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WAIROA WASTEWATER MODELLING

STAGE 1 – TRUNK MODEL DOWNSTREAM OF PUMP STATIONS

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WAIROA DISTRICT COUNCIL

OPUS INTERNATIONAL CONSULTANTS

Wairoa Wastewater Modelling

Stage 1- Trunk Model Downstream of Pump Stations

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Quality Assurance Statement	
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Executive Summary

Stage 1 of the study covers the model build and investigation of the network downstream of the pump stations. The main objective of the Stage 1 study is:

- To understand why some gullies connected to this pipe system are overflowing
- To establish the impact of sealing off the overflow at the downstream end of the rising main from Kopu Pump Station
- To establish the impact of increasing the flows from pump stations
- To determine whether there are any short sections of pipe that are reducing the performance (identify quick-win improvements).

The model was built from the asset data supplied by the Wairoa District Council. Asset data has some gaps and the missing invert levels were filled up by interpolation. The model was built using InfoWorks CS 11.5. The model was not calibrated against any measured flows. The pump flow rates in the model are the theoretical rates based on the pump manufactures specifications.

Dry weather flows in the model is population based, with per capita of 200 litres per person per day. The pattern was assumed based on typical diurnal patterns of residential areas.

a. Kopu Pump Station

Kopu pump station has three pumps. When all three pumps are operating the pump station has a theoretical capacity of 140 l/s. However the pumps are old and it is likely that the pump station has an actual operating flow rate less than this.

The model identified that:

1. The pipe full capacity of the 450mm sewer from SMF0200 to SMF0010 is 15l/s, due to the flat grade of the pipe. However this section of pipe is deep and will not immediately overflow.
2. The overflow at manhole SMF0280 starts to operate at approximately 60l/s. The manholes between SMF0280 to SMF0200 are only 500mm deep. If the overflow was not in place the manholes would overflow.

To reduce the frequency and volume of overflows, it is recommended to pressurise the sewer from SMF0280 to SMF0200 by sealing the manhole covers.

This 300mm diameter sewer is about 30 years old and is made of asbestos cement. It does not appear to have any service connections.

Assuming the condition of the manholes and the pipe is adequate to be pressurised up to 2 metres, the sewer can pass 100 l/s, i.e. a 40 l/s reduction in total overflows.

The recommended works to further investigate and then implement this recommendation are:

1. Undertake a CCTV inspection and inspection of manholes between SMF0280 to SMF0200 to confirm that there are no connections onto this main and to check that the pipe and manhole is suitable for pressurising the main to 2m.
2. Seal manholes SMF0280 to SMF0200.
3. Undertake a draw down test on the pump station to confirm its actual pumping rate. Throttle down the pump station if it is pumping more than 100l/s.
4. Install pressure relief point at SMF0280 to prevent over pressuring of the downstream sewer.

b. Other Sections of the Trunk Network

The pipe full capacity of the gravity main from SMF0850 to SMF0200 is adequate the service the upstream catchment. No other "bottle necks" were identified in the trunk networks downstream of the pump stations.

c. Next Stage of the Project

It is recommended to proceed into Stage 3 to determine the improvement options to the local collection system.

We propose to examine the flow data from the Magflow meters recently installed at pump stations. It is expected that this data will be able to be used to calibrate the model, avoiding the need to undertake flow gauging.

1. Introduction

1.1 Background

Wairoa District Council (WDC) engaged Opus Consultants in July 2011 to build a computer model of the trunk waste water collection system downstream of the pump stations. The consultancy work is divided into three stages.

Stage 1 – Model downstream of pump stations

Stage 2 – Flow gauging by others

Stage 3 – Extend the model and develop improvement options

Wairoa District Council has wastewater strategy to address the wet weather overflows (Reference "Issues and Strategies to Address Wet Weather Overflows from Wairoa Wastewater System, May 2011). As outlined in this document, the following modelling tasks were carried out by Opus Consultants.

The specific modelling tasks included in Stage 1 are:

- Construct a hydraulic model of the wastewater system between the pump stations and the wastewater treatment plant
- Investigate the performance of the section of the network downstream of the pump stations.

The main objective of the Stage 1 study is:

- To understand why some gullies connected to this pipe system are overflowing
- To establish the impact of sealing off the overflow at the downstream end of the rising main from Kopu Pump Station
- To establish the impact of increasing the flows from pump stations
- To determine whether there are any short sections of pipe that are reducing the performance (identify quick-win improvements.

1.2 Project Area

Wairoa is the northernmost town in the Hawke's Bay Region and is located on the northern shore of Hawke's Bay at the mouth of the Wairoa River. It is 120 kilometres northeast of Napier, and 98kilometres from Gisborne.

Wairoa is a manufacturing and a farming service town. The district has a population of 8480 resident population (2006 census) and the population within the Wairoa Township (the study area) is approximately 4000.

A location map and a wastewater catchment map are shown below:

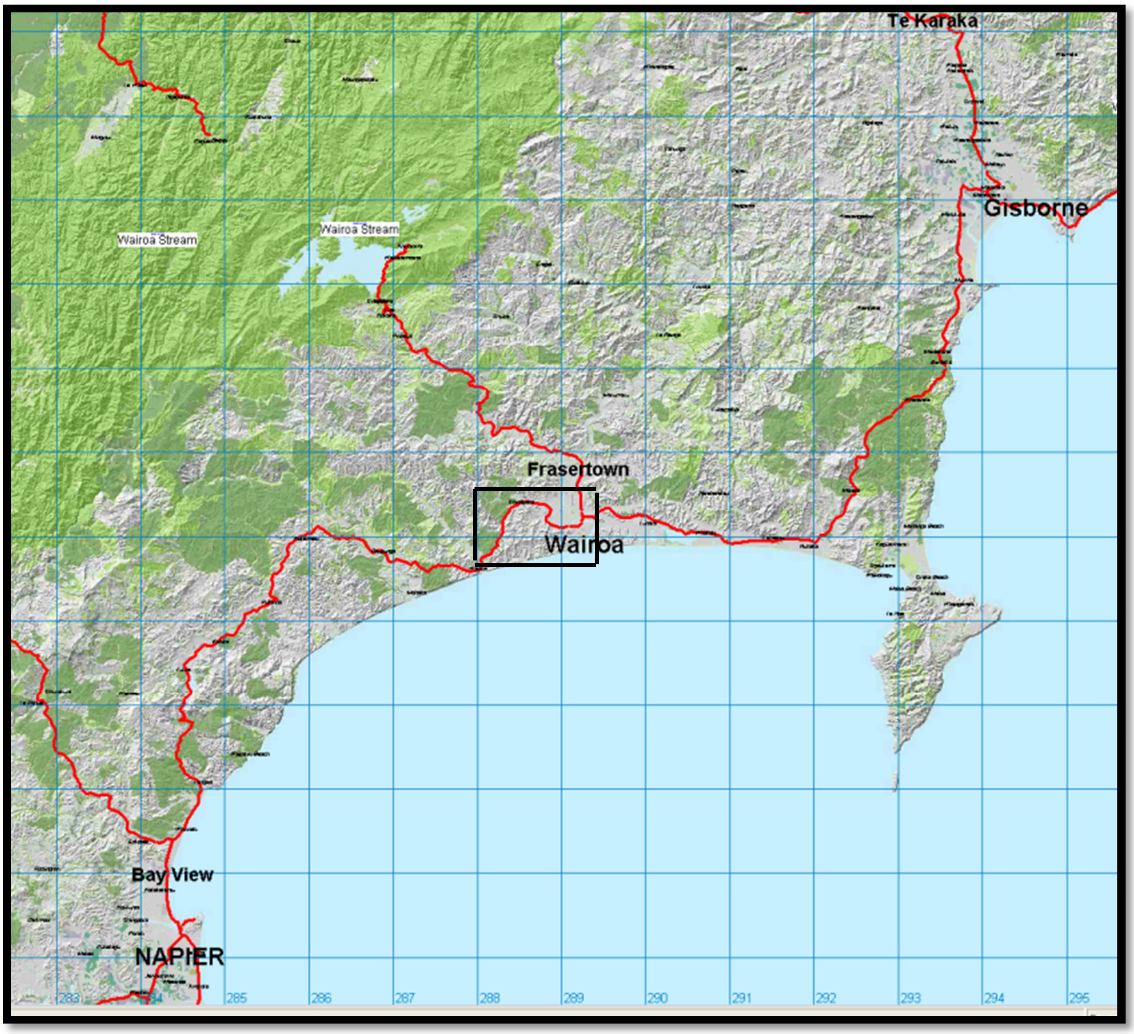


Figure 1 – Location Map

2 Wastewater Collection System

2.1 Overview

The existing wastewater collection system consists of 745 manholes, 40 km of gravity pipes with diameters ranging from 100mm to 450mm in diameter. There are 5 pump stations. The original network has been built in around 1948 and some improvements, extensions and replacements have been carried out at different periods since then. 70% of the pipe network is over 60 years old. 60% of the pipes by length are earthen ware (EW), 15% is asbestos cement (AC) and 10% is sulphide resistant cement concrete (CC-SR).

Five pump stations are:

- North Clyde PS
- Alexandra Park PS
- Kopu Road PS
- Rutherford Street PS
- Fitzroy Street PS

Three main pump stations (namely North Clyde, Alexandra Street and Kopu Road) service bulk of the area. Rutherford Street PS services a few properties along Kitchener Street. All these four pump stations discharge to a gravity system which runs down to Fitzroy Pump Station at the downstream end. This pump station pumps the flow to the treatment plant at the southernmost end of the area.

The service area is very flat and is encompassed by a large meander of the Wairoa River. The original sewer network built in 1948 consisted of three pump stations, viz. North Clyde, Alexandra Park and Kopu Road. The sewer flows were discharged into the Wairoa River through Kopu Road pump station.

