Appendix C:

Gisborne Wastewater Network Model Updates and Upgrade

Draft Report

Gisborne Wastewater Network Model Updates and Upgrades

Prepared for Gisborne District Council (Client)

Prepared by CH2M Beca Ltd (Beca)

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Revision History

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Document Acceptance

Action	Name	Signed	Date
Prepared by	Tracey Myers	T Myers	16/11/17
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on behalf of	CH2M Beca Ltd		

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Appendices

1 Introduction

1.1 Background

In the early 2000s, CH2M Beca (Beca) constructed a new wastewater network model using GIS data provided by Gisborne District Council (GDC). The model was used to review the effects of Inflow and Infiltration (I&I), and development. This model was calibrated against flow survey data in 2007.

In 2011, Beca wrote a report identifying upgrades required to achieve no sewer overflow events in a 2-year ARI (Annual Recurrence Interval) storm event. These upgrades were incorporated in the 2012-2022 LTP.

In 2014, Beca completed recalibration of the existing model. Part of the calibration process included updating the population in the model to include the 2012 census information. The model was then used to assess the storage volumes required for various return period rainfall events to contain the wastewater that would otherwise be discharged to the environment via Gisborne's emergency discharge sluice valves. The bulk reticulation that would be needed to transfer this wastewater directly to the WTTP was also determined. Both of these options were prohibitively expensive and did not address the sources of excessive stormwater entering the wastewater system.

Consequently in 2016, GDC refocussed the Wastewater Discharge Reduction Strategy, with the Drainwise Strategy being produced. The actions in the strategy are aimed at reducing the gross (direct) inflow of stormwater into the wastewater network by 85%. Key to achieving this target is substantially eliminating stormwater enterng the sewer via household gulley traps, stormwater down pipes and laterals.

Between 2014 and 2016 a number of improvements were made to improve the accuracy of the model. These included updating levels from survey data, adding in the Western Industrial network, and modelling the inlet to the Wastewater Treatment Plant to reflect the actual site details. These and other model improvements are outlined in Section 2.1.

Using the updated model further analysis has been undertaken with the targeted 85% reduction of gross stormwater inflow (termed "fast response" in the model), to determine if the wastewater network would remain under capacity with the reduced stormwater input. As per the Strategy, a two year ARI is the containment standard GDC wish to achieve. So the two year ARI is the design event the modelling is based on.

1.2 Purpose

The purpose of this report is to:

- Document the improvements that have been made to the model since the 2014 modelling work
- Outline the assumptions associated with the model and for specific scenarios modelled
- Demonstrate the capacity of the Gisborne wastewater network
- Assess the impact (remaining overflows) in the network with 85% of the gross inflows removed
- Test the impact in the network if the 85% gross inflow reduction is not achieved
- Propose and cost network upgrades to remove remaining modelled overflows from the network once 85% or 65% of the gross inflow is removed

2 Model Updates and Assumptions

2.1 Updates

The Gisborne wastewater network model was originally built in Mouse. Since being converted to InfoWorks in the mid-2000s it has been improved upon over the years. Further Improvements have been made to the model since 2014, including recommendations from an external model review in 2017. These model updates were necessary to correct previous inaccuracies in the original GIS data, reflect the current network and improve calibration for wet weather events.

The major model updates since 2014 are described below:

Gully Traps

A number of gully traps have been added to the model to enable modelling of some known on property flooding issues.

Pipes

There are a number of areas in the model that have been updated based on survey data. These areas include, Balance Street, De Lautour Road, Kara Street, Richardson Avenue, and Russell Street.

The levels in the siphons on Fitzherbert Street, Gladstone Road, Oak Street, and Peel Street have been updated from as-built drawings.

Pump Stations

Six pump stations have been added to the model. They are Aerodrome Road, Beetham Village, Dunstan Domestic, Dunstan Industrial, McDonald Road, and Sponge Bay. GDC provided a spreadsheet of the SCADA data as they have adjusted the pump start/stop levels in the pump stations. All of the pump stations have been amended to reflect this information.

Rising mains have been added into the network. They had previously been modelled as just a pump to the outlet manhole. The points where the pump meets the rising main have been amended to be break nodes instead of Manholes.

The rising main(s) servicing the Grant Road and Russell Street pump stations were updated. They were shown as connecting at the wrong location, and were not modelled with a shared rising main.

Steele Road pump station has had new pumps and a new rising main installed. These have been added in the model. Steele Road also has an emergency storage tank in case of power outages, and this has also been incorporated.

Bifurcations and Emergency Overflows

There are a large number of inter-catchment connections and bifurcations in the Gisborne wastewater network. These were previously modelled using levels from the GDC GIS data, which had the outgoing levels the same as the incoming. Surveys were carried out and the model has been updated to include the actual outgoing levels.

An inter-catchment connection at RUSSSM030 has been re-modelled as a closed sluice, after confirmation from GDC.

Inlet at Wastewater Treatment Plant

The inlet to the wastewater treatment plant had been modelled very simply, with all flows going to the inlet pump station, with no bypass. In reality, industrial flows enter the outfall pump station directly, and there is a bypass before the inlet pump station that allows flows greater than 450 l/s to enter the outfall pump station directly rather than go through the treatment plant. The model has been updated to correctly reflect this.

Sub-catchments

A number of sub-catchments were found to be connected to the wrong part of the sewer network. A model-wide assessment was completed using the laterals from the GIS, and any incorrectly assigned sub-catchments were amended.

The 2012 census was originally used to update the model populations. This was completed by assigning the mesh block data across the relevant sub-catchment areas. A review of the model suggested that the population would be better distributed based on the number of dwellings per sub-catchment. This has been completed for the whole model. The sub-catchment areas have also been adjusted to reflect the number of dwellings contained within them.

When the model was calibrated in 2007, five traders with significant flow had been included in the model. These were Bernard Matthews, Cedenco, Gisvin, Juken Nissho, and Tairawhiti Hospital. Based on the land parcel data and the addition of the western industrial wastewater network, the flows from Bernard Matthews and Juken Nissho have been reallocated to different manholes. Bernard Matthews previously entered a dummy node and now enters the Dunstan Road industrial pump station. Juken Nissho previously entered the Lytton Road industrial pump station, and now enters the McDonald Road pump station. The Cedenco, Gisvin, and Hospital flows remain unchanged.

No other trade flows had been included in the model. GDC requested that trade flows in the CBD be accounted for using the Code of Practice design flows. Commercial flows and types were identified using the GDC GIS database. The flow has then been calculated against the contributing catchment area of each premises.

During the 2014 calibration, the runoff area adjustments were applied uniformly across all catchments upstream of a flow monitor. Information was provided by GDC that some areas were affected by runoff more than others, e.g. upstream of the Oak Street pump station. These catchments were adjusted accordingly to reflect this information, leading to different areas upstream of a flow monitor having different runoff areas applied. The review of the model suggested that these areas should still be applied uniformly across the sub-catchments. This was completed for the affected areas.

The model was checked against the 2006 flow survey data and it was noted that a second peak occurs approximately 12 hours after the heavy rainfall and the fast response runoff peak. This occurs for 21 (out of 25) monitors. GDC confirmed that this matches reality where a number of properties suffer flooding, and after approximately 12 hours the floodwater drains into the sewer network via gulley traps. A rain-on-grid layer was supplied by GDC. After obvious rivers and streams were removed, duplicate sub-catchments were created in the network to specifically model the on-property flooding. Duplicate rainfall profiles were created with a 12 hour adjustment, and runoff area was added to the duplicate sub-catchments. These areas were then re-calibrated to reflect the information regarding the on-property flooding.

