Black Creek Stormwater

Model Build

PREPARED FOR WELLINGTON WATER | December 2022

We design with community in mind

Revision Schedule

Rev No	Date	Description	Signature of Typed Name (documentation on file)			
			Prepared by	**Checked by**	**Reviewed by**	**Approved by**
1	31/03/2015	Model Build	AO & KS			
2	18/03/2015	Model Build and Validation	AO & KS			
3	08/05/2015	Model Build, Validation and system performance	AO & KS			
4	10/06/2019	Sensitivity and free board assessment	AB	AO	TK	TK
5	15/12/2021	Model report combination	LC	AS	CH	
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Executive Summary

MWH, now Stantec (Stantec) was engaged by Wellington Water Ltd to develop a comprehensive model of the stormwater network for Black Creek in the suburb of Wainuiomata, in Hutt City. The objectives were to develop a model that included all the known storm water assets so that flooding hazards could be identified.

The model was built in Info works Integrated Catchment Modelling (ICM) prior to the Wellington Modelling Specifications thus shows some differences to the standard approach. Once the model was constructed, the model was validated against several rainfall events between 2010 to 2012 The validation process showed that the model results largely agreed with the previously recorded flooding issues with a majority of the flooding restricted to parks, roads and fields.

A sensitivity study was also completed, testing the importance of various aspects of the model such as tailwater conditions, rainfall, and culvert blockages. The results showed that the rainfall and culvert blockages had a significant impact while tailwater conditions had minimal impacts. A freeboard assessment was then completed using updated gauge corrected rainfall radar data from 2019 and outputs from the sensitivity study.

System performance was also conducted using 20 scenarios looking at different aspects of the model, including scenarios looking at the impact of development in the northern catchment. The analysis showed that the primary driver for flooding was network capacity and that development did have an impact in the northern reaches of the model and minimal impacts further downstream.

As the model was built prior to the stormwater specifications and because there are some newer developments and assets that are not included in the model, It is recommended that if this model was to be used for the planning of future infrastructures projects, the model should be re assessed and asset data should be confirmed.

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1 Introduction

MWH, now Stantec (Stantec), was engaged by Wellington Water Ltd (WWL) in 2014 to develop a comprehensive model of the stormwater network for Black Creek in Wainuiomata. The resulting Model is an integrated open channel, floodplain, and stormwater pipe model developed using Innovyse's Integrated Catchment Modelling (ICM) software. As part of the model build process a validation and system performance was also conducted. Sensitivity and freeboard assessments for the Black Creek catchment were then completed in 2019. This report is a combination of all of these steps and supersedes the (MWH, 2015 & Stantec, 2019) reports. It is important to note that the Black Creek model was built prior to the Wellington Modelling specifications, and therefore there are known differences to the specifications. The model build methodology and assumptions used are discussed throughout the document and are known by WWL. Although this document combines each of the stage of the model construction (model build, validation, sensitivity, freeboard, and system performance), in one report, each stage is linked to a different version of the model network as the work was completed separately from each other. Descriptions of which model network version the text aligns with is outlined in the report.

1.1 Catchment Overview

Black creek is a large tributary of the Wainuiomata River and is the main waterway that services the urban area of Wainuiomata, a Lower Hutt Suburb sitting between the eastern Hutt hills and the Orongorongo Range, see Figure 1-1. The stormwater model covers the majority of the urban area of Wainuiomata and the surrounding hill slopes.

1.1.1 Topography

The Black Creek catchment is characterised by steep valley side surrounding a wide flat bottom valley, which includes the suburb of Wainuiomata as seen in Figure 1-2. The valley sides rise steeply to the ridgeline of 380- 400m. The valley floor is relatively flat and wide and gently slopes towards the confluence of Black Creek with the Wainuiomata River Much of this valley has been converted for urban development.

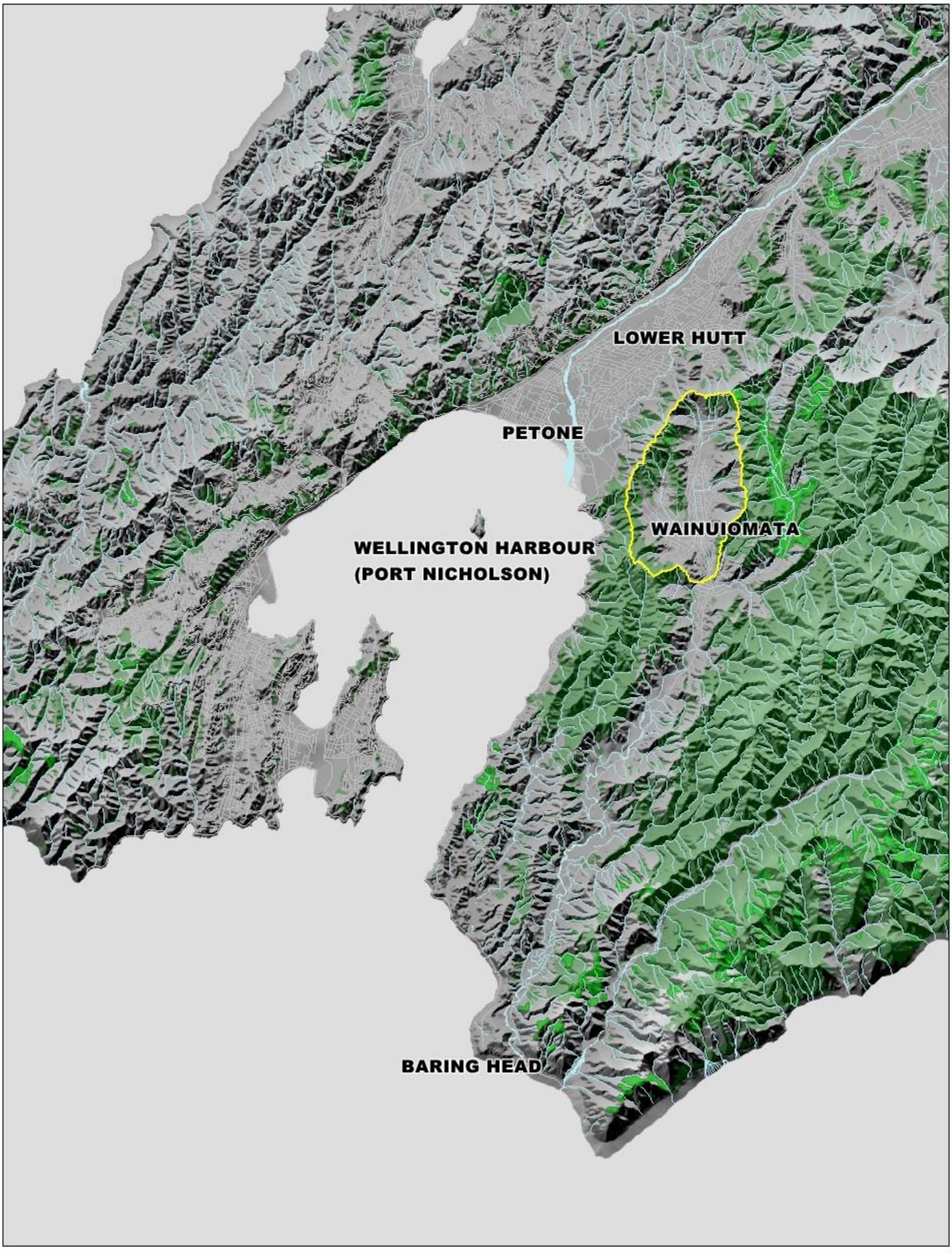


Figure 1-1: Catchment location map

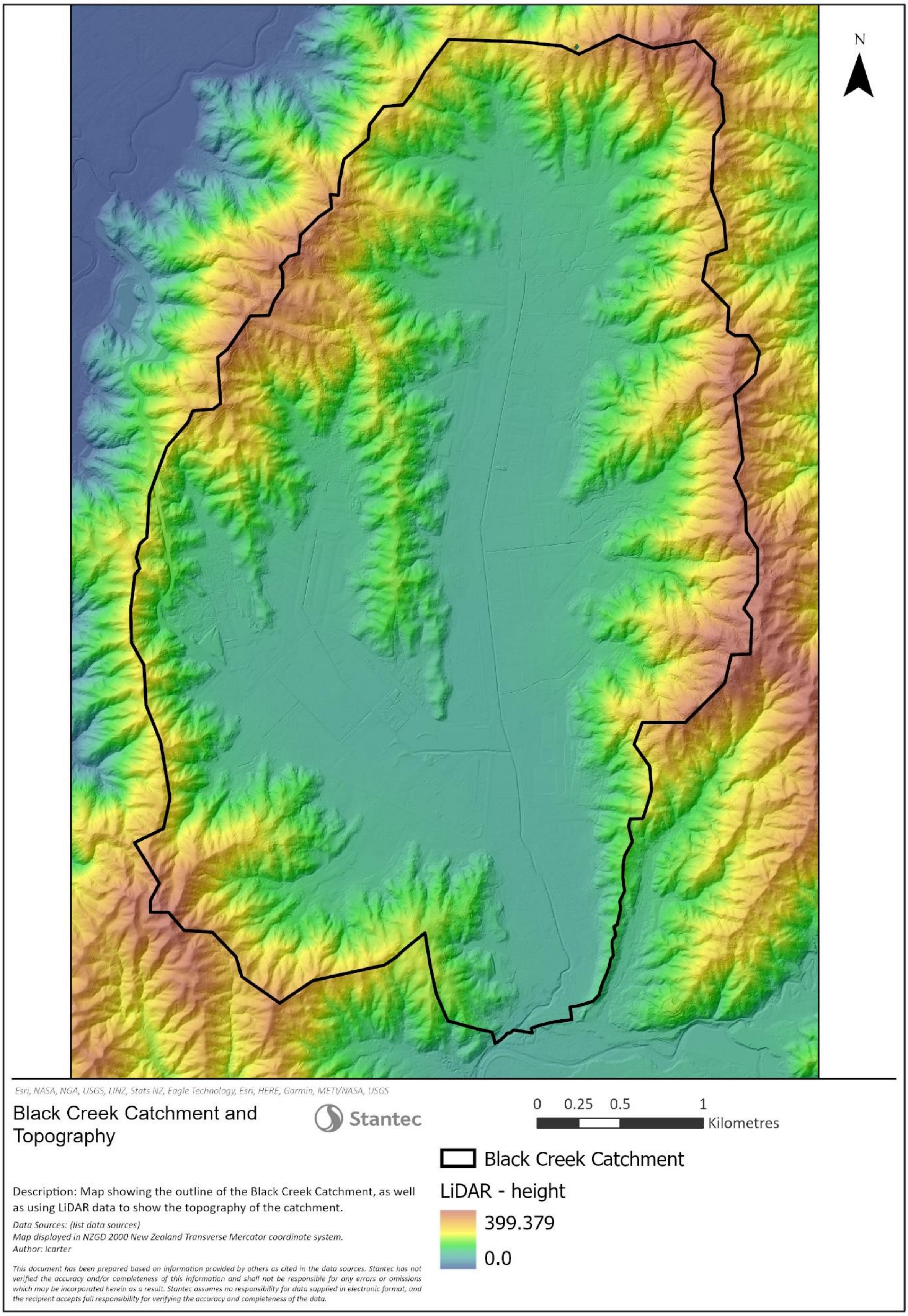


Figure 1-2: Black Creek catchment topography

1.1.2 Geology and Soils

The underlying geology for the catchment is primarily Quaternary sediments over laying Torlesse rock with lacustrine sediments left from the lake that previously resided in the valley (Begg, 1996). Between 50/80,0000 years to 20,000 years ago, the Black Creek Valley floor was covered in a lake, reflected in the stratigraphy with lacustrine sediments, with a depth of 30-5 m. These are then overlain with swamp sediments and alluvial gravels. When the Valley was developed, drainage of swampy lands was completed which explains the poor drainage in the Wainuiomata Valley.

The hill slopes are steepland soils from the Ruahine, Taita, Tawai and Rimutaka soil types, with characteristics explained int Table 1-1. The soils are assigned to drainage class ranges from 2.5 to 2.9, these values are intermediate between poorly drained, 2 and imperfectly drained, 3 values. The New Zealand Fundamental Soil Drainage Classification Layer from the New Zealand Land Resource Inventory (NZLRI, (Newsome, Wilde, & Willoughby, 2008)). The location of each of the above soils are shown in Figure 1-3.

Table 1-1: Soil Types of Wainuiomata (source Page, 1995)

Soil Name	Symbol	Description	Texture	Drainage
Taita hill soils	TH	Yellow-brown earths and related steepland soils. Soils are shallow and of low fertility.	Silt loam Clay loam	Moderately well to imperfectly drained
Tawai Steepland soils	TaS	Yellow-brown earths and related steepland soils. Soils are strongly leached yellow-brown earths.	Silt loam Clay loam	Moderately well to imperfectly drained
Ruahine Steepland soils	RuS	Steepland soils related to yellow-brown earths	Stony silt loam Silt loam Clay loam	Well drained
Rimutaka steepland soils	RmS	Steepland soils related to podzolised yellow-brown earths. Soils are podzolised, shallow and of very low fertility.	Silt loam Stony silt loam Stony sandy loam	Moderately well to well drained
Himatangi soils	Hm Hm +G	Flat, narrow alluvial valley floors with imperfectly to poorly drained soils developed from fine-grained alluvium. Gleyed recent soils. Intergrades between yellow-grey earths and yellowbrown earths	Silt loam Heavy silt loam Silty clay loam	Imperfectly to poorly drained

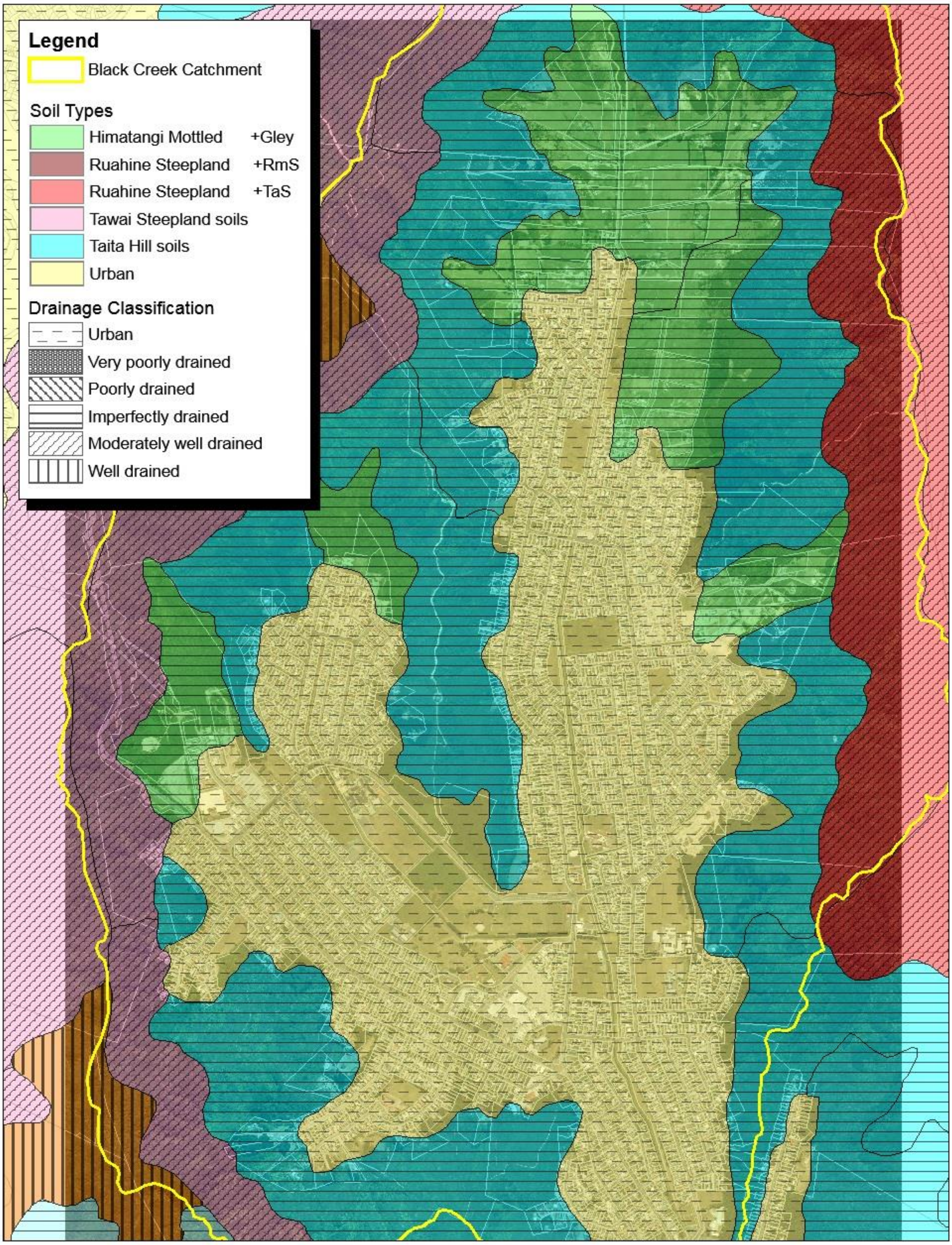


Figure 1-3: FSL drainage classification and soil types identified by NZLRI Land use Capability layer (Data reproduced with the permission of Landcare Research New Zealand Limited)

1.1.3 Land Use

There are three distinct land uses in the catchment. Urban residential which takes up a majority of the valley floor, farmland in the upper reaches of the valley, and native bush, pine trees or scrub in the hills as shown in Figure 1-4. Much of the farmland to the north is in the process of a land use change and is currently marked for future development in the Hutt City district plan as shown in Figure 1-5.

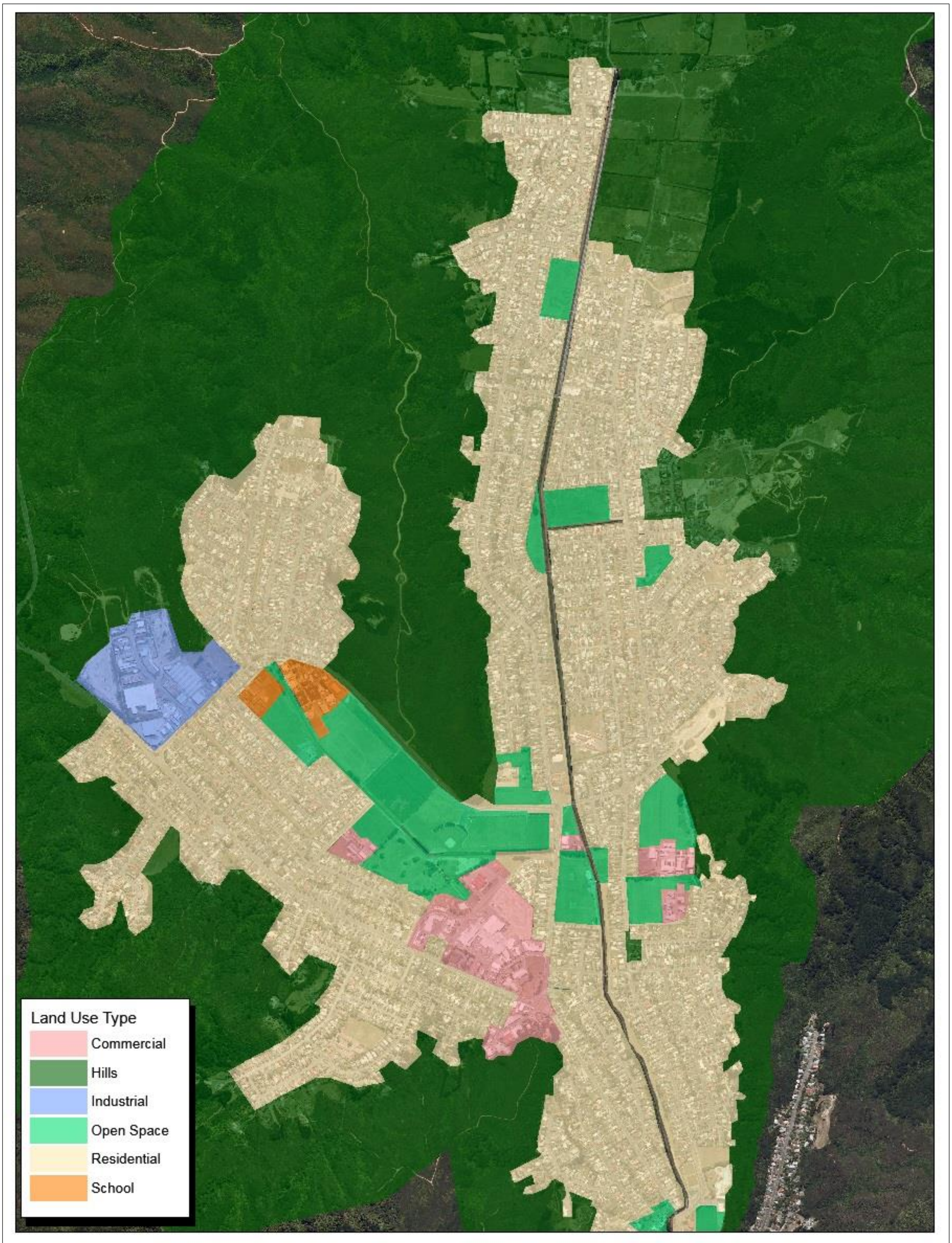


Figure 1-4: Current land use types

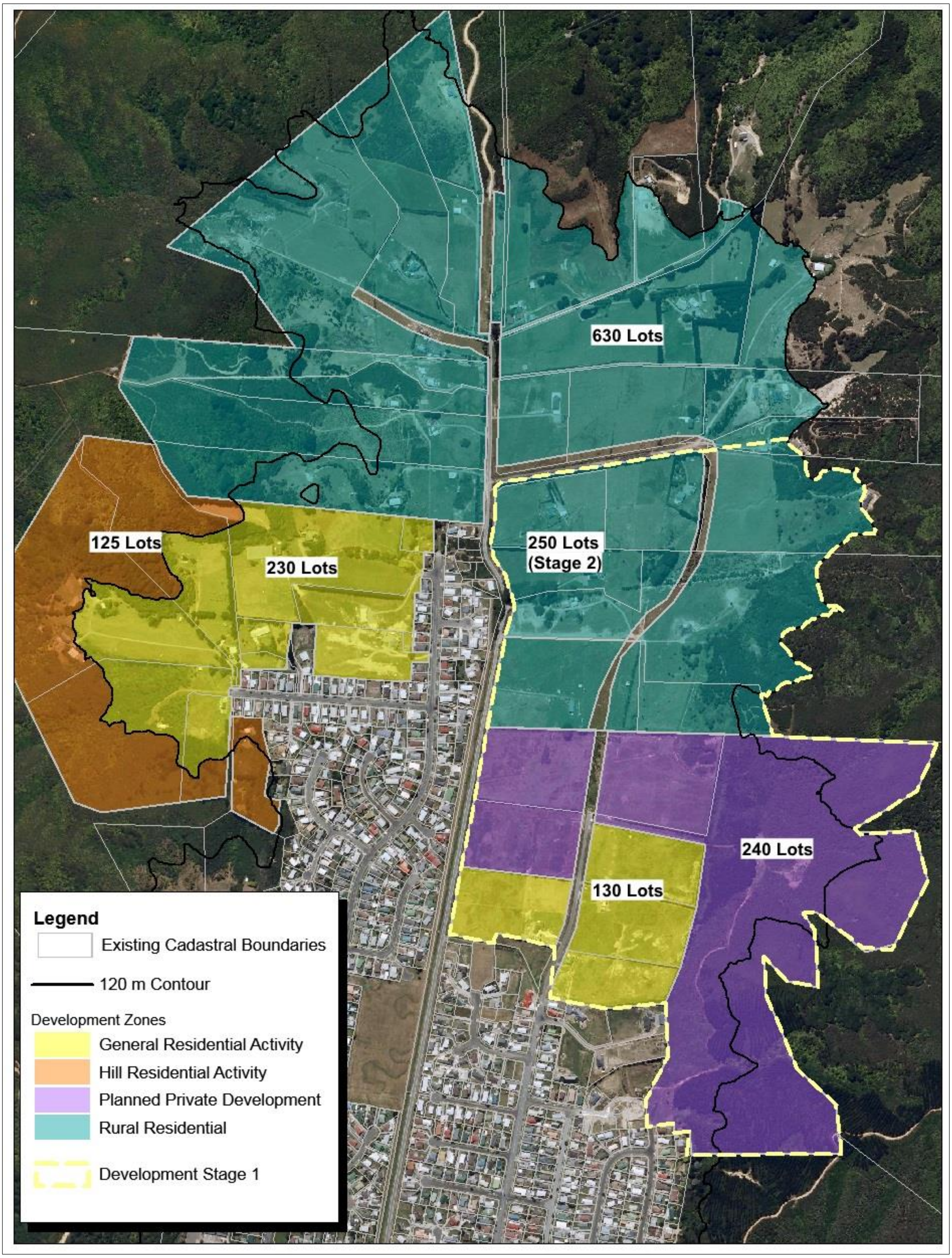


Figure 1-5: Planned development zones (from GHD report)

1.2 Background

Flooding has been a recorded issue in the Black Creek catchment and has been reported upon previously. In 2004, then Maunsell, now AECOM, completed an investigation into the flooding issues, which included a hydraulic model of the catchment, to aid in improving the capacity of Black Creek. Recommendations were made to widen sections of Black Creek, which included 2.75km of the channel in the following sections:

- Black Creek between Main Road and Nelson Crescent Bridge.
- Parkway Drain between Black Creek and Rata Street pedestrian bridge; and,
- Konini Drain between Parkway Drain and Konini Street culverts.

These upgrades were undertaken between 2005 and 2009.

The Upper Black Creek catchment has been targeting as an area for intense greenfield development, which was set out in the Hutt City Urban Growth Strategy 2012-2032. Therefore, a hydraulic model of the Black Creek catchment had to be developed to help quantify the existing capacity of the system, to identify the impact of the proposed development, and to highlight any flood mitigating options. The sensitivity testing and the freeboard stage of the model construction was completed with the intention of refining the baseline flood and allow an allowance for freeboard to be used for assigning minimum building floor levels for Wainuiomata.

2 Project Objectives

2.1 Activities and Scope

The scope of the multiple stages of the Wainiomata model, that have been combined into this report, are summarised below.

- Develop a rainfall – runoff model for land use development scenarios with current and future design rainfall events nominated by Wellington Water.
- Develop a model for the stormwater drainage system; primary piped mains and secondary flow paths, for the network extent, nominated by Wellington Water. This will then be reviewed and validated against the May 2011 event.
- High level assessment of effects of upper catchment development on the stormwater drainage system and the catchment hydrology.
- Determine the sensitivity of the design flood flows and design water levels against varying model parameters for specified scenarios as nominated by Wellington Water.
- Validate predicted flows and water levels by comparative referencing with gauged and reported information.
- Assess the capacity of the existing modelled drainage network system to convey runoff from current and future design rainfall events nominated by Wellington Water and for specific land use development scenarios.
- Assess the level of service provided compared to the desired level of service, and identification of the network systems that do not meet these standards, as nominated by Wellington Water.
- Analyse and tabulate design flows and water levels for land use development scenarios with current and future design rainfall events nominated by Wellington Water.
- Develop fluvial floodplain and flood hazard maps for rainfall event and development scenarios specified by Wellington Water.
- Establish Freeboard values following the methodology described in "Dynamic Freeboard Analysis – Tawa", Jacobs Memorandum, 04 October 2017.
- Establish a base case with 1% AEP rainfall with 20 % increase in rainfall intensity for climate change, assessing water depths and velocities.

This condensed report describes the work carried out by Stantec on all listed activities, with the incorporated feedback from WWL during the model build and validation process. As discussed in Section 1 the model was constructed pre modelling specifications and therefore contains differences. These differences are well understood by WWL.

Although this report covers each stage of the model construction (model build, validation, sensitivity, freeboard, and system performance), no updates were made to the models. Thus each stage is related to a separate model network within the database. Reference to which network the stage is related to is stated at the start of each section

3 Available Information

3.1.1 Drainage Network Data

Stormwater asset data was provided by Wellington Water in InfoNet and GIS format. The asset information provided in the InfoNet snapshot file data formed the basis of the current modelling. In the InfoNet asset data there were:

- 3373 nodes
  - Of these, 1477 were listed as manholes,
  - 169 had a chamber floor level provided.
- 3031 pipes
  - 2998 had internal dimensions specified
  - 175 had both upstream and downstream inverts given
  - 199 had an upstream invert recorded
  - 194 with a downstream invert recorded.
- Information obtained from the asset data provided by Wellington Water was flagged within ICM with the tag AS.
- In discussion with Wellington Water Stantec organised and undertook inspections of 35 manholes to provide approximate depth-to-inverts and pipe diameters.
- In addition to this Stantec reviewed 314 scanned as-built plans provided by Wellington Water to gather any available manhole or pipe information.
- Following the survey and review of available and relevant as-built plans, the number of nodes with available chamber floor levels was increased to 241.

The GIS asset data provided by Wellington Water did not include accurate information for the multiple stream crossings on Black Creek and Park Drain, however, as-built information was provided for three bridges (Main Road Bridge, Park Drain Bridge, and the new Bryan Heath Park pedestrian foot bridge). Stream crossing and bridge information was also available in the previous Hydrologic Engineering Centre, Rainfall Analysis System (HEC-RAS) model (geometry data from version 09-2009_WellingtonRd_noDevt_final) provided by Wellington Water. In addition to this Stantec staff visited all the accessible stream crossings, collecting photographs and, where possible, relative measurements of relevant dimensions. Details for the crossings and how they have been represented in the model are provided in Crossing Details Appendix H

Other features not captured in the supplied GIS data included three instream weirs and culvert inlet and outlet structure details. Several the culverts conveying streams beneath roads have weirs at their inlets, including the culverts conveying the two Black Creek tributaries across Wise St. Details of how these have been represented in the model are covered in Appendix A.

Stantec, is not aware of any detention or storage features that impact the stormwater network in the Wainuiomata catchment.

The GIS information provided did not include accurate information for the multiple stream crossings on Black creak, or the tributary Park Drain, however, the as built information was provided for three bridges:

- Main Road Bridge
- Park Drain Bridge
- Bryan Heath Park foot bridge.

Wellington Water provided stream crossing and bridge information that had been available in a previous HEC RAS model form 2009. Visual confirmation was also completed by Stantec staff.

3.1.2 Stormwater Network

Missing or inconsistent data was identified and marked. A review of the latest GIS from HCC is summarised in Table 3-1 and Table 3-2 below.

The following table shows a summary of the missing data by numbers. “Manholes” refers to any asset data listed as a “node” within InfoNet, and hence may include junctions and outfalls etc.

Table 3-1: Stormwater Pipe Asset Data Summary (full catchment data)

Asset	Classification	Priority	Number	Percentage of Total (%)
Nodes	Total Stormwater Manholes		1451	100%
	Ground Levels in GIS (or surveyed)	1	215	15%
	Missing chamber dimensions	2	1451	100%
	Missing chamber depths	1	1349	93%
Pipes	Total Stormwater Pipes		1861	100%
	Missing pipe material	1	33	2%
	Missing pipe width	1	11	1%
	Upstream and downstream inverts available	1	206	11%
Other	Outgoing pipe width is smaller than largest incoming pipe	1	29	2%
	Outgoing pipe invert is higher than incoming pipe invert	1	4	0%
	Downstream pipe area is smaller than an upstream pipe area	2	0	0%
	Invert is above cover level	2	1	0%
	Soffit is above cover level	2	1	0%
	Negative pipe gradients where there are upstream and downstream pipe inverts	2	11	1%

- Overall, the stormwater asset data is only available for 11% of pipe inverts. This is very low compared to the old NZWWA criteria (which has now been removed from the guidelines).
- 215 manholes have ground level information which is again considerably low. 102 manholes had attached additional information for manhole depth, which can be used to calculate the manhole's chamber floor level once a ground level is obtained.
- The GIS data supplied does not have chamber size. A likely chamber size can be inferred based on the diameters of the connected pipes.
- The percentage of pipes missing diameters is low at less than 1%. The number of missing pipe materials is also low at less than 2%. An unknown material is not always an issue for hydraulic modelling as slime growth results in similar roughness values regardless of material.
- The percentage of pipes with a downstream reduction in pipe diameter is not significant, at around 2%.
- Ignoring pipes with an invert level of 0, there are a very small number of pipes (less than 1% of total network) with negative gradients.

3.1.3 Other Stormwater Assets

There are no known purpose-built control structures in the study area, although there is a possibility that the bridge sites act as hydraulic controls.

Table 3-2: Ancillary Structures Data Summary

Feature	Total Number	Number in Key Routes	Comments
Manholes	1482	174	Limited survey data.
Inlets	95	7	No survey data available.
Outlets	195	39	No survey data available.
Flap valves	2	2	No survey data available
Bridges	4	4	As built drawings available for two sites
Weirs	-	-	No weirs were recorded in the GIS data. Weirs were added instream as part of the model build for river reach connections.
Ponds	**-**	**-**	No ponds or wetlands have been identified in the study area.

3.1.4 Hydrologic/Hydrometric Data

National Institute of Water and Atmospheric Research (NIWA)'s High Intensity Rainfall Design System (HiRDS) v3 was also available from the online form. HiRDS V3 is an online interface that provide estimates on rainfall at any location in New Zealand. The database is made up of climate observations from NIWA (National Institute of Water and Atmosphere) database. HiRDS takes topographic variation in recorded gauges into account

3.1.4.1 Rainfall Data

Wise Park Gauge data was provided by Wellington water which incorporated 5-minute interval data from 1st January 2005 to 29 July 2014. Hourly data was used from 30th July 2014 through to 1 January 2015, and this data was provided by HVWS. Wise Park, shown in Figure 3-1 is the only available long term rainfall data within the Black Creek Catchment and contains no major gaps in the data. Two other gauges, Wainuiomata at WTP and Wainuiomata at Reservoir are other rainfall gauges in the proximity, however, they do not sit within the boundary of Black Creek catchment.

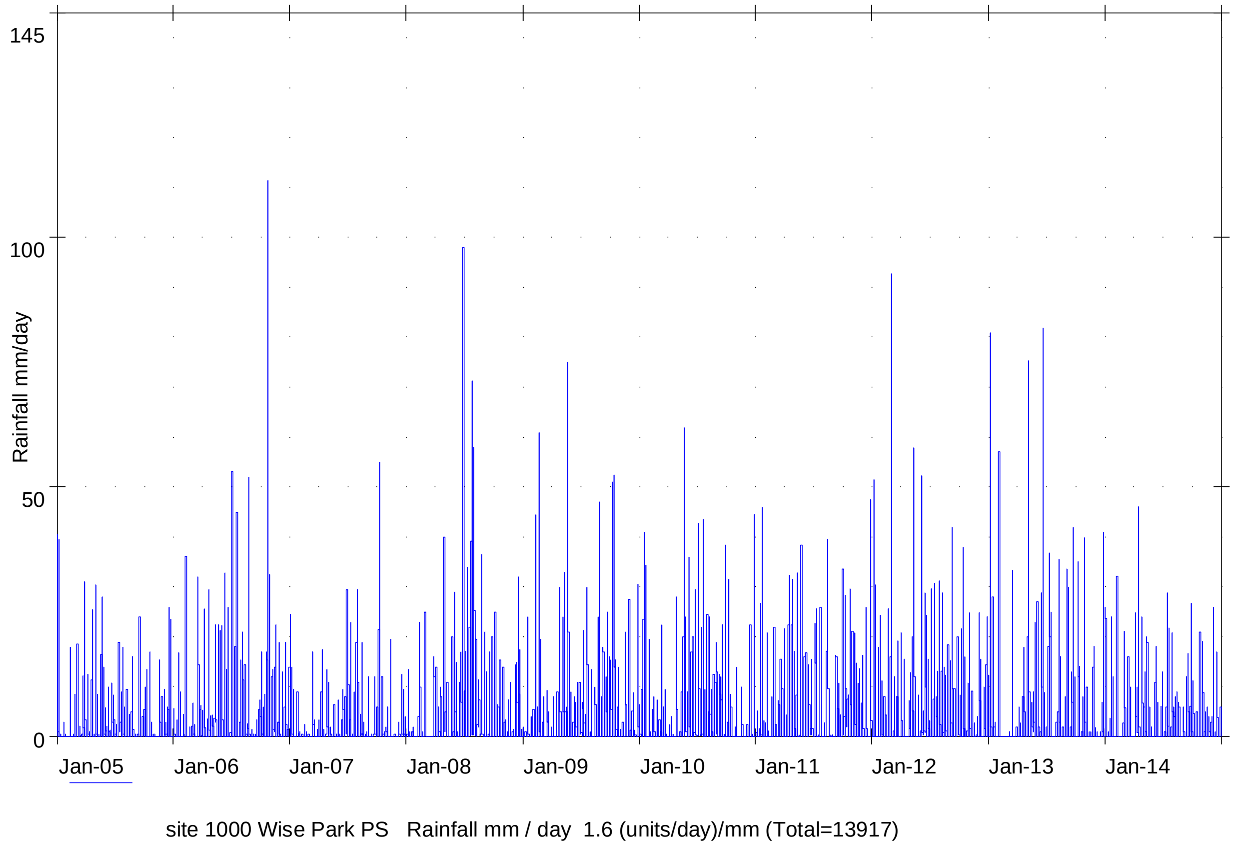


Figure 3-1: Wise Park Rainfall

Figure 3-2 is a double mass plot of wise park and Wainuiomata WTP rainfall. It shows no significant rainfall recording or exposure issues evident at wise park gauge from 2007 to 2011 and a gradual increase in recorded rainfall at WTP relative to Wise Park from the middle of 2011.

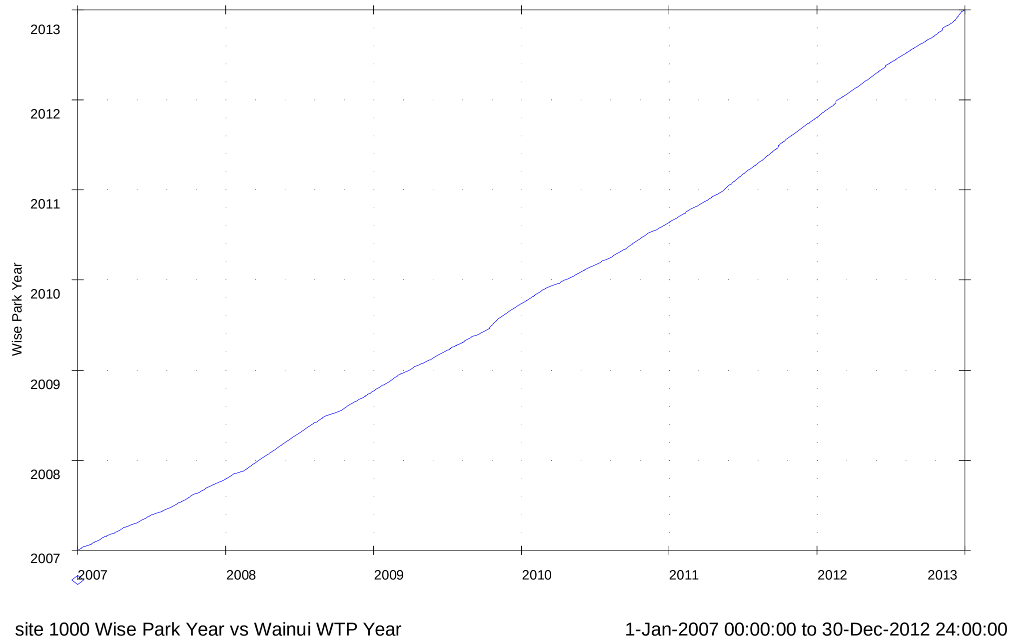


Figure 3-2: Double mass plot, Wise Park and Wainuiomata WTP

3.1.4.2 Water Level

Water level in Black Creek is recorded at Wellington Road Bridge and Main Road Bridge. Wellington Road data, Figure 3-3 spans 7 January 2010 to 1 March 2015 and has no gaps in the Data.

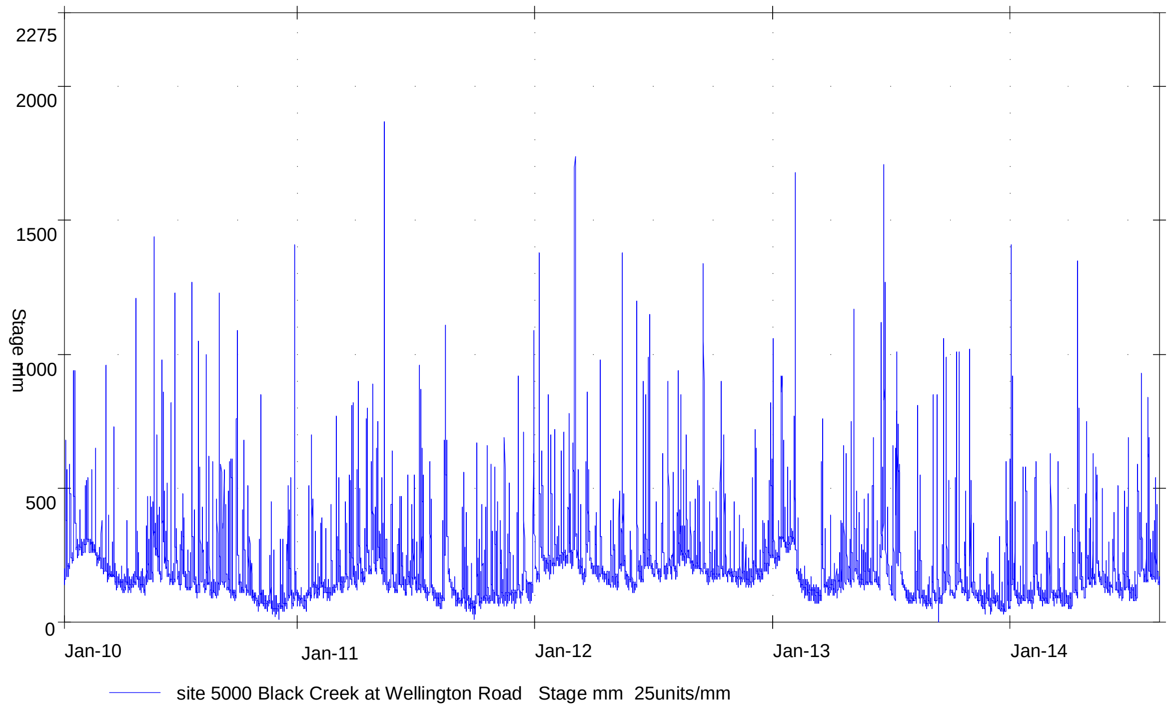


Figure 3-3: Wellington Road Water Level

There is recorded data for 24 February 2011 to 31 January 2014 for Black Creek at Main Road Bridge, Figure 3-4. However, there are 2 gaps in the data during this time. Both gauges, despite gaps, have reasonable data with no obvious shift in Bed level.

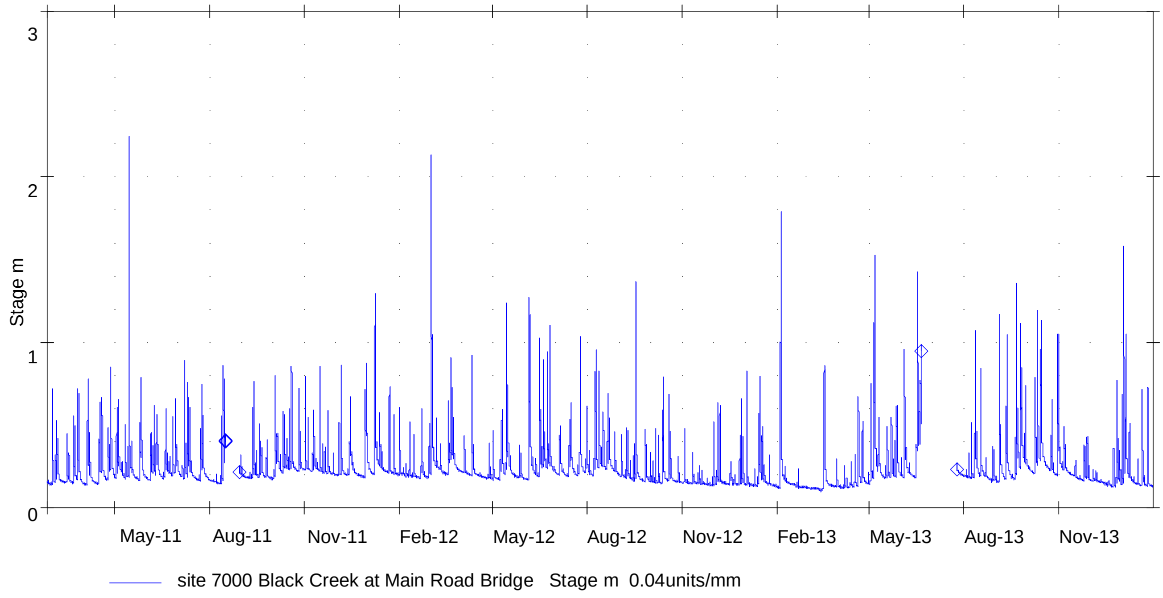


Figure 3-4: Main Road Water Level

Velocity data were also recorded using a Sontek Argonaut-SW acoustic doppler sensor installed on a straight section of stream approximately 300m downstream of Main Road Bridge for a period of three months over winter. The Argonaut SW cross-section was surveyed, and low flow velocity profiling was carried out to determine a low flow velocity index. High flows were not gauged so the validity of recorded flood flow velocities is less certain than for low flows.

The continuous three months flow data was used to develop a depth to discharge rating for the Main Road Bridge stage record as shown in Figure 3-5.

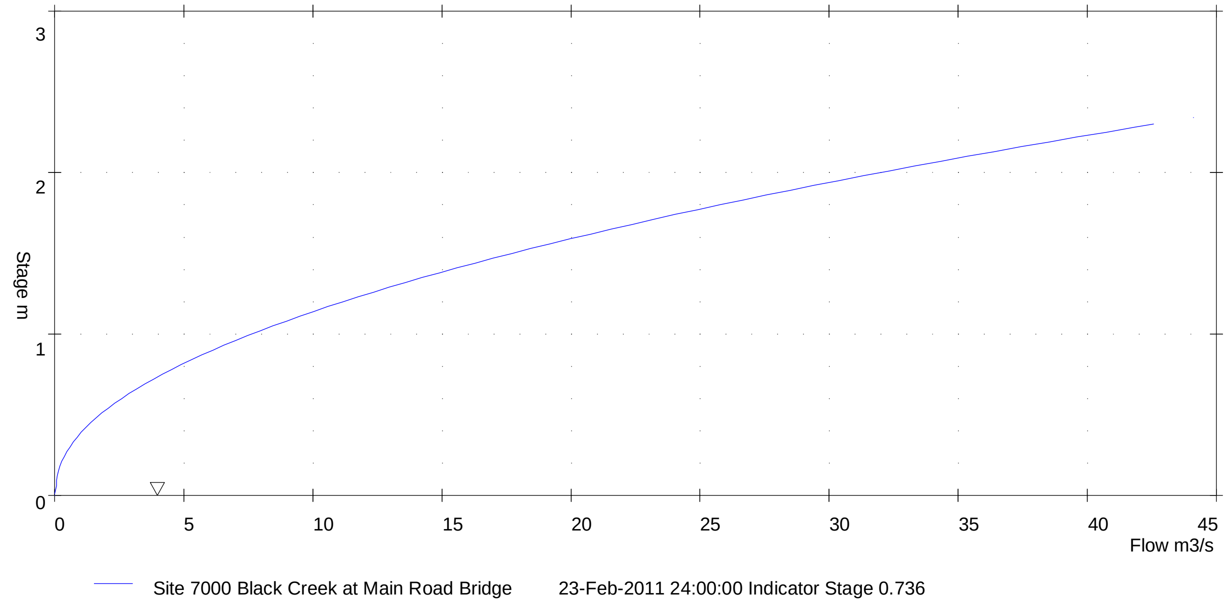


Figure 3-5: Main Road Bridge Rating Curve

Figure 3-6 shows the flow hydrograph produced by applying the rating developed by AWT to the Main Road Bridge stage record.

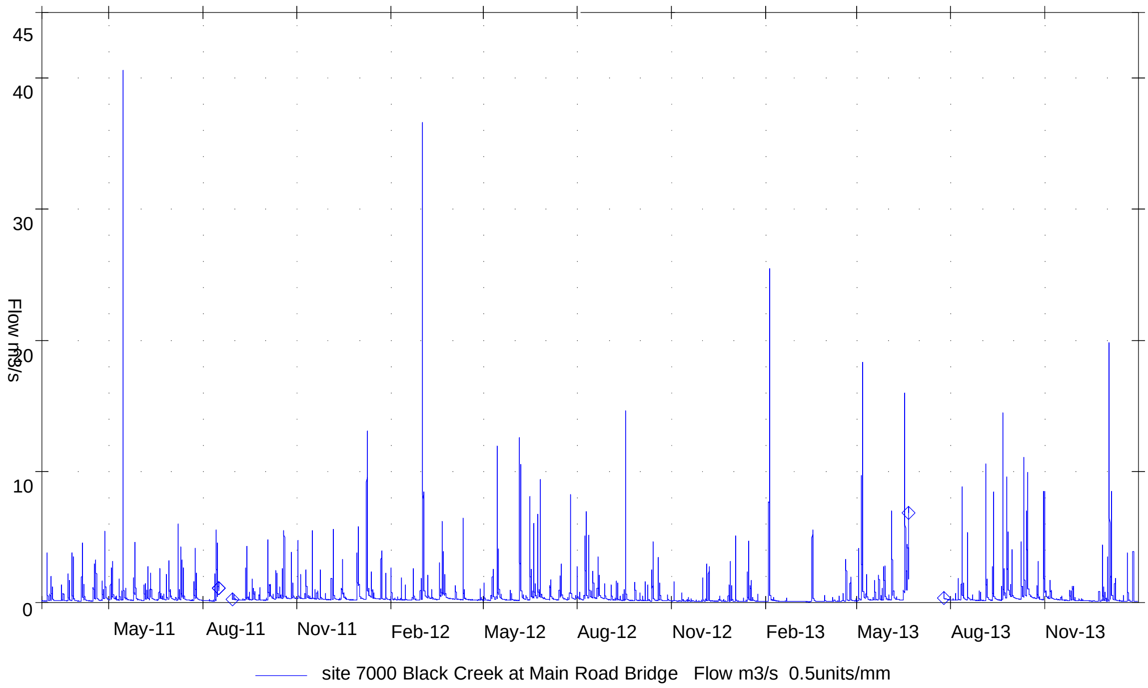


Figure 3-6: Main Road Bridge Flow Hydrograph

A comparison of Wellington Road and Main Road Bridge stage data, Figure 3-7 shows a distinctive rise in Wellington Road base flows at the end of 2012 compared with the Main Road Bridge record. This could be a result of channel aggradation of an issue with the level sensor.

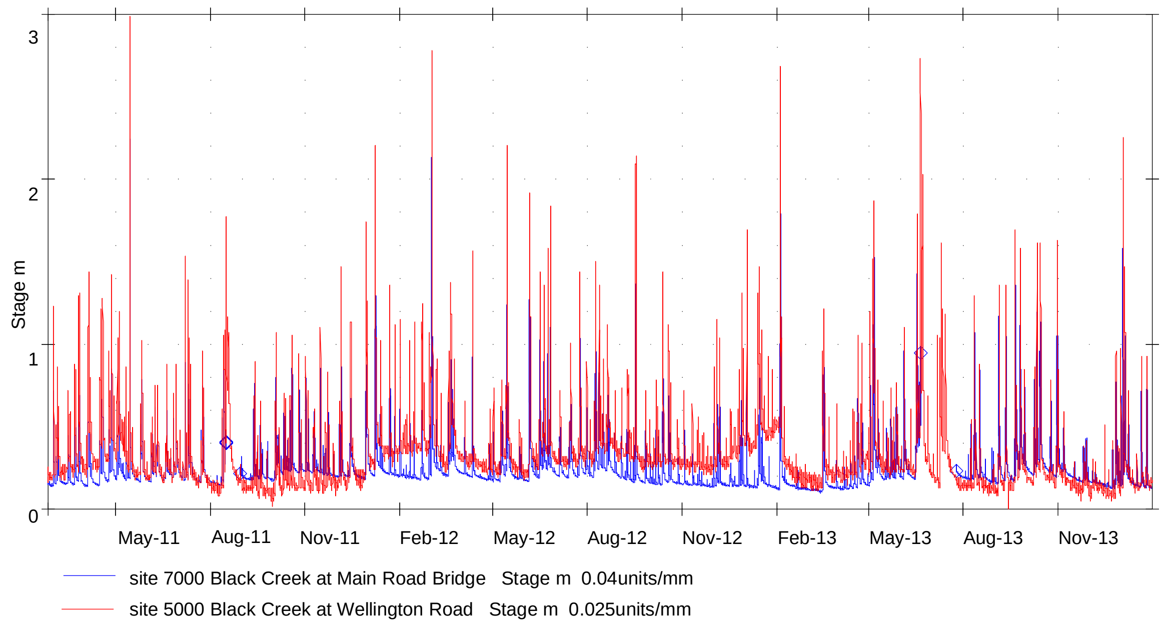


Figure 3-7: Comparison of Wellington Road and Main Bridge Road stage data

3.1.4.3 Topographical Data

Topography data for the model was provided by Wellington Water in the form of a 1m ASCII grid generated from LiDAR. The LiDAR was collected in 2013 and provided by Hutt City Council. A review of the grid generated from the LiDAR concluded it was adequate for use in the current hydraulic model.

In discussion with WWL it was decided to use to the grid to generate cross sections for all of the channels included in the 1-D extent of the model except downstream of Main Road Bridge where vegetation on the channel banks significantly reduced the accuracy of the LiDAR. This decision was reached following a comparison of selected surveyed cross-sections applied in the HEC-RAS model and sections cut from the grid along the same alignments. It was found that the sections generated from the grid adequately matched the surveyed sections. Additional advantages of using the LiDAR included:

- The elimination of the need to undertake the collection of surveyed cross-sections, where no previous surveyed sections existed.
- The LiDAR represented a more up-to-date version of the stream channels which were widened between 2005 and 2009 (Figure 3-8).
- The use of cross-sections cut from the grid required no additional processing when coupling to the 2-D extent which would be based on the same grid.

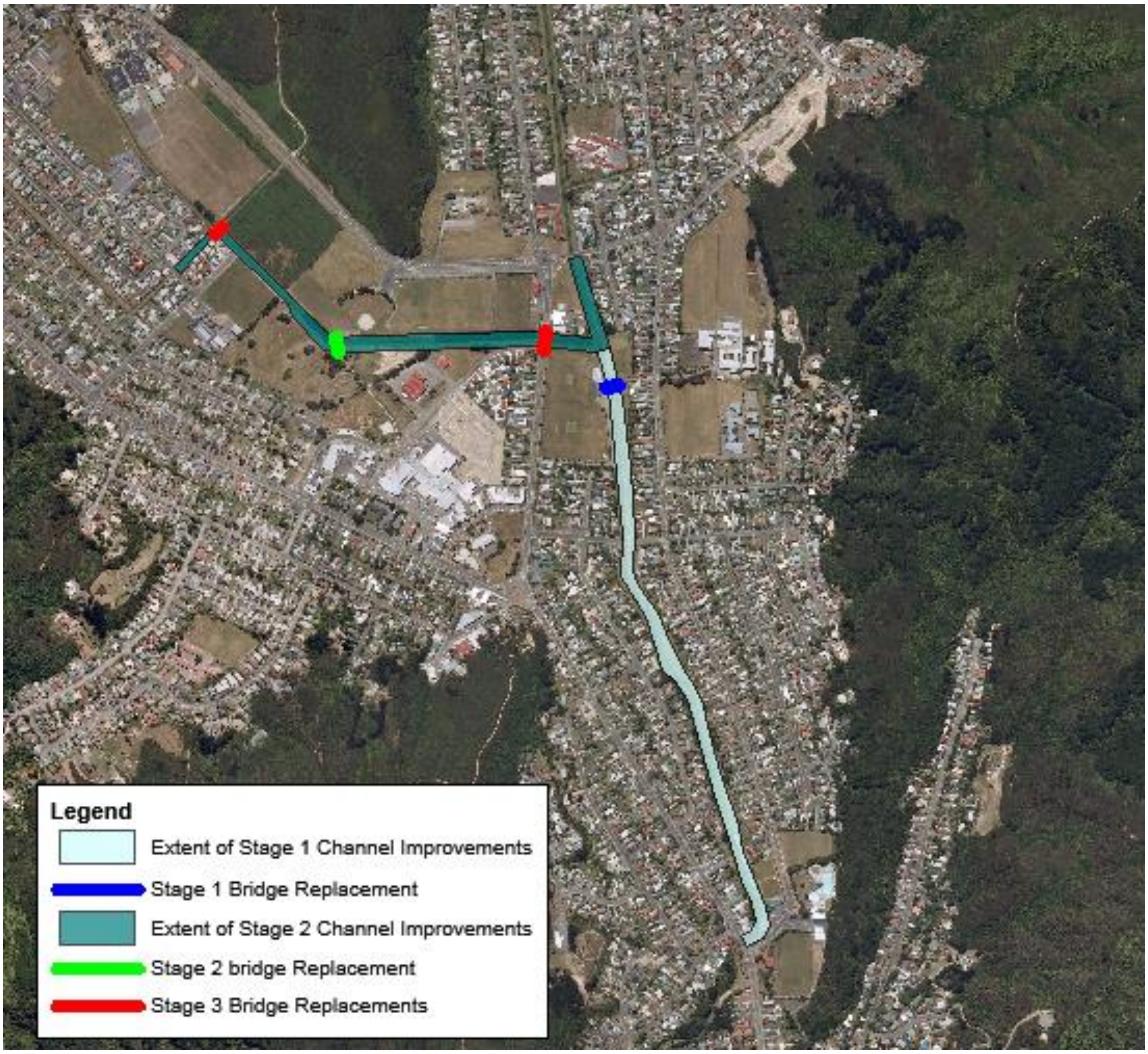


Figure 3-8: Extent of the channel improvement works carried out between 2004 and 2009

For the section of Black Creek extending from Main Rd Bridge to the confluence with the Wainuiomata River cross sections were sourced from the MIKE11 model built by Greater Wellington Regional Council in 1998. For the current modelling it is has been assumed that there have been no significant changes to this portion of the channel since 1998.

3.1.5 Operational Data

There are no stormwater pump stations, in the Wainuiomata system. However, a fairly comprehensive review of the general operational issues of the network, such as flooding, is covered in the scoping study undertaken by Wellington Water.

The Scoping Study carried out by Wellington Water identified 11 wastewater overflows in the catchment (Table 3-3). These have not been included in the current modelling. Modelling of the wastewater system was carried out in 2011-2012, however, the maximum event magnitude simulation was the 5yr ARI event, and the model results indicated that the overflow volumes were not significant compared to flows in the pipe and channel network.

Table 3-3: Wastewater overflows in the Black Creek Catchment (from Table 3 3, pg15, Wellington Water, 2014)

Node	Location	Overflow Volume (m³) 1 year ARI (current scenario)	Pipe Size (mm)
710002R00936	End of Rowe Pde	2946	300
710002R00866	End of Heath Street	1735	375
710006R00896	Rear 95 Main Road	1719	375
710011R00855	50 Fraser Street	572	200
710011R00874	36 Hyde Street	449	225
710003R00828	Footbridge Best Street	213	225
710015R00853	27 Fitzherbert Road	194	225
N/A	Wellington Road PS	164	455
760019R00968	24/26 Wood Street	114	150
710003R00847	End of Dunn Street	69	300
610003R00916	Ngaturi Park	0	100

3.1.6 Known Flooding Issues

Information related to previous recorded flooding incidents as well as previous studies regarding areas with historical and known flooding problems, include the following areas:

- Hair Street
- Gibbs Crescent
- Parenga
- Grimsby Grove
- Wise Street
- Norfolk Street
- Crowther Road junction Brookfield Lane
- Westminster Street
- Parts of Wellington Road
- Hine Road
- Donelly Drive
- Crescent Bridge,
- Best St. Bridge and
- Konini Street Bridge.

Recent stream widening and bridge replacement works have mitigated flooding in a number of locations. No recent flooding of note since 2004 – See Appendix C for the full list.

4 Model Build

Innovyze software Infoworks ICM version 5.0.3.10021, August 2014 was used for the development of the Black Creek Stormwater model. This was updated to ICM version 9.5 to allow for all model stages to be brought together in one database. The associated network for this section is the baseline, “Black Creek Base Scenario 2020 model”.

ICM allows for the integration of one- and two-dimensional domains, allowing both above and below ground elements of the catchment model to be incorporated, representative of all flow paths.

ICM, contains user defined flags that can be used to specify the data source. Flags were used to define all data sources as set out in WWL SW Regional specification as seen in Figure 4-1.

	Name	Display Colour	Obsolete	Description
	#I			Model Import
	#S			System Calculated
	#V			CSV Import
	AERL			From Aerial
	AN			Anecdotal Information
	ASB			From an Asbuilt
	CALC			Calculated value
	CCTV			From CCTV
	EJ			Engineering Judgement
	FI			From Field Inspection
	GIS			GIS Import
	GPS			From GPS survey
	IF			Inferred
	LIDR			Ground Levels inferred from LIDAR Data
	MI			Manhole Inspection
	MS			Modelling Specification
	PROP			Proposed
	RS			Regional Standard for Water Services
	SURV			Surveyed
	XX			Guessed or Suspect Data
*				

Figure 4-1: Asset Information source data flags

4.1 Hydrological Model

MWH (now Stantec) built the base model in 2014/2015 using the pre panel agreed Hortons runoff parameters and 24hour nested rainfall. In 2016 WWL updated and converted the model to use a SCS method and 12hour nested rainfall to match the panel modelling specifications at the time. These updates were then reviewed by Stantec in 2018/2019. The final methodology has been detailed below.

The catchment area draining to the Black Creek –Wainuiomata River confluence is approximately 16.5km2. This area contains 388 subcatchments delineated using the existing pipe network and land parcel information. In the area where development is planned in the future, such as the Upper Black Creek, the subcatchments were generated from the proposed stormwater network and follow the requirements of the Regional Specification (ranging from 0.5ha to 3ha). While this is potentially more detailed than required for the current phase of modelling it will allow for future detail to be added to modelled network. Where no pipe network is modelled the size of the subcatchments has not been restricted to the specification requirements.

The final hydrological model adopted, following the model validation process, included adjustments to the subcatchment extents. To ensure routing was properly represented, following the initial delineation of the model subcatchments and assessment of the original hydrology in the model, the subcatchments were aggregated by their inflow locations. That is, all subcatchments draining to a network node were aggregated into a single subcatchment and all runoff surface percentages were appropriately combined. This reduced the number of subcatchments to 200, from the original 388.

4.1.1 Runoff Surfaces

As stated above WWL updated the model hydrology to match the more recent specifications. This involved changing the original 4 runoff surfaces into separate runoff surfaces for each subcatchment. Each of these runoff surfaces were given initial abstraction values and impervious percentages based on the regional hydrology/landuse layers developed by the Wellington Water Stormwater Modelling Panel.

4.1.2 Modelling Proposed Development Subcatchments

A key component of this investigation is an assessment of potential impacts arising from proposed development in the upper Black Creek catchment. Figure 1-5 shows the area currently proposed for development and the expected land use. The area marked as the development zone is likely to be developed in 10 to 20 years' time. With development of the land, there is an expected pattern of more rainfall becoming runoff due to the likely hood of increased impervious land. These were based on the change from rural to residential land use.

To investigate the potential impacts of the development, the percentages of the runoff surfaces in the model catchments affected by the development zones have been adjusted to account for the change in land use The affected subcatchments are shown in Figure 4-2.

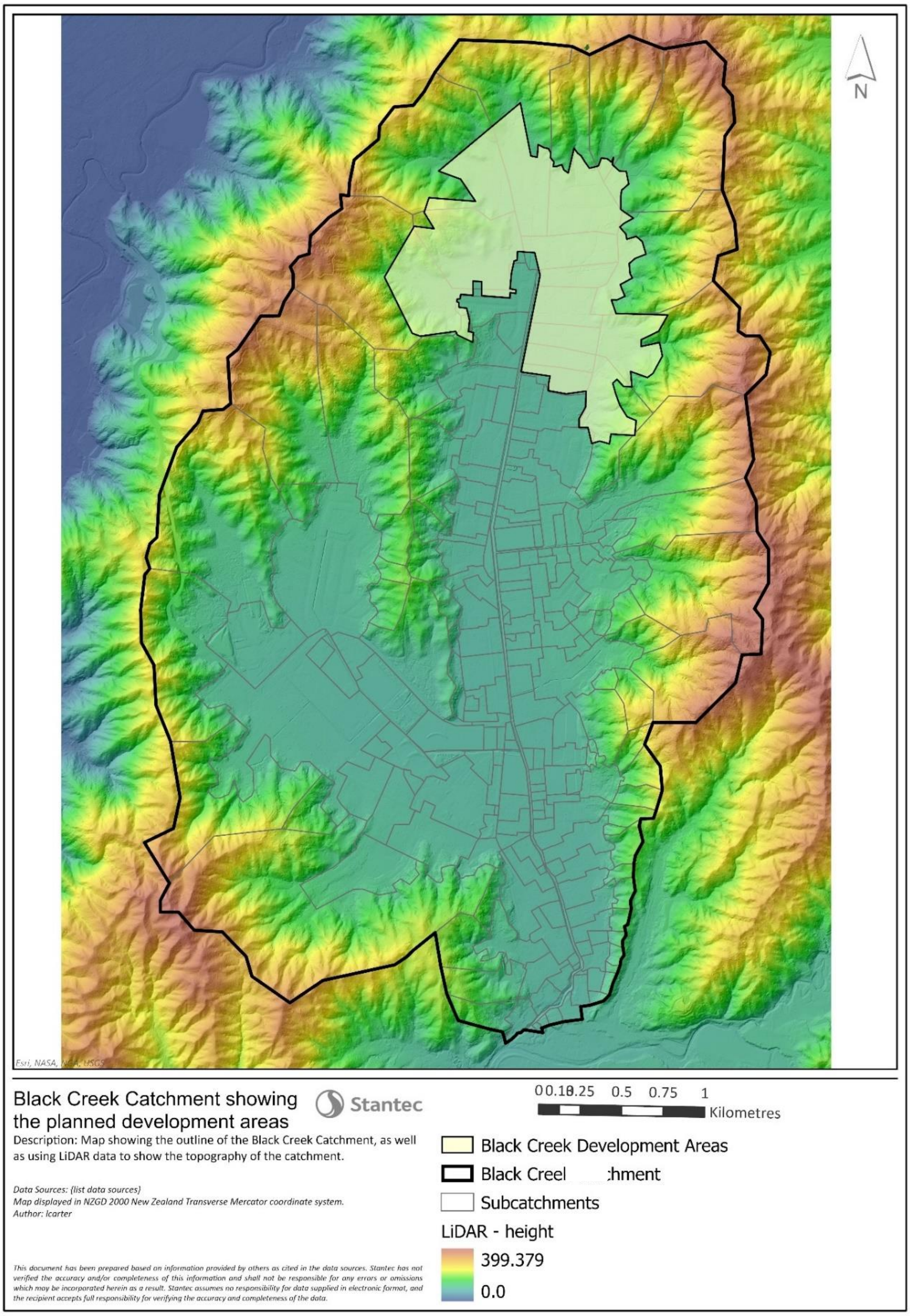


Figure 4-2: Subcatchments impacted by proposed development

4.1.3 Hydrology Layers

Regional hydrology layers were provided by WWL and Cardno. These layers contain CN (curve number), IA (initial abstraction) and Impervious percentage values across the region. No changes were made to these layers at the time of the construction of this model.

