Petone Stormwater Model Build

PREPARED FOR WELLINGTON WATER LIMITED | March 2023

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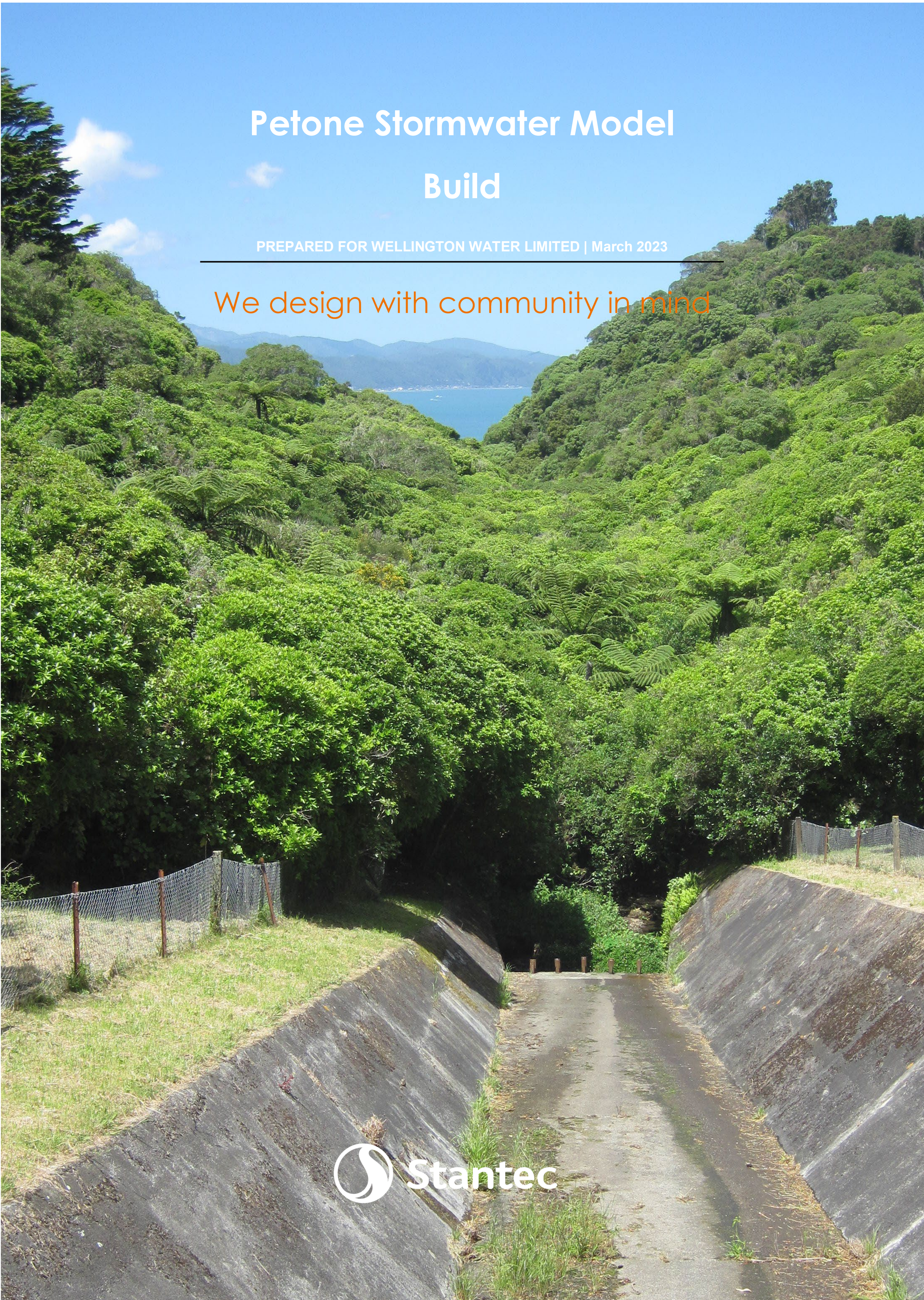

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Revision Schedule

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6	15/03/2023	Extended simulations update	AS	CH	CH	DW

Quality Statement

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Executive Summary

As part of the Wellington Water Ltd (WWL) Modelling Panel, Stantec built and validated an integrated 1D-2D stormwater model for the Petone region in Wellington. The aim was to develop a model that followed the Regional Modelling Specifications (Wellington Water Ltd, 2017), which includes all known public stormwater assets. The finished model would then be used to help understand current and future flooding hazards.

The model was built in InfoWorks Integrated Catchment Modelling (ICM) and then validated against the 2016 and 2015 high rainfall events in Wellington. This validation process showed that the model results largely agreed with observed flooding extents and depths, being only slightly more conservative than observed. Small differences were seen in localised areas due to errors in the asset information and other limitations discussed throughout the report. Results from the model are suited to mapping flooding during high magnitude events when including an appropriate freeboard to account for uncertainty. However, it is recommended that if the model is used as input to infrastructure design, the area of interest should be reassessed, and additional asset data should be collected.

This document has been compiled using the previous model build report submitted in 2019; Petone Stormwater Model Build (Stantec, 2019).

Contents

Revision Schedule i
Quality Statement ii
Executive Summary iii
Abbreviations ix
1 Introduction 1

1.1 Background 1
1.2 Catchment Overview 1
1.3 Study Objectives 3
1.4 Activities and Scope 3

2 Available Information 4

2.1 Previous Work 4
2.2 Drainage Network Data 4

2.2.1 Asset Data 4
2.2.2 Topographical Data 5
2.2.3 As-Built Data 5
2.2.4 Road Assessment and Maintenance Management (RAMMS) 5
2.2.5 Survey Data 5
2.2.6 HCC Online GIS Database 5
2.2.7 Reported Flooding Issues 5

2.3 Hydrologic/Hydrometric Data 10

2.3.1 Rainfall Data 10
2.3.2 Flow and Water Level Data 12
2.3.3 Tidal Data 12
2.3.4 Regional Hydrology Layers 12

3 Model Build 13

3.1 Model Build Overview 13
3.2 Hydrological Model 13

3.2.1 1D Subcatchment Delineation 13
3.2.2 1D Subcatchment Hydrology 15
3.2.3 HEC-HMS Hydrological Model – Korokoro Stream 18

3.3 Hydraulic Model 21

3.3.1 Hydraulic Model Overview 21
3.3.2 1D Reticulated Network 23
3.3.3 1D Channel Network 35
3.3.4 2D Network 37
3.3.5 1D – 2D Connectivity 40

3.4 Boundary Conditions 41

3.4.1 Rainfall 41

3.4.2 Hutt River 42
3.4.3 Korokoro Stream 43
3.4.4 Harbour 43
3.4.5 Model Limitations and Assumptions...44
3.4.6 Hydraulic Model Assumptions..45
3.4.7 Hydrological Model Assumptions 45
3.4.8 Initial Model Testing...45
3.4.9 Model Quality Assurance 46

4 Model Results...47
5 Model Validation 51

5.1 May 2015 Event...51

5.1.1 Alicetown Fish and Chips, Cuba Street / Montague Street...55
5.1.2 Swim Shop, Ministry of Swimming, Cuba Street (building next to fish and chip shop)......57
5.1.3 Hunting &Fishing (now Lifestyle Furniture), Victoria Street / Valentine Street 59
5.1.4 Honda Cars Service Centre, Victoria Street...60
5.1.5 Hutt Central School Playground, Railway Ave 62
5.1.6 1 Aglionby Street - Off Railway Ave 64
5.1.7 19a Railway Ave- Mayur Foodmart, Corner of Railway Ave & Herbert Street 67
5.1.8 General Observations..67
5.1.9 Conclusions..67

5.2 15 November 2016 Event 69
5.3 Conclusion..71
5.4 Validation Results..72

6 Sensitivity and Freeboard 74

6.1 Sensitivity Analysis 74

6.1.1 50% increase in rainfall...74
6.1.2 Inlets fully blocked 77
6.1.3 Inlets partially blocked 79
6.1.4 No Tidal Boundary 81
6.1.5 No Hutt River..82
6.1.6 No Pump Stations...84

6.2 Sensitivity Assessment Summary...85
6.3 Freeboard Methodology and Selection 85

6.3.1 Freeboard Allowance Selection 85
6.3.2 Freeboard Simulation Setup 87
6.3.3 Dynamic Freeboard Results 87

7 Conclusions..89
8 Recommendations..90
9 References..91

List of Appendices

Appendix A Asset Data Uncertainty
Appendix B Attenuation in the Western Hills
Appendix C Cross Section Surveys
Appendix D Manhole Survey Details
Appendix E Regional Hydrology Layers
Appendix F HEC-HMS Model Checks

Appendix G Data Flags
Appendix H Pipe Asset Data Updates
Appendix I Head/Discharge Curves for Sumps
Appendix J Petone and Hutt River Event Timing
Appendix K Event Timing - Rainfall and Hutt River Flows
Appendix L Event Timing - Petone ICM Output
Appendix M Stormwater Outlet Asset Data
Appendix N Flow Distribution - Birchville
Appendix O Useful As-builts
Appendix P DEM modifications
Appendix Q Rapid Flood Hazard Results

List of Tables

Table 2-1: Summary of available information on the stormwater nodes 4
Table 2-2: Summary of available information on the stormwater pipes smaller than 1000mm 4
Table 2-3:Summary of available information on the stormwater pipes greater than 1000mm..........4
Table 3-1: HEC-HMS subcatchment parameters 20
Table 3-2: HEC-HMS subcatchment parameters 21
Table 3-3: Comparison between the full capacity and surveyed capacity of the culverts in...........26
Table 3-4: Detention dam characteristics...31
Table 3-5: Stormwater pump characteristics 32
Table 3-6: Roughness coefficients...39
Table 3-7: Design rainfall intensities (mm/hr) for existing climate conditions. 42
Table 5-1: Water depth comparison at Alicetown Fish and Chips 55
Table 5-2: Water depth comparison at the Ministry of Swimming..57
Table 5-3: Water depth comparison outside Hunting and Fishing 59
Table 5-4: Water depth comparison outside Honda Cars...60
Table 5-5: Water depth comparisons outside Hutt Central School 62
Table 5-6: Water depth comparisons at 1 Aglionby street..64

List of Figures

Figure 1-1: Petone Model overview 2
Figure 2-1 Flooding from the Korokoro catchment on SH2 near Ulrich Aluminium (14 May 2015). . 6
Figure 2-2 Flooding from the Korokoro catchment on SH2 near Ulrich Aluminium (14 May 2015). . 6
Figure 2-3 Flooding on SH2 during the 14 May 2015 event 7
Figure 2-4 Korokoro Stream during the 14 May 2015 event. 7
Figure 2-5 SH2 following the 14 May 2015 event...8
Figure 2-6 Udy Street after the failure of Western Hills gravity pressure main (15 November 2016)8
Figure 2-7 Udy Street after the failure of Western Hills gravity pressure main (15 November 2016). 9
Figure 2-8 Road damage from the Western Hills gravity pressure main (15 November 2016). 9
Figure 2-9 Rainfall radar and gauge overview...11
Figure 2-10 Tide as recorded at Estuary Bridge and Queens Wharf gauges. November 2016 event 12
Figure 3-1 Subcatchment delineation for the Petone catchment showing total area 14
Figure 3-2 Composite SCS curve numbers assigned to each subcatchment. 16
Figure 3-3 Composite Initial Abstraction values assigned to each subcatchment.........................17
Figure 3-4 Korokoro HEC-HMS model overview...19
Figure 3-5 HEC-HMS schematic of the Korokoro catchment. 20
Figure 3-6 Petone Hydraulic model overview...22
Figure 3-7 Remaining negative grade and decreasing pipe diameters, post fixes.........................25
Figure 3-8 (a) A building over Korokoro Stream with multiple walls to separate flow. Photos Source (Cuttriss, 2017). (b) Model representation in ICM. 26

Figure 3-9 Manhole on river side of stop bank. Google Street View, taken by Duane Wilkins. 27
Figure 3-10 Known flap valve locations......27
Figure 3-11 : 1D detention dam schematisation in ICM. 29
Figure 3-12: Various detention dam structures. (a) Dowse Drive detention dam. (b) As-built drawing of the Dowse Dam, low flow inlet structure (Cuttriss, 2000) (c) ICM layout of Dowse Dam, showing a close up of the storage node and low flow structure. (d) Mulburry Street dam, spillway. (e) Overview of the Mulberry Street dam setup in ICM. 30
Figure 3-13 RAMMs Information along SH2. The areas highlighted in green boxes show gaps in the GIS fille......33
Figure 3-14 Subcatchment updates made to accommodate the SH2 network additions. 34
Figure 3-15 Building constructed over the top of the Korokoro Stream. Photo source (Cuttriss, 2017)......35
Figure 3-16: Korokoro Stream river reach and 1D pipe layout, showing the connection between culverts under the buildings and open river reaches. 36
Figure 3-17 2D mesh zones and mesh level zones......38
Figure 3-18 Manning's n roughness zones in the ICM model......40
Figure 3-19 12-hour nested rainfall profile 41
Figure 3-20 Hutt River levels comparison to GWRC Mike Model 43
Figure 3-21 12 Hour tidal boundary, existing climate 44
Figure 3-22 12 Hour tidal boundary with 1m of sea-level rise......44
Figure 4-1 ICM model export, 100y ARI nested flooding event, existing climate, extended 24hr simulation. 48
Figure 4-2 ICM model export, 10y ARI nested flooding event, existing climate, extended 24hr simulation. 49
Figure 4-3 ICM model export, 100y ARI nested flooding event with an additional 20% of rainfall and inflows for climate change, extended 48hr simulation. 50
Figure 5-1 2016 rainfall radar time series......52
Figure 5-2 2015 rainfall radar time series......53
Figure 5-3 Areas of recorded flooding during the May 2015 event, Image taken from (GHD, 2018) 54
Figure 5-4: Location of sites used to compare the model to observed levels. 56
Figure 5-5: Depth hydrograph at the entrance of the Ministry of Swimming 57
Figure 5-6:Depth hydrograph at the car park behind Ministry of Swimming and Alicetown Fish and Chips......58
Figure 5-7 Depth hydrograph at the end of Montague Street. 58
Figure 5-8 Depth hydrograph in front of Hunting and Fishing. 59
Figure 5-9 Depth hydrograph at the back entrance of Honda Cars 60
Figure 5-10 Honda Cars validation sites during the 2015 flood event......61
Figure 5-11 Depth hydrograph at the north-eastern side of the main school building. 62
Figure 5-12 ICM results export, Hutt Central School. 63
Figure 5-13 Depth hydrograph outside main entrance of 1 Aglionby Street. 64
Figure 5-14 Depth hydrograph at the back of no.1 Aglionby Street 65
Figure 5-15 Depth hydrograph at the intersection of Railway Ave and Aglionby Street 65
Figure 5-16 Location of depth hydrographs 66
Figure 5-17 Depth hydrograph in the road corridor near the Mayur food court entrance. 67
Figure 5-18 Flooding along Cornish Street in Lower Hutt on May 2015. Photo by David Morrison quoted in (Price, 2015). 68
Figure 5-19 Rainfall intensity; May 2015 and November 2016 events......69
Figure 5-20 Udy Street property flooding on 8th November 2016 (Weekes, 2016) Photographer Cameron Burnell/Fairfax NZ. 70
Figure 5-21 SH2 Flooding in the north bound lane (Radio New Zealand, 2016), Photographer Kate Gudsell......70
Figure 5-22 SH2 Flooding north bound lane (Radio New Zealand, 2016), Photographer Kate Gudsell. 71
Figure 5-23 ICM model export, May 2015 flood event......72
Figure 5-24 ICM Model Export, November 2016 flood even......73

Figure 6-1 ICM model export, 100y ARI nested flooding event, existing climate- 12hr simulaiton . 75
Figure 6-2 Difference in depth between base case scenario and a simulation with a 50% increase in rainfall intensity ... 76
Figure 6-3 Difference in depth between base case scenario and a simulation with full blockage of all inlets ... 78
Figure 6-4 Difference in depth between base case scenario and a simulation with partial blockage of all inlets ... 80
Figure 6-5 Difference in depth between the base case scenario and a simulation with the tidal boundary condition removed ... 81
Figure 6-6 Difference in depth between the base case scenario and a simulation with the Hutt River inflow removed ... 83
Figure 6-7 Difference in depth between the base case scenario and a simulation with the pump stations removed ... 84
Figure 6-8 Freeboard allowance applied across the Petone network ... 86
Figure 6-9 Maximum flood depths for the 100y ARI + 20% CC design event including dynamic freeboad ... 88

Abbreviations

Enter Abbreviation	Enter Full Name
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 Modeling
UMM	Urban Model Manager
DTM	Digital Terrain Model
DEM	Digital Elevation Model
ASL	Above Sea Level
ARI	Average 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
SH2	State Highway Two
1D	One dimensional
2D	Two Dimensional

1 Introduction

1.1 Background

Stantec were engaged by Wellington Water Ltd (WWL) to develop a comprehensive model of the stormwater network for Petone using InfoWorks Integrated Catchment Modelling (ICM) software. This work is part of a collaborative modelling panel established by WWL for the creation and maintenance of regional models across the Greater Wellington region to improve the understanding of flood risks. The initial build follows a consistent approach set by WWL as stated in Model Build Specifications v4 (Wellington Water Ltd, 2017).

This document contains the previous model build report submitted in 2019 - Petone Stormwater Model Build (Stantec, 2019). In addition, this report includes more recent work, such as sensitivity and freeboard analysis and model updates in response to external peer review.

1.2 Catchment Overview

Petone is a suburb of Lower Hutt city and extends along the south-western side of the Hutt River. Approximately half of the Petone urban area lies within a flat valley ranging from elevations of 0m to 5m above mean sea level and is bound to the south by Wellington Harbour (Figure 1 1). This part of the catchment is significantly affected by tide and river levels. The stormwater system includes five pump stations. The catchments to the west of the Petone flood plain, the Western Hills, range in elevation from 5m to 220m and account for the other half of the modelled urban area. The Western Hills catchments include five detention basins, designed to attenuate flood flows from the Western Hills before they reach the low-lying flood-prone regions of Petone. The Korokoro Stream outlets to the southwest of the Petone catchment near Wellington Harbour. The Korokoro catchment is predominantly rural but has a significant impact on flooding in Petone, particularly along State Highway 2.

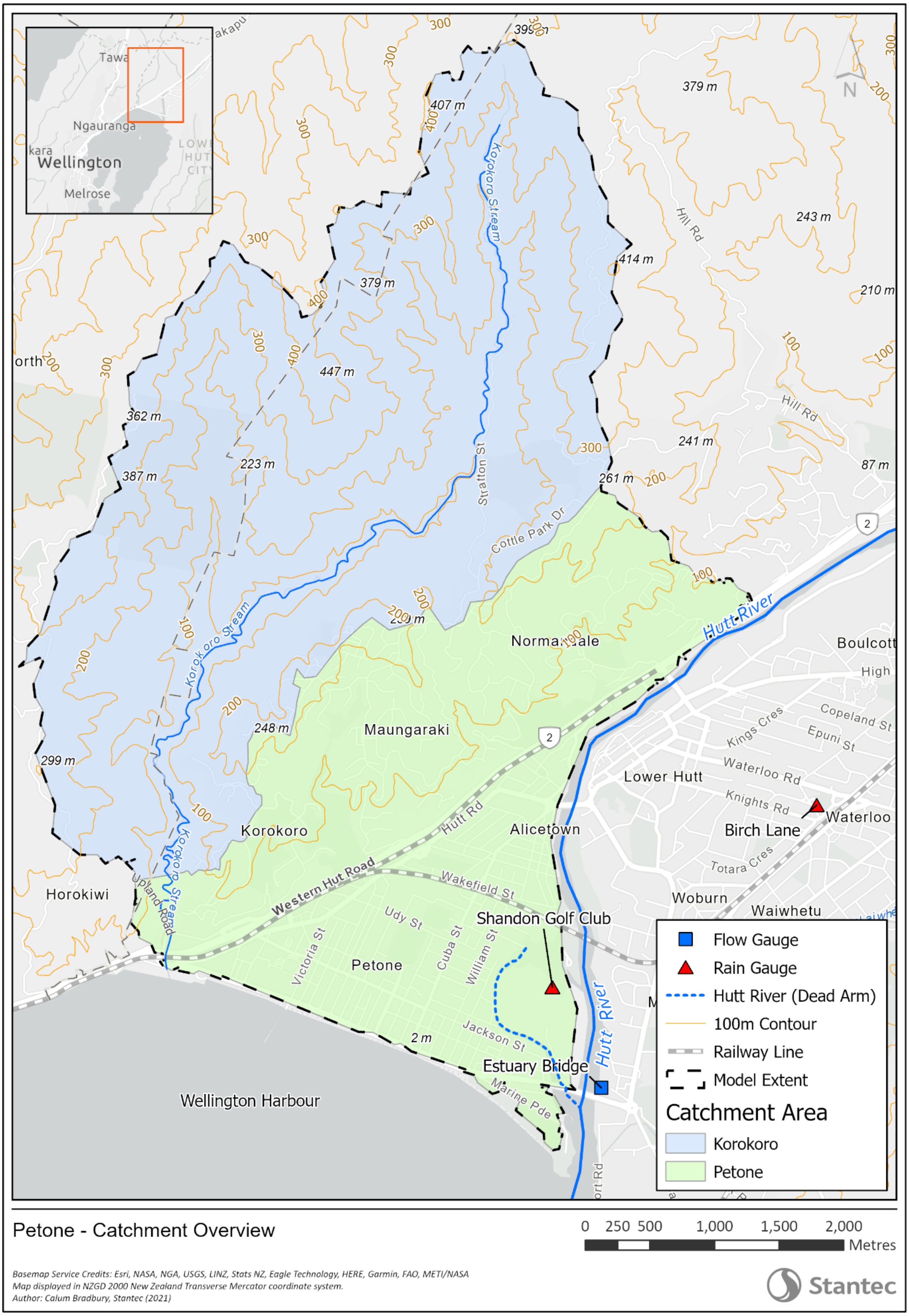


Figure 1-1: Petone Model overview

1.3 Study Objectives

The objective of the work is to develop a fully integrated 1D/2D hydrological and hydraulic model of the Petone stormwater network. The model will be used as a tool in the development of minimum building floor levels for future growth in Petone and for optioneering of network upgrades to help mitigate existing stormwater flooding.

1.4 Activities and Scope

- 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 will be incorporated into the model. Design events including the 10y ARI with existing climate, and the 100y ARI with existing and future climate would be simulated with the base model.

- Validate the model using reported flooding incidences and photographs for two storm events. It was agreed with WWL that the 14th of May 2015 and the 15th of November 2016 storm events would be simulated and analysed.

- Undertake sensitivity analysis on the model to understand the impact of different model parameters on flooding in Petone. These sensitivity scenarios would be used to inform the selection of freeboard values to be applied to the network before publishing of the flood maps.

- Undertake the freeboard assigning process following the Dynamic Freeboard Analysis (Jacobs, 2017; Jacobs, 2017) memorandum and simulate the freeboard run.

- 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 Available Information

2.1 Previous Work

A scoping study for the development of a hydraulic model for the Petone catchment was produced by MWH (now Stantec) in 2017, and is outlined in (MWH (now Stantec), 2017).This study included a Rapid Flood Hazard Assessment using InfoWorks ICM and a review of the available GIS data for the stormwater network in Petone. Key findings and recommendations from this study are outlined below and the RHFA results are shown in Appendix Q.

- Confirmed that approximately 80% of stormwater asset invert levels are unavailable. See Table 2 1 to Table 2
- Recommended obtaining survey cross-sections in two key areas. This survey information was conducted and incorporated into the model.
- The Rapid Flood Hazard Assessment identified a natural depression in central Petone which extends from Nelson Street (west) to William Street (east), and from Petone Recreation Ground (south) to Beaumont Ave (north). Water is expected to pool on the surface in this depression, likely causing widespread flooding. Other problem areas identified were Udy Street, State Highway 2 (SH2), the downstream section of the Korokoro Stream and areas around Dowse Drive & Percy Streams. It was expected that this depression would still encapsulate the main surface flooding issues in the detailed model.
- Other areas of interest which have been investigated and included in the detailed model build - as outlined in this document - are the Western Hills gravity pressure main, pump stations, detention dams, the Dead Arm of the Hutt River, and the Korokoro catchment.

2.2 Drainage Network Data

2.2.1 Asset Data

Stormwater asset data was provided by WWL in ESRI shapefile format. A summary of the input data is described below in Table 2-1 to Table 2-3.

Table 2-1: Summary of available information on the stormwater nodes

	Count	Missing Lid Level	Missing invert Level
Manhole	2590	*1767 (68%)*	2003 (77%)
Sump	2120	*All*	All
Inlet	110	-	103 (94%)
Inlet Structure	22	-	16 (73%)
Outlet	131	-	113 (86%)
Outlet Structure	29	-	17 (58%)
Pump Stations	5	-	-

Table 2-2: Summary of available information on the stormwater pipes smaller than 1000mm

	Count	Missing Upstream Invert (%)	Missing Downstream Invert	Missing Diameter
Gravity Mains	5022	4614 (92%)	4592 (91%)	32 (<1%)
Pressure Mains	13 (2 lines)	1 (8%)	-	-

Table 2-3:Summary of available information on the stormwater pipes greater than 1000mm

	Count	Missing Upstream Invert (%)	Missing Downstream Invert	Missing Dimensions
Gravity Mains	179	169 (95%)	169 (95%)	29 (16%)
Pressure Mains	23 (3 lines)	23 (100%)	23 (100%)	-

2.2.2 Topographical Data

Topographical data was sourced from HCC by WWL for use in the development of hydraulic models for the Panel. The topographical data was supplied as LiDAR points from which a Digital Terrain Model (DTM) with 1m x 1m cells was generated for use in the model. LiDAR data were collected in 2013 to Wellington 1953 vertical datum. The model build process was undertaken prior to the availability of the 2016 LiDAR data. At this stage, it has not been agreed to update to this newer data. For further information on available LiDAR data refer to Ground Model Assessment Summary Report (MWH (now Stantec), 2016) submitted in April 2016 to WWL.

2.2.3 As-Built Data

As-built plans provided by WWL in 2017 were reviewed to add or improve asset information. Data obtained from as-built plans were flagged in ICM. These plans were particularly useful for the Western Hills gravity pressure main, pump stations, flap-gates, and detention dams where the plans were used to confirm the shape of the pipes, invert levels, connectivity of different assets, pump curves and storage volumes.

2.2.4 Road Assessment and Maintenance Management (RAMMS)

The Road Assessment and Maintenance Management (RAMMS) database was used to fill in gaps in the GIS data. See 3.3.2.13 for more information.

2.2.5 Survey Data

2.2.5.1 Cross Sections

Based on recommendations in the Petone Scoping Study, Cuttriss Consultants completed cross-section surveys in the lower reaches of the Korokoro Stream and the Dead Arm of the Hutt River to provide additional information for the model build. These cross-sections provided bed levels as well as invert levels of selected pipes. Appendix C includes details of the survey.

2.2.5.2 Manholes

Manhole surveys were completed by Adamson Shaw to pick up the invert and orientations of pipes entering and exiting eight manholes with GIS flagged inverts in areas subject to significant surface flooding in a 10-year ARI event. See Appendix D for more details.

2.2.6 HCC Online GIS Database

The HCC online GIS database containing up-to-date pipe information such as diameters, materials and node types was used to include the recent (03/2017) pipe upgrade along Queen Street. As the inverts were unknown, they were assumed based on the inverts of connecting pipes and engineering judgement.

2.2.7 Reported Flooding Issues

Several locations in Petone were flooded on 14 May 2015 and 15 November 2016. This was largely the result of significant flows from the Western Hills of Petone including the Korokoro catchment, the stream adjacent to Dowse Drive, and Percy's Stream. Figure 2 1 through to Figure 2 5 below show the inundation of SH2 and several surrounding buildings including Ulrich Aluminium from the lower reaches of the Korokoro catchment.

Figure 2 6 through to Figure 2 8, show a major rupture of the Western Hills gravity pressure main after the 15 November 2016 event, which resulted in significant flooding and road damage along Udy Street.

Figure 2-1 Flooding from the Korokoro catchment on SH2 near Ulrich Aluminium (14 May 2015).

Figure 2-2 Flooding from the Korokoro catchment on SH2 near Ulrich Aluminium (14 May 2015).

Figure 2-3 Flooding on SH2 during the 14 May 2015 event

Figure 2-4 Korokoro Stream during the 14 May 2015 event.

Figure 2-5 SH2 following the 14 May 2015 event

Figure 2-6 Udy Street after the failure of Western Hills gravity pressure main (15 November 2016)

Figure 2-7 Udy Street after the failure of Western Hills gravity pressure main (15 November 2016).

Figure 2-8 Road damage from the Western Hills gravity pressure main (15 November 2016).

2.3 Hydrologic/Hydrometric Data

2.3.1 Rainfall Data

Rainfall data was gathered from the following sources:

1) Rainfall depths from HIRDS v3, NIWA’s rainfall database for various return period and storm durations. Rainfall depths or intensities are available for New Zealand from this online tool. Rainfall for estimated future climate conditions following the four RCP curves were also available.
2) Rainfall radar averaged over the Petone and Korokoro regions was received from WWL for the 2015 and 2016 high rainfall events as separate time series. See Figure 2-9
3) Recorded rainfall depths at the Shandon and Birch Lane rainfall gauges were also available for both the 2015 and 2016 high rainfall events.

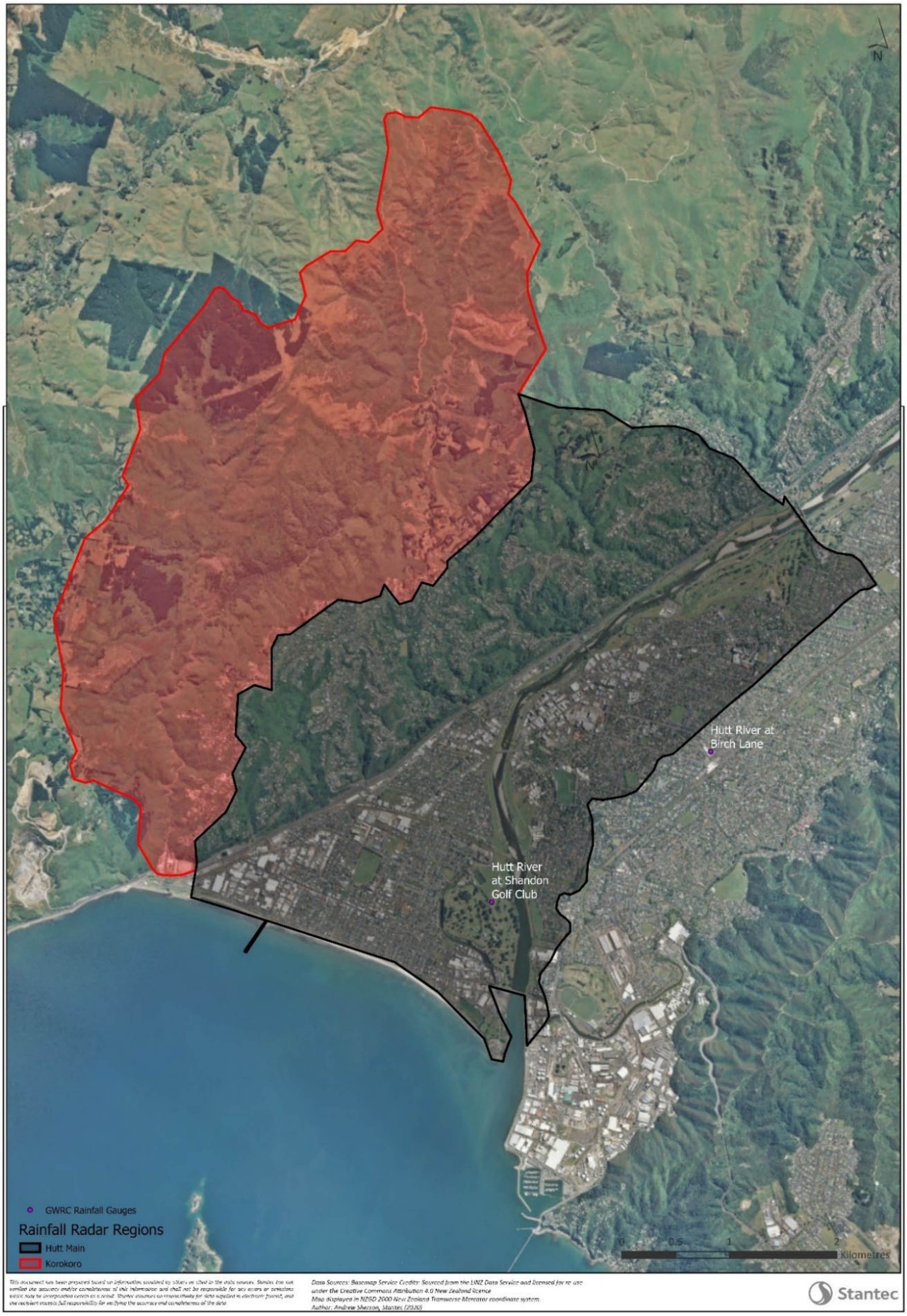


Figure 2-9 Rainfall radar and gauge overview

2.3.2 Flow and Water Level Data

2.3.2.1 Hutt River

Greater Wellington Regional Council (GWRC) developed a DHI hydraulic model of the Hutt River. Results from the GWRC model are available to obtain inflows to simulate levels along the Hutt River. This is discussed further in Section 3.4.2. Flow data for the Hutt River is also available from GWRC at the Taita Gorge gauging station (1766538, 5441948) approximately 10km north of the model extent. This data is available from 6th March 1979 to the present.

2.3.2.2 Korokoro Stream

A flow gauge was re-established on the Korokoro Stream in 2017 by GWRC. Previous data from this site are limited to water level and therefore frequency analysis of the data is not possible.

2.3.3 Tidal Data

Recorded river level was provided by GWRC for the Hutt River at Estuary Bridge gauge, (1759345, 5433683), which provided a record of tidal influence at the southern model boundary. 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), where data is available from 31st August 1994 to the present.

There is not a significant difference in tide timing between the two gauges, as shown in Figure 2-10, however there is a difference in the minimum and maximum levels. This level difference is driven by high levels in the Hutt River during flood events, raising the recorded level at Estuary Bridge.

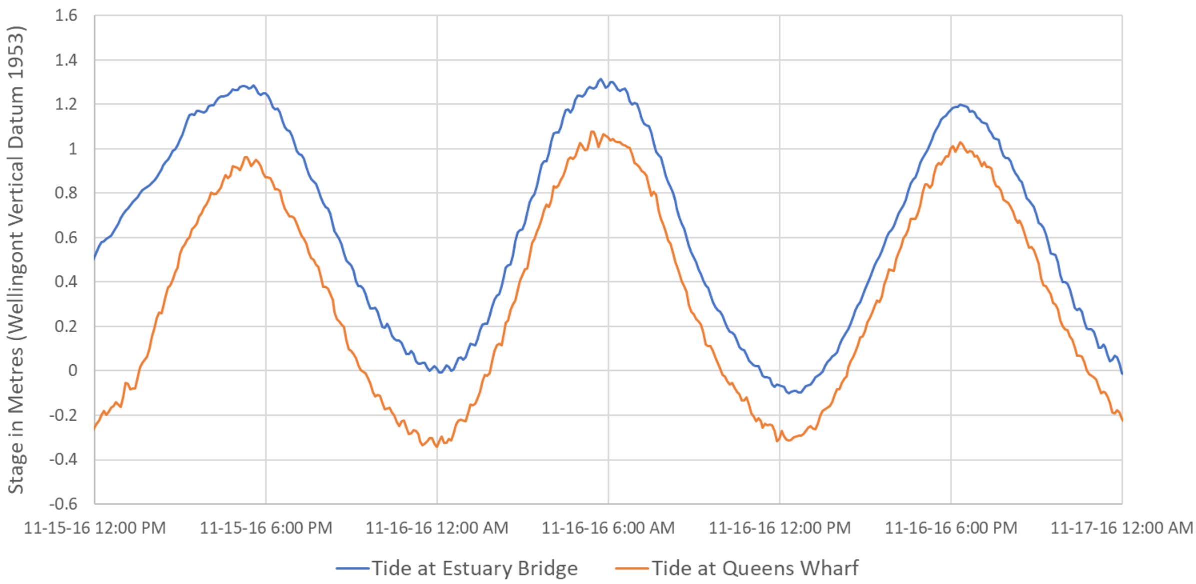


Figure 2-10 Tide as recorded at Estuary Bridge and Queens Wharf gauges. November 2016 event

2.3.4 Regional Hydrology Layers

Regional hydrology layers for the Wellington region consisting of 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 MWH (now Stantec), based on the hydrology guidelines provided in the *Quick Reference Guide for Design Storm Hydrology* (Cardno, 2016). The layers were formed using land-use, soil classifications and topography. An overview of the regional layers can be found in Appendix E . Edits to each layer, apart from roughness, are discussed in Section 3.2.2.

3 Model Build

3.1 Model Build Overview

A fully integrated 1D/2D model was developed for Petone. 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 runoff across the model surface and through the reticulated stormwater network. The software used in the initial development of the Petone Stormwater model was Urban Model Manager (UMM), developed by Awa Environmental. The UMM model was then exported and edited further within InfoWorks ICM Version 7.5, developed by Innovyze. The model has more recently been updated to ICM Version 9.5 and all recent work has been undertaken in this version, including validation and sensitivity. InfoWorks ICM (Integrated Catchment Modelling) is an integrated modelling platform that incorporates both urban and river catchments.