Other

A single sub-catchment had previously been used to model the western industrial area. GDC requested that this area be added into the model. This network includes the Aerodrome, Dunstan, and McDonald pump stations discussed above.

2.2 Assumptions

The following assumptions apply to the model in general. Assumptions related to specific scenarios are outlined in Section 3:

General

- All co-ordinates have been projected to the New Zealand Transverse Mercator (NZGD2000) co-ordinate system
- All levels have been converted to Gisborne District Council (GDC) Datum – an adjustment of + 4.106m from the Gisborne 1926 MSL Datum. Levels that are low spots or modelled as flooding have been checked against the LiDAR data and adjusted if necessary

Manholes

- Missing manhole cover levels have been calculated as 1.2m plus pipe diameter above the pipe invert level
- Unless based on survey data, manhole floor levels have been set to the default value (i.e- the level of the lowest connecting pipe)
- Manhole diameters have been assumed to be a minimum of 1050mm (apart from pump station wet wells)
- Flood type for nodes used to model rising main has been set to ‘Sealed’. All other flood types have been set to ‘Lost’ (i.e. water is lost from the system once the manhole cover level is exceeded)
- No storage compensation has been applied as no simplification has been undertaken

Pipes

- Missing invert levels at the very upstream end of the network were assigned based on a fixed cover level of 1.5m
- Missing invert levels have been determined by interpolation from upstream and downstream manholes
- Missing diameters have been determined by interpolation from upstream and downstream manholes
- Where inferred invert levels resulted in pipes which were unrealistically deep, or with insufficient cover, a more appropriate invert level has been assumed at the critical point, and affected invert levels re-adjusted
- For gravity sewers, the solution model has been set to ‘Full’. The manhole headloss coefficients have been assigned using the InfoWorks automated tool. The headloss type has been set to ‘Normal’
- Colebrook-White roughness coefficients have been determined based on the average condition for the pipe material: 1.5mm for cast iron pipes, 0.5mm for glass-reinforced plastic, 0.6mm for plastic pipes, and 3.0mm for asbestos cement, clay, concrete, earthenware, and glazed-earthenware pipes
- Missing pipe materials have been based on known pipe materials from upstream and downstream manholes

Pump Stations

- The majority of the pump flows are from the 2006 calibrated model or have been provided by GDC. The remainder are based on the 2014 calibration. The pump stations where the flows are estimated based on engineering judgement are Innes Industrial, Lytton Industrial, and Sponge Bay
- For rising mains, the solution model has been set to ‘ForceMain’. The manhole headloss coefficients have been set to 1. The headloss type has been set to ‘Fixed’

Property Ponding Flow

- The 12-hour delay in the property ponding flow has been assumed to be uniform across all of the areas where it occurs.

Subcatchments

- The removal of fast response is uniform across all sub-catchments that include it

3 Scenarios Modelled

3.1 4 and 6 x Average Dry Weather Flow

A first principles analysis was carried out to check the capacity of the wastewater network. It is understood that the network was originally designed to take 6 x ADWF in the upper catchments, and 4 x ADWF in the Interceptors. In order to model these scenarios two new networks were created, one for 4 x ADWF and one for 6 x ADWF. The ADWF and PDWF was calculated for each sub-catchment. The runoff area was then adjusted to create the difference between the PDWF and 4 or 6 * ADWF.

It has been assumed that all of the sub-catchments have runoff at either 4 * ADWF or 6 * ADWF, depending on the scenario being modelled.

3.2 Two Year ARI

A design rainfall event was created in order to run the model with a two year ARI storm. This storm event incorporates adjustments for climate change to 2051. A document entitled Tools for Estimating the Effects of Climate Change on Flood Flow was used to determine the adjustments to use for Gisborne. The document suggests an increase of between 0.5 and 2.2 ºc, with an average of 1.3 ºc. This increase uses 1990 as the starting point and no allowance has been made to start from the current year. Therefore, the design rainfall is conservative for the anticipated future average temperature increase.

HIRDS v3 was then used to obtain the current estimated rainfall depth for a two year ARI 24 hour duration storm event. A temperature increase of 1.3 ºc was used to obtain the 2051 rainfall depth.

The TP108 pattern was used to apply the rainfall depth across the 24-hour period. This rainfall event was then added to the model.

It has been assumed that the two year ARI event will fall the same across the entire catchment.

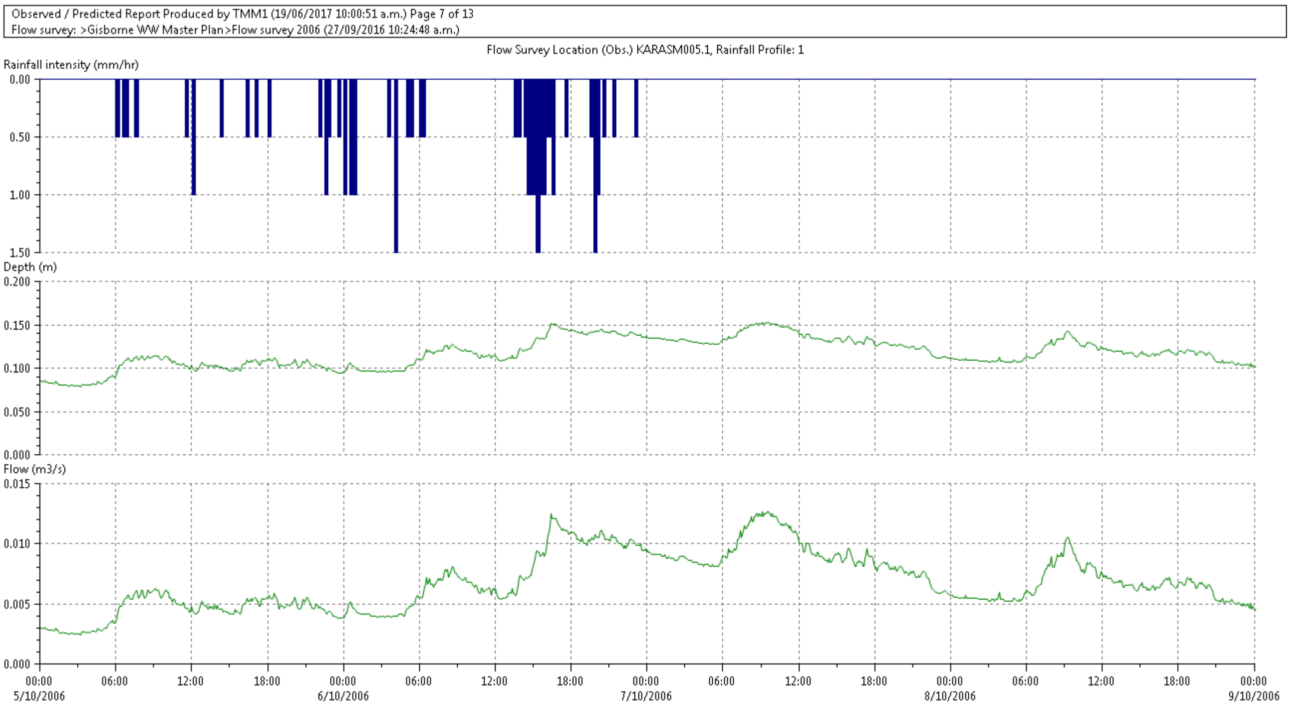
3.3 Removal of Stormwater

3.3.1 85% Fast Response Inflow

GDC have an overall objective of removing 85% of direct inflows from the wastewater system. In order to model this, 85% of the runoff area was removed from the runoff areas included in the sub-catchments. This was undertaken for all sub-catchments that illustrate the fast response inflow.

