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Te Puke Stormwater Model Build

Te Puke MIKE URBAN Stormwater Modelling
Stage 2: Modelling Report

November 2012

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Stage 2: Modelling Report

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Executive Summary

Opus has been commissioned by Western Bay of Plenty District Council (WBoP DC) to develop a comprehensive stormwater network model for the Te Puke catchment.

This report describes the process of building the stormwater model for Te Puke. It summarises the model results for the 5, 10, 50 and 100-year Average Recurrence Interval (ARI) rainfall events in Te Puke, allowing the performance of the stormwater network to be predicted and significant bottlenecks in the network to be identified.

A model was built in MIKE URBAN v2011 using data from various sources, following the recommendations of the May 2011 Opus report "Stage 1 – Gaps Analysis Report" which summarised available data. The model network can be summarised as follows:

- It contains all pipes, apart from rodding eye, catchpit and house connections.
- No rodding eyes or soakholes have been included.
- Catchpits have not been included in the model and the catchments have been directly connected to the model nodes. This assumes all surface water drains to the pipe network.
- The EastPack pond is the only storage basin shown in the model and attenuates the stormwater runoff from the site.
- All open drains in, and downstream of, Te Puke are included. They were modelled using aerial photos, contour lines and LiDAR data.
- The model includes six overland flow paths that were needed to ensure model connectivity.

Boundary conditions in the model include temporally-distributed rainfall, inflows from upstream catchments, and downstream water levels in the Kaituna River.

There are a total of 393 subcatchments in the model, ranging in size from 0.13 to 262 hectares. The Time-Area method (or Model A) was chosen to represent the hydrology of the catchment. In accordance with the Stage 1 report a sensitivity analysis was carried out on the following catchment parameters:

- Catchment imperviousness
- Surface flow velocity
- Hydrological reduction factor

The results show that within the range of reasonable values the model is not very sensitive to these catchment parameters. Surface flow velocity has the greatest effect but is still not very significant.

The model results show that the 30-minute storm duration is the most critical for each of the four return periods simulated. The same general pattern of network capacity exceedance was apparent for all storm durations, with bottlenecks being more pronounced for the higher return periods.

Portions of the network in these bottlenecks even have insufficient capacity to cope with the flows from a 5-year ARI event.

While the model is not calibrated it is nevertheless a useful tool in identifying potential bottlenecks, identifying upgrades as well as prioritising the upgrades.

Contents

1 Introduction

1.1 Background

The stormwater system of Te Puke consists of pipes and open drains that ultimately discharge into the Kaituna River. The piped network consists of 1366 pipes and 540 manholes. Only small portions of this stormwater system have been modelled on an ad-hoc basis in the past. This makes it difficult to get a good overview of the level of service that the stormwater system in Te Puke provides and to develop coherent long-term asset management strategies. Western Bay of Plenty District Council (WBoP DC) wishes to address this and use the modelling of this catchment as a pilot project for any modelling of other catchments. Furthermore, the results from this modelling can be used to produce flood hazard and extent maps as well as providing emergency management planning information.

In May 2011 Opus produced the Stage 1 – Gaps Analysis Report (Maas and Apirumanekul, 2011) that summarised the data available for a model to be built. It also identified the gaps in the data and the means for obtaining any missing data.

Following this report WBoP DC gathered the required asset data based on the May 2011 report and further missing data was obtained as the model build progressed.

1.2 Purpose

The purpose of this report is to describe the process of building the stormwater model for Te Puke using Mike Urban v2011 including all assumptions made during the build process. The report also summarises the model results for the 5, 10, 50 and 100-year Average Recurrence Interval (ARI) rainfall events in Te Puke as well as highlighting the significant bottlenecks in the network.

2 Catchment and Drainage Network Description

2.1 Catchment

Te Puke is located in the Bay of Plenty to the south east of Tauranga (Figure 2.1) and forms part of the area administered by Western Bay of Plenty District Council (WBoP DC).

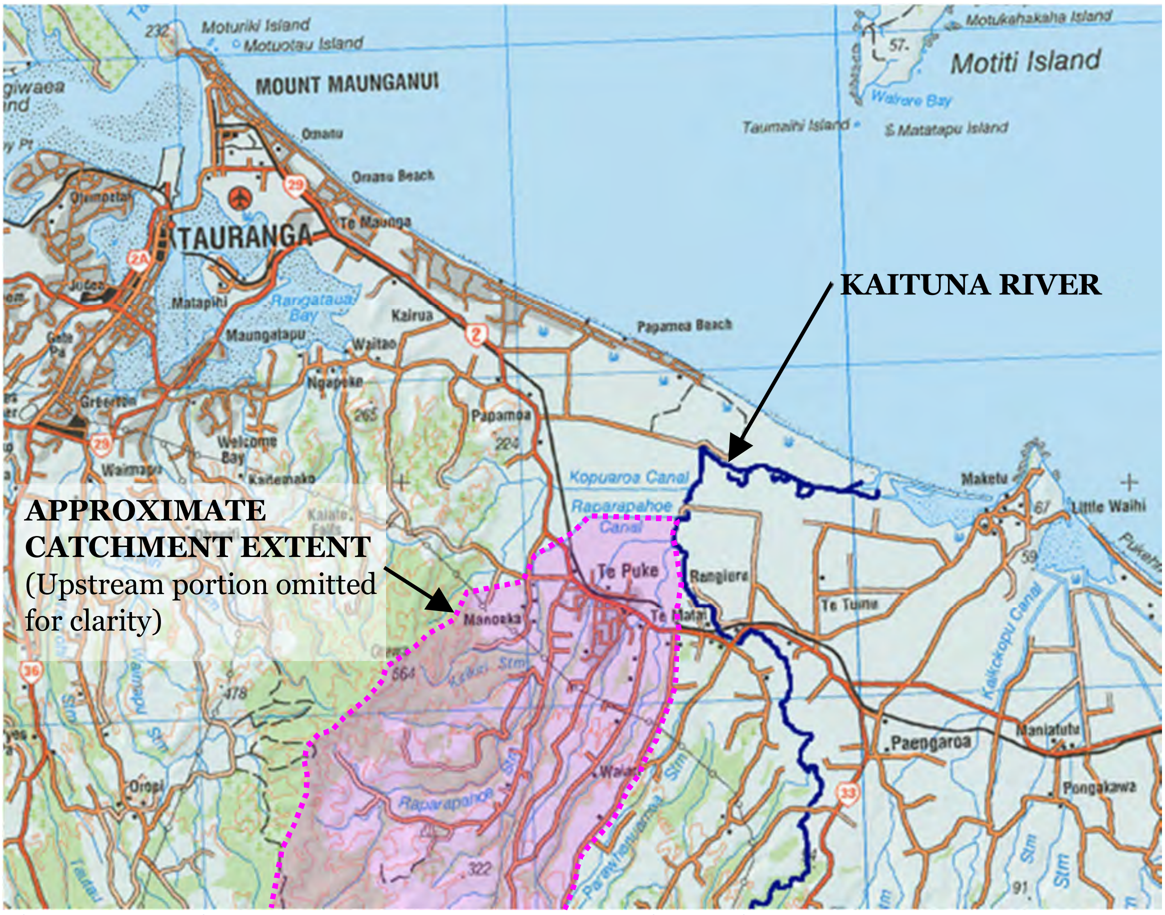


Figure 2.1 Location of Te Puke catchment and the Kaituna River

The urban area of Te Puke has been built just to the south of a swampy coastal flat on top of terraces between a number of watercourses. These watercourses and their tributaries originate from large catchments that are predominately rural with high producing grassland and orchards (refer Figure A.1). The watercourses themselves are deeply incised into the landscape with a mixture of indigenous forests and pine plantations.

The district Plan zones of Te Puke (refer Figure A.2) show that the urban portion of the catchment is predominantly zoned residential with a significant industrial zoned area.

2.2 Drainage Network

The stormwater system of Te Puke consists of pipes and open drains that ultimately discharge into the Kaituna River (refer Figure A.3). The piped network consists of 1366 pipes and 540 manholes.

3 Hydrometric Data Sources

3.1 Introduction

Hydrometric data consists of rainfall data, river water level and flow data. In the vicinity of Te Puke there are four hydrometric sites with suitable records (refer Figure A.4).

Te Puke Electronic Weather Station (EWS) is a rainfall site with a relatively short record and has only been operating since 2002.

There are two long term water level and flow records for the Kaituna River at Te Matai and Clarke's. There is also a long term record available at Raparapahoe River Above Drop Structure. The data for the latter would be suitable for deriving synthetic flows of smaller catchments that discharge into the stormwater system of Te Puke.

3.2 Rainfall Data

WBoP DC's development code (WBoP, 2009) specifies that the design rainfall tables in it should be used for the design of stormwater systems. These tables are based on information using HIRDS v1.5b.

Data from HIRDS

HIRDS is an acronym for High Intensity Rainfall Design System. It is a generalised procedure to obtain spatially and temporally consistent depth-duration-frequency design rainfalls for any location throughout New Zealand. HIRDS v3 is a more up-to date version than that used by WBoP DC and was deemed more suitable for this work. Table 3.1 gives values from HIRDS v3 for a point in the centre of the urban area of Te Puke.

Table 3.1 HIRDS v3 design rainfall table for the Te Puke area (rainfall depths in mm)

ARI	Duration				
	10-min	20-min	30-min	1-hr	2-hrs
2	10.1	15.6	20.0	30.6	41.5
5	13.3	20.4	26.2	40.2	54.3
10	16.0	24.5	31.4	48.2	64.9
20	19.0	29.1	37.4	57.3	77.0
50	23.7	36.4	46.7	71.7	96.0
100	28.1	43.0	55.3	84.8	113.3

Comparison with Recorded Rainfall Data

As identified in Section 3.1, a rainfall gauge site exists near Te Puke that has only been operating since 2002. Comparing the results from a statistical analysis on the data from this site (Table 3.2) with those from HIRDS v3 shows that the values are very similar even though the record from this site is relatively short. Hence it would appear that the values from HIRDS v3 are likely to be applicable to describe the rainfall in Te Puke.

Table 3.2 Te Puke EWS design rainfall table (2002-2011) (rainfall depths in mm)

ARI	Duration				
	10-min	20-min	30-min	1-hr	2-hrs
2.33	9.3	16.7	19.6	28.1	44.4
5	12.5	22.5	26.9	37.1	59.7
10	15.1	27.2	33.2	44.8	72.8
20	17.5	31.5	39.5	52.3	85.4
50	20.6	37.1	47.7	62.0	101.9
100	22.9	41.1	53.9	69.2	114.2

Storm Duration

In general the smaller the catchment to be analysed the smaller the time of concentration for that catchment. As the model consists of a large number of small subcatchments discharging into the stormwater system of Te Puke, the time of concentration for these catchments is likely to be small. As a result the critical storm duration is likely to be at most 1 hour and hence storm durations of 10 minutes, 30minutes and 1 hour were selected.

Temporal Distribution

HIRDS only provides the total depth for the rainfall event modelled. To be able to model the event itself these depths then have to be distributed temporally. In 2006 Opus produced a report for Environment Bay of Plenty (Welch, 2006) that examined the temporal distribution of rainfall around Tauranga. The report contained a number of different temporal patterns which were dependent on the duration of the storm. Table 3.3 is the temporal pattern of the 1-hour duration storm which is the shortest duration storm in that report. This temporal pattern was chosen for temporally distributing the rainfall in the MIKE URBAN model for Te Puke.

Table 3.3 Tauranga temporal rainfall distribution for 1-hour duration storms

% Total Storm Duration	% Total Storm Rainfall	% Incremental Rainfall
0	0.0	0.0
10	1.8	1.8
20	4.0	2.2
30	10.8	6.8
40	26.2	15.4
50	49.7	23.5
60	73.8	24.1
70	91.0	17.2
80	96.0	5.0
90	99.0	3.0
100	100.0	1.0

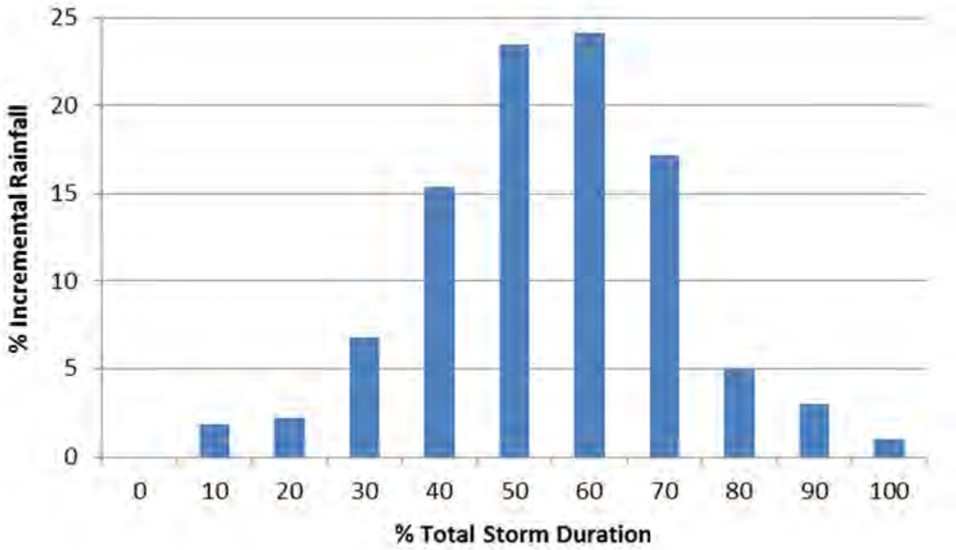


Figure 3.1 Temporal rainfall distribution for a 1-hour duration storm in Tauranga

3.3 Flow Gauging Data

There are ten rivers and streams that discharge into the area covered by the MIKE URBAN model of Te Puke. However, there are no flow records available for them. Therefore, the flow regimes of similar adjacent catchments need to be characterised, and their properties scaled and translated to those streams within Te Puke. Given the geographic proximity of the Raparapahoe River Above Drop Structure site this was selected to be used to determine the estimated peak discharges for the other locations by scaling them up based on the catchment area (Table 3.4)

Table 3.4 Estimated peak discharges for the eight catchments within Te Puke (m^3/s)

Name	Area (km^2)	Estimated Peak Discharges (m^3/s)					
		ARI 2.33	ARI 5	ARI 10	ARI 20	ARI 50	ARI 100
Raparapahoe Above Drop	52.00	49.00	62.20	70.60	77.40	84.90	89.70
Ohineangaanga Stream	16.10	15.20	19.29	21.89	24.00	26.33	27.82
Ohineangaanga Stm trib	1.00	0.94	1.19	1.35	1.48	1.63	1.72
Waiari Stream u/s SH2	72.22	67.89	86.66	98.22	107.61	117.72	124.94
Waiari Stream trib 1	4.58	4.32	5.48	6.22	6.82	7.48	7.90
Waiari Stream trib 2	2.06	1.94	2.47	2.80	3.07	3.37	3.56
Waiari Stream trib 3	0.74	0.70	0.89	1.00	1.10	1.21	1.28
Raparapahoe Canal trib 1	0.22	0.20	0.26	0.29	0.32	0.35	0.37
Raparapahoe Canal trib 2	0.16	0.15	0.19	0.22	0.24	0.26	0.28
Raparapahoe Canal trib 3	0.41	0.38	0.49	0.55	0.61	0.67	0.70

3.4 Kaituna River Levels

The water level in the Kaituna controls the downstream boundary of the MIKE URBAN model. While there are no water level or flow records in the immediate vicinity of Te Puke, there are long term records from both Te Matai and Clarke's. By interpolating between the two sites a synthetic water level was derived near the discharge points of the stormwater system of Te Puke (Table 3.5).

Table 3.5 Synthetic Kaituna River levels near Te Puke for various return periods

ARI (years)	Synthetic Te Puke Water Level (m)
2.33	2.607
5	3.044
10	3.360
20	3.636
50	3.966
100	4.195

3.5 Combining Probabilities

Rainfall in the catchments upstream of Te Puke arrives as flows in the watercourses and their tributaries that flow through Te Puke. Given that in most cases their catchments extend a considerable distance upstream and to much higher elevations, it is unlikely that a major inflow from upstream will coincide with an equivalent rainfall event in Te Puke itself.

Similarly the water levels in the Kaituna near Te Puke are as a result of rainfall in a large catchment not in the immediate vicinity of Te Puke with different rainfall-runoff characteristics. Hence it is also unlikely that a rainfall event will coincide with a water level in the Kaituna River that is as a result of a rainfall event of a similar magnitude.

Environment Bay of Plenty has produced guidelines (Everett, 2001) that deal with combining return periods of events in different catchments. These guidelines have been used to determine what flows from the upstream catchments and levels in the Kaituna River to use for the modelled events (Table 3.6).

Table 3.6 Combination of events used in the MIKE URBAN model

Model Overall ARI (years)	ARI of Event (years)	
	Te Puke Rainfall	Water Level/Flow in Watercourses
100	100	20
50	50	20
10	10	2.33
5	5	2.33

Given the uncertainty in the timing of the peaks of the downstream levels and upstream inflows with respect to the rainfall on the catchment itself the peak values of these levels and inflows have been applied as constant values during the simulations.

4 Model Build

4.1 Subcatchment Delineation

The overall catchment extent was defined as part of Stage 1. Using the contour lines, aerial photos, LiDAR data and the stormwater network itself this was then broken up into suitable subcatchments. These subcatchments were then further refined and attached to the model network as the model build progressed.

There are a total of 393 subcatchments in the model (refer Figure A.5) with catchment sizes ranging between 0.13 and 262 hectares. They tend to be smaller in the urban areas due to the density of the drainage network there.

The subcatchments were linked to the most conservative location in the network (i.e. as close as possible to the upstream end of the network within, or near, the subcatchment) where possible.

4.2 Subcatchment Hydrologic Parameters

There is insufficient data available to calibrate and/or verify the model. Hence there is no need to choose a complex hydrologic rainfall-runoff model and hence the Time-Area (or Model A) was chosen to represent the hydrology of the catchment. The input required for this model consists of the following parameters:

- Catchment Area
- Time of concentration
- Imperviousness
- Initial loss
- Hydrological Reduction
- Time-Area curve

In this rainfall-runoff model the volume of the runoff is controlled by the initial loss, size of the contributing area, catchment imperviousness and the hydrological reduction factor (DHI, 2010a). The shape of the runoff hydrograph is controlled by the concentration time and by the time-area (T-A) curve. These two parameters represent a conceptual description of the catchment reaction speed and the catchment shape.

The catchment area is calculated automatically each time it is created or modified. The remaining parameters can be rapidly assigned to the subcatchments using the catchment processing tool in MIKE URBAN.

Time of Concentration

The time of concentration for each catchment was calculated using the MIKE URBAN catchment processing tool with a mean surface flow velocity of 0.3 m/s. For flow in gutters along the road this value is appropriate, though for flow across land this may be on the high side. As a result, the modelled time of concentration may be shorter than is actually the case, leading to quicker catchment response with a higher peak flow. No calibration data exists so this value is used to model rainfall events since it produces conservative results. As part of the sensitivity analysis of the model, a much lower mean surface flow velocity value of 0.1 m/s will be used. The result of this is discussed in Section 5.4.

The resultant time of concentration of each of the catchment ranges from 1 to 147 minutes with 95% of the subcatchments having a time of concentration of 25 minutes or less.

Imperviousness

The Stage 1 report identified that no building footprint and/or impervious area outlines are available for the model build. In accordance with the recommendations in that report the catchment was split into imperviousness classes and representative samples were taken for each as required.

The imperviousness of the catchment is related to the land use. The District Plan zones as shown in Figure A.2 were used as a starting point for splitting the catchment into imperviousness classes. To limit the complexity of the model the catchment was split into only 5 imperviousness classes.

By examining the aerial photos the parks and rural areas within the residential, commercial and industrial zones were able to be identified and removed from those zones. Furthermore, the land covered by SH2 was identified and split off in a similar fashion. Any land not already covered by an imperviousness class was then classed as Parks and Rural. Figure A.6 and Table 4.1 show the final breakdown of the catchment covered by the MIKE URBAN model into imperviousness classes.

Table 4.1 Breakdown of catchment included in MIKE URBAN model

Imperviousness Class	Area (ha)	Proportion of Catchment
Residential	286.7	19.3%
Commercial	14.9	1.0%
Industrial	82.6	5.5%
Parks and Rural	1084.0	72.8%
Highway	20.6	1.4%
Total	**1488.8**	**100%**

The catchment is predominantly made up of the Parks and Rural imperviousness class. This is not surprising given that a large rural area downstream (or to the north) of Te Puke is included in the model. The urban part of the catchment is predominantly Residential with a significant Industrial component. Commercial land use represents the smallest area.

To determine the imperviousness of the Residential, Commercial and Industrial imperviousness classes, eight, two and three samples were taken, respectively (refer Figure A.6 for the locations of the samples). The samples were taken by randomly placing a 100 m by 100 m square (with a total surface area of 10,000 m^2 or one hectare) on the aerial photos and measuring the impervious area in each using CAD tools (refer to Figure B.1 to Figure B.11 for the samples). The results of the sampling is shown in Figure 4.1 and shows the minimum, maximum, average and mean impervious area for each class.

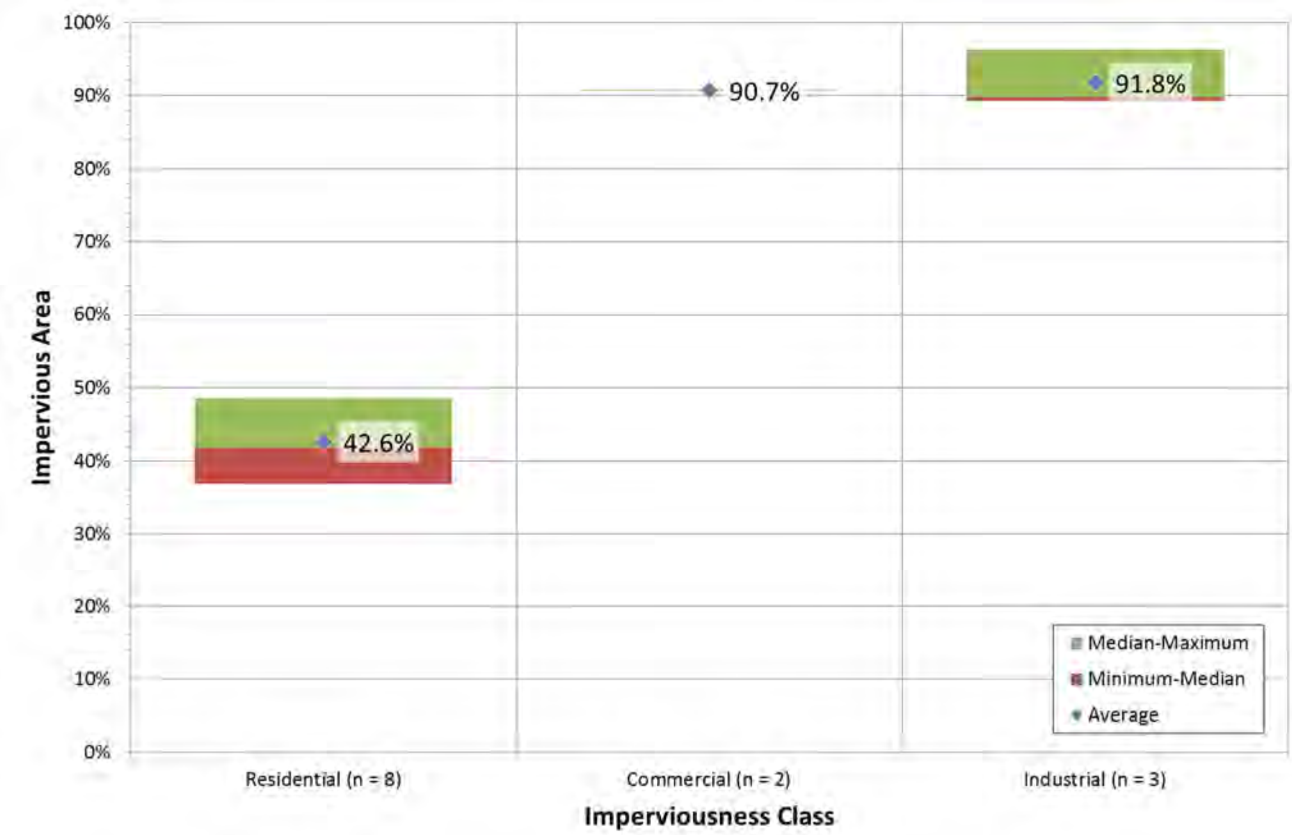


Figure 4.1 Range of imperviousness values for each imperviousness classes

The imperviousness of the remaining two imperviousness classes (Rural and Parks and Highway) is based on experience-based assumptions.

Table 4.2 shows the values that have been used in the MIKE URBAN modelling with the average used to generate the model results for the various design storms as well as the range values for the samples which were used in the sensitivity testing of the model. These imperviousness values were then assigned to the catchments using the MIKE URBAN catchment processing tool. Where a subcatchment contains more than one imperviousness class this processing tool assigns a weighted-average imperviousness value based on the area each represents.

Table 4.2 Imperviousness values used

Imperviousness Class	Impervious Area	
	Average	Range
Residential	42.6%	36.9 - 48.4%
Commercial	90.7%	90.7 - 90.8%
Industrial	91.8%	89.3 - 96.3%
Parks and Rural	5%	
Highway	90%	

Initial Loss

The rainfall initial loss value for the catchments was left at the default value of 0.0006 m when assigning catchment parameters using the MIKE URBAN catchment processing tool. For large events in an urban catchment such an initial loss is negligible and hence it is reasonable to set this initial loss at such a low value.

Hydrological Reduction

This factor represents a continuous hydrological loss that accounts for water losses caused by processes such as evapo-transpiration and imperfect imperviousness categorisation. The default value of 0.90 was used for all catchments when assigning catchment parameters using the MIKE URBAN catchment processing tool.

Time-Area Curve

The time-area curve accounts for the shape of the catchment and hence how the catchment reacts to the rainfall. Time Area curve 1 has been used for all catchments given that the majority of catchments are rectangular shaped. This curve has been assigned to each of the subcatchments using the MIKE URBAN catchment processing tool.

4.3 Piped Network Definition

The piped model network was imported from the GIS data supplied by WBoP DC using the import routines within MIKE URBAN. Most of the inconsistencies and gaps in the data were resolved in accordance with the Opus Stage 1 report (Maas and Apirumanekul, 2011). The remainder of the gaps and inconsistencies were brought to WBoP DC's attention and were corrected once they were inspected and/or surveyed. The status of the data in the model was recorded in MIKE URBAN (as detailed in Appendix C – Stormwater Modelling Data Status Flagging) to allow the origin of the data to be tracked.

The pipe roughness values used for both free flowing and pressurised pipes used for the various pipe materials in the model are given in Table 4.3.

Table 4.3 Values used to define pipe roughness

Material	Manning's M (=1/n)	Colebrook – White Equivalent Roughness (m)
Concrete (Normal)	75	0.0015
Plastic	80	0.001
Corrugated Steel	40	0.025

4.4 Open Channel Network Definition

The Stage 1 gap analysis report identified that the GIS layer supplied by WBoP DC contained a number of deficiencies including missing open drains, ambiguous lengths and badly defined upstream and downstream node names. To remedy this in an effective way the alignments of the open channel networks were digitised from scratch using the aerial photos, contour lines and LiDAR data provided. Nodes were added where necessary.

For open channels MIKE URBAN requires cross-sections to be defined at least at the start and the end of each open channel. Cross-sections were extracted from the LiDAR data using WaterRIDE Flood Manager, simplified where necessary, and entered into MIKE URBAN.

The photos (including aerial photos) of the open drains within the model do not show a large variation in roughness and from the photos examined it is possible to estimate the roughness of the channels. Table 4.4 details the roughness values used and Figure A.7 shows where they were applied to the open drains network.

Table 4.4 Open channel roughness values

Channel Type	Manning's Roughness	
	M (=1/n)	n
Channel - maintained grass	40	0.025
Channel - slight growth	30	0.033
Channel - overgrown	20	0.050
Rivers	30	0.033

4.5 Overland Flow Paths

During the course of the model build, and subsequently during the model runs, it became apparent that some overland flow paths were needed to allow the model to run and simulate the natural flow of water. The details of the added flow paths are given in Table 4.5.

Table 4.5 List of overland flow paths in the model

Model Link(s)	Description
FLOD0057, FLOD0094	Takes the flow from a surcharging sump (SWCP1540) through Fairhaven Park to SWCI0840
FLOD0128	Takes the overland flow from SWCO1022 through properties in Seddon Street, Magnolia Place and Ben Keys Street to the open drain to the north of Ben Keys Street
FLOD0160	Takes the flow from bubble-up manhole SWMH1436 along a driveway in a south-easterly direction to an open drain
FLOD0173	Takes the flow from SWMH1768 (which has been assumed to be a bubble-up manhole) through properties in Tynan Avenue and Boucher Street to SWCI0846
FLOD0181	Takes the flow from SWCO1013 through properties to sumps along the edge of Stock Road (SWCP1825 and SWCP1826)
FLOD0182, FLOD0183	Takes the flow from Stewart Street that is unable to enter the stormwater system between buildings to Queen Street

The alignment of the overland flow paths was determined by examining the aerial photos, LiDAR and contour lines. The cross-sections were either extracted from the LiDAR or by deriving dummy cross-sections that best approximated the likely shape of the overland flow path.

The channel roughness (Table 4.6) of the overland flow path was inferred from the aerial photos. These roughness values are included in Figure A.7.

Table 4.6 Channel roughness values used for the overland flow paths

Material	Manning's M (=1/n)
Channel - maintained grass	40
Channel - overgrown	20
Concrete (Smooth)	85

4.6 Hydraulic Structures

Catchpits

In accordance with the Stage 1 report (Maas and Apirumanekul, 2011) catchpits have not been included in the model and the catchments have been directly connected to the model nodes. Where the effect of a catchpit was necessary to return overland flow into the piped network, or vice-versa, then this was represented by a weir.

Weirs

Nine fictional weirs have been added to the model to facilitate the flow between the stormwater network and the overland flow paths. Brief descriptions of these weirs are given in Table 4.7.

Table 4.7 Fictional weirs included in the model

Weir ID	Description
Weir@SWCP1540	Overflow from SWCP1540 to overland flow path
Weir@SWCP1492	Overflow from SWCP1492 to overland flow path
Weir@SWMH1040	Bubble-up manhole outflow from SWMH1040 to overland flow path
Weir@SWMH1768	Schematic overflow from SWMH1768 to overland flow path as outlet is not known
Weir@SWCP1825	Flow from overland flow path into SWCP1825
Weir@SWCP1826	Flow from overland flow path into SWCP1826
Weir@SWJN0774	Overflow from SWJN0774 to overland flow path
Weir@FNMH0134	Allows water to flow from and to overland flow at FNMH0134
Weir@SWMH1427	Allows overland flow to return back to the piped network at SWMH1427

Storage Basins

The EastPack pond is the only storage basin to be included in the model. It located in the north of Te Puke off Quarry Road and detains the stormwater runoff from the site. It has been schematised in the model (refer Figure 4.2) as a storage basin (FNRE0145) that is connected to the remainder of the model network by three fictitious links (FLRE0174, FLRE0175 and FLRE0176). These links have been made hydraulically smooth (by setting Manning's M=1/n=1000) and short to ensure that they do not significantly add to the storage volume and the travel time of water through the pond.

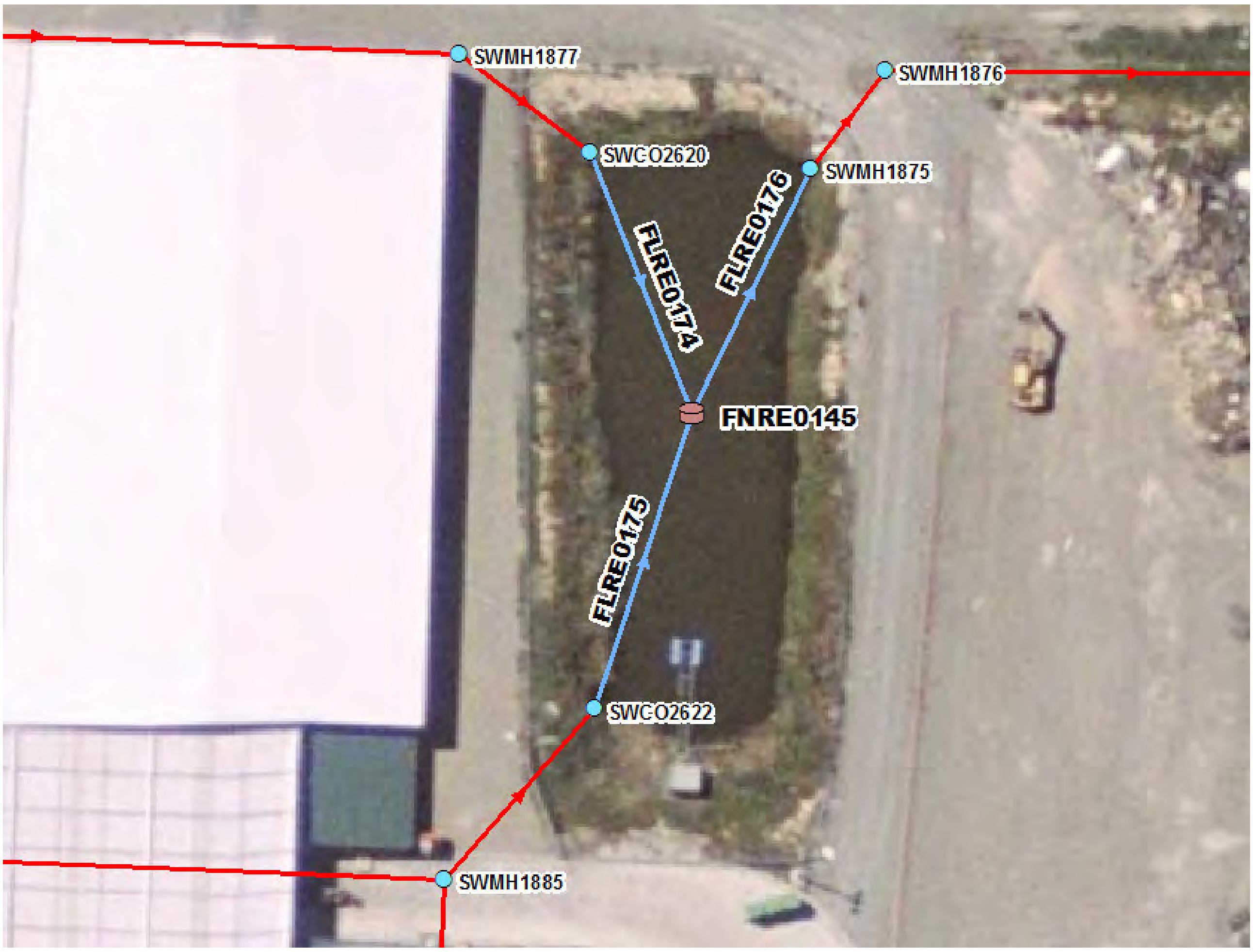


Figure 4.2 Schematisation of EastPack pond in MIKE URBAN model

The depth-storage relationship of the EastPack stormwater pond was modelled in MIKE URBAN on the basis of geometric data taken from the EastPack Ltd New Packhouse & Coolstore Facility Stormwater Layout Plan (appended as Figure A.9). Pond dimensions were used to calculate variations in cross-sectional area and water surface area in depth increments of 0.2m as detailed in Table 4.8.