The model build section 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

3.2 Hydrological Model

As per the Wellington Water Regional Stormwater Modelling Specifications v5 (Wellington Water Ltd, 2017) and the Quick Reference Guide for Design Storm Hydrology (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 is described in the following sections. The Korokoro catchment was modelled as a separate hydrological model in HEC-HMS due to its size as well as being predominantly rural. The HEC-HMS model for the Korokoro catchment also follows the specification set out in the *Quick Reference Guide* (Cardno, 2016).

3.2.1 1D Subcatchment Delineation

Subcatchments used in the hydrological model to calculate inflow to the hydraulic model were generated within UMM using a combination of custom tools and Arc Hydro tools available in ArcGIS.

2,869 subcatchments were delineated based on topography, sump locations and added streamlines, see Figure 3-1**.** Catchments were created for each sump and for individual buildings connected directly to the main network (not discharging to the roadside kerb). This was achieved by using GIS to intersect the private pipe network with the buildings they service, then intersecting the main network with the private pipes. Streamlines were drawn to break up larger catchments and allow more precise rainfall application to the model. Out of the 2,869 catchments, 226 were building footprints included as individual subcatchments that drain directly to the nearest manhole.

The autogenerated subcatchments from UMM ranged in size from 0.001ha to 49.415ha. The latest panel specifications suggest that catchments, other than those that just cover building footprints, should be between 0.1ha and 3.0ha. As the construction of the model started before the latest specification was issued, there are 34 catchments above 3.0ha, one of which is above 10ha. There are also 1,267 below 0.1ha, though many of these just cover building footprints. It is therefore recommended that any future works include a review of relevant subcatchment delineation.

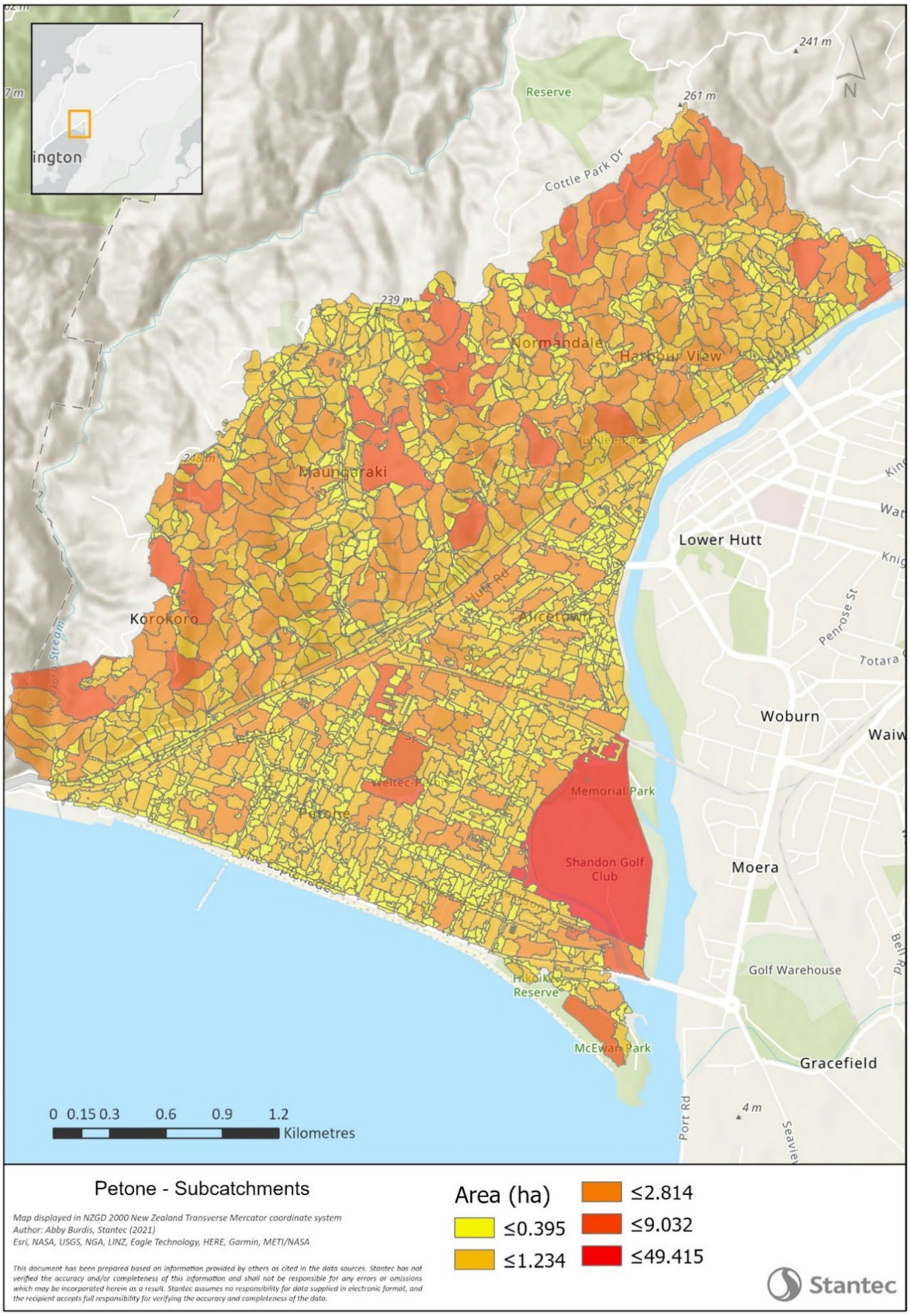


Figure 3-1 Subcatchment delineation for the Petone catchment showing total area

3.2.2 1D Subcatchment Hydrology

The Regional Hydrology Layers (discussed in Section 2.3.4) were analysed for any discrepancies/errors where the incorrect CN, IA, or roughness values had been applied. Some of the changes made to regional layers can be found in Appendix E . Instances of high IA and low CN values usually occurred at transition points from urban to rural areas where there were incorrect overlaps of the land use and soil type base layers. Where the IA was unrealistically high, the soil class was dropped one category from well-draining soil class A to a moderately draining soil class B. This gave a more realistic CN value of 46 and an IA value of around 29.8 for these areas.

3.2.2.1 Curve Numbers

Curve numbers were assigned for each subcatchment based on the Regional Curve Number Layer. A single composite value was determined using zonal statistics in ArcGIS with subcatchment polygons and the Regional Curve Number layer. Figure 3-2 shows the distribution of composite curve numbers for each subcatchment across the Petone and Korokoro catchments.

3.2.2.2 Initial Abstraction

Similar to the curve number assignment, Initial Abstraction values were assigned for each subcatchment based on the Regional Curve Number layer. A single composite value was determined using zonal statistics in ArcGIS with subcatchment polygons and the Regional Initial Abstraction layer. Figure 3-3**.** shows the distribution of composite initial abstractions for each subcatchment across the Petone and Korokoro catchments.

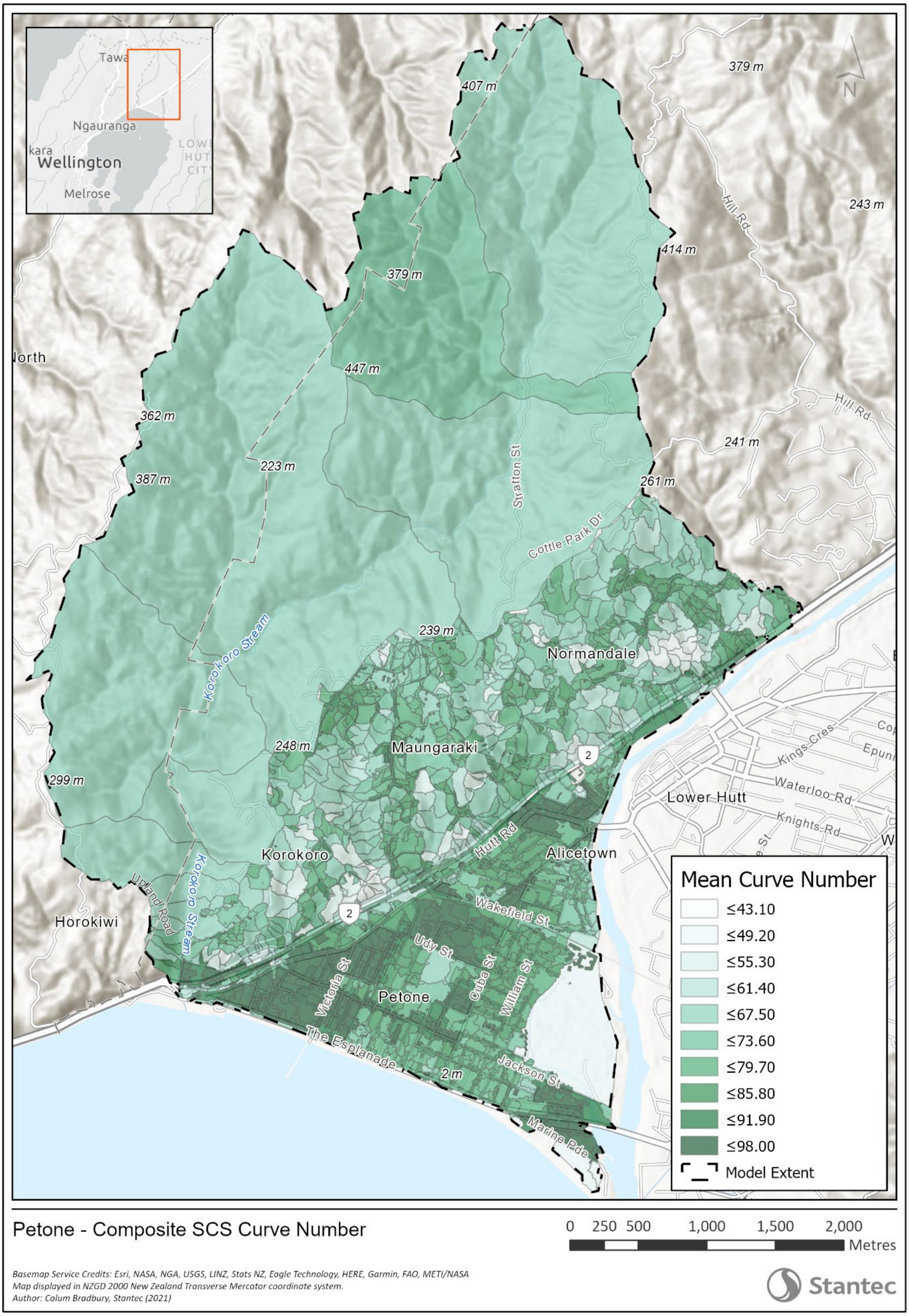


Figure 3-2 Composite SCS curve numbers assigned to each subcatchment.

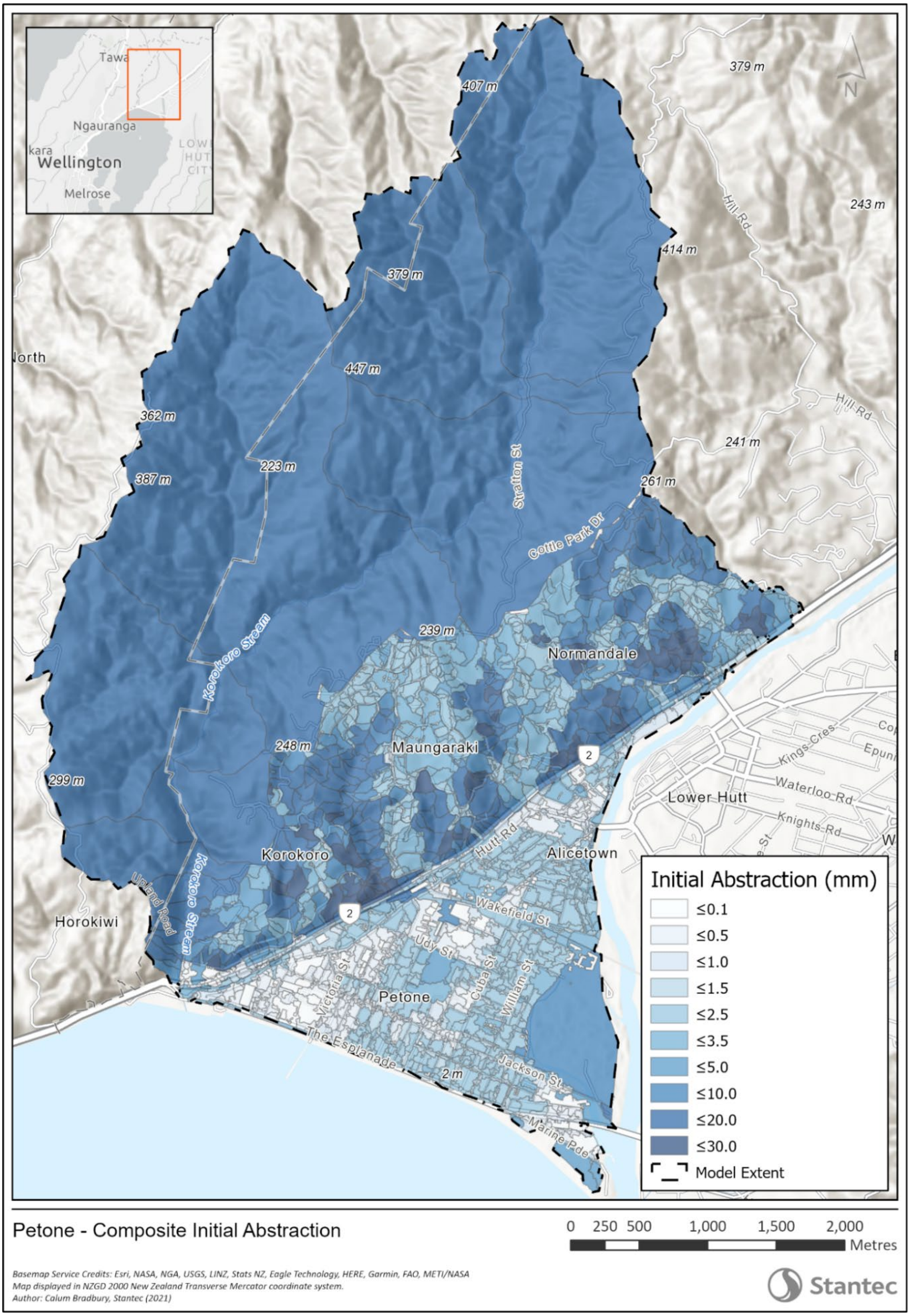


Figure 3-3 Composite Initial Abstraction values assigned to each subcatchment.

3.2.2.3 Time of Concentration

The WWL Hydrology Quick Reference Guide (Cardno, 2016) outlines the required components of runoff and associated time of concentration (ToC) calculations necessary to estimate a ToC for a model subcatchment. A summary of the approach is provided in the WWL Modelling Specifications V5 (Wellington Water Ltd, 2017). The method splits the calculation across four different flow regimes: overland sheet flow, shallow concentrated flow, gutter flow, and pipe flow. Calculations are conducted for each method by splitting up the automatically generated streamlines into sections based on roughness layers and road polygons. A final combined ToC is then assigned to each subcatchment.

3.2.3 HEC-HMS Hydrological Model – Korokoro Stream

The Korokoro Stream outlets to the southwest of the Petone catchment into the Wellington Harbour. The Korokoro catchment is predominantly rural but has a significant impact on flooding in Petone along SH2. A hydrological inflow has been applied to the Petone model through development of a HEC-HMS model and is described in the following sections.

The Korokoro HEC-HMS model was split into eight subcatchments to allow for river reach routing. Each of these subcatchments were defined following 5m contour lines developed from the 1m LINZ 2013 LiDAR. Streamlines representing the longest flow path from the top of the catchment to the bottom and the river reaches linking the subcatchments to the outlet were then manually drawn following these 5m contour lines. An overview of the Korokoro catchment is shown in Figure 3-4**.** and a schematic of the Korokoro HEC-HMS model is shown in Figure 3-5

Composite CN and IA values were assigned to each subcatchment as per the methodology described in Sections 3.2.2.1 and 3.2.2.2. The slope of each subcatchment and reach was then calculated by dividing the difference in elevation over the stream length and based on the longest water path, see Figure 3-4**.** & Table 3-1. A Manning's n roughness of 0.045 was assumed for each river reach. This was deemed appropriate for the natural streambed in this predominantly rural catchment.

Rainfall excess from each subcatchment was routed through river reaches using the Muskingum Cunge method and the parameters stated in Table 3-2**.** The resulting hydrograph was then input as an inflow boundary in ICM. No frequency analysis was undertaken for the Korokoro catchment as the nearby rainfall gauge's (Mill Road gauge) record has only been recording flow since 2017, even though it was installed around the1980s.

Validation of the HEC-HMS model and further details can be found in Appendix F

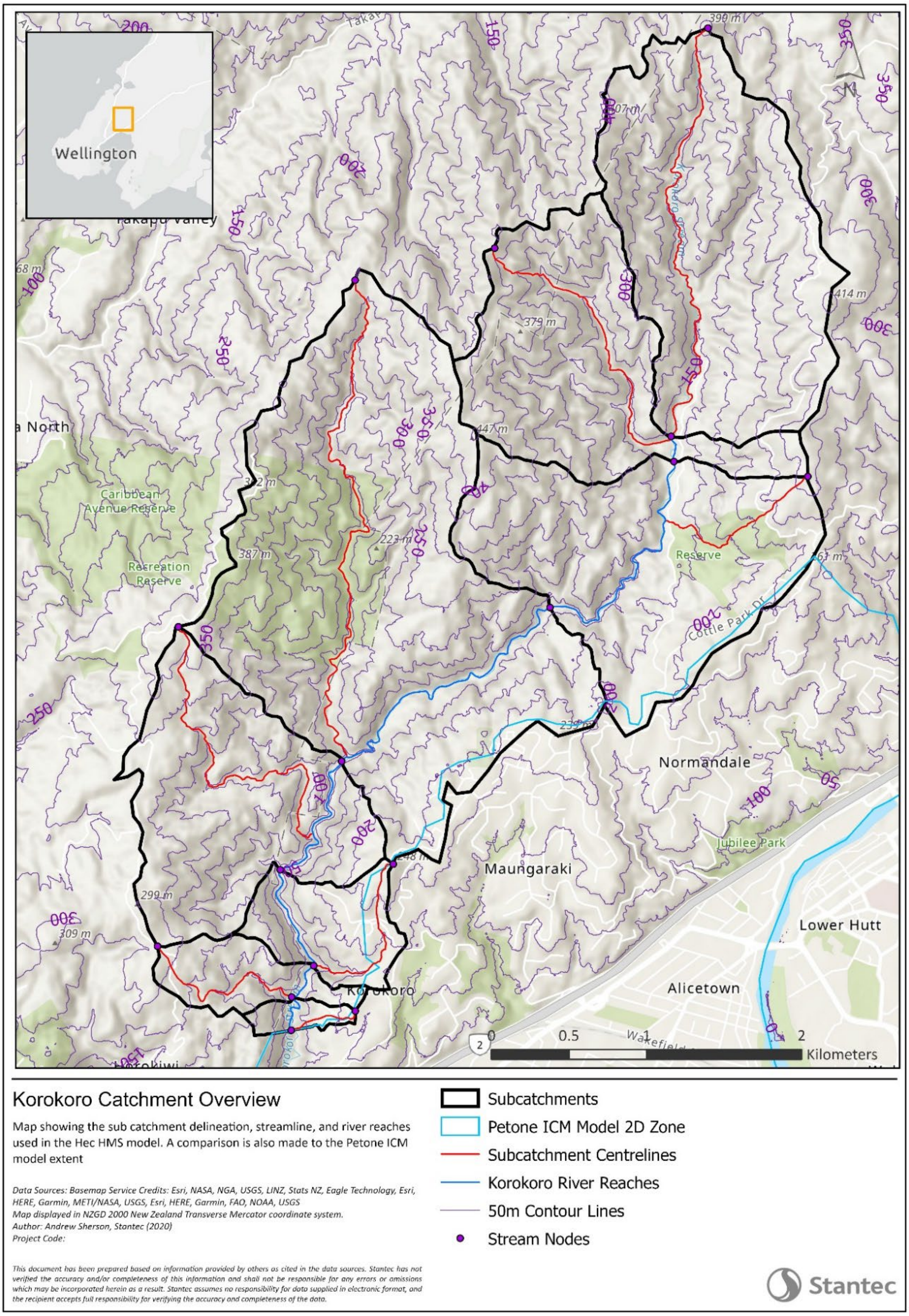


Figure 3-4 Korokoro HEC-HMS model overview.

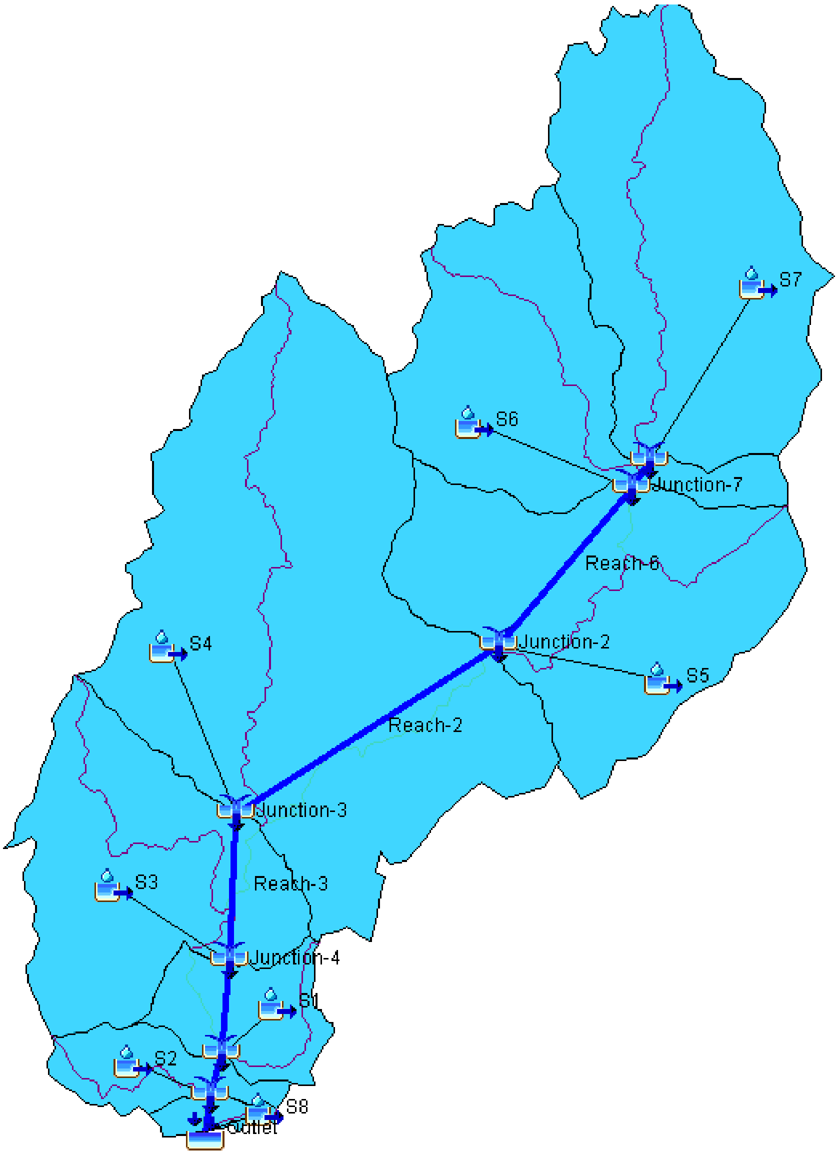


Figure 3-5 HEC-HMS schematic of the Korokoro catchment.

Table 3-1: HEC-HMS subcatchment parameters

Catchment	Area (km2)	Curve Number	Impervious Percent %	Initial Abstraction (mm)	Ramser Kirpich Tc (min)	Bransby Williams TC (min)	Average TC (min)	Lag (0.6x TC)
1	0.570	66.09	14.10	11.02	8.76	26.20	17.48	10.49
2	0.423	64.38	5.01	13.07	8.99	28.25	18.62	11.17
3	1.978	64.88	1.42	13.76	21.43	63.93	42.68	25.61
4	5.078	64.33	1.93	14.00	30.77	82.37	56.57	33.94
5	2.513	65.86	4.76	13.30	19.49	54.11	36.80	22.08
6	1.947	68.01	0.46	12.13	16.82	47.64	32.23	19.34
7	2.896	66.75	1.58	12.96	25.03	69.62	47.32	28.39
8	0.114	64.34	8.08	11.51	3.91	11.69	7.80	4.68

Table 3-2: HEC-HMS subcatchment parameters

	Length (m)	Top height (m)	Bottom height (m)	Slope (m/m)	Manning's n	Bottom width (m)	Side slope (xH:1V)
Reach 1	176	119.36	114.50	0.03	0.045	10	1
Reach 2	2116	79.86	45.65	0.02	0.045	10	1
Reach 3	1037	45.65	29.58	0.02	0.045	10	1
Reach 4	735	29.58	19.24	0.01	0.045	10	1
Reach 5	304	19.24	14.78	0.01	0.045	30	0.5
Reach 6	1837	114.50	79.86	0.02	0.045	10	1
Reach 7	294	14.78	14.16	0.00	0.045	30	0.5

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 subcatchment 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 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 inter-connections including sumps, manholes, bank lines, and inline banks (Section 3.3.5).

The model consists of over 6000 pipes, ~6000 nodes, two open channels/rivers, five pumping stations, five detention ponds, and 87 flap valves. As the asset data received had many missing attributes, an interpolation process was conducted in UMM to estimate asset information based on set rules outlined in the *Wellington Water Stormwater Modelling Specifications* (Wellington Water Ltd, 2017). As-built drawings were used where available, particularly for complex structures such as detention dams and pumping stations. Where data were unavailable, engineering judgement was used for many of the edits. For further details on the rules and assumptions applied in UMM, refer to the updated *WWL Stormwater Hydraulic Modelling Specifications* (Wellington Water Ltd, 2017). An overview of the hydraulic elements of the model is shown in Figure 3-6.

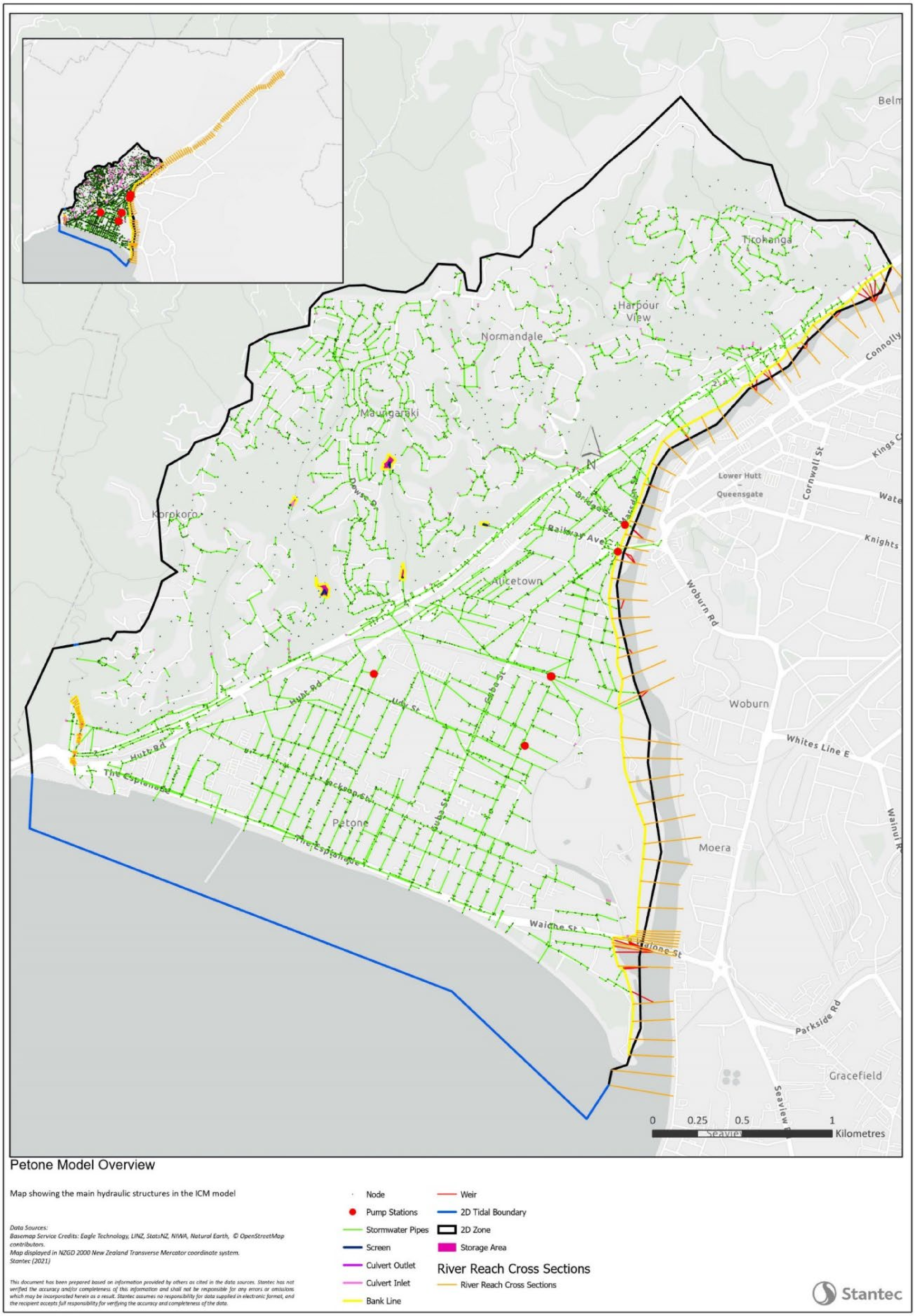


Figure 3-6 Petone Hydraulic model overview.

3.3.2 1D Reticulated Network

The 1D network model is used to simulate hydraulic processes within the reticulated stormwater network. The network includes all public stormwater pipes, manholes, sumps, flap valves, pumping stations and detention ponds. No private stormwater assets have been included unless specified.

The following stormwater asset GIS layers were supplied by HCC:

- Storm_fixtures_private
- Storm_pipes_private
- Storm_sumps
- Storm_sumps_unmatched
- Stormwater_fixtures
- Stormwater_fixtures_private
- Stormwater_lateral_fixtures
- Stormwater_lateral_pipes
- Stormwater_pipes
- Stormwater_pipes_private
- Tnz_culvert_fixtures
- Tnz_culvert_pipes

The model also includes some assets that were identified from the as-built drawings (Section 2.2.3), ortho-photos and site survey undertaken by Cuttriss Consultants (Section 2.2.5). Several pipes were added to the model to connect isolated drainage sumps to the network based on these drawings. Multiple sumps were added based on Google Street View.

3.3.2.1 Node Ground Levels

- Where node (manhole or sump) ground levels were available in the GIS data supplied by WWL, they have been used. In instances where there are no available data, the ground levels were updated from the DTM.

3.3.2.2 Node Invert Levels

- Where WWL's GIS data for pipes contained upstream or downstream invert levels, these were applied to the connected nodes.
- Where WWL's GIS node data contained depths and no invert level, data were extracted from the connected pipes and the invert level was calculated from the depth and the ground level.
- If no other invert level data were available, the invert was estimated based on the connected pipe diameters and an assumed pipe cover depth of 450mm.
- Where invert levels resulted in negative slopes, the invert levels were adjusted manually using engineering judgement.

3.3.2.3 Sumps

Sump lengths and widths were taken from the WWL Modelling Specifications V5 (Wellington Water Ltd, 2017) and in most cases are equal to 0.675m and 0.53m, respectively.

Unknown sump invert levels were based on the depth in the modelling specifications provided by WWL and ground levels for the sumps extracted from the DTM.

In instances where sump lead invert levels were not available, an assumed upstream invert level 0.45m above the sump invert level and an assumed downstream invert level based on an assumed slope of 5% was used. Where this would result in the downstream invert being below the invert of the downstream node or close to the ground level of the downstream node, the gradient was adjusted to 1% or 10%. If the adjusted levels were still too low or too high, the downstream invert was set to the manhole invert or based on ground level, pipe diameter and a minimum pipe cover of 100mm. Sump lead invert levels calculated using this method could be identified in the metadata using the "sump" code.

Sump head discharge curves, were attributed to each sump, based on the sump's configuration. These can be found in Appendix I Figure I 1. The curves were developed for the WWL Stormwater Modelling Panel and represent the behaviour of water at different levels.

3.3.2.4 Pipe Connectivity

- Connectivity of the network was checked for issues which included: pipes digitised in the wrong direction, pipes not connected to an outlet structure, sumps not connected to a pipe, sumps and pipes that exist in the network but not in the GIS layers, duplications, or bifurcations in the network.
- Errors that could be reasonably corrected without survey were resolved using engineering judgement and/or LiDAR information. This often involved adjusting link connectivity or splitting links and creating nodes.

- In the case of missing junctions, the split links were named with the asset ID of the original link suffixed with "_n", where n is an integer. The created node was labelled with the asset ID of the original link suffixed with "_a", or "_b" in the case where the link needs to be split multiple times.
- Similarly, where links do not terminate at a node, a node was created and labelled with the links' asset ID suffixed with '_a' or '_b'. In some instances, the coordinates of the new node were used as the ID, using the format 'coordinate number Y coordinate number X' (e.g., n1760102**X**5437057**Y**).
- 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 '_1' or with the asset ID of the upstream node suffixed with a 'u' and the connected downstream node suffixed with a 'd' (e.g., '610007R01060u423009R01060d')

3.3.2.5 Pipe Diameters

Of the 5,406 pipes in the network with GIS data, 189 showed a reduction in downstream diameter. 98% of these decreases have diameters based on information included in the GIS database. Only 2% of the interpolated or assumed diameters were updated using engineering judgement. The remaining 98% with GIS flagged values were not changed unless there was a significant jump in pipe size and as-built information was available. See Appendix H for examples of updates made to pipe diameters.

3.3.2.6 Pipe Losses

Pipe head losses have been calculated using ICM's head loss calculation through manholes, where the energy losses due to the turbulent transition between a conduit and a manhole were approximated. Calculations have been conducted network-wide using an inference tool that looks at the angle of the incoming pipe in relation to the manhole and the difference in invert level to calculate the energy lost using the following equation: $h = ku \times ks \times kv \times (\frac{v^2}{2g})$ (Innovyze, 2015)

Where Δh is head loss; ku is a user-defined factor calculated from the angle of approach, ks is the surcharge ratio coefficient calculated from the difference in the invert of the pipe and manhole, kv the velocity coefficient based on the direction of flow, v the flow velocity and g acceleration due to gravity.

3.3.2.7 Negative Pipe Gradient Fixes

There were widespread negative grades in the model (that is, pipes with a downstream invert higher than the upstream invert). These negative grades arose because of LiDAR inconsistencies and because 80% of assets derive their inverts from cover levels. In the initial model build process using Urban Model Manager (UMM), many of these negative grades were rectified using engineering judgement to interpolate and reorient missing or incorrect inverts. After further discussions with WWL around the impact of significant assumptions in the flatter areas of Petone where there was more LiDAR inconstancy and a more interconnected network, it was agreed to remove negative grades for all pipes that had assumed or interpolated levels. Each negative grade was then rectified if possible. A few examples of this rectification process can be found in Appendix H .

Not all negative grades could be rectified and were either consistent with the as-built drawing or could not be fixed without further survey. For instance, 10 of the 29 outlets draining to the harbour required negative grade upstream pipes due to inconsistencies in the ground level that could not be changed without significant assumptions. Some of these culverts may be buried by sand dunes, and so drain below ground level or drain further out into the harbour. Forty negative grades were flagged either from as-builts or from the GIS database. In these instances, it could not be assumed that these negative grades are incorrect. The remaining five negative grade pipes were pump station rising mains. Figure 3-6 shows the remaining pipes with negative gradients in the network.

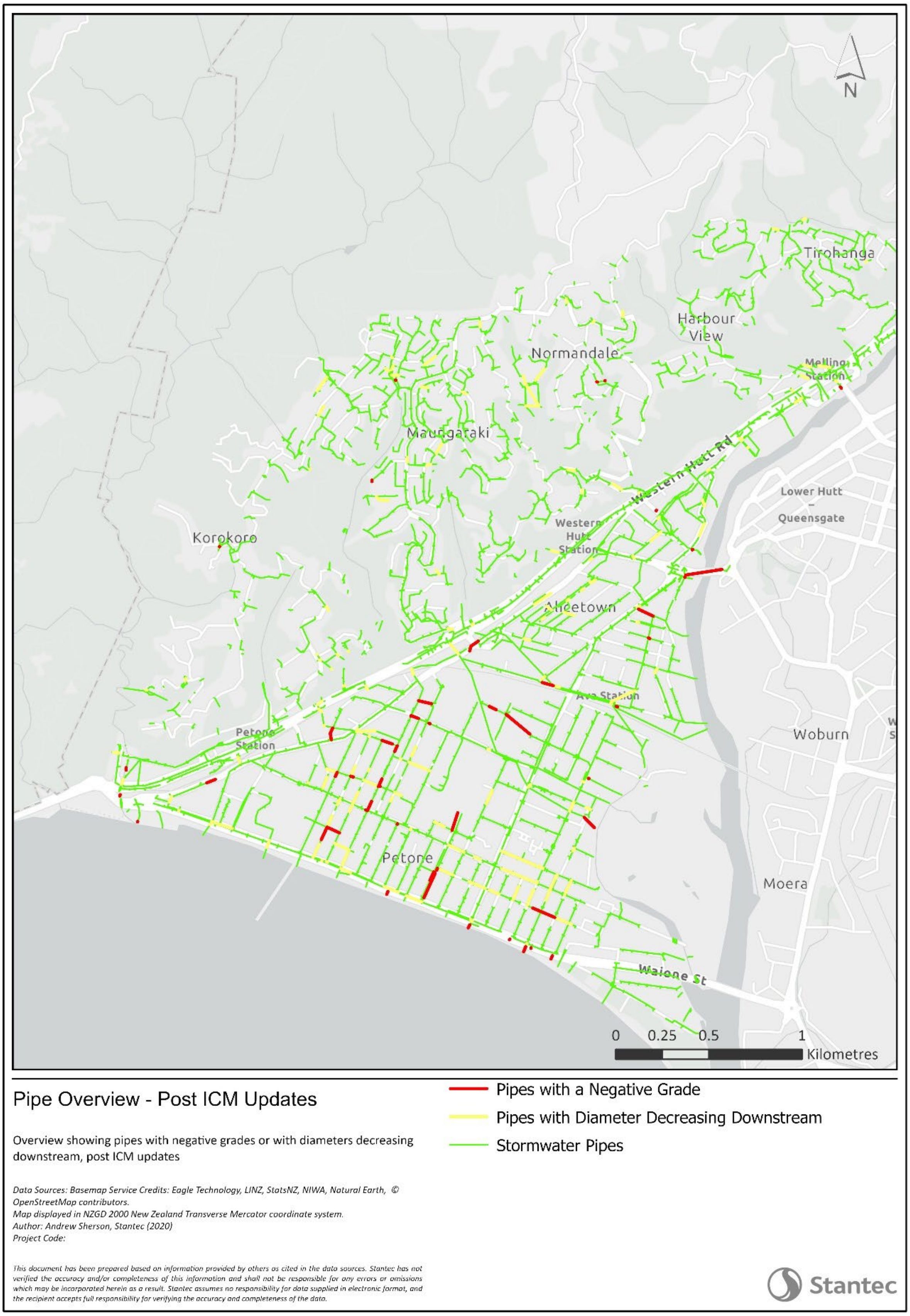


Figure 3-7 Remaining negative grade and decreasing pipe diameters, post fixes.