4.1.4 Inflows

There are four inflows used the Black Creek model. All four are tiny 'base flows' used to improve stability in the model and are applied at the top of river reaches. The nodes in which the inflows are modelled to join are shown in Table 4-1.

Table 4-1: Table showing the inflows at the river reaches in the model

Time	BCc0002 (m3/s)	BCc0004 (m3/s)	Side_CK_2 (m3/s)	Sice_CK_3 (m3/s)
01-01-2006 at 00:00	0.01	0.01	0.005	0.005
01-02-2006 at 00:00	0.01	0.01	0.005	0.005
01-03-2006 at 00:00	0.01	0.01	0.005	0.005
01-04-2006 at 00:00	0.01	0.01	0.005	0.005
01-05-2006 at 00:00	0.01	0.01	0.005	0.005
01-06-2006 at 00:00	0.01	0.01	0.005	0.005
01-07-2006 at 00:00	0.01	0.01	0.005	0.005
01-08-2006 at 00:00	0.01	0.01	0.005	0.005

4.2 Hydraulic Model

4.2.1 Method Used

A coupled 1-D and 2D model has been developed in Infoworks ICM to represent the mixture of piped (1D), channelised and the overland flow (2D) in the Black Creek Catchment in Wainuiomata.

The LiDAR digital terrain model was used to create the 2D mesh polygons to cover the valley floor up to the extent of the residential zone. Buildings are represented within the 2D simulation polygon using roughness zones. The mesh triangles have a minimum area of $2m^2$ and a maximum of $10m^2$.

The Black creek, as well as the Park Drain channels, have been modelled in 1 D based on cross sections cut from the ground model. Key pipe networks are represented in the 1D network, including manholes and outlet structures.

4.2.2 Hydraulic Model Extents

The extent of the current hydraulic model is shown in Figure 4-3 below. The model is made up of three components and covers most of the Wainuiomata Valley floor, only extending onto the hillside at selected locations.

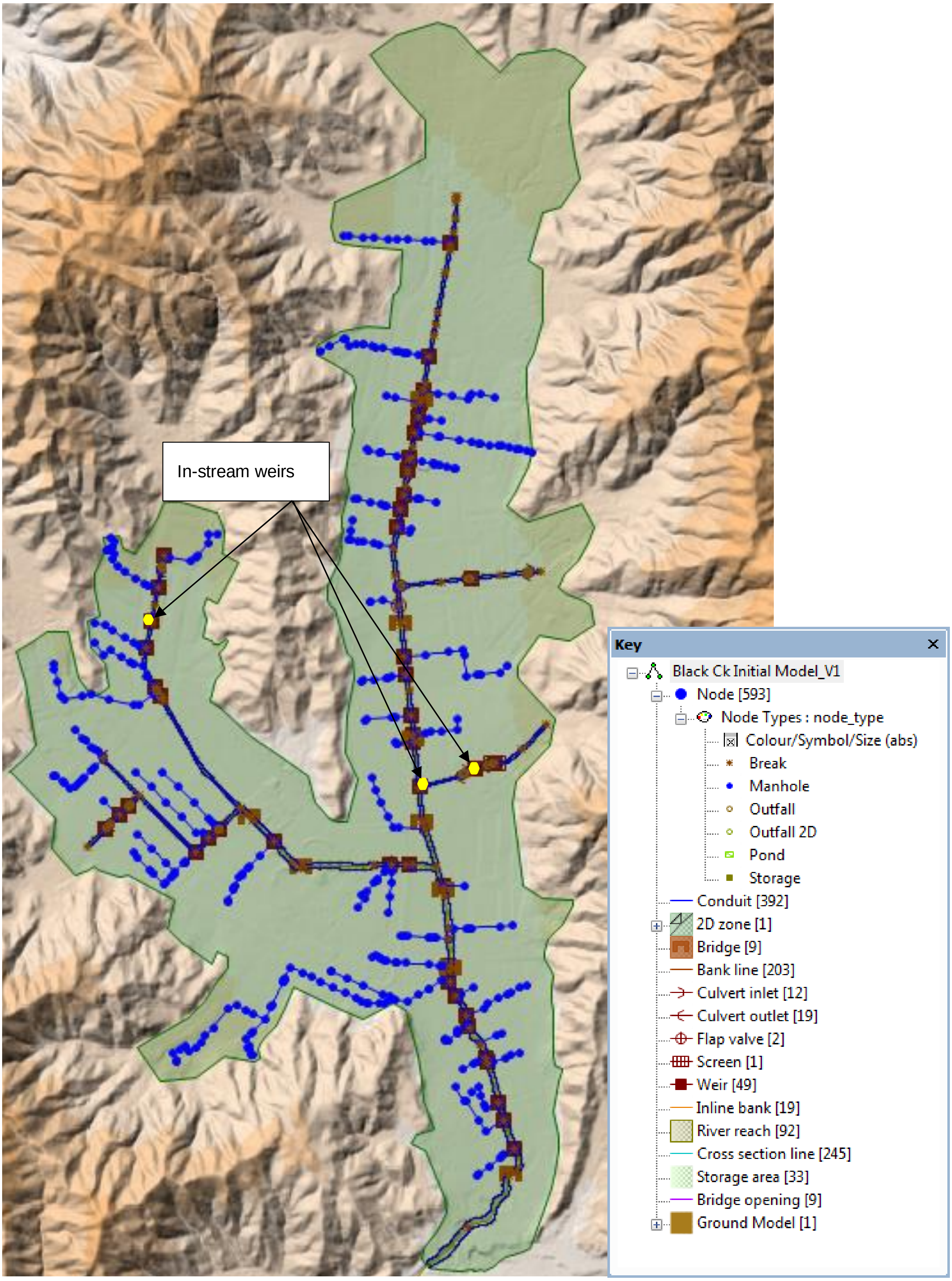


Figure 4-3: Hydraulic model extent and 1-D network features

4.2.3 1D Pipe Network

The Wainuiomata stormwater network includes forty-eight pipelines incorporated into the model. Of the 48, 33 of these were identified in the scoping study in Stage 1.

The pipe network comprises of 48 pipelines represented by 422 manholes and 392 individual pipes. See Table 4-2 for the breakdown of the network.

4.2.3.1 Manholes

- 372 out of the 422 manholes are expected to be real features, based on the asset information provided.
- The remaining 50 are dummy nodes included for connectivity purposes.
- Nodes with Asset IDs beginning with BCc have been assumed to be connectivity/dummy nodes.
- All manhole lid levels have been inferred from the ground model and flagged as “LIDR”, “INF”, or “IF”.
- The “Flood Type” of the 372 manholes, expected to be real features, has been set to either “2-D” or “sealed” depending on if the node falls within the 2D mesh extent.
- The 2-D Element Area Factor and Flooding Discharge Co-efficient for the majority of these manholes have been set to the default values of 1.0 and 0.5, respectively.
- The chamber and shaft plan areas for all the manholes have been calculated by ICM based on the connecting pipe diameters.
- The “Flood Type” of the connective nodes has been set to “Sealed” so no flow is lost at these locations, as they are not real assets.

4.2.3.2 Pipes

- The majority of the pipes are reinforced concrete. However, the asset data shows there are at least seven PVC pipes.
- In most cases the pipe network lines start at a manhole with a known depth-to-invert and end at an outlet that drains to an open channel.
- In selected cases a pipeline connects an inlet point along a distinct channel to a manhole with a known invert downstream. The inverts at these inlet locations have been inferred from the ground model.
- Pipe outlet levels have been inferred from the ground model, site photographs, and measurements taken by Stantec staff.
- There is some missing storage in the pipe network due to not all pipes being included in the model. However, this missing storage has been accounted for using the “Prune” function in ICM and pipe information provided by Wellington Water.

Table 4-2: Summary of Hydraulic Model Components

Hydraulic Model Components	Values
Total number of stormwater network system nodes	591
Total number of subcatchments	200
Total number of links (includes conduits, river reaches, & weirs)	493
Total number of stormwater network system pipes	392
Total number of overland flow path/open channel links	92
Total number of weirs	51
Total number of orifices	0
Total number of outlets	20

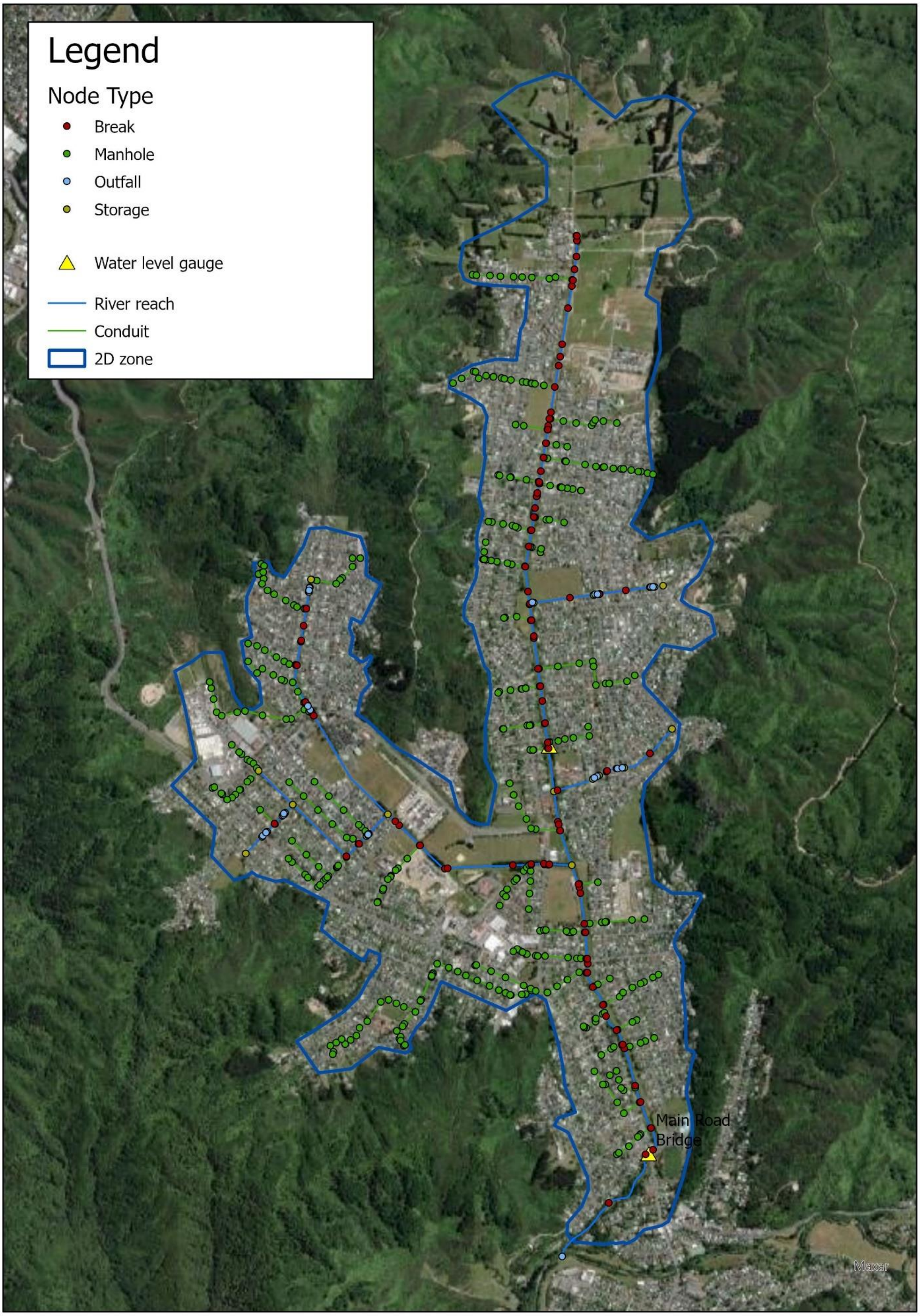
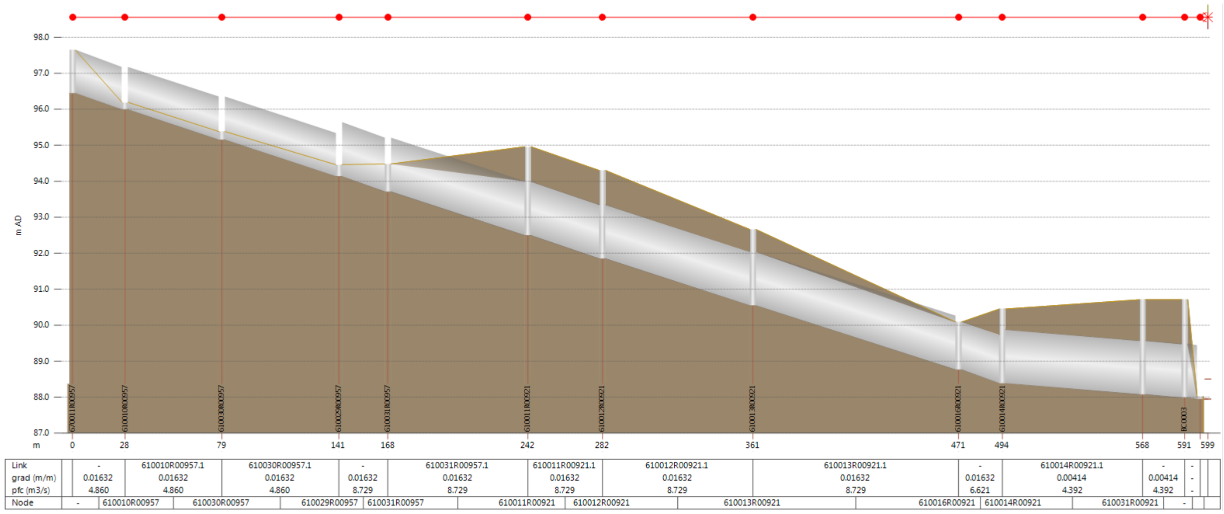


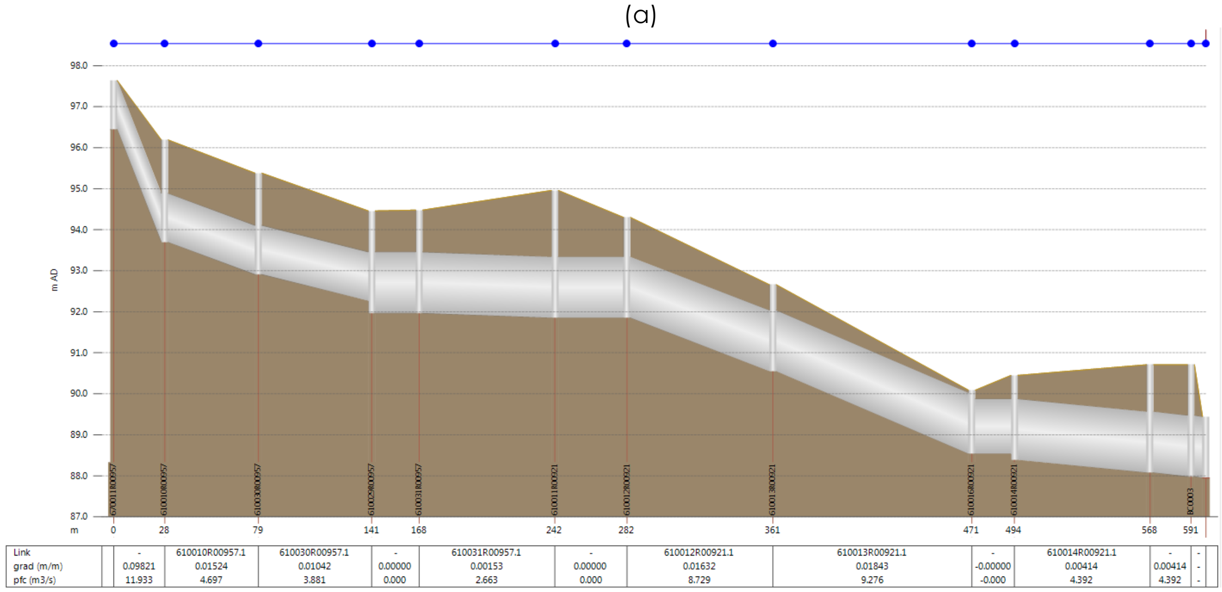
Figure 4-4: Modelled stormwater network

It should be noted that due to the limited coverage of the modelled stormwater network (Figure 4-4), any overland flow originating from under capacity portions of the network not included in the model will not be evident in the results and therefore the flooding in the catchment may be underestimated.

Pipe inverts and manhole chamber depths have been interpolated between the manholes with asset data and the outlets. On straight lines following a relatively constant grade all inferred levels have been flagged as “IN”. However, on a number of lines the ground level did not follow a constant grade and inferred pipe inverts rose above the ground model ground level. The Regional Specification does not provide guidance on the depth of cover to be adopted and as the pipes were installed prior to development of specifications for pipe installation, depths have been assumed. In these cases engineering judgement has been used to estimate a reasonable depth to invert at selected manholes along the line, taking into account depth-of-cover, these have been flagged as “EJ”. Any intermediate pipe inverts have then been inferred from the estimated manhole depths Figure 4-5 . Where this has been done, the pipe inverts and manhole chamber floor levels have been flagged as “XX”, Guessed or Suspect Data.



(a)



(b)

Figure 4-5: 1-D Pipe long sections. a) Initial straight-line interpolation, b) Implemented inverts in the model following updates from engineering judgment.

4.2.3.3 Outlets

All of the pipe outlets generally fall into three categories as follows:

- A pipe projecting out of the channel bank with no outlet structure (Figure 4-6and Figure 4-7),
- An outlet structure including a concrete apron and/or weir (Figure 4-8 and Figure 4-9),
- A narrow concrete channel, approximately the width of the pipe, to convey flow from the pipe to the stream (Figure 4-10).

At all but two of the pipe outlet locations the pipe network has been connected into the 1-D channel network using weir links. The weir width has been set either as the actual width of the existing weir in place or the same as the pipe where no weir is present. This approach has been adopted as all pipe outlets are situated above the bed of the channels they drain into. In ICM, weir links are able to cope with significant differences in pipe invert and bed level while keeping the model stable.

At two locations flap valves were present so flap valve links were used, (Figure 4-11).

Figure 4-6: Protruding outlet

Figure 4-7: Protruding outlet

Figure 4-8: Outlet with concrete apron and weir

Figure 4-9: Outlet with concrete apron and weir

Figure 4-10: Narrow concrete channel from outlet to stream

Figure 4-11: Flap valve

4.2.4 Channel Network

Black Creek can be broken down into six distinct channels, as shown in Figure 4-3. These channels consist of large altered channels and thus are relatively consistent shaped with long reaches. Within the model these channels are represented as river reaches, each with at least 2 cross sections and two bank lines.

The final 1D network comprises of 93 river reaches with 238 cross sections. All but 6 were created using the ground model with vertices along the sections specified every 0.5m. The remaining 6 were extracted from the Greater Wellington MIKE 11 model for the downstream section of Main Road Bridge.

Each river reach uses two bank lines to connect to the adjacent 2-D zone. The bank lines have been generated from the ground model with vertices spaced every 2m. The alignments of the bank lines have been digitised to pick up existing stop banks and any features identified during the Stantec staff site visits. The discharge co-efficient for the bank lines have been set at 1.0, representing the lower end of the range suggested for maintained grass embankments. The modular limits for the bank lines have been set at 0.9, the ICM default.

Twenty channel crossings have been schematised in model, including four road bridges (two of which have been represented as culverts), nine culverts and seven pedestrian bridges. At four of the culvert inlets there are weirs present. These have been represented in the model using weir links immediately upstream of the culvert inlet links.

Three in-stream weirs were identified during site visits by Stantec. The locations of these weirs are shown in Figure 4-3. The weirs have been included in the channel network using weir links between two break nodes. Storage polygons with no specified storage have been used to exclude the weir locations from the 2-D mesh

Six confluences/junctions between streams have been included in the 1-D channel network. These been modelled using storage nodes, and an associated storage polygon, to connect two upstream river reaches to a downstream reach. The storage polygons have been used to exclude the junction locations from the 2-D mesh where it can be included in the river reach boundaries. Storage values have been calculated for the storage nodes, so all channel storage is included in the model.

At the two largest confluences, Black Creek and Park Drive and Park Drive and Konini Drive, head losses have been estimated using headless coefficients based on the recommended coefficients for pipe networks. This is likely to be a conservative approach but does allow the impact of possible energy losses at the major confluences to be considered.

4.2.5 2D Network

The extent of the 2-D network is shown in Figure 4-3. The 2-D zone developed for the model is approximately 5.85km2 and covers the majority of the floor of the Wainuiomata Valley. The maximum triangle size for the TIN mesh has been set to 10m2 as per the WWL modelling specifications at the time of model construction, with a minimum area of 2m2.

Roughness zones have been used to vary surface roughness across the 2-D zone. A GIS layer of the footprints of buildings in Wainuiomata has been used to define buildings within the TIN by increasing the roughness to create a resistance to overland flow at the location of a building. Approximately 8500 building footprints have been applied the model. Just over half of the building footprints have been included in the mesh, that is, have defined boundaries in the generated TIN mesh. The decision of whether to include a roughness zone in the TIN mesh has been based, at least initially on the extent of flooding seen in the RFHA carried out by WWL.

Buildings located in areas that appear to be at risk of flooding have been included in the mesh generation process; while building located outside of these have been excluded. Figure 4-12 below, shows the distribution of the buildings to be excluded/to be included zones. The intention of excluding selected roughness zones is to reduce the number of small triangles generated in the smaller gaps between buildings in the model extent.

Roads, carparks, and recreational areas/playing fields are represented in the model using roughness zones. These roughness zones were excluded from the mesh generation process as seen in Figure 4-12.

A GIS layer of the kerb lines has been provided by Wellington Water. These have been included as break lines during the generation of the TIN mesh to improve the definition of the roads in the TIN.

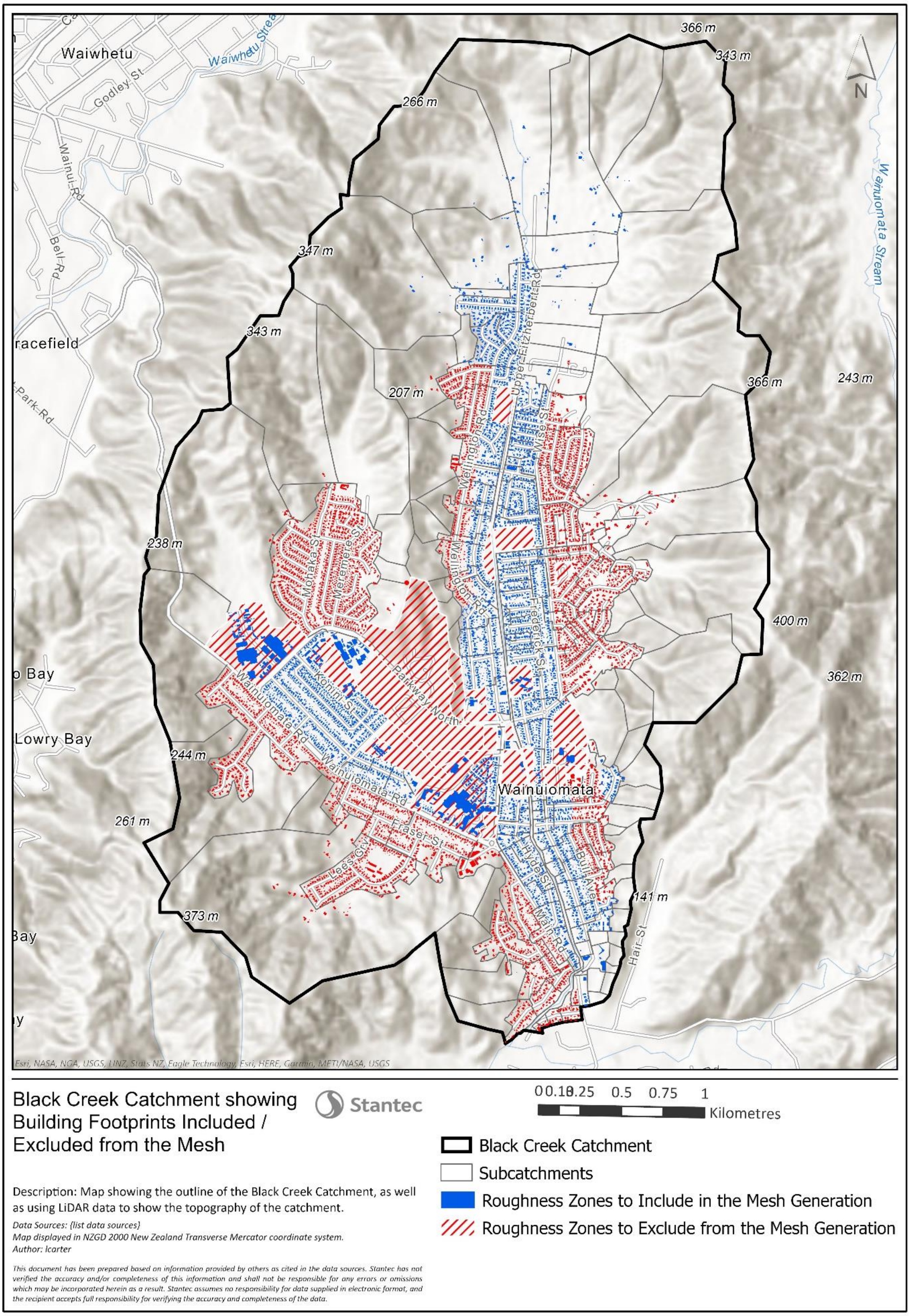


Figure 4-12: Building footprints

4.2.6 Energy Losses

4.2.6.1 1-D Pipe Network

Energy losses in the 1-D pipe network due to surface friction have been accounted for using the Colebrook-White Ks Roughness Values specified in Table 4-3 taken from the Regional Specification.

Table 4-3: Typical 1-D Colebrook-White Ks Roughness Values

Classification	Colebrook-White Ks Values
Pipe Material	
Vitreous Clay	1.5
Precast Concrete Pipe	1.5
Cast Insitu Concrete	3.0
PVC / PE	0.6
Corrugated Aluminium, PE or PP	30

To account for energy losses due to turbulence in the pipe network, headloss coefficients have been set using the ICM Inference tool. No benching information for the manholes was available at the time of the model build so full benching has been assumed initially for the calculation of the headloss coefficients. River reaches have a headloss applied at the junctions. Information was not collected during the review of the as-built plans or the manhole inspections due to time and budget constraints.

Some culverts between river reaches have been given Manning's n values to allow variable roughness. In most cases the top of the culvert was set with a lower manning's n, and the bottom with roughness representative of a stream bed.

4.2.6.2 1-D Channel Network

Energy losses in the 1-D channel network due to surface friction have been accounted for using the Manning's n Roughness Values specified in Table 4-4.

Table 4-4: Typical 1-D Mannings n Roughness Values

Classification	Manning's n Values
Open Channel	
Straight uniform channel in earth and gravel in good condition	0.0225
Unlined channel in earth and gravel with some bends and in fair condition	0.025
Channel with rough stony bed or with weeds on earth bank and natural streams with clean straight banks	0.03
Winding natural streams with generally clean bed but with some pools and shoals	0.035
Winding natural streams with irregular cross section and some obstruction with vegetation and debris	0.045
Irregular natural stream with some obstruction with vegetation and debris	0.06
Very irregular winding stream obstructed with significant overgrown vegetation and debris	0.1

All the channels in the network have been assigned a Manning's n value of 0.03. This is consistent with the previous HEC-RAS modelling carried out by AECOM. The guidance provided in the ICM help section has been used for inlet and outlet losses at the ends of pipes depending on the type of inlet or outlet (based on assessment of site photographs). Expansion and contraction losses have been calculated by ICM during the construction of each bridge element.

4.2.6.3 2-D Network

Energy losses in the 2-D network due to surface friction have been accounted for using roughness zones to set Manning's n Roughness Values specified in Table 4-5 below. The values are based on suggested values in Table 5.10, pg. 42 of the Regional Specification. In areas where there is no specified roughness zone, an intermediate value between unpaved/grass and gardens/dense vegetation was used to represent both land uses. Paved roads were set as 0.013 as roads are usually well maintained and smooth. Car parks were assumed to be poorly maintained or have plantings and dividing barriers in them so a roughness value of 0.02 was adopted.

Table 4-5: Manning's Value Applied to the 2D Zone of the Black Creek Hydraulic Model

Classification	Manning's n Values
Default value used for 2-D zone	0.06
Building	0.5
Road	0.013
Car Park	0.02
Recreational Area/Playing Field	0.03

4.3 Rainfall

The rainfall for the original model build was completed by Stantec using a 24hr nested rainfall profile, however, this was updated by Wellington Water, which Stantec reviewed. The update changed the profile to a 12 hour design (nested) storm.

Design rainfall hydrographs for both existing and future climate in the Black Creek catchment were created based on HIRDS v3 data following the Regional Specifications.

An aerial reductions factor of 0.97, as prescribed in Table 5.2, pg. 26, of the Regional Specification was applied to the rainfall data as, at least for the design events, a single rainfall record will be applied to all model subcatchments (with a total area of 16.5km2).

Rainfall depths from both existing and future climate scenarios (future equates to 20% additional rain) were taken from Hirds v3 from the approximate centroid of the Black Creek catchment as per the specifications laid out by the Regional SW Specifications

Table 4-6: Adopted HIRDS v3 Design Rainfall Depths for the Black Creek Catchment with Aerial reduction applied- existing climate

ARI	Duration									
	10m	20m	30m	60m	2h	6h	12h	24h	48h	72h
2	7	10.2	12.6	18.3	26.9	49.3	72.2	105.8	134.5	154.8
5	8.9	12.9	16	23.3	34	62.2	91	133.1	169.2	194.7
10	10.4	15.1	18.7	27.2	39.8	72.5	106	154.9	196.9	226.7
20	12.1	17.5	21.8	31.6	46.2	84.1	122.8	179.2	227.8	262.2
50	14.7	21.3	26.5	38.5	56.1	101.9	148.5	216.4	275.1	316.6
100	17	24.6	30.7	44.5	64.8	117.6	171.2	249.2	316.8	364.6

To test the validity of HIRDS design rainfall, a frequency analyses of the long-term record at the Wainuiomata at Reservoir rain gauge and the shorter record at Wise Park was carried out. The results of the frequency analyses were compared with HIRDS v 3 rainfall depths. The outcome of the assessment confirmed that HIRDS v3 provides appropriate design depths for the Black Creek catchment. Details of the analysis are included in Appendix A .

A comparison of design rainfall adopted for this assessment with previous depths used by AECOM in 2004 was also undertaken and is included in Appendix B

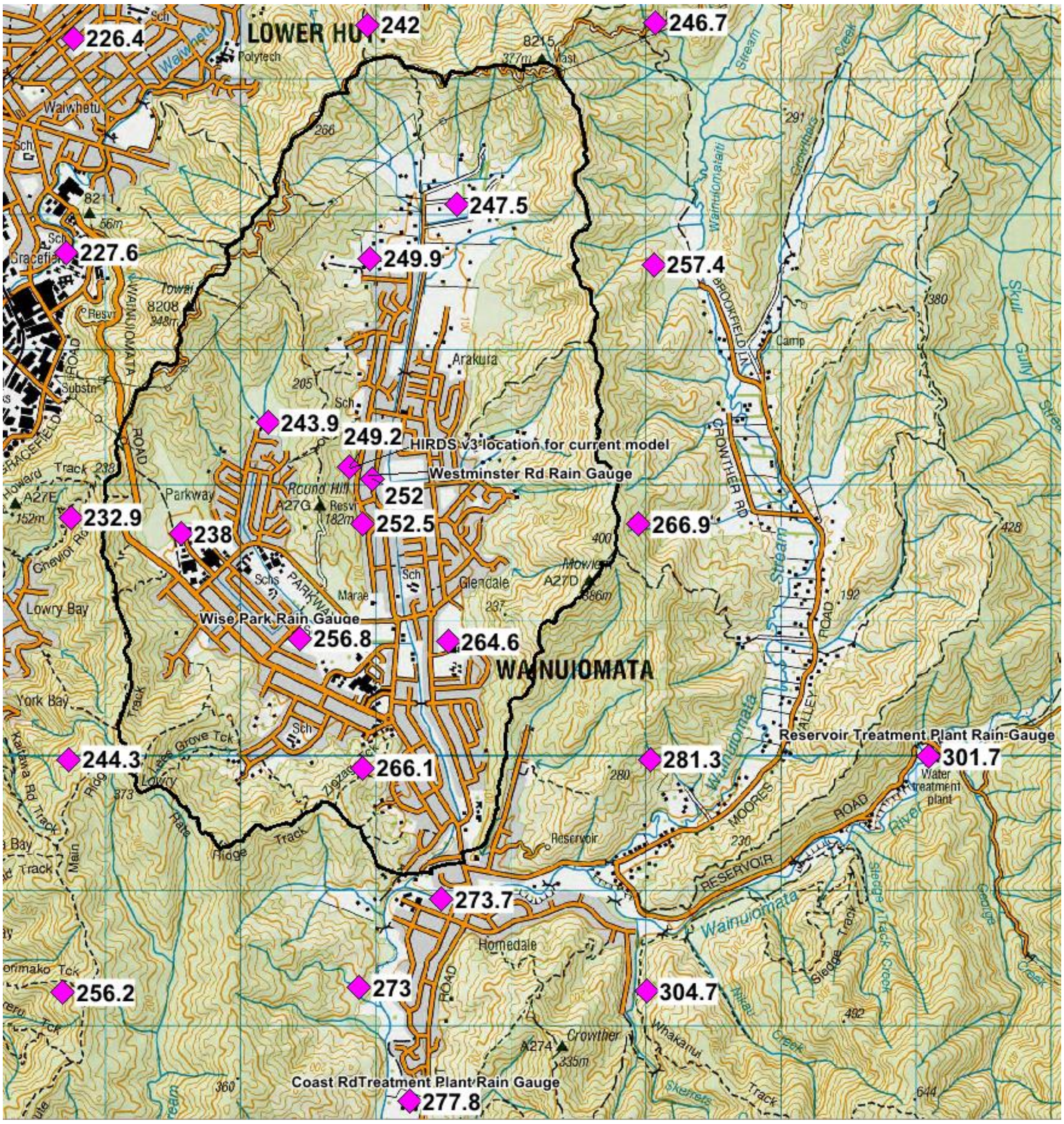


Figure 4-13: Spatial variation of HIRDS v3 100yr ARI 24hr rainfall across the Wainuiomata catchment

4.3.1 Design Rainfall Temporal Pattern

A comparison of several temporal profiles was undertaken to select an appropriate profile for the Black Creek catchment. These are listed in Table 4-7.

After consideration of the four alternatives, it was agreed with Wellington Water to adopt the HIRDS nested profile. Figure 4-14 shows an example of the adopted profile.

A nested 12-hour storm profile based on HIRDS v3 data was adopted as the temporal pattern for the design rainfall events. This method is recommended in the Regional Specifications.

The nested profile maximises rainfall intensities by incorporating selected short duration totals within those needed for longer durations at the same probability level. This method uses all durations for a given ARI without altering the original HIRDS v3 intensities.

Table 4-7: Summary of Temporal Profiles Considered

Profile	Source	Comment
TP108 Nested	TP 108	Based on the Chicago Method and tested in the Auckland Region. Produces peak 10-minute intensities 98% greater than the HIRDS nested method.
HIRDS Nested	HIRDS v3	HIRDS v3 IDF table compiled into symmetrical 5-minute durations.
Tomlinson	Waters of NZ, 1992	Averaged profile based on 17 extreme rainfall events recorded in the Wellington region between 1950 and 1979.
Average Variation	Pilgrim et al, 1969	Averaged profile method based on at site recorded events.

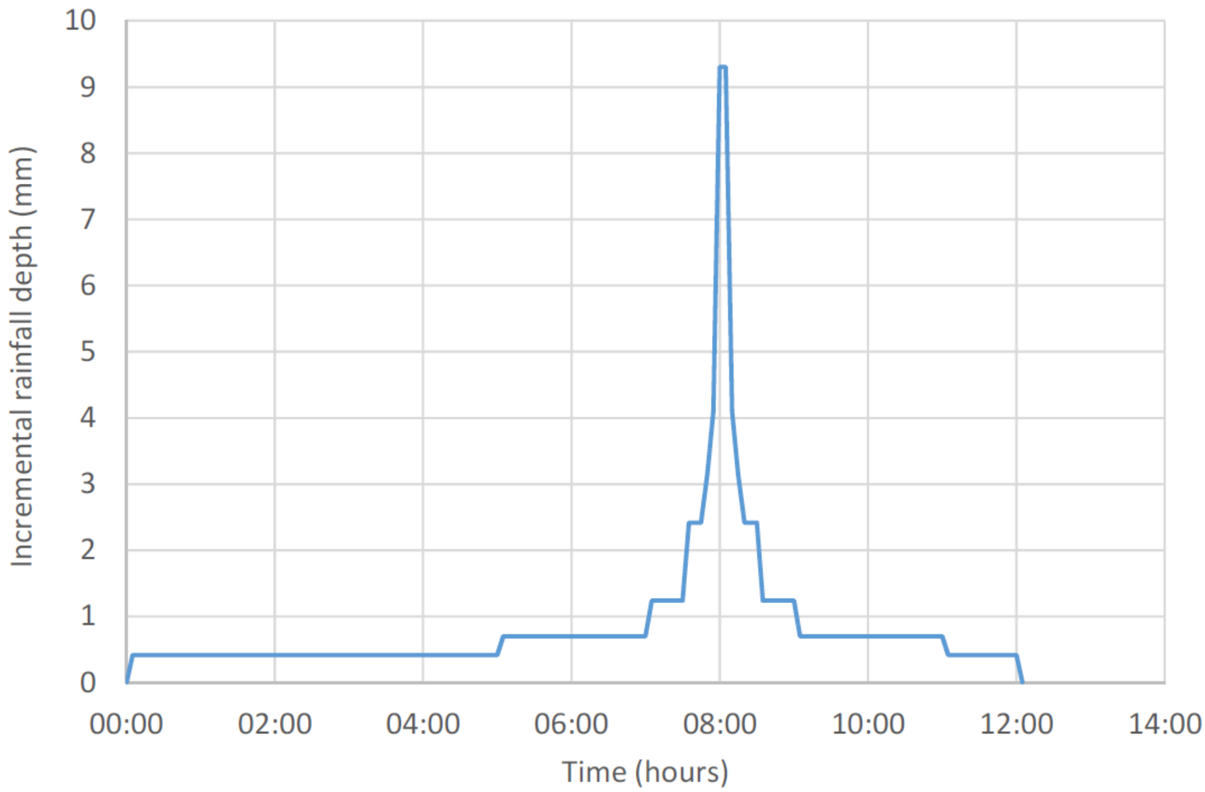


Figure 4-14: 12hr nested profile, example from (Cardno, 2016)

4.4 Boundary Conditions

A single outflow boundary for the 1-D channel network has been included at the confluence of Black Creek and the Wainuiomata River, approximately 0.65 km downstream of Main Rd Bridge. A assessment was conducted as part of the sensitivity process which showed that the levels in the Wainuiomata River had negligible impact on levels in Black Creek upstream of Main Rd Bridge. Thus, only a normal boundary was used. Adopting a normal condition boundary resulted in less instability in the model and did not require additional consideration of the coincidence of peak flows in Black Creek and Wainuiomata River.

Details of the sensitivity analysis are provided in Section 6.

4.4.1 2-D boundary

A normal boundary condition was for the 2-D zone. A normal boundary allows water to leave the model freely, acting in a similar way as the transition of flow from one cell to another.

4.5 Model Limitations and Assumptions

4.5.1 Model Limitations

Computational models are only as accurate as the inputs and the data available for validation. Wellington Water was the primary source of information for the project, LiDAR, recorded rainfall, recorded stream data, and the scanned as built plans. Stantec, supplied surveyed information from a series of manhole inspections.

The constraints and the limitations of the stormwater flood model are as listed:

- Only selected key stormwater lines were modelled. The selected lines were the inlets and outlets that inflow and discharge into the Black Creek and Parkway drains directly.
- A significant amount of the network has not been included in the model due to a lack of asset information.
- Manhole and pipe invert levels have been interpolated from available or surveyed data for a substantial amount of the network.
- Large parts of the model are based on LiDAR. Where the quality of the LiDAR is suspect or there have been changes made since the collection of LiDAR, the model will not represent the real life structures.
- Building floor levels are not applied.
- Only one trash screen has been included in the model for the entire catchment (SC2_us_b).
- Cross sections for the open channels have been generated from the LiDAR.

Limited coverage of the modelled stormwater network means that any overland flow that is occurring due to under capacity portions of the network have not been included into the model will not be evident in the results, meaning some flooding estimates may be marginally underestimated.

4.5.2 Model Assumption

4.5.2.1 Hydrological Model Assumption

Subcatchment inflows have been assigned to single manholes rather than distributed across all manholes within a subcatchment and /or slumps in the subcatchment. This could result in surcharged nodes due to focussed loading.

Rainfall has been applied as a nested 24hr event with an area factor incorporated, for all event magnitudes and distributed evenly across the Black Creek Catchment. This is not an accurate representation of a real-world event, creating possibly conservative peak flows. If more detailed results are required, spatially varied rainfall can be incorporated.

The Black Creek catchment possess short and sometimes inaccurate water level and rainfall information, which may result in the catchments response to the rainfall event differing from those modelled.

4.5.2.2 Hydraulic Model Assumptions

The following assumptions were made in the development of the Hydraulic model.

- The LiDAR generated ground model is an accurate representation of catchment topography.
- Manholes lid levels are adequately represented in the ground model.
- A constant gradient has been applied between known invert levels and those requiring interpolation. Exceptions to this are where there are significant changes in ground level gradient, or where a constant gradient would cause the pipe to be set to an invert above ground level. In these cases, manual adjustments to pipe and manholes levels have been made.
- At junctions, where a smaller diameter pipe discharges into a larger diameter pipe, the pipes were laid soffit to soffit.
- Manholes were assumed to be fully benched.
- No sediment was added to the pipes. There are two culverts exempt from this, under road bridges.
- Peak outflow from the Black Creek catchment occurs during the rising limb of the flood hydrograph in the Wainuiomata River.
- The stage-discharge relationship provided by Wellington Water for Main Rd Bridge is adequate for use.

4.5.3 Model Testing

The model was built using aerial photography, topographic information and GIS layers provided by Wellington Water. The stormwater network has been based on a combination of the Hutt City Council GIS network assets and a InfoNet files generated by Wellington Water. The Extent of the 2D simulation polygon (2D Zone) was defined based on the topography of the Wainuiomata Valley, with attention to covering the low-lying areas and extended upstream of all the branches of the pipe network.

4.5.3.1 Instability Tests

Initial model testing was carried out using two events:

- Baseflow, with no rainfall
- The 100yr ARI nested design storm applied to the WWL RFHA model.

The initial model tests showed some instabilities were being generated from the level boundary applied to the downstream end of the model network (the confluence with the Wainuiomata River) and at some of the links between the 1D and 2D networks at road crossings. To reduce the instabilities at the downstream boundary a normal condition boundary has been adopted following an investigation into the impact of water levels in the Wainuiomata River Appendix G

At the road crossings, time was spent adjusting the 1D and 2D components to provide an accurate and smooth transition between the networks.

4.5.3.2 Sensibility Check

A sensitivity check was carried out as part of the model verification process as established in Section 4.3. A 20yr ARI event was simulated with the existing case network in current climate conditions. The results were then compared against the extents in the GIS data provided by WWL, See Section 5.6. The results compared relatively well with the reported flood locations (Figure 4-15 & Figure 8-11) which are listed in Appendix C

4.5.4 Mass Balance Checks

By default, the ICM simulation engine undertakes mass balance checks at every simulation time step. If the cumulative Mass Balance error exceeds 0.01 m^3 any individual time step the simulation is automatically terminated (OPUS, 2015). This implies that any simulation that is completed is considered to have passed this check.

Volume balance information for each simulation is provided in the simulation log file. The volume balances for the two simulations used for initial testing are provided below (Table 4-8

Table 4-8: Volume Balance Summary for Initial Simulations

Design Storm Event	2D Volume Balance	
	Mass Error Balance (%)	Total Mass Error (m^3)
Baseflow – no rain	0.0	0.0
100yr ARI	0.0002	25.592

4.5.4.1 Quality assurance and Quality Checks.

An internal model review was completed, and any issues raised in the review process were investigated and addressed. WWL were provided multiple drafts of the modelling approached and any issues raised were addressed. The current model is deemed to be fit for the purpose of providing a preliminary assessment of network performance and investigating the impact of development in the upstream catchment.

4.5.5 Model Results

4.5.5.1 Discussion

The results of hydraulic modelling using the original (pre validation) and validation hydrology are fairly variable when compared against the recorded data for the selected validation events. The original hydrology for the May 2011event significantly overestimates peak flow at Main Rd Bridge and peak depth at both validation locations. The validation hydrology generates peaks within roughly 5% of the recorded data at both locations and the differences in flow, depth and timing are well within the WWL guidelines for calibration acceptance criteria of ±15% and ±1hr, respectively. Greater significance has been given to the results of the May 2011 validation event as this the largest event available for use, and is above the desired 5yr ARI, for use in model validation (as given in the WWL Regional Specification), and because of the available gauge corrected rain radar data. As a result, the adopted (validation) hydrology has been based on parameters used to generate the best fit to this event. It should be noted that the use of only one event provides a limited validation of the model, particularly for the longer acting hydrology elements such the ground infiltration component.

Two additional validation events were also simulated, however, due to limited rainfall data available for the events, and the low magnitude, little significance have been given to their results. It is recommended that further validation work is completed when additional record with larger magnitude events are available. The use of Gauge corrected rainfall radar for the additional validation work is highly recommended.

The original hydrology for the February 2012 and January 2013 events generally provides a closer fit for the peak depth, while the validation hydrology generates flows and hydrographs more similar in shape to the recorded data. This difference indicates the model channel cross-sections at the recorded locations may not be an accurate representation of the actual channel near the stream bed (at lower flows). The modelled channel sections have been cut using the ground model generated from the 2013 LiDAR survey. There are number of possible reasons why the LiDAR does not provide a good representation of the base of the Black Creek channel, however, the narrow low flow channel (resulting in poor coverage in the LiDAR points), poor representation of ground levels below water, and vegetation in and beside the active channel are likely to be the leading causes. The model results indicate the flow-depth relationship improves for the channel as flow increases, so it is considered that the model cross-sections are adequate for the current purposes of the model.

Rainfall for the February 2012 and January 2013 events was only available at two-point locations within the catchment. It is also very likely that the poor fit of the validation plots to the recorded data is a result of “missing rainfall” on the catchment

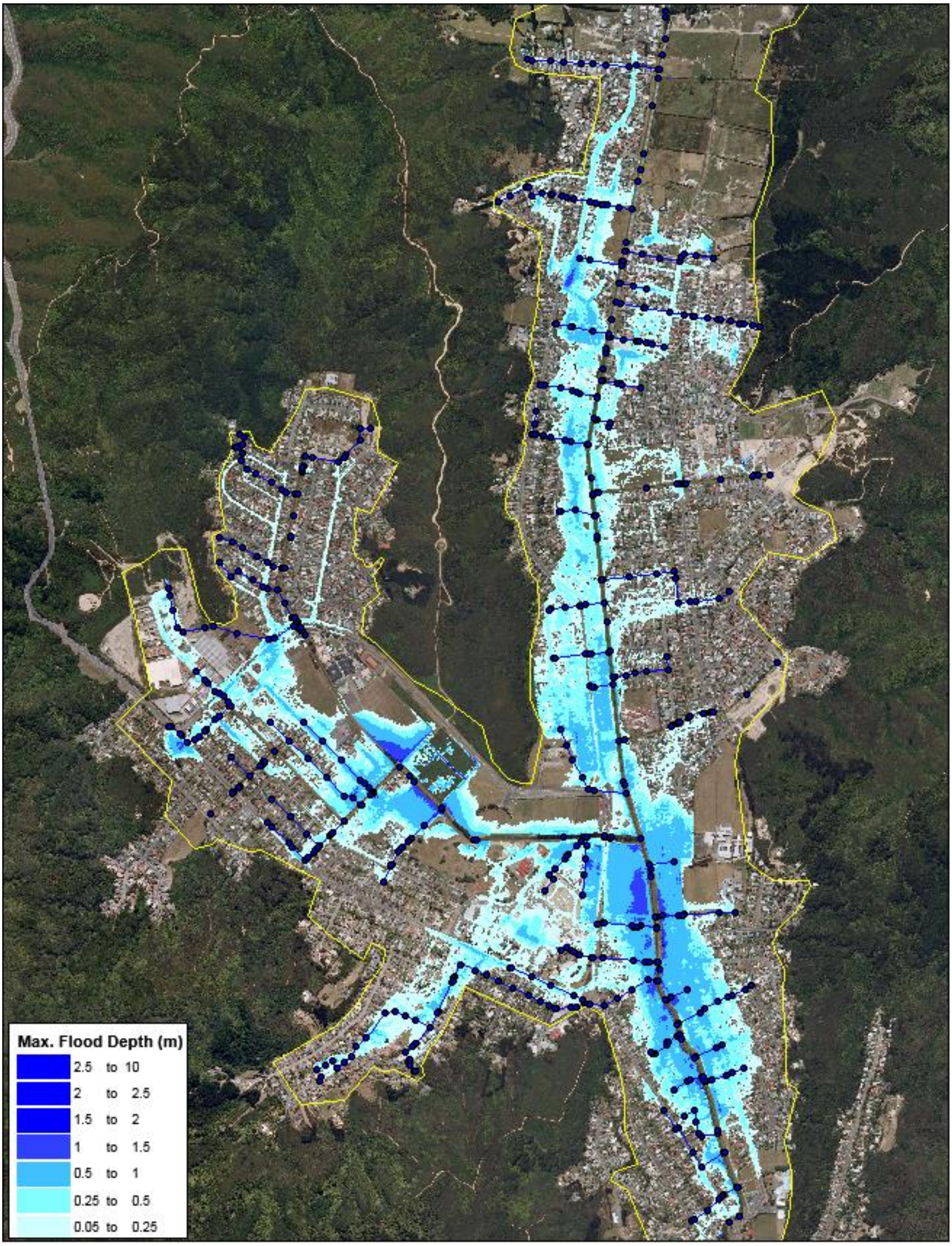


Figure 4-15: Maximum flood extent for the RFHA 100yr ARI event applied to the current 1D/2D hydraulic model

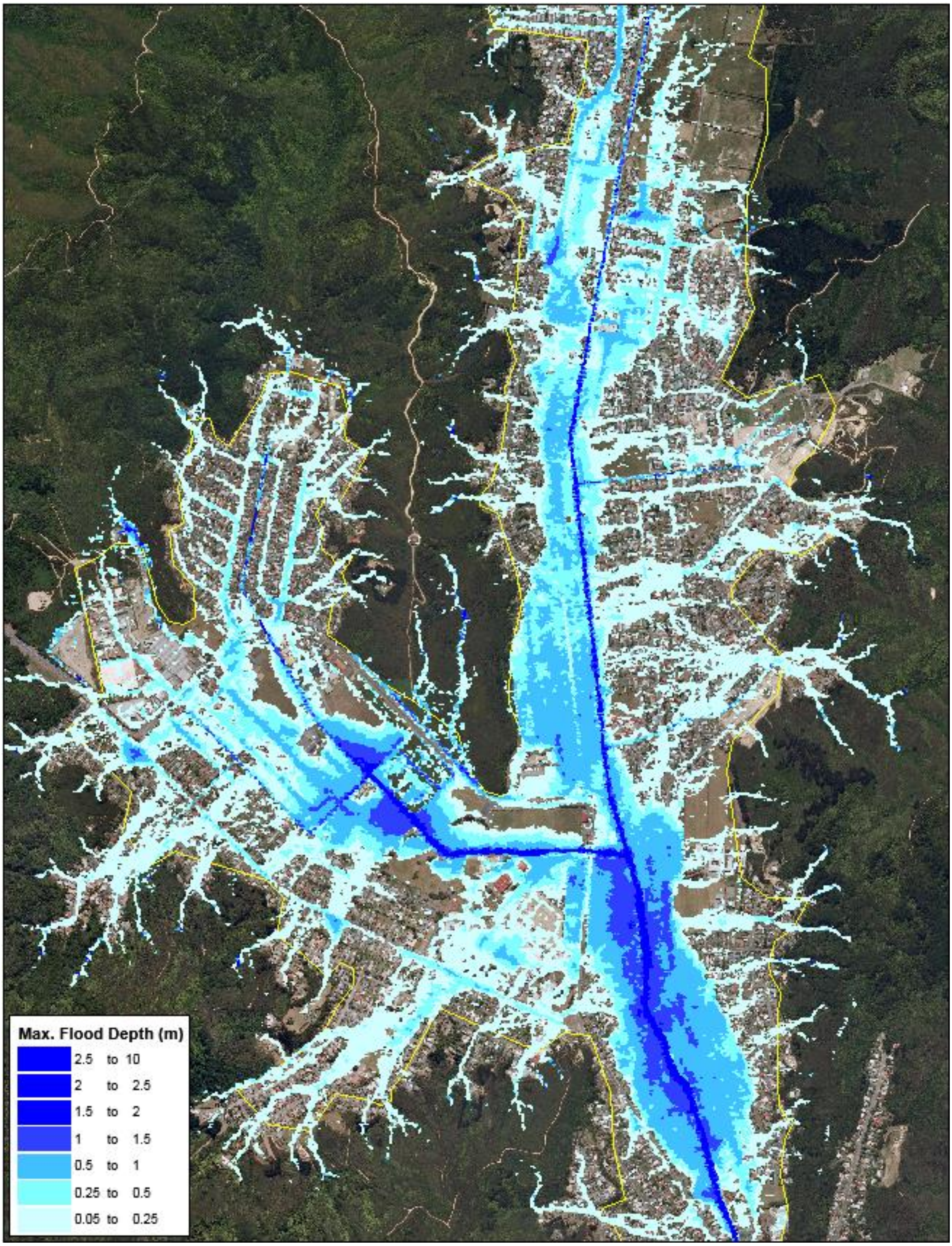


Figure 4-16: Maximum flood extent for the RFHA 100yr ARI event applied to the RFHA 2D hydraulic model

5 Validation

The work in this section was completed prior to the updates completed by Wellington Water. A review was undertaken after the updates as a sensitivity/ sense check to see if the validation had to be redone as seen in section 5.5.

The Black Creek Hydraulic model was validated by using recorded rainfall and water levels at the Wellington Road Bridge gauge for three flooding events. The AECOM MOUSE-HEC-RAS model and the Rapid Flood Hazard Assessment were also used for comparison for the hydraulic model.

Two separate model networks are associated with the validation stage. One for the 2011 event, which includes updates after the main validation works, and the 2012 and 2013 events used the old another for the remainder of the validation runs. “Black Creek Base Scenario April 19” and “Black Ck Validation RR” respectively. These rainfall events were chosen as they were recent events at the time of the validation and had observed flow data for comparison.

Model validation results are reported as “original” hydrology for the hydrological model with initial default parameters and as “validated” hydrology for the hydrological model with updated parameters based on the validation process.

Water level has been recorded at two locations on Black Creek since 2010. At a pedestrian bridge adjacent to McKay St called the Wellington Road Bridge recorder and at the Main Road Bridge seen in Figure 5-1. Flow data is also available for the Main Road Bridge recorder using a rating curve developed by AWT. Further details for these recorders are provided in Section 3.1.4.2 of this report

Figure 5-1 Location of the water level recordings

5.1 Hydrology Update.

In 2015 Stantec obtained recorded flows and levels at the Main Rd Bridge from Capacity (now Wellington Water) for the May 14th – 15th flood and compared them with simulated results for the event. Of interest are the 'Record' and 'Modelled – Validation Hydrology' profiles.

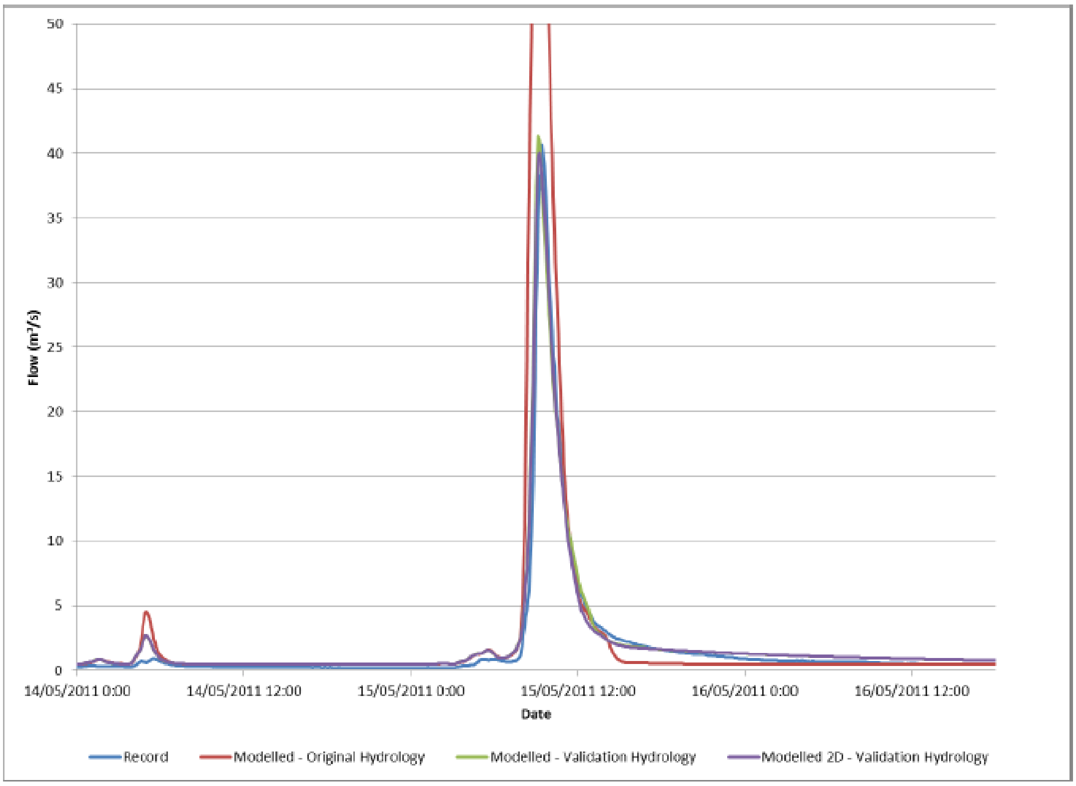


Figure 5-2 Flood hydrograph at Main Rd Bridge for the recorded and modelled May 2011 event. Note the profiles for Record and Modelled – T

Similar to previous findings, flows at the Main Rd Bridge recorder align well with results from the current, updated, model and recorded flows, as shown in Figure 4.16. Modelled flows at this location peak at 41.5 m^3/s where recorded data peaks at 40.6 m^3/s. This is a minor increase in modelled flow as a result of the hydrology update. Modelled flows from simulations carried out in 2015 peaked at 41.3 m^3/s. Due to the comparable results between pre-hydrology update flows and post-hydrology update flows, no further validation was considered necessary at this stage (largely due to the extensive model calibration undertaken in 2015 which examined a variety of parameters, detailed in the original model build report (MWH, 2015)).

Table 5-1: Recorded and modelled flows at Main Rd Bridge for the May 2011 event

Simulation	Maximum Flow (m^3/s)	Time of peak flow
Recorded Data	40.6	15/05/2011 9:25
2015 Model – pre-hydrology update	41.3	15/05/2011 9:10
2019 Model – post hydrology update	41.5	15/05/2011 9:10

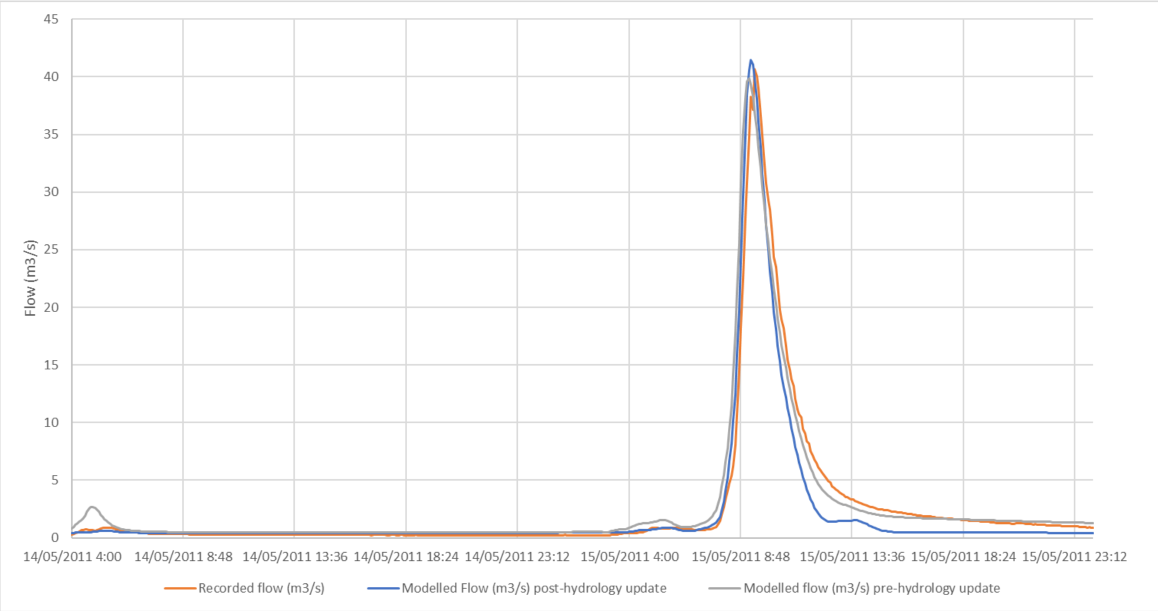


Figure 5-3: Main Rd Bridge flows (m^3/s) - Modelled pre-hydrology update vs modelled post-hydrology update vs recorded flows

Three rainfall events were selected from the Wise Park rain gauge and Westminster Road rain gauge. Table 5-2 provides details for the selected events

Table 5-2: Shows the Results from the Validation Events. Please Note the May 2011 has the Preferred ARI as per WWL Regional Specifications

	Wise Park Rain Gauge			Westminster Road Rain Gauge		
Date	Total Depth (mm)	Peak intensity (mm/hr)	Peak Intensity duration/ARI based on Hirds v3	Total Depth (mm)	Peak intensity (mm/hr)	Peak Intensity duration/ARI
14 – 17 May 2011	46	37.6 26.6	30min/10yr 1hr/10yr	NA	NA	NA
27 Feb – 5 Mar 2012	114.4	6	12hr/2yr	106.6	8.3 6.3	6hr/2yr 12h/2yr
31 Jan – 8 Feb 2013	62	24.2	30min/2yr	46.6	38.4	<1yr

Table 5-2 shows that only the May 2011 event was above the desired 5yr ARI for use in model validation (as given in the WWL Regional Specification).

Westminster Road rain gauge had no recorded data for the 14-17th May 2011 event

Rainfall intensities generated from the rainfall depths recorded during these events have been applied to the hydraulic model to investigate the resultant flow at Main Rd Bridge and water depths at Main Road Bridge and Wellington Road Bridge. The events were simulated first in a fully 1D network (pipes and open channel) for an initial assessment and then in the coupled 1D/2D network. For the 1D network, all manholes with flood type “2D” were set to flood type “Stored” to allow storage in the network due to flooding to be accounted for.

For all the events the model includes the channel widening carried out between 2005 and 2007. The development of the catchments is as described in Section 4.1and represents the stage of development at the time of the LiDAR data collection in 2013.

5.2 February 2012

Recorded rainfall for the February 2012 event was available from the Wise Park and Westminster Road rain gauges. Rainfall was applied to each subcatchment in the model via two rainfall zones. These zones were created by following natural catchment boundaries and topography with reference to the rainfall gauge locations as shown in Figure 5-4.