In early 1980s, the flows have been directed into a new gravity trunk sewer abandoning the outfall to Wairoa River. These flows were taken to the southernmost end of the area where a new treatment plant was then constructed. Fitzroy Street pump station at the terminal end was constructed to lift the flows into the treatment plant.

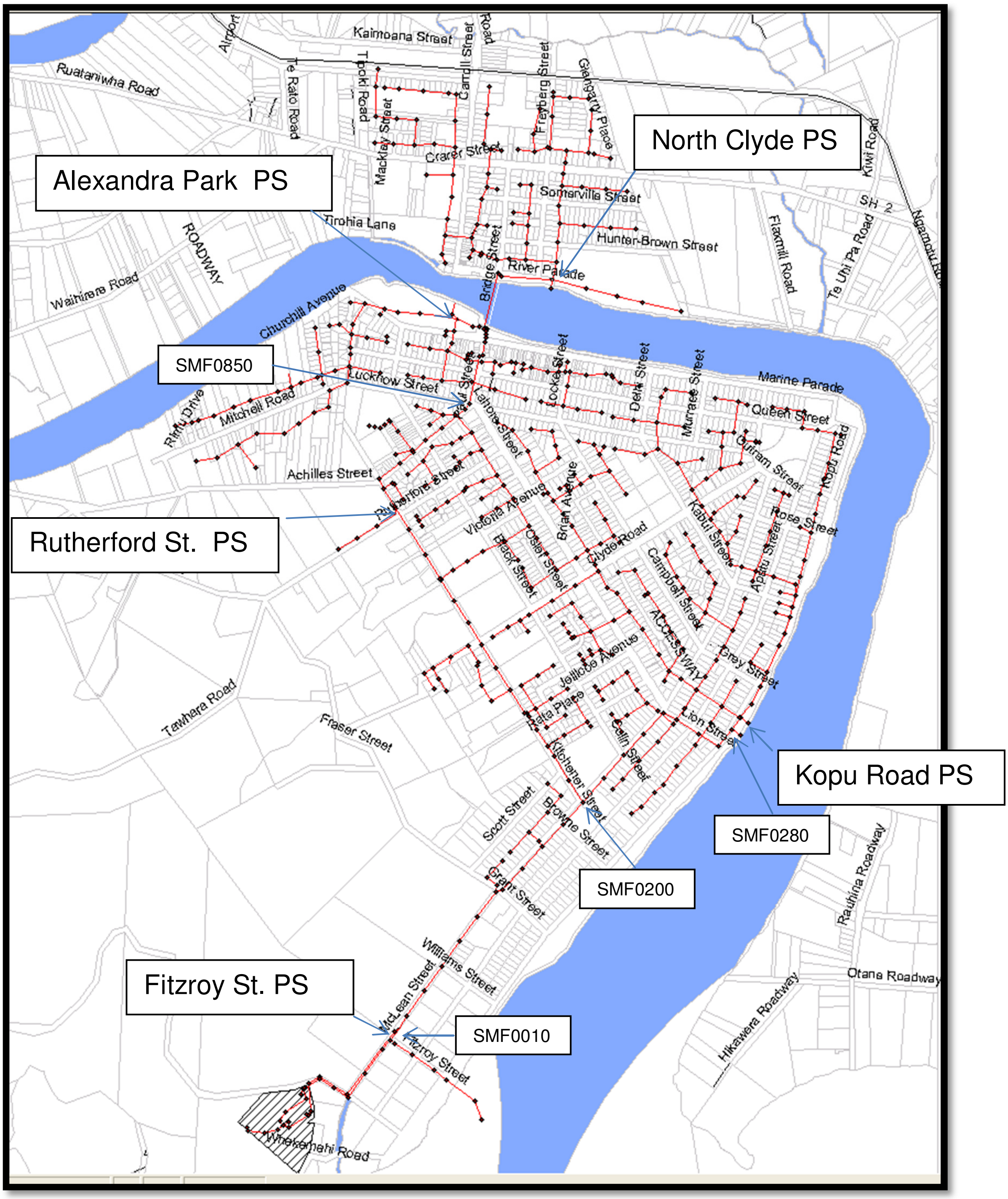


Figure 2 – Wastewater Service Area of Wairoa

2.2 Pump Stations

North Clyde PS has been built in 1949. It services the North Clyde area which is located on the northern bank of Wairoa River. The PS has two identical pumps Flygt 3102-180 with 430 impellers. The rising main from this pump station id 200mm in diameter and has a length of 400m which crosses Wairoa River under the road bridge.

Alexandra Park PS is identical with North Clyde PS, except that the rising main is 150mm in diameter and is shorter.

Kopu Road PS is the main pump station and services major part of the service area. Built in 1949, this pump station originally received flows from North Clyde and Alexandra Park pump stations in additional to its own gravity catchment and discharged the effluent into the Wairoa River. This has undergone complete changes in early 1980s. A common gravity main has been constructed to receive pumped flows from the three pump stations which transports the effluent to a new treatment plant located at the southernmost end. Kopu Road PS has three pumps, two identical Flygt 3102-180 with 430 impellers and one Flygt 3152 with unknown impeller.

Rutherford Street PS services just three properties and the flow from this pump station is considered insignificant for the stage 1 study.

Fitzroy Street PS has been constructed in 1980s as a lift station at the end of the gravity main to lift the flow to the sewer leading into the treatment plant. This pump station has four identical pumps (Flygt 3152-181 with 452 impellers. These four pumps operate simultaneously with the same start/ stop levels.

A schematic layout of the modelled pump stations is shown below.

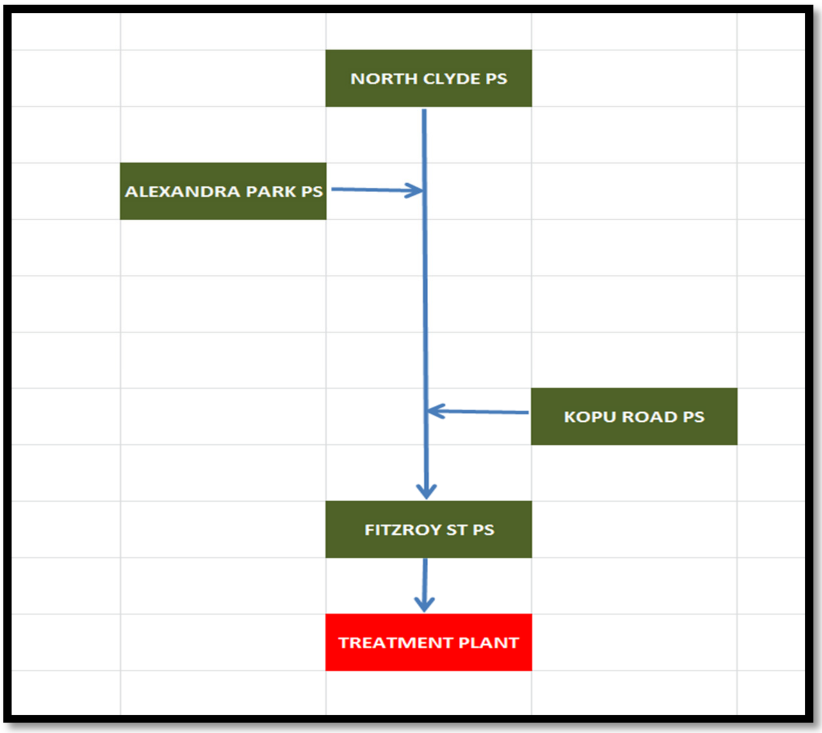


Figure 3 – Schematic Layout of the Modelled Pump Stations

Details of the pump stations are given in the table below. These details are extracted from the construction drawings supplied by WDC. Pumping rates were determined using the pump curves available from technical information supplied by the pump manufactures. (Note: It is possible that the actual pump rates are lower due to the wear and tear)

Table 1-Pump Station Details

Generic Name of PS	Pump Types	Number of pumps	Equivalent Wetwell Diameter (m)	Storage outside wetwell (m^3)
North Clyde PS	Flygt 3102-180 - 430	2	2300	20
Alexandra Park PS	Flygt 3102-180 - 430	2	2300	20
Kopu Road PS	Flygt 3102-180 – 430 Flygt 3152	2 1	5000	80
Fitzroy Street PS	Flygt 3152-181 -452	4	2900	-

Based on the information supplied by the WDC, it is evident that Kopu Road pump station to have frequent overflows during wet weather events. There is some information that suggests that overflows do occur at Alexandra Park and North Clyde pump stations during wet weather events, however, not as frequent as Kopu Road pump station. Information does not suggest that overflows do occur at Fitzroy Street.

3 Model Build

3.1 Asset Data

3.1.1 Manhole and Pipe Data

WDC supplied the manhole and pipe asset data in MapInfo Table file format. In addition, separate Excel spread sheets of the manhole lid levels and the depth to invert was supplied.

Manhole lid levels and invert levels in the MapInfo Table files were found to be inconsistent and incomplete. There were a lot of data gaps. The invert levels did not represent the true flow direction. The coordinates and the connectivity appeared to be of acceptable quality.

Manhole lid levels and the depth to invert supplied by WDC were good quality and therefore were used for the model build. However, the inverts of one of the most critical sewers, which is the gravity sewer downstream of Kopu Road Rising main to the trunk sewer (SMF0280 to SMF0200, 300mm in diameter) were found to be erroneous. Both the GIS and the spreadhseet were found to have the same asset data. Invert levels of three manholes were adjusted to reflect the direction of flow.