3.3.2.8 1D Culverts

The lower reaches of the Korokoro Stream flow underneath multiple buildings near SH2. Flow was conveyed underneath each building via a 1D culvert with a unique shape, based on survey data. Each conduit had a bottom Manning's n of 0.045, to represent stream roughness, and a top Manning's n of 0.013 for smooth concrete. Culvert inlets and outlets connected these culverts to the river reaches through a break node to calculate head losses. The culvert inlet head loss equations followed standard principals in ICM, in which typical inlet structure parameters were selected based on site photos and the presence or absence of head walls.

In some situations, the channel under the buildings had multiple pylons or other structures that split the flow. To model these split channels, multiple separate culverts were used to convey water. Any 2D outfall that was in the 2D zone, under a building was changed to a break node to convey water into the culverts without constricting flow along the riverbed.

(a) (b)

Figure 3-8 (a) A building over Korokoro Stream with multiple walls to separate flow. Photos Source (Cuttriss, 2017). (b) Model representation in ICM.

For the model runs, it was assumed that each culvert has the same capacity as that which was surveyed. Any sedimentation, such as that depicted in Figure 3 7, was assumed to remain. This assumption was made as it is difficult to know how much of the sediment will become mobile during a flood. This assumption results in modelling about half of the full capacity of the three pipes shown above, see Table 3 3.

Table 3-3: Comparison between the full capacity and surveyed capacity of the culverts in

Culvert	Modelled area with blockage included (m^2)	Total area (m^2)
1	2.8	2.8
2	2.3	4.2
3	0.6	3.8
Total	5.7	10.8

Several bridges also span the Korokoro Stream. Like the building underpasses, these bridges were modelled as unique culverts. Culverts were chosen instead of ICM bridge connections as the bridge links were found to be unstable due to 1) the proximity of the bridge structures to each other and 2) there was not enough information to construct them without making significant assumptions. Instead, a culvert with an overtopping weir structure was modelled.

Head losses were accounted for using culvert inlet elements which enabled the selection of inlet control parameters based on typical culvert inlet shapes in ICM.

3.3.2.9 Overtopping Structures

For the larger bridges with known top widths, a broad crested weir was used to model any overtopping flow. These weirs had a width based on the distance between cross-sections, a height and crest level from the survey data and a runoff coefficient based on the Henderson 1966 equation $Cd = 0.65/(L+H/P)^{0.5}$ where L is the length of the weir, P the height, and H the upstream level of water above the crest (assumed to be 0.5m). The deck of the bridge was then voided out with a storage area so water could not flow across the 2D surface and fall in-between the bank lines. For smaller structures, like footbridges that had no measured top width, sharp-edged weirs were used. The sharp-crested weirs were given a runoff coefficient of 0.58, following typical ICM weir parameters stated in 'Essential water and wastewater calculations for engineers and operators' (John & Paul, 2007).

3.3.2.10 Flap Valves

Flap valves were added to each outfall along the Hutt River following WWL recommendations, as the available GIS files did not include this information. Since there was limited information regarding the type of flap valve, the ICM default head loss of 1.0 was selected for the discharge coefficient, the invert and diameter of the flap valve were then copied from the outlet and pipe information respectively.

Penstocks were also added to 23 manholes just upstream of the Hutt River, following as-builts & discussions with WWL operators. These penstocks were added to prevent water bypassing the outlet flap valves by entering the network through manholes on the Hutt River side of the stop bank (Figure 3-9).

Figure 3-9 Manhole on river side of stop bank. Google Street View, taken by Duane Wilkins.

Following a short investigation into the outlets along the harbour and their impact on flooding in the network, five standard flap valves with a discharge coefficient of 1 and a diameter the same as the attached culvert were added following WWL recommendations. The locations of these flap valves are shown in Figure 3-9**.** The outlet classified as "unknown" was assumed to also be a standard flap valve.

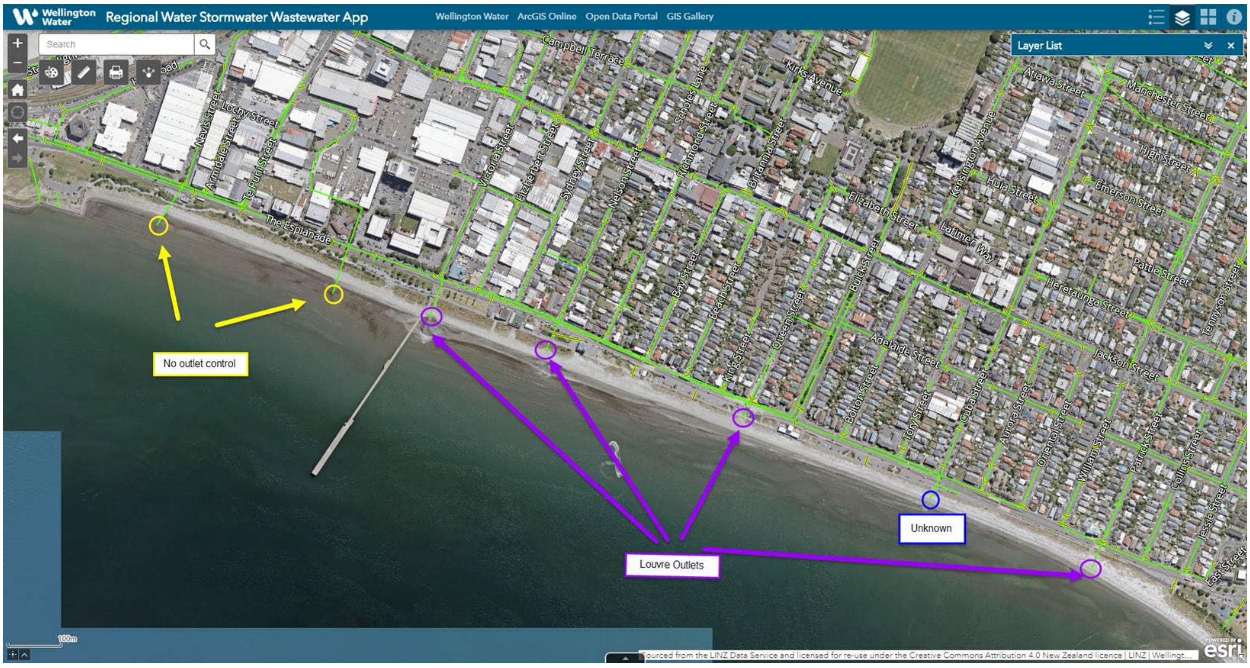


Figure 3-10 Known flap valve locations

3.3.2.1 Detention Dams

Flood detention dams were modelled as 1D elements to allow control over storage volumes and outflows. The total volume of each dam was modelled using storage nodes with ground levels derived from the DTM and an assumed flood height of 10m above ground level to allow for over bank spill. These volume to area curves defined the total storage volume for different water levels in each dam using the dam shape, derived from the DEM. The calculation was undertaken using GIS tools and was based on the closest 0.2m contour line to as-built bank levels.

Once constructed, an iterative frustum equation was run to correct any issues in calculating the volume from the DTM. Overall, the Frustum equation was used to create volume to area curves based on the total volume from as-built drawings. The curve and equation then modified the areas for each height, producing volumes that aligned with the as-builts.

The storage nodes were then joined to a storage area that covered the full extent of the dam, drawn by following the closest contour line to the dam crest height. The resulting storage area shape was then checked against the as-built drawings to confirm that it matched the dam structure. The main purpose of the storage area was to void the dam from the 2D zone to prevent any double counting of water ponding on the 2D surface as well as in the 1D storage node. A bank line was then drawn around the edge of the storage area, connecting the storage node to the 2D zone. Dummy nodes were then inserted to connect the various link types, such as weirs and screens together (see Figure 3-10). Overall, the structure for each dam was based on as-built information.

Each dam included a low flow structure to allow water to discharge from the dam. The low flow structures were constructed from screens, weirs, orifices, drop structures/manholes, conduits, and culvert inlets in ICM (Figure 3-11). Since not all the required information could be obtained from the as-built information, some assumptions had to be made. For example, the width and number of bars in the screens had to be assumed from site photographs, weir runoff coefficients were assumed to follow typical ICM values of 0.58, and culvert inlets were assumed to follow standard designs set out in ICM in 'Essential water and wastewater calculations for engineers and operators' (John & Paul, 2007). Overall, only a few assumptions were required, as most information for the dams were detailed in the as-built drawings. For example, the weir used to represent flow over the Dowse Dam low inlet structure (Figure 3-11) has a width based on the circumference of the orifice in the trash gate, stated in the as-built drawings.

Some detention dams had extra spillway structures, these structures were built in a similar way, where weirs were used to control overflow levels (Table 3-4).

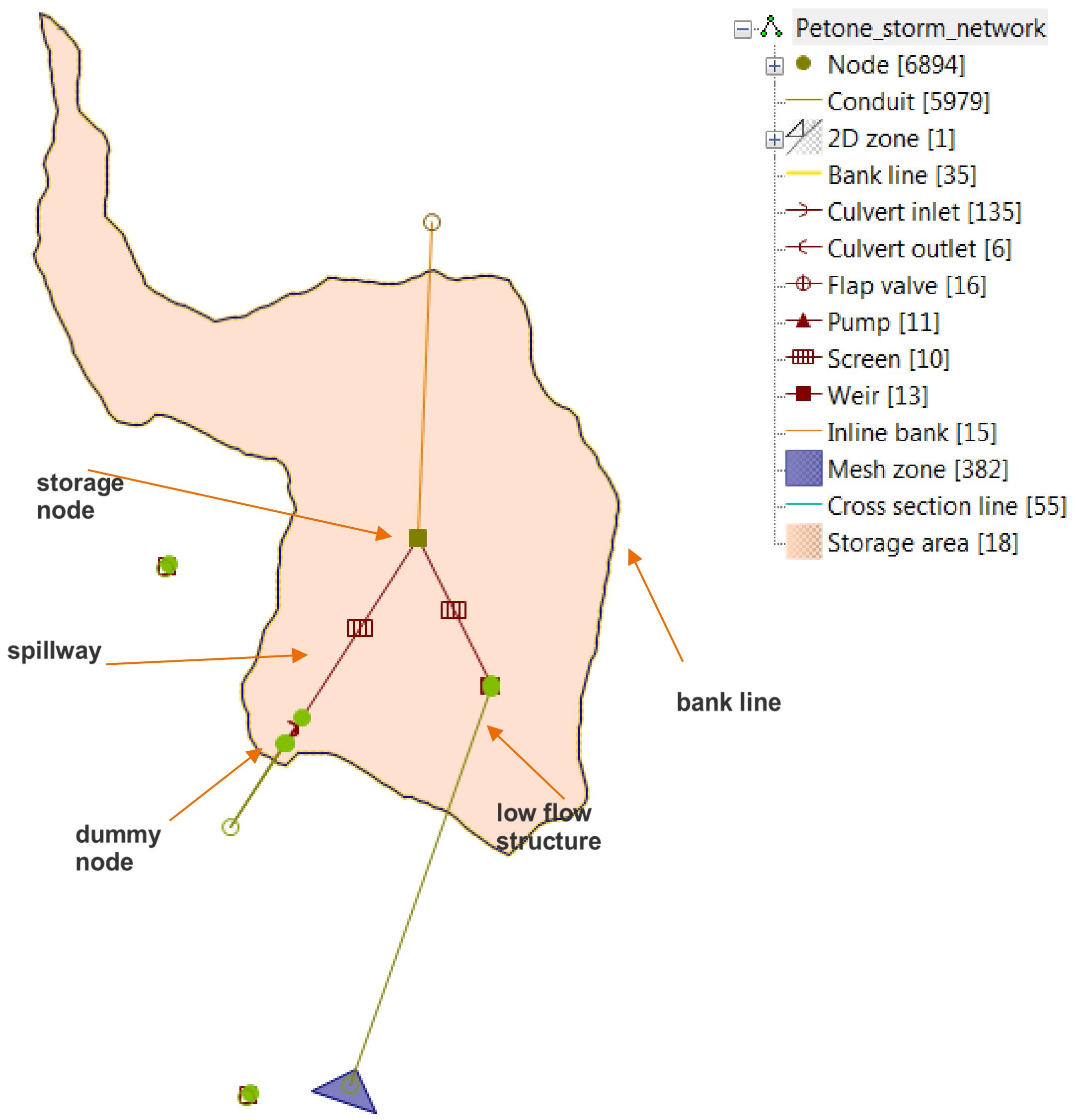


Figure 3-11 : 1D detention dam schematisation in ICM.

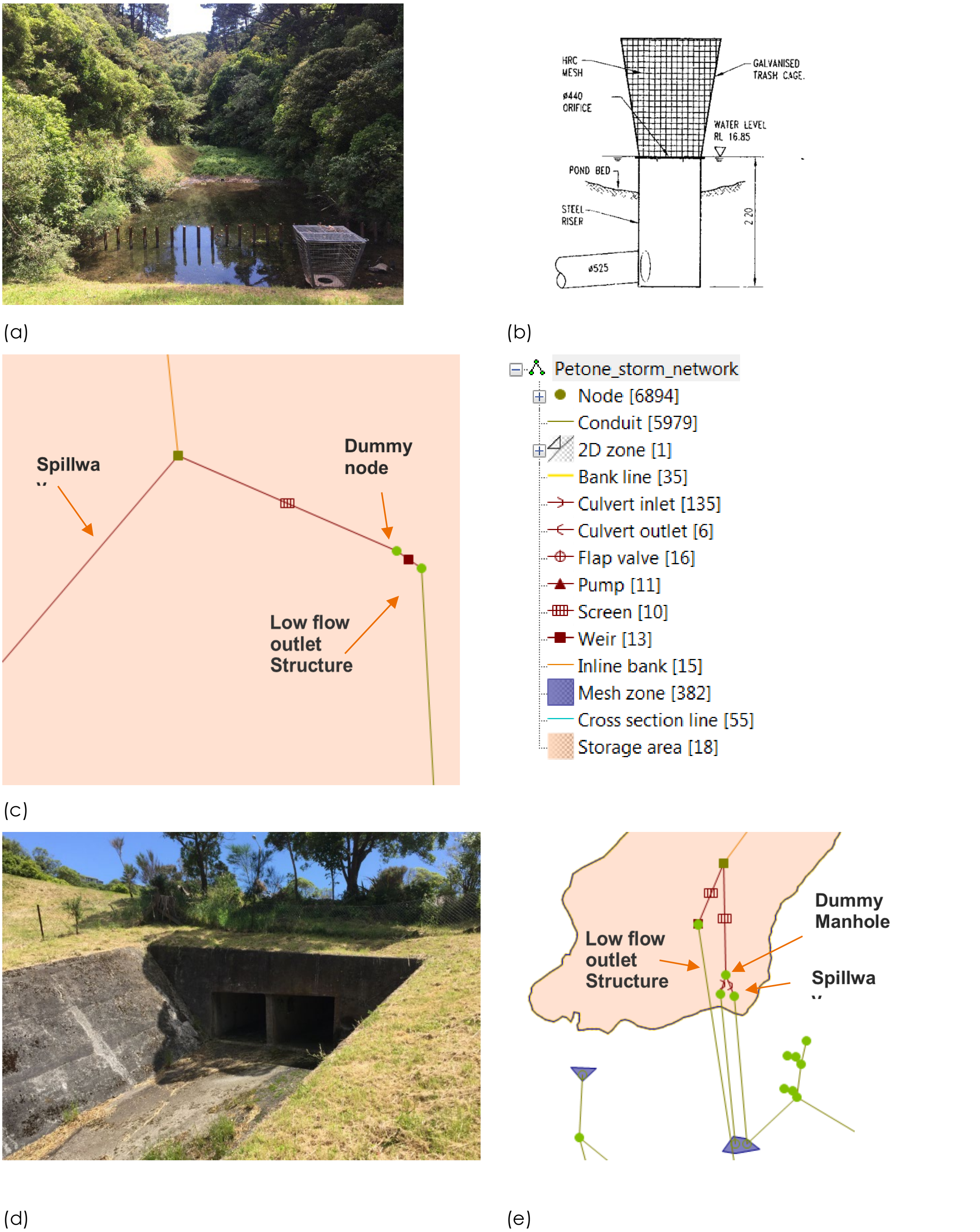


Figure 3-12: Various detention dam structures. (a) Dowse Drive detention dam. (b) As-built drawing of the Dowse Dam, low flow inlet structure (Cuttriss, 2000) (c) ICM layout of Dowse Dam, showing a close up of the storage node and low flow structure. (d) Mulburry Street dam, spillway. (e) Overview of the Mulberry Street dam setup in ICM.

Table 3-4: Detention dam characteristics

Detention Dam	Dam Crest level (m)	Volume (m^3)	Low flow Outlet structure Invert	Outlet structure components	Spillway level	Spillway structure
Mulberry Street	120.2	12,310	112.9 side orifice 113.2 top orifice	Trash cage over an Inlet drop structure with a side and top orifice	118.5	Debris arrestor and two buried concrete box culverts underneath the road connected to a concrete spillway.
Stanhope Grove	29.4	13,400	22.299 side orifice 22.39 top orifice	Large trash cage over an inlet drop structure with a side and top orifice	28.18	Debris arrestor, three concrete box culverts under the road connected to a concrete spillway
Dowse Drive	21.2	4,260	16.85	Trash cage over inlet drop structure with one orifice	21.00	Large flat concrete spillway with short walls
Barberry Grove	93.4	700	92.017 and 91.617 (low flow pipes)	Three outlet flow structures. Only one known. (Trash cage hat above a manhole drop structure)	N/A	None, has a high-level outflow structure instead with an invert of 93.017 for the drop structure and 93.317 for the trash cage hat
Jacaranda Grove	30.3m	2,100	25.39 (main inlet structure) 29.6 (secondary inlet pipe)	Trash cage over inlet drop structure with one orifice	29.94	Concrete culvert under an earth dam, connected to a concrete spillway that drains back into the network through an unknown pipe

The two Korokoro Dams were not included in the model as they are assumed to not have enough storage capacity to greatly affect the results from the 15km^2 catchment. This is based on apparent high levels of sedimentation and constant discharge as shown in

. Both Korokoro Dams were decommissioned in the late 20th century and have uncontrolled spillways (Engineering New Zealand, 2019). Like the Korokoro Dam, no information was available for Percy's Reserve pond structure and so was left out of the model. The dam is not expected to add a significant amount of storage to attenuate floodwaters as the pond is kept full.

3.3.2.2 Pump Stations

There are five stormwater pumping stations in the Petone area and all have been included in the model as 1D model components. The following describes how the pumps have been schematised in the model:

- The pumps were added as pump links with characteristics based on as-built drawings. These drawings contained pump curves, storage volumes and invert levels for most pumps (**Table 3-5**). All but two pumps had defined pump curves that could be established from the drawings or pump serial numbers and were input to ICM as rotodynamic pumps. Those that did not have an associated pump curve were assumed to have a constant discharge and were added as fixed level pumps.
- The total storage for each pump was modelled as a manhole with a defined volume based on the as-built drawings.
- Flap valves were used to model any valve structures connected to the pumps or manholes preventing water flowing in the wrong direction, the diameters of which were assumed to be the same size as the connected links.
- Screens were placed upstream some of the pumps, following the as-built drawings. If the dimensions were not stated in the as built, they were assumed based on the size of the connected storage area. The size of the bars and gaps was assumed based on engineering judgement.
- Rising mains were set to "forced mains" within ICM to remain pressurised in the simulation. Attached manholes had their flood type set to "sealed" to maintain pressure through the system.

Table 3-5: Stormwater pump characteristics

Pumping station	Location	No. of Pumps	Duty / Standby / Assist	Pump Type	Name/serial No	Capacity (l/s)	Inlet pipe Dia.	Discharge to	Data Source
John Street	123 Richmond Street	2	Duty, Duty	Rotodynamic	3200 AMO	Pump 1 = 138 Pump 2 = 138 (pump 1 & 2 =276 assumed double pump)	600mm	Stormwater manhole 76m away	As-builts and Pump manual
Marsden Street	Abutment of Hutt River near Bridge Street	2	Duty, Duty	Fixed	Pump 1: CP3126 proposed (3300 as-built) Pump 2: 70-75BL submersible	Pump 1 = 240 Pump 2 = 56 (pumps 1 & 2 = 296) (Estimated from Pump type)	525mm	300mm connected to 100mm outlet to manhole 3m from wet well. 525mm discharge to Hutt River	As-builts, Pump manual, and Manufacturer information
William Street	104 William Street	3	Duty, Duty, Standby (not in model)	Rotodynamic		1000 per pump (Assumed)	1350mm	1200mm gravity main to the 'Dead' Arm of the Hutt with floodgates	As-builts, Pump manual, and Manufacturer information
Te Mome	Wakefield Street Lot DP 73017	3	Duty, Duty, Duty/ (possible standby)	Fixed		Pump 1 = 0 - 850, Pump 2 = 0-790, Pump 3 =0-440 Total = 2080 (stated in Pump manual, however it is not known if all three operate together)	1200mm	1050mm rising main	As-builts and Pump Manual
Tama Street	Embankment of the Ewen Bridge	2	Duty	Rotodynamic		Pump 1 = 45, Pump 2 = 111	450mm	150mm delivery main to manhole	As-builts and Pump Manual

3.3.2.3 2020 Updates to SH2 Network

The stormwater assets provided by WWL in GIS format did not include large sections of asset information along SH2 (see Figure 3-12.). The agreed approach with WWL was to include RAMMs node information in the model to fill in data gaps. Since the RAMMs information covers only nodes, the connections between these nodes and the network are assumed. The following assumptions were made:

- highway crossings occurred only at specific culvert sites
- culverts drain to the closest similar sized downstream pipe
- drainage paths followed the general shape of the terrain
- sumps always connect to manholes

The assumed pipe layouts were then discussed with WWL and refined.

New subcatchments were created for any new sumps (see Figure 3-13). Following advice from WWL, the subcatchments were drawn draining to groups of sumps rather than each individual sump like the rest of the network. This approach was used as the sumps are tightly packed together and generally drain to the same pipe main. Each subcatchment was drawn manually following contour lines and the existing catchments, replacing the larger catchments that existed before the added network, see Figure 3-13.

The time of concentration for these additional catchments was calculated following the methodology discussed in Section 3.2.2.3, to match UMM calculations for the rest of the model. It was assumed that the maximum flow length occurred for overland sheet flow (50m in urban environments), shallow concentrated flow for the remaining undeveloped sections, and gutter flow occurred along the road kerb and channel. SCS land use values were then calculated from the regional hydrology layers (Section 2.3.4).

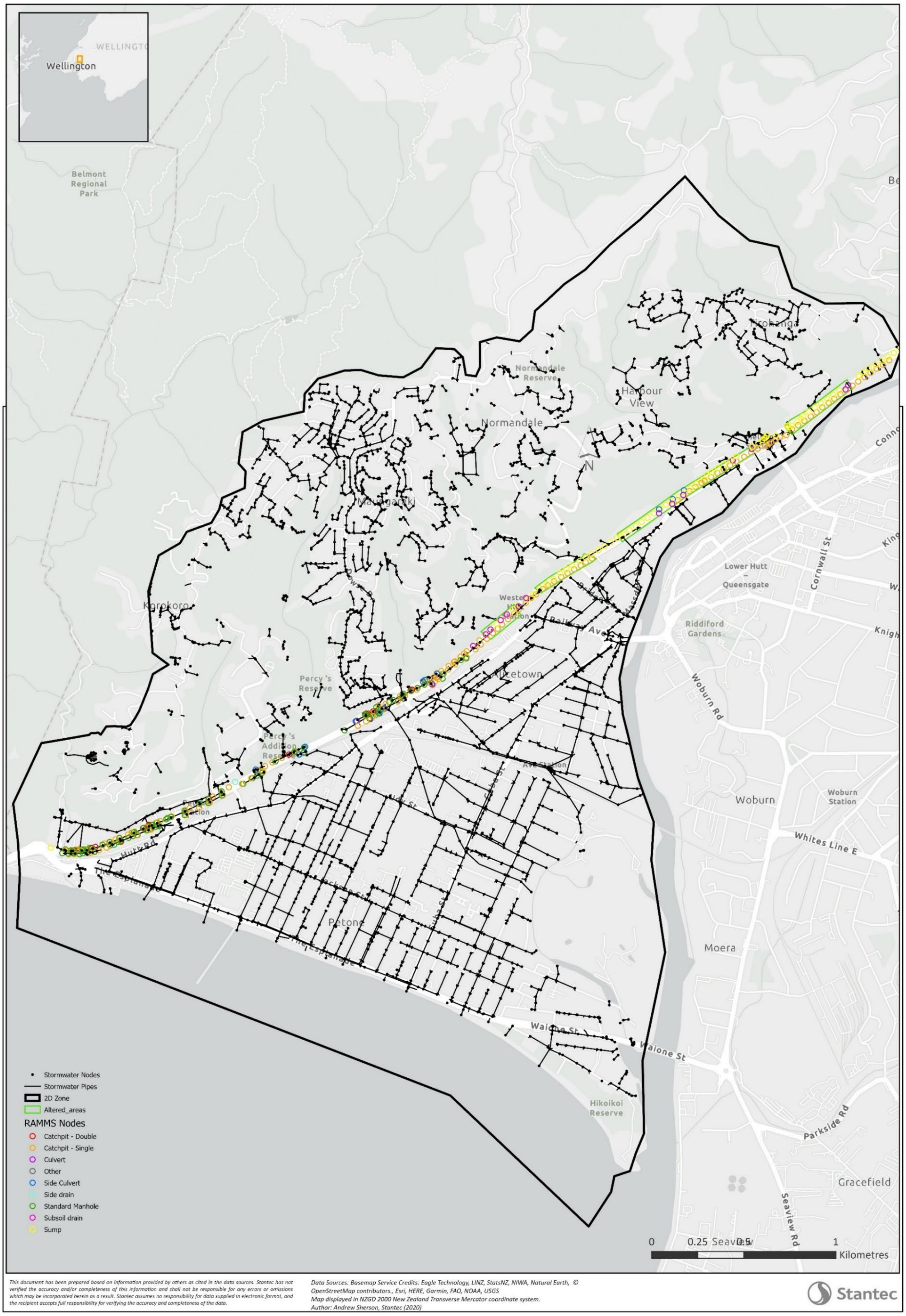


Figure 3-13 RAMMs Information along SH2. The areas highlighted in green boxes show gaps in the GIS fille

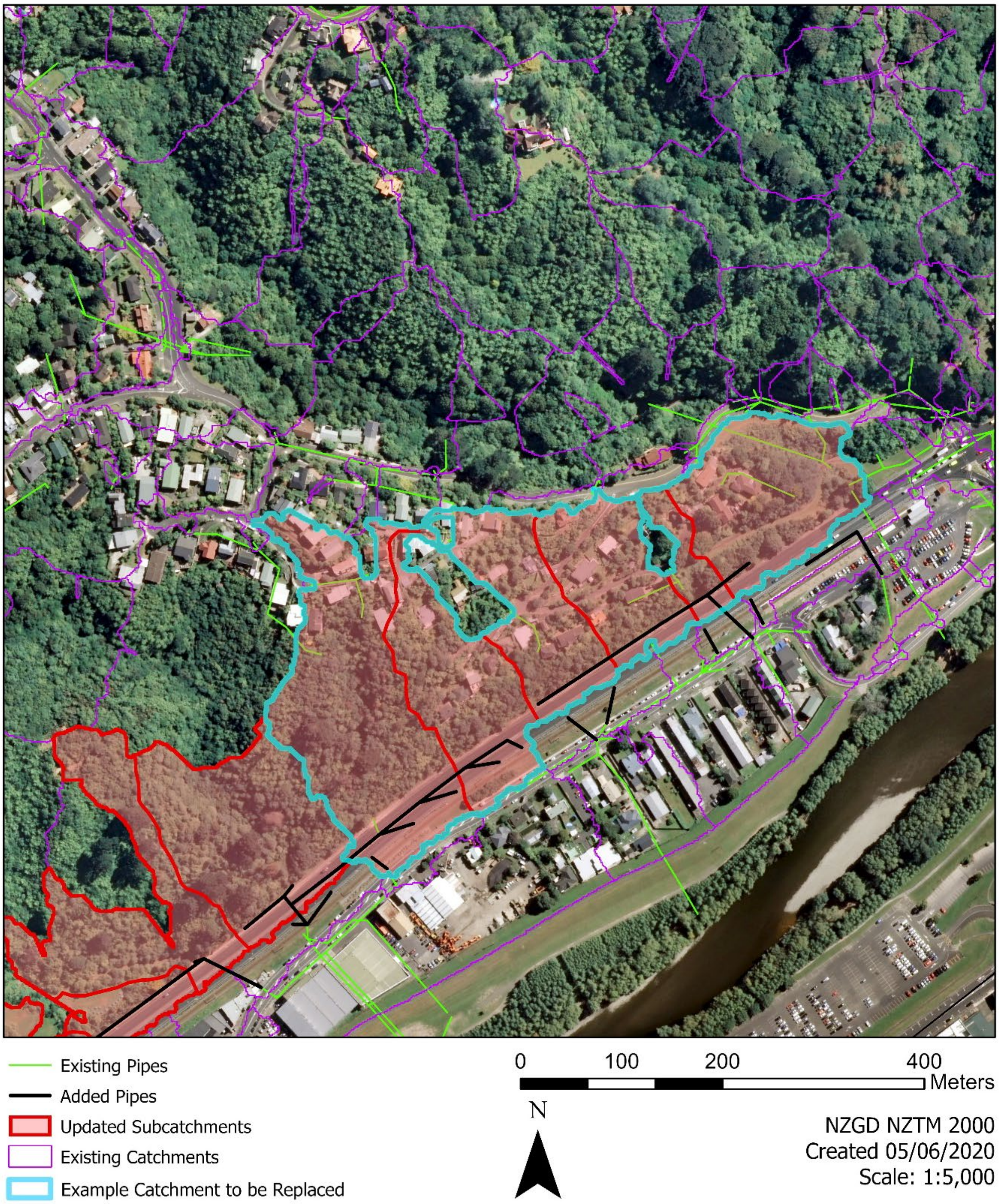


Figure 3-14 Subcatchment updates made to accommodate the SH2 network additions.

3.3.3 1D Channel Network

The lower reaches of the Korokoro Stream were modelled as 1D river reaches between other 1D model components (Figure 3-17). River reaches were used instead of relying on the 2D model to convey flow because several buildings were constructed over the top of the Korokoro Stream, which constricts flood flows (Figure 3-15.). These channel sections were not picked up in the DTM.

Cross-sections of the stream bed were surveyed by Cuttriss Consultants to understand the layout of the riverbed and adjacent building levels. From these cross-sections the river reaches were built, using interpolated sections every 10m. Bank lines were then drawn based on the edges of the cross-section lines. All points outside the bank crest were removed.

Inline banks were also drawn across the river on the upstream and downstream boundaries of the river reaches, allowing water to spill into the building polygons that sit over of the stream. The inline banks were purposely drawn over the top of the building footprints and were given a level based on the surveyed floor level of the building plus 200mm, to represent the flood level. Each bank line was given a discharge coefficient of 0.8 and a modular limit of 0.9, as per the WWL Modelling Specifications V5 (Wellington Water Ltd, 2017). The Hutt River is also modelled as a 1D river reach boundary condition and is discussed in Section 3.4.2.

Figure 3-15 Building constructed over the top of the Korokoro Stream. Photo source (Cuttriss, 2017)

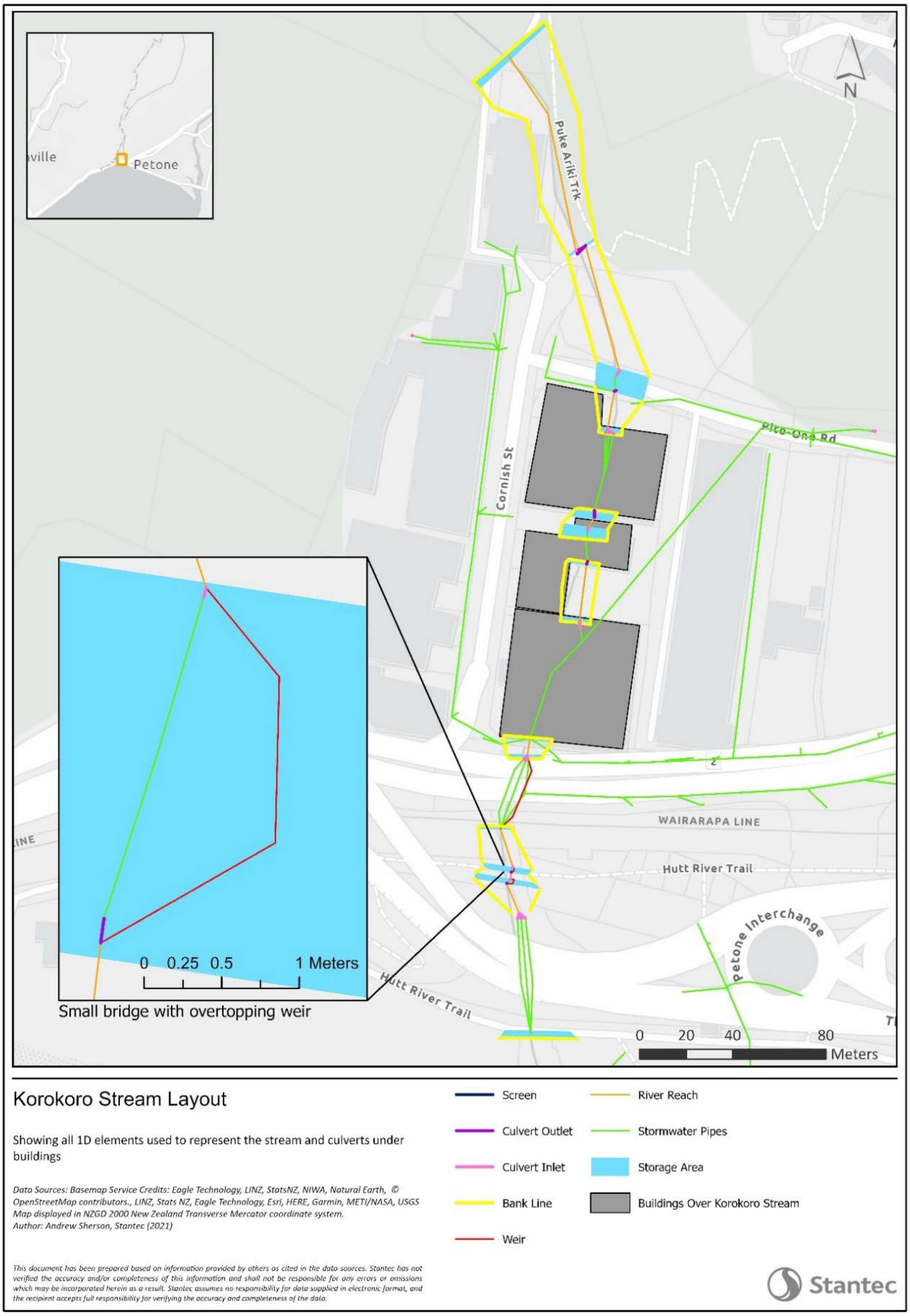


Figure 3-16: Korokoro Stream river reach and 1D pipe layout, showing the connection between culverts under the buildings and open river reaches.

3.3.4 2D Network

3.3.4.1 2D Mesh

The 2D zone developed for the model has an area of approximately 1.76km^2 and covers the urban area of Petone and parts of the Hutt River and Wellington Harbour. The small catchment to the southeast of the model associated with the Horokiwi Quarry was not included as the runoff from the quarry drains directly to the harbour and therefore does not directly impact the Petone stormwater network.

The maximum triangle size for the TIN mesh was set to 4m^2, with a minimum element area of 2m^2, except where specific mesh zones were placed. Each mesh triangle has a ground level based on the DTM or connected outfall level.