3.3.2 Property Ponding / Flooding

As explained in Section 2.1 review of flow gauging records during rainfall events revealed that in many areas a second flow peak is observed hours after the initial peak wastewater flow. An example of this is shown in Figure 3.1 below for the flow gauge installed at the junction of De Lautour Road and Kara Street.



	Rainfall			Depth		Flow		
	Depth (mm)	Peak (mm/hr)	Average (mm/hr)	Min (m)	Max (m)	Min (m3/s)	Max (m3/s)	Volume (m3)
Rain	7.511	1.500	0.078					
Observed				0.078	0.153	0.002	0.013	2371.551

Figure 3-1 Flow Gauge Showing Second Flow Peak

Analysis of the flow gauges that demonstrated the secondary peak revealed they were all down stream of areas where flooding and ponding on private property were known to occur, often above household gulley trap levels. Figures 3.2 and 3.3 provide photographic evidence of such ponding.

Figure 3-2 Ponding on Private Property during a 1 in 10-Yr ARI Event

Figure 3-3 Height of Flooding During the Same Event (indicated by the mop)

It has been postulated that, hours after the main rainfall event and peak flow, many properties are draining the ponded stormwater through their gulley traps, resulting in the observed secondary peaks. Removal of 85% of the fast response removal explained in Section 3.3.1 was not accounting for this phenomenon, even though in effect it is a direct inflow.

Removing 85% of the property ponding flow contribution is consistent with GDC's overall objective to remove 85% of direct inflows from the wastewater system. Therefore to allow for this 85% of the runoff area was also removed from the subcatchments representing the second peak.

It has been assumed that 85% removal of the property ponding flow will be achieved by GDC's private property flooding reduction initiatives.

3.4 Future Growth Projection for 2051

The population in the current model is based on the 2012 Census. The modelled population is 31,311. GDC provided a spreadsheet including the projected growth to 2051. The population in 2051 is expected to be 41,228, an increase of 9,917. The areas that are forecast to have the largest increase in population are Taruheru and Wainui. The Taruheru Block population has been increased from 1,606 to 5,935.

A new scenario was created with the population updated to the 2051 projection. With 100% of the gross inflows entering the network, the increase in wastewater flow causes flooding in three new areas (six overflows in total), compared to the two year ARI event with the current population (refer Section 4.2).

The new flooding areas are where the Wainui subdivision connects into the network, where the Taruheru block connects into Campion Road and the Back Ormond Road catchment. With 85% of gross inflows removed, the only remaining flooding area is the Back Ormond Road pump station.

As the population increase makes minimal difference to wet weather wastewater flows, the overflows and upgrades (in Sections 4.2 and 5) are based on the current scenario, rather than the future growth projection.

4 Model Results and Outputs

4.1 Pipe Grades and Velocities

Figures A1 and A2 in Appendix A show that a large proportion of the Gisborne wastewater network including interceptors is very flat, with grades of less than 1:200.

Due to the flat grades, wastewater velocities are very low with many areas less than 0.5m/s even during the 2 year ARI event (refer Figures A3 - A6). At velocities less than 0.5m/s, solids in the wastewater will settle out in the pipes, effectively reducing the capacity and increasing the likelihood of network overflows.

It should also be noted that in many instances upgrading existing pipe sizes will not provide significantly more flow capacity. Velocities will decrease and the larger pipe sizes will effectively provide more storage within the network.

4.2 Network Overflows

4 & 6 x ADWF

No overflows occur at 4 x ADWF.

Two manholes flood in the 6 x ADWF Scenario. Figure B1 in Appendix B shows the location of the modelled flooding.

One location is Back Ormond Road pump station, which has a large development (Taruheru) entering it directly. This flooding is unrealistic as in reality the development will have a network providing attenuation prior to the pump station -ie- the peak flow in the model is likely to be reduced in reality. The GDC GIS does not show any reticulation for Taruheru however. This likely explains the modelled overflow at the Back Ormond Road pump station for the 2051 growth projection scenario.

The other overflow location is 203 Stanley Road, which is a known flooding location. The modelled volume of this overflow is only 0.5 m^3.

These results show that the Gisborne wastewater network has been designed adequately to convey 6 x ADWF.

2-Yr ARI Storm Events

Table 4-1 shows the number of flooding manholes and the total flood volume for the two year ARI event, with and without 85% of gross inflows removed. Flooding occurs over a 90-hour period. Figures B2 to B4 show the location of the flooding.

Table 4-1 - Number of Flooding Manholes

	2 Year ARI All Gross Inflow Modelled	85% of Fast Response Removed	85% of Property Ponding Flow also Removed
Number of Flooding Manholes	154	29	4
Total Flood Volume (m^3) over whole event	56,017	4,318	95

Table 4-2 shows the flood volumes from manhole which have the top 10 flood volumes for any of the above scenarios.

Table 4-2 - Flood Volumes for Top 10 Overflows (any scenario)

Manhole ID	All Gross Inflow Modelled (m^3)	85% of Fast Response Removed (m^3)	85% of Property Ponding Flow also Removed
ADARSM010	1,433*		
BLCESM005	1,907*		
CHEESM005	618	299*	
FOX_SM035	4,161*	47	
GANTSM015	273	251*	
HALLSM010	3,948*		
HETASM010	552	265*	
HRGASM020	575	124*	
HURASM005	1,055	488*	42
INUISM025	821	231*	
INUISM060	339	123*	
ORMDSM030	1,396*		
RIVRSM005	2,253*	916*	0.1
RIVRSM020	459	456*	
RUSSSM065	3,440*		
RUSSSM085	1,897*		
RUSSSM090	2,251*		
RUTESM060	1,056	57	53
RUTESM085	1,290	0.5	0.4
STAFSM015	1,562*		
TUKUSM010	947	398*	
Flood Volume of Top 10 overflows over whole event	**24,248**	**3,656**	**95**

*Top 10 flooding volume manholes for each scenario

These results illustrate the significance of gross stormwater inflows including the on property ponding with respect to total wet weather wastewater flows and overflow volumes.

4.2.1 Sensitivity Analysis

If 85% gross inflow removal is not achieved other scenarios have been considered to provide a sensitivity analysis. Model simulations were undertaken to predict the number and volume of overflows if only 65% or 75% reduction was achieved. This includes removal of 65% or 75% of the property ponding secondary peak flows as explained previously.

If 75% of gross inflows were removed, 20 modelled manholes flood with a total overflow volume of 1,534m^3. Figure B5 shows the location of these flooding points.

If 65% of gross inflows were removed, 38 modelled manholes flood with a total overflow volume of 5,661m^3. Figure B6 shows the location of these flooding points.

4.3 Wastewater Treatment Plant Inflows

The Gisborne wastewater treatment plant is designed to treat a maximum domestic flow of 38,880m³/d (a constant 450 l/s over 24 hours). Currently, any flows above 450 l/s flow over a weir and enter the outfall pump station directly, effectively bypassing the treatment process. The treatment plant daily volume resource consent limit is 33,000m³/d.

The updated model was also run to estimate flows entering the WWTP for various scenarios. For the two Year ARI modelled flows, approximately 39,747m³ enters the treatment plant during a 24 hour period, with about 30,598m³ of network spills in the same period.

Table 4-3 shows the modelled inflows to the plant and the network spills associated with each of the gross inflow removal scenarios.

