Eastern Lower Hutt Stormwater Model Build

PREPARED FOR WELLINGTON WATER LIMITED | March 2022

We design with community in mind

Stantec

Revision Schedule

Rev No.	Date	Description	Signature or Typed Name (documentation on file)			
			Prepared by	Checked by	Reviewed by	Approved by
1	29.01.2021	Draft 1 for External Peer Review	Calum Bradbury/ Andrew Sherson	Abby Burdis	Colin Horne	David McDonald
2	29.03.2021	Draft 2 for External Peer Review	Calum Bradbury/ Andrew Sherson	Abby Burdis	Colin Horne	David McDonald
3	10/09/2021	Draft 3 Sensitivity	Calum Bradbury/ Andrew Sherson	Abby Burdis	Colin Horne	David McDonald
4	10/03/2022	Review update	Calum Bradbury	Andrew Sherson	Colin Horne	David McDonald
5	25/08/2022	Freeboard Update	Ben Caldwell	Andrew Sherson	Avinash Gangurde	Stephan Pretorius

PROJECT MANAGER	**PROJECT TECHNICAL LEAD**
David MacDonald	Colin Horne/Andrew Sherson

PREPARED BY		
Ben Caldwell		31 / 08 / 2022
CHECKED BY		
Andrew Sherson		31 / 08 / 2022
REVIEWED BY		
Avinash Gangurde		01 / 09 / 2022
APPROVED FOR ISSUE BY		
Stephan Pretorius		02 / 09 / 2022

WELLINGTON
Level 15, 10 Brandon Street, Wellington 6011
PO Box 13-052, Armagh, Christchurch 8141
TEL +64 381 6700

STATUS Final | Project No 310103203

Executive Summary

Stantec as part of the Wellington Water Ltd (WWL) Modelling Panel completed the build, validation, and calibration of an integrated 1-D/2-D model of the Lower Hutt area to the east of the Hutt River (Eastern Lower Hutt). The objective was to develop a model that followed the Regional Stormwater Hydraulic Modelling Specifications Version v5 by Wellington Water Ltd (2017) and included all known stormwater assets so that flooding hazards could be understood. This work is part of Wellington Water Ltd overarching plan to model the urban centres in the Wellington Region to aid local councils and decision makers.

The model was built in InfoWorks Integrated Catchment Modelling (ICM) version 9.5 and then validated and calibrated against the 2004 and 2016 high rainfall events in Wellington. This calibration process involved a more in-depth investigation than previous WWL models due to the availability of historical gauged information and the Governance Board partnership with Greater Wellington Regional Council (GWRC) who had aims of improving their understanding of the flooding risks of the Waiwhetū stream.

The validation process showed that the model results largely agreed with flood levels and flow along the Waiwhetū Stream during both events. Overall, the results from the model are suited to district plan mapping, and catchment-scale analysis during high magnitude events. It is, however, recommended that if the model is used as input to detailed infrastructure design, the area of interest should be reassessed, and asset data should be confirmed.

Model sensitivity analysis was undertaken in 2021 with the intention of understanding the sensitivity of the stormwater network to a range of different model parameters. These would be used to select values for freeboard allowance to develop a baseline flood extent to be used for assigning minimum building floor levels for Eastern Lower Hutt (ELH).

A freeboard analysis was completed in 2022 using agreed uplift with Wellington Water to provide the 100-year water levels used for input into district plan maps.

Table of Contents

EXECUTIVE SUMMARY **1**

ABBREVIATIONS .... **VIII**

1.0 INTRODUCTION **9**
1.1 BACKGROUND 9
1.2 CATCHMENT OVERVIEW 9
1.2.1 Hutt River 9
1.2.2 Wellington Harbour 9
1.2.3 Waiwhetū Stream 9
1.2.4 Urban Network 11
1.3 ACTIVITIES AND SCOPE 11

2.0 AVAILABLE INFORMATION **11**
2.1 PREVIOUS WORK 11
2.2 DRAINAGE NETWORK DATA 12
2.2.1 Asset Data 12
2.2.2 As-Built Data 12
2.2.3 Pumping Stations 12
2.2.4 Topographical Data 13
2.2.5 Cross Section Data 13
2.2.6 Reported Flood Issues 13
2.2.7 Site Visit 18
2.3 HYDROLOGIC/HYDROMETRIC DATA 18
2.3.1 Rainfall Data 18
2.3.2 Flow Data 19
2.3.3 Tidal Data 19
2.3.4 Regional Hydrology Layers 19

3.0 MODEL BUILD **20**
3.1 MODEL BUILD OVERVIEW 20
3.2 HYDROLOGICAL MODEL 20
3.2.1 Subcatchment Delineation 20
3.2.2 1-D Hydrology 22
3.3 HYDRAULIC MODEL 27
3.3.1 Hydraulic Model Overview 27
3.3.2 1-D Reticulated Network 29
3.3.3 1-D River Reach Network 34
3.3.4 2-D Model Build 40
3.3.5 1-D/2-D Connections 44
3.4 BOUNDARY CONDITIONS 45
3.4.1 Rainfall Data 45
3.4.2 Hutt River 46
3.4.3 Tide 47

4.0 RESULTS **49**
4.1 MODEL LIMITATIONS AND ASSUMPTIONS 49
4.1.1 Model Limitations 49
4.1.2 Hydraulic Model Assumptions 49

4.1.3 Hydrological Model Assumptions 49
4.2 INITIAL MODEL TESTING 49
4.2.1 Stability Tests.................... 49
4.3 PRELIMINARY RESULTS.................... 50
4.4 MODEL QUALITY ASSURANCE 54

5.0 CALIBRATION.................... 54
5.1 WHITES LINE EAST GAUGE 54
5.2 ALTERATIONS TO THE WAIWHETŪ STREAM 55
5.3 QUEENS WHARF AS A TIDAL BOUNDARY 56
5.4 SENSITIVITY TESTING FOR CALIBRATION 56
5.4.1 Curve Number Adjustment 56
5.4.2 Stream Roughness Adjustments 56
5.5 RESULTS OF CALIBRATION 57

6.0 VALIDATION 61
6.1 15-16 FEBRUARY 2004 RAINFALL EVENT 62
6.1.1 Model Inputs 63
6.1.2 Comparison to Observed Data 66
6.2 14-15 NOVEMBER 2016 RAINFALL EVENT 76
6.2.1 Model Inputs 76
6.2.2 Comparison to Observed Data 78
6.3 LIMITATIONS AND ASSUMPTIONS.................... 90
6.4 SUMMARY.................... 90

7.0 SENSITIVITY AND FREEBOARD 91
7.1 SENSITIVITY ANALYSIS 91
7.1.1 50% increase in rainfall 91
7.1.2 Inlets Fully Blocked 93
7.1.3 Inlets Partially Blocked 95
7.1.4 No Tidal Boundary 97
7.1.5 No Hutt River Inflow 99
7.1.6 All Bridges Blocked 101
7.1.7 All Pumps Turned Off 103
7.2 SENSITIVITY CONCLUSIONS.................... 105
7.3 FREEBOARD ANALYSIS.................... 105
7.3.1 Freeboard Allowance Selection 105
7.3.2 Freeboard Simulation Setup.................... 105
7.3.3 Dynamic Freeboard Results.................... 105

8.0 CONCLUSIONS AND RECOMMENDATIONS 109

9.0 REFERENCES.................... 110

LIST OF TABLES

Table 2-1 Summary of available data at GWRC rainfall gauges. Coordinates in New Zealand Geodetic Datum (NZGD) 2000. 18
Table 3-1 Summary of subcatchment area.................... 22
Table 3-2 InfoWorks ICM node summary, count of public and private assets 29
Table 3-3 Missing invert data summary, count of public and private assets 30

Table 3-4 InfoWorks ICM conduit diameter summary, count of public and private assets 30
Table 3-5 InfoWorks ICM conduit material summary, count of public and private assets, and conduit roughness 31
Table 3-6 Summary of inlet parameters applied to culvert inlets within InfoWorks ICM 32
Table 3-7 Pump station capacity and on/off levels 33
Table 3-8 Manning's N roughness characteristics 35
Table 3-9 Overview of surface roughness Manning's N 42
Table 3-10 HIRDSv4 Rainfall depths for the central ELH point (1762492, 5436908 NZGD2000) 46
Table 4-1 Mass and volume errors in simulations 50
Table 5-1 Summary of calibration data at the Whites Line East gauge. 61
Table 6-1 Summary of model data. 63
Table 6-2 Total rainfall depths for the gauges applied to the February 2004 simulation. 64
Table 6-3 Summary of key model inputs. 76
Table 6-4 Summary of rainfall radar data (November 2016). 76
Table 9-1 Pump station asset information summary 130
Table 9-2 Data flags and description as used in InfoWorks ICM 131
Table 9-3 Difference between HIRDSv4 and HIRDSv3 at the Birch Lane GWRC rain gauge. 136
Table 9-4 InfoWorks ICM bridge parameters. All default values excluding inverts and skew angle. 137
Table 9-5 Flood level comparison. 138

LIST OF FIGURES

Figure 1-1 Eastern Lower Hutt catchment overview 10
Figure 2-1 Overview of the digital terrain model produced using the three data sources. 14
Figure 2-2 Flooding issues (blue) reported to WWL. February 2004 event. Sourced from WWL. 15
Figure 2-3 Flooding from the Waiwhetū Stream during 16th of February 2004 event (Westlake & Manolache, 2016) 16
Figure 2-4 Impact of the Waiwhetū Stream Bursting its Banks During 2016 Event. Photo from Cameron Burnell quoted in (Weekes, 2016) 16
Figure 2-5 Waiwhetū Stream flooding during the November 2016 event. Photo from Newshub 15/11/2016. 17
Figure 2-6 Waiwhetū Stream flooding during the November 2016 event. Photo from Melissa Nightingale – NZ Herald 15/11/2016. 17
Figure 2-7 Tide as recorded at the Estuary Bridge and Queens Wharf gauges. November 2016 event. 19
Figure 3-1 Overview of subcatchment delineations 21
Figure 3-2 Initial abstraction is reduced to account for construction of new residential housing, and an associated increase in impervious area. 23
Figure 3-3 Curve number is reduced from an unrealistically high number on a school field. 24
Figure 3-4 Composite SCS curve numbers applied to subcatchments. 25

Figure 3-5 Composite initial abstraction values applied to subcatchments. 26
Figure 3-6 Eastern Lower Hutt stormwater network overview. 28
Figure 3-7 Overview of 1-D pond construction. 34
Figure 3-8 All 1-D river reaches, and the Manning's N applied as their bed roughness. 36
Figure 3-9 Open channel crossing overview including culverts and bridges. 37
Figure 3-10 A) Bell Road bridge connection in InfoWorks ICM, b) image of bridge from Google Maps. 38
Figure 3-11 Cleary St bridge site photo 38
Figure 3-12 Bridge opening at Cleary Street. US surveyed cross section in grey, bridge deck in pink, and opening shown in white 39
Figure 3-13 Wai-iti Crescent channel crossing example. A) Model schematisation, B) Google Street View photo 39
Figure 3-14 Concrete wall mesh zone 40
Figure 3-15 Overview of mesh zones used to reduce the total number of mesh elements. 41
Figure 3-16: Manning's n roughness as applied in the 2-D model. 43
Figure 3-17 Layout of a typical connection between a river reach and a culvert inlet/outlet. This example shows a stream crossing modelled as a culvert. 44
Figure 3-18 HIRDSv4 rainfall data overview showing the location of the GWRC Birch Lane rainfall gauge, and a depth range of approximately 5mm. Storm: 100-year ARI, 1-hour duration. 45
Figure 3-19 24-hour tidal boundary with existing conditions (1.11m peak) 47
Figure 3-20 24-hour tidal boundary with 1m sea level rise (2.1m peak) 47
Figure 3-21 Overview of the model components used to apply the 1.11m and 2.1m tides 48
Figure 4-1 Preliminary flood extents for the nested 100-year ARI design storm including a 20% climate change increase 51
Figure 4-2 Preliminary flood extents for the nested 100-year ARI design storm 52
Figure 4-3 Preliminary flood extents for the nested 10-year ARI design storm 53
Figure 5-1 Overview of Waiwhetū Stream flow data recorded at Whites Line East from 2001 to mid-2020. 54
Figure 5-2 Cross sections altered to account for the 2009-2010 Waiwhetū channel improvements. 55
Figure 5-3 Overview of channel bed roughness as applied to the Waiwhetu Stream 57
Figure 5-4: Recorded and modelled stage (mm) at Estuary Bridge. Data is unavailable for the December 2019 event. Modelled stage includes the combined impact of discharge to the Hutt River, the Taita Gorge inflow, and the Queens Wharf tide. 58
Figure 5-5 Comparison between observed and modelled level at Whites Line East 59
Figure 5-6 Comparison between observed and modelled flow at Whites Line East. 60
Figure 6-1 Frequency analysis of the Whites Line East gauge data (31st May 1978 to 21st May 2020). 62
Figure 6-2 24-hour rainfall totals for the period beginning the 15th of February 2004 at 9am (Figure 3, from (Watts & Gordon, 2004)). 63

Figure 6-3 Rainfall banding as applied to the 2004 model simulation (spatial rainfall zones 1, 2, 3 and 4) and based on Figure 6-2 from Watts and Gordon (2004). 65
Figure 6-4 Overview of data used for validation of the February 2004 event. 66
Figure 6-5 Hutt Park aerial imagery and approximate extent in relation to model simulation results. 67
Figure 6-6 Aerial imagery of Riverside Drive upstream of the Bell Road bridge and approximate extent in relation to model simulation results. 68
Figure 6-7 Aerial imagery of Te Whiti Park, the Whites Line East gauge and approximate extent in relation to model simulation results. 68
Figure 6-8 Seaview/Gracefield Aerial Imagery and approximate extent in relation to model simulation results. 69
Figure 6-9 Photo of flooding facing west at 6 Port Road combined with model simulation extents. 70
Figure 6-10 Photo downstream of the Whites Line East bridge, model simulation extents and cross section. 71
Figure 6-11 Photo of flooding, model simulation extents and cross section at the 199 Riverside Drive footbridge crossing the Waiwhetū Stream. 72
Figure 6-12 Photo of observed and modelled flooding extents at the Cleary Street-Mission Street Junction. 73
Figure 6-13 Overview of the difference between flood levels recorded by GWRC and model simulation levels. 74
Figure 6-14 Detailed view of locations with a GWRC recorded flood level/model simulation level difference of greater than 0.25m. 75
Figure 6-15 Rainfall radar catchment overview. 77
Figure 6-16 Overview of validation data (November 2016) 78
Figure 6-17 Observed and modelled flood extents at Kokiri Marae. 79
Figure 6-18 Observed and modelled flood extents on Hayward Terrace. 80
Figure 6-19 Observed flood extents, modelled flood extents and cross section at 199 Riverside Drive. 81
Figure 6-20 Observed stream level and modelled cross section at the Cleary Street Bridge 82
Figure 6-21 Observed and modelled water level at the Rossiter Avenue Bridge. 82
Figure 6-22 Observed and modelled flood extents at 321 Riverside Drive. 83
Figure 6-23 Observed and modelled stream levels at the Burnside Street footbridge. 84
Figure 6-24 Observed and modelled stream levels at the Norton Park Avenue bridge. 85
Figure 6-25 Observed flooding extents, and modelled flooding extents and cross section at the Rumgay Street-Riverside Drive junction. 86
Figure 6-26 Observed and modelled flood extents at the Heather Grove-Riverside Drive junction. 87
Figure 6-27 Observed and modelled stream levels at the Tilbury Street bridge (downstream). 88
Figure 6-28 Observed and modelled stream levels at the Tilbury Street bridge (upstream). 89
Figure 6-29 Observed and modelled flooding at 28 Cornwall Street, Lower Hutt 89

Figure 7-1 Difference in depth between the base scenario and a 50% increase in rainfall intensity ... 92
Figure 7-2 Difference in depth between the base scenario and a full blockage of all inlets and culverts ... 94
Figure 7-3 Difference in depth between the base scenario and a partial blockage of all inlets and culverts ... 96
Figure 7-4 Difference in depth between the base scenario and the removal of the tidal boundary ... 98
Figure 7-5 Difference in depth between the base scenario and no flow in the Hutt River ... 100
Figure 7-6 Difference in depth between the base scenario and a blockage of all bridges ... 102
Figure 7-7 Difference in depth between the base scenario and the removal of all pump links ... 104
Figure 7-8 Zone boundaries for the split model sections used in the freeboard simulation ... 106
Figure 7-9 Freeboard allowance applied across the Eastern Lower Hutt network ... 107
Figure 7-10 Maximum flood depths for the 100yr ARI +20% CC design event including dynamic freeboard ... 108
Figure 9-1 Surveyed cross section locations ... 112
Figure 9-2 SCS curve number layer overview. ... 127
Figure 9-3 Initial abstraction layer overview. ... 128
Figure 9-4 Roughness surface layer overview ... 129
Figure 9-5 Head-discharge curves applied to all sumps and manholes in the model. X-axis: Discharge (m^3s^{-1}), Y-axis: Head (m). ... 132
Figure 9-6 Example surveyed cross section. Purple circles show removed vertices ... 133
Figure 9-7 Example updated cross section ... 134
Figure 9-8 Standard U-shaped channel ... 134
Figure 9-9 Standard trapezoidal cross section ... 135

LIST OF APPENDICES

APPENDIX A – SURVEY DATA

APPENDIX B – ADDED ASSETS

APPENDIX C – INTERPOLATIONS AND AUTOMATIONS

APPENDIX D – REGIONAL HYDROLOGY LAYERS

APPENDIX E – PUMP STATION DATA

APPENDIX F – DATA FLAGS

APPENDIX G – HEAD-DISCHARGE CURVES

APPENDIX H – 1-D RIVER REACH CROSS SECTIONS

APPENDIX I – HIRDSV4 COMPARISON

APPENDIX J – INFOWORKS ICM BRIDGE PARAMETERS

APPENDIX K – GWRC RECORDED FLOOD LEVELS (2004)

Abbreviations

WWL	Wellington Water Limited
HCC	Hutt City Council
GWRC	Greater Wellington Regional Council
LINZ	Land Information New Zealand
NIWA	National Institute of Water and Atmospheric Research
ELH	Eastern Lower Hutt
ICM	Integrated Catchment Modeller
UMM	Urban Model Manager
DTM	Digital Terrain Model
ASL	Above Sea Level
ARI	Annual Recurrence Interval
RFHA	Rapid Flood Hazard Assessment
HIRDS	High Intensity Rainfall Design System
SCS	Soil Conservation Service
IA	Initial Abstraction
CN	Curve Number
ToC	Time of Concentration
LiDAR	Light Detection and Ranging
GIS	Geographical Information System
SQL	Structured Query Language
ESRI	Environmental Systems Research
NZGD	New Zealand Geodetic Datum
%IMP	Impervious Percentage
MWH	Montgomery Watson Harza

1.0 INTRODUCTION

1.1 BACKGROUND

Wellington Water Ltd (WWL) commissioned Stantec in 2019 to build a coupled 1-D/2-D model for Eastern Lower Hutt (ELH), covering the Lower Hutt area east of the Hutt River. This work is part of the collaborative Stormwater Modelling Panel formed by WWL to build and maintain stormwater models used for assessing urban flood risk across the Wellington Region. This report will outline the initial model build, model calibration, validation, model sensitivity, and freeboard assessment. The model calibration and validation stages were undertaken with collaboration between WWL, Stantec, and Greater Wellington Regional Council (GWRC) to ensure suitability of the model as part of the Waiwhetū Governance Group.

The model build predominately follow the standard approach set out by WWL in the Regional Stormwater Modelling Specifications v5 (Wellington Water Ltd, 2017). Slight differences may occur around the modelling of the Waiwhetū stream to incorporate the aims and requirements of both WWL and GWRC.

1.2 CATCHMENT OVERVIEW

The ELH catchment covers the Lower Hutt suburbs east of the Hutt River from the Wellington Harbour in the south to Pomare in the north, encompassing an area of 31.5km^2. Most of the ELH urban area lies on a flat floodplain with elevations ranging from approximately 1m to 10m above sea level (ASL). The low-lying urban encompasses approximately ¾ of the catchment area, excluding only the Eastern Hills. The Eastern Hills encompass the remaining ~¼ of the ELH catchment, extending up into steep forested terrain with elevations of up to ~350m (ASL). See Figure 1-1 for an overview of the ELH catchment.

1.2.1 Hutt River

The Hutt River, located along the north-western and western boundary of the ELH catchment, is the major water course for the Hutt Valley. The river is constrained by stop banks on either side of the channel, which were designed to contain the flows from approximately a 440-year Annual Reassurance Interval (ARI) storm event. The event was based on recorded climate and flow data (Greater Wellington Regional Council, 2001). Historically, much of the flat land within the Eastern Lower Hutt catchment formed the Hutt River flood plain.

1.2.2 Wellington Harbour

The catchment is bound in the south by the Wellington Harbour which influences levels in the lower, flatter reaches of the Hutt River, Awamutu Stream, and Waiwhetū Stream. The flap gates at the Opahu pumping station limit the impact of the tide on Opahu Stream levels.

1.2.3 Waiwhetū Stream

The Waiwhetū Stream runs alongside the base of the Eastern Hills, collecting runoff from the hills and the surrounding urban suburbs. There are numerous structures within the stream that may constrict flow, including 15 road bridges, several foot bridges, and multiple pipe crossings. These constrictions are considered in Section 3.3.2. The northern reaches of the stream, along Eastern Hutt Road, are diverted through a 1800mm diameter pipe which collects flow from the Eastern Hills and the suburbs of Wingate and Taitā. The pipe then discharges into a regular concrete channel at Rata Street that conveys flows to Balgownie Grove, where the stream then reverts to its natural channel before draining into the mouth of the Hutt River.

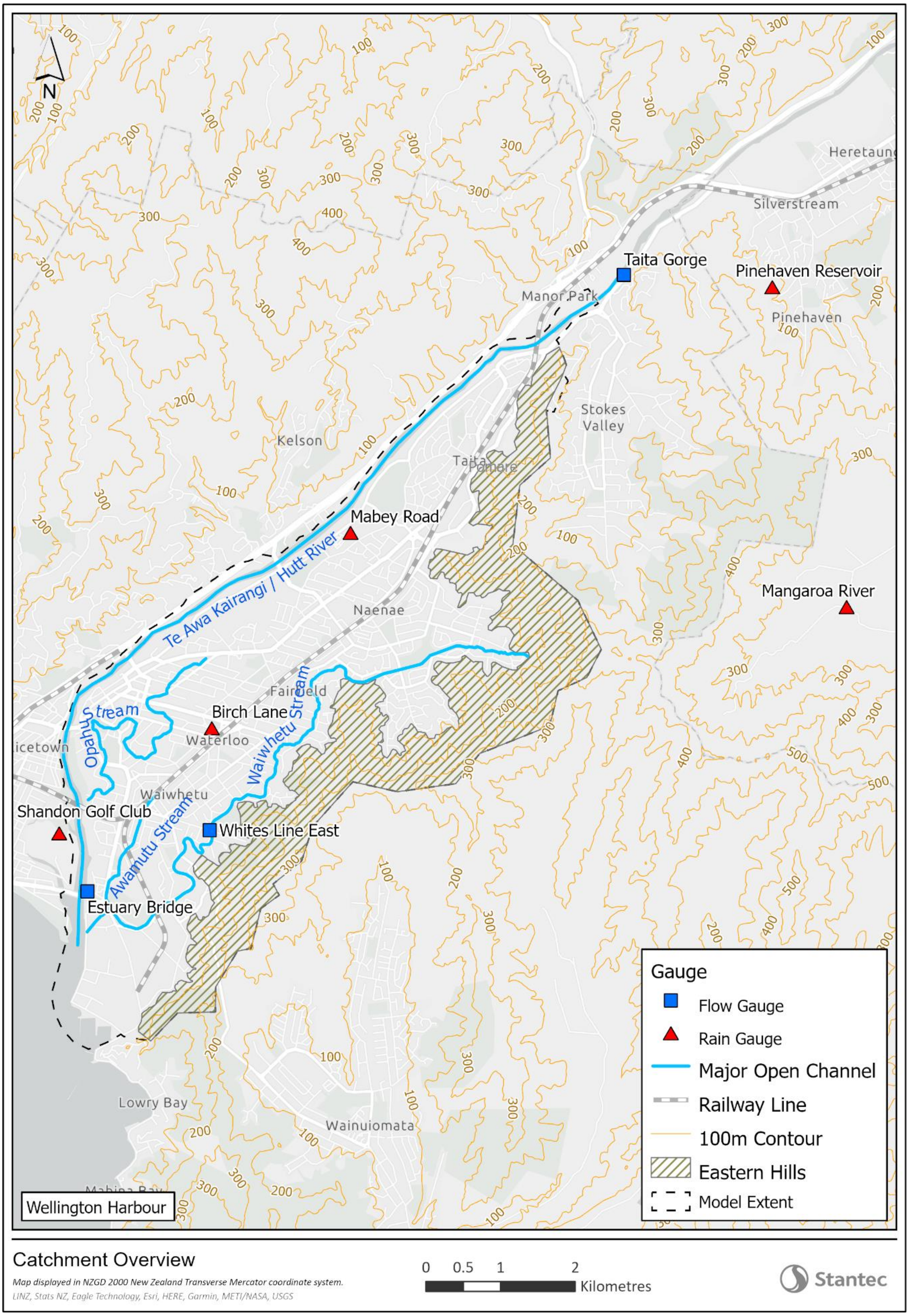


Figure 1-1 Eastern Lower Hutt catchment overview

1.2.4 Urban Network

The stormwater network in the Eastern Lower Hutt catchment contains approximately 240km of stormwater pipes, many of which are close to or below sea level. Five pump stations have been constructed to help push water up and into the natural stream channels which then gravitate from the catchment. Summary tables outlining the stormwater assets are available in Section 2.1. In addition to the Waiwhetū Stream, the Opahu and Awamutu streams drain the Lower Hutt area south of Boulcott and west of Waterloo Station, respectively.

1.3 ACTIVITIES AND SCOPE

The objective of the project is to develop a coupled 1-D/2-D stormwater model that incorporates the full public stormwater network including well-defined overland flow paths such as the Waiwhetū, Opahu, and Awamutu Streams. The model will be used by WWL, HCC, and GRWRC for use in future development, and network optioneering. The following objectives are discussed in detail in the report:

1. Short review of GIS information including, As Builts and hydrology layers.
2. Develop a fully integrated 1D/2D hydrological and hydraulic model following the Regional Stormwater Hydraulic Modelling Specifications v5, (Wellington Water Ltd, 2017). All public stormwater assets including the 1D pipe network, 1D channel network, and the 2D ground surface are to be incorporated into the model. Design events including the 10-year ARI with existing climate, and the 100-year ARI with existing and future climate would be simulated with the base model.
3. Validate the model using reported flooding incidences and photographs for two storm events. It was agreed with WWL that the November 2016 and February 2004 events would be simulated and analysed.
4. Calibrate the model specifically for the Waiwhetū stream using flow at the Whites Line East gauge. The agreed calibration events are the 13th-18th January 2004, 2nd - 4th March 2012, 15th May 2015, 15th November 2016, and 8th December 2019 storm events.
5. Undertake sensitivity analysis on the model to understand the impact of different model parameters on flooding in Stokes Valley. These sensitivity scenarios would be used to inform the selection of freeboard values to be applied to the network before the publishing of flood maps.
6. Undertake the freeboard assigning process following the Dynamic Freeboard Analysis (Jacobs, 2017) memorandum and simulate the freeboard run.
7. Develop a report outlining the model build process including all relevant model details, the validation process and findings and the sensitivity and freeboard process and findings.

2.0 AVAILABLE INFORMATION

2.1 PREVIOUS WORK

Stantec undertook a scoping study in 2019 to understand the catchment, determine the extent of available information, identify areas where more information would be required, and highlight regions of interest where significant flooding issues occurred, (Sherson, Kerr, Mulay, & Paine, 2019). As part of the scoping study, a rapid flood hazard assessment (RFHA) model was developed to assist in understanding flooding the catchment. This scoping study highlighted that the majority of the Eastern Lower Hutt public stormwater network (90%) has no known inverts, and that there are a range of bridge crossings, tunnels and channels that would need to be investigated and modelled.

GWRC completed a survey of the Waiwhetū Stream and the Awamutu Stream (the main tributary into the Waiwhetū Stream) in 2019, see Appendix A – Survey Data.

Additional known previous studies include:

- GWRC DHI models.

- Managing flood risk from the Hutt River, Wellington, New Zealand and impacts on insurance (Westlake & Manolache, 2016).
- Waiwhetū Stream Hydraulic Modelling 2005-2011 by GWRC in 2011.
- Vulnerability and adaptation to increased flood risk with climate change – Hutt Valley study (Lawrence, Tegg, Reisinger, & Quade, 2011).
- GWRC Waiwhetū Flood Management Plan (Greater Wellington Regional Council 2010)
- The 15-16 February 2004 storm in the Wellington region: Hydrology and meteorology (Watts & Gordon, 2004).
- Flood hydrology of the Waiwhetū Stream (Keenan 2004).
- Hutt River Floodplain management plan (Greater Wellington Regional Council, 2001).

2.2 DRAINAGE NETWORK DATA

2.2.1 Asset Data

Stormwater asset data was provided by WWL as an Innovyze InfoNet database containing all known public stormwater asset information.

Private asset data was supplied by WWL in the following Environmental Systems Research Institute (ESRI) Shapefiles:

- Links - storm_pipes_private.shp.
- Nodes - storm_fixtures_private.shp.

There are many critical attributes missing in both the public and private asset data as identified by Stantec while completing the RFHA assessment (Sherson, Kerr, Mulay, & Paine, 2019). These are discussed in detail in Section 3.3.2.

2.2.2 As-Built Data

Over 4000 as-built drawings were provided by WWL for the Lower Hutt region. Application of the as-built data to the model is described in Section 3.3.2.1.

2.2.3 Pumping Stations

There are five pumping stations throughout the catchment. These include pumping stations on Guthrie Street, Riverside Drive, Randwick Road, Richmond Grove and Parkside Road. Details for the pump stations and pumping capacity are provided in 0. Pumps are discussed further in Section 3.3.2.10, including where data has been assumed.

2.2.4 Topographical Data

Topographical data was sourced from Hutt City Council (HCC) by WWL for use in the development of hydraulic models for the WWL Stormwater Panel. HCC collected LiDAR data in 2016 which was used to generate a 1m resolution digital terrain model (DTM) in the Wellington 1953 vertical datum. The HCC LiDAR data did not cover the tops of the Eastern Hills, so to avoid the exclusion of this part of the Eastern Lower Hutt catchment from the model, Stantec combined the HCC data with the Greater Wellington Regional Council (GWRC) 2013 1m DTM. This involved converting the GWRC data to the Wellington 1953 vertical datum from the NZ2016 vertical datum. As the area not covered by the HCC data is outside of the 2-D zone, it will have limited impact on the model beyond extraction of gradients for calculation of Time of Concentration (Section 3.2.2.3).

Wellington Harbour bathymetry, created from survey points around the harbour, referenced in Chart NZ 4633, was also merged with the 2016 HCC DTM to allow the model to extend into the harbour as the model boundary. Both the GWRC 2013 DTM and the harbour bathymetry were added to fill in gaps in the HCC 2016 LiDAR as shown in Figure 2-1. The merged DTM was then used to represent topography within the 2-D zone of the hydraulic model. All model elements are converted to the Wellington 1953 vertical datum if required, all model levels are therefore in the Wellington 1953 vertical datum.

2.2.5 Cross Section Data

Surveyed cross sections covering the lower 7.2km of the Waiwhetū Stream (2/3rds) and the full extent of the Awamutu Stream were received from GWRC as Excel spreadsheets and shapefiles. Additional cross sections were built using the DTM or interpolated in InfoWorks ICM to maintain the cross section maximum separation distance recommended by the Regional Stormwater Hydraulic Modelling Specifications (Wellington Water Ltd 2017). GWRC also supplied cross sections for the Hutt River as used in a previous Mike 11 DHI model.

2.2.6 Reported Flood Issues

The Eastern Lower Hutt region has a history of flooding; the Waiwhetū Stream has overtopped its banks multiple times in recent history, including in the February 2004 Lower North Island floods, the 15th of November 2016 rainfall event, and the 14th of May 2015 rainfall event (see Figure 2-3 to Figure 2-6). In all three of these events, flooding was most prominent around the Waiwhetū Stream where much of the Lower Hutt Stormwater network drains. Data has been collated from a range of sources including WWL, GWRC, and news websites for several storms and their associated flood events over the past 20 years including:

- February 2004.
- March 2012.
- May 2015.
- November 2016.
- December 2019.

The 2004 event was particularly destructive, flooding more than 50 households and causing widespread problems across the Eastern Lower Hutt catchment. A GWRC Watts and Gordon (2004) report into the hydrology and meteorology of the February 2004 flood event provides valuable insight into rainfall distribution and river flows across the Wellington region. An indicative figure showing flooding issues reported to WWL during the 2004 event had been provided, see Figure 2-2. Numerous photos are provided in Section 6.0 of observed flooding during the February 2004 and November 2016 flood events as part of model validation.

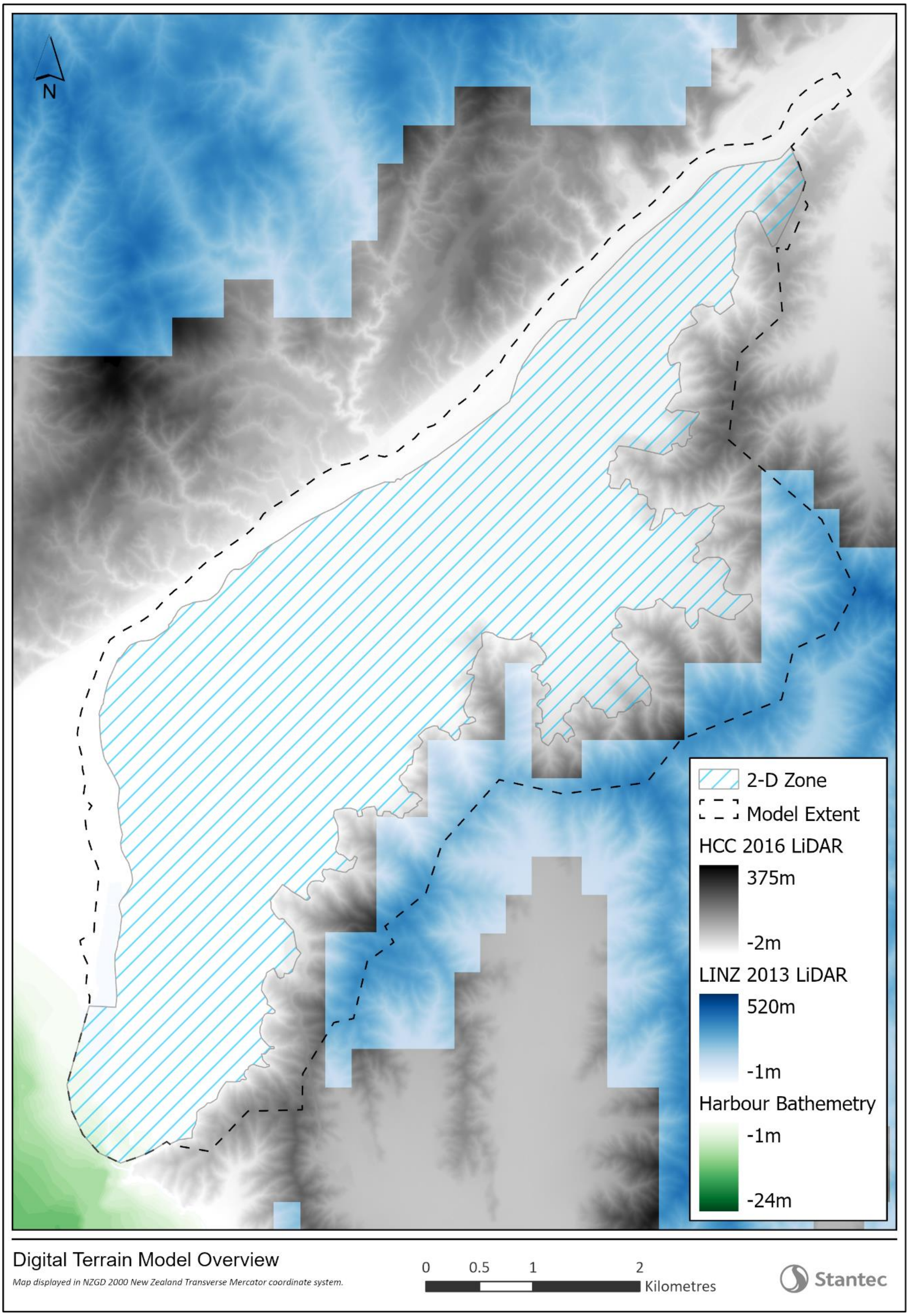


Figure 2-1 Overview of the digital terrain model produced using the three data sources.

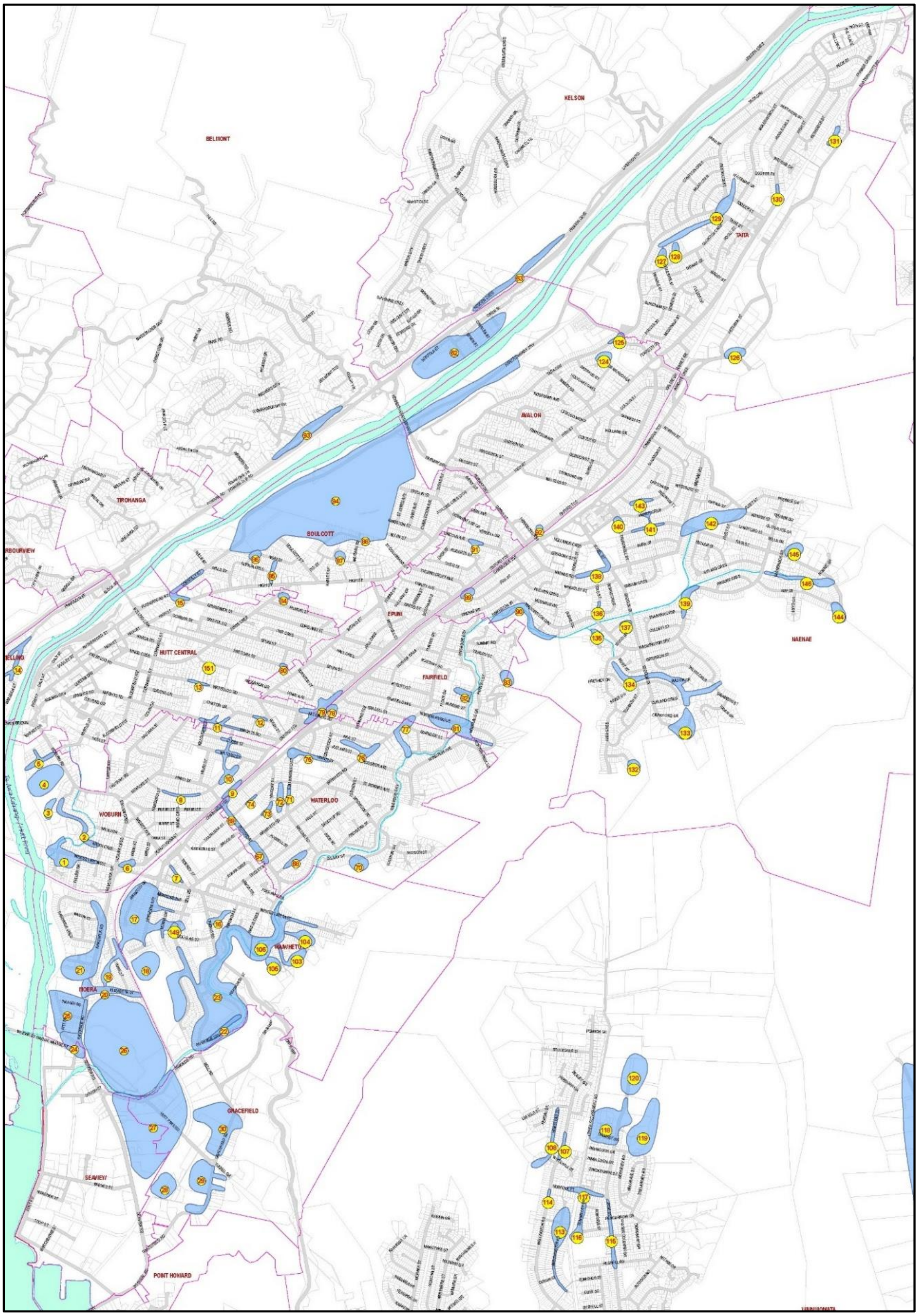


Figure 2-2 Flooding issues (blue) reported to WWL. February 2004 event. Sourced from WWL.

Figure 2-3 Flooding from the Waiwhetū Stream during 16th of February 2004 event (Westlake & Manolache, 2016)

Figure 2-4 Impact of the Waiwhetū Stream Bursting its Banks During 2016 Event. Photo from Cameron Burnell quoted in (Weekes, 2016).

Figure 2-5 Waiwhetū Stream flooding during the November 2016 event. Photo from Newshub 15/11/2016.

Figure 2-6 Waiwhetū Stream flooding during the November 2016 event. Photo from Melissa Nightingale – NZ Herald 15/11/2016.

2.2.7 Site Visit

Multiple photos and drawings of channels structures, pipes, overpasses, and bridges were taken across three days focusing on open channels and inlet structures in ELH that have not yet been surveyed. These observations will supplement the available survey, as-built drawing, or GIS data as required. They will also guide application of model parameters and coefficients where appropriate. Use of site visit observations will be covered in detail in Section 2.3.

2.3 HYDROLOGIC/HYDROMETRIC DATA

2.3.1 Rainfall Data

Total rainfall depths for different return period events are available from NIWA's HIRDS v4, a system that uses historical observed rainfall to produce predictions for future events across varying future climate scenarios (NIWA, 2018).

2.3.1.1 Rain Gauge Data

Recorded rainfall is available from GWRC for five gauges within or nearby the Eastern Lower Hutt catchment, these are detailed in Table 2-1 and their locations shown in Figure 1-1. Additional rain gauge data was identified at several gauges managed by Upper Hutt City Council (UHCC) including Gibbons Street, Heretaunga Dam, Perry Street, and Pinehaven. This data was not used due to limitations in its temporal availability and availability in the 5-minute time step required by the Regional Stormwater Hydraulic Modelling Specifications (Wellington Water Ltd, 2017).

Table 2-1 Summary of available data at GWRC rainfall gauges. Coordinates in New Zealand Geodetic Datum (NZGD) 2000.

Name	X	Y	Start Date	End Date	Time Step	Notes
Hutt River at Shandon Golf Club	1758967	5434461	03/04/2000	-	5 minutes	
Hutt River at Birch Lane	1761012	5435870	27/07/2001	-	5 minutes	
Hutt River at Mabey Road Depot	1762873	5438489	10/11/1995	-	5 minutes	Unavailable for the February 2004 storm event
Mangaroa River at Tasman Vaccine Limited	1769523	5437495	03/05/1968	-	5 minutes	
Pinehaven Stream at Pinehaven Reservoir	1768527	5441783	03/08/2010	-	5 minutes	

2.3.1.2 Rainfall Radar

Rainfall radar data is available for the Wellington Region, with 5-minute interval spatially averaged profiles calculated for key catchments. This data is available from 2008 onwards. Total rainfall depths, and depths at each timestep can be visualised using the Mott MacDonald Moata interface at a 500m-by-500m resolution, improving understanding of the storms used for validation. For details on how this rainfall was applied to the model, see Section 5.0.

2.3.2 Flow Data

2.3.2.1 Hutt River

Flow data for the Hutt River is available from GWRC at the Taitā Gorge gauging station (1766538, 5441948 NZGD 2000 approximately 1.2km north of the model extent. This data is available from 6th March 1979 to present.

2.3.2.2 Waiwhetū Stream

Stage (water surface level relative to a known datum) and flow along the Waiwhetū stream is available from GWRC at the Whites Line East gauge (1760984, 5434504 NZGD2000). This data is available from 31st May 1978 to present.

2.3.3 Tidal Data

The observed stage was provided by GWRC for the Hutt River at Estuary Bridge (1759345, 5433683 NZGD2000), recording tidal influence on levels in the Hutt River. This data is available from 28th September 1976 to 11th November 2019. Tidal data after 11th November 2019 is available from GWRC for the Wellington Harbour at Queens Wharf gauge (1749036, 5428137 NZGD 2000), where data is available from 31st August 1994 to present. The Estuary Bridge data was provided in the Wellington 1953 vertical datum, whereas the Queens Wharf data required conversion to the Wellington 1953 vertical datum following guidance provided by GWRC.

There is not a significant difference in tide timing between the two gauges, as shown in Figure 2-7, however there is a difference minimum and maximum level. The potential impact of this difference is discussed further in Section 5.3. Application of the tide as a boundary condition within the model is discussed in detail in Section 3.4.

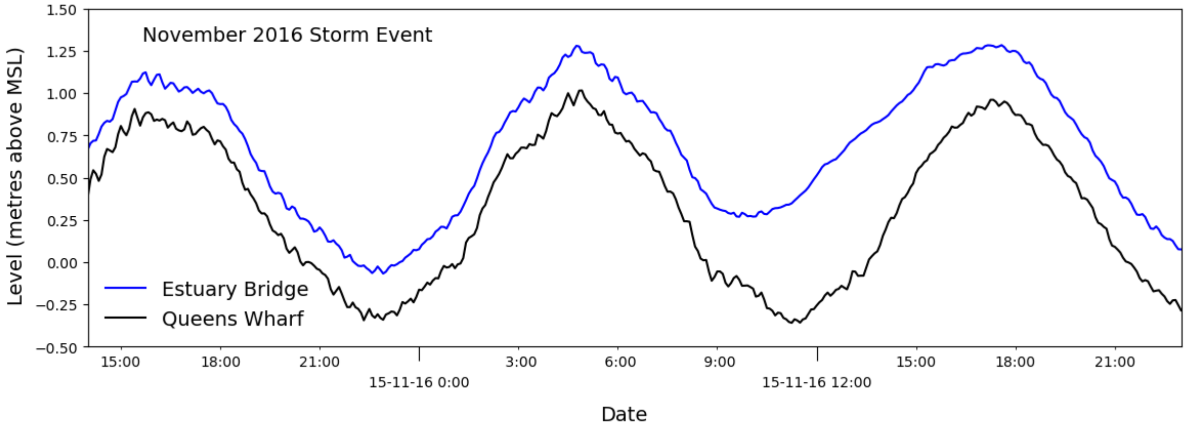


Figure 2-7 Tide as recorded at the Estuary Bridge and Queens Wharf gauges. November 2016 event.

2.3.4 Regional Hydrology Layers

Regional hydrology layers for the Wellington region consisting of SCS curve number, initial abstraction, Percentage Impervious (%IMP), and Manning's n roughness are available for use. The layers were developed for WWL using GIS software in 2016 by Montgomery Watson Harza (MWH) (now Stantec), based on the hydrology guidelines provided in the Quick Reference Guide for Design Storm Hydrology, (Cardno, 2016). The layers were calculated using land use, soil classifications, and topography. Figures providing an overview of the regional hydrology layers are available in Appendix D – Regional Hydrology Layers. Section 3.2 provides detail on application of these layers to the hydrological model.

3.0 MODEL BUILD

3.1 MODEL BUILD OVERVIEW

A fully integrated 1-D/2-D model was developed for the Eastern Lower Hutt catchment. The 1-D/2-D coupled model includes both a hydrological model for the conversion of rainfall into runoff, and a hydraulic model for conveyance of this runoff across the surface and through the reticulated stormwater network. The majority of the model build was undertaken using InfoWorks ICM version 9.5; however, some initial model development was carried out using Urban Model Manager (UMM). UMM is an ESRI ArcMap software add-in developed by Awa Environmental designed to facilitate building stormwater models by standardisation, automation, and interpolation following the WWL Regional Stormwater Modelling Specifications v5, (Wellington Water Ltd, 2017).

The model build described below has been broken into the three major model components: the hydrological model build, the hydraulic model build, and the modelled boundary conditions.

The following projection and vertical datum have been adopted:

- **Projection** – NZGD 2000 New Zealand Transverse Mercator.
- **Vertical Datum** – Wellington Vertical Datum 1953.

All data provided by WWL was already in this format, however several other data sources required conversion. Data flags were used to preserve and record data origin, see Appendix F – Data Flags.

3.2 HYDROLOGICAL MODEL

The hydrological model covers all aspects of rainfall, boundary conditions and inflows. The full extent of the hydrological model is shown by the subcatchment coverage in Figure 3-1 below. As per the Regional Stormwater Modelling Specifications v5 by Wellington Water Ltd, (2017) and the Quick Reference Guide for Design Storm, (Cardno, 2016). the Soil Conservation Service (SCS) method is used in the hydrological model for catchment runoff estimation. This method requires the input of catchment area, curve number (CN), initial abstraction (IA), and time of concentration (ToC). Each parameter and its application are described in the following sections.

3.2.1 Subcatchment Delineation

In total 7884 subcatchments were delineated for application of rainfall to the model following the Quick Reference Guide for Design Storm Hydrology recommendations. Out of the 7884 catchments, 2659 are building footprints included as individual subcatchments that drain directly to the nearest manhole. Larger catchments were broken up to allow more precise application of rainfall. Subcatchments were generated automatically for each sump and each building with a direct connection to the public stormwater network using a series of custom automations from the standard tools available with ESRI ArcGIS Spatial Analyst. See Appendix C – Interpolations and Automations.

The subcatchments range in size from 0.001 ha to 73.152 ha, see Table 3-1 for a summary and Figure 3-1 for an overview. The WWL Regional Stormwater Modelling Specifications v5 recommend that subcatchments not connected to buildings should be between 0.1 ha and 3.0 ha. Overall, there are 89 catchments above 3.0 ha, and 3716 below 0.1 ha. Therefore, it is recommended that a review of these small and large subcatchments be undertaken in future model updates.