Manhole No.	GIS data supplied by WDC			Adjusted Invert (m) for model
	Lid Level (m)	Depth to Invert (m)	Invert level (m)	
SMF0280	4.885	-	-	4.40
SMF0270	4.400	1.5	3.900	3.90
SMF0260	3.910	1.6	2.310	3.50
SMF0250	4.058	1.7	2.358	3.00
SMF0240	4.341	1.8	2.541	2.541
SMF0230	4.010	1.9	2.110	2.110
SMF0220	4.072	2.0	2.072	2.072
SMF0210	3.946	2.1	1.846	1.846
SMF0200	3.748	3.9	-0.152	-0.152

3.1.2 Pump Station Data

Four as-built drawings of the pump stations (one drawing per pump station) were received. The drawings did not have a scale nor major dimensions. The details on the drawings were very sketchy. A narrative statement provided by the client together with the drawings suggested that some of the levels on the drawings are inconsistent.

Pump start / stop levels, high alarm levels were supplied by the Council. These levels were not consistent with the manhole and ground levels. For example, the top of the pump station wet well of Kopu PS as per the as-built records is 14.58m. However, the ground levels and the lid levels of the surrounding manholes as per the spread sheet supplied by WDC is approximately 4.8m. There was no information on the level datum used for the two level sets. It may be possible that the level datum used is different. Pump station levels were adjusted to ensure consistency. In this example, the invert levels of the pump station were dropped by 9.8m.

Pump types were also supplied with the narrative statement. The start / stop levels supplied by the WDC appear to have been based on the information supplied by the maintenance contractor.

3.2 Time Varying Data

WDC supplied the daily flow volumes to the treatment plant, pond water levels, daily rainfall depths and the daily water consumption. The data covered a period of approximately 2 years (2010 and 2011).

Telemetry data of North Clyde PS, Alexandra Park PS, Kopu Road PS and Fitzroy Street PS were requested from WDC at the commencement of the project. This data was not received until 22 November 2011. The data supplied has gaps and is not considered as best of quality. Stage 1 study has already been completed by then without the telemetry data. Due to the time constraints it was decided that Stage 1 be completed without using the telemetry data and this to be included in Stage 2.

It was our intention to use the telemetry data of pump start / stop times converted to a flow hydrograph with five minutes time steps to carry out a brief calibration of the pump stations. Due to the absence of these data in time, we were compelled to use the daily flow volumes at Fitzroy Street PS.

The figure below is a graph of two years of daily records at Fitzroy Street pump station. The top graph shows the treatment pond water level in metres (in black with scale on left Y axis), and daily inflow volume in cubic metres and daily dry weather flow in cubic metres (assumed as 80% of the water consumption) and the scale as shown on right Y axis.

The bottom graph shows the inflow volume in cubic metres (right Y axis) and the rainfall depths in millimetres (left Y axis).

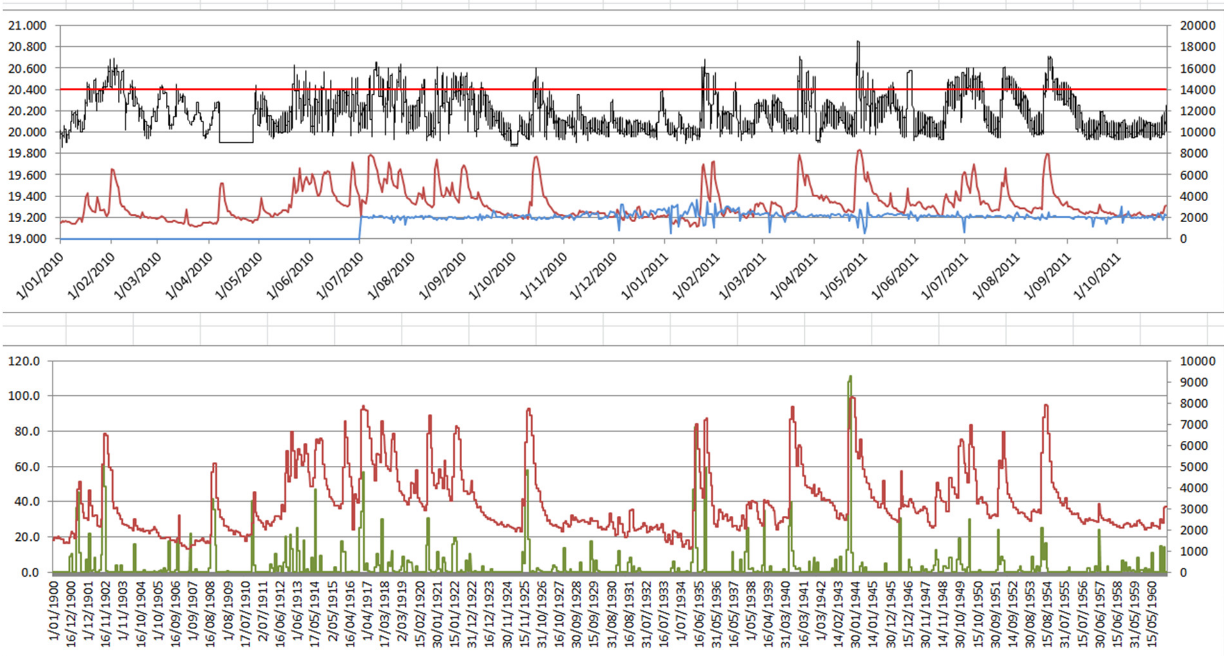


Figure 4- Daily flows at Fitzroy Street PS

It could be seen that the average daily dry weather flow is 1800 m^3 and the maximum daily wet weather flow during the two years is 9000 m^3. Thus there is an increase by five times from dry weather to wet weather flow volumes. This ratio for the peak flow rate will be lot higher and based on our experience in other projects, we would expect the peak wet weather flow <u>rate</u> to be about 6 to 10 times the peak dry weather flow rate. A ratio of 2 for the peak wet weather flow to peak dry weather flow rate is generally acceptable and is often used in Engineering Standards.

3.3 Model Build

Asset data was imported in to InfoWorks to build a network model. The manhole lid levels and invert levels supplied as an Excel spread sheet were used to replace the levels in the Mapinfo Table files. This was done as we found that the latter is more accurate and consistent.

The network was trimmed leaving only the network downstream of the pump stations. Catchment loading was based on the populations. It was assumed that number of people per household (pphh) is 4. Stage 1 Model extent is shown in figure below.

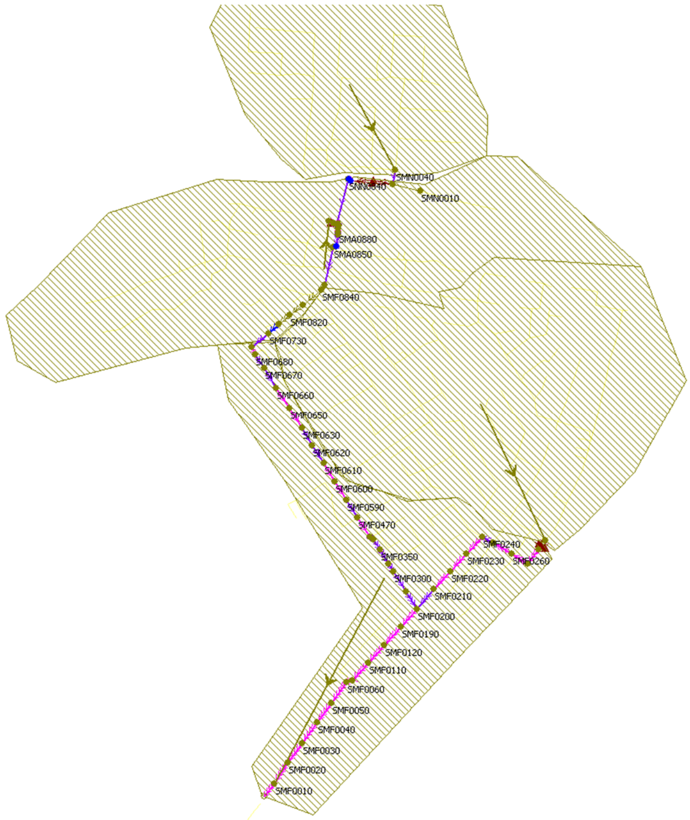


Figure 5 – Model Extent - Stage 1

Pumps were modelled as rotary pumps with the Head-Discharge relation. Rising main losses were simulated using Colebrook- White method with a friction value of 0.6mm.

Based on the flow measurements supplied by WDC, the average daily dry weather flow at the Fitzroy Street PS is 1800 m^3 which suggests that per capita wastewater discharge is 200 litres per person per day. A standard diurnal pattern for residential areas was used. This was based on flow measurements that were carried out by Opus in other similar towns.

The diurnal pattern used is shown below.