All buildings larger than 500m^2 were voided out of the mesh using simplified building footprints. These building footprints were then simplified to reduce the number of vertices and to remove small gaps between the building polygons to ensure no small mesh elements were slowing down the simulation.

3.3.4.2 Porous Walls

- Two large porous walls were added to the model to restrict flows and cause ponding where water is stored behind large wall-like structures. These were at the following locations:
- The SH2 median barrier. This wall was based on aerial pictures and Google Street View and assumed to have a height of 1.2m and a porosity of 0.
- Petone Esplanade wall adjacent to the beach. The 0.5m coastal wall has many entrances and holes, which were added to ICM as breaks in the wall. The wall varies in height, with some isolated sections up to 2m high. Overall, the wall was considered non-porous, despite having very small holes in the bottom of the coastal barrier, as they are generally blocked with debris.

Two smaller walls were also added to the model; these were the bounding wall along the spillway of the Dowse Dam which had an assumed height of 0.4m and a 1.2m high solid wall along the side of SH2 across the Korokoro Stream. Both were assumed to be non-porous.

3.3.4.3 Mesh Zones

Mesh zones were added to each inlet and outlet to create a wide flat region at the same level as the invert of the inlet or outlet Figure 3-16. This process is automated in UMM which creates a triangle larger than the standard mesh element size at the right invert. The exact size of the triangle was set based on the diameter of the pipe with larger pipes receiving larger triangles. Further manual tweaks were then done to some of the triangles, removing those that were not necessary and causing problems, or increasing the triangle size to help pipes larger than 300mm to pick up the expected flows.

3.3.4.4 Mesh Level Zones

Pito One Drain

A mesh level zone was added along the northern side of Pito One Road (Figure 3-16) to model a small open drain. The mesh level zone was created in GIS software following the dimensions and layout in the as-built drawings PSWXX2356-7A and PSWXX2758-1A before being imported into ICM. All known invert levels were included as set elevations at various vertices along the drain. All points in-between these set inverts were interpolated.

Korokoro Stream

A small mesh level zone was added to remove a small mound in the DTM that was blocking the main channel of the Korokoro Stream causing it to bifurcate down a nearby road. The mesh level zone follows the shape of the main channel and was given an upper-level limit equal to the level of the stream at the upstream end of the mesh level zone (Figure 3-16).

Korokoro Mouth

A patch was required to connect the mouth of the Korokoro Stream to the harbour due to the inclusion of harbour bathymetry, and because sections of the Korokoro mouth were missing from the supplied HCC 2013 DTM. This connection was accomplished by inserting a mesh level zone with a max level of -0.31m to match the surveyed level of the Korokoro outlet.

The Dead Arm of the Hutt

A small mesh level zone was added at the outlet of the Dead Arm of the Hutt to correct the DTM at the junction between the two water bodies, allowing flow to drain from the Dead Arm of the Hutt (Figure 3-16). The mesh level zone was inserted with an upper limit linked to the upstream invert of the culvert that conveyed water under the Esplanade Bridge, discharging into the Hutt River.

Western Hills

Mesh level zones were added in the Western Hills to help fix blockages in the stream channels caused by discrepancies in the DTM from vegetation impacting the LiDAR (Figure 3-16). Each mesh level zone extended across blockages where pond depths exceeded more than 3m for multiple mesh elements. In total 18 mesh level zones were created to correct the most significant issues. The elevations for each mesh level zone were determined for each vertex where the upstream and downstream vertices were specified from the ground model and everything in-between interpolated. Mesh level zones were used to fix these discrepancies rather than break lines as the artefacts in the DTM were generally larger than a few elements wide. Thus, an alteration of multiple element elevations was required.

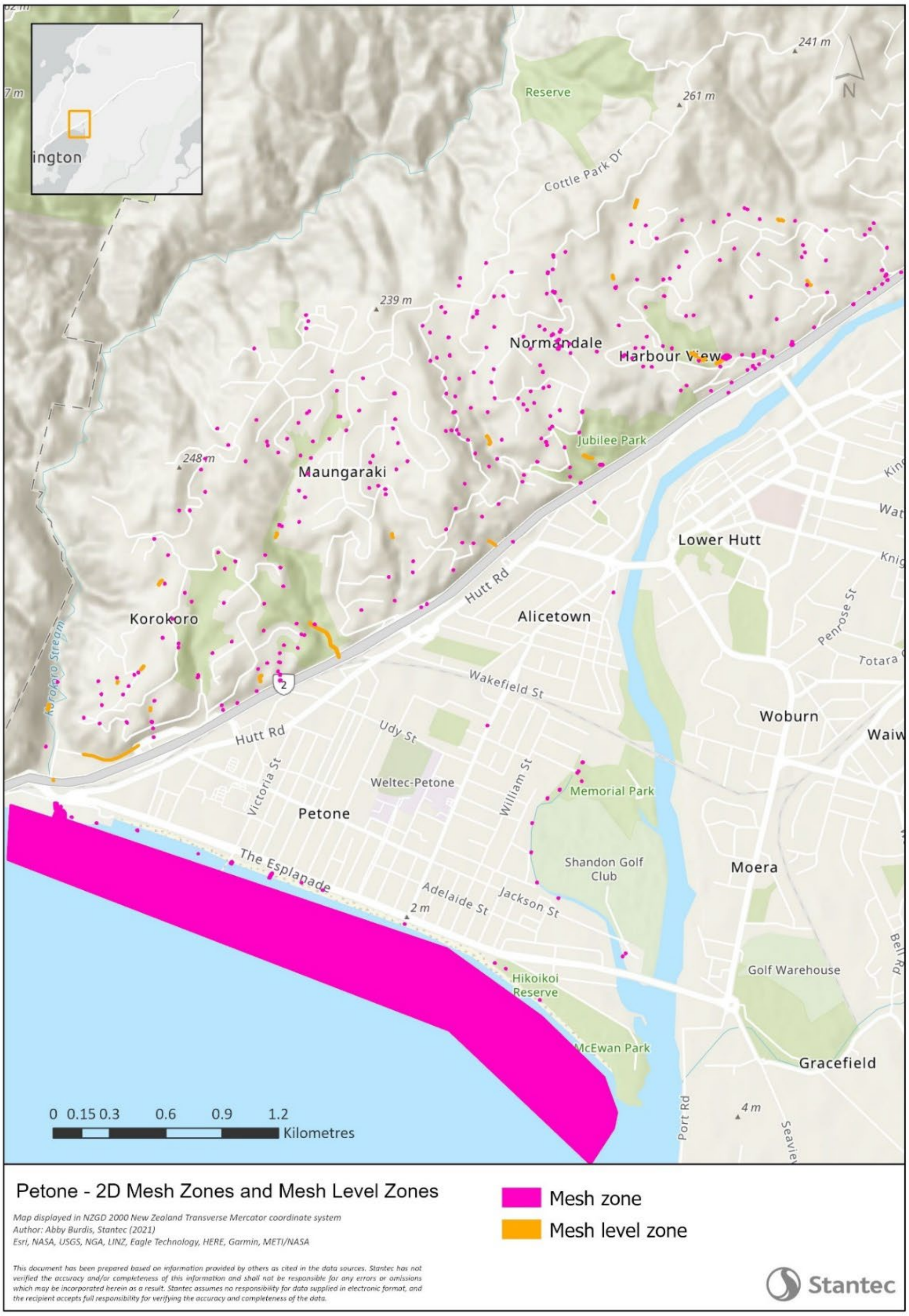


Figure 3-17 2D mesh zones and mesh level zones

3.3.4.5 2D Surface Roughness

Energy losses in the 2D network due to surface friction have been accounted for using roughness zones to set Manning's n roughness values across the 2D zone. The Roughness Zones have been adopted from the Regional Hydrology Layers discussed in Section 2.3.4.Figure 3-17 shows roughness values applied to the different surface types identified across the Wellington Region. These roughness layers were checked with no changes deemed necessary. This is due to the high-level nature of the layers and limited problems with soil types unlike alterations made to the hydrology layers described in Section 3.2.2.Figure 3-17 shows an overview of the roughness coefficients used in the Petone catchment.

Table 3-6: Roughness coefficients

Ground cover	Manning roughness coefficient n	Comment
Roads and footpaths	0.02	Upper limit from Regional Specification (value indicated by WWL)
River	0.05	"Winding with some weeds and large stones" in Ven Te Chow table (value indicated by WWL)
Vegetation: alpine	0.05	The maximum value of "rock cut channel, jagged and irregular" in Ven Te Chow table
Vegetation: bare	0.04	"Rock cut channel, jagged and irregular" in Ven Te Chow table (value indicated by WWL)
Vegetation: forest	0.1	"Medium to dense brush in summer" in Ven Te Chow table (value indicated by WWL)
Vegetation: impervious	0.05	"Impervious" here means water-logged (value indicated by WWL)
Vegetation: pasture	0.05	The maximum value of "high grass" in Ven Te Chow table (value indicated by WWL)
Vegetation: scrub/flax	0.08	The maximum value of "light brush and trees in summer" in Ven Te Chow table (value indicated by WWL)
Vegetation: urban open space	0.05	"Urban open space" can be any of forest/ open field /landscaped garden/ pavement
Recreational area, playing field	0.05	Upper limit from Regional Specification
Non-residential properties: pavement	0.02	Upper limit from Regional Specification because assumed to be poorly maintained or have plantings and dividing barriers
Non-residential properties: building	0.5	Upper limit from Regional Specification
Residential properties: pavement	0.02	In line with Regional Specification (value indicated by WWL)
Residential properties: grass	0.04	In line with Regional Specification (value indicated by WWL)
Residential properties: trees	0.15	Upper limit from Regional Specification
Residential properties: small, fenced backyards	0.1	
Residential properties, high density, limited open space.	0.2	Residential areas with a high subdivision count or closely packed properties. The building footprints do not have a separate roughness definition
Residential properties: building	0.5	Upper limit from Regional Specification

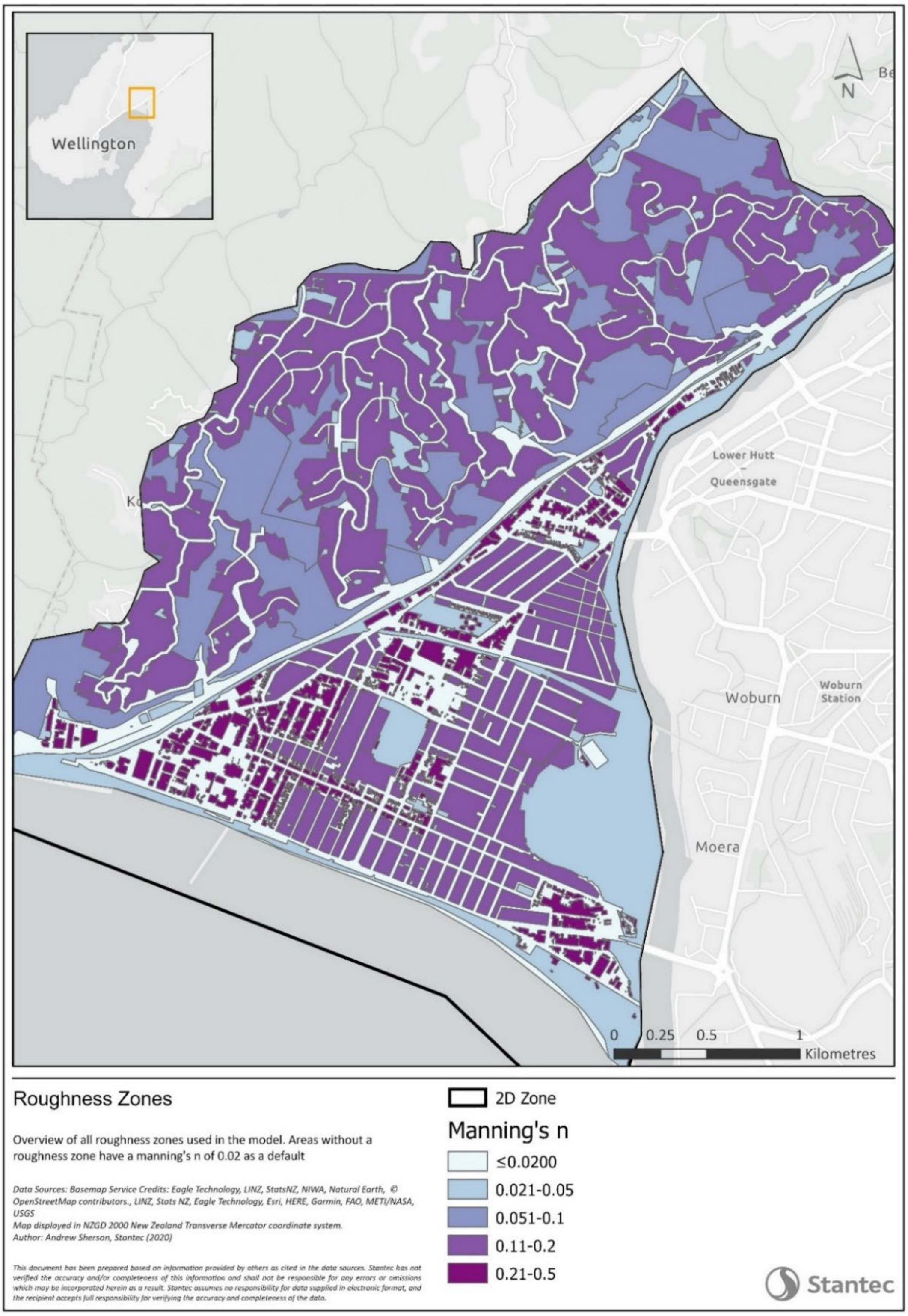


Figure 3-18 Manning's n roughness zones in the ICM model

3.3.5 1D – 2D Connectivity

All modelled 1D and 2D structures are interconnected. At points of transition, such as bank lines and 2D nodes, flow is transferred between each domain in all directions. There are four key links between the 1D pipe network and the 2D surface in the Petone model including: 2D nodes, 2D outfalls, bank lines, and Inline banks.

3.3.5.1 2D Nodes and Sumps

Manholes with a connection to the 2-D surface are modelled using the “2D” flood type and sumps with a flood type “Gully 2D”. These nodes can transfer water to the 2D mesh elements they are located in. For “2D” flood type nodes, flow between the node and the 2D mesh element is calculated using a weir equation where the circumference of the node (manhole) is the weir width. For flood type “Gully 2D” flow is transferred between the 1D and 2D via a custom head discharge curve (**Appendix I)**.

3.3.5.2 2D Outfalls

At a few locations, the pipe network outlet discharges directly onto the 2D surface. At these locations, an Outfall 2D node has been used. Flow between Outfall 2D nodes and its corresponding 2D mesh element is calculated as a vortex control with a nominal head discharge relationship.

3.3.5.3 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 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* (Wellington Water Ltd, 2017).

3.3.5.4 Inline banks

Inline banks are used at connections between river reaches and the reticulated network, or at most stream crossings. Flow across bank lines is calculated using an irregular weir equation. The discharge co-efficient values for the inline banks have been set at 0.8, and the modular limit set at 0.7 as per the *Regional Stormwater Hydraulic Modelling Specifications* (Wellington Water Ltd, 2017).

3.4 Boundary Conditions

3.4.1 Rainfall

Rainfall profiles for 10-year ARI, 100-year ARI, and 100-year ARI + 20% events were used as specified by WWL. Rainfall depths for these events were extracted from HIRDS v3. Nested rainfall profiles were produced following the methods outlined in *SCS Rainfall Runoff Model Calibration* (Cardno, 2016) which spreads the rainfall across a 12 hour period and includes intensities from 12 hours to 10 minutes (Figure 3-18).

Total rainfall depths for the nested profiles were extracted for the point with the following NZTM co-ordinates from HIRDS v3:

X: 1757845

Y: 5434786

Present-day rainfall intensity values for this location are provided in Table 3-7 below. A 12 hour nested rainfall profile was used, with the peak occurring at approximately 60% of the storm duration, as per WWL hydrology recommendations (Cardno, 2016) and shown in

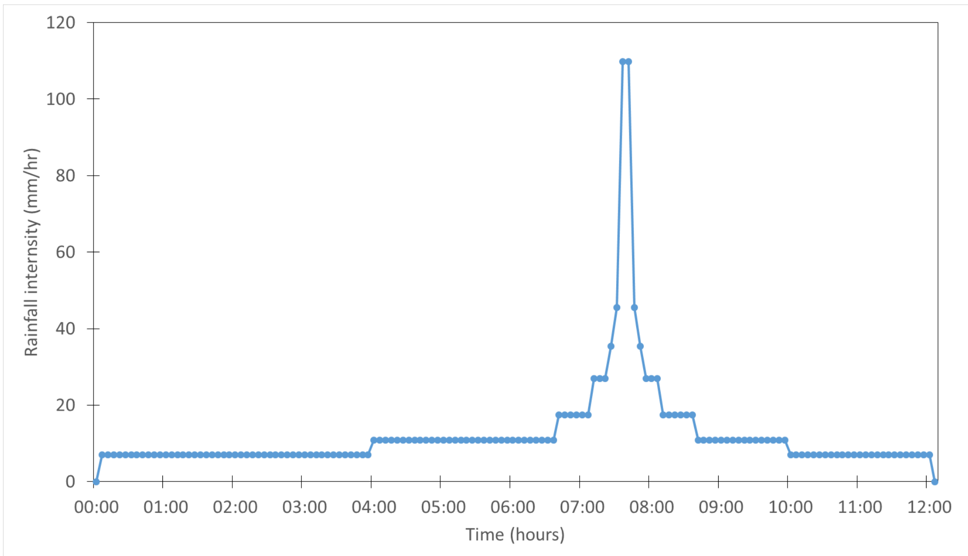


Figure 3-19 12-hour nested rainfall profile

Table 3-7: Design rainfall intensities (mm/hr) for existing climate conditions.

ARI(y)	AEP	10m	20m	30m	60m	2h	6h	12h	24h	48h	72h
1	0.633	37.8	27.6	22.8	16.5	11.7	6.7	4.8	3.4	2	1.5
2	0.5	41.4	29.7	24.6	17.8	12.6	7.2	5.1	3.6	2.2	1.6
5	0.2	52.2	37.8	31.4	22.7	15.9	9.2	6.5	4.5	2.8	2.1
10	0.1	61.2	44.4	36.8	26.6	18.7	10.7	7.5	5.3	3.2	2.4
20	0.05	71.4	51.6	42.8	31	21.8	12.4	8.7	6.1	3.7	2.8
30	0.033	78	56.4	46.8	33.8	23.8	13.5	9.5	6.7	4	3
40	0.025	82.8	60	49.8	36	25.2	14.4	10.1	7.1	4.3	3.2
50	0.02	87	63	52.2	37.8	26.4	15.1	10.6	7.4	4.5	3.4
60	0.017	90.6	65.7	54.2	39.3	27.5	15.7	11	7.7	4.7	3.5
80	0.012	96.6	69.6	57.8	41.8	29.2	16.6	11.6	8.1	4.9	3.7
100	0.01	100.8	73.2	60.6	43.8	30.6	17.4	12.2	8.5	5.2	3.9

Nested profiles are helpful for assessing a stormwater system's conveyance capacity where there is a range of subcatchment areas within a study area. Using these profiles reduces the number of times the model needs to be run to generate a flood map. However, given there are detention storages within the Petone system, storm volume can be as important as peak flow rates in determining maximum flood depths. In future studies of system performance, it will be important to use a range of rainfall durations (and associated total depths) for the ARI of interest. For these model runs a standard temporal profile (as opposed to a nested profile) should be used to understand the potential attenuation provided by the detention dams.

3.4.2 Hutt River

The Hutt River was modelled in 1D using river reaches built from 330 cross sections supplied by GWRC in 2018, see Figure 3-6.. All received cross sections were used to allow flows to be added close to where the Taita Gauge site is located. Thus, the model contains river reach sections that extend beyond the modelled area.

Minor adjustments were applied to the cross sections. These included 1) removing sections that extended beyond the stop banks 2) incorporating interpolated cross sections to allow outlets to connect closer to their actual location and to improve stability at the confluence with the Dead Arm of the Hutt where there is a rapid change in cross section width.

In total, 27 river reach sections were required to model the Hutt River to allow the outlets to connect to the 1D environment at break nodes between each section. Following the specifications, outlets were changed to break nodes and connected via a weir structure with a coefficient of 0.85 and a width representing the diameter of the attached pipe.

The 10 & 100y ARI flows applied at the top of the modelled Hutt River section, starting near the Taita Gauge, were obtained directly from NIWA's Online Flood Frequency Tool showing estimated flows from recorded data. No further flow was added or lost from the model until water reached the boundary with the 2D modelled area. Although this approach does not account for additional water entering the Hutt River from catchments in-between Taita and the top of the 2D area of the Petone model, it still provides reasonable flow rates. This is backed up by GWRC's DHI model results suggesting that there is a loss of flow moving downstream along the Hutt River to the harbour, which may be a result of overtopping of some sections of the stop bank and groundwater infiltration. Thus, inserting estimated flows along the Hutt River past the Taita gauge at the top of the river reach into the Hutt River should provide reasonable or slightly conservative flows, as shown by Section 5.

A roughness value of 0.03 was set for the cross sections for consistency with the Eastern Lower Hutt Model. This gives reasonable levels when compared to the GWRC DHI model as shown in Figure 3-20. There is some variance near the harbour where tidal influences become more significant, and up the top of the catchment near Taita Gauge. However, as the model boundary does not interact with the 1D channel till about cross section 500 these variances are not a significant issue.

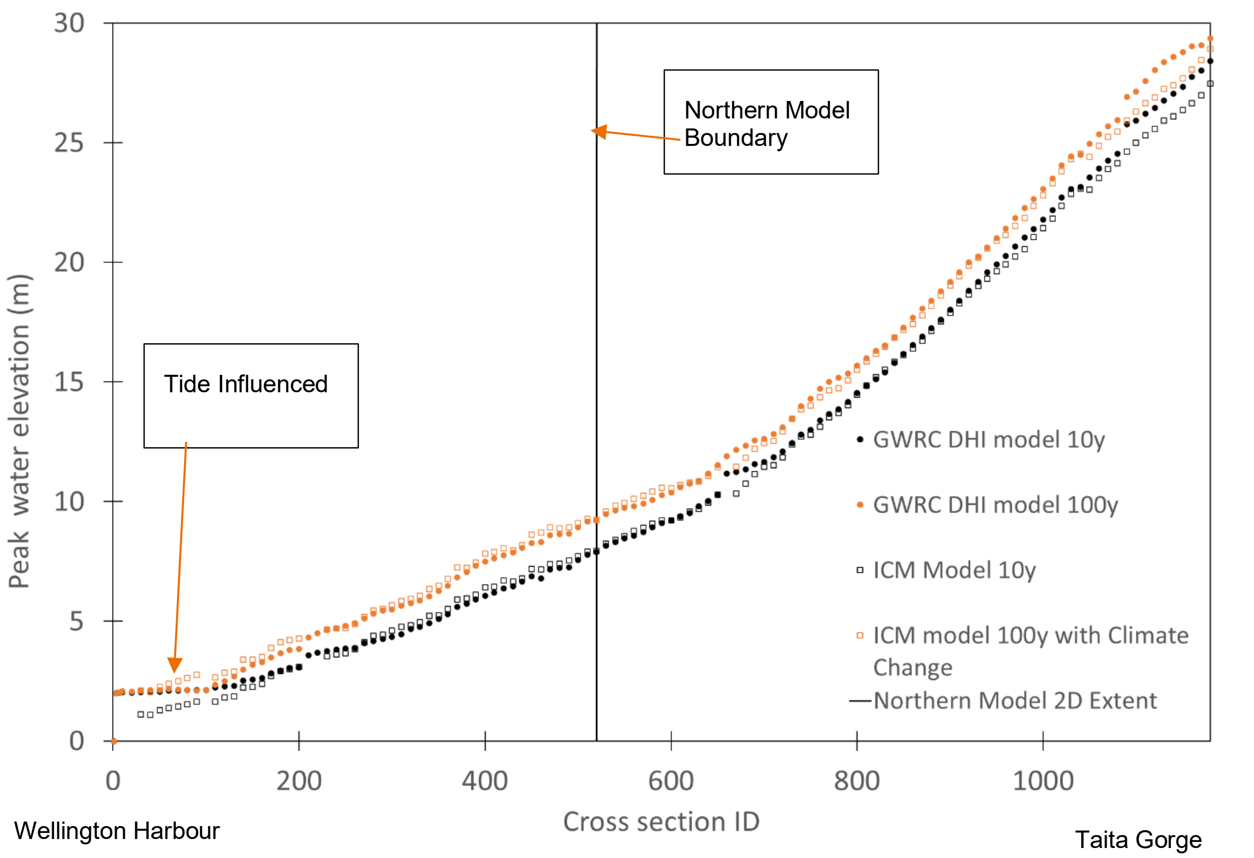


Figure 3-20 Hutt River levels comparison to GWRC Mike Model

3.4.3 Korokoro Stream

Flows obtained from the Korokoro Stream HEC-HMS model described in Section 3.2.2.3 were applied directly to the 2D surface through an inflow boundary. The 2D inflow then entered a 1D river reach where survey cross sections were available. Both 1D and 2D domains were used as there was little known about the upper reaches of the Korokoro catchment, while there was an abundance of information available for the lower reaches which required more precise modelling.

3.4.4 Harbour

A standard 12-hour tidal cycle with a 1.11m peak was added as a tidal boundary to the Petone ICM model (see Figure 3-19.). This 12hr cycle is repeated for longer simulation durations. The peak was calculated from a 0.855m Mean High Water Spring level and a 0.25m barometric increase.

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 as a result of tidal conditions, following current stormwater specifications (NIWA, 2012) and (Wellington Water Ltd, 2017). Timing of the peak tide was shifted to match peak flooding within the modelled Petone floodplain which is a conservative approach.

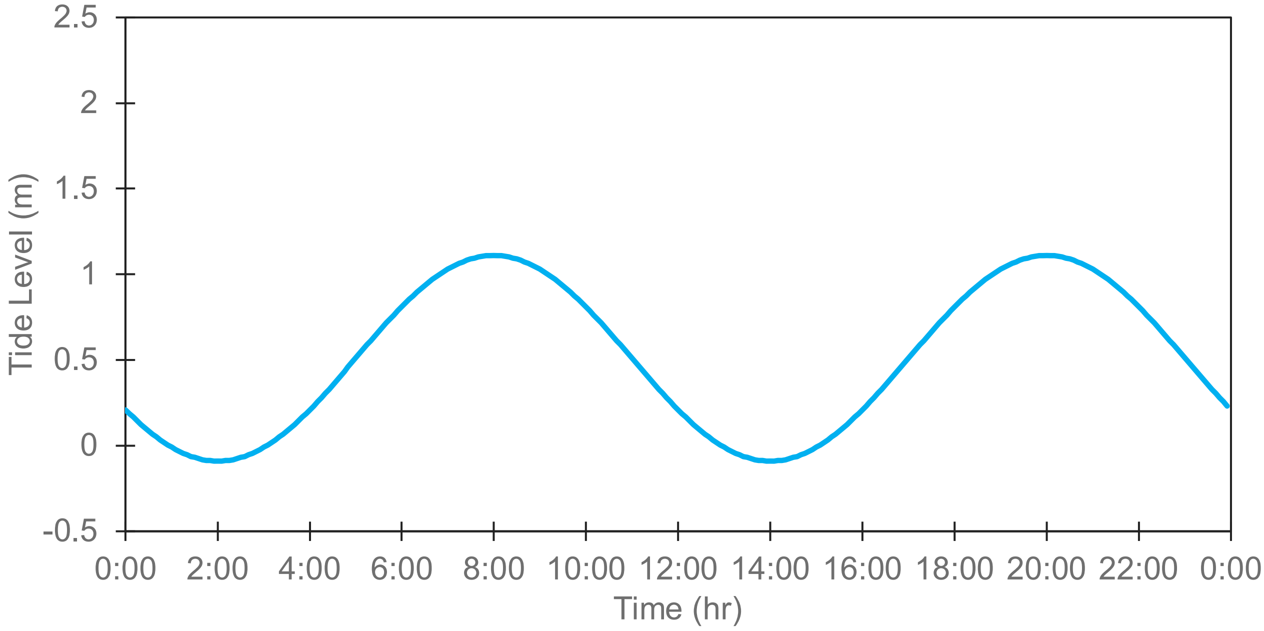


Figure 3-21 12 Hour tidal boundary, existing climate

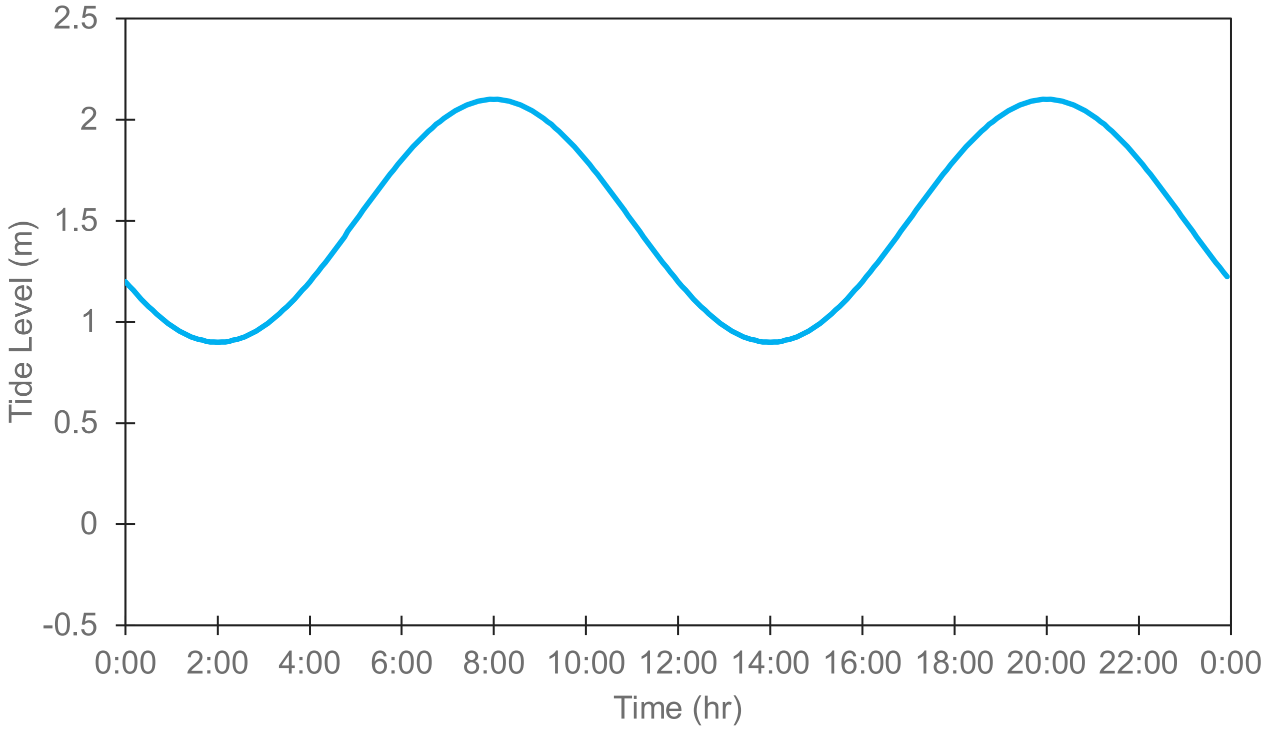


Figure 3-22 12 Hour tidal boundary with 1m of sea-level rise.

A single initial condition was set up for all model runs. The initial condition was a region-wide base water level of 0.21m to match initial tide conditions for the existing climate scenario and 1.2m for sea-level rise for climate change scenarios (see Figure 3-19**.** and Figure 3-20**.**).

3.4.5 Model Limitations and Assumptions

3.4.5.1 Model Limitations

Computational models are only as accurate as the information put into them, and the data available to verify their accuracy. The primary sources of information for this investigation were available asset data provided by (WWL), LiDAR provided by WWL, recorded stream and rainfall data provided by WWL, scanned as-built plans provided by WWL, and site information confirmations by MWH, now part of Stantec.

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

- Manhole and pipe levels for a substantial amount of the network have been interpolated from available or surveyed data.
- Large parts of the model are based on LiDAR. Where the quality of the LiDAR is suspect or there have been changes made since the collection of LiDAR, the model will not represent the real-life structures. Further information on the quality of LiDAR can be referred to in Ground Model Assessment Summary Report submitted by MWH in April 2016 to WWL.
- Building floor levels have not been examined and no DEM adjustments have been made to represent appropriate floor levels.
- Where survey data were not available, fences and walls that may constrict flow paths are not represented in the model.
- There is approximately 39,000m^3 of potential artificial storage available in the Western Hills, likely due to poor ground density in the LiDAR causing artificial dams or depressions. This may have some impact on modelled flood levels in Petone.
- The sensitivity and freeboard process as described in section 6 uses shorter 12hr duration simulations. During the peer review process the model was shown to not reach peak depth in some locations, thus longer simulations were run. The longer runs showed increases in water levels around the Dead Arm of the Hutt, that were within freeboard allowances. Thus, It has been confirmed by Wellington Water that this is OK, but may need to be addressed at a later date when the model is updated.

3.4.6 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.
- Manhole lid levels are adequately represented in the ground model.
- The interpolation rules applied in UMM are appropriate.
- No sediment has been added to the pipes.
- The scoping study indicated that the Pito-One drain would be best modelled as a 1D channel. After further consideration, the preferred approach was to model the drain as a mesh-level zone based on as-built drawings.
- Steep pipes, with gradients greater than 10% were assumed to have no head loss at the upstream end of the culvert, following discussions with WWL and the standard ICM process.

3.4.7 Hydrological Model Assumptions

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

The subcatchments delineated and parameterised within UMM are assumed to be appropriate and have been correctly parameterised.

3.4.8 Initial Model Testing

3.4.8.1 Layout Checks

As described elsewhere in this report, the model has been built using aerial photography, topographic information, GIS layers and as-built information provided by WWL, HCC, and NZTA. The extent of the 2D simulation polygon (2D zone) was defined based on the LiDAR and the reticulated storm water network, ensuring that it covered all low-lying areas and extended upstream of any network branches.

3.4.8.2 Stability Tests

The log results for each model run were checked for any significant errors or mass balance problems. No significant issues were found. The mass balance errors were below 5%.

3.4.8.3 Sensibility Check

The model database and run setups have been checked following WWL QA documentation. A high-level check has been completed for each run to pick up on any anomalies. Some issues were identified and rectified. These are described below.

3.4.8.4 Inlet Checks

Using the 10y and 100y ARI model results, each surface water flow path was checked to make sure it was being picked up by the 2D outfalls, if possible. If there was potential capacity in the outfall and downstream pipes, the outfall was flagged for further investigation. For each flagged 2D outfall, an investigation was conducted around why the pipe was not picking up flow from the ground model. In most instances it was due to inaccuracies in the LiDAR and GIS information where inlets were not at the low points of valleys. Where water was not entering the pipe, the mesh zones on the inlet was increased.

3.4.9 Model Quality Assurance

An internal check and review process of the model build was conducted using an expanded version of the WWL model check and review process. During the check and review process various concerns were raised by the internal reviewer. The concerns were raised with WWL and rectified where practical and in discussion with WWL. Issues that could not be addressed at this stage of the model build have been described throughout the report and in Appendix A and Appendix B . No model calibration has been undertaken.

4 Model Results

The model was used to simulate the 10-year ARI event without allowance for climate change and the 100-year ARI events with and without climate change. The results are included below.
Both the 10-year and 100-year ARI results show flooding in the central urban area of Petone, particularly between the Petone Recreation Grounds and Te Mome Rd, Udy and Cuba Streets. Flooding is a result of the shallow basin-like topography of the Petone area exacerbated by elevated flood levels in the Hutt River which limit the ability of stormwater to drain from the Petone catchment. At this stage of the Petone catchment modelling, the Hutt River has been represented by a constant inflow to the model, providing a continuous, high water level across the network outlets to the river. As a result, there are currently a few locations within the model that do not drain and have a constant water level following the end of the simulated rainfall events due to the persistently high river levels, the distance from any pump stations and being in depressions with little or no piped network to drain them. This potentially creates an unrealistic degree of flooding in these low-lying areas of the model.

Despite the issues noted above, most of the model does drain properly. In addition, the validation results discussed below show clear peaks throughout the model during recorded events and thus reinforce that levels in the Hutt River dictate how the network functions.

The Korokoro catchment shows significant flooding around Cornish Street and the surrounding buildings, especially those occupied by Ullrich Aluminium which is flooded in both the 10-year and 100-year ARI events with water depths higher than floor levels. These results are similar to the observed conditions which show extensive flooding at the outlet of the Korokoro catchment and across SH2 (see Figure 2-1 to Figure 2-5.).

The Western Hills catchments, due to their steep gradients have much shorter peak durations lasting less than two hours. Generally, flooding is confined to valleys such as Maungaraki, Percy Reserve and Harbour View. Detention dams constructed in five of these catchments are included in the model and are designed to attenuate flows.