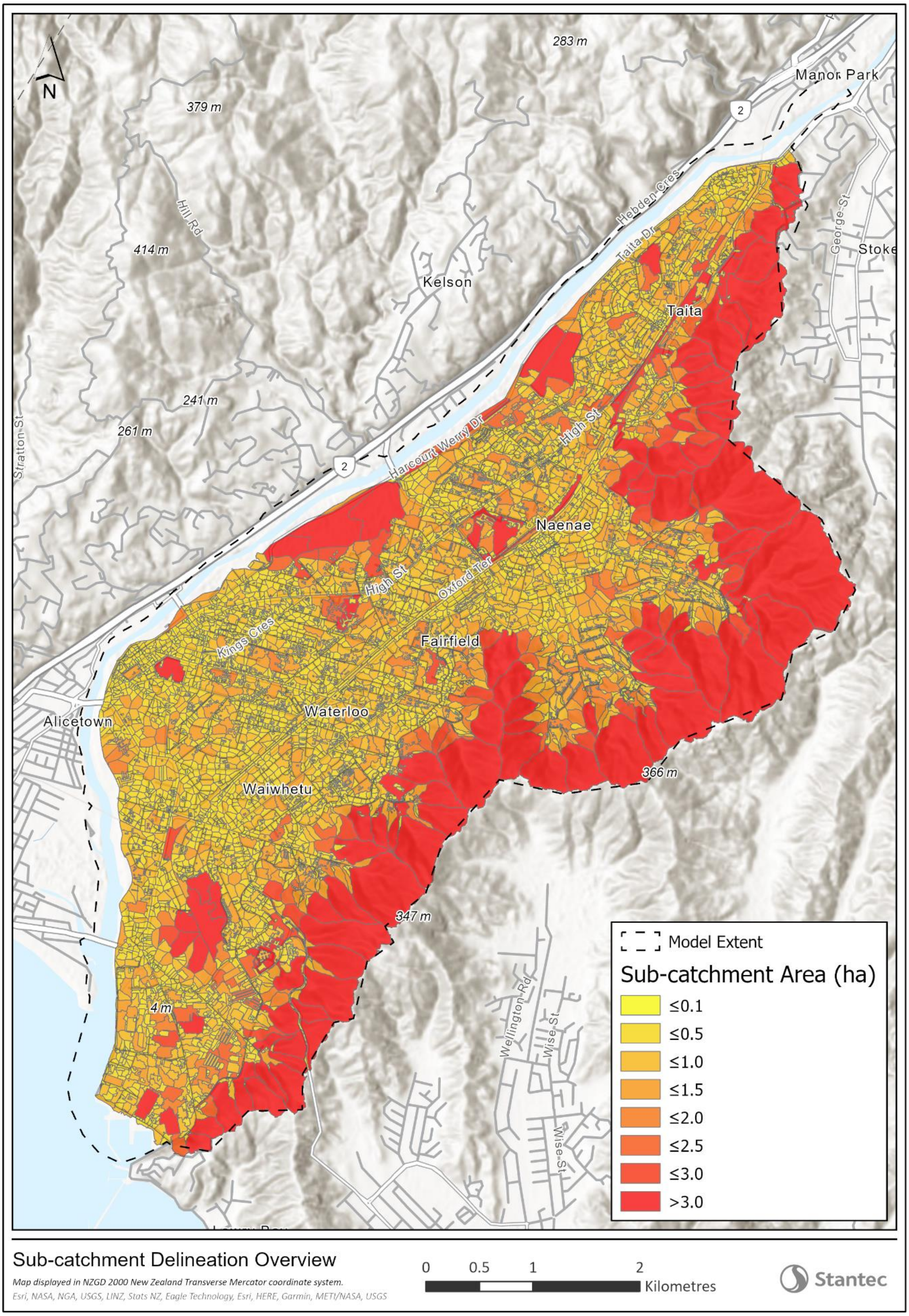


Figure 3-1 Overview of subcatchment delineations

Table 3-1 Summary of subcatchment area

Subcatchment Area (ha)	Count	Percentage (%)
Less than or equal to 0.1	3716	47.13%
0.1 to 0.5	2886	36.61%
0.51 to 1.0	796	10.10%
1.01 to 1.5	258	3.27%
1.51 to 2.0	78	0.99%
2.01 to 2.5	33	0.42%
2.51 to 3.0	28	0.36%
Greater than 3.0	89	1.13%

3.2.2 1-D Hydrology

As per the Regional Stormwater Modelling Specifications v5 by Wellington Water Ltd, (2017) and the Quick Reference Guide for Design Storm Hydrology by Cardno, (2016), the Soil Conservation Service (SCS) runoff curve number method is used as the hydrological model for catchment runoff estimation. This method requires catchment area, curve number (CN), initial abstraction (IA), and time of concentration (ToC).

3.2.2.1 Updates to Regional Hydrology Layers

Small inconsistencies are known to be present in the regional hydrology layers that result from missing data, new constructions, and base soil layer issues. It was agreed with WWL that the IA, CN, and roughness layers would be reviewed to identify and correct any significant anomalies.

Figure 3-2 and Figure 3-3 provide an example of updates made to initial abstraction values and curve numbers in the regional hydrology layer. In Figure 3-2, initial abstraction is reduced to account for the increase in impervious area due to construction of new residential housing. In Figure 3-3, curve number is reduced as a school field was incorrectly given a curve number of 98.

Figure 3-2 Initial abstraction is reduced to account for construction of new residential housing, and an associated increase in impervious area.

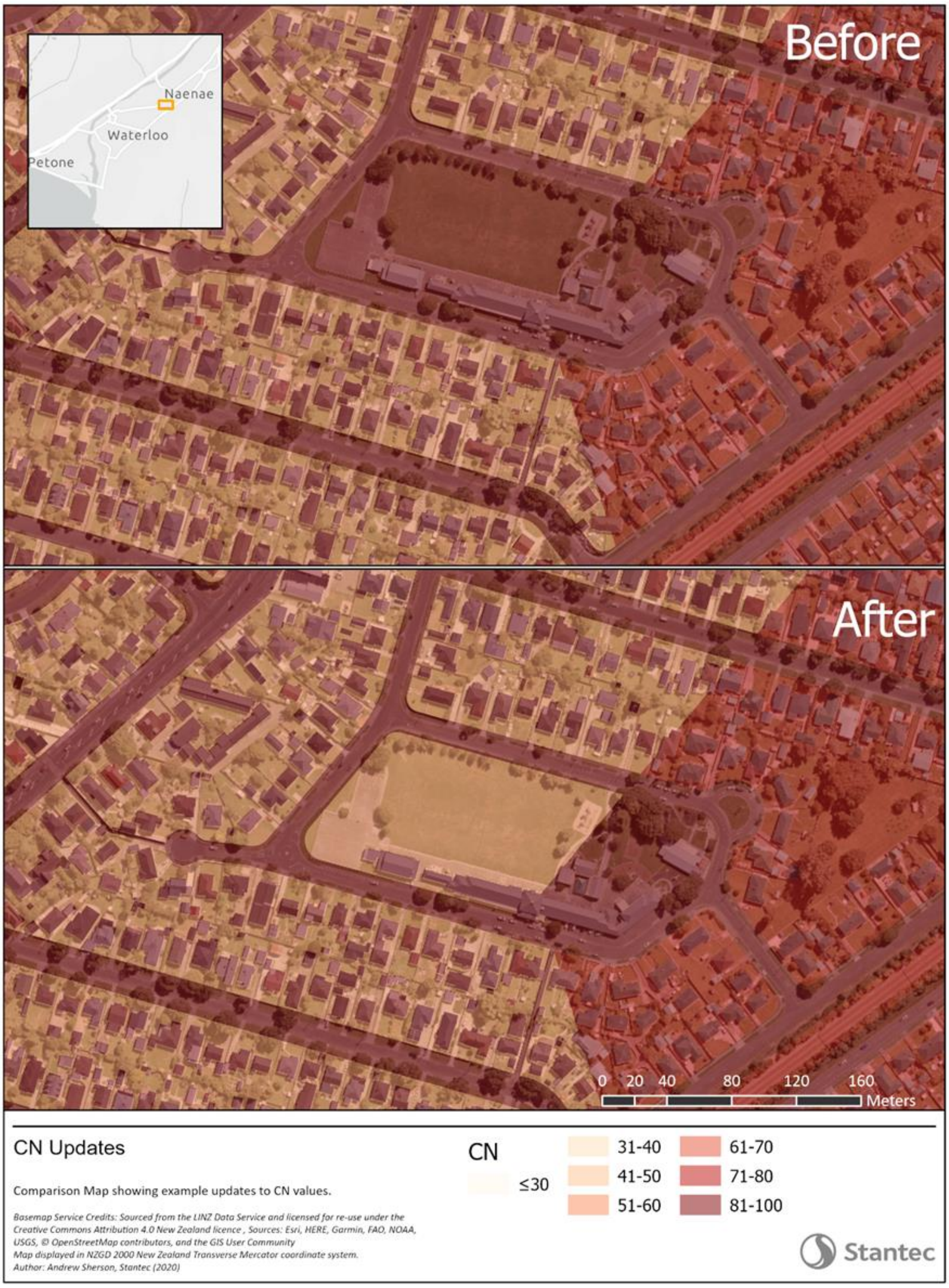


Figure 3-3 Curve number is reduced from an unrealistically high number on a school field.

3.2.2.2 SCS Curve Number and Initial Abstraction

Composite CN and IA values were assigned to each subcatchment by using the zonal statistics tool in ESRI ArcGIS on the Regional Hydrology Geographical Information System (GIS) layers (Section 2.3.4). CN is applied directly to the subcatchments, and IA was applied using a unique runoff surface for each subcatchment. Overviews of the composite CN and IA values are shown in Figure 3-4 and Figure 3-5.

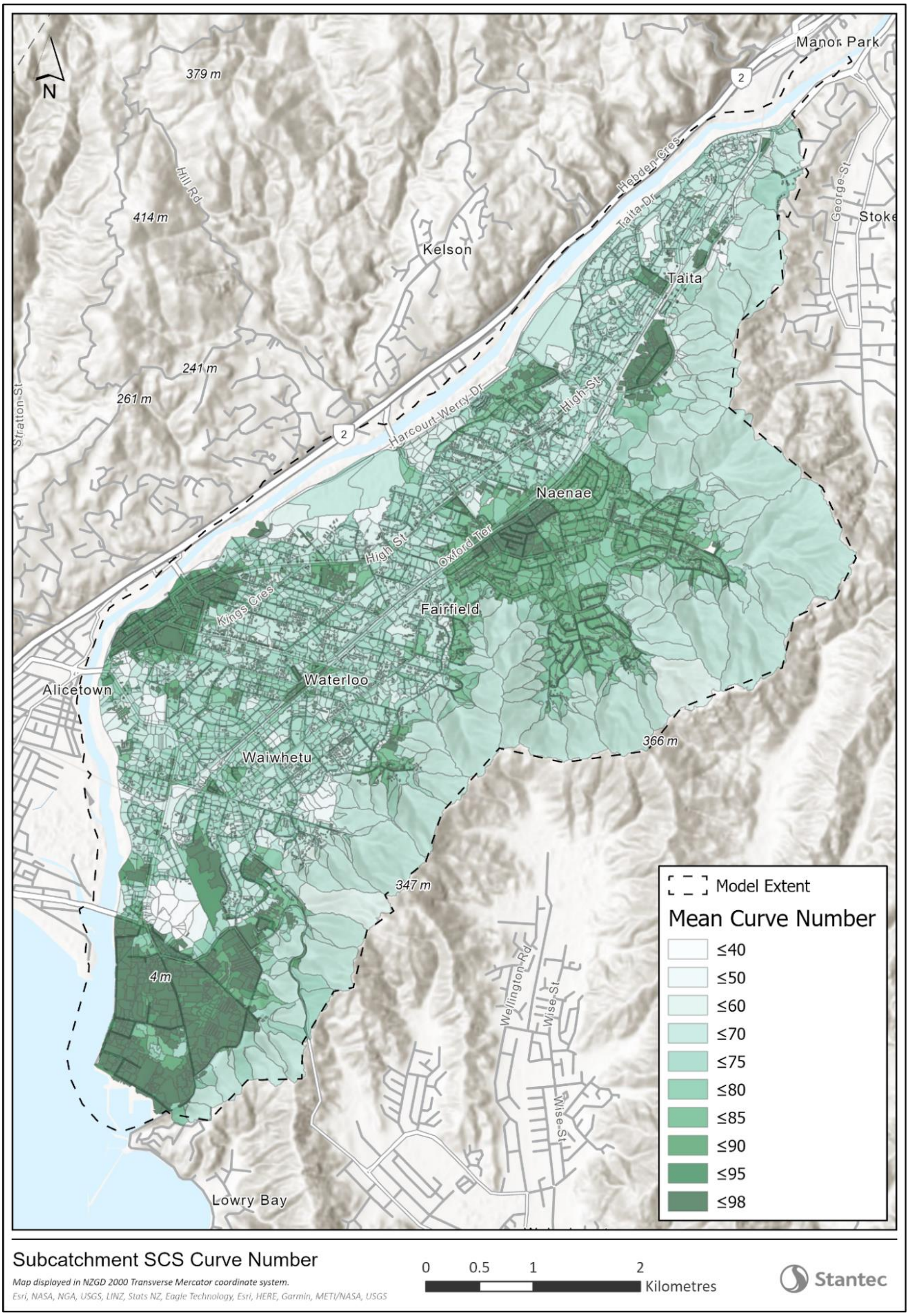


Figure 3-4 Composite SCS curve numbers applied to subcatchments.

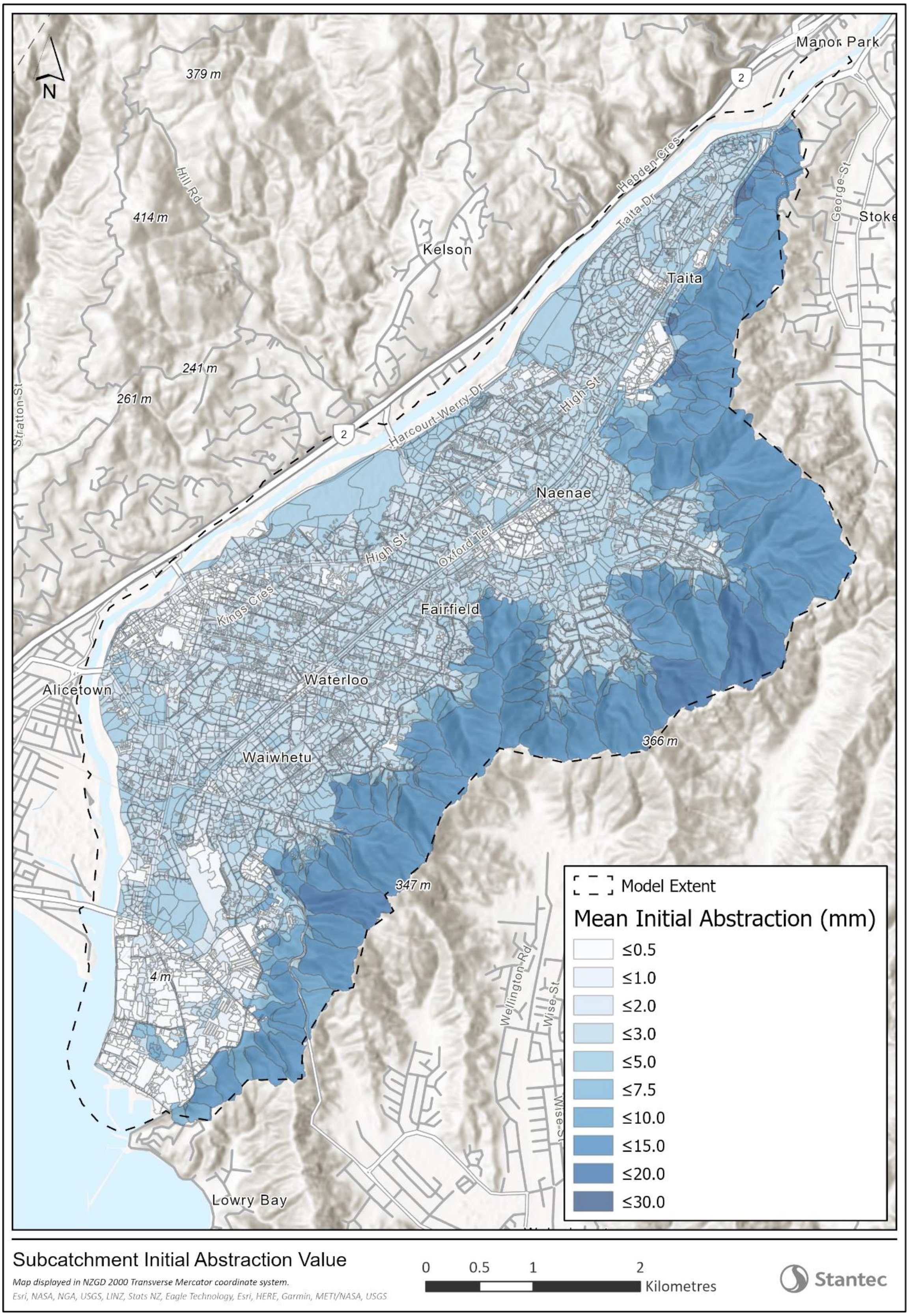


Figure 3-5 Composite initial abstraction values applied to subcatchments.

3.2.2.3 Time of Concentration

Time of concentration (ToC) was applied using methods outlined in the Quick Reference Guide for Design Storm Hydrology, (Cardno, 2016). The eight undeveloped subcatchments with longest flow paths of greater than 1000m were calculated using the mean of the Ramser Kirpich and Bransby Williams equations. Most subcatchments are developed/mixed use or have a channel length less than 1000m, therefore their ToC was calculated using the following components:

- Overland flow.
- Shallow concentrated flow.
- Gutter flow.

A summary of a practical approach to applying these different components in the context of a model is provided in the WWL Modelling Specifications V5, (Wellington Water Ltd, 2017). As the subcatchments are directly connected to either the reticulated network or a 1-D river reach, the pipe flow component is not required. An automated GIS workflow was developed to extract longest flow paths for each subcatchment, split the flow path into the component parts, and calculate a total ToC. See further details in Appendix C – Interpolations and Automations.

3.3 HYDRAULIC MODEL

3.3.1 Hydraulic Model Overview

The hydraulic model is used to simulate hydraulic processes within the stormwater network. It covers conveyance of generated runoff from the hydrological model across the surface and through the reticulated stormwater network.

The hydraulic model is made up of the following:

- 1-D components including the reticulated pipe network and open channels (Sections 3.3.2 and 3.3.3).
- 2-D components as defined by the 2-D zone covering the flexible 2-D mesh area that is generated using the ground model for surface flow hydraulic calculations (Section 3.3.4).
- 1-D/2-D connections including sumps, manholes, bank lines, and inline banks (Section 3.3.5).

The extent of the hydraulic model is shown in Figure 3-6, with an overview of the modelled network including the 1-D pipes and river reaches, and the extent of the 2-D zone. The hydraulic model covers the full Eastern Lower Hutt catchment as outlined in Section 1.2.

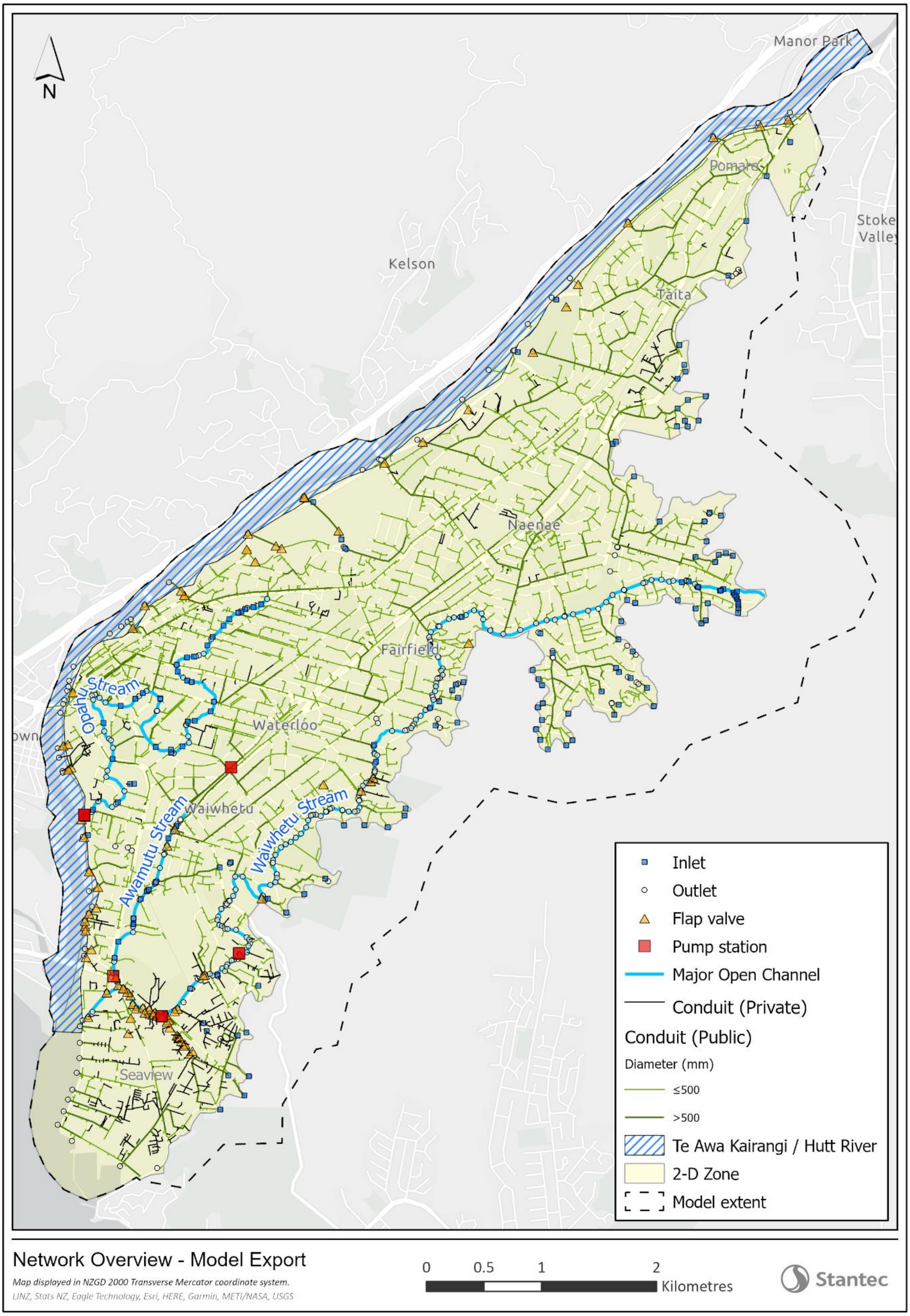


Figure 3-6 Eastern Lower Hutt stormwater network overview.

3.3.2 1-D Reticulated Network

3.3.2.1 InfoNet As-Built Update

Following WWL's guidance, Stantec used as-built information to check and update the received InfoNet Database. Due to the large number of as-built drawings (~4000), and because approximately 90% of the HCC as-built information was already incorporated into the InfoNet database correctly, not all as-built drawings were checked. As agreed with WWL, the modellers focused on checking all inlets, outlets, and pipes with a diameter of 500mm or greater. Approximately 200 as built drawings were checked, and information added to the model where necessary.

3.3.2.2 Stormwater Nodes

Nodes are used in InfoWorks ICM to represent manholes, sumps, storage, breaks connecting river reaches. Dummy nodes, or manhole with a "sealed" flood type in InfoWorks ICM, are used to represent connections and junctions that do not require representation on the surface. Dummy nodes may be used to connect inline banks at inlets or outlets, or to connect sump leads to the network where no manhole is present. Storage nodes are used to represent storage at the ends of open channels and to represent ponds modelled as part of the 1-D network (see Section 3.3.2.11). Table 3-2 provides a summary of the modelled nodes.

Table 3-2 InfoWorks ICM node summary, count of public and private assets

Type	No. of Public	No. of Private
Manholes	5,289	555
Sealed Manholes	995	289
Sumps	4,471	322
Storage Nodes	328	2
Break Nodes	257	-
Outfall Nodes	371	-

3.3.2.3 Manholes

All manholes are modelled using the "Gully 2-D" flood type in InfoWorks ICM to provide a connection to the 2-D surface. This allows application of a "Manhole" head discharge curve (see Appendix G – Head-Discharge Curves), better controlling the amount inflow into the reticulated network as every subcatchment is directly connected to a 1-D network element, with the majority connected to manholes. Head discharge curves are applied as part of importing the UMM export into InfoWorks ICM.

Some network connectivity issues were rectified by creating additional nodes. Where links did not terminate at a node, or where a new junction node was required, a node was added and labelled with the links' asset ID suffixed with "_a" or "_b" (see Appendix B – Added Assets).

Manholes within the model which do not require a connection to the 2-D surface are modelled with a "Sealed" flood type. These are typically used where dummy manholes are required, or at pipe junctions where no manhole is present.

3.3.2.4 Sumps

All sumps are modelled using the "Gully 2-D" flood type, with a head discharge curve corresponding to the sump type (0). The head discharge curves are applied as part of the UMM export into InfoWorks ICM. In addition to sumps identified in the asset data provided by WWL, sumps were added when identified in as-built drawings, observed on site visit, or observed using Google Street view (see Appendix B – Added Assets).

Sump invert levels were based on the minimum depth in the modelling specifications provided by WWL (1.2m), and ground levels for the sumps were extracted from the DTM. Sumps are represented in the model as circular nodes. This does not affect the hydraulic calculation as the model only considers the depth volume relationship of the nodes not their geometry.

3.3.2.5 Stormwater Conduits

The InfoWorks ICM network contains 11,568 conduits, of which 10,433 are part of the public stormwater network and 1,135 are private assets. Several key attributes were missing from many of the conduits supplied in the InfoNet database (public stormwater network), and the ESRI Shapefiles (private assets). InfoWorks ICM has numerous tools for identification of connectivity issues, interpolation of missing data, and a scripting interface for automating workflows. Further as-built drawings were consulted as required and where available, modelling/engineering judgement was used where appropriate. Flags can be used to identify the data origin, see Appendix F – Data Flags. Table 3-3 to Table 3-5 summarise the amount of missing data that required inference, the range of conduit diameters, and the range of conduit materials.

Table 3-3 Missing invert data summary, count of public and private assets

Type	Count	Missing Upstream Invert	Missing Downstream Invert
Public Main	10,433	9,450 (91%)	9,392 (90%)
Private Main	1,135	1,023 (90%)	1,025 (90%)

Invert levels were inferred in UMM, however the network exported from UMM contained numerous negative gradients. Inference of approximately 90% of conduit inverts is challenging, especially considering that the Eastern Lower Hutt area is predominately low gradient.

Several interpolation processes were carried out to improve pipe parameterisation and reduce the number of negative gradients. These involved SQL queries and ruby scripts run within the InfoWorks ICM user interface. A full outline of the InfoWorks ICM interpolation process can be found in Appendix C – Interpolations and Automations. Changes were only made to pipes that had unknown inverts or inverts calculated from the ground model. No changes were made to known as built or GIS sourced information unless the information was likely incorrect or unreliable. Flags were used to identify if invert levels were known, interpolated, or set by the user.

Table 3-4 InfoWorks ICM conduit diameter summary, count of public and private assets

Diameter (mm)	Public	Private
< 225	926	28
225 to < 375	6,121	914
375 to < 600	1,308	128
600 to <1050	890	24
>= 1050	255	15
Unknown	933	26

All missing conduit diameters were assigned a default of 300mm on export from UMM. These assumed diameters were checked for suitability using SQL queries to identify downstream decreases in diameter. As built drawings were consulted where anomalous transitions in conduit diameter were detected, and in their absence, diameter was inferred using the surrounding network or modelling/engineering judgment.

Table 3-5 InfoWorks ICM conduit material summary, count of public and private assets, and conduit roughness

Material	Public	Private	1/Manning's N
Reinforced Concrete	8,420	584	75
Earthenware Ceramic	1,500	2	70
Plastic	193	47	80
Asbestos Concrete	183	2	77
Iron	1	-	70
Unknown	136	500	75

Roughness values as 1/Manning's n are taken from the Regional Stormwater Hydraulic Modelling Specifications by Wellington Water Ltd (2017), conduit roughness where material is unknown are assumed as "Concrete (Normal)" with a value of 75.

3.3.2.6 Public Stormwater Pipes

Invert levels were applied using as-built values and GIS data where possible. However, as outlined above, many pipes and manholes were without invert levels and required interpolation from nearby assets and the DTM. The interpolation process was based on a combination of 600mm minimum cover calculated using pipe diameter and DTM elevation, and any nearby survey, as-built drawing, or GIS data. This process prevented inference of negative gradients and ensured network connectivity whilst following the WWL Regional Stormwater Modelling Specifications v5, (Wellington Water Ltd, 2017).

Numerous additional pipes were added using engineering judgment during network connectivity troubleshooting (see Appendix B – Added Assets). Where it was necessary to add a link to connect a drainage sump to the network, the link was labelled with the asset ID of the sump suffixed with "_a". Links split where a junction was missing were named with the asset ID of the original link suffixed with "_n", where n is an integer.

3.3.2.7 Private Pipes

Although most of the privately owned network are small individual residential property connections, there are a significant number of larger private pipes with diameters up to 1200mm located in commercial and industrial areas. Following discussion with WWL it was agreed that these private pipes would need to be included in the model.

This was done by:

1) Identifying areas where private pipe connections were critical to the connectivity of subcatchments and/or conveyed significant flows.
2) Clipping down the private network to assets with diameters larger than 150mm.
3) Adding nodes to pipe junctions, where missing in the database. Any added dummy nodes were given the flood type 'sealed' to prevent the node surcharging to the surface where there was no chamber to do so.
4) Importing all clipped down nodes and pipes into InfoWorks ICM including any additional dummy nodes.

3.3.2.8 Conduit Headlosses

As per the Regional Stormwater Hydraulic Modelling Specifications (Wellington Water Ltd 2017), all conduit headlosses are calculated using the inference tool in InfoWorks ICM. Following InfoWorks ICM guidelines, headlosses are removed for conduits with a gradient of >10%.

3.3.2.9 Culvert Inlet Losses

Energy losses at culvert inlets were applied using culvert inlet elements within InfoWorks ICM and the inlet parameters have been selected following the InfoWorks ICM recommendations (Ramsbotom, Day, & Rickard, 1997). Table 3-6 provides a summary of inlet parameters used within the model.

Table 3-6 Summary of inlet parameters applied to culvert inlets within InfoWorks ICM

Description	Equation	K	M	c	Y	Inlet Headloss Coefficient (Ki)
Circular conduit/concrete/headwall/square edge	A	0.0098	2.000	0.0398	0.67	0.50
Circular conduit/concrete/headwall/socket end of pipe	A	0.0078	2.000	0.0292	0.74	0.30
Rectangular conduit/concrete/headwall/wingwall (30° – 75°)/square edge	A	0.0260	1.000	0.0385	0.81	0.30
Rectangular conduit/concrete/headwall/wingwall (15°)/square edge	A	0.0610	0.750	0.0400	0.80	0.50
Rectangular conduit/concrete/headwall/wingwall (0°)/square edge	A	0.0610	0.750	0.0423	0.82	0.70
Rectangular conduit/concrete/headwall/20mm chamfers	B	0.5150	0.667	0.0375	0.79	0.50

A screen element cannot follow a culvert inlet element as the culvert inlet requires a river reach immediately upstream (Buck & Allitt, 2019). In this circumstance only a screen element is used as this is assumed to be the dominant control on headloss. There are 18 screen elements incorporated in the model, identified following site visits or in as-built drawings.

3.3.2.10 Pump Stations

There are five pumping stations throughout the catchment. These include pumping stations on Guthrie Street, Riverside Drive, Randwick Road, Richmond Grove and Parkside Road. Appendix E – Pump station data provides a description of the pump stations and pumping capacity and Table 3-7 provides an overview of the key level assumptions made whilst schematising the pump stations.

Table 3-7 Pump station capacity and on/off levels
Most pump on/off levels required some assumptions due to the lack of available data on their operation.

Pump Station Name	No. of Pumps	Pump Capacity (l/s)	ON (Level)	OFF (Level)
Guthrie Street Subway	1	4.2 (As built)	2.10m (Assumed – surrounding network)	2.00m (Assumed – surrounding network)
	2	5.6 (As built)	2.10m (Assumed – surrounding network)	2.00m (Assumed – surrounding network)
Riverside Drive	1	42 (As built)	0.46m (Assumed – surrounding network)	0.36m (Assumed – surrounding network)
	2	56 (As built)	0.46m (Assumed – surrounding network)	0.36m (Assumed – surrounding network)
Randwick Road (Seaview Roundabout)	1	650 (As built)	-0.53m (Assumed – surrounding network)	-1.20m (Assumed – surrounding network)
Opahu Stream	1	2,600 (As built)	2.40m (Assumed – WWL data)	2.05m (Assumed – WWL data)
	2	2,600 (As built)	2.55m (Assumed – WWL data)	2.20m (Assumed – WWL data)
	3	2,600 (As built)	2.60m (Assumed – WWL data)	2.00m (Assumed – WWL data)
Park Side Road	1	1,100 (As built)	-0.56m (As built)	-1.73m (As built)
	2	1,100 (As built)	-0.13m (As built)	-1.33m (As built)
Naenae Station (DUMMY)	1	4.2 (Assumed)	10.37m (Assumed – surrounding network)	10.30m (Assumed – surrounding network)
Naenae Station (DUMMY)	2	5.6 (Assumed)	10.37m (Assumed – surrounding network)	10.30m (Assumed – surrounding network)
Epuni Station (DUMMY)	1	4.2 (Assumed)	6.72m (Assumed – surrounding network)	6.65m (Assumed – surrounding network)
Epuni Station (DUMMY)	2	5.6 (Assumed)	6.72m (Assumed – surrounding network)	6.65m (Assumed – surrounding network)

All pump stations excluding Naenae and Epuni Stations were schematised using as built drawings; therefore, levels could be assumed with a reasonable level of confidence. In order to provide some drainage capability, dummy pumping stations were added to the station railway underpasses at Naenae and Epuni Stations using the Guthrie Street pump station as a template following discussion with WWL. Guthrie Street was chosen as a template as it is low capacity and drains a railway underpass.

3.3.2.11 Ponds

Ponds are modelled as 1-D storage areas where they are connected to the public stormwater network. All other ponds are assumed to be adequately represented in the ground model and resulting 2-D mesh.

Figure 3-7 shows an example of a typical 1-D pond layout in InfoWorks ICM. The storage node, associated storage area, and ground model are used to create a storage array representing the pond volume. Bank lines and inline banks provide a connection to the 2-D surface (discussed further in Section 3.3.5).

Figure 3-7 Overview of 1-D pond construction.

3.3.2.12 Flap Valves

Flap valves were added to each outfall along the Hutt River and Waiwhetū Stream following GIS information and WWL recommendations. There are 108 flap valves incorporated in the model. Flap valves were added by inserting a new sealed manhole in between the conduit and the outfall and then inserting the flap valve link from the sealed manhole to the outfall. Where invert levels were not available in as-built drawings or in GIS inverts were assumed from connected conduits. As there was limited information regarding the type of flap valve, the default InfoWorks ICM value of 1.00 was selected for the discharge coefficient resulting in no assumed headloss.

3.3.2.13 Railway Underpasses

Five underpasses were identified underneath the Wellington-Wairarapa railway line. These included the Guthrie Street underpass and Epuni, Naenae, Taitā and Pomare stations. A pump station was identified at Guthrie Street; however, no information was available for pumps stations at Epuni, Naenae or Taitā stations; therefore, dummy pump stations were set up at these locations (see Section 3.3.4.8). The Pomare Station underpass did not require drainage due to the absence of overland flow at this location.

3.3.3 1-D River Reach Network

There are numerous river reaches within the ELH catchment. The majority of these have been modelled using 1-D river reaches in InfoWorks ICM as small channels are not well represented in the available LiDAR, often due to the extensive vegetation cover. River reaches were digitised following low points in the DTM starting between transition points such as bridges and culverts. The cross sections were either taken from survey data, built using standard cross section shapes, built using levels taken from the DTM, or were interpolated using the inbuilt InfoWorks ICM tool. Figure 3-8 provides an overview of all 1-D river reaches included in the model.

River reaches have been included upstream of all inlet locations for the reticulated network. These river reaches provide velocity to flow entering conduits and as result the model will provide a better representation of energy losses at the inlets.

In total 412 river reaches were built with 1217 cross sections to model the river reaches as 1-D elements in InfoWorks ICM. Survey cross sections were used where available, with some minor modifications to remove points not required, for example water level (see Appendix H – 1-D River Reach Cross Sections). Additional cross sections were built using the ground model or interpolated in InfoWorks ICM to satisfy recommendations in the Regional Stormwater Hydraulic Modelling Specifications by Wellington Water Ltd (2017) for cross section interval. Break nodes are used to allow for the connection of outlets from the reticulated network, lateral inflows from subcatchments not included in the pipe network, or to mark the start or end of a structure. Storage nodes are used at most river reach junctions, inlets, and outlets.

3.3.3.1 1-D River Reach Energy Losses

As shown in Figure 3-8, a uniform bed roughness was not applied across all rivers reach sections used to model the river reaches. Three Manning's N roughness values were used to capture contrasts in channel characteristics across the catchment, as shown in Table 3-8. These values were selected based on discussion with WWL and GWRC, site observations, and consideration of values provided in the Regional Stormwater Hydraulic Modelling Specifications and Roughness Characteristics of New Zealand Rivers (Hicks & Mason, 1998).

Table 3-8 Manning's N roughness characteristics

Manning's N	Application in InfoWorks ICM	Classification
0.030	Altered channels	River reaches with engineered banks, or a partially concrete bed. May include obstructions.
0.055	Natural reaches of the Waiwhetū and Opahu	Predominantly natural river reach, cross section shape changes frequently, frequent obstructions.
0.080	Steep inlet river reaches along the Eastern Hills	Steep natural river reaches, bed is frequently vegetated.

3.3.3.2 Open Channel Crossings

There are numerous open channel crossings present across the Eastern Lower Hutt catchment including road bridges, rail bridges, and pipe crossings. Modelling of open channel crossings is restricted to larger crossings that are likely to impact flood extents. Figure 3-9 provides an overview of the distribution of open channel crossings as included in the InfoWorks ICM model.

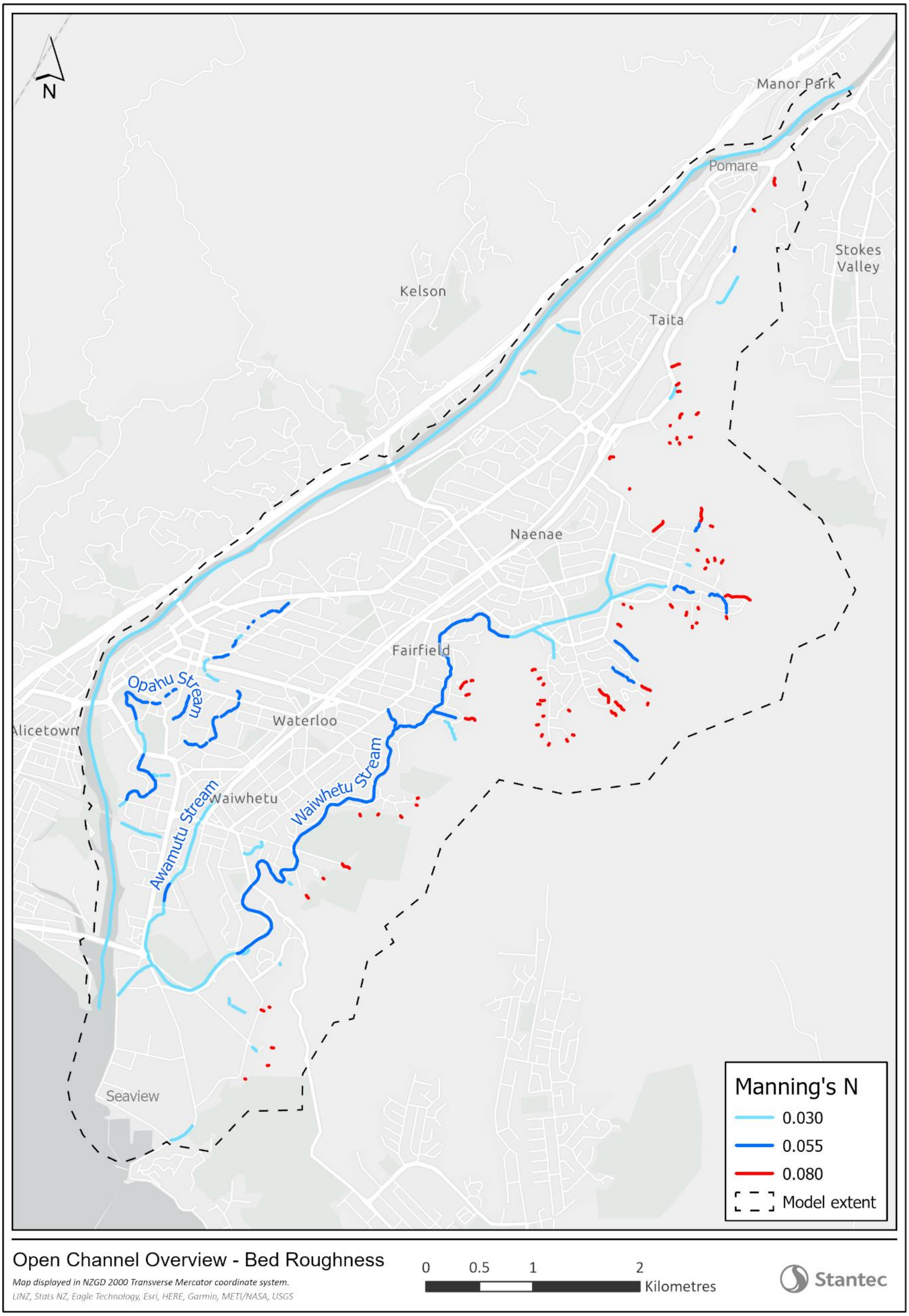


Figure 3-8 All 1-D river reaches, and the Manning's N applied as their bed roughness.

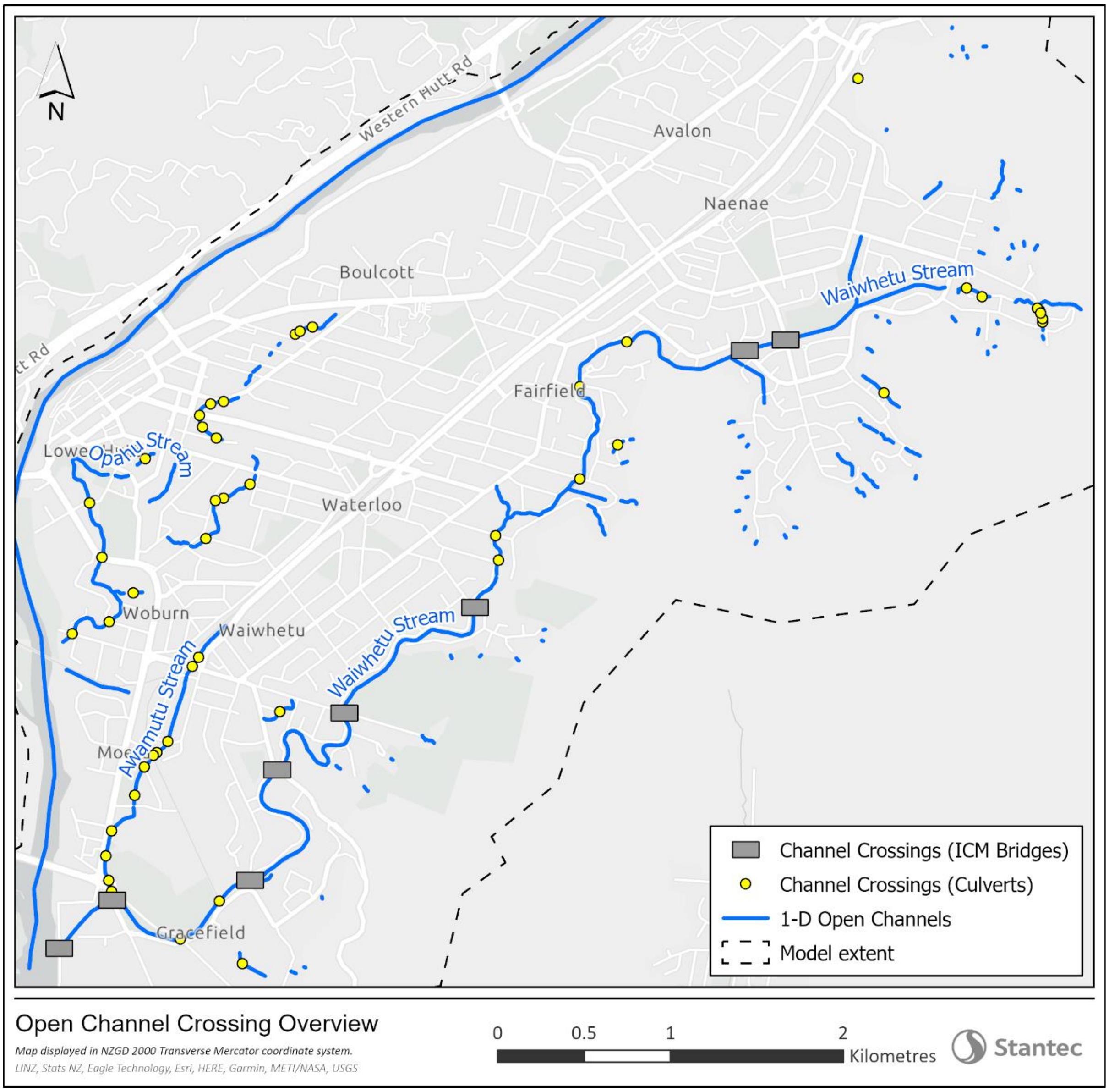
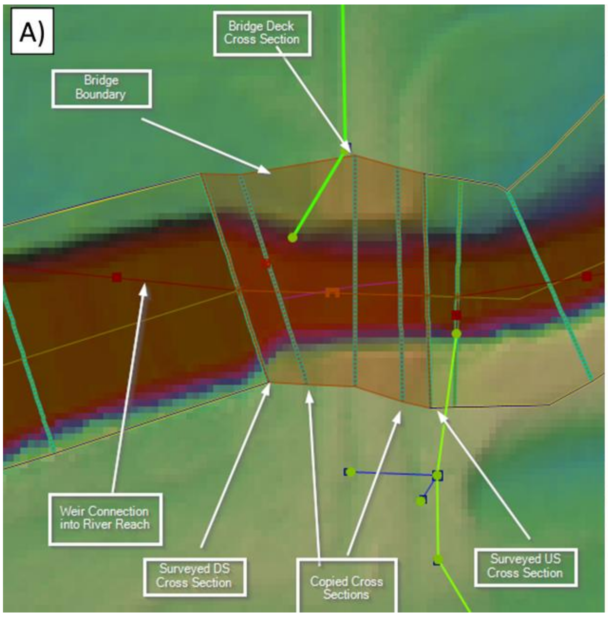


Figure 3-9 Open channel crossing overview including culverts and bridges.

3.3.3.3 Bridges

An InfoWorks ICM bridge structure was used for each of the eight larger crossings along the Waiwhetū Stream. Bridges were used in these instances as the size of the structure allowed for inclusion of the required contraction and expansion cross sections in addition to the upstream and downstream cross sections of connected river reaches. Bridge deck levels were taken as equal to road level at either side of the bridge, as most bridges have been removed from the HCC 2016 DTM used to produce a ground model. See Figure 3-10 for an example of bridge schematisation.

No channel crossings were modelled using InfoWorks ICM bridge structures on the Opahu and Awamutu Streams as they are small streams and bridge structures are notoriously unstable model elements in InfoWorks ICM.



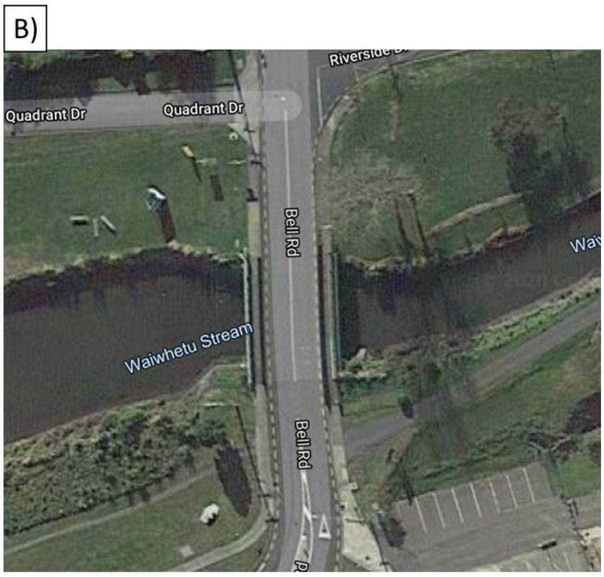


Figure 3-10 A) Bell Road bridge connection in InfoWorks ICM, b) image of bridge from Google Maps.

A bridge opening was defined based on the downstream cross section of the river reach upstream of the bridge (see Figure 3-12 for an example). A mix of symmetrical and non-symmetrical shapes were used for the bridges depending on the complexity of the opening. For some bridges including the Cleary Street Bridge shown in Figure 3-11 and Figure 3-12 site photos and observations were used to aid in determining the schematisation of the bridge.

Figure 3-11 Cleary St bridge site photo

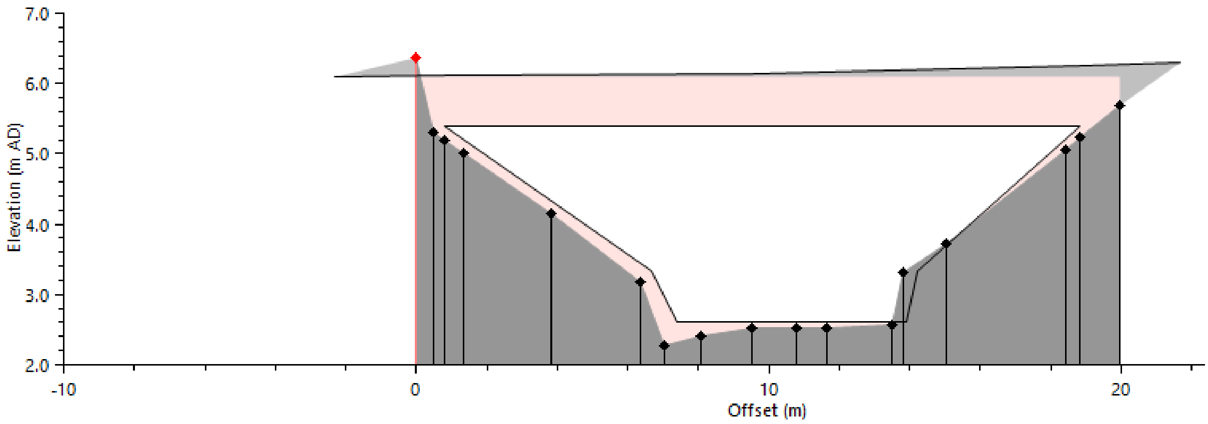


Figure 3-12 Bridge opening at Cleary Street. US surveyed cross section in grey, bridge deck in pink, and opening shown in white

All InfoWorks ICM bridge parameters including contraction and expansion losses were kept as the InfoWorks ICM defaults, see Appendix J – InfoWorks ICM Bridge Parameters.