MIKE URBAN uses these parameters to calculate depth-varying water volume and velocity in the pond according to the method outlined in the MIKE URBAN user manual (DHI, 2010b). Figure 4.3 shows the resultant depth-storage relationship in MIKE URBAN for the EASTPACK pond.

Table 4.8 Details of EastPack pond as entered in the model

RL (m)	H (m)	Cross-section area (m²)	Surface area (m²)
47.0	0.0	0.0	600.0
47.2	0.2	3.1	633.4
47.4	0.4	6.2	667.4
47.6	0.6	9.5	702.2
47.8	0.8	13.0	737.8
48.0	1.0	16.5	774.0
48.2	1.2	20.2	811.0
48.4	1.4	23.9	848.6
48.6	1.6	27.8	887.0
48.8	1.8	31.9	926.2
49.0	2.0	36.0	966.0
49.2	2.2	40.3	1006.6
49.4	2.4	44.6	1047.8
49.6	2.6	49.1	1089.8
49.9	2.9	54.9	1143.4

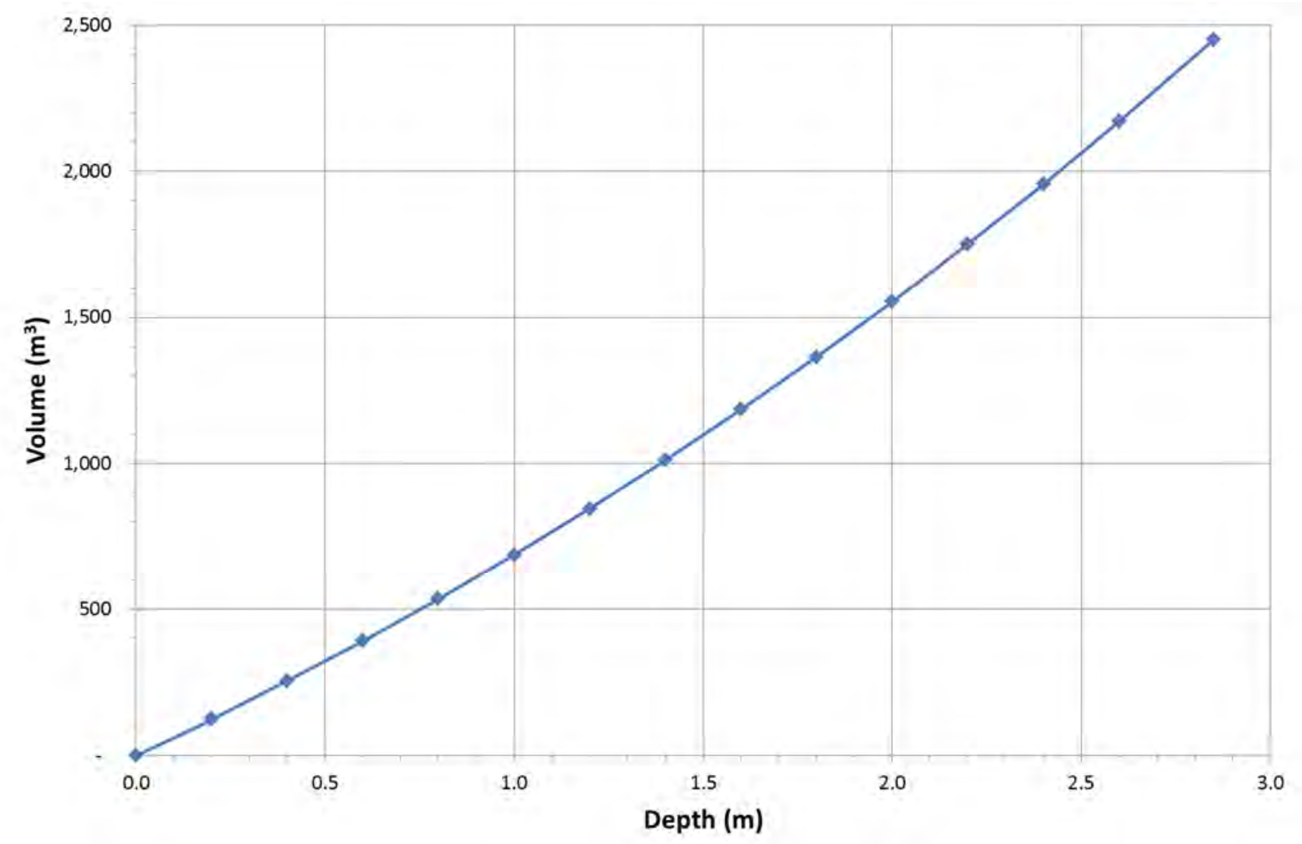


Figure 4.3 Depth-storage relationship of the EastPack Pond in the model

Soakholes

There are 75 Soakholes located in urban Te Puke. In the Stage 1 report (Maas and Apirumanekul, 2011) it was proposed to not include them in the MIKE URBAN model but to include their effect by reducing the runoff from the relevant subcatchments. Following careful consideration it has been decided to not include this effect either for the following reasons:

i. In large events (50 and 100-year ARI events) the reduction effect on the run-off will be minimal as the soakholes will be swamped by the sheer volume of water.
ii. It is not clear as to how well maintained the soakholes are and hence it is possible that they may not be as effective at allowing water to soak away into the ground for smaller events (5 and 10-year ARI events).

By not including the soakholes in the model network or their effect on the catchment runoff the model will produce conservative results.

4.7 Boundary Conditions

The boundary conditions represent the inflows from upstream catchments and the downstream water levels in the Kaituna River. The ten upstream inflows have been added to the model as constant network loads. The three downstream water levels have been added to the model as constant external water levels. The location and magnitude of all the boundary conditions are shown in Figure A.8. Refer to Sections 3.3 to 3.4 for the derivations and selection of the values used.

4.8 Hot Start Runs

The preliminary model runs for each of the events showed the creation of a large hydraulic shock in the model as a result of suddenly introducing the upstream inflows into a dry model. To prevent this from distorting the effect of a rainfall event on the model network the model was first run for 6 hours with all but the inflows from the Waiari Stream and Raparapahoe River as constant inflows with the required magnitude. The magnitude of the Waiari Stream and Raparapahoe River inflows were such that the flows for these had to be gradually brought in over a period of 4 hours. The model results at the end of the 6 hour period were then used as hot start (or starting condition) for the model runs of the events themselves. Hot start runs were run for both the 2.33 and 20 year ARI boundary events.

4.9 Sensitivity Tests

Given that insufficient data exists for calibration or verification it is important that sensitivity tests be carried out to investigate the sensitivity of the model to variations in key model parameters related to the rainfall-runoff parameters. Table 4.9 shows the test values used, where those for the imperviousness were grouped in two scenarios being a high imperviousness scenario and a low imperviousness scenario. These sensitivity tests were run for the 50-year ARI, 30 minute duration storm as this is the critical storm duration for the design return period for the stormwater network. Comparing the results for this event will give a good indication of the sensitivity of the model results to these calibration parameters.

Table 4.9 Details of values used in sensitivity tests

Parameter	Original Value	Sensitivity Test Value(s)
Imperviousness - Residential - Commercial - Industrial	 42.6% 90.7% 91.8%	 36.9%, 48.4% 90.7%, 90.8% 89.3%, 96.3%
Catchment mean surface velocity	0.3m/s	0.1m/s
Hydrological reduction	0.9	1.0

4.10 Scenarios

Table 4.10 details the eighteen scenarios that have been set up in MIKE URBAN's scenario manager. These represent the various events and the associated hot start runs required to run them. Also included are the sensitivity tests performed on the model

Table 4.10 MIKE URBAN scenarios

Scenario Name	Event	
	Rainfall in Te Puke	Inflow from Upstream & Kaituna River Water Levels
Hot Start Runs		
Short Duration ARIs	-	2.33yr ARI (refer Section 4.8)
Hot Start	-	20yr ARI (refer Section 4.8)
Rainfall Event Runs		
5y 10m	5yr ARI, 10min duration	2.33yr ARI, steady
5y 30m	5yr ARI, 30min duration	2.33yr ARI, steady
5y 1h	5yr ARI, 1hr duration	2.33yr ARI, steady
10y 10m	10yr ARI, 10min duration	2.33yr ARI, steady
10y 30m	10yr ARI, 30min duration	2.33yr ARI, steady
10y 1h	10yr ARI, 1hr duration	2.33yr ARI, steady
Base	50yr ARI, 10min duration	20yr ARI, steady
50y 30 m rainfall	50yr ARI, 30min duration	20yr ARI, steady
50y 1 h rainfall	50yr ARI, 1hr duration	20yr ARI, steady
100y 10 m rainfall	100yr ARI, 10min duration	20yr ARI, steady
100y 30 m rainfall	100yr ARI, 30min duration	20yr ARI, steady
100y 1 h rainfall	100yr ARI, 1hr duration	20yr ARI, steady
Sensitivity Tests		
Low imperviousness	50yr ARI, 30min duration	20yr ARI, steady
High imperviousness	50yr ARI, 30min duration	20yr ARI, steady
Reduced SFV	50yr ARI, 30min duration	20yr ARI, steady
Increased HR	50yr ARI, 30min duration	20yr ARI, steady

Note in the above, the "Reduced SFV" represents the sensitivity test with a reduced catchment mean Surface Flow Velocity, and "Increased HR" represents the increased Hydrological Reduction sensitivity test.

5 Model Results

5.1 Introduction

The Mike URBAN hydraulic model of the Te Puke stormwater system has been run for four different return period events, each for three different durations, as well as a number of sensitivity model runs. In order to estimate the performance of the network the following parameters have been determined from each of the simulations for the pipes:

- $\mathbf{Q_{Manning}}$ is the pipe-full capacity and has a constant value based on the pipe diameter, slope and roughness;
- $\mathbf{Q_{max}}$ is the absolute maximum flow in the pipe during a simulation run;
- $\mathbf{H_{max}}$ is the maximum head reached in the pipe during a simulation run;
- **D** is the pipe diameter.

The ratio $\mathbf{Q_{max} / Q_{Manning}}$ indicates how much of the pipe capacity is utilised by the flow in the pipe and hence whether there is any spare capacity in the pipe. A value less than 1 indicates that the pipe is not running full at any point during the simulation. A value greater than 1 indicates that the pipe is surcharged and the flow is pressurised.

The ratio $\mathbf{H_{max} / D}$ represents the filling of the pipe and indicates whether the pipe is surcharged. A value greater than 1 indicates that the pipe is surcharged by depth.

5.2 Results for all ARIs

The 30-minute storm duration was shown to be the most critical for each of the four return periods simulated. The same general pattern of network capacity exceedance was apparent for all storm durations, with the bottlenecks being more pronounced for the longer return periods.

The performance of the stormwater system is shown to be hindered by a series of bottlenecks that generate pressurised flow in the pipes and potentially surcharging from manholes. This effect is described in MIKE URBAN by the $Q_{max} / Q_{Manning}$ ratio and is expanded on below.

5.3 Significant Bottlenecks

Values of $Q_{max} / Q_{Manning}$ of greater than 1 indicate sections in the system where flow in the pipe is pressurised and likely to induce surcharging in the manholes. These constriction effects can occur where a reduction in pipe diameter within the system causes surcharging of manholes upstream of the constriction (e.g. No. 3 Road – Figure E.2), or where a low grade means pipes are unable to convey stormwater fast enough (e.g. Valley Road – Figure E.3). Pressurised flow that is modelled in pipes in the upper-most reaches of the system (e.g. Boucher Avenue – Figure E.4, Commerce Lane – Figure E.5) do not necessarily indicate inadequate pipe sizes but rather reflect the fact that entire catchments are connected to the upstream node of the pipe model.

WBoP have indicated that the desired level of service is such that it can cope with a 50-year ARI event so the identification of significant bottlenecks has been done on the basis of the results from the critical (30-minute) duration storm with this return period.

The results (refer Figure D3.4) show many potential bottlenecks in the system with the following being the most significant:

- No. 3 Road (Figure E.2)
- Atuaroa Avenue (Figure E.8)
- Ben Keys Street (Figure E.7)
- Boucher Avenue (Figure E.4)
- Cameron Road (Figure E.9)
- Commerce Lane & Jocelyn Street (Figure E.5)
- Dunlop Road (Figure E.12)
- Princess St (Figure E.6)
- Queen Street (Figure E.10)
- Slater Place (Figure E.11)
- Valley Road (Figure E.3)

5.4 Sensitivity Tests

Catchment imperviousness

Using the low and high end values of the catchment imperviousness ranges (refer Figure D.5.2 and Figure D.5.4) do not significantly affect the model results. Hence the model results are not very sensitive to errors or changes in catchment imperviousness.

Reduced Surface Flow Velocity

Reducing the catchment surface flow velocity from 0.3 m/s to 0.1 m/s (i.e. by 66%) increases the catchment times of concentration and as a result the peak discharges in the network decrease by approximately 14% on average. Hence errors in this calibration parameter are not likely to lead to significant errors in model results. This can be seen when comparing Figure D.3.4 with Figure D.5.6 where the changes in the Q_{max} / $Q_{Manning}$ ratio are few and relatively small.

Increased Hydrological Reduction

Increasing the Hydrological Reduction factor (Figure D.5.8) by 11% does not have a significant effect on the model results.

6 Conclusions

Following the May 2011 gap analysis report on the data available on the stormwater system of Te Puke (Maas and Apirumanekul, 2011) the missing data has been collected. The stormwater model for Te Puke using Mike Urban v2011 has been built and run for the 5, 10, 50 and 100-year ARI rainfall events in Te Puke each for 10, 30 and 60-minute storm durations.

The results have allowed the performance of the stormwater network to be predicted and show the following deficiencies in the network with respect to the 50-year ARI storm:

- No. 3 Road (Figure E.2)
- Atuaroa Avenue (Figure E.8)
- Ben Keys Street (Figure E.7)
- Boucher Avenue (Figure E.4)
- Cameron Road (Figure E.9)
- Commerce Lane & Jocelyn Street (Figure E.5)
- Dunlop Road (Figure E.12)
- Princess St (Figure E.6)
- Queen Street (Figure E.10)
- Slater Place (Figure E.11)
- Valley Road (Figure E.3)

Furthermore, the results show that portions of the network in the above bottlenecks have insufficient capacity to cope with the flows from a 5-year ARI event.

No calibration data was available and hence in accordance with respect to the Stage 1 report a sensitivity analysis was carried out on the following catchment parameters:

- Catchment imperviousness
- Surface flow velocity
- Hydrological reduction factor

The results show that, within the range of reasonable values, the model is not very sensitive to these catchment parameters. Surface flow velocity has the greatest effect but is still not very significant.

While the model is not fully calibrated it is nevertheless a useful tool in identifying potential bottlenecks, identifying upgrades as well as prioritising the upgrades.

7 References

DHI (2010a), *"MOUSE – Runoff Reference Manual"*, software reference manual produced by DHI, July 2010.

DHI (2010b), "*MOUSE – Pipe Flow Reference Manual*", software reference manual produced by DHI, July 2010.

Everett S C (2001), *"Environment BOP – Bay of Plenty Regional Council - Hydrological and Hydraulic Guidelines"*, a report prepared by The Bright Spark Group for Environment BOP, Guidelines Report 2001/04, 16 July 2001.

Harrison Grierson (2001), "EastPack Ltd New Packhouse & Coolstore Facility Te Puke Quarry Road – Stormwater Layout Plan", a drawing produced by Harrison Grierson for EastPack Ltd, drawing number 10096DR1, March 2001.

Maas F J and Apirumanekul C (2011), *"Te Puke Mike Urban Stormwater Modelling – Stage 1: Gap Analysis"*, a report prepared by Opus International for Western Bay of Plenty District Council, Reference 3-50909.00, May 2011.

WBoP DC (2009), "*Development Code: Design"*, Western Bay of Plenty District Council, September 2009.

Welch C A (2006), *"Tauranga City Temporal Rainfall using the Average Variability Method"*, a report prepared by Opus International Consultants for Environment Bay of Plenty, Reference 3-50449.00, May 2006.