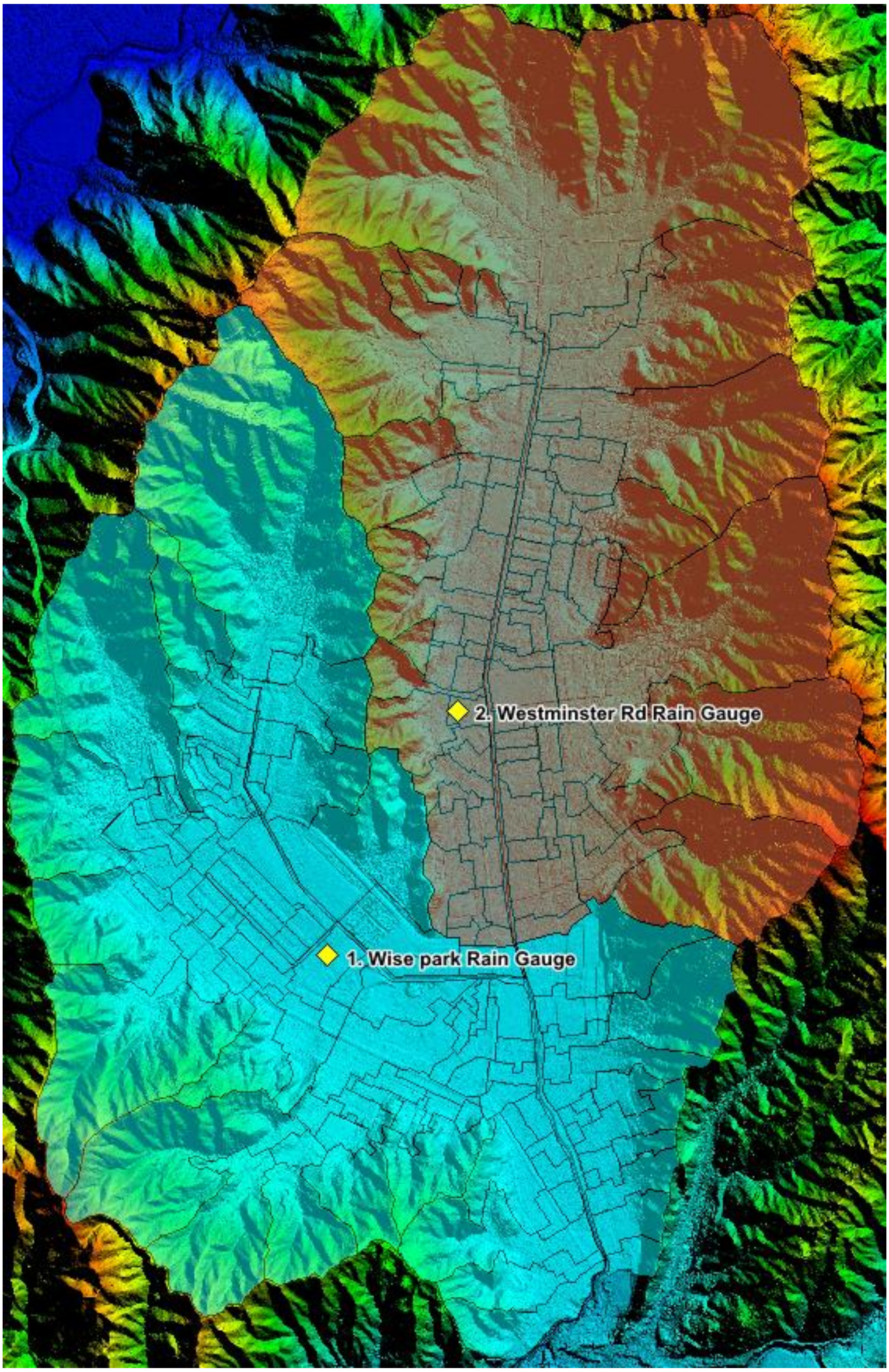


Figure 5-4: Rainfall zones based on the location of the Westminster Road and Wise Park rain gauges.

Due to the low ARI of the rainfall event, the 1D network was only used to complete the simulations. Table 5-3 and Figure 5-5 show the recorded and modelled flow at Main Rd Bridge. Similar to the May 2011 the original hydrology significantly overestimates the peak flow in Black Creek during the event, and while the validation hydrology provides a better fit, the simulated peak flow is not within the required ±15%.

A comparison of the recorded and the modelled depths at Main Road Bridge and Wellington Road Bridge showed that the original hydrology peaks closer to the recorded peaks, see Figure 5-6. Although the plots indicate that the validation hydrology generates a depth profile that is closer in shape to the recorded data.

Table 5-3 Main Road Bridge Model Validation Flow Results - Feb 2012

Simulation	Flow (m^3/s)	% Difference to recorded data	R^2	Time	Recording interval	Time difference
Recorded data	16.2	0	NA	3/03/2012 9:45	5 minute	NA
Original Hydrology – 1D network	32.8	102	0.7112	3/03/2012 7:55:00	5 minute	110 minutes
Validation Hydrology – 1D network	19.4	19.6	0.9108	3/03/2012 9:45:00	5 minute	0 minutes

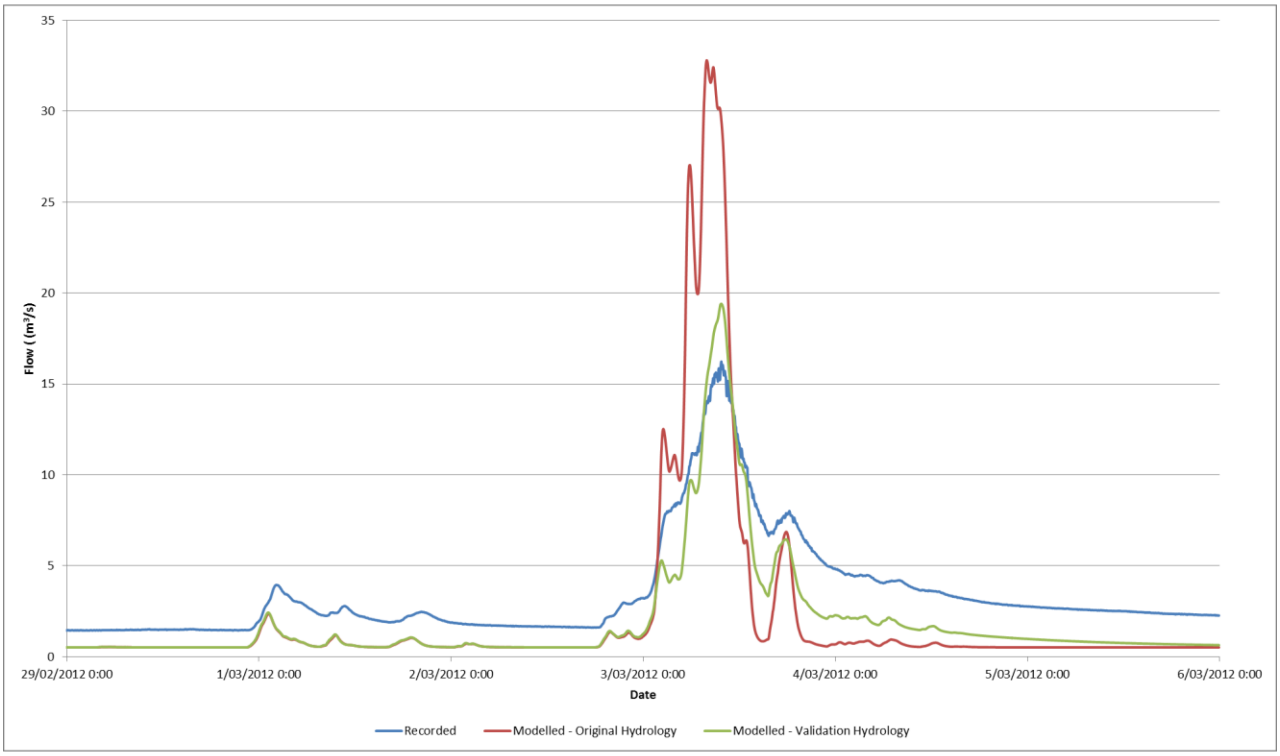


Figure 5-5: Flow-time plot at Main Rd bridge for the recorded and modelled Feb 2012 event

Table 5-4: Main Road Model Validation Depth Results - February 2012

	Depth (m)	% Difference to recorded data	R^2 value	Time	Recording interval	Time difference
Recorded data	2.137	NA	NA	3/03/2012 9:45	5 minute	NA
Original Hydrology – 1D network	2.33	8.94%	0.7948	3/03/2012 7:55:00	5 minute	110 minutes
Validation Hydrology – 1D network	1.77	17.0%	0.8258	3/03/2012 9:45:00	5 minute	0 minutes

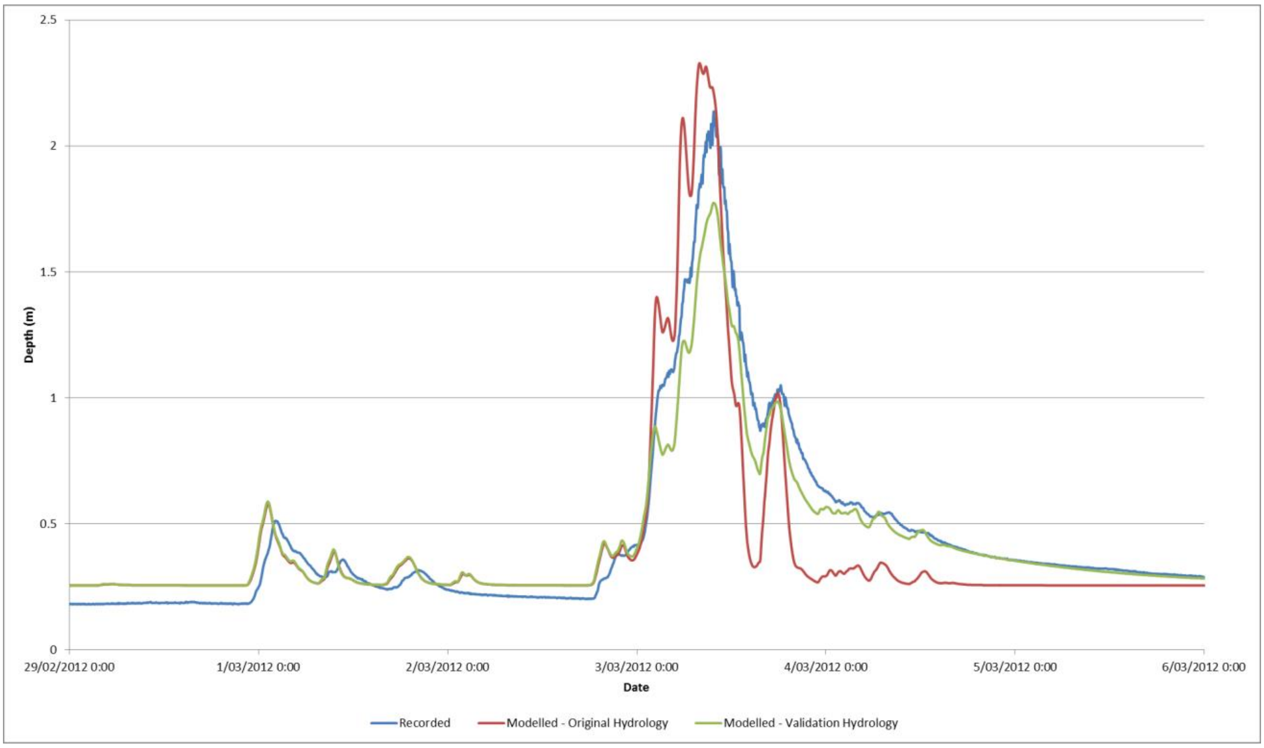


Figure 5-6: Stage-time plot at Main Rd Bridge for the recorded and modelled February 2012 event

Table 5-5: Wellington Road Model Validation Depth Results - February 2012

	Depth (m)	% Difference to recorded data	Time	Recording interval	Time difference
Recorded data	1.74	NA	3/03/2012 9:45:00	30 minute	NA
Original Hydrology – 1D network	1.71	2.0%	3/03/2012 7:40: 00	5 minute	125 minutes
Validation Hydrology – 1D network	1.19	31.6%	3/03/2012 9:40: 00	5 minute	5 minutes

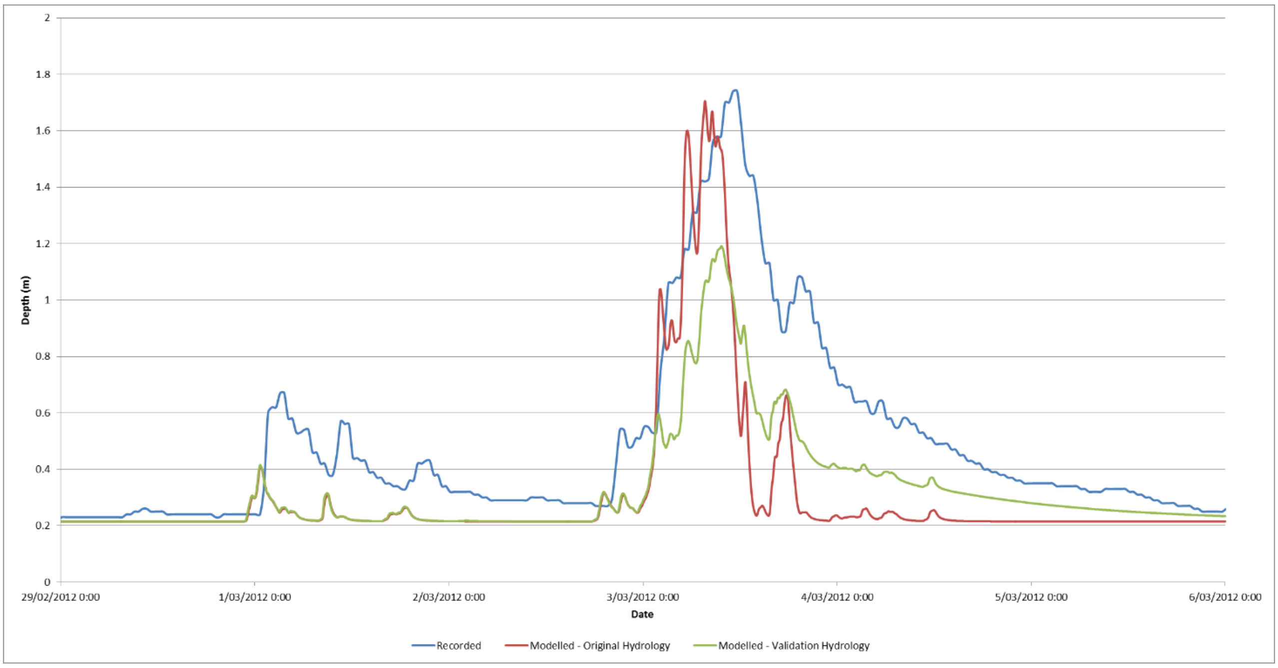


Figure 5-7: Stage-time plot at Wellington Rd Bridge for the recorded and modelled February 2012 event

5.3 January 2013

Recorded rainfall for the January 2013 event was available from the Wise Park and Westminster Road rain gauges. The same rainfall zones as shown in Figure 5-4 for the 2012 event were also used to distribute the correct rainfall to each subcatchment in the model.

The Main Road Bridge recorded flows were used for the validation of the model as, see Table 5-6. As with the previous events the original hydrology significantly overestimates the peak generated during the event. The validation hydrology provides a far better fit for peak flow in Black Creek, however the full hydrographs for the validation and original hydrology do not compare well to the recorded data.

The plots of simulated depths at Main Rd Bridge show a similar result to the flow plots, with validation hydrology providing a better, but not ideal, fit to the recorded data (and Figure 5-8). At Wellington Rd Bridge, neither the original hydrology nor the validation hydrology compare well to the recorded depths at this location (Figure 5-9).

Table 5-6: Main Road Bridge Model Validation Flow Results - Jan 2013

Simulation	Peak Flow (m^3/s)	% Difference to recorded data	R^2	Time	Recording interval	Time difference
Recorded data	16.2	0	NA	3/03/2012 9:45	5 minute	NA
Original Hydrology – 1D network	40.1	195	0.4815	4/02/2013 19:35:00	5 minute	15 minutes
Validation Hydrology – 1D network	14.2	4.0	0.7553	4/02/2013 19:30:00	5 minute	20 minutes

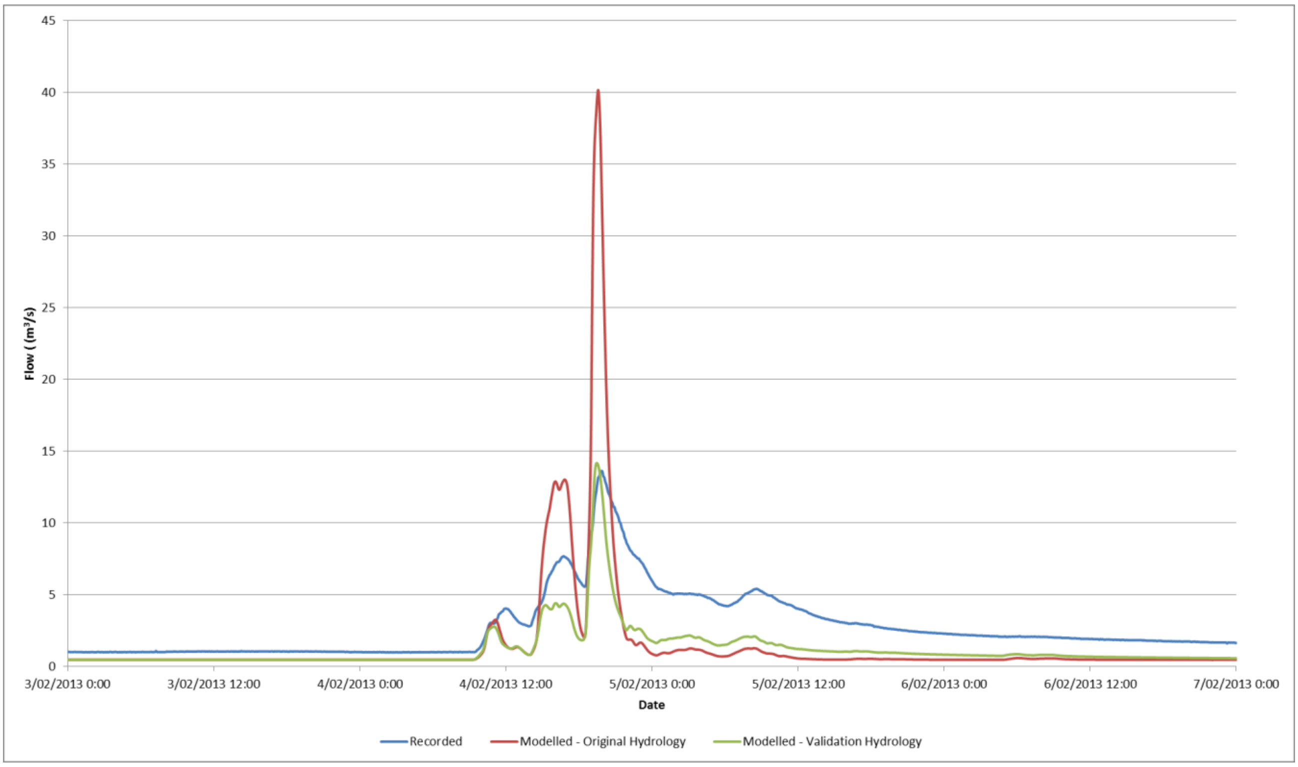


Figure 5-8: Flow-time plot at Main Road bridge for the recorded and modelled Jan 2013 event

Table 5-7: Main Road Model Validation Depth Results - February 2013

	Peak depth (m)	% Difference to recorded data	R^2 Value	Time	Recording interval	Time difference
Recorded data	1.79	NA	NA	4/02/2013 19:50:00 a.m.	5 minute	NA
Original Hydrology – 1D network	2.57	43.8	0.6258	4/02/2013 19:35:00	5 minute	15 minutes
Validation Hydrology – 1D network	1.50	16.4	0.9057	4/02/2013 19:30:00	5 minute	20 minutes

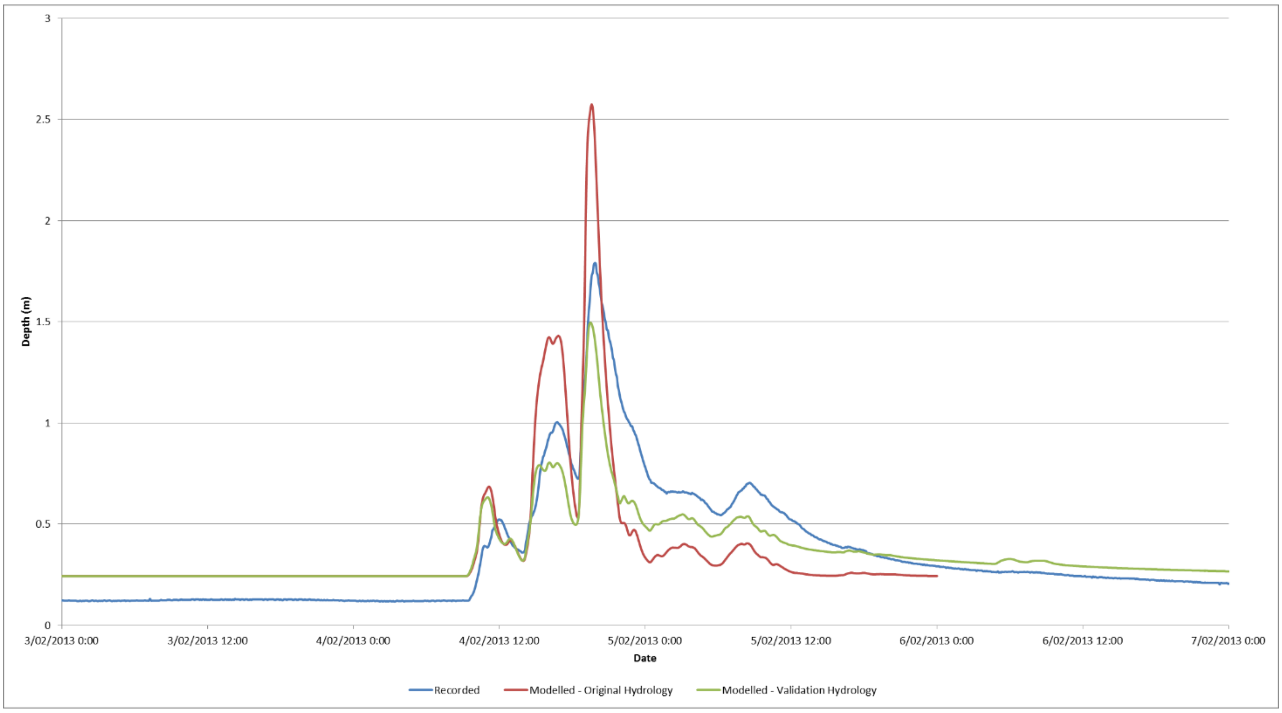


Figure 5-9: Stage-time plot at Main Road Bridge for the recorded and modelled January 2013 event

Table 5-8: Wellington Road Model Validation Depth Results - January 2013

	Peak depth (m)	% Difference to recorded data	Time	Recording interval	Time difference
Recorded data	1.68	NA	4/02/2013 20:38:00	30 minutes	NA
Original Hydrology – 1D network	1.87	11.1	4/02/2013 19:25:00	5 minutes	73 minutes
Validation Hydrology – 1D network	0.98	41.9	4/02/2013 19:30	5 minutes	68 minutes

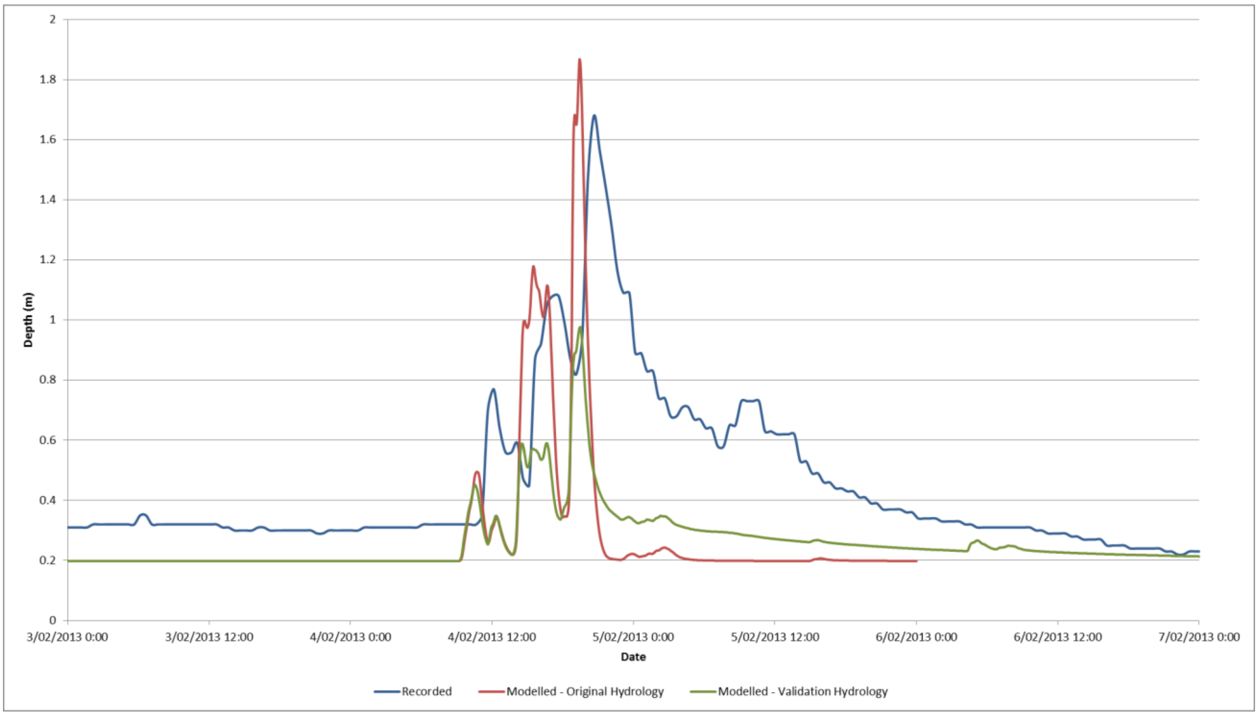


Figure 5-10: Stage-time plot at Wellington Road Bridge for the recorded and modelled January 2013 event

5.4 May 2011

The May 2011 event was only available from the Wise Park rain gauge; however, Wellington Water was able to supply gauge corrected rainfall radar data for this event. A static image of the distribution of maximum 1hr rainfall depths recorded during the event Figure 5-11,was used to develop three rainfall zones Figure 5-12 within which a scaled dataset of the Wise Park rainfall record was applied. These zones, and the scaling factors applied to the Wise Park data, are provided below. A tidal boundary condition was used in this more recent simulation, whereas previous simulations completed earlier in the investigation did not include boundary conditions. Further results have shown that boundary conditions do not have a significant impact on the results.

Table 5-9: Rainfall Zones Developed from Available Gauge Corrected Radar

ID	Zone	Average 1hr maximum depth from GCR (mm)	Scaling factor applied
1	Low	20	1.0
2	Medium	30	1.5
3	High	42	2.1

Three model simulations were run for this event, as follows .

- Wise Park rainfall applied to all subcatchments with scaling factors based on GCR data applied using the original hydrological parameter – 1D network.
- Wise Park rainfall applied to all subcatchments with scaling factors based on GCR data applied using new hydrological parameters and subcatchment layout – 1D network.
- Wise Park rainfall applied to all subcatchments with scaling factors based on GCR data applied using new hydrological parameters and subcatchment layout – 2D network.

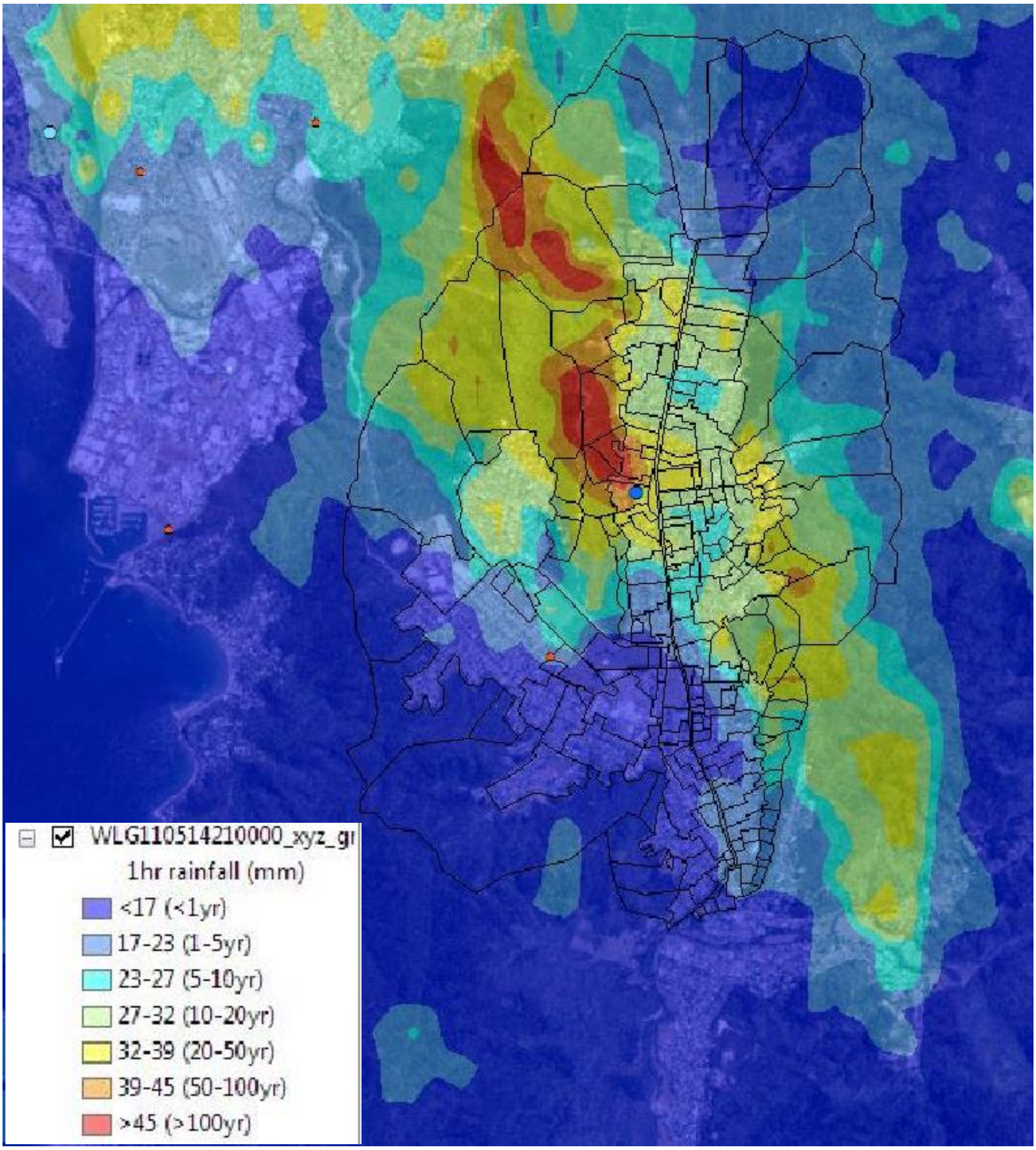


Figure 5-11: Gauge corrected rainfall radar plot of the maximum 1hr rainfall depths recorded during the 14-17 May 2011 event

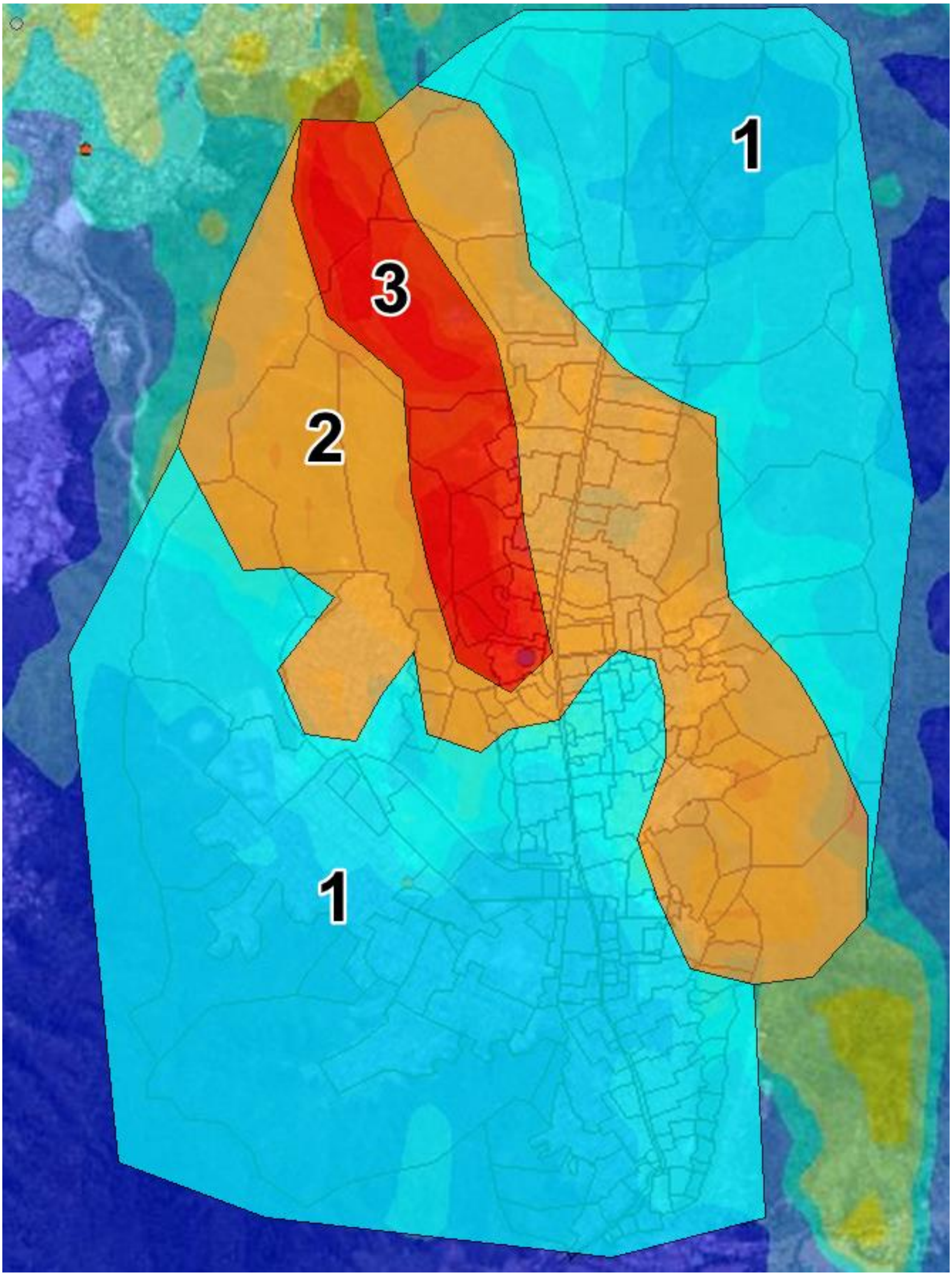


Figure 5-12: Rainfall zones developed from gauged corrected rain radar

Peak flows generated at the Main Road Bridge were used for the validation of the model. As with the previous examples, the original hydrology significantly overestimates the peak flow at the bridge. Therefore, multiple simulations using different parameters for the Runoff Routing value, Horton Limiting value, and Horton Decay value for the pervious surfaces were conducted to generate a hydrograph closer to the recorded data. The final hydrological parameters and results are shown in Table 5-10 to Table 5-12 Appendix B.

A Ground Infiltration Model (GIM) was also included in the final hydrology to improve the fit of the receding limb in the modelled event to the recorded hydrograph. The ground infiltration parameters applied in the model are given in and Table 5-13.

Table 5-10: Main Road Bridge Model Validation Flow Results - May 2011

Simulation	Flow (m^3/s)	% Difference to recorded data	R^2	Time	Recording interval	Time difference
Recorded data	40.6	0	NA	15/05/2011 9:25:00	5 minute	NA
Wise Park Original Hydrology – 1D network	95.1	134	0.8505	15/05/2011 9:05	5 minute	20 minutes
Wise Park Validation Hydrology – 1D network	41.3	1.6	0.9623	15/05/2011 9:10	5 minute	15 minutes
Wise Park Validation Hydrology – 2D network	40.0	1.6	0.9722	15/05/2011 9:15	5 minute	10 minutes

Table 5-11: Impervious Runoff Surfaces

Runoff surface ID	Description	Runoff routing type	Runoff[1] routing value	Runoff volume type	Surface type	Ground Slope (m/.m)	Initial loss type	Initial loss value (m)	Routing model	Fixed runoff coefficient
1	Roads/ Paved	Abs	0.013	Fixed	Impervious	0	Abs	0.0003	SWMM	0.8
2	Roofs	Abs	0.013	Fixed	Impervious	0.05	Abs	0.0003	SWMM	1

Table 5-12: Pervious Runoff Surface

Runoff surface ID	Description	Runoff routing type	Runoff [2] routing value	Runoff volume type	Surface type	Initial loss type
3	Wooded	Absolute	0.1	Horton	Pervious	Absolute
4	Lawn, Paddocks	Absolute	0.1	Horton	Pervious	Absolute

Runoff surface ID	Initial loss value (m)	Routing model	Horton initial (mm/hr)	Horton limiting (mm/hr)	Horton decay (1/hour)	Horton recovery (1/hour)	Initial loss porosity
3	0.002	SWMM	10	2	0.25	1.5	1
4	0.002	SWMM	5	2	0.25	1.5	1

[1] This value is a *Manning's n* corresponding to the surface type
[2] This value is a *Manning's n* corresponding to the surface type

Table 5-13: Ground Infiltration Model Parameters

Ground infiltration ID	Soil depth (m)	Percolation coefficient	Baseflow coefficient	Infiltration coefficient	Percolation threshold	Percolation percentage infiltrating	Porosity of soil	Porosity of ground
10	1	0.75	0.01	0.2	5	30	30	50

Table 5-14 and Table 5-15 provide details for the recorded and modelled depths at Main Rd Bridge and Wellington Rd Bridge. As with flow, the original hydrology generated significantly greater peak depths at both locations, while the validation hydrology provides a reasonably good fit for the peak depth and receding limb of the depth profile (Figure 5-13 and Figure 5-14).

Table 5-14: Main Road Bridge Model Validation Depth Results – May 2011

Simulation	Depth (m)	% Difference to recorded data	R^2	Time	Recording interval	Time difference
Recorded data	2.248	0	NA	15/05/2011 9:25:00	5 minute	NA
Wise Park Original Hydrology – 1D network	3.4	51.9	0.8337	15/05/2011 9:05	5 minute	20 minutes
Wise Park Validation Hydrology – 1D network	2.2	3.8	0.9578	15/05/2011 9:10	5 minute	15 minutes
Wise Park Validation Hydrology – 2D network	2.1	5.4	0.9621	15/05/2011 9:15	5 minute	10 minutes

Table 5-15: Wellington Road Bridge Model Validation Depth Results – May 2011

Simulation	Depth (m)	% Difference to recorded data	Time	Recording interval	Time difference
Recorded data	1.87	NA	15/05/2011 9:08:50	30 minute	NA
Wise Park Original Hydrology – 1D network	3.3	75.0	15/05/2011 8:50	5 minute	18 minutes
Wise Park Validation Hydrology – 1D network	1.9	0.6	15/05/2011 8:55	5 minute	13 minutes
Wise Park Validation Hydrology – 2D network	1.9	1.7	15/05/2011 8:55	5 minute	13 minutes

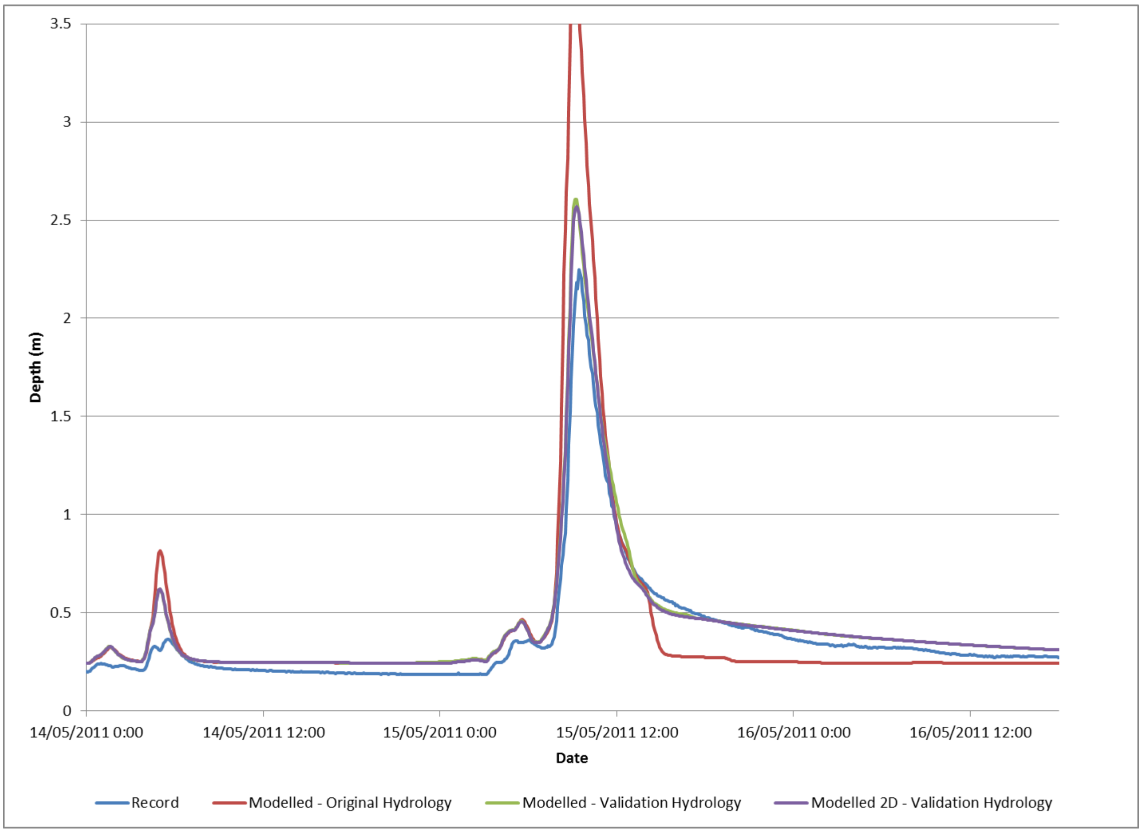


Figure 5-13: Stage-time plot at Main Road Bridge for the recorded and modelled May 2011 event

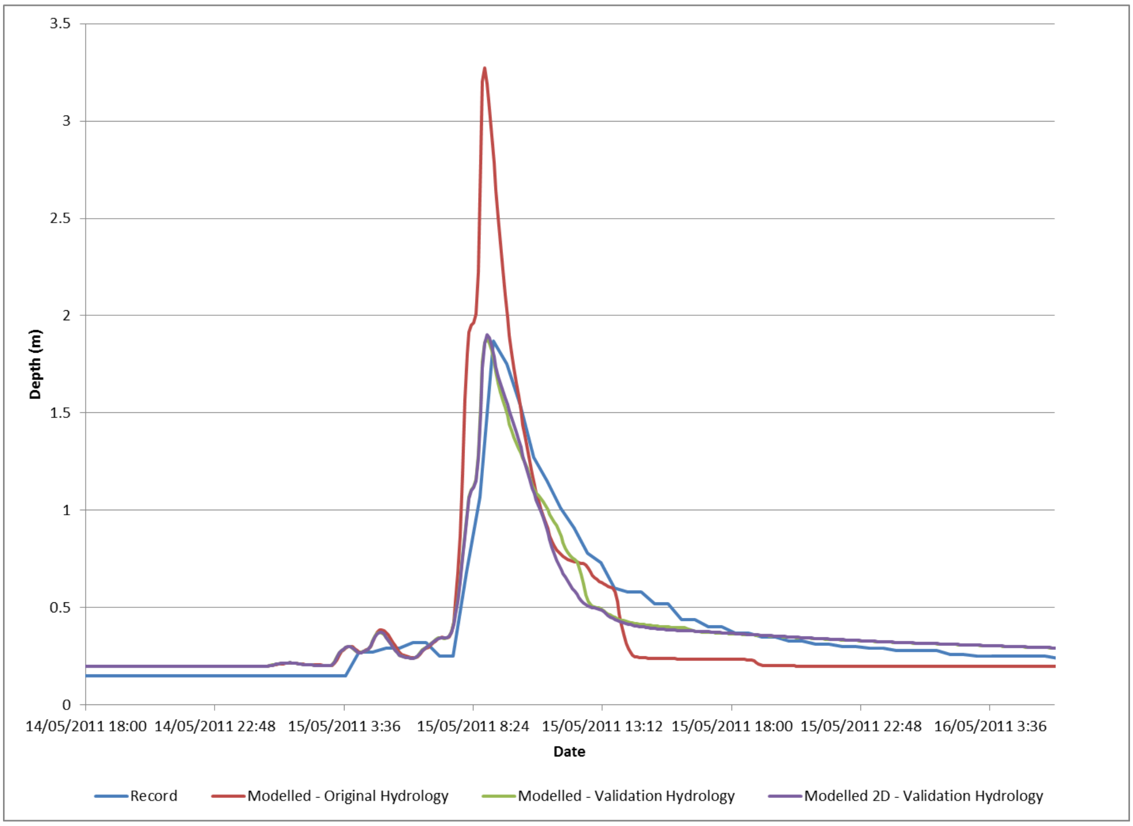


Figure 5-14: Stage-time plot at Wellington Road Bridge for the recorded and modelled May 2011 event

As part of the model calibration/validation process the model subcatchments were also adjusted following internal review. The original subcatchments were aggregated by their inflow locations to ensure routing was properly represented. All runoff surface percentages were then also updated following the aggregation. The aggregation reduced the number of subcatchments to 200, from the original 388. The original 388 are present in the ICM model and available for future use when additional network data becomes available.

5.5 2019 Validation Review

Following updates to the model by WWL a validation review was conducted. The May 2011 event was rerun (see section 3.1.4.1) with the hydrology updates and a minor increase in flow from 41.3m^3/s to 41.5 m^3/s was observed, see Table 5-16. As there was negligible change in the modelled flow no further validation was considered necessary.

Table 5-16: Recorded and Modelled Flows at Main Bridge for the May 2011 Event

Simulation	Maximum Flow (m^3/s)	Time of peak flow
Recorded Data	40.6	15/05/2011 9:25
2015 2D Model – pre-hydrology update	41.3	15/05/2011 9:10
2019 2D Model – post hydrology update	41.5	15/05/2011 9:10

5.6 Validation conclusions

The validation results show that the model validates well to the larger 2011 event with the use of rainfall radar and updated hydrology. However, the validation of the smaller 2013 and 2012 events, with more limited spatial variation, is not as close. The modelled flow is peakier than observed data through the Main Road bridge for the 2012 event, and the model under predicts the lower flows at the start and end of the 2012 event. Depths in the channel are closer to observed data at the start and end of the event, however, the model is under predicting depth during the peak by ~17%. The Wellington Road bridge also shows this under prediction where the updated hydrology depths are 31% lower than the observed data. Similar patterns can be observed in the 2013 event with under predictions of flow at the start and end of the rainfall event at Main Road bridge, lower depths at Main Road bridge, and under predictions at the Wellington Road bridge. However, the peak flow is much closer at Main Road bridge.

To improve the validation, further analysis of the spatial variability of the rainfall during the 2012 and 2013 events could be conducted, following a similar approach to the 2011 event.

6 Sensitivity

6.1 Sensitivity Analysis

A sensitivity analysis was completed for the coupled 1D/2D model to understand the influence of different parameters on flood extents, to aid in the creation of freeboard levels. It was agreed with WWL that Stantec would examine the effects of a rainfall increase of 50%, the partial blockage of inlets, the full blockage of inlets and a higher tailwater. Wave action from wind and vehicles was not considered following discussions with WWL. The maximum floods depths were compared to the 100yr ARI 12hr nested profile with 20% allowance form climate change, referred to as the ‘base case’.

A separate network was created for this stage of works, of which two versions have been included with this report. The high tailwater run is the only scenario connected with the second version of the network “Black Ck High Tailwater Run”. All other sensitivity simulations are related to the “Black Ck_Sensitivity_Network_SCS” network. Two versions of the network were supplied to maintain consistency with previous delivery of the model to WWL.

Please note all maps in the following section show the difference in depth between the base scenario and the sensitivity simulation.

6.1.1 50% Increase in Rainfall

50% increase of rainfall intensity was applied to the base climate change scenario (Figure 6-1). The result was a significant increase in flooding which exceeded 0.5m in some areas, The network is particularly sensitive to an increase in rainfall.

The majority of the areas that are impacted by the increase in rainfall are the low elevation areas which are already prone to flooding. The additional rainfall leads to small increases in surface water depth along the steeper hills. It is not until the water slows and pools where more significant increases in water depth are observed. For example, A large increase can be seen around Main Road to the south of the catchment. Less significant changes are seen around Parkway, as shown in Figure 6-1.

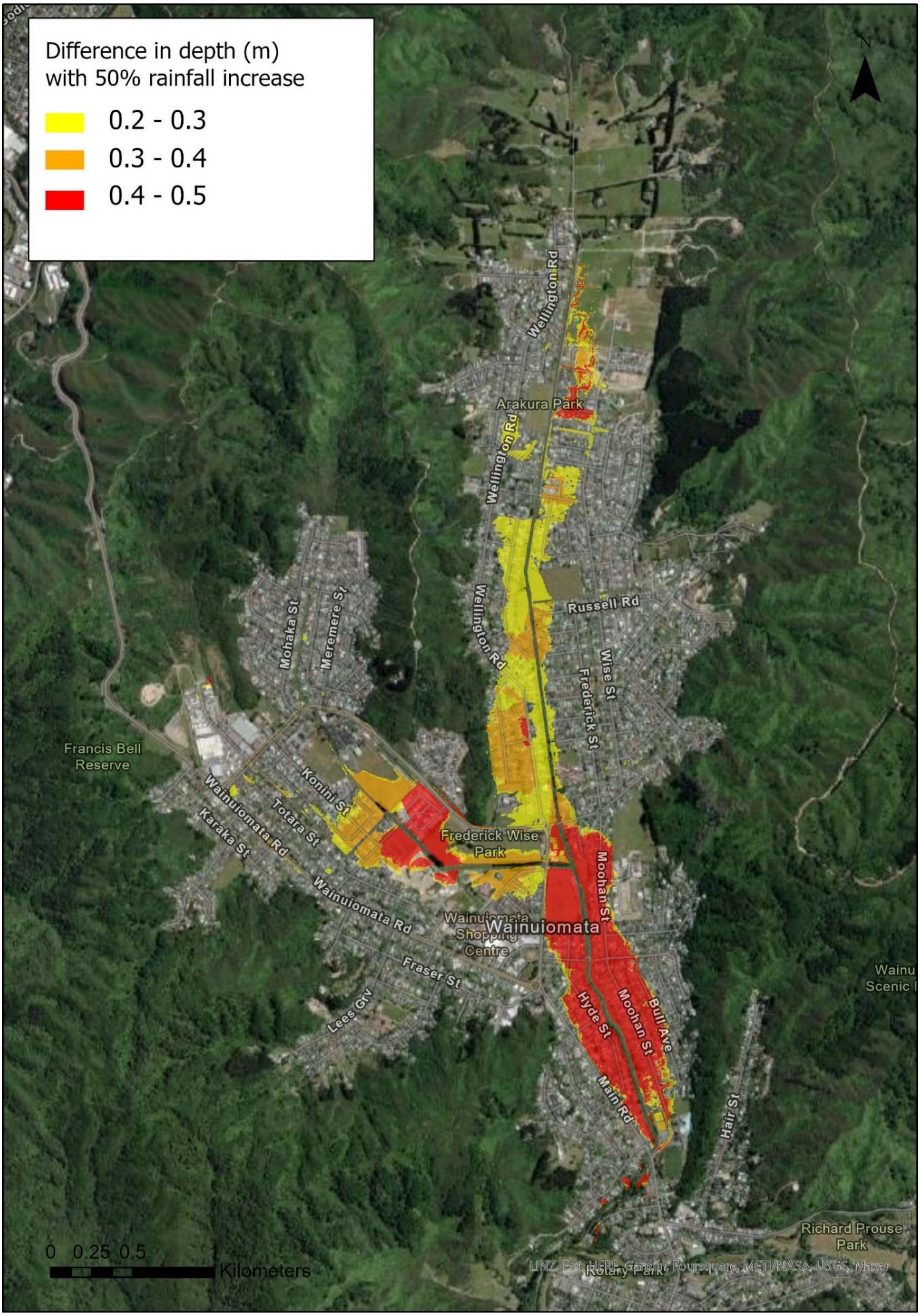


Figure 6-1: Difference in depth between base case scenario Cand a simulation with a 50% increase in rainfall intensity.

6.1.2 Inlets fully blocked

Inlets were fully blocked at the previously recognised 12 inlets (Figure 6-22). This blockage was accomplished by filling the conduits downstream of the inlets with sediment to approximately 90% of the culvert height. A depth of around 90% was used, as ICM requires a small space for base flow and the preissmann slot. For example, a 600mm conduit could only be filled with 540mm of sediment.
The results show that the network is moderately sensitive to full blockages at culvert inlets particularly around Frederick Wise Park, near the end of Parkway, and to a lesser extent, at the south end of Upper Fitzherbert Road. At these locations water is unable to pass through the bridges (modelled as culverts) along the main channels leading to localised increases in water depth. No change is observed in the upper reaches of the network as culvert inlets or small river reaches were not used to pick up sub catchment flows due to the model being constructed prior to the WWL modelling specifications. Instead, runoff is directly input into the nearest manhole.

Some areas modelled showed a decrease in the flood depth, these were downstream of blocked inlets and therefore would experience reduced flooding due to lower flow through the culverts. Above

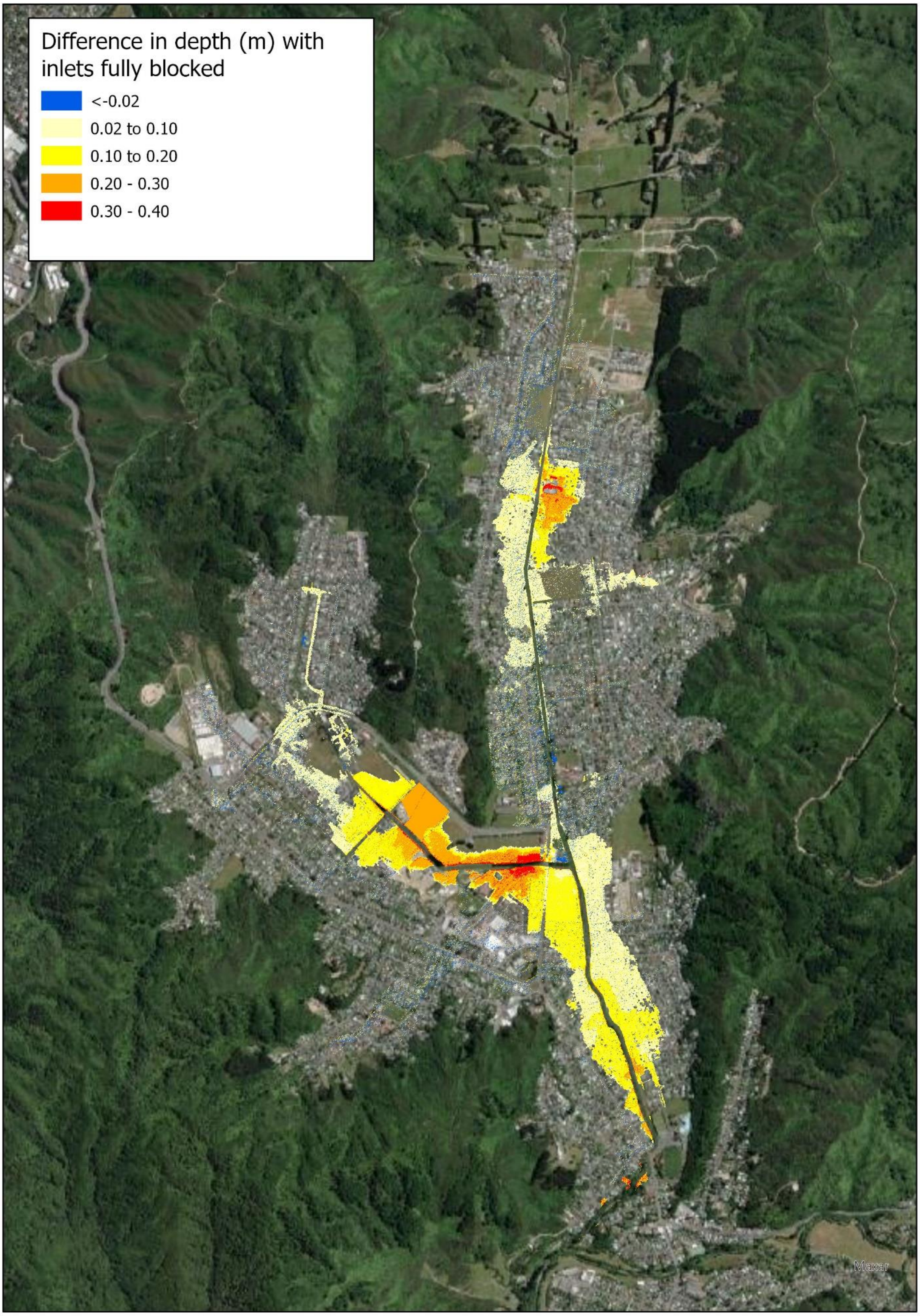


Figure 6-2: Difference in depth between base case scenario and a simulation with full blockage of all inlets.

6.1.3 Inlets partially blocked

Conduits immediately downstream of inlets were filed with sediment to half the dimeter of the culvert for this simulation (3). A total of 12 inlets were partially blocked. The results show that the network is slightly sensitive to the blocking of inlets near Frederick Wise Park and in other localised minor areas.

There is no area significantly susceptible to the partial blockage, as water is still able to flow through the culverts. The greatest amount of difference can be seen were Black Creek flows below Fitzherbert Street.

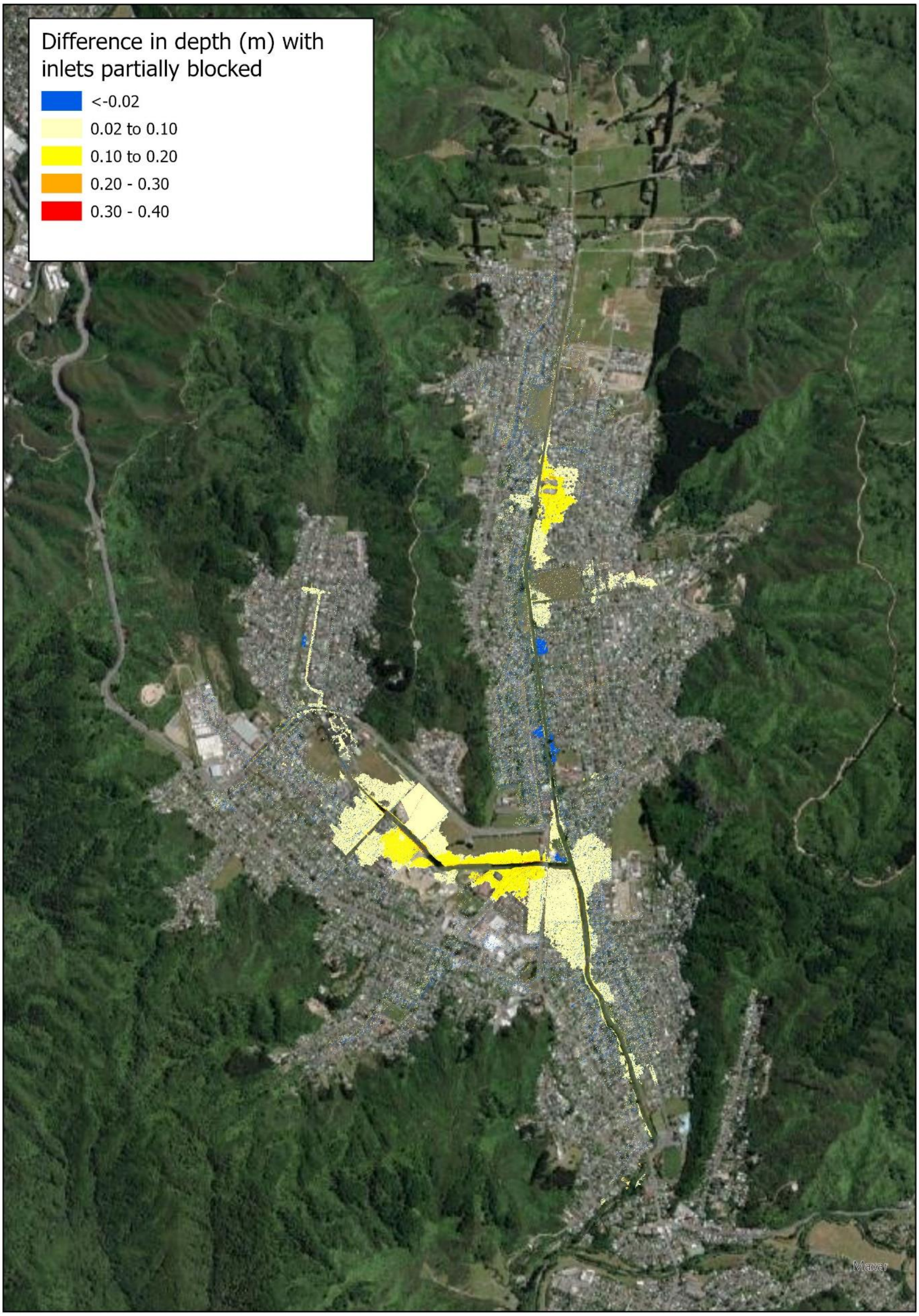


Figure 6-3: Difference in depth between the base case scenario and a simulation with partial blockage of all inlets.

6.1.4 High Tailwater

The tailwater level was raised by 1m to determine the sensitivity of the network to the flows in the Wainuiomata River (Figure 6-4). The network was not sensitive to this boundary condition, no difference in depth between the existing case and the high tailwater simulations exceeded 0.2m with the majority of the change being less than 0.05m. This low level of sensitivity occurs due to the distance that the Wainuiomata River is from the catchment and the gradient of Black Creek which rises quickly above the tailwater level. Thus the tailwater does not have a significant impact on the direction or velocity of flow.

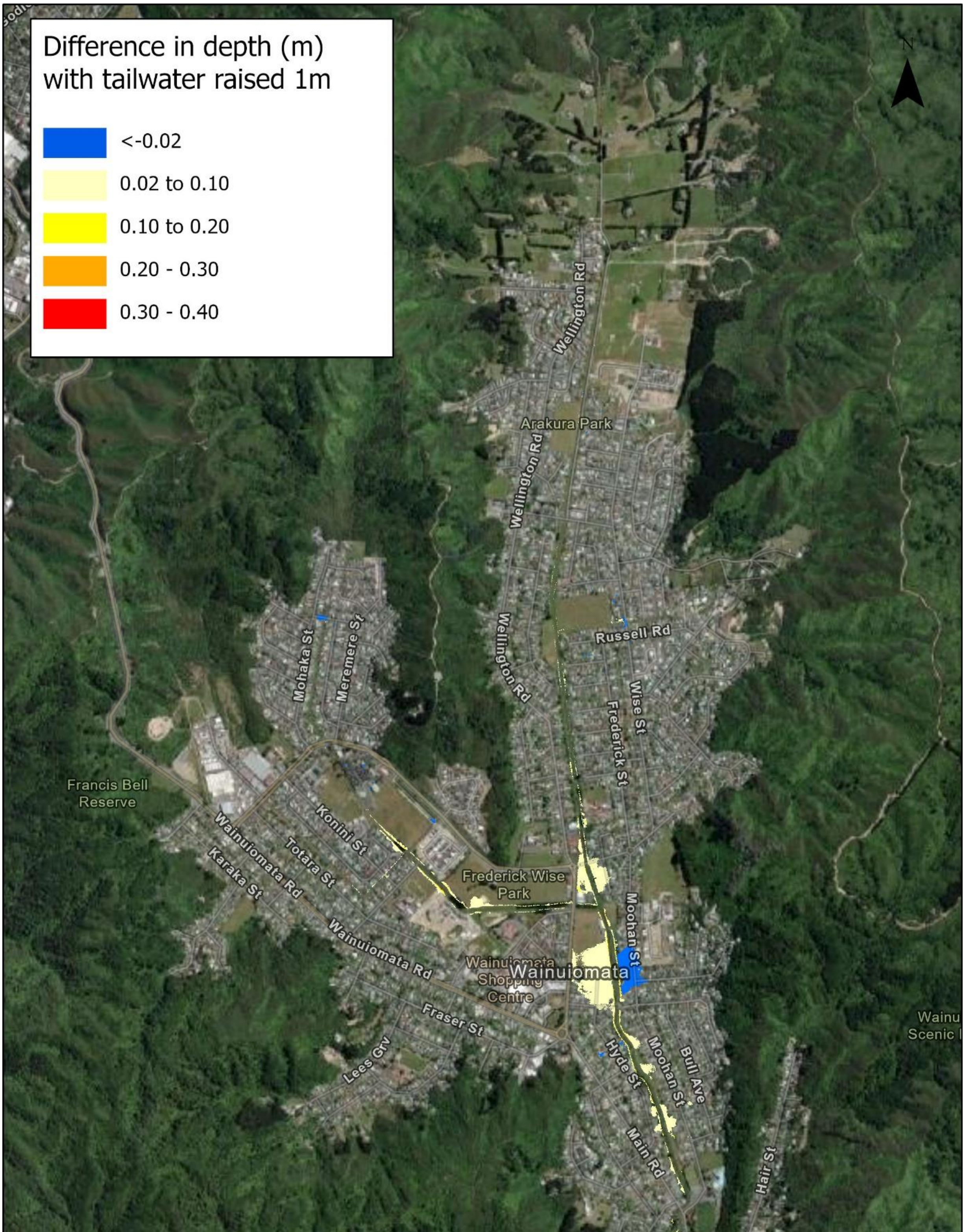


Figure 6-4: Difference in depth (m) between the base case scenario and a simulation with a 1m increase in the tailwater boundary condition.

7 Freeboard

7.1 Freeboard Allowance Selection

Freeboard values were determined by examining the results from the scenarios in 5 and determining how sensitive the Black Creek network was to these key model parameters. After discussions with Wellington Water, the freeboard allowances shown in Figure 7-2 were applied across the network using a dynamic simulation.

7.2 One Velocity Head (V2/2g)

The base case maximum results were used to calculate velocity head values in the Black Creek Catchment, as shown in Figure 7-1. This simulation was completed to identify the areas of risk within the network that may have required further consideration in determining appropriate freeboard values. Only one area, near the top of the catchment, on Stockdale Street, exceeded 0.5V2/2g. As this was an isolated incident, confined to the road itself, this was not considered further during the freeboard selection process.