3.3.3.4 1-D Culverts

Most channel crossings on the Waiwhetū Stream, and all channel crossings elsewhere in the catchment were modelled as culverts. Overtopping of the inlet or crossing has been modelled using inline bank elements in most cases. In a small number of cases weirs were used, mostly for footbridges that were represented as special shaped culvert but not wide enough to require 2-D overflow/inline banks. Figure 3-13 shows the model set up of a typical channel crossing modelled as a culvert. These weirs were given an ICM standard discharge coefficient of 0.85.

Figure 3-13 Wai-iti Crescent channel crossing example. A) Model schematisation, B) Google Street View photo

3.3.4 2-D Model Build

3.3.4.1 2-D Mesh Development

The modelled 2-D zone covers the urban area of Eastern Lower Hutt up to the maximum extent of the residential area and a small section of the Wellington Harbour. Mesh triangles applied throughout the 2-D zone had a minimum area of 2m^2 and a maximum area of 4m^2, except in coastal regions where elements ranged in size from 10-50m^2 and 25-100m^2. Element sizes of 5-20m^2 are applied to undeveloped areas across the ELH catchment such as parks and the Eastern Hills to reduce the total number of mesh elements as shown in Figure 3-15. At 21.4km^2, the 2-D zone is relatively large requiring approximately 5.8 million mesh elements.

3.3.4.2 Mesh Zones and Mesh Level Zones

As part of the UMM export to InfoWorks ICM, mesh level zones were created for each manhole with ground levels equal to the corresponding node lid level. The mesh zones around 2-D outfalls were required to provide a consistent level for discharge and were set slightly larger than the standard manhole mesh polygons. Each 2-D outfall mesh zone has a ground level based on the DTM.

Multiple mesh or mesh level zones were also added into the model to rectify the DTM where the ground surface was not represented appropriately. For example, concrete walls on the sides of channels (see Figure 3-14**Error! Reference source not found.**), or the bridge deck of smaller stream crossings where the bridge deck had been removed from the DTM. These mesh or mesh level zones had levels taken either from known surveyed levels or set based on engineering judgement.

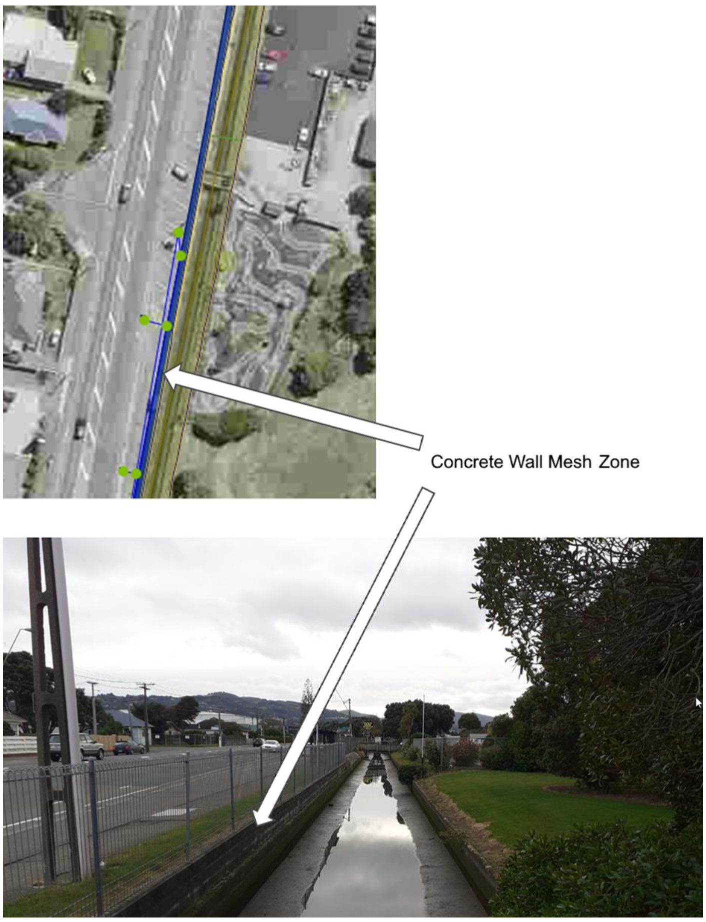


Figure 3-14 Concrete wall mesh zone

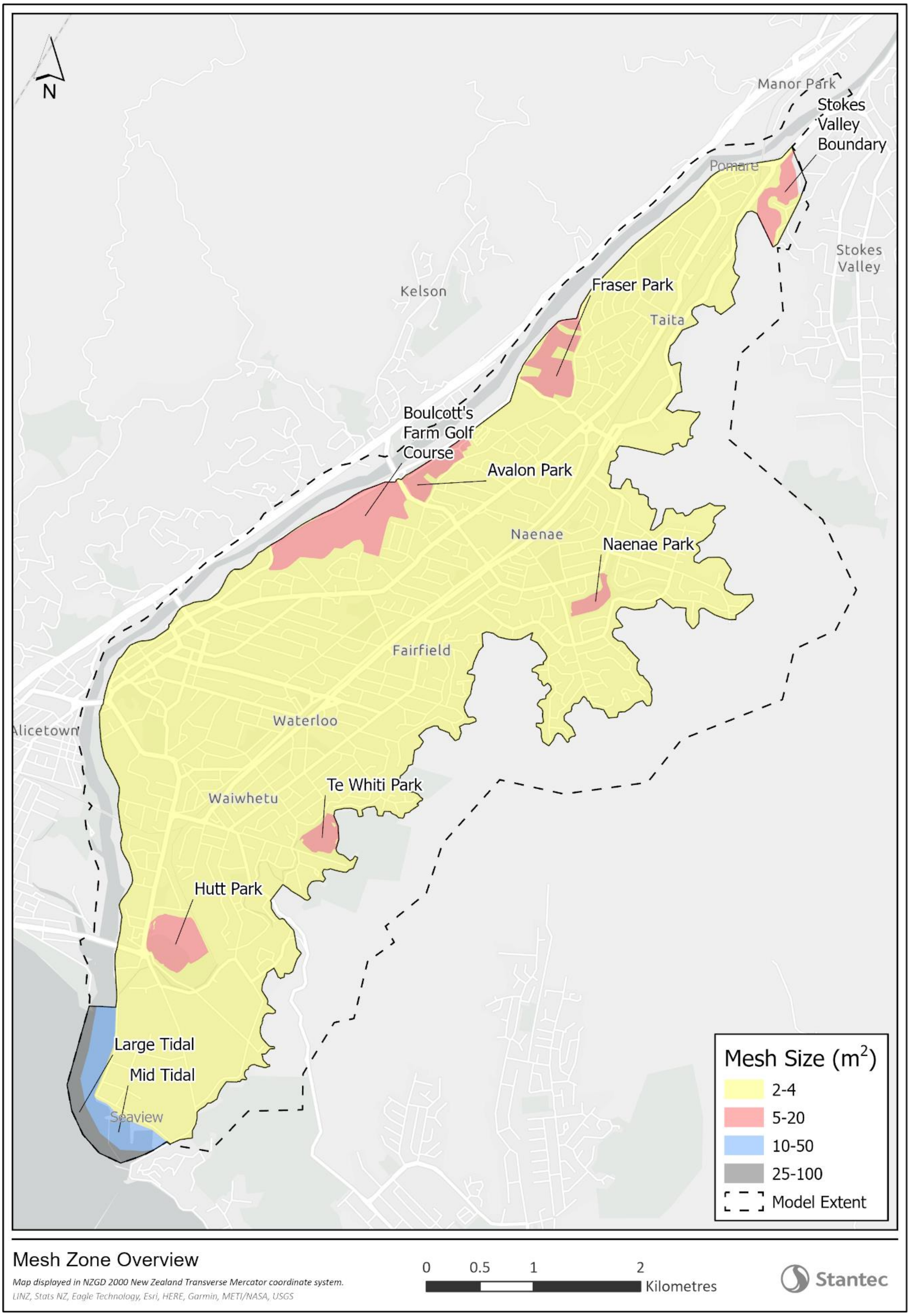


Figure 3-15 Overview of mesh zones used to reduce the total number of mesh elements

3.3.4.3 Surface Roughness

Surface roughness was applied as roughness zone polygons based on the regional roughness layer outlined in Section 2.3.4. To reduce the number of polygons, a default Manning's N roughness of 0.02 was applied to the 2-D zone, and any polygons with a Manning's N of 0.02 were not imported. A Manning's N of 0.02 was selected as this provided the largest reduction in polygons with interior holes (these are not compatible with InfoWorks ICM). Table 3-9 and Figure 3-16 provide an overview of the surface roughness values applied within the model.

Table 3-9 Overview of surface roughness Manning's N

Surface Type	Manning's N
Roads and footpaths	0.02
Vegetation: bare	0.04
Recreational area/playing field	0.05
Vegetation: scrub/fax	0.08
Vegetation: forest	0.10
HCC: residential properties	0.20
Non-residential properties: buildings	0.50

A high-level check and update to the roughness zones was completed, and a total of five alterations were made. Of these five changes, no alterations were made to the polygon geometries, apart from adding in a small section in the south of the catchment to model a treatment swale. Thus, it is expected that small inconsistencies may be present which may have a small impact on the results at a local scale. This is because the larger polygons cannot represent the highly varied surface roughness typical of many urban areas. Thus, further detailed investigations are recommended when using the model at a local scale. Overall, uncertainty in surface roughness polygons is insignificant at regional scale as the main overland flow paths roughness values are applied directly to 1-D elements and are therefore not reliant upon the 2-D roughness layer.

3.3.4.4 Building Voids

Building footprints were obtained from the LINZ Data Service and all footprints with an area greater than 500m^2 were included in the model as void polygons. The LINZ polygons were simplified by removing vertices within 3m of each other and removing any interior holes within the building shape to remove possible InfoWorks ICM mesh generation errors. Finally, ~10 buildings had to be manually edited as their footprints extended into river reach boundaries. This editing process involved using the aerial imagery to digitise the buildings more accurately, removing overlaps with river reach boundaries. The simplified building footprints are then used as voids in generation of the 2-D zone mesh, excluding them from the 2-D surface.

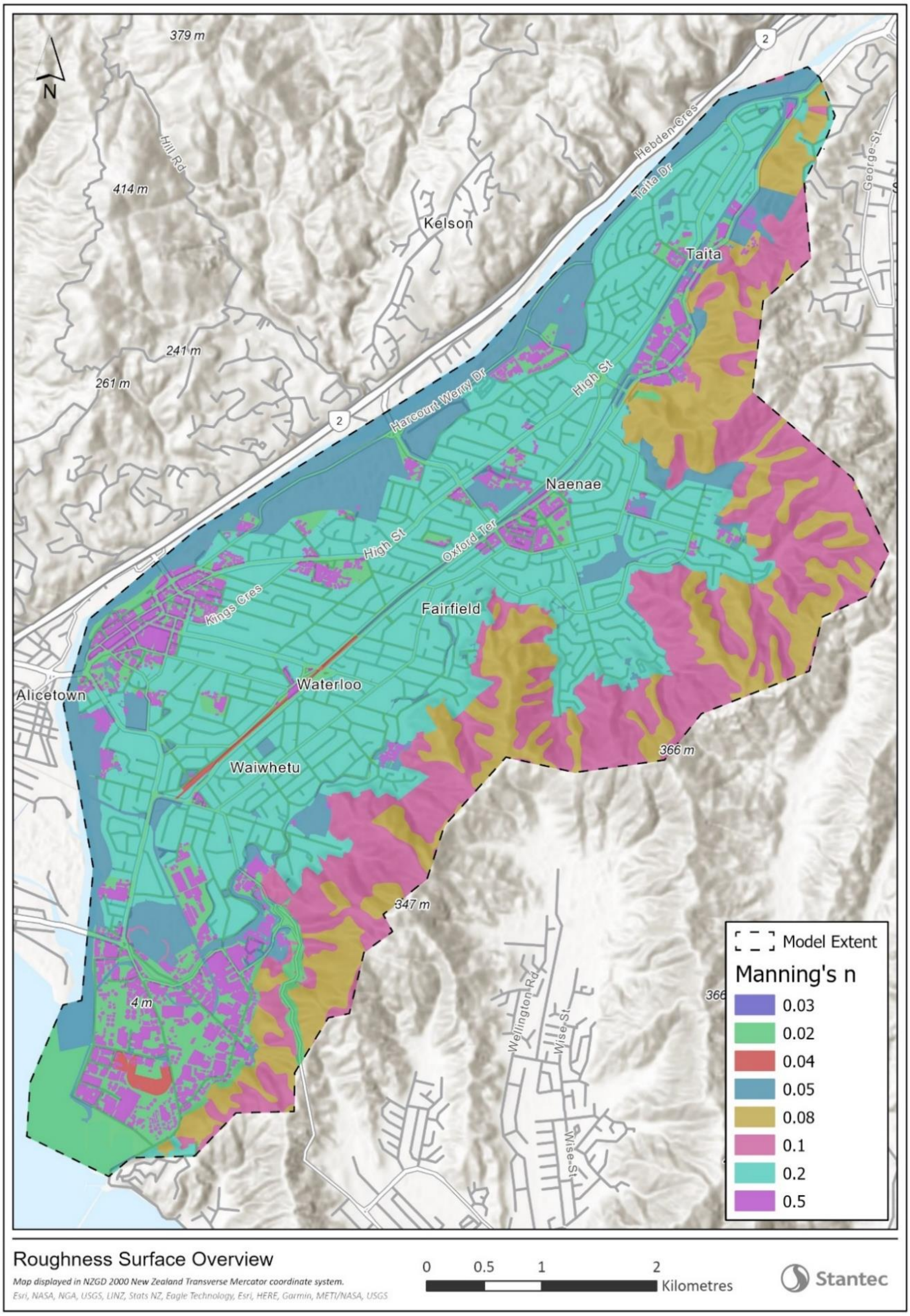


Figure 3-16: Manning's n roughness as applied in the 2-D model.

3.3.5 1-D/2-D Connections

All modelled 1-D and 2-D structures are interconnected. In points of transition, such as bank lines and 2-D nodes, flow is transferred between each domain in all directions. There are three key links between the 1-D pipe network and the 2-D surface in the ELH model including: 2-D nodes, bank lines, and inline banks.

3.3.5.1 2-D Nodes and Sumps

As discussed in Sections 3.3.2.3 and 3.3.2.4, manholes and sumps with a connection to the 2-D surface are modelled using the "Gully 2-D" Flood Type and a custom head discharge curve (Appendix G – Head-Discharge Curves.) These nodes can transfer flow between the 1-D network and the 2-D mesh elements that the nodes are located in.

3.3.5.2 Bank lines

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 or surveyed cross section ends. Flow across bank lines is calculated using an irregular weir equation. The alignment of the bank lines has been digitised to represent an appropriate top-of-bank location. The discharge co-efficient values for the bank lines have been set at 0.8, and the modular limit set at 0.7 as per the Regional Stormwater Hydraulic Modelling Specifications v5 (Wellington Water Ltd, 2017).

3.3.5.3 Inline Banks

Inline banks are used at connections between river reaches and the reticulated network, or at most stream crossings that are not modelled using an InfoWorks ICM bridge structure (see Section Figure 3-17**Error! Reference source not found.**). They are also used when modelling ponds in the 1-D network (See Section 3.3.2.11). Flow across bank lines is calculated using an irregular weir equation.

Figure 3-17 provides an overview of a typical culvert outlet into a river reach. The inline bank allows for flow over the bank line providing a connection to the 2-D surface upstream of the river reach. Sealed manholes rather than break nodes are used to provide increased model stability.

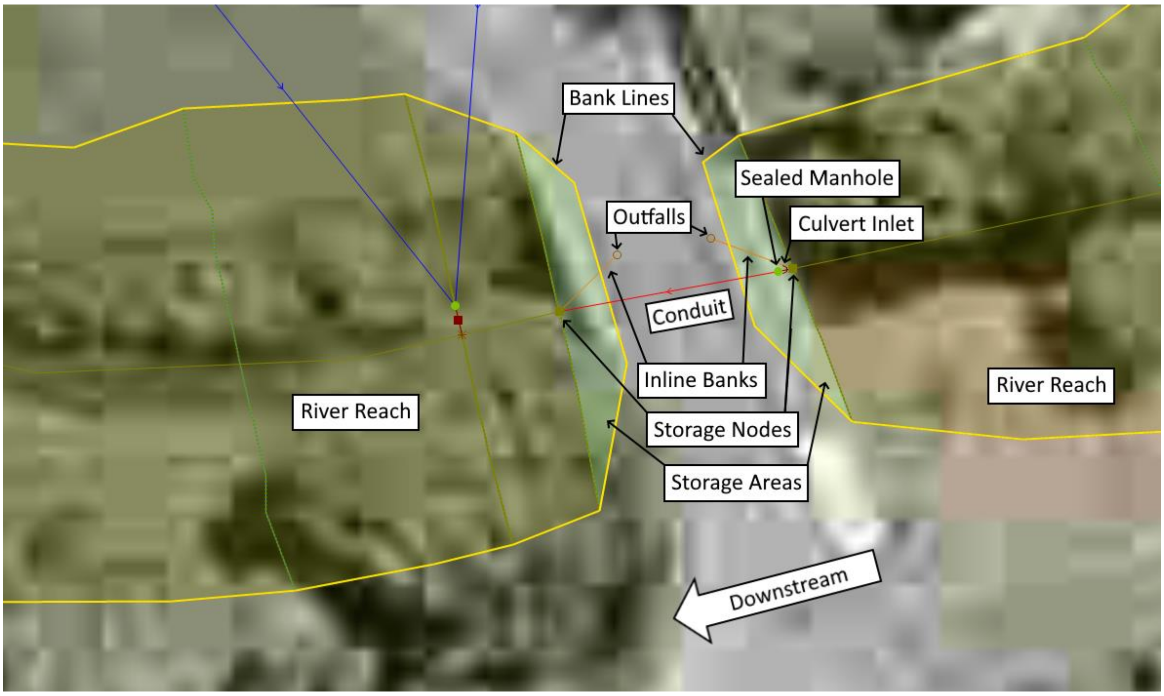


Figure 3-17 Layout of a typical connection between a river reach and a culvert inlet/outlet. This example shows a stream crossing modelled as a culvert.

3.4 BOUNDARY CONDITIONS

3.4.1 Rainfall Data

For model testing and simulation of the standard scenarios outlined in WWL Regional Stormwater Modelling Specifications v5, HIRDSv4 rainfall data was extracted for a central point within the Eastern Lower Hutt catchment (1762492, 5436908 NZGD2000). This point was chosen after consideration of the potential for an orographic control on rainfall by the Eastern Hills. Several sample points along the hills produced no significant difference in rainfall depths as the HIRDSv4 data is based on a 2km-by-2km grid (shown in Figure 3-18). The difference between HIRSv4 (Henderson, Collins, Doyle, & Watson, 2018) and HIRDSv3 by NIWA (2010) is not considered significant in development of a nested rainfall profile, as demonstrated in 0.

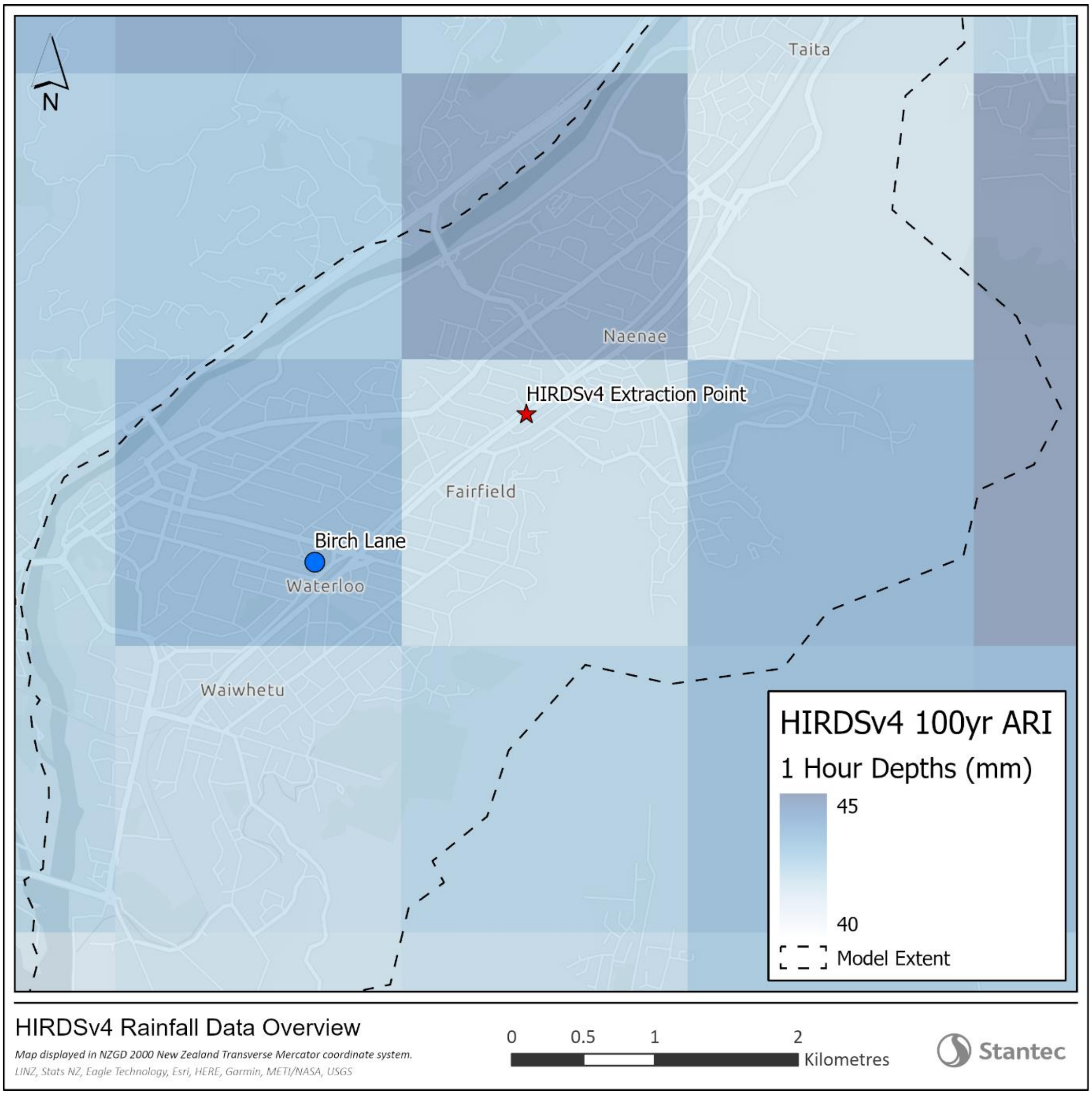


Figure 3-18 HIRDSv4 rainfall data overview showing the location of the GWRC Birch Lane rainfall gauge, and a depth range of approximately 5mm. Storm: 100-year ARI, 1-hour duration.

As agreed with WWL, 12-hour nested design rainfall profiles were developed for simulation. These include the 100-year ARI with a 20% rainfall intensity increase for climate change, 100-year ARI with existing climate, and a 10-year ARI with existing climate. The design rainfall profiles have been generated using the rainfall depths summarised in Table 3-10.

Table 3-10 HIRDSv4 Rainfall depths for the central ELH point (1762492, 5436908 NZGD2000)

ARI (years)	AEP (%)	Rainfall Depth (mm)						
		10m	20m	30m	1h	2h	6h	12h
1.58	63.3	7.07	9.68	11.7	16.2	22.6	37.6	51
2	50.0	7.77	10.6	12.8	17.8	24.7	41.1	55.6
5	20.0	10.2	13.9	16.8	23.2	32.1	53	71.5
10	10.0	12.1	16.4	19.7	27.2	37.6	61.9	83.3
20	5.0	14	19	22.9	31.5	43.3	71.1	95.5
30	3.3	15.2	20.6	24.8	34	46.8	76.7	103
40	2.5	16.1	21.8	26.1	35.9	49.4	80.7	108
50	2.0	16.8	22.7	27.2	37.4	51.4	83.9	112
60	1.7	17.4	23.5	28.1	38.6	53	86.5	116
80	1.2	18.3	24.7	29.6	40.5	55.6	90.6	121
100	1.0	19	25.6	30.7	42.1	57.7	93.9	125
100 plus 20% climate change increase	n/a	22.8	30.8	36.9	50.5	69.2	112.6	150.5
250	0.4	22.1	29.7	35.5	48.5	66.2	107	143

3.4.2 Hutt River

The western boundary of the Eastern Lower Hutt model follows the Hutt River stop banks. This will likely control levels and discharge capacity at stormwater outfalls on the Hutt River side of the stop bank. After discussion with WWL, it was decided that the Hutt River should be modelled at a continuous 10-year ARI flow of 1345m^3 at the Taitā Gorge gauging station (Henderson, Collins, Doyle, & Watson, 2018) close to the northeast boundary of the ELH catchment area. A 10-year ARI was chosen as it provides a raised tailwater condition that could occur in a high rainfall event, especially when rain also occurs in the wider Hutt River catchment. The 10-year flow provides slightly conservative values without significantly changing the ARI of the event at a 100-year ARI scale, the main event WWL was interested in. Applying this level across other events such as the 10-year ARI provides a consistent boundary condition for comparison. A static flow rate simplifies the boundary condition and agrees with WWL's requirements and previous modelling in the region which showed that the Hutt River has a long and drawn-out hydrograph peak, producing levels that stay high for the majority of the stormwater model run, See the Petone 2021 stormwater model build report (Sherson, Burdis, & Kerr, 2021).

It was recommended that this flow be modelled as a 1-D river reach using surveyed cross sections as supplied by GWRC.

3.4.3 Tide

A standard 12-hour tidal cycle with a 1.11m astronomical high spring tide that includes a 0.25m barometric increase was added as a tidal boundary to the ELH InfoWorks ICM model, see Figure 3-19.

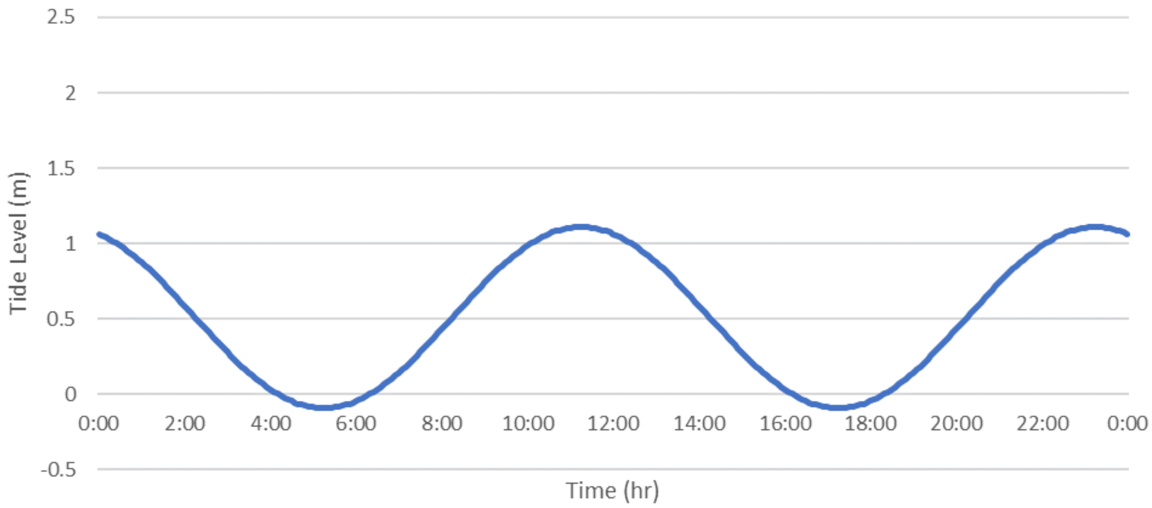


Figure 3-19 24-hour tidal boundary with existing conditions (1.11m peak)

To model sea-level rise, a further 1m was added to the tide for climate change runs as shown in Figure 3-20. This level-controlled boundary was created to represent the changing levels in the harbour due to tidal conditions, following current stormwater specifications in (NIWA, 2012) and (Wellington Water Ltd, 2017). Timing of the peak tide was shifted to match peak discharge in the lower reaches of the Waiwhetū Stream at approximately 11 hours to ensure a conservative approach.

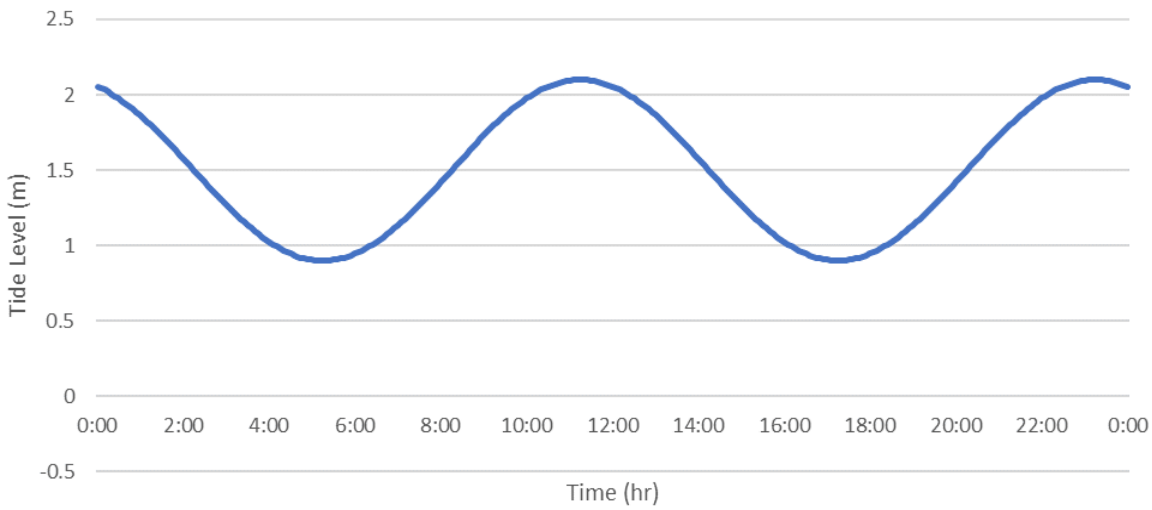


Figure 3-20 24-hour tidal boundary with 1m sea level rise (2.1m peak)

The existing climate, oscillating tide was controlled using a level file for the 2-D tidal boundary, the Hutt River outfall, the Waiwhetū Stream outfall, and eight 1-D outfalls directly connected to Wellington Harbour (see Figure 3-21). Outfalls draining directly to the harbour were not modelled in 2-D to improve stability as the model scope does not include investigating any potential impact of stormwater runoff on Wellington Harbour. The climate change tide with a 2.1m peak could not be applied in this manner as a large proportion of southern section of the ELH catchment is below 2.1m elevation, resulting in numerous instabilities within InfoWorks ICM.

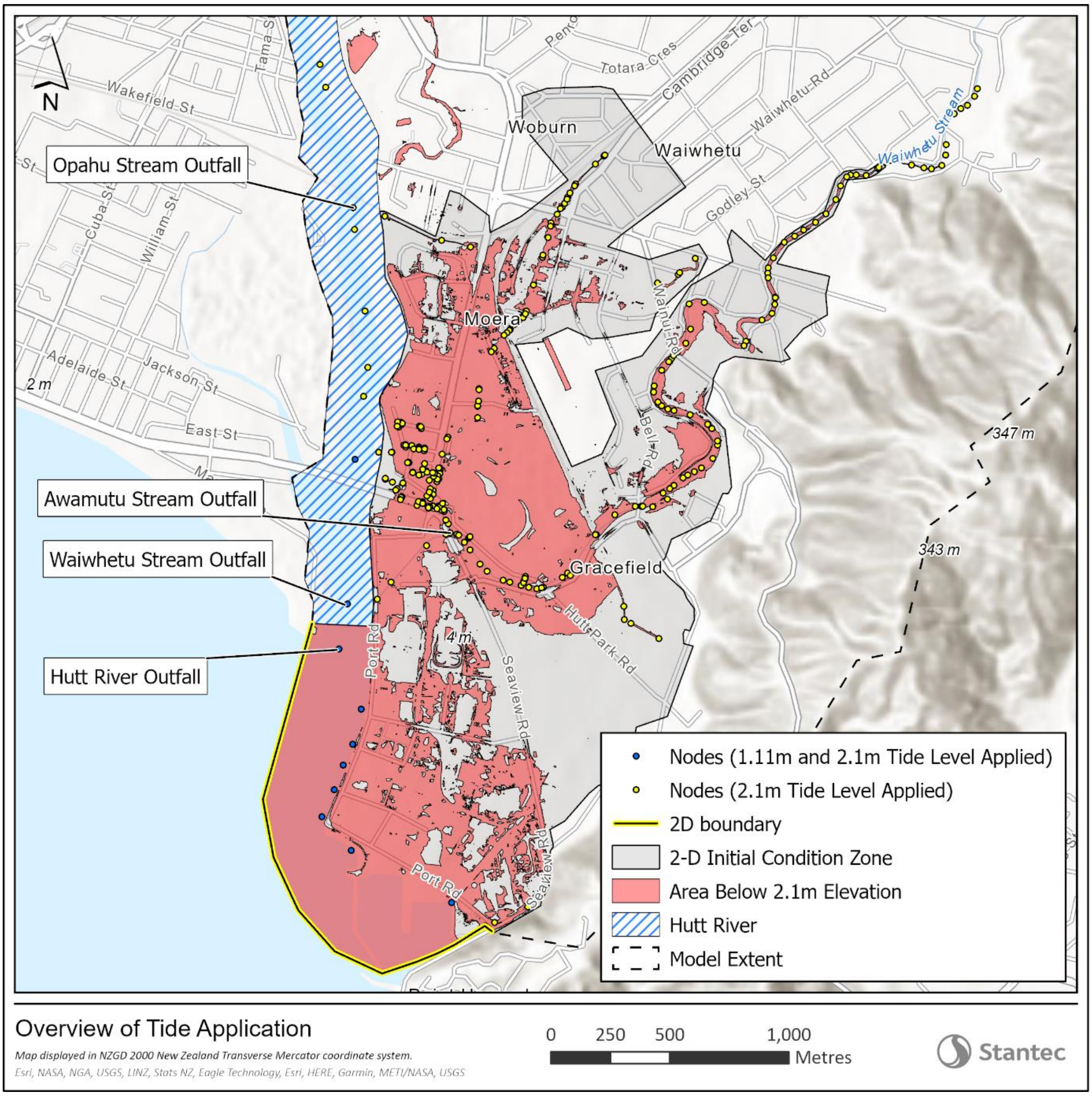


Figure 3-21 Overview of the model components used to apply the 1.11m and 2.1m tides

Figure 3-21 shows the model components used to apply the larger 2.1m tide. Due to instabilities in InfoWorks ICM, all nodes that would be flooded and were connected to river reaches (all nodes symbolised in Figure 3-21) are initially controlled using a level file applying the receding limb of the tide. After low tide, the nodes controlled by the level file reduced to the 10 used to apply the 1.11m tide (see Figure 3-21). The 2-D boundary was controlled using the level file for the entire simulation; however, a 2-D initial condition was required to stabilise the model and apply an initial level of 2.1m across the southern portion of the model. Figure 3-21 shows the area covered by this initial condition, and the area within which is below 2.1m ASL.

The initial condition is intended to exclude areas that are below 2.1m ASL but otherwise isolated from any potential tidal influence. The Opahu Stream is not modelled as tidal as the flap gates present at its confluence with the Hutt River should isolate it from tidal influence.

4.0 RESULTS

4.1 MODEL LIMITATIONS AND ASSUMPTIONS

Computational models are as accurate as the information available to develop them and the data available to verify their accuracy.

Due to the limited extent of pipe invert data as well as other limitations discussed in this section, results from the model are suited to mapping flooding during high magnitude events including an appropriate freeboard to account for uncertainty. Where the model is to be used as input to infrastructure design it is recommended that the area of interest be re-assessed, and additional asset data collected as required to provide an appropriate level of detail.

4.1.1 Model Limitations

The limitations of the model are listed below:

- Manhole and pipe levels for a substantial amount of the network, (~90%) have been interpolated using the in-built InfoWorks ICM interpolation tool.
- Large parts of the model are based on LiDAR which is likely to have an accuracy of no better than +/- 0.2m and less accuracy in vegetated areas. Changes to local topography may also have occurred following capture of the LiDAR in 2013 and 2016. The uncertainty associated with modelled assets based on LiDAR including ground levels should be considered. Further information regarding the LiDAR used in the development of the model is described in Ground Model Assessment Summary Report (MWH, 2016).
- Building floor levels are not included in the model.
- Various structures that may affect flood flows and level are not included in the model. These include road tunnels, walls, fences etc.

4.1.2 Hydraulic Model Assumptions

The following assumptions have been applied in the development of the hydraulic model:

- The LiDAR generated ground model was assumed to be an accurate representation of catchment topography. As discussed in Section 4.1.1 above, this may not always be the case.
- The interpolation rules applied are appropriate.
- No sediment has been added to the pipes.
- Steep pipes, with gradients greater than 10% were assumed to have no headloss at the upstream end of the culvert, following discussions with WWL and the standard InfoWorks ICM process.

4.1.3 Hydrological Model Assumptions

The following assumptions were applied in the development of the InfoWorks ICM hydrological model:

- The automated subcatchment delineation and ToC calculations are assumed to be appropriate.
- WWL methodology for representing hydrology is appropriate for Eastern Lower Hutt.
- Regional hydrology layers provide an accurate representation of subcatchment scale hydrological processes.

4.2 INITIAL MODEL TESTING

4.2.1 Stability Tests

The log results for each model run were checked for any significant errors or mass balance problems. No significant issues were found. Mass and volume balance errors are reported below in Table 4-1.

Table 4-1 Mass and volume errors in simulations

Simulation	Total Mass Error (m^3)	Mass Balance Error (m^3)	Volume Balance Error (%)
100-year ARI Climate Change	0.0903	523.0	0.0018
100-year ARI Existing Climate	0.0944	363.6	0.0017
10-year ARI Existing Climate	0.0763	255.8	0.0021

4.3 PRELIMINARY RESULTS

Preliminary results are presented for the following events:

- Nested 10-year ARI design storm (Figure 4-1)
- Nested 100-year ARI design storm (Figure 4-2)
- Nested 100-year ARI design storm including a 20% climate change increase (Figure 4-3).

The reported flooding issues (shown in Figure 2-2) following the February 2004 event are broadly consistent with preliminary model results, indicating the model predicts flooding in areas with known flooding issues. Particular problem areas include the Waiwhetū Stream, Hutt Park, and suburbs adjacent to the Eastern Hills. Naenae is shown to be particularly susceptible to flooding due to runoff from the Eastern Hills, with flooding along roads and pooling closer to the Waiwhetū Stream during the design events.

Low lying, poorly drained areas are shown to be susceptible to flooding at the 10-year ARI design event, with much more widespread flooding present in the 100-year ARI and 100-year ARI (20% climate change) design events. The Hutt Valley railway line is also bounded by flooding in all design events, consistent with its raised elevation above the valley floor. Flood extents and depths are considered in detail as part of model validation (Section 6.0).

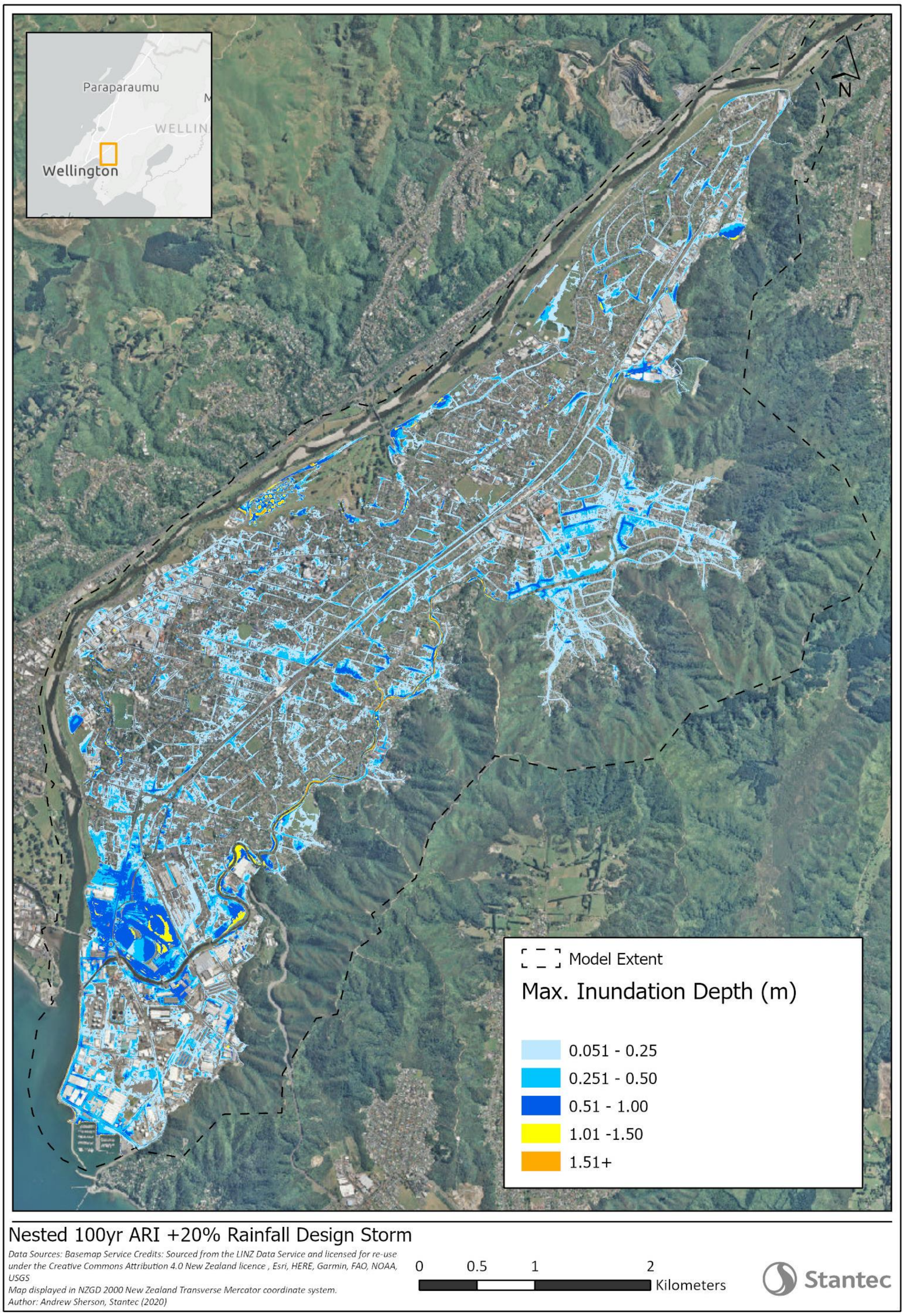


Figure 4-1 Preliminary flood extents for the nested 100-year ARI design storm including a 20% climate change increase

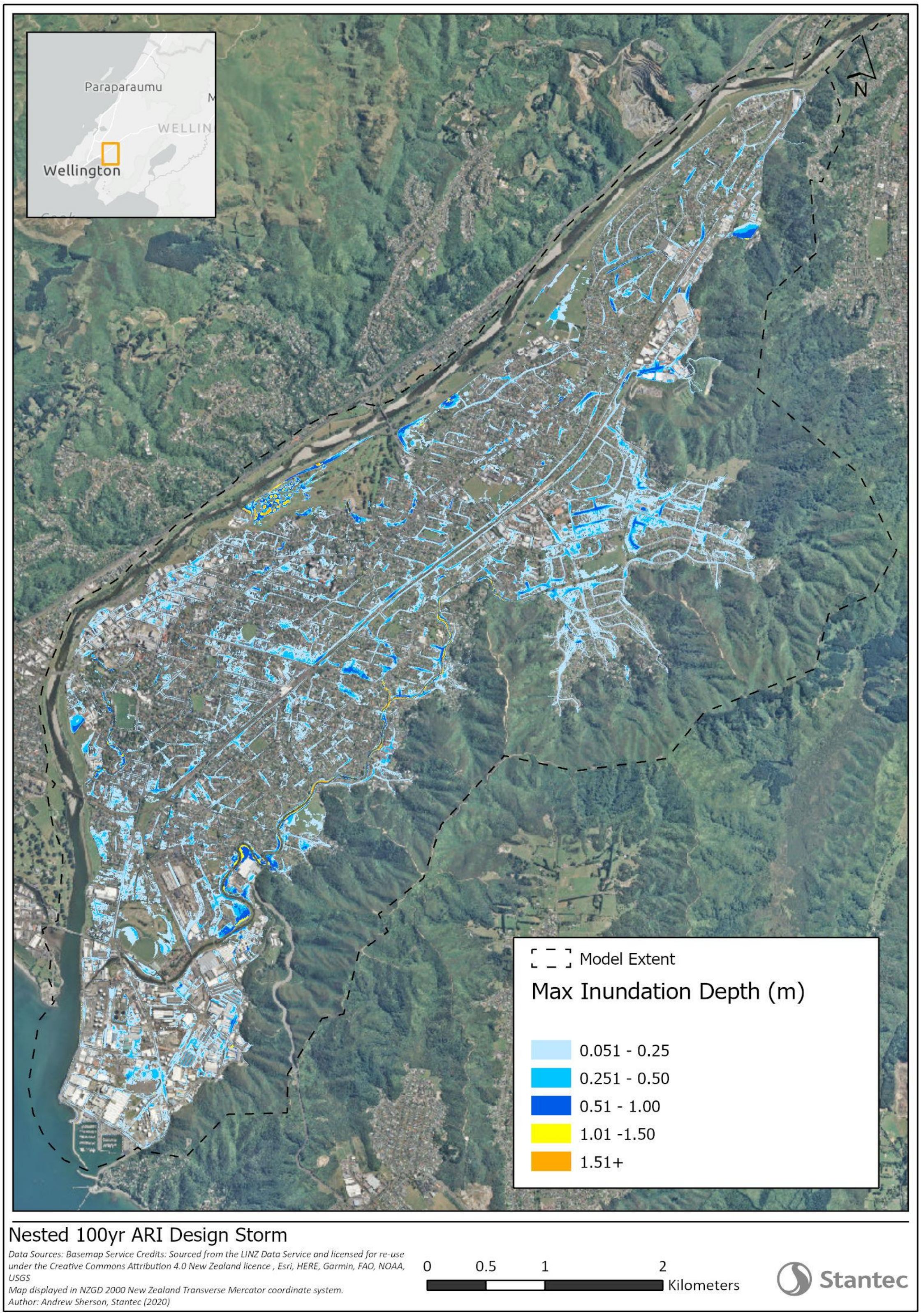


Figure 4-2 Preliminary flood extents for the nested 100-year ARI design storm

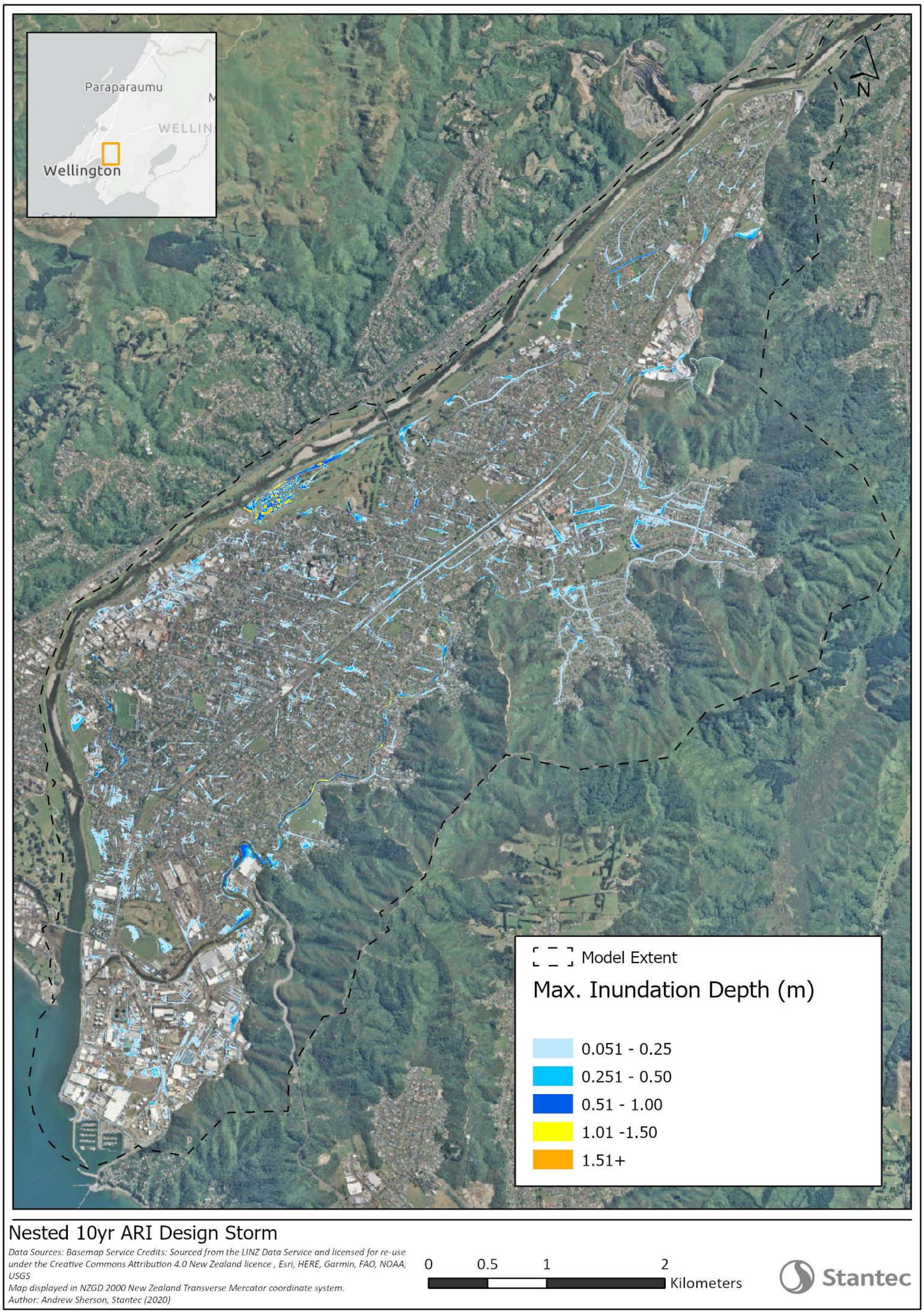


Figure 4-3 Preliminary flood extents for the nested 10-year ARI design storm

4.4 MODEL QUALITY ASSURANCE

An internal check and review process of the model build was conducted, in combination with an external review by WSP in April 2021 following the WWL model check and review process. During the check and review process, various concerns were raised. Most of these concerns were addressed, and all significant concerns were raised with WWL and rectified where possible.

