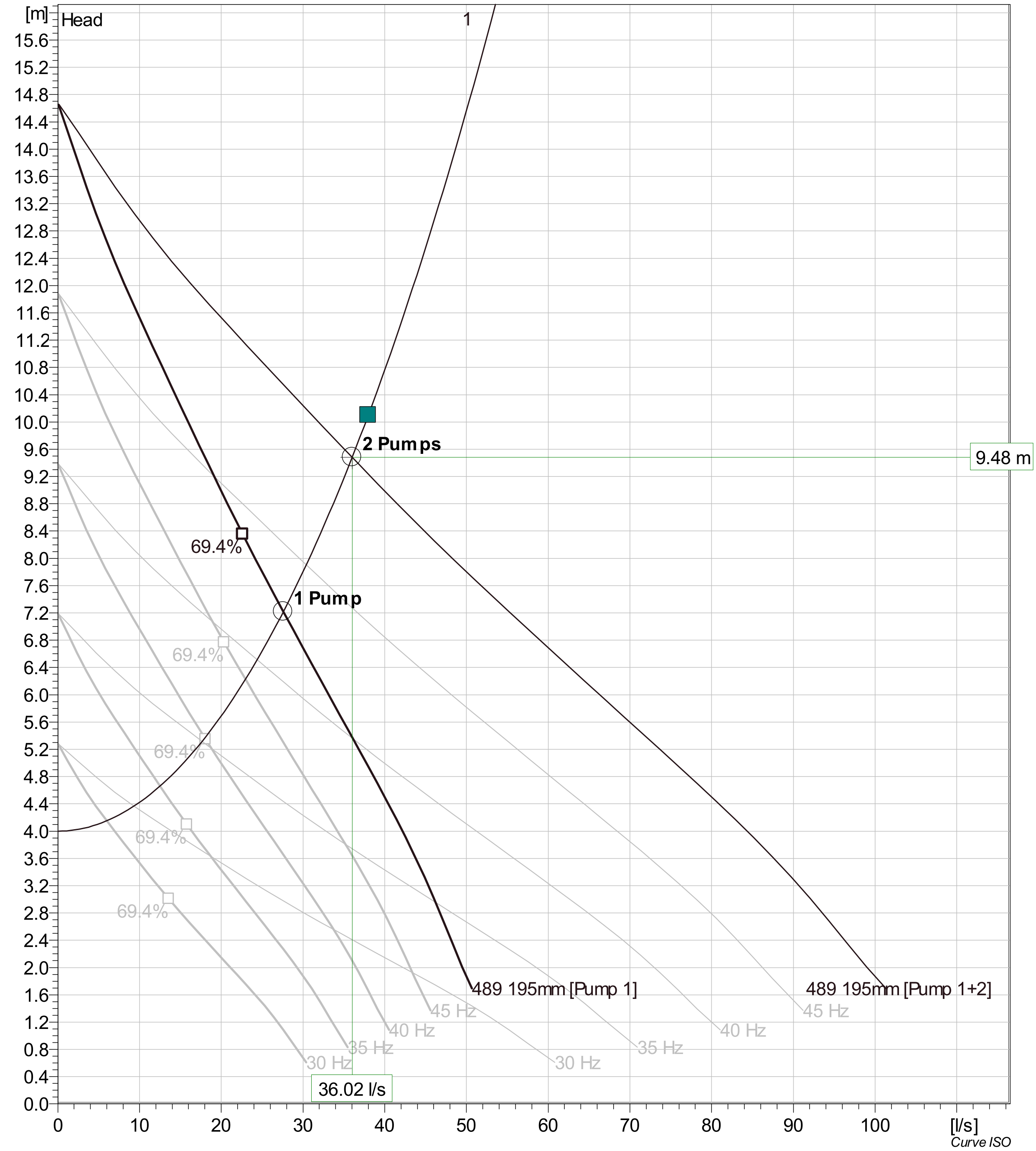
FLYGT



Curve ISO

Project	Project ID	Created by	Created on	Last update
			8/28/2017	

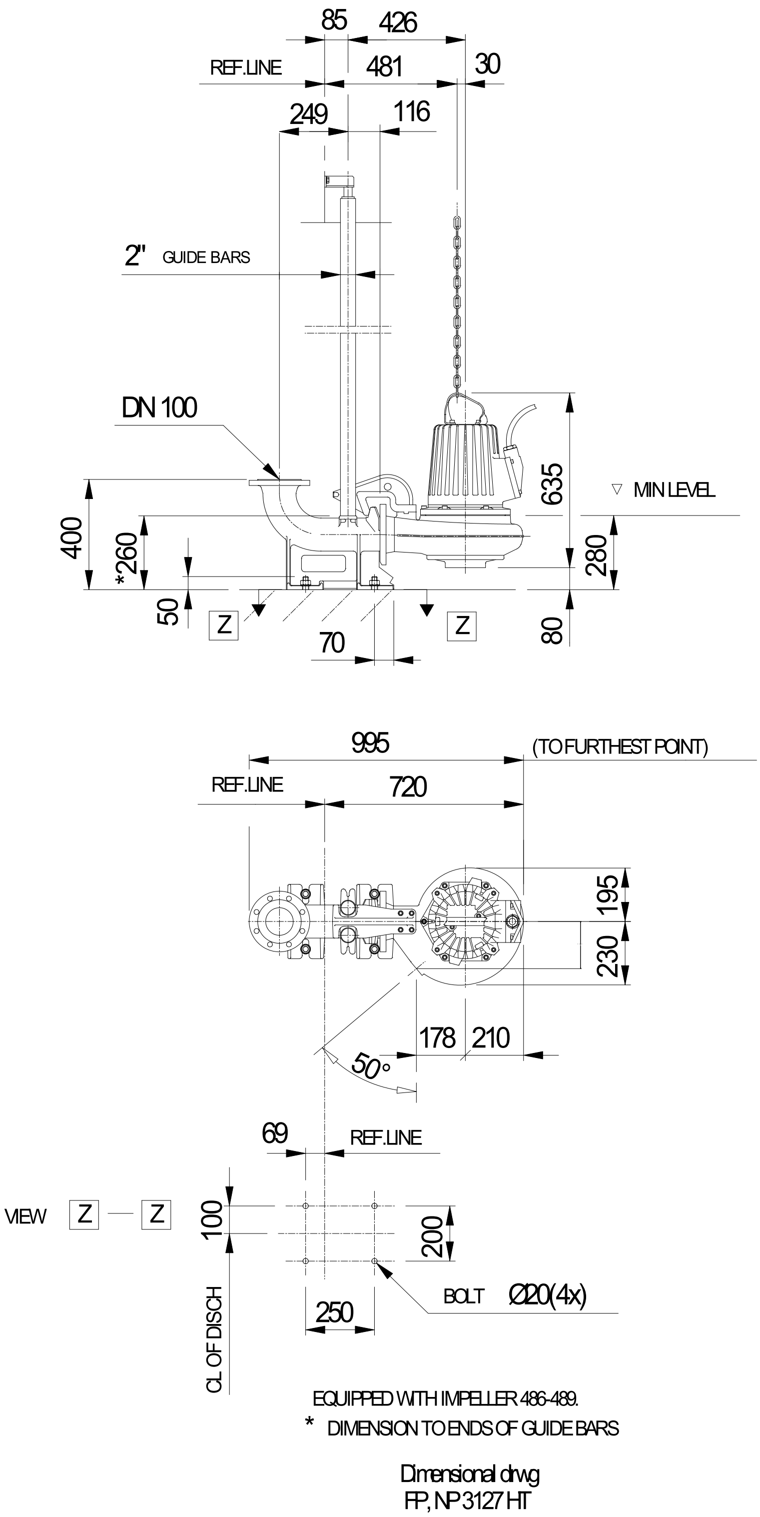
NP 3127 HT 3~ Adaptive 489
VFD Analysis



Pumps running /System	Frequency	Flow	Head	Shaft power	Flow	Head	Shaft power	Pump eff.	Specific energy	NPSHre
2 / 1	50 Hz	18 l/s	9.48 m	2.45 kW	36 l/s	9.48 m	4.91 kW	68.2 %	0.0459 kWh/m³	2.83 m
2 / 1	45 Hz	15.2 l/s	7.91 m	1.75 kW	30.4 l/s	7.91 m	3.5 kW	67.4 %	0.04 kWh/m³	2.42 m
2 / 1	40 Hz	12.2 l/s	6.51 m	1.18 kW	24.4 l/s	6.51 m	2.37 kW	65.8 %	0.0361 kWh/m³	2.04 m