Appendix A - Plans

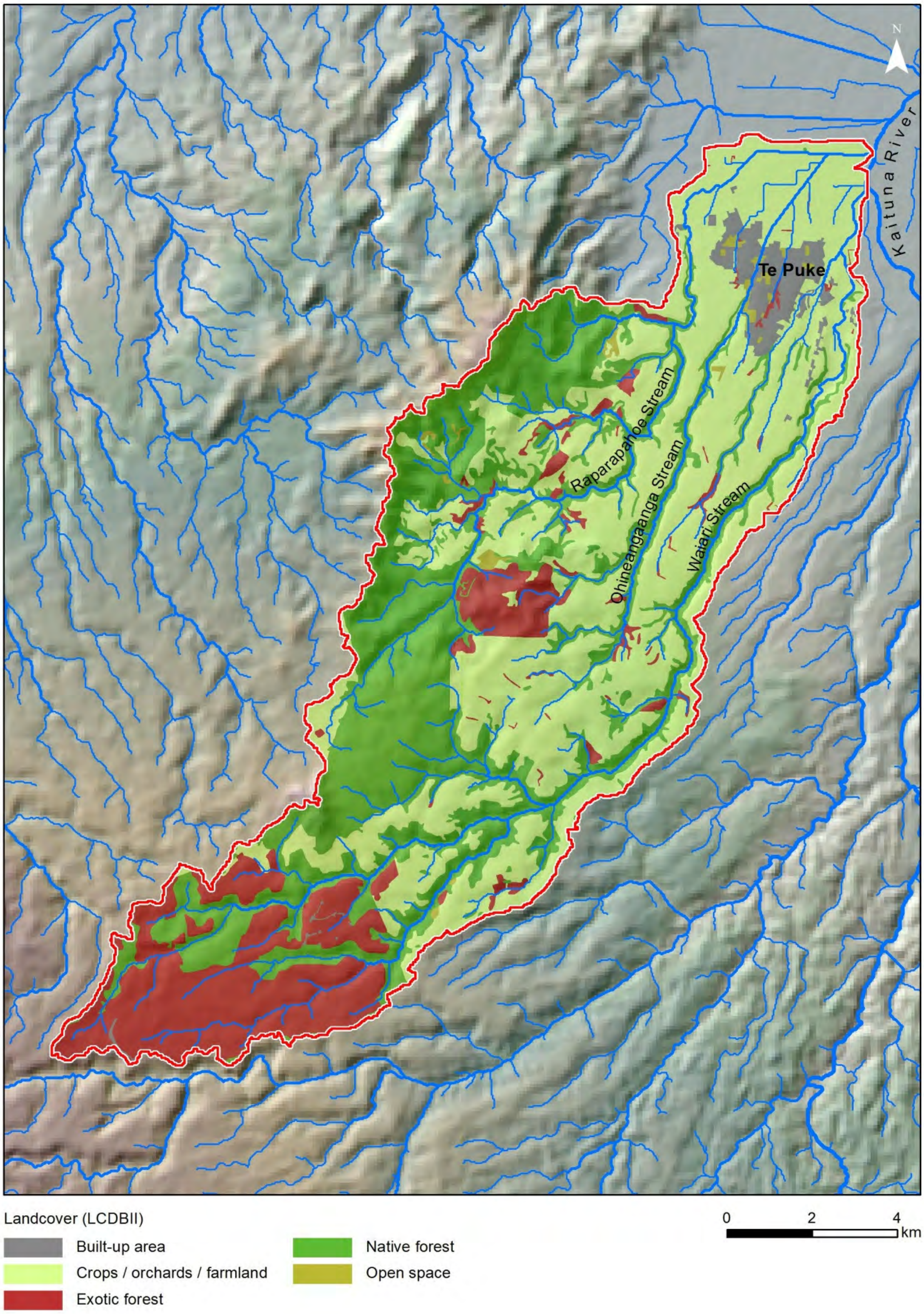


Figure A.1 Land use of the Te Puke catchment

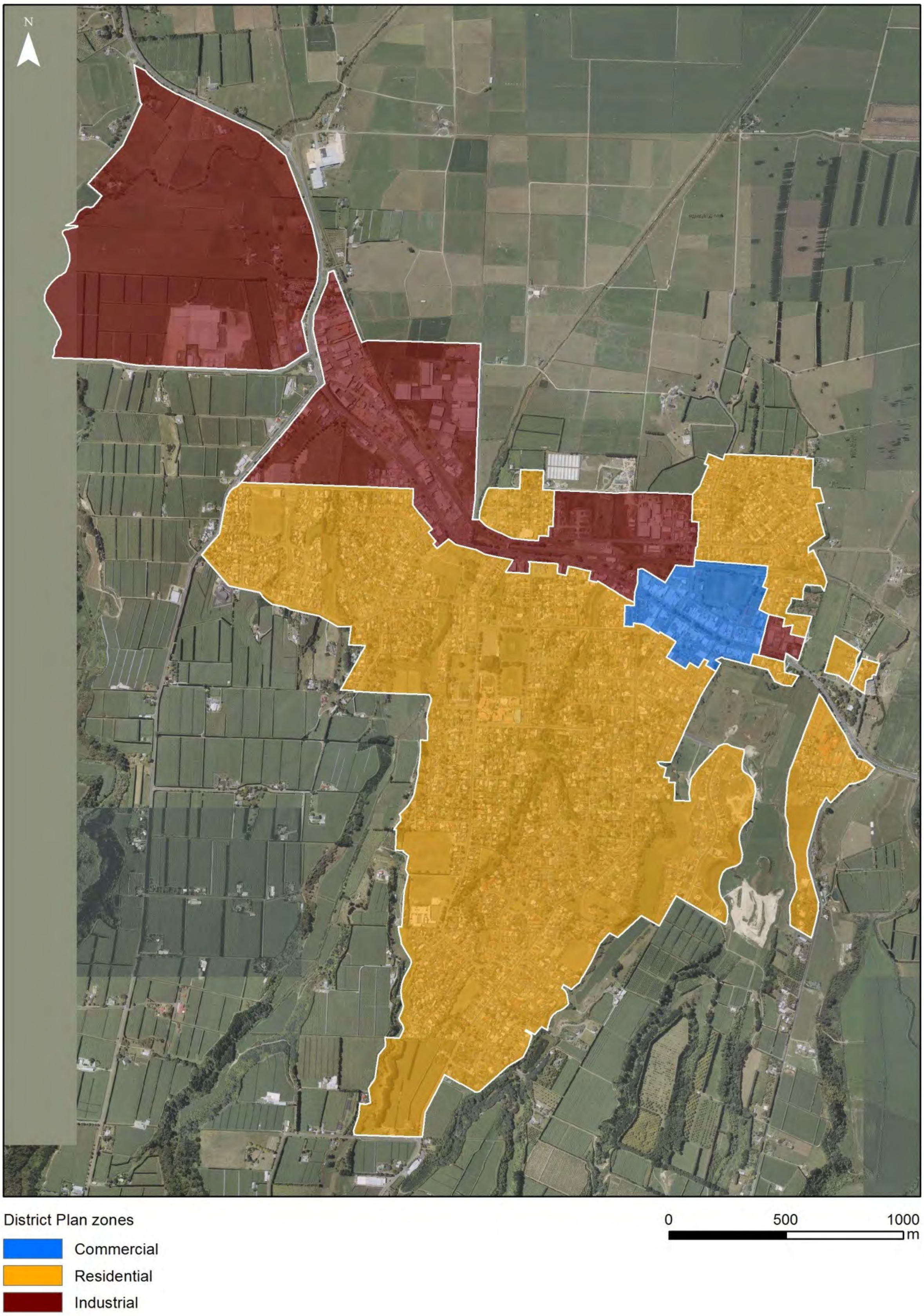


Figure A.2 District Plan zones in Te Puke

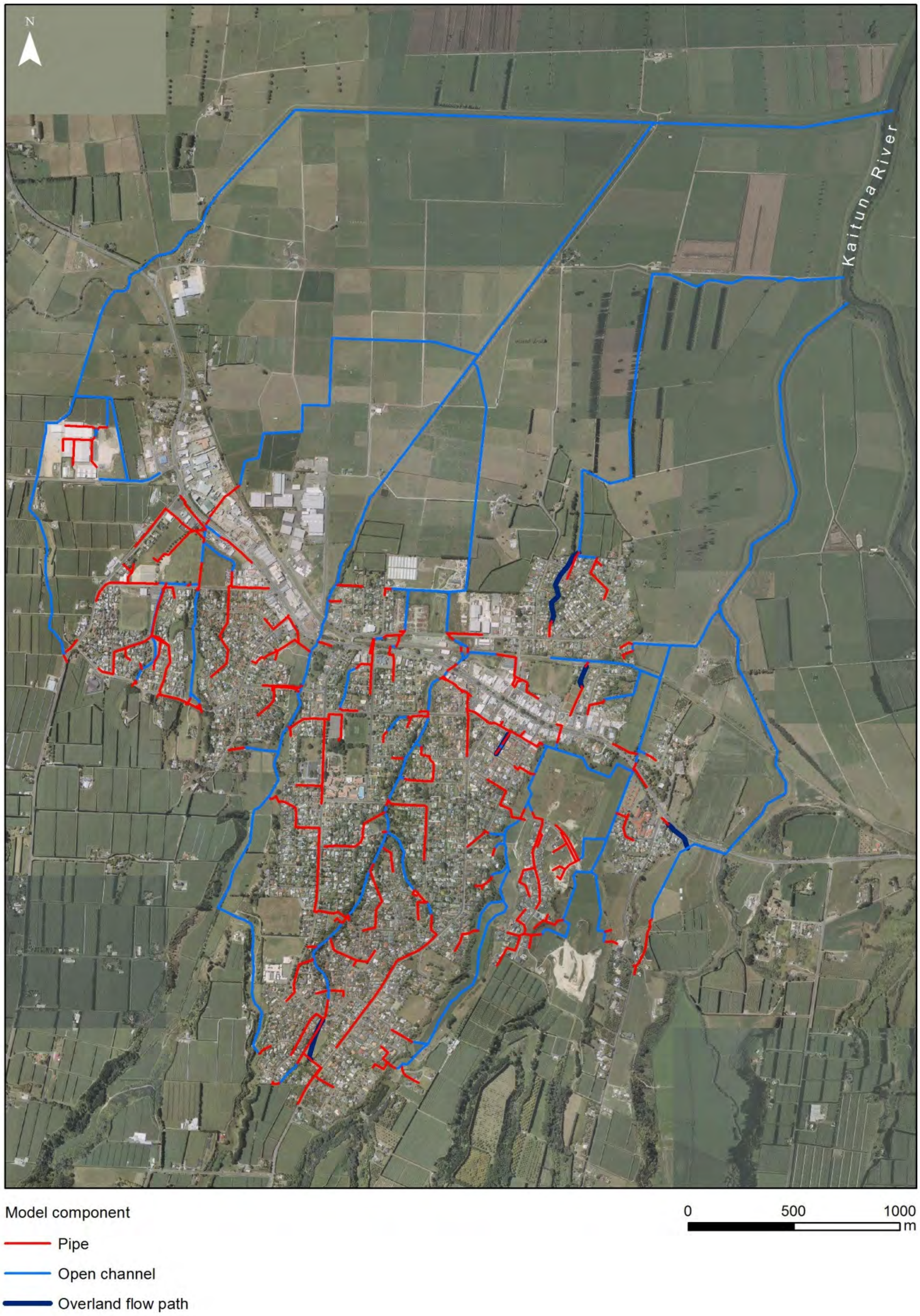


Figure A.3 Stormwater drainage network of Te Puke in the MIKE URBAN model

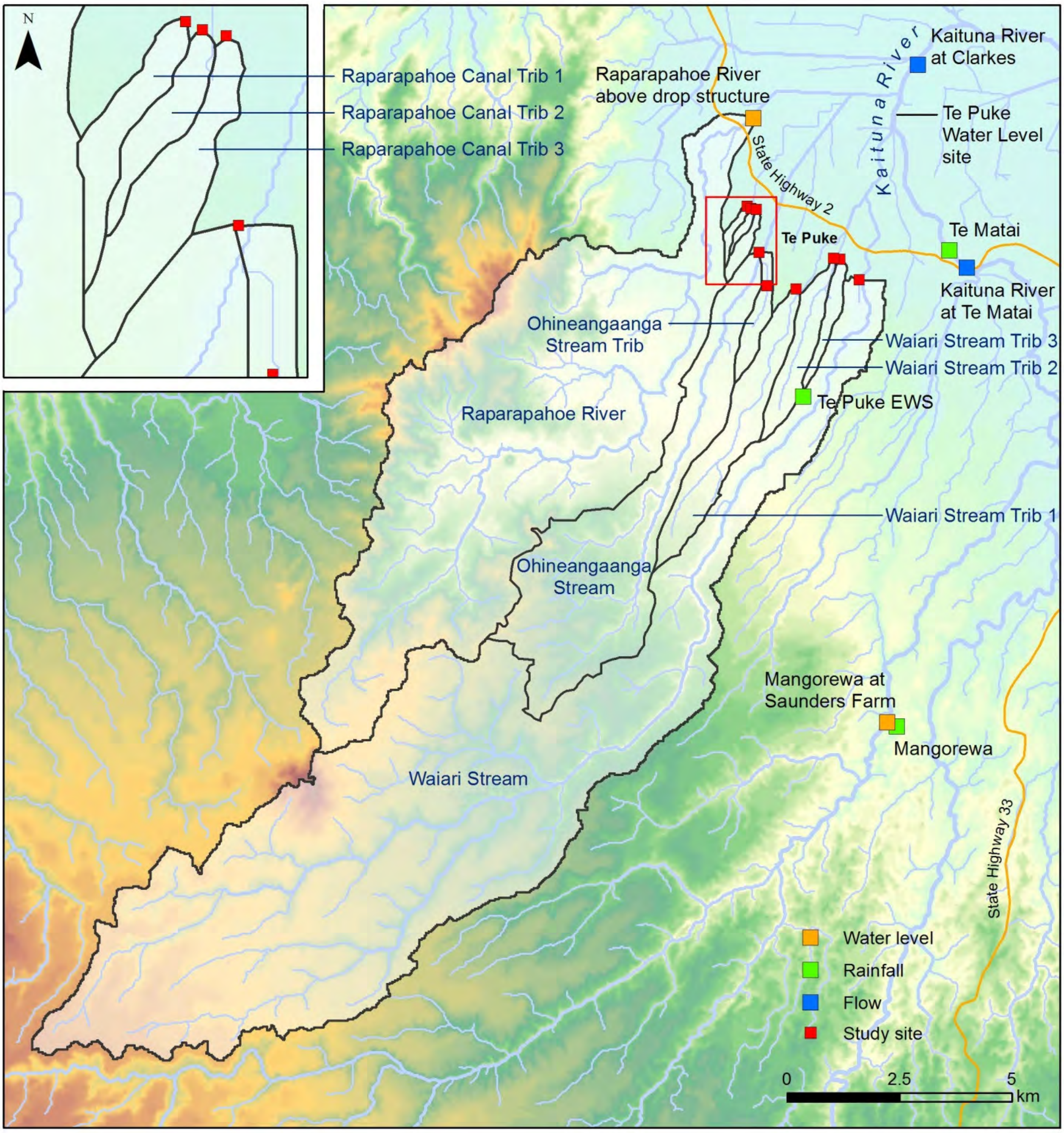


Figure A.4 Location of the flow stations, rainfall sites and study catchments

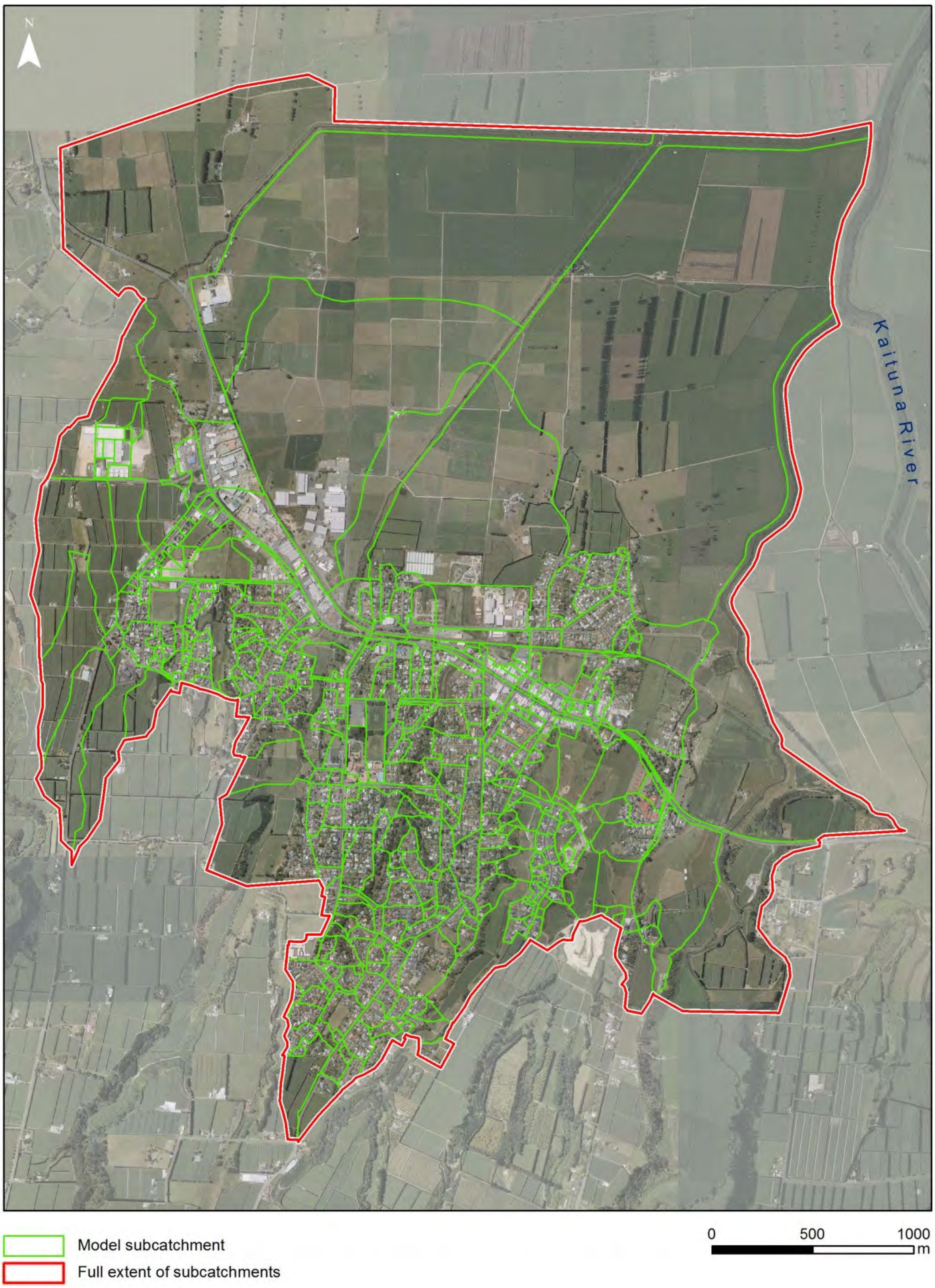


Figure A.5 Plan of subcatchments

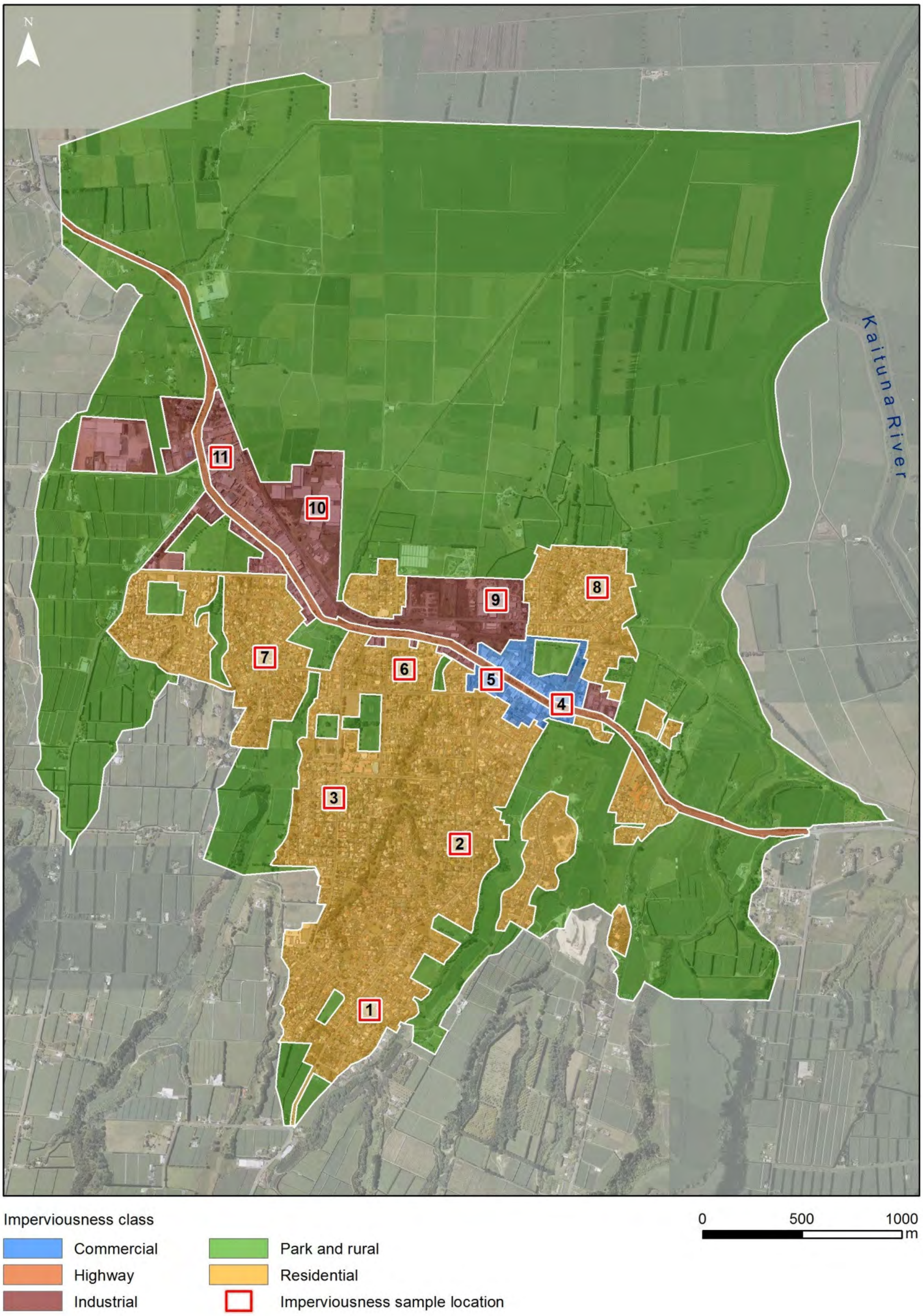


Figure A.6 Imperviousness classes and sample locations

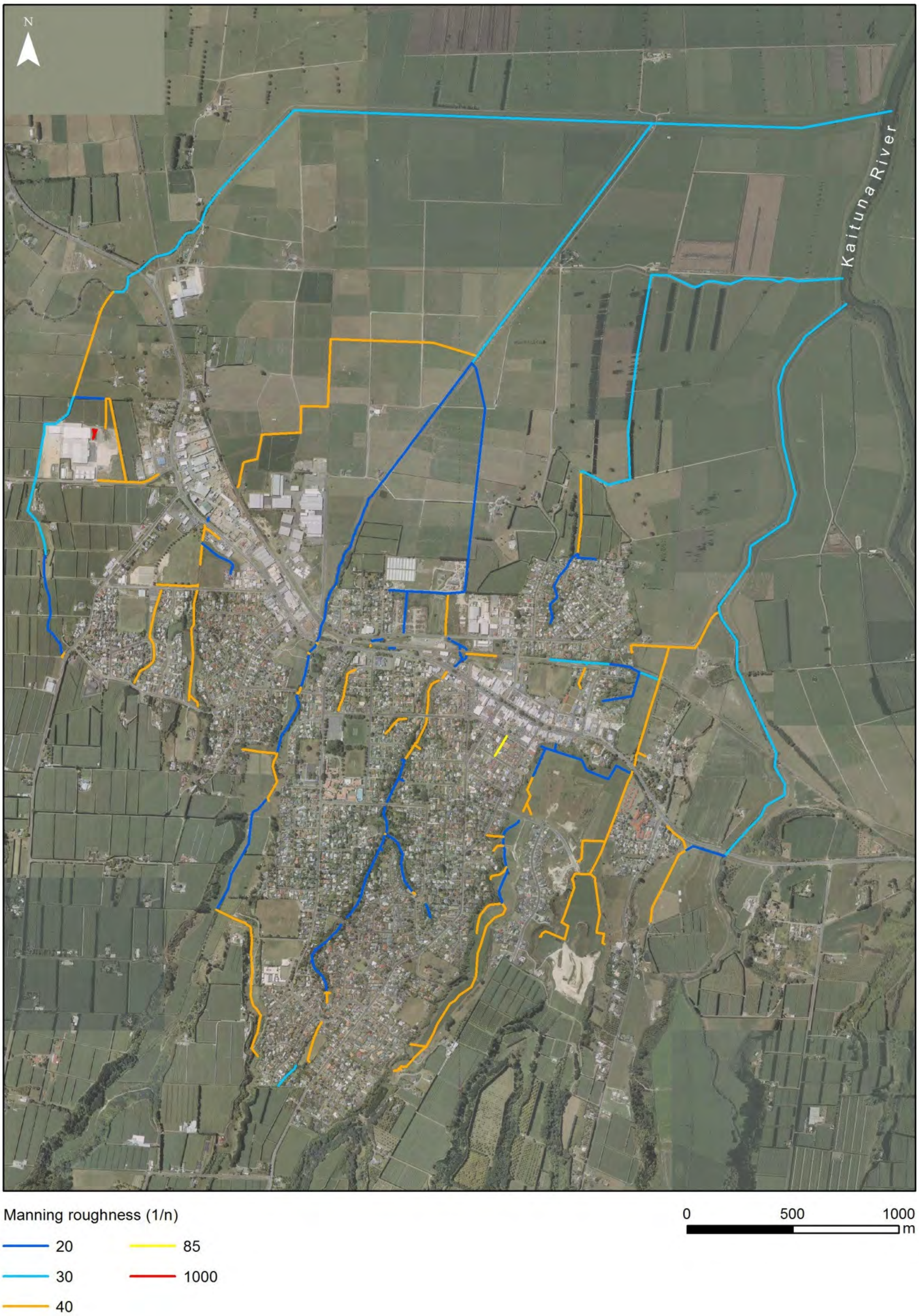
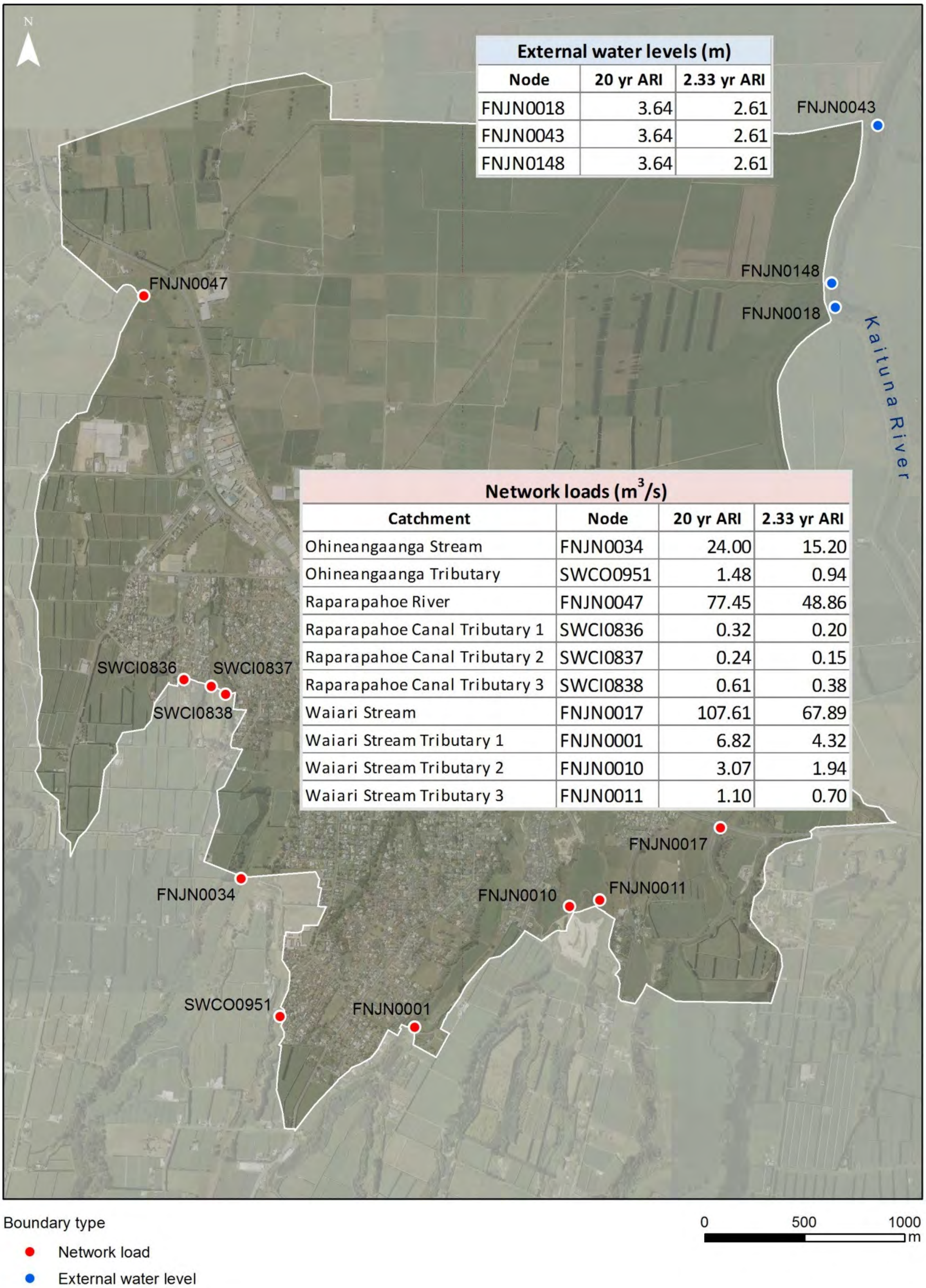


Figure A.7 Overview of channel (including overland flow paths) roughness



External water levels (m)

Node	20 yr ARI	2.33 yr ARI
FNJN0018	3.64	2.61
FNJN0043	3.64	2.61
FNJN0148	3.64	2.61

Network loads (m^3/s)

Catchment	Node	20 yr ARI	2.33 yr ARI
Ohineangaanga Stream	FNJN0034	24.00	15.20
Ohineangaanga Tributary	SWCO0951	1.48	0.94
Raparapahoe River	FNJN0047	77.45	48.86
Raparapahoe Canal Tributary 1	SWCI0836	0.32	0.20
Raparapahoe Canal Tributary 2	SWCI0837	0.24	0.15
Raparapahoe Canal Tributary 3	SWCI0838	0.61	0.38
Waiari Stream	FNJN0017	107.61	67.89
Waiari Stream Tributary 1	FNJN0001	6.82	4.32
Waiari Stream Tributary 2	FNJN0010	3.07	1.94
Waiari Stream Tributary 3	FNJN0011	1.10	0.70

Figure A.8 Magnitude and location of the boundary conditions applied to the MIKE URBAN model

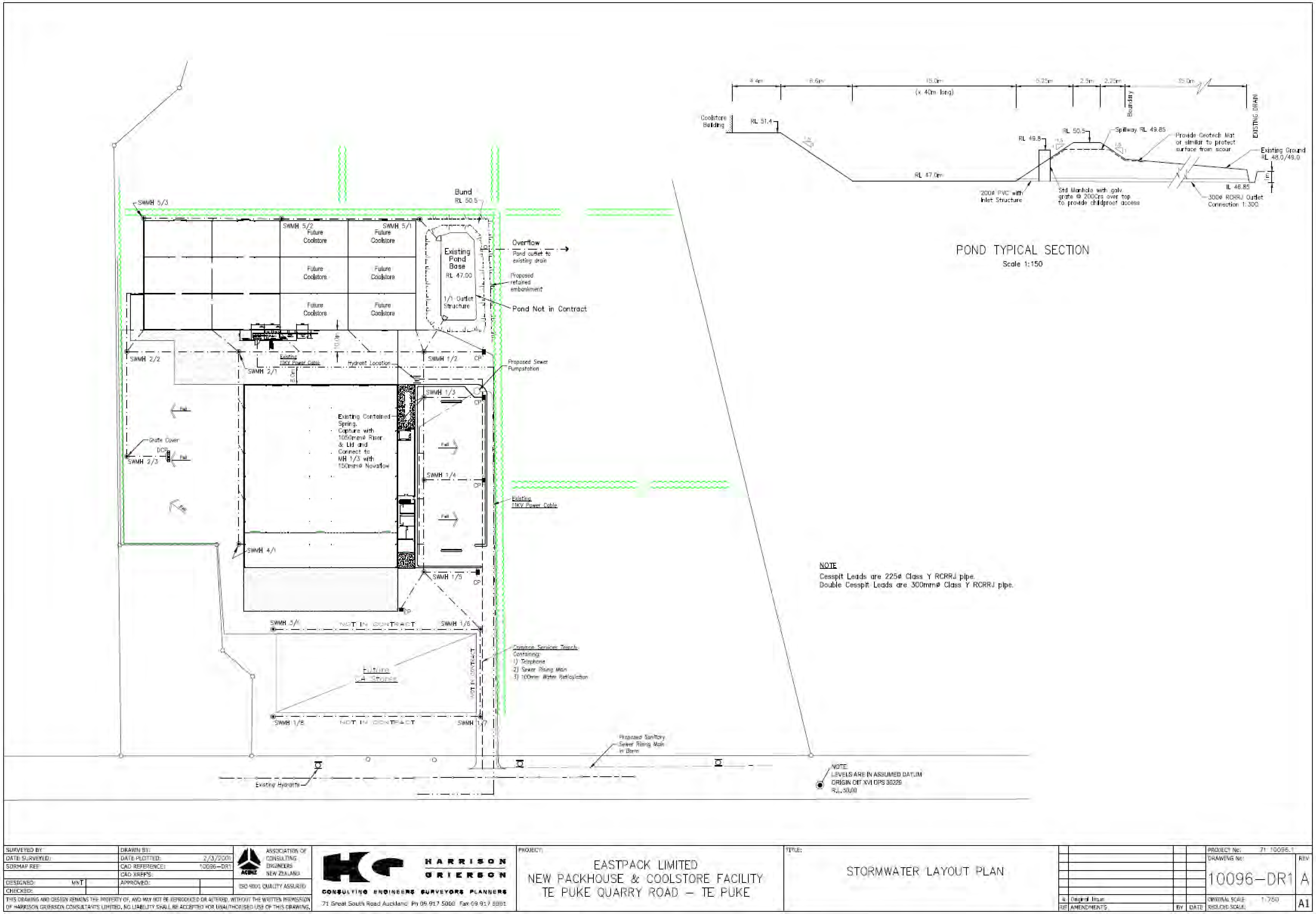


Figure A.9 EastPack Ltd stormwater layout and pond details (source: Harrison Grierson, 2001)

Appendix B – Impervious Area Samples

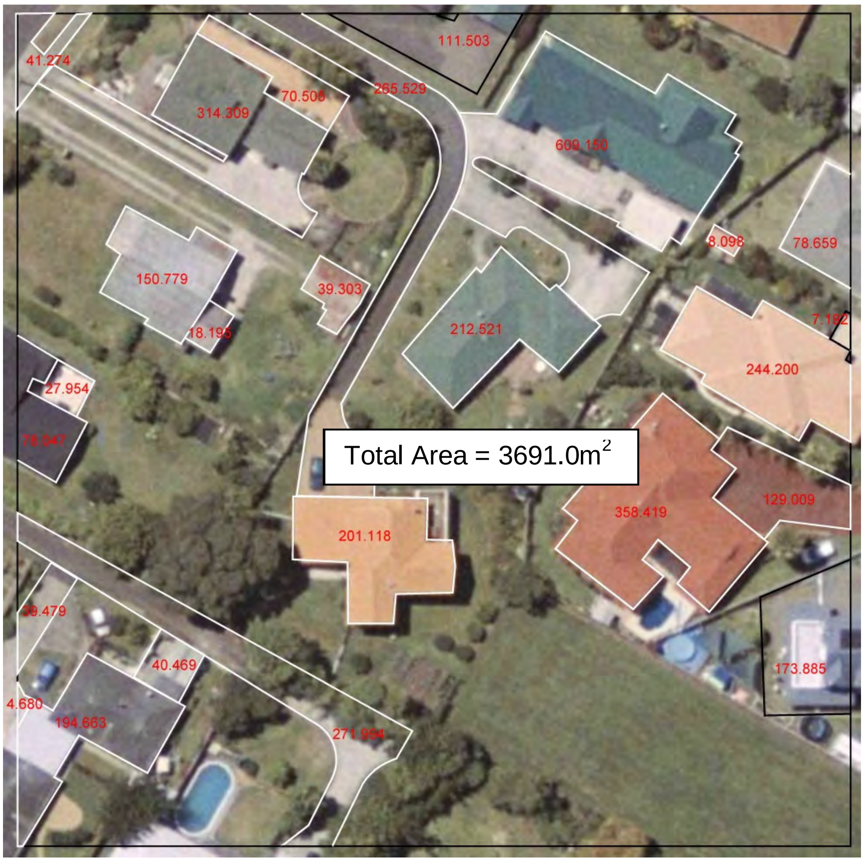


Figure B.1 Impervious area sample 1 (Residential)

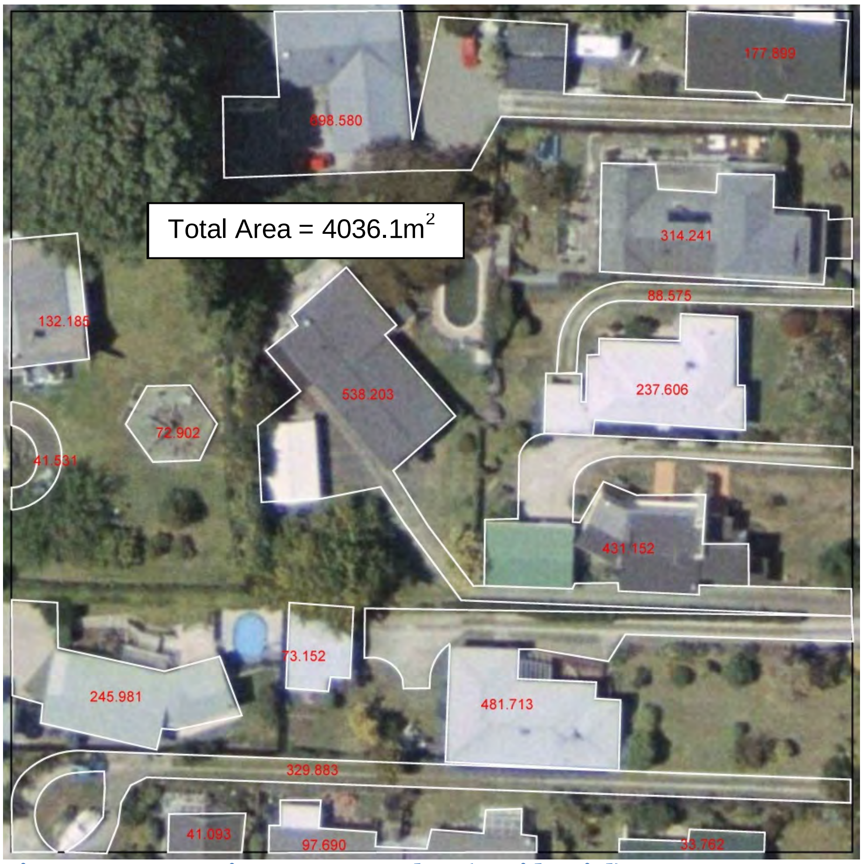


Figure B.2 Impervious area sample 2 (Residential)

Figure B.3 Impervious area sample 3 (Residential)

Figure B.4 Impervious area sample 4 (Commercial)

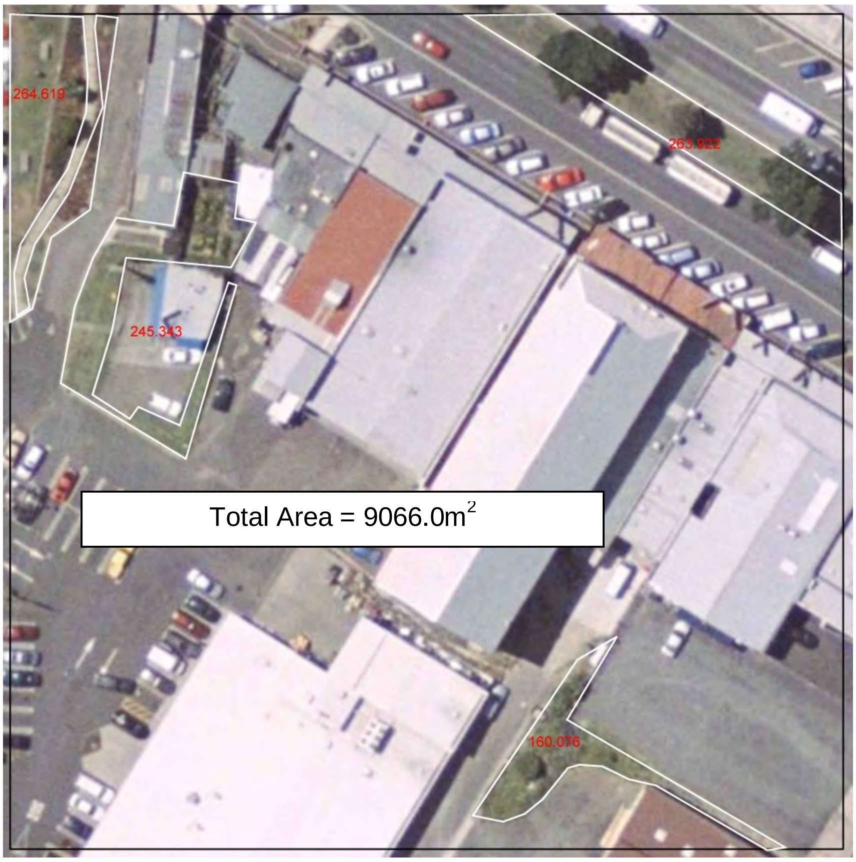


Figure B.5 Impervious area sample 5 (Commercial)

Figure B.6 Impervious area sample 6 (Residential)

Figure B.7 Impervious area sample 7 (Residential)

Figure B.8 Impervious area sample 8 (Residential)

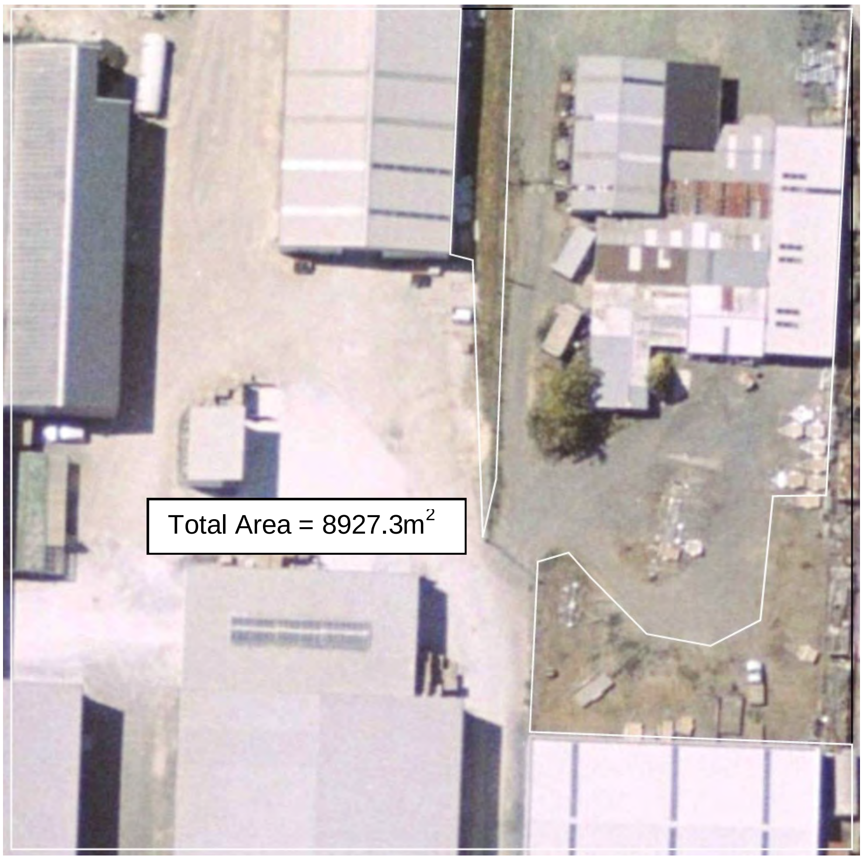


Figure B.9 Impervious area sample 9 (Industrial)

Figure B.10 Impervious area sample 10 (Industrial)

Figure B.11 Impervious area sample 11 (Industrial)

Appendix C – Stormwater Modelling Data Status Flagging

C.1 Status

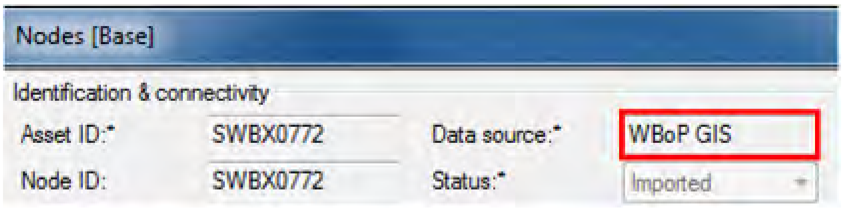
The assets have been split up into four groups:

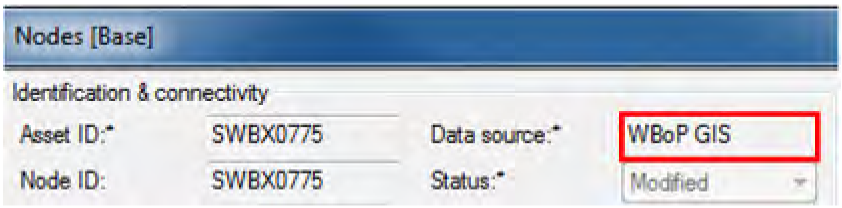
- **Imported:** Assets that have been imported from WBoP DC's GIS.
- **Modified:** Assets that have been imported from WBoP DC's GIS and subsequently modified.
- **Digitised:** Assets that are currently not in WBoP DC's GIS and have been digitised and added as a result of the model build.
- **Fictitious:** Items added for modelling purposes.

C.2 Data Source

Imported & Modified

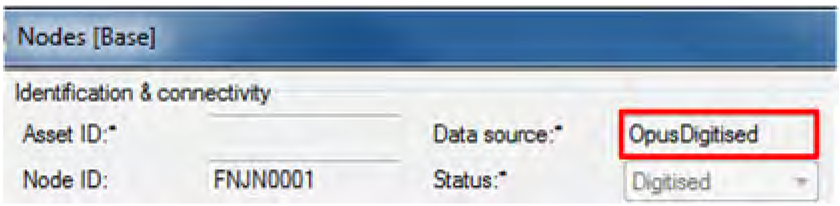
= ***WBoP GIS***

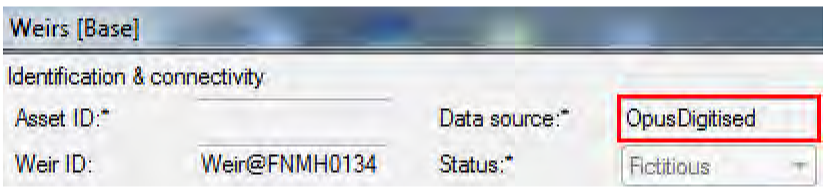




Digitised & Fictitious

= ***OpusDigitised***

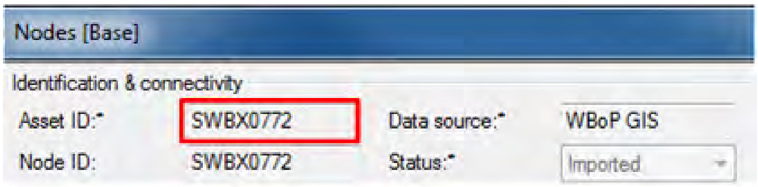


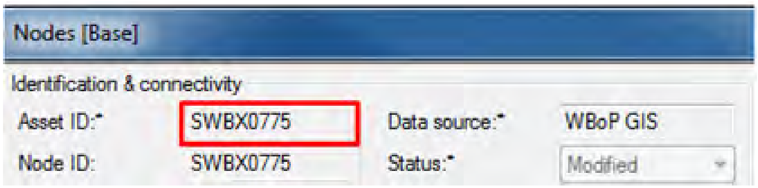


C.3 Asset ID

Imported & Modified

= ***GIS ID***





Digitised & Fictitious

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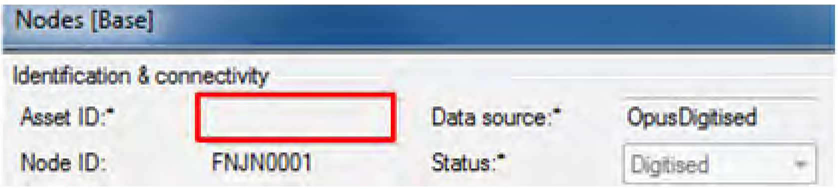
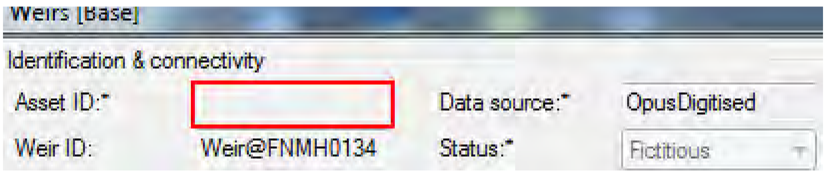
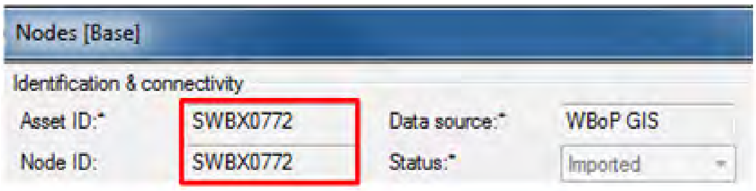


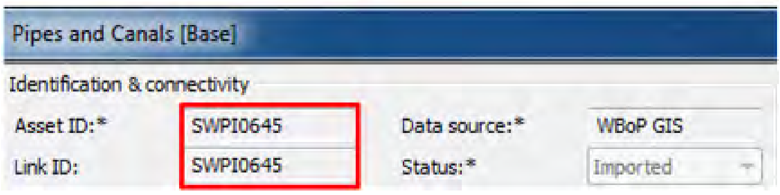


C.4 NodeID / LinkID

Imported

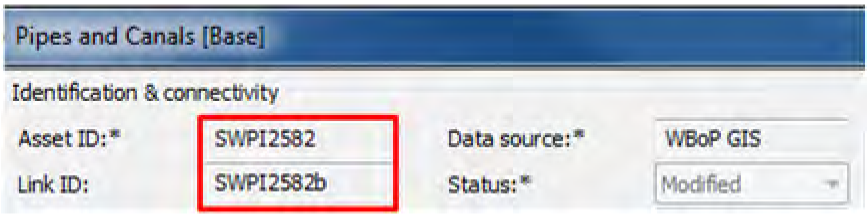
= ***Asset ID***



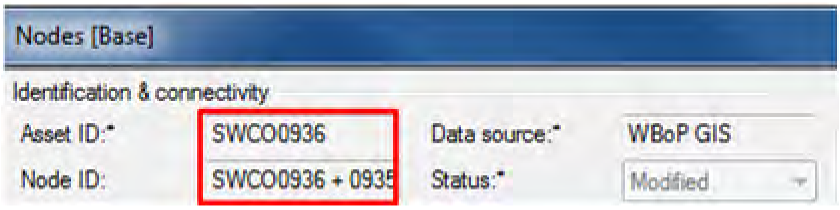


Modified

... split asset pipes:



... combined outlets:



Digitised

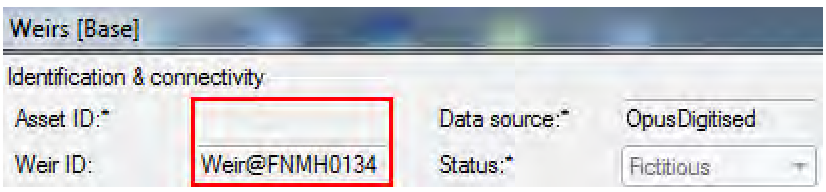
These have been labelled as per WBoP DC As-built Specification but with the SW prefix replaced with **FN** / **FL** (refer Table C.1) and a consecutive 4-digit number starting at 0001.

Table C.1 Prefixes of new digitised assets used in model build

Asset	Prefix	Type
Stormwater Pipe	FLPI	Link
Stormwater Open Drain	FLOD	Link
Stormwater Inlet	FNCI	Node
Stormwater Outlet	FNCO	Node
Stormwater Junction	FNJN	Node
Stormwater Manhole	FNMH	Node

Fictitious

Nine fictional weirs have been added to the model to facilitate the flow between the stormwater network and the overland flow paths. The WeirID reflects the real asset name that they are attached to.