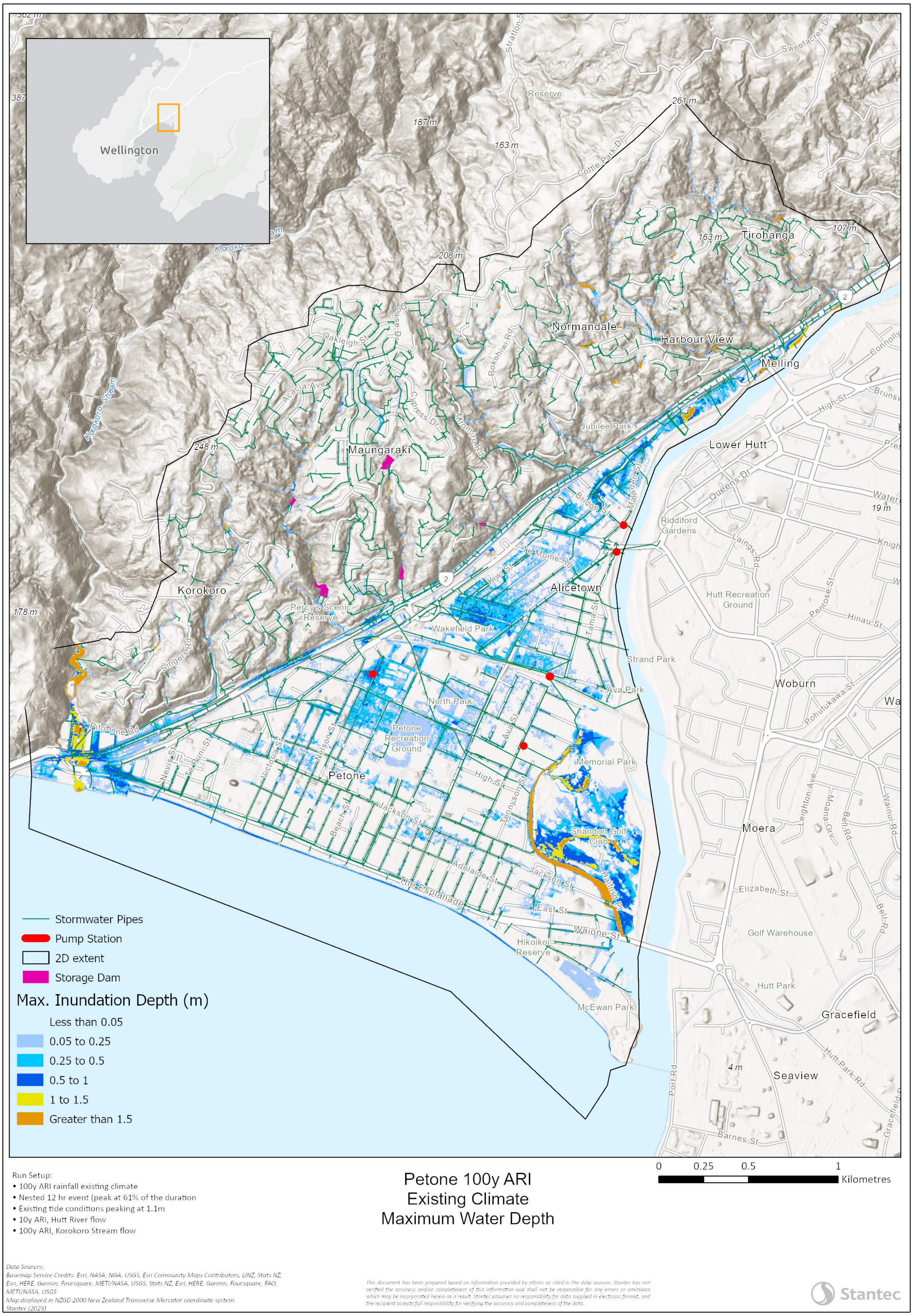


Figure 4-1 ICM model export, 100y ARI nested flooding event, existing climate, extended 24hr simulation.

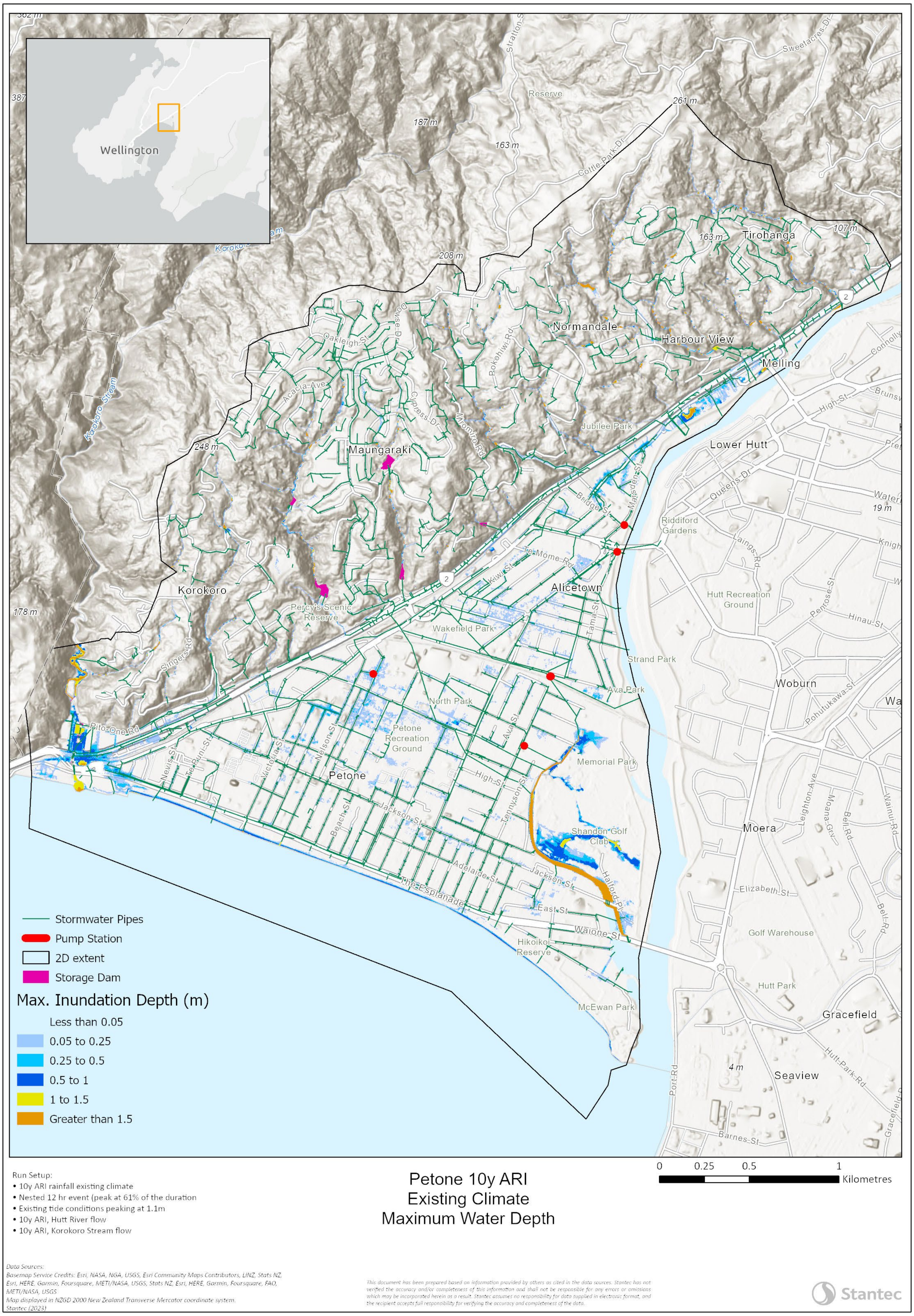


Figure 4-2 ICM model export, 10y ARI nested flooding event, existing climate, extended 24hr simulation.

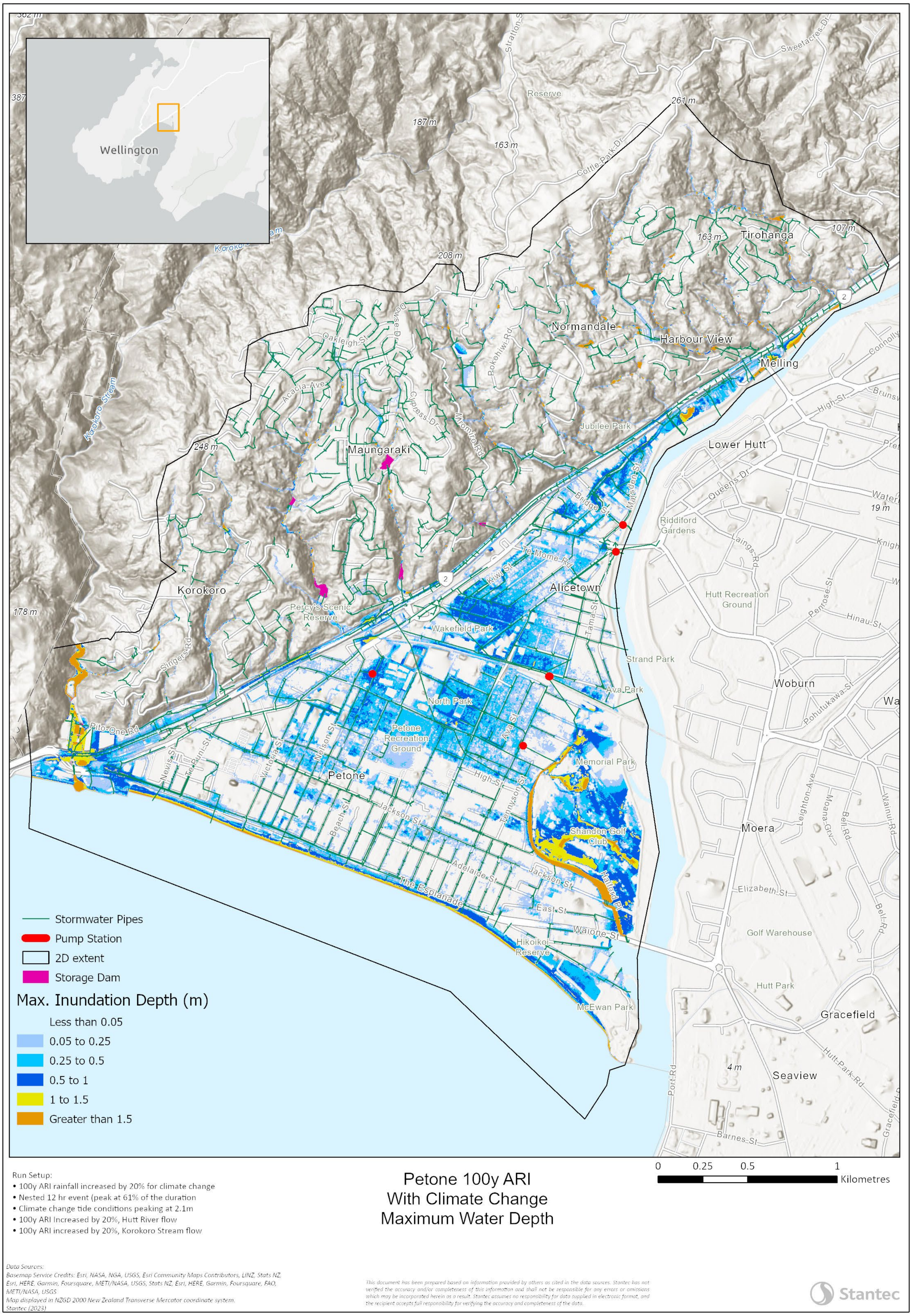


Figure 4-3 ICM model export, 100y ARI nested flooding event with an additional 20% of rainfall and inflows for climate change, extended 48hr simulation.

5 Model Validation

Two validation events were selected by WWL for validating the model. The 2015 and 2016 events were selected as they occurred close to the construction of the model to align with available asset information and community memory. The 2015 event was estimated to have an ARI of around a 50-100y ARI (GHD, 2018), while the 2016 event had an estimated return period of 5-30years (Greater Wellington Regional Council, 2016). The 2015 event has detailed observed levels outlined in 'Flooded Alicetown properties report from Hutt City rainfall event 14 May 2015' (GHD, 2018), which allowed for direct comparisons of water depths across multiple locations.

The validation runs are not directly comparable to the above 10y and 100y ARI runs as a nested rainfall and steady-state high flow through the Hutt River were used for the design runs, providing conservative results.

Following discussions with WWL and the provision of radar rainfall it was determined that rainfall radar information would be used over rainfall gauge information for validation. The rainfall radar averaged across the Petone "Hutt Main" catchment was applied to the entire 2D mesh, while the Korokoro rainfall radar was applied as inputs to the Korokoro HEC-HMS model. Both rainfall radar recordings for the catchments are similar with only small differences in intensity for brief periods, see Figure 5-1**.** & Figure 5-2. In addition, the total rainfall depths agree with the Shandon Golf Course rainfall gauge, being only 10% higher in total depth, which is reasonable considering that the rainfall radar is averaged across a wide area.

Taita Gorge flow gauge records of each event were applied to the top of the Hutt River 1D river reach which begins close to the gauge site. Tide measured at the Queens Wharf gauge was applied as a boundary condition along the harbour. Queens wharf was chosen as the tide level as it was the closest tidal gauge not effected by levels in the Hutt River. If the Estuary Bridge gauge was used there would be risk of doubling the impacts of flow on the levels in the Hutt River at the mouth.

Validation of the model was undertaken by comparing simulation results against flood levels recorded (formally and anecdotally) during the two events.

5.1 May 2015 Event

The following section compares flooding impacts reported in the *Alicetown Report* (GHD, 2018) for the May 2015 event to results from the Petone ICM stormwater Model. Seven properties with specific flooding impacts described in the GHD report are discussed, comparing described levels with the stormwater model results.

It should be noted that all levels quoted in the GHD report were derived from interviews with property owners and/or managers and measurements of road, kerb, and building floor levels a few months after the event. They therefore contain a degree of uncertainty.

Properties discussed:

- Alicetown Fish and Chips, Cuba Street/ Montague Street
- Swim Shop, Ministry of Swimming, Cuba Street
- Honda Cars Service Centre, Victoria Street
- Hunting & Fishing, Victoria Street /Valentine Street
- Hutt Central School playground, Railway Ave
- 1 Aglionby Street- off Railway Ave
- 19a Railway Ave Corner of Railway Ave and Herbert Street

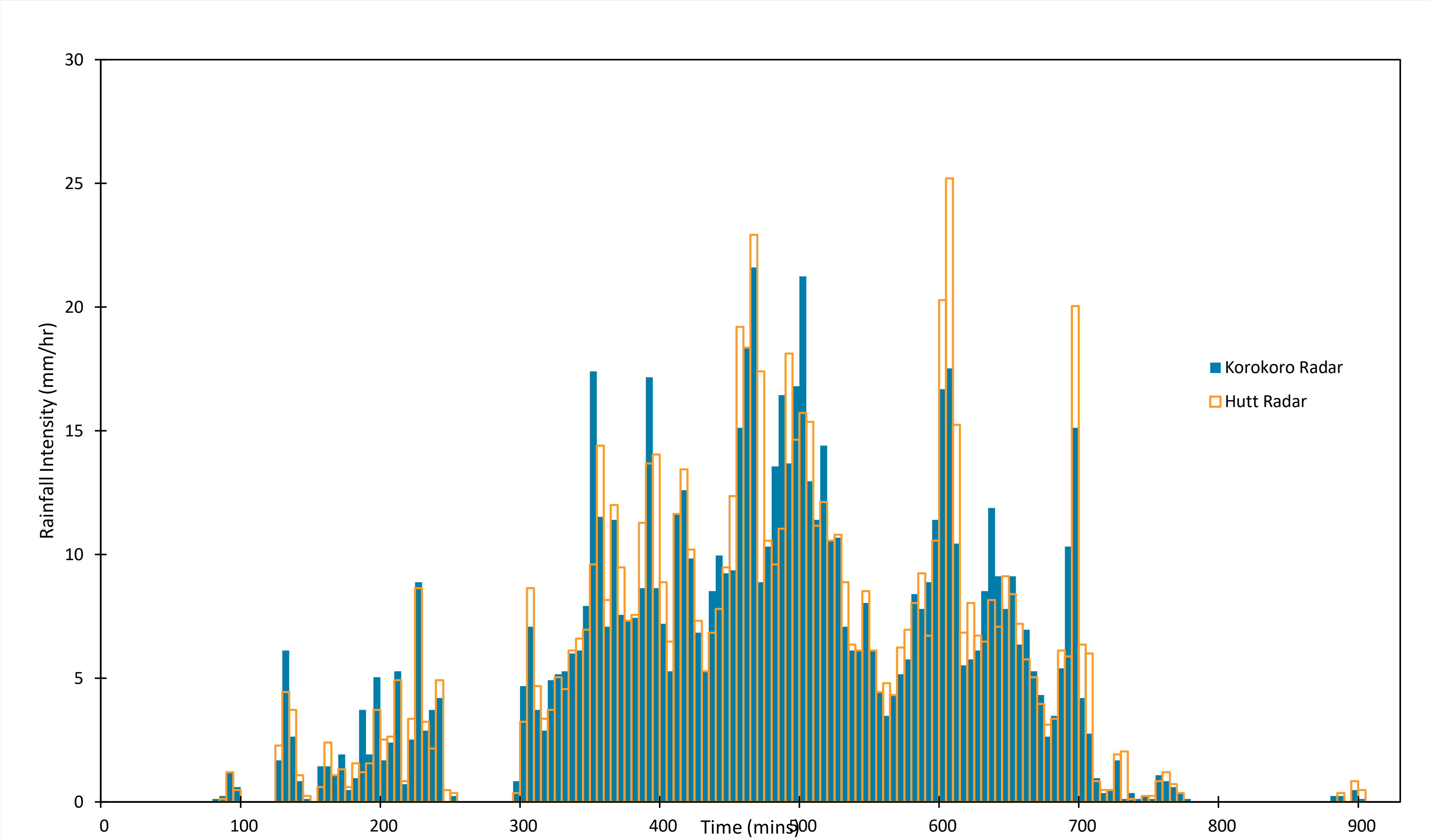


Figure 5-1 2016 rainfall radar time series.

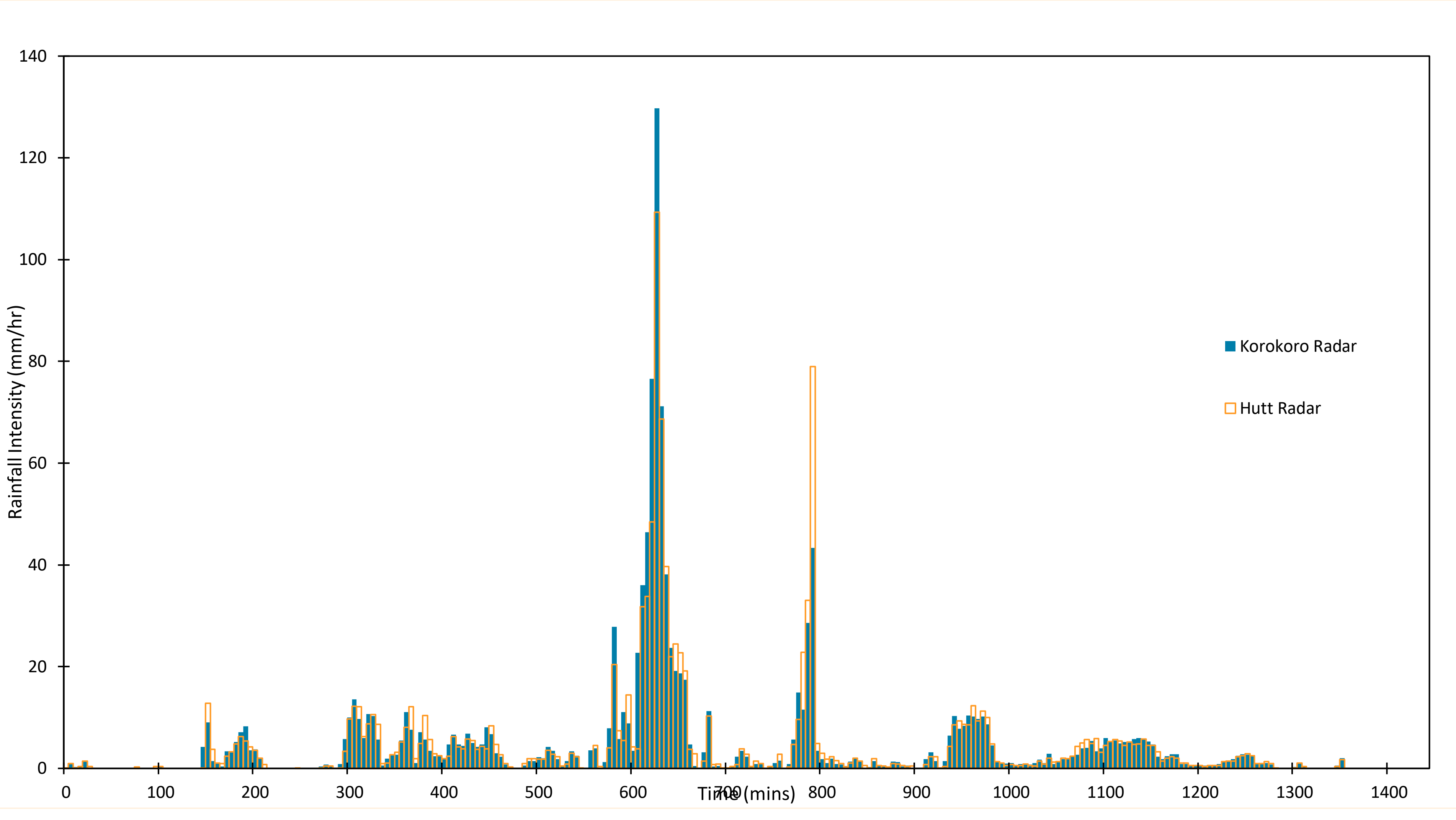


Figure 5-2 2015 rainfall radar time series

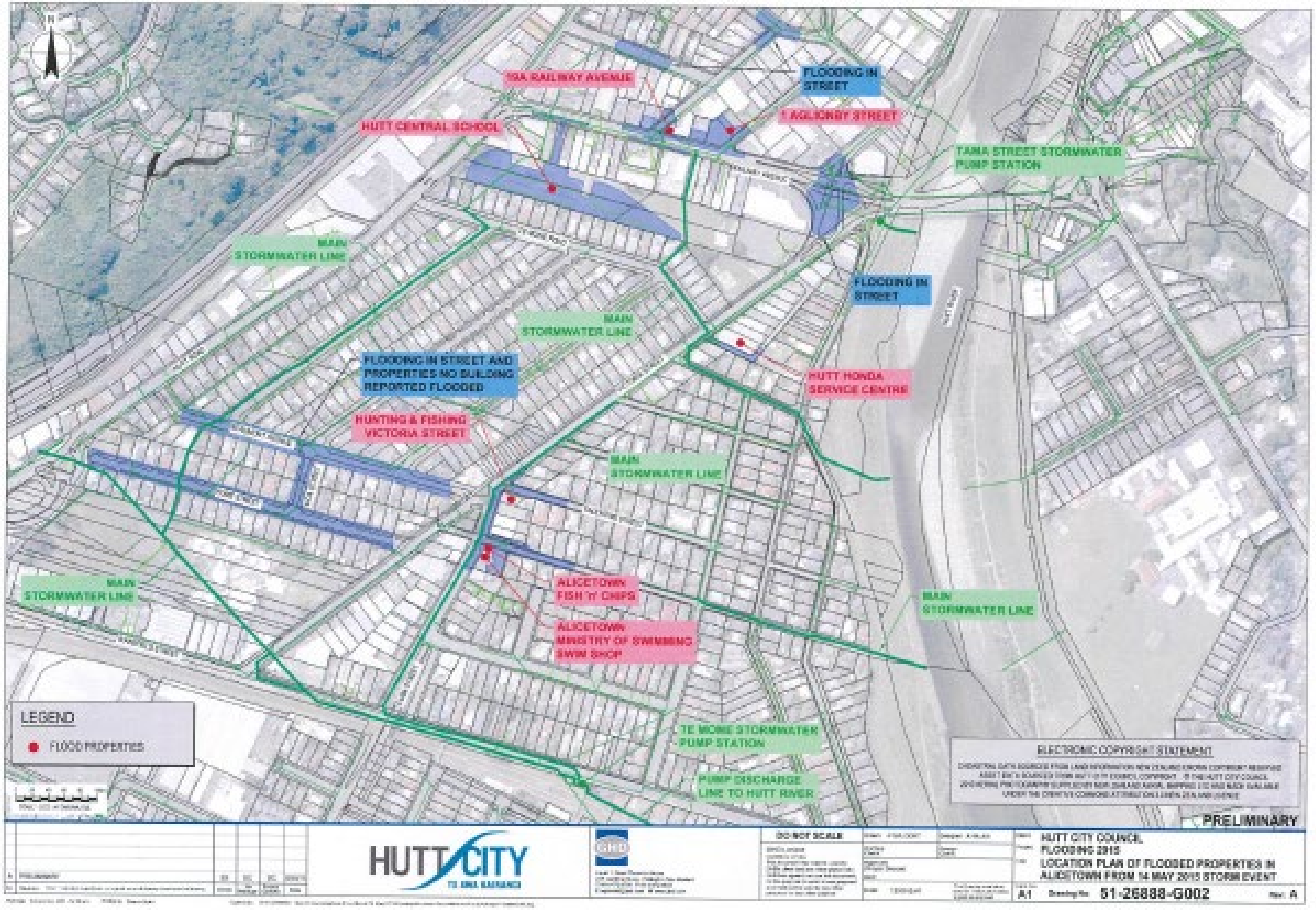


Figure 5-3 Areas of recorded flooding during the May 2015 event, Image taken from (GHD, 2018)

5.1.1 Alicetown Fish and Chips, Cuba Street / Montague Street

The GHD report gave a summary of flooding impacts, providing a single flood depth outside the front door of the fish and chips shop based on the owner's observations. This value was compared to the modelled level at the same location and it was found that the modelled maximum depth of 0.1m matched the observed value reported in the GHD report (see

Table 5-1 & Table 5-4)Table 5-4: Water depth comparison outside Honda Cars

Table 5-1: Water depth comparison at Alicetown Fish and Chips

Location	GHD Reported Observed Depth	Modelled Depth (Max)
Front door	0.1m	~0.1m

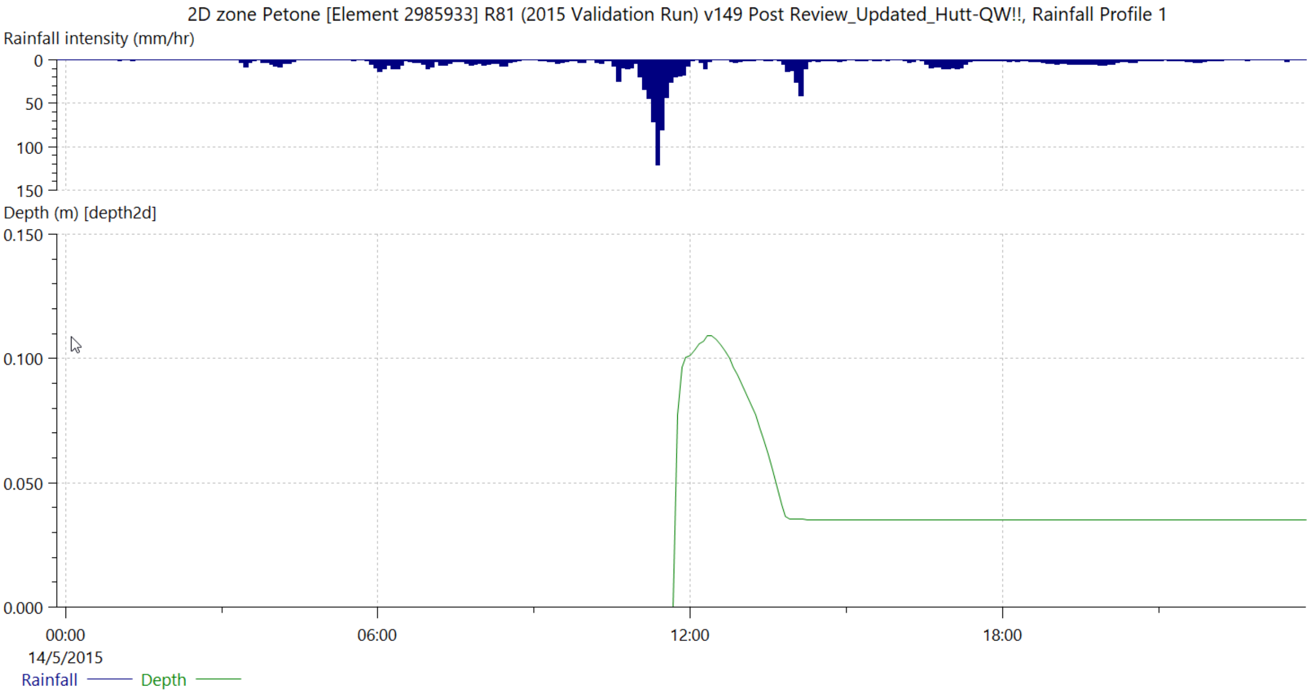


Figure 5-4 Depth hydrograph at the front of Alicetown Fish and Chips

Figure 5-4: Location of sites used to compare the model to observed levels.

5.1.2 Swim Shop, Ministry of Swimming, Cuba Street (building next to fish and chip shop)

The GHD report stated that flood levels at the Ministry of Swimming were high enough to enter the building but were stopped by the owner laying down towels. This agrees with model results which show a slightly lower depth in the swim shop than the fish and chip shop (Table 5-2 and Figure 5-6).

Table 5-2: Water depth comparison at the Ministry of Swimming

Location	GHD Reported Observed Depth	Modelled Depth (Max)
Front door	Up to door level but stopped by towels	~0.07m

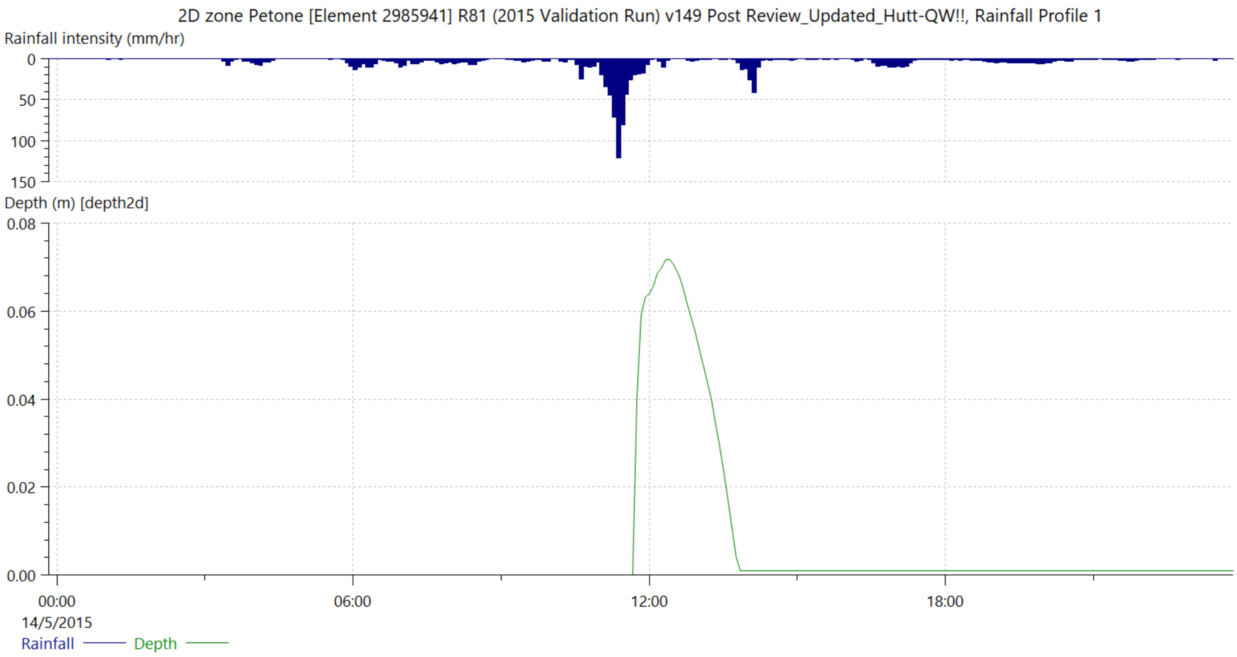


Figure 5-5: Depth hydrograph at the entrance of the Ministry of Swimming

The GHD report also showed a significant amount of water in the carpark and along the end of Montague Street because of a depression in the road. This agrees with the model results which show 0.1m of water in the carpark and depths of up to 0.34m at the end of Montague Street.

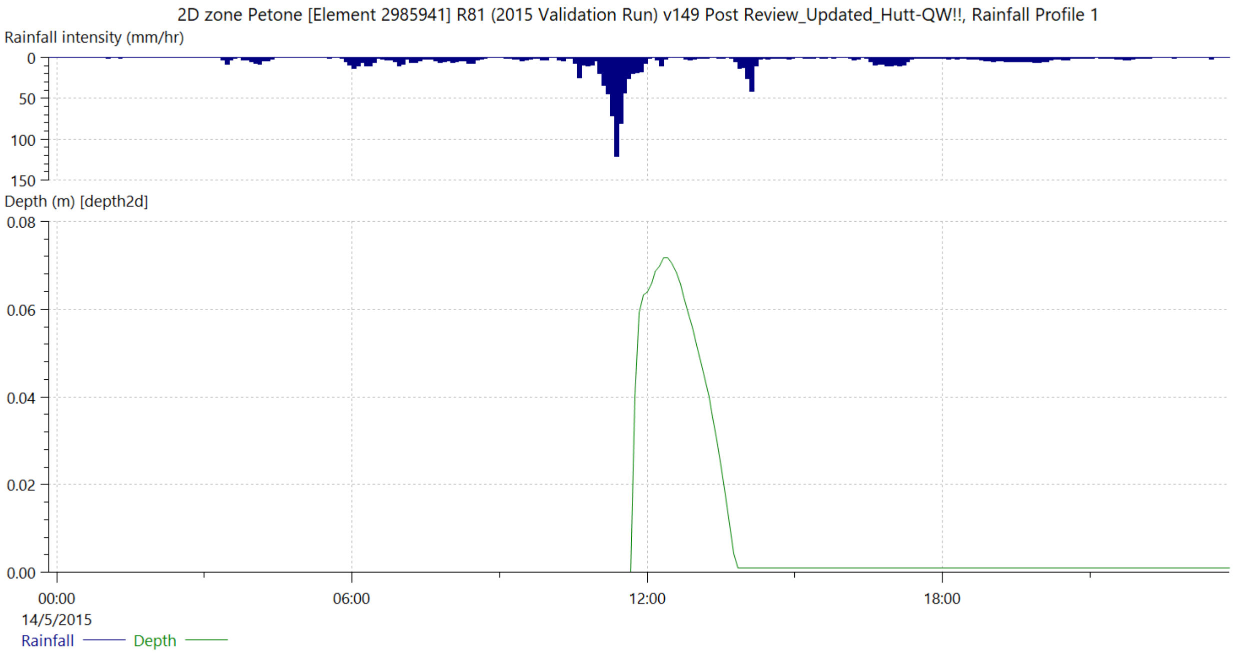


Figure 5-6:Depth hydrograph at the car park behind Ministry of Swimming and Alicetown Fish and Chips

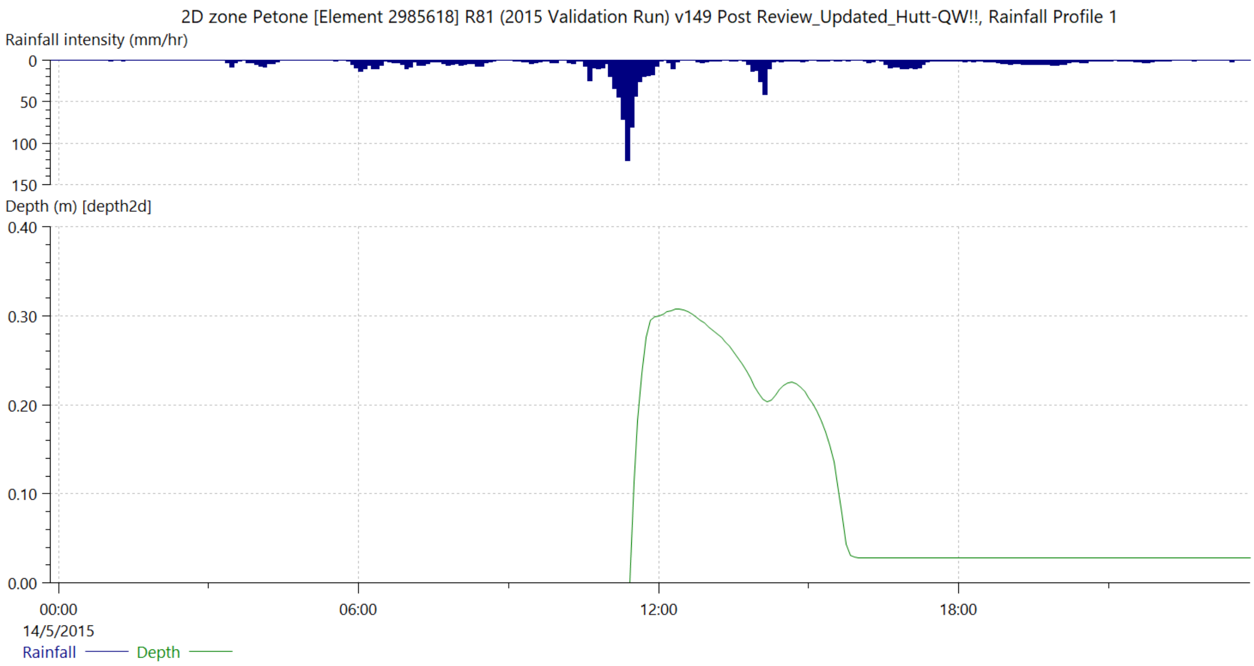


Figure 5-7 Depth hydrograph at the end of Montague Street.