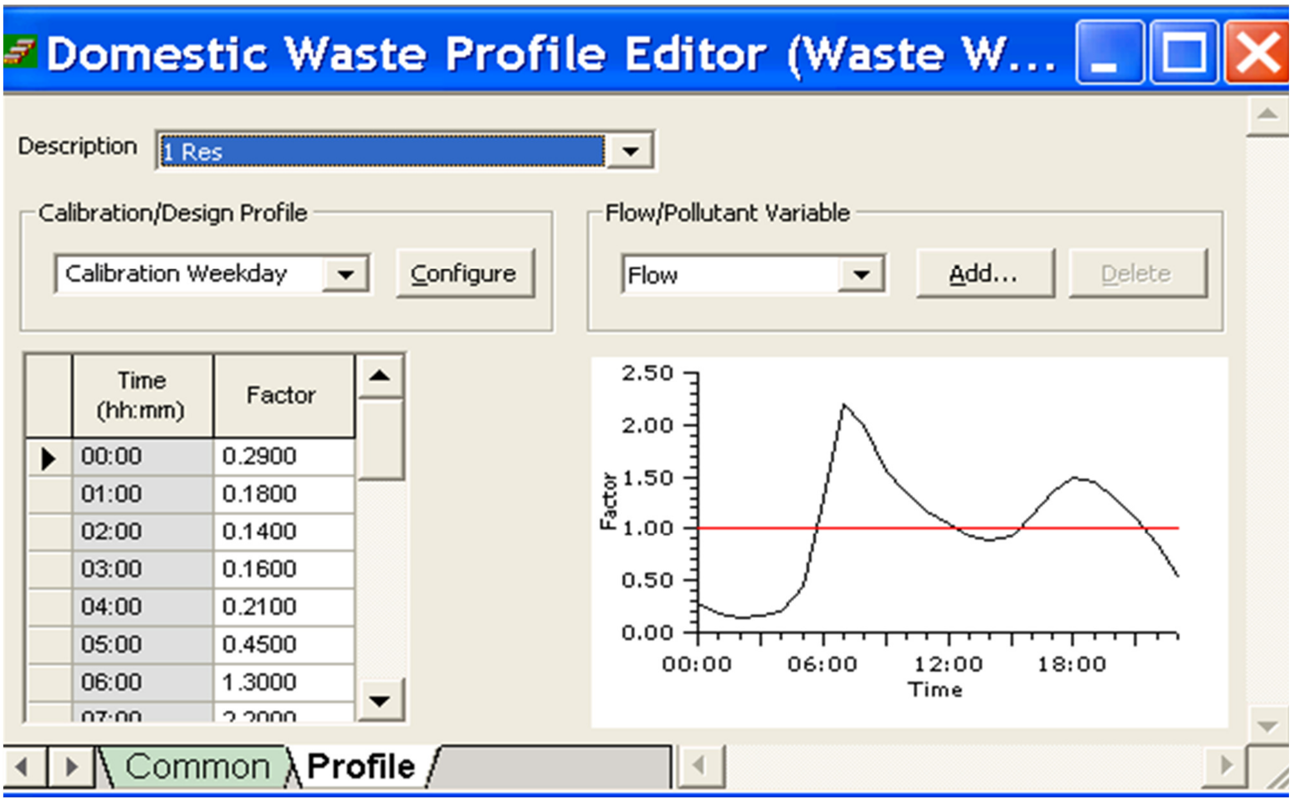


Figure 6- Dry Weather Flow Pattern

Pump station details used on the model are tabulated below:

PS name	Pump Type	Start Level	Stop Level	Equivalent wet well diameter
North Clyde	3102-180	1.06	0.66	2.3m
	3102-180	1.21	0.66	
Alexandra Park	3102-180	-0.09	-0.17	2.3m
	3102-180	0.71	-0.17	
Kopu Road	3102-180	2.76	2.51	5.0m
	3102-180	2.81	2.51	
	3152-181*	3.21	2.51	
Fitzroy Street	3152-181	0.304	-0.146	2.9m
	3152-181	0.304	-0.146	
	3152-181	0.304	-0.146	
	3152-181	0.304	-0.146	

*Denotes assumed

Rising main lengths and diameters in the model are based on the spread sheet data provided by the WDC. It is to be noted that some of the information in the narrative statement provided by WDC were contradictory. For example, the length and the diameter of the Kopu PS rising main in the spread sheet (and also GIS) is 43m and 200mm respectively. However, in the narrative statement, this is 128m and 300mm. We used the details as supplied by the spread sheet when any inconsistencies were found.

4 Network Performance

4.1 Scope

The main objective of the network assessment is to check the capacity of the pump stations in relation to that of the associated gravity sewers. It was identified that some of the pump station performance requires review as the capacity of the gravity system downstream is either too large or too small.

Performance was assessed under a regime of flow conditions from peak dry weather flow to five times the peak DWF. Based on the daily flow volumes measured at Fitzroy Street PS, the ratio of the peak daily wet weather flow volume to the dry weather flow volume is about 5. Therefore, a ratio for the peak flow rate to average dry weather flow in the range of 8 to 10 can be expected from the network. Therefore, the range of flow conditions investigated would cover the extreme wet weather flow conditions experienced by the network.

4.2 Model Results

Gravity Network:

In general, the gravity main from the North Clyde PS and Alexandra Park PS up to the connection point of the Kopu Road gravity main (SMF0200) has sufficient capacity for wet weather flows (4 x DWF) and the capacity of this section of the main is consistent with the pump flow rates. However, the section downstream up to the Fitzroy Street PS is under capacity.

It can be seen from the model outputs that the pipe full capacity (with abbreviation "r.pfc" on the hydraulic long sections in Appendix A) of the sewer from SMF0200 to SMF0010 is as low as 8 l/s, when the required capacity for peak dry weather flow is 58 l/s. It is to be noted that the average pipe full capacity is about 15 l/s. For wet weather flows this is severely under capacity. The capacity of the sewer from SMF0280 to SMF0200 is generally adequate for wet weather flow (equal to 2 x Peak DWF), except for one short length. However, the combined pipe full capacity of the entire length can be considered as adequate.

Thus the main issue of the trunk system is the inadequate capacity of the 450mm sewer from manholes SMF 0200 to SMF0010. This section however is relatively deep and therefore can afford to surcharge. Under surcharging conditions, it can pass flow over 140 l/s. With surcharging in the trunk sewer, the branch sewer from Kopu Road pump station (SMF0280 to SMF0200) is affected causing manhole overflows.

The hydraulic grade lines along the trunk mains are shown on Appendix A.

Pump Stations

Pump performance is summarised in the table below:

PS name	2 x DWF		3 x DWF		4 x DWF		5 x DWF	
	Peak Flow l/s	No of pumps in operation	Peak Flow l/s	No of pumps in operation	Peak Flow l/s	No of pumps in operation	Peak Flow l/s	No of pumps in operation
North Clyde PS	32	1	34	1	36	1	38	1
Alexandra Park PS	34	1	34	1	56	2	56	2
Kopu Road PS	50	1	98	2	98	2	140	3

Although the North Clyde PS has two installed pumps, only one is used for the range of flow investigated. Other pump stations utilize all the installed pumps as the wet weather inflow increases.

It is very important to cross check these flow rates with the actual rates. As the pumps are now several years old, their efficiency may have dropped resulting a reduced flow rate. It is possible some of these rates are as low as 50% (This will be done during Stage 3).

PS overflows

The only pump station overflow that operates for the range of discharges investigated are the Kopu Road PS. This begins to overflow for the scenario 5 x DWF. The peak inflow to Kopu Road PS for this scenario is 116 l/s and the peak outflow (pumping rate with all three pumps in operation) is 140 l/s. The capacity of the gravity sewer downstream of the pump station is approximately 60 l/s. This suggests that Kopu Road PS has excessive capacity which the downstream network cannot handle. The question is whether the actual pumping rate from the pump station with all three pumps in operation is 140 l/s or less. This will be investigated during Stage 3 of this study.

4.3 Capacity Optimisation

Options investigated:

1. Investigate a scenario with the Kopu Road PS peak capacity as 100 l/s which may reflect the actual pumping rate.
2. Upgrade the capacity of the gravity main from the Kopu Road pump station to manhole SMF0200.
3. Seal manholes along the gravity sewer in Option 2 above to allow surcharging and to prevent overflows
4. Upgrade the capacity of the gravity main from manhole SMF0200 to Fitzroy Street PS.

Option 1

This option is a theoretical exercise to investigate the performance downstream network in the event that due to wear and tear, the pumping rate of Kopu Road pump station has dropped down from the theoretical 140 l/s to 100 l/s.

Option 2

This will be an expensive option to upgrade 760 metres of the existing 300mm diameter pipe. An alternative to this is the Option 3.

Option 3

This option appears a cost effective option. This option was further investigated.

Option 4

Although the main cause that limits the capacity of the overall trunk system is the 450mm sewer downstream of manhole SMF0200, it is not considered as a cost effective option to upgrade this.

This section is relatively deep and therefore can afford to surcharge. Under surcharging conditions, it can pass flows over 140 l/s. With surcharging in the trunk sewer, the branch sewer from Kopu Road pump station (SMF0280 to SMF0200) is affected by the backwater causing manhole overflows. This surcharging is not causing a major problem as the surcharging significantly reduces at manhole SMF0210 (refer Figure 10 – surcharging at manhole due to downstream backwater is about 100mm only).

Thus, upgrading the downstream would not resolve the issues completely. This means that this option would not work alone and needs to be implemented with Option 3. This makes Option 4 the most expensive option.

Discussion of Option 3

Our approach to optimise the network capacity considers the following:

- Capacity of the pump station
- Capacity of the downstream network

The optimised solution for the network lies at the point where the upstream capacity (pump capacity) is equal to the downstream capacity (enhanced).

Kopu pump station has three pumps and under the extreme wet weather flow conditions, they theoretically pump 140 l/s. (compared to DWF x 5 of is 116 l/s). The actual pumping rate with all three pumps in operation may be close to 100 l/s (needs to be verified).

The maximum flow that the gravity sewer from SMF0280 to SMF0200 can pass without causing manhole overflows is approximately 60 l/s. There is a shortfall of 40 l/s. One of the options that was considered to enhance the capacity is to pressurise this line. This sewer does not appear to have any service connections. This 300mm diameter sewer is about 30 years old and is made of asbestos cement. Condition of the manholes and the pipe is not known. If the sewer is pressurised up to 2 metres, it can pass 100 l/s. Figure 7 and 8 below shows the schematics of the Kopu Road PS and the downstream network – Existing and Proposed respectively.