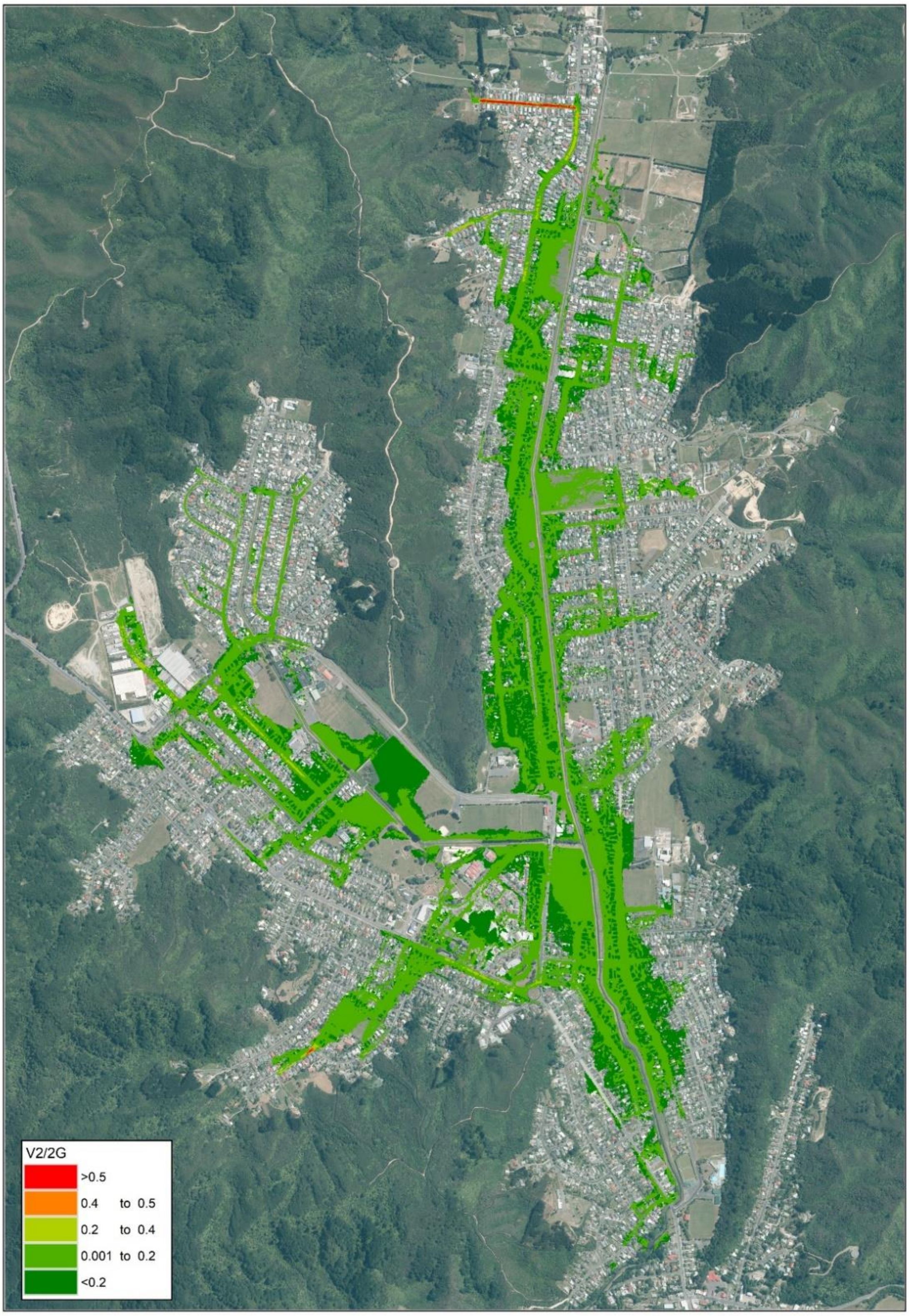


Figure 7-1 V2/2G calculations for the 100yr ARI 12hr event with 20% climate change adjustment

7.3 Freeboard Simulation

A separate network, “Black Creek Freeboard April 19”: scenario “Freeboard”, made to be devoid of all 1D attributes, was used for the Freeboard analysis. This network was developed following the method outlined in “Dynamic Freeboard Analysis – Tawa”, 2017, Jacobs Memorandum, where the agreed freeboard values were added to maximum flood depths (m) for the 100yr ARI 12hr nested profile with +20% allowance for climate change. These resulting depths were processed in GIS and imported back into the network as IC Zone – hydraulics (2D) polygons. All 1D network elements were deleted from the model including all subcatchments, 1D pipes, and 1D river reaches. IC Zone – hydraulics (2D) polygons were then used to create initial 2D conditions . A simulation including only the Initial Conditions (2D) file was then run for five minutes. This allowed the maximum water levels to spread naturally across the catchment to establish baseline flood extent to be used for assigning minimum building floor levels.

The resulting maximum freeboard depths are shown below in Figure 7-3.

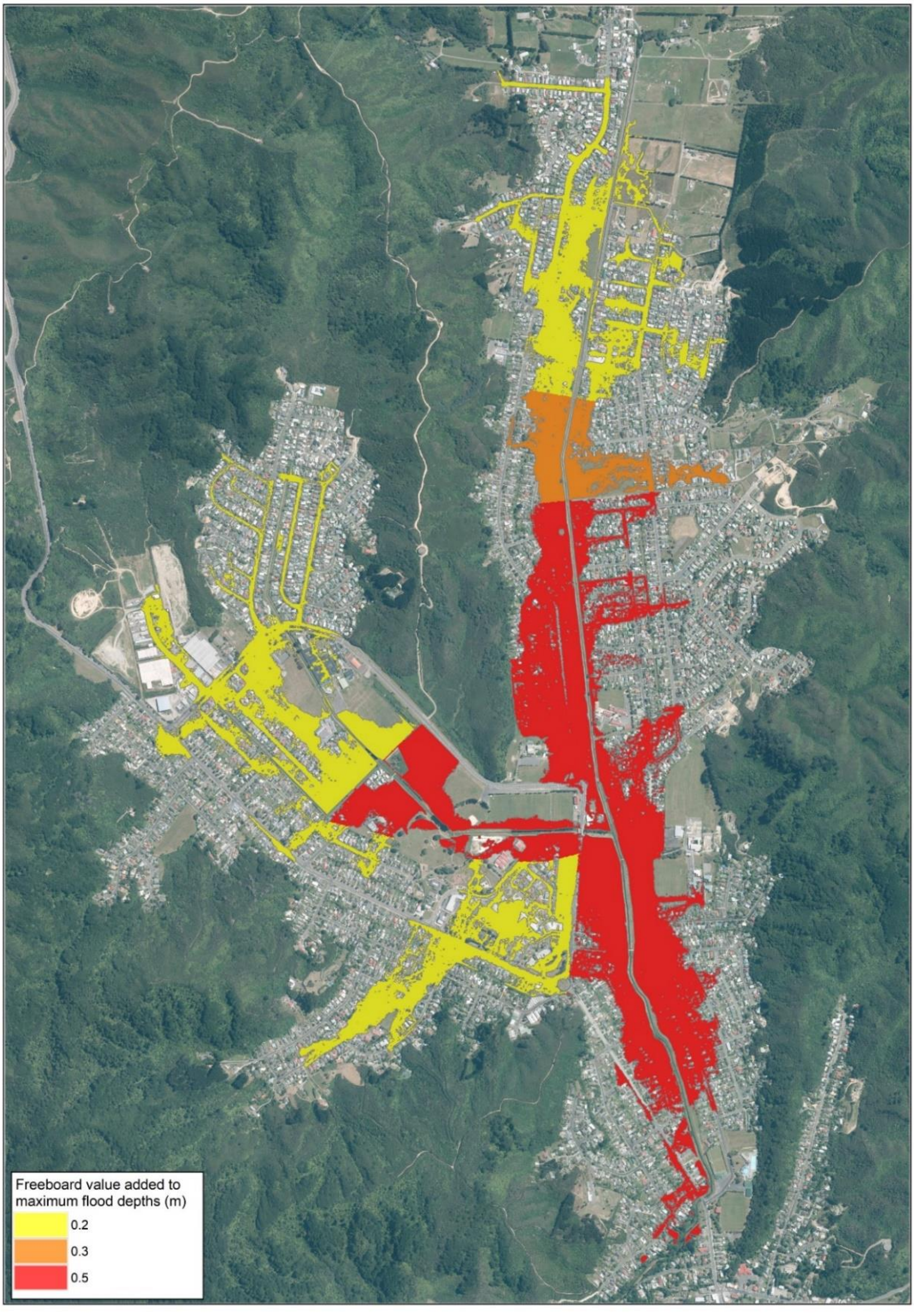


Figure 7-2: Freeboard added to the maximum flood depth for final freeboard simulation

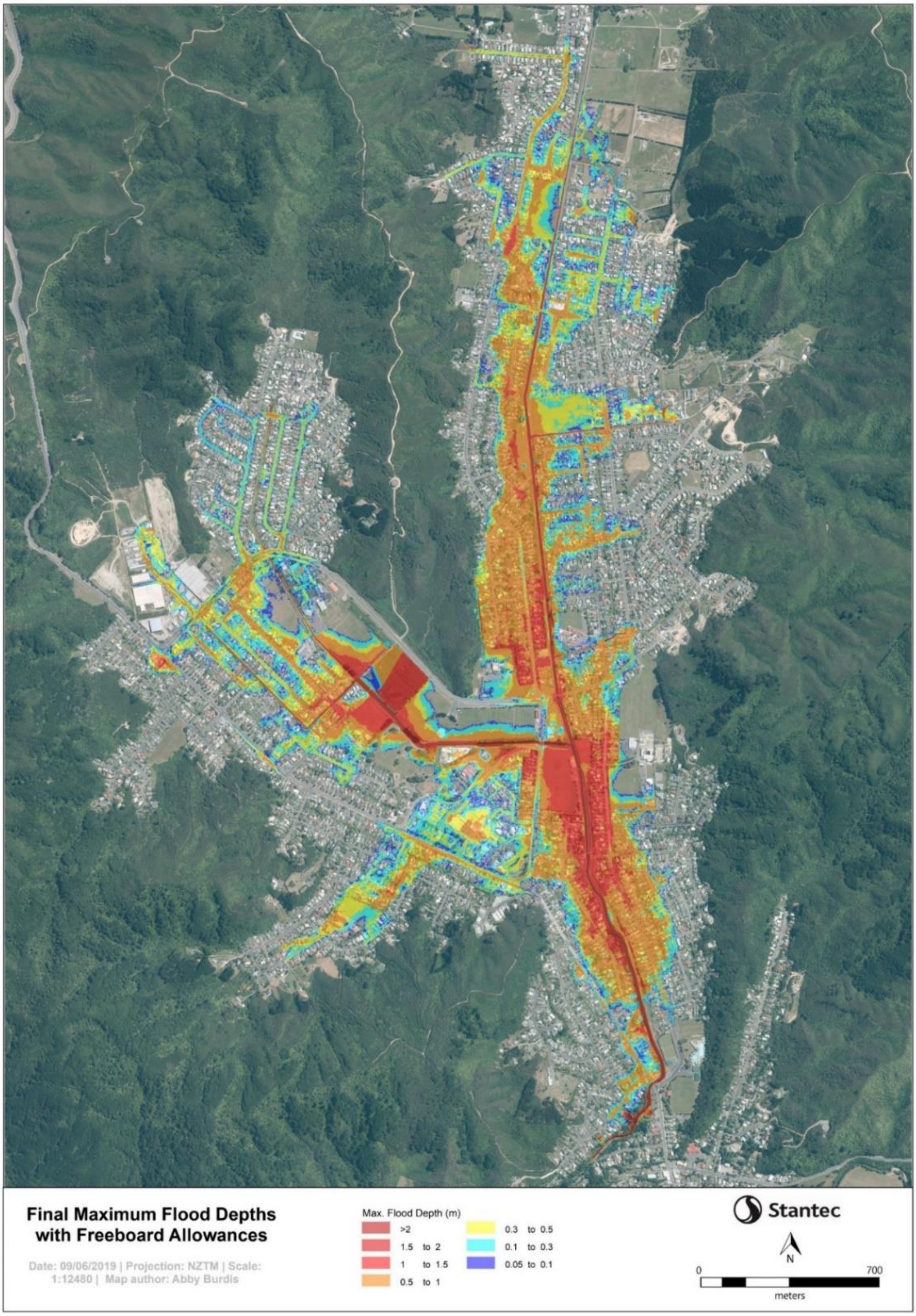


Figure 7-3: Maximum flood depths (m) with freeboard allowances

8 System Performance Assessment

A system performance assessment was completed to understand the potential impact of development and land use changes in the upper Black Creek catchments. At the time of the initial model build, there was a proposal in place for significant greenfield development of the semi-rural area at the northern end of Upper Fitzherbert Road.

The development will result in a change in runoff surfaces present in that portion of catchment which may have an impact on flows into Black Creek and downstream water levels.

In addition to investigating changes in runoff, an assessment of the limited portion of the stormwater network has also been conducted.

Twenty model scenarios have been selected for the system performance assessment. These include five event magnitudes, with and without climate change to 2090 and before and after full development as it is currently proposed.

Two separate model networks were used to represent the before and after development situations. These are “Black Ck System_Performance_Existing” and “Black Ck System_Performance_Development” The key difference between the before and after development scenarios, is the percentage of each runoff surface type in the catchments covering the development area. The pipe and channel network remains the same between the scenarios as no information on the proposed stormwater assets in the development area was available, at the time of the assessment.

8.1 Design Storm Model Scenarios

Table 8-11 provides a summary of the rainfall depths applied for the nominated storm events

Table 8-1: Summary of Existing and Future Design Rainfall Depths for Nominated Storm Events (using an area reduction factor of 0.97)

Design Storm Event	Existing 24-hour rainfall depth (mm)	Future 24-hour rainfall depth (mm)	Percentage difference
5yr ARI	129.1	143.8	11.4
10yr ARI	154.9	175.4	13.2
20yr ARI	179.2	206.3	15.1
50yr ARI	216.4	252.8	16.8
100yr ARI	249.2	291.1	16.8

8.2 Water Balance for the Catchment

Table 8-2 provides a summary of the catchment water balances for the modelled scenarios. The table shows an increase in the percentage of rainfall becoming runoff with an increase in event magnitude and climate change. This is an expected pattern as runoff is likely to increase with increasing rainfall volumes and intensities.

The table also shows there is an increase in runoff following the development of the upper Black Creek catchment, as would be expected with increased areas of impervious ground. The magnitude of the difference between the pre and post development runoff is greatest in the lower ARI events, and becomes smaller with increasing event magnitude.

Table 8-2 : Summary of the Water Balance of the Catchment

Model Scenario		Catchment Rainfall Volume (m^3)	Catchment Runoff Volume (m^3)	Catchment Runoff Volume (%)
Existing Development	Current 5yr ARI	2316362.7	919180.0	39.7
Existing Development	Future 5yr ARI	2378747.5	1135446.5	47.7
Existing Development	Current 10yr ARI	2486272.5	1229453.1	49.4
Existing Development	Future 10yr ARI	2815325.0	1529063.3	54.3
Existing Development	Current 20yr ARI	2996039	1650967	55.1
Existing Development	Future 20yr ARI	3311294.5	1984900.2	59.9
Existing Development	Current 50yr ARI	3473407.4	2135059.8	61.5
Existing Development	Future 50yr ARI	4057657.3	2680094.5	66.1
Existing Development	Current 100yr ARI	3999880.1	2626599.2	65.7
Existing Development	Future 100yr ARI	4672412.3	3258775.1	69.7
Proposed Development	Current 5yr ARI	2136362.7	967574.4	45.3
Proposed Development	Future 5yr ARI	2378747.5	1181971.1	49.7
Proposed Development	Current 10yr ARI	2486272.5	1276474.2	51.3
Proposed Development	Future 10yr ARI	2815325.0	1577103.5	56.0
Proposed Development	Current 20yr ARI	2996039	1676397	56.0
Proposed Development	Future 20yr ARI	3311294.5	2034470.2	61.4
Proposed Development	Current 50yr ARI	3473407.4	2185103.7	62.9
Proposed Development	Future 50yr ARI	4057657.3	2731768	67.3
Proposed Development	Current 100yr ARI	3999880.1	2680892.6	67.0
Proposed Development	Future 100yr ARI	4672412.3	3312016.1	70.9

8.3 Impact of Development in the Upper Black Creek Catchment

Table 8-3 provides the difference in runoff volumes originating from the model subcatchments affected by the proposed development (Figure 4-2). The magnitude of increase varies between the subcatchments. However, there is a net increase in runoff volume of 54,402m3 for the 100yr ARI event and 48,464m^3 for the 5yr ARI event following development.

Table 8-3 Runoff Volumes (m3) from Subcatchments Affected by Proposed Development

Subcatchment ID	5yr			100yr		
	Pre Development	Post Development	% Increase	Pre Development	Post Development	% Increase
110SC	2457	2457	0.0	5550	5550	0.0
116SC	2050	2090	2.0	4396	4438	1.0
117SC	2189	2189	0.0	4605	4605	0.0
183SC	7290	8665	18.9	23417	24898	6.3
185SC	2130	2130	0.0	4747	4747	0.0
19SC	8098	8098	0.0	19396	19396	0.0
328SC	3281	5101	55.5	8896	10963	23.2
330SC	18610	27006	45.1	71670	82151	14.6
331SC	20792	27418	31.9	73779	81457	10.4
341SC	5062	7984	57.7	13966	17355	24.3
370SC	10431	12752	22.2	26471	29168	10.2
372SC	22320	23781	6.5	75593	77098	2.0
380SC	2530	2530	0.0	7401	7401	0.0
3SC	72023	92821	28.9	272870	294898	8.1
114	1054	1096	4.0	2393	2447	2.3
383	1688	2555	51.4	4612	5596	21.3
329	905	1447	59.8	2481	3076	24.0
326	1472	2225	51.2	3899	4732	21.4
327	1082	1584	46.4	2939	3505	19.3
Net Increase	**48,464 m^3**	**26.1%**		**54,402 m^3**	**8.6%**	

Figure 8-1 and Figure 8-2 show channel long sections with maximum water levels reached in Black Creek during the 100yr and 5yr ARI events. The long section for the 100yr ARI event, Figure 8-1, shows that there is an increase in the water level up to approximately the Norfolk St Bridge. For the 5yr ARI event, Figure 8-2, the increase in water level appears to extend the full way down Black Creek channel to Main Rd Bridge, however flow remains within the channel.

Figure 8-3 and Figure 8-4 show flow hydrographs in Black Creek at a cross section location immediately downstream of all the proposed development discharge locations (chainage 851m in Figure 8-1). The post development hydrograph shows a “peakier” response to the storm events which would be expected given the significant change to the ground surface as a result of development.

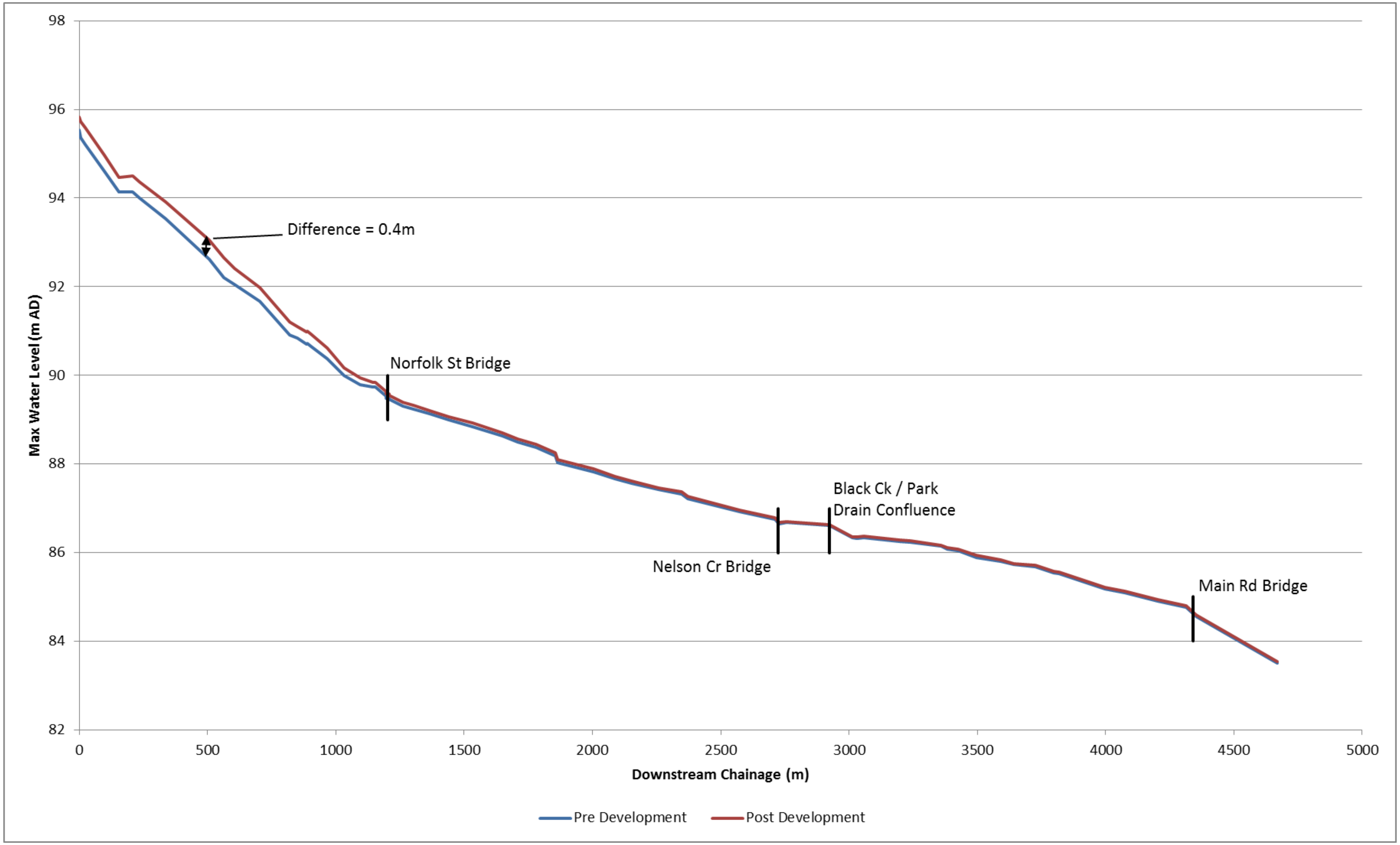


Figure 8-1: Water levels in Black Creek pre and post development during the 100yr ARI event under current climate conditions

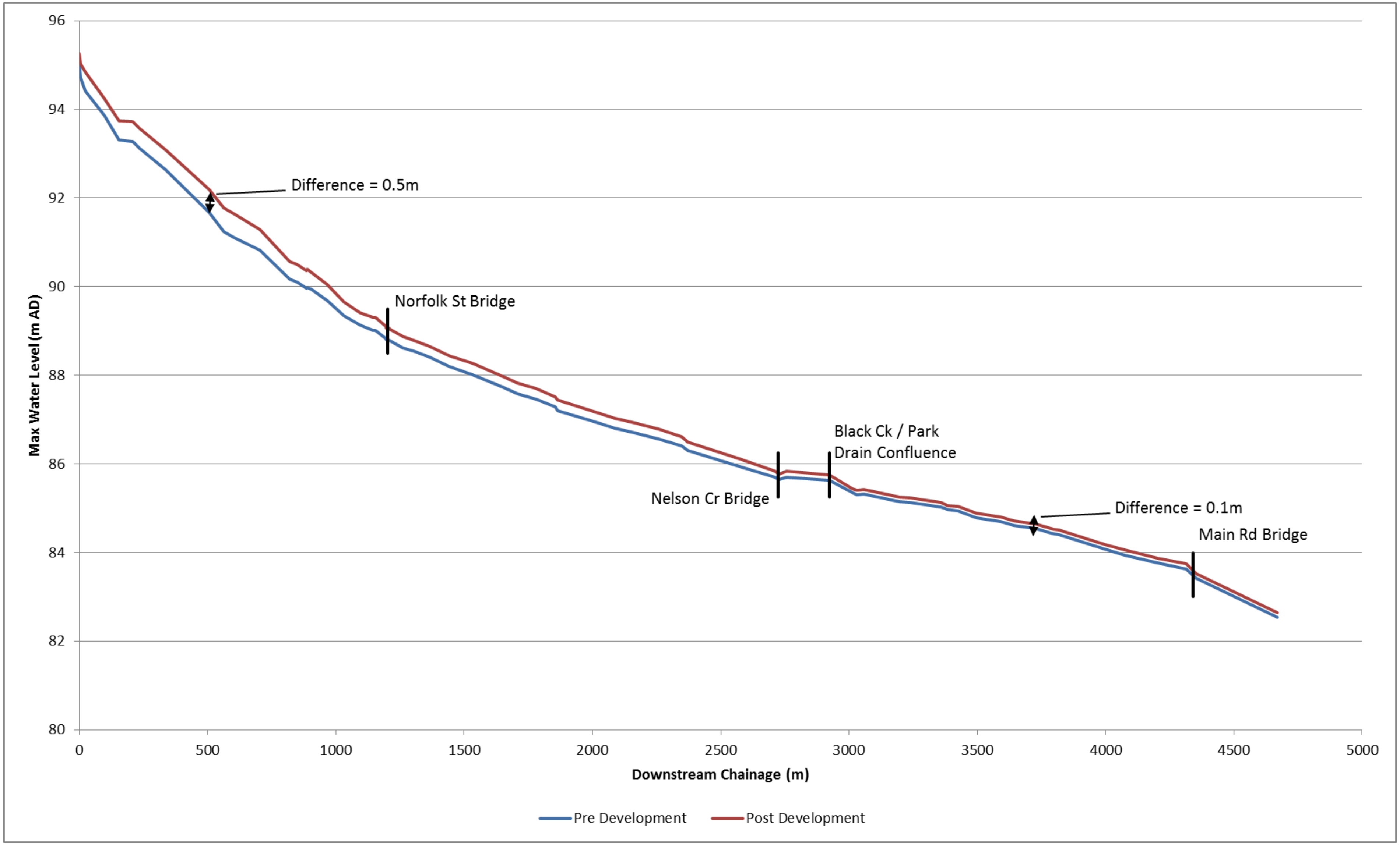


Figure 8-2: Water levels in Black Creek pre and post development during the 5yr ARI event under current climate conditions

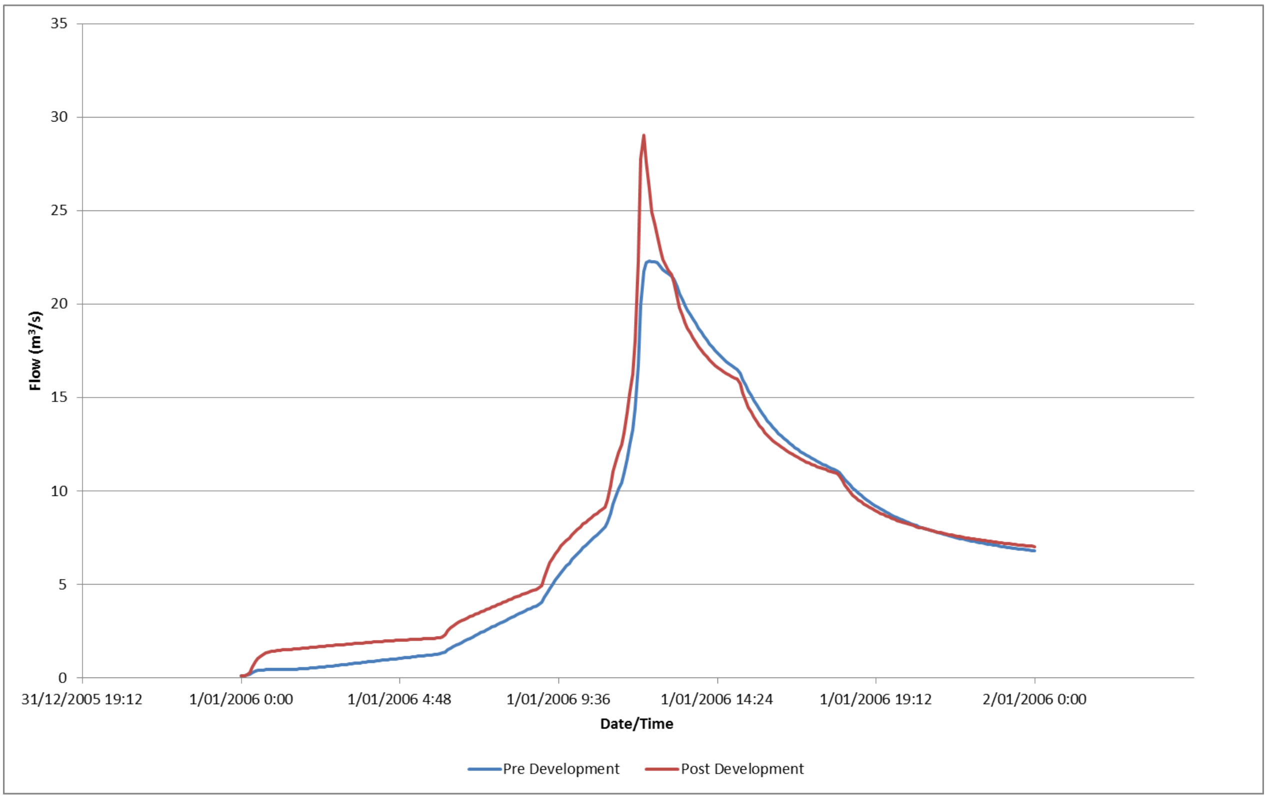


Figure 8-3: Flow for the 100yr ARI event in Black Creek channel at chainage 851m

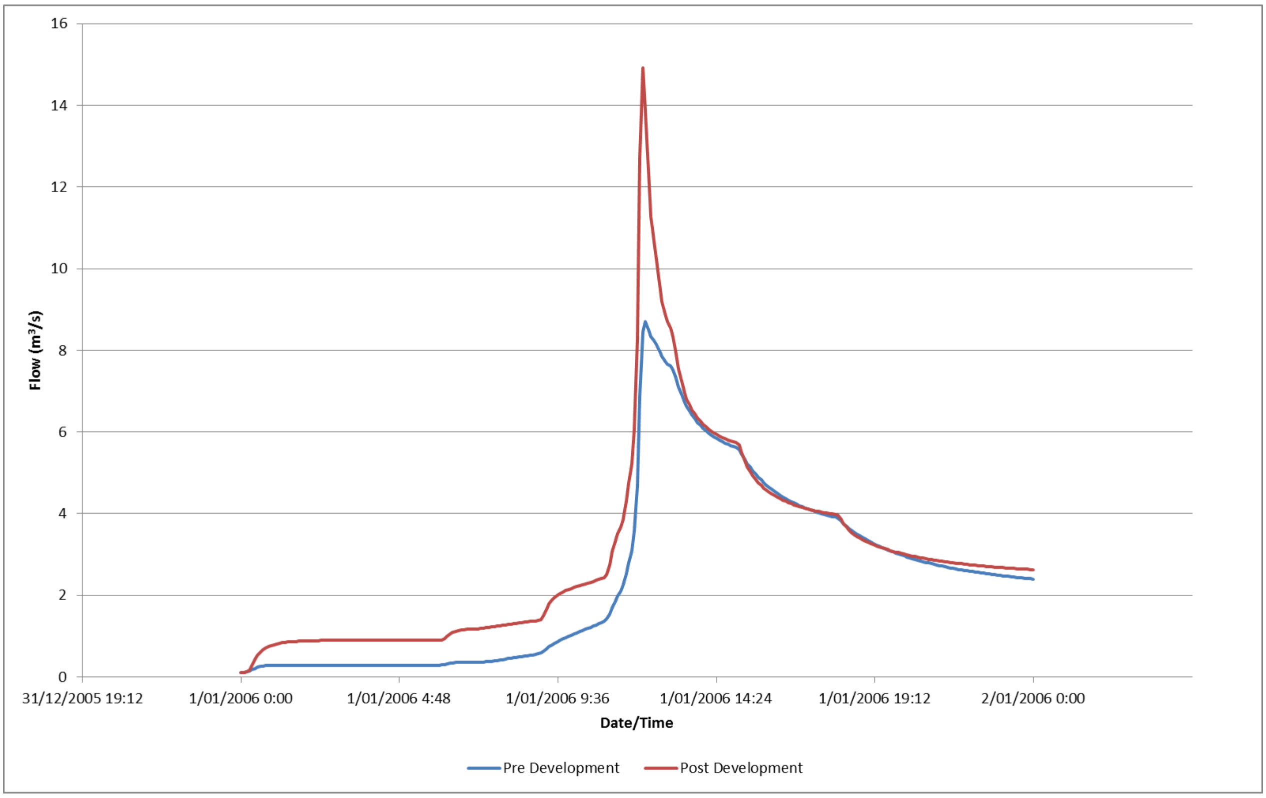


Figure 8-4: Flow for the 5yr ARI event in Black Creek channel at chainage 851m

8.4 Capacity of the Existing Primary Pipe System

An assessment of the existing primary pipe system was undertaken by assessing the flow conditions during the various simulations. Table 8-4 provides the percentage of pipes operating under each flow condition for the modelled scenarios. The flow conditions are as follows:

- Flow Condition 0 – Pipe operating under free flow conditions (ICM result field MaxSurchargeState = <1)
- Flow Condition 1 – Pipe operating under backwater conditions (ICM result field MaxSurchargeState = 1)
- Flow Condition 2 – Pipe operating under pressurised conditions (ICM result field MaxSurchargeState = 2)

The results in Table 8-4 show that a significant portion, approximately 40% or greater, of the modelled pipe network is under capacity and flowing under pressurised conditions, even for the 5yr ARI event. Of interest is that between 45% to 50% of the network is flowing under backwater conditions for all the event magnitudes and climate scenarios. Similarly there is little variation in the pipe flow conditions between the pre and post development simulations.

It should be noted that the model includes only a limited portion of the Wainuiomata stormwater network. As a result, the values in the table below may represent a conservative estimate of the network performance.

Table 8-4: Flow Conditions of Existing Pipe Network

		Current Climate			Future Climate		
		% pipes	% pipes	%pipes	% pipes	% pipes	% pipes
5yr	Pre Development	14.3	46.7	39.0	8.2	44.9	46.9
	Post Development	13.5	47.7	38.8	8.2	45.4	46.4
10yr	Pre Development	6.1	47.4	46.4	5.1	47.4	47.4
	Post Development	6.9	46.9	46.2	5.1	48.2	46.7
20yr	Pre Development	5.1	48.2	46.7	4.8	47.4	47.7
	Post Development	5.1	49.5	45.4	4.8	48.2	46.9
50yr	Pre Development	4.8	48.0	47.2	4.3	47.2	48.5
	Post Development	4.8	48.2	46.9	4.3	47.2	48.5
100yr	Pre Development	4.3	47.2	48.5	2.6	49.7	47.7
	Post Development	4.3	47.2	48.5	2.6	49.7	47.7

A series of maps have been produced to show thematically the spatial distribution of the pipe flow conditions and manhole performance. Manholes have been thematically displayed to differentiate between surcharged, surcharged where the top water level reaches above the outlet pipe soffit, and where the top water level reaches above ground level. For the manholes where the top water level reaches above ground level, the volume of water overflowing from the manhole has been represented by point size. In this case the ICM result field “Flood Volume” has been used to represent the overflow volumes.

Sixteen maps have been generated, four for each event magnitude. These maps are provided in Appendix E . The pre and post development results for the 5yr ARI event (under current climate conditions) are provided below in Figure 8-5 and Figure 8-6. The maps indicate there is some difference in the network performance between the model simulations, with increased flooding from manholes upstream of Mary Crowther Park in the post development scenario. There is a negligible difference between the development scenarios, downstream.

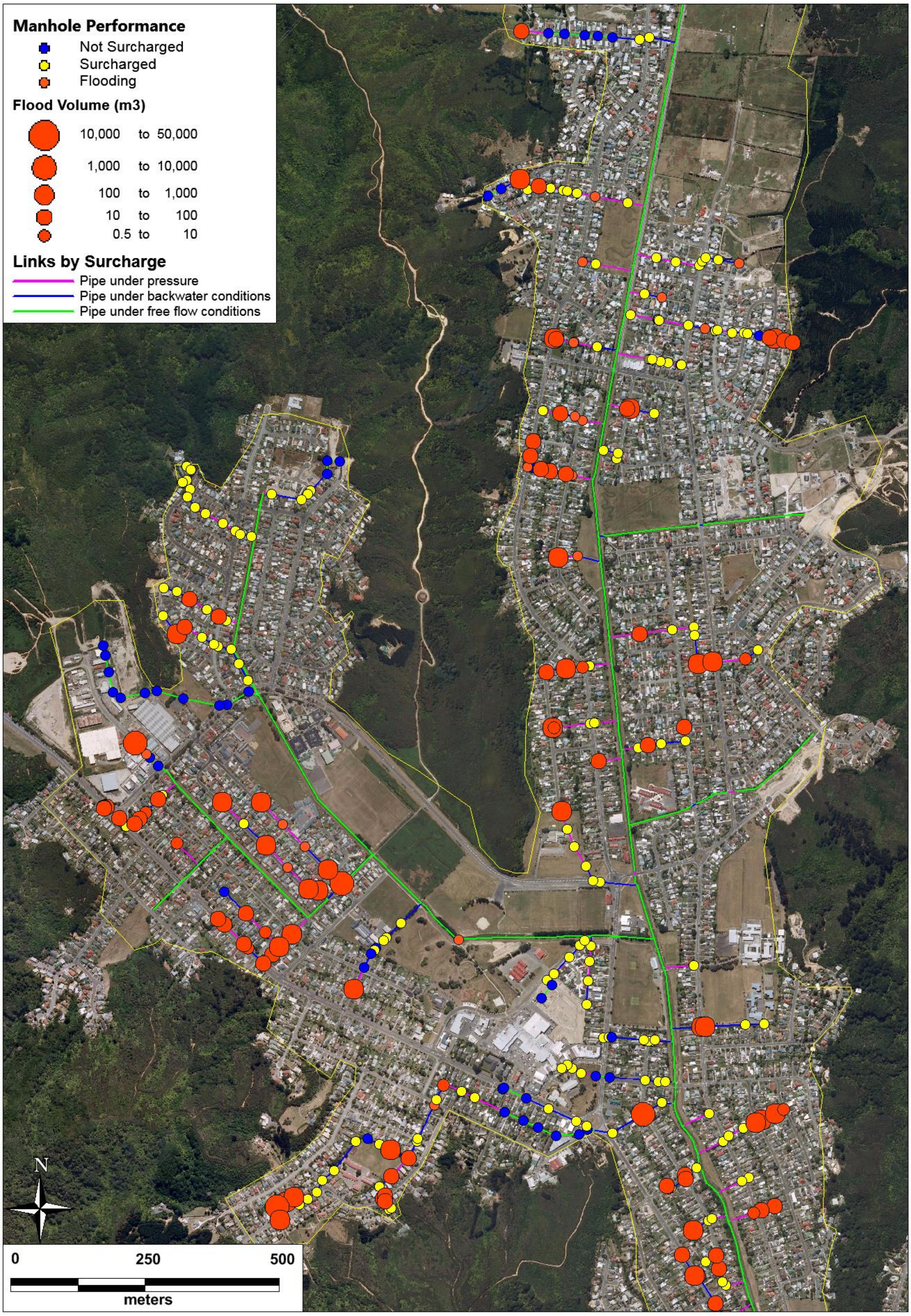


Figure 8-5: Wainuiomata SW network performance for the 5yr ARI event – pre development

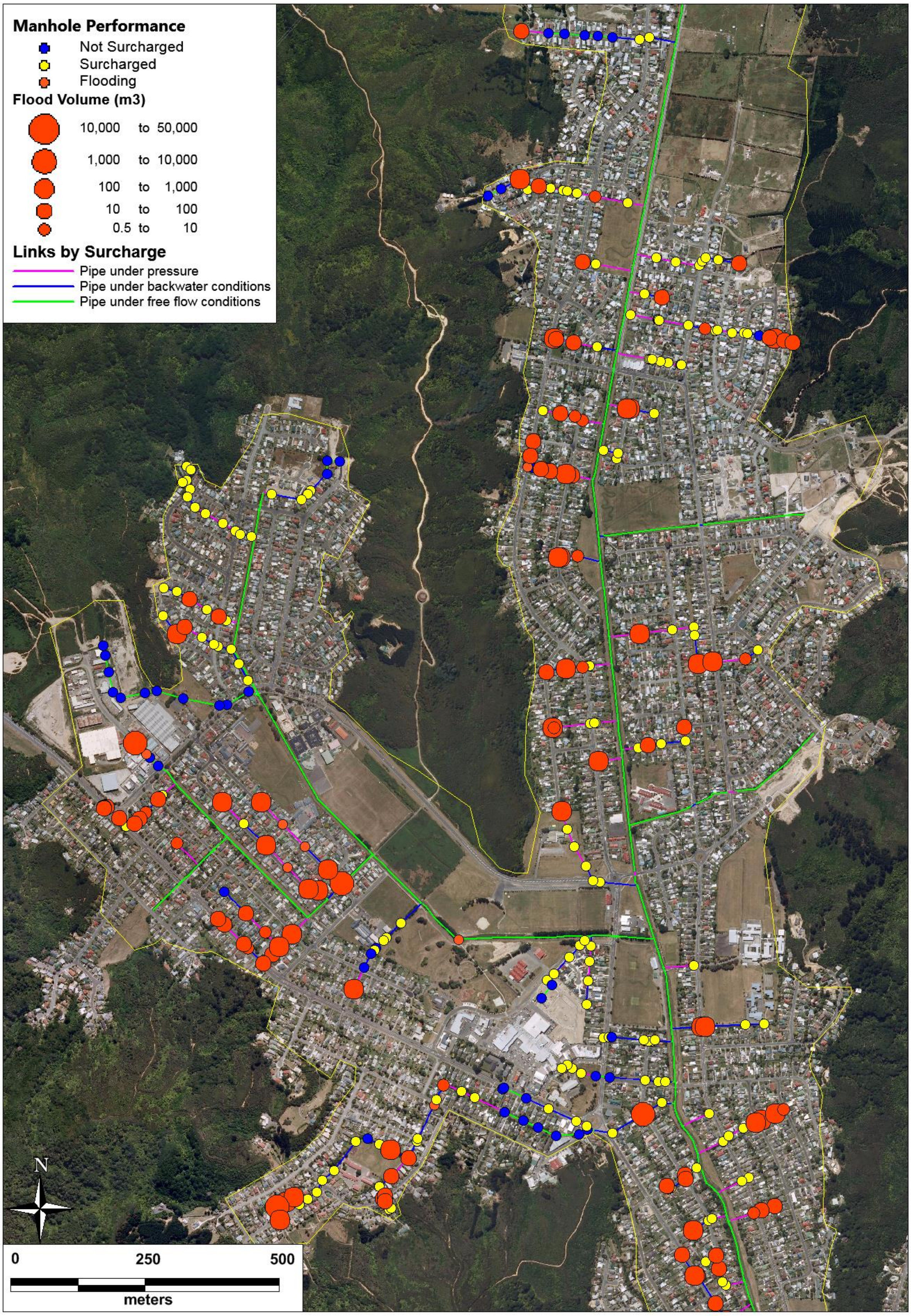


Figure 8-6: Wainuiomata SW network performance for the 5yr ARI event – post development

8.5 Culvert and Bridge Capacity

Table 8-5 and Table 8-6 provide the design flow and capacity information for the modelled bridges and culverts. Table 8-6 shows that while the capacity of most of the culverts is exceeded during a 100yr ARI event, only 4 of the 17 culverts modelled are under capacity for the 10yr ARI event.

Table 8-7 shows which of the culverts and bridges were overtopped and/or surcharged during the 10yr and 100yr ARI events for both the pre and post development scenarios**.** Table 8-6 shows that the majority of the culverts perform well under the current climate conditions and development scenario. Only 5 culverts appear to be surcharged or overtopped. The results also show there is no difference in culvert performance under the proposed development scenario for the 10yr ARI event, although water levels are generally slightly higher under the proposed development conditions.

Similarly, there is practically no difference between the development scenarios for the 100yr ARI event, except at the pedestrian bridge near Weymouth Grove. At this location it appears water level is approximately 0.4m higher under the proposed development conditions. This fits with the long sections in Figure 8-1, as the pedestrian bridge is situated approximately 300m upstream of the Norfolk Rd Bridge.

Table 8-5: Selected Design Flows through the Modelled Bridges

Location	Link ID	Type	MPD Design Flow (m3/s)	
			10% AEP	1% AEP
Federick Wise Park	670002R00868.1	Bridge	15.2	28.3
Nelson Cr	BC0044.1	Bridge	28.6	42.6
Weymouth Gr	BC_br_1_us.1	Bridge	17.9	29.3
Best St	BCc0067.1	Bridge	44.5	67.8
Main Rd	BCc0102.1	Bridge	52.7	85.7
Fitzherbert Park	BCc0106.1	Bridge	42.8	61.5
McKay St	BCc0112.1	Bridge	27.3	40.7
Edmonds St	BCc0126.1	Bridge	25.2	40.2
Park Drain	PWDr_B_us.1	Bridge	13.6	18.5

Table 8-6 Summary of Capacity and Design Flow for Culverts

Location	Link ID	Type	Shape	Width (mm)	Height (mm)	Culvert Capacity		MPD Design Flow (m3/s)		10% AEP Capacity Available
						Full Flow (m3/s)	Inlet Control (m3/s)	10% AEP	1% AEP	
Frederick St	SC2_ds_a.1	Culvert	CIRC	750	750	0.6	0.6	1.3	1.7	No
Wise St	670111R00967.1	Culvert	CIRC	750	750	1.9	0.6	1.4	2.2	Yes
Parkway	670033R00921.1	Culvert	CIRC	900	900	1.6	1.0	1.8	2.0	No
Parkway	670033R00921a.2	Culvert	CIRC	900	900	1.6	1.0	1.8	2.0	No
Manutuke St	670005R00899.1	Culvert	CIRC	900	900	2.7	1.0	1.1	2.5	Yes
Manutuke St	670005R00899a.1	Culvert	CIRC	900	900	2.7	1.0	1.1	2.5	Yes
Wise St	SC1_ds_c.1	Culvert	CIRC	900	900	2.5	1.0	2.4	2.7	No
Totara St	670013R00949.1	Culvert	RECT	1200	1200	1.9	1.9	0.6	1.1	Yes
Totara St	670013R00949.2	Culvert	RECT	1200	1200	1.9	1.9	0.6	1.1	Yes
Wainuiomata Rd	670088R00955.1	Culvert	RECT	1200	1200	1.9	1.9	0.5	1.4	Yes
Wainuiomata Rd	670088R00955.2	Culvert	RECT	1200	1200	1.9	1.9	0.5	1.4	Yes
Ashburn Rd	BC0197.1	Culvert	CIRC	1200	1200	7.9	2.0	0.8	2.3	Yes
Konini St	670012R00888a.1	Culvert	RECT	1400	1200	6.0	1.9	1.5	2.0	Yes
Konini St	670012R00888.1	Culvert	RECT	1400	1200	6.0	1.9	1.5	2.0	Yes
Russell Rd	BCc0094.2	Culvert	CIRC	1500	1500	7.8	3.6	3.2	4.8	Yes
Fitzherbert Rd	670071R00853.1	Culvert	RECT	6500	3350	201.9	9.0	16.2	29.5	Yes
Norfolk St	BC0172.1	Culvert	RECT	10000	2100	107.0	4.5	20.6	31.4	Yes

Table 8-7: Summary of Maximum Water Levels Upstream of Culverts, and Bridges Compared to Overtopping Level

Location	Link ID	Surcharge Level (m RL)	Overtop Level (m RL)	Upstream Max. Water Level (m RL)			
				Pre Development		Post Development	
Federick Wise Park	670002R00868.1	87.0	87.0	86.3	87.1	86.4	87.1
Nelson Cr	BC0044.1	85.9	86.4	86.1	86.9	86.2	86.9
Weymouth Gr	BC_br_1_us.1	91.4	91.4	90.4	90.9	90.7	91.3
Best St	BCc0067.1	86.2	86.6	85.4	86.2	85.5	86.3
Main Rd	BCc0102.1	85.6	86.2	84.1	85.0	84.2	85.0
Fitzherbert Park	BCc0106.1	86.8	86.9	85.7	86.5	85.8	86.5
McKay St	BCc0112.1	87.0	87.3	86.7	87.4	86.9	87.5
Edmonds St	BCc0126.1	87.6	88.2	87.6	88.4	87.8	88.4
Park Drain	PWDr_B_us.1	87.0	87.2	86.5	87.2	86.6	87.2
Frederick St	SC2_ds_a.1	89.5	90.5	89.9	90.6	89.9	90.6
Wise St	670111R00967.1	91.5	93.8	92.7	94.7	92.7	94.7
Parkway	670033R00921.1	90.2	90.5	90.8	90.8	90.8	90.8
Parkway	670033R00921a.2	90.2	90.5	90.7	90.8	90.7	90.8
Manutuke St	670005R00899.1	96.5	98.2	96.5	98.1	96.5	98.1
Manutuke St	670005R00899a.1	96.5	98.2	96.5	98.1	96.5	98.1
Wise St	SC1_ds_c.1	90.8	92.5	91.9	92.6	91.9	92.6
Totara St	670013R00949.1	89.4	89.5	88.9	89.1	88.9	89.1
Totara St	670013R00949.2	89.4	89.5	88.9	89.1	88.9	89.1
Wainuiomata Rd	670088R00955.1	91.6	93.1	91.0	91.4	91.0	91.4
Wainuiomata Rd	670088R00955.2	91.6	93.1	91.0	91.4	91.0	91.4
Ashburn Rd	BC0197.1	97.9	97.9	97.7	98.5	97.7	98.5
Konini St	670012R00888a.1	86.5	86.7	87.0	87.5	87.0	87.5
Konini St	670012R00888.1	86.5	86.7	87.0	87.5	87.0	87.5
Russell Rd	BCc0094.2	87.8	88.5	88.1	88.9	88.3	88.9
Fitzherbert Rd	670071R00853.1	87.0	87.8	86.1	86.9	86.2	86.9
Norfolk St Bridge	BC0172.1	89.7	90.3	89.0	89.6	89.3	89.7

8.6 Floodplain Mapping

Flood depth and extent mapping has been carried out for the 10yr, 50yr and 100yr ARI events, as per the WWL Regional Specification requirements as seen in Appendix E . Figure 8-7and Figure 8-8 show the modelled extent of flooding for the pre and post development scenarios for the 10yr ARI event under current climate conditions. Figure 4-3 and Figure 4-3 show the modelled extent of flooding for the 100yr ARI event under current climate conditions.

The 10yr ARI event indicates there is some flooding across Wainuiomata, although most of the flooding that does occur is restricted mainly to roads, parks and fields. The maps show that most flooding originates from the stormwater network rather than open channels. Exceptions to this are seen at the Parkway and Konini St culverts where overtopping of the road occurs. Flow from Park drain also appears to spill into Wainuiomata High School field. The results also indicate that there is little difference between the pre and post development scenarios.

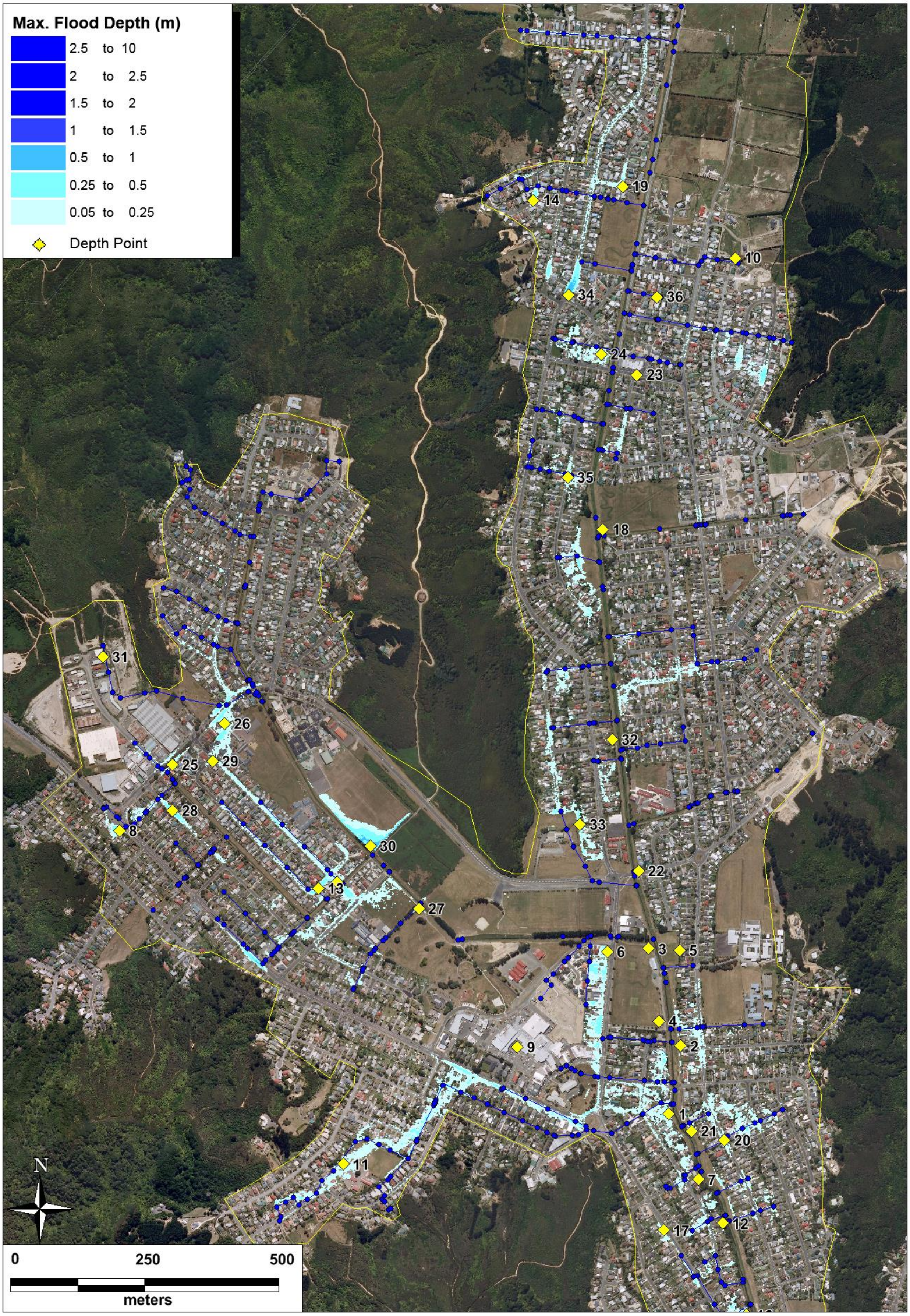


Figure 8-7: Maximum flood depth during the 10yr ARI event under current climate condition - pre development

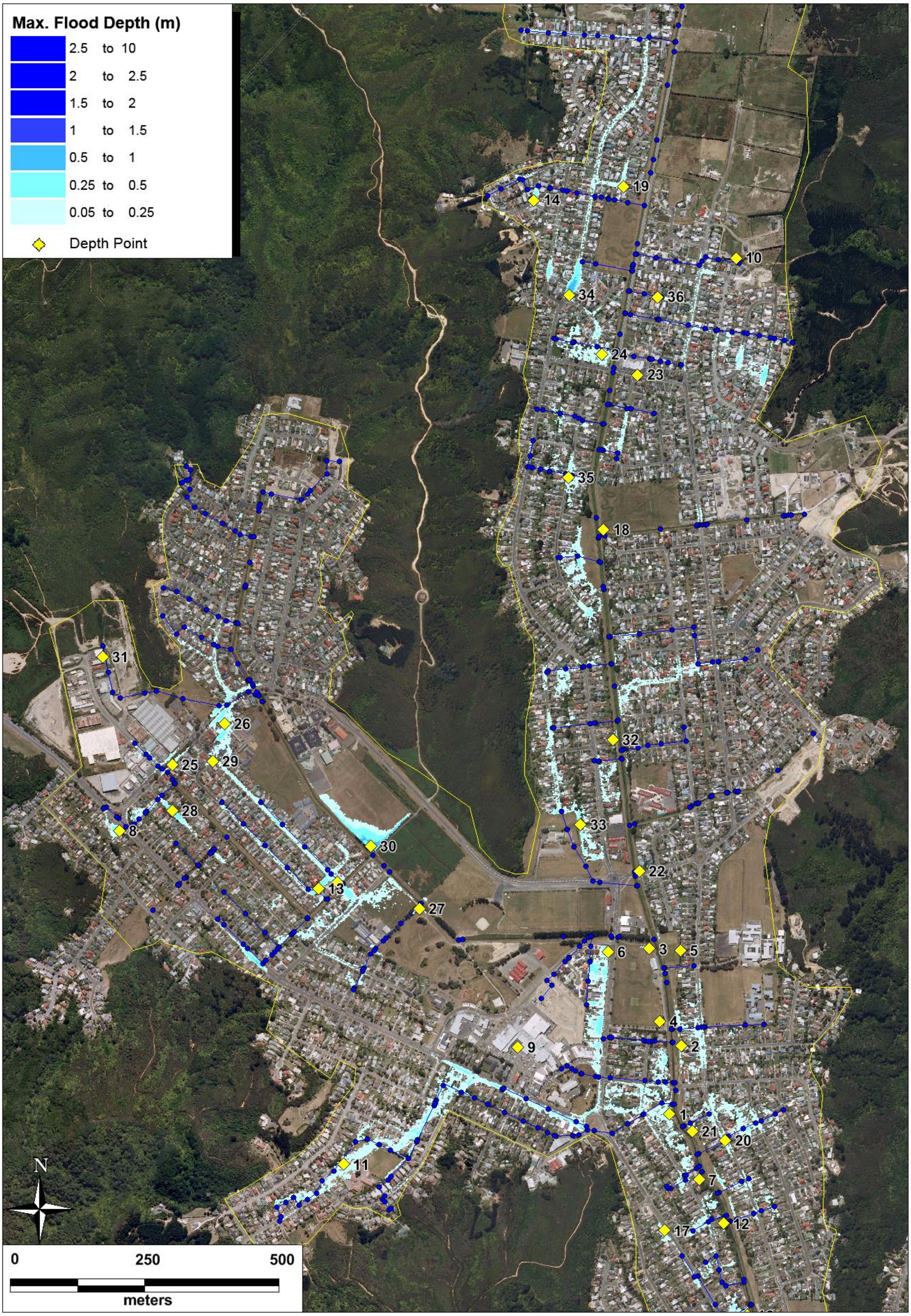


Figure 8-8: Maximum flood depth during the 10yr ARI event under current climate conditions - post development

The results of the simulations for the 100yr ARI event magnitude show extensive flooding across Wainuiomata, with inundation not just restricted to roads and parks. For example water breaks out of the channel at several locations downstream of Ashforth St, see Figure 8-9 and Figure 8-10. Water also leaves the Main Black Creek channel from the confluence of Black Creek and Park Drain to approximately Best St. The culvert crossings at Parkway, Konini St, Totara St, Frederick St, and the two crossing on Wise St are all surcharged. As is the case in the 10yr ARI simulations there appears to be little difference between the pre and post development scenarios.

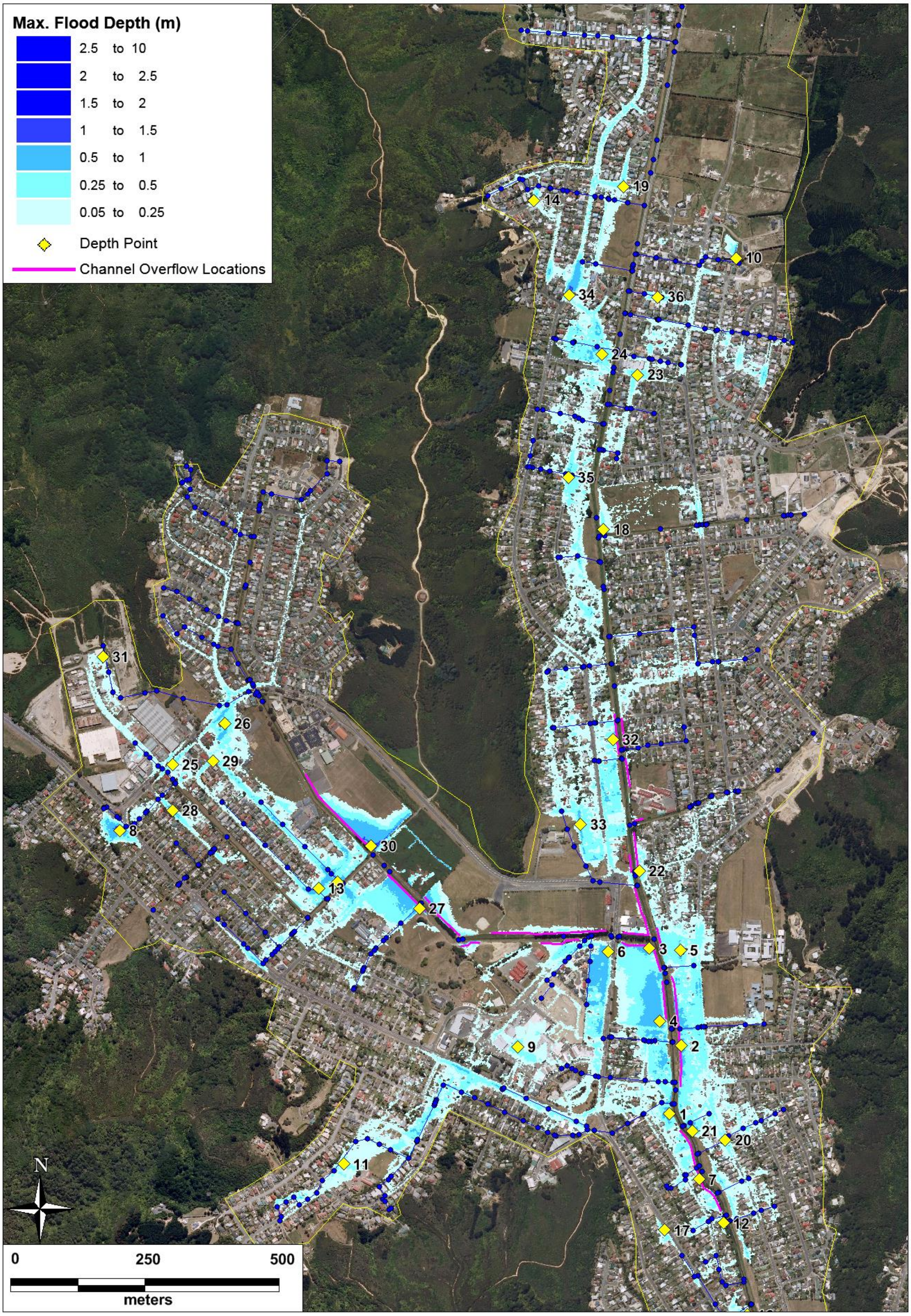


Figure 8-9: Maximum flood depths during the 100yr ARI event under current climate conditions - pre development

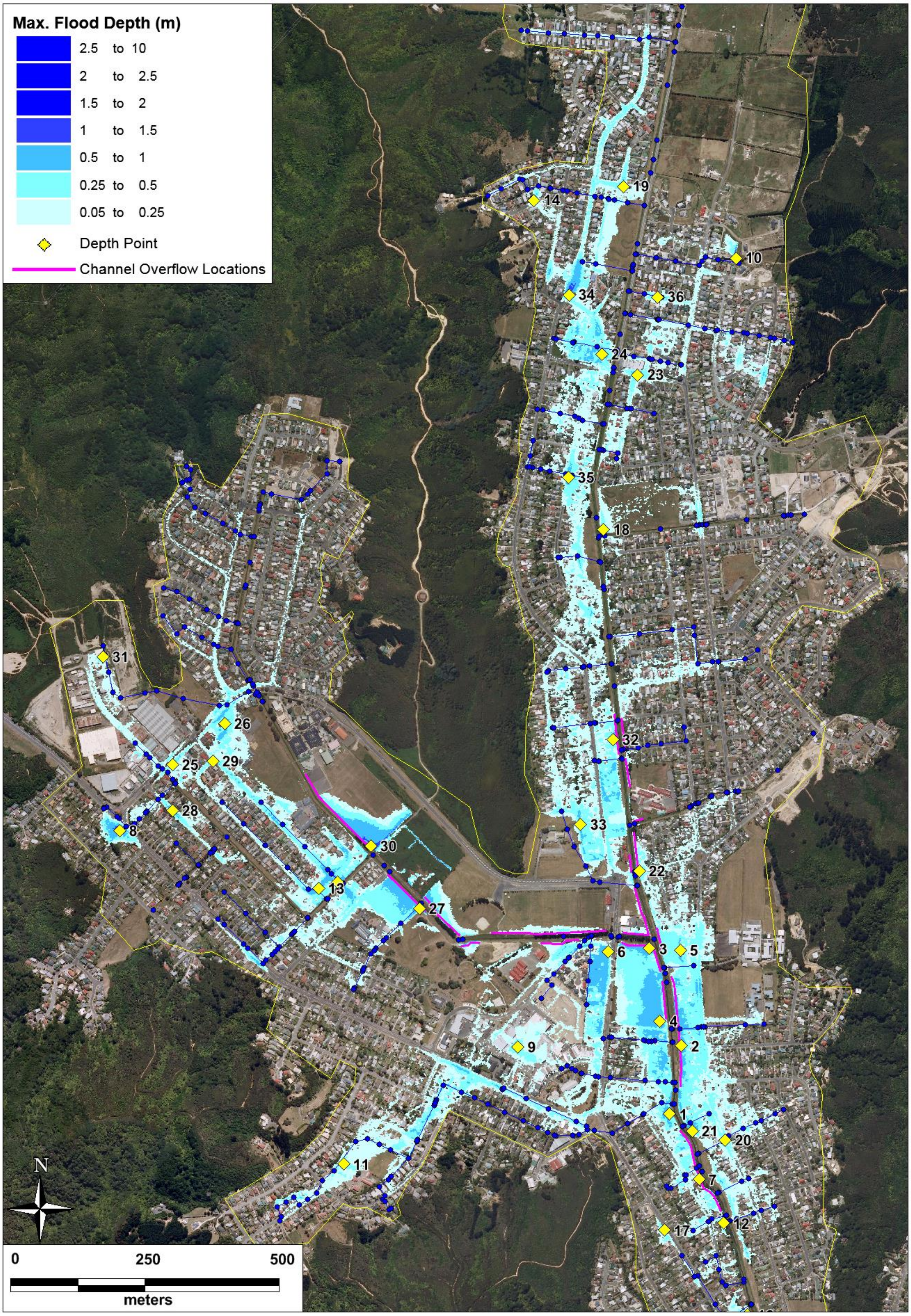


Figure 8-10: Maximum flood depths during the 100yr ARI event under current climate conditions - post development

8.6.1 Maximum Water Depths

During the 10yr ARI event only three locations are inundated with water over 0.5m in depth (locations 30, 8, and 34), see Table 8-8. All of these locations are situated in open spaces or roads. The residential areas at risk of flooding during the 10yr ARI event are as follows:

- Between Parkway and Totara St, from a mixture of network surcharges and channel overtopping (locations 26 and 29).
- Totara St, from a mixture of network surcharges and channel overtopping (locations 29 and 15).
- Between Holland St and Wainuiomata Rd, from network surcharges (location 11).
- Between Fitzherbert Rd and Hyde St and Black Creek, from network surcharges (locations 1 and 7).
- Between upper Bull Ave and Black Creek, from network surcharges (location20).
- Between the Strand and Fitzherbert Rd, from network surcharges (location 6).
- Between Castlerea St, Wise St and Ashforth St, from network surcharges.
- Between Wellington Rd, Norfolk St and Black Creek, from network surcharges (location 24).