Table 4-3 - Wastewater Treatment Plant Inflows

ARI	85% Gross Inflows Removed		75% Gross Inflows Removed		65% Gross Inflows Removed	
	Inflow to WWTP (m^3)	Network Spills (m^3)	Inflow to WWTP (m^3)	Network Spills (m^3)	Inflow to WWTP (m^3)	Network Spills (m^3)
2yr	16,563	95	21,192	1,534	28,578	5,661
5yr	19,334	430	24,387	3,617	30,027	6,085
10yr	21,145	1,006	26,525	5,636	30,944	8,867

It should be noted that if network upgrades were implemented to contain all spills for each of the scenarios in Table 4.3 then the total daily volume to the treatment plant would be the sum of the “inflow to WWTP” and “Network Spills”(ie- what were spills are conveyed to the treatment plant).With the above scenarios, this shows that only one will exceed the plant capacity (65% removed with a 10yr ARI). However, all of the 65% scenarios exceed the current consent limit.

Network upgrades (refer Section 5) to remove overflows and convey all flow to the WWTP are only being investigated for the adopted 2 year ARI containment standard.

5 Network Upgrades

5.1 85% Gross Inflow Removal Upgrades

Table 5-1 shows the upgrades applied in the model that results in the four overflows being removed for the 85% gross inflow reduction (including property ponding flows) scenario. Figure C1 in Appendix C shows the location of the modelled upgrades.

Table 5-1 - Network Upgrades if 85% Gross Inflow Reduction Achieved

Location	Upgrades		
	150mm Pipe to 225mm	Number of New Manholes Required	Other
Harris Street	64m (HARISM055 to HARISM050)	2	
Rutene Road	N/A	2	Construct a 150mm cross-connection from RUTESM060 to RUTE-ISMH09 (Rutene Interceptor)
Riverside Road	N/A	1	Construct a separate rising/falling main (513m rising, 282m falling main both 150mm diameter)*

* Russell Street PS currently shares a rising main with Grant Road PS, with the flow from Grant Road taking precedence. Russell Street PS can achieve a flow of 17 l/s when it runs without Grant Road PS. However, during a two year ARI storm Russell Street would need to run for approximately 8 hours, while Grant Road was not operational. A separate rising main would therefore be required to allow both pump stations to continue pumping at the same time at the required rates during such periods.

5.2 75% Gross Inflow Removal Upgrades

Table 5-2 shows the upgrades applied in the model that results in the 20 overflows being removed for the 75% gross inflow reduction (including property ponding flows) scenario. Figure C2 in Appendix C shows the location of the modelled upgrades.

Table 5-2 - Network Upgrades if 75% Gross Inflow Reduction Achieved

Location	Recommended Upgrade				
	150mm Diameter to 225mm	200mm Diameter to 225mm	225mm Diameter to 300mm	300mm Diameter to 375mm	Other
a) Upstream of Grant Road PS					Increase pump rate to 10 l/s. Current flow is 7.7 l/s, and the shared rising main is 150mm diameter (looking at constructing separate rising main for Russell St PS)
b) Cambridge Terrace	123m (OXFDSM005 to RANFSM055)				
c) Rutene Road					Construct two 150mm diameter cross-connections (RUTESM095 to RUTE-ISMH14, and RUTESM060 to RUTE-ISMH09) 47m length total
d) Harris Street	205m (HARISM070 to HARISM050)				Abandon connection from HETASM010 to HETASM005
e) Fox Street	148m (FOX_SM065 to RUSSSM030 and RUSSSM015 to SPS015) – partial cost is renewal	156m (RUSSSM030 to RUSSSM015)			
f) Riverside Road					Increase Russell St PS flow to 25 l/s. Construct a separate rising/falling main (513m rising, 282m falling main both 150mm diameter)* Current flow is 17l/s.
g) Adair/Russell Streets	343m (WHITSM055 to ORMDSM085) – whole cost is renewal!!				
h) Clifford/Hall Streets	71m (HALLSM010 to ORMDSM075		443m (ORMDSM085 to ORMDSM070)		
i) Russell Street	319m (RUSSSM062 to RUSSSM060 and WHITSM025 to SHENSM005)				Abandon connection from RUSSSM085 to RUSSSM080
j) Stafford Stree	203m (STAFSM010 to STOUSM025)				

* Russell Street PS currently shares a rising main with Grant Road PS, with the flow from Grant Road taking precedence. Russell Street PS can achieve a flow of 17 l/s when it runs without Grant Road PS. However, during a two year ARI storm Russell Street would need to run for approximately 8 hours, while Grant Road was not operational. A separate rising main would therefore be required to allow both pump stations to continue pumping at the same time at the required rates during such periods.

5.3 65% Gross Inflow Removal Upgrades

Table 5-3 describes the upgrades applied in the model that results in the 38 overflows being removed for the 65% gross inflow reduction (including property ponding flows) scenario. Figure C3 in Appendix C shows the location of the modelled upgrades.

Table 5-3 - Recommended Upgrades after 65% Gross Inflow Removed

Location	Recommended Upgrade				
	150mm Diameter to 225mm	200mm Diameter to 225mm	225mm Diameter to 300mm	300mm Diameter to 375mm	Other
k) 51 Island Road					Increase Island Road PS flow to 10 l/s. Current flow is 4.9 l/s, and the rising main is 90mm diameter
l) Upstream of Grant Road					Increase pump rate to 10 l/s. Current flow is 7.7 l/s, and the shared rising main is 150mm diameter (looking at constructing separate rising main for Russell St PS)
m) Upstream of Graham Road PS					Increase pump rate to 10 l/s. Current flow is 6.5 l/s, and the rising main is 100mm diameter
n) Cambridge Terrace/Oxford Street	235m (OXFDSM025 to RANFSM055)		354m (RANFSM055 to WNUISM105)		Abandon connection from RANFSO02 to RANFSM055
o) Belfast Crescent/Cambridge Terrace	1.026km (BELFSM010 to RAKASM005)				
p) Rutene Road					Construct two 150mm diameter cross-connections (RUTESM095 to RUTE-ISMH14, and RUTESM060 to RUTE-ISMH09) 47m length total
q) Harris Street	365m (HARISM085 to HARISM050)		178m (HARISM050to RUTE-ISMH06B)		Abandon connection from HETASM010 to HETASM005
r) Fox/Waiteata Streets	238m (FOX_SM030 to RUSSSM030)		156m (RUSSSM030 to RUSSSM015)		
s) Riverside Road					Increase Russell St PS flow to 30 l/s. Construct a separate rising/falling main (513m rising, 282m falling main both 150mm diameter)* Current flow is 17l/s.
t) Balance/Russell Streets	343m (RUSSSM085 to CLIFSM080)				
u) Clifford/Hall Streets	212m (CLIFSM080 to BLCESM008 and HALLSM010 to ORMDSM075)		665m (ORMDSM095 to ORMDSM070)		
v) Richardson Avenue/Russell Street			483m (RUSSSM060 to ORMDSM070)		Abandon connection from RUSSSM085 to RUSSSM080
w) Stafford Street	444m (WHITSM005 to CLIFSM035, and STAFSM010 to STOUSM025)				
x) Downstream of i, j, k, l, and m convergence point.			5m (ORMDSM025 to ORMDSM020)	813m (ORMDSM070 to ORMDSM015)	

*Russell Street PS currently shares a rising main with Grant Road PS, with the flow from Grant Road taking precedence. Russell Street PS would need to achieve a flow of 30 l/s when it runs without Grant Road PS. However, during a two year ARI storm Russell Street would need to run for approximately 8 hours, while Grant Road was not operational. A separate rising/falling main would therefore be required to allow both pump stations to continue pumping at the same time at the required rates during such periods.

5.4 Costs

Cost estimates for the upgrades required to eliminate remaining overflows if 85% of the gross inflows are removed are shown in table 5.4. Cost estimates for the 75% gross inflow removal are shown in table 5.5. Cost estimates for the 65% gross inflow removal are shown in table 5.6.