2 / 1	35 Hz	8.83 l/s	5.32 m	0.747 kW	17.7 l/s	5.32 m	1.49 kW	61.7 %	0.0356 kWh/m³	1.69 m
2 / 1	30 Hz	4.66 l/s	4.37 m	0.419 kW	9.32 l/s	4.37 m	0.838 kW	47.6 %	0.0476 kWh/m³	1.4 m
1 / 1	50 Hz	27.6 l/s	7.22 m	2.86 kW	27.6 l/s	7.22 m	2.86 kW	68.4 %	0.0351 kWh/m³	2.7 m
1 / 1	45 Hz	23 l/s	6.23 m	2.03 kW	23 l/s	6.23 m	2.03 kW	69.1 %	0.0312 kWh/m³	2.28 m
1 / 1	40 Hz	18 l/s	5.37 m	1.36 kW	18 l/s	5.37 m	1.36 kW	69.4 %	0.0288 kWh/m³	1.91 m
1 / 1	35 Hz	12.5 l/s	4.66 m	0.84 kW	12.5 l/s	4.66 m	0.84 kW	68.1 %	0.0291 kWh/m³	1.6 m
1 / 1	30 Hz	5.99 l/s	4.15 m	0.442 kW	5.99 l/s	4.15 m	0.442 kW	55.3 %	0.0402 kWh/m³	1.36 m