In general, the water depths generated by the model at these locations are approximately 0.2m or less. locations 15 and 13 are the only locations where depths reach close 0.4m and 0.5m, respectively (Table 8-9). The results in Table 8-10 indicate that of the locations listed above 15, 13, 6, 20, and 24 were inundated for the longest period, ranging from approximately 5 (location 13) to 12 hours (location 24).

There is also no significant difference between the results generated in the post development scenario compared to the pre development results (Table 8-9), with an increase depth greater than 0.1m at only one of the locations (location 4, under the future climate conditions).

As shown in the flood extent maps, Table 8-8 and Table 8-9 show significant flooding during the 100yr ARI event. Water depths are 0.5m or greater at over half the locations interrogated in the model and over 0.2m at 34 of the locations. The maximum depth reached is 1.17m at the corner of Parkway and Wainuiomata Rd (location 8). Flooding at 21 of the locations provided in the Table 8-11 lasted approximately 12 hours or longer, with the maximum duration of inundation being 16.92 hours at location 15 on Konini St. It should be noted that these durations of inundation may have been truncated by the end of the simulation rather than water draining from a location.

For the 100yr ARI event the model simulations indicate that the majority of flooding originates from the stormwater network rather than Black Creek or Park Drain, except adjacent to Black Creek and Park Drain; downstream of the Park Drain and Black Creek confluence; and the Konini St, Totara St, and Parkway culvert crossings where the flooding is a result of the stormwater network surcharging and channel overflows. There is also no significant difference between the results generated in the post development scenario compared to the pre development results for the 100yr ARI event Table 8-8.

The predicted climate change does have some impact on flood depths and extent. The increase in flood depths as a result of climate change ranges from 0m to 0.4m. The greatest increase in depth (0.35m) occurs in Hyde St (location 12).

Table 8-8: Maximum Flood Depths at Selected Locations Under Current Climate Conditions

ID	Location	10yr		50yr		100yr	
		Pre	Post	Pre	Post	Pre	Post
1	Best St/Black Creek	0.21	0.21	0.36	0.38	0.57	0.59
2	Black Creek at Wright St	0.00	0.04	0.53	0.56	0.76	0.79
3	Bryan Heath Park RB	0.06	0.14	0.55	0.57	0.74	0.76
4	Bryan Heath Park RB2	0.00	0.00	0.70	0.72	0.89	0.92
5	Bryan Heath Park LB	0.00	0.00	0.49	0.50	0.62	0.64
6	Corner of Fitzherbert& the Strand	0.22	0.22	0.71	0.72	0.80	0.81
7	Corner of Hyde & Arthur St	0.00	0.00	0.00	0.00	0.11	0.14
8	Corner of Parkway Dr & Wainuiomata Rd	0.55	0.55	1.07	1.07	1.17	1.17
9	Countdown Car Park	0.00	0.00	0.51	0.51	0.59	0.59
10	End of Trelawney St	0.00	0.00	0.47	0.57	0.69	0.71
11	Fernlea School	0.13	0.13	0.24	0.24	0.29	0.29
12	Hyde St near number 28	0.01	0.01	0.27	0.30	0.51	0.54
13	Karamu St	0.29	0.30	0.57	0.57	0.67	0.67
14	Kendal Grove	0.35	0.36	0.45	0.45	0.47	0.47
15	Konini St	0.39	0.39	0.67	0.66	0.77	0.77
16	Main Rd near number 71	0.23	0.23	0.30	0.30	0.32	0.32
17	Main Rd near number 40	0.21	0.21	0.32	0.33	0.38	0.38
18	Mary Crowther Park	0.00	0.00	0.31	0.34	0.40	0.41
19	Matthews Rd	0.01	0.02	0.16	0.18	0.21	0.23
20	Moohan St around number 67	0.12	0.12	0.23	0.23	0.32	0.35
21	Moohan St at number 79	0.03	0.03	0.23	0.26	0.51	0.55
22	Nelson Crescent Bridge	0.00	0.00	0.21	0.23	0.38	0.40
23	Norfolk & Honey St	0.00	0.00	0.23	0.30	0.38	0.42
24	Norfolk St between Upper Fitzherbert & Wellington Road	0.22	0.27	0.52	0.55	0.59	0.62
25	Parkway Dr at Waiu St	0.30	0.30	0.50	0.50	0.56	0.56
26	Parkway Dr near Mohaka St	0.15	0.15	0.36	0.36	0.41	0.41
27	St Claudine Thevenet School Field	0.05	0.10	0.59	0.59	0.76	0.75
28	Totara St around 61	0.45	0.45	0.57	0.57	0.61	0.61
29	Upper Konini St	0.06	0.06	0.26	0.26	0.33	0.33
30	Wainuiomata High School Field	0.64	0.64	0.96	0.94	1.07	1.16
31	Waiu St /Mountain Bike Park	0.00	0.00	0.38	0.38	0.46	0.46
32	Wellington Road near Devon St	0.00	0.00	0.21	0.29	0.36	0.40
33	Wellington Road near Parkway D	0.40	0.41	0.63	0.64	0.72	0.74
34	Wellington Road, Whitehall Rd Intersection	0.52	0.58	0.80	0.84	0.88	0.94
35	Westminster Road around no. 61	0.19	0.22	0.43	0.45	0.50	0.53
36	Wimbledon Grove	0.00	0.08	0.41	0.43	0.44	0.46

Table 8-9 Maximum Flood Depth at Selected Locations Under Future Climate Conditions

ID	Location	10yr		50yr		100yr	
		Pre	Post	Pre	Post	Pre	Post
1	Best St/Black Creek	0.24	0.24	0.59	0.62	0.80	0.83
2	Black Creek at Wright St	0.22	0.25	0.78	0.79	0.95	1.00
3	Bryan Heath Park RB	0.30	0.34	0.76	0.78	0.93	0.96
4	Bryan Heath Park RB2	0.08	0.27	0.91	0.94	1.10	1.13
5	Bryan Heath Park LB	0.28	0.37	0.64	0.66	0.82	0.84
6	Corner of Fitzherbert& the Strand	0.43	0.44	0.81	0.82	0.89	0.90
7	Corner of Hyde & Arthur St	0.00	0.00	0.14	0.17	0.42	0.45
8	Corner of Parkway Dr & Wainuiomata Rd	0.80	0.80	1.17	1.17	1.22	1.22
9	Countdown Car Park	0.25	0.25	0.59	0.59	0.66	0.66
10	End of Trelawney St	0.00	0.00	0.70	0.72	0.75	0.76
11	Fernlea School	0.18	0.18	0.29	0.29	0.34	0.34
12	Hyde St near number 28	0.05	0.05	0.54	0.56	0.83	0.89
13	Karamu St	0.42	0.42	0.68	0.68	0.78	0.78
14	Kendal Grove	0.40	0.40	0.47	0.47	0.49	0.49
15	Konini St	0.51	0.52	0.78	0.78	0.88	0.88
16	Main Rd near number 71	0.27	0.27	0.32	0.32	0.53	0.57
17	Main Rd near number 40	0.26	0.26	0.39	0.39	0.45	0.45
18	Mary Crowther Park	0.10	0.14	0.40	0.42	0.47	0.51
19	Matthews Rd	0.09	0.10	0.21	0.24	0.25	0.30
20	Moohan St around number 67	0.16	0.17	0.34	0.37	0.53	0.55
21	Moohan St at number 79	0.06	0.07	0.54	0.58	0.79	0.81
22	Nelson Crescent Bridge	0.00	0.00	0.39	0.42	0.54	0.56
23	Norfolk & Honey St	0.00	0.00	0.38	0.42	0.47	0.51
24	Norfolk St between Upper Fitzherbert & Wellington Road	0.38	0.43	0.59	0.63	0.66	0.70
25	Parkway Dr at Waiu St	0.35	0.35	0.57	0.57	0.62	0.62
26	Parkway Dr near Mohaka St	0.27	0.27	0.41	0.41	0.46	0.46
27	St Claudine Thevenet School Field	0.29	0.30	0.77	0.78	0.89	0.92
28	Totara St around 61	0.53	0.53	0.61	0.61	0.67	0.67
29	Upper Konini St	0.16	0.16	0.33	0.33	0.39	0.39
30	Wainuiomata High School Field	0.77	0.77	1.15	1.10	1.31	1.30
31	Waiu St /Mountain Bike Park	0.15	0.15	0.47	0.47	0.53	0.53
32	Wellington Road near Devon St	0.00	0.07	0.37	0.42	0.51	0.56
33	Wellington Road near Parkway D	0.49	0.51	0.73	0.75	0.84	0.88
34	Wellington Road, Whitehall Rd Intersection	0.67	0.70	0.89	0.95	0.97	1.06
35	Westminster Road around no. 61	0.27	0.30	0.50	0.54	0.63	0.68
36	Wimbledon Grove	0.15	0.31	0.45	0.46	0.48	0.49

Table 8-10: Inundation Time (hours) where Depth is Greater than 50mm, Current Climate Conditions

ID	Location	10yr		50yr		100yr	
		Pre	Post	Pre	Post	Pre	Post
1	Best St/Black Creek	2.17	2.17	4.75	4.75	6.83	6.83
2	Black Creek at Wright St	0.00	0.00	3.00	3.08	4.08	4.08
3	Bryan Heath Park RB	0.25	0.83	3.83	3.83	5.00	5.08
4	Bryan Heath Park RB2	0.00	0.00	11.50	11.58	11.67	11.75
5	Bryan Heath Park LB	0.00	0.00	11.67	11.75	11.83	11.92
6	Corner of Fitzherbert& the Strand	9.67	9.67	11.33	11.50	11.75	11.83
7	Corner of Hyde & Arthur St	0.00	0.00	0.00	0.00	1.17	1.33
8	Corner of Parkway Dr & Wainuiomata Rd	11.92	11.92	12.17	12.17	12.42	12.42
9	Countdown Car Park	0.00	0.00	11.67	11.67	11.75	11.75
10	End of Trelawney St	0.00	0.00	11.92	12.00	12.00	12.08
11	Fernlea School	1.25	1.25	2.50	2.50	3.92	3.92
12	Hyde St near number 28	0.00	0.00	2.42	2.42	3.58	3.58
13	Karamu St	5.33	5.33	11.33	11.33	14.83	14.83
14	Kendal Grove	11.92	12.00	12.17	12.17	12.58	12.58
15	Konini St	7.42	7.42	14.92	14.92	16.92	16.92
16	Main Rd near number 71	1.42	1.42	3.58	3.58	4.42	4.42
17	Main Rd near number 40	11.58	11.58	12.00	12.00	12.75	12.75
18	Mary Crowther Park	0.00	0.00	11.92	12.00	12.00	12.08
19	Matthews Rd	0.00	0.00	1.75	1.83	2.33	2.33
20	Moohan St around number 67	11.08	11.08	11.92	12.00	12.00	12.00
21	Moohan St at number 79	0.00	0.00	2.25	2.25	3.50	3.50
22	Nelson Crescent Bridge	0.00	0.00	1.83	1.92	3.25	3.25
23	Norfolk & Honey St	0.00	0.00	11.75	11.83	11.83	11.92
24	Norfolk St between Upper Fitzherbert & Wellington Road	11.50	11.58	11.92	11.92	12.50	12.50
25	Parkway Dr at Waiu St	11.83	11.83	12.00	12.00	12.33	12.33
26	Parkway Dr near Mohaka St	1.08	1.08	4.42	4.42	6.92	6.92
27	St Claudine Thevenet School Field	0.00	0.58	3.17	3.17	4.17	4.25
28	Totara St around 61	11.92	11.92	12.00	12.00	12.17	12.17
29	Upper Konini St	0.17	0.17	3.17	3.17	4.58	4.58
30	Wainuiomata High School Field	7.08	7.08	14.58	14.58	15.42	15.42
31	Waiu St /Mountain Bike Park	0.00	0.00	12.08	12.08	12.08	12.08
32	Wellington Road near Devon St	0.00	0.00	1.92	2.00	3.33	3.25
33	Wellington Road near Parkway D	11.83	11.83	12.00	12.00	12.25	12.25
34	Wellington Road, Whitehall Rd Intersection	11.75	11.92	12.00	12.00	12.08	12.08
35	Westminster Road around no. 61	11.92	11.92	12.08	12.17	12.25	12.33
36	Wimbledon Grove	0.00	0.42	2.75	2.75	3.58	3.42

Table 8-11: Inundation time (hours) where Depth is Greater than 50mm, Future Climate Conditions

ID	Location	10yr		50yr		100yr	
		Pre	Post	Pre	Post	Pre	Post
1	Best St/Black Creek	3.25	3.25	7.25	7.25	9.17	9.17
2	Black Creek at Wright St	1.25	1.33	4.25	4.25	5.50	5.58
3	Bryan Heath Park RB	1.92	2.17	5.17	5.17	6.58	6.58
4	Bryan Heath Park RB2	10.50	11.08	11.67	11.17	11.83	11.92
5	Bryan Heath Park LB	11.25	11.50	11.83	11.33	12.00	12.08
6	Corner of Fitzherbert& the Strand	10.75	10.75	11.83	11.25	11.92	11.92
7	Corner of Hyde & Arthur St	0.00	0.00	1.33	1.58	2.92	3.08
8	Corner of Parkway Dr & Wainuiomata Rd	12.00	12.00	12.42	11.83	13.33	13.33
9	Countdown Car Park	11.50	11.50	11.83	11.25	11.92	11.92
10	End of Trelawney St	0.00	0.00	12.00	11.50	12.08	12.08
11	Fernlea School	1.58	1.58	3.92	3.92	5.25	5.25
12	Hyde St near number 28	0.00	0.17	3.75	3.67	4.75	4.75
13	Karamu St	7.42	7.42	15.00	14.42	16.83	16.83
14	Kendal Grove	12.00	12.00	12.58	12.00	13.42	13.42
15	Konini St	10.08	10.08	17.08	16.50	17.67	17.67
16	Main Rd near number 71	1.83	1.83	4.50	4.58	6.42	6.58
17	Main Rd near number 40	11.67	11.75	12.83	12.25	13.67	13.67
18	Mary Crowther Park	11.17	11.83	12.00	11.50	12.25	12.33
19	Matthews Rd	0.92	1.08	2.42	2.42	3.25	3.33
20	Moohan St around number 67	11.58	11.58	12.00	11.42	12.25	12.33
21	Moohan St at number 79	0.67	0.75	3.67	3.67	4.75	4.92
22	Nelson Crescent Bridge	0.00	0.00	3.33	3.33	4.33	4.42
23	Norfolk & Honey St	0.00	0.00	11.83	11.33	11.92	12.00
24	Norfolk St between Upper Fitzherbert & Wellington Road	11.75	11.75	12.58	12.00	13.50	13.50
25	Parkway Dr at Waiu St	11.92	11.92	12.33	11.75	13.08	13.08
26	Parkway Dr near Mohaka St	1.67	1.67	7.17	7.17	9.25	9.25
27	St Claudine Thevenet School Field	1.42	1.42	4.42	4.42	5.83	5.83
28	Totara St around 61	12.00	12.00	12.17	11.58	12.33	12.33
29	Upper Konini St	1.00	1.00	4.83	4.83	7.08	7.08
30	Wainuiomata High School Field	9.00	9.08	15.58	15.00	17.08	17.08
31	Waiu St /Mountain Bike Park	11.92	11.92	12.08	11.50	12.25	12.25
32	Wellington Road near Devon St	0.00	0.42	3.50	3.42	4.42	4.42
33	Wellington Road near Parkway D	11.92	11.92	12.25	11.67	12.75	12.83
34	Wellington Road, Whitehall Rd Intersection	11.92	11.92	12.08	11.50	12.42	12.50
35	Westminster Road around no. 61	12.00	12.00	12.33	11.83	12.58	12.67
36	Wimbledon Grove	0.92	1.67	3.58	3.50	4.33	4.25

8.6.2 Flood Hazard Mapping

The maximum values for each triangle in the 2-D mesh zone were exported for the flood hazard maps. The mesh triangles were assigned an appropriate classification value based on the criteria in Table 8-12: Flood Hazard Classification Category. A thematic map was then developed based on the three classifications, potential hazard, minor hazard and significant hazard as detailed in Table 8-12. Flood Hazard maps are presented in Appendix F

The flood hazard mapping indicates that in the 100yr event there are a number of areas with a significant hazard. Some of these are in parks and playing fields, but others are on main thoroughfares such as Wellington Road, Wainuiomata Road, and in residential areas such as around Hyde St and Best St.

The areas classified as significant hazard are inundated for a range of durations, from 2 hours on Matthews Road and Kendall Grove to up to 12 hours at Parkway Drive near Mohaka Street.

The proposed development scenario does not result in new areas of significant hazard, but it does increase the extent of the areas classified as being a hazard. One notable location is Wellington Road above Enfield St, where the extent of significant hazard is increased by the development work.

Table 8-12: Flood Hazard Classification Category

Hazard Classification	Description	Depth – Velocity Criteria
1	Potential Hazard	0.05 m < Depth < 0.1 m
2	Minor Hazard	0.1 m ≤ Depth < 0.3 m & Velocity < 2.0 ms-1
3	Significant Hazard	Depth ≥ 0.3 m or Depth ≥ 0.1 m & Velocity ≥ 2.0 ms-1

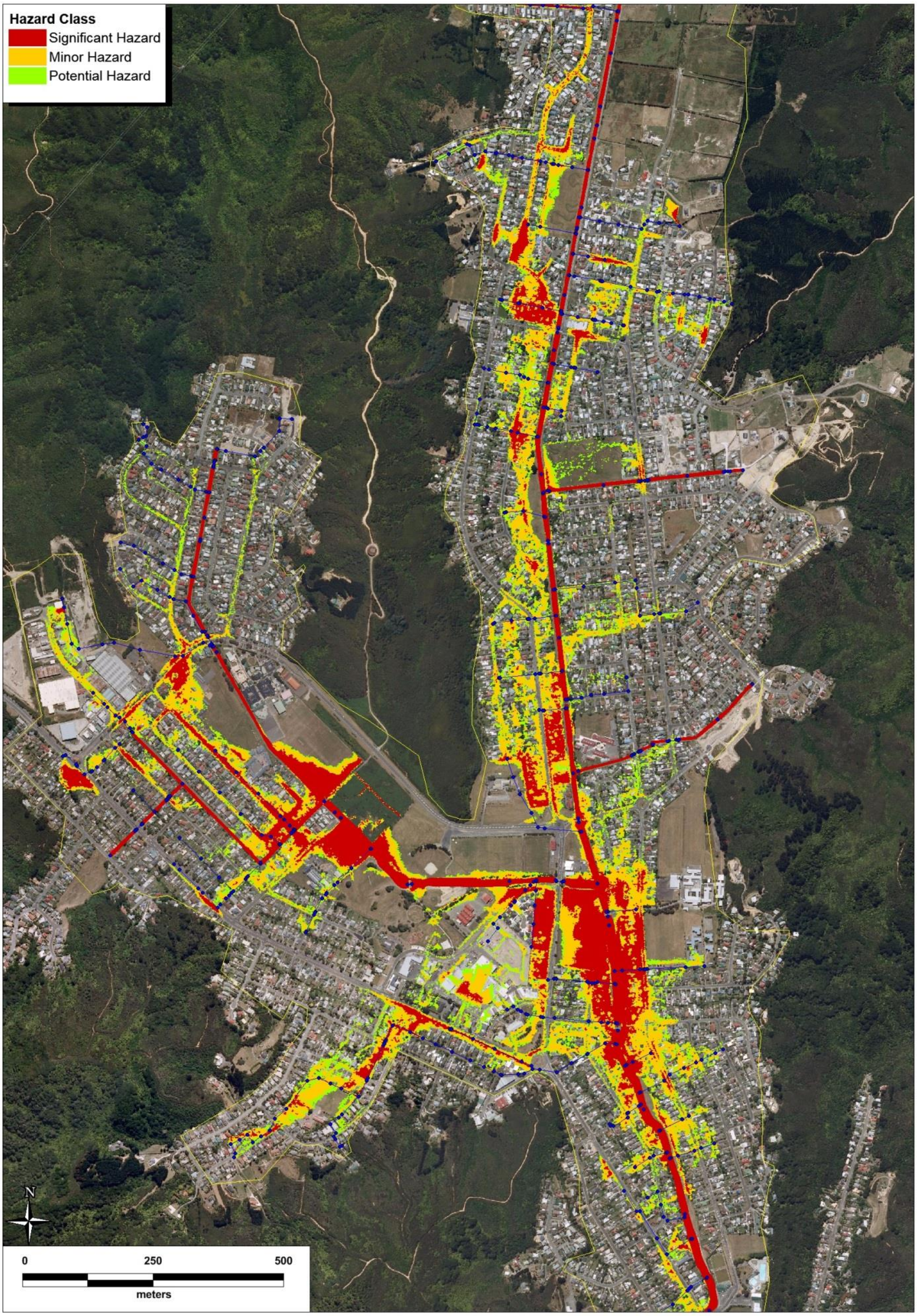


Figure 8-11: Flood Hazard Map for the 100yr ARI event under current development and Climate

8.6.3 Culvert and Bridge Capacity

Table 8-13 and Table 8-14 provide the design flow and capacity information for the modelled bridges and culverts. Table 8-14 shows that while the capacity of most of the culverts is exceeded during a 100yr ARI event, only 4 of the 17 culverts modelled are under capacity for the 10yr ARI event.

Table 8-15 shows which of the culverts and bridges were overtopped and/or surcharged during the 10yr and 100yr ARI events for both the pre and post development scenarios**.** As in Table 8-14, Table 8-15 shows that the majority of the culverts perform well under the current climate conditions and development scenario. Only 5 culverts appear to be surcharged or overtopped. The results also show there is no difference in culvert performance under the proposed development scenario for the 10yr ARI event, although water levels are generally slightly higher under the proposed development conditions.

Similarly, there is practically no difference between the development scenarios for the 100yr ARI event, except at the pedestrian bridge near Weymouth Grove. At this location it appears water level is approximately 0.4m higher under the proposed development conditions. This fits with the long sections in Figure 8-1 as the pedestrian bridge is situated approximately 300m upstream of the Norfolk Rd Bridge.

Table 8-13: Selected Design Flows through the modelled Bridges

Location	Link ID	Type	MPD Design Flow (m3/s)	
			10% AEP	1% AEP
Frederick Wise Park	670002R00868.1	Bridge	15.2	28.3
Nelson Cr	BC0044.1	Bridge	28.6	42.6
Weymouth Gr	BC_br_1_us.1	Bridge	17.9	29.3
Best St	BCc0067.1	Bridge	44.5	67.8
Main Rd	BCc0102.1	Bridge	52.7	85.7
Fitzherbert Park	BCc0106.1	Bridge	42.8	61.5
McKay St	BCc0112.1	Bridge	27.3	40.7
Edmonds St	BCc0126.1	Bridge	25.2	40.2
Park Drain	PWDr_B_us.1	Bridge	13.6	18.5

Table 8-14: Summary of Capacity and Design Flow for Culverts

Location	Link ID	Type	Shape	Width (mm)	Height (mm)	Culvert Capacity		MPD Design Flow (m3/s)		10% AEP Capacity Available
						Full Flow (m3/s)	Inlet Control (m3/s)	10% AEP	1% AEP	
Frederick St	SC2_ds_a.1	Culvert	CIRC	750	750	0.6	0.6	1.3	1.7	No
Wise St	670111R00967.1	Culvert	CIRC	750	750	1.9	0.6	1.4	2.2	Yes
Parkway	670033R00921.1	Culvert	CIRC	900	900	1.6	1.0	1.8	2.0	No
Parkway	670033R00921a.2	Culvert	CIRC	900	900	1.6	1.0	1.8	2.0	No
Manutuke St	670005R00899.1	Culvert	CIRC	900	900	2.7	1.0	1.1	2.5	Yes
Manutuke St	670005R00899a.1	Culvert	CIRC	900	900	2.7	1.0	1.1	2.5	Yes
Wise St	SC1_ds_c.1	Culvert	CIRC	900	900	2.5	1.0	2.4	2.7	No
Totara St	670013R00949.1	Culvert	RECT	1200	1200	1.9	1.9	0.6	1.1	Yes
Totara St	670013R00949.2	Culvert	RECT	1200	1200	1.9	1.9	0.6	1.1	Yes
Wainuiomata Rd	670088R00955.1	Culvert	RECT	1200	1200	1.9	1.9	0.5	1.4	Yes
Wainuiomata Rd	670088R00955.2	Culvert	RECT	1200	1200	1.9	1.9	0.5	1.4	Yes
Ashburn Rd	BC0197.1	Culvert	CIRC	1200	1200	7.9	2.0	0.8	2.3	Yes
Konini St	670012R00888a.1	Culvert	RECT	1400	1200	6.0	1.9	1.5	2.0	Yes
Konini St	670012R00888.1	Culvert	RECT	1400	1200	6.0	1.9	1.5	2.0	Yes
Russell Rd	BCc0094.2	Culvert	CIRC	1500	1500	7.8	3.6	3.2	4.8	Yes
Fitzherbert Rd	670071R00853.1	Culvert	RECT	6500	3350	201.9	9.0	16.2	29.5	Yes
Norfolk St	BC0172.1	Culvert	RECT	10000	2100	107.0	4.5	20.6	31.4	Yes

Table 8-15: Summary of Maximum Water Levels Upstream of Culverts, and Bridges Compared to Overtopping Level

Location	Link ID	Surcharge Level (m RL)	Overtop Level (m RL)	Upstream Max. Water Level (m RL)			
				Pre Development		Post Development	
				10% AEP	1% AEP	10% AEP	1% AEP
Federick Wise Park	670002R00868.1	87.0	87.0	86.3	87.1	86.4	87.1
Nelson Cr	BC0044.1	85.9	86.4	86.1	86.9	86.2	86.9
Weymouth Gr	BC_br_1_us.1	91.4	91.4	90.4	90.9	90.7	91.3
Best St	BCc0067.1	86.2	86.6	85.4	86.2	85.5	86.3
Main Rd	BCc0102.1	85.6	86.2	84.1	85.0	84.2	85.0
Fitzherbert Park	BCc0106.1	86.8	86.9	85.7	86.5	85.8	86.5
McKay St	BCc0112.1	87.0	87.3	86.7	87.4	86.9	87.5
Edmonds St	BCc0126.1	87.6	88.2	87.6	88.4	87.8	88.4
Park Drain	PWDr_B_us.1	87.0	87.2	86.5	87.2	86.6	87.2
Frederick St	SC2_ds_a.1	89.5	90.5	89.9	90.6	89.9	90.6
Wise St	670111R00967.1	91.5	93.8	92.7	94.7	92.7	94.7
Parkway	670033R00921.1	90.2	90.5	90.8	90.8	90.8	90.8
Parkway	670033R00921a.2	90.2	90.5	90.7	90.8	90.7	90.8
Manutuke St	670005R00899.1	96.5	98.2	96.5	98.1	96.5	98.1
Manutuke St	670005R00899a.1	96.5	98.2	96.5	98.1	96.5	98.1
Wise St	SC1_ds_c.1	90.8	92.5	91.9	92.6	91.9	92.6
Totara St	670013R00949.1	89.4	89.5	88.9	89.1	88.9	89.1
Totara St	670013R00949.2	89.4	89.5	88.9	89.1	88.9	89.1
Wainuiomata Rd	670088R00955.1	91.6	93.1	91.0	91.4	91.0	91.4
Wainuiomata Rd	670088R00955.2	91.6	93.1	91.0	91.4	91.0	91.4
Ashburn Rd	BC0197.1	97.9	97.9	97.7	98.5	97.7	98.5
Konini St	670012R00888a.1	86.5	86.7	87.0	87.5	87.0	87.5
Konini St	670012R00888.1	86.5	86.7	87.0	87.5	87.0	87.5
Russell Rd	BCc0094.2	87.8	88.5	88.1	88.9	88.3	88.9
Fitzherbert Rd	670071R00853.1	87.0	87.8	86.1	86.9	86.2	86.9
Norfolk St Bridge	BC0172.1	89.7	90.3	89.0	89.6	89.3	89.7

9 Conclusions and Recommendations

A coupled 1D-2D hydraulic model was developed in ICM following the agreed modelling methodology. The model was then validated against three rainfall events with a good fit for the 2011 event, and poor to ok fits for the 2012 and 2013 events. As the 2011 event included a more detailed analysis of spatial rainfall variability it is recommended to do a more detailed investigation into the spatial variability of the 2013 and 2012 events to improve model performance.

A sensitivity check was also completed investigating the impact of different model parameters including a 50% increase in rainfall intensity, partial and full blockage of inlets, a 1m increase in tailwater levels, and a $V^2/2G$ calculation. Analysis showed that the network was mostly sensitive to the 50% increase in rainfall.

Appropriate freeboard values were assigned across the network following results of the sensitivity analysis and discussions with Wellington Water.

- HiRDS was confirmed to provide appropriate design rainfall depths in the Black Creek Catchment. The HiRDS nested profile was selected for the modelling completed in this report.
- Comparison between the AECOM modelling report and the current Stantec model shows that that the 2004 study, the rainfall was 40% less, and therefore showed significantly less flooding than the model in this report.
- The model results show that approximately 40% of the modelled network is under capacity for the 5yr ARI event. This increases to nearly 50% of the network for the 100yr ARI event. The percentage of the network with available capacity, and not operating under backwater conditions, is approximately 14% and 4% for the 5yr ARI and 100yr ARI events respectively.
- Development in the upper reaches of the Black Creek Catchment does increase runoff. However, no significant impact in the performance of the network.
- Climate change will impact the depths and extents of flooding based on the model due to increased rainfall volume and intensity.
- Due to stormwater network overflows, the flood extents are generally restricted to roads, parks and fields in the lower magnitude events. For higher magnitude events, there is more overflow from open channels at road crossings.
- Although a 100 Ari event leads to flooding mainly in parks and fields, some main thoroughfares, such as Wellington Road and Wainuiomata Road become significant flooding hazards.

It should be noted though that only a limited amount of the stormwater network was modelled, significant portions have been interpolated. It is possible that this has led to an underestimation of the capacity of the stormwater network, and therefore a conservative estimate of flooding in the catchment. Similarly, no channel survey work was undertaken for this investigation as it was considered that the LiDAR provided an appropriate level of accuracy for large storm events. However, the representation of the open channels within the model, particularly in the vicinity of the culverts, would benefit from detailed survey.

Following this investigation, several recommendations are provided below as potential future work to improve this model:

- The inclusion of the non-modelled stormwater network.
- Survey of network assets to account for missing attribute information as outlined in Table 3-1.
- A survey of the channel cross sections along Black Creek and its tributaries , as the model calibration work indicates the channel cross sections cut from the LiDAR ground model may not provide an accurate representation of channel capacity at low flows. The channel survey should also include a detailed survey of the culvert inlets and outlets within each channel.
- It is also strongly recommended that gauge corrected rainfall radar is used in any additional validation work.

10 References

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Appendices

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Appendix A Design Rainfall Analysis

A.1 Design Depths

Since AECOM carried out their investigation of the Black Creek catchment in 2004 there has been a new version of HIRDs v3 developed by NIWA which takes into consideration additional rainfall data. The Wise Park rainfall gauge is only available in archived form from 2005 and data from this gauge would not have been available to AECOM for their study.

Stantec carried out frequency analyses for the 1 hour, 2 hour, 4 hour, 6 hour and 24 hour events for the Wainuiomata Reservoir rain gauge and for the 1 hour, 2 hour and 6 hour events for the Wise Park rainfall gauge using TIDEDA hydrometric software. Data from January 1890 to August 2014 (124 years) was collected for the 24 hour analysis at the Wainuiomata Reservoir gauge and from August 1998 to August 2014 (16 years) for the 1 hour, 2 hour, 4 hour, 6 hour events. All frequency analysis for Wise Park was carried out from January 2005 to January 2015 (10 years).

The frequency analyses were compared with HIRDS v3 data for each site (see Figure 10-1 and Figure 10-2 below)

The estimated rainfall depths for a given ARI and duration from the frequency analysis of Wise Park data were less than the output from HIRDS v3. However, there are is 10 years of data for Wise Park. The Wainuiomata Reservoir frequency analysis, which is based on a longer record (124 years for 24 hour data, 16 years for the sub daily events) has a fairly close match with results from HIRDS v3.

Based on these results, design rainfall depths from HIRDs v3 are considered appropriate for use in the Black Creek catchment and so were adopted for use in the Phase 2 analysis.

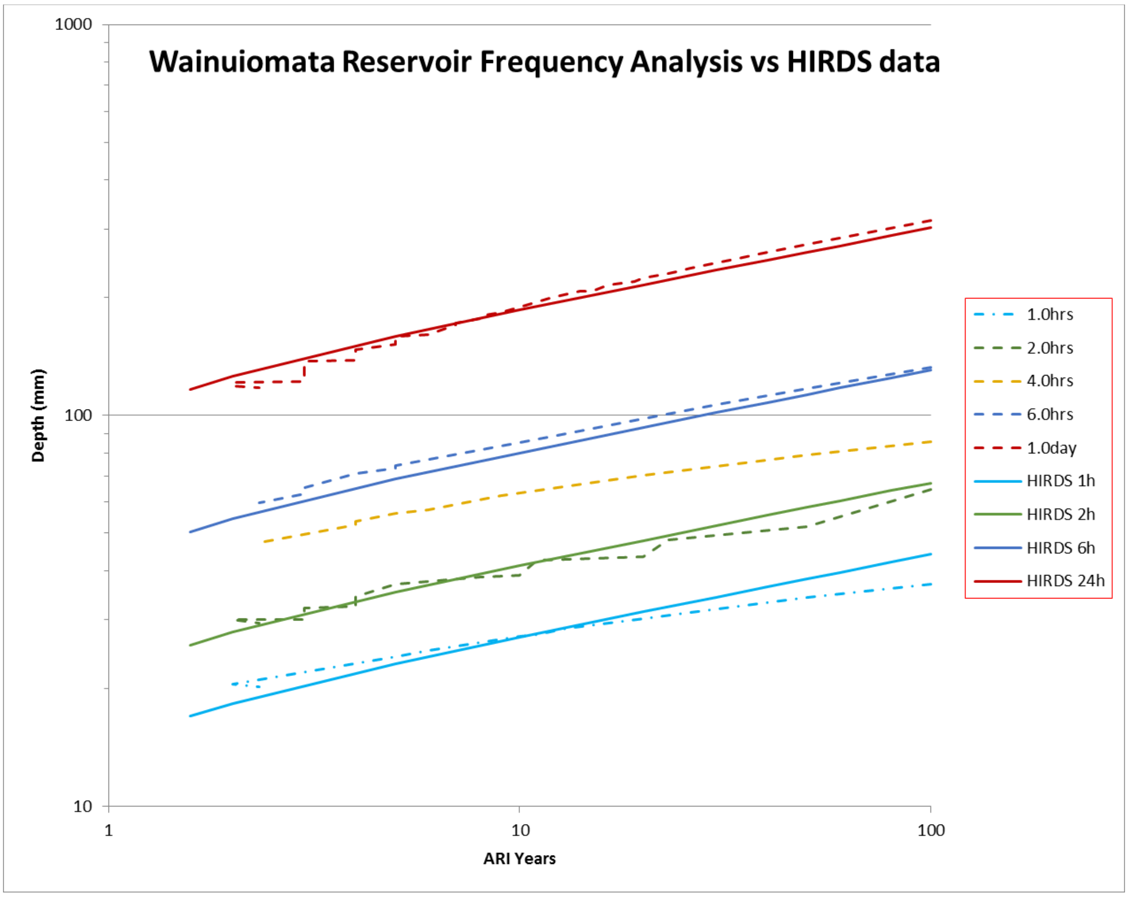


Figure 10-1: Wainuiomata Reservoir rainfall gauge frequency analysis compared with HIRDs data

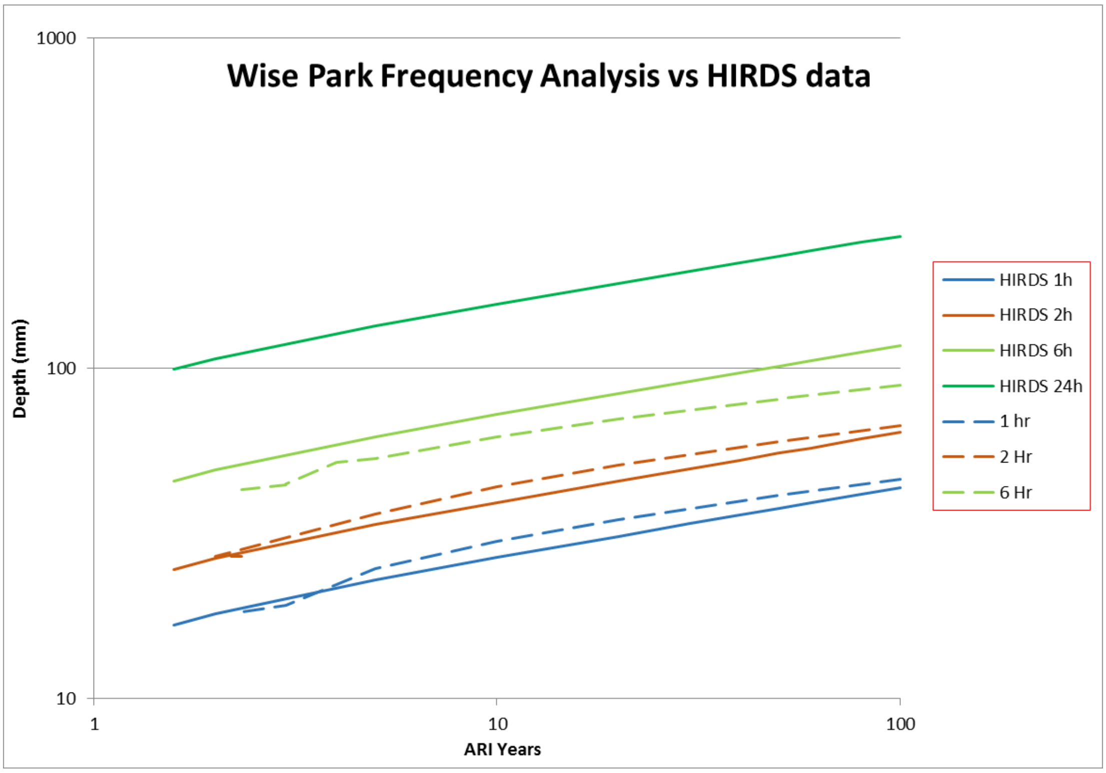


Figure 10-2: Wise Park rainfall gauge frequency analysis compared with HIRDs data

A.2 Comparison of Rainfall Temporal Profiles

The 12 largest events for Wainuiomata and Wise Park were extracted for 1 hour, 2 hour and 6 hour durations. The temporal distributions at both gauges were then compared. (Figure 10-3 to Figure 10-8).

The results show that rainfall distributions are similar at each gauge for the same rainfall event. Therefore, the Wainuiomata Reservoir gauge, which has a longer record and larger events than Wise Park is considered representative of the rainfall distribution likely within the Black Creek Catchment.

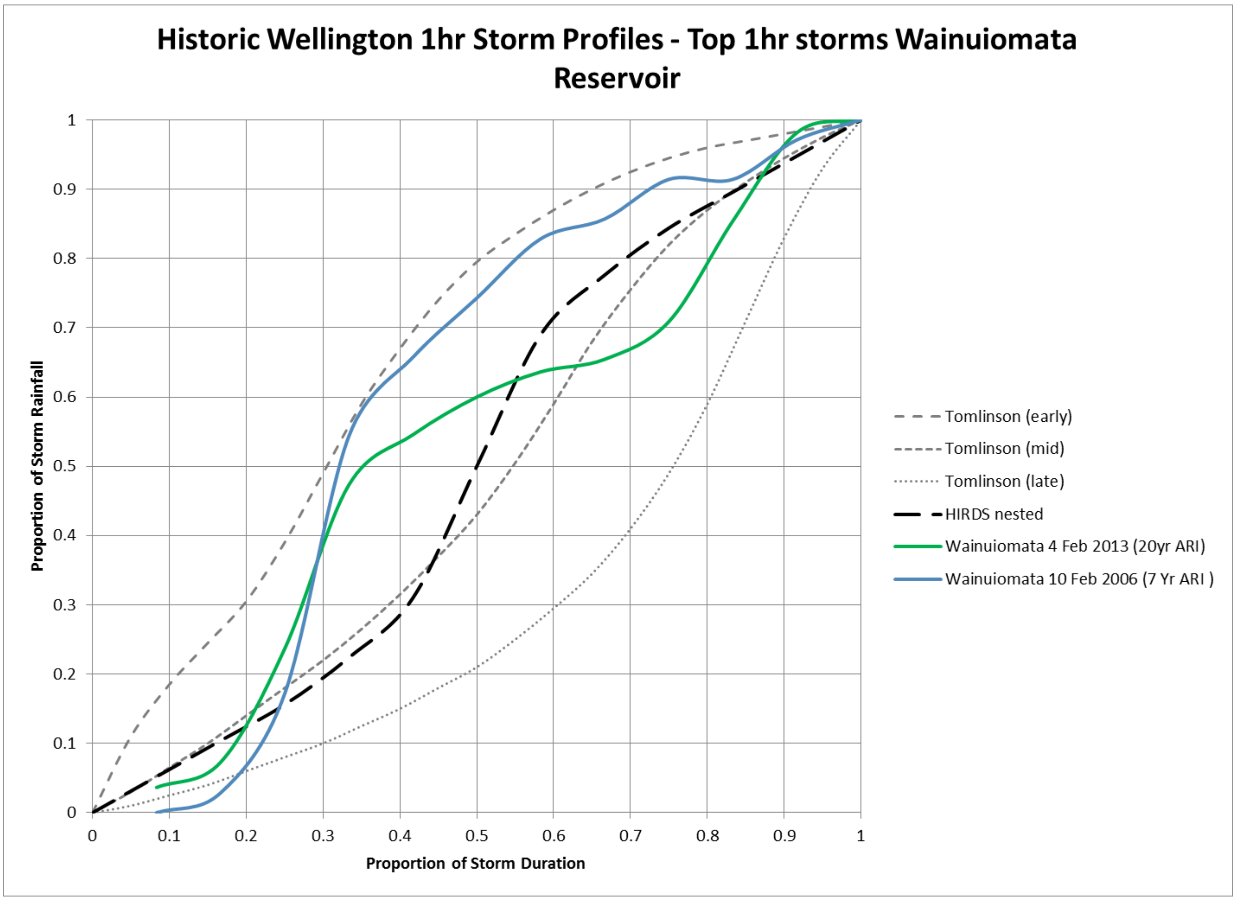


Figure 10-3: Wainuiomata Reservoir rainfall gauge 1 hour event profiles

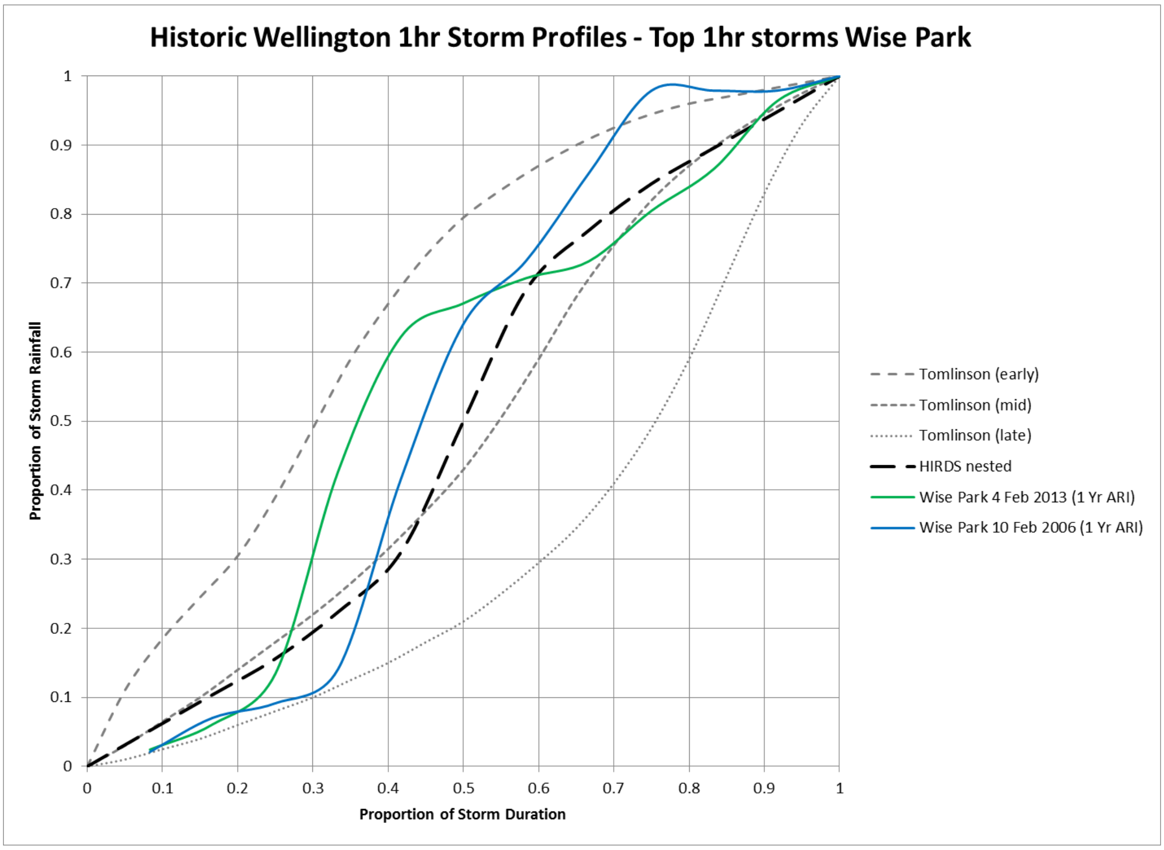


Figure 10-4: Wise Park rainfall gauge 1 hour event profiles

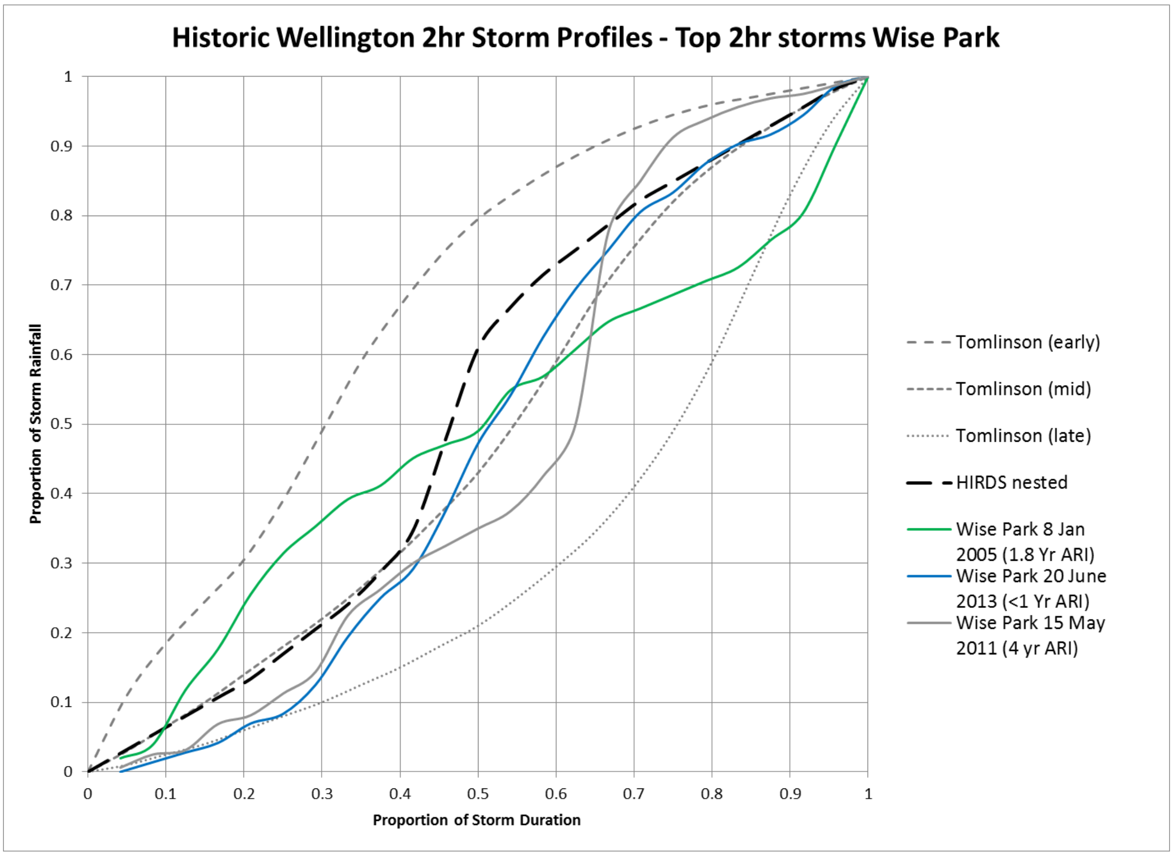


Figure 10-5: Wainuiomata Reservoir rainfall gauge 2 hour event profiles

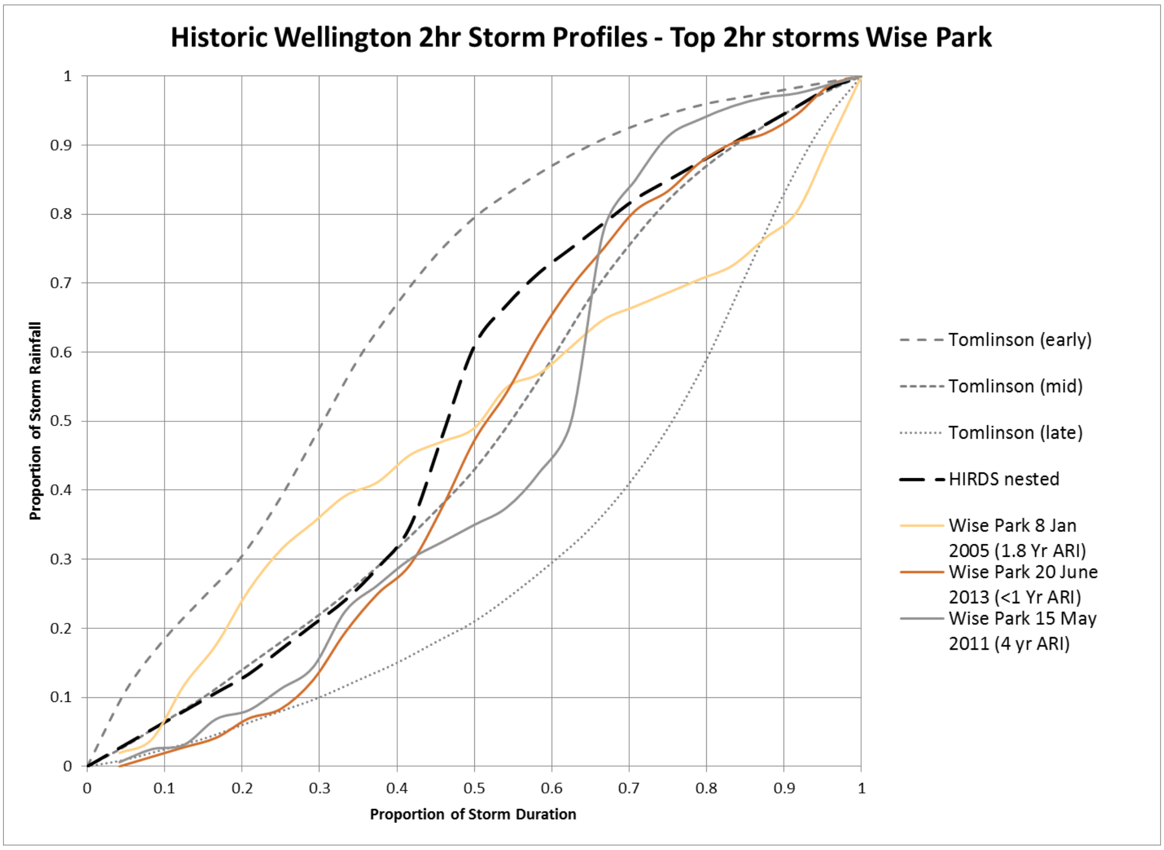


Figure 10-6: Wise Park rainfall gauge 2 hour event profiles (May 2011 event is also included as this is a validation event)

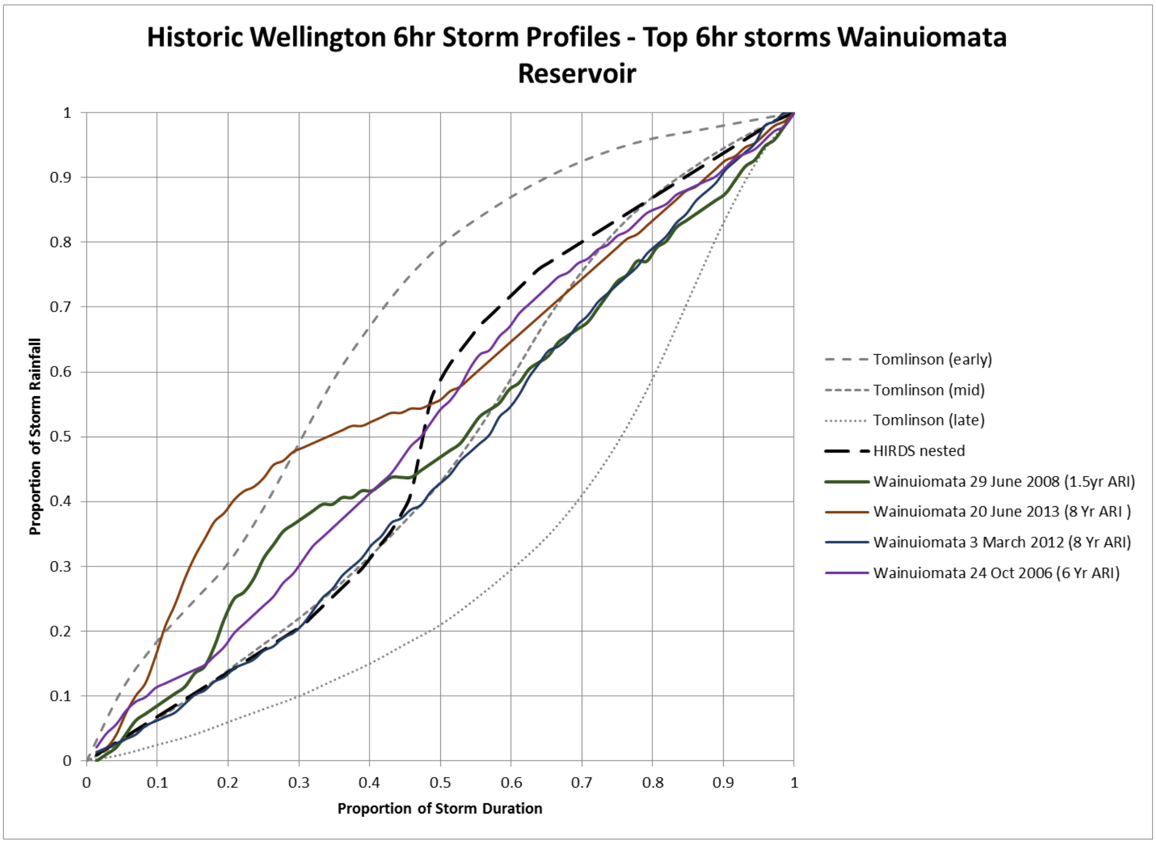


Figure 10-7: Wainuiomata Reservoir rainfall gauge 2 hour event profiles

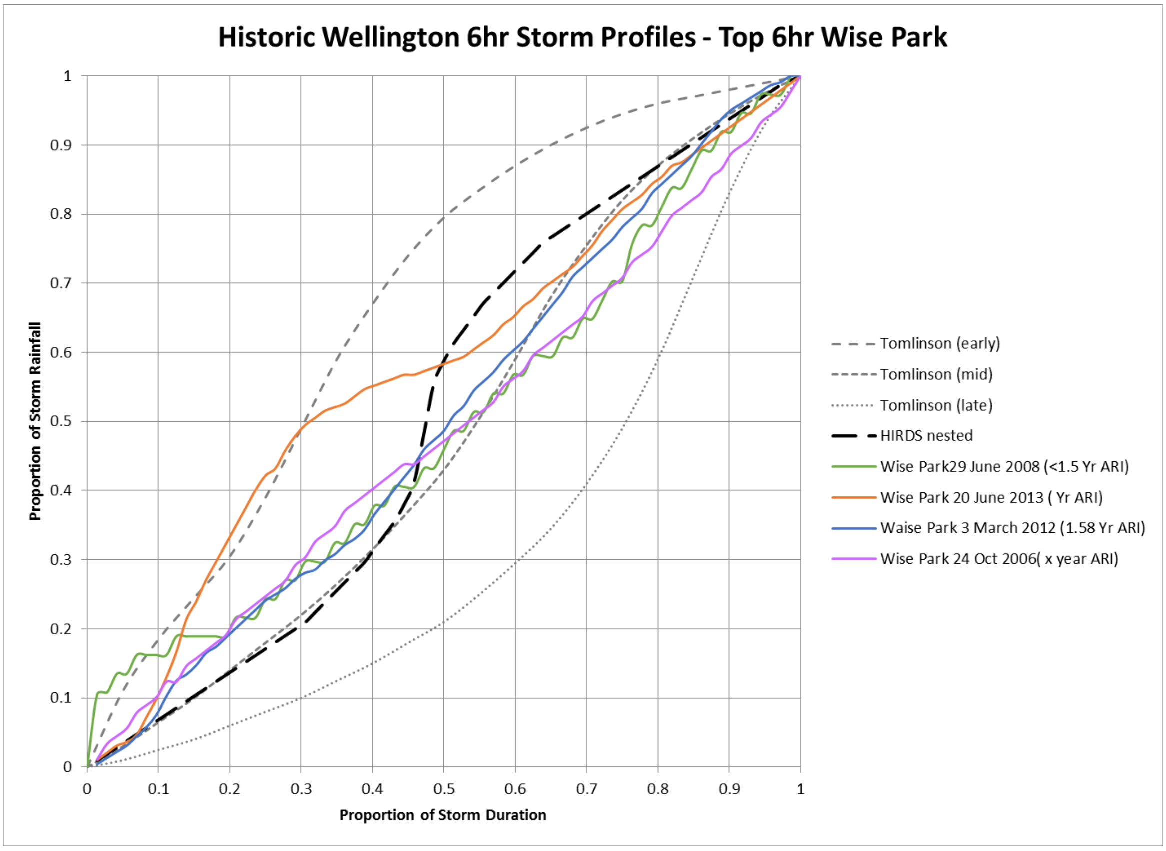


Figure 10-8: Wise Park rainfall gauge 2 hour event profiles

A.3 Comparison of Recorded Events with HIRDS

A comparison of 1, 2 and 6 hour storm events at the Wainuiomata Reservoir with respect to HIRDs design rainfall depths was completed. This identified, for example, whether a 6 hour, 20yr event has nested within it 20yr events of shorter durations for example 10 minute 20yr and 1 hour 20yr events.

Figure 10-9 to Figure 10-11 demonstrate that shorter duration events of the same ARI are not likely to be seen in the same event and in fact there is considerable variability with regard to ARI within the components of an individual storm.

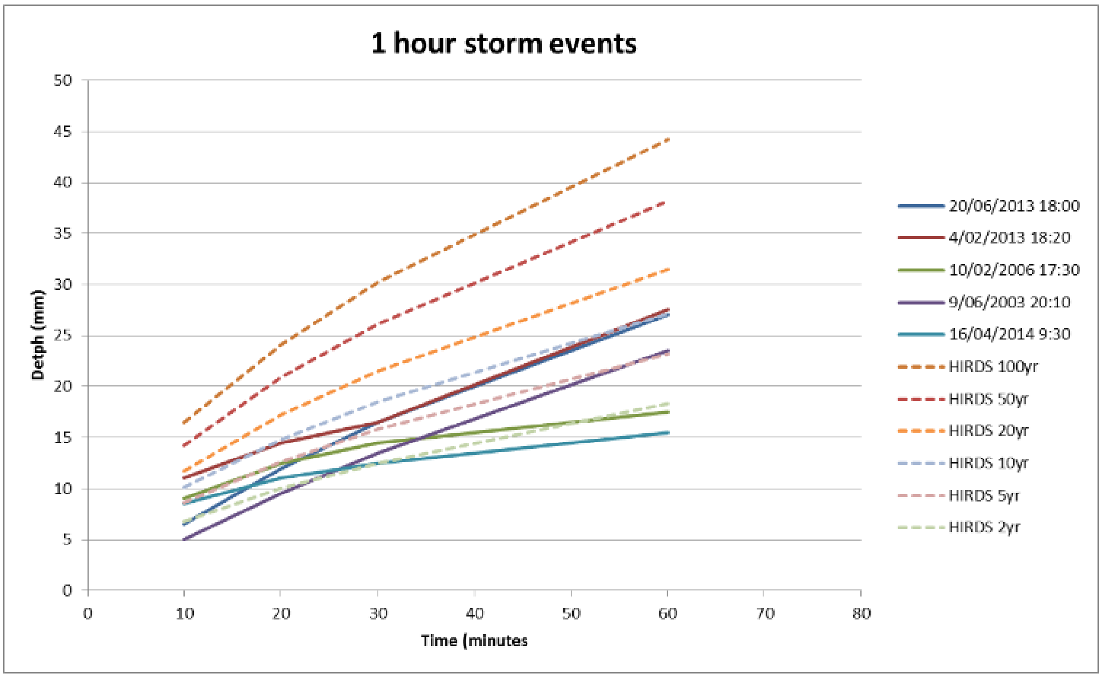


Figure 10-9: Wainuiomata Reservoir rainfall gauge 1 hour events compared to HIRDs

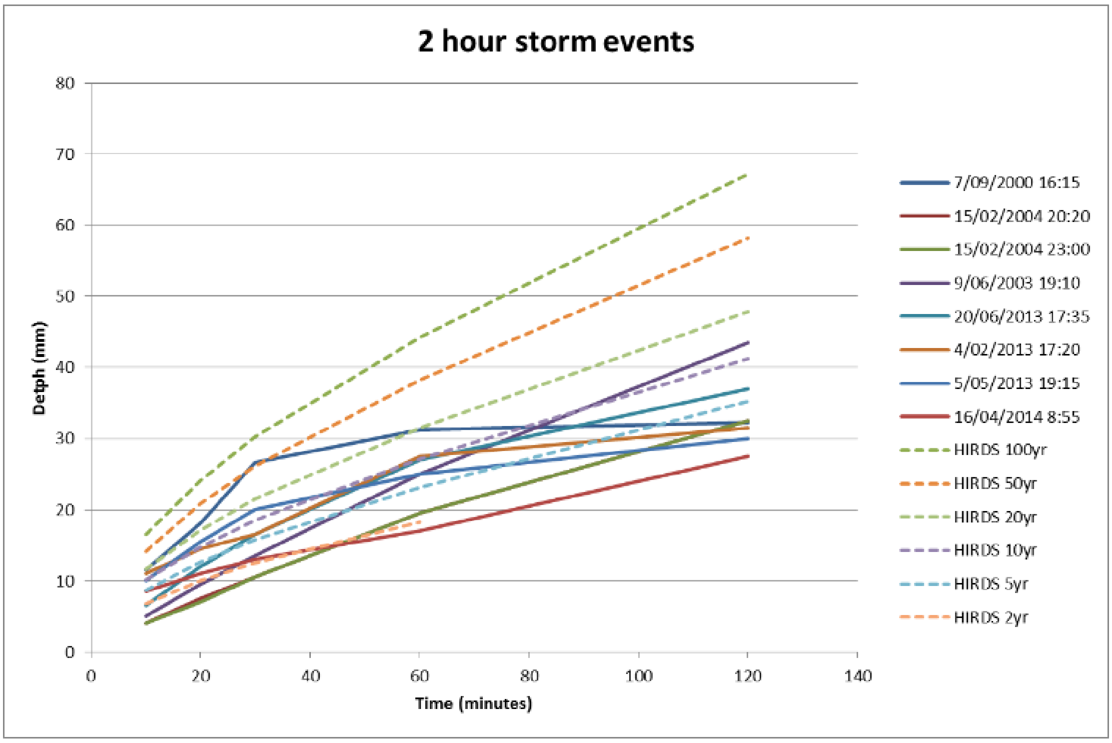


Figure 10-10: Wainuiomata Reservoir rainfall gauge 2 hour events compared to HIRDs

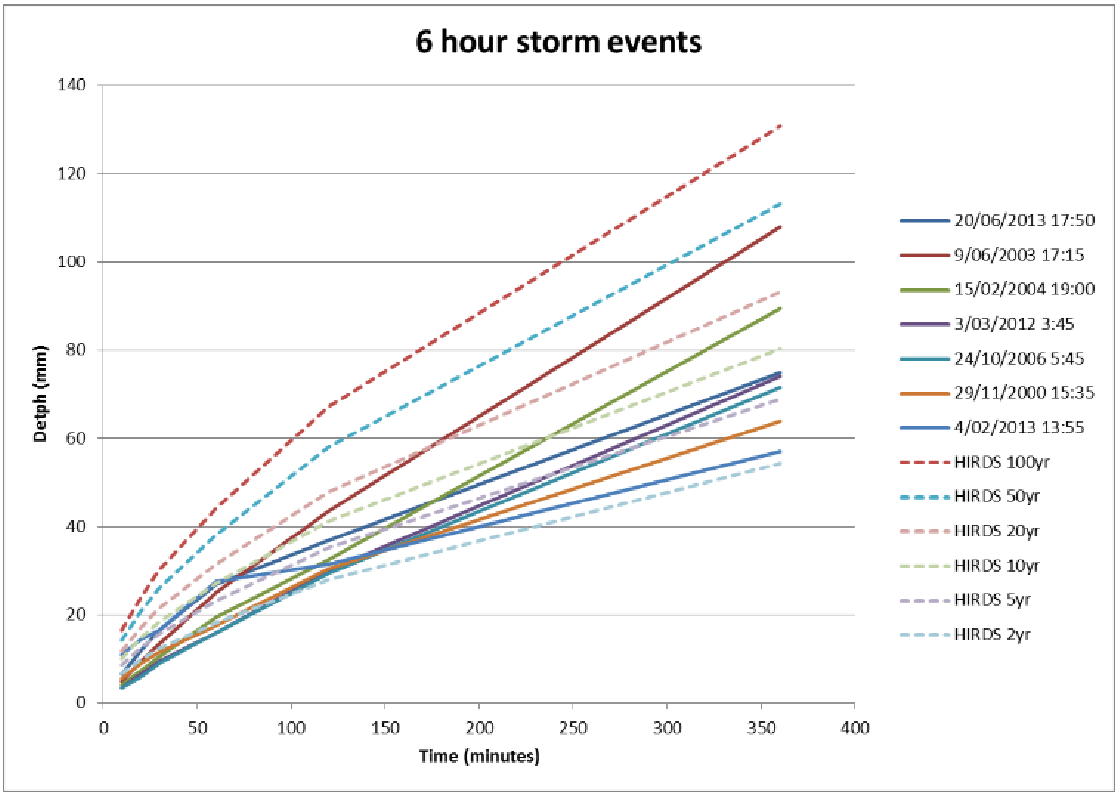


Figure 10-11: Wainuiomata Reservoir rainfall gauge 6 hour events compared to HIRDs

For the Rapid Flood Hazard assessment the TP 108 nested design rainfall distribution method was adopted. This method uses the 24 hour rainfall depth to calculate a uniform intensity across the 24 hours (termed I/24). This value was then multiplied by factors for each time period. and is based on a study of storm events in Auckland.