Table 5-4 - Cost of Upgrades if 85% Gross Inflow Removed

Location	Cost of Upgrade
Harris Street	$54,900
Rutene Road	$34,900
Russell St PS and rising main	$429,400
TOTAL	**$519,200**

Table 5-5 - Cost of Upgrades if 75% Gross Inflow Removed

Location	Cost of Upgrade
a) Grant Road PS	$13,000
b) Cambridge Terrace	$121,800
c) Rutene Road	$55,500
d) Harris Street	$191,900
e) Fox Street	$202,000
f) Russell Street PS and rising main	$429,400
g) Adair/Russell Streets	$317,400
h) Clifford/Hall Streets	$543,100
i) Russell Street	$256,900
j) Stafford Street	$228,300
TOTAL	**$2,358,900**

Table 5.6 - Cost of Upgrades if 65% Gross Inflow Removed

Location	Cost of Upgrade
k) Island Road PS	$23,600
l) Grant Road PS	$13,000
m) Graham Road PS	$18,500
n) Cambridge Terrace	$539,800
o) Belfast Crescent	$756,700
p) Rutene Road	$55,500
q) Harris Street	$555,800
r) Fox Street	$275,000
s) Russell Street PS & rising/falling main	$429,400
t) Balance Street	$317,400
u) Clifford Street	$906,600
v) Richardson Avenue	$482,000
w) Stafford Street	$408,800
x) Convergence Point	$967,300

Location	Cost of Upgrade
TOTAL	**$5,749,400**

The basis of the gravity reticulation cost estimates is:

- GDC 2016 replacement valuation rates for manholes, gravity mains and service connections increased by 30% to allow for inflation and the current market.
- 1050mm manholes are costed for pipes up to 225mm, and 1500mm manholes for pipes 300mm and larger.
- Service lines from the main to property connections are based on 100mm pipes with an average length of 12m. The GIS property boundary layer has been used to determine the number of replacement connections off each upgraded pipe.
- The replacement valuation rates include 15% for design and construction monitoring.
- A further amount of $7,000 has been added per cross connection, to the Rutene Road estimate to allow for this all being a road crossing.
- No further contingency has been added to the above costs basis. GDC may wish to add a contingency for budgeting purposes. It is understood the replacement valuation rates are currently being reviewed.

An assessment of the existing pump stations has not yet taken place. Therefore for the pumped flow upgrades the basis of the cost estimates is:

- Pump supply prices based on the upgraded pumped flows from the model have been provided by Xylem.
- A factor of 100% has been applied to pump supply prices to allow for installation and contingency.
- It is assumed that the new pumps can be installed in the existing wet wells without significant modifications and that no power supply or electrical upgrades are required.
- The Russell St rising main rate used of $540/m (inclusive of design and construction monitoring) is based on recent construction contracts around the north island.
- For budgeting purposes GDC may wish to add further contingencies where appropriate due to the above unknowns.

6 Conclusion and Recommendations

The Gisborne wastewater network has been designed adequately to convey six times the average dry weather flow. However, stormwater inflow and groundwater infiltration during wet weather results in wastewater flows well in excess of 6 x ADWF. Modelling results show that gross stormwater inflow, both fast response inflow and delayed inflow from private property stormwater ponding, contribute to the majority of overflows from the wastewater network.

Therefore GDC's Wastewater Discharge Reduction Plan to reduce the gross inflow of stormwater into the wastewater network by 85% is a prudent approach. Based on a two year ARI rainfall event (the adopted containment standard) with 85% of the gross inflow removed from the model, only three overflows remain. The estimated cost of three upgrades (Rutene Road cross connection, Harris Street upgrade and Russell Street pump station rising/falling main) to remove these overflows is $519,200. Design of these items could proceed now.

Model sensitivity analysis predicts that if only 75% of the gross inflows are removed then 20 overflows totalling 1,530m^3 remain for the two year ARI event. 10 separate network upgrades have been added to the model to remove all of these flooding nodes. Cost estimates have been made for these options, totalling $2.36M.

Similarly if only 65% of the gross inflows are removed then 38 overflows totalling 5,660m^3 remain for the two year ARI event. 14 separate network upgrades have been added to the model to remove all of these flooding nodes. Cost estimates have been made for these options, totalling $5.75M.

It would not be prudent for GDC to commit all of the 65% or 75% gross inflow removal only expenditure to wastewater network upgrades, when the majority of overflow reductions should be achieved by preventing stormwater entering the sewerage system (the Drainwise Strategy).

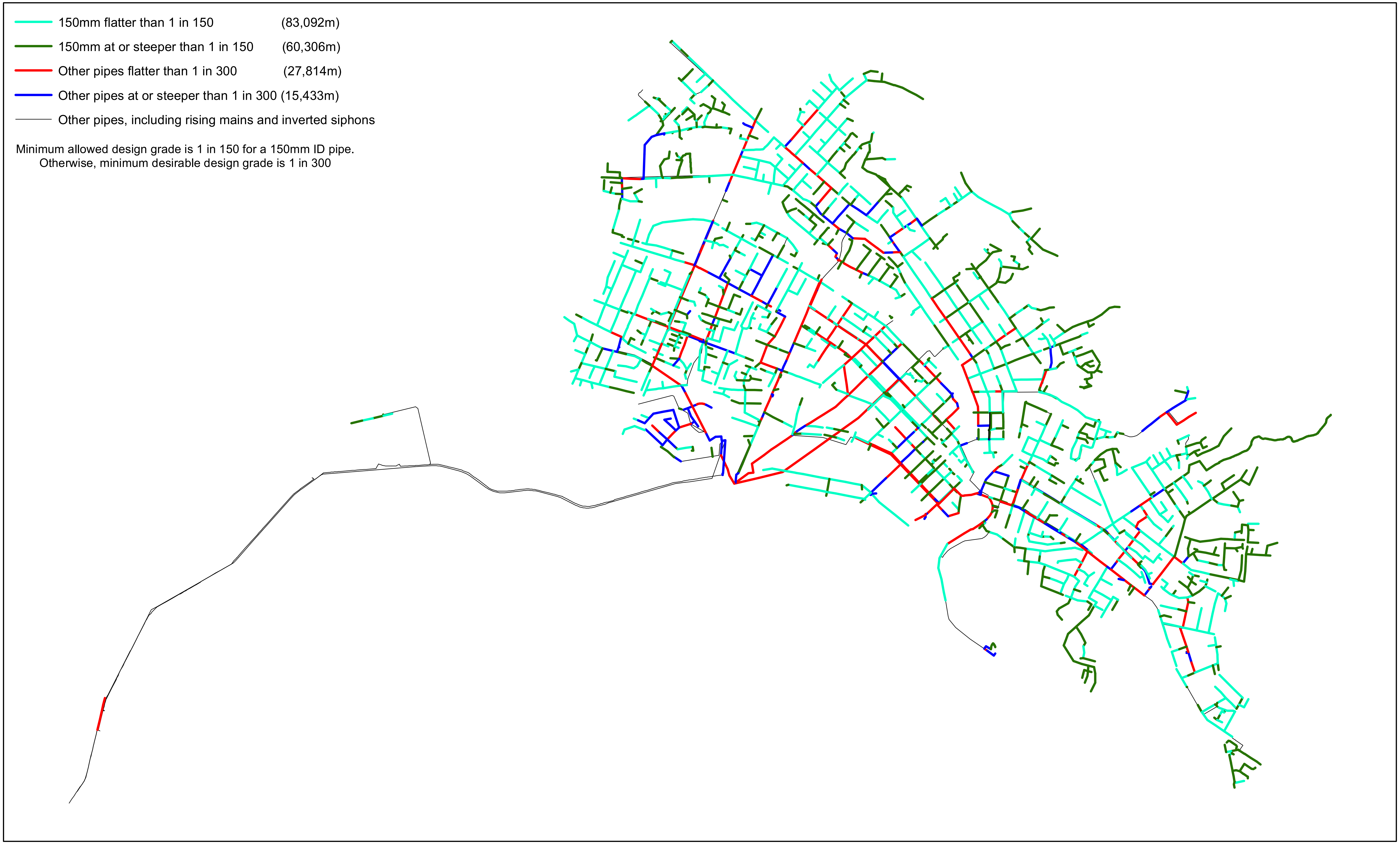
It is therefore recommended that GDC implement the network upgrades based on achieving 85% gross inflow removal. Success of the Drainwise Strategy can then be monitored and additional network upgrades presented in Tables 5.2, 5.3 and Appendix C implemented in the future if necessary.