The condition of the assets still needs to be investigated. Hydraulically, pressurising the sewer to enhance the overall capacity is achievable. It would be necessary to remove the existing overflow to Wairoa River at the manhole SMF0280. This may be replaced with a high level overflow as a relief point if the ground levels in the vicinity permit. This high level overflow may come into action when the hydraulic head inside the sewer at manhole SMF0280 exceeds 2m.

The capacities of two main pump stations (North Clyde and Alexandra Park) are consistent with the peak flow from the servicing catchment. It is not necessary to reconfigure these stations at this stage. However, after detailed modelling in Stage 3 of the study, we may investigate the opportunity to optimise the network capacity in relation to the pump capacity. The pipe full capacity of the gravity main from SMF0850 to SMF0200 is adequate to service the upstream catchment. There are no “bottle necks”.

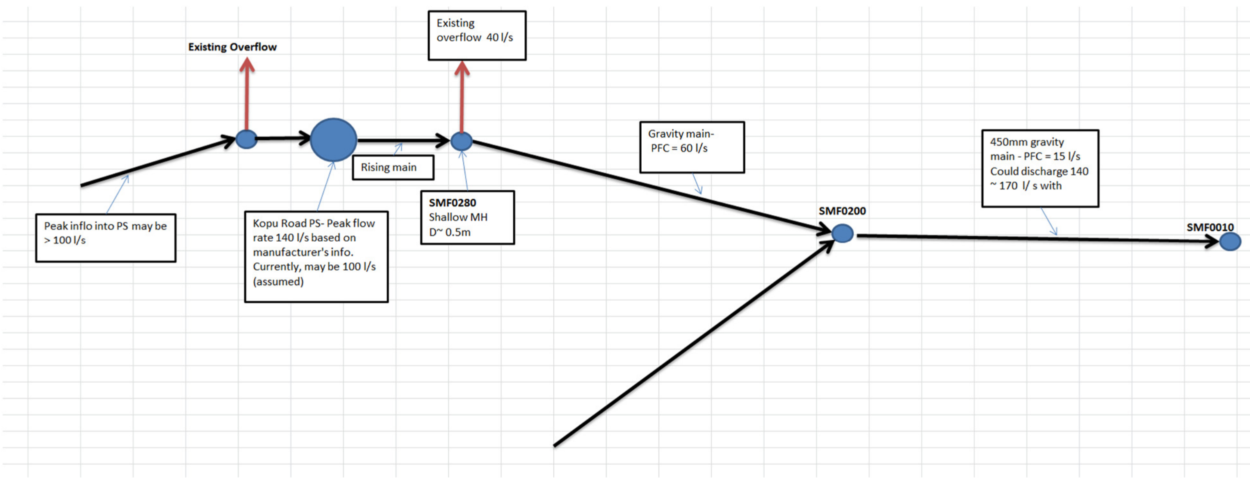


Figure 7 – Schematic of Kopu Road PS and downstream – Existing

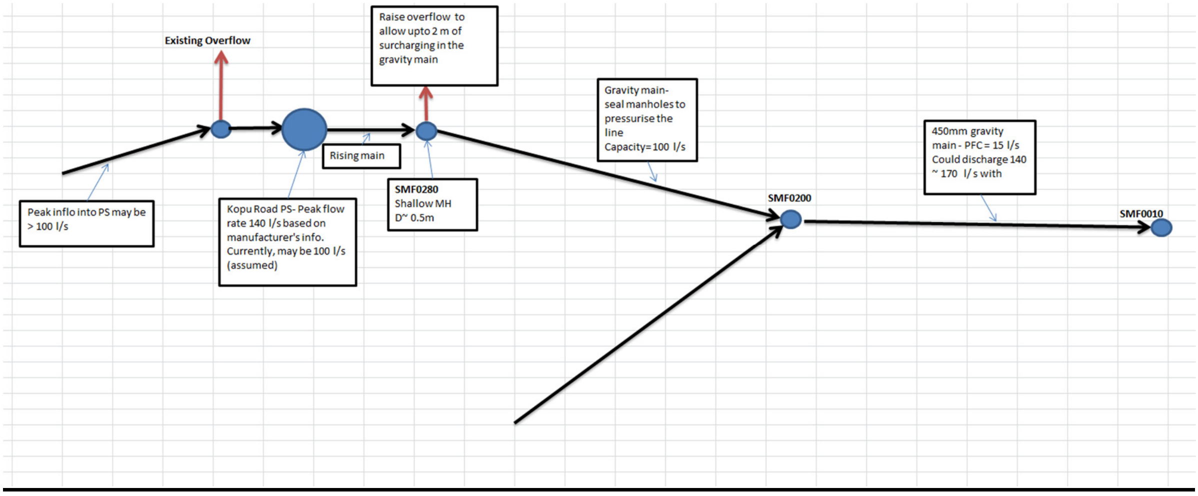


Figure 8 - Schematic of Kopu Road PS and downstream – Proposed

Impact on the treatment plant.

Containing the overflows with the implementation of Option 3 will increase the flow to the treatment plant. The additional flow to the treatment plant will be small compared to the total flow volume. For example, for 5 x DWF scenario, this will be less than 2%.

5 Stage 3 Methodology

Stage 3 is to extend the model to include the network within the pump catchments and identify upgrading requirements. Having carried out Stage 1, we are now in a better position to offer the Council an improved methodology which we believe will give a better outcome at a lesser cost.

The previous methodology consisted of two stages, Stage 2 and Stage 3. Stage 2 is flow gauging by an external flow gauging contractor and the Stage 3 is to calibrate the model. We are aware that the Council has installed magflow meters at the North Clyde, Alexandra Park and Kopu Road pump stations and we believe 6 months of continuous flow data will be available when we commence Stage 3 in January 2012. We propose to use these instead of the flow monitoring, giving the Council a cost saving.

Our amended methodology is based on the following assumptions:

- The magflow data is available and is of good quality. The time step is 5 or 6 minutes.
- Rainfall records are available at the same time steps or as a minimum, if the gauges are tipping bucket type with bucket 0.1mm, time at every tip is required.
- There are at least three rain events which had a total depth of 10 mm in one hour.
- Telemetry data of pump start / stop times of each pump for the same period is available (6 months from July to December 2011)

Our revised methodology is as follows:

We will convert the telemetry data (pump start / stop times) using a spread sheet programme developed by us in-house to generate an inflow hydrograph. We will use the magflow data to cross check the pump flow rates. The inflow hydrograph thus created will give the inflow pattern into each pump station. We will then use the derived inflow hydrograph to calibrate the pump catchment in a conventional manner.

This methodology will create additional work for us and we are happy to negotiate with WDC additional fees. We believe the additional fees will be much less than what the Council would have otherwise spent on flow monitoring.

6 Recommendations

It is recommended to investigate:

- The structural integrity of the sewer from manhole SMF0280 to SMF0200 and determine its ability to withstand hydraulic head up to 2 metres.
- A suitable site near or at manhole SMF0280 to provide a high level overflow into Wairoa River.
- The sewer from manhole SMF0280 to SMF0200 to confirm that there are no service connections.

It is also recommended to proceed into Stage 3 to determine the improvement options to the local collection system. As previously informed by WDC, we expect the newly installed magflow meters at pump stations to be operational. We propose to use these instead of the flow gauging proposed (Stage 2), provided that the data are suitable. We recommend that the data be examined to assess the suitability for model calibration.

Appendix A - Hydraulic grade lines along the trunk mains

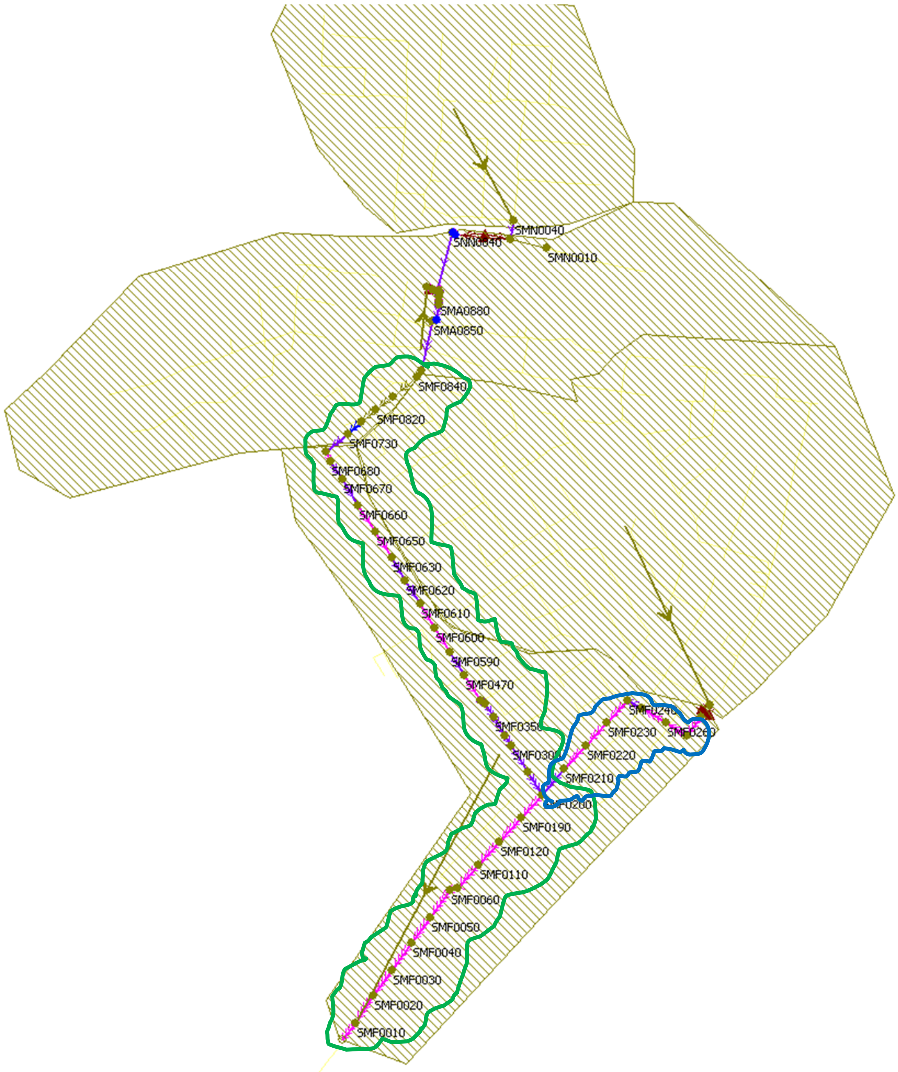


Figure 9 – Modelled Sewer Layout

The following long sections show the hydraulic grade lines along the two branches of the trunk sewers. The top long section in each figure is along the sewer shown in green and the bottom long section is that shown in blue.