5.1.3 Hunting &Fishing (now Lifestyle Furniture), Victoria Street / Valentine Street

Modelled flooding in the vicinity of Hunting and Fishing, (now Lifestyle Furniture) is similar to the observed flooding, although is slightly deeper, see Table 5-3. It is possible this small variance occurs because the model depths were taken from outside the building (as the building has been excluded from the model mesh) rather than inside where the observations in the GHD (2018) report were made. In addition to similar flood levels, the modelled sumps behaved as observed. For example, the sumps outside Hunting and Fishing surcharge for up to six hours after the peak of the storm, which agrees with the observation that ponding was not being drained during the flood at the sumps in the GHD report.

Table 5-3: Water depth comparison outside Hunting and Fishing

Location	GHD Reported Observed Depth	Modelled Depth (Max)
Inside the building	0.05-0.1m	~0.18m

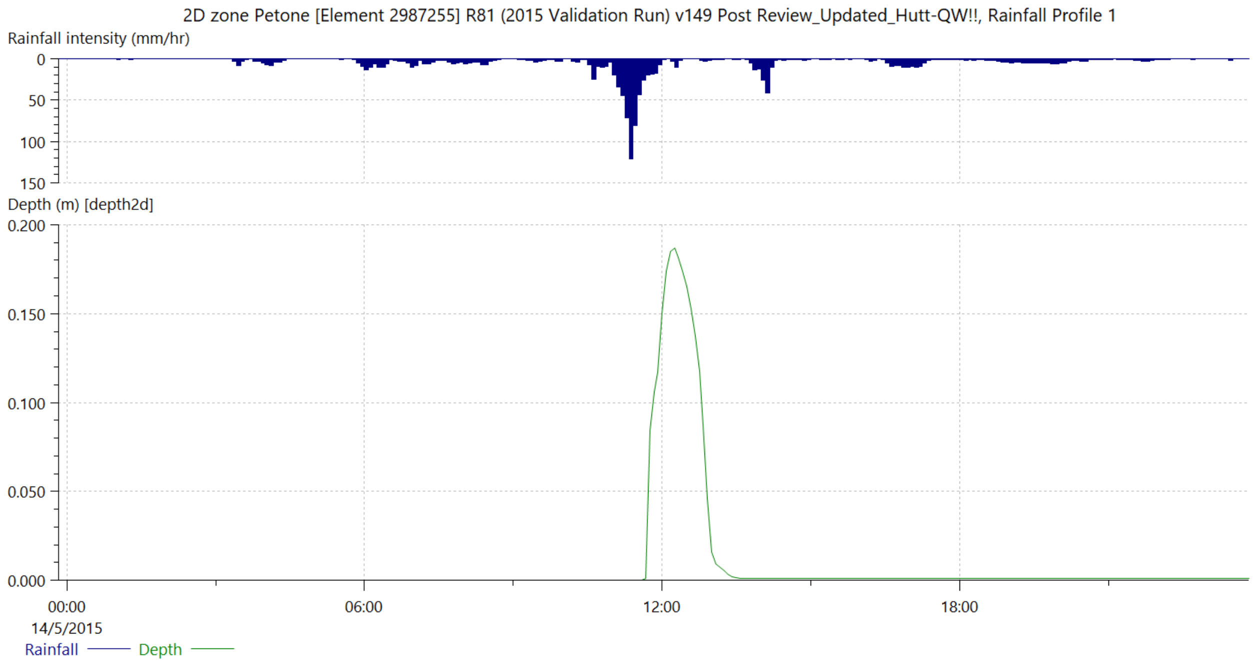


Figure 5-8 Depth hydrograph in front of Hunting and Fishing.

5.1.4 Honda Cars Service Centre, Victoria Street

Both modelled and observed flooding show that a similar amount of water ponded and then entered the building from the back-service entrance, as opposed to entering from the road. The depths in the model along the service lane were around 0.3m and were close to overtopping onto Victoria St. This agrees with the GHD report as stated below.

Table 5-4: Water depth comparison outside Honda Cars

Location	GHD Reported Observed Depth	Modelled Depth (Max)
Inside back of building Location 1)	0.15m	~0.15m
Side of building along access way (location 2)	"water can pond in the service lane to a depth of approx. 0.4m before flowing out onto Victoria St and then flowing south along Victoria St. With the flood level of the building approx. 0.3m below the back of the footpath the building floor would also be flooded, which was the case in the storm event".	0.2-0.3m

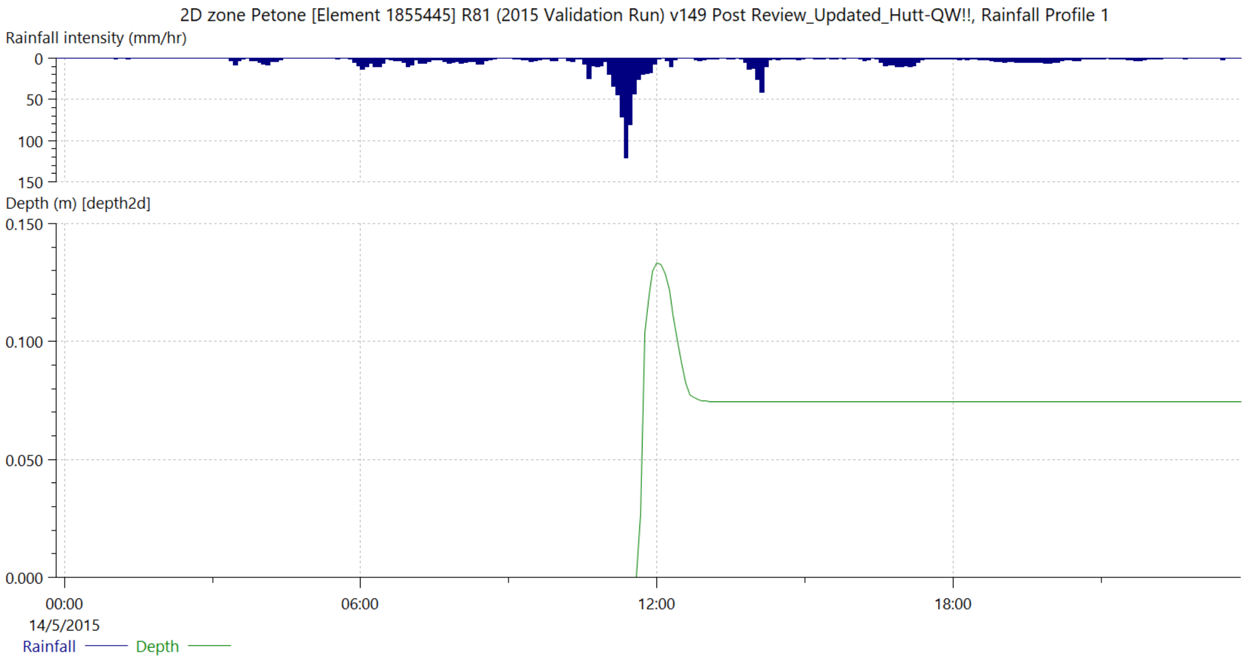


Figure 5-9 Depth hydrograph at the back entrance of Honda Cars

Figure 5-10 Honda Cars validation sites during the 2015 flood event.

5.1.5 Hutt Central School Playground, Railway Ave

At Hutt Central School there are differences between the modelled and observed flood extent. The model shows a large pool of water along the northern face of Wellington Automotive, which was not observed in 2015. This is because in the model the automotive building blocks flooding into the school grounds or courtyard that would usually be able to flow through the lot. This results in a difference of around 0.5m between observed and modelled depths (see Table 5-5). However, the flooding does seem to originate from a similar location as stated in the GHD report.

Table 5-5: Water depth comparisons outside Hutt Central School

Location	GHD Reported Observed Depth	Modelled Depth (Max north-eastern side of the main school building)
In courtyard	0.05-0.1m	~0.48m

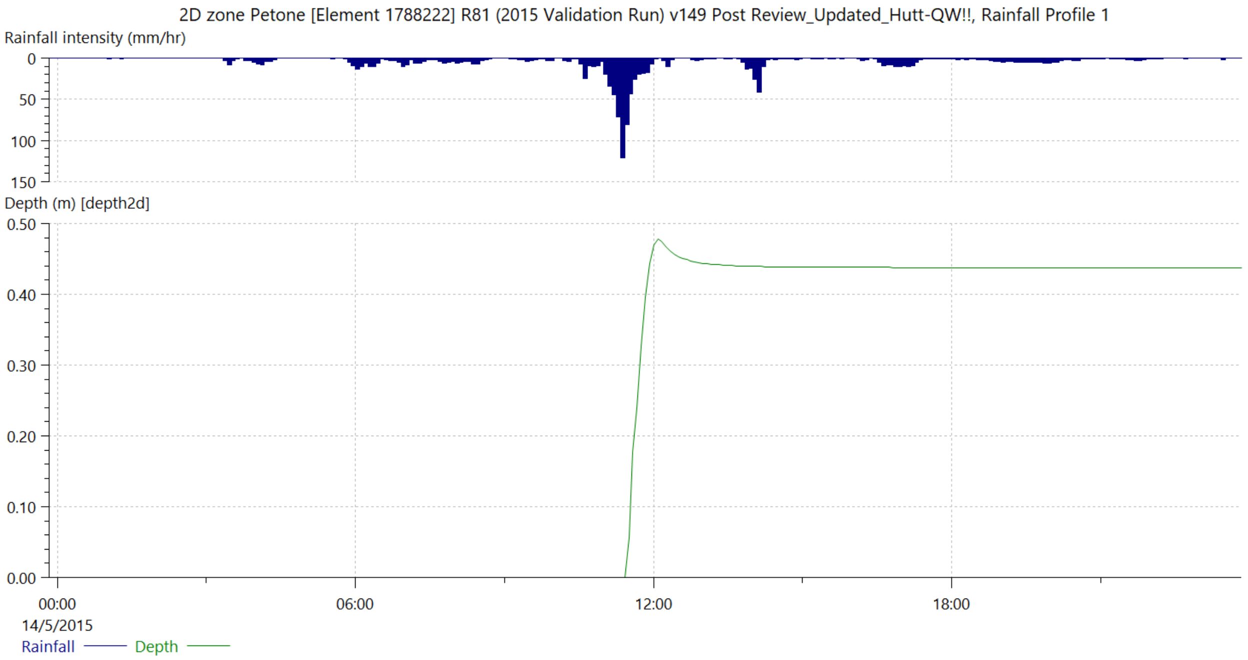


Figure 5-11 Depth hydrograph at the north-eastern side of the main school building.

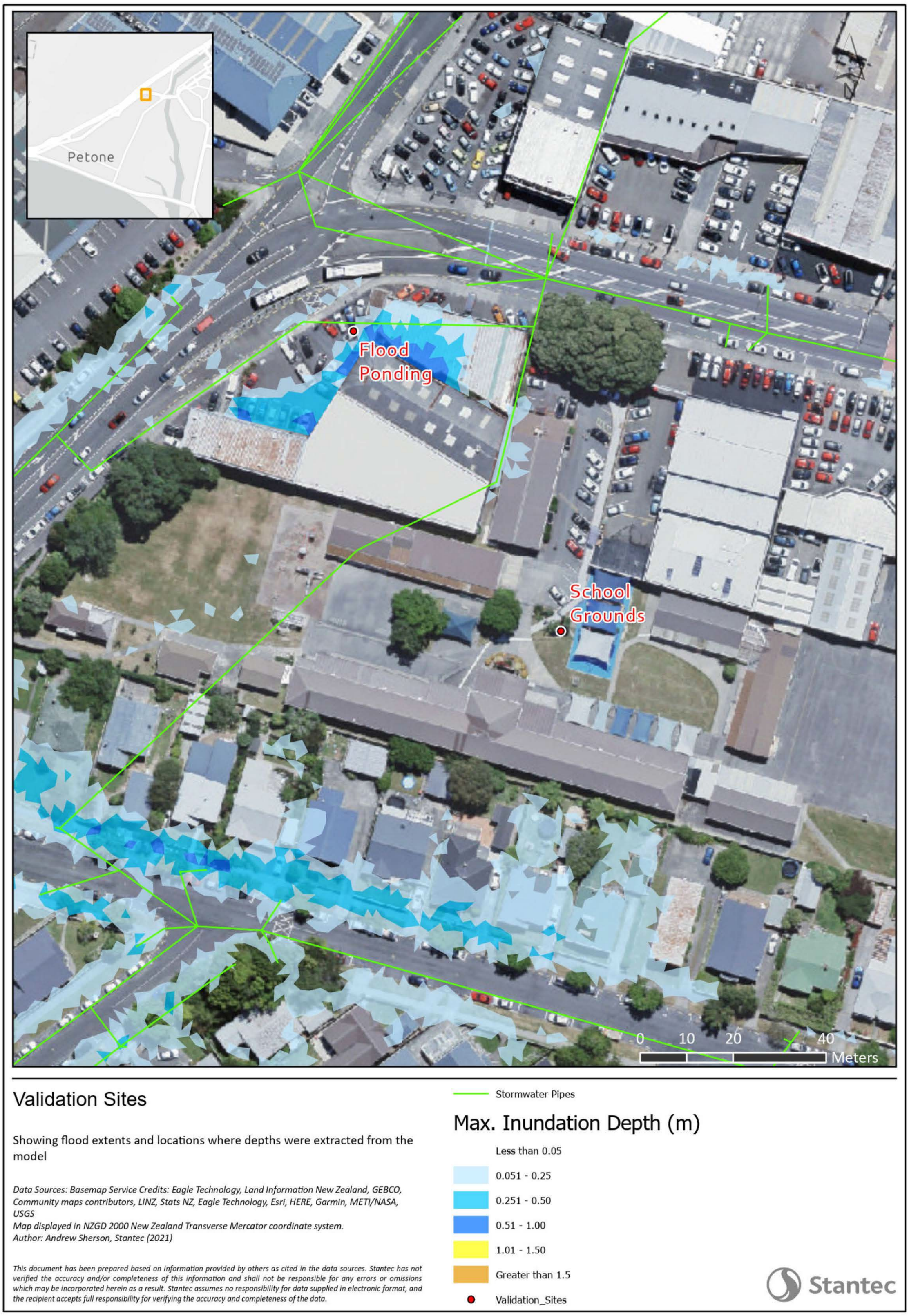


Figure 5-12 ICM results export, Hutt Central School.

5.1.6 1 Aglionby Street - Off Railway Ave

The GHD report states flooding was widespread in the vicinity of the intersection of Aglionby Street and Railway Ave, reaching up to 85m into Aglionby Street from the intersection.
The model shows a similar situation where floodwaters cover most of the street (Aglionby Street is 190m long), with most of the flooding in the first half of the street and the intersection. This corresponds with the GHD observations.

Table 5-6: Water depth comparisons at 1 Aglionby street

Location	GHD Reported Observed Depth	Modelled Depth
Outside main entrance	0.15-0.2m	~0.3m
Back of building	0.05m	~0.09m
Road intersection	Knee height	~0.2m

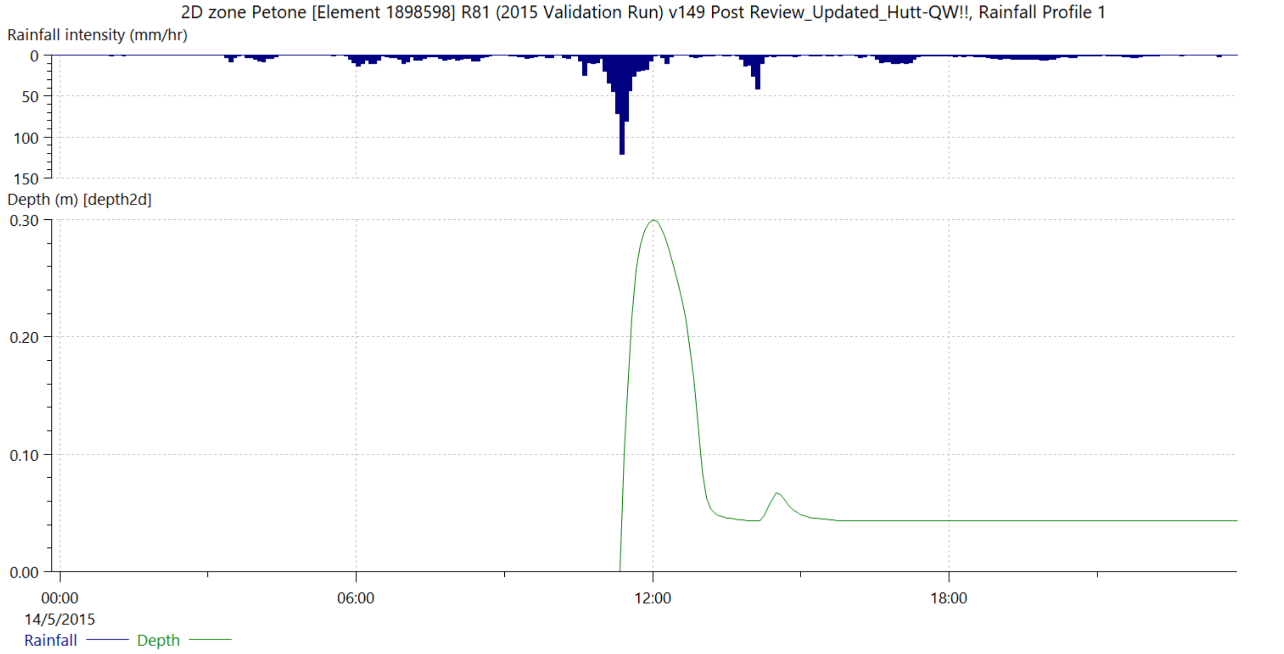


Figure 5-13 Depth hydrograph outside main entrance of 1 Aglionby Street.

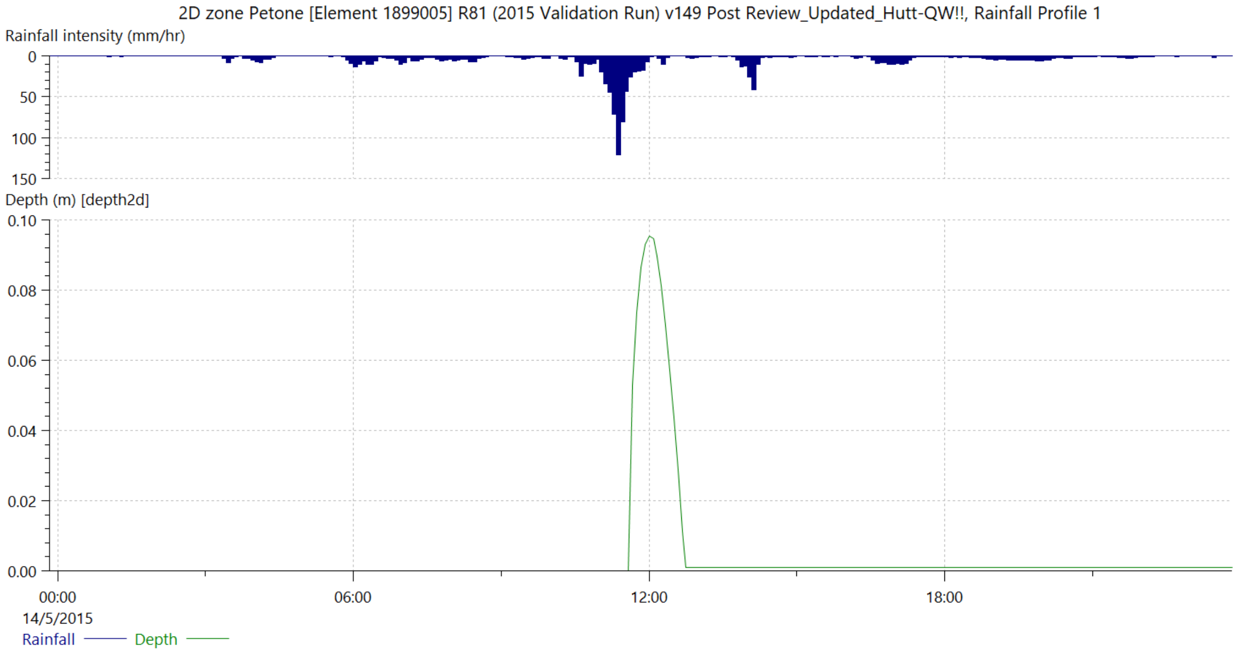


Figure 5-14 Depth hydrograph at the back of no.1 Aglionby Street

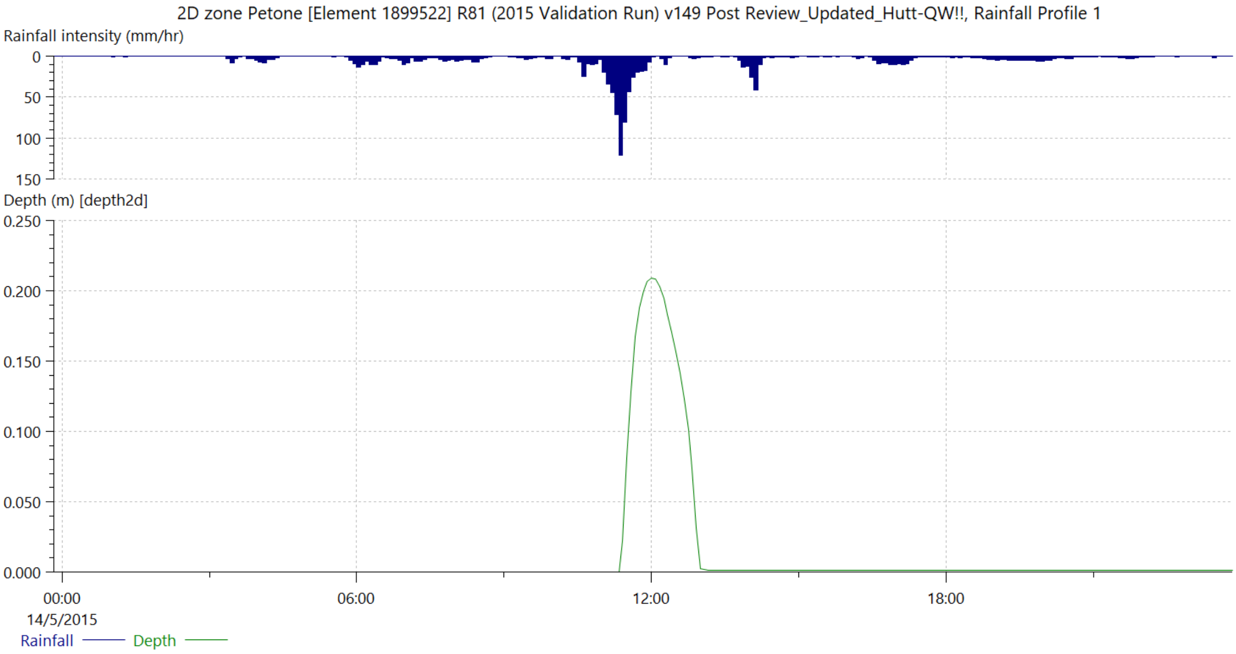


Figure 5-15 Depth hydrograph at the intersection of Railway Ave and Aglionby Street

Figure 5-16 Location of depth hydrographs

5.1.7 19a Railway Ave- Mayur Foodmart, Corner of Railway Ave & Herbert Street

Modelled flood extents at Mayur food court: 19a Railway Ave, are similar to the observed drawings shown in the appendices of the GHD report with most of the flooding occurring at the top of Herbert Street and along Railway Ave.

The modelled and observed depths are also similar near the entrance of Mayur Foodmart. At this location, no building interior flooding was observed which agrees with the model where there is no flooding along the footpath and ~ 0.07m of flooding in the road corridor along the gutter.

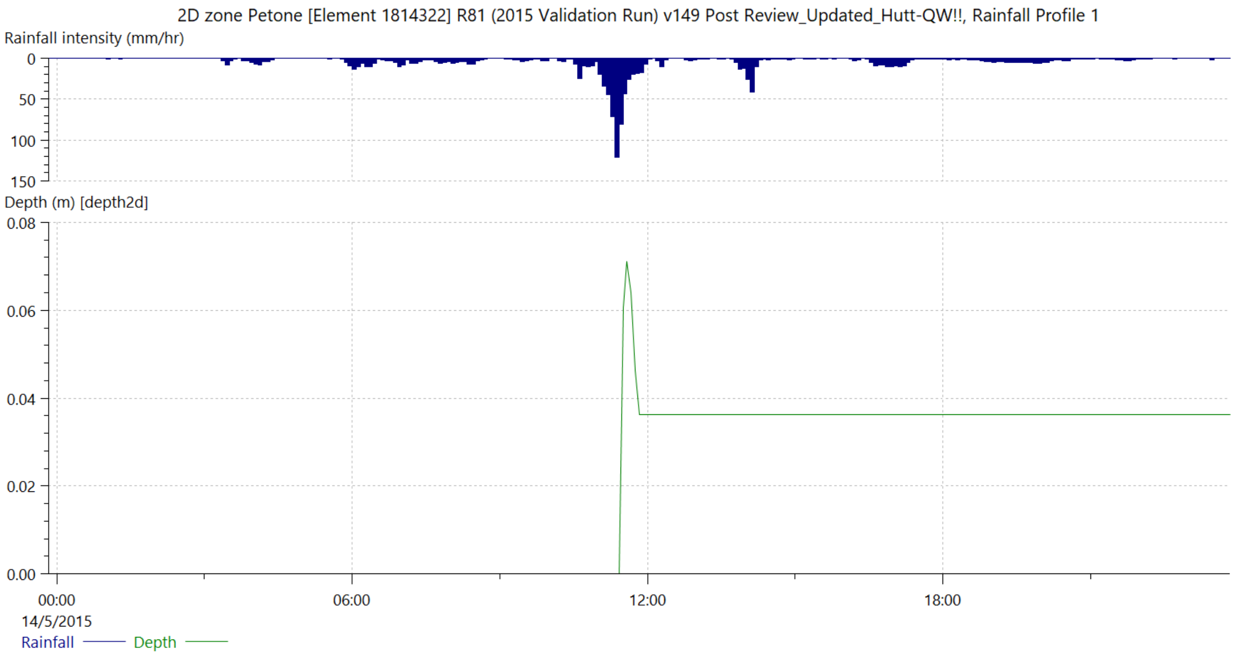


Figure 5-17 Depth hydrograph in the road corridor near the Mayur food court entrance.

5.1.8 General Observations

- The GHD report states that Beaumont Ave and Hume Street were both flooded extensively in the 2015 event. However, despite the reported widespread flooding, there was no recorded internal building damage.
- The model results show a similar situation, as both streets are flooded, but have depths ranging from 0.1-0.3m. This is not high enough to flood most properties which typically have raised floor levels, indicated by steps up to the front door.
- The majority of photos taken in the Petone region are linked to flooding that occurred along SH2 at the mouth of the Korokoro Stream. See Figure 2-1 to Figure 2-5**.** & Figure 5-20. This agrees with the model results, as the Korokoro Stream mouth experiences some of the highest water depths outside the harbour and river channels.

5.1.9 Conclusions

Model results are generally similar to the observed flooding extents and levels discussed in the GHD report. There are some differences in a few locations with regard to flood depths, but these differences are generally small and often attributable to the nature of the model construction.

Overall, the model appears to be slightly more conservative than the recorded flood observations.

Figure 5-18 Flooding along Cornish Street in Lower Hutt on May 2015. Photo by David Morrison quoted in (Price, 2015).

5.2 15 November 2016 Event

The November 2016 event had a lower peak rainfall intensity than the 2015 event, as shown in Figure 5-21, and was estimated to have an ARI of 5-30y. Flooding was widespread causing damage in Porirua and the Hutt Valley and closed multiple roads including both SH1 and SH2 (NZ Herald, 2016). Most of the damage in the Petone region occurred as a result of the rupture of the Western Hills gravity main which caused significant flooding around Udy Street as shown by Figure 2-6 to Figure 2-8**.** and Figure 5-22.. There was limited complaints information supplied by the client and online that was not related to the rupture of the pipe.

As the rainfall event occurred one day after the Kaikoura Earthquake it is believed that the rupture of the gravity pressure main may have been influenced by this event. Outside Udy Street, the only other region with recorded flooding information was along SH2 as shown by Figure 5-23**.** And Figure 5-24. This agrees with the model results which show almost no surface flooding apart from at the mouth of the Korokoro Stream, see **.**, as the rupture of the gravity pressure main was not modelled. Therefore, the model provides a reasonable representation of the event. Without additional modelling, the flooding from the Udy Street rupture cannot be further analysed.

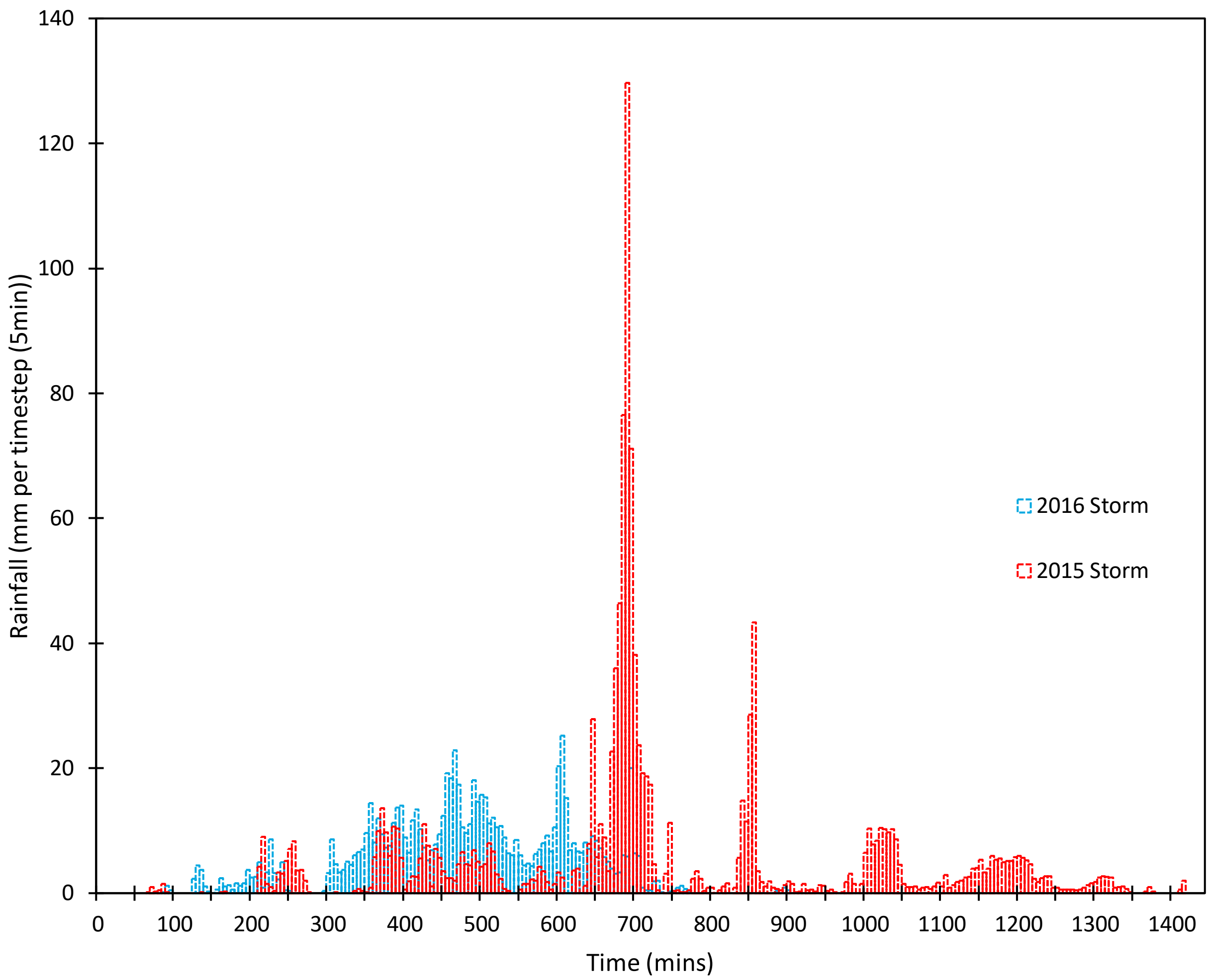


Figure 5-19 Rainfall intensity; May 2015 and November 2016 events.

Figure 5-20 Udy Street property flooding on 8th November 2016 (Weekes, 2016) Photographer Cameron Burnell/Fairfax NZ.

Figure 5-21 SH2 Flooding in the north bound lane (Radio New Zealand, 2016), Photographer Kate Gudsell

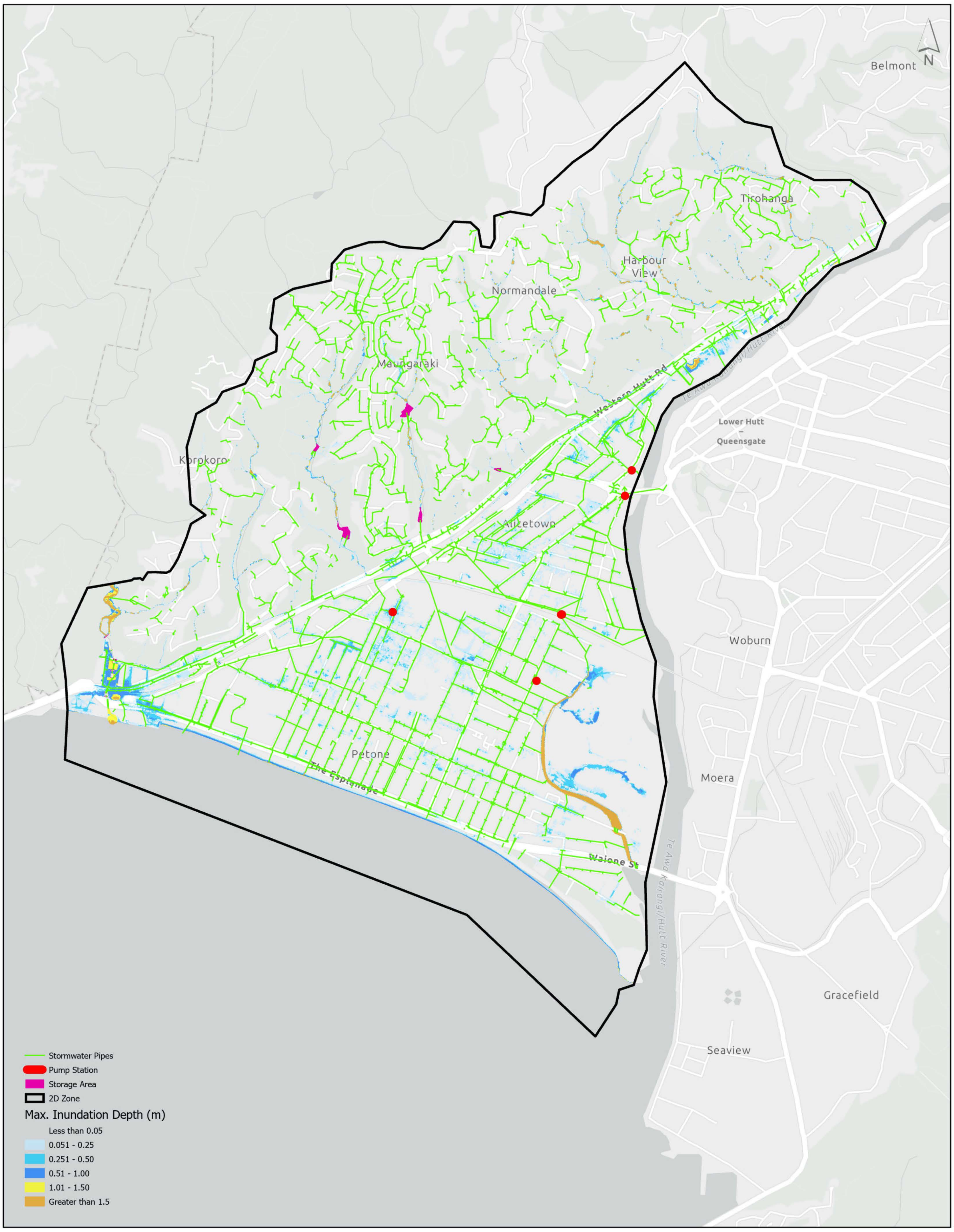
Figure 5-22 SH2 Flooding north bound lane (Radio New Zealand, 2016), Photographer Kate Gudsell.

5.3 Conclusion

The two rainfall events show that the model is producing results close to the observed flooding levels in both a high and low intensity, longer duration storm.

5.4 Validation Results

The ICM model produces similar results that are slightly more conservative than the observed values.



Run Setup:
- Observed rainfall radar 24hour duration
- Observed Hutt River Flow at Taita Gorge
- Observed tidal stage at Queens Wharf
- Modelled Korokoro Stream flow

Petone May 2015 Event
Maximum Water Depths

0 0.25 0.5 1 Kilometres

This document has been prepared based on information provided by others as cited in the data sources. Stantec has not verified the accuracy and/or completeness of this information and shall not be responsible for any errors or omissions which may be incorporated herein as a result. Stantec assumes no responsibility for data supplied in electronic format, and the recipient accepts full responsibility for verifying the accuracy and completeness of the data.