Project	Project ID	Created by	Created on	Last update
			8/28/2017	

NP 3127 HT 3~ Adaptive 489
Dimensional drawing

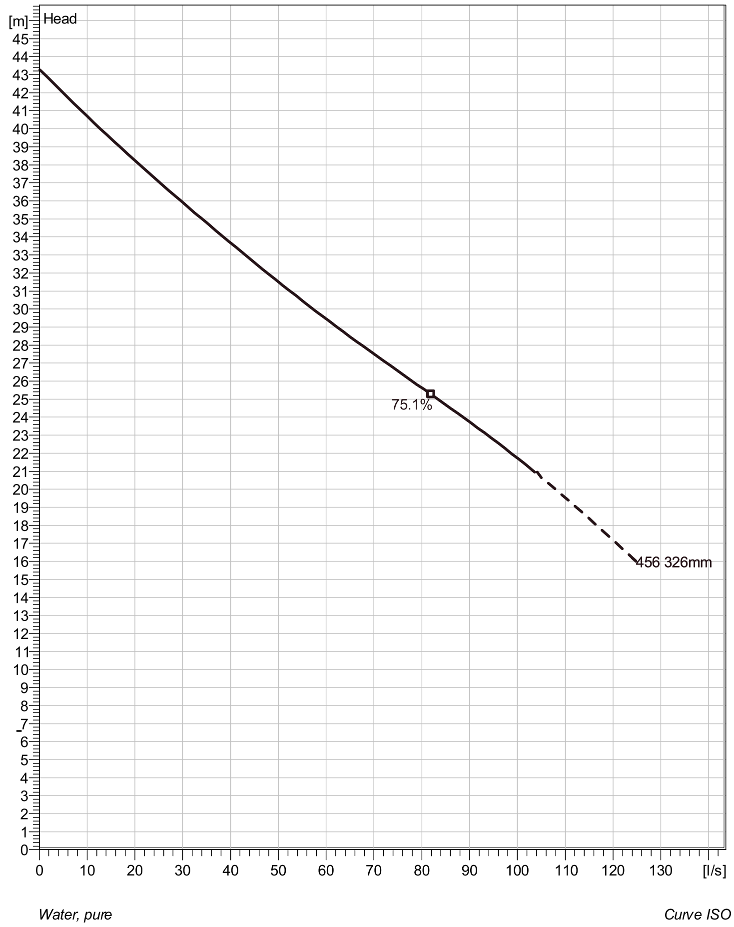


Weight

Project	Project ID	Created by	Created on	Last update
			8/28/2017	

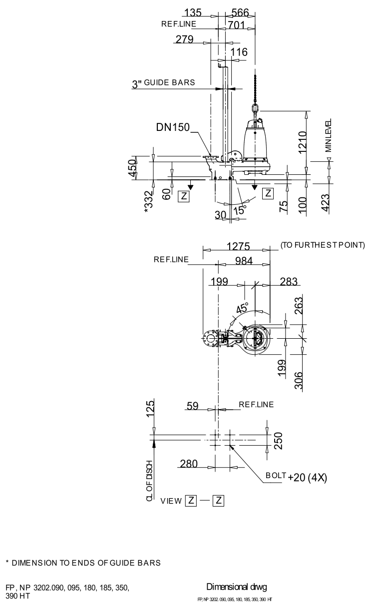
NP 3202 HT 3~ 456

Technical specification



Note: Picture might not correspond to the current configuration.

Installation: P - Semi permanent, Wet



General

Patented self cleaning semi-open channel impeller, ideal for pumping in most waste water applications. Possible to be upgraded with Guide-pin® for even better clogging resistance. Modular based design with high adaptation grade.