It is to be noted that the hydraulic long sections represents the peak flow conditions of each scenario. For example, scenario DWF x 5 has a peak flow factor equivalent to 5 x 2.2 (from diurnal profile). In other words, 11x ADWF (average dry weather flow). Compare this with 5 x ADWF generally used for engineering designs.

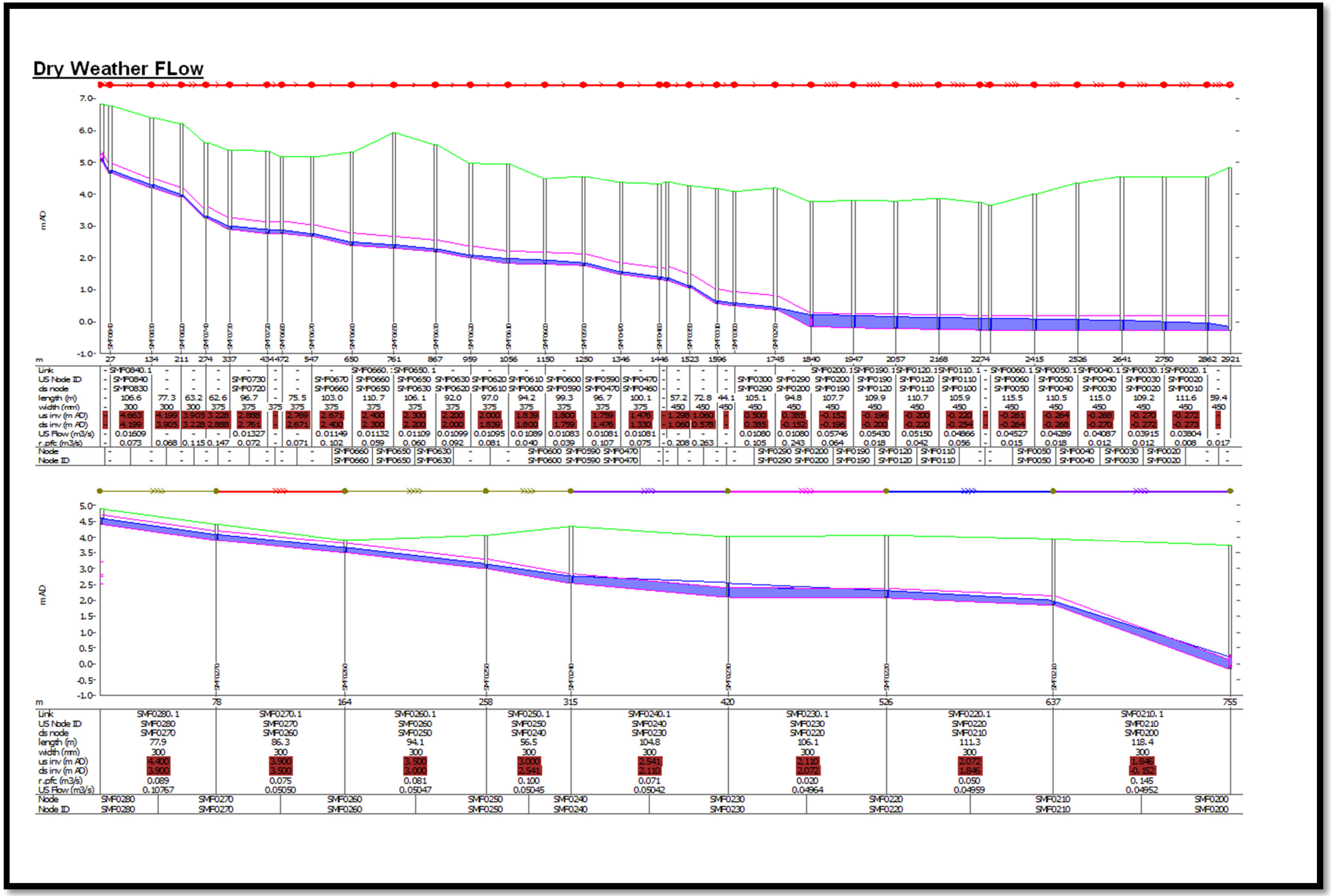


Figure 10 – Long Section – Peak Dry Weather Flow (DWF)

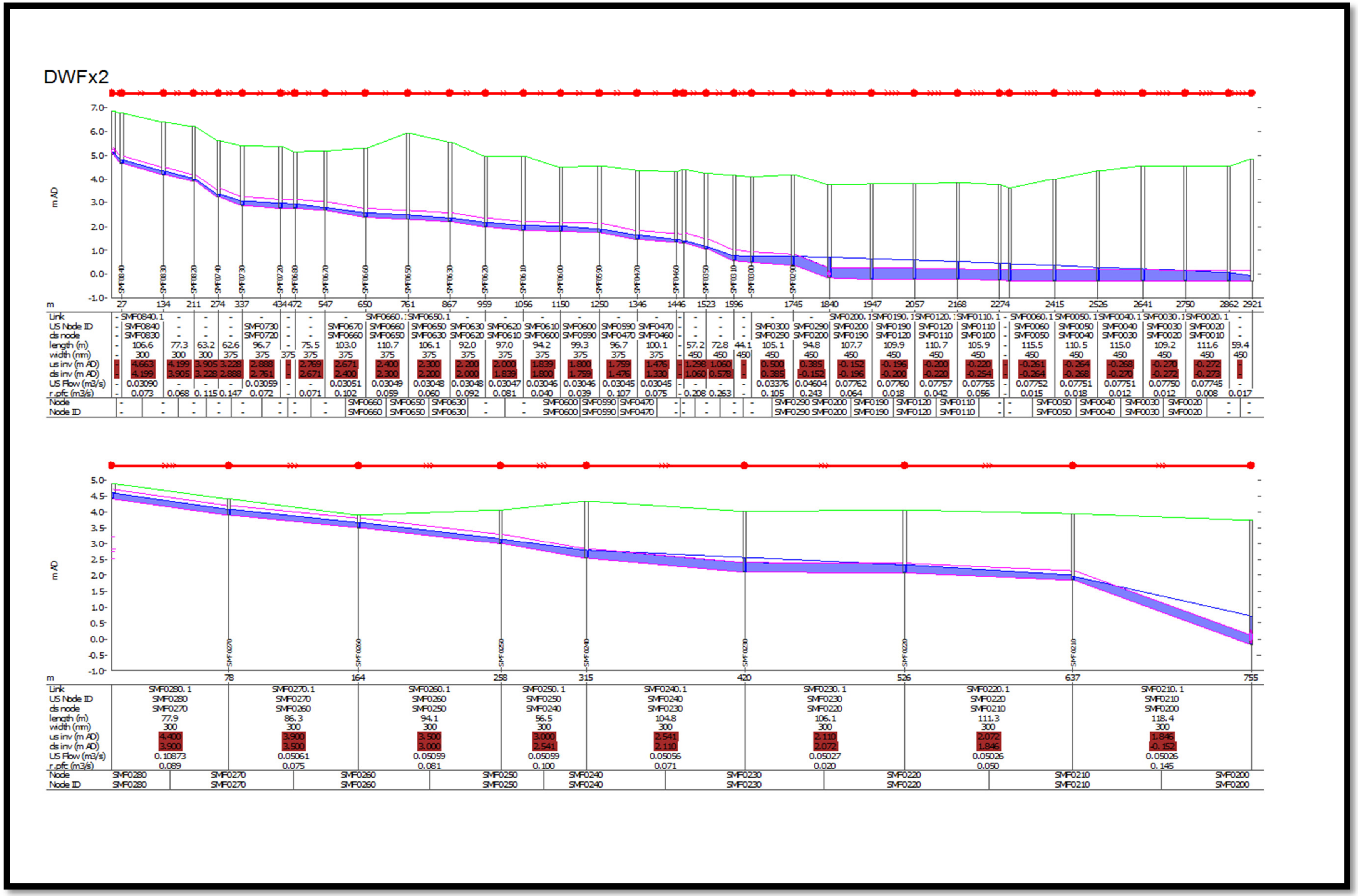


Figure 11 - Long Section – Peak Dry Weather Flow x 2 (DWFx 2)