A comparison of the TP 108 profiles with HIRDS depths for the Wise Park showed that the intensities for the shortest durations were overestimated and the longer durations underestimated as summarised in Table 10-1 and Table 10-2.

Table 10-1: Comparison of Rainfall Depth (100Yr ARI temporal profile)

Duration	Depth in period (mm)	
	TP108	HIRDs
10 min	28.0	17
20 min	43.1	24.6
30 min	53.3	30.7
60 min	75.5	44.5
2 hr	101.9	64.8
6 hr	156.5	117.6
12hr	179.6	171.2
24hr	249.2	249.2

Table 10-2: Comparison of Rainfall Intensity (100Yr ARI temporal profile)

Time (hrs:mins)	Rainfall Intensity (mm/hr)	
	TP 108	HIRDs Nested
0:00	3.530	6.5
6:00	7.6837	8.93
9:00	9.968	13.2
10:00	14.537	13.2
11:00	22.843	20.3
11:30	39.457	27.6
11:40	49.84	36.6
11:50	90.335	45.6
12:00	168.21	102
12:10	61.262	45.6
12:15	61.262	36.6
12:20	43.61	27.6
12:30	30.112	27.6
13:00	17.652	20.3
14:00	12.46	13.2
15:00	7.7875	8.93

A.4 Rainfall Temporal Distribution

The temporal distribution of the storms was compared against the design rainfall storms of Tomlinson, HIRDS nested, and the TP 108 nested design storms for 1, 2 and 6 hours (Figure 10-13, Figure 10-14 and Figure 10-15).

The Average Variability method (Pilgrim et al 1969) was then used to obtain a representative storm temporal distribution for the 1 hour, 2 hour and 6 hour patterns. The results showed that there were more individual bursts than the HIRDs or Tomlinson's patterns (Figure 10-16 to Figure 10-22).

After discussion with Wellington Water it was decided that the Nested HIRDS pattern was the most appropriate for the Black Creek catchment.

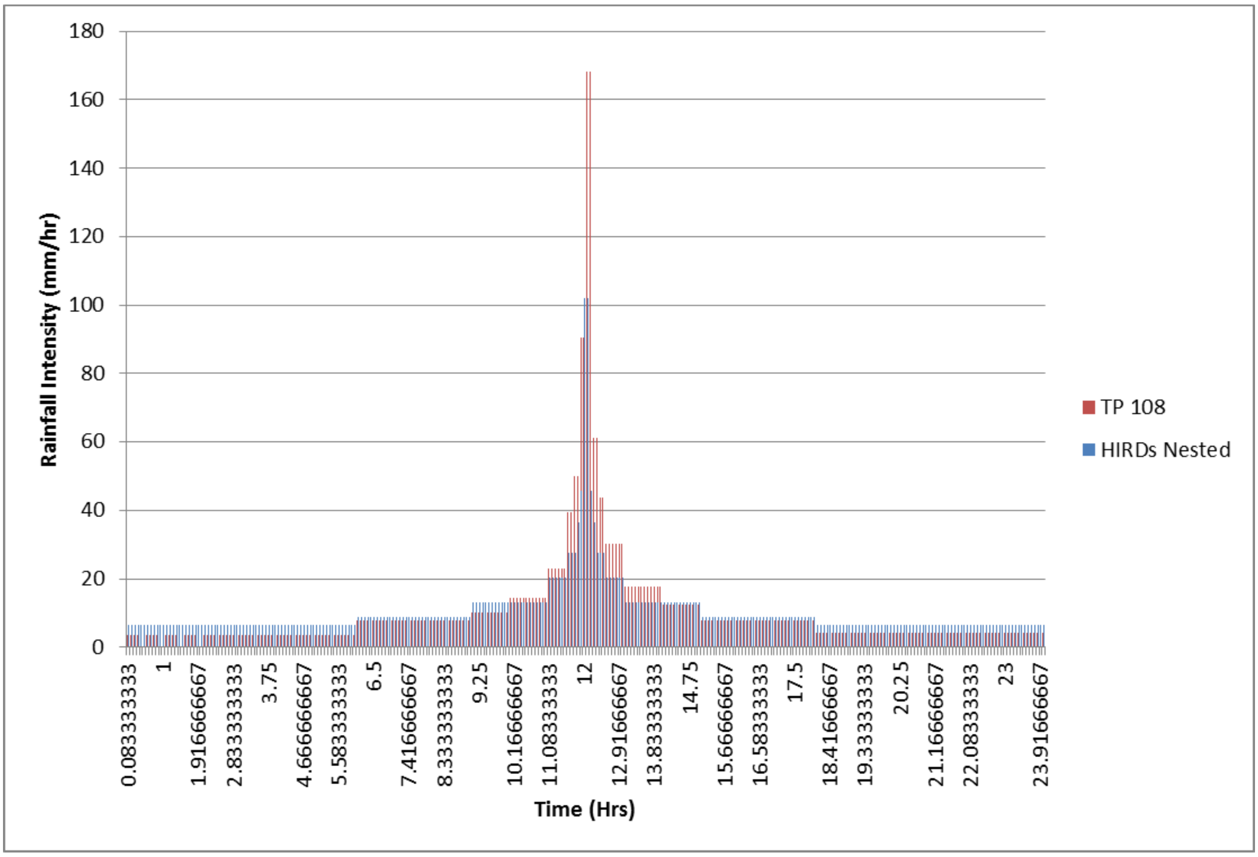


Figure 10-12: Comparison of TP 108 and average variability nested profiles

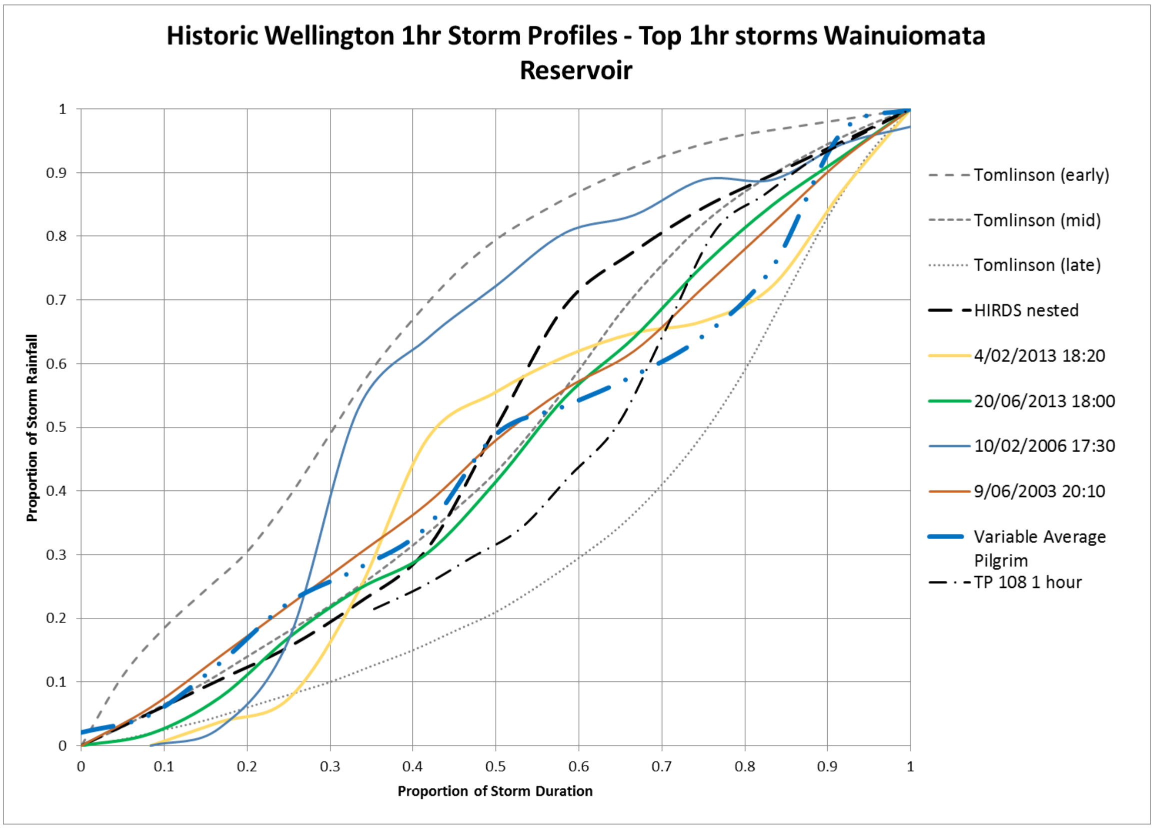


Figure 10-13: Temporal distributions for 1 hour storms at Wainuiomata Reservoir and design storms

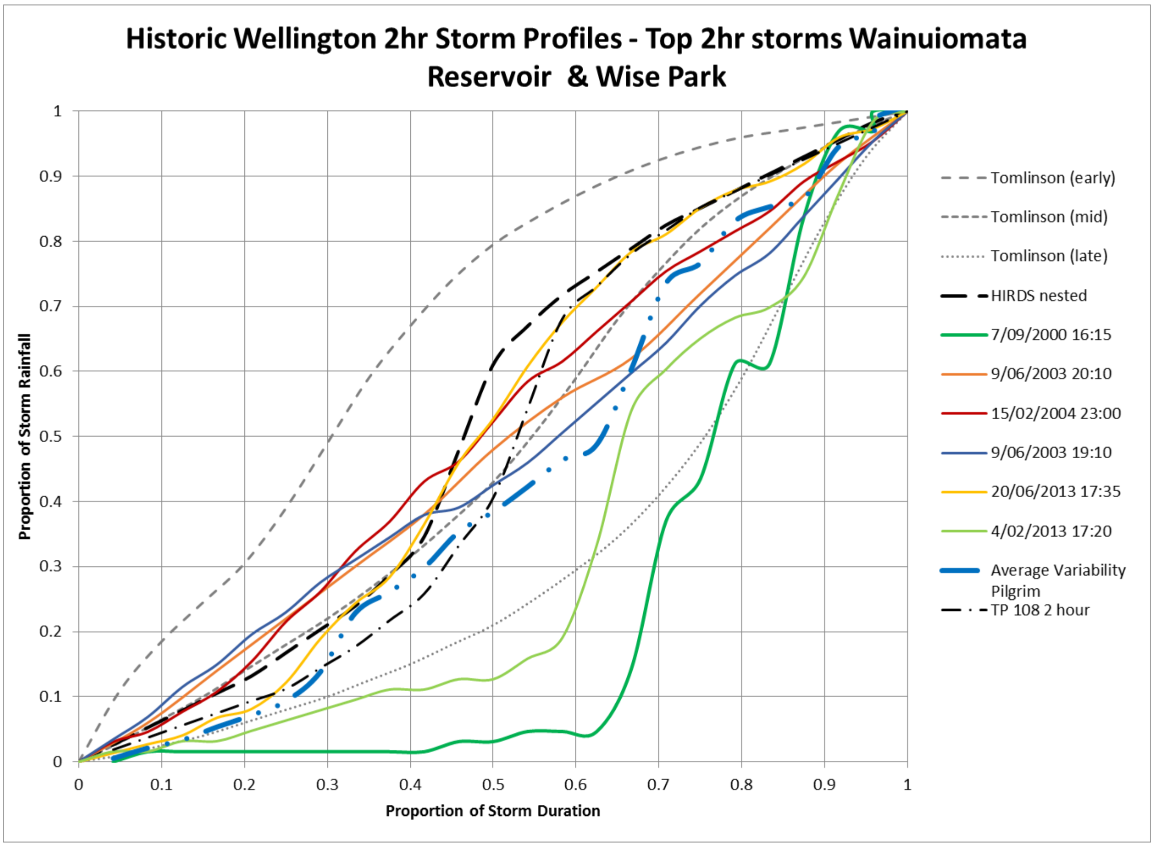


Figure 10-14: Temporal distributions for 2 hour storms at Wainuiomata Reservoir and design storms

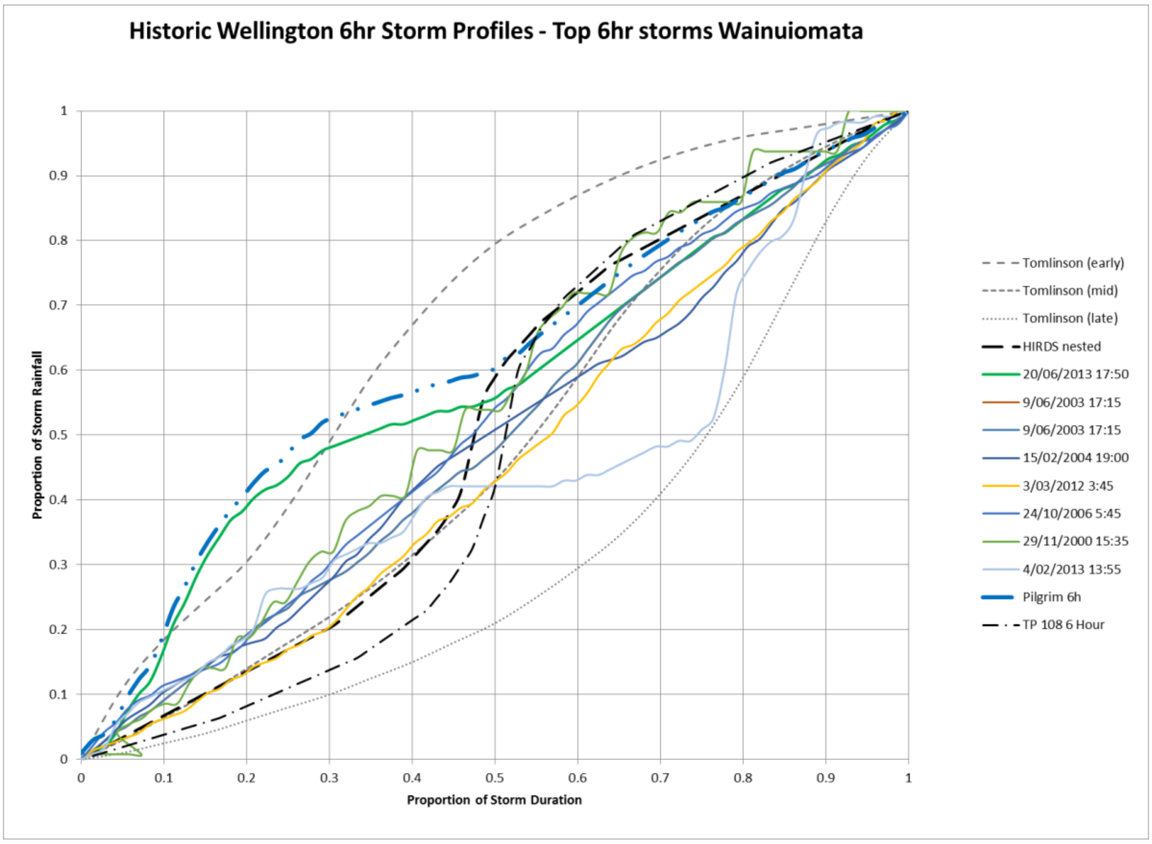


Figure 10-15: Temporal distributions for 6 hour storms at Wainuiomata Reservoir and design storms

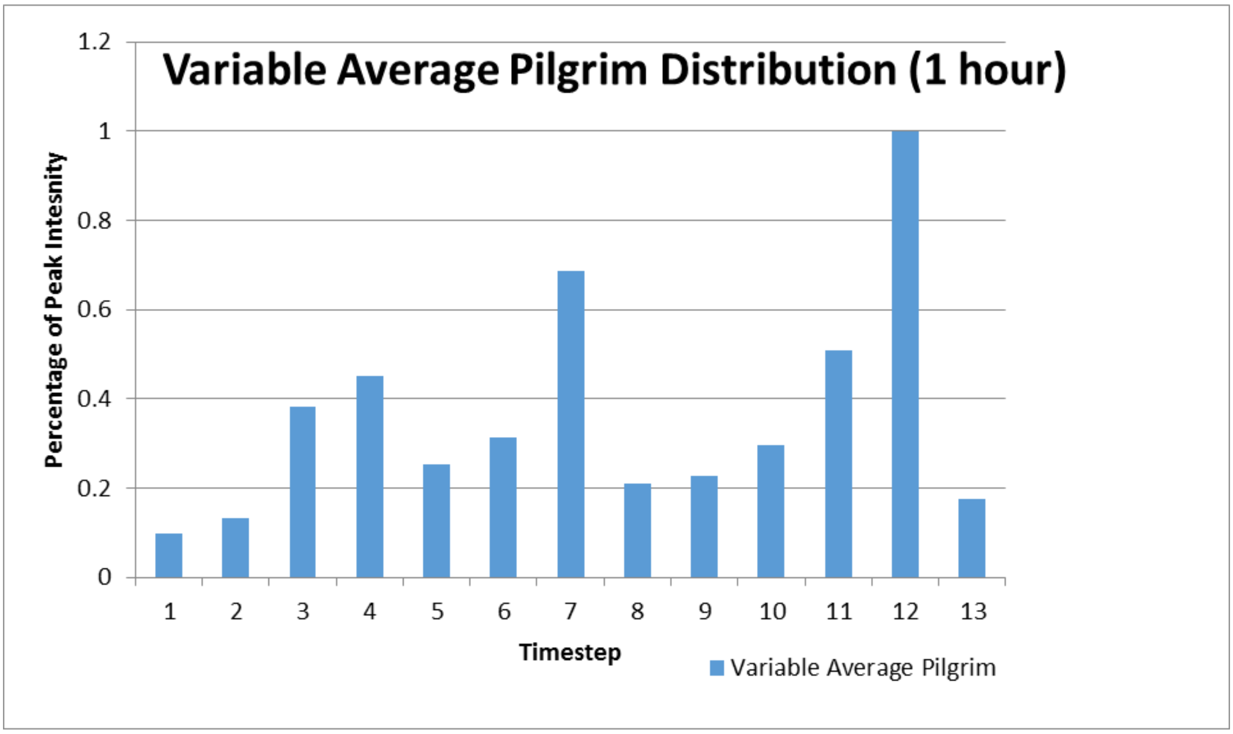


Figure 10-16: Temporal distributions for 1 hour storms using the Average Variability Method

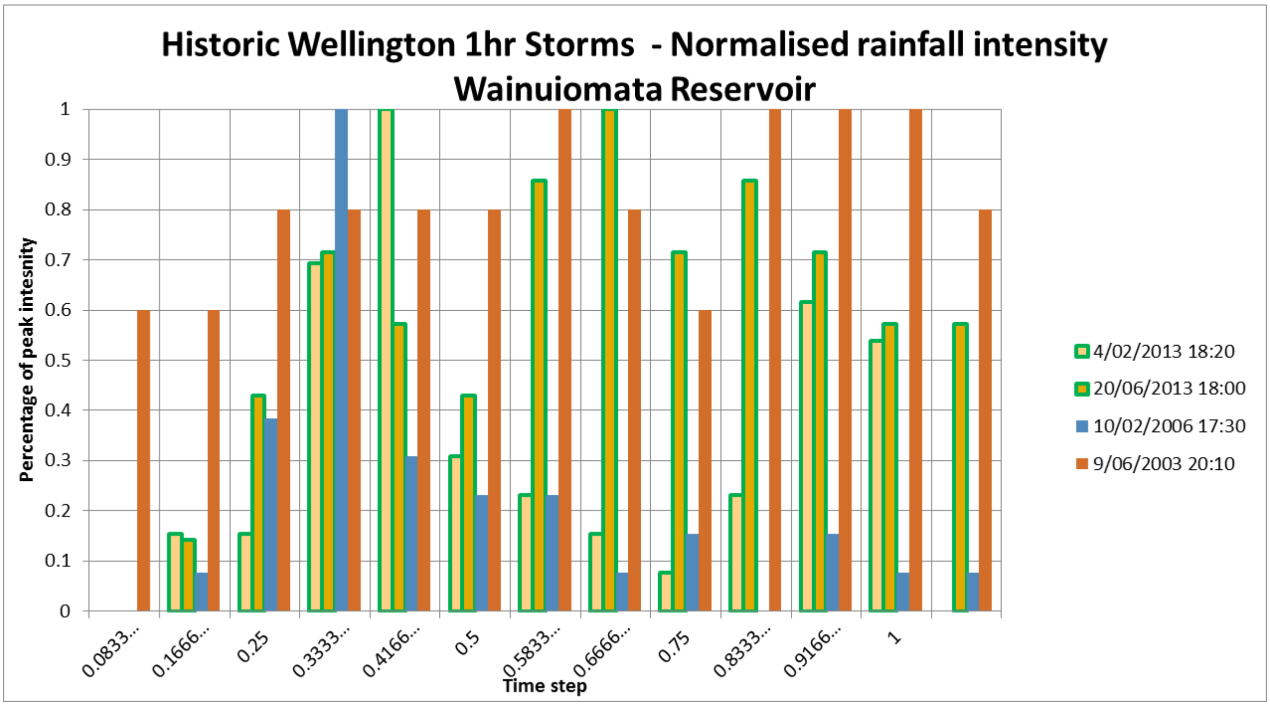


Figure 10-17: Temporal distributions for 1 hour storms at Wainuiomata Reservoir

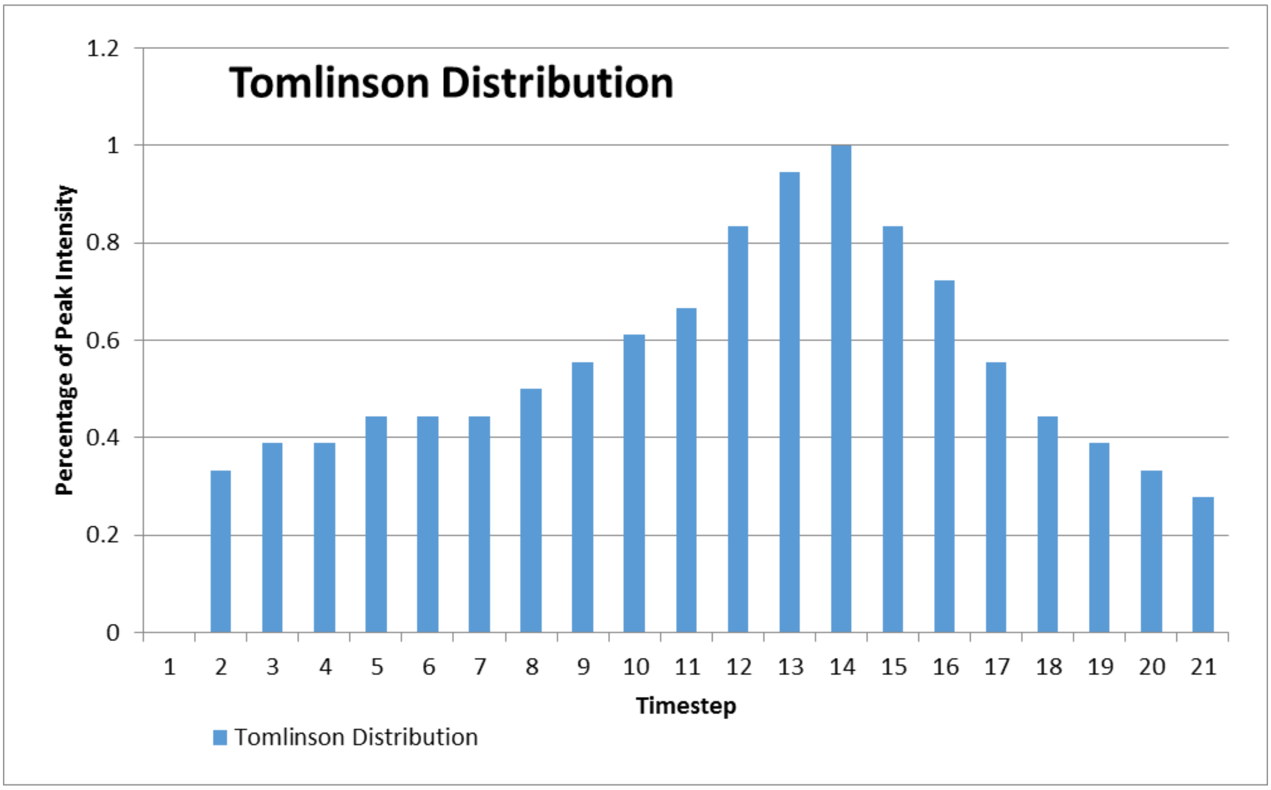


Figure 10-18: Normalised Tomlinson temporal distribution

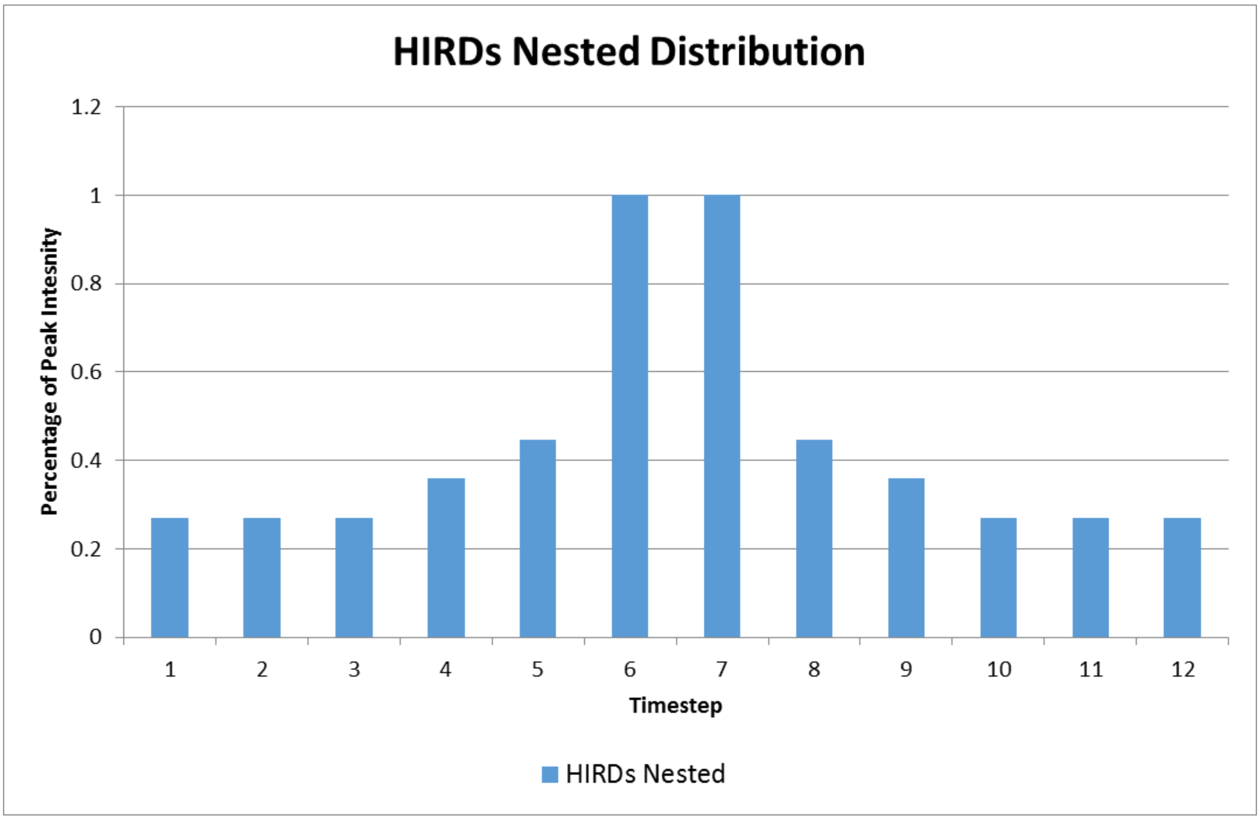


Figure 10-19: Normalised HIRDs nested temporal distribution

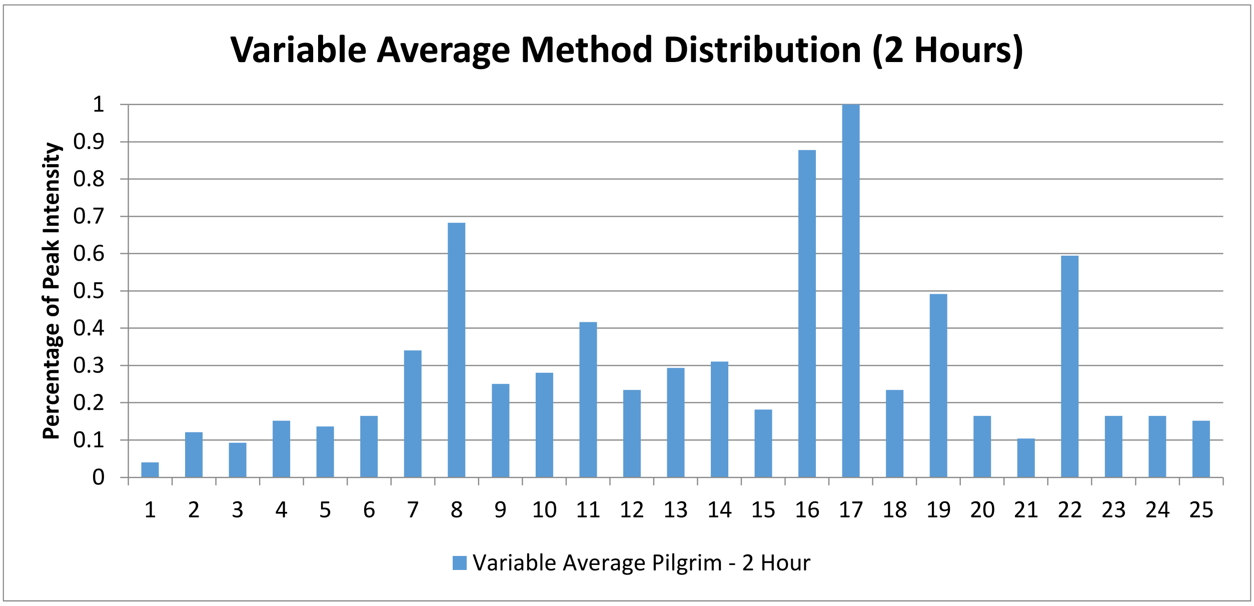


Figure 10-20: Temporal distributions for 2 hour storms using the average variability method

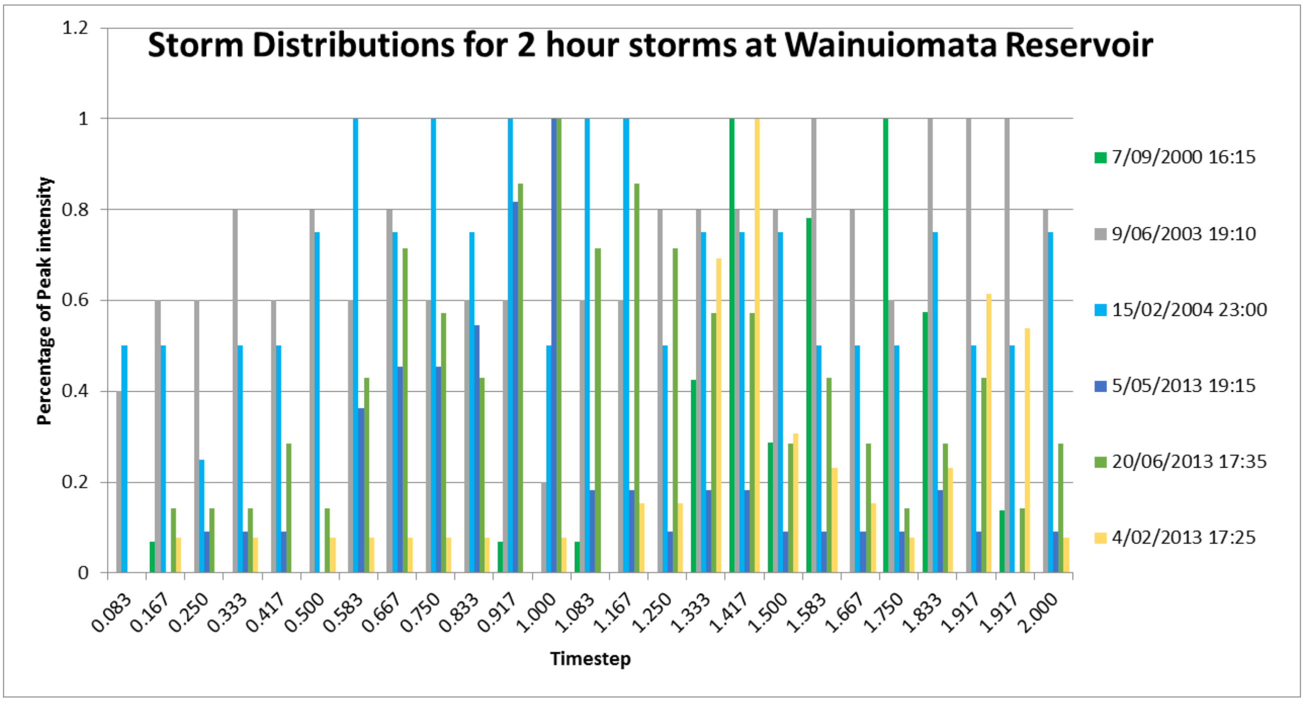


Figure 10-21: Temporal distributions for 2 hour storms at Wainuiomata Reservoir

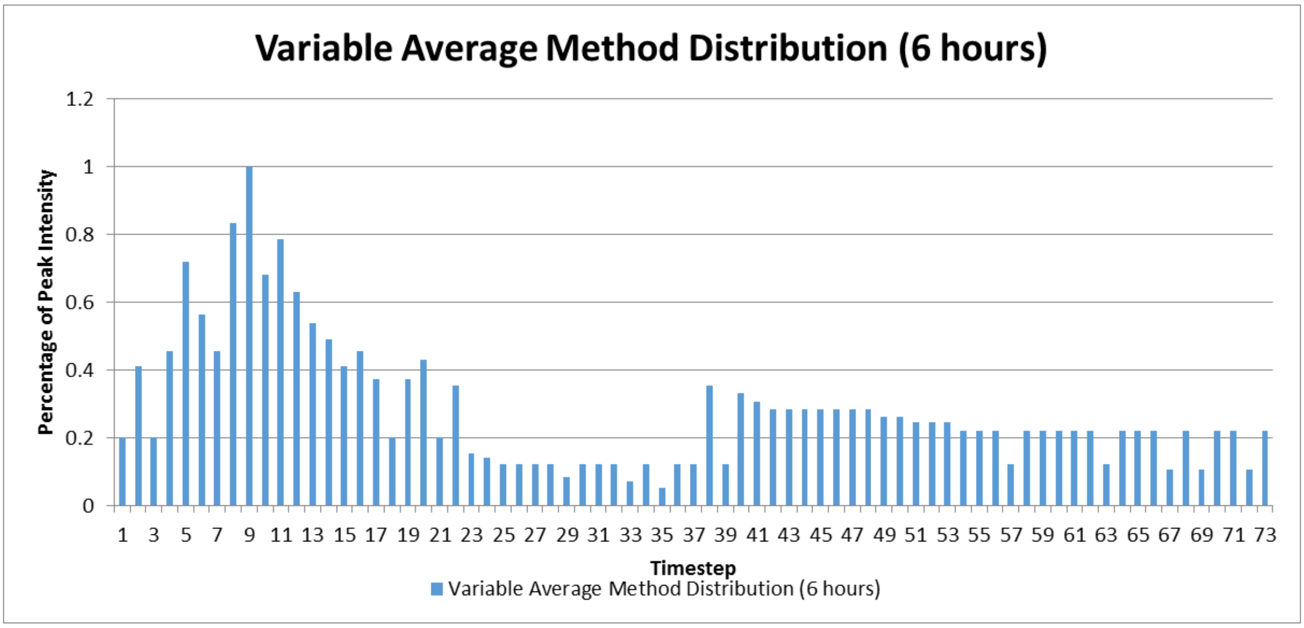


Figure 10-22: Temporal distributions for 6 hour storms using the Average Variability Method

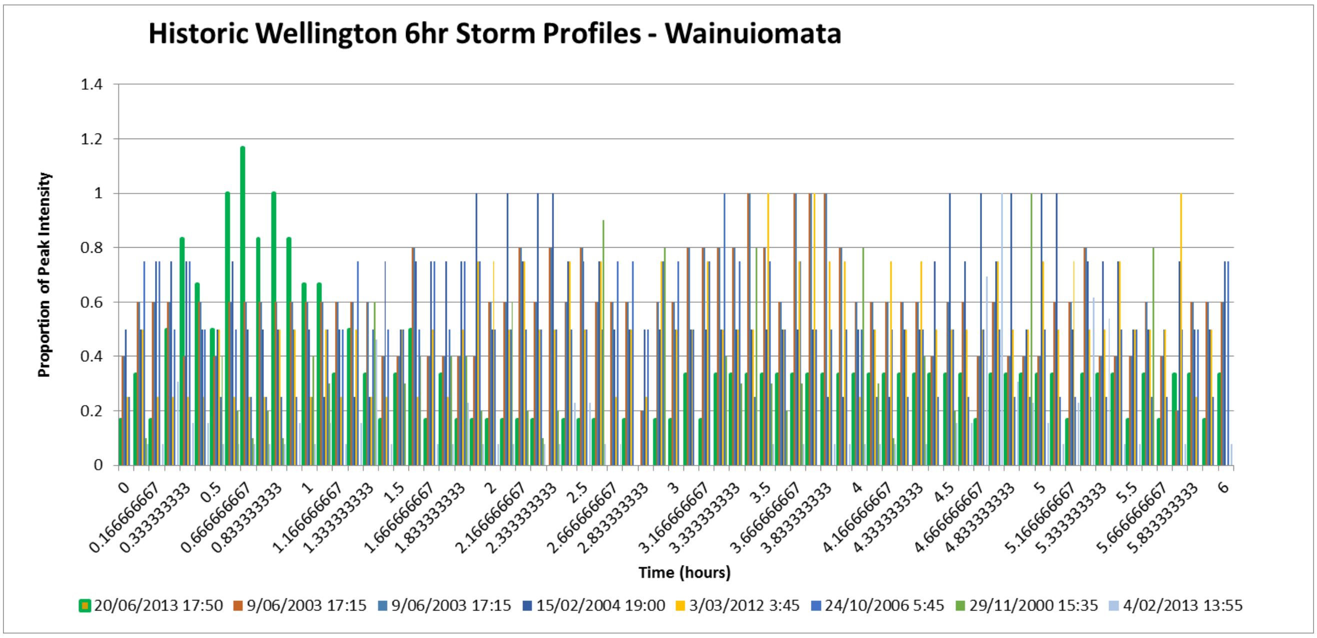


Figure 10-23: Temporal distributions for 6 hour storms at Wainuiomata Reservoir

Appendix B AECOM 2004 Hydrological Model

Previous hydrological modelling carried out by AECOM during their 2004 investigation was undertaken in MOUSE. Their model divided the Black Creek catchment into 34 subcatchments. The hydrological parameters of these catchments (catchment area, slope, length etc.) were developed using available GIS data. Ten representative properties were selected and their impervious surfaces were digitised based on aerial photography to estimate the percentage impervious for each catchment. The average impervious area for the residential urban area was assessed be 49% pervious, road reserves 30% pervious, and the forest / pasture areas 95% pervious (to account for small portions of roads, farm buildings etc.) on average.3 No information is available regarding the runoff volume generation and runoff routing approaches applied in the AECOM modelling.

The 2004 AECOM investigation developed design rainfall based on two investigations of the Wainuiomata Catchment by the Greater Wellington Regional Council in 1998. The investigation adopted rainfall intensities reported in the GRWC 1998 report and applied these to the rainfall profile recorded during the storm event on 15-16 August 2004. The event rainfall profile was normalised to be 24 hours in duration and then scaled to the required return period. It is reported in the available AECOM documents that the peak 1 and 2 hour rainfall profiles were nested within the normalised 24 hour design profile as it is expected that the critical duration for the Black Creek catchment lies within these two durations. Figure 10-24 shows the design rainfall profile applied, and Table 10-4 provides the 24 hour depths used and the resultant peak flows from the model.

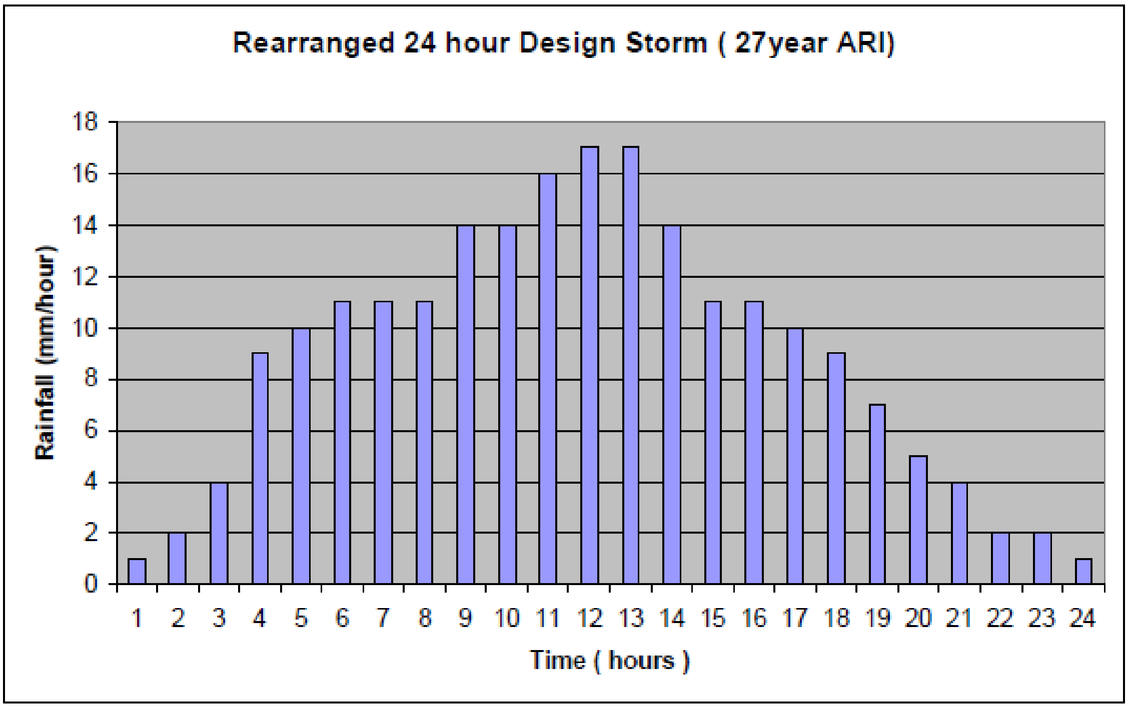


Figure 10-24: AECOM Normalised 24hr design storm (from Figure 3.9, pg 13, AECOM, 20054)

[3] Black Creek Wainuiomata Upgrading Model Development Report October 2004, Page 5

[4] AECOM: Black Creek Wainuiomata Upgrading Issues & Options Report April 2005

Table 10-3: AECOM Design Storms (from Table 3.1, pg 14, AECOM, 2005)

Storm	Total Rainfall (mm)	Peak Flow Rate (m^3/s) (Hydrological Model Output)
50 Year ARI	155	60.5
75 Year ARI	168	65.0
100 Year ARI	175	72.0
August 2004	199.5 (over 6 days)	32.0

It should be noted that there is significant variation in the design rainfall totals applied in the 2005 modelling and the current model. Table 10-4. shows the difference in the 24 hour rainfall depths for the 50yr and 100yr ARI events between the previous and current modelling.

Table 10-4: Design 24 hours Rainfall Depths

	2005 Model (mm)	Current Model (mm)	Percentage Difference (%)
50yr ARI	155	216.4	40
100yr ARI	175	249.2	42

Appendix C Reported Flooding Issues

Previous catchment reports and catchment knowledge have highlighted the following areas with historical flooding:

Table 10-5: Table 9 5 Historical Flooding

Location	Date	Source
A key factor influencing the design of the Hutt City stormwater system is the city's geography. The Hutt Valley is shouldered by hills on the eastern and western sides. Wainuiomata is a basin virtually surrounded by hills. Problems can be experienced in severe storms, with large quantities of debris from steep gullies being swept into intakes to the stormwater system, sometimes overwhelming them. Wainuiomata and the Hutt Valley floor, which receive the rapid run-off from the hills, are relatively flat with only limited natural drainage.	General	HCC Stormwater Asset Management Plan 2007
Much of Wainuiomata is surrounded by steep, undeveloped hillsides which can convey large quantities of debris into stormwater intakes during severe storms.	General	HCC Stormwater Asset Management Plan 2007
The majority of the stormwater pipe system was designed with the capacity to accommodate rainfall with a 5yr ARI and can be expected to be overloaded when more severe rainfall is experienced. Secondary stormwater flood-paths, to safely convey floodwater when the capacity of the primary system was exceeded, were generally not provided when areas were developed. New stormwater drains are designed with the capacity to accommodate rainfall with an ARI between 10 and 50 years, depending on the risk in individual situations. Secondary stormwater flood-paths are provided where practical (the provision of secondary flood-paths in established areas is often impractical) and are required in new "greenfields" developments.	General	HCC Stormwater Asset Management Plan 2007
Hair Street (flood area 66) Hill catchments either side and a lack of capacity. Various improvements have been undertaken.	2000	Historical Flooding Maps
Gibbs Crescent (flood area 68) Watercourse from Hair street run through this area and cause flooding at no 31. Improvements include bunding and secondary flow path improvements. Flooding at 39 Parenga likely to a result of flood levels in the Wainuiomata River.	2003 and 2004	Historical Flooding Maps
Grimsby Grove. Flooding influenced by Black Creek. Also in 2004 as a result of overflows from Wise Street. Bunding constructed in 2005 to improve this situation	2004	Historical Flooding Maps
Wise Street. Drainage paths were altered as part of subdivision work in 2000 to 2004. This area needs to be considered carefully in relation to future developments/building heights	2004	Historical Flooding Maps
Norfolk Street (flood area 54). Low ae on ponding map. Influenced by flood levels in Black Creek	2004	Historical Flooding Maps
Crowther Road / Brookfield Lane (flood area 86) Flooding from Wainuiomata Stream	2004	Historical Flooding Maps
Westminster Street and Wellington Road (flood area 56) Both 75 and 52 are adjacent to street sumps and local flooding has occurred due to flood levels in Black Creek. Some road flooding reported in Wellington Road	2004	Historical Flooding Maps

Location	Date	Source
Ashford Street (flood area 57). Flood levels along Ashford street are likely to be affected by levels in Black Creek. Road flooding in Wellington road near the intersection with Devon Street. Unit 46/5 Parkway was flooded – could be influenced by levels in Black Creek.	2004	Historical Flooding Maps
Wainuiomata Road near Rata Street (flood area 58). Hill catchments overflow into low area of road	2004	Historical Flooding Maps
Konini Street near Rata Street (flood area 59) Flooding also reported in Fitzherbert Road and the Strand	2004	Historical Flooding Maps
Hine Road (flood area 65) Runoff from hillside discharging through properties onto road	2004	Historical Flooding Maps
Donelly Drive (flood area 55). During the February 2004 event a lot of stormwater built up in this area as a result of overflows from other systems and also an influence from Black Creek. Wise Street, Russell to Norfolk. Secondary overflows have historically occurred at the culvert between 92 and 94 Wise Street. Improvements have been undertaken to intake and secondary paths	2004 (Feb)	Historical Flooding Maps
Coast Road (flood area 87) Flooding from Wainuiomata River	2004 Feb	Historical Flooding Maps
Moohan Street (flood area 67) Flooding associated with high water levels in Black Creek.	2004 Feb	Historical Flooding Maps
Since 2004 there has been no major flooding reported in the Black Creek Catchment and limited cleaning of inlets and watercourses. The reduction in flooding incidents could in part be due to the reduction in frequency of severe storms of-late.	2005	John Keeler Capacity Stormwater Operator (CONFIRM database checked for trigger locations)
Based on model results the most extensive flooding would occur along the Parkway and Konini Drains and at the confluence with Black Creek. These floods typically are located around the area where the Parkway drain enters Black Creek as well as by Nelson Crescent Bridge, Best St. Bridge and Konini Street Bridge.	2005	Model Report

Appendix D Model Scenarios and Simulations

Table 10-6: Model Simulation Matrix for System Performance Assessment

Simulation	Landuse	Design Storm Event	Rainfall	Downstream Boundary
1	Existing Development	5yr ARI	Current 5yr ARI	Normal condition
2	Existing Development	5yr ARI	Future 5yr ARI	Normal condition
3	Existing Development	10yr ARI	Current 10yr ARI	Normal condition
4	Existing Development	10yr ARI	Future 10yr ARI	Normal condition
5	Existing Development	20yr ARI	Current 20yr ARI	Normal condition
6	Existing Development	20yr ARI	Future 20yr ARI	Normal condition
7	Existing Development	50yr ARI	Current 50yr ARI	Normal condition
8	Existing Development	50yr ARI	Future 50yr ARI	Normal condition
9	Existing Development	100yr ARI	Current 100yr ARI	Normal condition
10	Existing Development	100yr ARI	Future 100yr ARI	Normal condition
11	Proposed Development	5yr ARI	Current 5yr ARI	Normal condition
12	Proposed Development	5yr ARI	Future 5yr ARI	Normal condition
13	Proposed Development	10yr ARI	Current 10yr ARI	Normal condition
14	Proposed Development	10yr ARI	Future 10yr ARI	Normal condition
15	Proposed Development	20yr ARI	Current 20yr ARI	Normal condition
16	Proposed Development	20yr ARI	Future 20yr ARI	Normal condition
17	Proposed Development	50yr ARI	Current 50yr ARI	Normal condition
18	Proposed Development	50yr ARI	Future 50yr ARI	Normal condition
19	Proposed Development	100yr ARI	Current 100yr ARI	Normal condition
20	Proposed Development	100yr ARI	Future 100yr ARI	Normal condition

Appendix E Hydraulic Model Result Maps

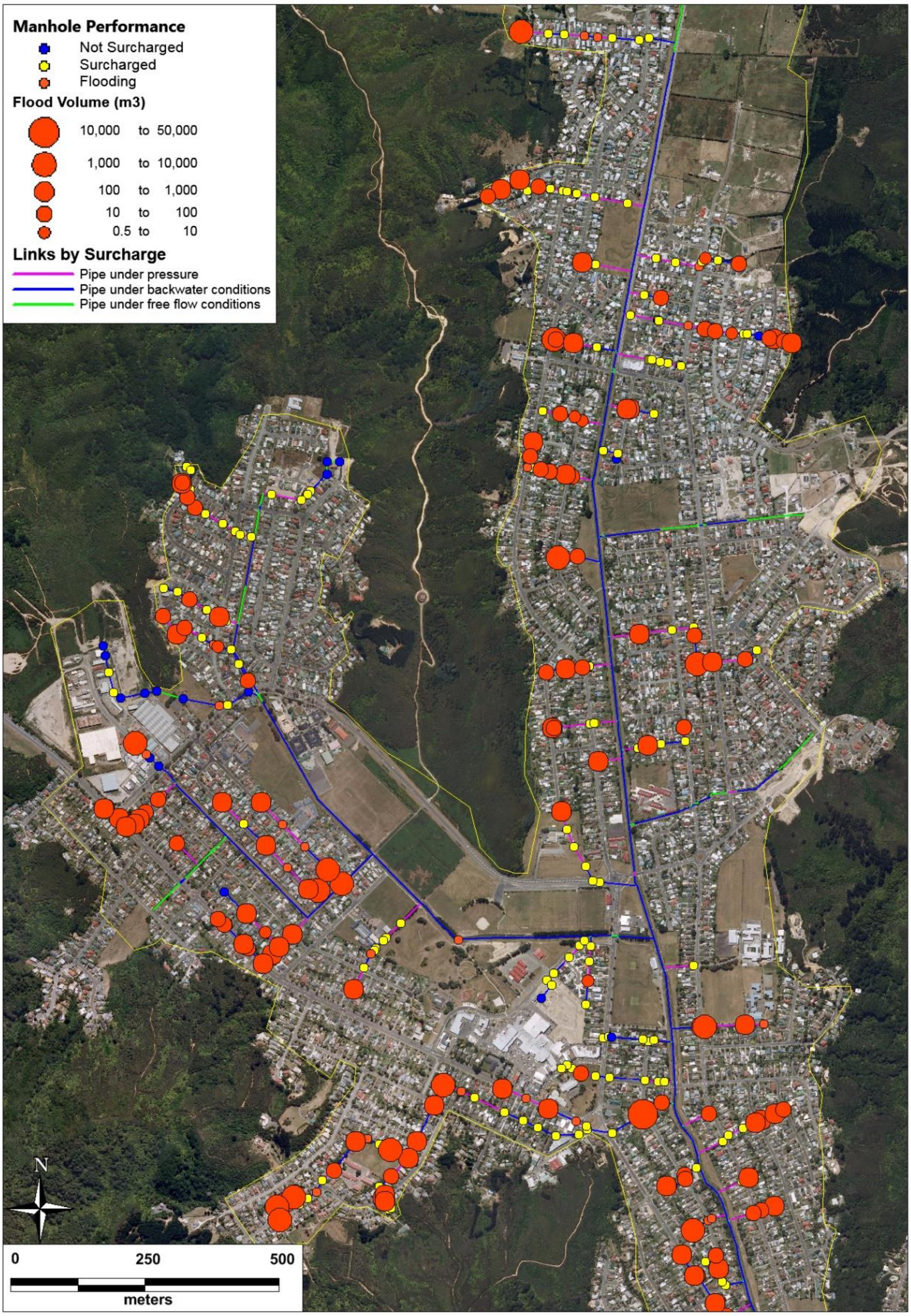


Figure 10-25: Wainuiomata SW network performance for the 10yr ARI event (current climate) – pre development

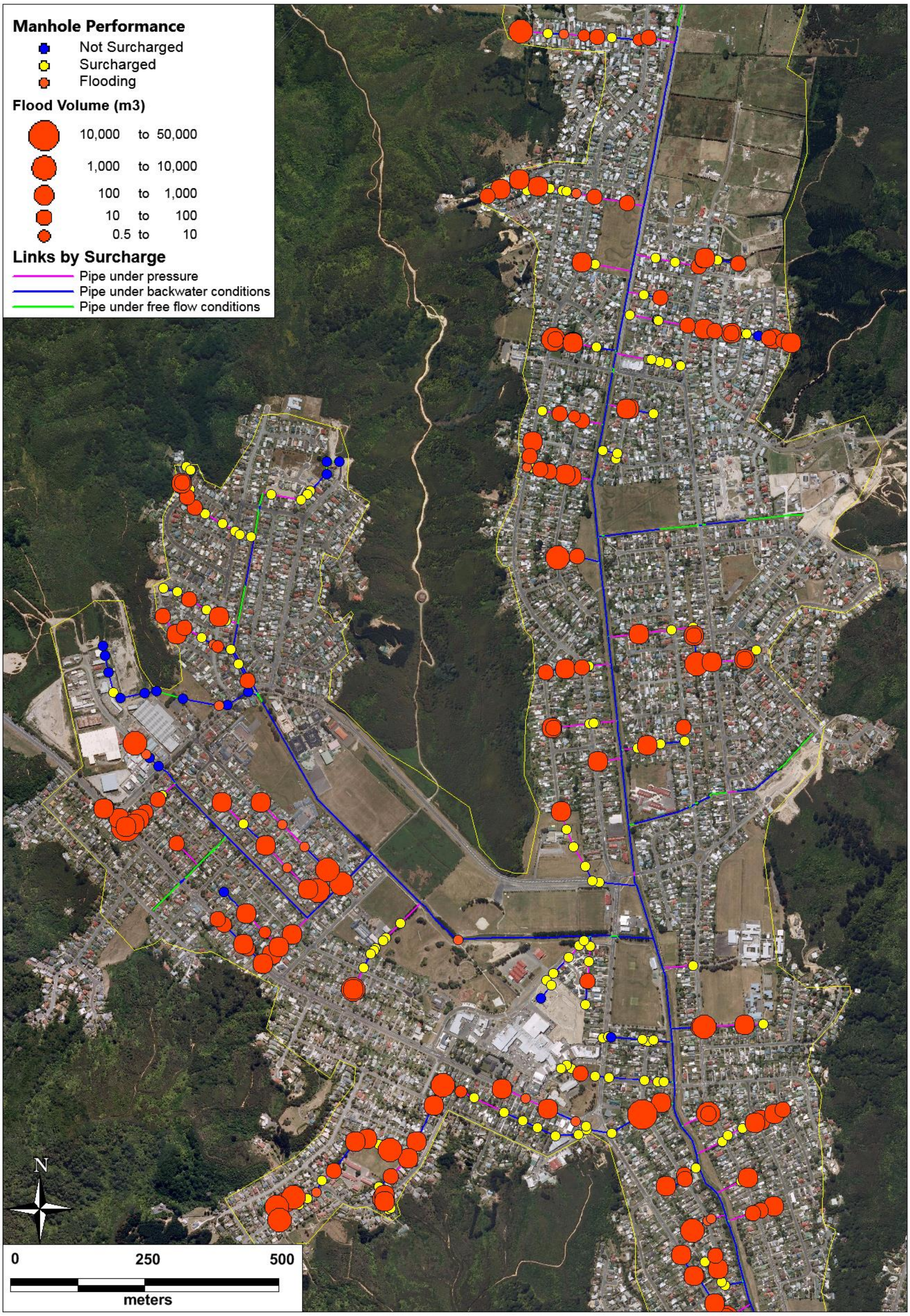


Figure 10-26: Wainuiomata SW network performance for the 10yr ARI event (current climate) – post development

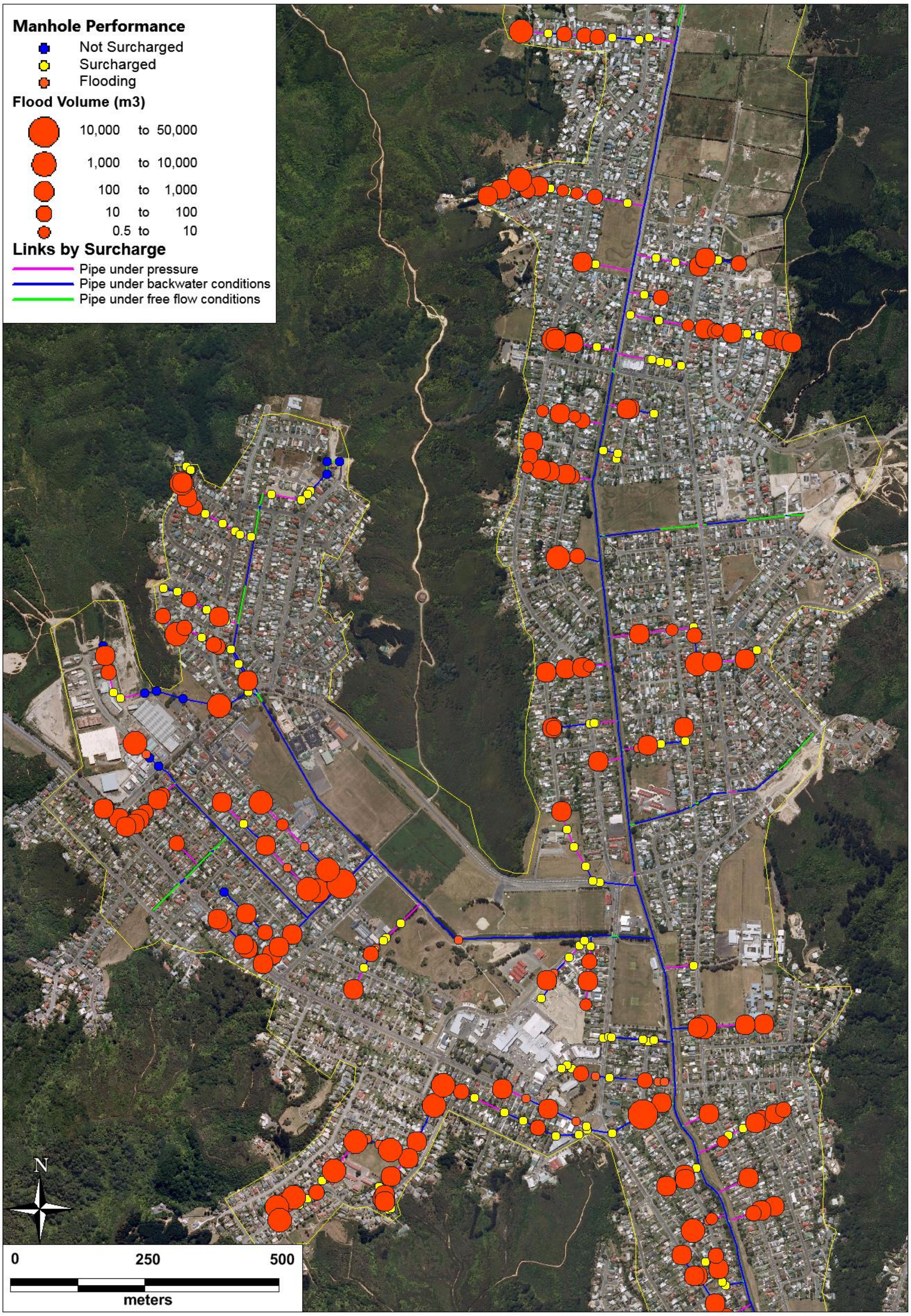


Figure 10-27: Wainuiomata SW network performance for the 20yr ARI event (current climate) – pre development

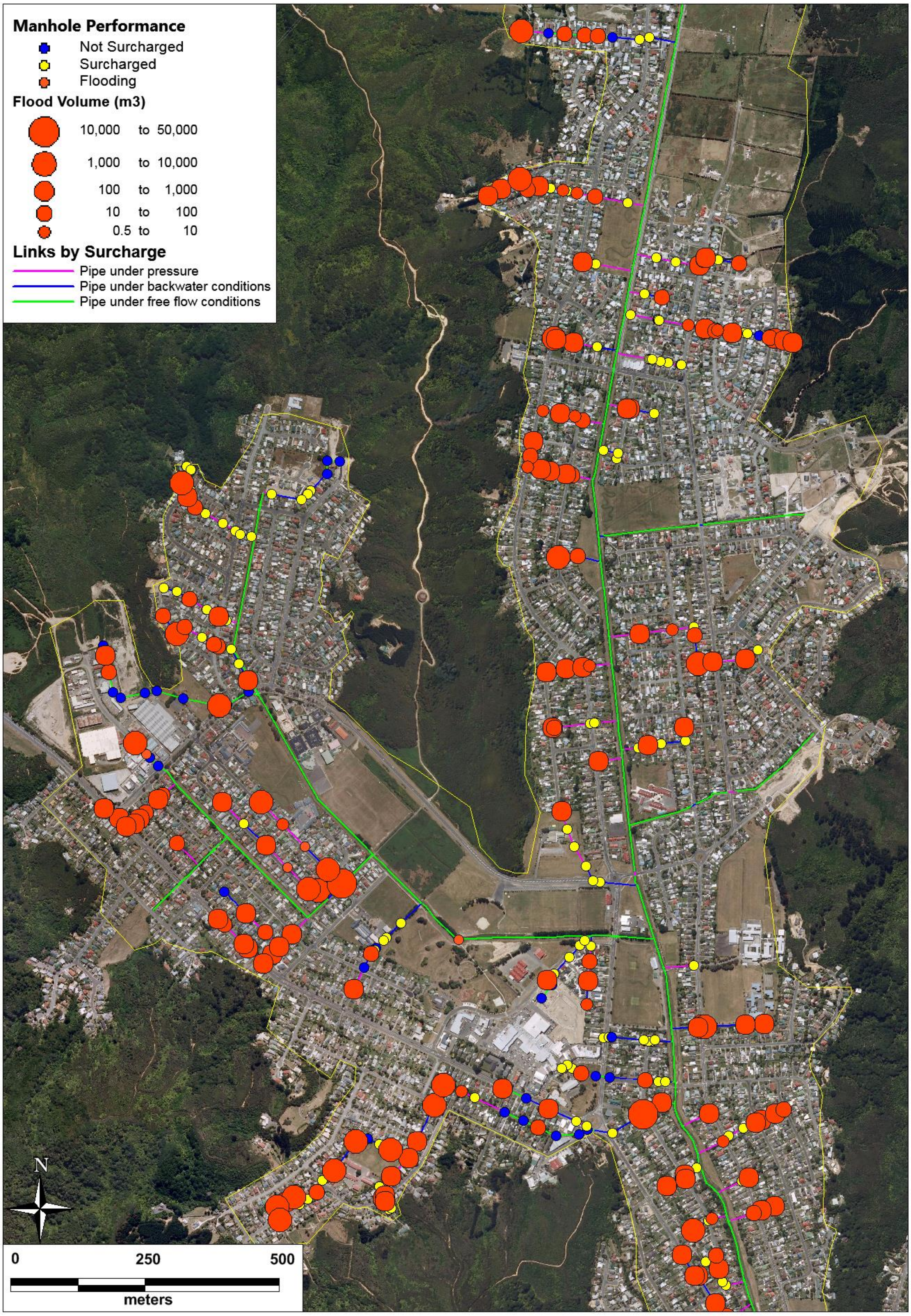


Figure 10-28: Wainuiomata SW network performance for the 20yr ARI event (current climate) – post development