Impeller

Impeller material	Grey cast iron
Discharge Flange Diameter	150 mm
Suction Flange Diameter	200 mm
Impeller diameter	326 mm
Number of blades	2

Motor

Motor #	N3202.180 30-19-4AA-W 30KW Standard
Stator variant	1
Frequency	50 Hz
Rated voltage	400 V
Number of poles	4
Phases	3~
Rated power	30 kW
Rated current	54 A
Starting current	360 A
Rated speed	1475 rpm
Power factor	
1/1 Load	0.88
3/4 Load	0.84
1/2 Load	0.74
Motor efficiency	
1/1 Load	90.5 %
3/4 Load	91.5 %
1/2 Load	91.0 %

Configuration

Project	Project ID	Created by	Created on	Last update
			8/25/2017	

NP 3202 HT 3~ 456

Performance curve

Pump

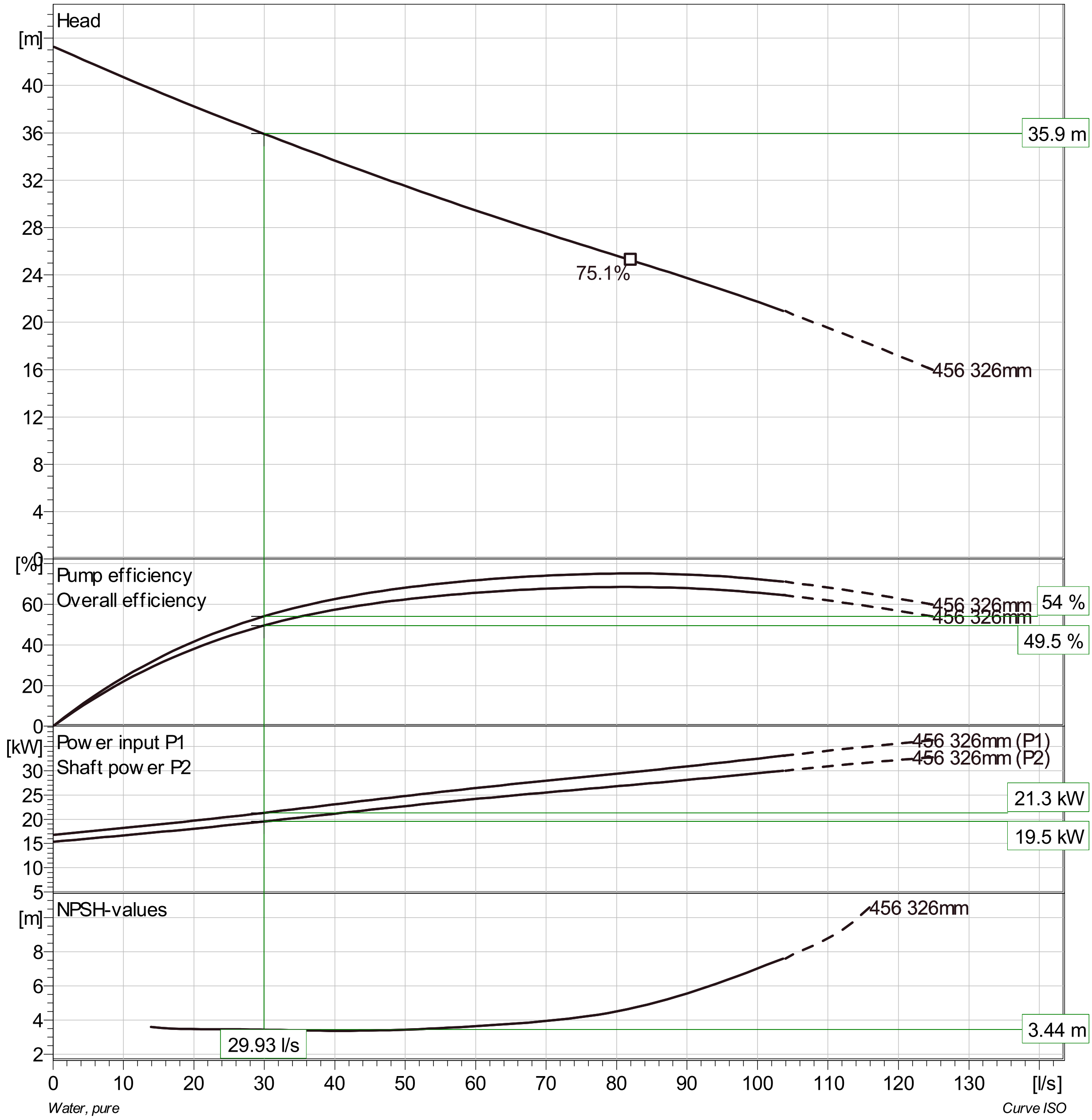
Discharge Flange Diameter	150 mm
Suction Flange Diameter	200 mm
Impeller diameter	326 mm
Number of blades	2

Motor

Motor #	N3202.180 30-19-4AA-W 30KW
Stator variant	1
Frequency	50 Hz
Rated voltage	400 V
Number of poles	4
Phases	3~
Rated power	30 kW
Rated current	54 A
Starting current	360 A
Rated speed	1475 rpm