5.0 CALIBRATION

Following the completion of the model build, it was agreed between WWL, GWRC, and Stantec that selected notable rainfall events would be used to compare the model results with recorded flows and levels at the Whites Line East gauge in the Waiwhetū. Five significant flood events were identified over the previous 20 years including: 13th - 18th February 2004, 2nd - 4th March 2012, 15th May 2016, 15th November 2016, and 8th December 2019. These events were selected based on reported flooding, data availability, and discussion with WWL and GWRC.

5.1 WHITES LINE EAST GAUGE

Model calibration focused on recorded level, discharge, and total flood volume at the Whites Line East gauge (see Figure 1-1) as this provides the best continuous quantitative record of potential flood issues within the ELH catchment. Figure 5-1 provides an overview of the Waiwhetū Stream flow record at Whites Line East from 2001 to 2020, with calibration event peaks highlighted.

Due to a lack of suitable gauging events, there is considerable uncertainty for flows of above 20 m^3s^{-1} using the Whites Line East rating curve (provided by GWRC). An alternative rating curve to better represent higher flow events is under development by GWRC, with the current and preliminary alternative rating both (shown in Figure 5-6). This uncertainty only impacts the conversion of recorded stage (mm) to flow (m^3/s^{-1}) for the February 2004 event, resulting in a peak recorded discharge range of 28.9 m^3s^{-1} (original) to 36.8 m^3s^{-1} (alternative). Model performance can still be assessed using observed level at the Whites Line East gauge as levels are more important to the calibration/validation process because they control flood extent, the key driver for WWL.

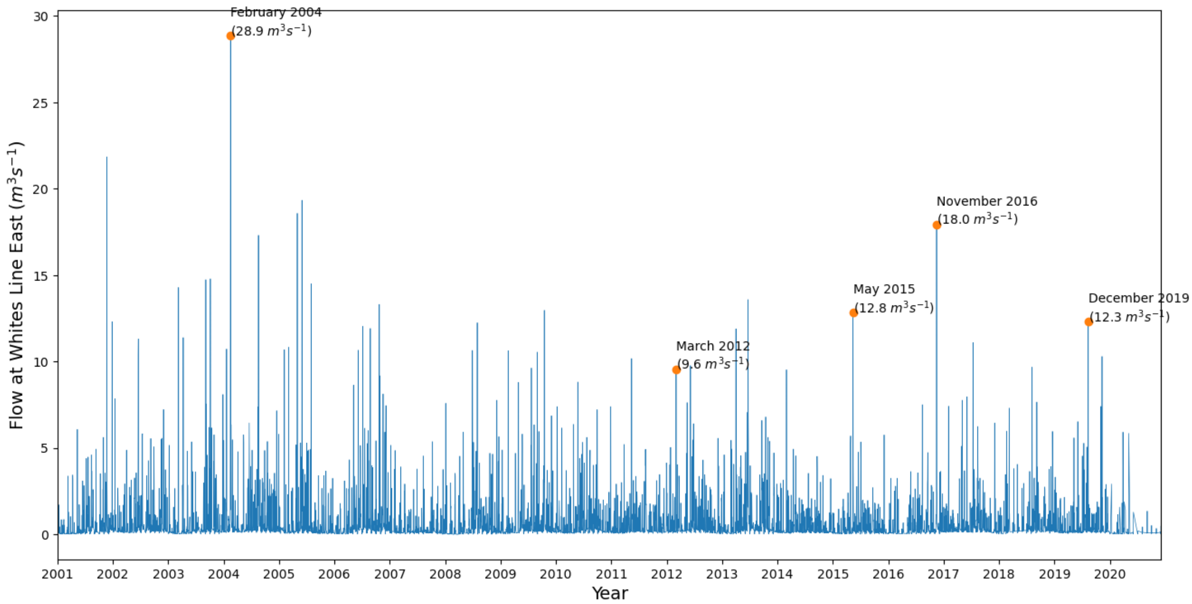


Figure 5-1 Overview of Waiwhetū Stream flow data recorded at Whites Line East from 2001 to mid-2020.

5.2 ALTERATIONS TO THE WAIWHETŪ STREAM

Major works were carried out in the Waiwhetū channel from the Bell Road bridge downstream to the Hutt confluence between 2009 and 2010. This involved removal of 12,000m³ of contaminated sediments, and channel upgrades to address capacity issues highlighted by the 2004 storm event. To account for the pre-upgrade channel capacity, a scenario was built into the model using 2005 cross sections for the Waiwhetū Stream as shown in Figure 5-2. This scenario helped improve model calibration and is only used for simulating the February 2004 storm event.

Figure 5-2 Cross sections altered to account for the 2009-2010 Waiwhetū channel improvements.

5.3 QUEENS WHARF AS A TIDAL BOUNDARY

Tide as recorded at the Estuary Bridge gauge represents a combination of both tide and flow in the Hutt River. The Queens Wharf tidal record has been used as a boundary condition for all calibration event model simulations as level in the Hutt River is accounted for using the Taitā Gorge gauged flow as an inflow. Queens Wharf provides a tidal record independent of levels in the Hutt River, reducing the potential for overestimation of level in the Hutt River. The combination of inflow based on the Taitā Gorge gauge, model discharge into the Hutt River, and the Queens Wharf tide should result in modelled levels at the Estuary Bridge gauge similar to observed stage. This is demonstrated in Figure 5-4, also indicating no significant difference in tide timing between Estuary Bridge and Queens Wharf.

5.4 SENSITIVITY TESTING FOR CALIBRATION

Initial model results showed typically higher peaks and a more rapid receding limb than the recorded events were showing at Whites Line East gauge. Total volumes for the events, however, were reasonable. Following discussion with WWL and GWRC, it was agreed that sensitivity testing would be undertaken to look at factors that affect flow though the network. Both curve number adjustments and stream roughness adjustments were trialled for the calibration scenarios and results are discussed in the following sections.

5.4.1 Curve Number Adjustment

High level testing of the SCS curve number (CN) was carried out by applying a uniform 20% reduction in CN across the catchment (excluding building polygons). This provided a uniform reduction in total volume, peak flow, and peak level but did not impact the relative pattern of discharge at White Line East. Flow volumes also became lower than recorded for most calibration events. It was therefore concluded that a widespread alteration of CN would not improve model calibration. Detailed consideration of land use change, and major modification of the Regional Hydrology Layers would fall outside of the scope of the current project.

5.4.2 Stream Roughness Adjustments

Roughness values along the natural reaches of the Waiwhetū Stream were adjusted within a Manning's n range of 0.03 to 0.08 based on site observations and standard river characteristics (Hicks & Mason, 1998). Following multiple adjustments and qualitative investigation of model performance, a Manning's N of 0.055 was adopted for a large portion of the Waiwhetū Stream as it provided the most consistent match between modelled and observed stage and flow. A channel roughness of 0.055 was therefore applied to natural river reaches, excluding the Hutt River (see Figure 3-8), concrete constructed channel sections, and minor channels upstream of inlet locations. The Manning's N values selected for the modelled channels following calibration are discussed in Section 3.3 and shown in Figure 5-3.

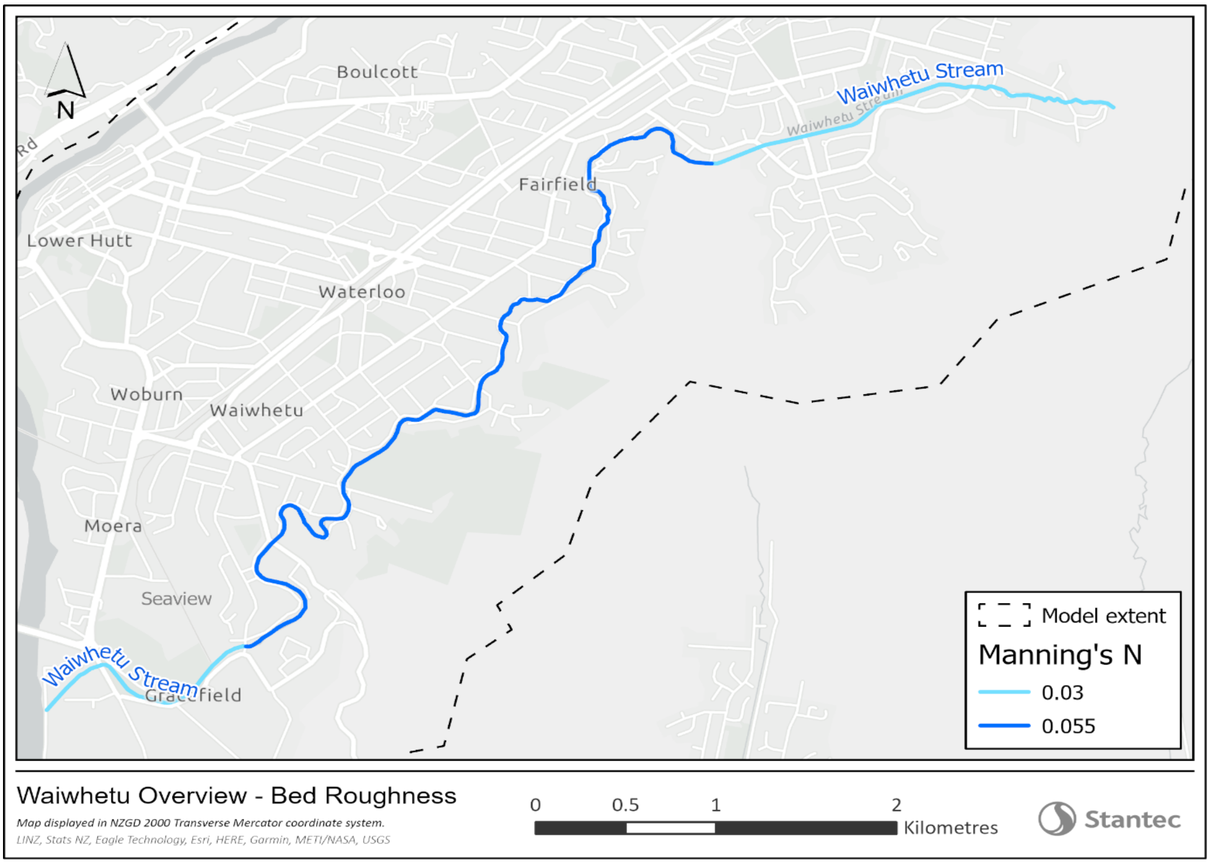


Figure 5-3 Overview of channel bed roughness as applied to the Waiwhetu Stream

5.5 RESULTS OF CALIBRATION

Figure 5-4 shows a comparison between the modelled and observed stage at Estuary Bridge. Timing generally matches well between modelled and observed stage, however some differences are observed in the February 2012 and May 2015 events. There are also some differences between peak stage particularly for the May 2015 and November 2016 events. The precise cause of these differences is unknown. However, application of the tide using observed levels at Queens Wharf is considered to be appropriate in order to prevent potential double counting of Hutt River flow that may have occurred if the Estuary Bridge gauge was adopted for the tide. Estuary Bridge data was not available for the December 2019 event.

Figure 5-5 and Figure 5-6 provide a comparison of stage and flow for the five selected calibration events following adjustments made to stream roughness. Table 5-1 summarises modelled and observed peak flow, level, and total volume at the Whites Line East gauge. The model calibrates reasonably well at the Whites Line East gauge. Peak flows and levels are generally within ~+20% and total volumes are within ~+5 to 15%. The May 2015 event calibrates closest based on peak flow, level, and total volume. However, the model over-predicts flow at some of the peaks, in particular during the March 2012 and December 2019 events. This is likely due to rapid runoff occurring as a result of the hydrological methods adopted.

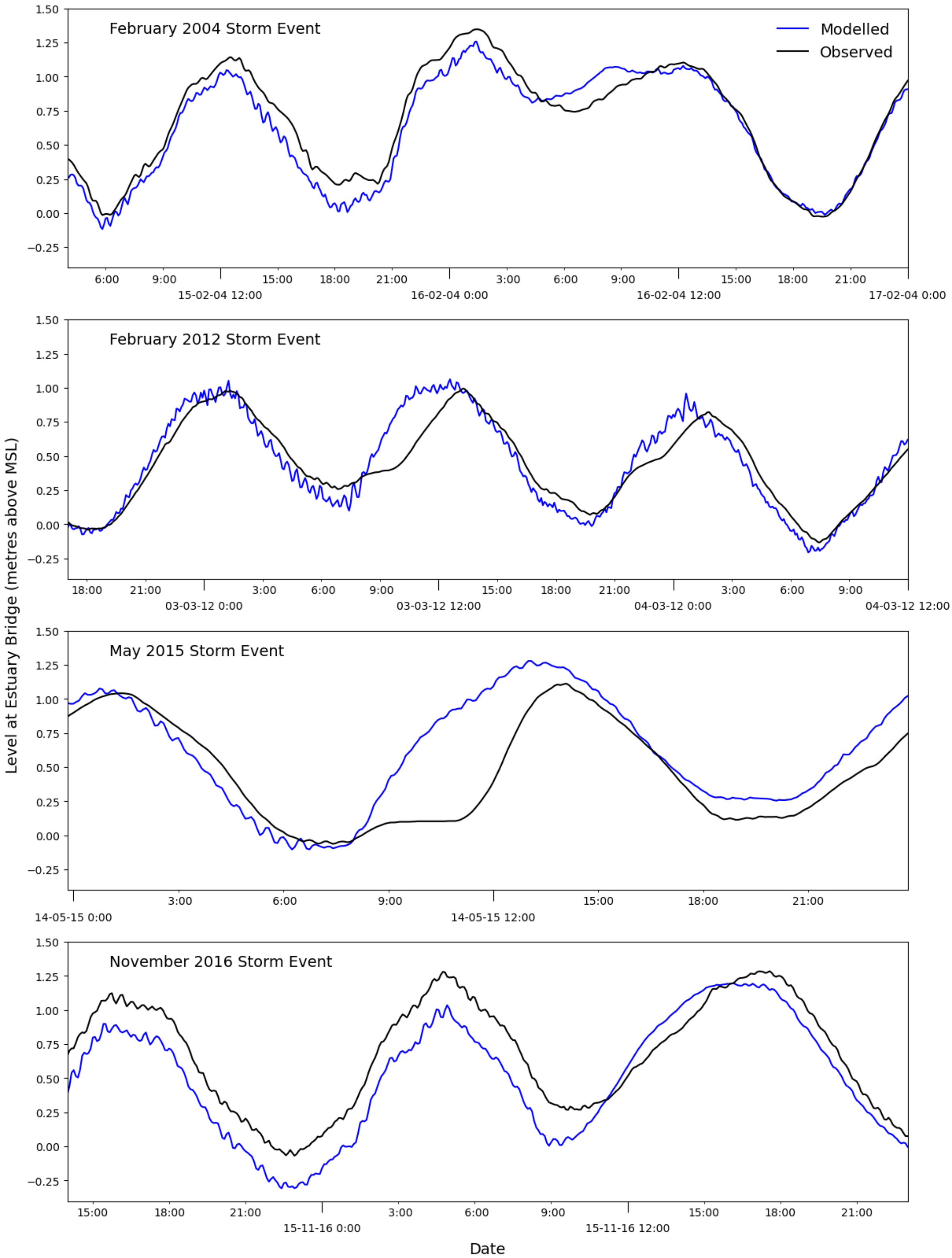


Figure 5-4: Recorded and modelled stage (mm) at Estuary Bridge. Data is unavailable for the December 2019 event. Modelled stage includes the combined impact of discharge to the Hutt River, the Taita Gorge inflow, and the Queens Wharf tide.

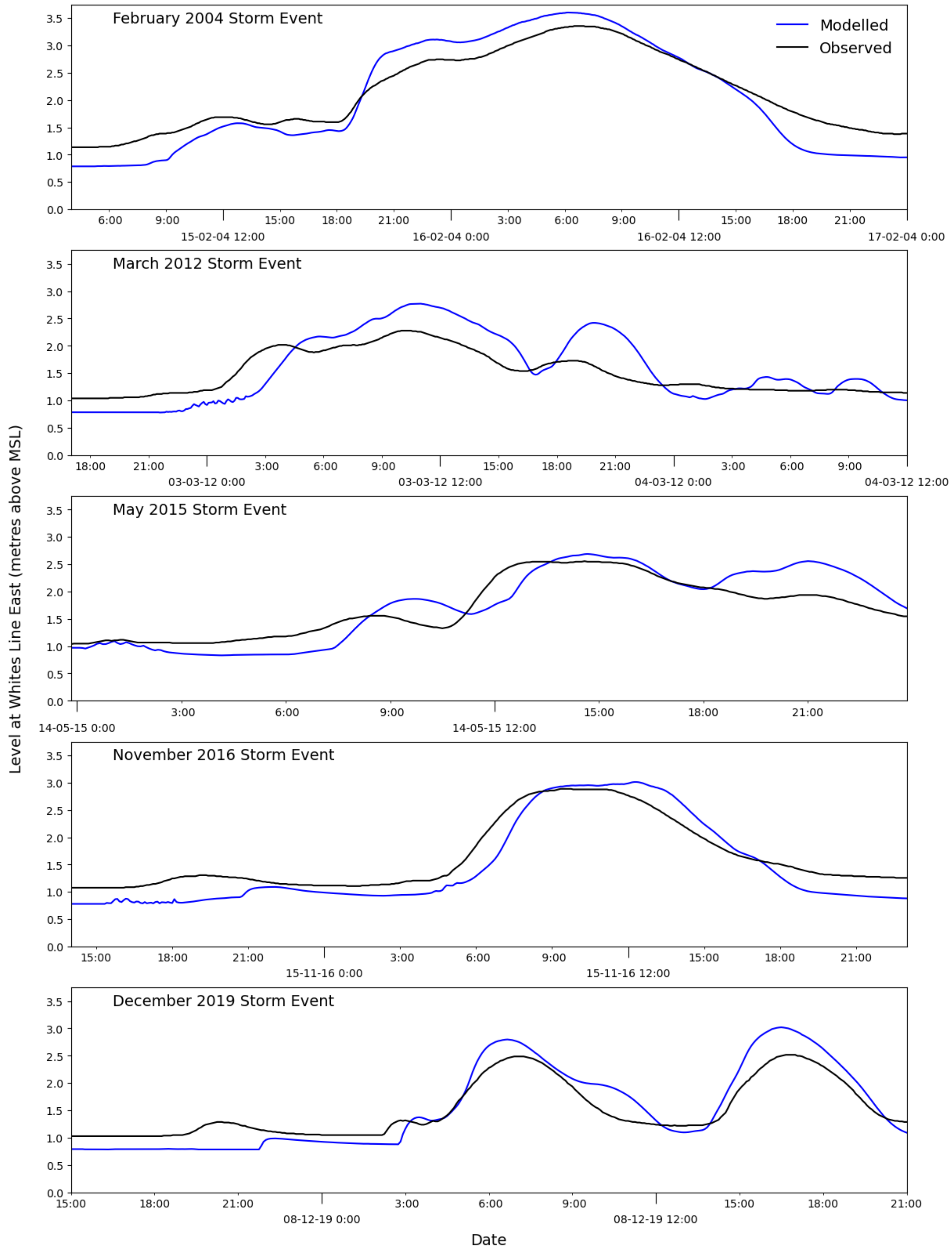


Figure 5-5 Comparison between observed and modelled level at Whites Line East

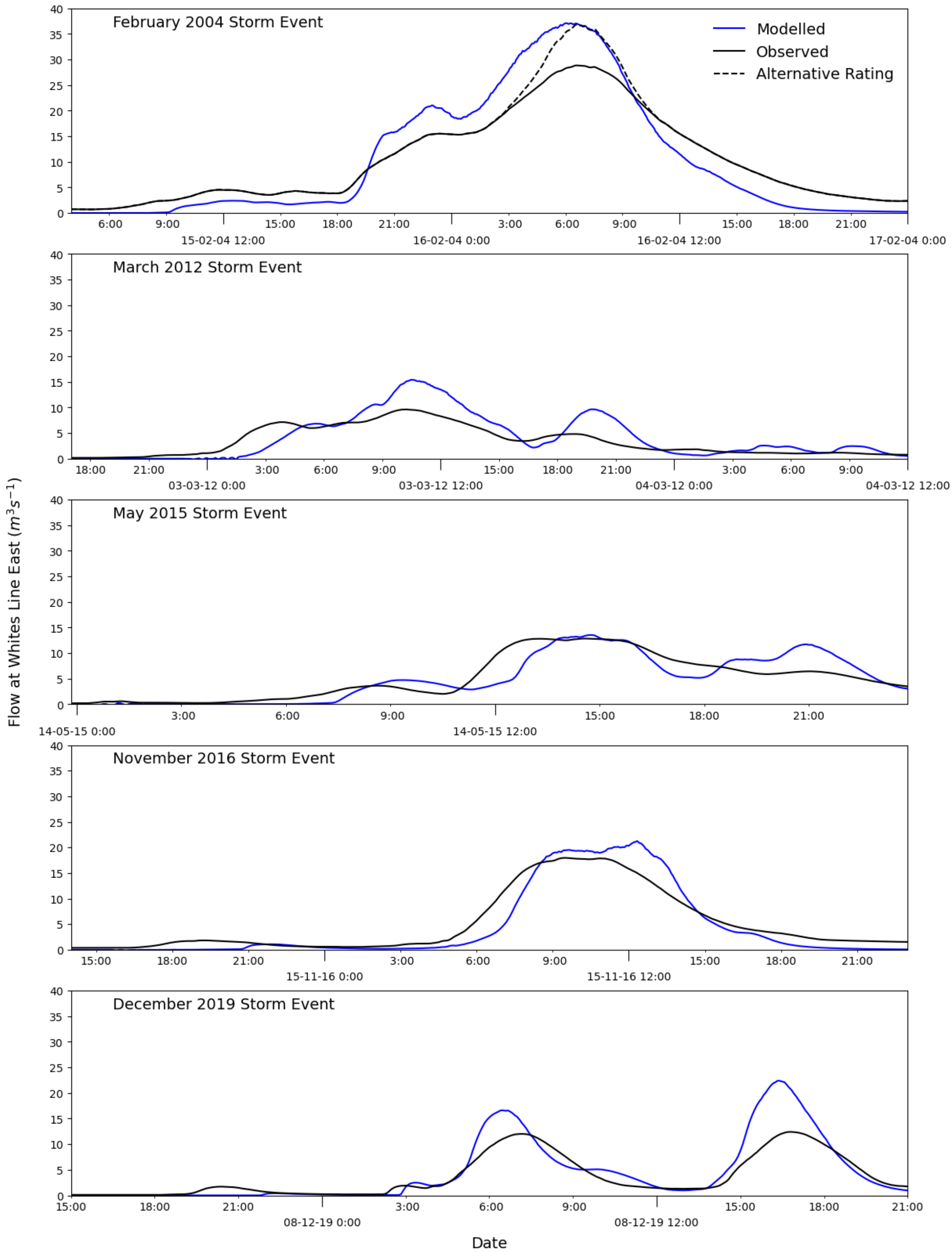


Figure 5-6 Comparison between observed and modelled flow at Whites Line East.

Table 5-1 Summary of calibration data at the Whites Line East gauge.

Event	Peak Level (m MSL)		Peak Discharge (m^3s^{-1})		Total Volume (m^3)	
	Observed	Modelled	Observed	Modelled	Observed	Modelled
December 2019	2.5	3.0	12.4	22.4	392,022	479,012
November 2016	2.9	3.0	18.0	21.2	599,244	526,852
May 2015	2.6	2.7	12.8	13.4	443,292	422,512
March 2012	2.3	2.8	9.6	15.4	524,841	599,738
February 2004	3.4	3.6	28.9 to 36.8	37.1	1,630,446 to 1,744,482	1,631,330

6.0 VALIDATION

The following section discusses how the model flood extents compare to observed flood extents using photographs and some reported flood levels. Although the same high rainfall events are used for the calibration the purpose and outcomes are different.

The February 2004 and November 2016 storm events were selected for validation from the five model calibration events by the Waiwhetū Governance group (WWL, GWRC, WSP, and Stantec). The 2004 and 2016 events were chosen because of the amount of available information including flood reports, recorded levels, photos, rain gauge data, and flow gauge data. They are also of particular significance in the community due to the widespread flooding around the Waiwhetū Stream that occurred in February 2004, and the relative recency of the November 2016 event. The two events have the largest recorded peak flows in the Waiwhetū Stream since 2004.

Frequency analysis of the Whites Line East gauge data indicates an ARI of approximately 10 years for the 2016 peak of 18.0 m^3s^{-1}, and an ARI yet to be confirmed by GWRC for the 2004 event. Figure 6-1 shows the frequency distribution and annual peak discharge, with the individual events discussed in more detail below.

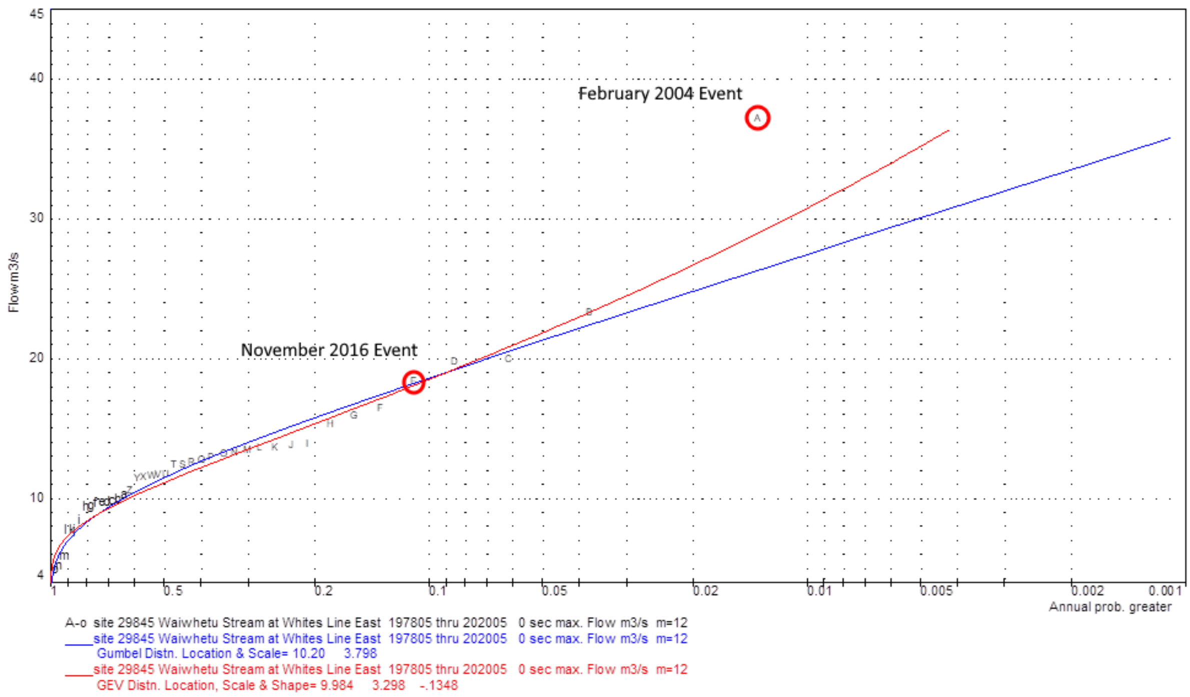


Figure 6-1 Frequency analysis of the Whites Line East gauge data (31st May 1978 to 21st May 2020).

6.1 15-16 FEBRUARY 2004 RAINFALL EVENT

Starting on the 15th of February 2004, the Lower North Island experienced a sustained rainfall event that caused widespread flooding in the Wellington region and further north in Whanganui, Taranaki, and Manawatu. Southerly gales were sustained for most of the event on February 15th and 16th, with the storm moving slowly east across the lower North Island. Total recorded rainfall depths over a 24-hour period reached 200mm at some gauges within the Hutt catchment, with the event characterised by sustained, long duration rainfall. Figure 6-2 shows the 24-hour rainfall depths observed across the Wellington region. Flooding in areas to the north of Wellington were hit the hardest with an estimated 220mm of rainfall in 24 hours.

The February 2004 event was highly unusual with a weather system more typical of winter, but with higher temperatures resulting in higher precipitation. Although not a typical event, the severity of the storm means that it was important to the community across the Eastern Lower Hutt catchment, particularly along the Waiwhetū Stream where the most severe flooding occurred.

A return period is hard to determine for flow in the Waiwhetū Stream as the recorded flows were the highest since the gauge was installed in 1978. Based on the flood frequency analysis of the Whites Line East gauge data the recorded peak of 36 m^3s^{-1} represents an ARI of approximately 100 years, however as shown in Figure 6-1 the event does not correspond well to the projected frequency distribution. This indicates considerable uncertainty around the predicted ARI, which also correlated poorly with estimated rainfall return periods of up to 50 years. The Whites Line East gauge rating is under continued investigation by GWRC for the 2004 event.

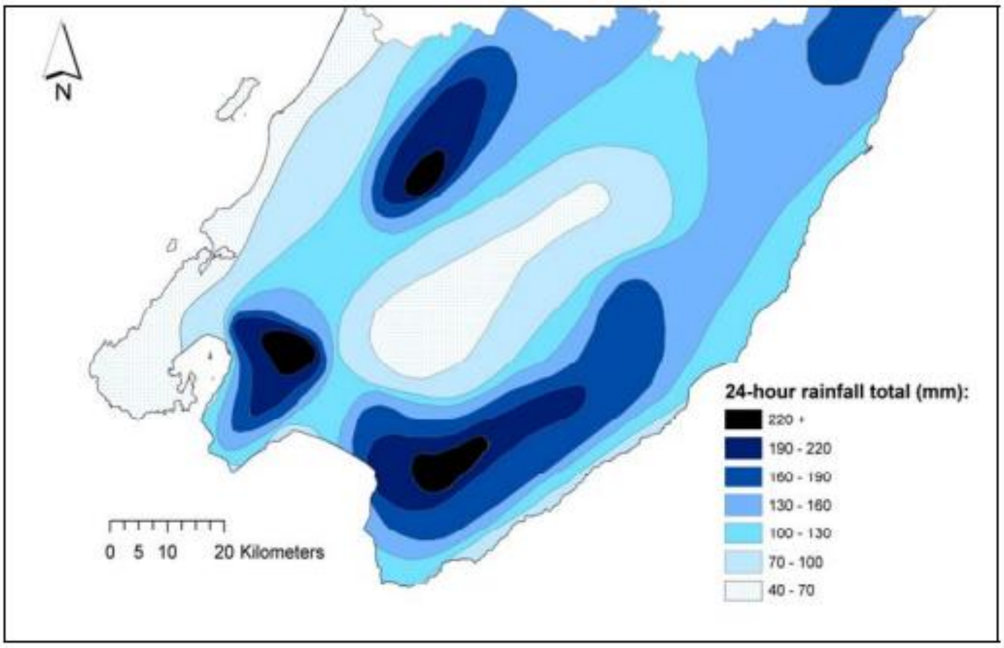


Figure 6-2 24-hour rainfall totals for the period beginning the 15th of February 2004 at 9am (Figure 3, from (Watts & Gordon, 2004)).

6.1.1 Model Inputs

Tide level data from the Queens Wharf gauge was applied to the model representing the tidal boundary condition in Wellington Harbour as per the methodology described in Section 3.4.3. The recorded flow of the Hutt River at the Taitā Gorge gauge was applied as an inflow at the northern end of the model, allowing the Hutt River to be modelled as a boundary condition with a peak flow of 1067 m^3s^{-1}.

Table 6-1 Summary of model data.

Key Model Inputs	
Start	15/02/2004 00:00
End	17/02/2004 00:00
Rainfall	Rainfall gauges applied to banding based on Watts and Gordon (2004) shown in Figure 6-2.
Tide	Level at Queens Wharf (GWRC)
Inflow	Hutt River flow at Taitā Gorge (GWRC)
Waiwhetū at Whites Line East peak	16/02/2004 06:35
Hutt at Taitā Gorge peak	16/02/2004 07:35
High Tide at Queens Wharf	15/02/2004 12:20, 16/02/2004 01:20, 16/02/2004 12:35

Rainfall radar was not available for the February 2004 event (available from 2008 onwards), resulting in larger uncertainty around rainfall distribution within the Eastern Lower Hutt catchment when compared to the November 2016 event. This uncertainty was compounded by a lack of rainfall data at the Mabey Road gauge (location shown in Figure 6-3).

A GWRC report on the hydrology and meteorology of the 2004 event provided guidance (Figure 6-2), with spatial rainfall zones developed based on rainfall banding as per the 2004 report. The recorded rainfall profiles at the gauges in Table 6-2 were applied to the model using the spatial rainfall zones shown in Figure 6-3. Due to a lack of suitable rainfall data, the Mangaroa River at Tasman profile was applied to spatial rainfall zones 2 and 3 to ensure a more conservative approach (greater total rainfall depth). However, significant uncertainty remains around rainfall distribution within the catchment.

The 2004 model simulation scenario included modified cross sections on the lower reaches of the Waiwhetū downstream of the Bell Road bridge as discussed in Section 5.2. The three largest pumping stations at Opahu Stream, Randwick Road (Seaview Roundabout), and Parkside Road were removed as they were either not built or not complete during the February 2004 event.

Table 6-2 Total rainfall depths for the gauges applied to the February 2004 simulation.

Gauge	Start	End	Total Depth (mm)	Spatial Rainfall Zone
Birch Lane	15/02/2004 00:00	17/02/2004 00:00	218.5	1
Mangaroa River at Tasman	15/02/2004 00:00	17/02/2004 00:00	180.5	2 and 3
Shandon Golf Club	15/02/2004 00:00	17/02/2004 00:00	87.5	4

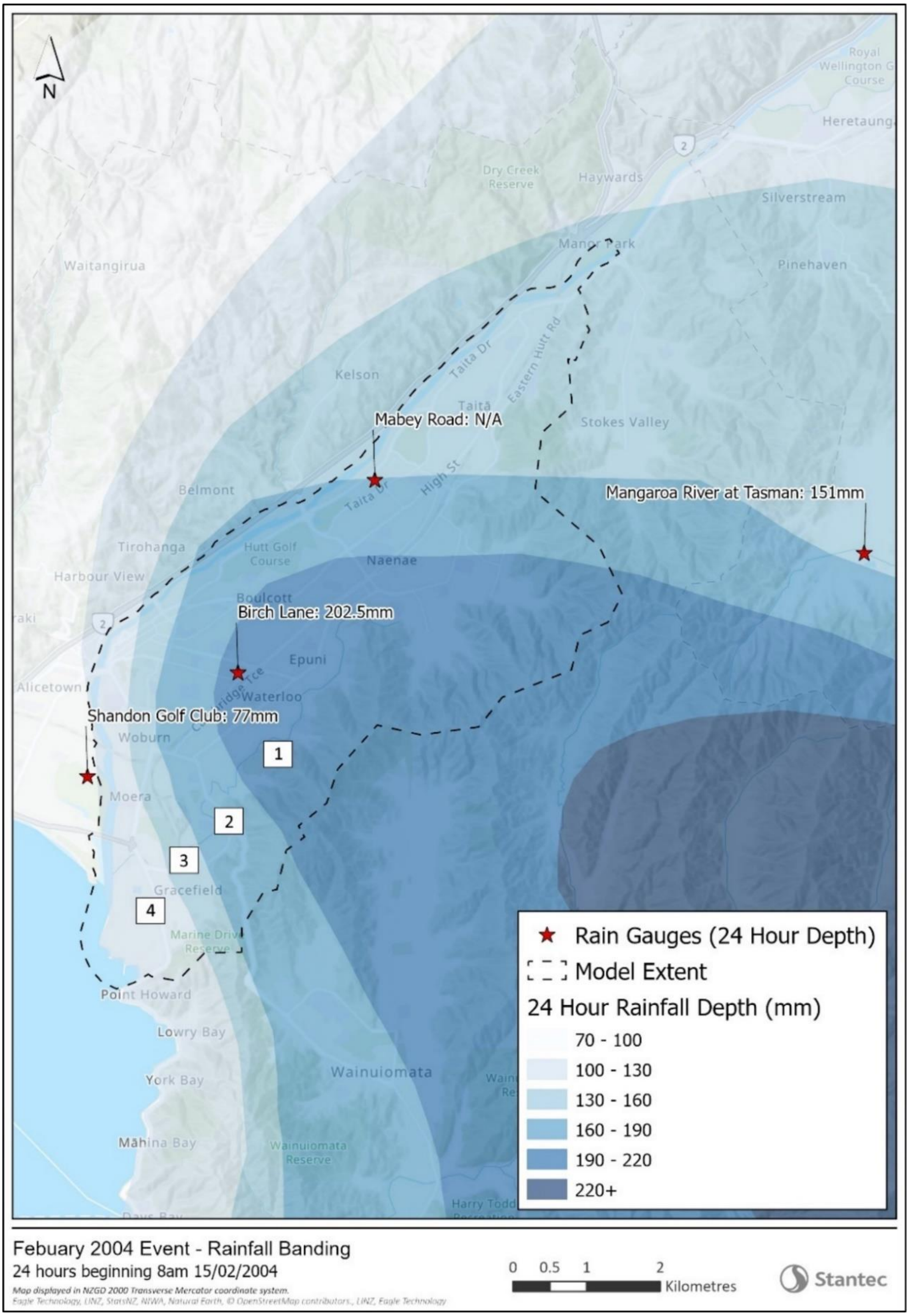


Figure 6-3 Rainfall banding as applied to the 2004 model simulation (spatial rainfall zones 1, 2, 3 and 4) and based on Figure 6-2 from Watts and Gordon (2004).

6.1.2 Comparison to Observed Data

The available observed flooding information included aerial imagery, photos taken on the ground, and 34 flood level observations recorded by GWRC. These flood observations are compared with model simulation results from Figure 6-5 to Figure 6-14. An overview of all data available for validation is shown in Figure 6-4.

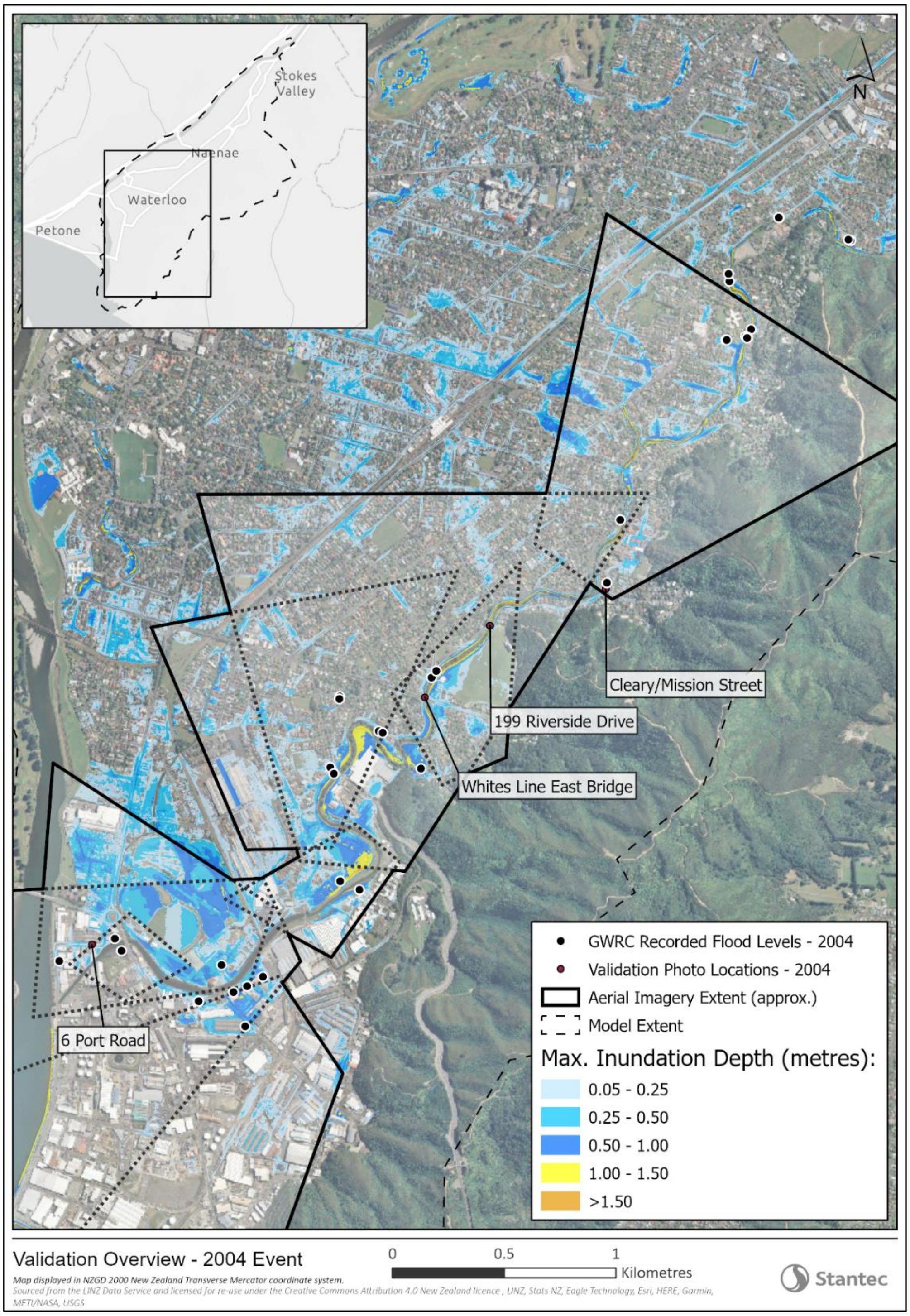


Figure 6-4 Overview of data used for validation of the February 2004 event.

6.1.2.1 Aerial Imagery

Aerial imagery of flooding extents was captured during the February 2004 event. Figure 6-5 to Figure 6-8 provide examples of key areas where flood extents are well captured (see Figure 6-4 for the full extent). The imagery was captured after the peak although the exact timing is uncertain.

The aerial imagery generally correlates well with modelled flood extents at Hutt Park, Whites Line East, and the Gracefield/Seaview area. Flood extents do appear to be slightly overestimated around Riverside Drive in Figure 6-6, however due to uncertainty around the timing of the aerial imagery some inconsistencies are to be expected. The imagery provides a useful qualitative indication that the model provides a reasonable representation of observed flooding extents.

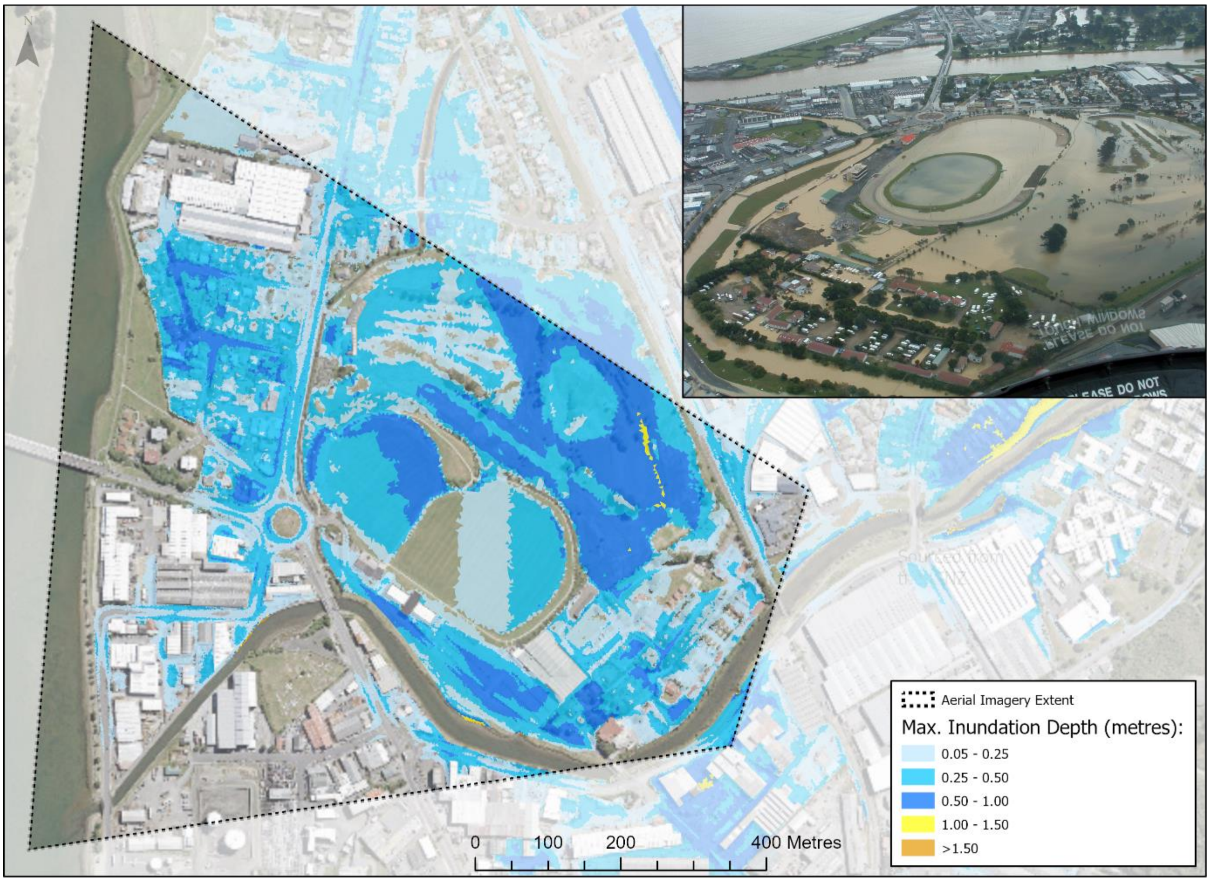


Figure 6-5 Hutt Park aerial imagery and approximate extent in relation to model simulation results.

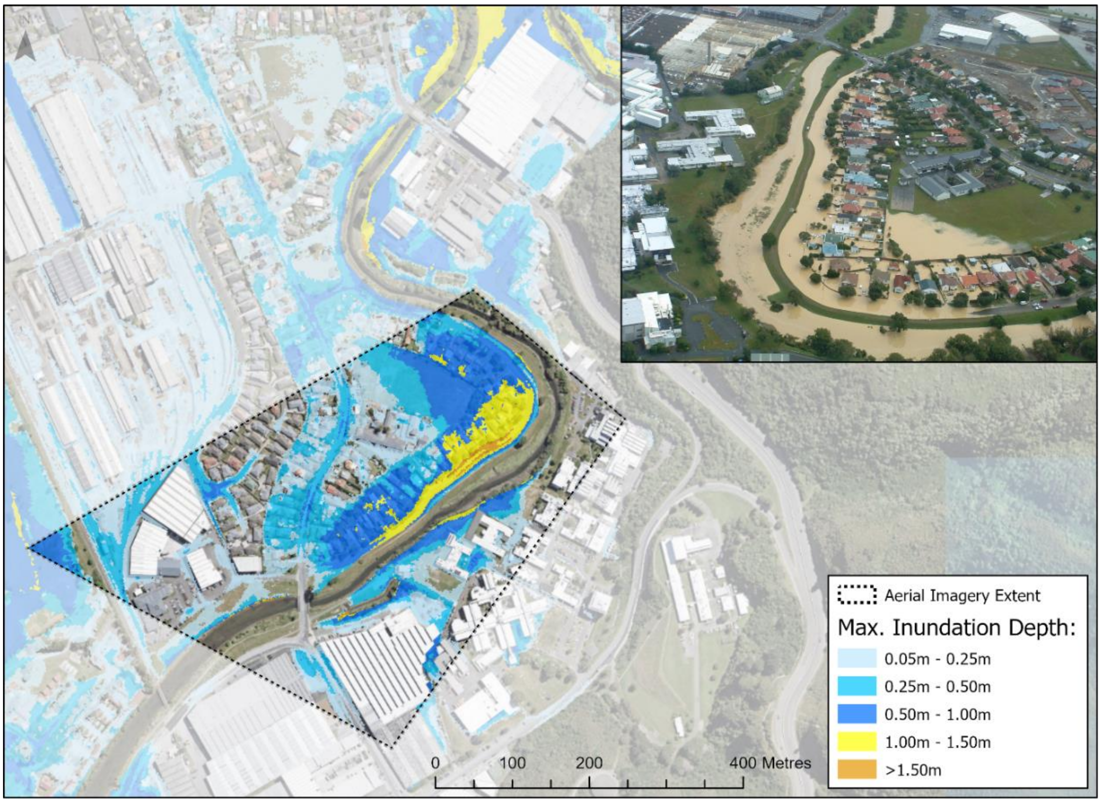


Figure 6-6 Aerial imagery of Riverside Drive upstream of the Bell Road bridge and approximate extent in relation to model simulation results.