Should the 75% or 65% gross inflow removal scenario upgrades be required, these should be considered in conjunction with the existing renewals programme of replacing old asbestos cement (AC) and earthenware pipes throughout the network. Many of the proposed upgrades involve replacing AC or earthenware pipes, with larger pipes. The cost of replacing these pipes with a pipe of the same size should be funded from the renewals budget, with only the additional cost attributable to larger pipe being capital funded.

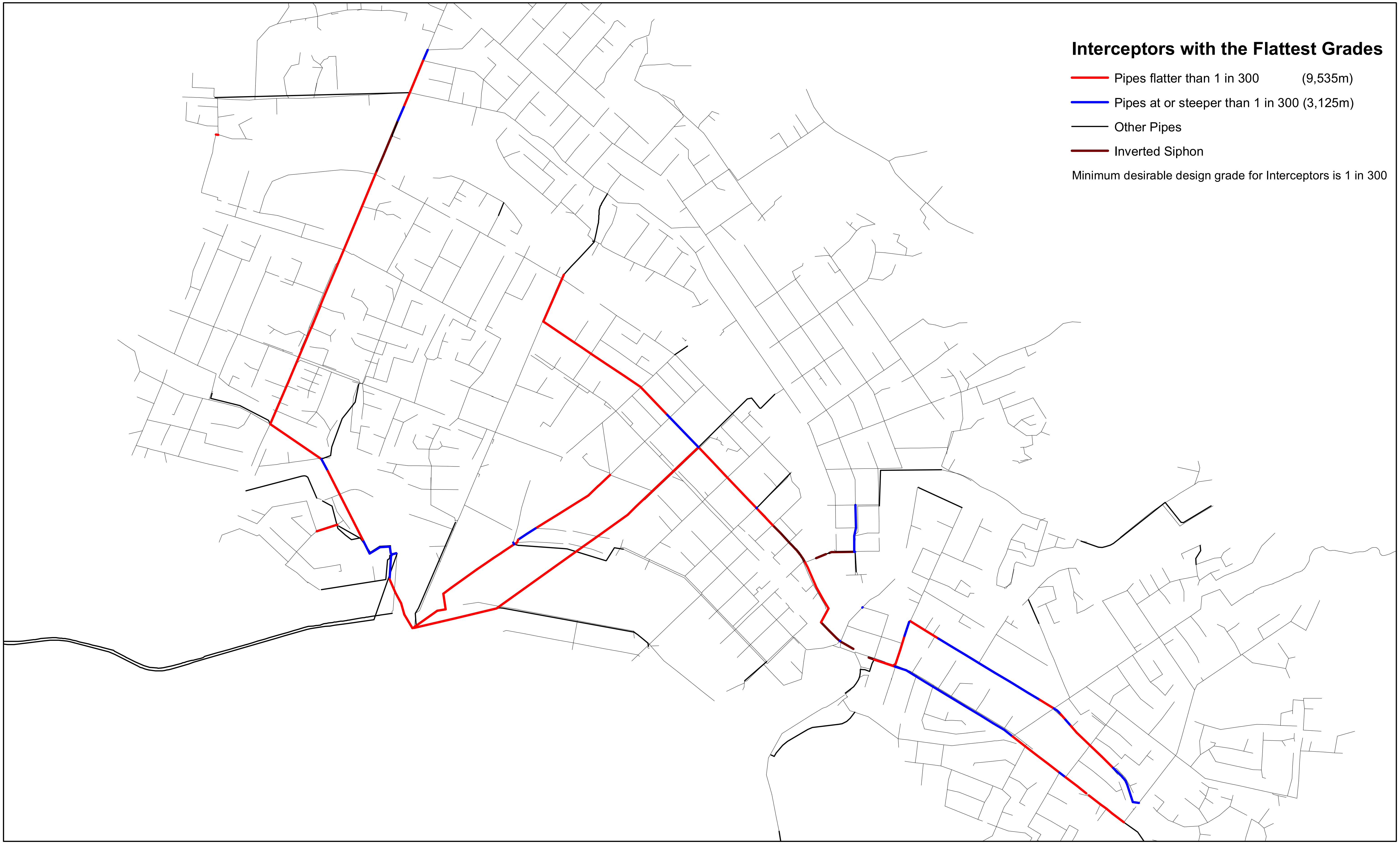
At a network level, flat pipe grades resulting in low wastewater velocities and solids deposition also need to be considered to optimise GDC's pipe jetting programme.