Data Sources: Basemap Service Credits: Eagle Technology, LINZ, StatsNZ, NIWA, Natural Earth, © OpenStreetMap contributors.
Map displayed in NZGD 2000 New Zealand Transverse Mercator coordinate system.
Author: Andrew Sherson, Stantec (2021)

Stantec

Figure 5-23 ICM model export, May 2015 flood event

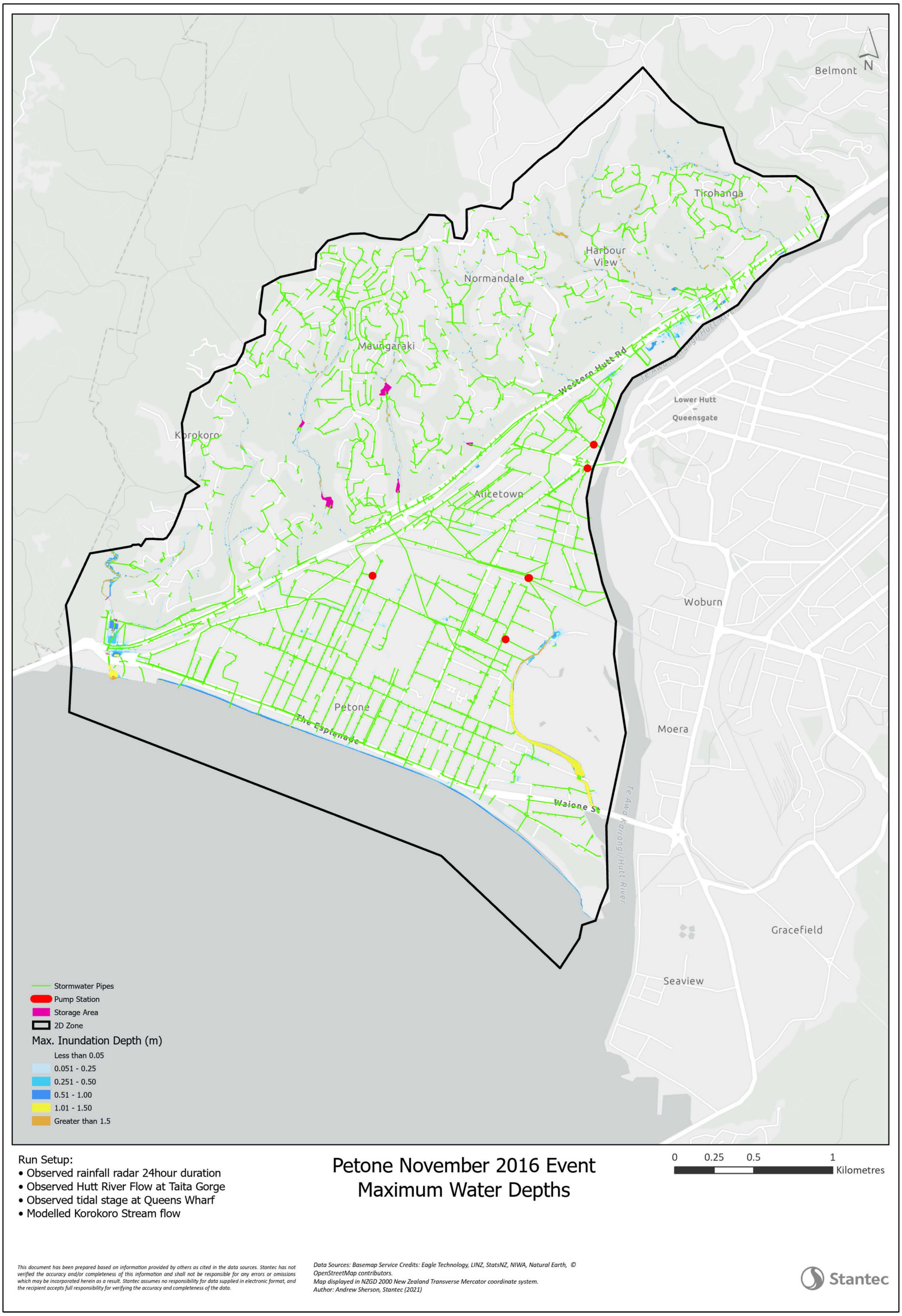


Figure 5-24 ICM Model Export, November 2016 flood even

6 Sensitivity and Freeboard

6.1 Sensitivity Analysis

The following section describes the sensitivity scenarios that were examined to inform the freeboard selection process. It was agreed that Stantec would examine the effects of the following model simulations:

- Rainfall (50% increase)
- All inlets fully blocked
- All inlets partially blocked (50%)
- No Tailwater for receiving tide (involved removing tidal boundary to assess the impact of the tidal boundary on flooding in Petone)
- No Hutt River inflow (involved removing the Hutt River inflow to assess the impact of the Hutt River level on flooding in Petone)
- Pumps turned off

All scenarios were compared with maximum flood depths (m) from what is referred to as the base case scenario, that is; the 100yr ARI 12hr nested profile with +20% allowance for climate change, see Figure 6-1**.** As discussed in section 3.4.5, longer duration simulations were completed after the completion of the sensitivity and freeboard process. The sensitivity and freeboard process discussed below therefore uses a shorter simulation period of 12hrs.

All resultant sensitivity maps show the difference in depth between the base case scenario and the sensitivity simulation.

6.1.1 50% increase in rainfall

For this scenario, a 50% increase of rainfall intensity was applied to the base climate change scenario as shown in Figure 6-1**.**. The network is highly sensitive to the increase in rainfall; much more so than other scenarios that are described in the following sections. The model is particularly sensitive around the depression south of the Hutt Valley railway line and north of Jackson Street due to the low-lying land. Water generally pools in this area relying on pumps to then discharge the stormwater to the Hutt River.

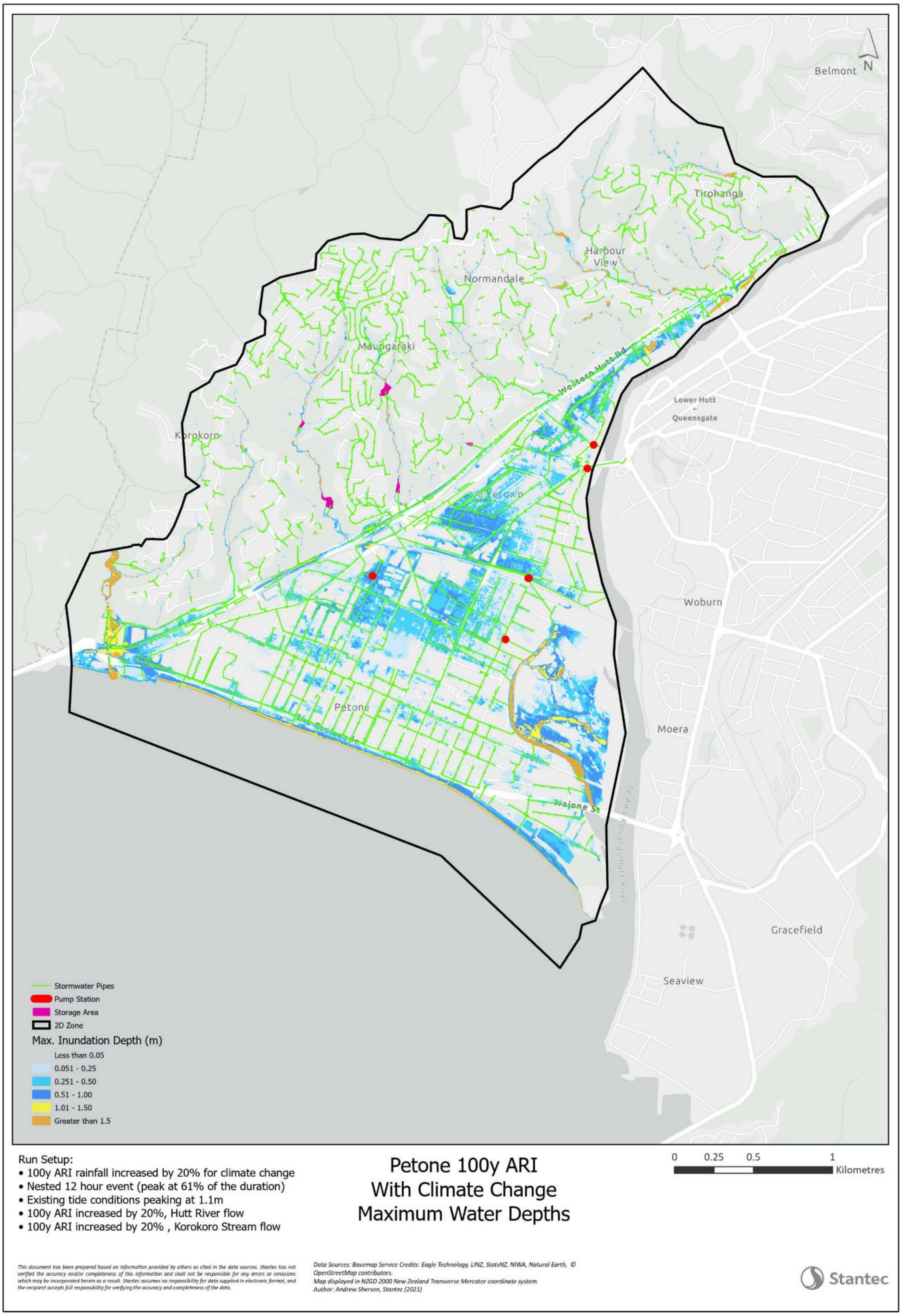


Figure 6-1 ICM model export, 100y ARI nested flooding event, existing climate- 12hr simulaiton

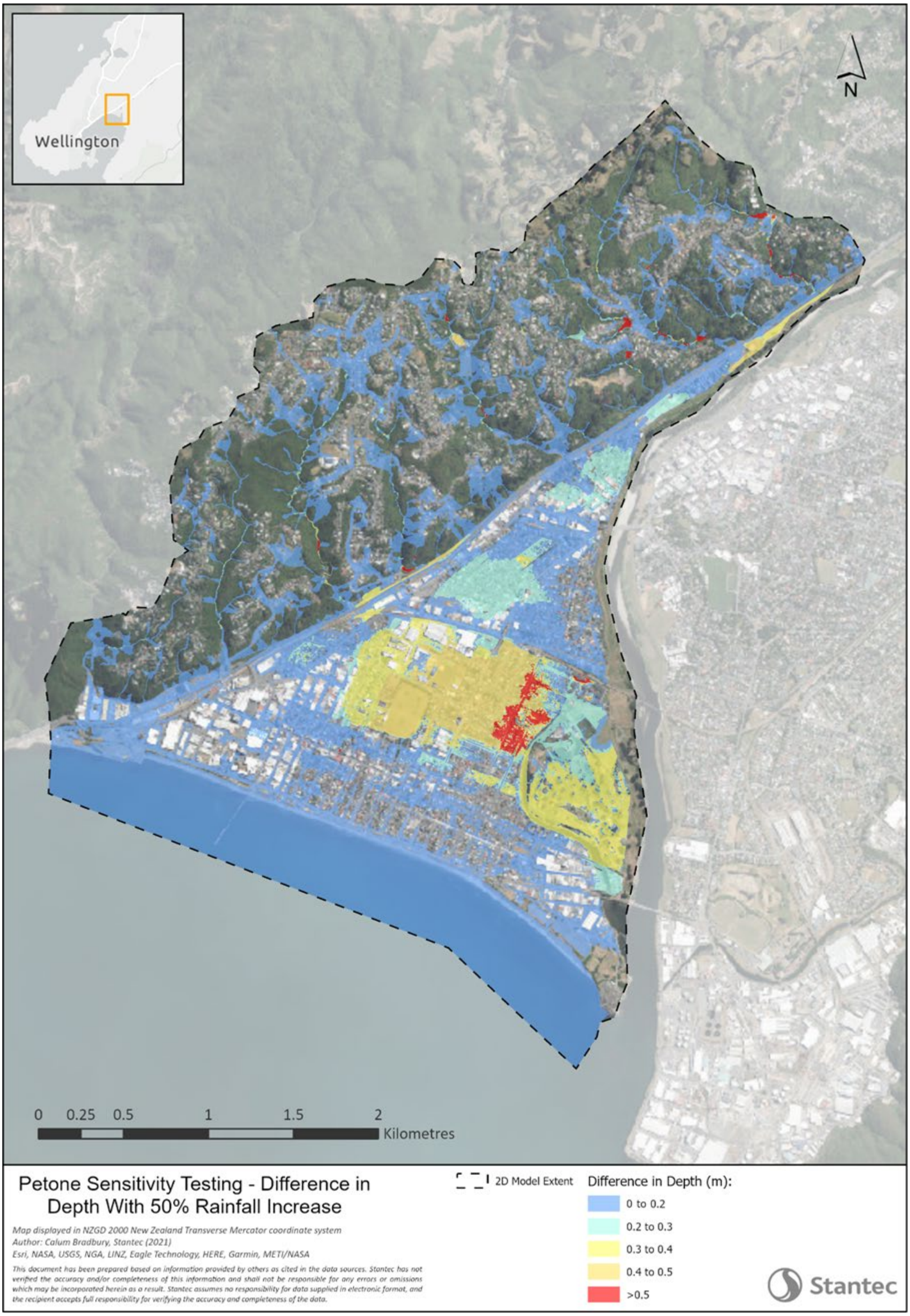


Figure 6-2 Difference in depth between base case scenario and a simulation with a 50% increase in rainfall intensity

6.1.2 Inlets fully blocked

All 139 Inlets were blocked by filling the conduits immediately downstream of inlets with sediment up to 90% of the conduit height. A 100% blockage was not used as InfoWorks does not allow sediment depth to be equal to the pipe diameter. This is because the conduit still requires base flow from the preissmann slot accounting for up to 10% of conduit space. For example, a 600mm conduit could only be filled with 500mm sediment due to the 100mm of base flow.

Smaller pipes required slightly larger percentages of space to run, due to the baseflow requirement. In total five pipes required a lower sediment depth percentage of 83% and one pipe with a 100mm diameter, required a sediment depth of 75% of the pipe diameter. These lower percentages are not considered to have a significant effect on results.

Results in Figure 6-3**.** show that the network is moderately sensitive to the blocking of inlets in a few areas (particularly to the very north of the model) but insensitive at all other locations. This insensitivity is likely due to most sub catchments being directly connected to the network. Surface flow occurs when the network has limited or no capacity. Therefore, even though the inlets are not blocked in the baseline scenario, water may not be able to freely enter the network during the peak of the storm.

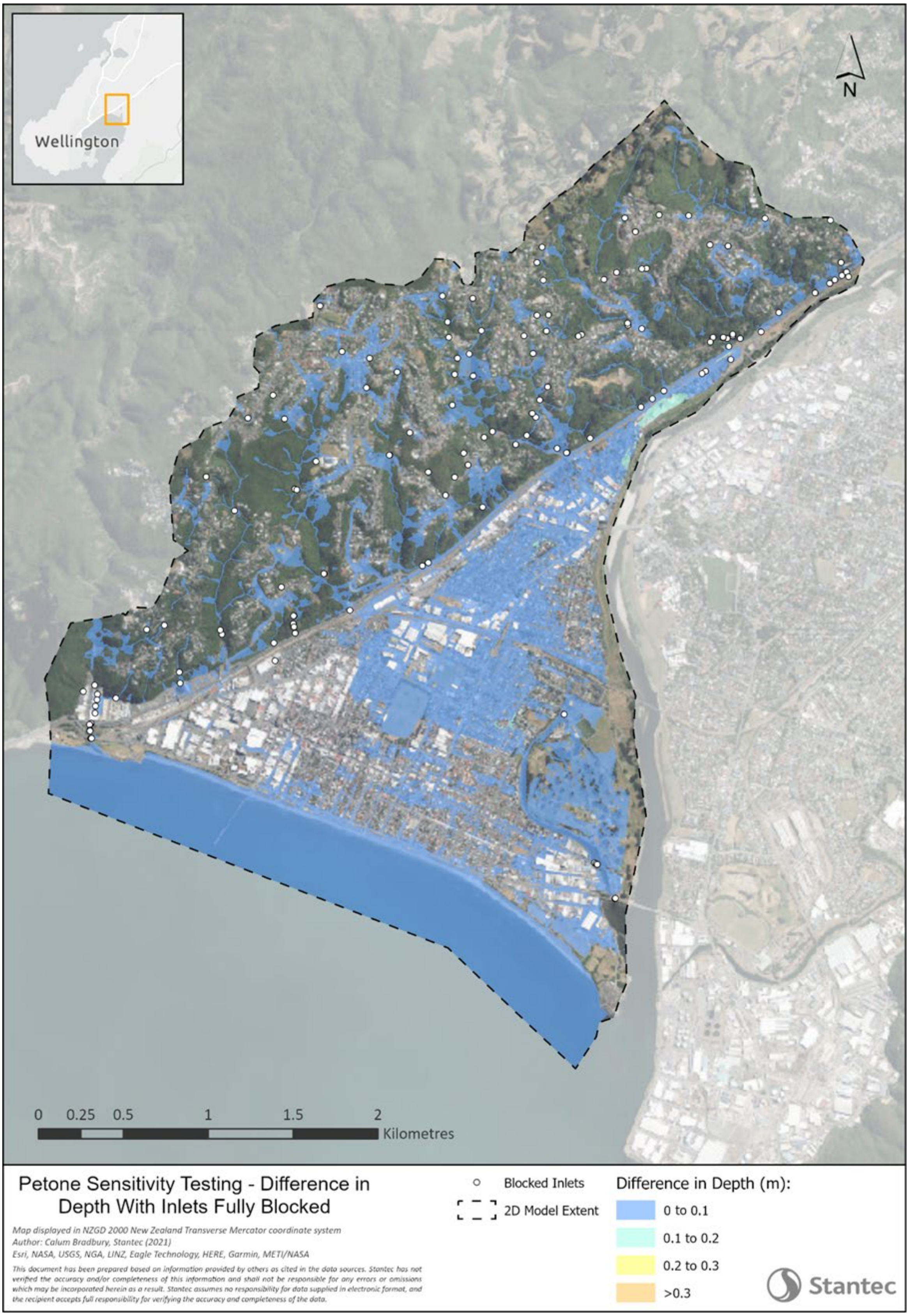


Figure 6-3 Difference in depth between base case scenario and a simulation with full blockage of all inlets

6.1.3 Inlets partially blocked

Conduits immediately downstream of inlets were filled with sediment to half the diameter of the culvert. A total of 139 pipes directly downstream of inlets were included in this scenario. The results in Figure 6-2**.** show that the network is insensitive to the inlets being partially blocked. This insensitivity is due to similar reasons discussed in the full blockage scenario above.

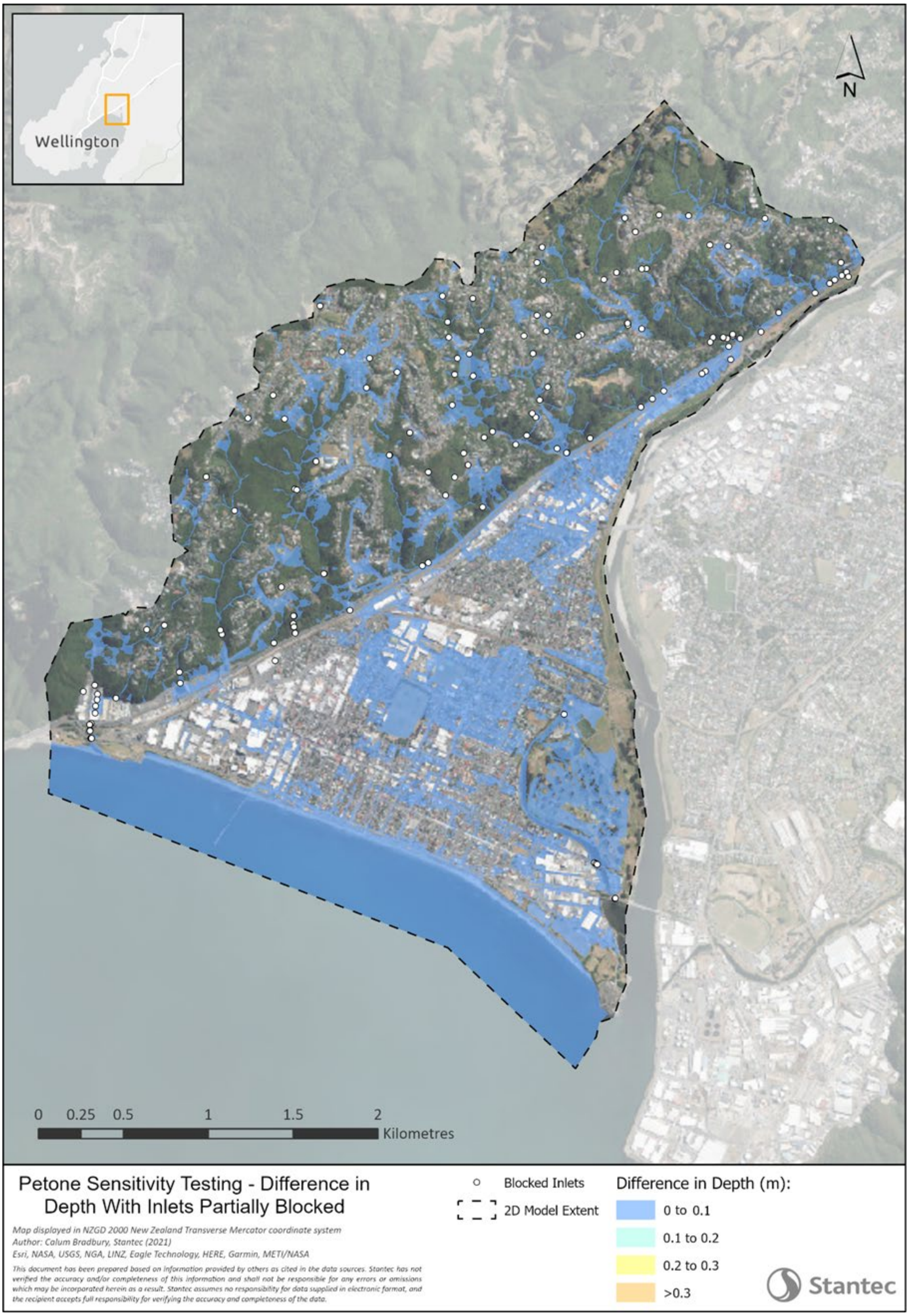


Figure 6-4 Difference in depth between base case scenario and a simulation with partial blockage of all inlets

6.1.4 No Tidal Boundary

The tailwater level boundary was removed to determine the sensitivity of the network to the tide on the waterfront. The base case scenario was already modelled with future sea level rise (Mean High Water Spring +1m) therefore this simulation was to determine how sensitive the network was to the removal of the tide. Results in Figure 6-4**.** show that the network is sensitive to the tailwater boundary conditions along the waterfront and further inland in a few isolated locations as these areas rely on gravity to drain to the harbour.

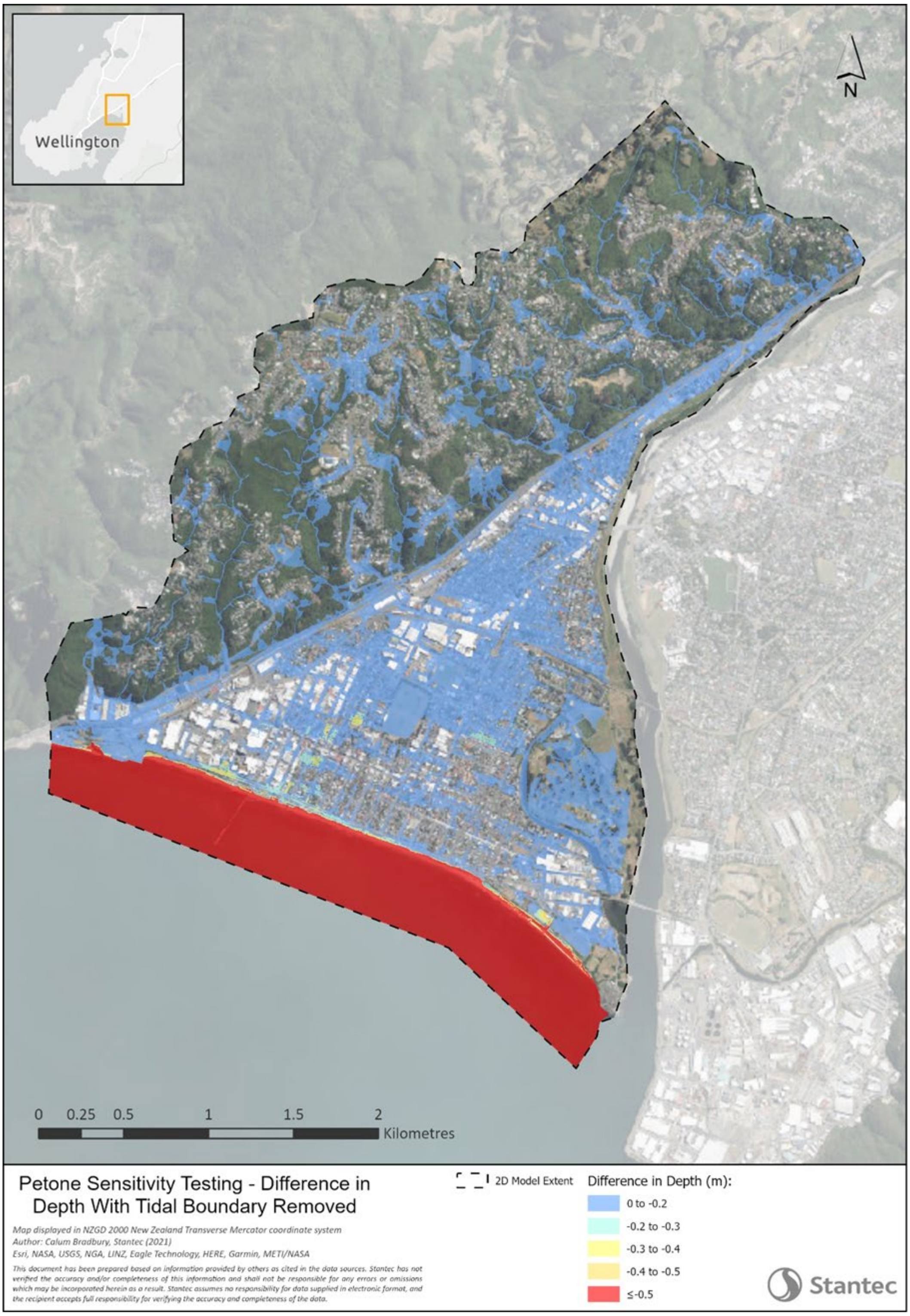


Figure 6-5 Difference in depth between the base case scenario and a simulation with the tidal boundary condition removed

6.1.5 No Hutt River

For this scenario, the Hutt River inflow was removed to assess the impact of the Hutt River level on flooding in Petone, but the tidal conditions are still modelled. Results in Figure 6-5**.** show the greatest difference in depth was found to be in 3 main areas. The northern and southern most regions are directly related to less river inundation with the removal of the boundary condition. The northern most section is located on the Hutt River side of State Highway 2, which acts as the stop bank in this area, so is directly influenced by levels in the Hutt River. Inundation in the Southern section of the model occurs as there is no defined Hutt River stop bank in this location. Changes in flood depths in the central hotspot occur due to the pumps being able to operate more efficiently.

As expected, the western side of the model showed little sensitivity to the removal of the Hutt River inflow. There are a couple of additional isolated areas showing a depth difference greater than 0.2m.

Figure 6-6 Difference in depth between the base case scenario and a simulation with the Hutt River inflow removed

6.1.6 No Pump Stations

For this scenario, all pump stations were deleted from the model to assess the impact of the pumps on flooding in Petone. Results in Figure 6-6**.** show the greatest difference in depth was found to be on William Street close to the Wilford Primary School, with a difference of 0.2 to 0.3m extending from High Street to the Hutt Valley railway line. This increase in flooding is related to the removal of the largest pump station Te Mome along Wakefield St. As expected, the western side of the model showed little sensitivity to the removal of the pump stations.

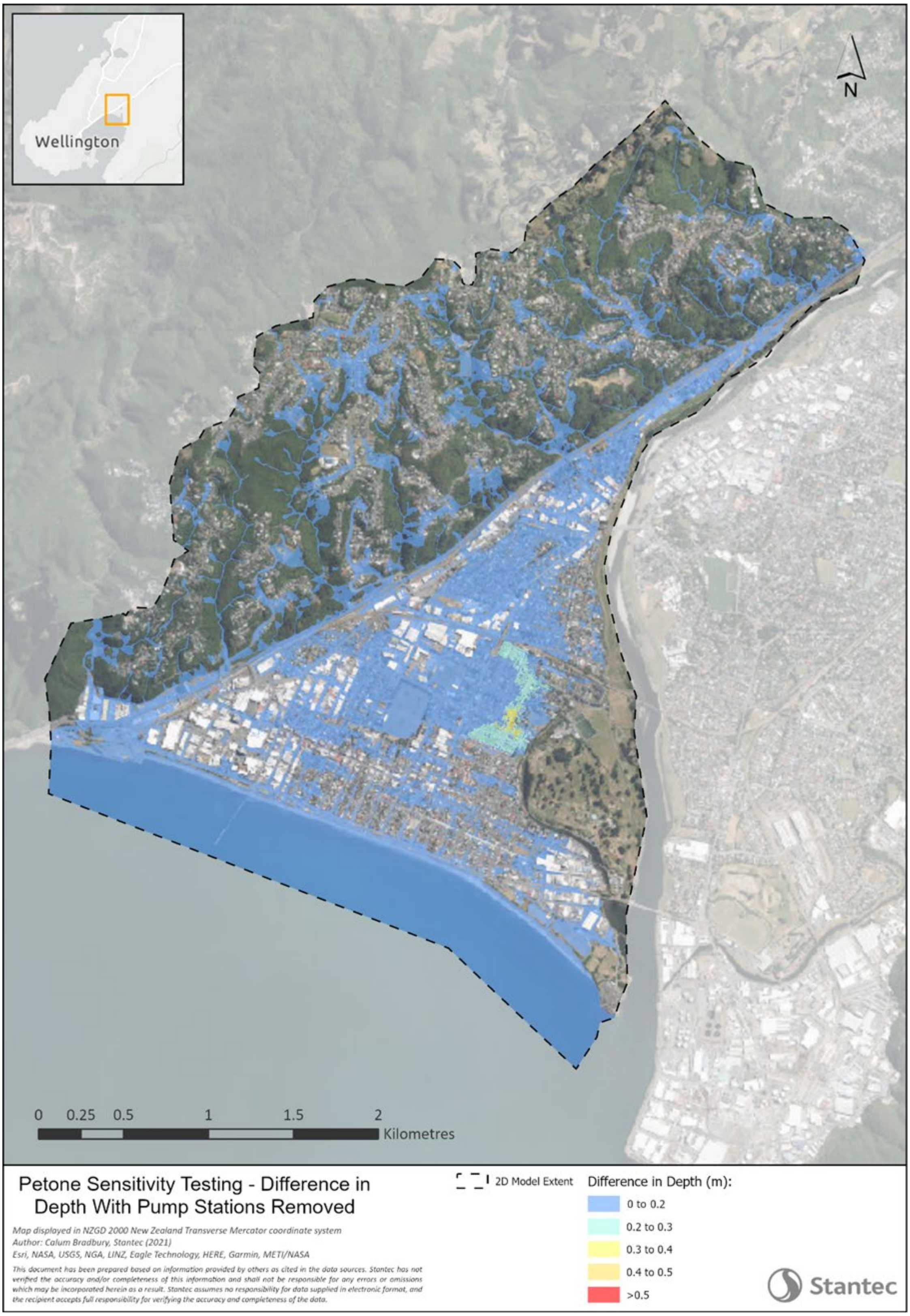


Figure 6-7 Difference in depth between the base case scenario and a simulation with the pump stations removed

6.2 Sensitivity Assessment Summary

Stantec has undertaken a sensitivity assessment for the Petone catchment. Model sensitivity was determined by simulating the following scenarios: 50% increase in rainfall intensity, inlets partially blocked, inlets fully blocked, tidal boundary removal, Hutt River inflow removal and pump station removal. Analysis showed that the network was most sensitive to the 50% increase in rainfall with depth increases of over 0.5m in some areas, particularly in the lower reaches of the catchment. .

6.3 Freeboard Methodology and Selection

6.3.1 Freeboard Allowance Selection

A freeboard margin is applied to 100yr ARI (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 6.1 which investigates how sensitive the Petone network is to these key model parameters. After discussions with Wellington Water, the freeboard allowances shown in Figure 6-7 were applied across the network.

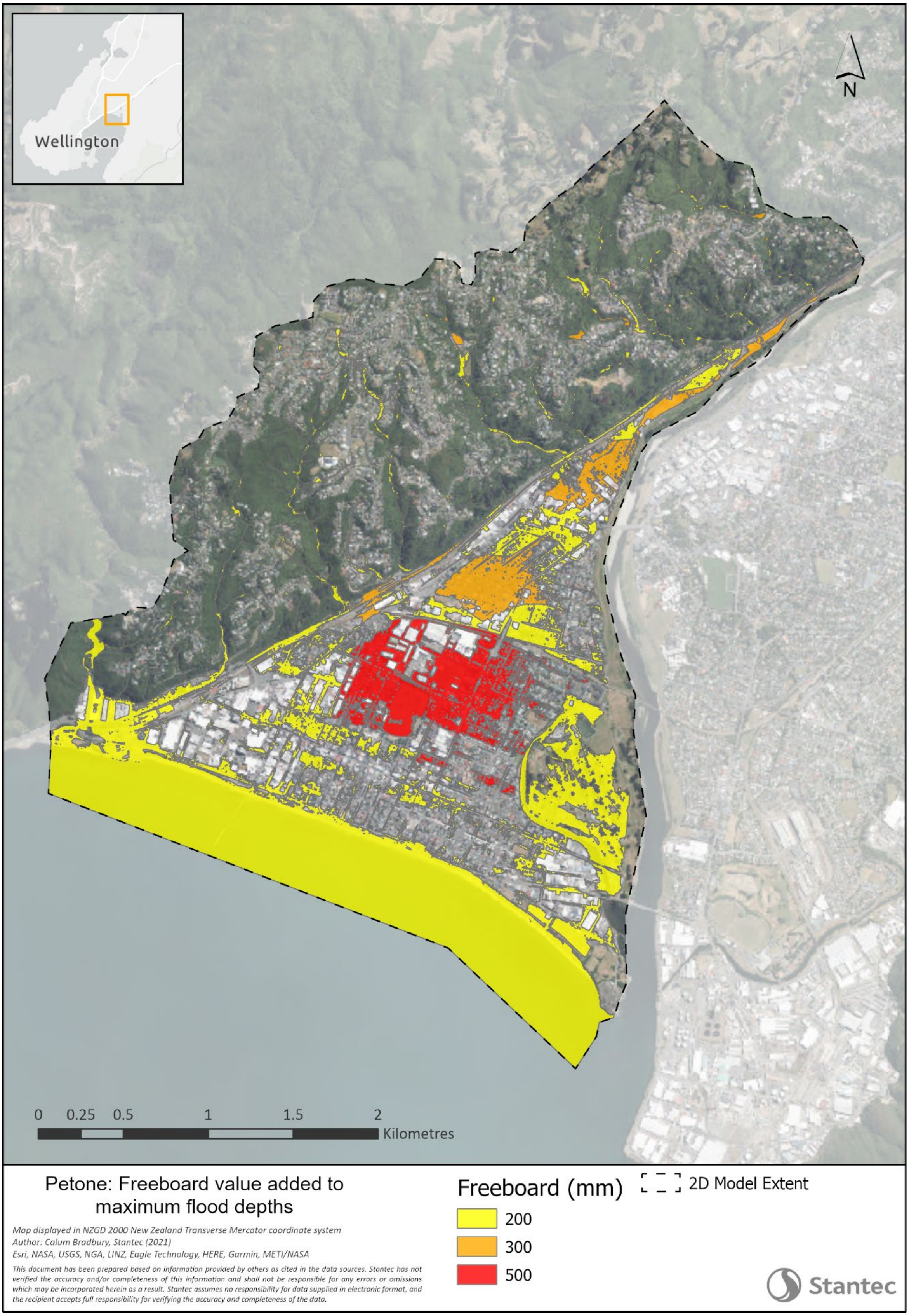


Figure 6-8 Freeboard allowance applied across the Petone network

6.3.2 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. A simulation including only the Initial Conditions (2D) file was then run for five minutes. This allowed the maximum water levels to spread naturally across the catchment to establish baseline flood extents to be used for assigning minimum building floor levels for Petone.

6.3.3 Dynamic Freeboard Results

Maximum flood depths with freeboard allowance are shown below in Figure 6-8.