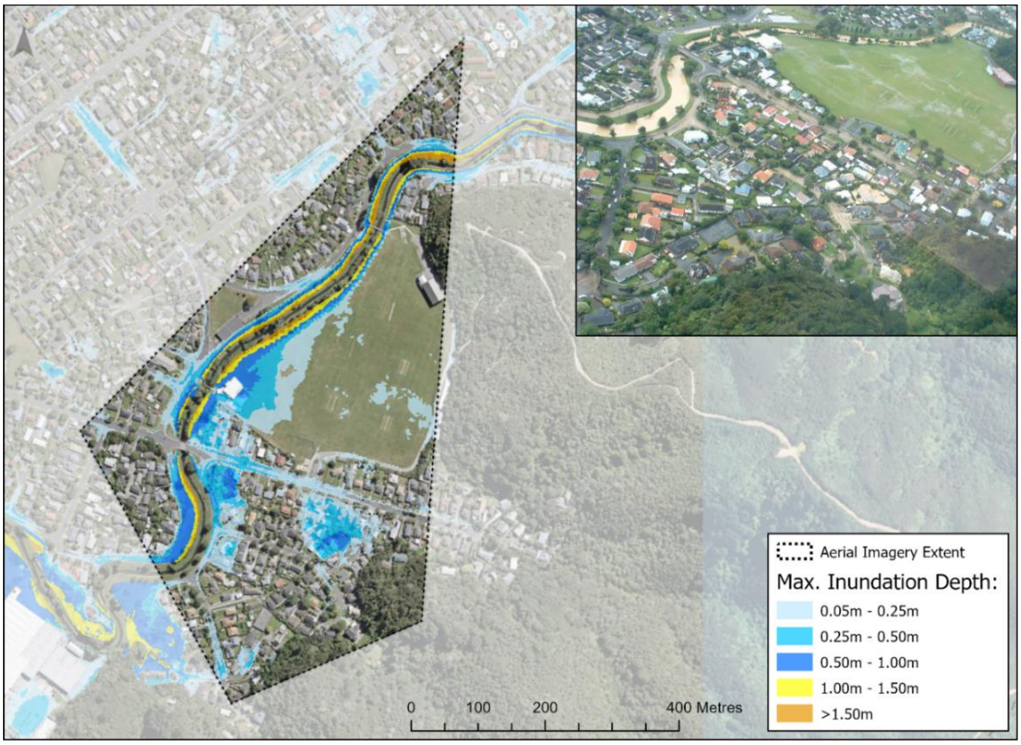


Figure 6-7 Aerial imagery of Te Whiti Park, the Whites Line East gauge and approximate extent in relation to model simulation results.

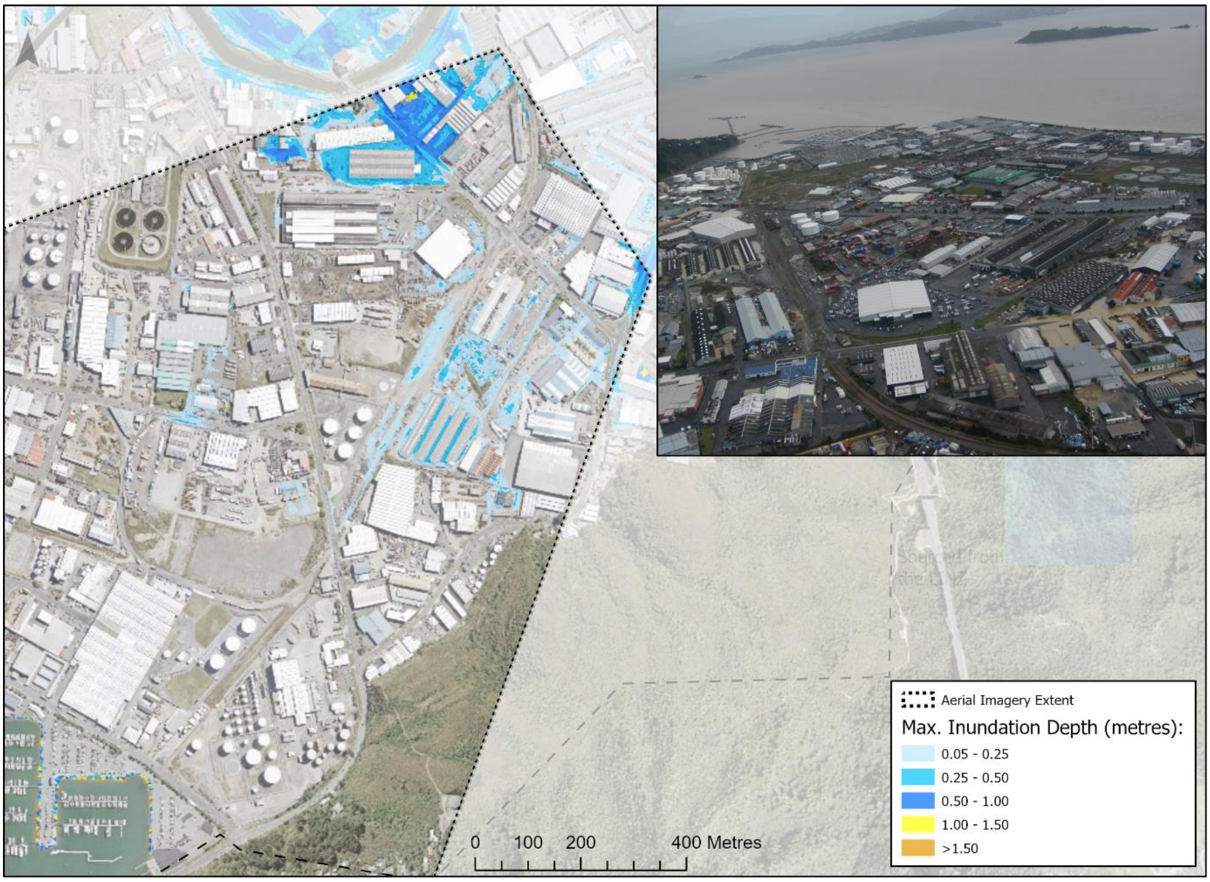


Figure 6-8 Seaview/Gracefield Aerial Imagery and approximate extent in relation to model simulation results.

6.1.2.2 Site Photos

Four photos were available showing flooding at various points along the Waiwhetū Stream, allowing more detailed comparison between observed and modelled flooding when compared to the aerial imagery. These photos are presented below with comparisons to modelled flood extents and cross sections.

6 Port Road

Figure 6-9 Photo of flooding facing west at 6 Port Road combined with model simulation extents.

Flooding was observed across the entire width of Port Road as shown in Figure 6-9. Although estimation of depths from the photo is challenging, the modelled depth of between 0.05m and 0.50m along Port Road appears to be consistent with observed flooding. The model does not predict full width inundation of Port Road closer to the Hutt River, however it is not possible to confirm inundation extent on Port Road close to the Hutt River using the available photo. Inundation in this location is assumed to have occurred due to overbank flow from the Waiwhetū Stream, which is consistent with observation in the community that channel capacity was an issue in this location.

Whites Line East Bridge

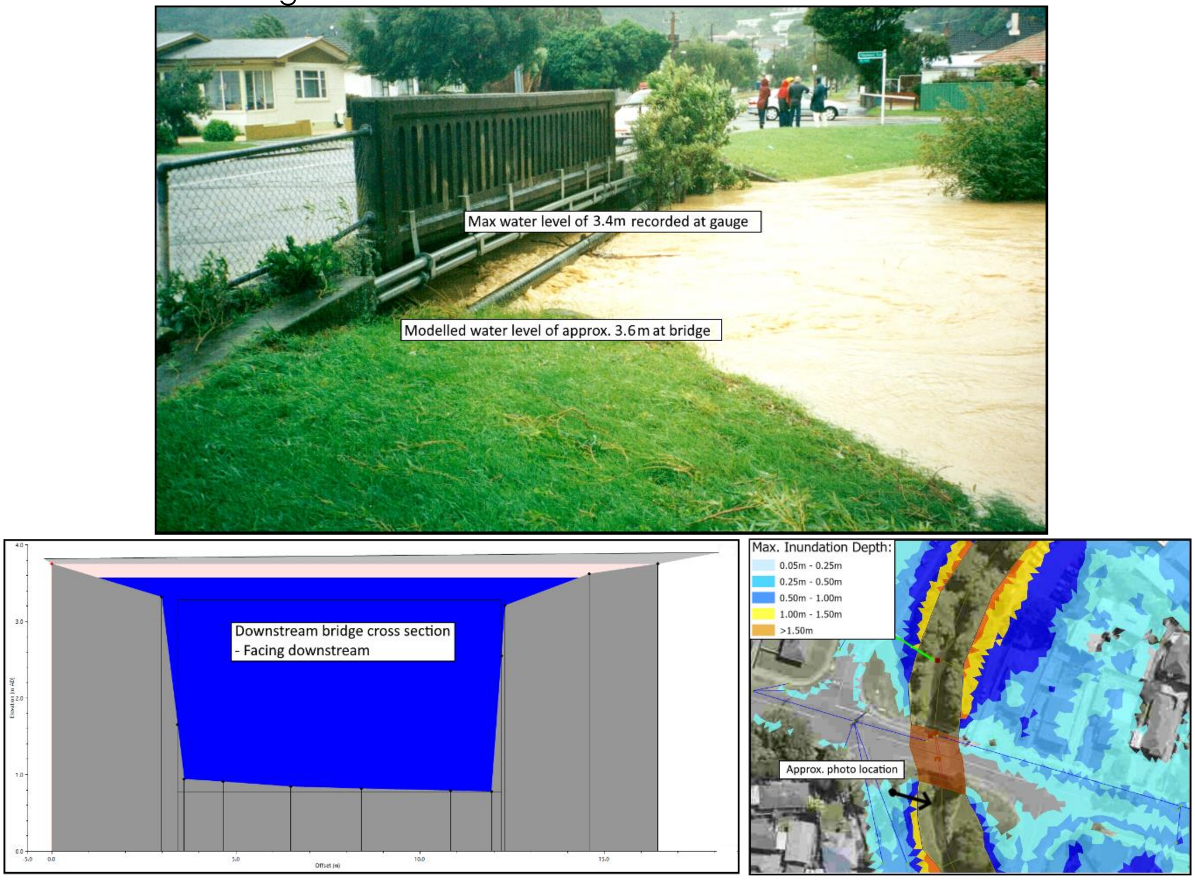


Figure 6-10 Photo downstream of the Whites Line East bridge, model simulation extents and cross section.

Figure 6-10 shows the downstream side of the Whites Line East bridge close to conveyance capacity (the gauge is on the upstream side). Modelled water level is approximately 3.6m compared to the peak of 3.4m recorded at the gauge, however it is unclear whether any bank overtopping occurred at the gauge to account for this difference. No flooding is visible from the photo, however there is a slight rise in Whites Line East heading east from the stream crossing which may be obscuring view of surface flooding evident in the aerial imagery (Figure 6-7). Timing of the photo is also unclear but is likely to be after the 06:35 peak flow.

199 Riverside Drive

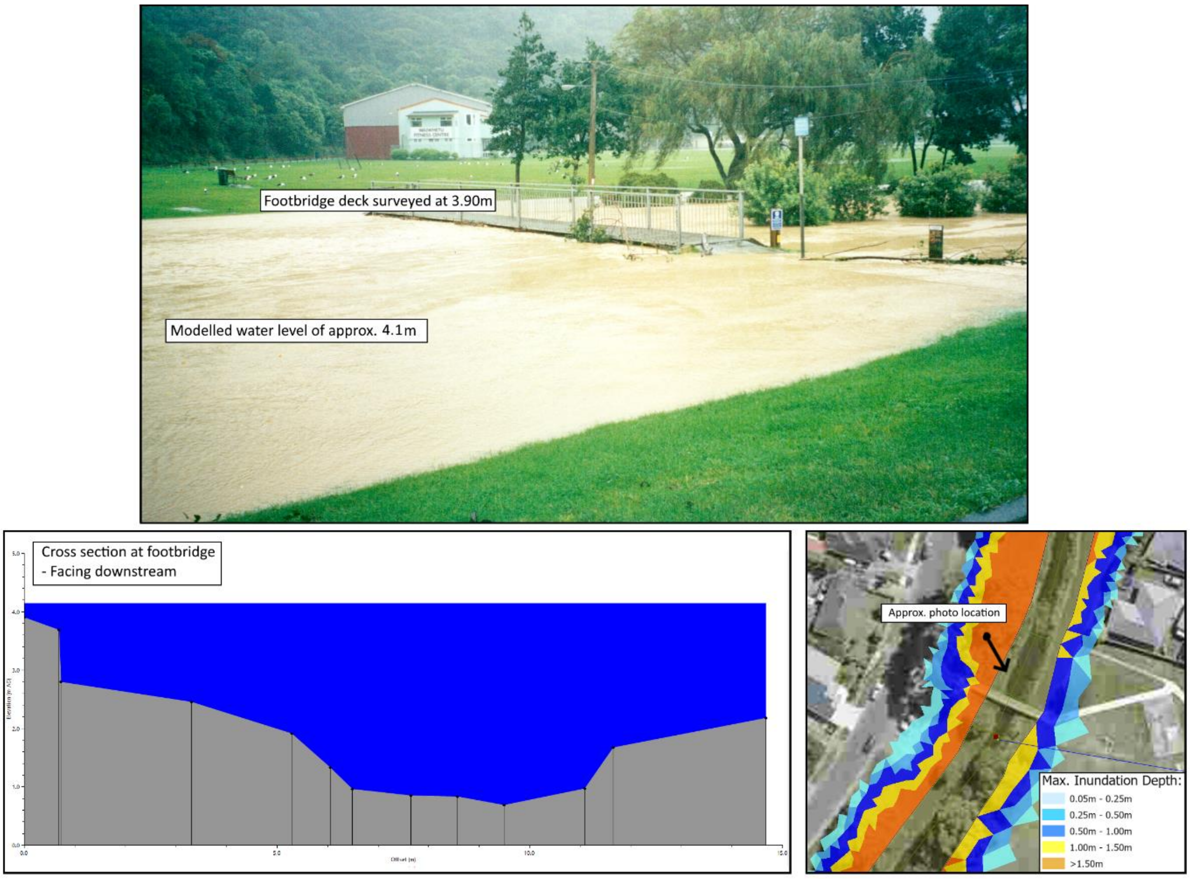


Figure 6-11 Photo of flooding, model simulation extents and cross section at the 199 Riverside Drive footbridge crossing the Waiwhetū Stream.

Flood extents observed at the footbridge crossing the Waiwhetū Stream close to 199 Riverside Drive appear to be consistent with modelled extents as shown in Figure 6-11. There is no indication of inundation of properties or the road in this location, which is consistent with the model results. Peak water level was modelled as approximately 4.1m at the footbridge, and the footbridge deck was surveyed at 3.9m. As with the other photos, timing is uncertain but likely to be after the peak.

Cleary Street-Mission Street Junction

Figure 6-12 Photo of observed and modelled flooding extents at the Cleary Street-Mission Street Junction.

Flooding extents shown in Figure 6-12 indicate that the modelled and observed flood extents are consistent. Depths are hard to estimate but as extent is constrained by topography, it is assumed that observed and modelled depths are similar. The timing of this photo is also unknown and likely to be post-peak, however as levels recede this location is separated from the Waiwhetū Stream and therefore surface flooding is more likely to persist after the peak stream level.

6.1.2.3 GWRC Recorded Flood Levels

GWRC provided 34 recorded flood levels sourced from debris marks, flood levels in properties, etc. These levels allow for a detailed, quantitative comparison between modelled and observed flood levels. Figure 6-13 provides an overview of all recorded flood levels and the difference between modelled and observed levels, Figure 6-14 provides a more detailed view where differences are greater than 0.25m. For further details around the GWRC recorded flood levels see Appendix J – InfoWorks ICM Bridge Parameters (GWRC Recorded Flood Levels (2004)).

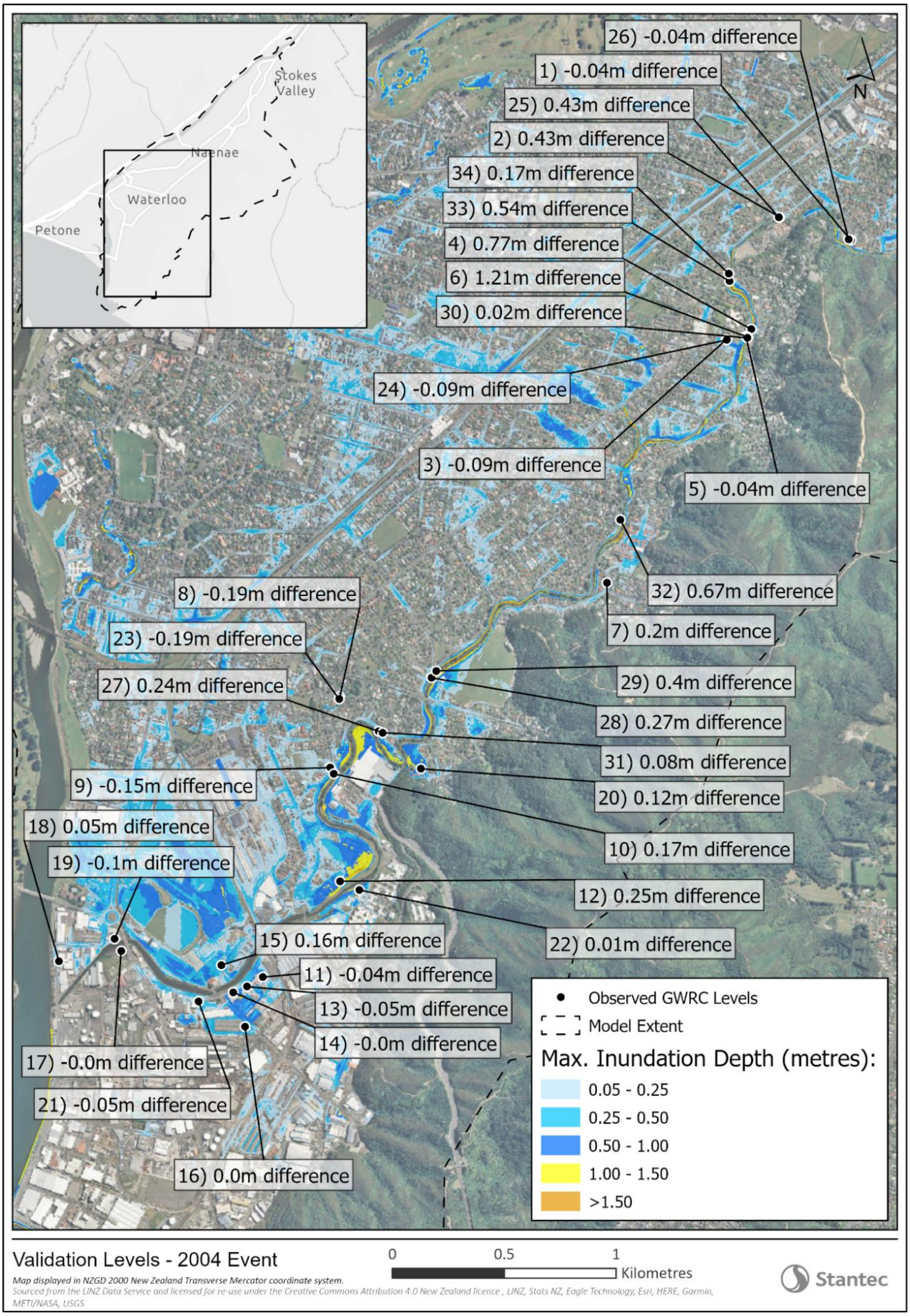


Figure 6-13 Overview of the difference between flood levels recorded by GWRC and model simulation levels.

Over 70% of locations have an observed/modelled difference of less than 0.25m, and all differences greater than 0.25m are overestimated, rather than underestimated in the model. This indicates that observed/modelled levels are generally consistent, but where there are more significant differences the modelled levels are more conservative. As shown in Figure 6-14, all locations with larger differences are on the boundary of 1-D river reaches. This may explain the larger differences as where banks are not fully represented in the 1-D river reaches, transitions in slope may not be well captured by the 2-4m^2 mesh triangles.

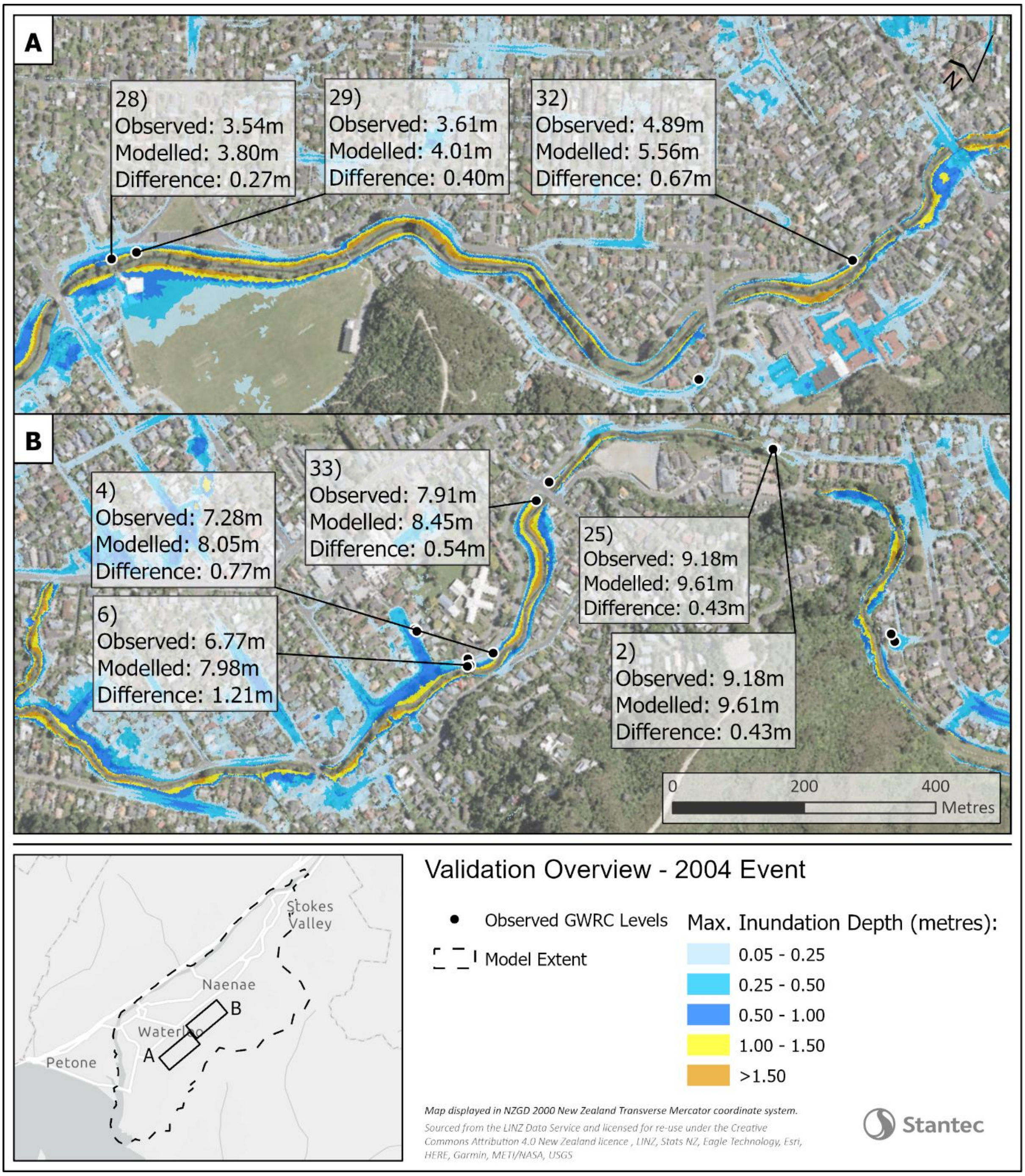


Figure 6-14 Detailed view of locations with a GWRC recorded flood level/model simulation level difference of greater than 0.25m.

6.2 14-15 NOVEMBER 2016 RAINFALL EVENT

The 2016 event began in the evening of November 14th, with a severe weather warning issued for strong northerly winds and heavy rain. Based on the flood frequency analysis of the Whites Line East gauge data the recorded peak of 18.0 m^3s^{-1} represents an ARI of approximately 10 years. As shown in Figure 6-1, this event corresponds well to the projected frequency distribution and provides confidence in the predicted 10-year ARI.

6.2.1 Model Inputs

Tide level data from the Queens Wharf gauge was applied to the model representing the tidal boundary condition in Wellington Harbour as per the methodology described in Section 3.4.3. The recorded flow of the Hutt River at the Taitā Gorge gauge was applied as an inflow at the northern end of the model, allowing the Hutt River to be modelled as a boundary condition with a peak flow of 1057 m^3s^{-1}.

Table 6-3 Summary of key model inputs.

Key Model Data	
Start	14/11/2016 14:00
End	15/11/2016 23:00
Rainfall	Catchment averaged rainfall radar
Tide	Level at Queens Wharf (GWRC)
Inflow	Hutt River flow at Taitā Gorge (GWRC)
Waiwhetū at Whites Line East peak	15/11/2016 10:50
Hutt at Taitā Gorge peak	15/11/2016 13:50
High Tide at Queens Wharf	14/11/2016 15:25, 15/11/2016 04:55, 15/11/2016 17:20

Catchment averaged rainfall radar profiles were available for the 2016 event; provided by WWL via the Mott MacDonald Moata interface. The Moata interface provides rainfall profiles generated by averaging recorded depths from a 500m-by-500m grid across large catchments. The three catchments from the Moata interface which cover the ELH model are shown in Figure 6-15. Table 6-4 shows the meta data of the three profiles used.

The profiles are linked to generalised catchments that cover a large area, are averaged, and cannot be modified to better match each storm in question.

Table 6-4 Summary of rainfall radar data (November 2016).

Rainfall Radar Catchment	Start	End	Total Depth (mm)
Hutt	14/11/2016 14:00	15/11/2016 23:00	84.5
Waiwhetū	14/11/2016 14:00	15/11/2016 23:00	91.4
Avalon	14/11/2016 14:00	15/11/2016 23:00	90.4

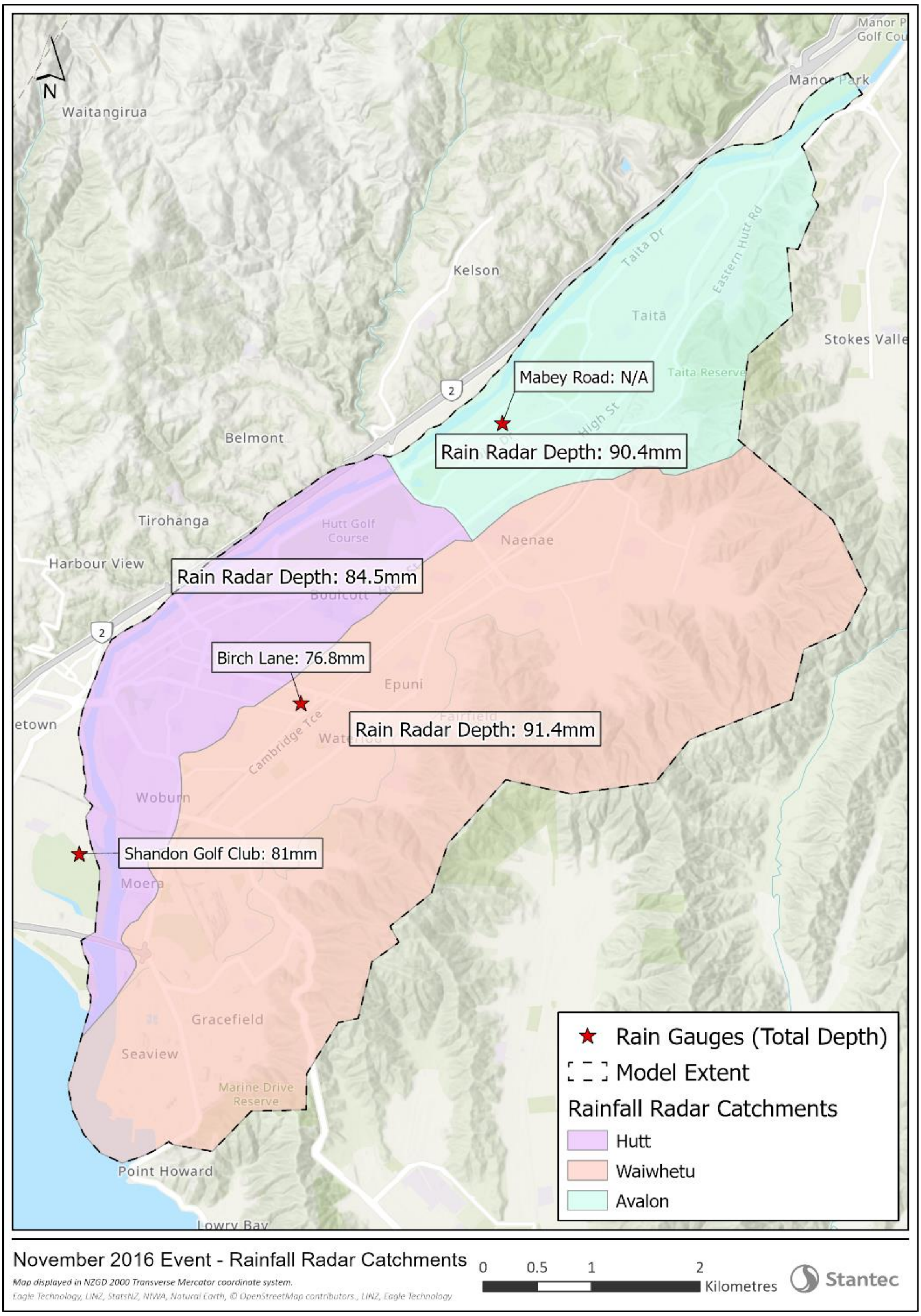


Figure 6-15 Rainfall radar catchment overview.

6.2.2 Comparison to Observed Data

Validation data is limited to photos provided by GWRC and sourced from various news websites, as shown in Figure 6-16. This data is for the most part focussed on the Waiwhetū Stream, the main source of observed flooding during the November 2016 event.

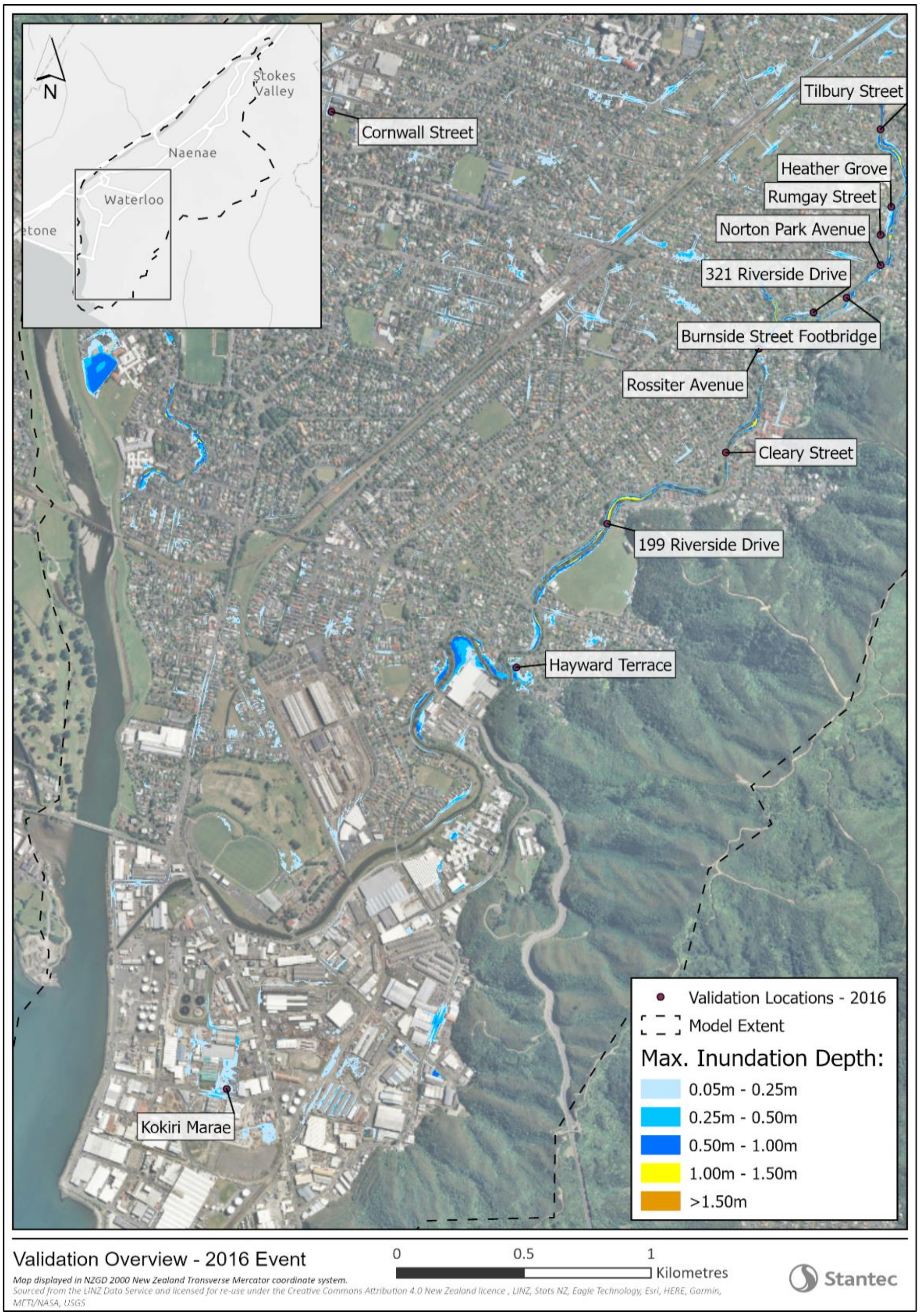


Figure 6-16 Overview of validation data (November 2016)

6.2.2.1 Site Photos

Twelve photos were available showing flooding at various locations in the Eastern Lower Hutt catchment, with the majority along the Waiwhetū Stream. As presented below, these allow for detailed comparison between observed and modelled stream levels and flooding extents.

Kokiri Marae

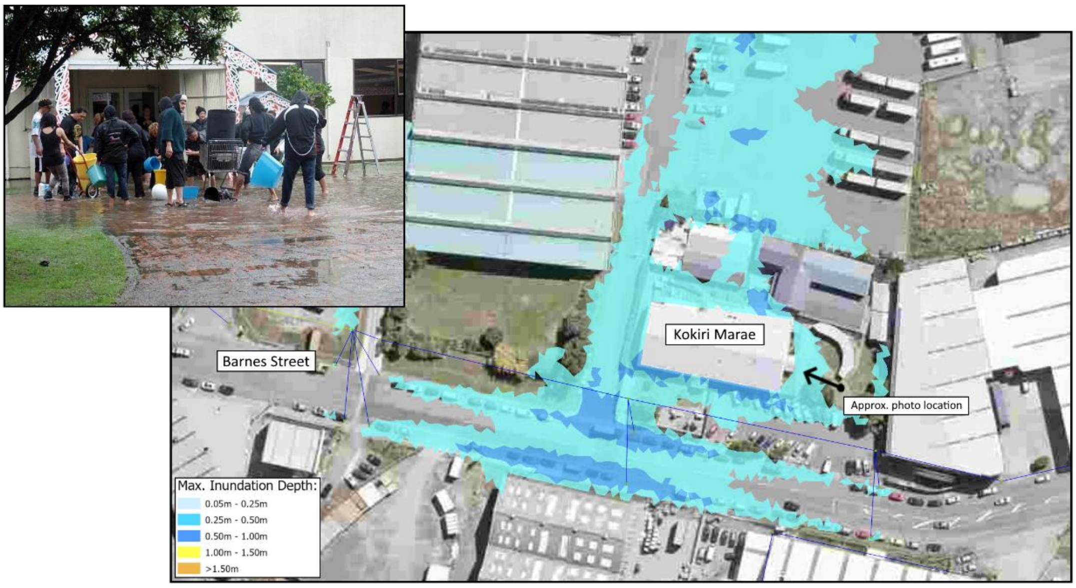


Figure 6-17 Observed and modelled flood extents at Kokiri Marae.

The extents are consistent between observed and modelled flooding in Figure 6-17, however no detailed information is available for the observed flood depths or levels. Flooding around the entrance of Kokiri Marae does appear to be consistent with a modelled depth of no greater than 0.25m. Time of capture for the photo is unknown, however it is assumed to be close to the peak water level of mid to late morning.

Hayward Terrace

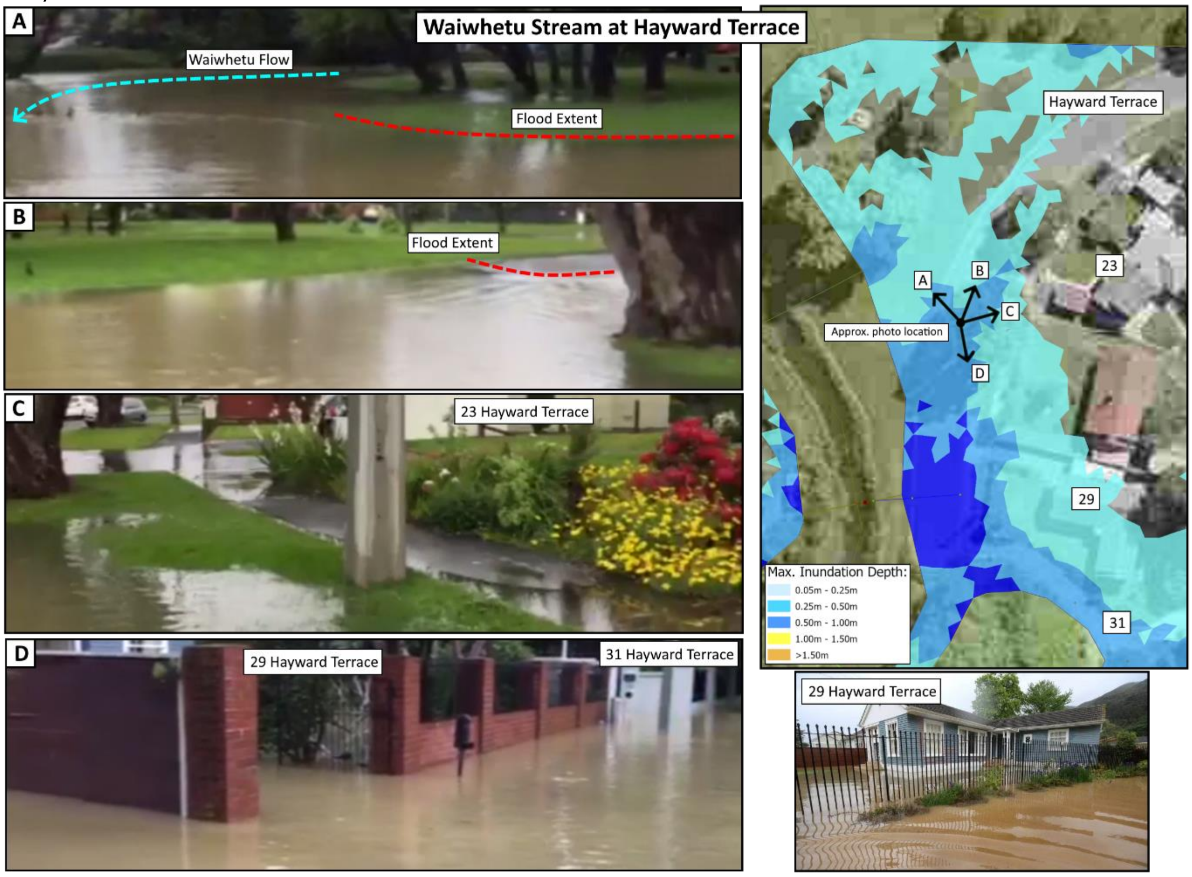


Figure 6-18 Observed and modelled flood extents on Hayward Terrace.

Observed flooding around 23 to 31 Hayward Terrace is shown in Figure 6-18. Timing of the images is unknown but is assumed to be close to the peak of mid-late morning. Flood extent relative to inundation of properties and the road appears to be consistent between modelled and observed flooding. Observed levels and depths are unknown, however they are also assumed to be consistent with modelled levels as like flood extents they are dependent on local topography.

199 Riverside Drive

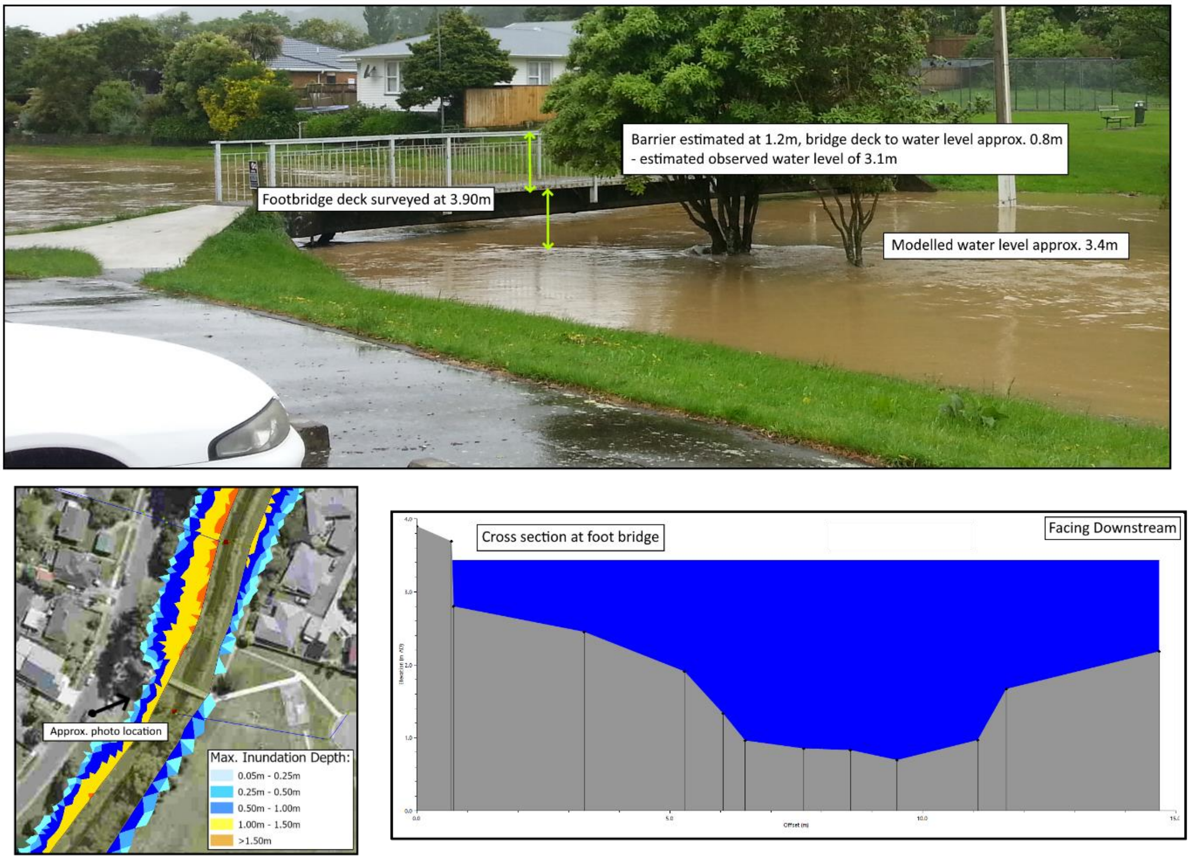


Figure 6-19 Observed flood extents, modelled flood extents and cross section at 199 Riverside Drive.

The photo shown in Figure 6-19 was captured at 12.45pm, slightly after the mid-late morning peak identified in the Waiwhetū Stream at the Whites Line East gauge. No overbank flow is observed or predicted by the model simulation at 199 Riverside Drive, however a footbridge crossing the Waiwhetū Stream provides a reference point to assist in estimation of stream level. Observed stream level is estimated at 3.1m, and modelled level is approximately 3.4m. Observed/modelled difference is approximately 0.3m.

Cleary Street Bridge

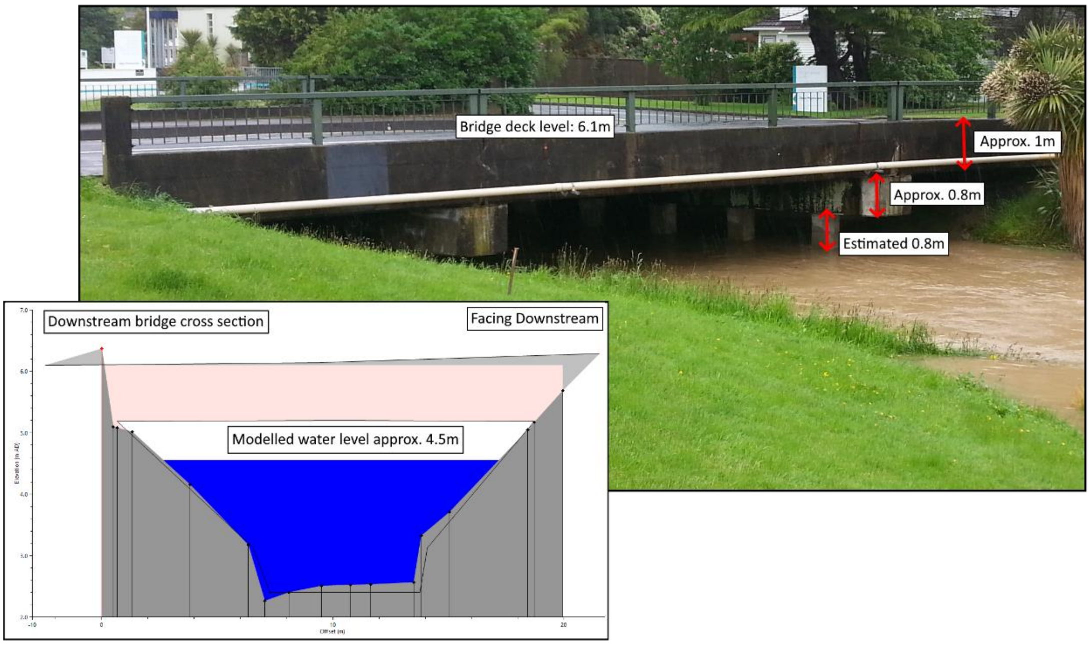


Figure 6-20 Observed stream level and modelled cross section at the Cleary Street Bridge.

The Cleary Street bridge photo shown in Figure 6-20 was captured at 12.40pm. Although this was after the peak in the Waiwhetū Stream, no significant drop in stream level is evident as no debris line is visible. The bridge deck allows stream level to be estimated at 3.5m, approximately 1m below the peak modelled stream level at this location. This may be a result of the observed stream level having dropped more than is apparent in the photo.

Rossiter Avenue Bridge

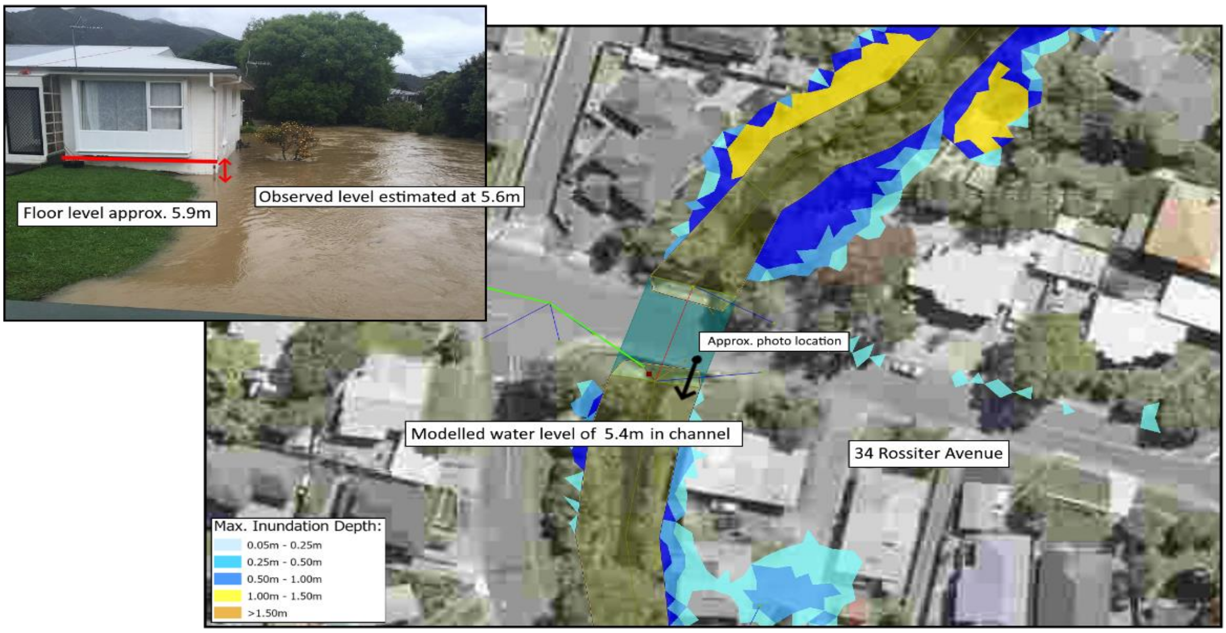


Figure 6-21 Observed and modelled water level at the Rossiter Avenue Bridge.

Time of capture for the photo shown in Figure 6-21 is unknown but is assumed to be close to the mid-late morning peak stream level in the Waiwhetū Stream. Observed stream level is estimated at 5.6m adjacent to 34 Rossiter Avenue using LiDAR data and the photo, and modelled stream level is approximately 5.4m in the channel. This indicates an observed/modelled difference of approximately 0.2m, suggesting stream levels in this location are reasonably consistent between observations and the model simulation.