Power factor	
1/1 Load	0.88
3/4 Load	0.84
1/2 Load	0.74

Motor efficiency	
1/1 Load	90.5 %
3/4 Load	91.5 %
1/2 Load	91.0 %

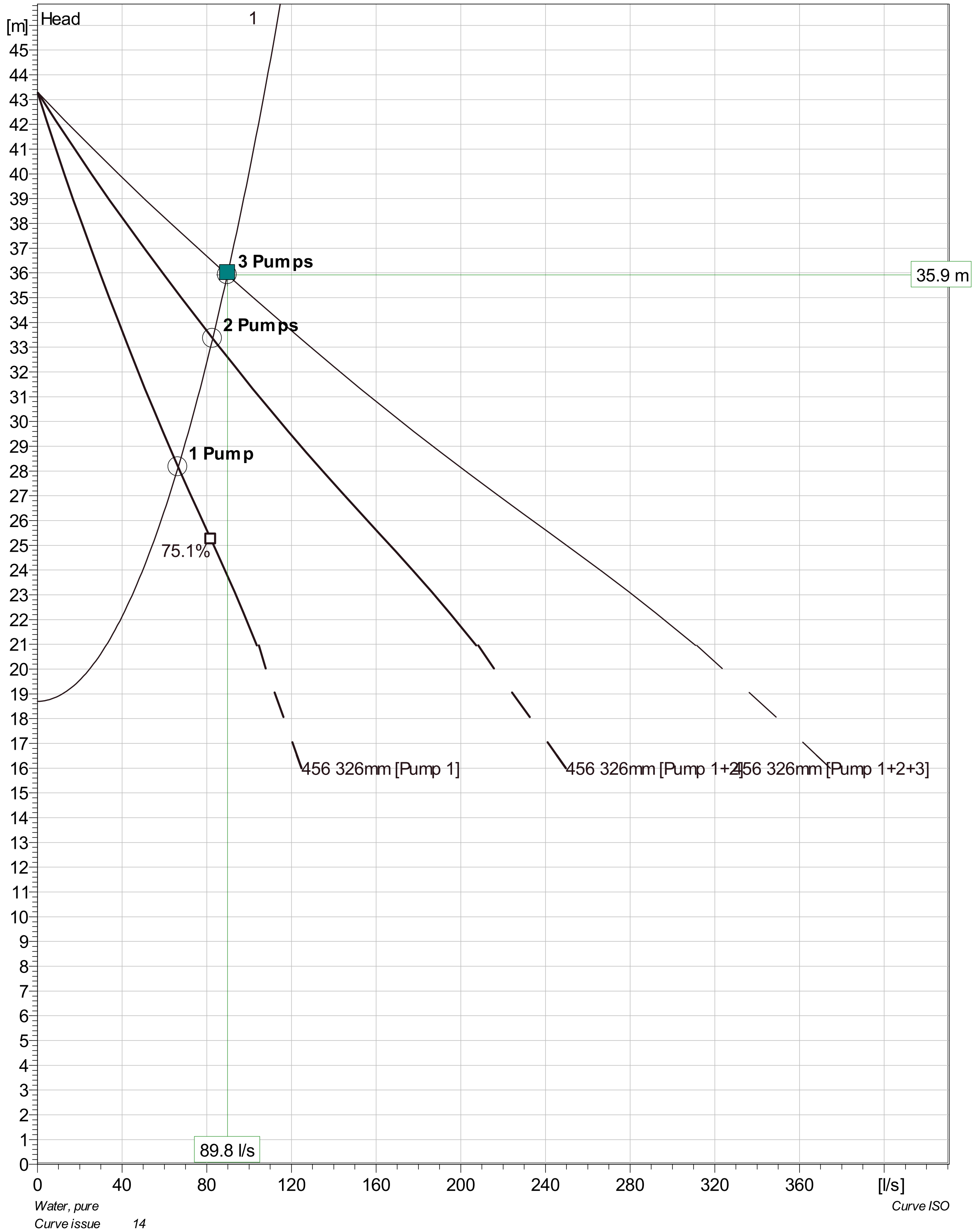


Duty point		Guarantee	
Flow	**Head**	**ISO:1999**	**Grade**
30 l/s	36 m	No	

Project	Project ID	Created by	Created on	Last update
			8/25/2017	

NP 3202 HT 3~ 456

Duty Analysis

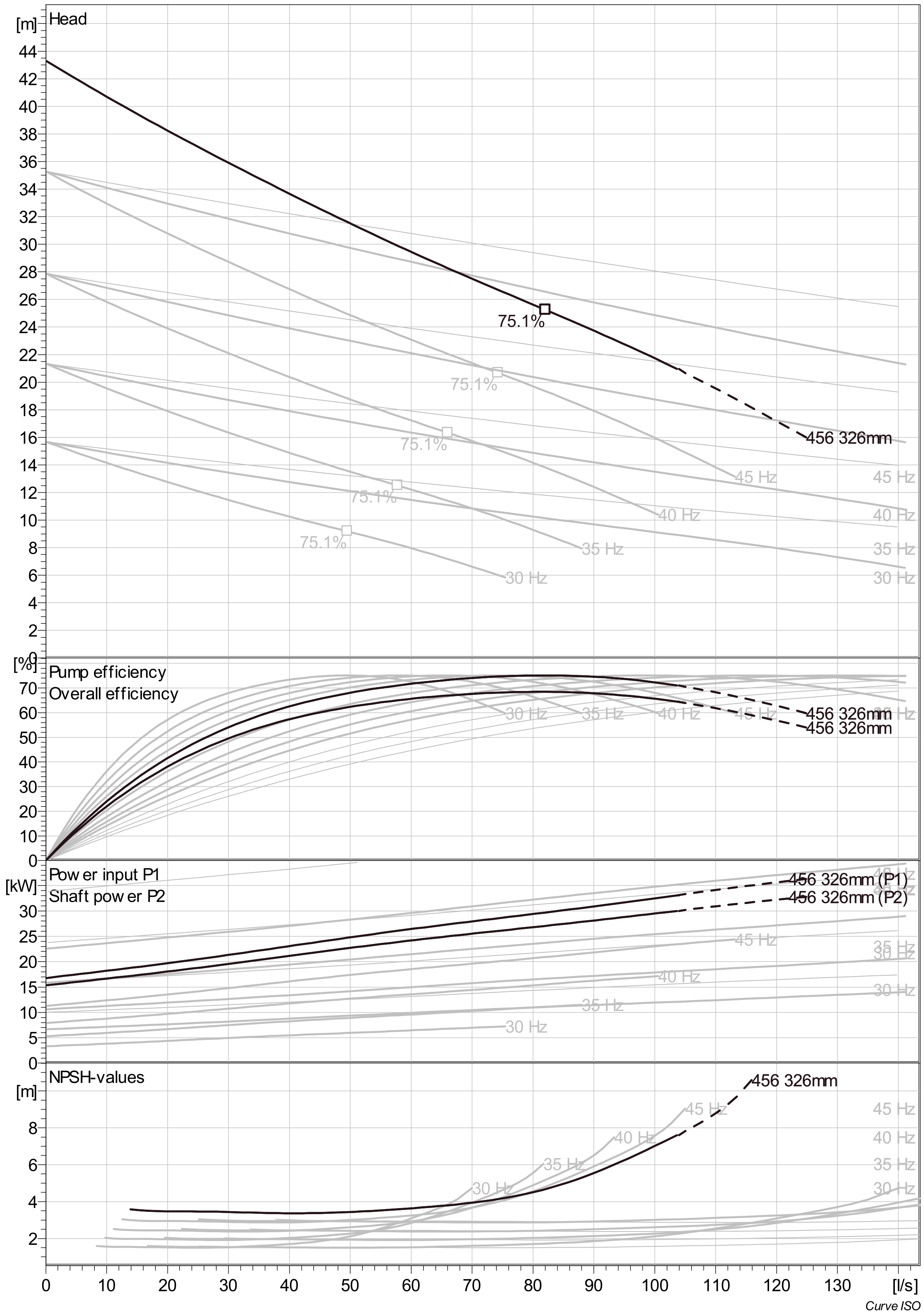


Pumps running /System	Individual pump Flow	Head	Shaft power	Total Flow	Head	Shaft power	Pump eff.	Specific energy	NPSHre
3 / 1	29.9 l/s	35.9 m	19.5 kW	89.8 l/s	35.9 m	58.6 kW	54 %	0.198 kWh/m³	3.44 m
2 / 1	41.4 l/s	33.4 m	21.4 kW	82.8 l/s	33.4 m	42.7 kW	63.4 %	0.156 kWh/m³	3.37 m
1 / 1	66.6 l/s	28.2 m	25.1 kW	66.6 l/s	28.2 m	25.1 kW	73.4 %	0.114 kWh/m³	3.82 m