The links and nodes representing the EastPack pond are other fictitious nodes and links inserted into the model. They have been given a prefix of **FN** / **FL** depending on the type (refer Table C.2).

Table C.2 Fictitious nodes and links inserted into the model to represent the EastPack pond

Asset	Prefix	Type
Fictitious (Storage Basin) Link	FLRE	Link
Fictitious (Storage Basin) Node	FNRE	Node

C.5 Other Data Status Flag Codes

MIKE URBAN allows a data status flag to be set for a number of key model parameters such as invert levels ground level, diameter and pipe length. Table C.3 gives a description of the status flags used.

Table C.3 Data status flag codes used for key model parameters

Flag	Applies To	Description
Assumed	Nodes & Links	The value was assumed based on other information and experience with stormwater assets
Calculated	Nodes & Links	The value was calculated from other relevant information
GIS	Nodes & Links	The value was imported directly from GIS
Interpolated	Nodes & Links	The value was interpolated based on other nearby values
LiDAR/DEM	Nodes & Links	The values was derived from the LiDAR data
Link	Nodes	The value was derived from the relevant value from the attached link
Modified	Nodes & Links	The value was modified for modelling purposes
Node	Links	The value was derived from the relevant value from the attached node
Surveyed	Nodes & Links	The value was obtained through field measurements and/or surveying

Appendix D – Model Results

D.1 5-year ARI Rainfall Event

Figure D.1.1 Pipe filling – 5-year ARI, 10 minute duration
Figure D.1.2 Pipe capacity – 5-year ARI, 10 minute duration
Figure D.1.3 Pipe filling – 5-year ARI, 30 minute duration
Figure D.1.4 Pipe capacity – 5-year ARI, 30 minute duration
Figure D.1.5 Pipe filling – 5-year ARI, 1 hour duration
Figure D.1.6 Pipe capacity – 5-year ARI, 1 hour duration