321 Riverside Drive

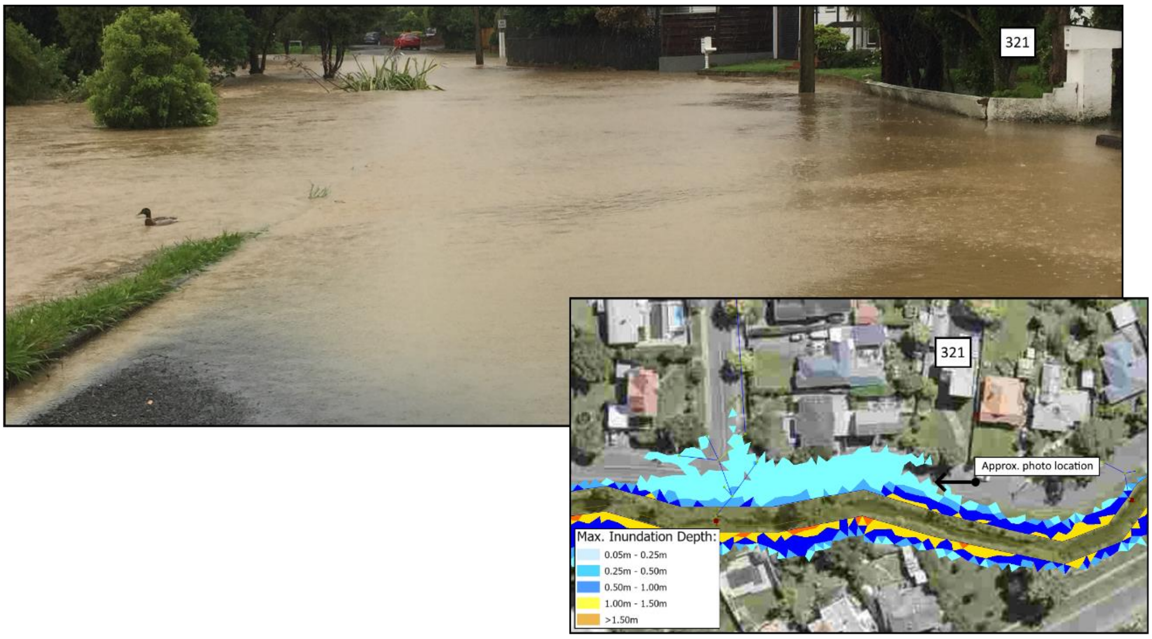


Figure 6-22 Observed and modelled flood extents at 321 Riverside Drive.

The photo in Figure 6-22 was captured at 12.05pm, close to the mid-late morning peak level in the Waiwhetū Stream. Observed and modelled flood extents appear to be consistent, however there is no data available for observed depths or levels in this location. Consistency between flood extents suggests that observed flooding is well represented in the model simulation.

Burnside Street Footbridge

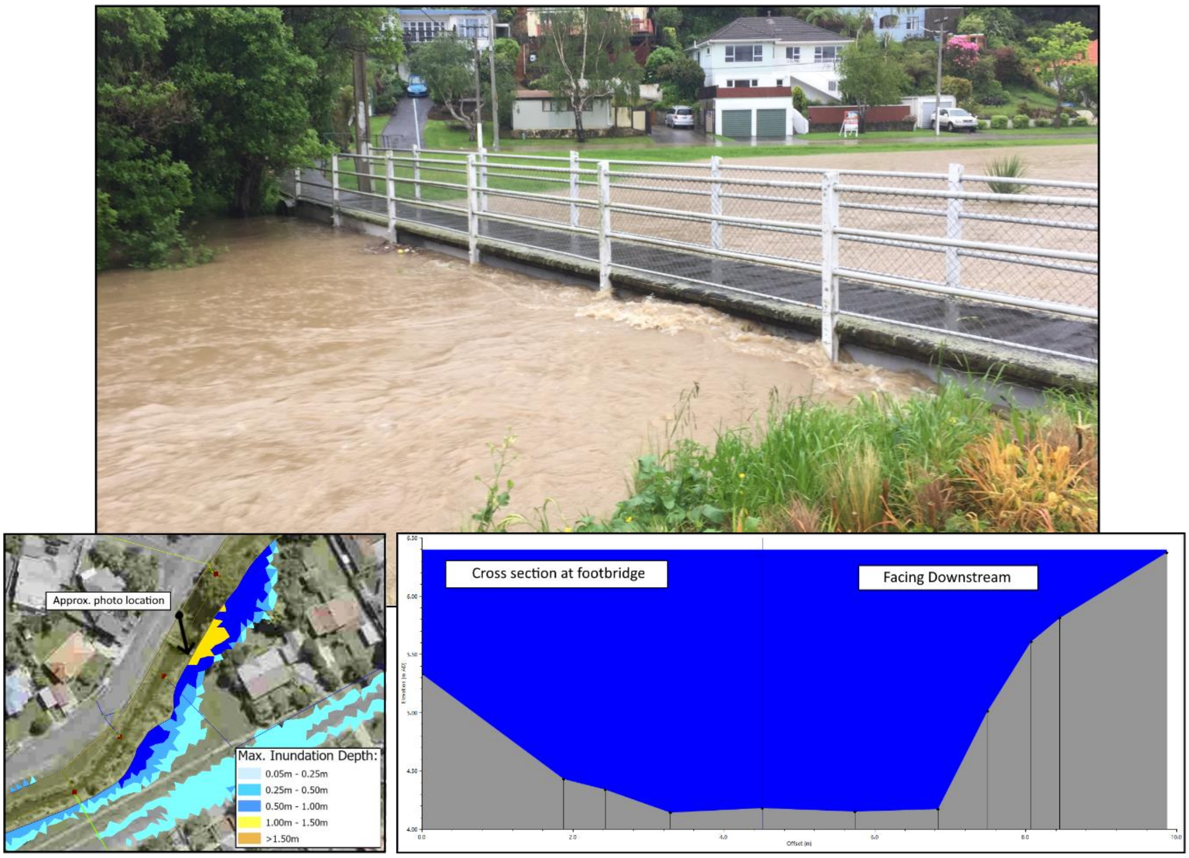


Figure 6-23 Observed and modelled stream levels at the Burnside Street footbridge.

Figure 6-23 includes a photo captured at 9.30am, very close to or before the mid-late morning peak level of the Waiwhetū Stream. Flood extents appear to be consistent between observations and model simulation results. Inundation of the road but not in the properties is both observed and modelled. Observed stream level is estimated at 6.0m using the footbridge deck, and modelled stream level is approximately 6.3m. This difference of approximately 0.3m indicates that modelled and observed stream levels are reasonably consistent.

Norton Park Avenue Bridge

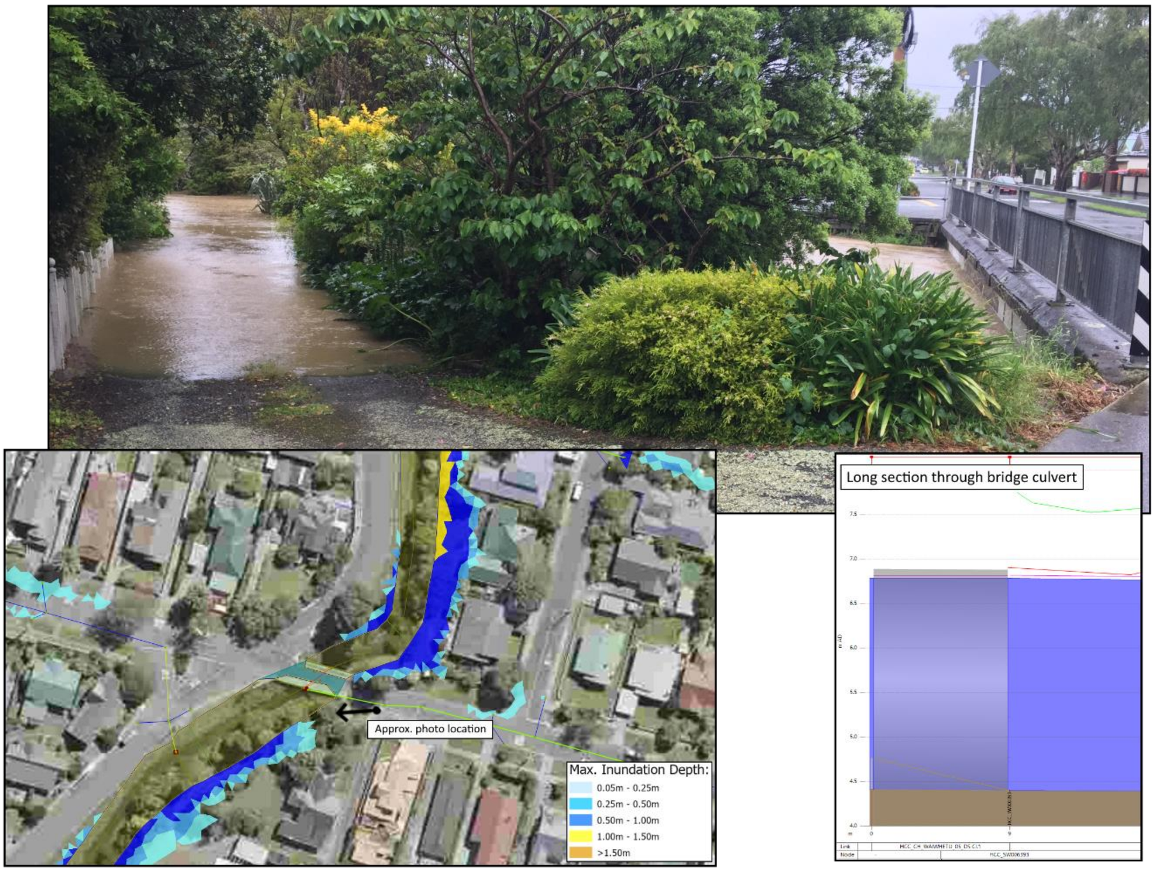


Figure 6-24 Observed and modelled stream levels at the Norton Park Avenue bridge.

The photo of the downstream side of the Norton Park Avenue bridge in Figure 6-24 was taken at 9.25am, close to or before the Waiwhetū Stream peak. Extents are consistent as no overbank flow was observed or modelled. The bridge is observed to be at or close to capacity and is also close to capacity at the peak in the model simulation.

Rumgay Street-Riverside Drive Junction

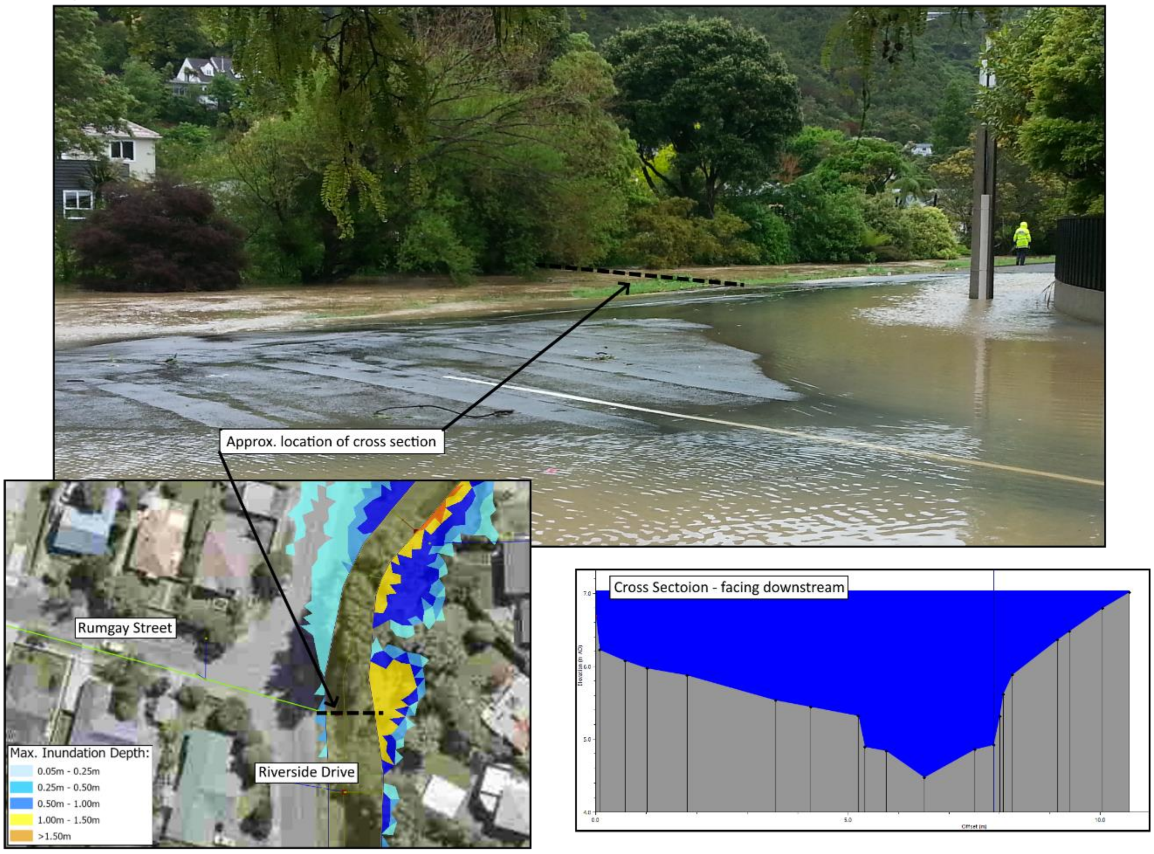


Figure 6-25 Observed flooding extents, and modelled flooding extents and cross section at the Rumgay Street-Riverside Drive junction.

The photo in Figure 6-25 was taken close to or after the Waiwhetū Stream peak at 12.10pm. There are some inconsistencies between observed and modelled flood extents, however the modelled channel is shown to be at bank full capacity by the channel cross section. Extents in this location may be particularly sensitive to slight inaccuracies in elevation due to removal of trees or buildings from the ground model, and incomplete representation of the surface due to the 2-D mesh element size of 2-4m². Observed and modelled levels in the Waiwhetū Stream appear to be consistent.

Heather Grove-Riverside Drive Junction

Figure 6-26 Observed and modelled flood extents at the Heather Grove-Riverside Drive junction.

The timing of the photo in Figure 6-26 is unknown but is assumed to be close to the mid-late morning peak level in the Waiwhetū Stream. Flood extents appear to be fairly consistent between observations and the model simulation, however there may be some minor under-representation of flooding in the model. No level data is available.

Tilbury Street Bridge

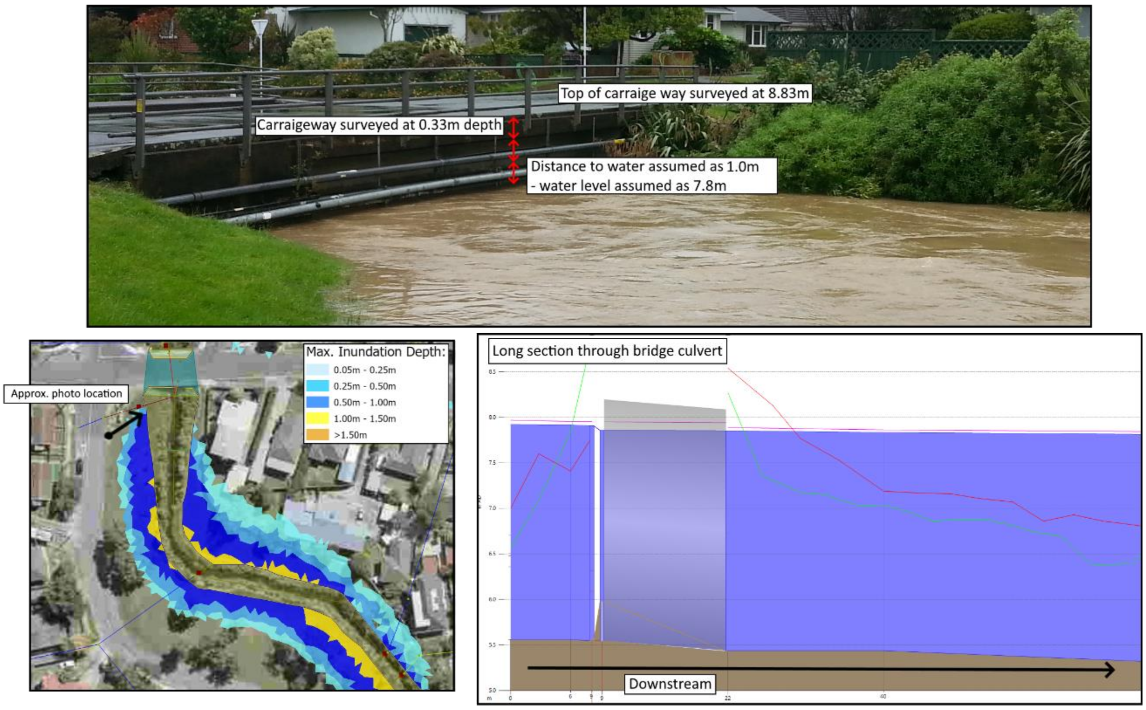


Figure 6-27 Observed and modelled stream levels at the Tilbury Street bridge (downstream).

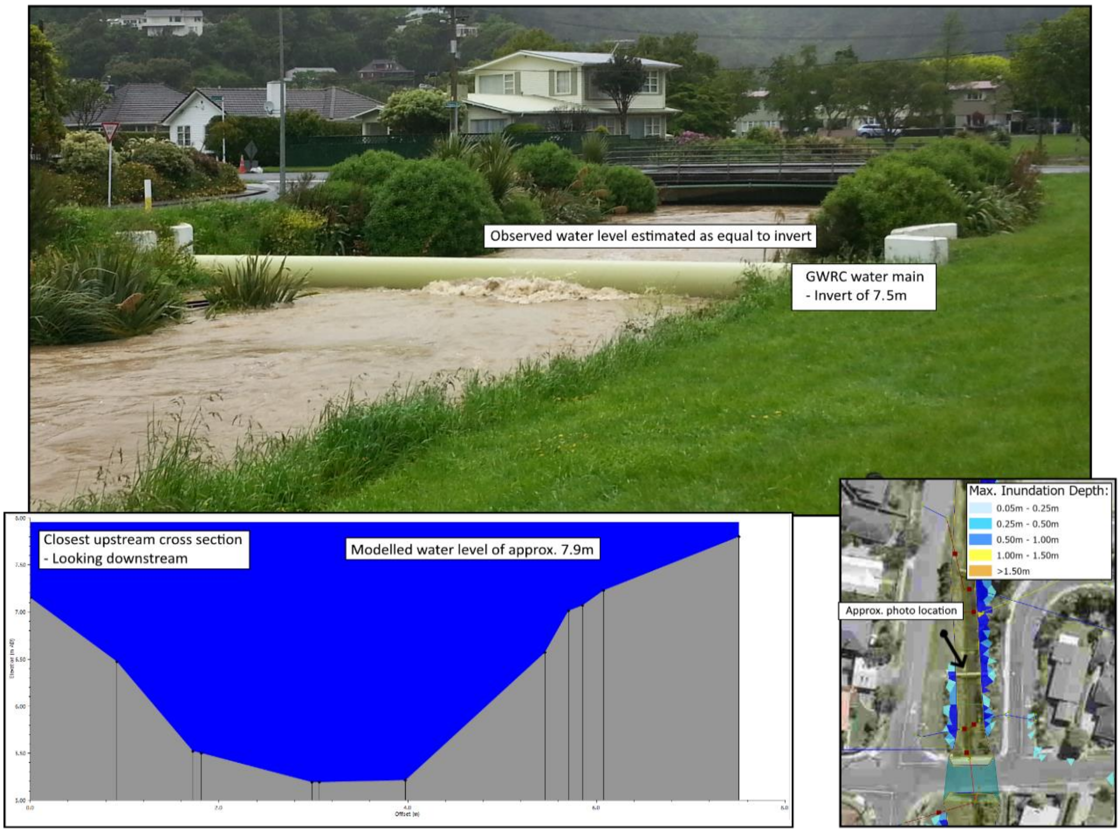


Figure 6-28 Observed and modelled stream levels at the Tilbury Street bridge (upstream).

The photos in Figure 6-27 and Figure 6-28 were captured at 12pm, close to or after the mid-late morning peak level in the Waiwhetū Stream. Observed and modelled flood extents appear to be consistent in both figures. At the pipe crossing upstream of the bridge there is a difference of approximately 0.4m between observed and modelled stream levels. However observed and modelled stream levels on the downstream side of the bridge are approximately equal at 7.8m, indicating reasonable confidence in modelled stream levels.

Cornwall Street

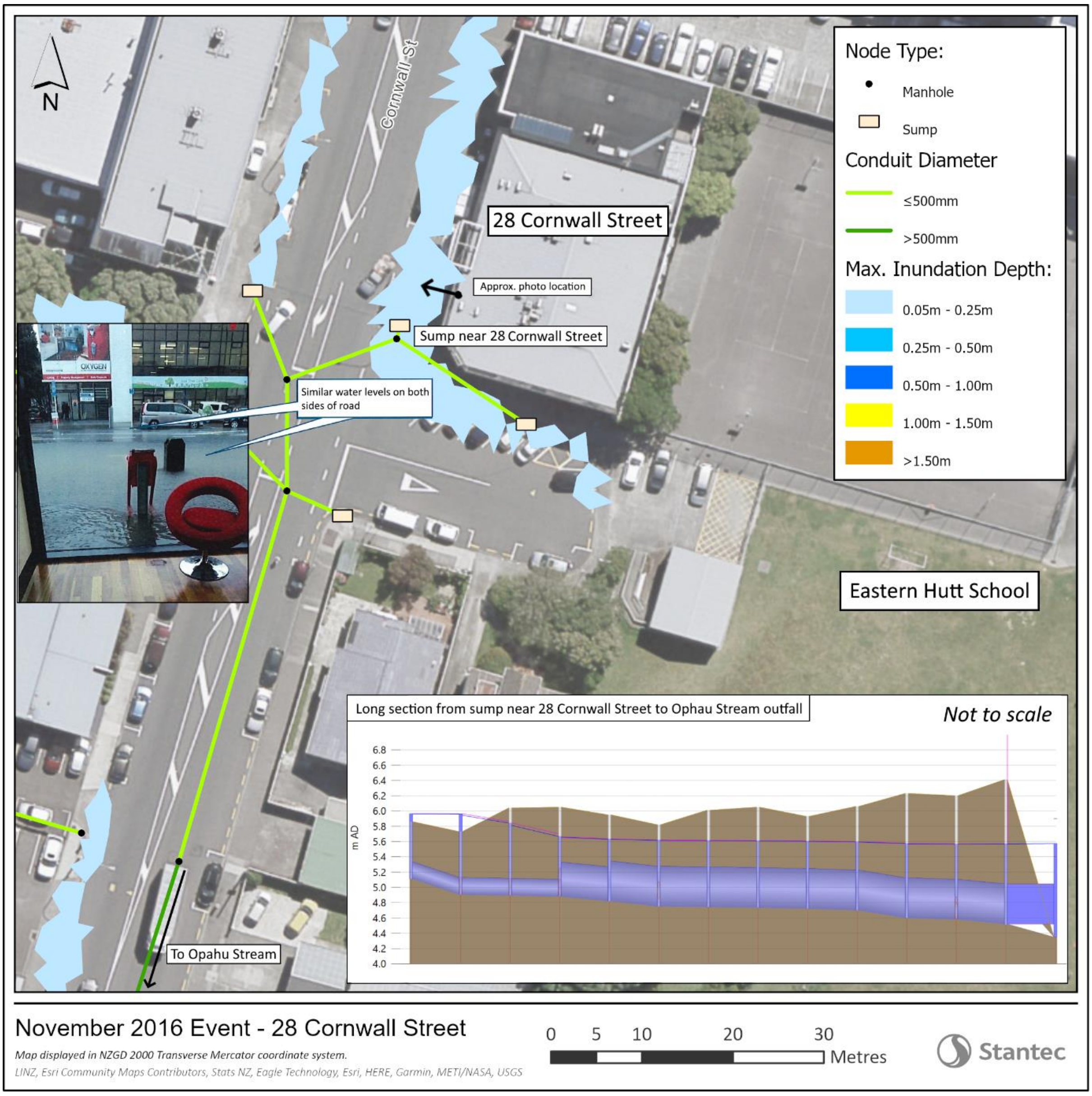


Figure 6-29 Observed and modelled flooding at 28 Cornwall Street, Lower Hutt

Flooding at and around 28 Cornwall Street was one of the only locations in central Lower Hutt with recorded flooding during the November 2016 event (excluding areas where blockages were identified as the likely cause). As shown in Figure 6-29, the road was partially inundated and there was potential for flooding to enter property at 28 Cornwall Street. The stormwater main was confirmed to be clear of debris during the event, but the stormwater network in this location is identified as sensitive to level at the outfall to the Opahu Stream (Donnelly, Pitchforth, & Telfer, 2017).

Modelled and observed flood extents appear to be consistent, and the long section in Figure 6-29 demonstrates that capacity of the stormwater system is limited by level in the Opahu Stream. However, depths may be slightly underestimated. This can be accounted for as stormwater collected by private assets at Eastern Hutt School is pumped into the public stormwater network to an unknown manhole close to 28 Cornwall Street (Donnelly, Pitchforth, & Telfer, 2017). This additional inflow is not accounted for in the current hydraulic model.

6.3 LIMITATIONS AND ASSUMPTIONS

The validation exercise provides a high-level understanding of the performance of the Eastern Lower Hutt stormwater model. Validation based on the February 2004 and November 2016 events is subject to several limitations:

- Limited data is available for validation. Significant uncertainty is present regarding the time many of the photos were captured, and the timing of peak flooding.
- No actual flood levels were obtained for the November 2016 event (excluding level at the Whites Line East gauge). Validation of the November 2016 event is based on comparisons between photos and model simulation results.
- No consideration has been given to the functionality of structures, including bridges, screens and the losses associated with these.
- Telemetry data from pumping stations has not been considered as part of validation.
- Spatial distribution of rainfall is assumed to be adequately represented by the 2016 rainfall radar data and 2004 rainfall banding.
- No allowance for antecedent conditions were made due to the agreed runoff method used, where only initial abstraction and continuing losses are applied.

6.4 SUMMARY

Validation of the February 2004 event with the available information indicates that flooding is consistent between observations and the model simulation results. It is characterised by the following:

- Aerial imagery and site photos indicate that extents are generally consistent between observed and modelled flooding. Where there are inconsistencies, the modelled extents tend to be slightly overestimated indicating that the model is more conservative in its replication of observed flooding.
- Where estimations of observed stream levels are made, the model simulation results tend to slightly overestimate stream levels. This supports the model as more conservative in its replication of observed flooding.
- Comparisons between modelled levels and the recorded flood levels provided by GWRC indicate that in most locations, modelled and observed levels are consistent. At all locations where the difference is greater than 0.25m the model overestimates level, further supporting the model more conservative in its replication of observed flooding.

The November 2016 event validation is characterised by:

- Site photos indicate that extents are consistent between observed and modelled flooding. Several exceptions have been identified along the Waiwhetū Stream; however, these can be accounted for by potential inaccuracies in representation of channel banks.
- Numerous estimations of level within the Waiwhetū Stream are made. In general, they indicate the model either well replicates or overestimates observed flood levels.
- Recorded flooding data is limited for the central Lower Hutt area; however, the available data indicates that the modelled and observed flood extents are similar. Potential underestimates of level can be accounted for by additional inflow from private reticulation.

Both validation events demonstrate a reasonable level of confidence that the model well replicates observed flooding. Most inconsistencies identified between observed and modelled flooding are slight extent or level overestimations within the model. The model can therefore be considered to provide a comparable to slight overestimation in its replication of observed flooding.

7.0 SENSITIVITY AND FREEBOARD

7.1 SENSITIVITY ANALYSIS

Stantec was engaged by WWL in 2021 to undertake a model sensitivity analysis to understanding the sensitivity of the stormwater network to a range of different scenarios, as follows:

- 100-year existing climate, with a 50% increase rainfall.
- All inlets are fully blocked. This includes both inlets from the eastern rural catchments and smaller culverts under roads along the Waiwhetū, Opahu and Awamutu streams.
- All inlets are partially blocked. This was assumed to be 50% blocked.
- No tidal boundary.
- No Hutt River inflow.
- All bridges blocked.
- All pumps 'off'.

The results of the sensitivity scenarios were examined and compared against a baseline 100-year with 20% increase in rainfall for climate change, see Figure 4-1, to inform the freeboard selection process. This baseline included the 2.1m tide (sea level rise) and a 10-year ARI Hutt River inflow (1345m^3s^{-1}) as per Section 3.4. At the request of WWL no climate change uplift was applied to the Hutt River inflow. Both WWL and GWRC were engaged in the scenario selection to understand the impact of different variables on flooding extents.

7.1.1 50% increase in rainfall

The total rainfall depth for a 100-year event in existing climate was increased by 50% and converted to a 12-hr nested rainfall profile for use in in this scenario. The results show that the model is very sensitive to the increase in rainfall, particularly in areas that already pond in the baseline scenario around open channels, as shown in Figure 7-1. Ponding significantly increases in areas where flow cannot drain due to network capacity, high levels in the Hutt River, and high tides. In particular, north of the Hutt Valley railway line close the Opahu Stream, within Avalon Park, and upstream of the Boulcott Golf Course club rooms. Additional ponding is also observed in Naenae Park likely due to downstream capacity of the Waiwhetū Stream, and across Moera due to the tidal boundary condition preventing outflow to the Hutt River and Wellington Harbour.

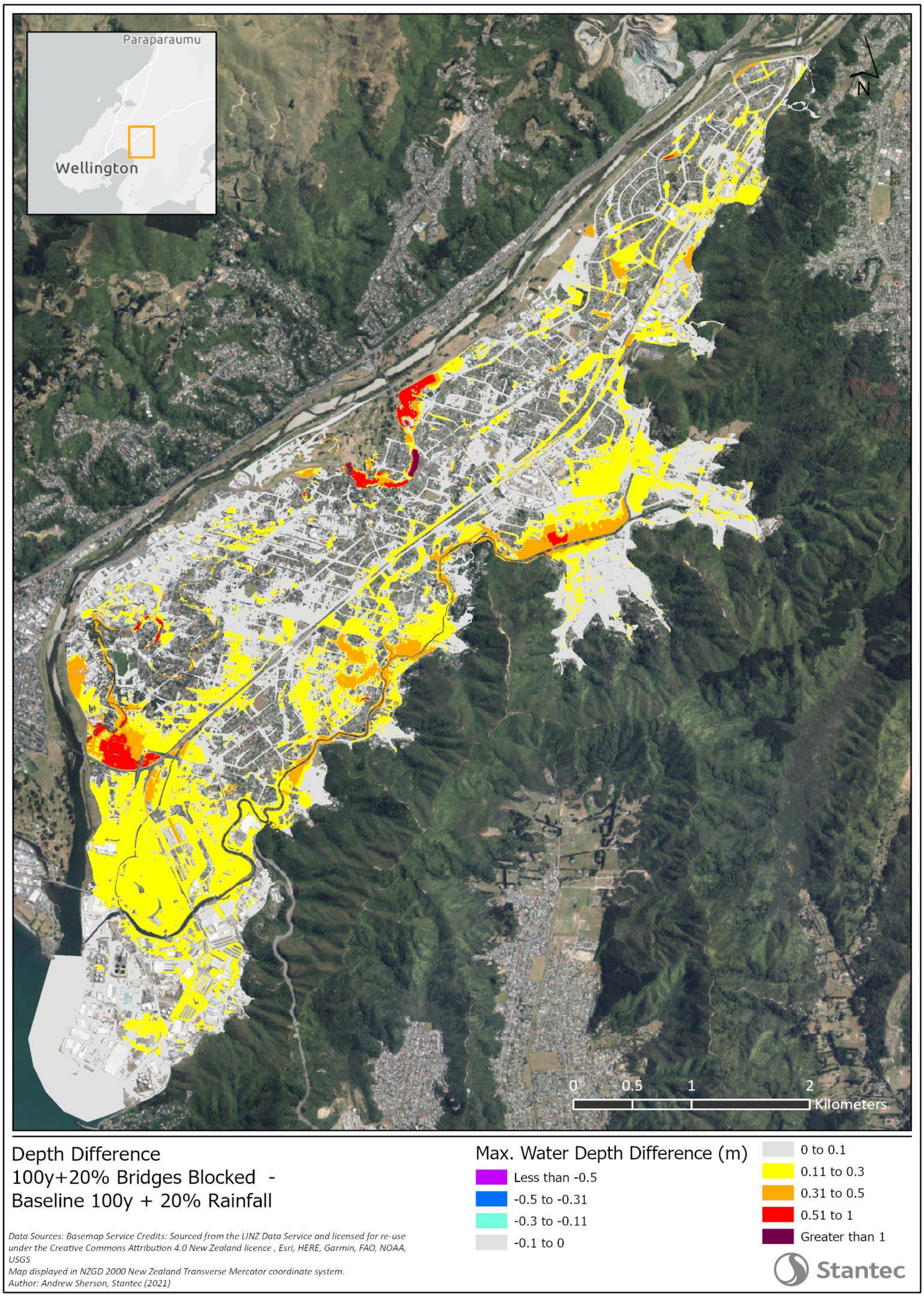


Figure 7-1 Difference in depth between the base scenario and a 50% increase in rainfall intensity

7.1.2 Inlets Fully Blocked

All inlets in the model were blocked by setting sediment levels in the pipes to 90% of the pipe height for pipes >200mm in diameter and 75% for pipes =<200mm in diameter. These sediment levels maintain the model's ability to simulate with a Priessman slot (which accounts for 10% of conduit space). A total of 168 pipes were blocked as part of this scenario.

The model is moderately sensitive to the blocking of inlets, particularly around the Opahu and Awamutu stream where conveyance is restricted by road culverts, see Figure 7-2. For example, ponding occurs behind the Hutt Valley railway line embankment due to blockages of large culverts. Flood depth increases also occur in the Waiwhetū Stream because of blockages of the smaller bridges (bridge spans less than 20m were modelled as culverts). The impacts of these blockages are fairly local with limited changes occurring along the northern and southern section's where the larger crossings are modelled as bridges and were not blocked in this scenario. In the Eastern Hills there are small, localised increases around the inlets. However, these changes in depth are not very large due 1) the inlets are already full and causing overland flow in a baseline 100-year event with climate change 2) any additional overland flow is shallow due to the gradients of the roads in the hills. Instead, the changes are seen where the water slows down and pools such as in Naenae and around the Waiwhetū stream. Increased ponding depths are also observed at the Hutt River stop bank in Naenae due to blockage of some culverts underneath the stop bank.

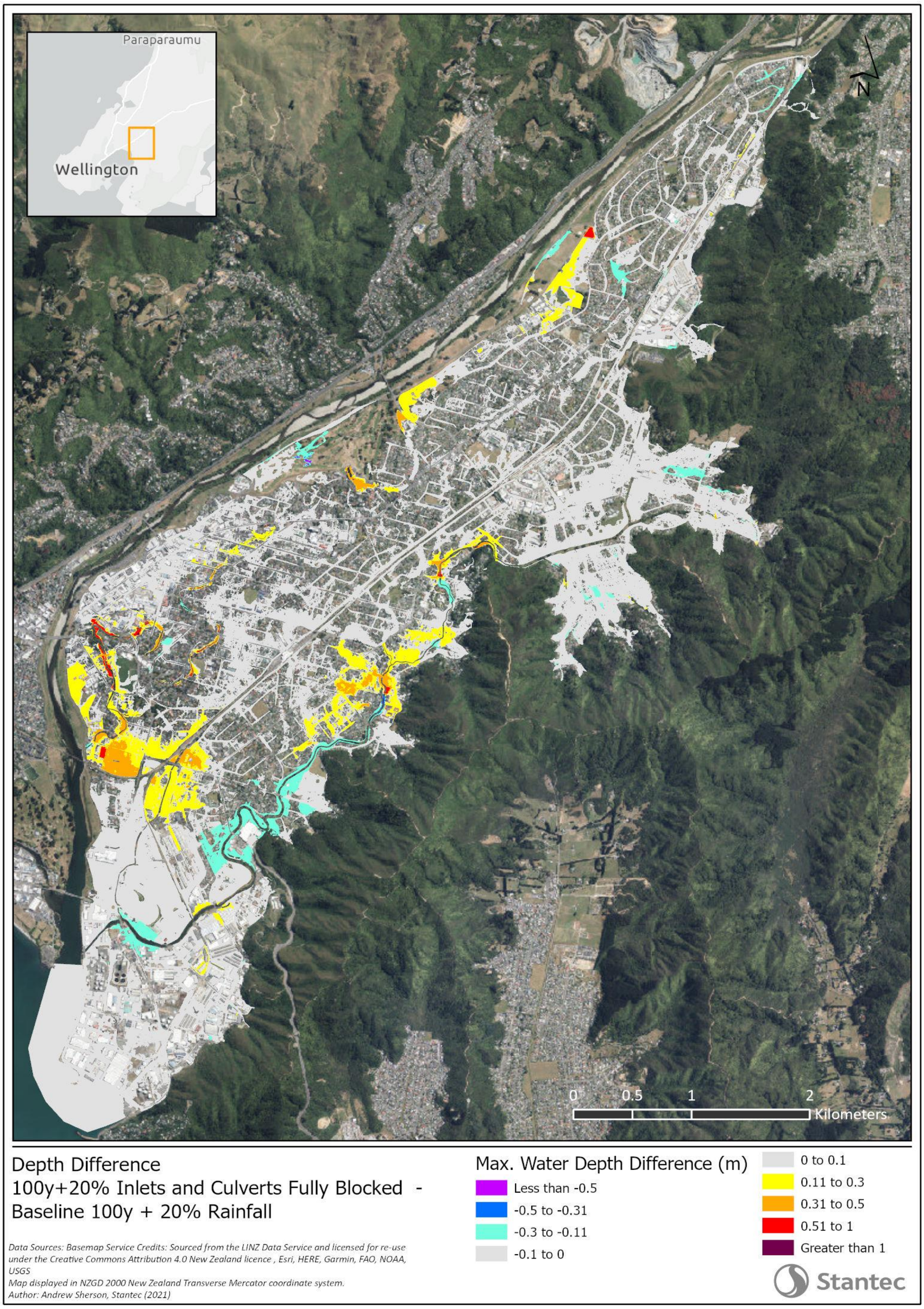


Figure 7-2 Difference in depth between the base scenario and a full blockage of all inlets and culverts

7.1.3 Inlets Partially Blocked

Similar to the scenario described in Section 7.1.2, all pipes downstream of river reaches, screens, outlets and culvert inlets were blocked, this time to 50% of the pipe height. Overall, partial blockage was applied to 168 pipes as part of this scenario.

The model shows the catchment is moderately sensitive to partial blockage of the inlets and culverts with increases in water depths occurring primarily around the Waiwhetū, Opahu, and Awamutu streams. Similar to the full blockage scenario, there are small, localised increases in flood depths around the inlets at the foot of the Eastern Hills, as shown in Figure 7-3. The full blockage locations that had the highest sensitivity are also impacted in the partially blocked scenario, with the largest increases in ponding observed behind the Hutt Valley railway line embankment. Ponding along the Hutt River stop bank is not observed to the same level as the full blockage scenario as the loss in pipe capacity is not as significant.

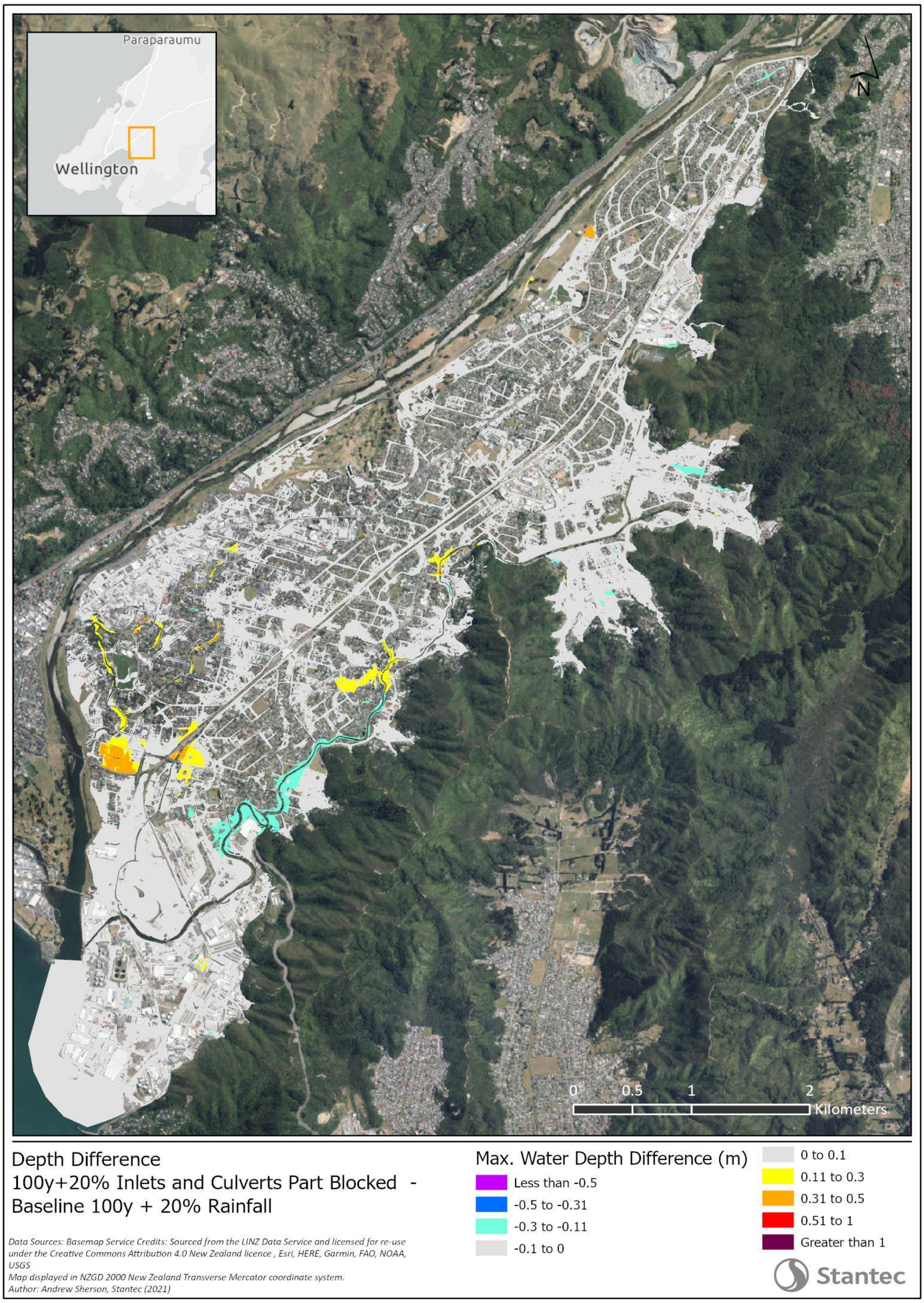


Figure 7-3 Difference in depth between the base scenario and a partial blockage of all inlets and culverts

7.1.4 No Tidal Boundary

The tidal level boundary was removed to understand the impact of sea level on flooding in the ELH catchment.

Results in Figure 7-4 show that southern section of the model around Moera and Seaview is very sensitive to tidal levels. This is due to the low elevation of this area, with large areas close to current sea level and below the 2.1m high tide applied to account for sea level rise. Sensitivity to tidal levels extends up the Awamutu Stream and Waiwhetū Stream as far as Whites Line East. However, tidal sensitivity does not extend further upstream. Some sensitivity is also observed in the Hutt Valley High School playing fields due to a reduced tailwater impact at the Hutt River stop bank.

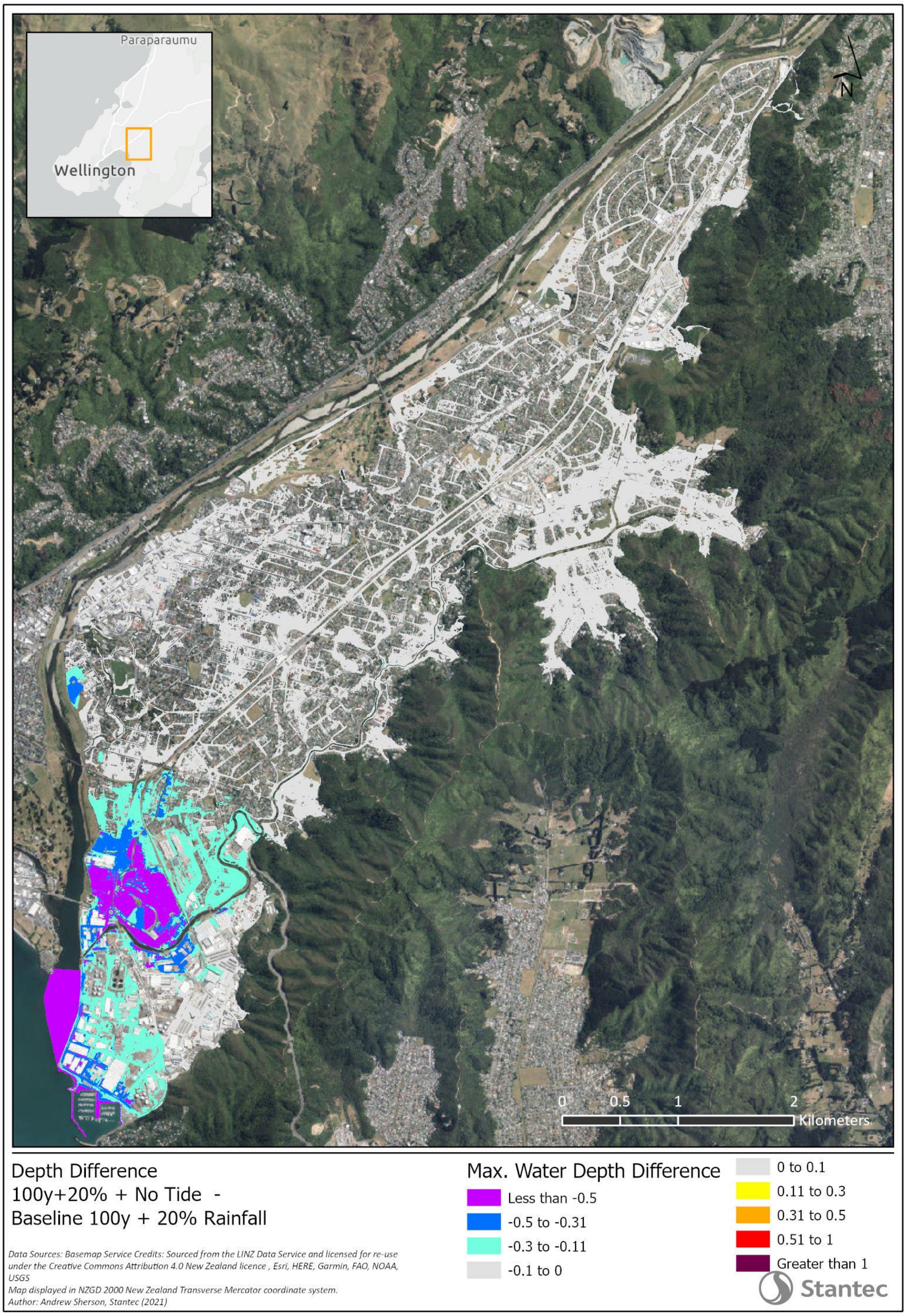


Figure 7-4 Difference in depth between the base scenario and the removal of the tidal boundary

7.1.5 No Hutt River Inflow

For this scenario, the Hutt River inflow was removed for the simulation to understand the impact of the Hutt River's on flooding in ELH.

The model is moderately sensitive to the Hutt River due to backwater effects on the pipe and channel networks, see Figure 7-5. The most significantly impacted areas of the model are those on alongside the river such as around the Boulcott Golf Course, and areas of along the edge of the stop banks. The region around the Opahu stream is moderately sensitive to levels in the Hutt River as the stream relies on a pump station when the Hutt River is too high to discharge to.

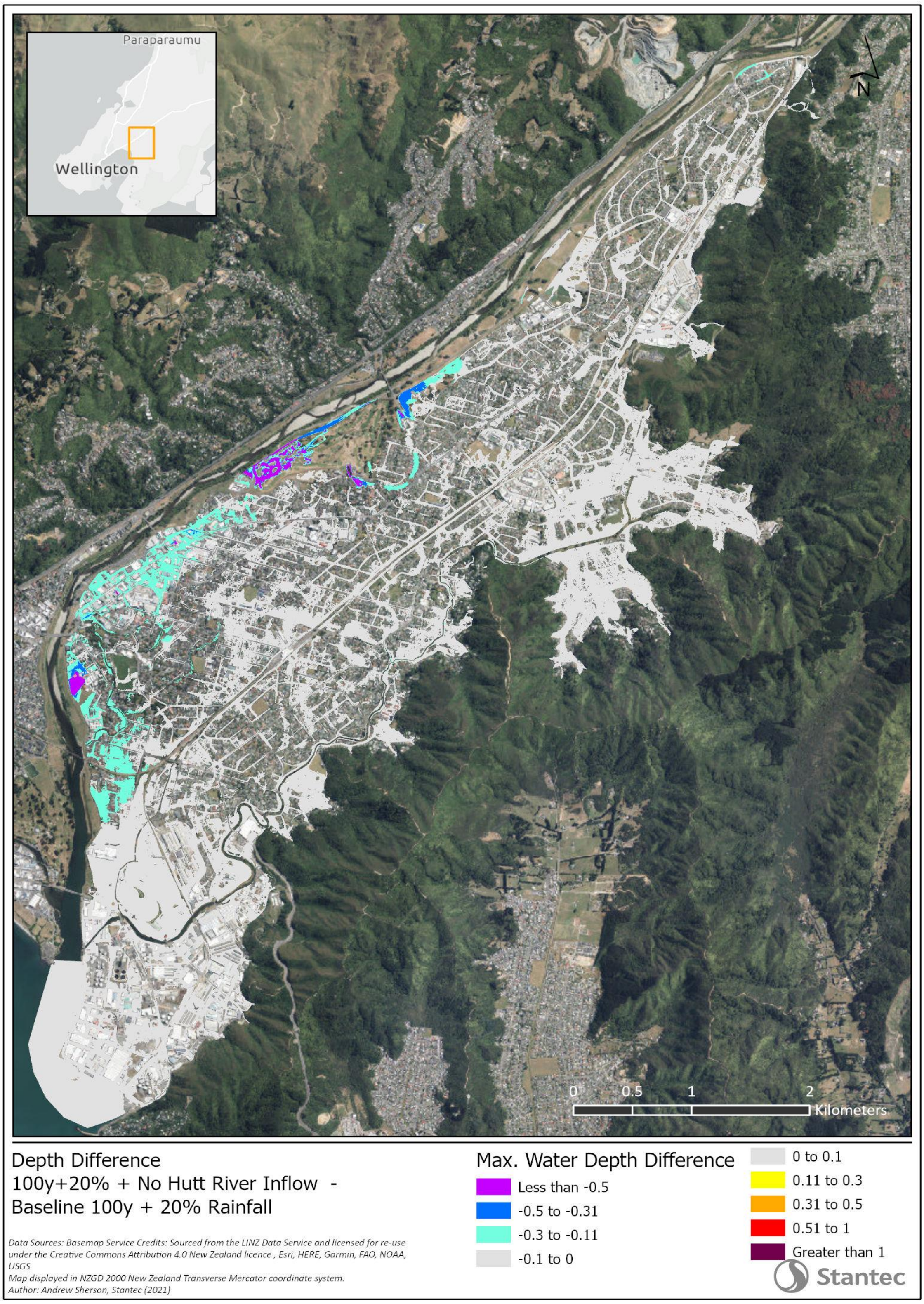


Figure 7-5 Difference in depth between the base scenario and no flow in the Hutt River

7.1.6 All Bridges Blocked

For this scenario, all bridge structures in the model were set to have no conveyance capacity. This was accomplished by setting the width of the bridge opening to 0.3m, removing ~95% of the available capacity. The remaining ~5% was left in the model to allow it to run. The purpose of the scenario was to understand the impact on flooding in the network if bridges were blocked.