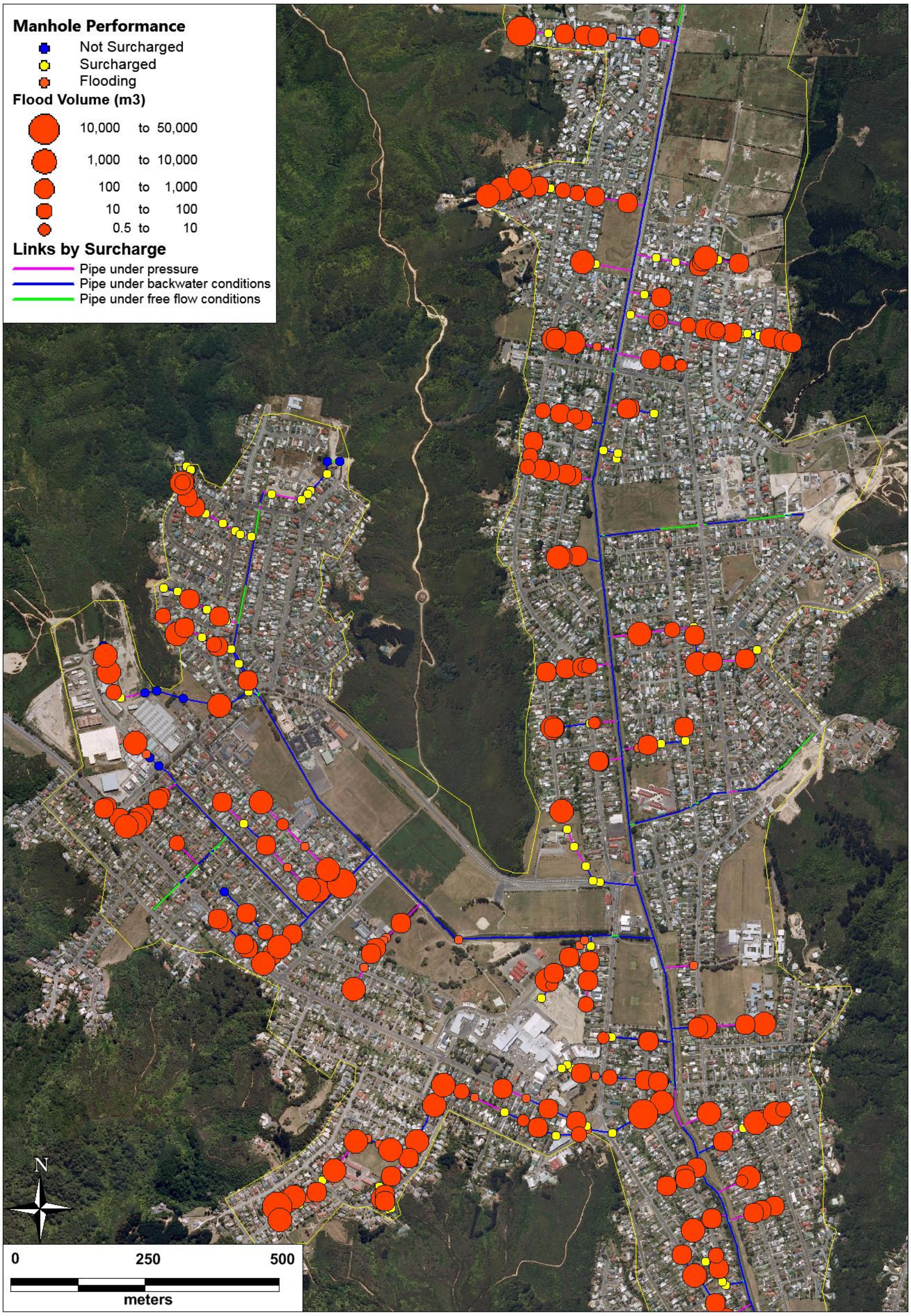


Figure 10-29: Wainuiomata SW network performance for the 50yr ARI event (current climate) – pre development

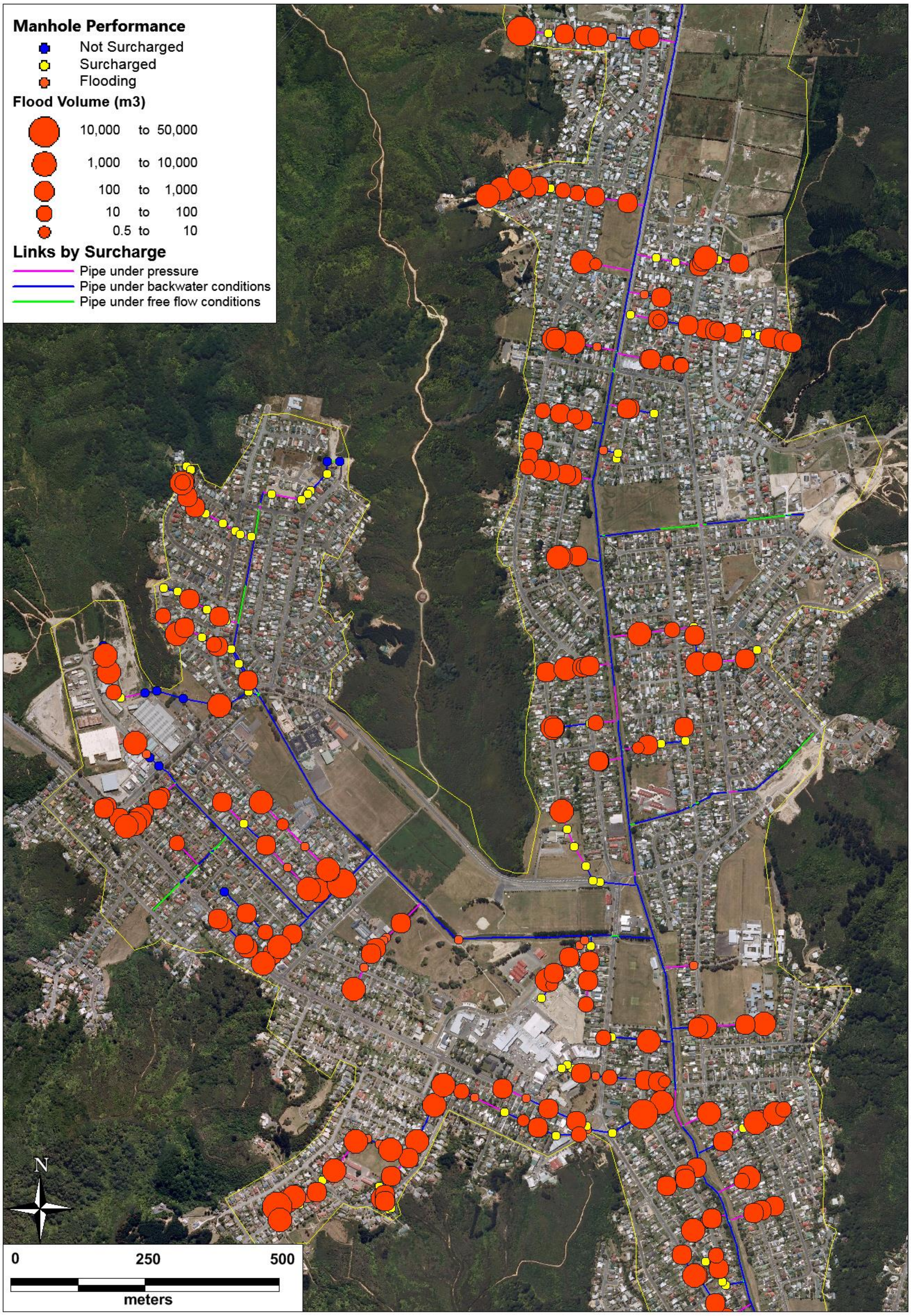


Figure 10-30: Wainuiomata SW network performance for the 50yr ARI event (current climate) – post development

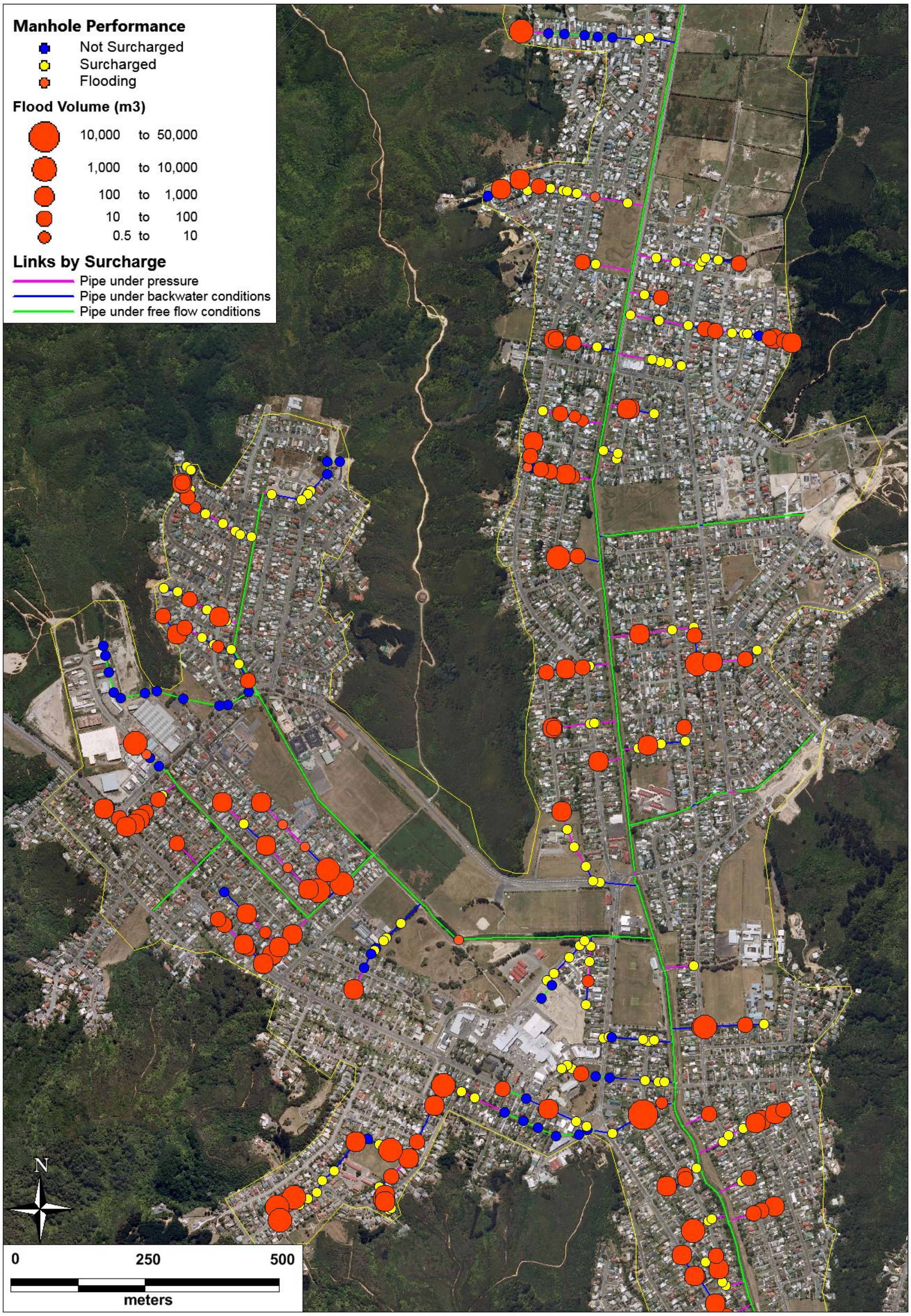


Figure 10-31: Wainuiomata SW network performance for the 5yr ARI event (future climate) – pre development

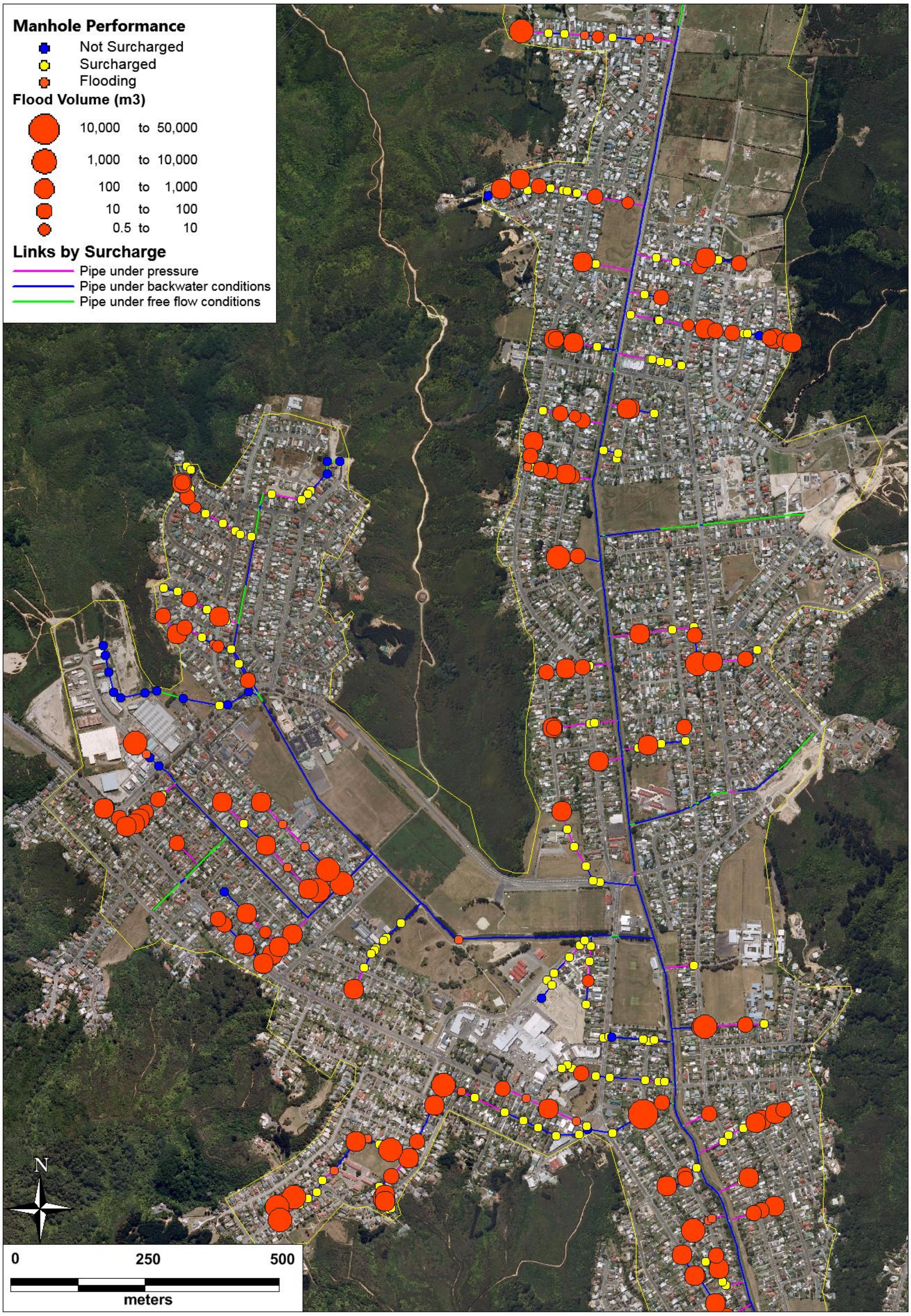


Figure 10-32: Wainuiomata SW network performance for the 5yr ARI event (future climate) – post development

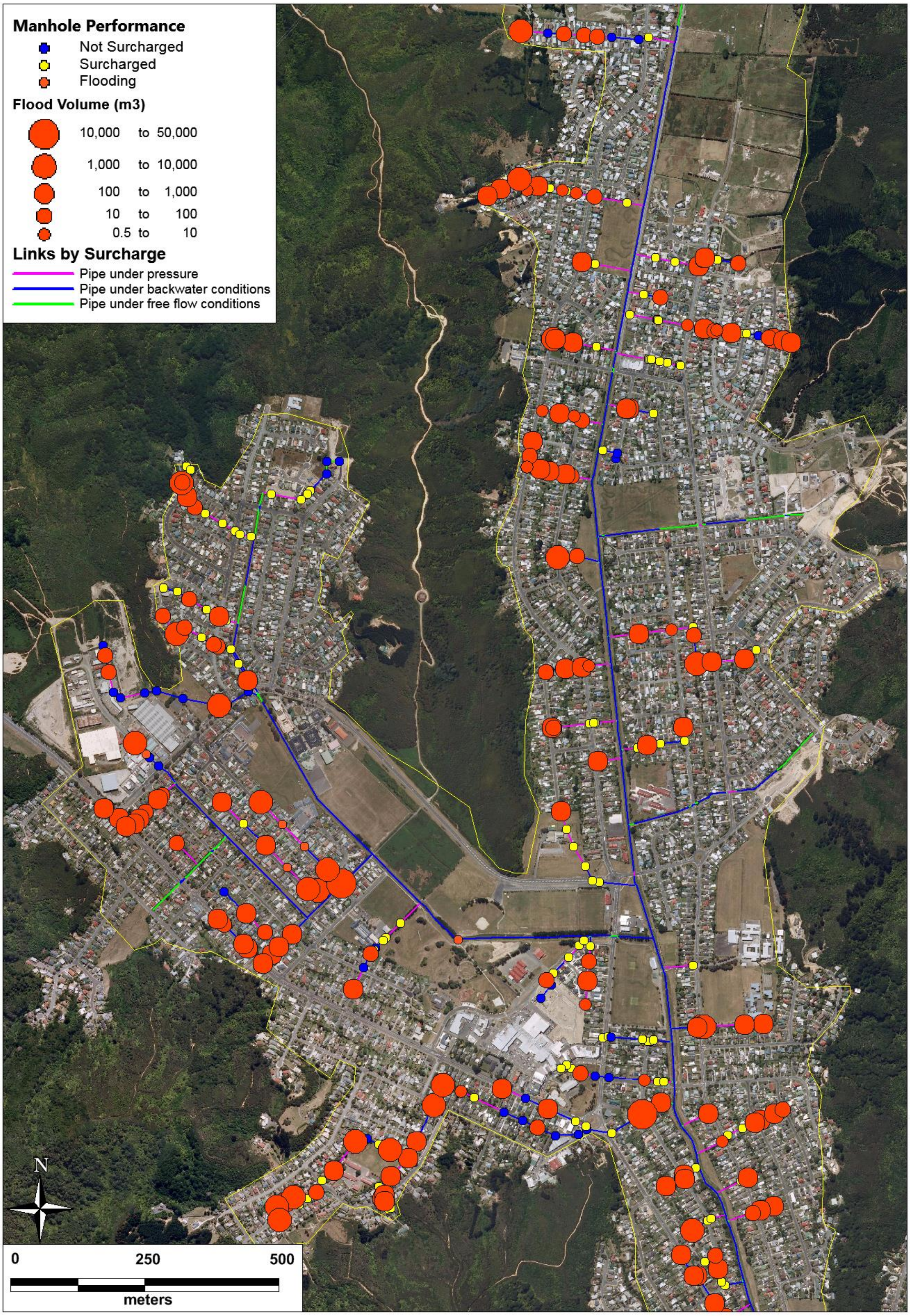


Figure 10-33: Wainuiomata SW network performance for the 10yr ARI event (future climate) – pre development

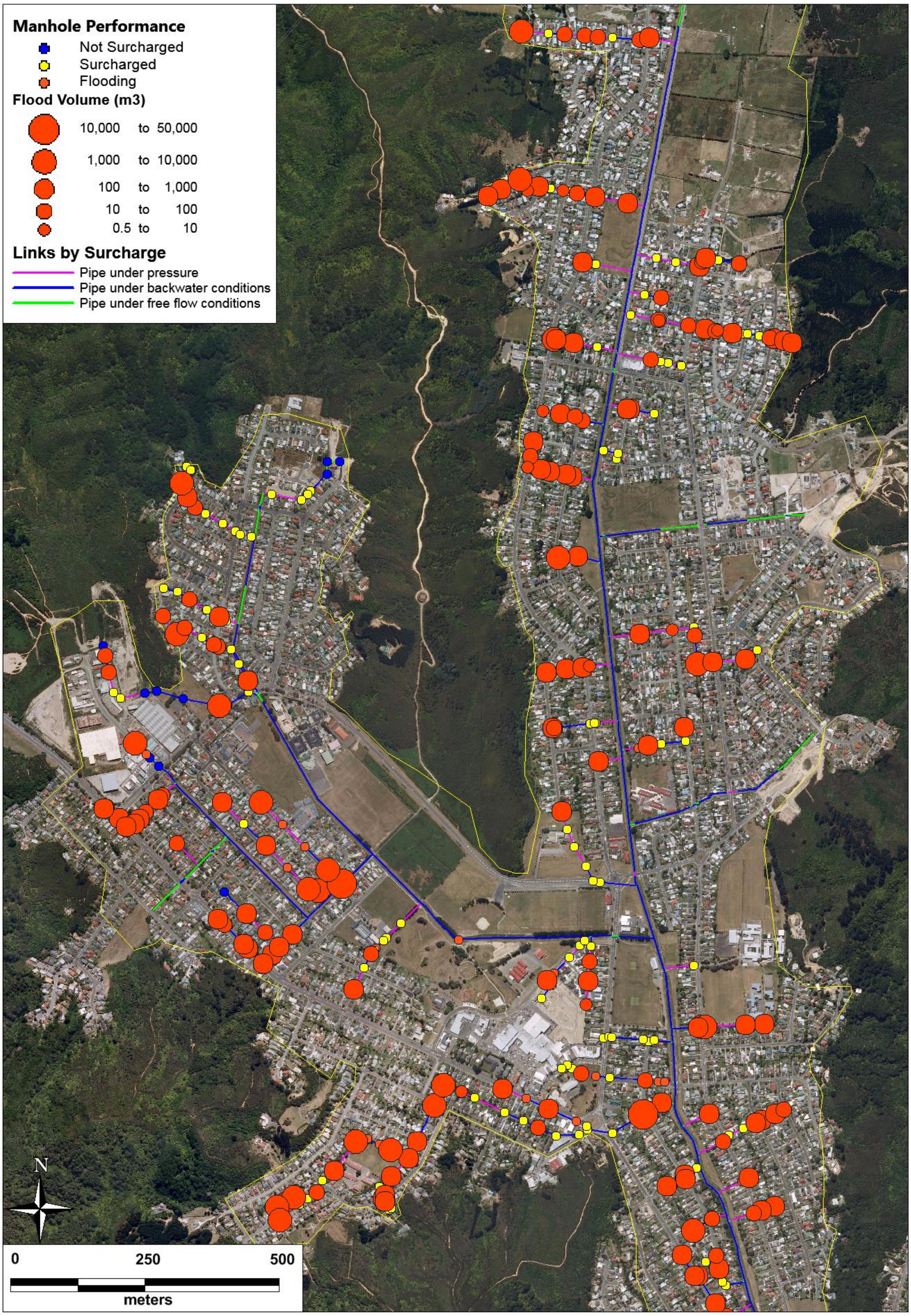


Figure 10-34: Wainuiomata SW network performance for the 10yr ARI event (future climate) – post development

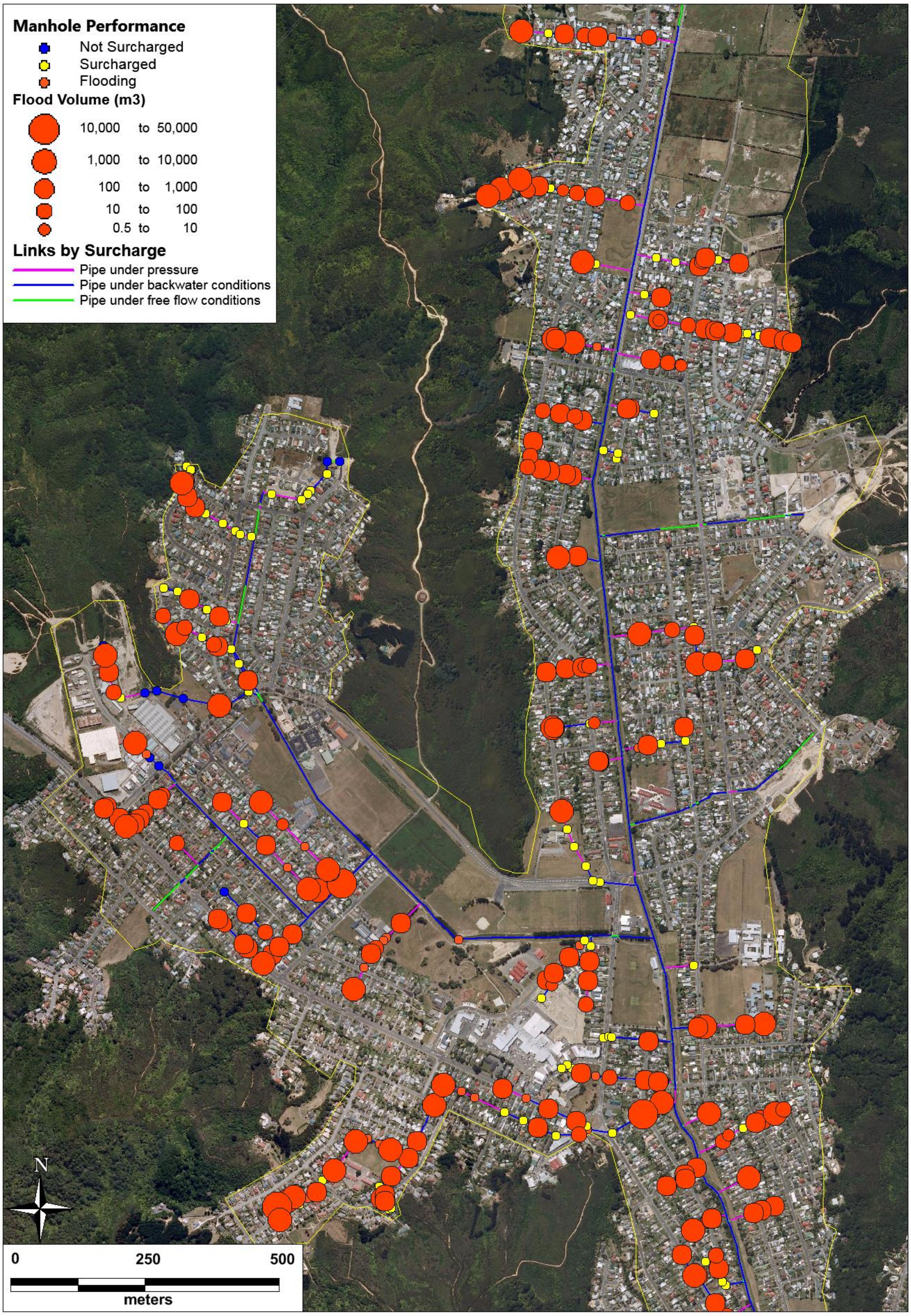


Figure 10-35: Wainuiomata SW network performance for the 20yr ARI event (future climate) – pre development

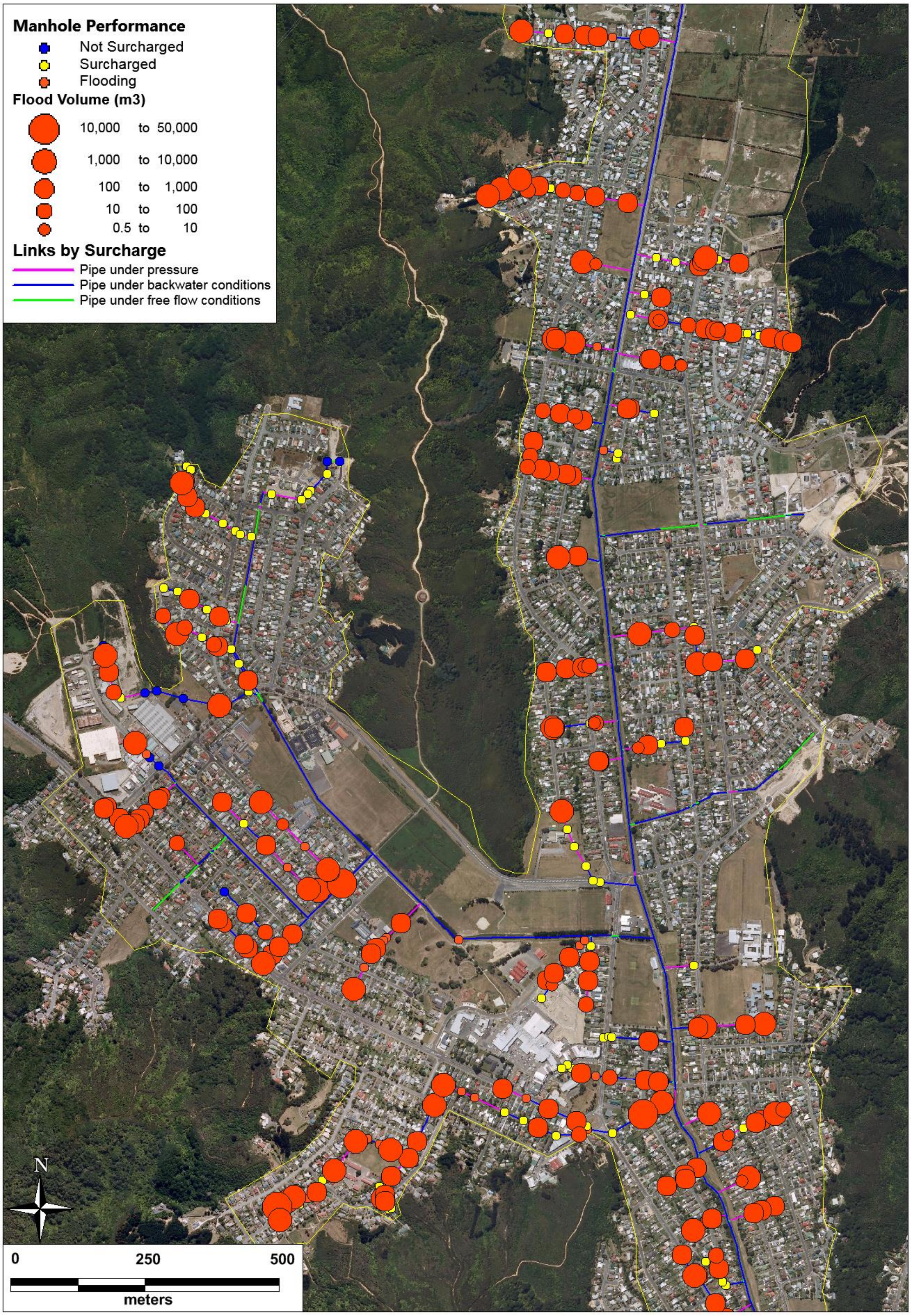


Figure 10-36: Wainuiomata SW network performance for the 20yr ARI event (future climate) – post development

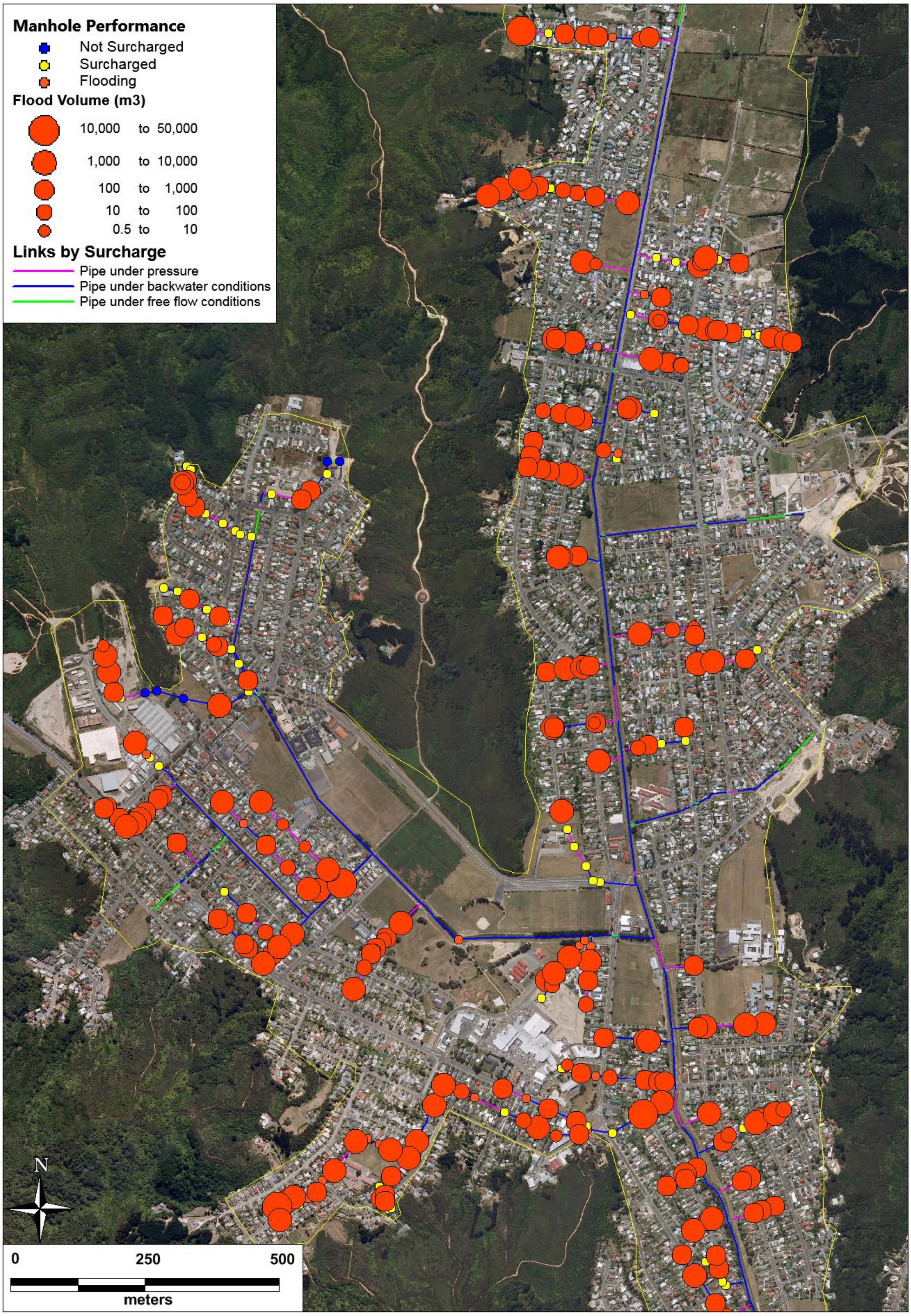


Figure 10-37: Wainuiomata SW network performance for the 50yr ARI event (future climate) – pre development

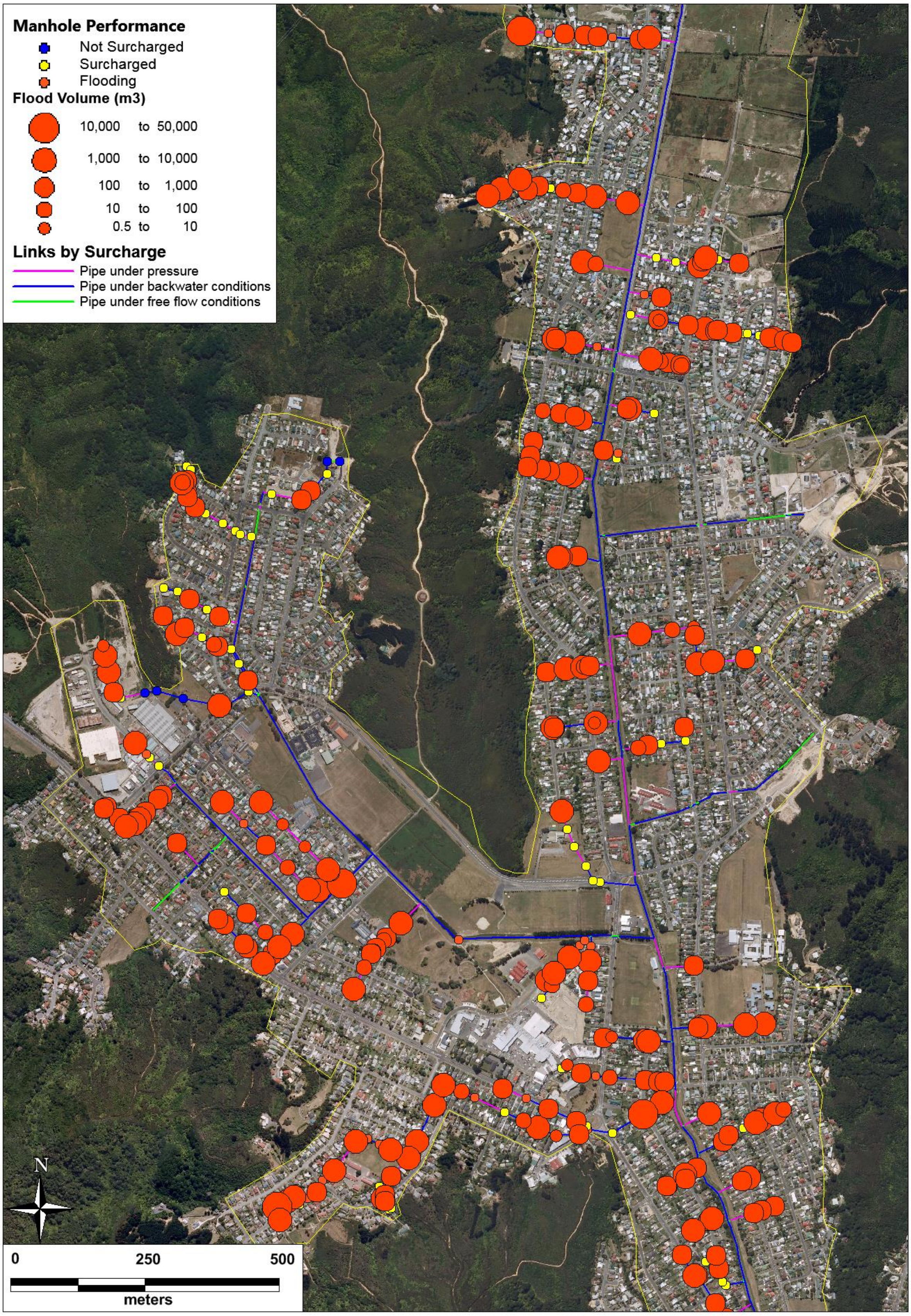


Figure 10-38: Wainuiomata SW network performance for the 50yr ARI event (future climate) – post development

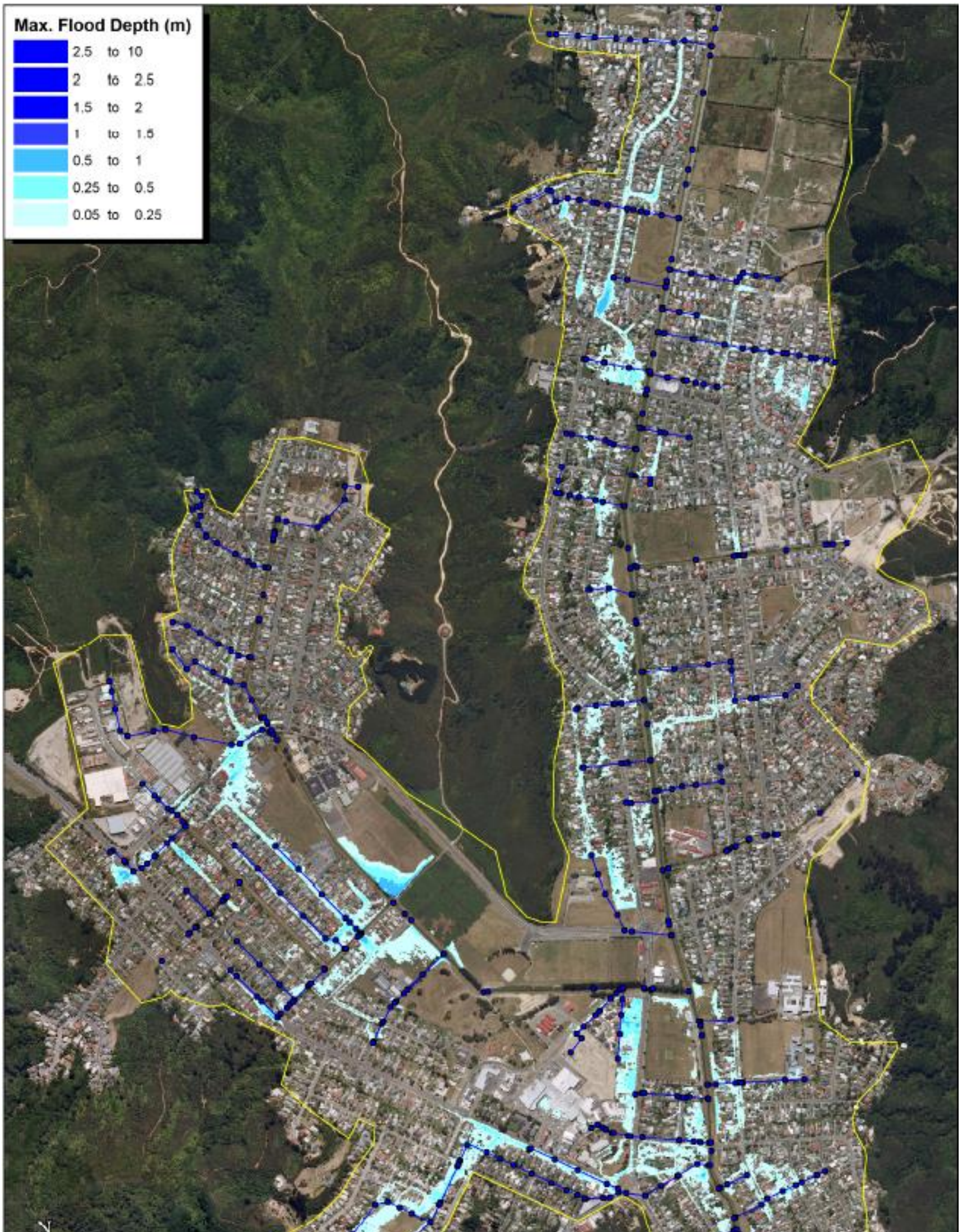


Figure 10-39: Maximum flood depths during the 10yr ARI event under future climate conditions – pre development

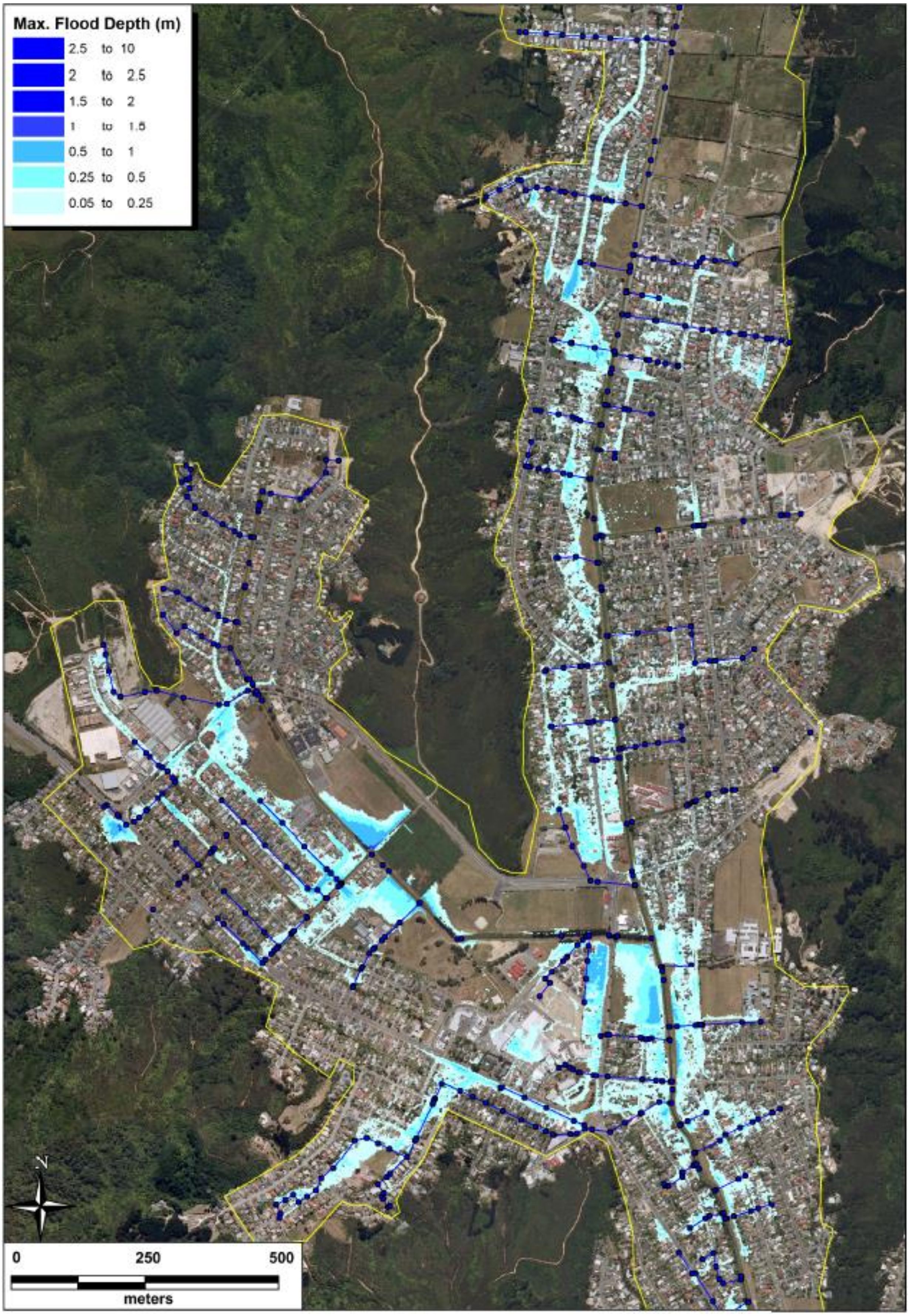


Figure 10-40: Maximum flood depths during the 50yr ARI event under current climate conditions – pre development

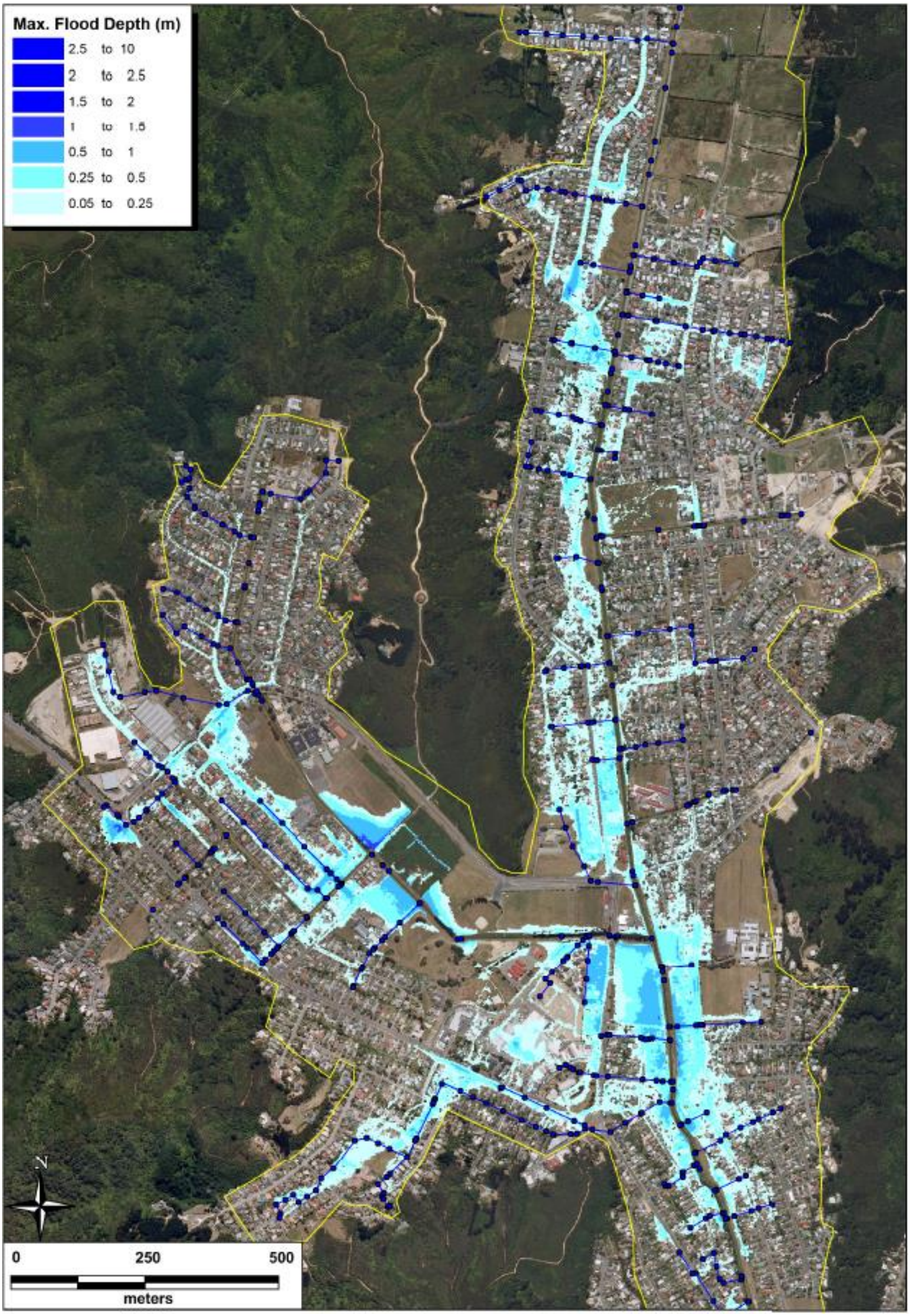


Figure 10-41: Maximum flood depths during the 50yr ARI event under future climate conditions – pre development

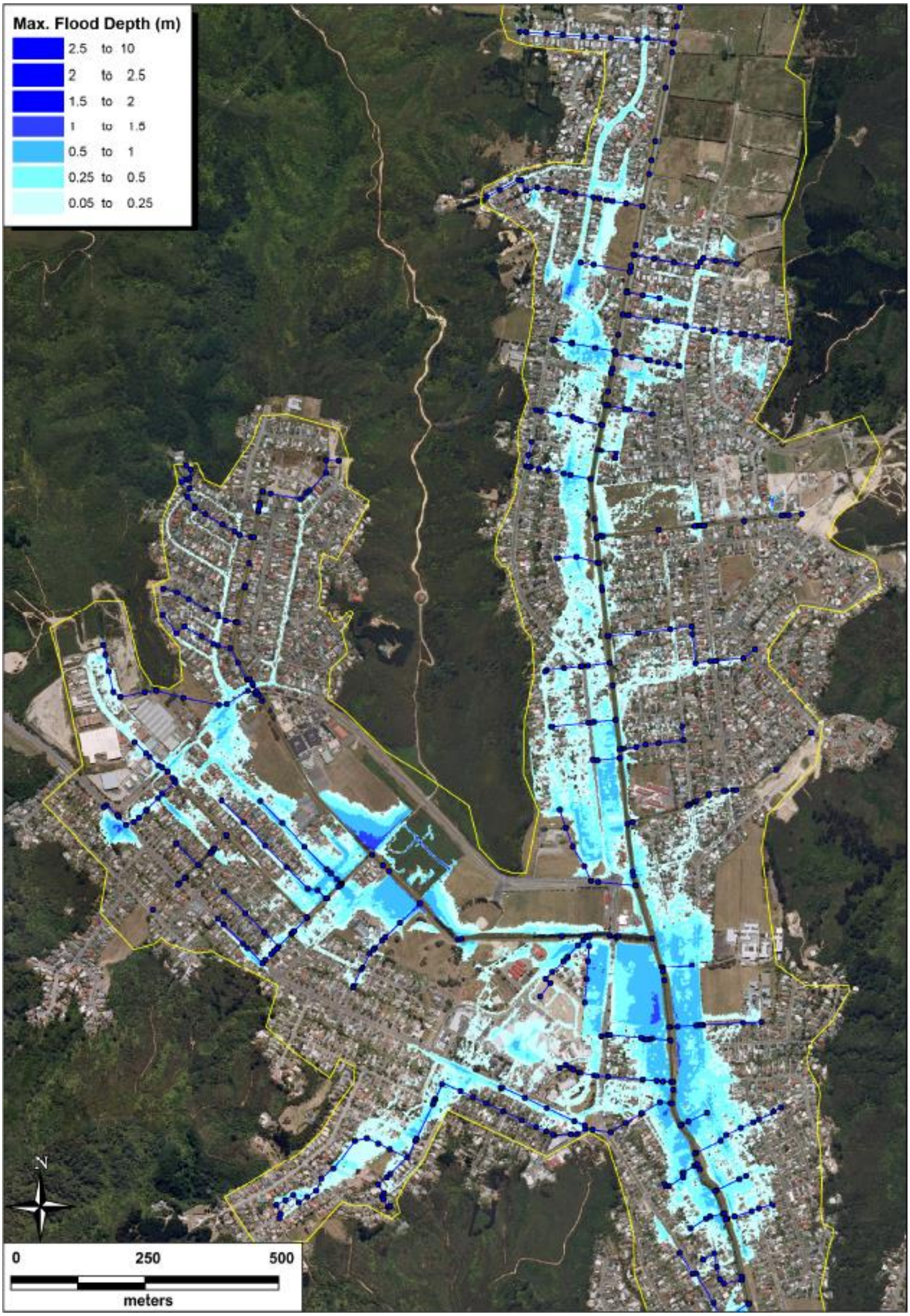


Figure 10-42: Maximum flood depths during the 100yr ARI event under future climate conditions – pre development

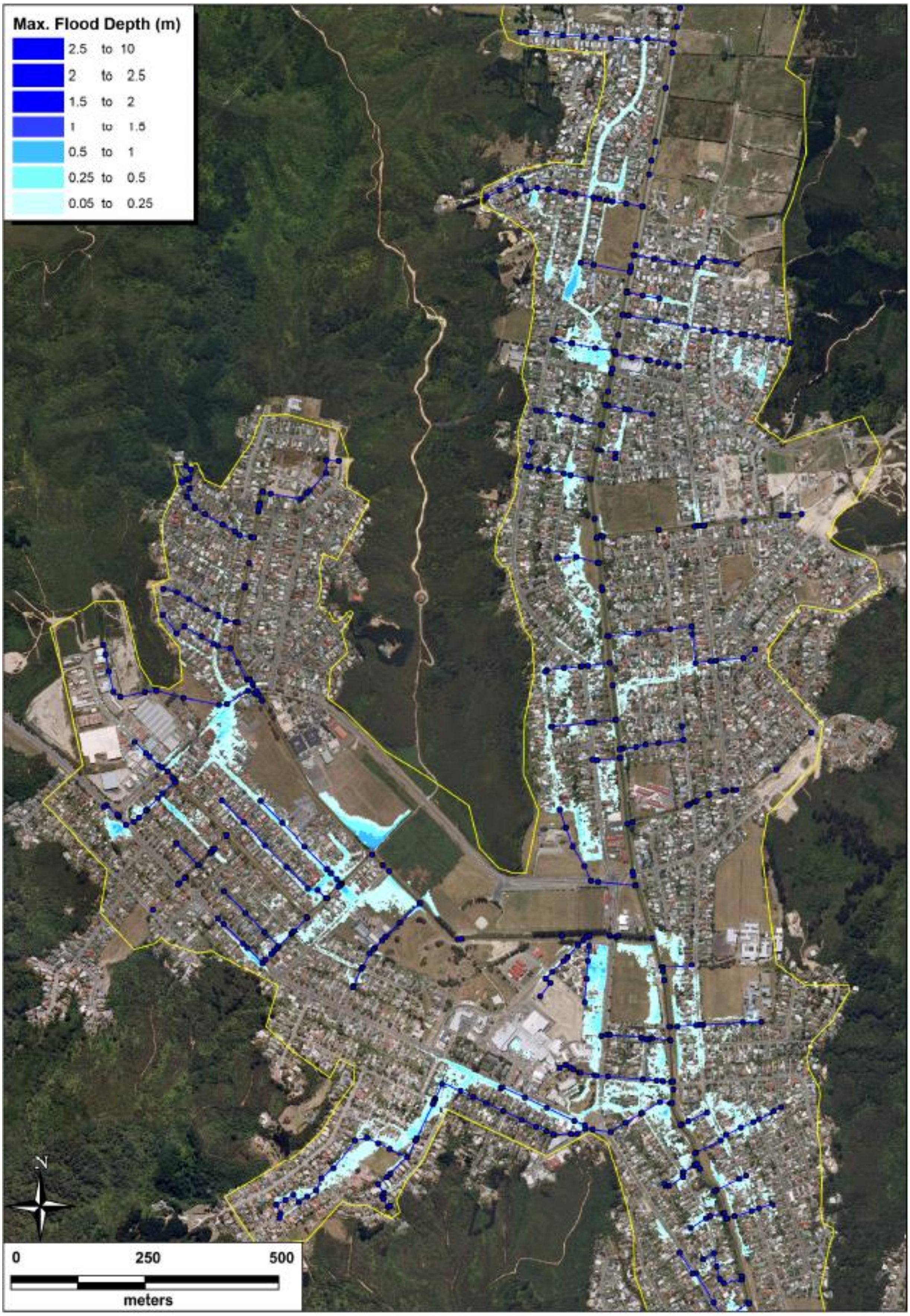


Figure 10-43: Maximum flood depths during the 10yr ARI event under future climate conditions – post development

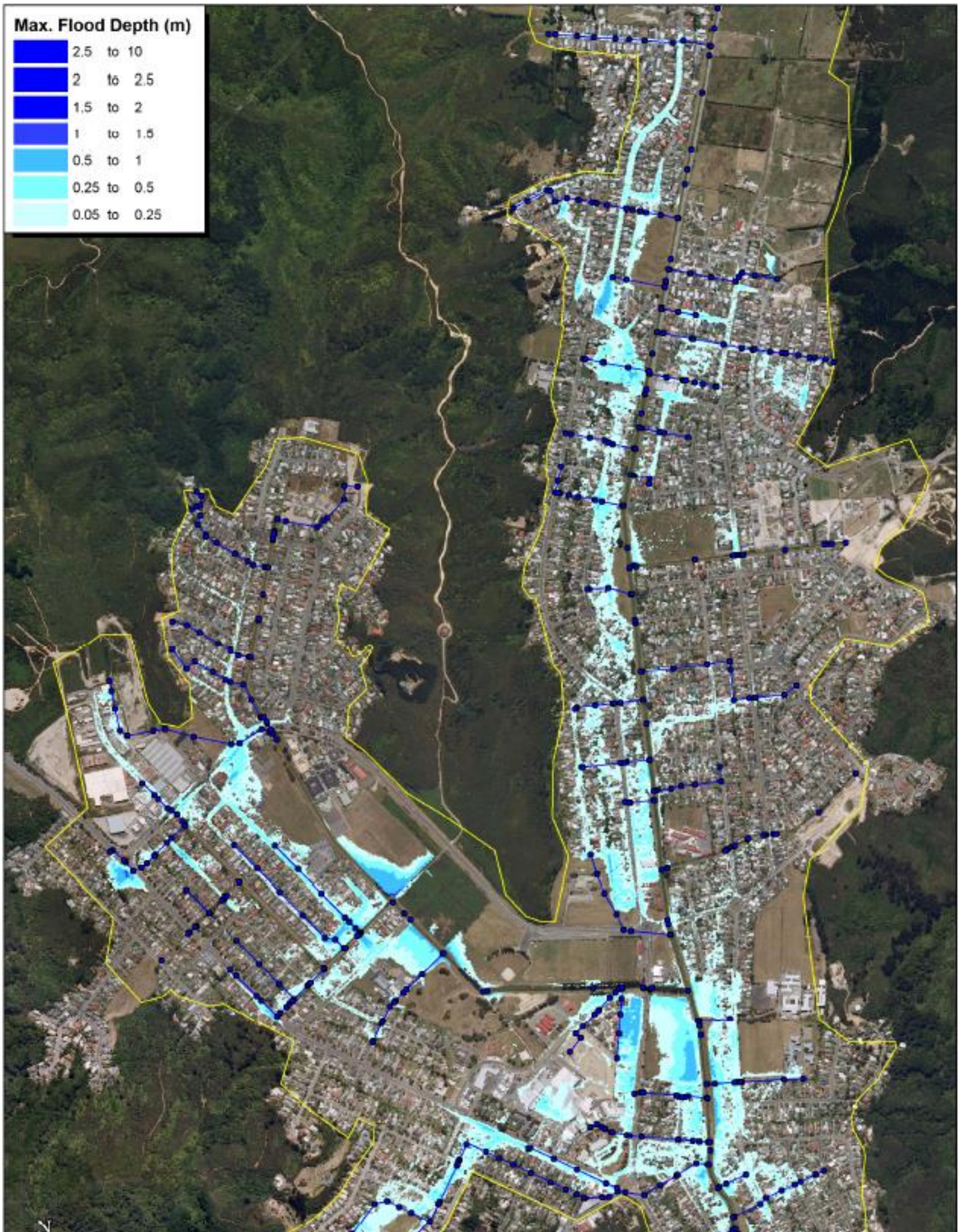


Figure 10-44: Maximum flood depths during the 50yr ARI event under current climate conditions – post development

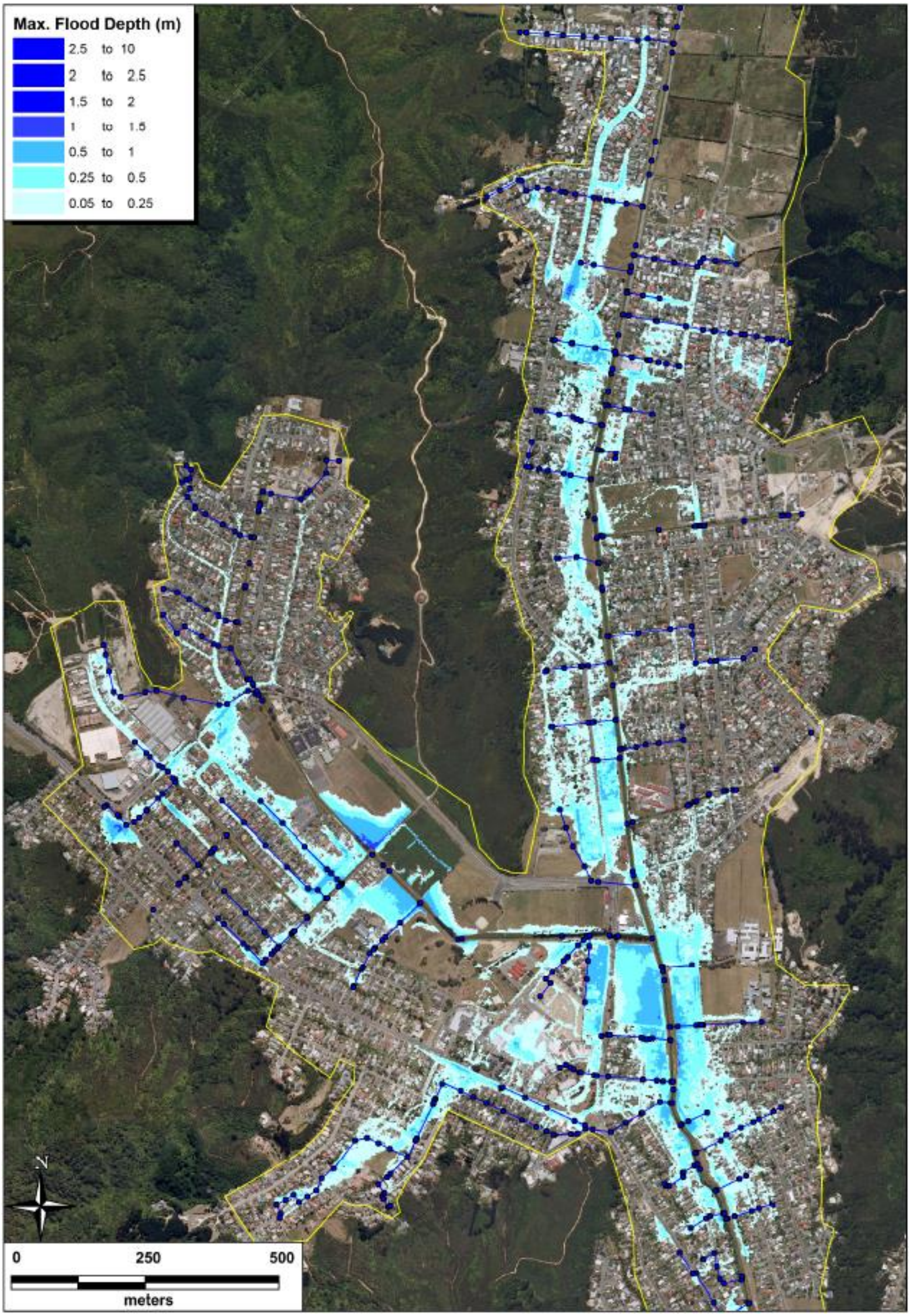


Figure 10-45: Maximum flood depths during the 50yr ARI event under future climate conditions – post development