Project	Project ID	Created by	Created on	Last update
			8/25/2017	

NP 3202 HT 3~ 456

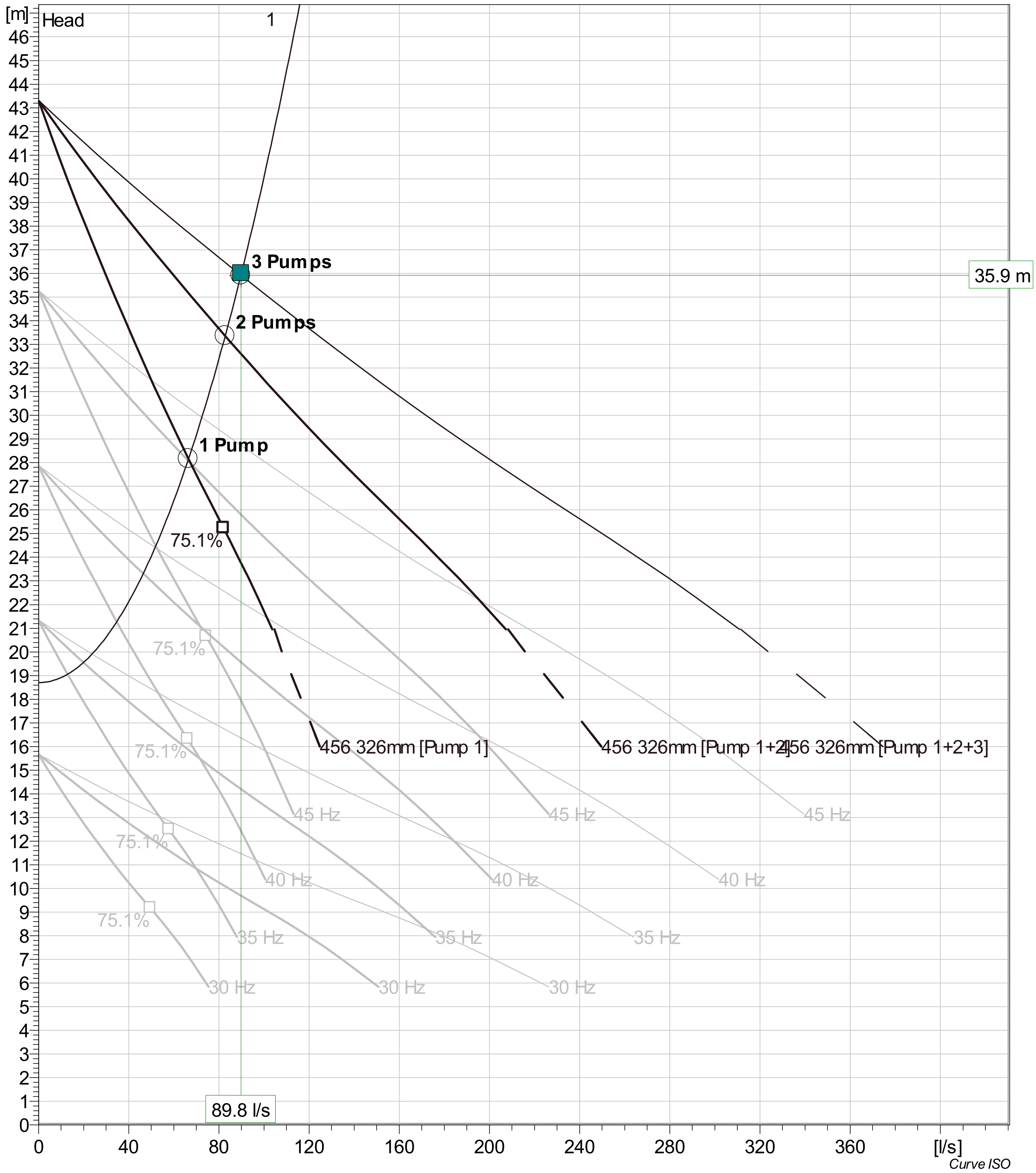
VFD Curve



Project	Project ID	Created by	Created on	Last update
			8/25/2017	

NP 3202 HT 3~ 456

VFD Analysis

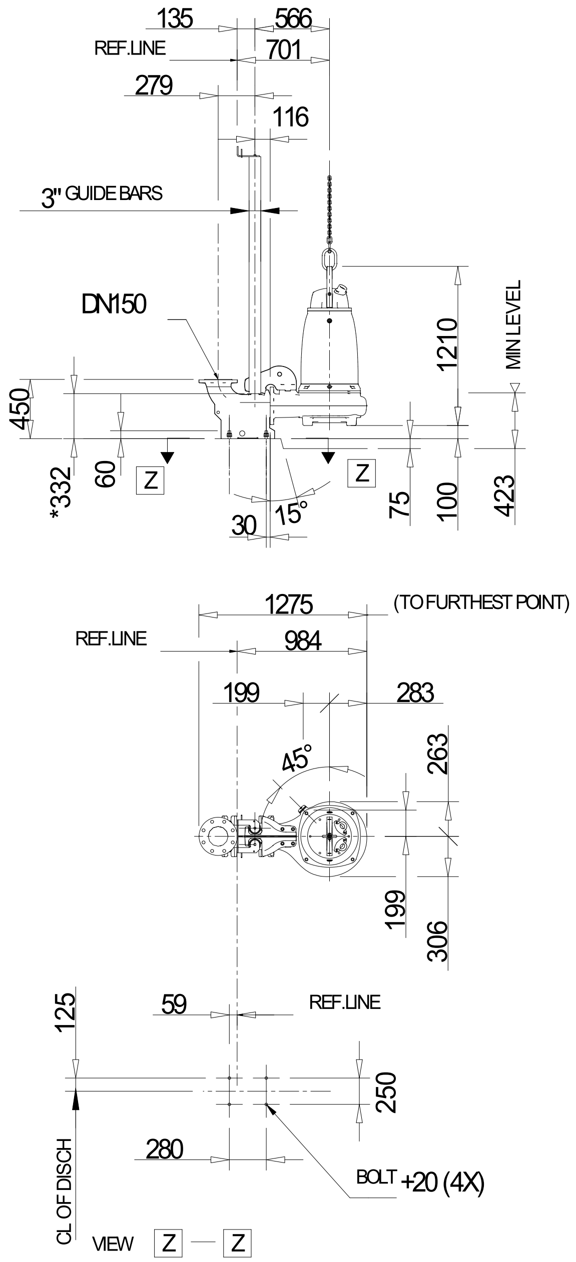


Pumps running /System	Frequency	Flow	Head	Shaft power	Flow	Head	Shaft power	Pump eff.	Specific energy	NPSHre
3 / 1	50 Hz	29.9 l/s	35.9 m	19.5 kW	89.8 l/s	35.9 m	58.6 kW	54 %	0.198 kWh/m³	3.44 m
3 / 1	44.8 Hz	23.7 l/s	29.5 m	13.6 kW	71 l/s	29.5 m	40.9 kW	50.1 %	0.176 kWh/m³	2.9 m
3 / 1	39.8 Hz	16.7 l/s	24.1 m	9.17 kW	50.2 l/s	24.1 m	27.5 kW	43 %	0.17 kWh/m³	2.41 m
3 / 1	34.8 Hz	7.21 l/s	19.7 m	5.64 kW	21.6 l/s	19.7 m	16.9 kW	24.7 %	0.256 kWh/m³	
3 / 1	29.8 Hz	SPECENERGY[KL,bp2| speedidx3]								
2 / 1	50 Hz	41.4 l/s	33.4 m	21.4 kW	82.8 l/s	33.4 m	42.7 kW	63.4 %	0.156 kWh/m³	3.37 m
2 / 1	44.8 Hz	32.4 l/s	27.7 m	14.7 kW	64.8 l/s	27.7 m	29.5 kW	59.7 %	0.139 kWh/m³	2.84 m