The results show that the model is significantly impacted by the blockages of the bridges. Particularly around the Waiwhetū stream and low-lying areas of Moera, as shown by Figure 7-6. This is likely due to the blockage of the Port Rd and Seaview bridges, located at the mouth of the Waiwhetū stream. Flow backs up in the low-lying areas and behind the stop banks as key discharge locations are blocked.

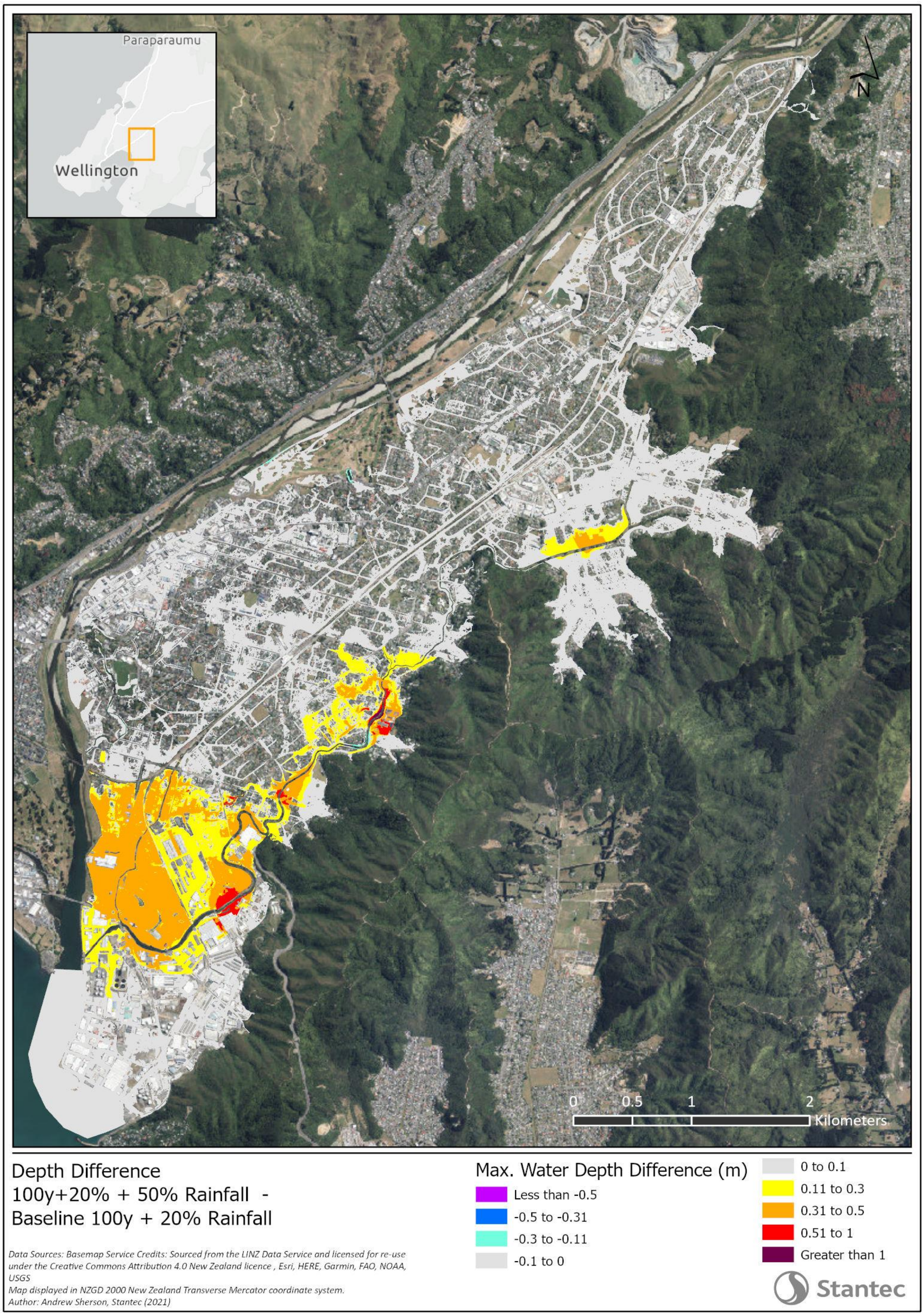


Figure 7-6 Difference in depth between the base scenario and a blockage of all bridges

7.1.7 All Pumps Turned Off

For this scenario, all pumps were 'turned off' to assess the impact of pumps on flooding in ELH. This was achieved by removing all pump links and connecting the remaining links where appropriate.

Results show that the model is particularly sensitive to the Opahu Stream pump. Significant increases in depth occur in the lower portion of the Opahu Stream catchment due to reliance on the pumping station for discharge to a high Hutt River. The Hutt River stop bank and Hutt Valley railway line embankment prevent flows from leaving the Opahu Stream catchment. All other pumps are located near sea level which appears to minimise the impact of turning pumps off as shown in Figure 7-7. This is also due to inability of the pumps to displace volumes significant to the 100-year ARI (20% climate change) flood extents.

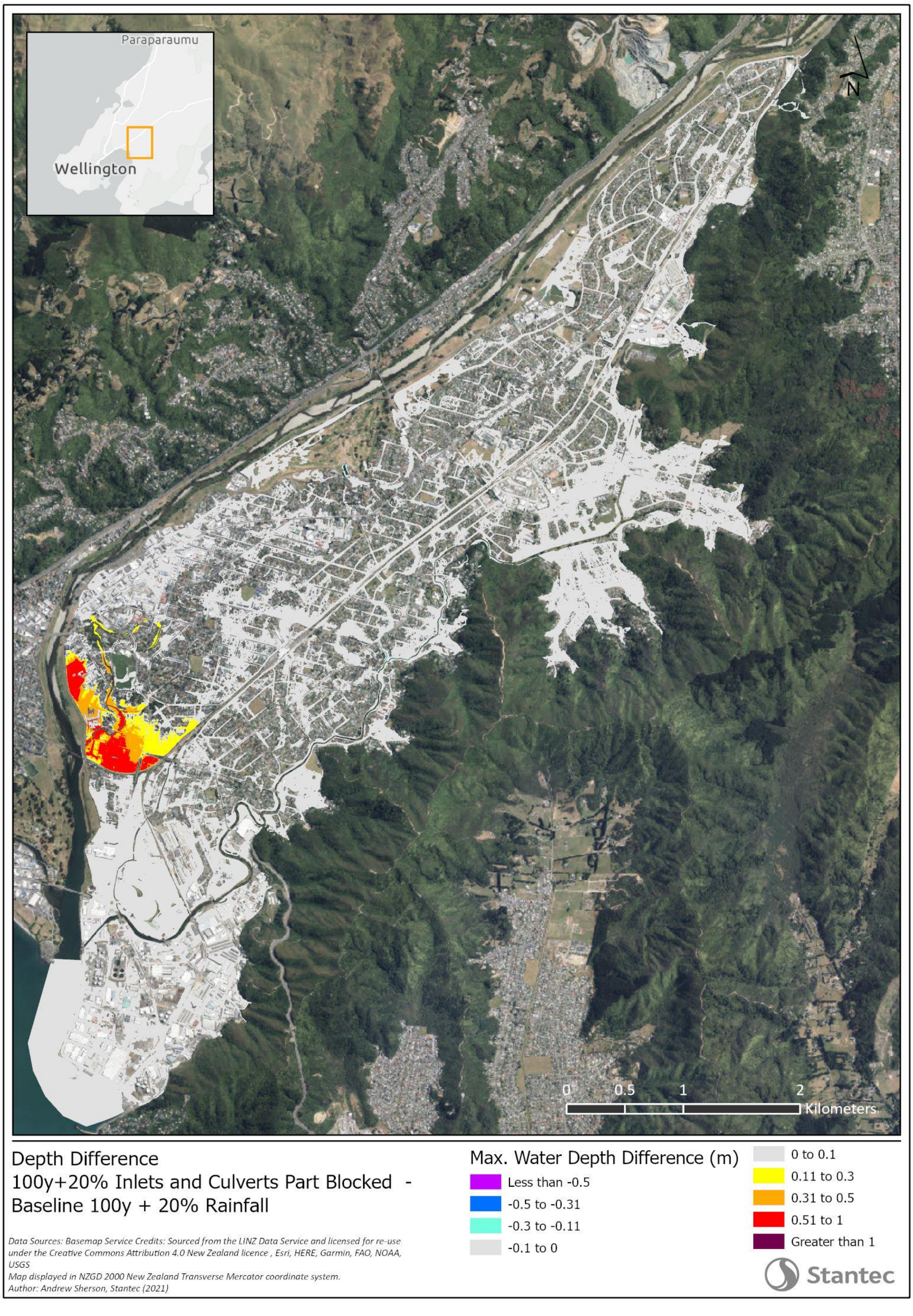


Figure 7-7 Difference in depth between the base scenario and the removal of all pump links

7.2 SENSITIVITY CONCLUSIONS

The ELH network is significantly sensitive to the increase in rainfall, blockages of bridges, and the removal of the tide, and is moderately sensitive to all other scenarios. The regions most affected are those that already experience flooding around the existing water courses such as around the Waiwhetū Stream. It is in these areas where flood depths alter the most. Some new flooding locations do occur but are closely linked to increasing flood extents around these flood prone areas shown in the baseline model results in section 4.3.

7.3 FREEBOARD ANALYSIS

Following the sensitivity study, Stantec was engaged by WWL to undertake a dynamic Freeboard process with the aim of generating maps for district plans. The following section briefly describes the process and presents the final map with the freeboard.

7.3.1 Freeboard Allowance Selection

A freeboard margin is applied to the 100yr ARI event, with a 20% climate change uplift, to guide future planning and define minimum building floor levels. Freeboard values were determined from the sensitivity analysis described in Section 7.1 which investigates how sensitive the Eastern Lower Hutt network is to these key model parameters. After discussions with Wellington Water around the sensitivity results, the freeboard allowances shown in Figure 7-9 were applied across the network.

7.3.2 Freeboard Simulation Setup

A freeboard simulation was developed as described in "Dynamic Freeboard Analysis – Tawa", Jacobs Memorandum, 04 October 2017. The agreed freeboard values were added to maximum flood depths for the 100yr ARI 12hr nested profile with +20% allowance for climate change. These were processed in GIS and imported back into the network as Initial Condition (IC) Zone – hydraulics (2D) polygons. All 1D network elements were deleted including all sub-catchments, 1D pipes, and 1D river reaches. The IC Zone – hydraulics (2D) polygons were then used to create an Initial Conditions 2D database object. The model was then cut into eight segments, shown in Figure 7-8, with overlapping mesh cells at the boundary. This segmentation was required due to the size of the Eastern Lower Hutt model and the required initial validation processing time. A five minute simulation was run for each segment, with only the Initial Conditions (2D) file. This allowed the maximum water levels to spread naturally across the catchment. The segmented results were then merged back together, taking the maximum water depths in any overlapping regions. Clipping of mesh cells into smaller, directly overlapping, segments was conducted in Seaview where there were differently aligned mesh elements.

7.3.3 Dynamic Freeboard Results

Maximum flood depths with freeboard allowance are shown below in Figure 7-10.

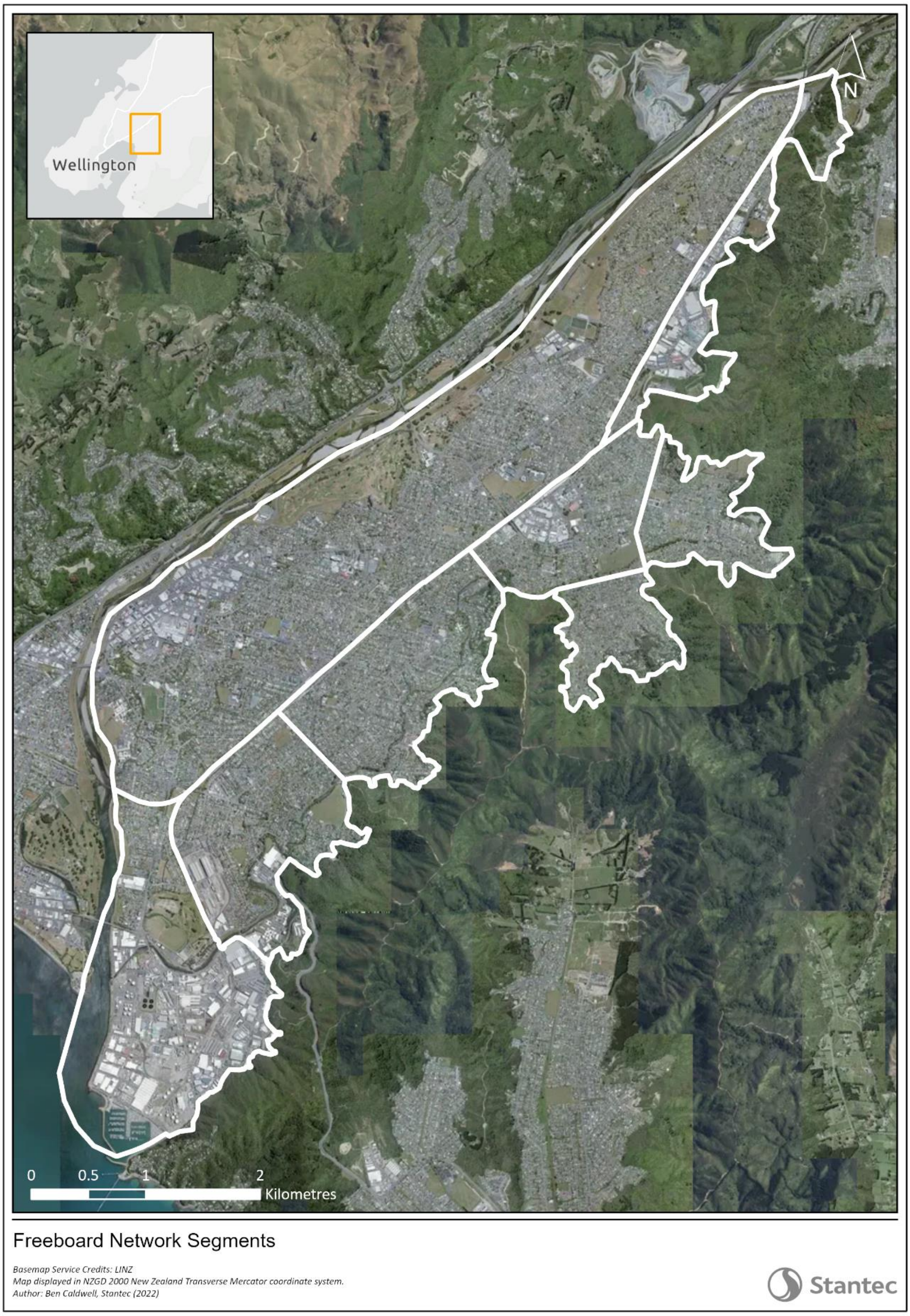


Figure 7-8 Zone boundaries for the split model sections used in the freeboard simulation

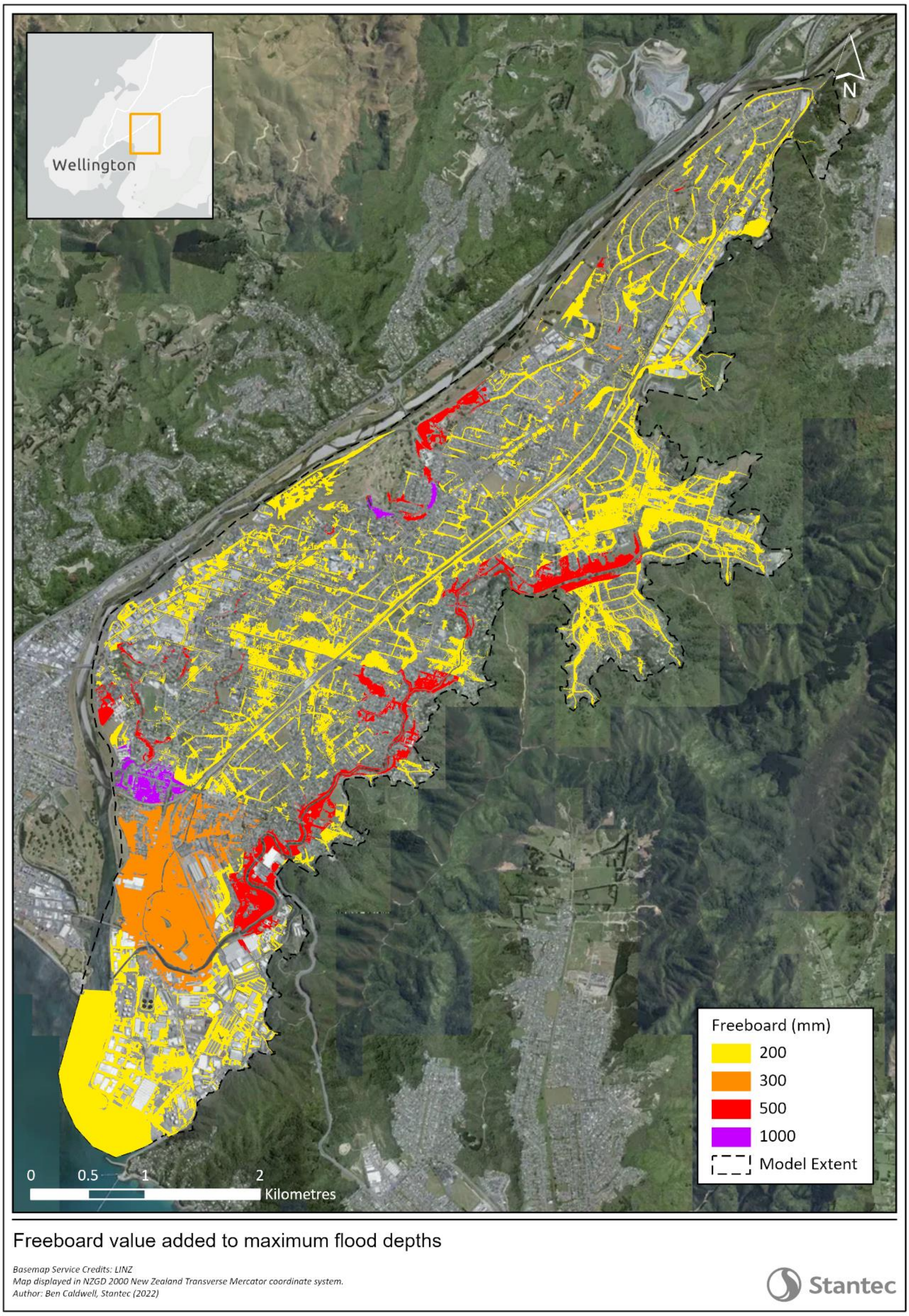


Figure 7-9 Freeboard allowance applied across the Eastern Lower Hutt network

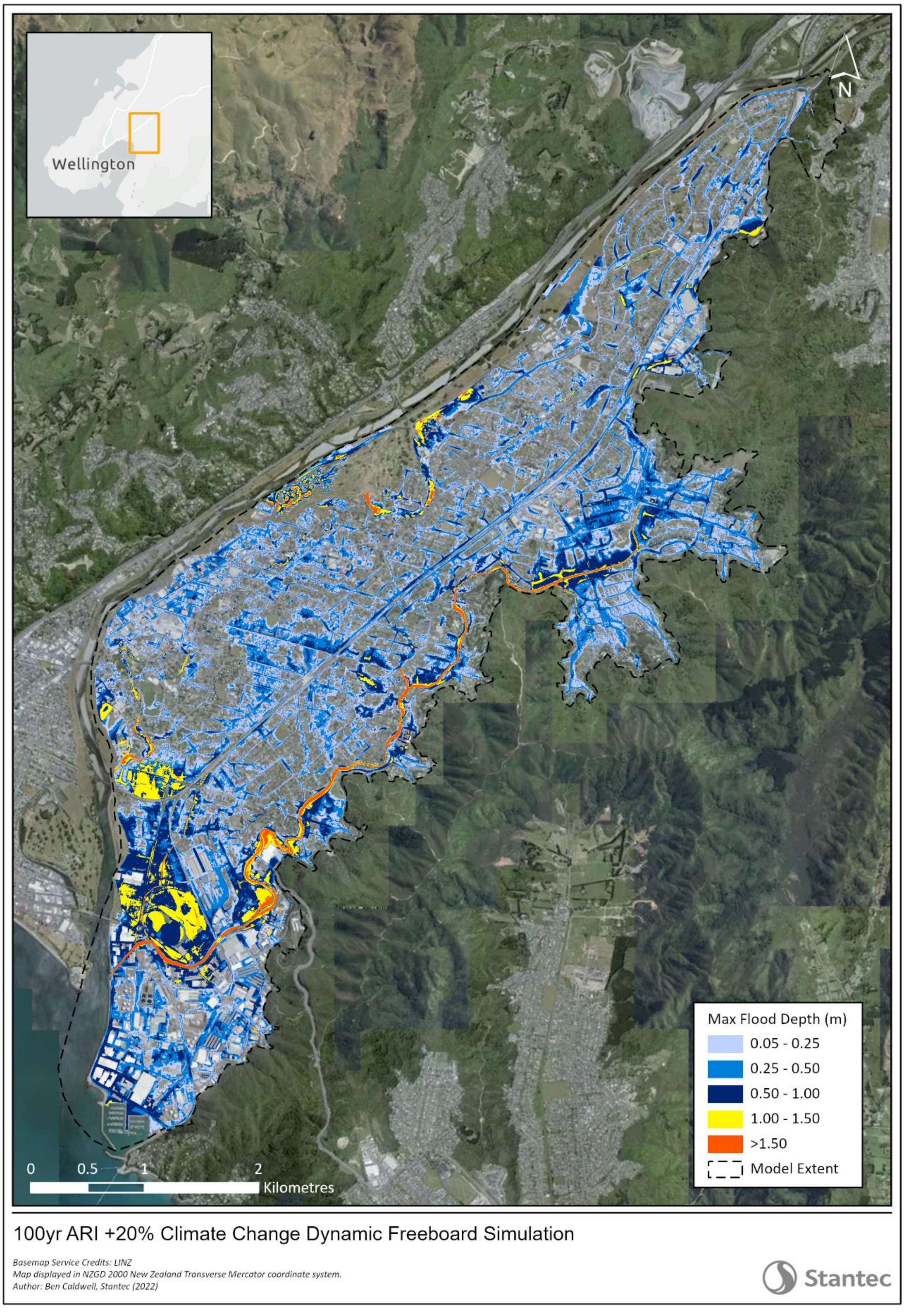


Figure 7-10 Maximum flood depths for the 100yr ARI +20% CC design event including dynamic freeboard. Flooding in the Harbour is not shown.

8.0 CONCLUSIONS AND RECOMMENDATIONS

This report describes the build, validation, calibration, sensitivity and freeboard of the Eastern Lower Hutt stormwater model.

Both UMM and ICM were used to develop an integrated 1D-2D model following the WWL Stormwater Model Specifications. Hydrological parameters of the model were calculated from regional GIS layers and application of the hydrological method descried in the Quick Reference Guide for Design Storm Hydrology, (Cardno, 2016).

The ICM model contains all known public stormwater assets including pipes, manholes, sumps, pumps, and weirs. The private network was also included for pipes larger than 150mm in diameter, to incorporate the network around the Seaview Industrial Area. The full 2D Mesh extent includes all of the Lower Hutt urban area to the east of the Hutt River and to the south of Stokes Valley, excluding Eastbourne. Multiple streams including the Opahu, Waiwhetu, and Awamutu were included with careful attention being placed on the Waiwhetu Stream following known flooding issues and GWRC guidance.

Design rainfall profiles for the 10-year ARI event with existing climate and 100-year ARI event with existing and future climate were simulated along with the 2004 and 2016 rainfall events.

The model results show that the model validates well with both validation events where flooding is concentrated around the Waiwhetu, Opahu, and Awamutu streams. Overall, the model is sensitive to sea level rise, increases in rainfall, pump stations, and blockages of bridges and culverts in some locations.

Due to the significant requirement of interpolation of unknown asset attributes such as inverts where 80% are unknown, it is recommended that before any detailed assessments at property levels are undertaken a review of the asset information is conducted.

It is also recommended to further investigate how well the Lidar used in the model represents the small uncontrolled ponds.

9.0 REFERENCES

Awa. (2017). *Regional stormwater hydraulic modelling specifications.* Wellington, NZ: Author.

Buck, G., & Allitt, R. (2019). The importance of culvert inlets. *CIWEM UDG Spring Conference.* Richard Allitt Associated Ltd.

Capacity Infrastructure Services Ltd. (2013). *Regional stormwater hydraulic modelling specifications.* Upper Hutt City Council, Hutt City Council & Wellington City Council. Wellington, New Zealand: Wellington Water.

Cardno. (2016). *Quick reference guide for design storm hydrology: standard parameters for hydraulic modelling.* Wellington Water. Lower Hutt, New Zealand: Cardno.

Donnelly, S., Pitchforth, T., & Telfer, B. (2017). *Flooded property assessment report, No.28 cornwall street, Lower Hutt.* Lower Hutt: Mott MacDonald.

Fuller, I. C., & Heerdegen, R. G. (2005). The February 2004 floods in the Manawatu, New Zealand: hydrological significance and impact on channel morphology. *Journal of Hydrology, New Zealand, 44*(2), 75-90.

Greater Wellington Regional Council. (2001). *Hutt River floodplain management plan - For the Hutt River and its environment.* Wellington: Author.

Henderson, R. D., Collins, D. G., Doyle, M., & Watson, J. (2018). *Regional flood estimation tool for New Zealand, Final report part 2.* Wellington: NIWA.

Hicks, & Mason. (1998). *Roughness characteristics of New Zealand rivers.* Wellington: NIWA.

Jacobs. (2017). *Dynamic freeboard analysis.* Wellington: Jacobs.

Lawrence, J., Tegg, S., Reisinger, A., & Quade, D. (2011). *Vulnerability and adaptation to increased flood risk with climate change - Hutt Valley summary.* The New Zealand Climate Change Research Institute Victoria University of Wellington.

MWH. (2016). *Wellington Water SW Modelling Panel- Ground model preparation work package: Summary Report- Revision 2.* Wellington, New Zealand: Author.

NIWA. (2010). *High intensity rainfall design system, Version 3*.

NIWA. (2012). *Assessing the storm inundation hazard for coastal margins around the Wellington Region.* Wellington, NZ: Author.

NIWA. (2018). *High intensity rainfall design system, Version 4*. Retrieved from https://hirds.niwa.co.nz/

Ramsbotom, D., Day, R., & Rickard, C. (1997). *Culvert Design Manual.* R168: Construction Industry Research and Information Association.

Sherson, A. K., Burdis, A., & Kerr, T. (2021). *Petone model build report.* Wellington, New Zealand: Stantec.

Sherson, A., Kerr, T., Mulay, G., & Paine, A. (2019). *Eastern Lower Hutt rapid flood hazard assessment.* Wellington: Stantec.

Watts, L., & Gordon, M. (2004). *The 15-16 February 2004 storm in the Wellington region.* Wellington: Greater Wellington Regional Council. Retrieved from http://www.gw.govt.nz/assets/council-publications/Environment%20Management_20040310_163624.pdf

Weekes, J. (2016, 11 15). Flooding closes roads and cut off homes in Hutt Valley. Wellington, New Zealand: Stuff . Retrieved from https://www.stuff.co.nz/national/86481083/flooding-closes-roads-and-cut-off-homes-in-hutt-valley

Wellington Water Ltd. (2017). *Regional stormwater hydraulic modelling specifications v5.* Wellington, New Zealand: Author.

Westlake, S., & Manolache, D. (2016). Managing flood risk from the Hutt River, Wellington, New Zealand and impacts on insurance. *Floodplain Management Australia.*

Appendices

We design with community in mind

APPENDIX A – SURVEY DATA

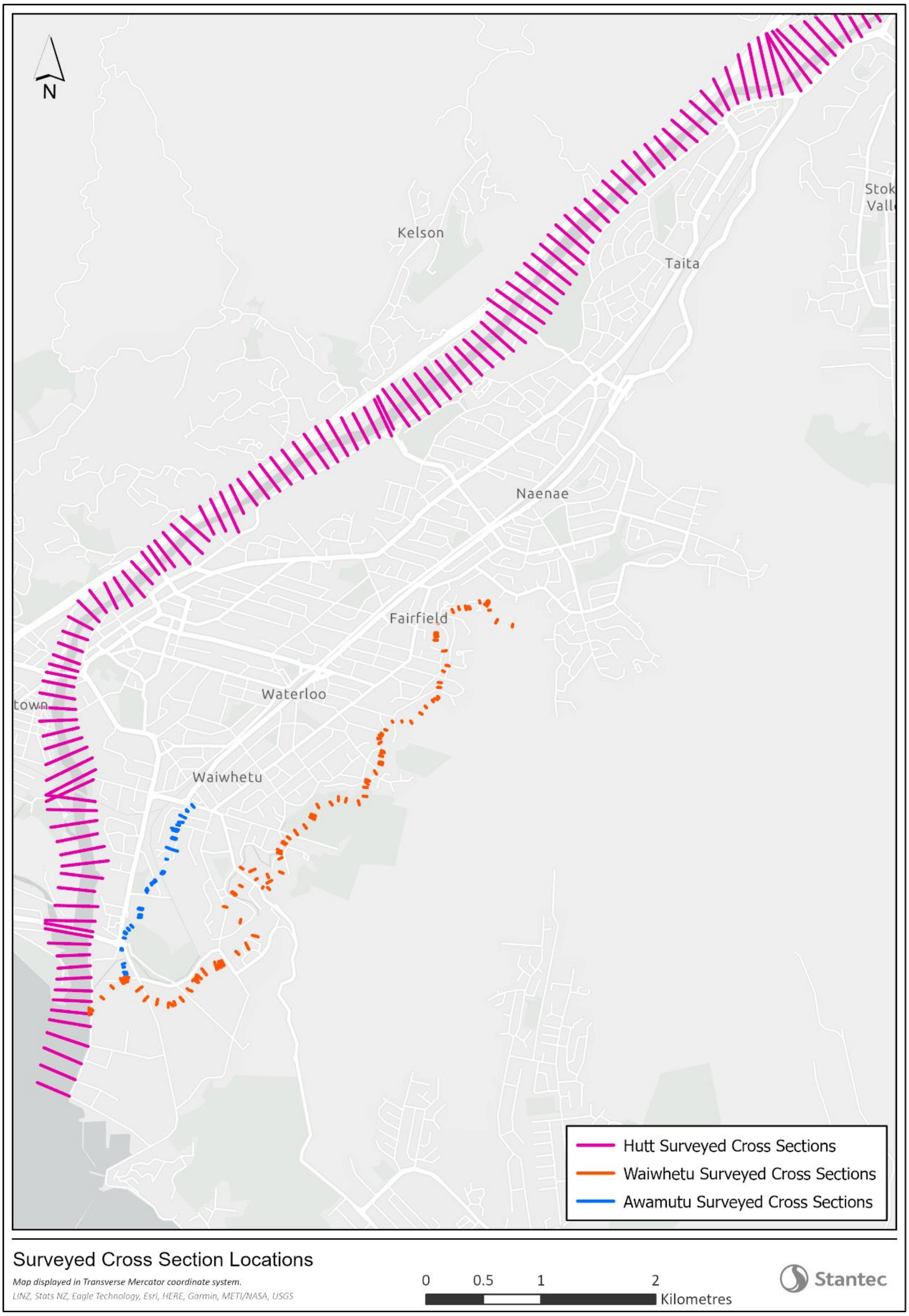


Figure 9-1 Surveyed cross section locations

APPENDIX B – ADDED ASSETS

Node ID	Node Type
610048R00270_a	Manhole
610085R00212_b	Outfall 2-D
670024R00551_b	Manhole
additional_outlet_1	Manhole
Epuni_Sump_1	Manhole
Epuni_Sump_2	Manhole
Epuni_Sump_3	Manhole
Epuni_US_OF_a	Outfall
Epuni_US_OF_b	Outfall
HC_SW022841_a	Manhole
HCC_CH_AWAMUTU_001_DS break_b	Manhole
HCC_CH_SW007332_HCC_CH_SW007332_b	Storage
HCC_CH_SW011029_downStream_b	Manhole
HCC_CH_SW027142_HCC_CH_SW027142_b	Storage
HCC_CH_WAIWHETŪ_02_HCC_SW007229_a	Break
HCC_CH_WAIWHETŪ_32_b_HCC_CH_WAIWHETŪ_32_c	Storage
HCC_CH_WAIWHETŪ_33_HCC_CH_WAIWHETŪ_33_c	Break
HCC_SW003628_a	Manhole
HCC_SW003628_b	Manhole
HCC_SW003628_c	Manhole
HCC_SW003628_d	Manhole
HCC_SW003676_b	Manhole
HCC_SW003703_b	Manhole
HCC_SW003706_b	Manhole
HCC_SW003708_b	Manhole
HCC_SW003710_b	Manhole
HCC_SW003711_b	Manhole
HCC_SW003719_b	Manhole
HCC_SW003737_b	Manhole
HCC_SW003739_b	Manhole
HCC_SW003740_b	Manhole
HCC_SW003786_b	Manhole
HCC_SW003789_b	Manhole
HCC_SW003790_b	Manhole
HCC_SW003826_b	Manhole
HCC_SW003827_a	Manhole
HCC_SW004233_b	Manhole
HCC_SW004450_b	Manhole
HCC_SW004664_b	Manhole

Node ID	Node Type
HCC_SW004709_A	Manhole
HCC_SW004709_B	Manhole
HCC_SW005250a	Manhole
HCC_SW005371_a	Manhole
HCC_SW005635_b	Manhole
HCC_SW005922_a	Manhole
HCC_SW005922_c	Manhole
HCC_SW005922_d	Manhole
HCC_SW005922_e	Manhole
HCC_SW006275_assumed Sump	Manhole
HCC_SW006381	Storage
HCC_SW006393_b	Manhole
HCC_SW006460_b	Manhole
HCC_SW006566_a	Manhole
HCC_SW006708a	Manhole
HCC_SW006868_a	Manhole
HCC_SW007210_b	Manhole
HCC_SW007210_c	Manhole
HCC_SW007256_b	Manhole
HCC_SW007266_b	Manhole
HCC_SW007288_b	Manhole
HCC_SW007289_b	Manhole
HCC_SW007298_b	Manhole
HCC_SW007314_b	Manhole
HCC_SW007325_b	Manhole
HCC_SW007335_b	Manhole
HCC_SW009016_a	Manhole
HCC_SW009016_b	Manhole
HCC_SW009044_b	Manhole
HCC_SW009265_a	Manhole
HCC_SW009429_a	Manhole
HCC_SW009430_a	Manhole
HCC_SW010398a	Manhole
HCC_SW010399a	Manhole
HCC_SW010726_a	Manhole
HCC_SW010877_b	Manhole
HCC_SW010953_a	Manhole
HCC_SW010953_b	Manhole
HCC_SW011001_b	Manhole

Node ID	Node Type
HCC_SW011959_a	Manhole
HCC_SW015227b	Manhole
HCC_SW015228d	Manhole
HCC_SW015228e	Manhole
HCC_SW015228f	Manhole
HCC_SW017194_a	Manhole
HCC_SW017359_a	Manhole
HCC_SW017499_a	Manhole
HCC_SW017499_b	Manhole
HCC_SW017559_1	Manhole
HCC_SW021434_a	Manhole
HCC_SW021594_a	Manhole
HCC_SW021681_a	Manhole
HCC_SW021898_a	Manhole
HCC_SW022842_b	Manhole
HCC_SW022869_a	Manhole
HCC_SW022988_2	Manhole
HCC_SW022998	Manhole
HCC_SW023004_2	Manhole
HCC_SW023042_2	Manhole
HCC_SW023313	Manhole
HCC_SW023313_a	Manhole
HCC_SW023314	Manhole
HCC_SW023399_a	Manhole
HCC_SW023400_a	Manhole
HCC_SW024072	Manhole
HCC_SW026644_b	Manhole
HCC_SW027141a	Manhole
HCC_SW027142_a	Manhole
HCC_SW027143_b	Outfall 2-D
HCC_SW028547_b	Manhole
HCC_SW028547_c	Manhole
HCC_SW07313_a	Manhole
HCC_SWP005300_a	Manhole
HCC_SWP010138a	Manhole
HCC_SWP022593_a	Manhole
HCC_SWP023443_b	Manhole
HCC_SWP023444_a	Manhole
HCC_SWP023444_b	Manhole
HeathRd_lower_ds_OF	Outfall
HeathRd_lower_US	Storage

Node ID	Node Type
HeathRd_lower_us_OF	Outfall
HeathRd_upper_DS	Storage
HeathRd_upper_ds_OF	Outfall
HeathRd_upper_OF	Outfall
HeathRd_upper_US	Storage
Hutt_22_break	Break
Hutt_break_10	Break
Hutt_Break_11	Break
Hutt_Break_11_b	Break
Hutt_break_13	Break
Hutt_break_14	Break
Hutt_break_15	Break
Hutt_Break_15_b	Break
Hutt_break_16	Break
Hutt_break_17	Break
Hutt_break_18	Break
Hutt_break_19	Break
Hutt_break_2	Outfall
Hutt_break_20	Break
Hutt_break_21	Break
Hutt_break_22	Break
Hutt_break_23	Break
Hutt_break_24	Break
Hutt_break_25	Break
Hutt_break_26	Break
Hutt_break_27	Break
Hutt_break_28	Break
Hutt_break_29	Break
Hutt_break_3	Break
Hutt_break_30	Break
Hutt_break_31	Break
Hutt_break_32	Break
Hutt_break_33	Break
Hutt_break_34	Break
Hutt_break_4	Break
Hutt_break_5	Break
Hutt_break_6	Break
Hutt_break_7	Break
Hutt_break_8	Break
Hutt_break_9	Break
Hutt_US_temp	Break

Node ID	Node Type
Industrial_ds_OF	Outfall
industrial_inferred_outlet	Manhole
Industrial_lower_ds_OF	Outfall
Industrial_lower_ds_storage	Storage
Industrial_lower_us_OF	Outfall
Industrial_us_OF	Outfall
LSWXX3188-1A_1	Manhole
LSWXX3188-1A_2	Manhole
LSWXX3188-1A_3	Manhole
LSWXX3188-1A_4	Manhole
Naenae	Outfall 2-D
Naenae Station underpass	Manhole
Naenae_US_OF_a	Outfall
Naenae_US_OF_b	Outfall
New Leighton Ave Bridge	Break
New Leighton Ave Bridge_break	Manhole
Nikau_Gr_break	Storage
Nikau_Gr_culvert_in	Manhole
Nikau_Gr_OF	Outfall
Nikau_Gr_OF_a	Outfall
Nikau_Gr_OF_b	Outfall
Nikau_Gr_OF_c	Outfall
Nikau_Gr_OF_d	Outfall
Nikau_Gr_storage_a	Storage
Nikau_Gr_storage_b	Storage
Nikau_Gr_storage_c	Storage
Nikau_Gr_storage_d	Storage
Norton Grove RR	Storage
Norton Grove RR_ILB	Outfall
Opahu Outlet Penstock Break	Manhole
Opahu Stream PS	Manhole
Opahu Stream PS DS a	Manhole
Opahu Stream PS DS b	Manhole
Opahu Stream PS DS c	Manhole
Opahu Stream PS US	Manhole
Opahu Stream PS_Screen1	Manhole
Opahu Stream PS_Weir	Manhole
Parkside Road PS	Manhole
Parkside Road PS_Dummy	Manhole
Parkside Road PS_Dummy_b	Manhole
Parkside Road PS_Dummy_c	Manhole

Node ID	Node Type
Pomare Station Underpass2	Manhole
Pomare_US_OF	Outfall
Pomere Station Underpass1	Outfall 2-D
randwick_cres_break	Break
randwick_cres_OF_ds	Outfall
randwick_cres_US	Storage
randwick_cres_us_OF	Outfall
Rata St Storage	Storage
Rata St Storage out	Outfall
Rata St Trash Gate	Manhole
Rata Street Culvert1	Manhole
Rata Street Culvert10	Storage
Rata Street Culvert10_IB	Outfall
Rata Street Culvert11	Manhole
Rata Street Culvert12	Storage
Rata Street Culvert12_IB	Outfall
Rata Street Culvert13	Storage
Rata Street Culvert13_IB	Outfall
Rata Street Culvert14	Manhole
Rata Street Culvert2	Storage
Rata Street Culvert2_IB	Outfall
Rata Street Culvert3	Storage
Rata Street Culvert3_IB	Outfall
Rata Street Culvert4	Manhole
Rata Street Culvert5	Storage
Rata Street Culvert5_IB	Outfall
Rata Street Culvert6	Storage
Rata Street Culvert6_IB	Outfall
Rata Street Culvert7	Storage
Rata Street Culvert7_IB	Outfall
Rata Street Culvert8	Manhole
Rata Street Culvert9	Storage
Rata Street Culvert9_IB	Outfall
Riverside Drive PS	Manhole
Seaview Roundabout PS	Manhole
Seaview Roundabout PS_Dummy	Manhole
Taitā Station Underpass	Manhole
Taitā Station Underpass2	Outfall 2-D
Taitā_US_OF	Outfall
Waiwhetū Flume ILB left	Outfall
Waiwhetū Flume ILB Right	Outfall

Node ID	Node Type
Waiwhetū_1_junction_out	Outfall
wastewater_outlet	Manhole
wastewater_outlet_break	Break
WhitesLine_beak	Storage
WhitesLine_OF_lower	Outfall
WhitesLine_OF_upper	Outfall
WhitesLine_scruffy	Manhole
WhitesLine_storage	Storage

Node ID	Node Type
Wilkie Cres Added Culvert	Storage
Wilkie Cres Added Culvert2	Storage
Wilkie Cres Added Culvert3	Manhole
Wilkie Cres Added Culvert4	Outfall
Wilkie Cres Added Culvert5	Outfall
XXXX000002	Manhole

US node ID	Suffix
HCC_SW021982	1
HCC_SW024977	1
HCC_SW017248	1
HCC_SW021422	1
HCC_SW024585	1
HCC_SW021410	1
HCC_SW028549	1
HCC_SW022985	1
HCC_SW024608	1
HCC_SW006455	1
HCC_SW021439	1
HCC_SWP013796_aUpStreamBreak	1
HCC_SW021376	1
HCC_SW024578	1
HCC_SW023337	1
HCC_SW022610	1
HCC_SW022740	1
HCC_SW022723	1
HCC_SW021689	1
HCC_SW024600	1
HCC_SW024601	1
HCC_SW023350	1
HCC_SW006453	1
HCC_SW006868_a	1
HCC_SW028551	1
HCC_SW021488	1
HCC_SW022832	1
HCC_SW023992	1
HCC_SW022295	1
HCC_SW022797	1
HCC_SW024500	1
HCC_SW023344	1

US node ID	Suffix
HCC_SW021428	1
HCC_SW022169	1
HCC_SW021416	1
HCC_SW006449	1
HCC_SW024644	1
HCC_SW024625	1
HCC_SW023484	1
HCC_SW021867	1
HCC_SW022031	1
HCC_SW021421	1
HCC_SW028550	1
HCC_SW023006	1
HCC_SW022710	1
HCC_SW024613	1
HCC_SW023067	1
HCC_SW024278	1
HCC_SW024646	1
HCC_SW015228f	1
HCC_SWP023444_a	1
HCC_SW006450	1
HCC_SW024449	1
HCC_SW009016_a	1
HCC_SW021973	1
HCC_SW017184	1
HCC_SW021424	1
HCC_SW021673	1
HCC_SW021718	1
HCC_SW022041	1
HCC_SW004223	1
HCC_SW021656	1
HCC_SW017311	1
HCC_SW021374	1

US node ID	Suffix
HCC_SW021936	1
HCC_SW022370	1
HCC_SW017504	1
HCC_SW005925	1
HCC_SW022763	1
HCC_SW022776	1
HCC_SW024783	1
HCC_SW023405	1
HCC_SW022815	1
HCC_SW022998	1
HCC_SW023008	1
HCC_SW022598	1
HCC_SW022728	1
HCC_SW022727	1
HCC_SW023836	1
HCC_SW021437	1
HCC_SW024452	1
HCC_SW021560	1
HCC_SW017543	1
HCC_SW022759	1
HCC_SW021681	1
HCC_SW021788	1
HCC_SW004222	1
HCC_SW017499	1
HCC_SW023335	1
HCC_SW022835	1
HCC_SW017399	1
HCC_SW023073	1
HCC_SW021784	1
HCC_SW021584	1
HCC_SW021419	1
HCC_SW017325	1
HCC_SW022990	1
HCC_SW027026	1
HCC_SW022988	1
HCC_SW024572	1
HCC_SW022436	1
HCC_SW021414	1
HCC_SW022296	1
HCC_SW024507	1
HCC_SW028546	1

US node ID	Suffix
HCC_SW021955	1
HCC_SW022510	1
HCC_SW021434	1
HCC_SW022033	1
HCC_SW022318	1
HCC_SW023015	1
HCC_SW024318	1
HCC_SW022989	1
HCC_SW024387	1
HCC_SW023533	1
HCC_SW023319	1
HCC_SW023314	1
HCC_SW021885	1
HCC_SW023129	1
HCC_SW023387	1
HCC_SW024621	1
HCC_SW024581	1
HCC_SW024311	1
HCC_SWP013753_aUpStreamBreak	1
HCC_SW006443	1
HCC_SW017331	1
HCC_SW022734	1
HCC_SW023942	1
HCC_SW022166	1
HCC_SW023343	1
HCC_SW024430	1
HCC_SW022779	1
HCC_SW023014	1
HCC_SW024156	1
HCC_SW021386	1
HCC_SW006456	1
HCC_SW024531	1
HCC_SW024571	1
HCC_SW015228	1
HCC_SW017326	1
HCC_SW023393	1
HCC_SW021397	1
HCC_SW024523	1
HCC_SW021399	1
HCC_SW023380	1
HCC_SW005922_c	1

US node ID	Suffix
HCC_SW022527	1
HCC_SW022479	1
HCC_SW023391	1
HCC_SW024455	1
HCC_SW022560	1
HCC_SW021551	1
HCC_SW023378	1
HCC_SW022709	1
HCC_SW022339	1
HCC_SW023330	1
HCC_SW021375	1
HCC_SW021441	1
HCC_SW021377	1
HCC_SW021600	1
HCC_SW023007	1
HCC_SW022369	1
HCC_SW023353	1
HCC_SW024226	1
HCC_SW022444	1
HCC_SW023363	1
HCC_SW023336	1
HCC_SW021975	1
HCC_SW021365	1
HCC_SW024532	1
HCC_SW021690	1
HCC_SW021430	1
HCC_SW021431	1
HCC_SW023401	1
HCC_SW021395	1
HCC_SW021898	1
HCC_SW021466	1
HCC_SW024456	1
HCC_SW022630	1
HCC_SW023388	1
HCC_SW022629	1
HCC_SW021442	1
HCC_SW021417	1
HCC_SW017389	1
HCC_SW021897	1
HCC_SW021549	1
HCC_SW022040	1

US node ID	Suffix
HCC_SW022742	1
HCC_SW023074	1
HCC_SW022088	1
HCC_SW026905	1
HCC_SW022402	1
HCC_SW024603	1
HCC_SW023316	1
HCC_SW017330	1
HCC_SW024647	1
HCC_SW024184	1
HCC_SW024182	1
HCC_SW023923	1
HCC_SW021550	1
HCC_SW009061	1
HCC_SW021361	1
HCC_SW021548	1
HCC_SW024441	1
HCC_SW022977	1
HCC_SW021948	1
HCC_SW022738	1
HCC_SW023534	1
HCC_SW023374	1
HCC_SW022841	1
HCC_SW023118	1
HCC_SW021871	1
HCC_SW024145	1
HCC_SW021981	1
HCC_SW021735	1
HCC_SW022561	1
HCC_SW021557	1
HCC_SW022762	1
HCC_SW022036	1
HCC_SW021367	1
HCC_SW021432	1
HCC_SW021582	1
HCC_SW022445	1
HCC_SW021390	1
HCC_SW024810	1
HCC_SW017558	1
HCC_SW022289	1
HCC_SW021837	1

US node ID	Suffix
HCC_SW023527	1
HCC_SW017258	1
HCC_SW017259	1
HCC_SW017266	1
HCC_SW024055	1
HCC_SW023341	1
HCC_SW023477	1
HCC_SW017497	1
HCC_SW004017	1
HCC_SW015227b	1
HCC_SW024566	1
HCC_SW024802	1
HCC_SW024586	1
HCC_SW022816	1
HCC_SW022286	1
HCC_SW028547	1
HCC_SW021444	1
HCC_SW022622	1
HCC_SW023376	1
HCC_SW022215	1
HCC_SW023036	1
HCC_SW024008	1
HCC_SW023005	1
HCC_SW021983	1
HCC_SW017352	1
HCC_SW023349	1
HCC_SW024468	1
HCC_SW022530	1
HCC_SW023358	1
HCC_SW023348	1
HCC_SW021834	1
HCC_SW022277	1
HCC_SW021594	1
HCC_SW022736	1
HCC_SW021415	1
HCC_SW022034	1
HCC_SW022278	1
HCC_SW023000	1
HCC_SW022999	1
HCC_SW026689	1
HCC_SW017324	1

US node ID	Suffix
HCC_SW017353	1
HCC_SW023050	1
HCC_SW022997	1
HCC_SW024450	1
HCC_SW021366	1
HCC_SW017556	1
HCC_SW023329	1
HCC_SW023352	1
HCC_SW023321	1
HCC_SW017183	1
HCC_SW017341	1
HCC_SW023970	1
HCC_SW023347	1
HCC_SW024564	1
HCC_SW017351	1
HCC_SW022165	1
HCC_SW023049	1
HCC_SW021354	1
HCC_SW022430	1
HCC_SW024227	1
HCC_SW022755	1
HCC_SW023371	1
HCC_SW022024	1
HCC_SW017318	1
HCC_SW021520	1
HCC_SW023044	1
HCC_SW023045	1
HCC_SW000001	1
HCC_SW023491	1
HCC_SW017381	1
HCC_SW024657	1
HCC_SW017457	1
HCC_SW024435	1
HCC_SW024448	1
HCC_SW028389	1
HCC_SW024615	1
HCC_SW023400	1
HCC_SW017501	1
HCC_SW017502	1
HCC_SW023384	1
HCC_SW026569	1