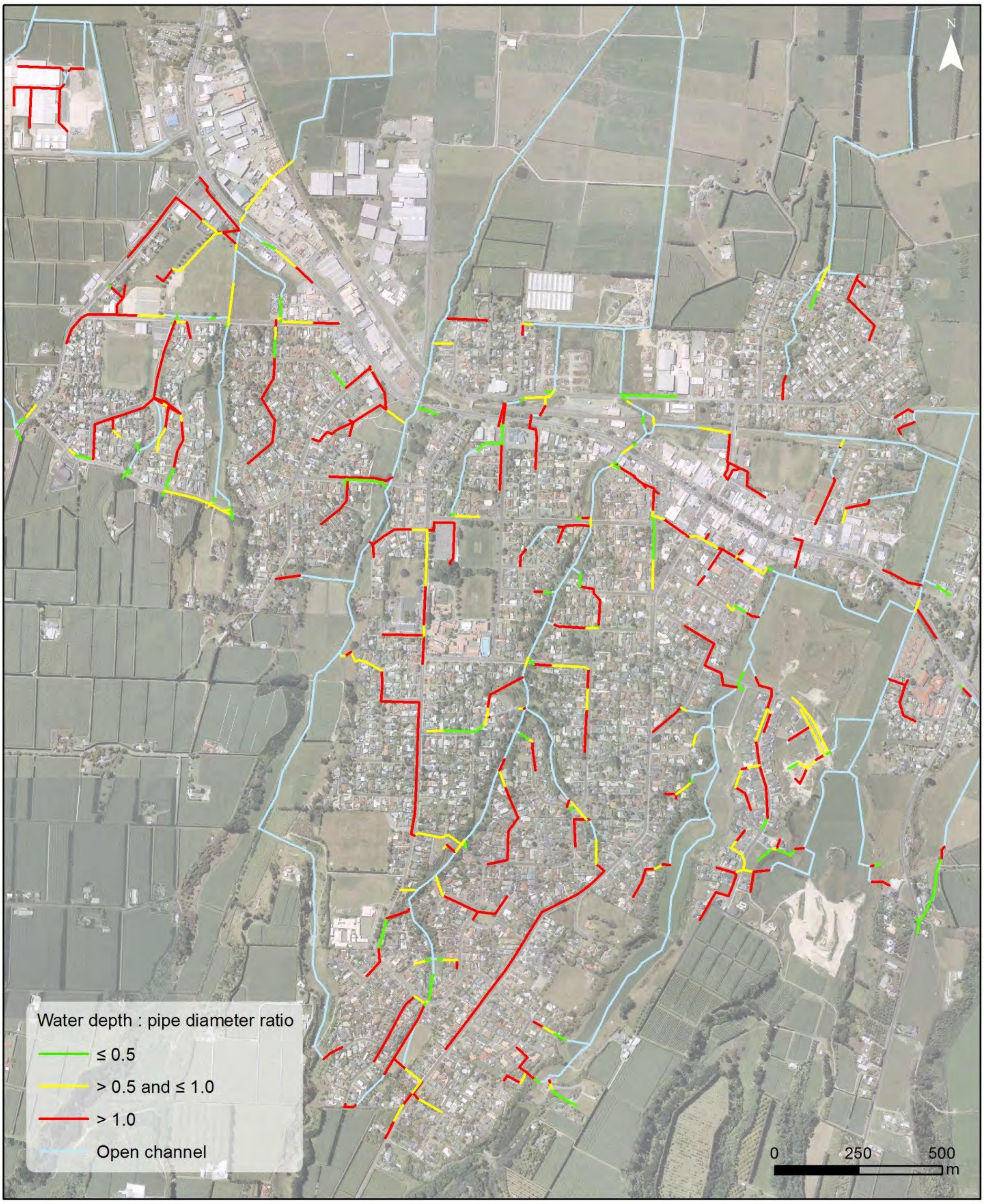


Figure D.1.1 Pipe filling – 5-year ARI, 10 minute duration

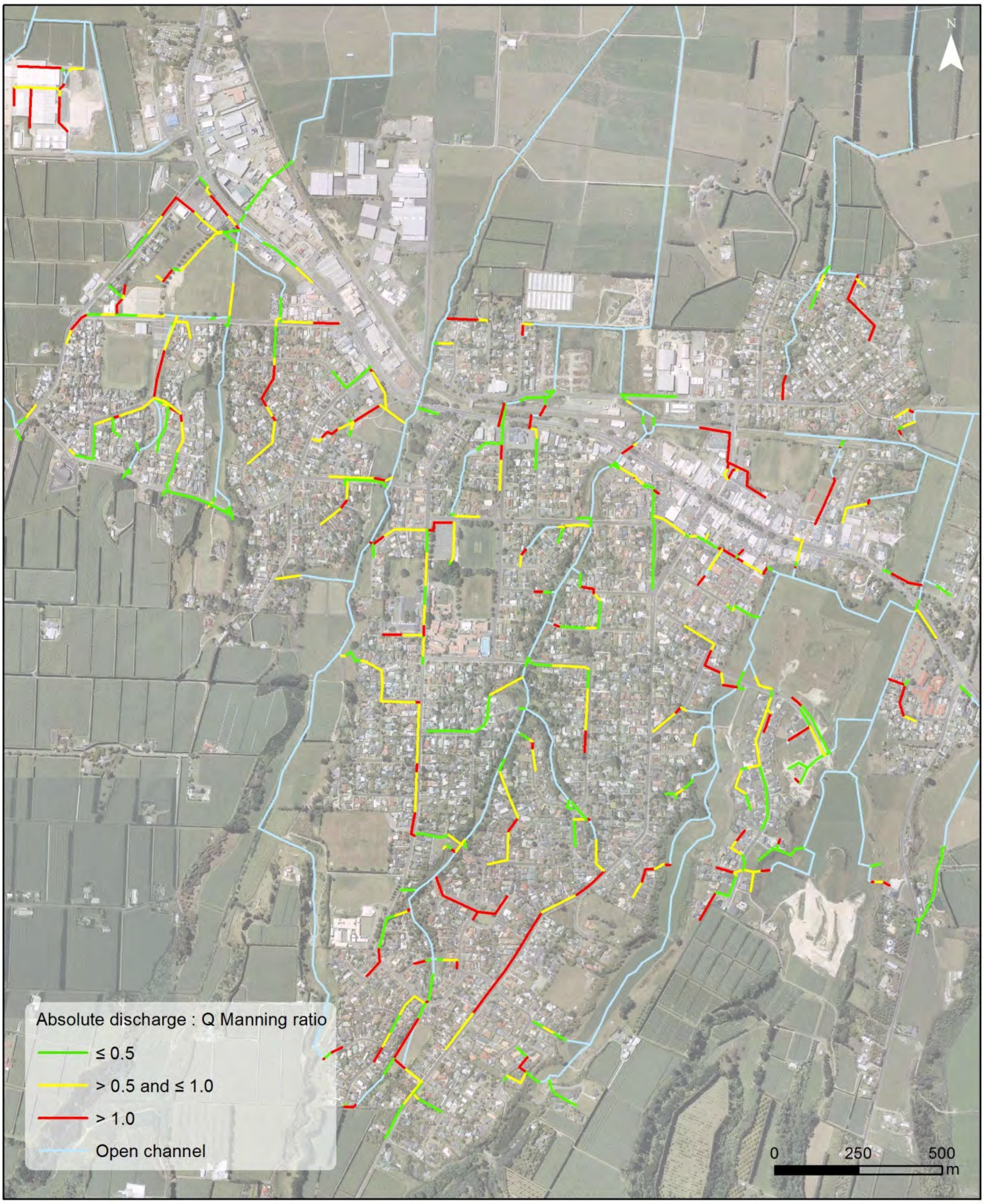


Figure D.1.2 Pipe capacity – 5-year ARI, 10 minute duration

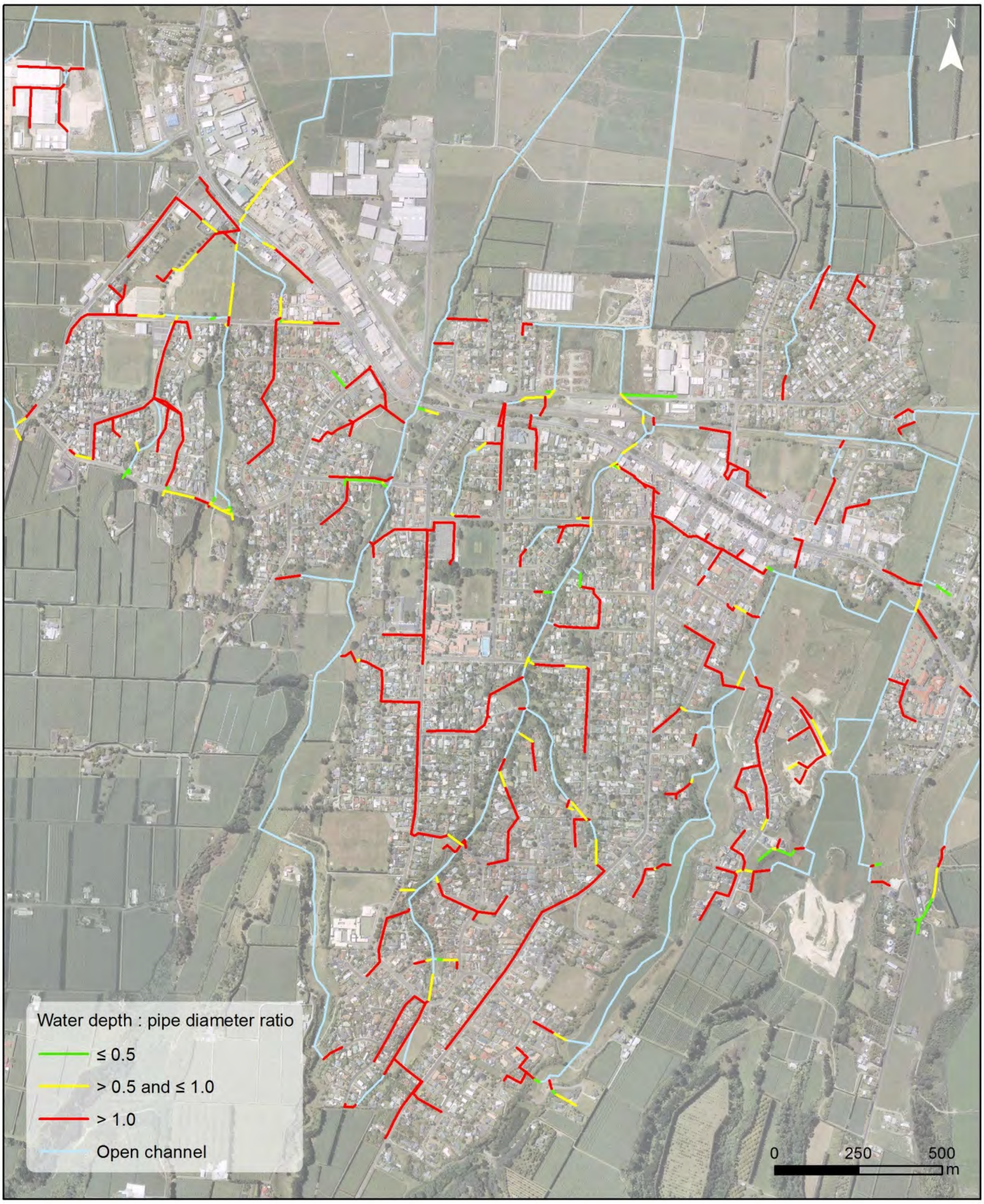


Figure D.1.3 Pipe filling – 5-year ARI, 30 minute duration

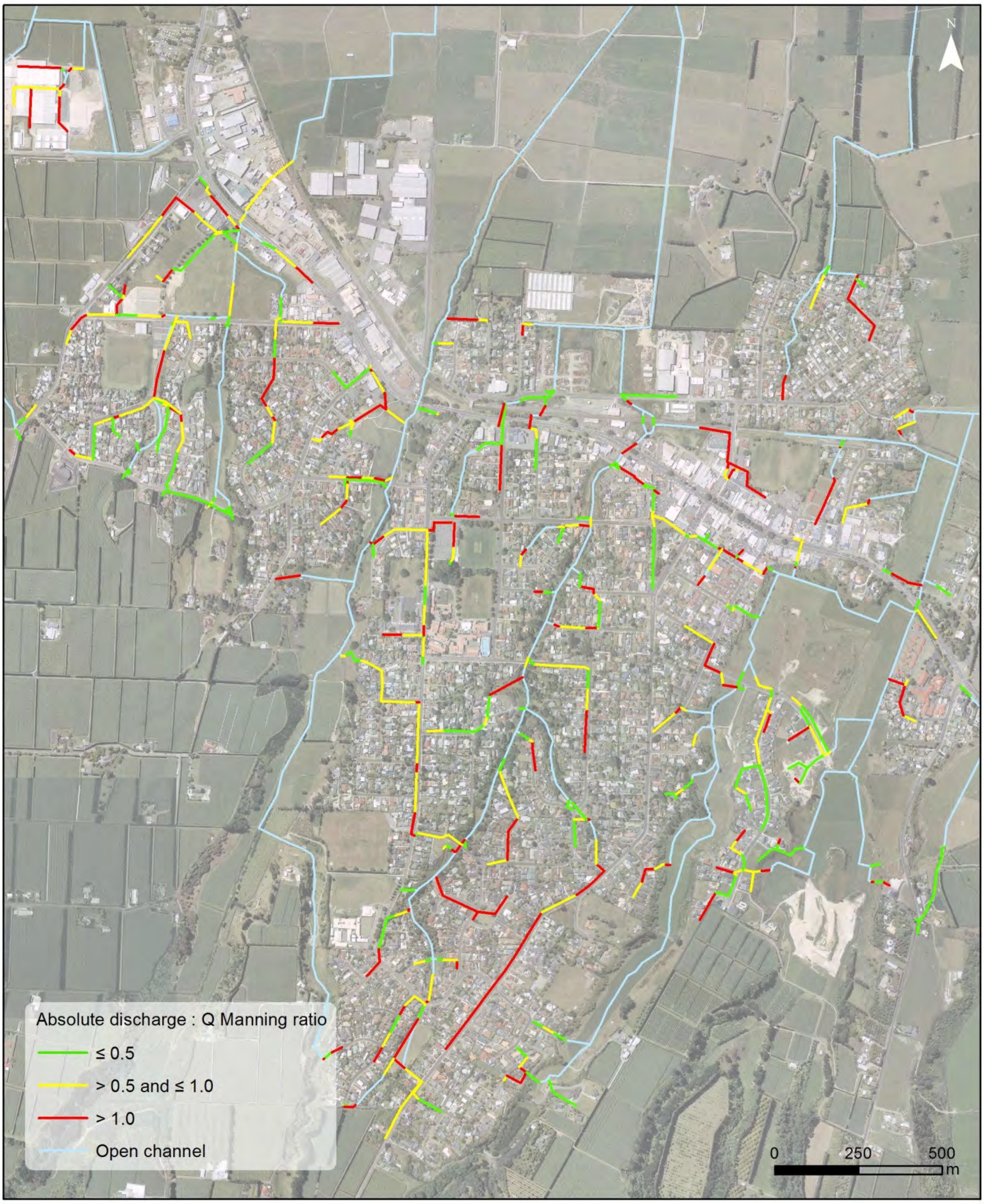


Figure D.1.4 Pipe capacity – 5-year ARI, 30 minute duration

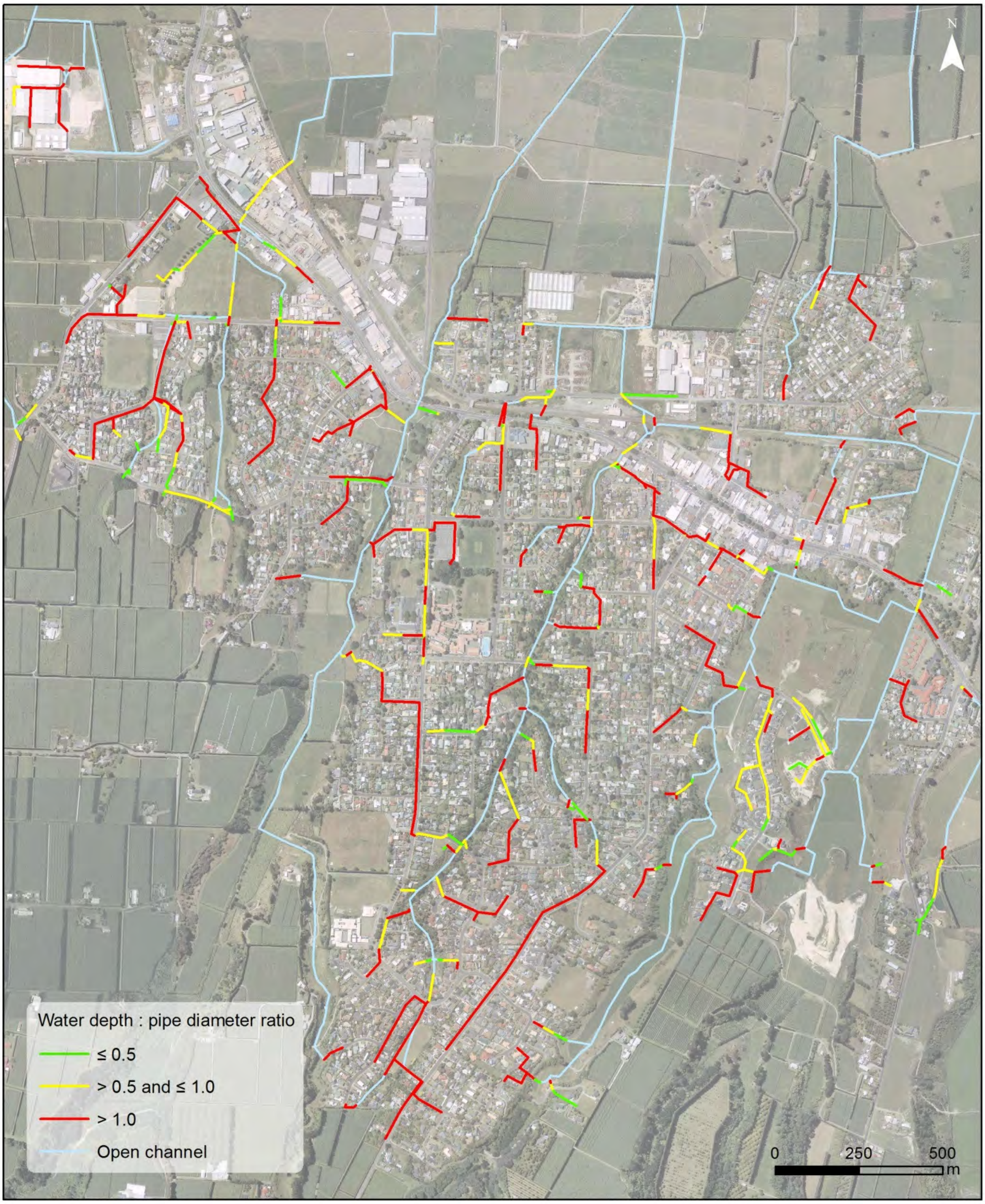


Figure D.1.5 Pipe filling – 5-year ARI, 1 hour duration

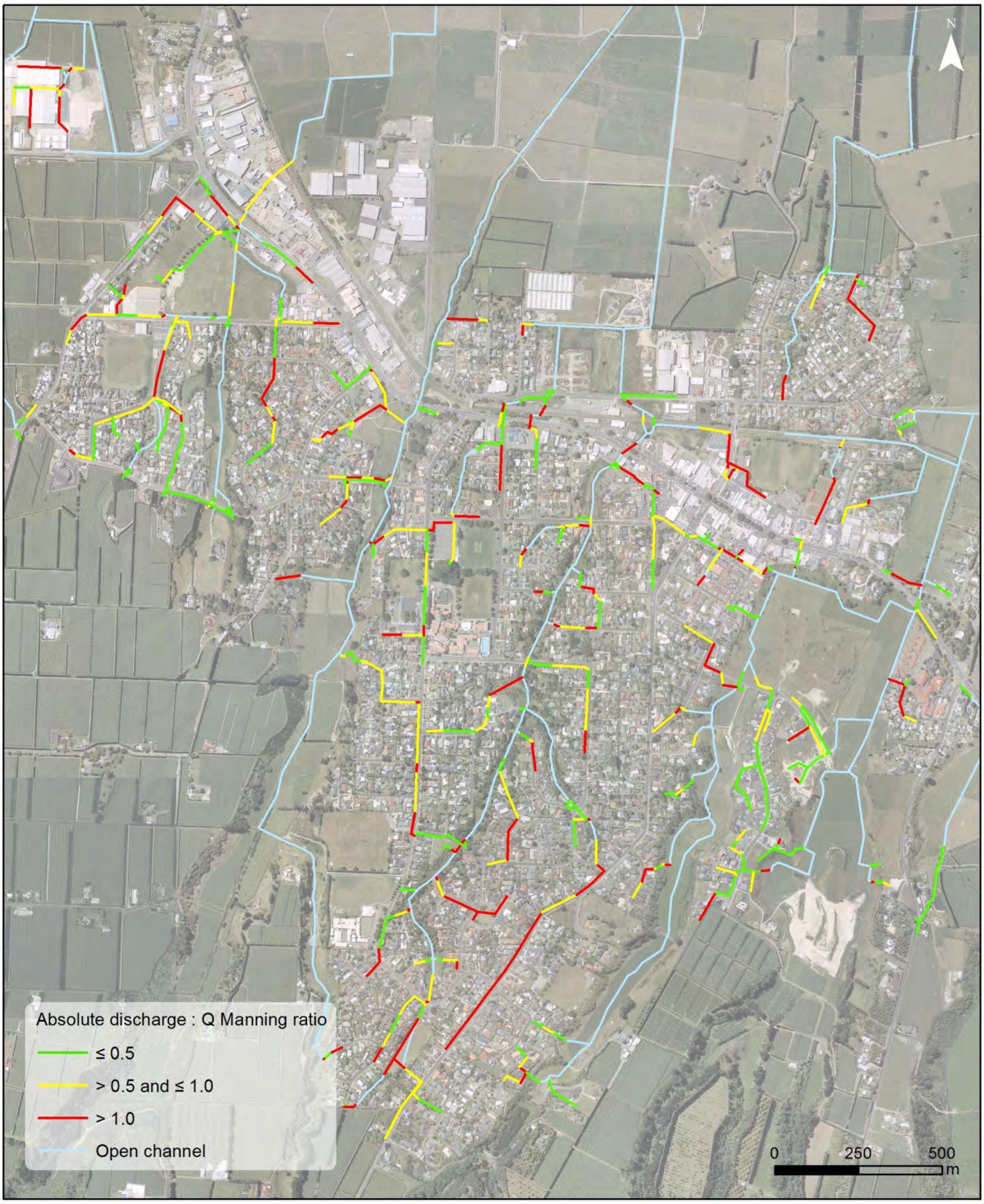


Figure D.1.6 Pipe capacity – 5-year ARI, 1 hour duration

D.2 10-year ARI Rainfall Event

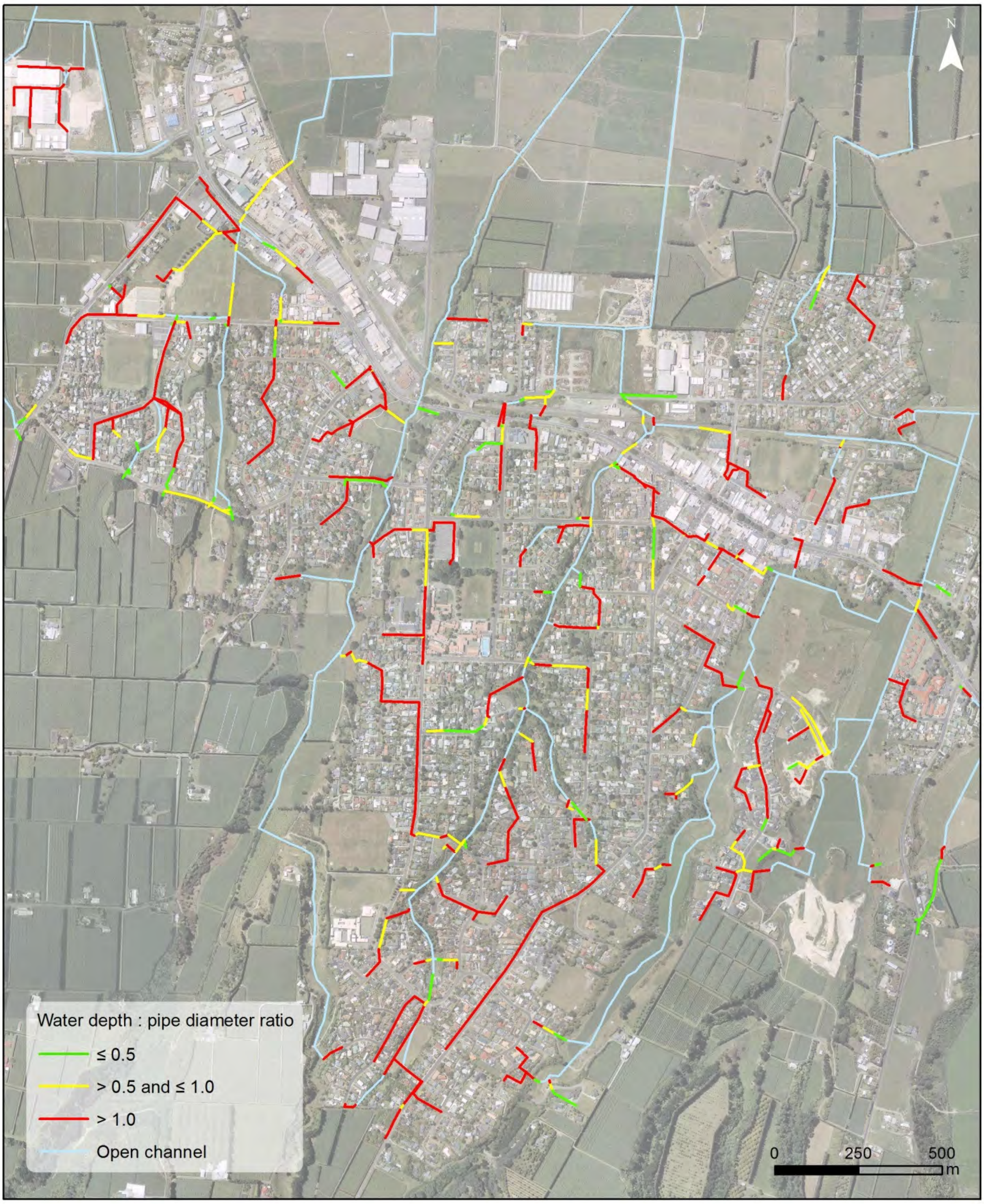


Figure D.2.1 Pipe filling – 10-year ARI, 10 minute duration

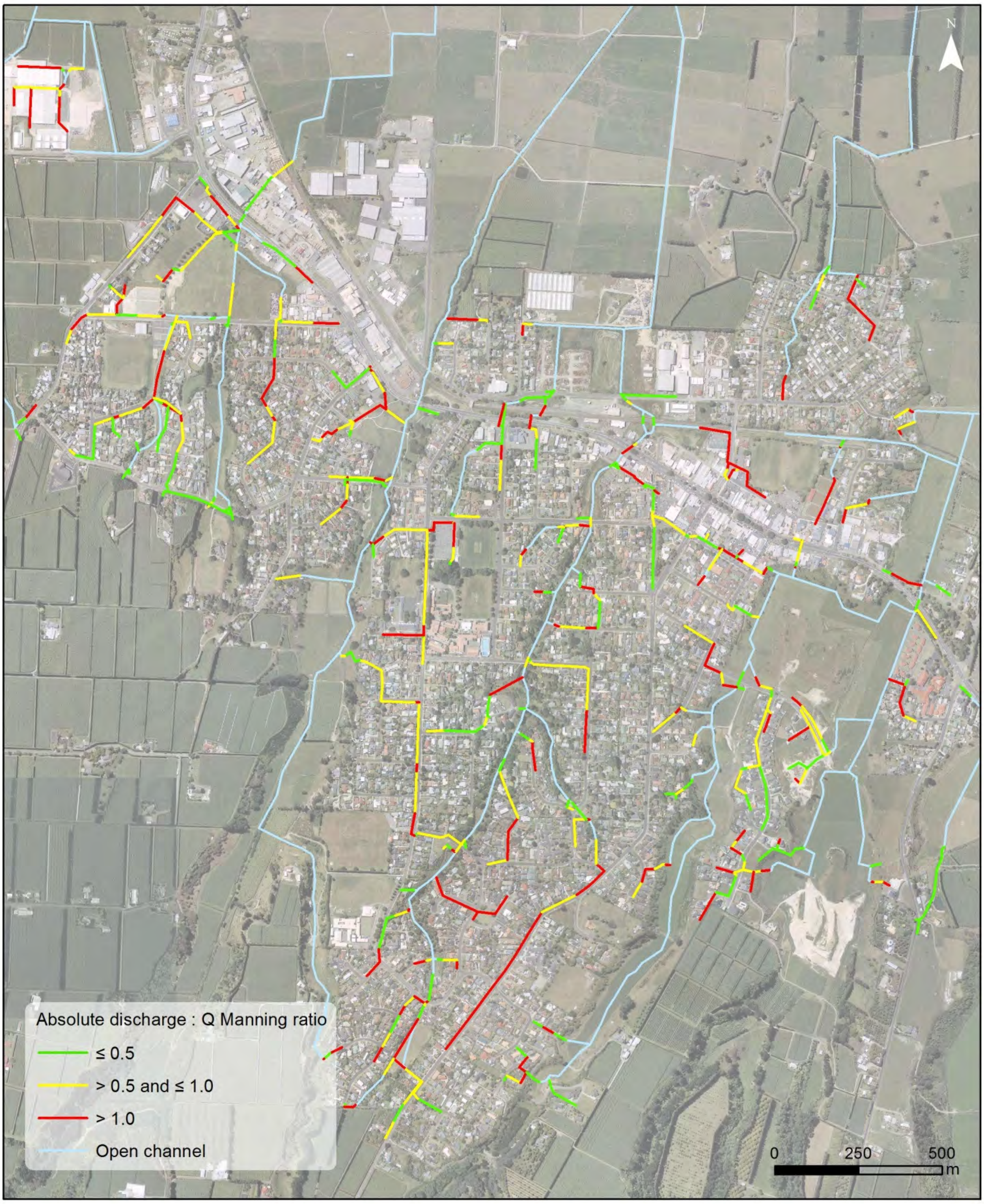


Figure D.2.2 Pipe capacity – 10-year ARI, 10 minute duration

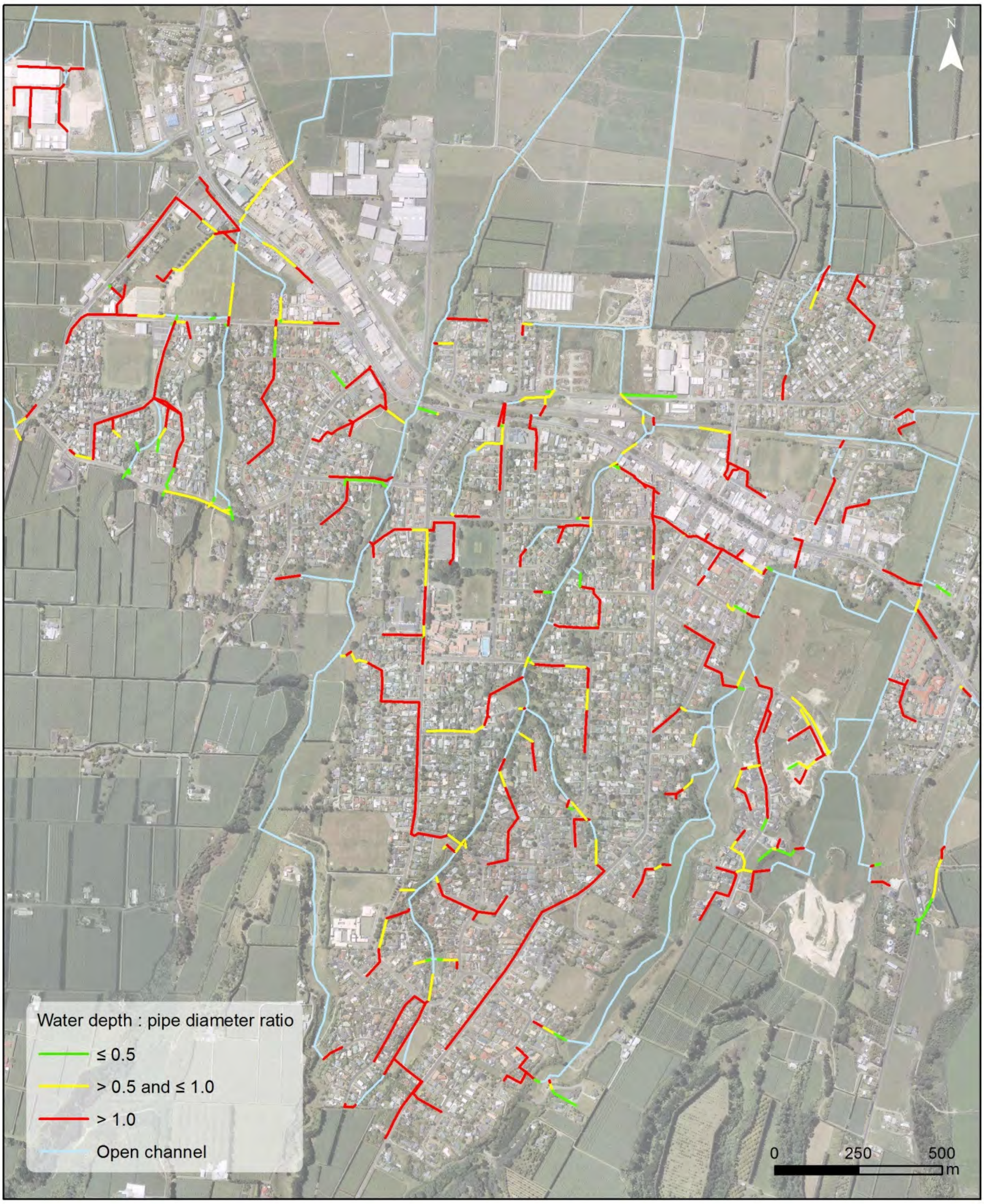


Figure D.2.3 Pipe filling – 10-year ARI, 30 minute duration

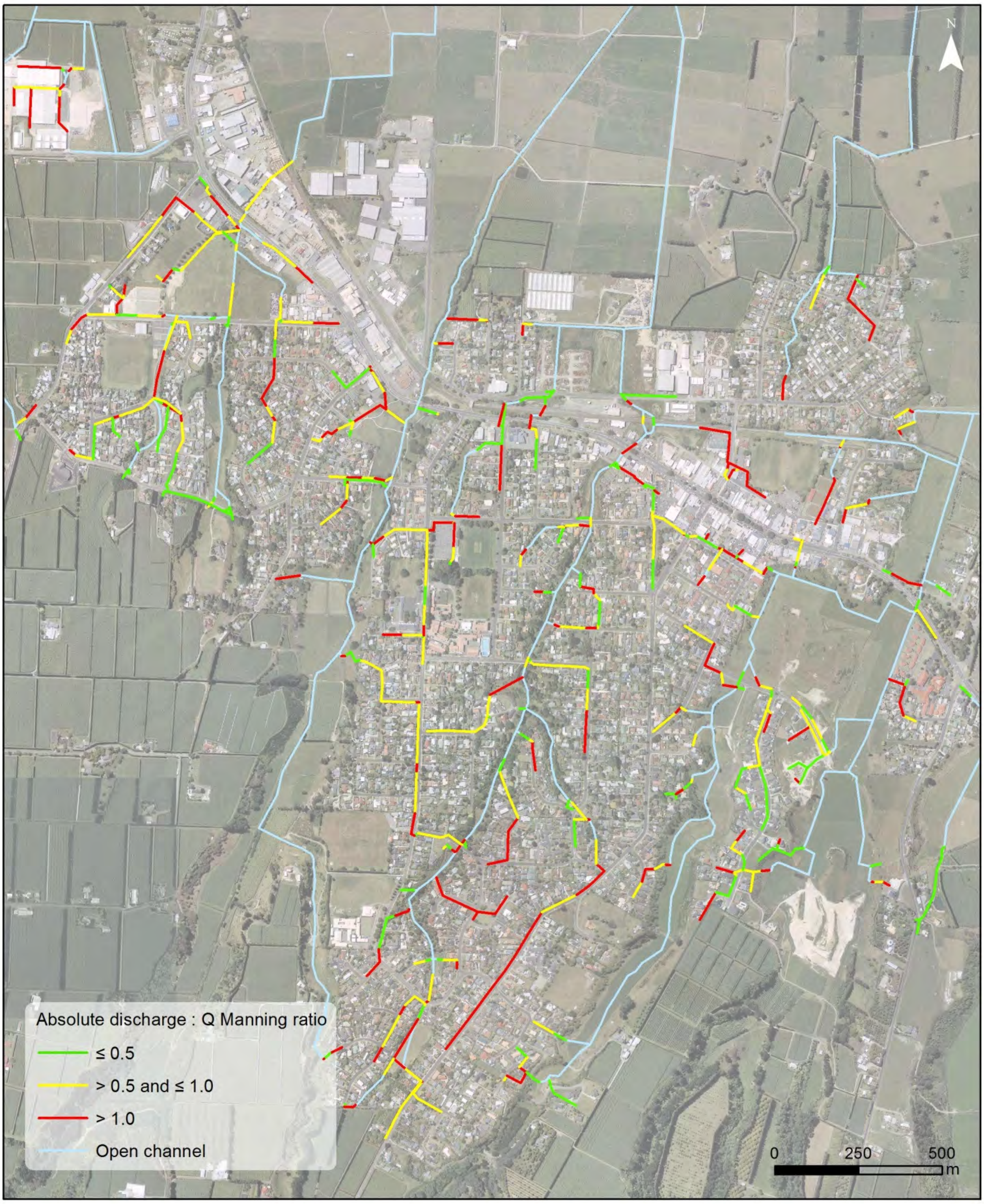


Figure D.2.4 Pipe capacity – 10-year ARI, 30 minute duration

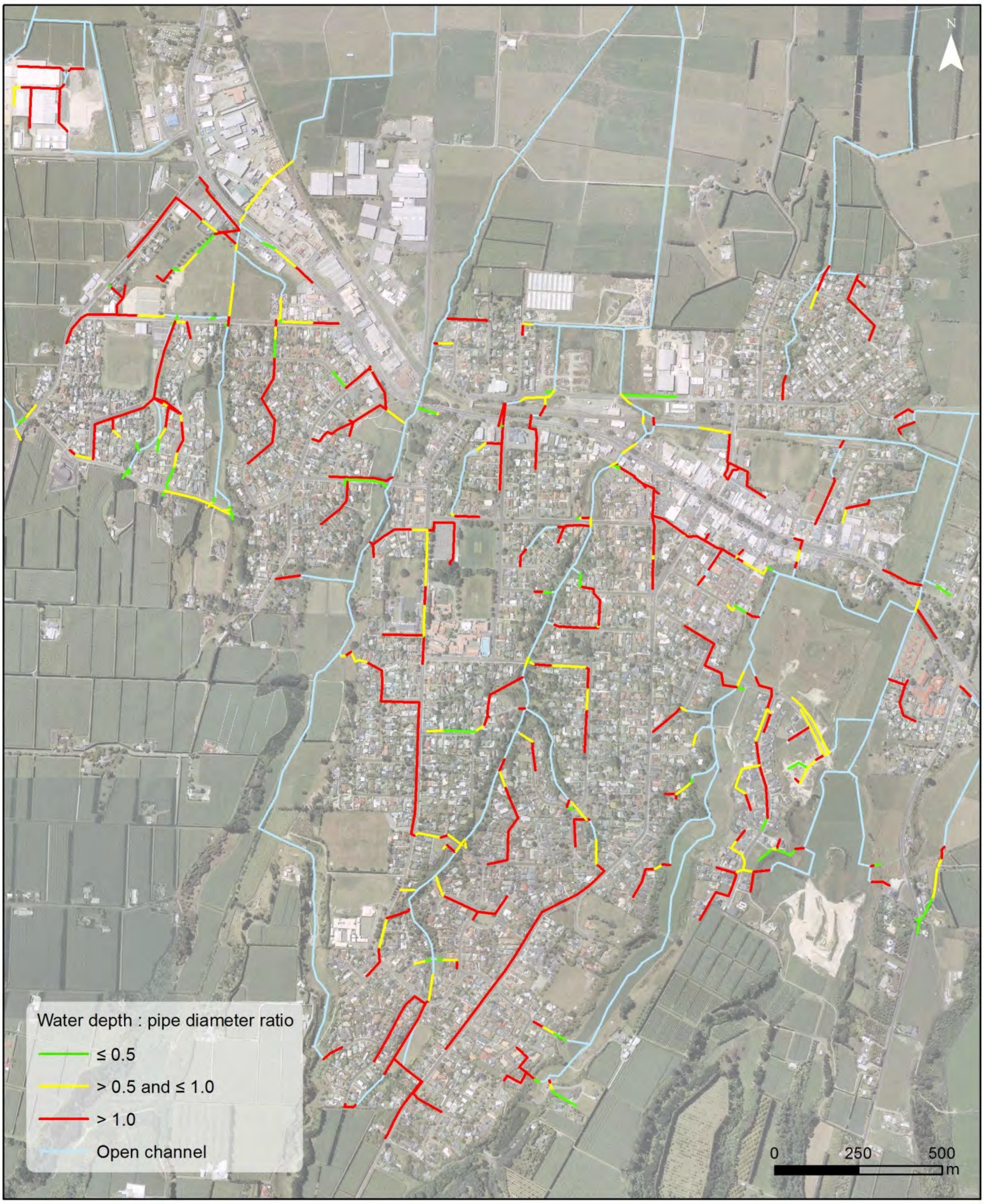


Figure D.2.5 Pipe filling – 10-year ARI, 1 hour duration

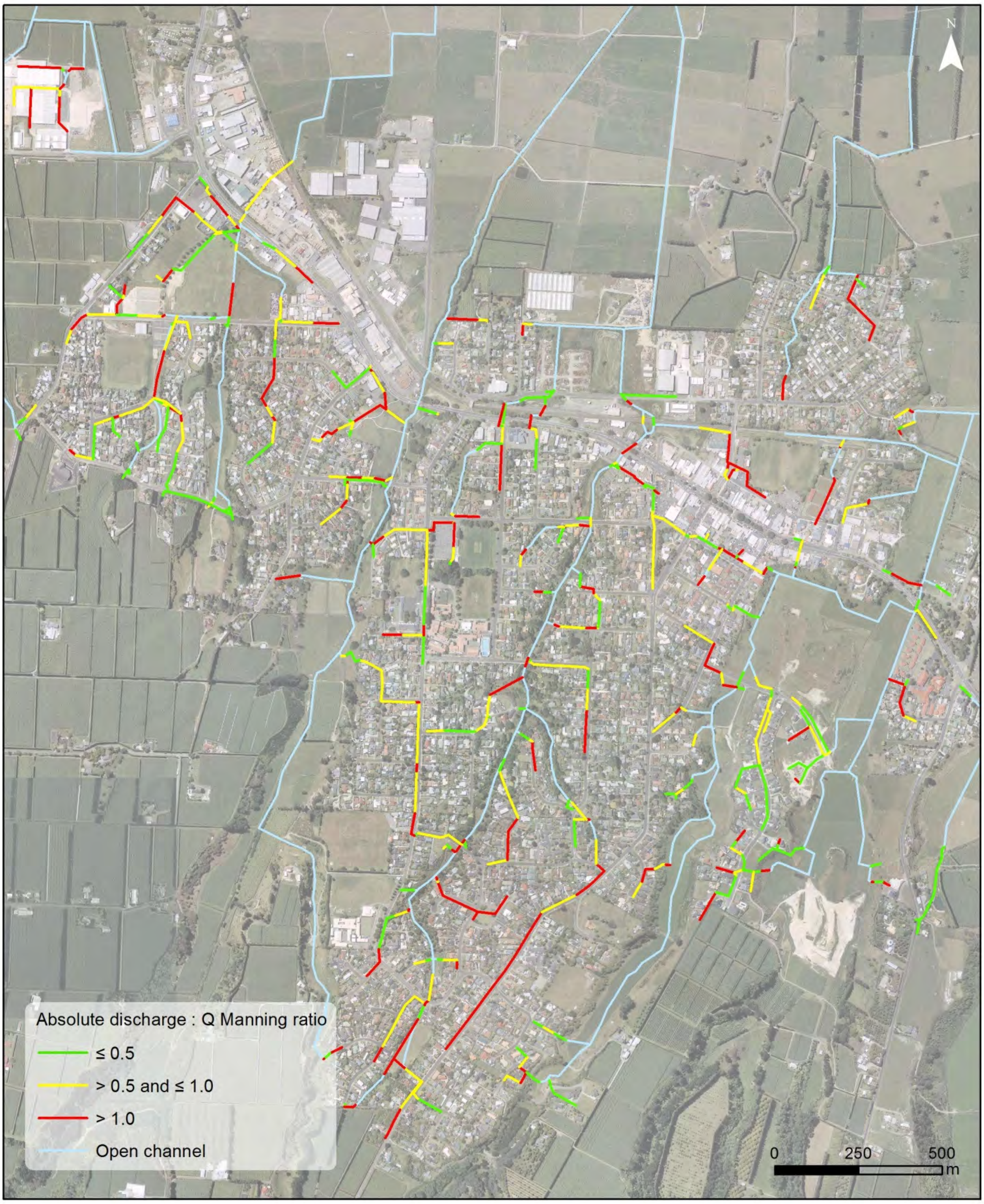


Figure D.2.6 Pipe capacity – 10-year ARI, 1 hour duration

D.3 50-year ARI Rainfall Event

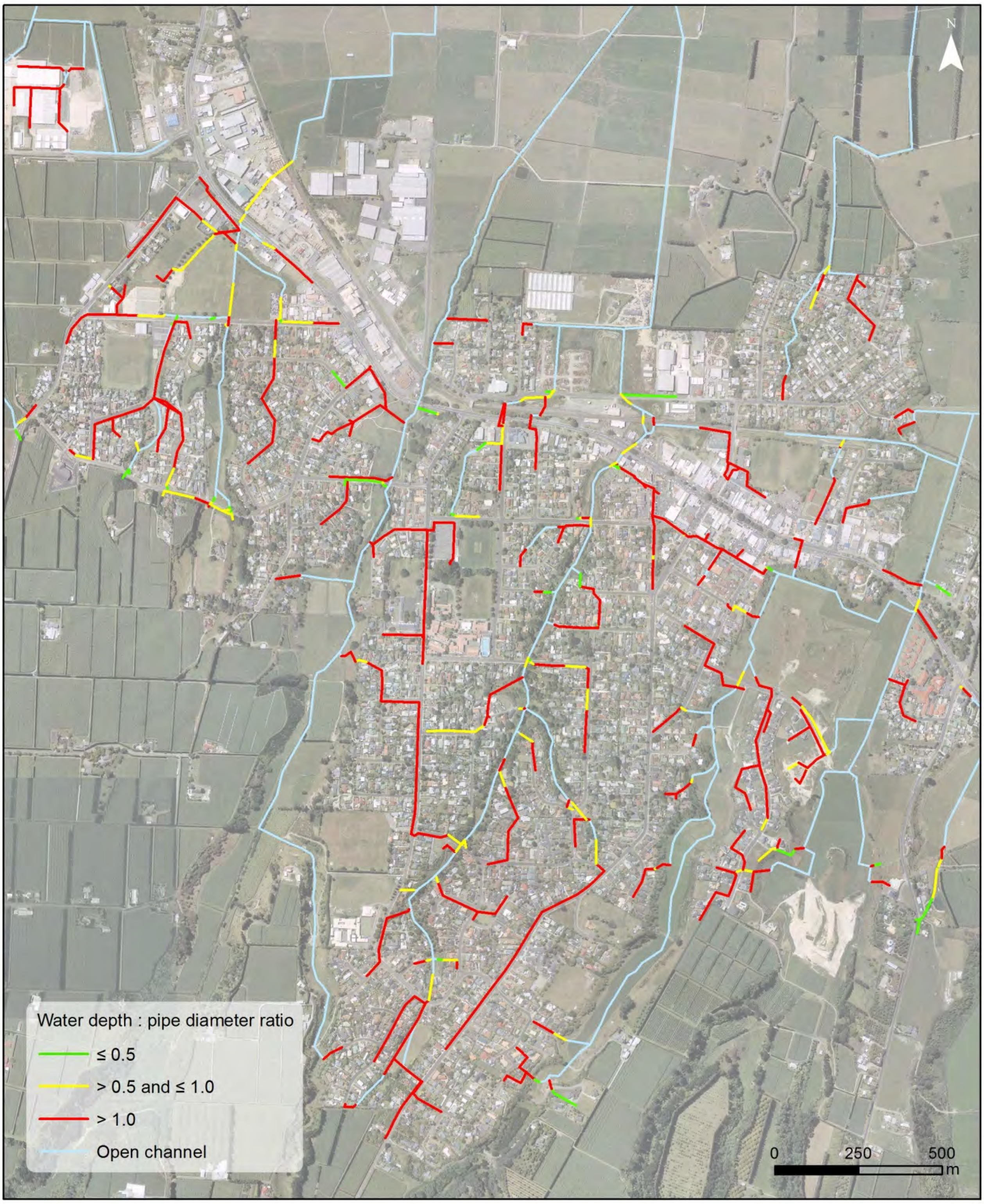


Figure D.3.1 Pipe filling – 50-year ARI, 10 minute duration

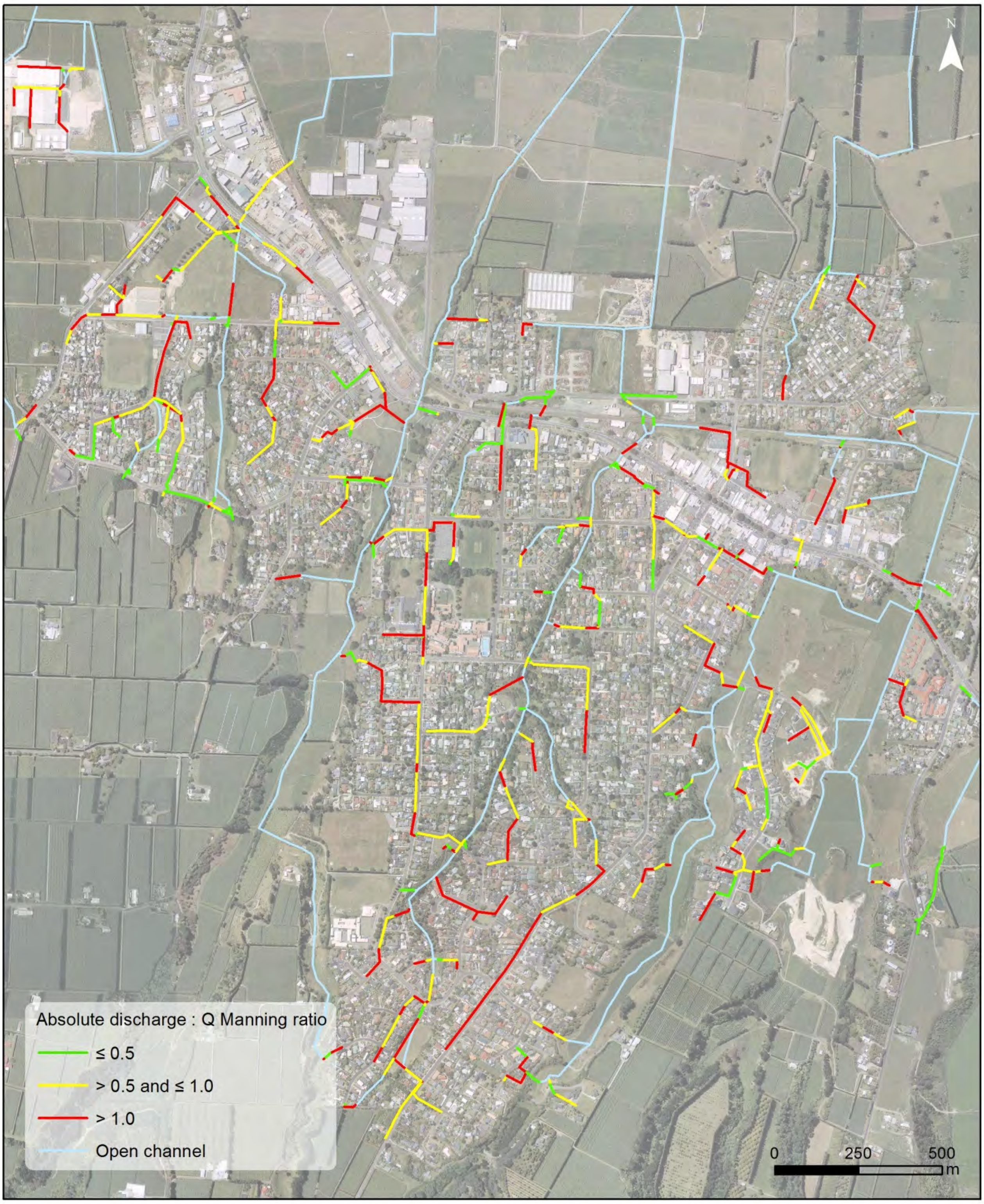


Figure D.3.2 Pipe capacity – 50-year ARI, 10 minute duration

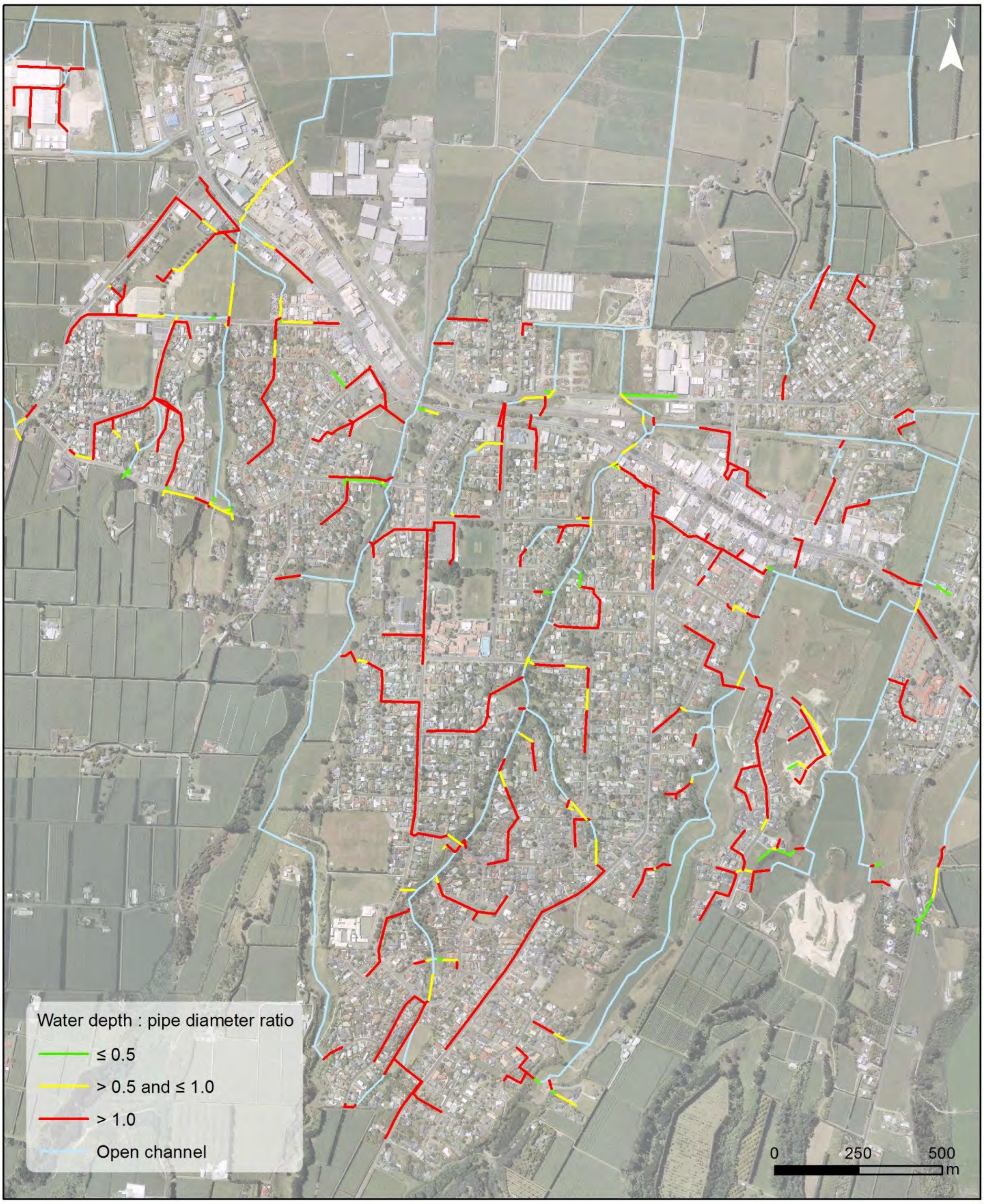