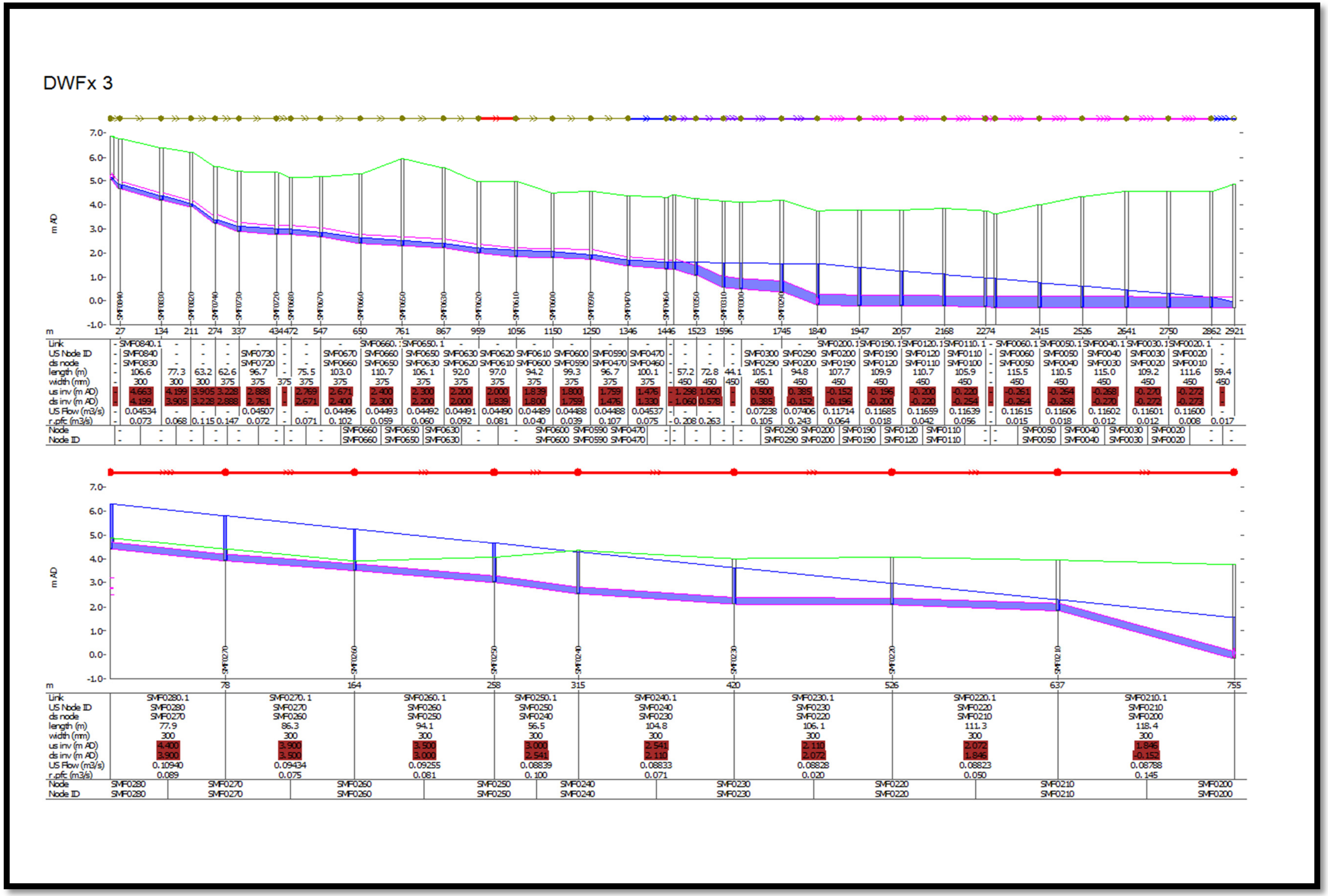


Figure 12- Long Section – Peak Dry Weather Flow x 3 (DWFx 3)

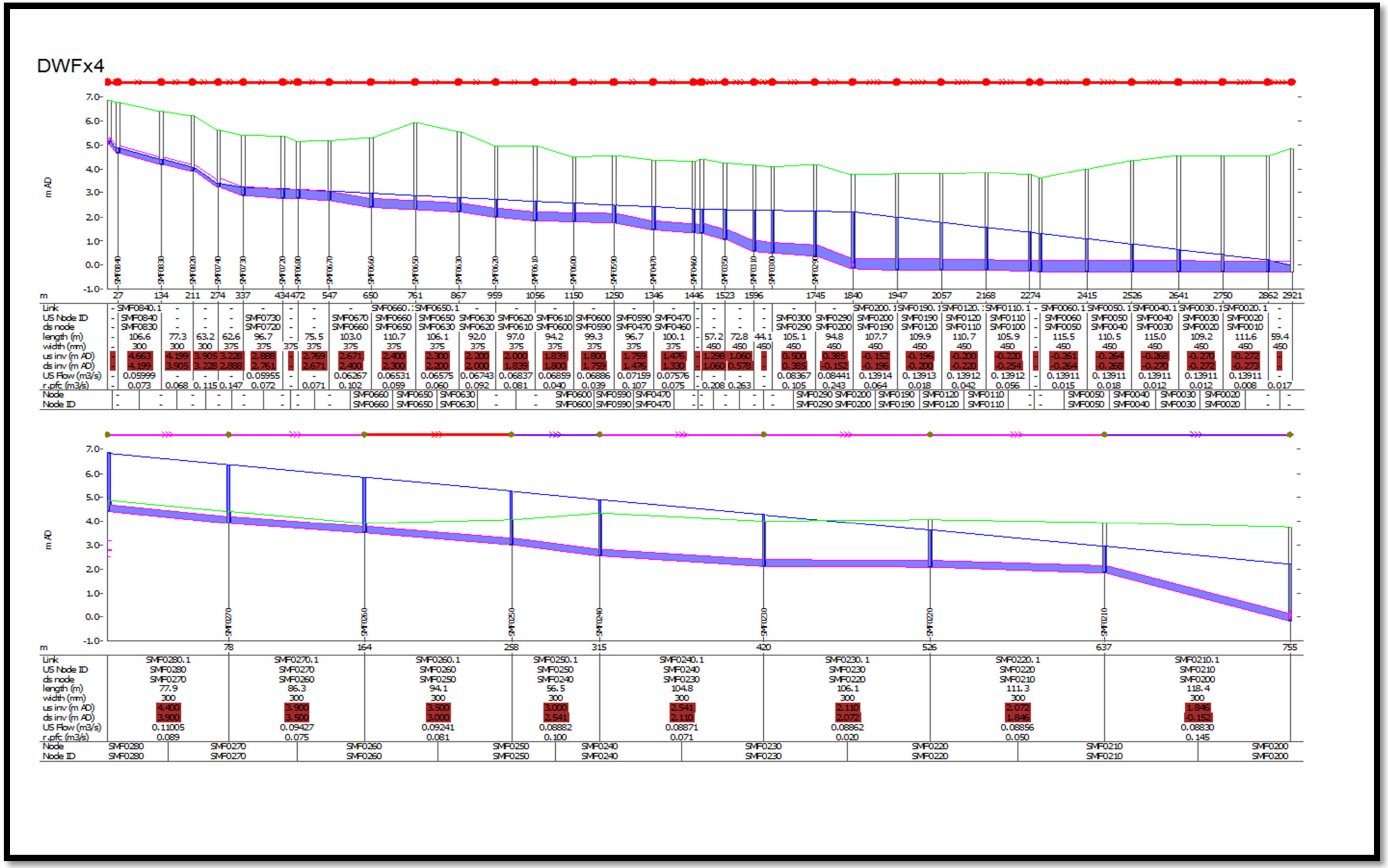


Figure 13 - Long Section – Peak Dry Weather Flow x 4 (DWFx 4)