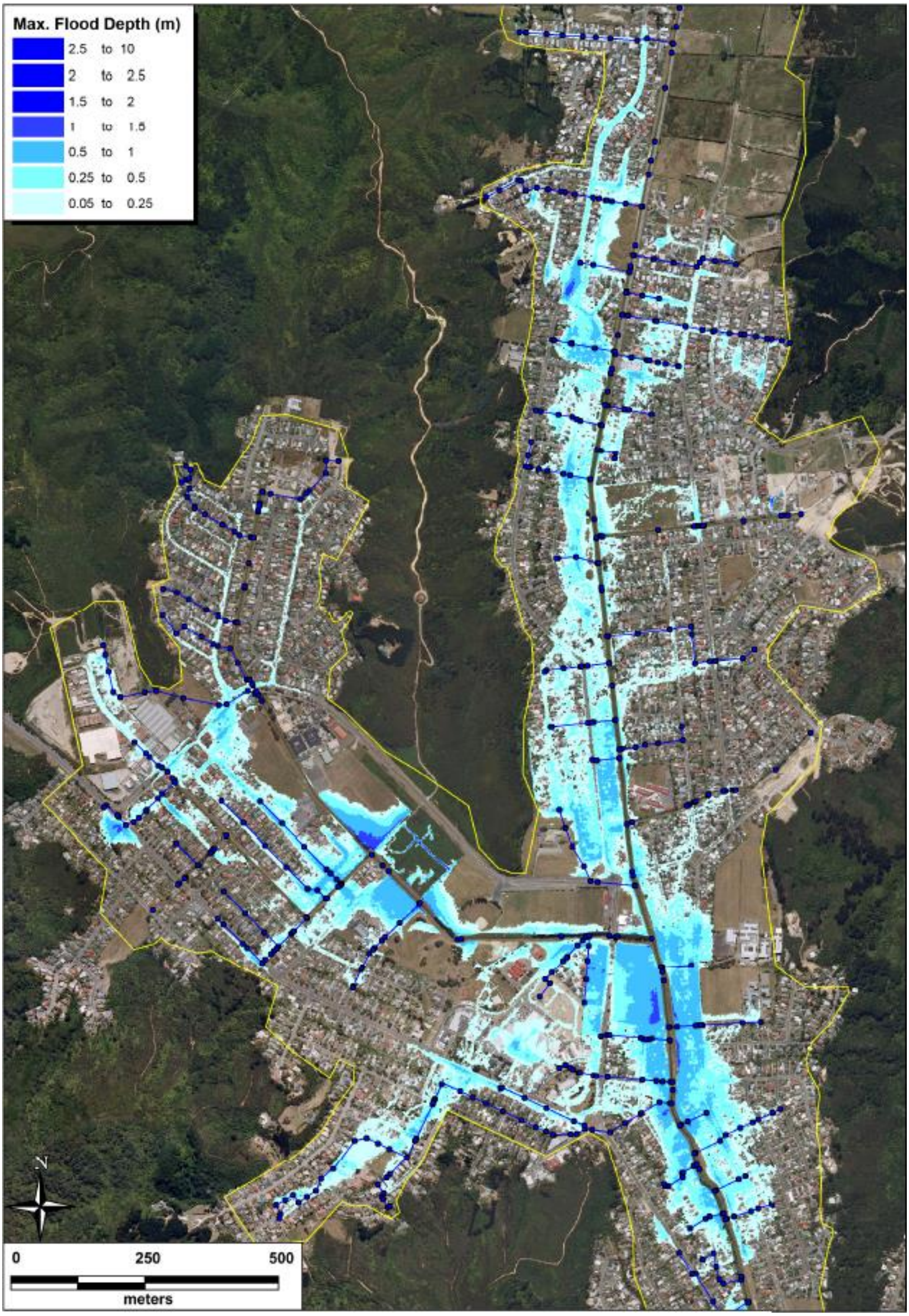


Figure 10-46: Maximum flood depths during the 100yr ARI event under future climate conditions – post development

Appendix F Flood Hazard Maps

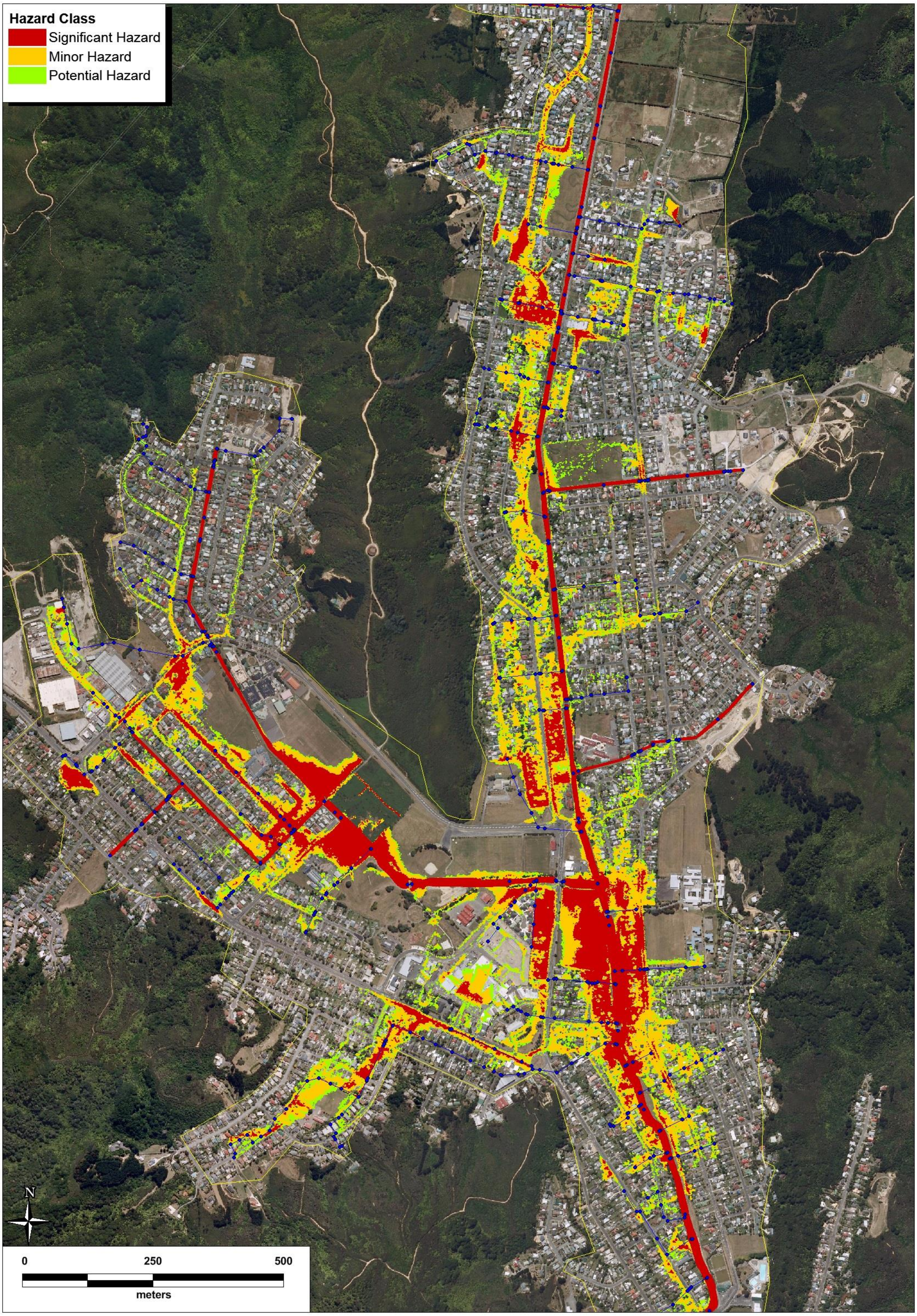


Figure 10-47: Hazard map of existing development for current climate

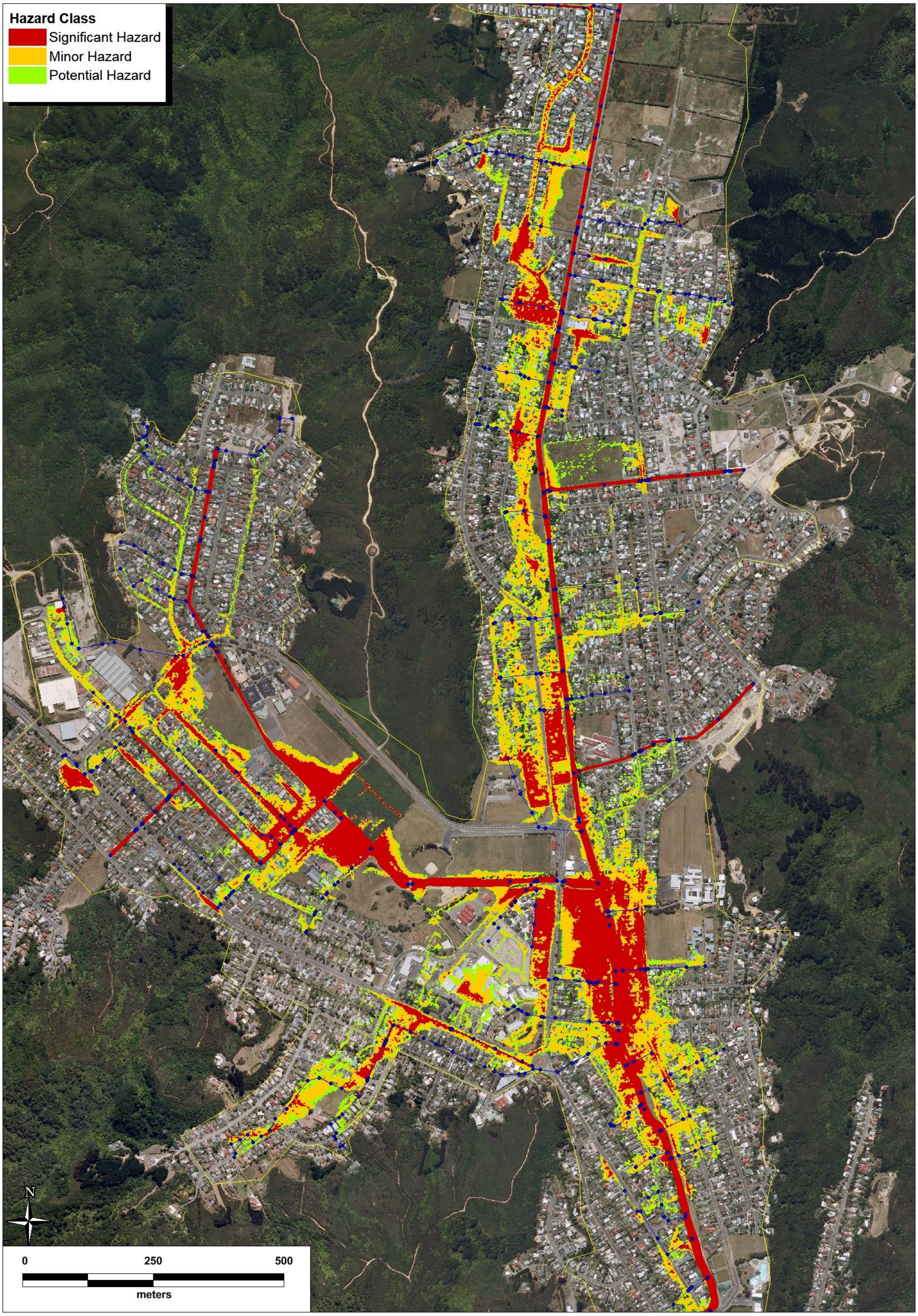


Figure 10-48: Hazard map of future development for current climate

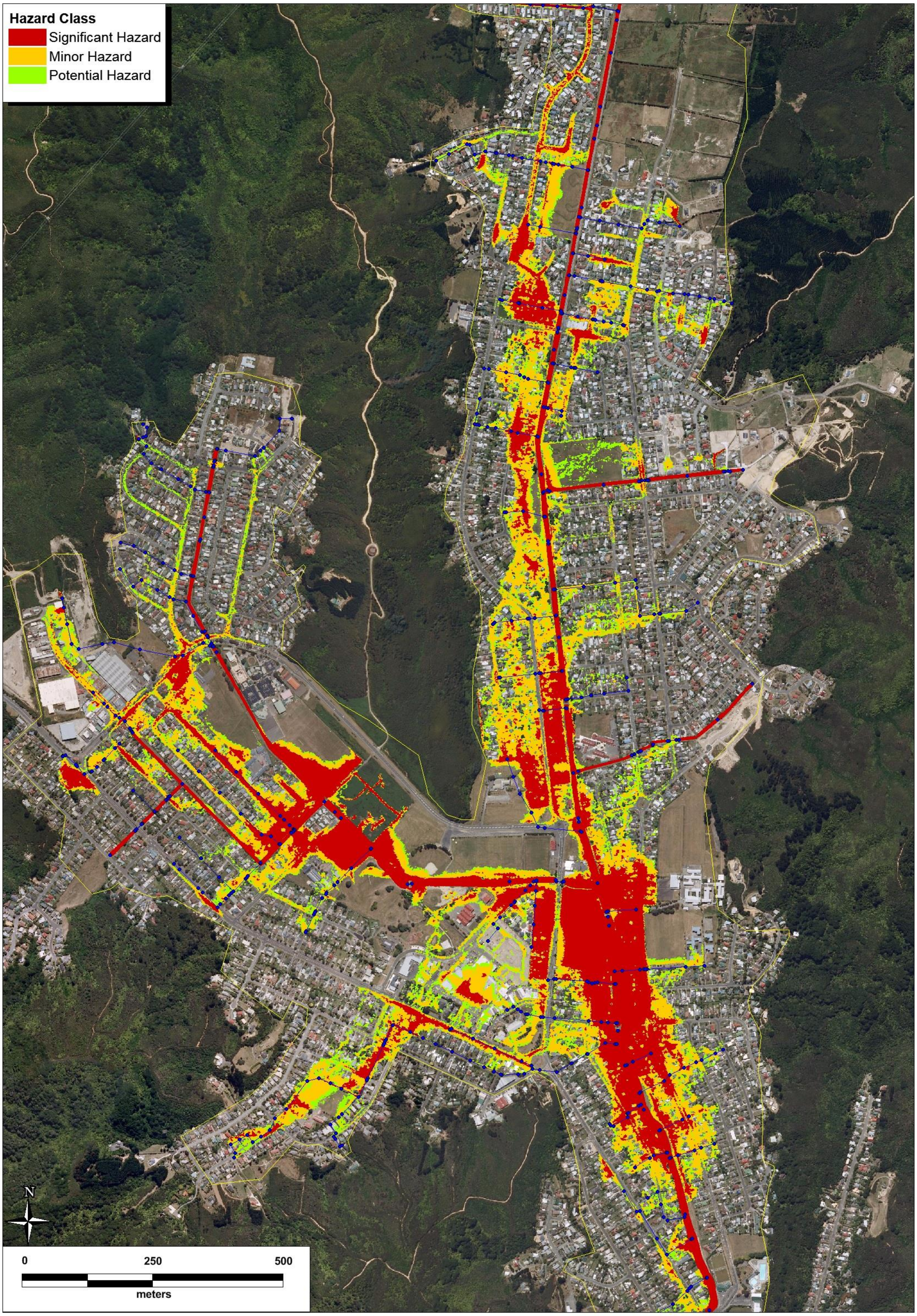


Figure-10-49: Hazard map of existing development for future climate change (2090)

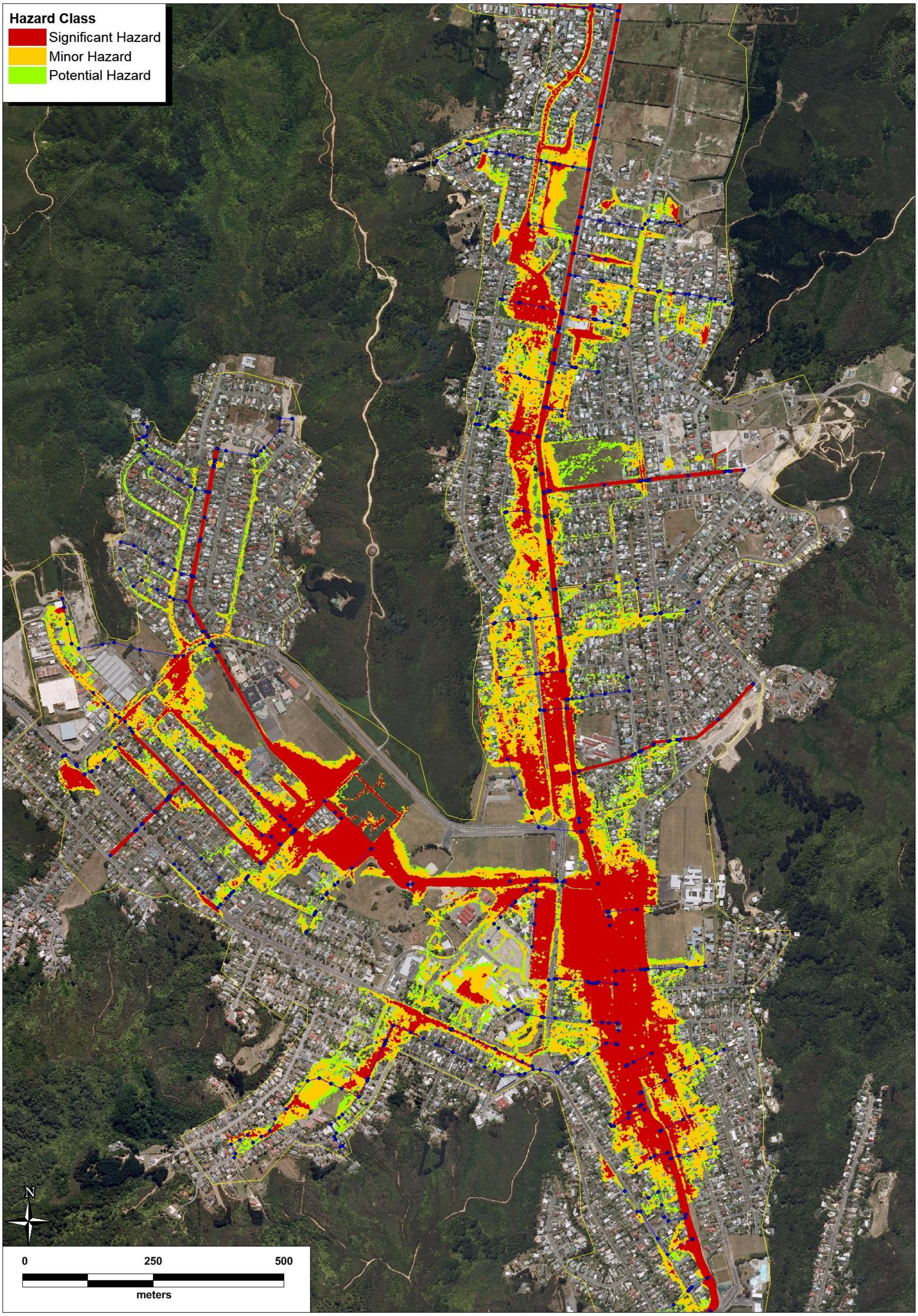


Figure 10-50: Hazard map of future development for future climate change (2090)

Figure 10-51: Matthews Rd

Figure 10-52: Kendal Grove

Figure 10-53: Wellington Road , Whitehall Rd Intersection

Figure 10-54: Norfolk St between Upper Fitzherbert and Wellington Road

	
Figure 10-55: Wimbledon Grove	**Figure 10-56: Intersection of Norfolk and Honey St**

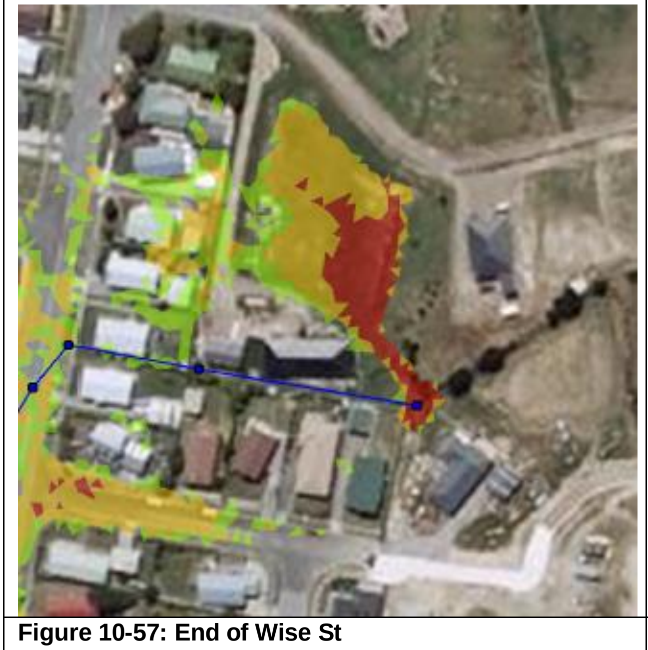



Figure 10-57: End of Wise St	**Figure 10-58: Westminster Road around no. 61**

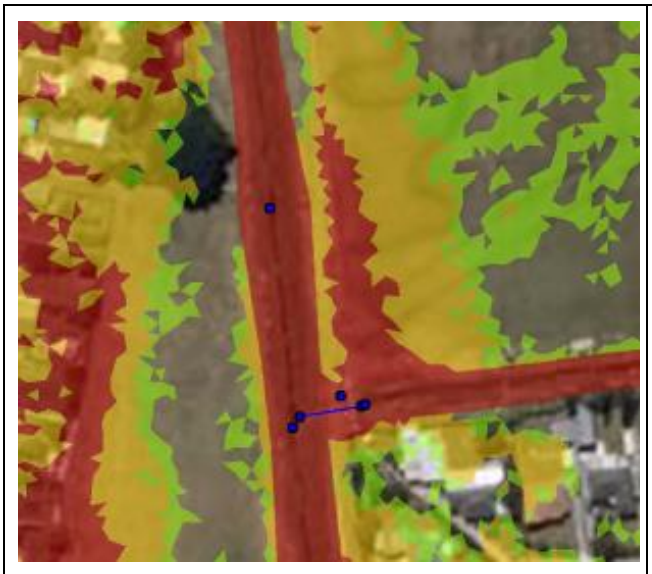
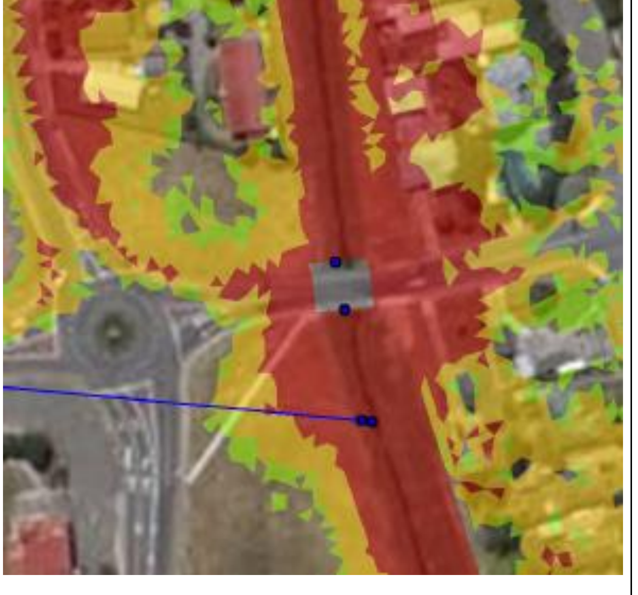


Figure 10-59: Mary Crowther Park	**Figure 10-60: Wellington Road near Devon St**
	
Figure 10-61: Wellington Road near Parkway Drive	**Figure 10-62: Nelson Crescent Bridge**

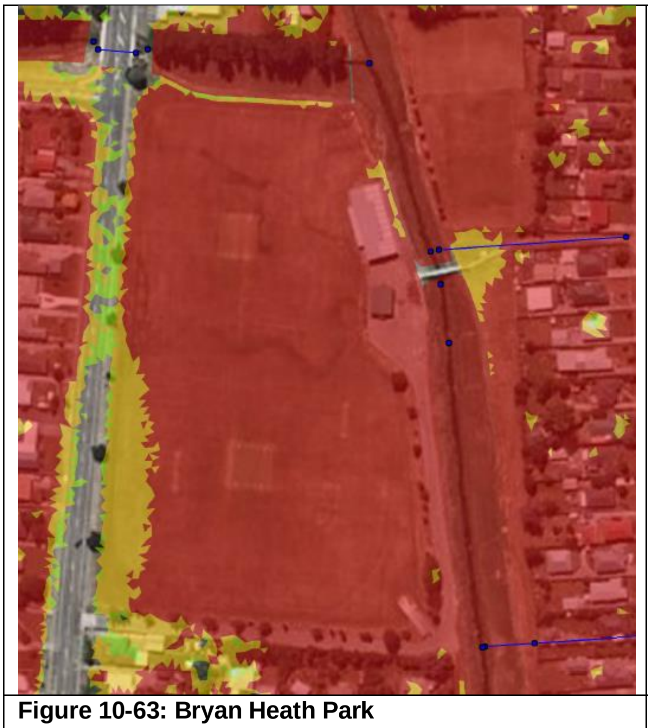
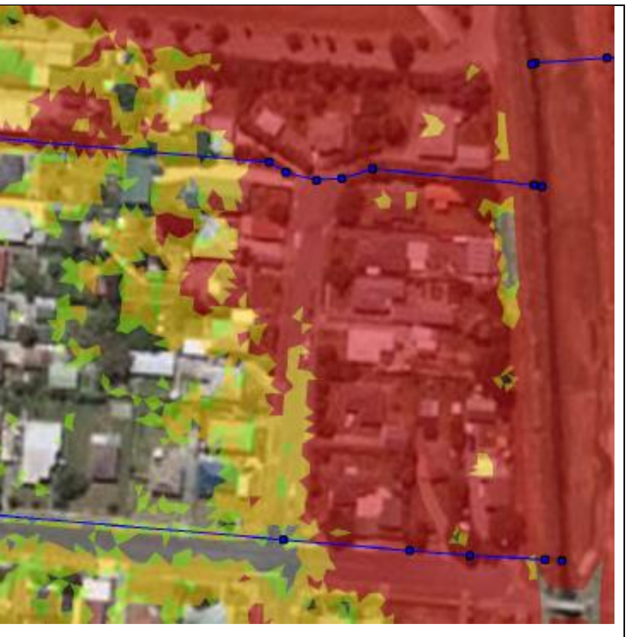
	

Figure 10-63: Bryan Heath Park	**Figure 10-64: Hyde St**

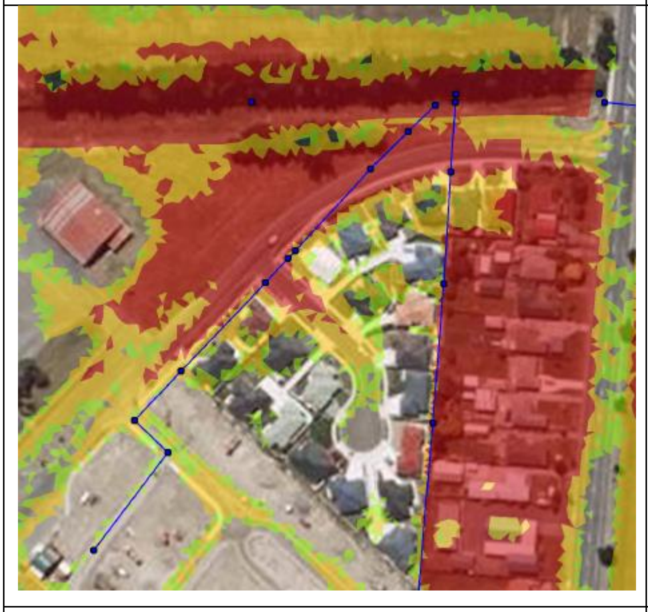
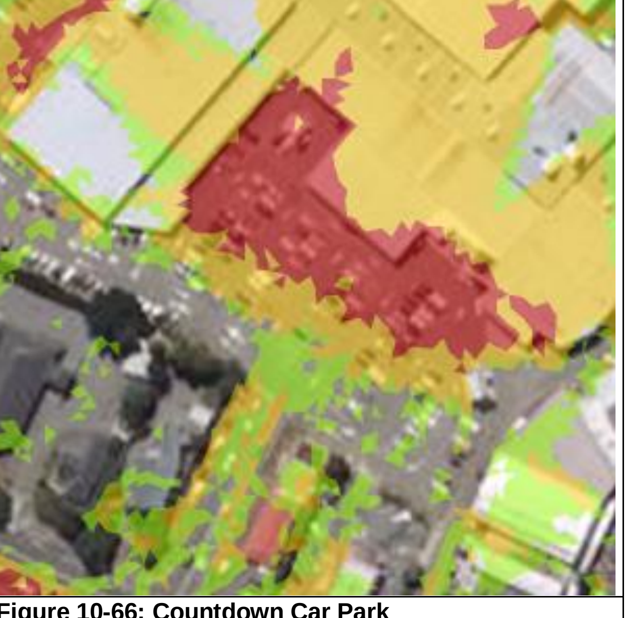

	

Figure 10-65: Coner of Fitzherbert & the Strand	**Figure 10-66: Countdown Car Park**

Figure 10-67: Best St /Black Creek

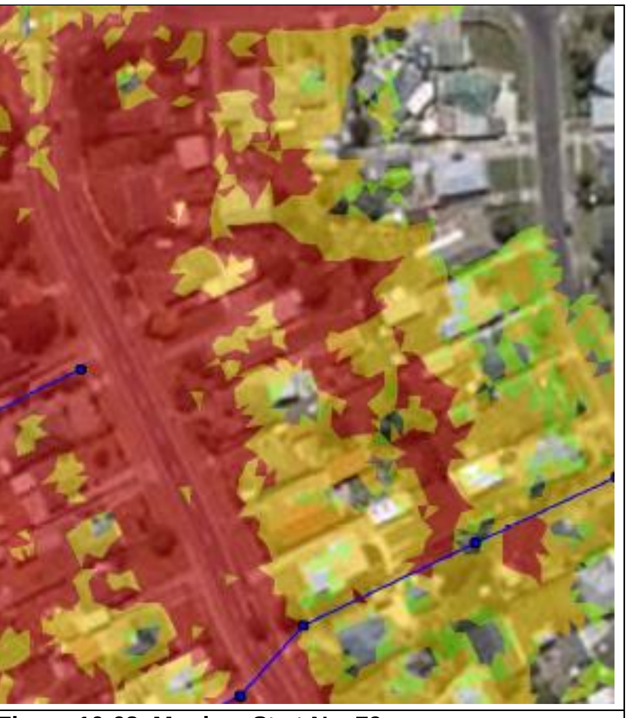


Figure 10-68: Moohan St at No. 79

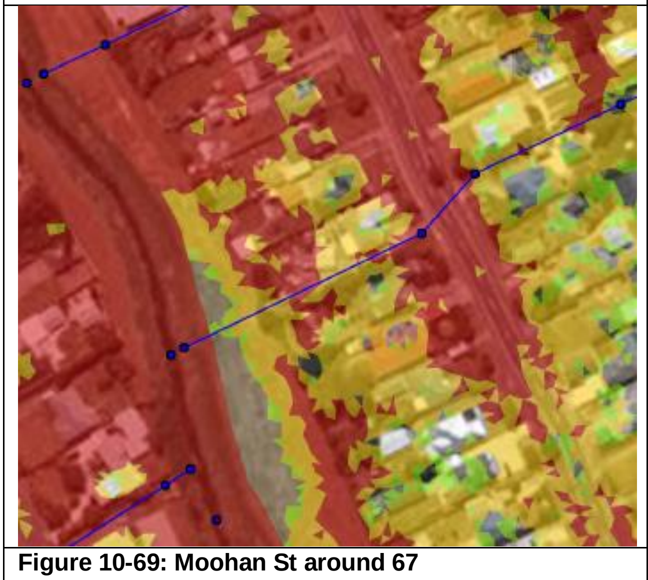


Figure 10-69: Moohan St around 67

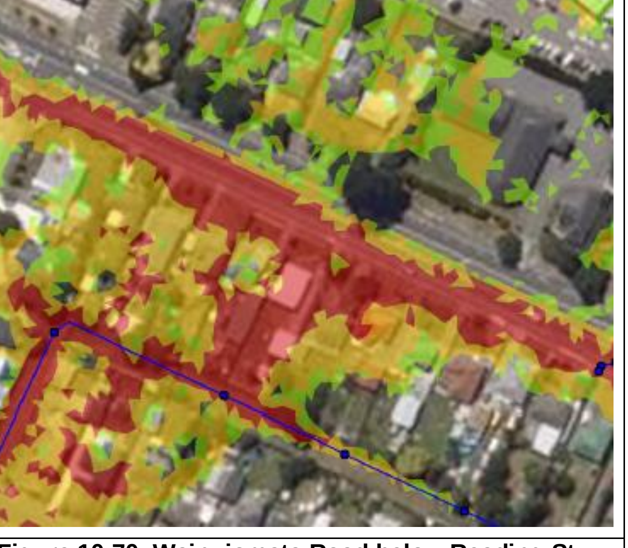


Figure 10-70: Wainuiomata Road below Reading St

Figure 10-71: Fernlea School, Lees Grove

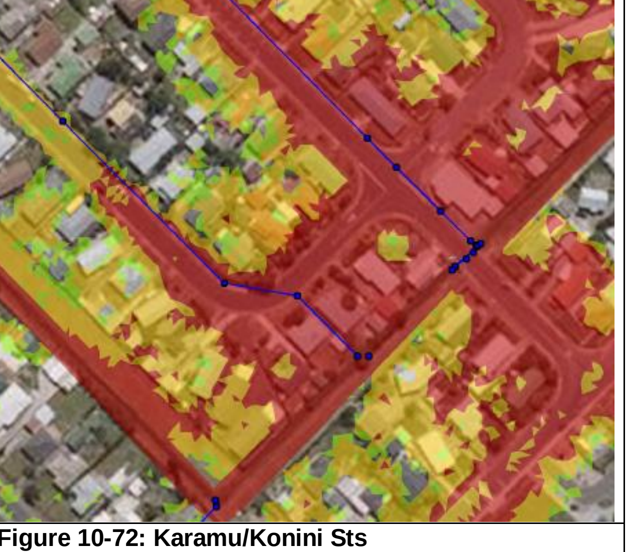


Figure 10-72: Karamu/Konini Sts

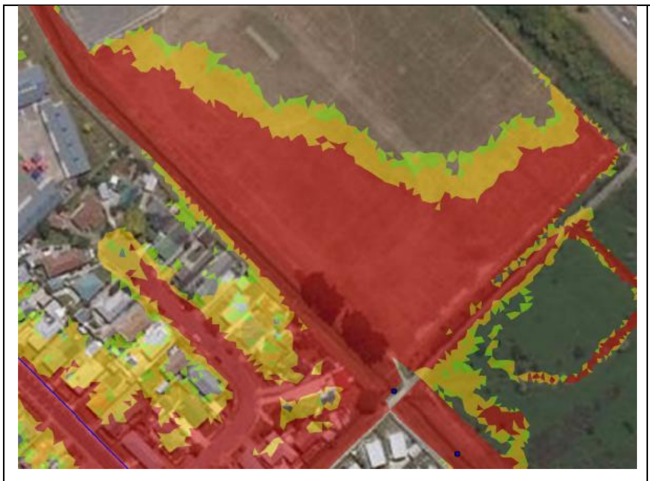


Figure 10-73: Wainuiomata High School playing field

Figure 10-74: Corner of Parkway Drive and Wainuomata Rd

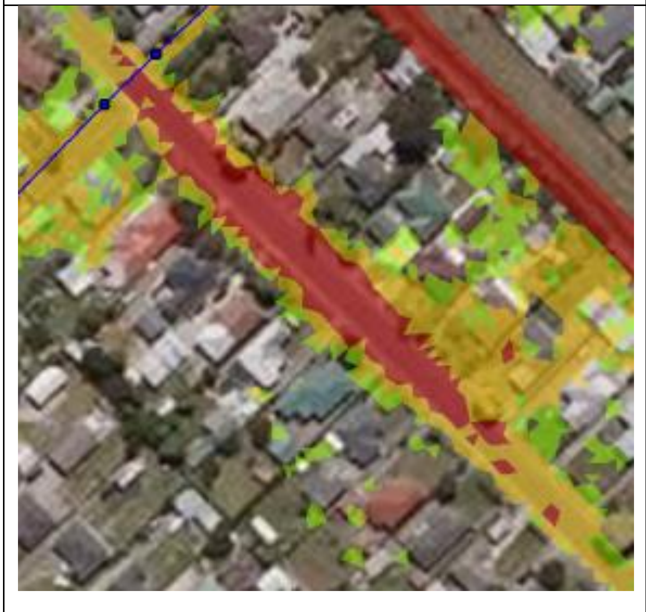


Figure 10-75: Totara St around 61

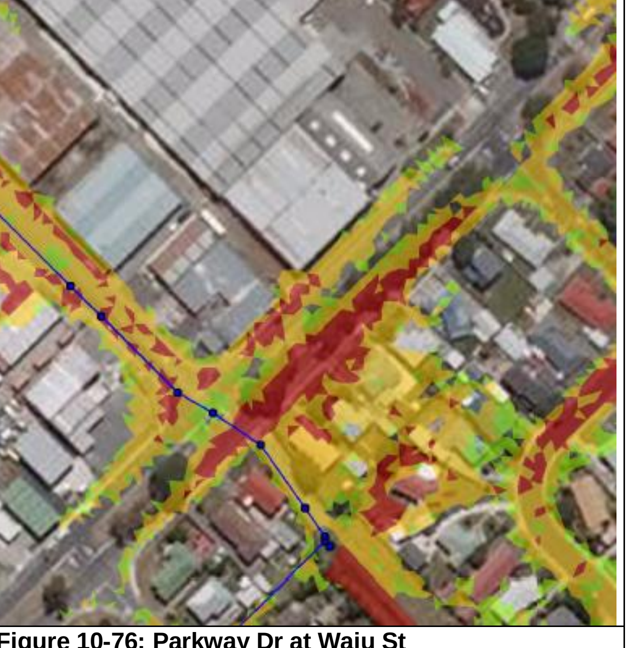


Figure 10-76: Parkway Dr at Waiu St

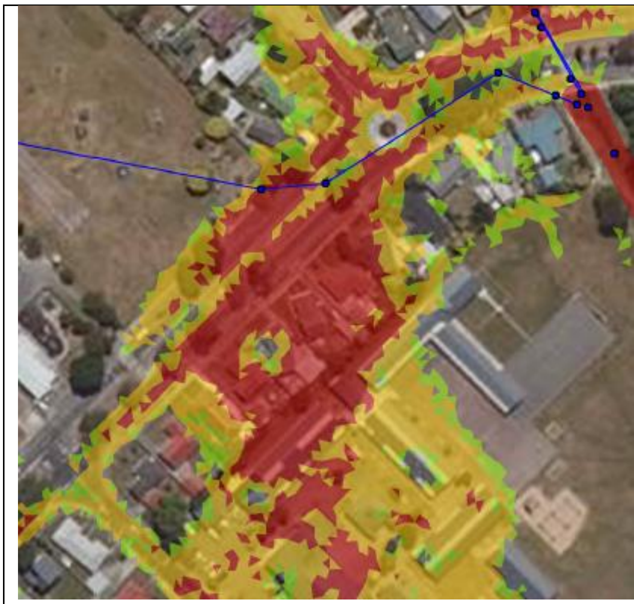


Figure 10-77: Parkway Dr near Mohaka St

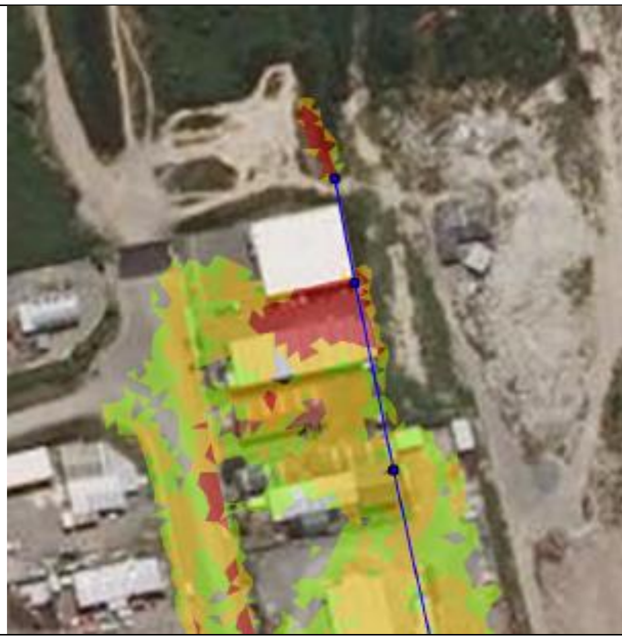


Figure 10-78: WaiuSt /Mtnbike Park

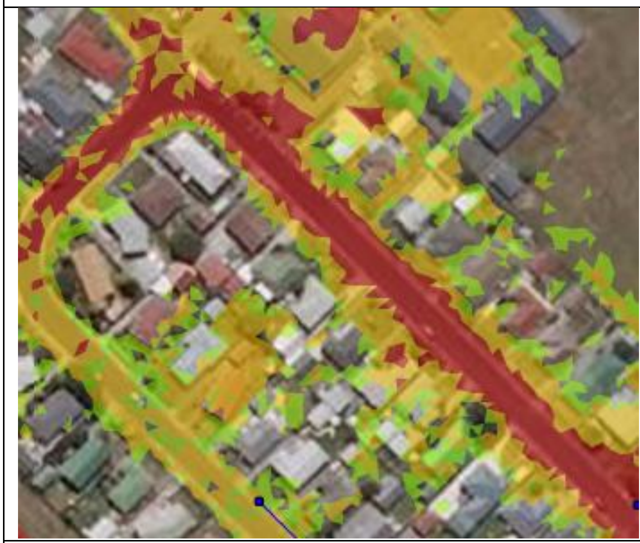


Figure 10-79: Upper KoniniSt

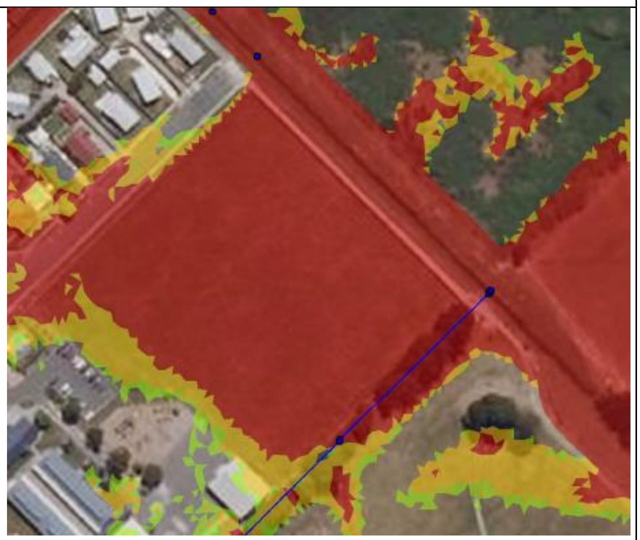
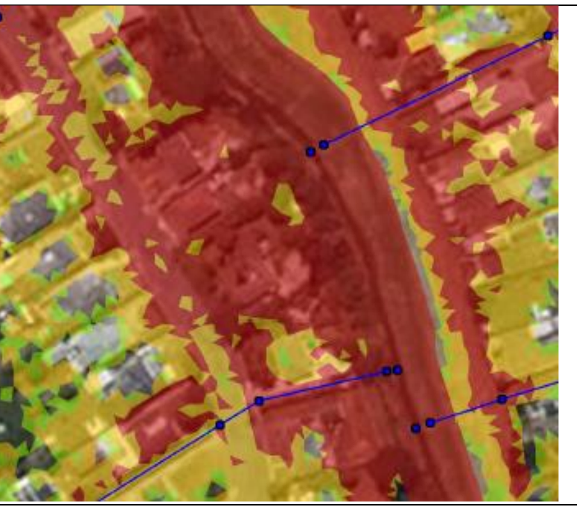


Figure 10-80: St Claudine Thevenet School Playing Field

Figure 10-81: Black Creek at Wright St

Figure 10-82: Main Rd near No. 40

	
Figure 10-83: Corner of Hyde & Arthur St	**Figure 10-84: Hyde St near 28**

Appendix G 1D Network Boundary Condition Assessment

Black Creek discharges into the Wainuiomata River approximately 0.65km downstream of Main Rd Bridge. A sensitivity analysis has been carried out to assess the impact of the levels in the Wainuiomata River on levels along Black Creek. To do this a small portion of the Wainuiomata River was modelled approximately 350m upstream and 350m downstream of the confluence with Black Creek. Design flow hydrographs for the modelled section of the Wainuiomata River were obtained from the Regional Council MIKE11 model and applied to the current model. A range of flows were applied to the Wainuiomata River portion of the network to ensure a range of water levels were achieved in the channel. The maximum water level reached during the sensitivity scenarios at the confluence was 83.0m which corresponds with a peak stage of 83m in the vicinity of the confluence in the MIKE 11 model for the 100yr ARI, with climate change, scenario. Variation in the head loss expected at the confluences have also included in the assessment.

The time to peak in the Wainuiomata River hydrograph at the confluence is approximately 15.5 hours. In Black Creek model the time to peak for a 24-hour event was 12.75 hours. No adjustment was made to align the peaks in the model runs for this assessment. This is because it is likely that during a uniformly distributed rainfall event the peak flow from Black Creek would reach the confluence before the peak in the Wainuiomata River.

The results of the analysis show that the varied water levels in the Wainuiomata River have minimal impact on water levels (approximately 0.1m or less) in Black Creek upstream of Main Rd Bridge. There is also very little difference in water levels when an open, normal condition, boundary is applied to the model.

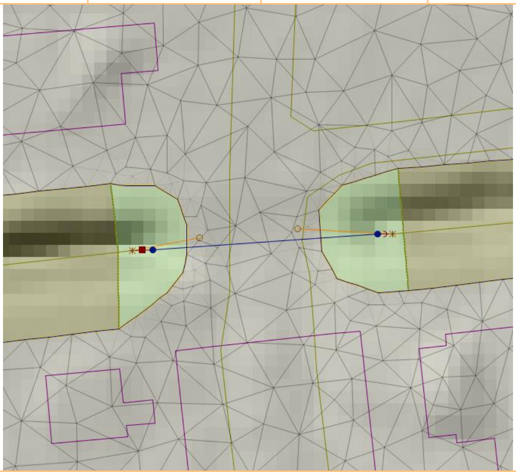
Table 10-7 shows the water levels in Black Creek at selected locations along the modelled reach for the various scenarios modelled.

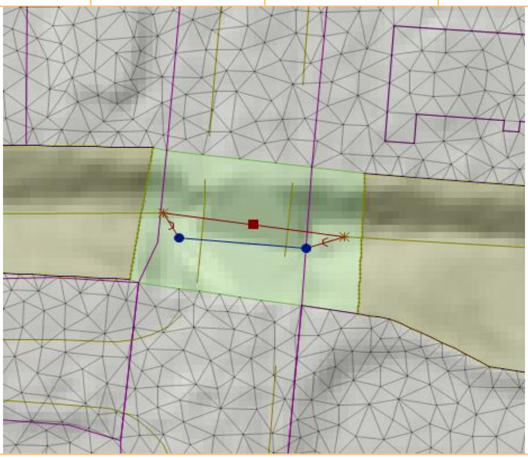
Table 10-7 Peak Water levels under selected downstream boundary condition scenarios

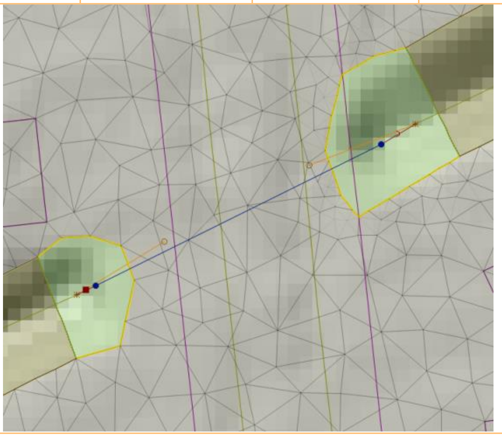
Scenario	300m upstream of confluence	Main Rd Bridge	Best St Bridge	Park Drain – Black Creek confluence	Norfolk St Bridge	Upstream boundary of model network
10yr – Current climate	82.95	84.13	85.50	85.97	89.15	95.30
10yr – Future climate	82.96	84.13	85.50	85.97	89.15	95.30
10yr – Future climate + 150m^3/s	83.19	84.18	85.50	85.97	89.15	95.30
10yr – normal condition boundary	82.97	84.13	85.50	85.97	89.15	95.30
10yr – Future climate + 150m^3/s, medium headloss	83.33	84.21	85.51	85.97	89.15	95.30
10yr – Future climate + 150m^3/s, high headloss	83.46	84.25	85.51	85.97	89.15	95.30
100yr – Current climate	83.56	84.85	86.19	86.62	89.63	95.72
100yr – Future climate	83.56	84.85	86.19	86.62	89.63	95.72
100yr – Future climate + 150m^3/s	83.66	84.87	86.19	86.62	89.63	95.72
100yr – normal condition boudary	83.58	84.85	86.19	86.62	89.63	95.72
100yr – Future climate + 150m^3/s, medium headloss	83.82	84.90	86.20	86.62	89.63	95.72
100yr – Future climate + 150m^3/s, high headloss	83.99	84.94	86.20	86.62	89.63	95.72

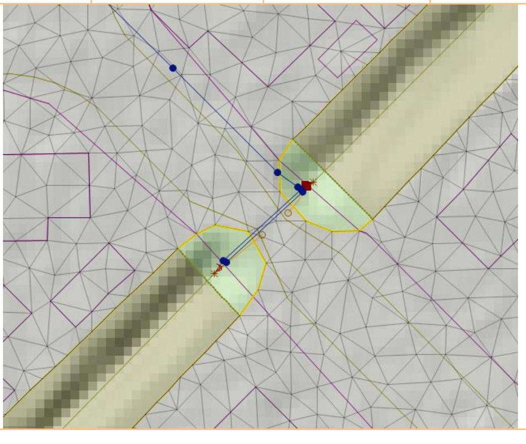
Appendix H Crossing Details

.1 Road Crossings

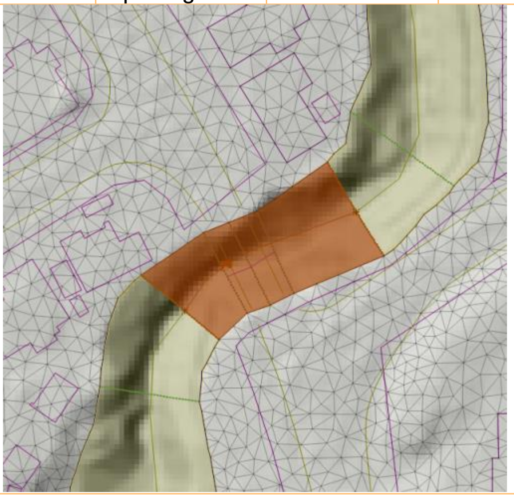
Bridge Location	Crossing Type	Height (m)	Width (m)	Road Level (m AD)	Inlet Type	Outlet Type	Overtopping Type
Ashburn Rd	Circular Culvert	1.2	1.2	98.1	Culvert inlet	Weir	Inline Bank
				No Image Available			

Bridge Location	Crossing Type	Height (m)	Width (m)	Road Level (m AD)	Inlet Type	Outlet Type	Overtopping Type
Fitzherbert Rd	Box Culvert	3.35	6.5	87.8	Culvert Inlet	Culvert Outlet	Weir
							

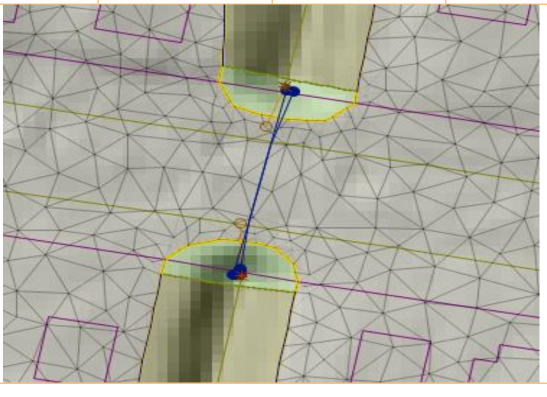
Bridge Location	Crossing Type	Height (m)	Width (m)	Road Level (m AD)	Inlet Type	Outlet Type	Overtopping Type
Frederick St Stream 2	Circular Culvert	0.75	0.75	90.6	Culvert in	Weir	Inline bank
							

Bridge Location	Crossing Type	Height (m)	Width (m)	Road Level (m AD)	Inlet Type	Outlet Type	Overtopping Type
Konini St	Twin Box Culverts	1.2	1.4	86.7	Culvert inlet	Weir	Inline bank
							

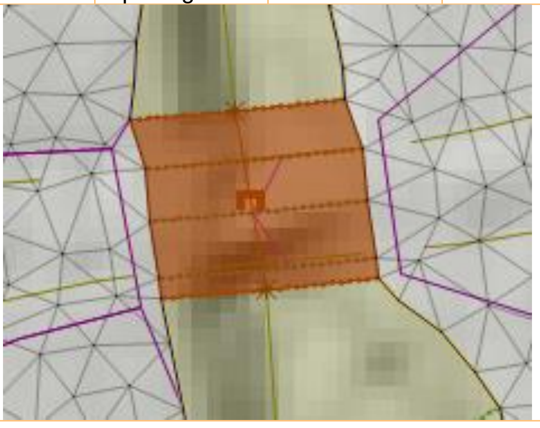
Bridge Location	Crossing Type	Height (m)	Width (m)	Road Level (m AD)	Inlet Type	Outlet Type	Overtopping Type
Main Rd	Bridge – user defined opening	4.7	15	86.2	ICM Bridge Link	ICM Bridge Link	ICM Bridge Link



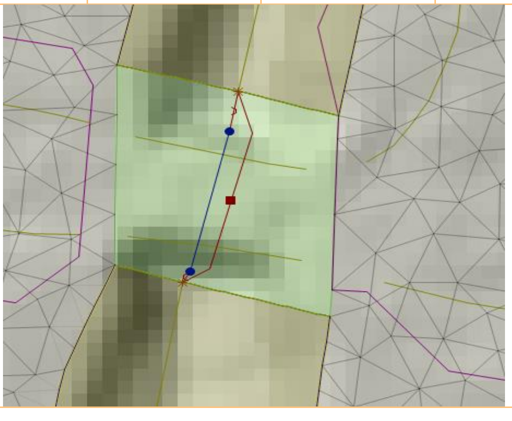
Bridge Location	Crossing Type	Height (m)	Width (m)	Road Level (m AD)	Inlet Type	Outlet Type	Overtopping Type
Manutuke	Twin Box Culverts	0.9	0.9	98.0	Culvert Inlet	Weir	Inline Bank

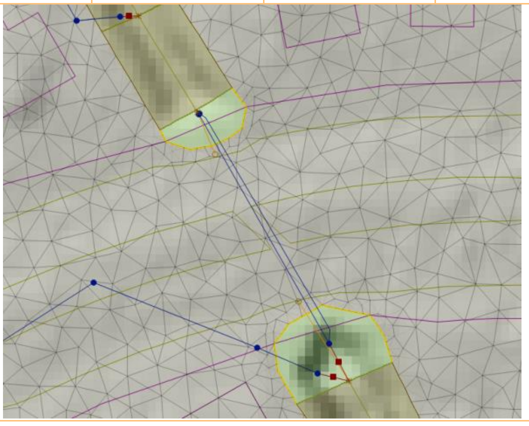


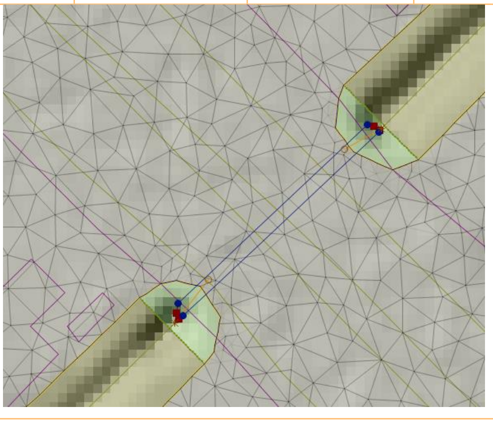
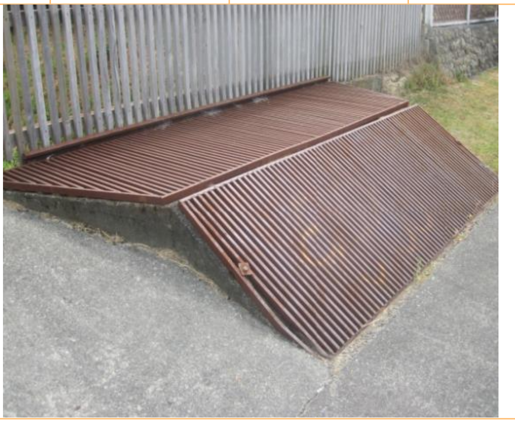
Bridge Location	Crossing Type	Height (m)	Width (m)	Road Level (m AD)	Inlet Type	Outlet Type	Overtopping Type
Nelson Cr	Bridge – user defined opening	2.1	10	86.4	ICM Bridge Link	ICM Bridge Link	ICM Bridge Link



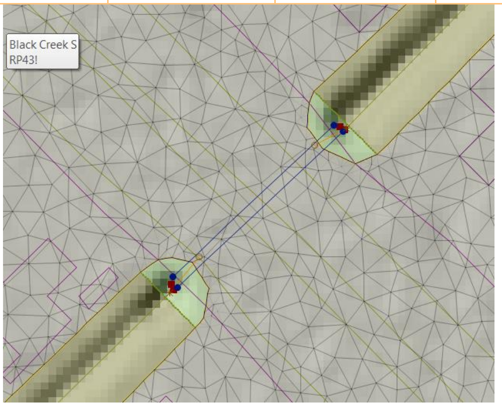
Bridge Location	Crossing Type	Height (m)	Width (m)	Road Level (m AD)	Inlet Type	Outlet Type	Overtopping Type
Norfolk St	Box Culvert	2.1	10	90.3	Culvert inlet	Culvert Outlet	Weir



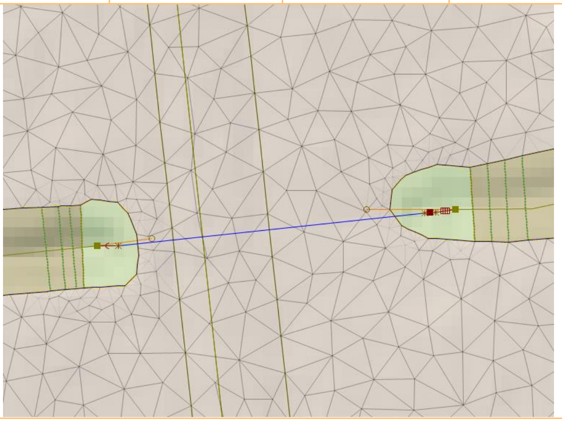
Bridge Location	Crossing Type	Height (m)	Width (m)	Road Level (m AD)	Inlet Type	Outlet Type	Overtopping Type
Parkway Dr	Twin Circular Culverts	0.9	0.9	91.0	Culvert inlet	Weir	Inline Bank
							

Bridge Location	Crossing Type	Height (m)	Width (m)	Road Level (m AD)	Inlet Type	Outlet Type	Overtopping Type
Totara st	Twin Box Culverts	1.2	1.2	89.6	Weir	Weir	Inline bank
							

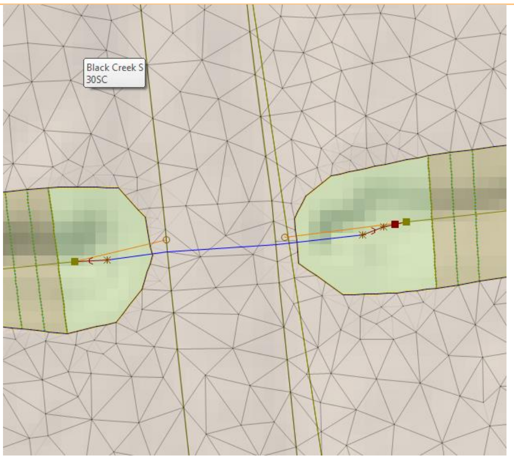
Bridge Location	Crossing Type	Height (m)	Width (m)	Road Level (m AD)	Inlet Type	Outlet Type	Overtopping Type
Wainuiomata Rd	Twin Box Culverts	1.2	1.2	89.6	Weir	Weir	Inline bank



Bridge Location	Crossing Type	Height (m)	Width (m)	Road Level (m AD)	Inlet Type	Outlet Type	Overtopping Type
Wise St Stream 2	Circular Culvert	0.9	0.9	92.6	Screen to Weir	Culvert outlet	Inline bank

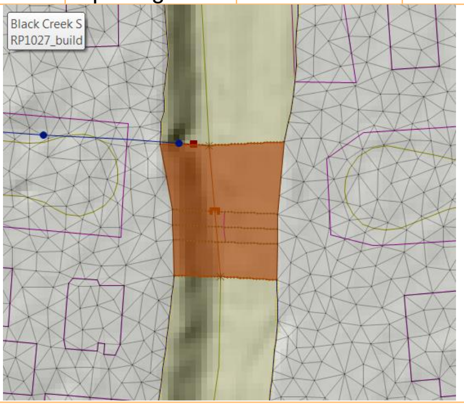


Bridge Location	Crossing Type	Height (m)	Width (m)	Road Level (m AD)	Inlet Type	Outlet Type	Overtopping Type
Wise St Stream 1	Circular Culvert	0.9	0.9	92.6	Weir to culvert inlet	Culvert outlet	Inline bank

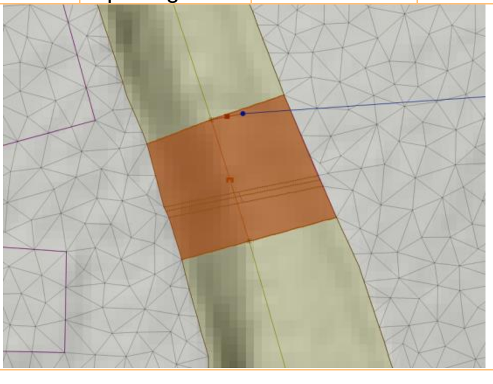

Black Creek S
30SC

.2 Footbridge Crossings

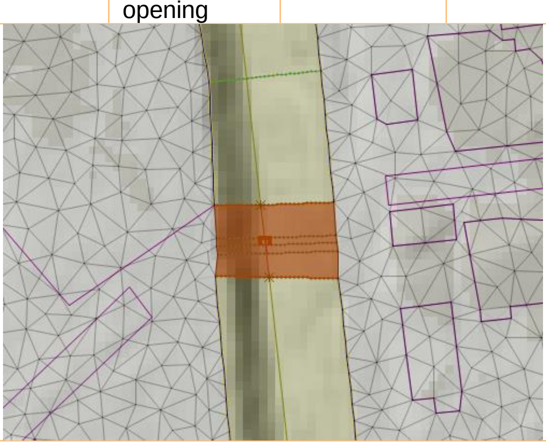
Bridge Location	Crossing Type	Height (m)	Width (m)	Road Level (m AD)	Inlet Type	Outlet Type	Overtopping Type
Best St	Bridge – user defined opening	3.7	18	86.6	ICM Bridge Link	ICM Bridge Link	ICM Bridge Link



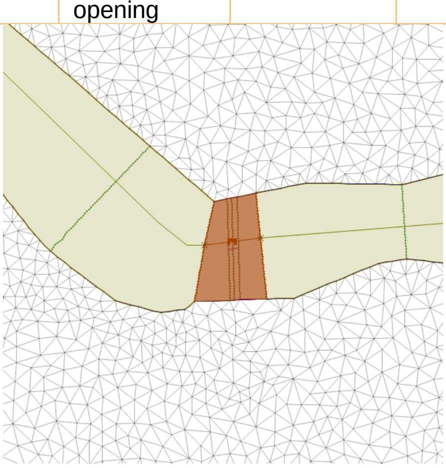
Bridge Location	Crossing Type	Height (m)	Width (m)	Road Level (m AD)	Inlet Type	Outlet Type	Overtopping Type
Bryan Heath Park	Bridge – user defined opening	3.85	17	86.9	ICM Bridge Link	ICM Bridge Link	ICM Bridge Link

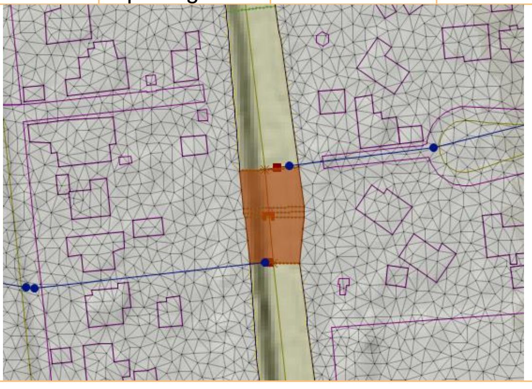


Bridge Location	Crossing Type	Height (m)	Width (m)	Road Level (m AD)	Inlet Type	Outlet Type	Overtopping Type
Edmonds St	Bridge – user defined opening	2.2	10	88.2	ICM Bridge Link	ICM Bridge Link	ICM Bridge Link

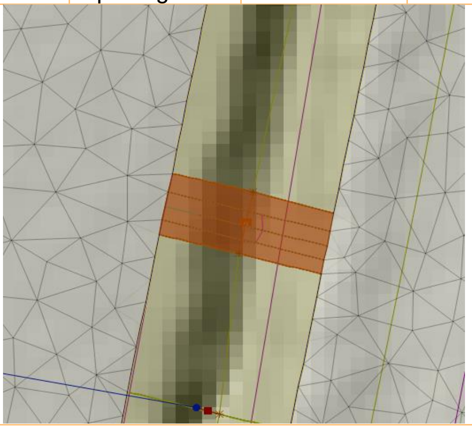


Bridge Location	Crossing Type	Height (m)	Width (m)	Road Level (m AD)	Inlet Type	Outlet Type	Overtopping Type
Frederick Wise Park	Bridge – user defined opening	2.5	22	86.9	ICM Bridge Link	ICM Bridge Link	ICM Bridge Link



Bridge Location	Crossing Type	Height (m)	Width (m)	Road Level (m AD)	Inlet Type	Outlet Type	Overtopping Type
McKay St	Bridge – user defined opening	2.4	15	87.3	ICM Bridge Link	ICM Bridge Link	ICM Bridge Link
							

Bridge Location	Crossing Type	Height (m)	Width (m)	Road Level (m AD)	Inlet Type	Outlet Type	Overtopping Type
Rata St	Bridge – user defined opening	2	20	87.2	ICM Bridge Link	ICM Bridge Link	ICM Bridge Link
							

<table>
<tr><th>Bridge Location</th><th>Crossing Type</th><th>Height (m)</th><th>Width (m)</th><th>Road Level (m AD)</th><th>Inlet Type</th><th>Outlet Type</th><th>Overtopping Type</th></tr>
<tr><td>Weymouth Gr</td><td>Bridge - user defined opening</td><td>2.7</td><td>10</td><td>91.2</td><td>ICM Bridge Link</td><td>ICM Bridge Link</td><td>ICM Bridge Link</td></tr>
<tr><td colspan="4"></td><td colspan="4"></td></tr>
</table>

CREATING COMMUNITIES

Communities are fundamental. Whether around the corner or across the globe, they provide a foundation, a sense of belonging. That's why at Stantec, we always **design with community in mind**.

We care about the communities we serve—because they're our communities too. We're designers, engineers, scientists, and project managers, innovating together at the intersection of community, creativity, and client relationships. Balancing these priorities results in projects that advance the quality of life in communities across the globe.

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