Figure D.3.3 Pipe filling – 50-year ARI, 30 minute duration

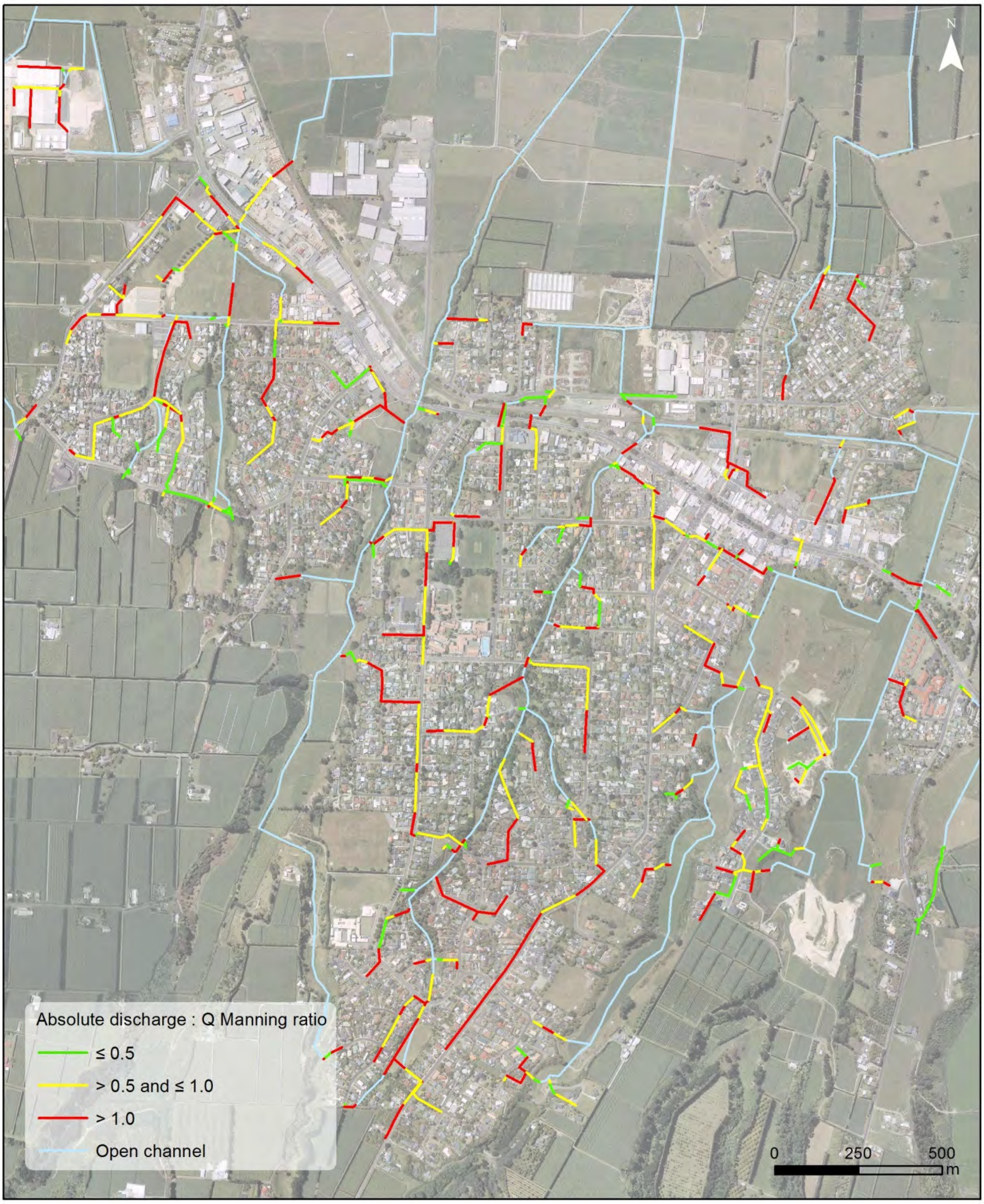


Figure D.3.4 Pipe capacity – 50-year ARI, 30 minute duration

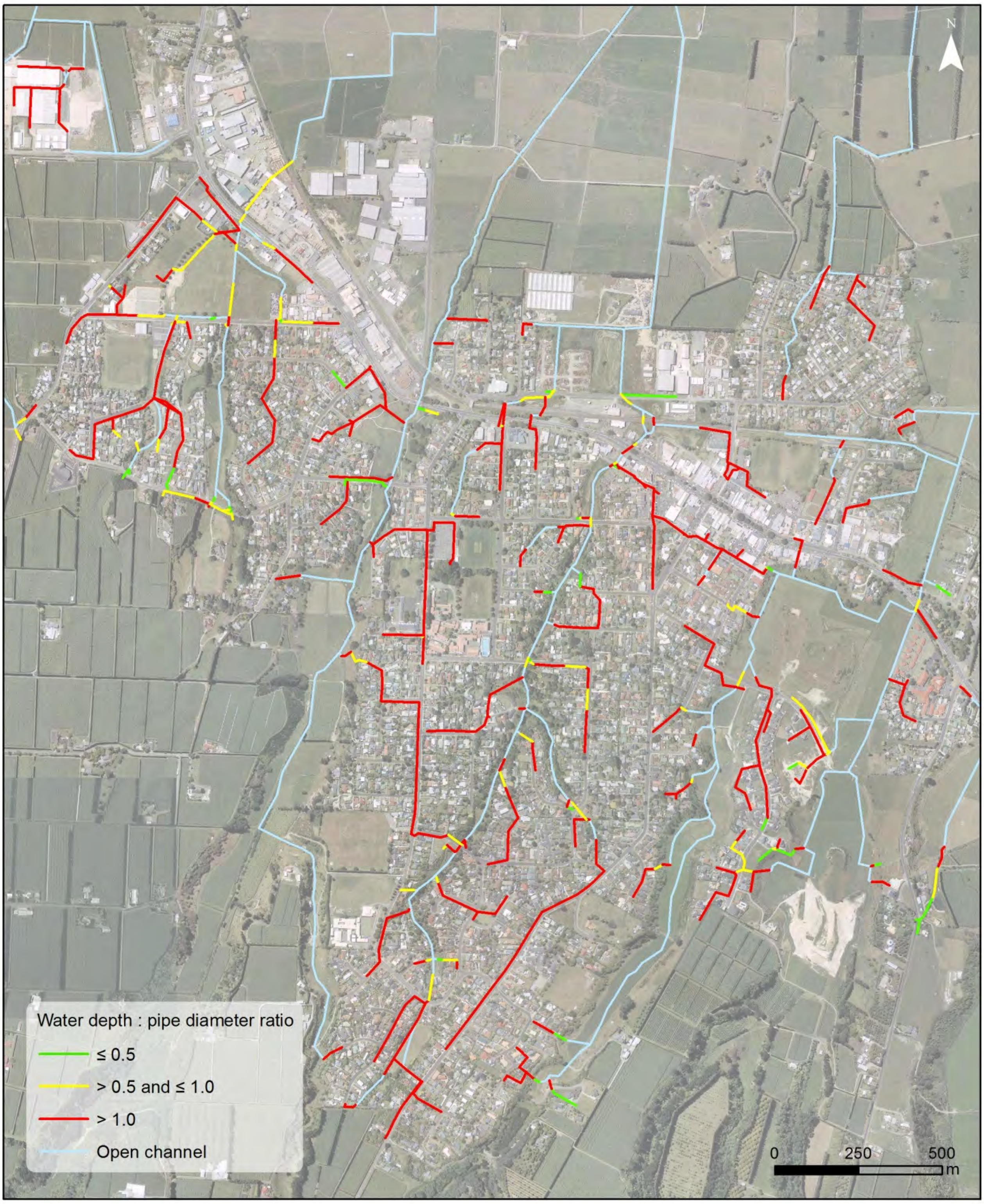


Figure D.3.5 Pipe filling – 50-year ARI, 1 hour duration

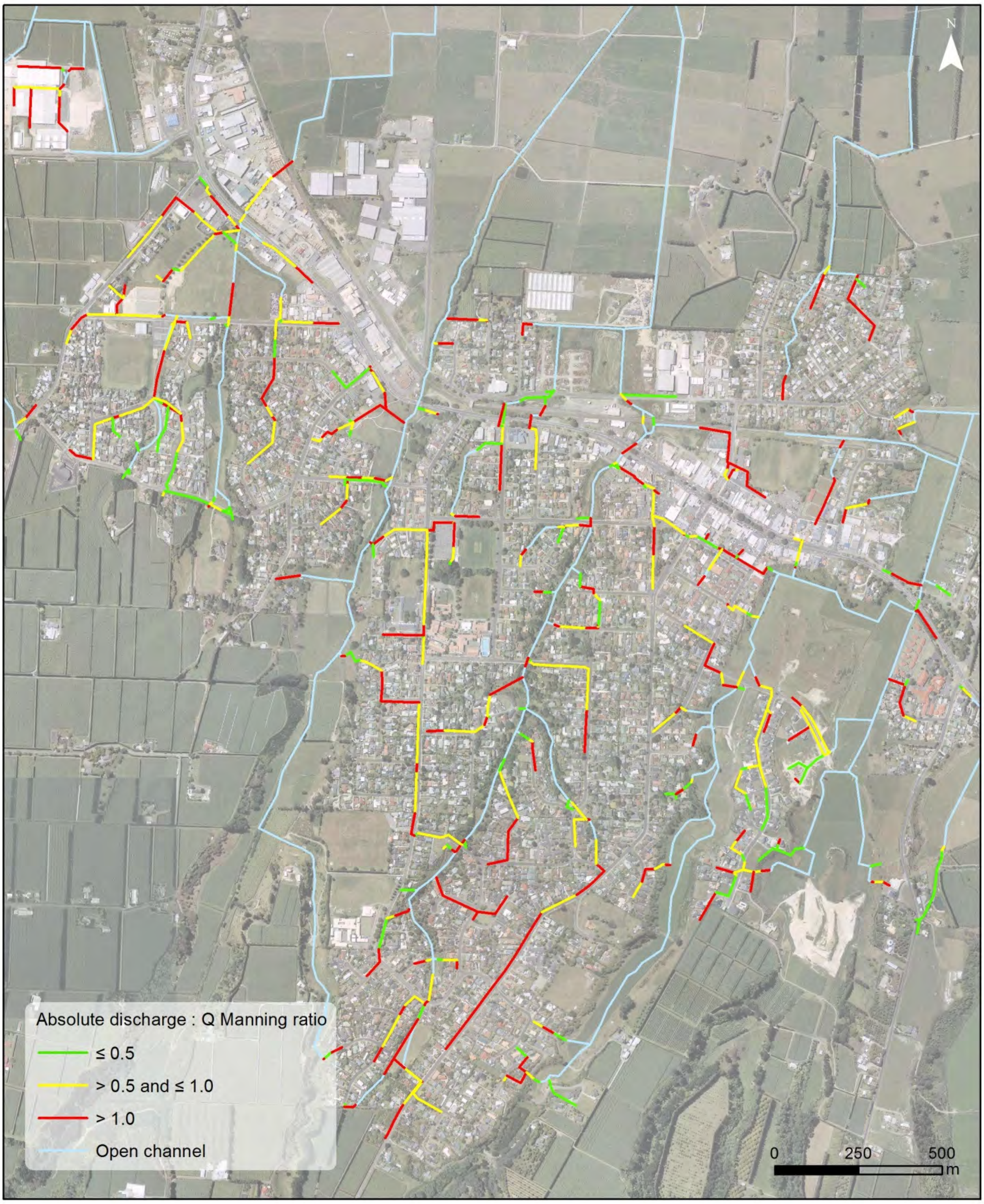


Figure D.3.6 Pipe capacity – 50-year ARI, 1 hour duration

D.4 100-year ARI Rainfall Event

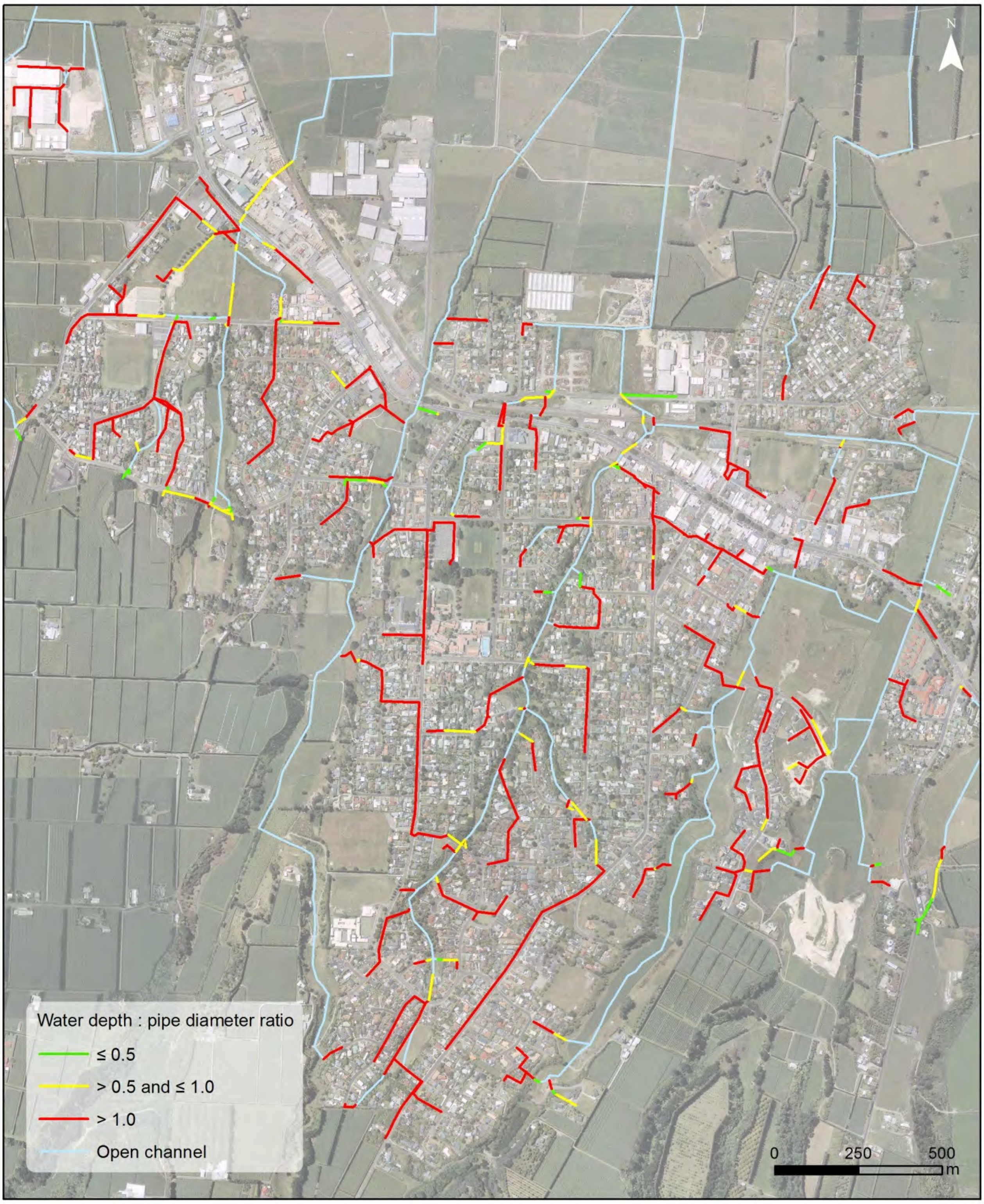


Figure D.4.1 Pipe filling – 100-year ARI, 10 minute duration

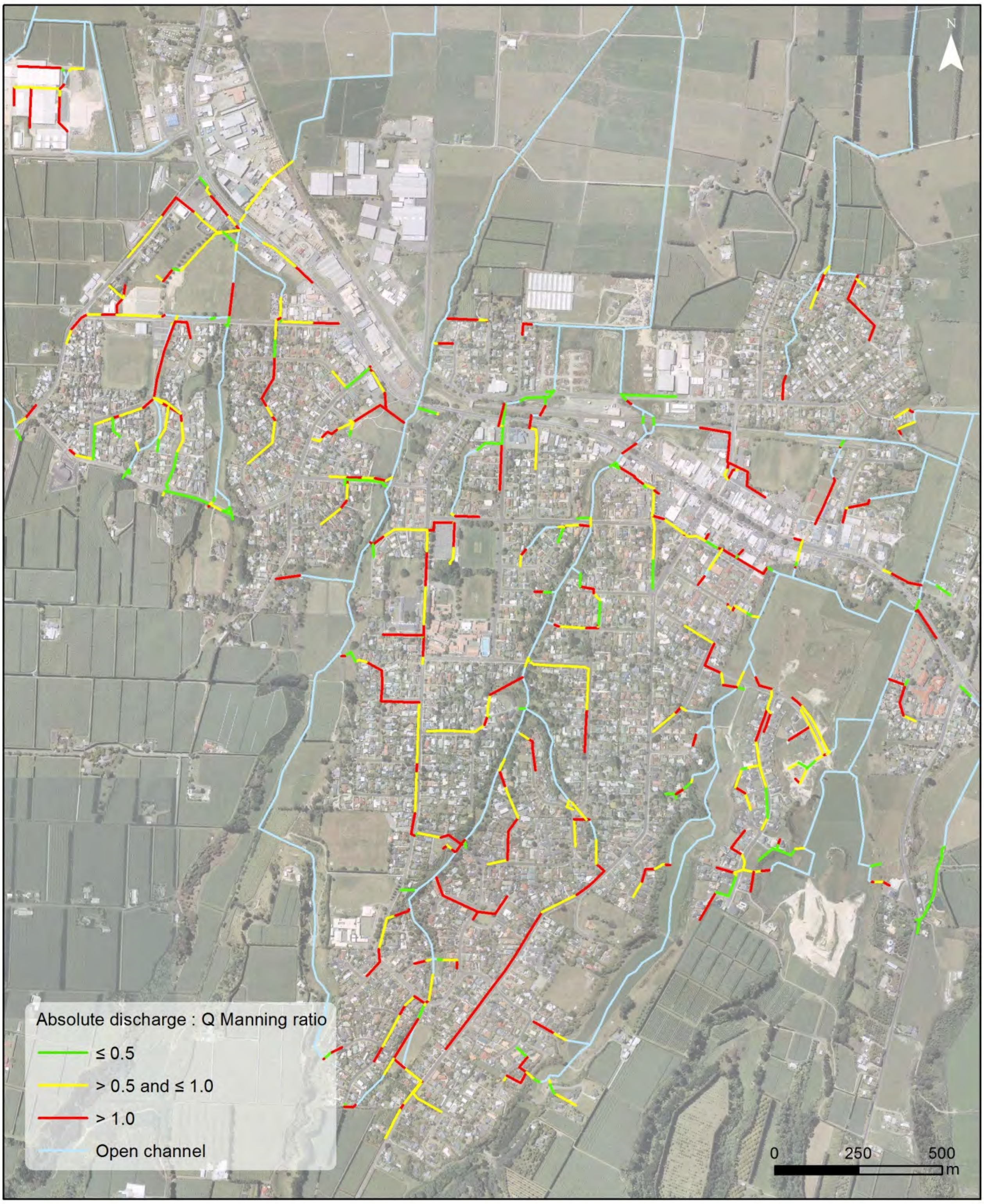


Figure D.4.2 Pipe capacity – 100-year ARI, 10 minute duration

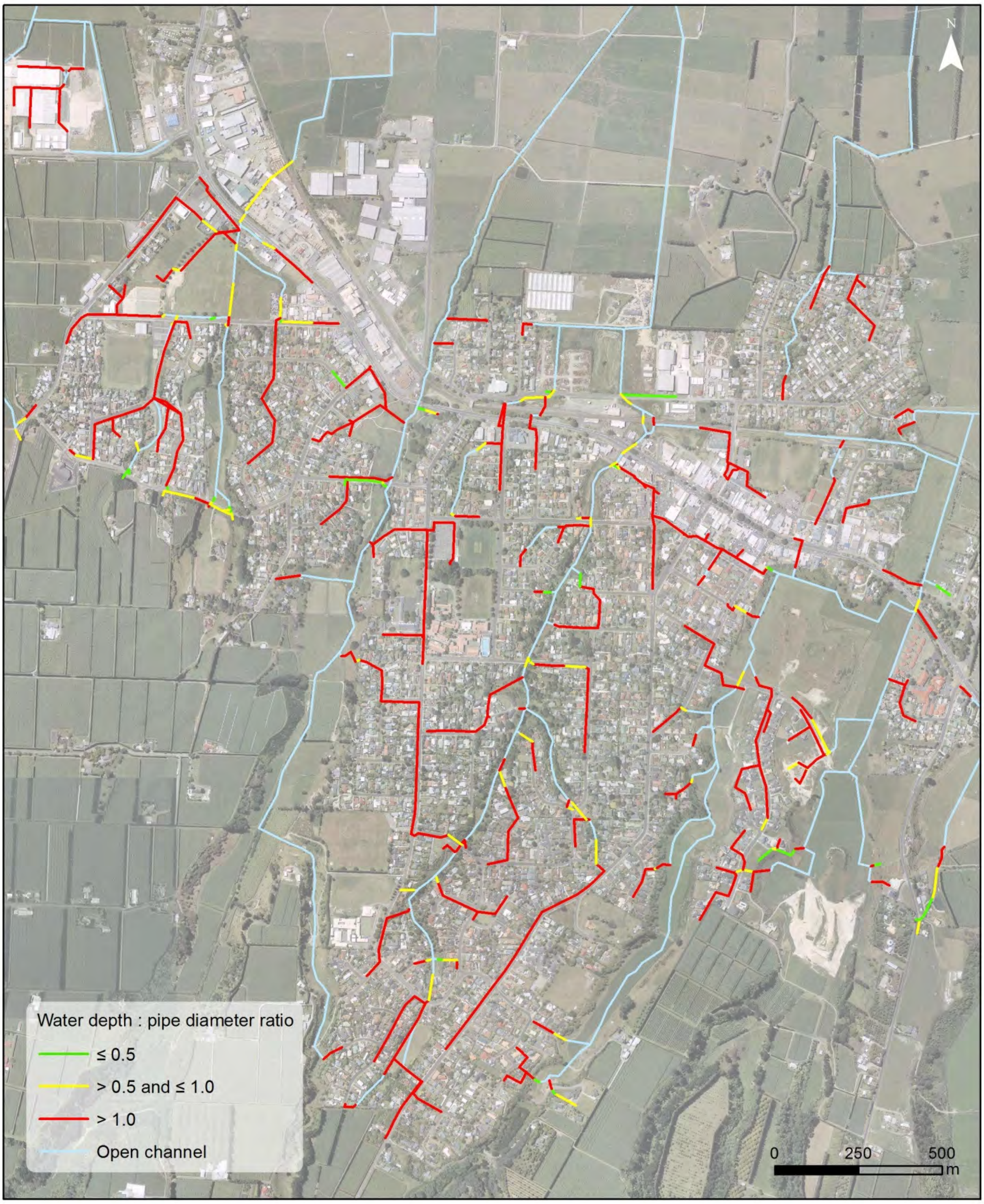


Figure D.4.3 Pipe filling – 100-year ARI, 30 minute duration

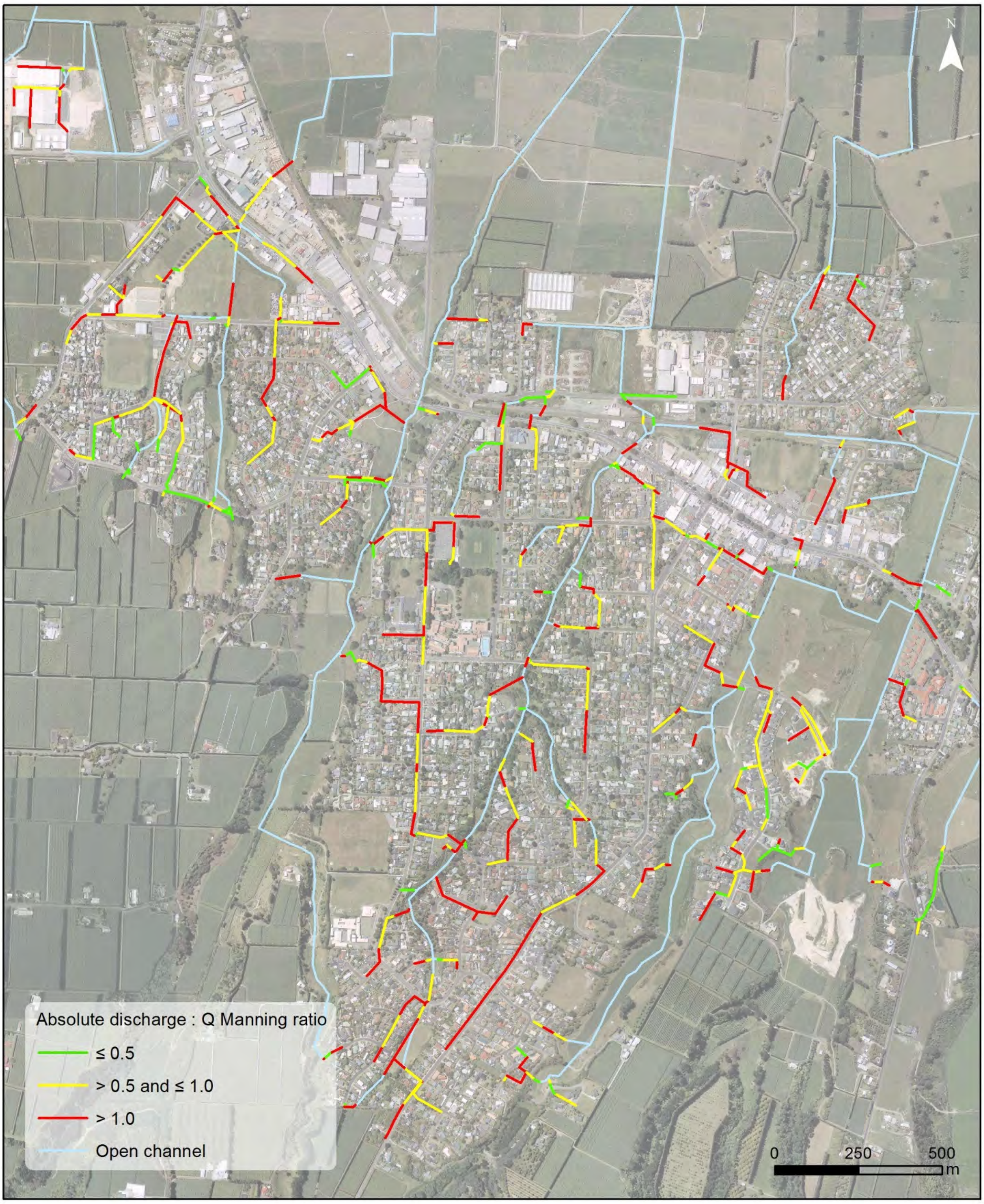


Figure D.4.4 Pipe capacity – 100-year ARI, 30 minute duration

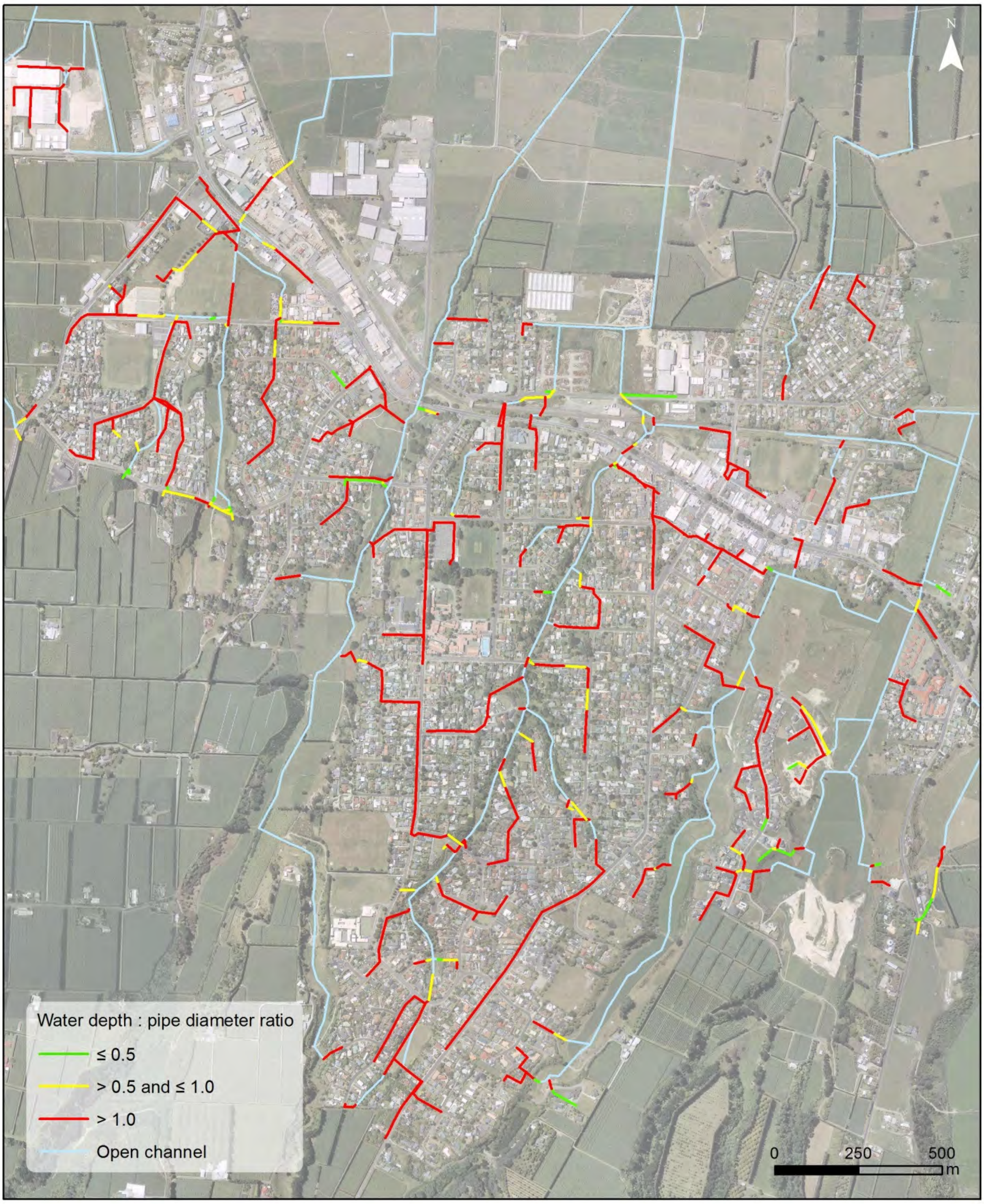


Figure D.4.5 Pipe filling – 100-year ARI, 1 hour duration

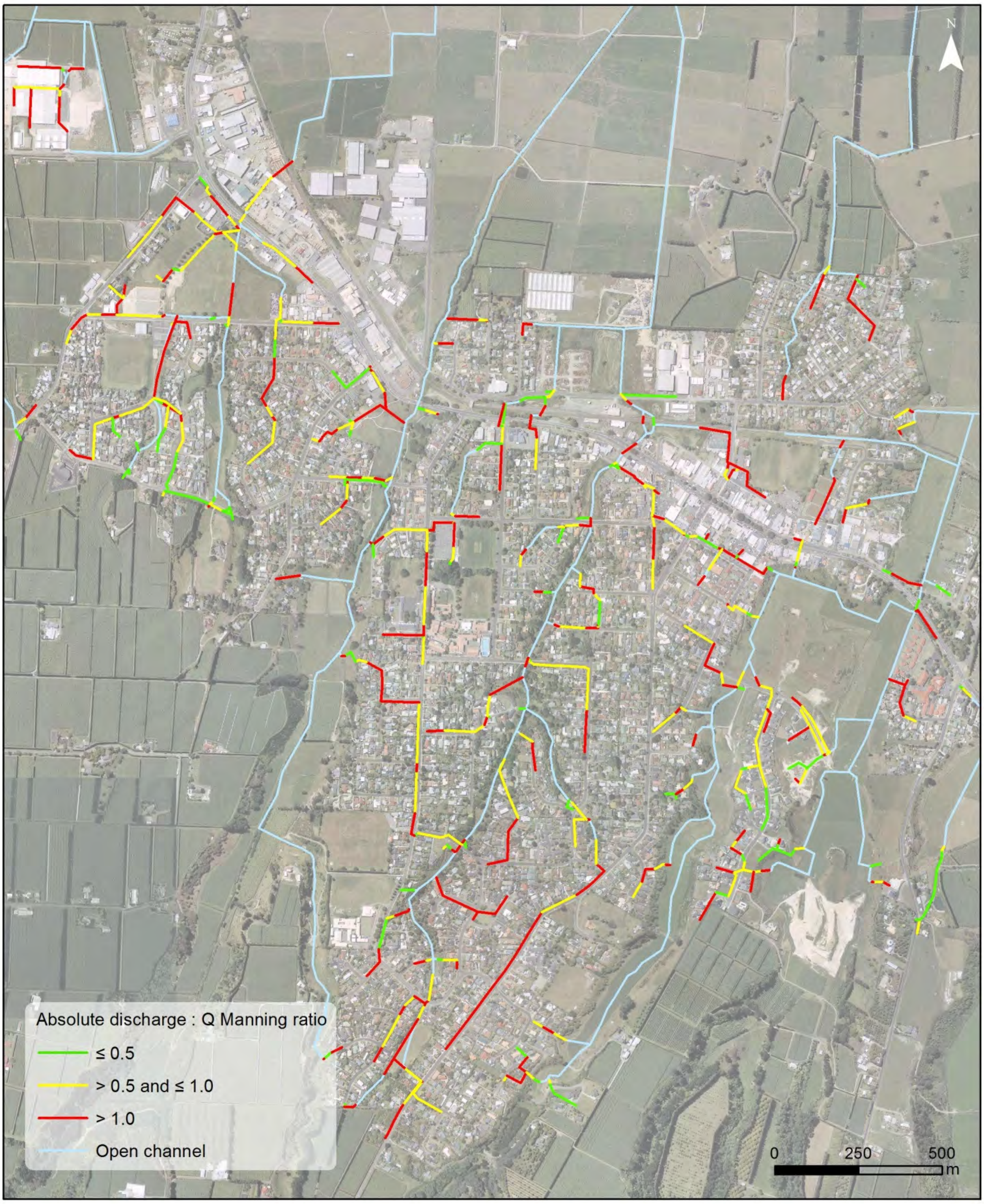


Figure D.4.6 Pipe capacity – 100-year ARI, 1 hour duration

D.5 Sensitivity Tests

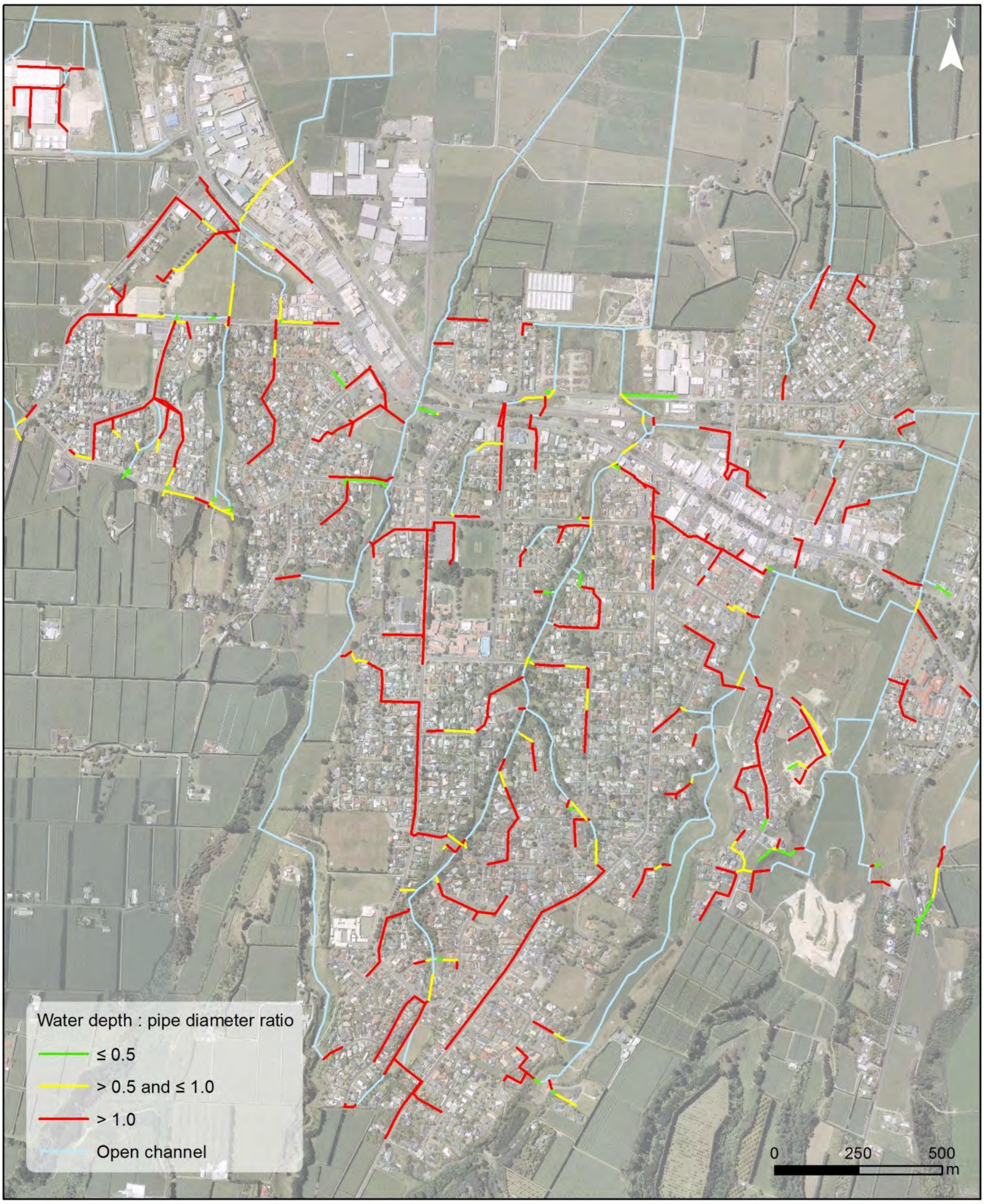


Figure D.5.1 Low Imperviousness, pipe filling – 50-year ARI, 30 minute duration

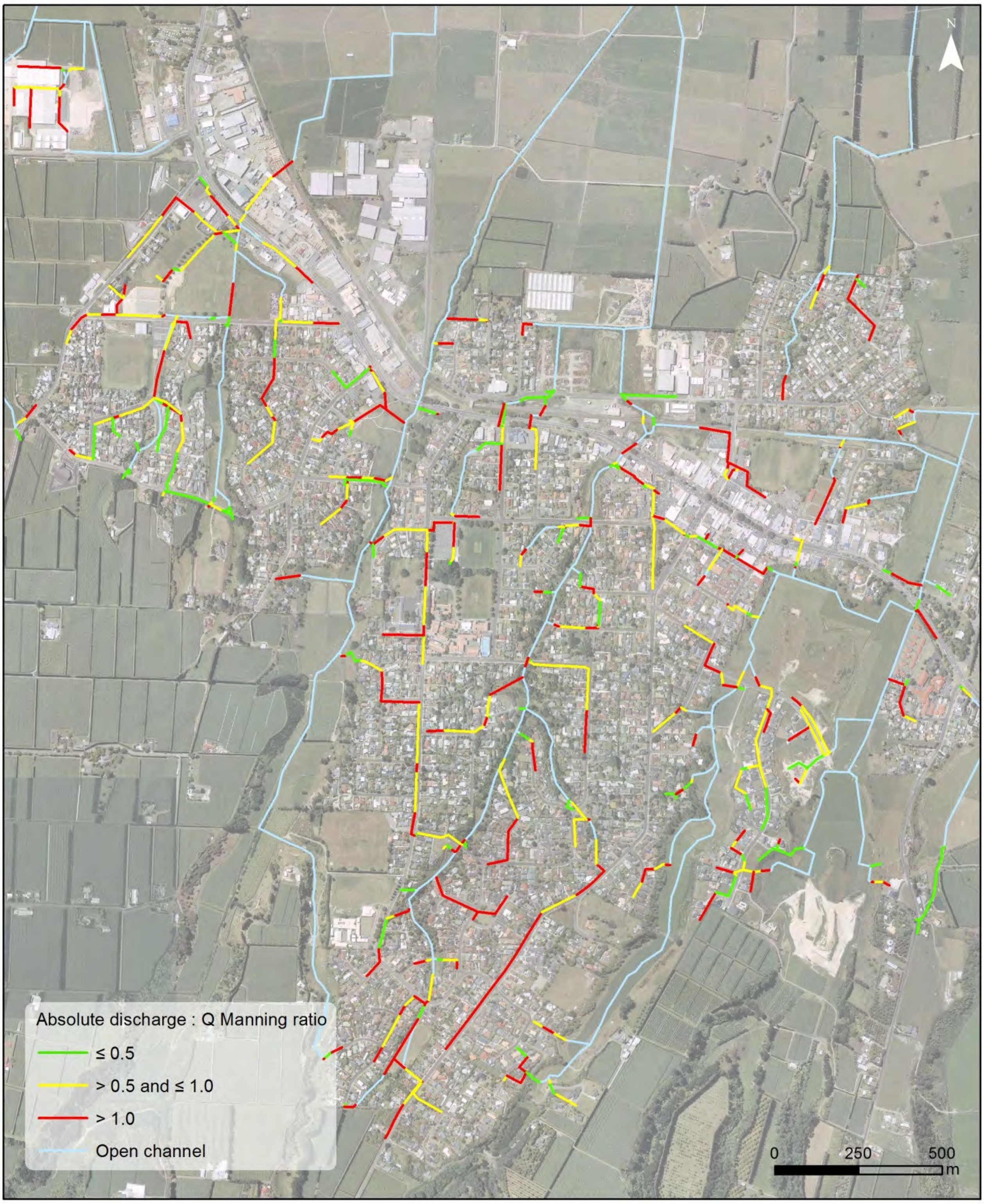


Figure D.5.2 Low Imperviousness, pipe capacity – 50-year ARI, 10 minute duration

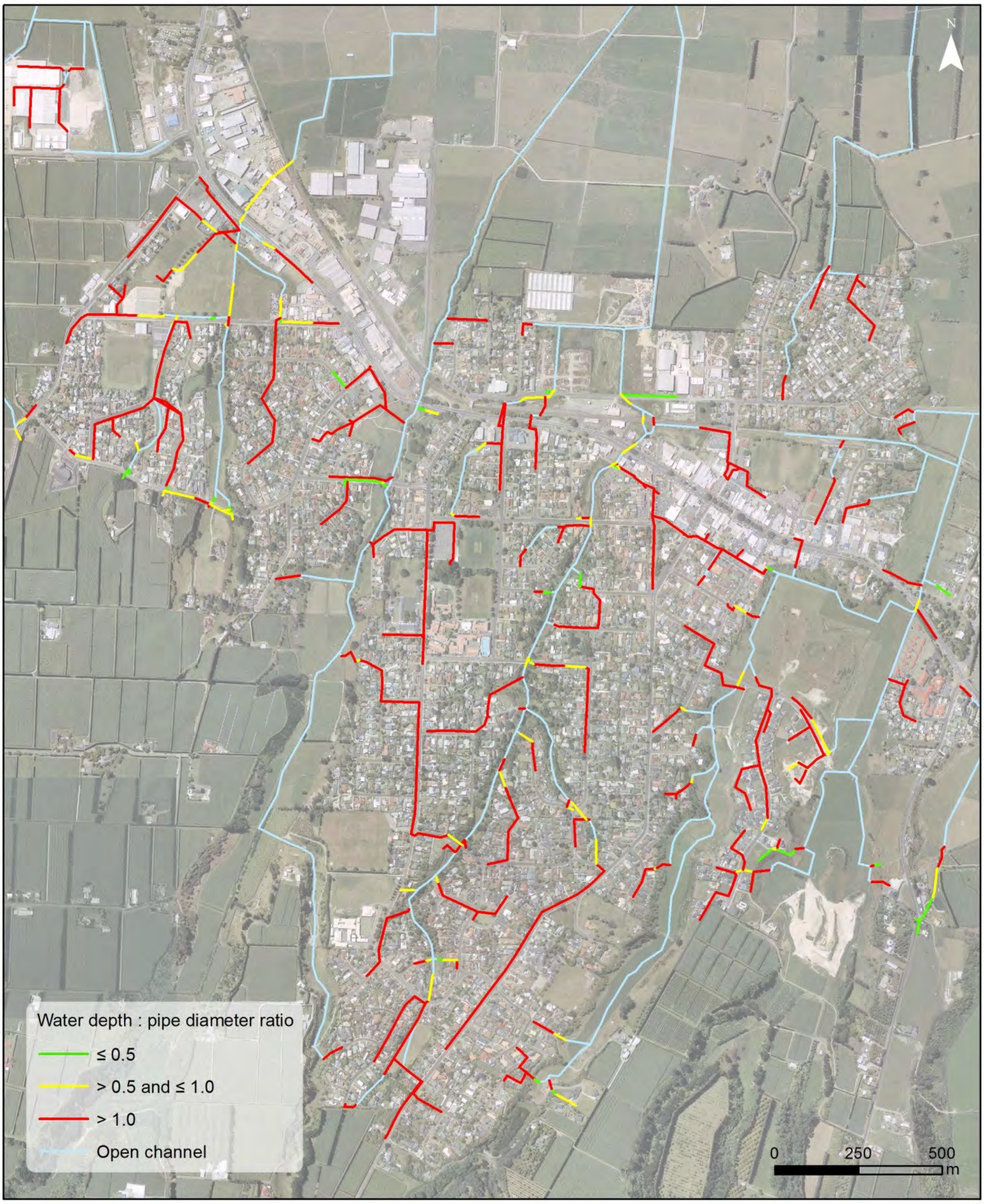


Figure D.5.3 High Imperviousness, pipe filling – 50-year ARI, 30 minute duration

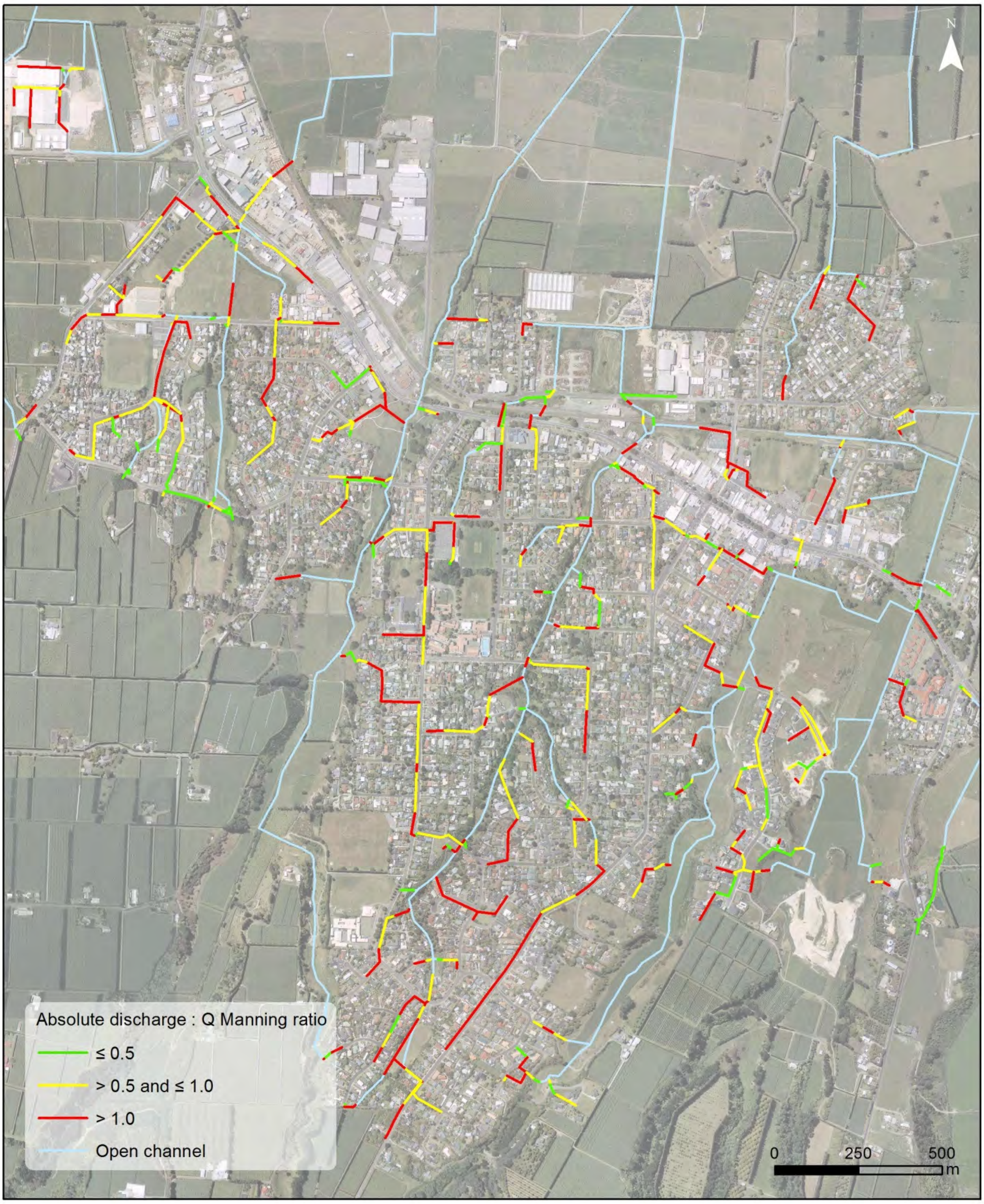


Figure D.5.4 High Imperviousness, pipe capacity – 50-year ARI, 10 minute duration