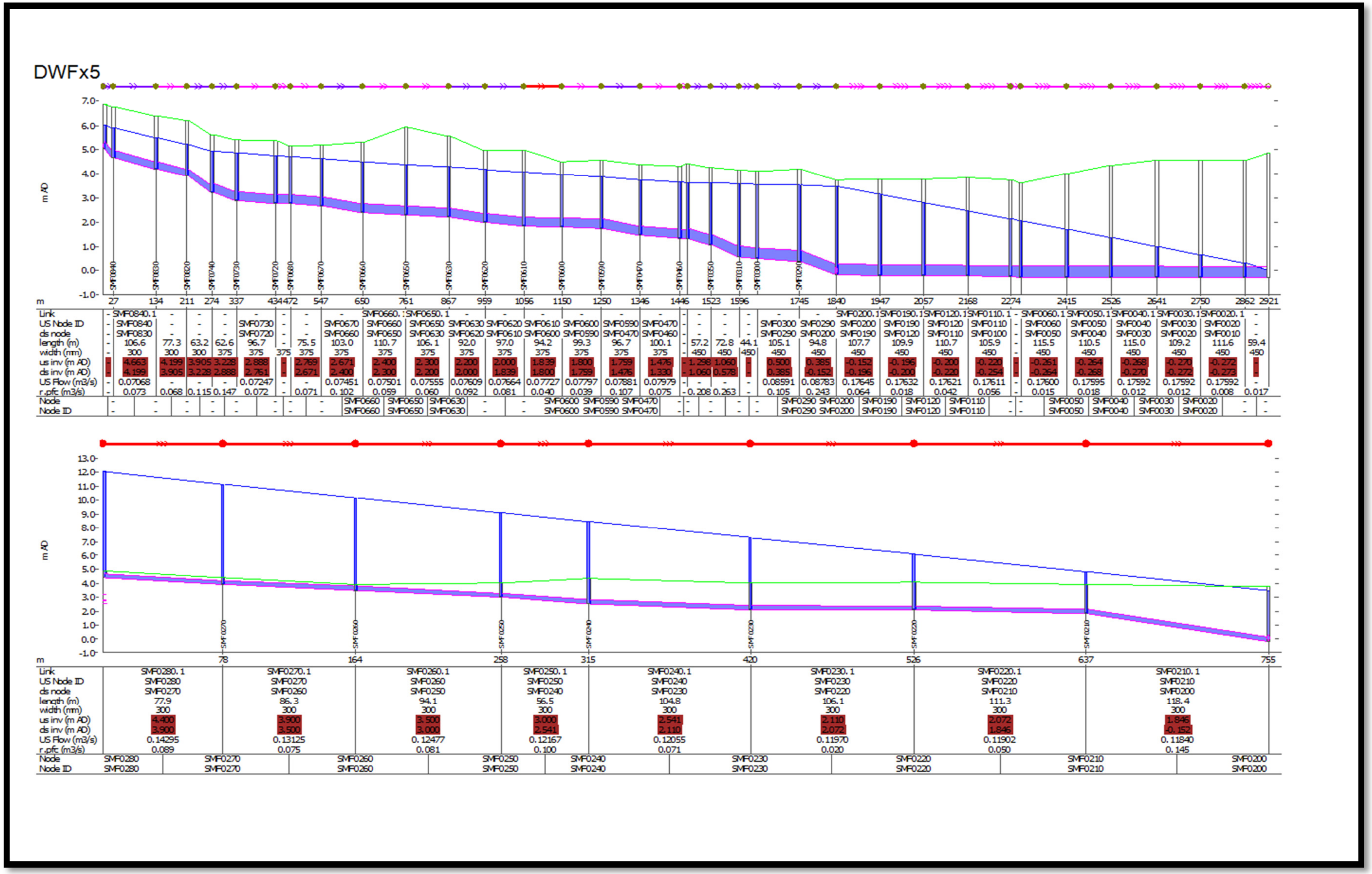
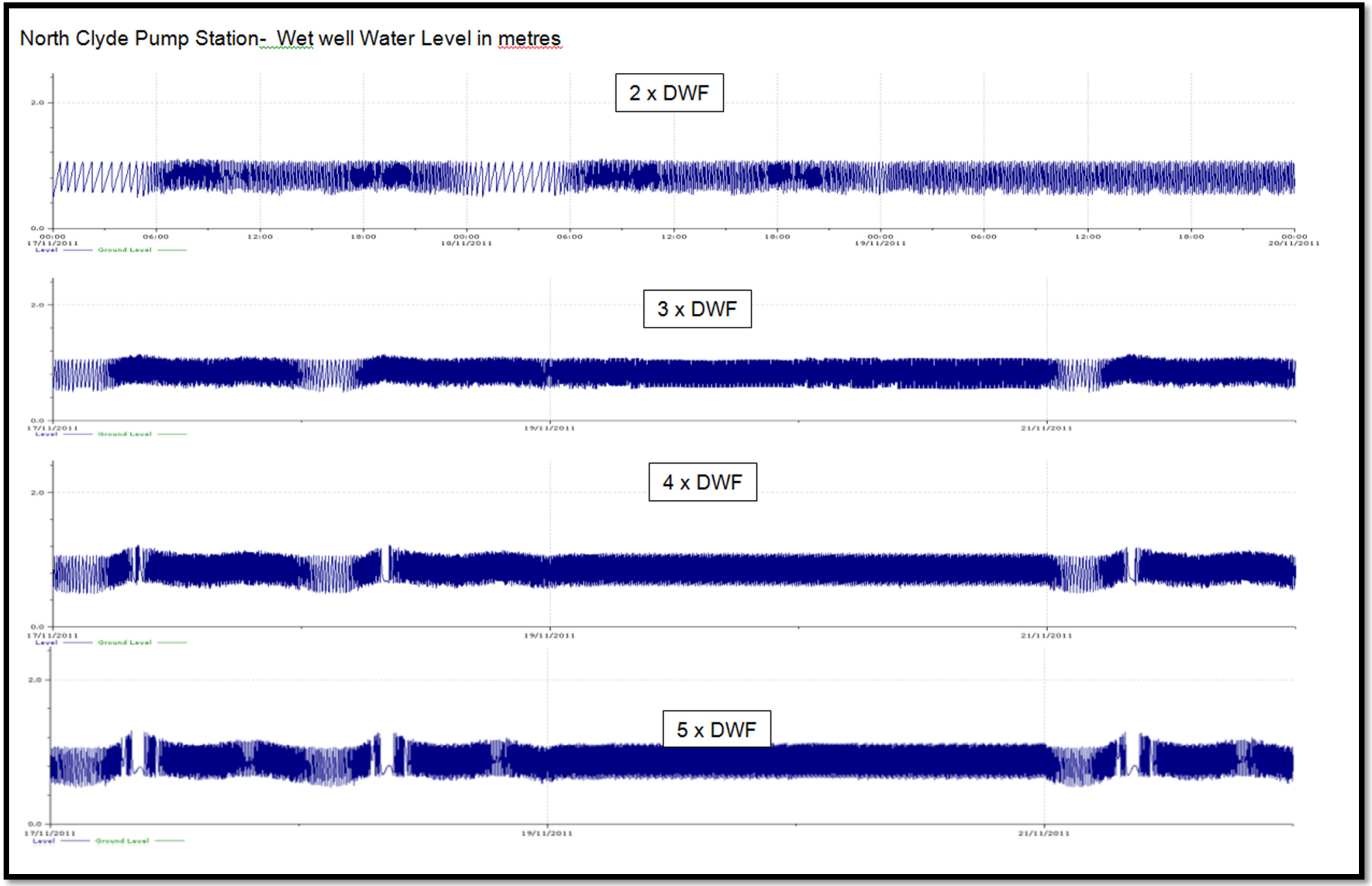
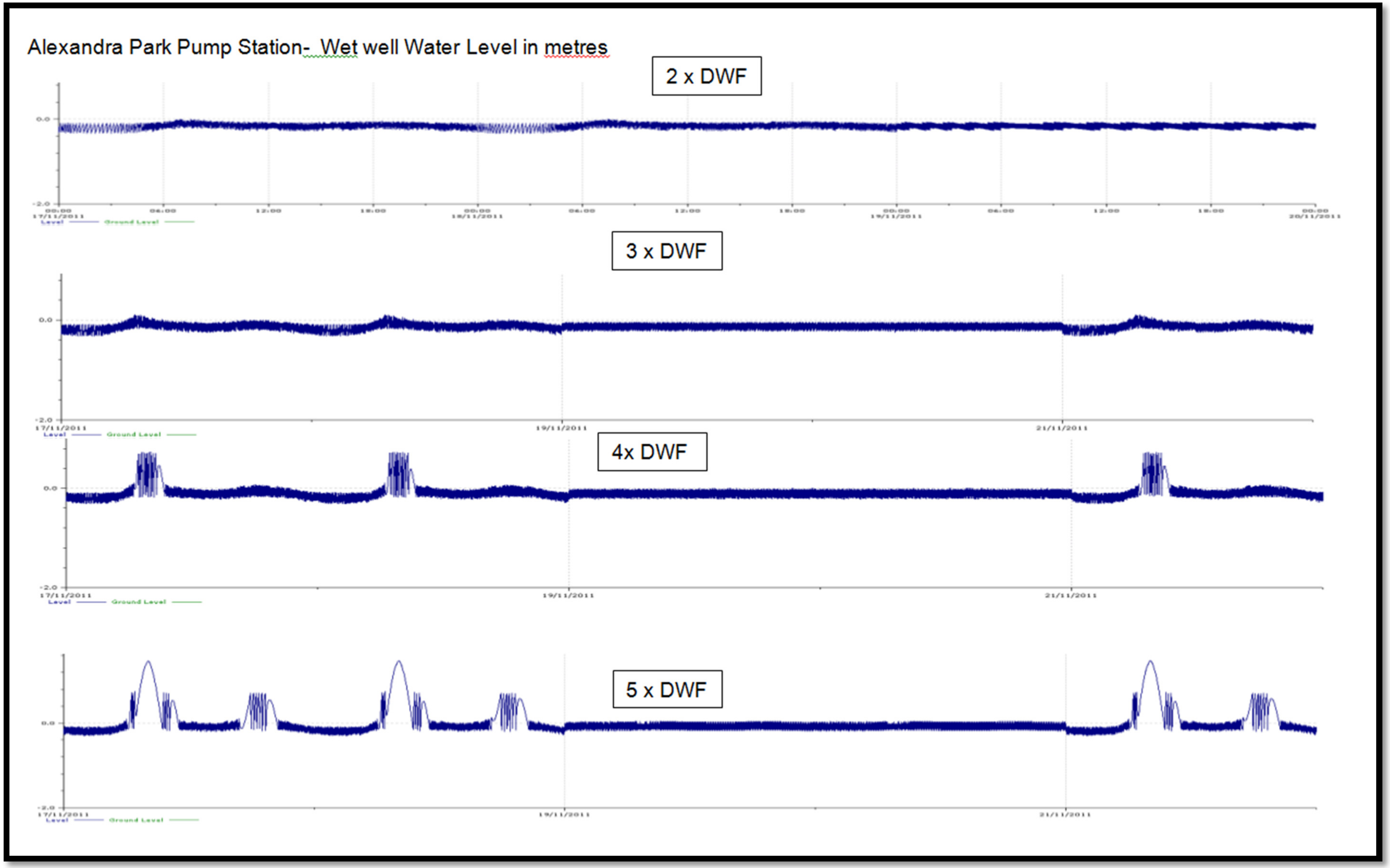


Figure 14 - Long Section – Peak Dry Weather Flow x 5 (DWFx 5)


North Clyde Pump Station- Wet well Water Level in metres
2 x DWF
3 x DWF
4 x DWF
5 x DWF
2.0
0.0
17/11/2011
19/11/2011
21/11/2011
Level
Ground Level


Alexandra Park Pump Station- Wet well Water Level in metres
2 x DWF
3 x DWF
4x DWF
5 x DWF

Kopu Road Pump Station- Wet well Water Level in metres