US node ID	Suffix
HCC_SW022212	1
HCC_SW023069	1
HCC_SW022817	1
HCC_SW024460	1
HCC_SW017561	1
HCC_SW023296	1
HCC_SW023523	1
HCC_SW023722	1
HCC_SW022085	1
HCC_SW021418	1
HCC_SW023394	1
HCC_SW023791	1
HCC_SW021464	1
HCC_SW028251	1
HCC_SW022050	1
HCC_SW021360	1
HCC_SW005922_d	1
HCC_SW005922_a	1
HCC_SW005922_e	1
HCC_SW017503	1
HCC_SW021394	1
HCC_SW021412	1
HCC_SW023404	1
HCC_SW021696	1
HCC_SW022810	1
HCC_SW021409	1
HCC_SW024265	1
HCC_SW023297	1
HCC_SW022100	1
HCC_SW021994	1
HCC_SW021995	1
HCC_SW021923	1
HCC_SW023399	1
HCC_SW023872	1
HCC_SW021370	1
HCC_SW024153	1
HCC_SW024454	1
HCC_SW017329	1
HCC_SW021688	1
HCC_SW021895	1
HCC_SW021896	1

US node ID	Suffix
HCC_SW017319	1
HCC_SW017359	1
HCC_SW021868	1
HCC_SW021869	1
HCC_SW024181	1
HCC_SW022976	1
HCC_SW023909	1
HCC_SW024411	1
HCC_SW021429	1
HCC_SW023012	1
HCC_SW023051	1
HCC_SW021355	1
HCC_SW024550	1
HCC_SW022455	1
HCC_SW021433	1
HCC_SW021389	1
HCC_SW017282	1
HCC_SW023324	1
HCC_SW017499_a	1
HCC_SW024434	1
HCC_SW028507	1
HCC_SW022834	1
HCC_SW024378	1
HCC_SW021851	1
HCC_SW024598	1
HCC_SW024510	1
HCC_SW021862	1
HCC_SW023013	1
HCC_SW024253	1
HCC_SW023644	1
HCC_SW021519	1
HCC_SW022071	1
HCC_SW023367	1
HCC_SW021427	1
HCC_SW023313	1
HCC_SW017194	1
HCC_SW024187	1
HCC_SW023035	1
HCC_SW024533	1
HCC_SW009016_b	1
HCC_SW022216	1

US node ID	Suffix
HCC_SW023967	1
HCC_SW023968	1
HCC_SW023033	1
HCC_SW017313	1
HCC_SW006452	1
HCC_SW006451	1
HCC_SW023023	1
HCC_SW021559	1
HCC_SW022562	1
HCC_SW022514	1
HCC_SW023917	1
HCC_SW022358	1
HCC_SW021864	1
HCC_SW021865	1
HCC_SW024225	1
HCC_SW022842	1
HCC_SW023003	1
HCC_SW021926	1
HCC_SW021691	1
HCC_SW022498	1
HCC_SW023406	1
HCC_SW021664	1
HCC_SW022174	1
HCC_SW021476	1
HCC_SW021477	1
HCC_SW022451	1
HCC_SW004948	1
HCC_SW024078	1
HCC_SW023326	1
HCC_SW021663	1
HCC_SW021612	1
HCC_SW017345	1
HCC_SW024220	1
HCC_SW022541	1
HCC_SW022542	1
HCC_SW024540	1
HCC_SW024539	1
HCC_SW021785	1
HCC_SW022262	1
HCC_SW022264	1
HCC_SW022281	1

US node ID	Suffix
HCC_SW017372	1
HCC_SW023239	1
HCC_SW023001	1
HCC_SW024606	1
HCC_SW023331	1
HCC_SW024762	1
HCC_SW021863	1
HCC_SW022072	1
HCC_SW017498	1
HCC_SW022741	1
HCC_SW023002	1
HCC_SW023351	1
HCC_SW017360	1
HCC_SW023377	1
HCC_SW017344	1
HCC_SW021595	1
HCC_SW017250	1
HCC_SW024607	1
HCC_SW024412	1
HCC_SW022780	1
HCC_SW022077	1
HCC_SW022422	1
HCC_SW023282	1
HCC_SW004206	1
HCC_SW026805	1
HCC_SW022993	1
HCC_SW004214	1
HCC_SW023029	1
HCC_SW023566	1
HCC_SW021838	1
HCC_SW017185	1
HCC_SW017245	1
HCC_SW023342	1
HCC_SW017281	1
HCC_SW017544	1
HCC_SW017256	1
HCC_SW022045	1
HCC_SW021393	1
HCC_SW022604	1
HCC_SW022987	1
HCC_SW017431	1

US node ID	Suffix
HCC_SW024604	1
HCC_SW028510	1
HCC_SW024524	1
HCC_SW017374	1
HCC_SW007236_aUpStreamBreak	1
HCC_SW024978	1
HCC_SW021400	1
HCC_SW022980	1
HCC_SW017314	1
HCC_SW017312	1
HCC_SW023398	1
HCC_SW07313_a	1
HCC_SW021401	1
HCC_SW022429	1
HCC_SW021653	1
HCC_SW021672	1
HCC_SW022991	1
HCC_SW017253	1
HCC_SW021553	1
HCC_SW024352	1
HCC_SW017339	1
HCC_SW021425	1
HCC_SW021674	1
HCC_SW023306	1
HCC_SW021402	1
HCC_SW028547_b	1
HCC_SW017194_a	1
HCC_SW006208	1
HCC_SW023011	1
HCC_SW021655	1
HCC_SW023969	1
HCC_SW021364	1
HCC_SW022240	1
HCC_SW021697	1
HCC_SW021423	1
HCC_SW023354	1
HCC_SW023483	1
HCC_SW021870	1
HCC_SW021353	1
HCC_SW022032	1
HCC_SW024417	1
HCC_SW017298	1
HCC_SW017328	1
HCC_SW021392	1
HCC_SW028529	1
HCC_SW028547_c	1
HCC_SW022517	1
HCC_SW017553	1
HCC_SW023284	1
HCC_SW022428	1
HCC_SW017340	1
HCC_SW017350	1
HCC_SW022431	1
HCC_SW021539	1
HCC_SW022086	1
HCC_SW022757	1
HCC_SW022756	1
HCC_SW021475	1
HCC_SW021523	1
HCC_SW023345	1
HCC_SW017579	1
HCC_SW023402	1
HCC_SW024775	1
HCC_SW023327	1
HCC_SW017342	1
HCC_SW024451	1
HCC_SW022743	1
HCC_SW023058	1
HCC_SW022493	1
HCC_SW017252	1
HCC_SW024501	1
HCC_SW021369	1
HCC_SW022035	1
HCC_SW021396	1
HCC_SW021398	1
HCC_SW021937	1
HCC_SW017568	1
HCC_SW027149	1
HCC_SW021629	1
HCC_SW023994	1
HCC_SW024072	1
HCC_SW028545	1

US node ID	Suffix
HCC_SW028509	1
HCC_SW021652	1
HCC_SW024582	1
HCC_SW015227	1
HCC_SW022251	1
HCC_SW023010	1
HCC_SW021892	1
HCC_SW023004	1
HCC_SW022927	1
HCC_SW023726	1
HCC_SW024288	1
HCC_SW024287	1
HCC_SW023916	1
HCC_SW022509	1
HCC_SW004218	1
HCC_SW023054	1
HCC_SW006454	1
HCC_SW021391	1
HCC_SW023056	1
HCC_SW022070	1
HCC_SW017347	1
HCC_SW024499	1
HCC_SW023729	1
HCC_SW022586	1
HCC_SW021438	1
HCC_SW024503	1
HCC_SW024508	1
HCC_SW023375	1
HCC_SW021552	1
HCC_SW023392	1
HCC_SW026651	1
HCC_SW003565	1
HCC_SW022979	1
HCC_SW023359	1
HCC_SW024453	1
HCC_SW023379	1
HCC_SW021413	1
HCC_SW023390	1
HCC_SW023325	1
HCC_SW024575	1
HCC_SW022432	1

US node ID	Suffix
HCC_SW024638	1
HCC_SW017308	1
HCC_SW026525	1
HCC_SW023469	1
HCC_SW010877_b	1
HCC_SW006580	1
HCC_SW003208	1
HCC_SW003904	1
HCC_SW003640	1
HCC_SW026662	1
HCC_SW021594_a	1
HCC_SW015219	1
HCC_SWP007792_1UpStreamBreak	1
HCC_SW026934	1
HCC_SW026502	1
HCC_SW026503	2
HCC_SWP009672_1UpStreamBreak	1
HCC_SW026953	1
HCC_SW026598	1
HCC_SW026802	1
HCC_SW026506	3
HCC_SW026956	1
HCC_SW027144	1
HCC_SW010399a	1
HCC_SW009044_b	1
HCC_SW005250a	1
HCC_SW026800	1
HCC_SW007161	2
HCC_SW026960	1
HCC_SW026807	1
HCC_SWP007796_1UpStreamBreak	1
HCC_SWP011849_1UpStreamBreak	1
HCC_SW027020	1
HCC_SW003365	1
HCC_SW026701	1
HCC_SW026505	2
HCC_SW026686	1
HCC_SWP006095_1UpStreamBreak	1
HCC_SW009555	1
HCC_SW026843	1
HCC_SW010953_a	2

US node ID	Suffix
HCC_SWP009674_1UpStreamBreak	1
HCC_SW017559_1	1
HCC_SW006747	1
HCC_SW027138	1
HCC_SW026904	1
HCC_SW027070	1
HCC_SW026504	3
HCC_SW026507	1
HCC_SW026865	1
HCC_SW006496	1
HCC_SW027040	1
HCC_SWP010744_1UpStreamBreak	1
HCC_SW006906a	1
HCC_SW027943	1
HCC_SW003644	1
HCC_SW028449	1
HCC_SW027019	2
HCC_SW010231	1
HCC_SW010278	1
HCC_SWP007774_1_1UpStreamBreak	1
HCC_SW023333	1
HCC_SW027933	1
HCC_SW005321	1
HCC_SW017359_a	1
HCC_SW010046	1
HCC_SW021681_a	1
HCC_SW026829	1

US node ID	Suffix
HCC_SW003320	1
HCC_SW009058	1
HCC_SW005632	1
HCC_SW026747	1
HCC_SW009082	1
HCC_SW010250	1
HCC_SWP009341_1_1UpStreamBreak	1
HCC_SW027137	1
HCC_SW027077	1
HCC_SW026796	1
HCC_SW027017	1
HCC_SWP023500_1UpStreamBreak	1
HCC_SW028448	1
HCC_SW003656	1
HCC_SW003183	1
HCC_SW023403_manhole	1
HCC_SW028344	1
HCC_SW004046	1
HCC_SW026672	1
HCC_SW026806	1
HCC_SW005158	1
HCC_SW027145	1
HCC_SW003891	1
HCC_SW026403	1
HCC_SW009018	1
HCC_SW021434_a	1
HCC_SW026957	1

APPENDIX C – INTERPOLATIONS AND AUTOMATIONS

Subcatchment Delineation and ToC calculation	
Step	Process
Delineation	1. Rasterise road centrelines and kerb lines
	2. Create 'IsNull' raster for centrelines, kerb lines, and sumps.
	3. Use the 'IsNull' raster for centrelines and kerb lines in the Raster Calculator to raise/lower the base DEM as appropriate.
	4. Fill modified DEM.
	5. Use sump 'IsNull' raster in the Raster Calculator to lower modified DEM and force drainage to sumps.
	6. Calculate 'Flow Direction', 'Flow Length', and 'Sink' rasters for the modified DEM.
	7. Run the 'Watershed' tool using the 'Flow Direction' and 'Sink rasters' to delineate subcatchments above sumps. Polygonise output.
Longest flow path extraction	Script based. The following steps are repeated for each of the approx. 5000 sumps.
	1. Extract the area from the Length raster.
	2. Calculate the longest flow distance within the subcatchment.
	3. Use this longest flow distance to extract a single raster cell as origin point for longest flow path. If there are multiple, then multiple flow paths will be delineated per subcatchment.
	4. Run a rolling ball analysis using the 'Cost Path' tool.
	5. Convert the output to a polyline.
Filtering of longest flow paths to remove multiple lines per subcatchment.	Some catchments get multiple longest flow path origin cells which result in multiple flow paths per catchment. The solution is to sort again using the polyline output.
	1. Select flow paths intersecting the subcatchment and count the total number.
	2. Do nothing if there are no flow paths
	3. Save the flow path to the refined folder if count = 1
	4. If count > 1 sort the selection descending by length.
	5. Loop through the selection and export the first feature, then break the loop.
	6. The result should be one flow path per subcatchment.
	7. Several checks were performed to verify each subcatchment contains a flow path. If required, repeat parts of the process to fill in errors or gaps.
Extract times of concentrations for each flow path.	This part was developed in ArcPro ModelBuilder.
	1. Split flow lines according to criteria described in Cardno Quick Reference Guide 2016.

	2. Extract elevations at each end of each portion of flow (Overland, shallow concentrated, gutter)
	3. Calculate ToC of each portion then sum.
	4. As per Cardno Quick Reference Guide 2016, calculate the mean of Ramser-Kirpich/Bransby-Williams where appropriate.

Automation of Invert Interpolation	
Step	Method
Controlling interpolation preference according to relative upstream network area.	A ruby script was written to assign a p*seudo* stream order to every link. This is based on a count of the total upstream links and biased using conduit diameter.
Selection according to stream order.	Another ruby script was written to trace upstream following the stream order and selecting a continuous conduit path to inference.
Iterative application of InfoWorks ICM inference tool.	Of the selected path all Non-GIS/Survey/Asbuilt inverts were set to zero and the InfoWorks ICM inference tool applied. The conduits were then tested using a minimum cover of 0.6m based on ground levels at nodes, and minimum cover applied wherever it was not met. The previously inferred values were then removed, and the inference process repeated using the inverts where minimum cover was required. This process was then repeated starting at the highest stream order and moving down stream orders until all inverts were inferred. Invert data flags were key to controlling this process.
Manual adjustments.	All negative grades remaining were manually check and adjusted using engineering judgment, or tailored application of the InfoWorks ICM inference tool.

APPENDIX D – REGIONAL HYDROLOGY LAYERS

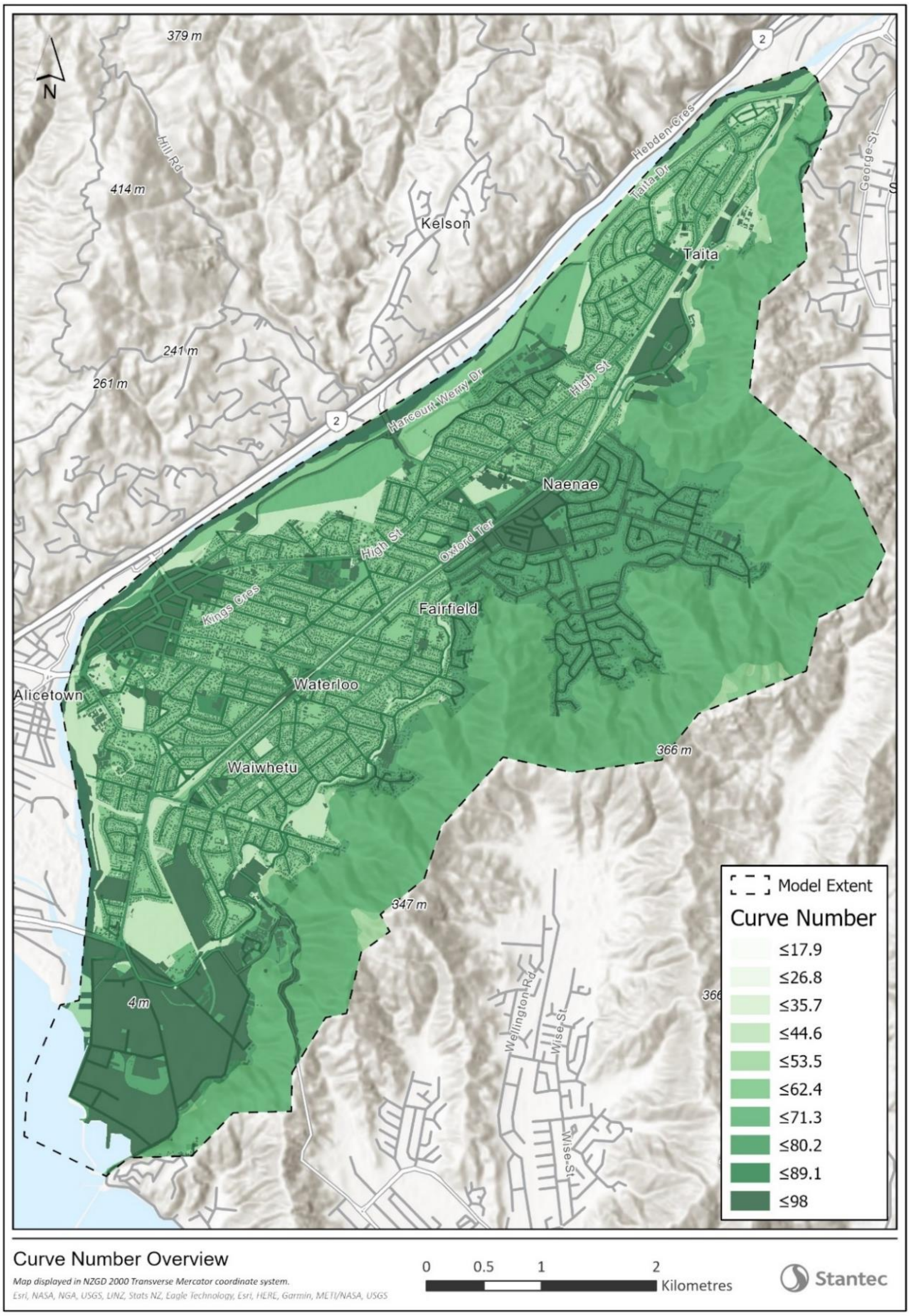


Figure 9-2 SCS curve number layer overview.

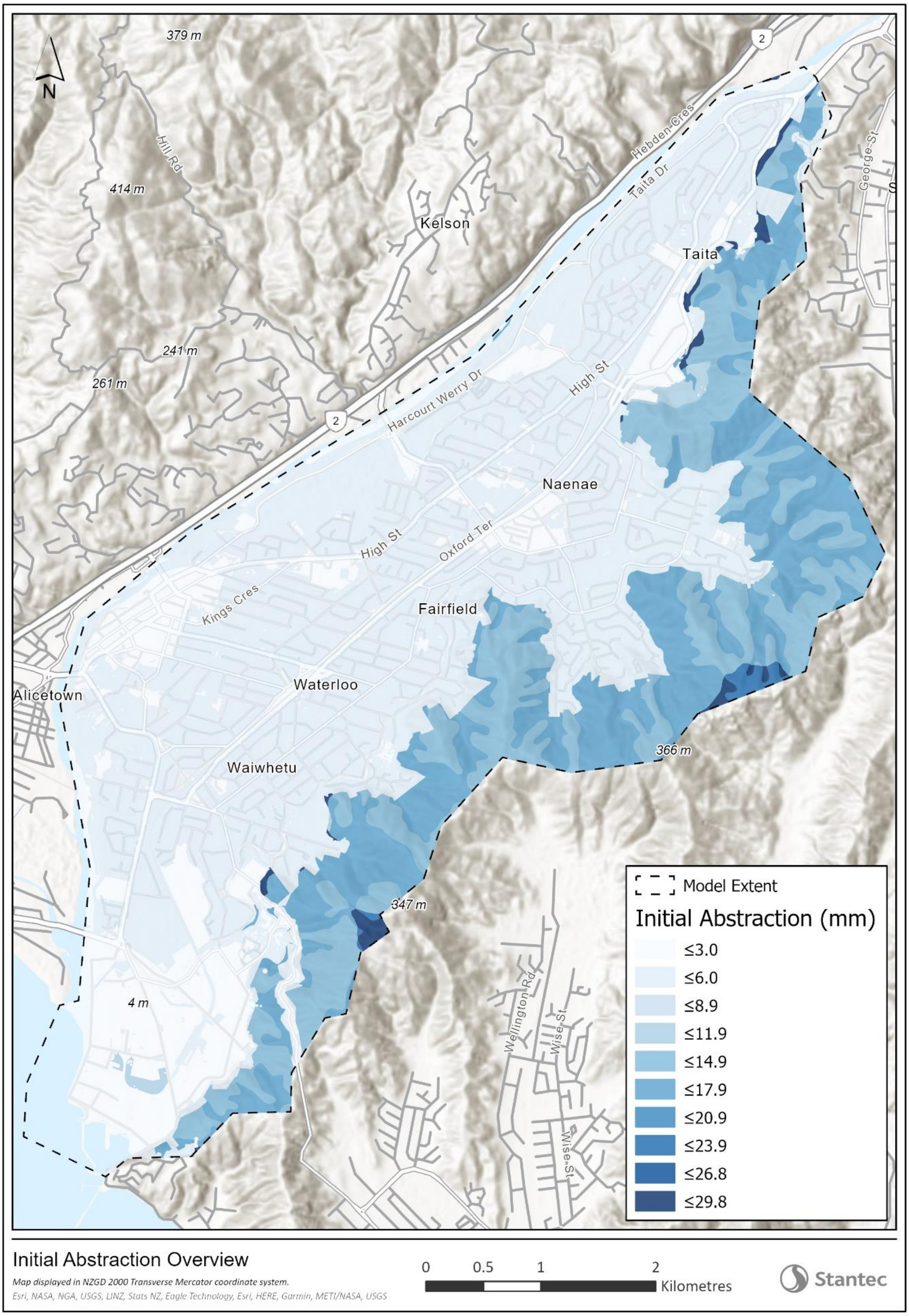


Figure 9-3 Initial abstraction layer overview.

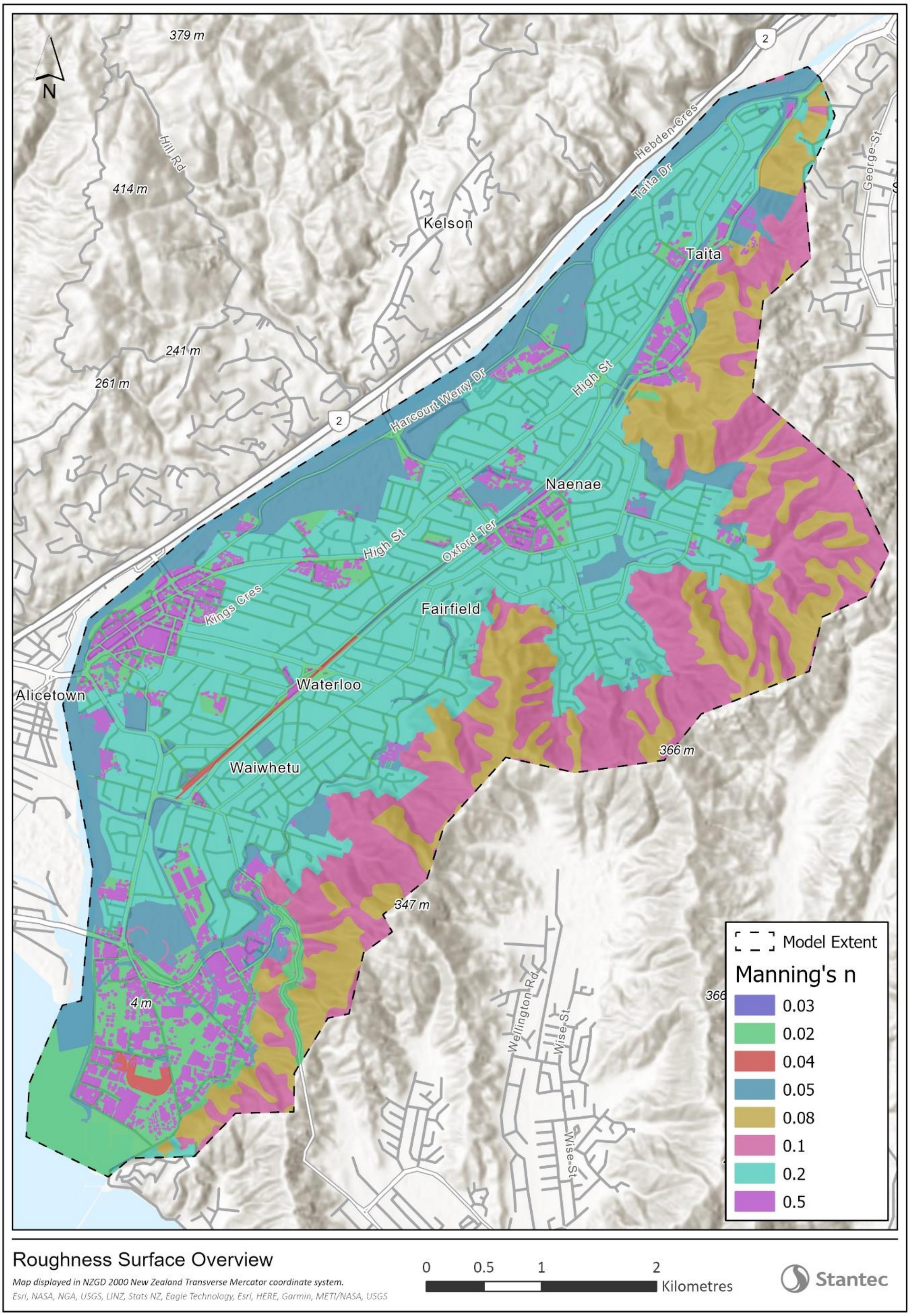


Figure 9-4 Roughness surface layer overview

APPENDIX E – PUMP STATION DATA

Table 9-1 Pump station asset information summary

Pump Station Name	No. of Pumps	Pump Model	Pump Regime	Pump Type (Modelled)	Pump Capacity (l/s)	ON	OFF	Power (kW)	Pump Curve
Guthrie Street Subway	1	FLYGT - Mini 30 submersible	Duty/Assist Duty/Standby	Roto Dynamic	4.2	Unknown	Unknown	0.75	Ready 8S MT 1~ 2p (With rated power 0.9kW)
	2	FLYGT - Mini 30 submersible		Roto Dynamic	5.6	Unknown	Unknown	0.75	Ready 8S MT 1~ 2p (With rated power 0.9kW)
Riverside Drive	1	TSURUMI - 337 - 6B submersible 8.3A, 1000rpm, 6 pole	Duty/Duty	Roto Dynamic	42	Unknown	Unknown	3.75	Provided (With rated power 0.7kW)
	2	SARLIN		Roto Dynamic	56	Unknown	Unknown	3.9	Not Provided
Randwick Road (Seaview Roundabout)	1	FLYGT - PL 7055/680 3~ 725D3	-	Roto Dynamic	650	Unknown	Unknown	37	PL 7055/680 3~ 725D3
Opahu Stream	1	FLYGT - PL7101/835	Duty/Duty/Duty	Roto Dynamic	2,600	2.48m above wet well floor	2.05m above wet well floor	-	PL7101_865(230kW)
	2	FLYGT - PL7101/835		Roto Dynamic	2,600	2.55m above wet well floor	2.28m above wet well floor	-	PL7101_865(230kW)
	3	FLYGT - PL7101/835		Roto Dynamic	2,600	2.68m above wet well floor	2.00m above wet well floor	-	PL7101_865(230kW)
Park Side Road	1	FLYGT LS100 PRESSURE TRANSDUCERS	Duty/Duty	Fixed	1,100	2.7m above floor level	1.5m above floor level Or at 1100l/s	-	Unknown
	2	FLYGT LS100 PRESSURE TRANSDUCERS		Fixed	1,100	3.1m above floor level	1.9m above floor level	-	Unknown

APPENDIX F – DATA FLAGS

Table 9-2 Data flags and description as used in InfoWorks ICM

Flag	Description
#D	InfoWorks ICM system default
ABM	As built
DEM	Value from ground model
DEP	UMM assumed to conform to sump depth requirements
GIS	GIS value
HCAL	Calculated headloss using InfoWorks ICM headloss inference
INF1	Inferred using InfoWorks ICM invert inference
MCRB	Minimum cover (Ruby script)
PCOV	Minimum cover (UMM)
PINF	Inverts inferred using InfoWorks ICM inference tool
PIPE	UMM assumed from adjacent pipe
SITE	Confirmed on site
SOFF	Invert level adjusted to connect at soffit
SUMP	UMM assumed to conform to sump lead requirements
SURV	Survey data
UMM	UMM applied default value
USER	User inferred value
WWL	Pump station data provided by Wellington Water

APPENDIX G – HEAD-DISCHARGE CURVES

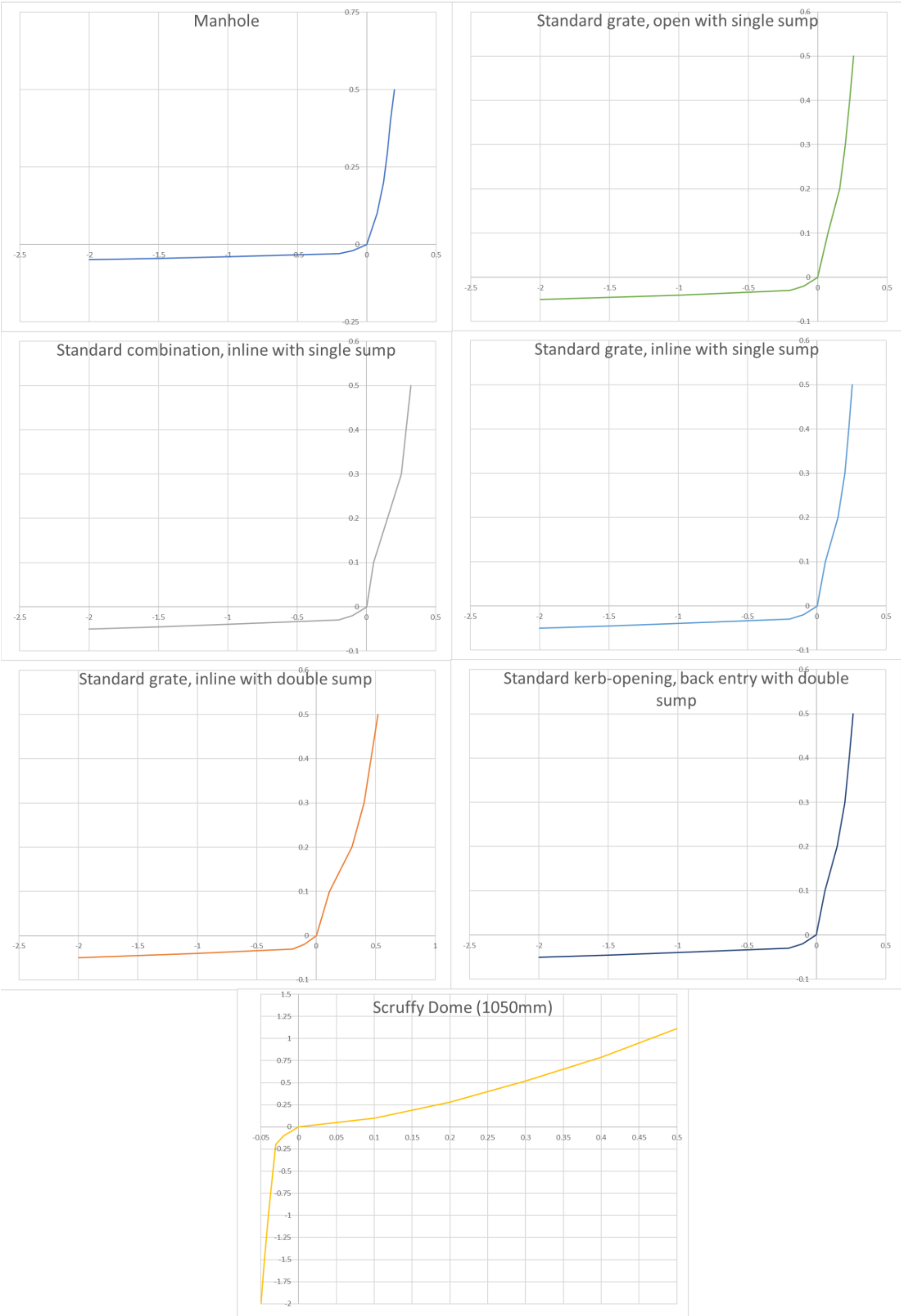


Figure 9-5 Head-discharge curves applied to all sumps and manholes in the model. X-axis: Discharge (m^3s^{-1}), Y-axis: Head (m).

APPENDIX H – 1-D RIVER REACH CROSS SECTIONS

The surveyed cross sections for the Waiwhetū and Awamutu streams from GWRC required edits to remove data that would cause issues in UMM and InfoWorks ICM. These changes included removal of water level and bridge deck data points, and all data points beyond the crest of the bank, as shown in Figure 9-6.

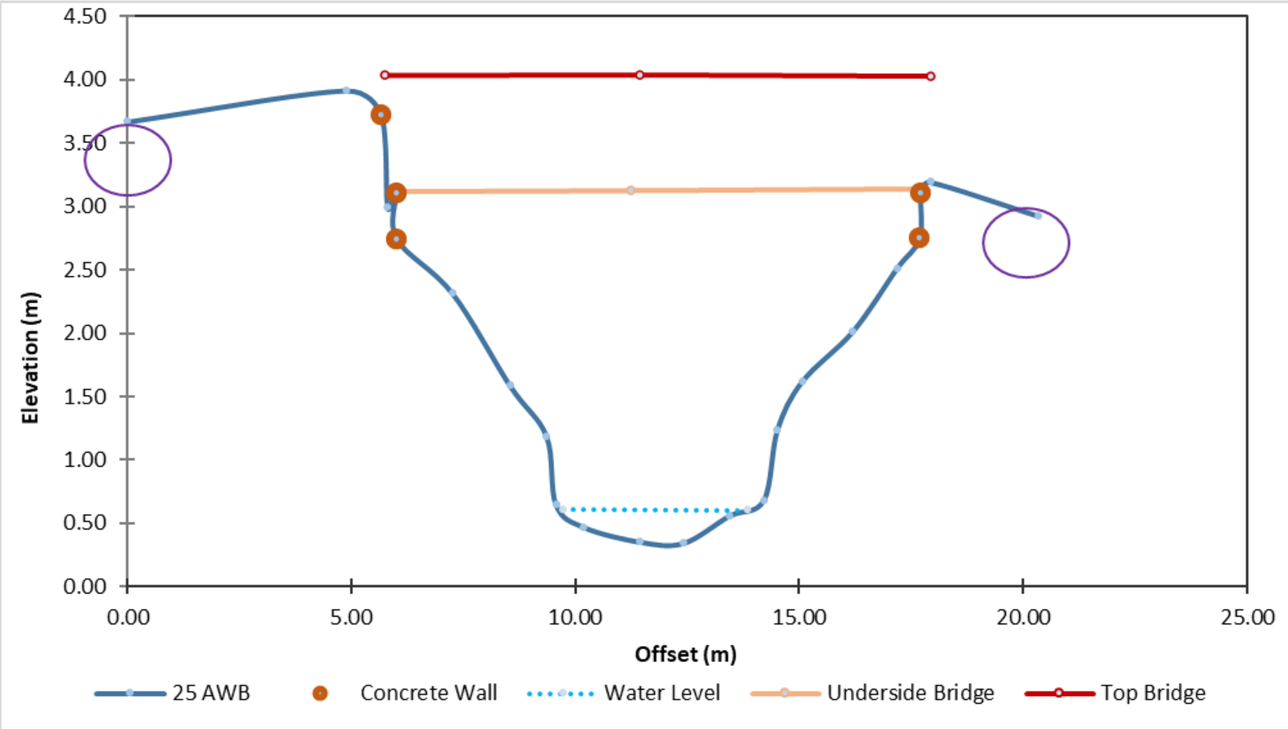


Figure 9-6 Example surveyed cross section. Purple circles show removed vertices

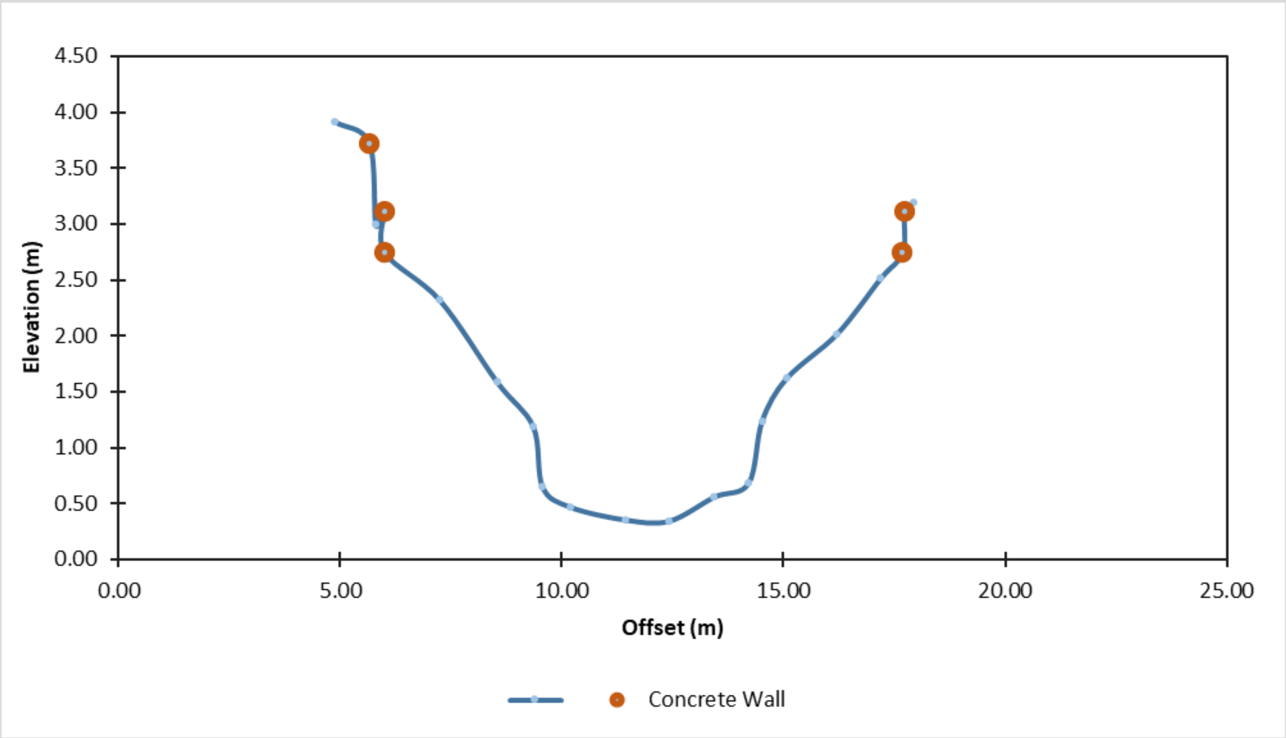


Figure 9-7 Example updated cross section

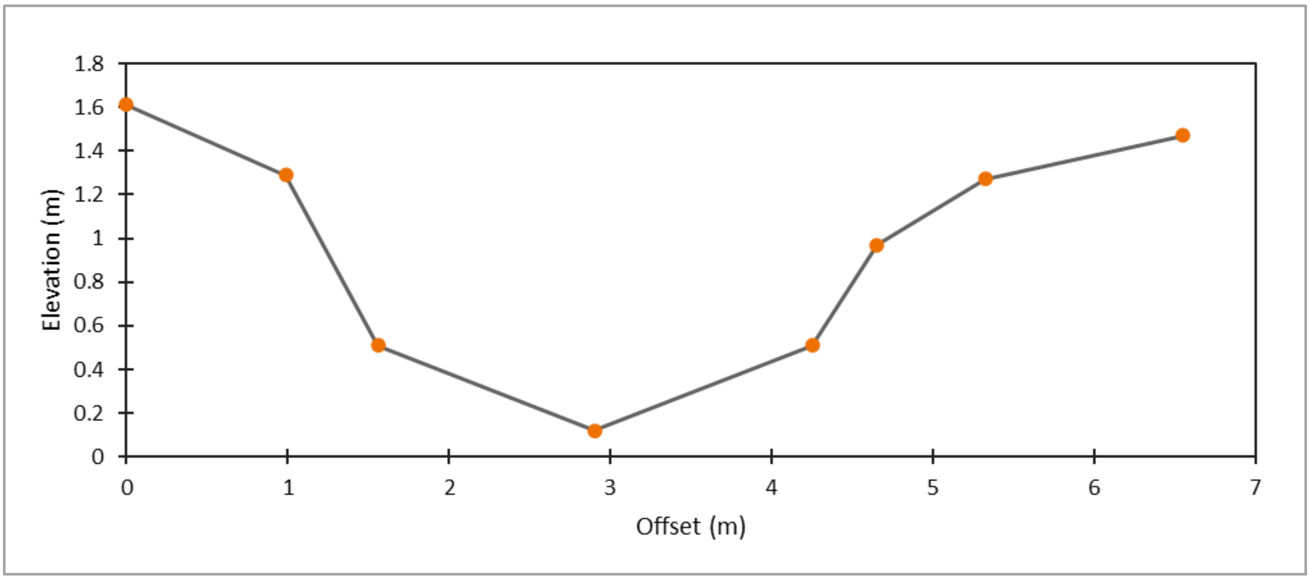


Figure 9-8 Standard U-shaped channel

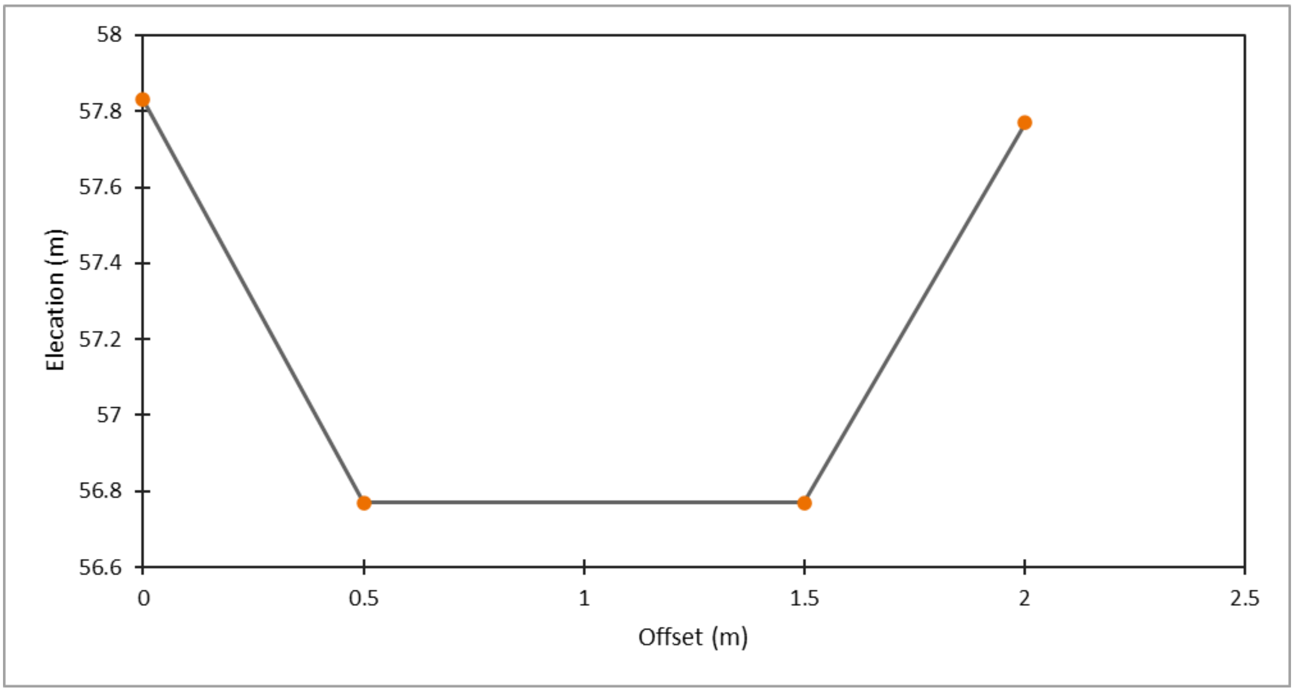


Figure 9-9 Standard trapezoidal cross section

APPENDIX I – HIRDSV4 COMPARISON

Table 9-3 Difference between HIRDSv4 and HIRDSv3 at the Birch Lane GWRC rain gauge.

ARI		10m	20m	30m	60m	2h	6h	12h	24h	48h
1.58	1yr	4%	1%	0%	-1%	-2%	-5%	-10%	-18%	-15%
2	2yr	5%	3%	2%	0%	0%	-4%	-8%	-17%	-14%
5	5yr	9%	6%	5%	3%	2%	-1%	-6%	-15%	-12%
10	10 yr	11%	7%	6%	3%	3%	-1%	-6%	-15%	-13%
20	20 yr	12%	7%	5%	3%	2%	-2%	-7%	-17%	-15%
50	50 yr	10%	6%	4%	1%	0%	-5%	-10%	-20%	-18%
100	100 yr	8%	4%	2%	-2%	-3%	-8%	-14%	-24%	-23%

APPENDIX J – INFOWORKS ICM BRIDGE PARAMETERS

Table 9-4 InfoWorks ICM bridge parameters. All default values excluding inverts and skew angle.

Link ID	Discharge Coefficient	Modular Limit	US Headloss Type	US Headloss Coefficient	DS Headloss Type	DS Headloss Coefficient	US Invert	DS Invert	Skew Angle	Contraction Loss	Expansion Loss	Reverse Contraction Loss	Reverse Expansion Loss
HCC_CH_WAIWHETŪ_02_upstream.1	1.7	0.9	Fixed	0	Fixed	0	8.45	8.33	-1.49	0.3	0.5	0.5	0.3
HCC_CH_WAIWHETŪ_03_upstream.1	1.7	0.9	Fixed	0	Fixed	0	7.65	7.57	23.55	0.3	0.5	0.5	0.3
HCC_CH_WAIWHETŪ_10_upstream.1	1.7	0.9	Fixed	0	Fixed	0	2.3	2.25	-25.36	0.3	0.5	0.5	0.3
HCC_CH_WAIWHETŪ_11_upstream.1	1.7	0.9	Fixed	0	Fixed	0	0.78	0.78	-24.58	0.3	0.5	0.5	0.3
HCC_CH_WAIWHETŪ_12_upstream.1	1.7	0.9	Fixed	0	Fixed	0	-0.17	-0.17	8.9	0.3	0.5	0.5	0.3
HCC_CH_WAIWHETŪ_13_upstream.1	1.7	0.9	Fixed	0	Fixed	0	-0.63	-0.97	3.25	0.3	0.5	0.5	0.3
HCC_CH_WAIWHETŪ_16_upstream.1	1.7	0.9	Fixed	0	Fixed	0	-1.22	-1.25	11.59	0.3	0.5	0.5	0.3
HCC_CH_WAIWHETŪ_17_upstream.1	1.7	0.9	Fixed	0	Fixed	0	-1.24	-1.24	-18.59	0.3	0.5	0.5	0.3

APPENDIX K – GWRC RECORDED FLOOD LEVELS (2004)

Table 9-5 Flood level comparison.

Received Level Data			InfoWorks ICM Model Results		
ObjectID	Recorded	Type	Max Level	Difference	Notes
1	10.49	Flood Level	10.47	-0.02	Within mesh element
2	9.18	Flood Level	9.61	0.43	Within mesh element
3	7.97	Debris Mark in Driveway	7.88	-0.09	Within mesh element
4	7.28	Flood Level	8.06	0.78	Within mesh element
5	7.97	Debris Mark	7.93	-0.04	Within mesh element
6	6.77	Debris Mark	7.99	1.22	Within mesh element
7	5.01	Flood Level	5.21	0.2	Within mesh element
8	2.65	Flood Level	2.47	-0.18	Within mesh element
9	2.97	Flood Level at Floor level House	2.82	-0.15	Within mesh element
10	2.65	Flood Level base of Green Light Pole	2.83	0.18	Within mesh element
11	2.11	Flood Level	2.07	-0.04	Within mesh element
12	2.26	Flood Level	2.51	0.25	Within mesh element
13	2.12	Flood Level	2.07	-0.05	Within 5 metres of mesh element
14	2.07	Flood Level	2.06	-0.01	Within 5 metres of mesh element
15	1.76	Flood Level	1.92	0.16	Within mesh element
16	2.06	Flood Level	2.06	0	Within mesh element
17	1.81	Debris Mark	1.67	-0.14	Level in river reach
18	1.72	Flood Level	1.78	0.06	Within 5 metres of mesh element
19	1.89	Flood Level	1.8	-0.09	Within mesh element
20	3.2	Flood Level in House	3.32	0.12	Within mesh element
21	2.04	Flood Level at First Step of Office	1.98	-0.06	Within mesh element
22	2.52	Flood Level at Air Con Unit	2.53	0.01	Within 5 metres of mesh element
23	2.65	Floodmark	2.47	-0.18	Within 5 metres of mesh element
24	7.97	Floodmark	7.88	-0.09	Within 5 metres of mesh element
25	9.18	Floodmark	9.61	0.43	Within mesh element

Received Level Data			InfoWorks ICM Model Results		
ObjectID	Recorded	Type	Max Level	Difference	Notes
26	10.49	Floodmark	10.47	-0.02	Within 5 metres of mesh element
27	2.82	n/a	3.07	0.25	Within mesh element
28	3.54	Floodmark	3.81	0.27	Within 5 metres of mesh element
29	3.61	Floodmark	4.03	0.42	Within mesh element
30	7.97	Floodmark	8	0.03	Within mesh element
31	2.99	Floodmark	3.27	0.28	Within 5 metres of mesh element
32	4.89	Floodmark	5.56	0.67	Within mesh element
33	7.91	Floodmark	8.46	0.55	Within mesh element
34	8.43	Floodmark	8.63	0.2	Within 5 metres of mesh element

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.

New Zealand offices:
Alexandra, Auckland, Balclutha, Christchurch, Dunedin, Gisborne, Greymouth, Hamilton, Hastings, Napier, Nelson, Palmerston North, Queenstown, Tauranga, Wellington, Whangārei