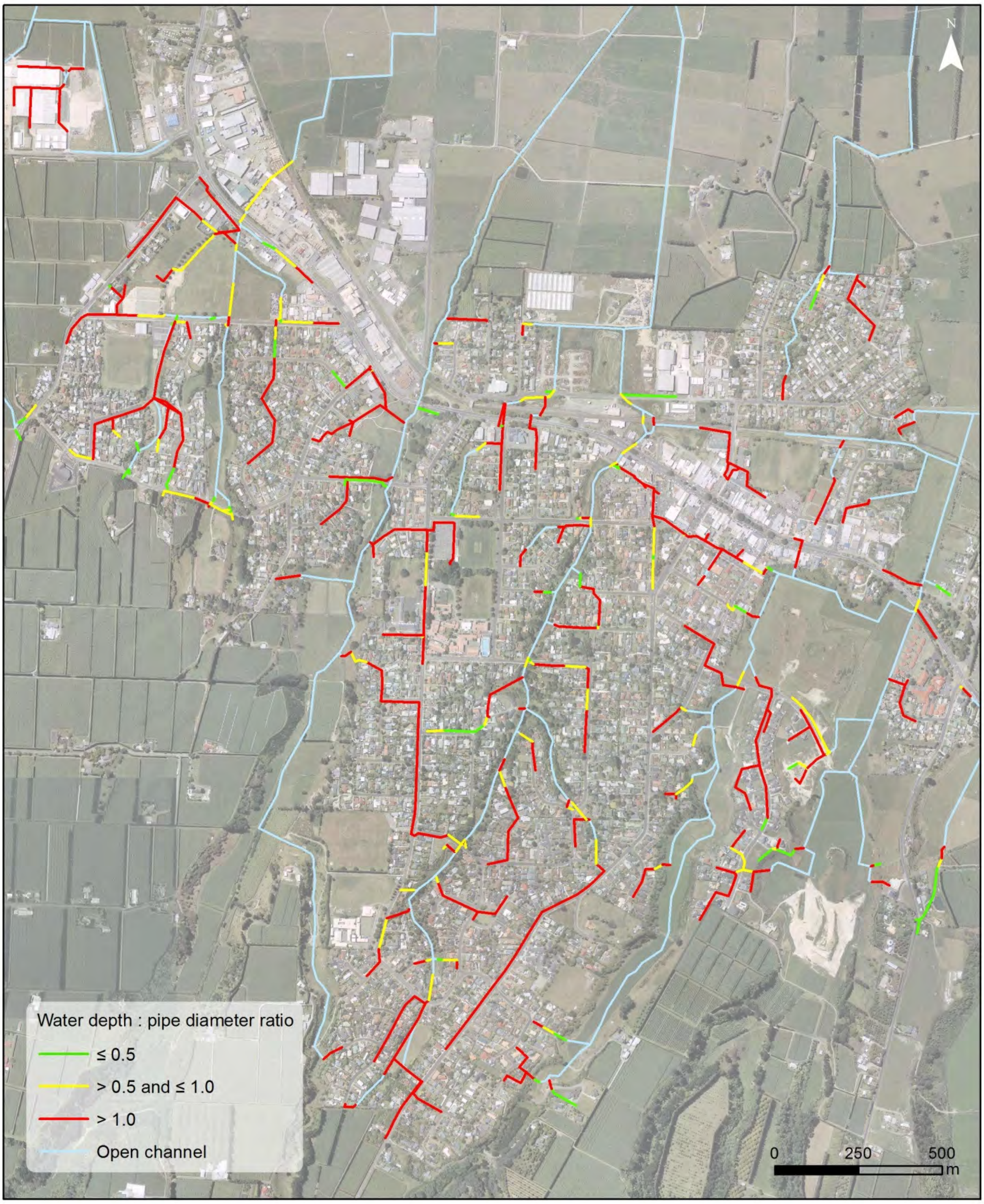


Figure D.5.5 Reduced Surface Flow Velocity, pipe filling – 50-year ARI, 30 minute duration

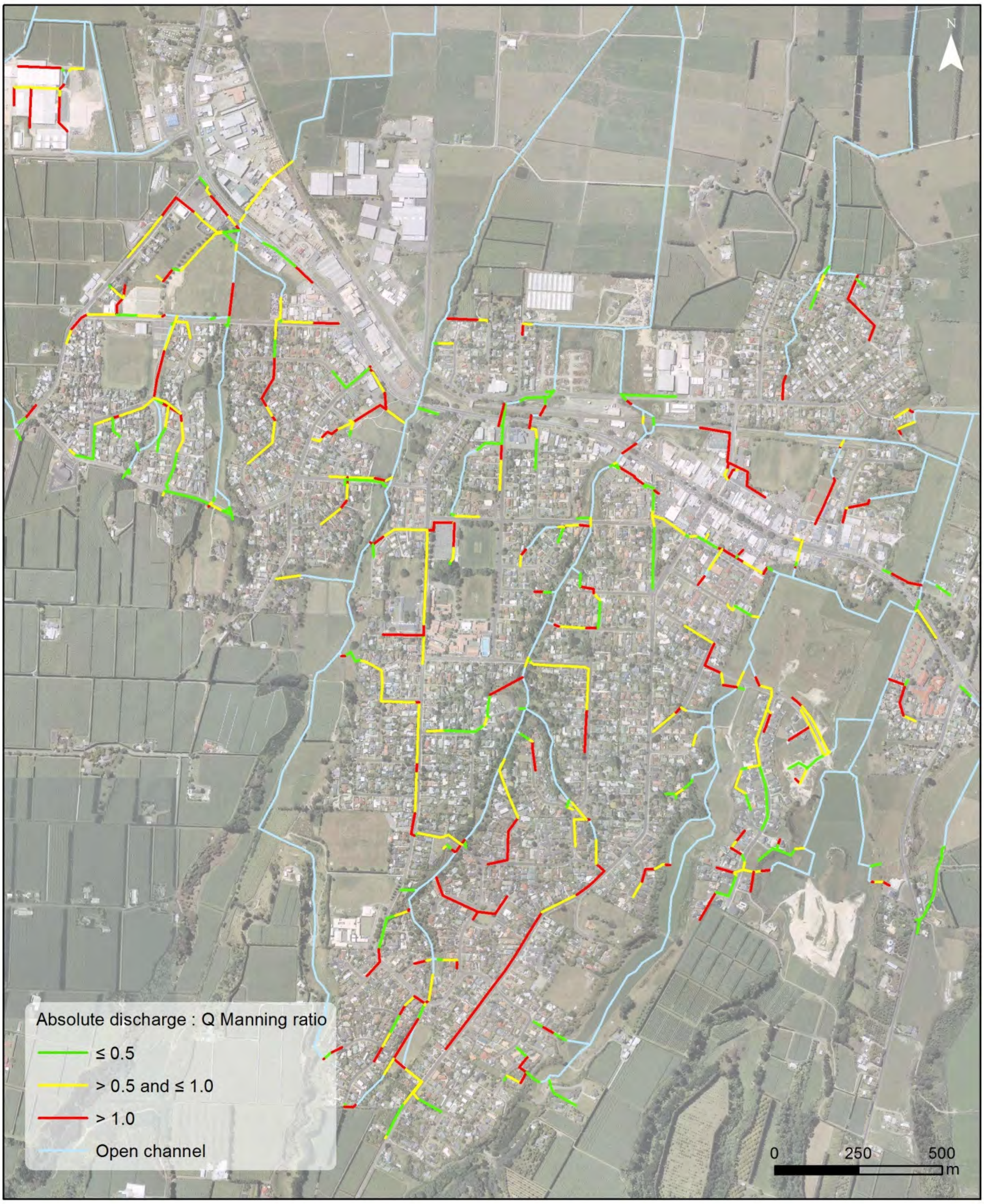


Figure D.5.6 Reduced Surface Flow Velocity, pipe capacity – 50-year ARI, 10 minute duration

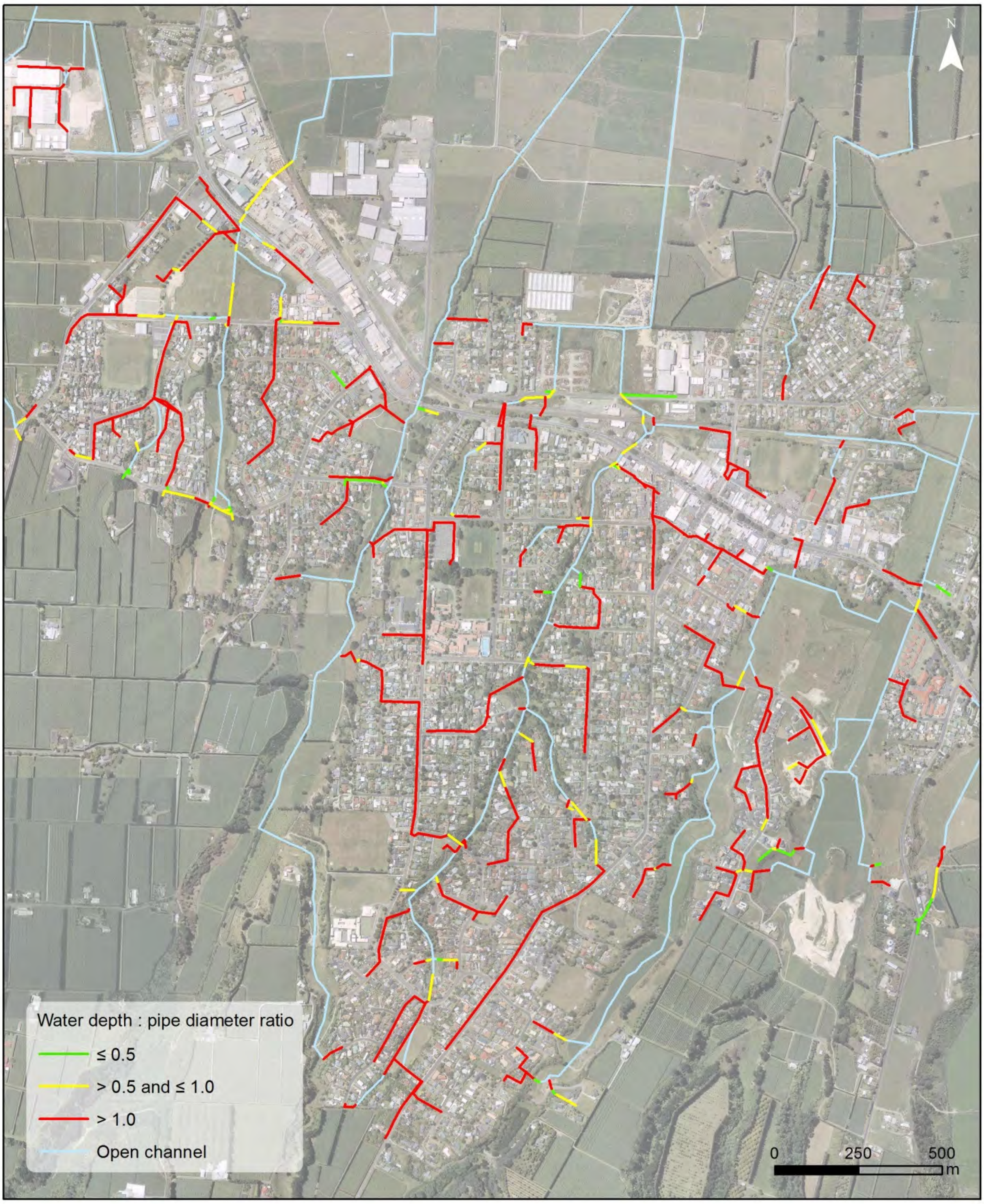


Figure D.5.7 Increased Hydrological Reduction, pipe filling – 50-year ARI, 30 minute duration

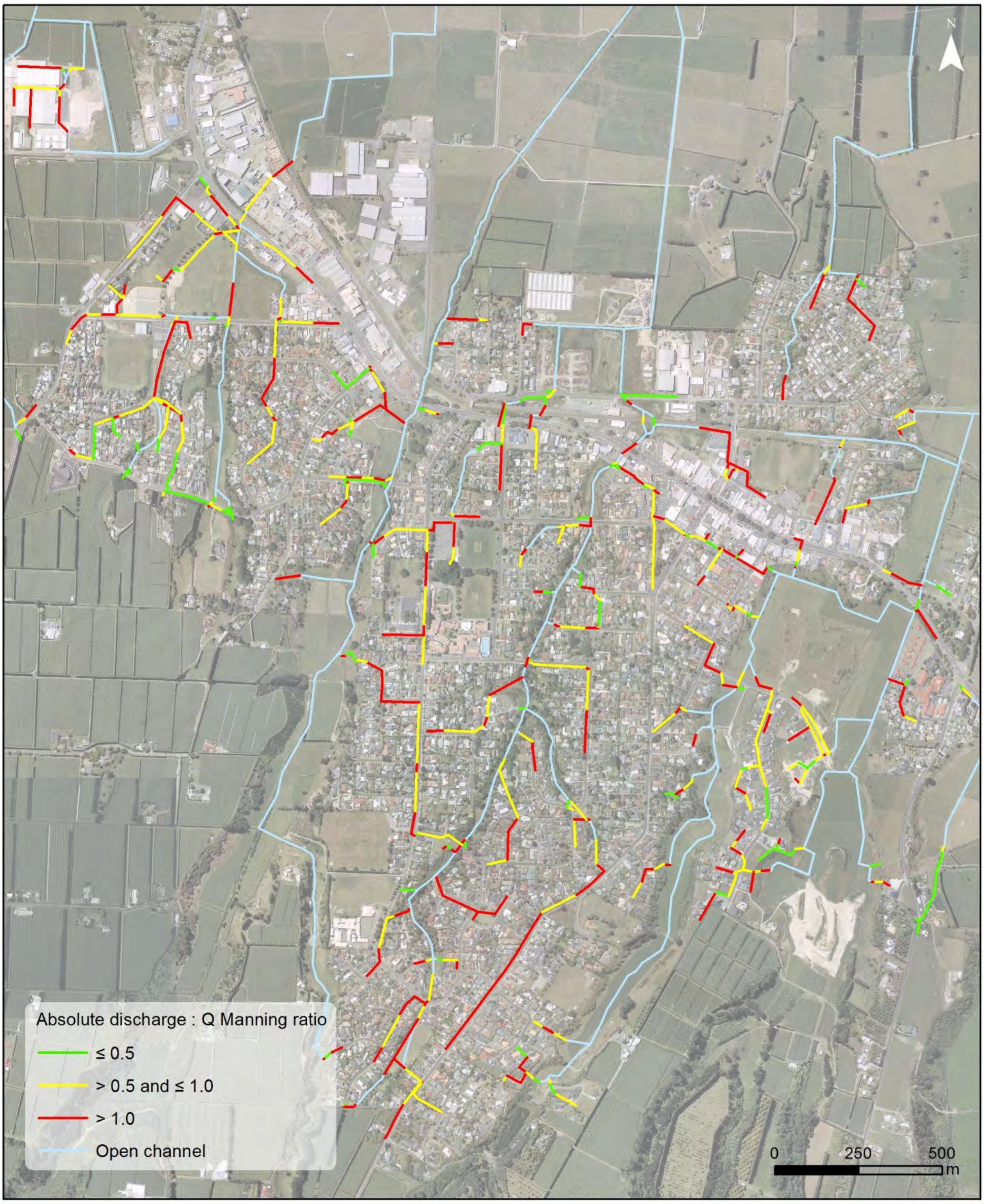


Figure D.5.8 Increased Hydrological Reduction, pipe capacity – 50-year ARI, 10 minute duration

Appendix E – Bottleneck Long Sections

Figure E.1 Location of bottlenecks
Figure E.2 Long-section of No3 Rd bottleneck with 50-year ARI, 30minute duration maximum HGL
Figure E.3 Long-section of Valley Rd bottleneck with 50-year ARI, 30minute duration maximum HGL
Figure E.4 Long-section of Boucher Ave bottleneck with 50-year ARI, 30minute duration maximum HGL
Figure E.5 Long-section of Commerce Lane & Jocelyn St bottleneck with 50-year ARI, 30minute duration maximum HGL
Figure E.6 Long-section of Princess St bottleneck with 50-year ARI, 30minute duration maximum HGL
Figure E.7 Long-section of Ben Keys St bottleneck with 50-year ARI, 30minute duration maximum HGL
Figure E.8 Long-section of Atuaroa Ave bottleneck with 50-year ARI, 30minute duration maximum HGL
Figure E.9 Long-section of Cameron Rd bottleneck with 50-year ARI, 30minute duration maximum HGL
Figure E.10 Long-section of Queen St bottleneck with 50-year ARI, 30minute duration maximum HGL
Figure E.11 Long-section of Slater Pl bottleneck with 50-year ARI, 30minute duration maximum HGL
Figure E.12 Long-section of Dunlop Rd bottleneck with 50-year ARI, 30minute duration maximum HGL

Figure E.1 Location of bottlenecks

Note: The identifiers on this plan are the same as the figure numbers on the following pages, for example, the reference "E6" in the above figure refers to Figure E.6.

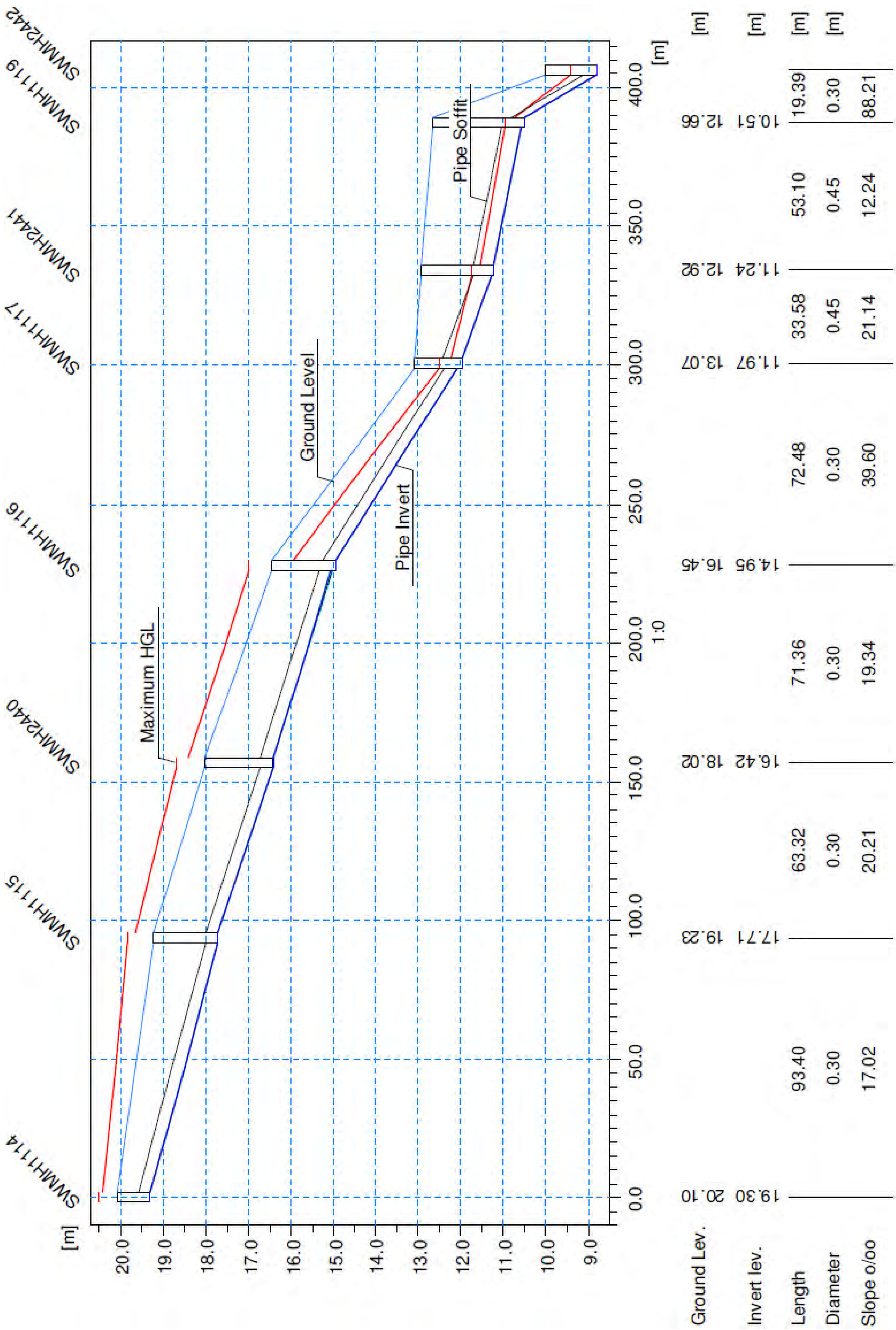


Figure E.2 Long-section of No3 Rd bottleneck with 50-year ARI, 30minute duration maximum HGL

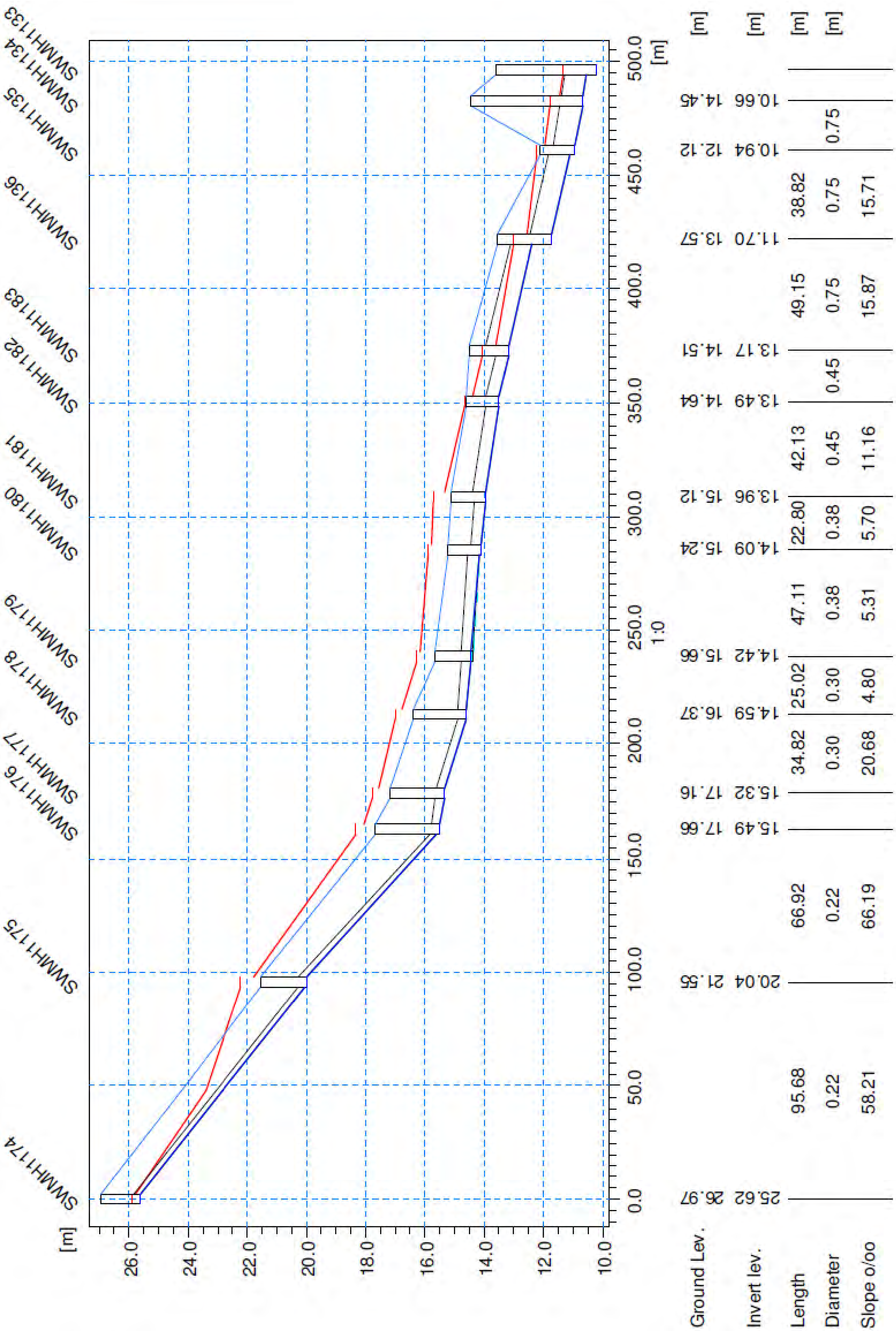


Figure E.3 Long-section of Valley Rd bottleneck with 50-year ARI, 30minute duration maximum HGL

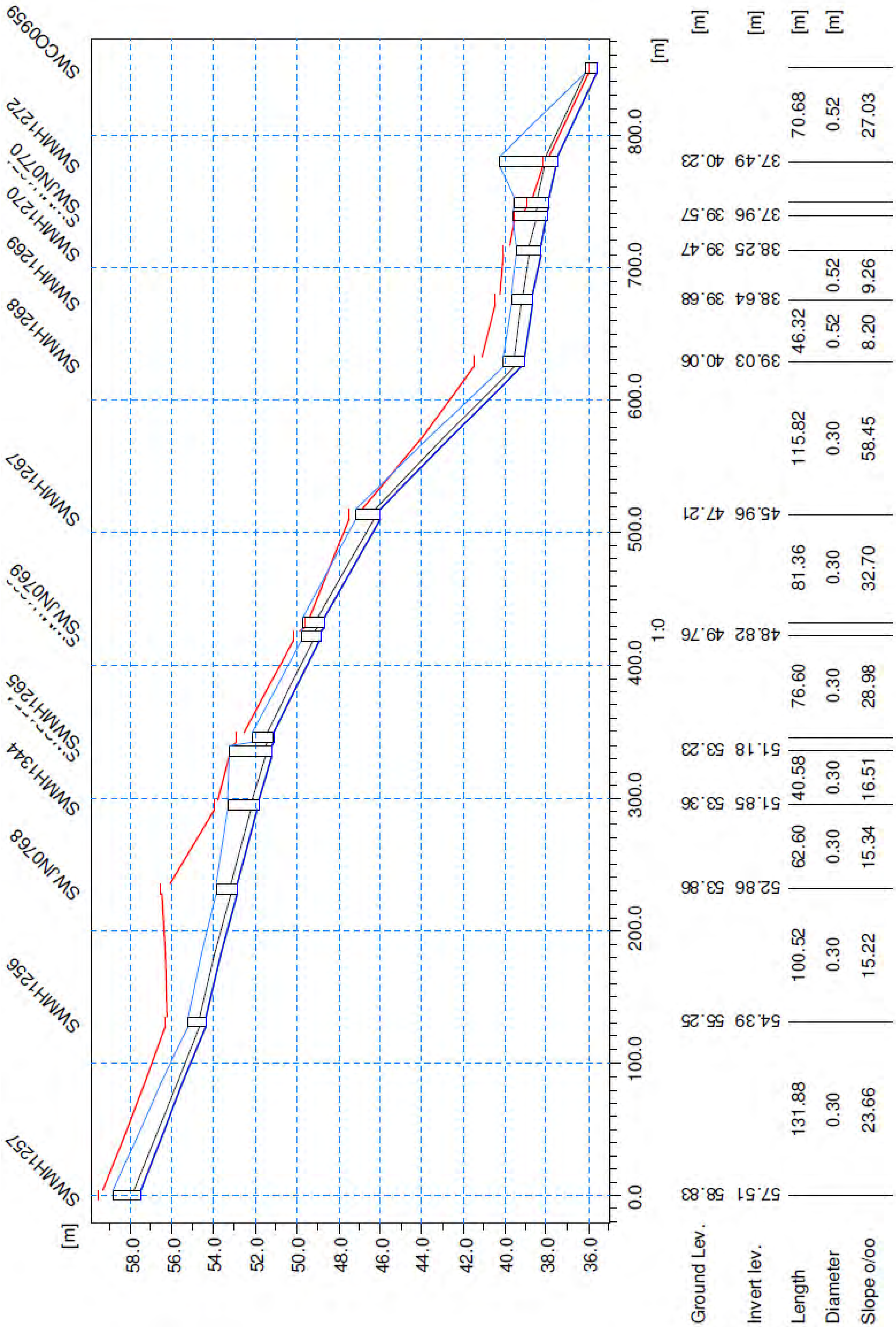


Figure E.4 Long-section of Boucher Ave bottleneck with 50-year ARI, 30minute duration maximum HGL

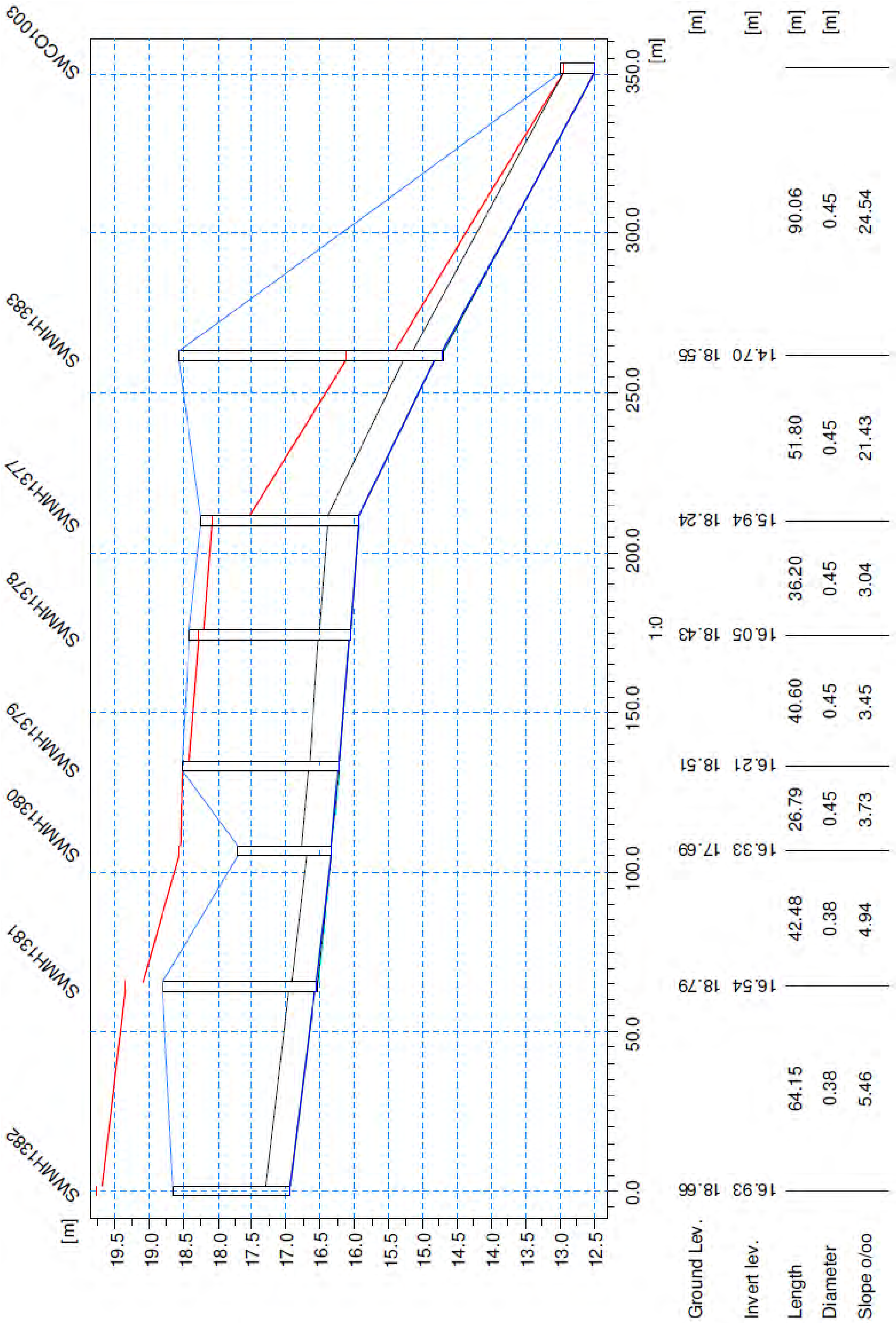


Figure E.5 Long-section of Commerce Lane & Jocelyn St bottleneck with 50-year ARI, 30minute duration maximum HGL