Appendix A

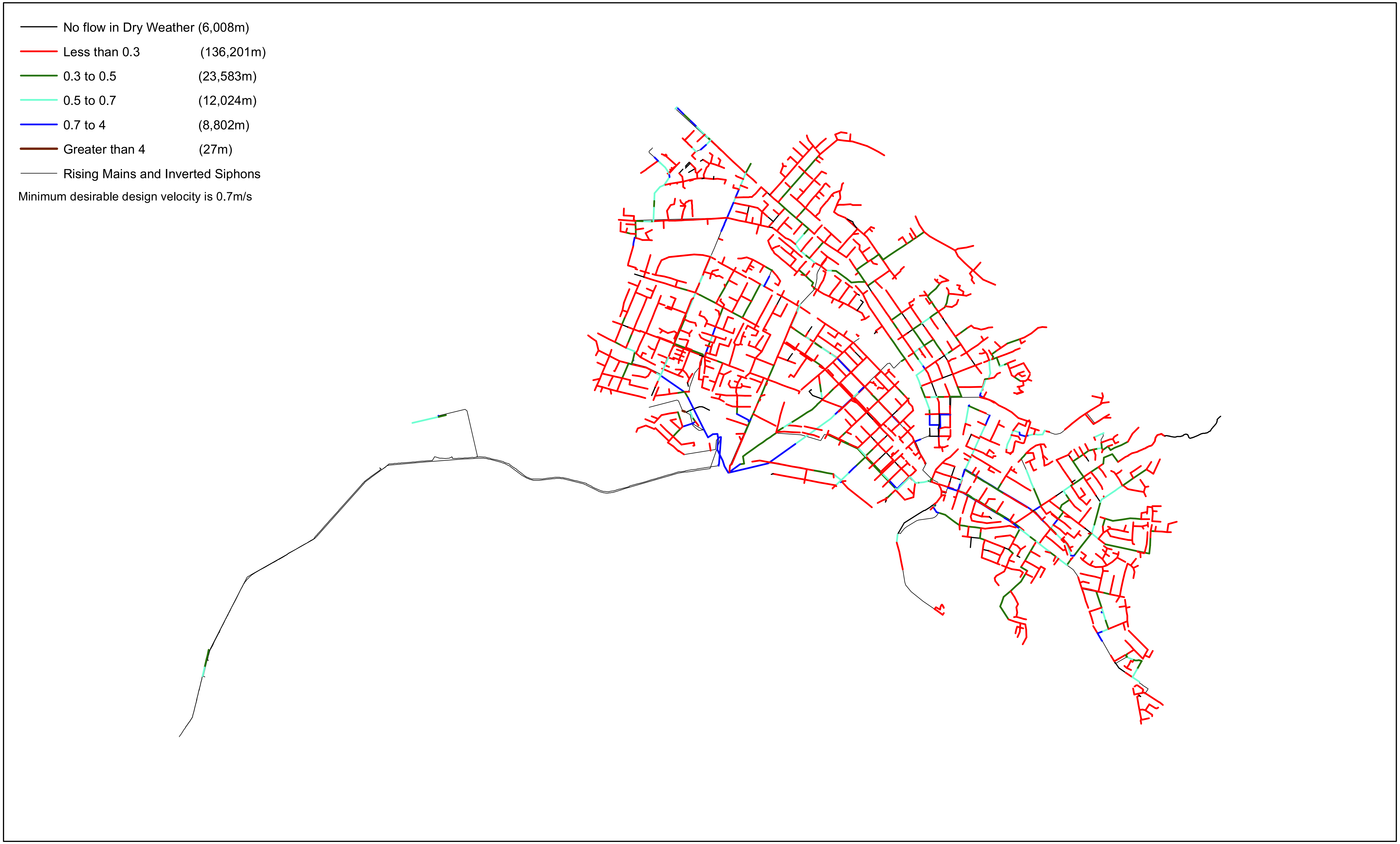
Pipe Grade & Velocity Plans



Gisborne Wastewater Network
Figure A1: Pipes with the Flattest Grades

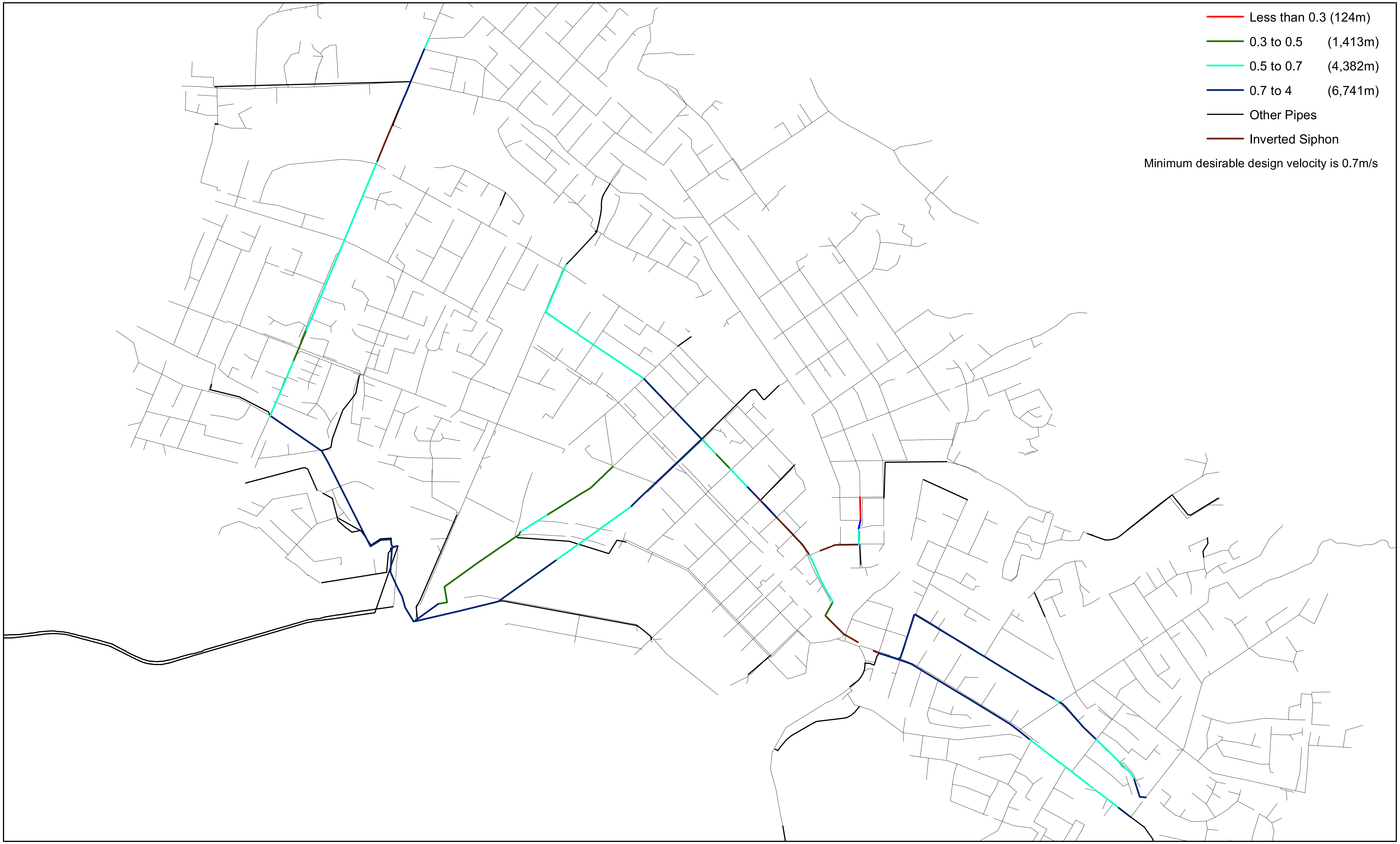


Gisborne Wastewater Network
Figure A2: Gradient of Interceptors



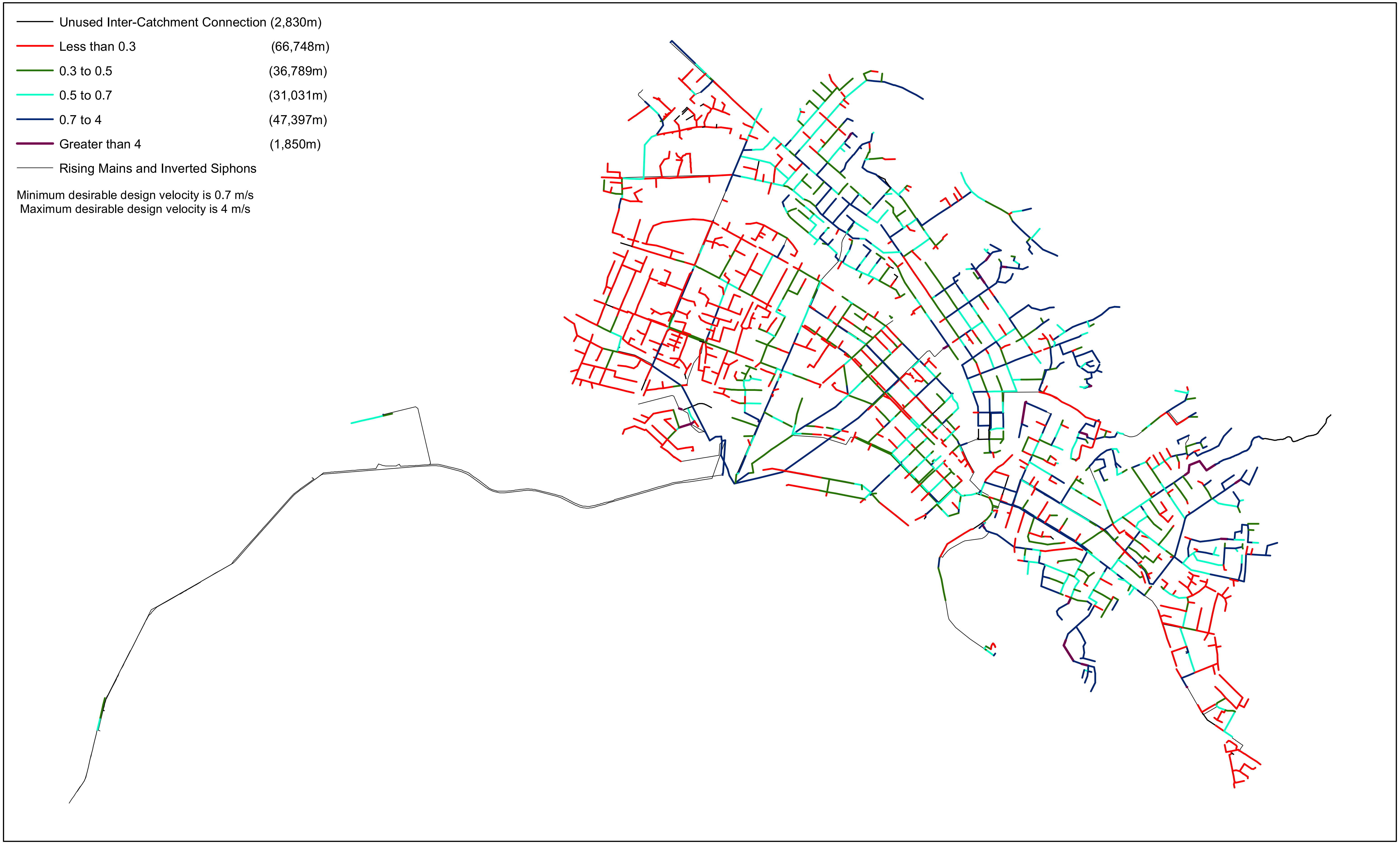
Gisborne Wastewater Network

Figure A3: Pipe Velocity in Dry Weather Flow



Gisborne Wastewater Network

Figure A4: Velocity in Interceptors in Dry Weather Flow



Gisborne Wastewater Network
Figure A5: Pipe Velocity During a 2-Yr ARI Event

Gisborne Wastewater Network
Figure A6: Interceptor Velocity During a 2-Yr ARI Event

Appendix B

Overflow Plans

Gisborne Wastewater Network
Figure B1: Flooding Manholes with 6*ADWF

Gisborne Wastewater Network
Figure B2: Flooding Manholes During 2-Yr ARI Event

Gisborne Wastewater Network

Figure B3: Flooding Manholes During 2-Yr ARI Event with 85% Fast Response Removed