2 / 1	39.8 Hz	22.5 l/s	23 m	9.72 kW	44.9 l/s	23 m	19.4 kW	52.2 %	0.135 kWh/m³	2.39 m
2 / 1	34.8 Hz	8.98 l/s	19.4 m	5.76 kW	18 l/s	19.4 m	11.5 kW	29.6 %	0.21 kWh/m³	
2 / 1	29.8 Hz	SPECENERGY[KL,bp1| speedidx3]								
1 / 1	50 Hz	66.6 l/s	28.2 m	25.1 kW	66.6 l/s	28.2 m	25.1 kW	73.4 %	0.114 kWh/m³	3.82 m
1 / 1	44.8 Hz	50.6 l/s	24.2 m	17 kW	50.6 l/s	24.2 m	17 kW	70.6 %	0.103 kWh/m³	2.98 m
1 / 1	39.8 Hz	33.3 l/s	21.1 m	10.8 kW	33.3 l/s	21.1 m	10.8 kW	63.7 %	0.101 kWh/m³	2.34 m
1 / 1	34.8 Hz	11.4 l/s	19 m	5.92 kW	11.4 l/s	19 m	5.92 kW	35.8 %	0.171 kWh/m³	1.97 m
1 / 1	29.8 Hz	SPECENERGY[KL,bp0| speedidx3]								

Project	Project ID	Created by	Created on	Last update
			8/25/2017	

NP 3202 HT 3~ 456

Dimensional drawing



* DIMENSION TO ENDS OF GUIDE BARS

FP, NP 3202.090, 095, 180, 185, 350, 390 HT

Dimensional drwg

FP,NP3202.090,095,180,185,350,390 HT

Project	Project ID	Created by	Created on	Last update
			8/25/2017	

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