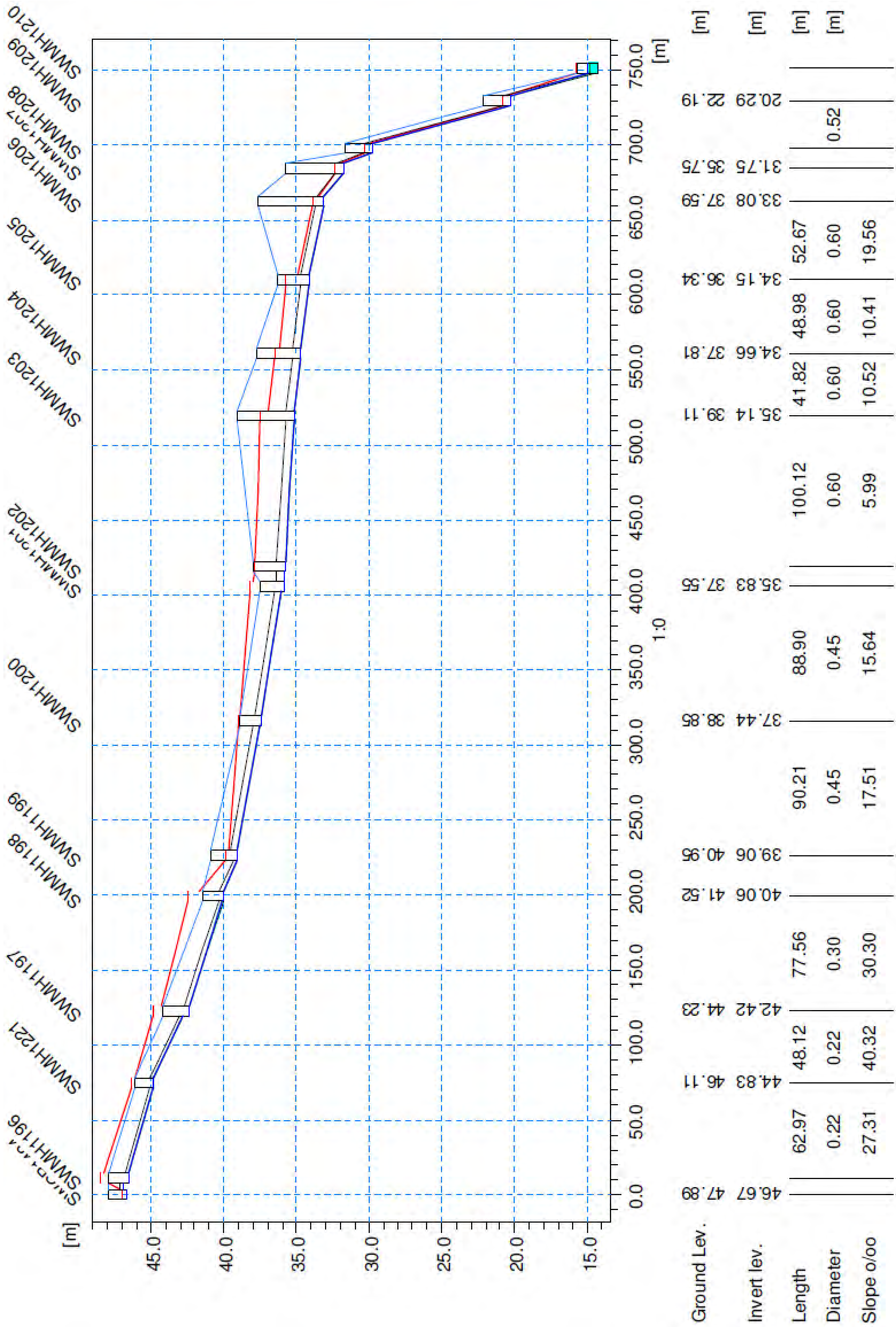


Figure E.6 Long-section of Princess St bottleneck with 50-year ARI, 30minute duration maximum HGL

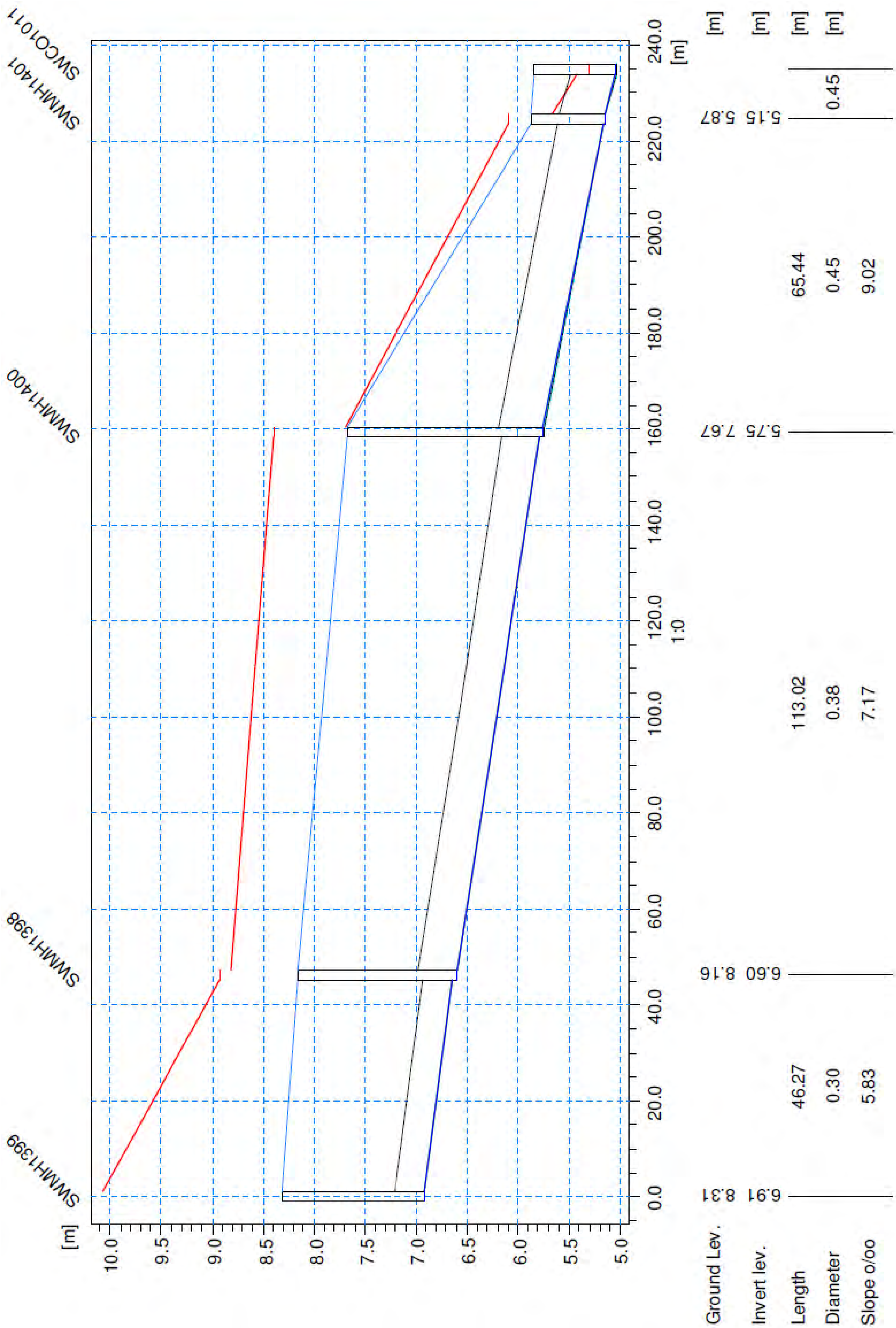


Figure E.7 Long-section of Ben Keys St bottleneck with 50-year ARI, 30minute duration maximum HGL

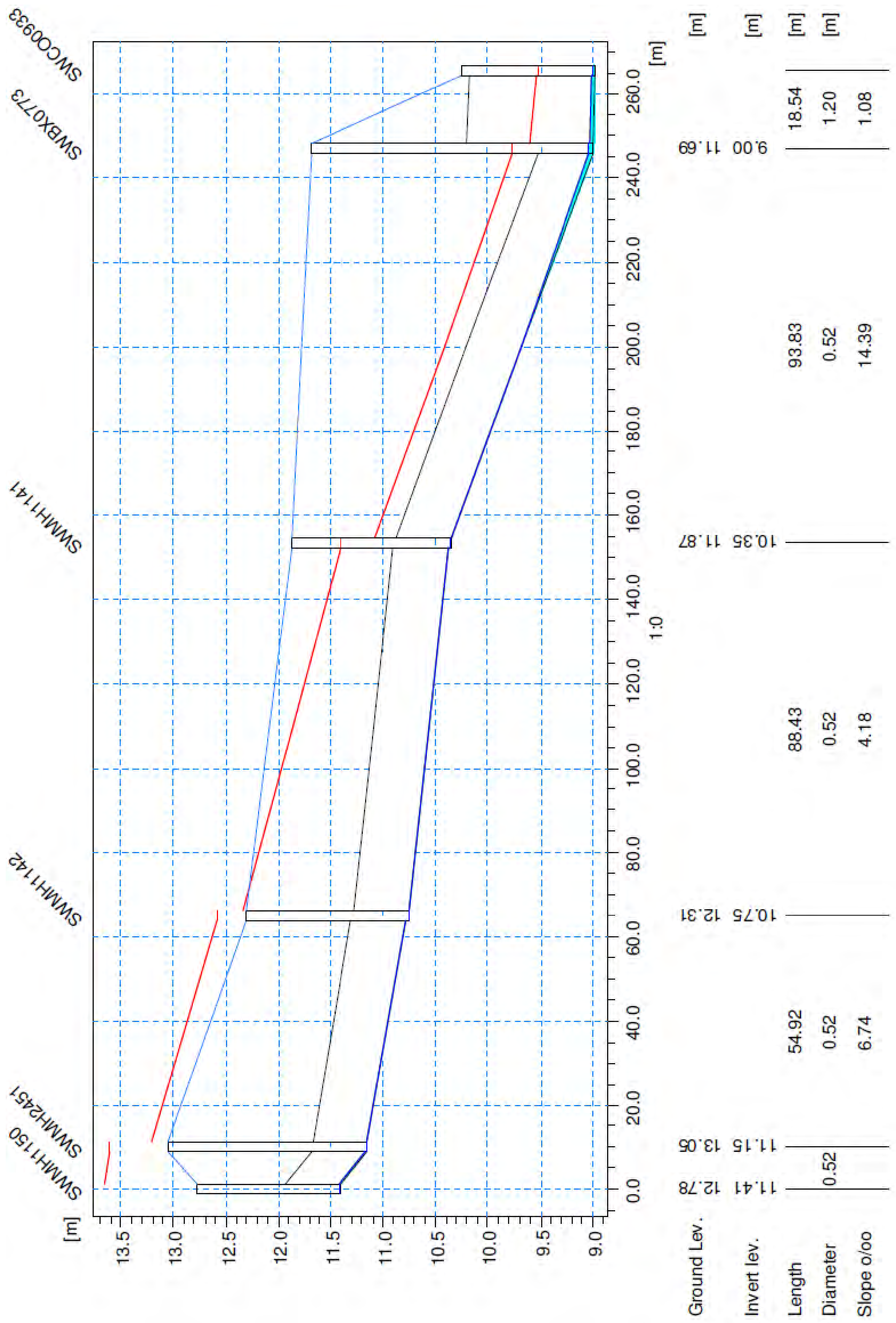


Figure E.8 Long-section of Atuaroa Ave bottleneck with 50-year ARI, 30minute duration maximum HGL

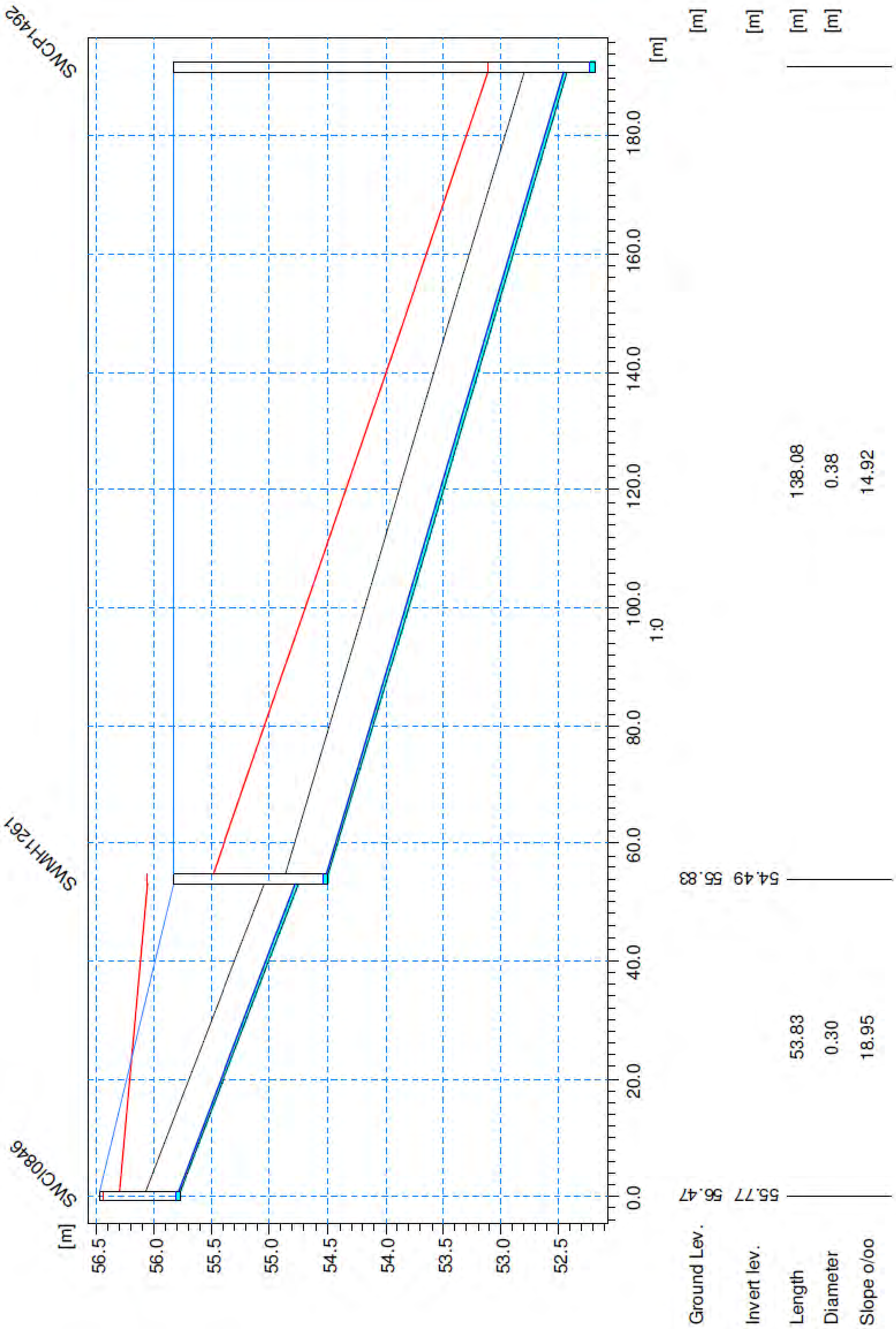


Figure E.9 Long-section of Cameron Rd bottleneck with 50-year ARI, 30minute duration maximum HGL

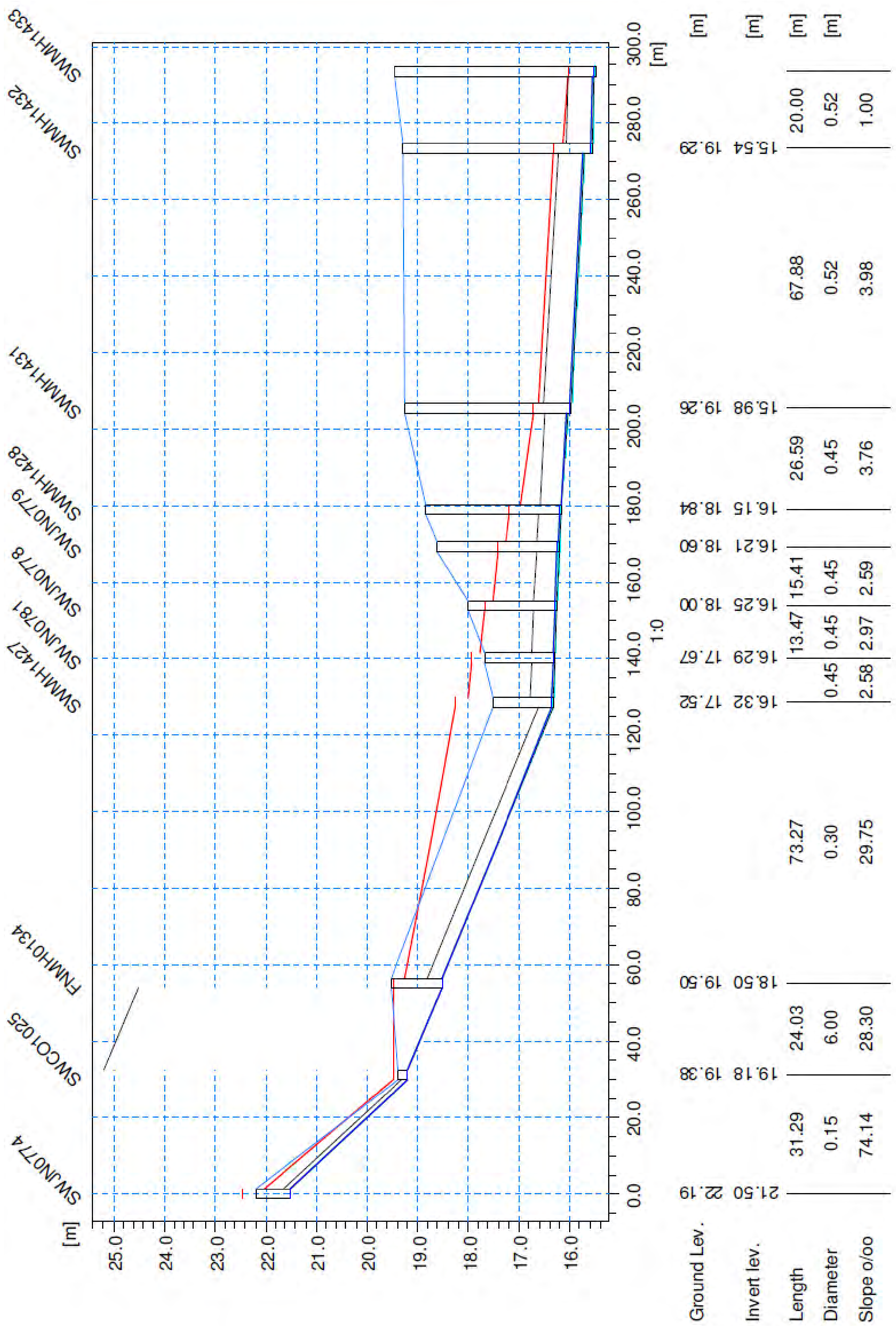


Figure E.10 Long-section of Queen St bottleneck with 50-year ARI, 30minute duration maximum HGL

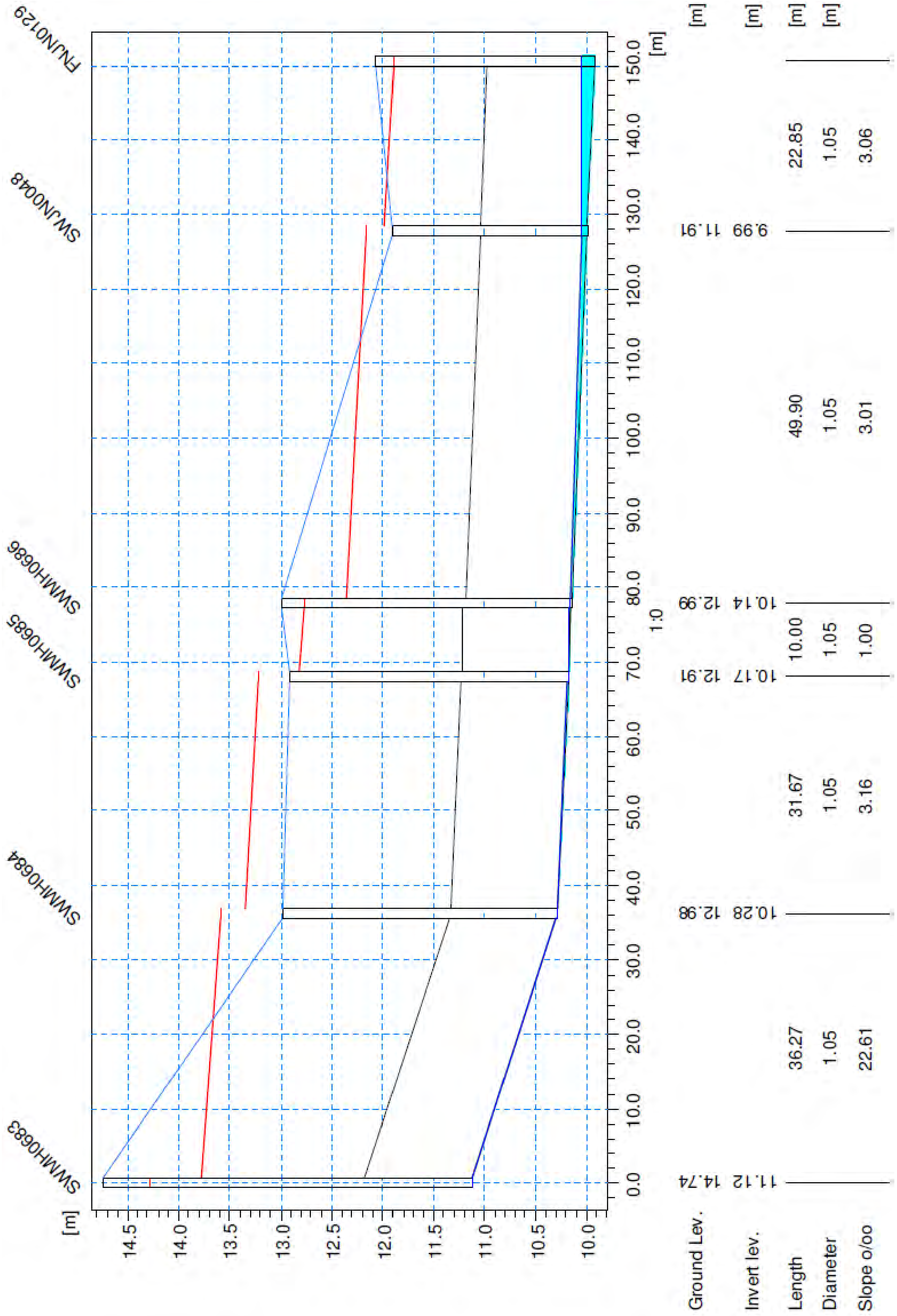


Figure E.11 Long-section of Slater Pl bottleneck with 50-year ARI, 30minute duration maximum HGL

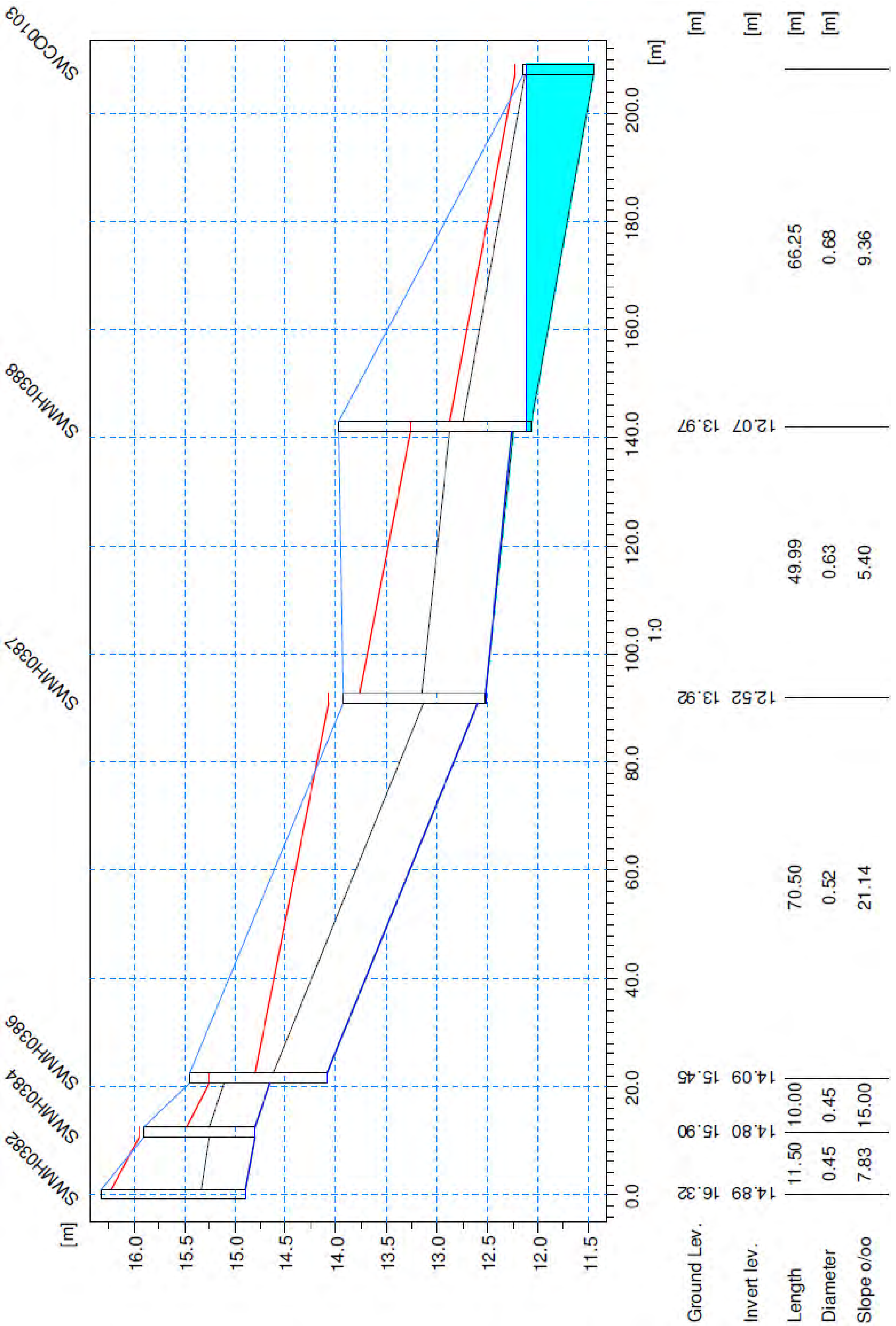


Figure E.12 Long-section of Dunlop Rd bottleneck with 50-year ARI, 30minute duration maximum HGL

<u>Note:</u> The other branch of this bottleneck is shown on the following page

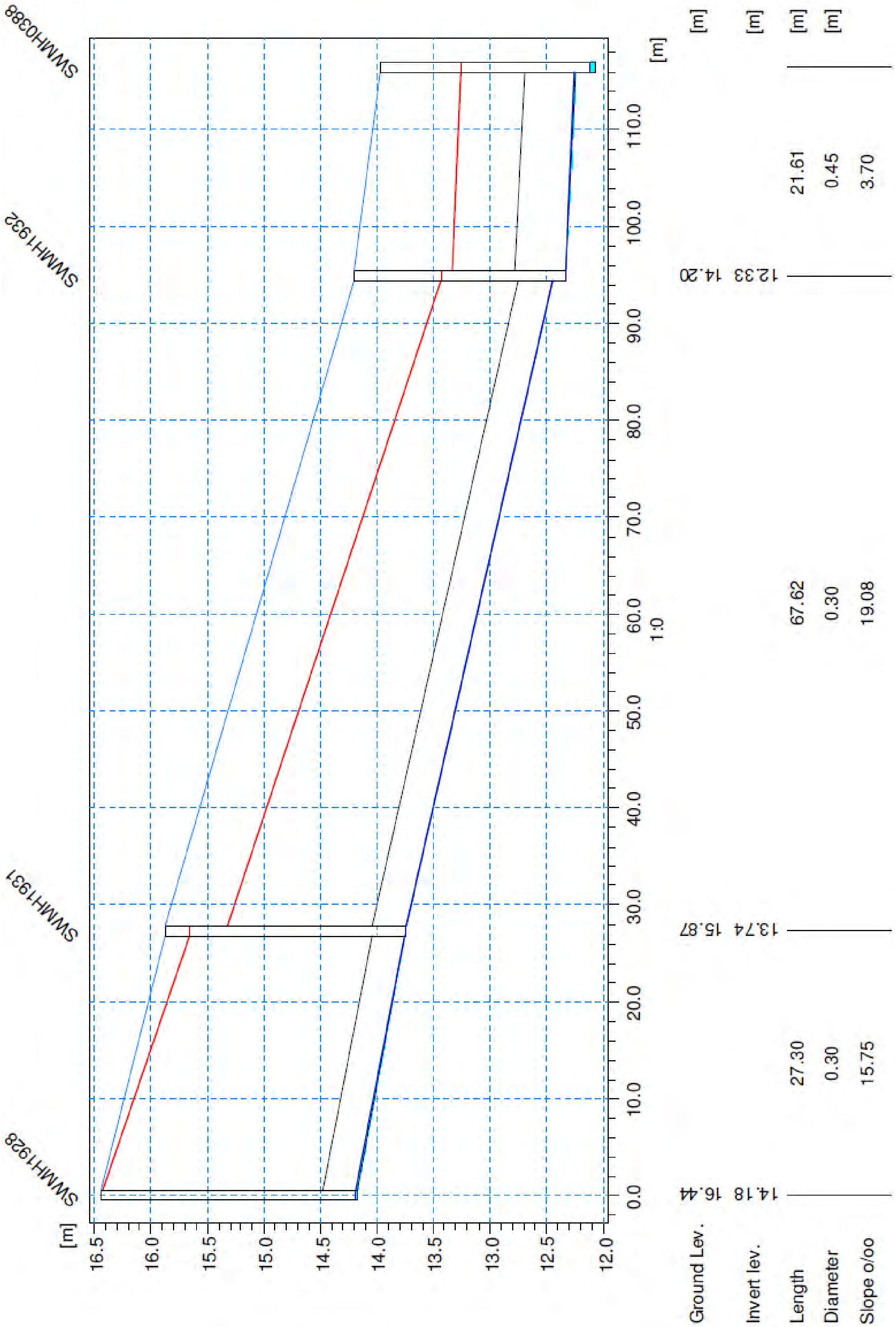


Figure E.12 Long-section of Dunlop Rd bottleneck with 50-year ARI, 30minute duration maximum HGL

Note: The other branch of this bottleneck is shown on the preceding page

Appendix F – Model Log Sheet

Scenario	Boundary conditions			Initial conditions
	Rainfall (Time series)	Water levels (Constant)	Inflows (Constant)	
Hot start				
Hot start	-	See Figure A.8	See Figure A.8	Dry model
Hot start 2.33 y	-	See Figure A.8	See Figure A.8	Dry model
Rainfall events				
5 year 10 min	Intensity 10-min 5y rainfall.dfs0	See Figure A.8	See Figure A.8	Hotstart 2.33yShort duration ARIs.PRF
5 year 30 min	Intensity 30-min 5y rainfall.dfs0	See Figure A.8	See Figure A.8	Hotstart 2.33yShort duration ARIs.PRF
5 year 1 hr	Intensity 1-hr 5y rainfall.dfs0	See Figure A.8	See Figure A.8	Hotstart 2.33yShort duration ARIs.PRF
10 year 10 min	Intensity 10-min 10y rainfall.dfs0	See Figure A.8	See Figure A.8	Hotstart 2.33yShort duration ARIs.PRF
10 year 30 min	Intensity 30-min 10y rainfall.dfs0	See Figure A.8	See Figure A.8	Hotstart 2.33yShort duration ARIs.PRF
10 year 1 hr	Intensity 1-hr 10y rainfall.dfs0	See Figure A.8	See Figure A.8	Hotstart 2.33yShort duration ARIs.PRF
50 year 10 min	Intensity 10-min 50y rainfall.dfs0	See Figure A.8	See Figure A.8	HotstartHotstart.PRF
50 year 30 min	Intensity 30-min 50y rainfall.dfs0	See Figure A.8	See Figure A.8	HotstartHotstart.PRF
50 year 1 hr	Intensity 1-hr 50y rainfall.dfs0	See Figure A.8	See Figure A.8	HotstartHotstart.PRF
100 year 10 min	Intensity 10-min 100y rainfall.dfs0	See Figure A.8	See Figure A.8	HotstartHotstart.PRF
100 year 30 min	Intensity 30-min 100y rainfall.dfs0	See Figure A.8	See Figure A.8	HotstartHotstart.PRF
100 year 1 hr	Intensity 1-hr 100y rainfall.dfs0	See Figure A.8	See Figure A.8	HotstartHotstart.PRF
Sensitivity tests				
Low imperviousness	Intensity 30-min 50y rainfall.dfs0	See Figure A.8	See Figure A.8	HotstartHotstart.PRF
High imperviousness	Intensity 30-min 50y rainfall.dfs0	See Figure A.8	See Figure A.8	HotstartHotstart.PRF
Reduced SFV	Intensity 30-min 50y rainfall.dfs0	See Figure A.8	See Figure A.8	HotstartHotstart.PRF
Increased HR	Intensity 30-min 50y rainfall.dfs0	See Figure A.8	See Figure A.8	HotstartHotstart.PRF

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