Gisborne Wastewater Network

Figure B4: Flooding Manholes During 2-Yr ARI Event with 85% of Fast Response and Property Ponding Removed

Gisborne Wastewater Network

Figure B5: Flooding Manholes During 2-Yr ARI Event with 75% of Fast Response and Property Ponding Removed

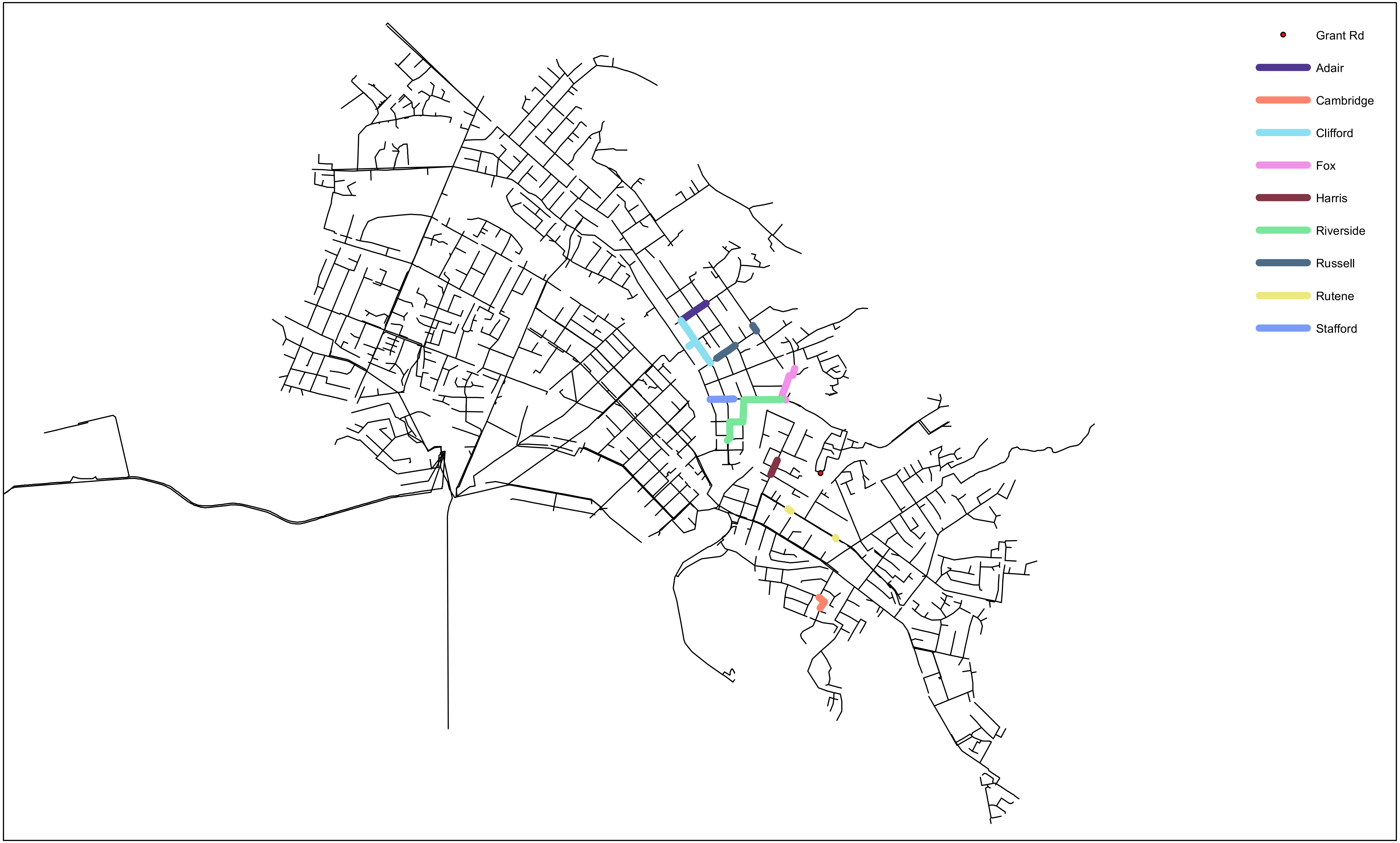
Gisborne Wastewater Network

Figure B6: Flooding Manholes During 2-Yr ARI Event with 65% of Fast Response and Property Ponding Removed

Appendix C

Upgrade Plans

Gisborne Wastewater Network
Figure C1: Upgrades Required to Remove Flooding During 2-Yr ARI Event with 85% of Fast Response and Property Ponding Removed



Gisborne Wastewater Network

Figure C2: Upgrades Required to Remove Flooding During 2-Yr ARI Event with 75% of Fast Response and Property Ponding Removed

Gisborne Wastewater Network
Figure C3: Upgrades Required to Remove Flooding During 2-Yr ARI Event with 65% of Fast Response and Property Ponding Removed