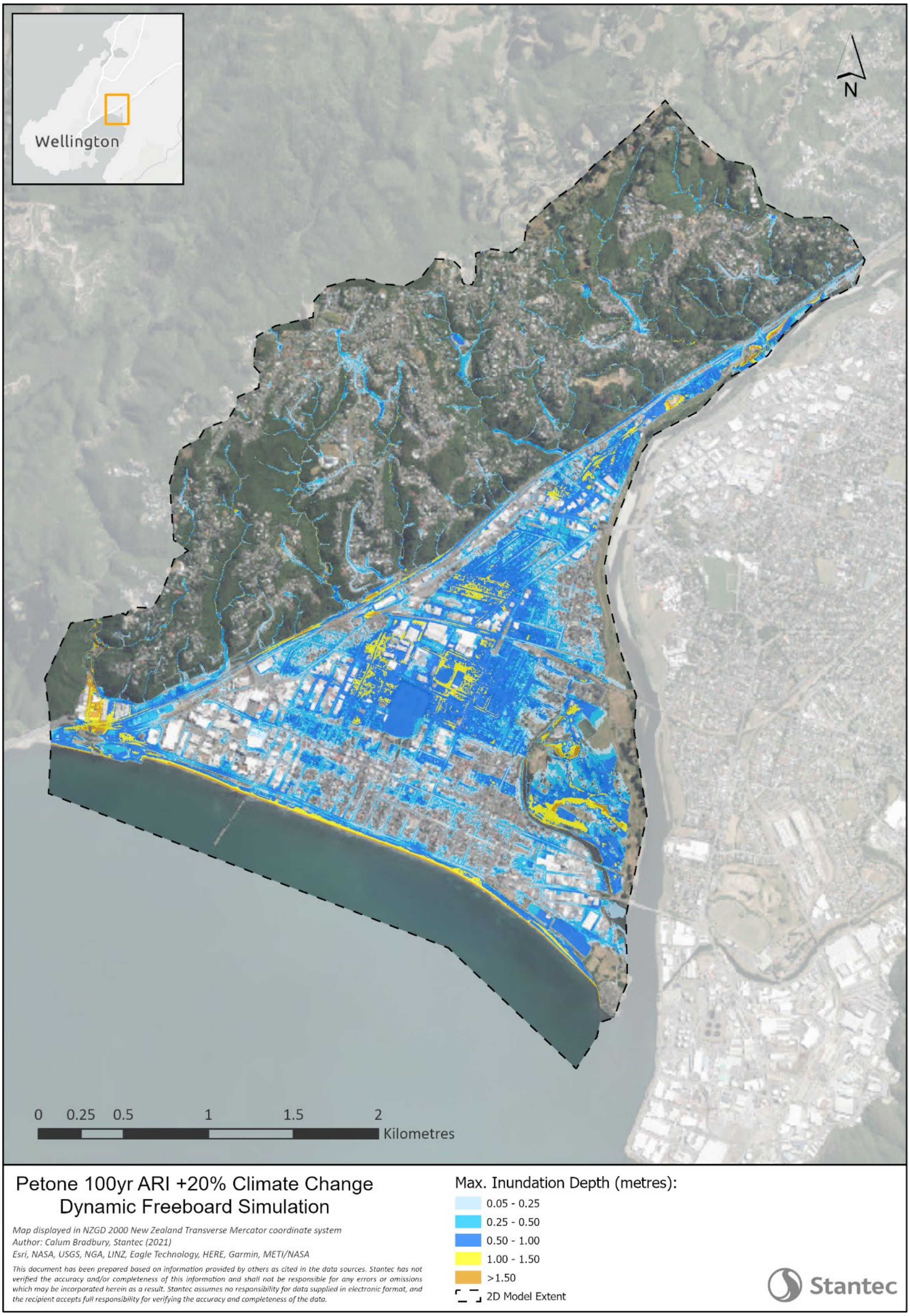


Figure 6-9 Maximum flood depths for the 100y ARI + 20% CC design event including dynamic freeboad

7 Conclusions

This report describes the build phase of the Petone stormwater model.

Both UMM and ICM were used to develop the 1D and 2D features in the model. Hydrological parameters were calculated from regional GIS layers and application of the hydrological method descried in the *Quick Reference Guide for Design Storm Hydrology* (Cardno, 2016).

A 2D mesh was then developed for Petone and a small portion of the Wellington Harbour. Pump stations, detention dams, the Korokoro Stream, and the Hutt River were included in the model as 1D structures. Design rainfall profiles for the 10-year ARI event with existing climate and the 100-year ARI event with existing and future climate were simulated.

The model results show that flooding mainly occurs around the central regions of Petone and around the mouth of the Korokoro Stream. The results also indicate that the extent of flooding is influenced by flood levels in the Hutt River which in the design event cases, have been set as a constant high level.

The model calibrated well to the 2015 event and shows only slightly conservative results.

Increasing rainfall caused the largest change in flood depths when undergoing a sensitivity analysis.

Future users of the model should be aware of the key limitations that have been identified through the course of this study. These are fully documented within this report and include:

Invert information was not available for 80% of the network. Extensive interpolation was required.

There is approximately 39,000m^3 of artificial storage in the Western Hills, likely due to poor LiDAR resolution causing artificial ponds or depressions. This may have some impact on modelled flood levels in Petone.

The freeboard and sensitivity process uses shorter duration simulations.

8 Recommendations

Recommended next steps to improve model performance:

- The model contains some very steep pipes. Currently, there is no standard approach for modelling these and it is recommended that a standard approach be developed for future model builds and if necessary, retrospectively applied to the Petone model.
- Many subcatchments in the model have areas of less than 0.1 hectares, created by the subcatchment automation process. It is recommended that this apparent anomaly be investigated and options for reducing the number of very small catchments investigated.
- Sections of the network are missing from the GIS database, particularly around SH2. Site investigations and an in-depth study of the available as-builts are recommended.
- Approximately 80 per cent of network inverts are unknown and/or interpolated. It is recommended that the model be updated with this information as additional data becomes available.
- To determine the performance of the detention dams and their impact on flood attenuation it is recommended that a range of rainfall durations (and associated total depths) be tested with a standard rainfall temporal profile (as opposed to a nested profile).
- Based on HIRDS data there appears to be no significant variations in rainfall distribution between the elevated hill areas of the catchment and the Petone floodplain. If relevant rainfall data become available in the future, this assumption should be checked.
- A transition to Hirds v4 needs to be considered for any further analysis.
- The calibration of the Hec HMS model should be completed at a later stage in agreement with WWL.
- The freeboard and sensitivity process should be redone during future model updates with longer simulation runtimes.

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Appendices

We design with community in mind

Appendix A Asset Data Uncertainty

As discussed in section 3.3.2.3 the received GIS data was missing some key assets along SH2. This missing information was filled in using RAMMS data. Because the RAMMS data is not complete and is often absent of key information, some uncertainty on the assets along SH2 remains. For instance, Google Street View shows a missing central drainage structure near Melling. As shown in Figure A1and Figure A2. There are small entrances in the road barrier along the southbound lane but not in the northbound lane. This indicates some form of drainage which is not in the asset information.

Figure A1 SH2 southbound lane, showing small drainage inlets near Melling. Image from Google Street View.

Figure A2 SH2 Northbound Lane near Melling. Image from Google Street View.

A.1 Possibly Misaligned Assets

When looking at the results, it was found that some pipe inlets were not located in low points. This occurred primarily along the Western Hills. This possible misalignment could lead to an over prediction of flooding in areas where the pipes cannot pick up the overland flow. For example, the culvert shown in Figure A3 and Figure A4 picks up very little overland flow, increasing local flooding along SH2.

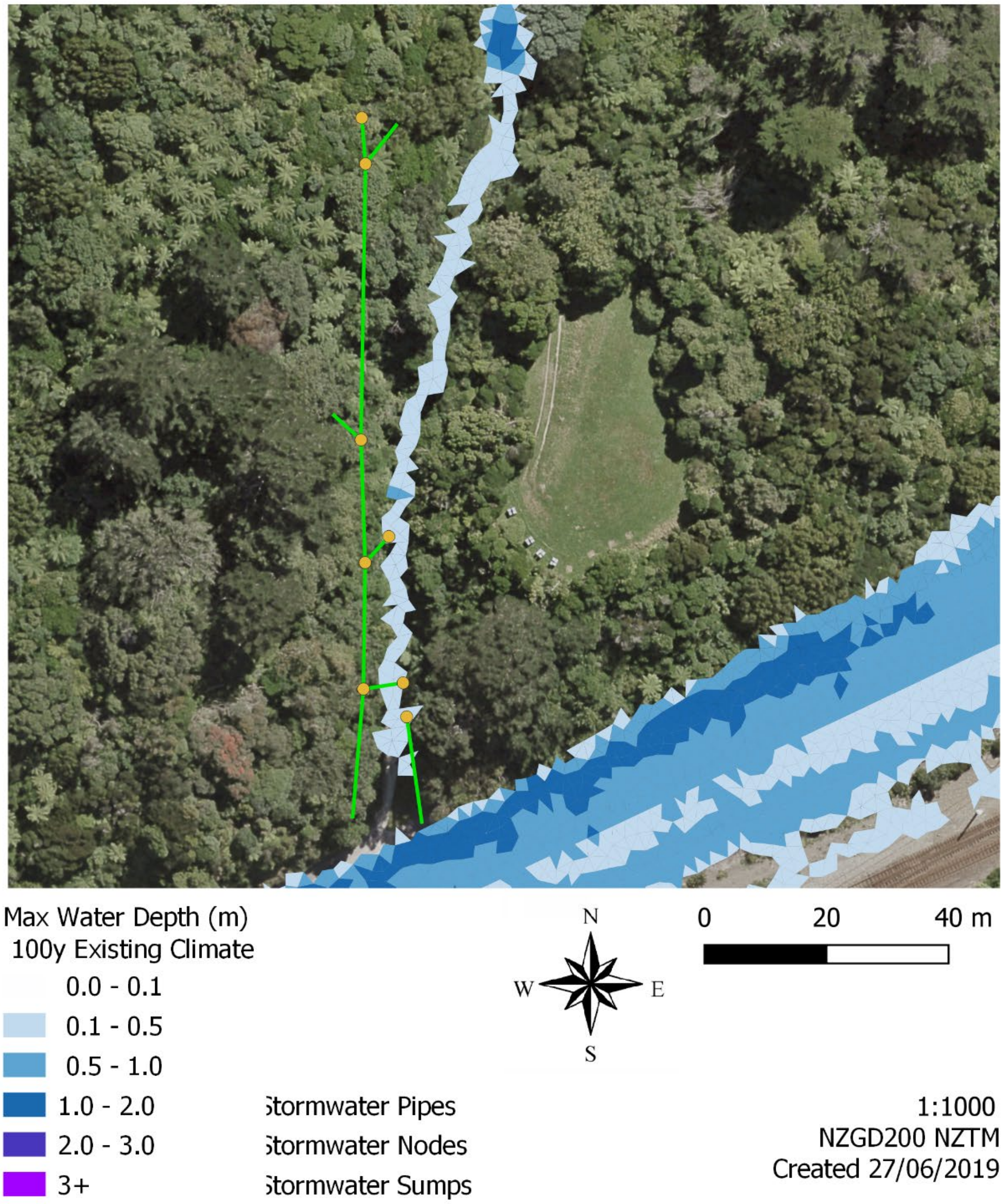


Figure A3: Possible incorrect pipe alignment in the model, or DTM issue.

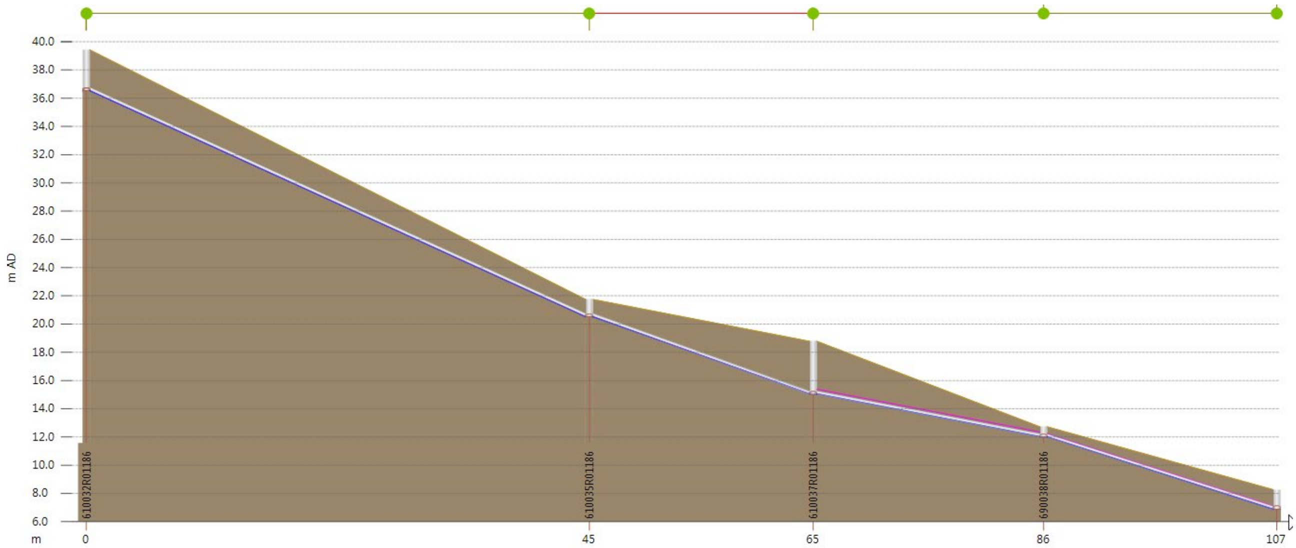


Figure A4 Conduit long section, showing max water levels.

A.2 Korokoro Dam

As stated in Section 3.3.2, the Korokoro dam was not included in the HEC-HMS model used to calculate Korokoro Stream inflows into the ICM model as the schematics of the structure were not received and the dam was assumed to not have a significant impact on the max flows. The validation runs confirm this where depths across SH2 are around 0.7m in some areas making it undrivable, agreeing with observed flooding shown in Figure 2-3 to Figure 2-5 and discussed in Radio New Zealand (2015). The overall flood extents also match observed flooding impacts showing the inundation of SH2, Cornish Street and the nearby buildings.

A.3 Empty Pipes

There are ~300 pipes that do not have any water flowing through them in the 100y ARI existing climate run, see Figure A5. These empty pipes are primarily caused by the automatic subcatchment-to-node-link process conducted in UMM, which links each subcatchment to the closest downstream sump. Due to atomisation, some pipes are left without any inflow as they are upstream of the closest sump. Overall, these empty pipes occur primarily in the Western Hills where the gradient is too steep for backward flow to occur. It is expected that these empty pipes will only have a small impact on the overall flood mapping, but may have an impact at a site level, and thus it is recommended when doing any site-specific investigations, a more detailed model investigation is conducted.

Figure A5 Empty pipes overview.

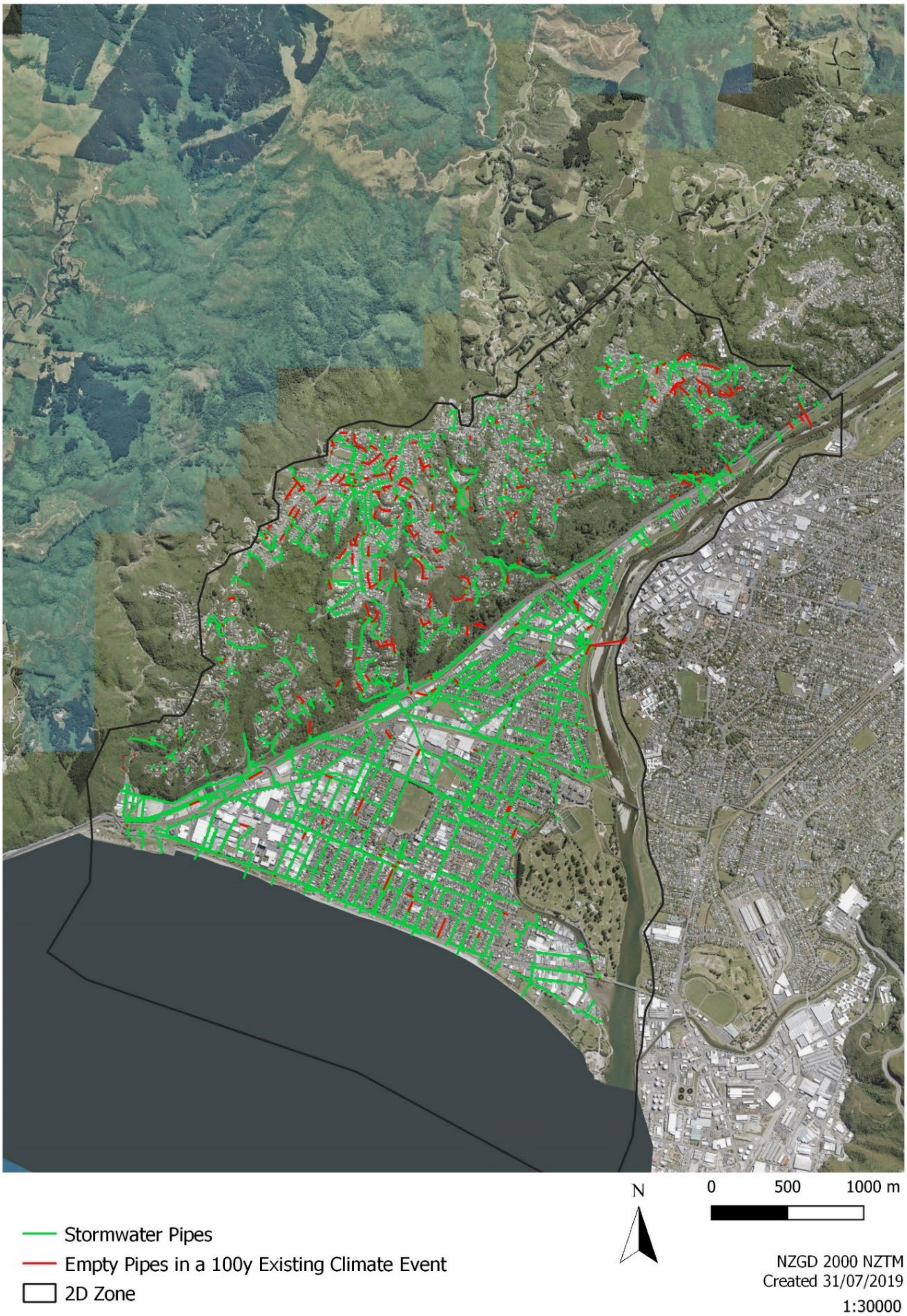


Figure A5 Empty pipes overview.

Appendix B Attenuation in the Western Hills

There is about 114,000m^3 of storage volume in the Western Hills when including all depressions in the hills, excluding the five known detention dams. In comparison, the main Petone depression, in the flatter areas of Petone has a storage volume of 867,000m^3 if full to the spill level of 2.44m (Figure B1). This means the Western Hills can attenuate around 13% of the total capacity of the main Petone depression and may underpredict flooding. However, much of this storage is tangible. For example,Figure B1 shows a significant depression at the intersection of Viewmont Drive and Harbour View Road which has a full capacity of 10,000m^3 and Figure B2 has a full capacity of 9,000m^3. There are many more examples of these manmade and or natural storages in the Western Hills and together make up a significant amount of storage, although are not classified in the council database as detention dams like the Stanhope Dam.

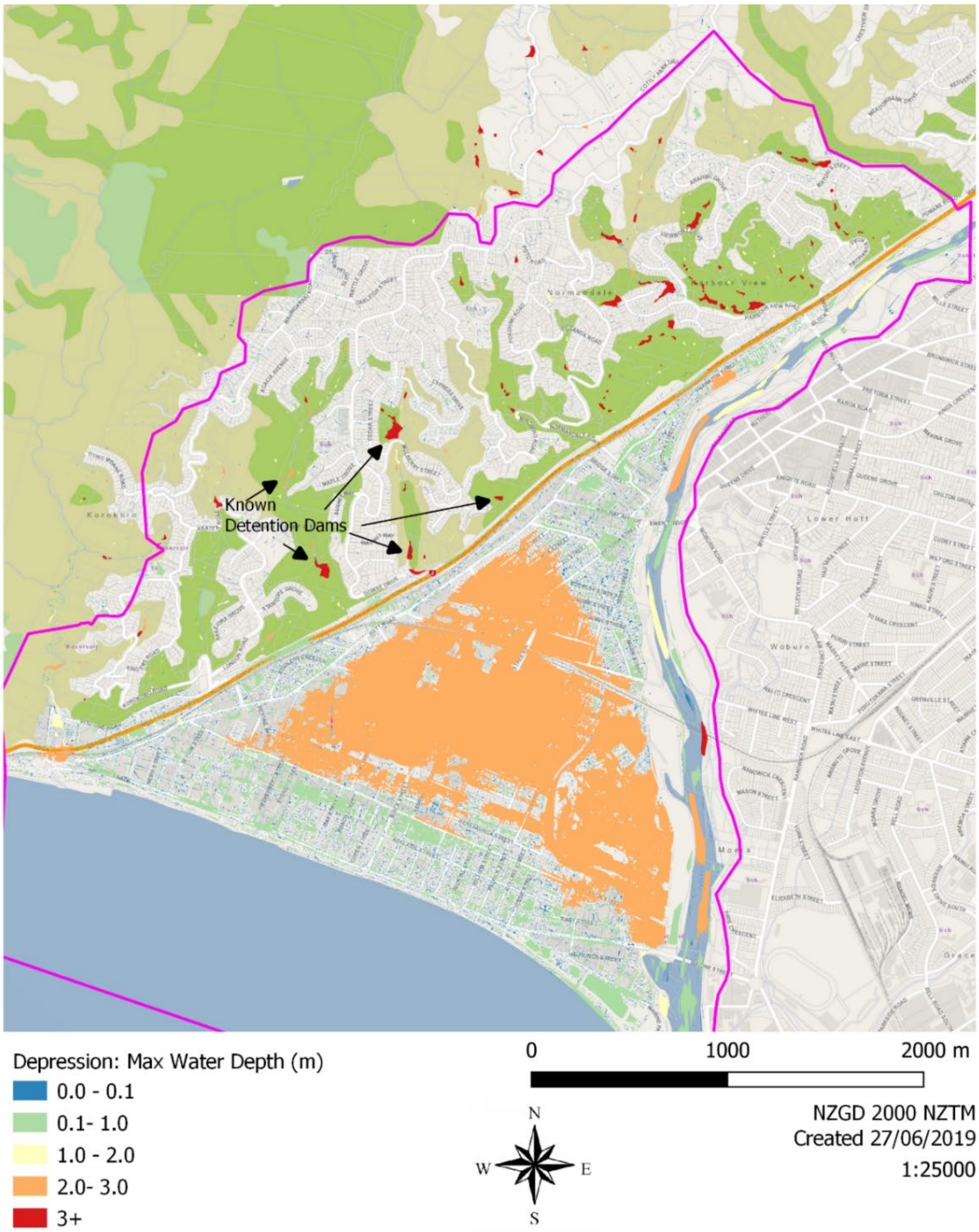


Figure B1 Model depressions overview.

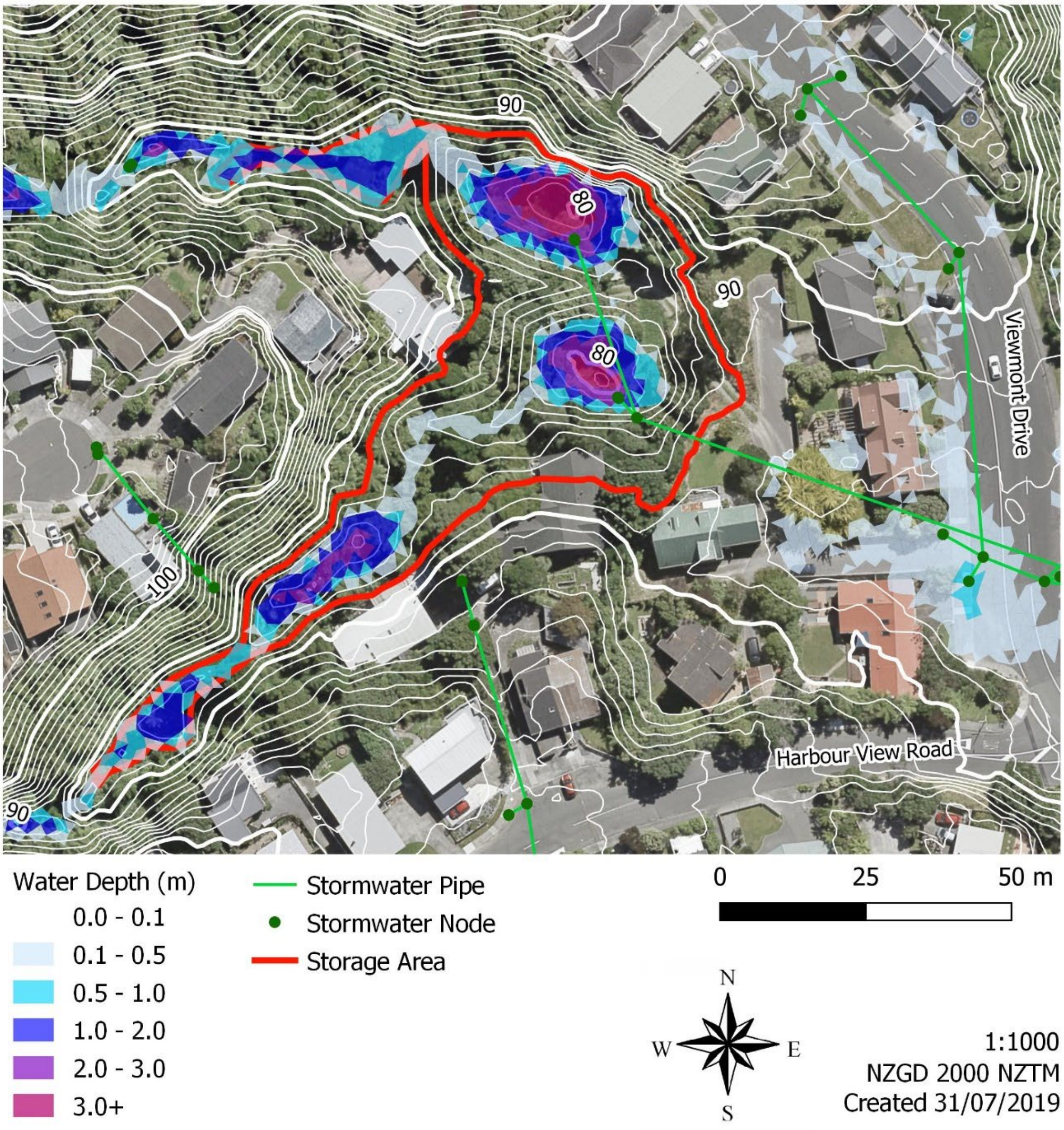


Figure B2 Manmade storage in the Western Hills. Water levels are from a 100y ARI existing climate event.

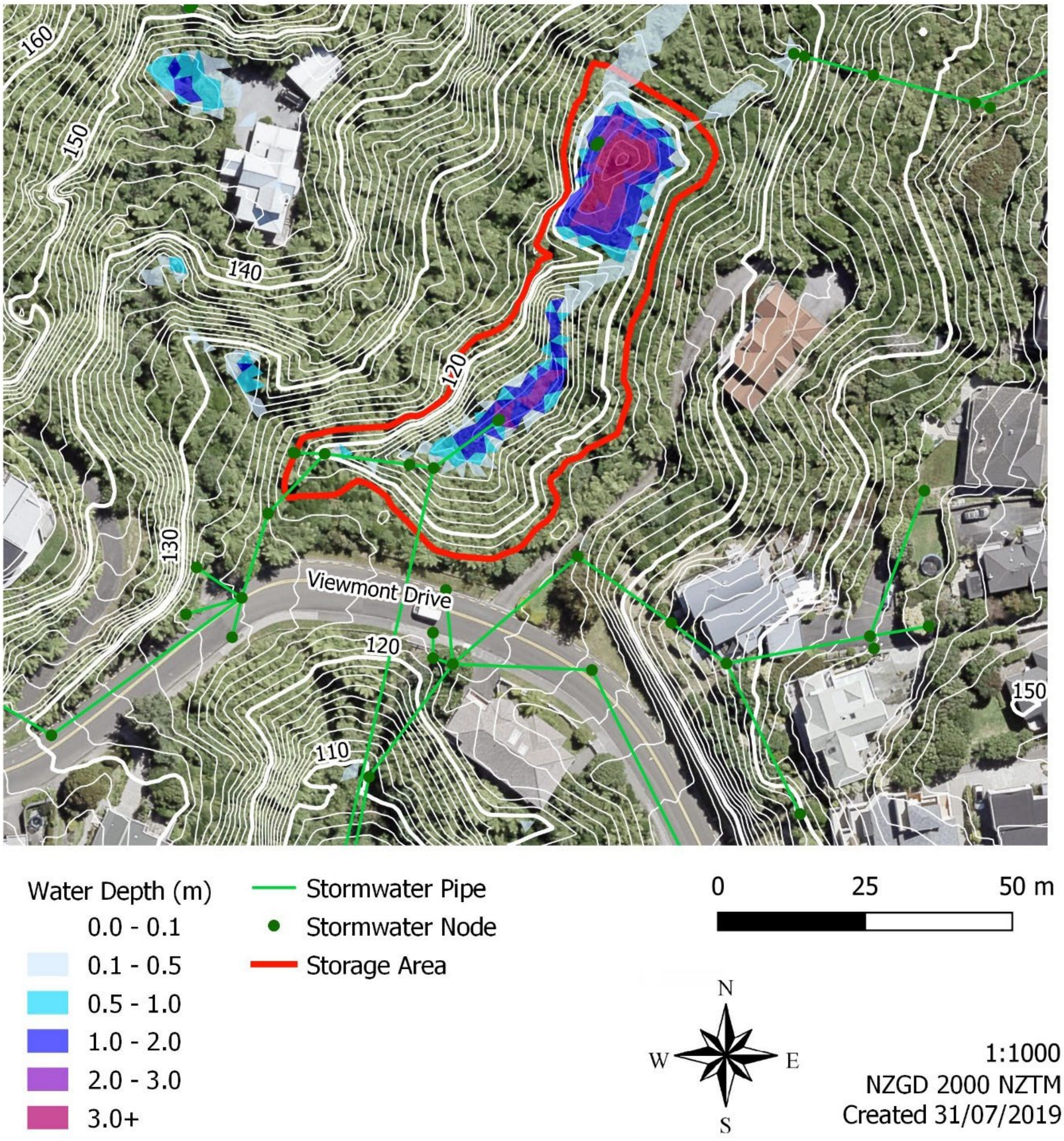


Figure B3 Manmade storage in the Western Hills. Water levels are from a 100y ARI existing climate event.

B.1 Artificial Storage

Flood water attenuation also occurs because of inconsistencies in the DTM as a result of vegetation affecting the LiDAR. This attenuation, unlike the detention dams or real depressions discussed above may not exist, and therefore lead to model results with lower flood peaks than what would occur.

During the model build, 18 mesh level zones were added to remove those artefacts causing the most significant attenuation. Mesh level zones were only added if there were no observable reasons for blockages causing water depths over 3m, as shown in Figure B4.

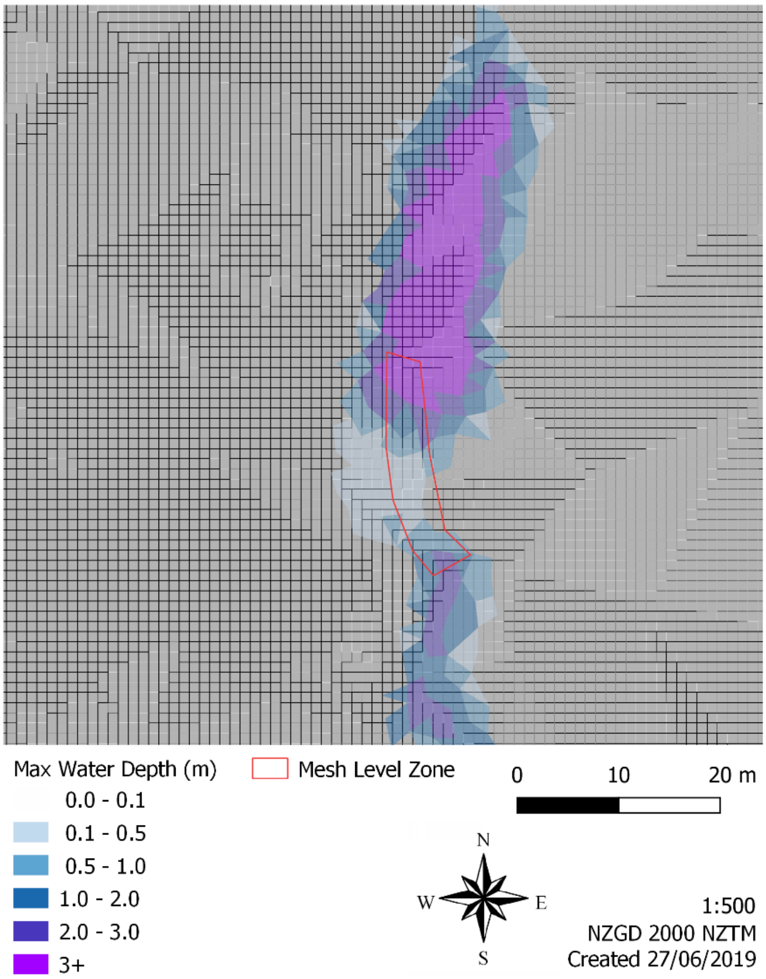


Figure B4 Mesh level zone example with 100y ARI Existing climate results-prior to including mesh level zone.

However, not all significant storages could be checked or rectified. For example, there is a significant depression behind 23 West Point Ave where water ponds up to 6m behind a tennis court, as shown in Figure B5 In this location, there are private pipes that drain the tennis court. However, no private pipes have been included in the model as they were out of scope and number in the tens of thousands, most of which are property connections to the roadside kerb. Therefore, there may still be attenuation occurring in places where it should not.

In total, after ignoring known, natural, and manmade depressions with connected outflows, and areas where mesh level zones were included to help resolve DTM issues, there is 39,000m^3 of artificial storage that may or may not exist. It is difficult to know how this storage volume will impact the results, however, it is less than 5% of the total volume of the main Petone depression, and thus is expected to not have a significant impact at a regional level. However, it could have a substantial impact at a finer scale and thus it is recommended to investigate further before undergoing any site-specific flood level investigations.

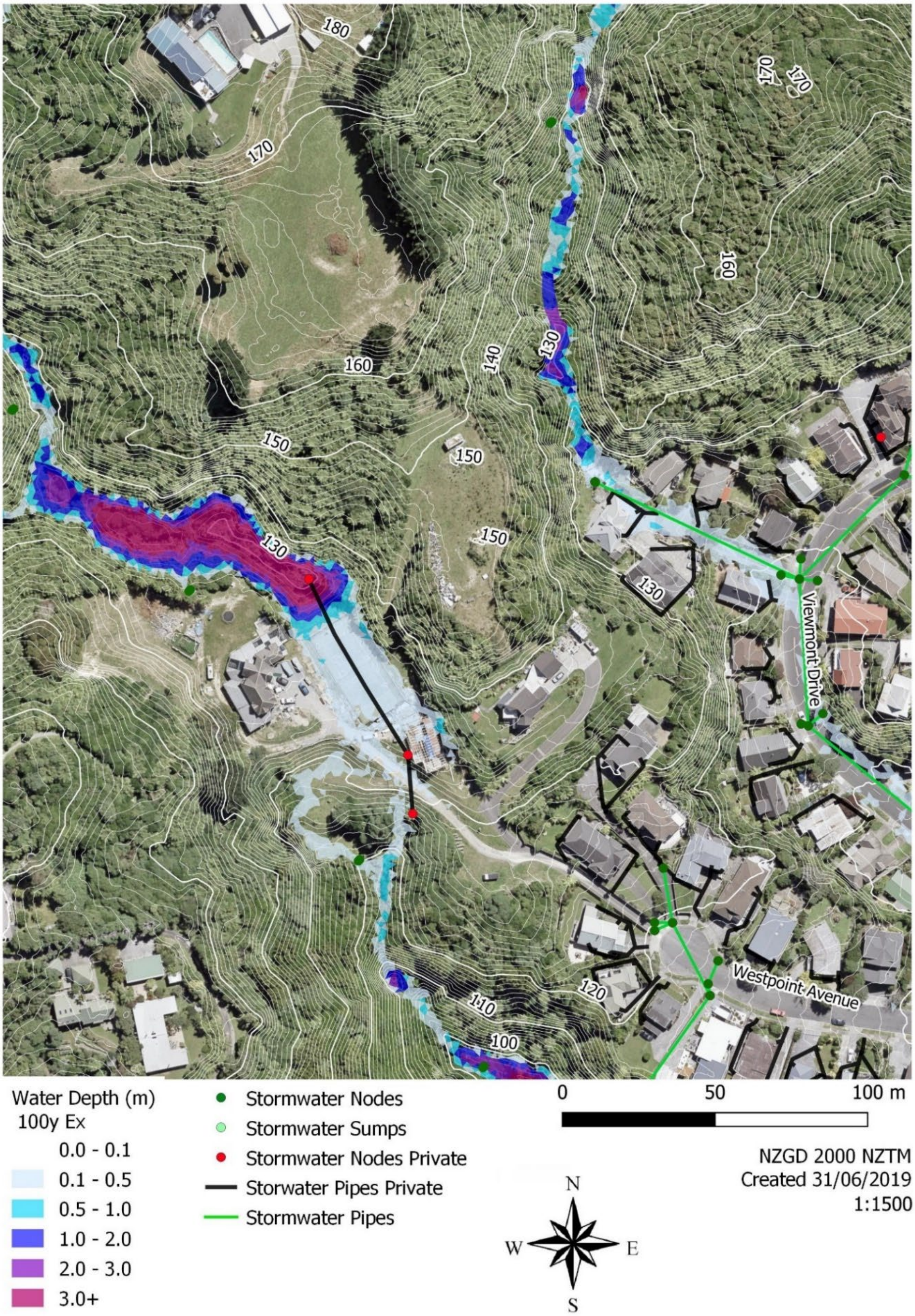


Figure B5 Ponding behind 23 Westpoint Ave

Appendix C Cross Section Surveys

Appendix D Manhole Survey Details

Two manhole surveys were completed by Adamson Shaw to pick up the invert and orientations of pipes entering and exiting eight manholes with GIS flagged inverts in areas subject to significant surface flooding in a 10-year event. Surveys were completed to check the validity of the GIS flagged information, received from council GIS layers.

The first of these surveys focused on picking up the invert levels and orientations of pipes around the Moa Street to Kiwi Street connection (see Figure D1). The survey showed that the 600mm pipe shown to be curved in an older as-built drawing does exist, though there is no evidence for its curvature further upstream. The survey also revealed that there was no direct connection between Kiwi St and Moa St, allowing for separate drainage and less surface flooding.

The second string of manhole surveys were primarily focused on determining the inverts of three manholes linked to a bulk 1200mm line through Petone which appeared to be daylighting from a mix of as-built levels and GIS levels (see Figure D2). The survey returned the correct pipe inverts and an improved ground level confirming that the 1200mm pipe had almost no cover, where one of the manholes was welded straight onto the pipe with no chamber.

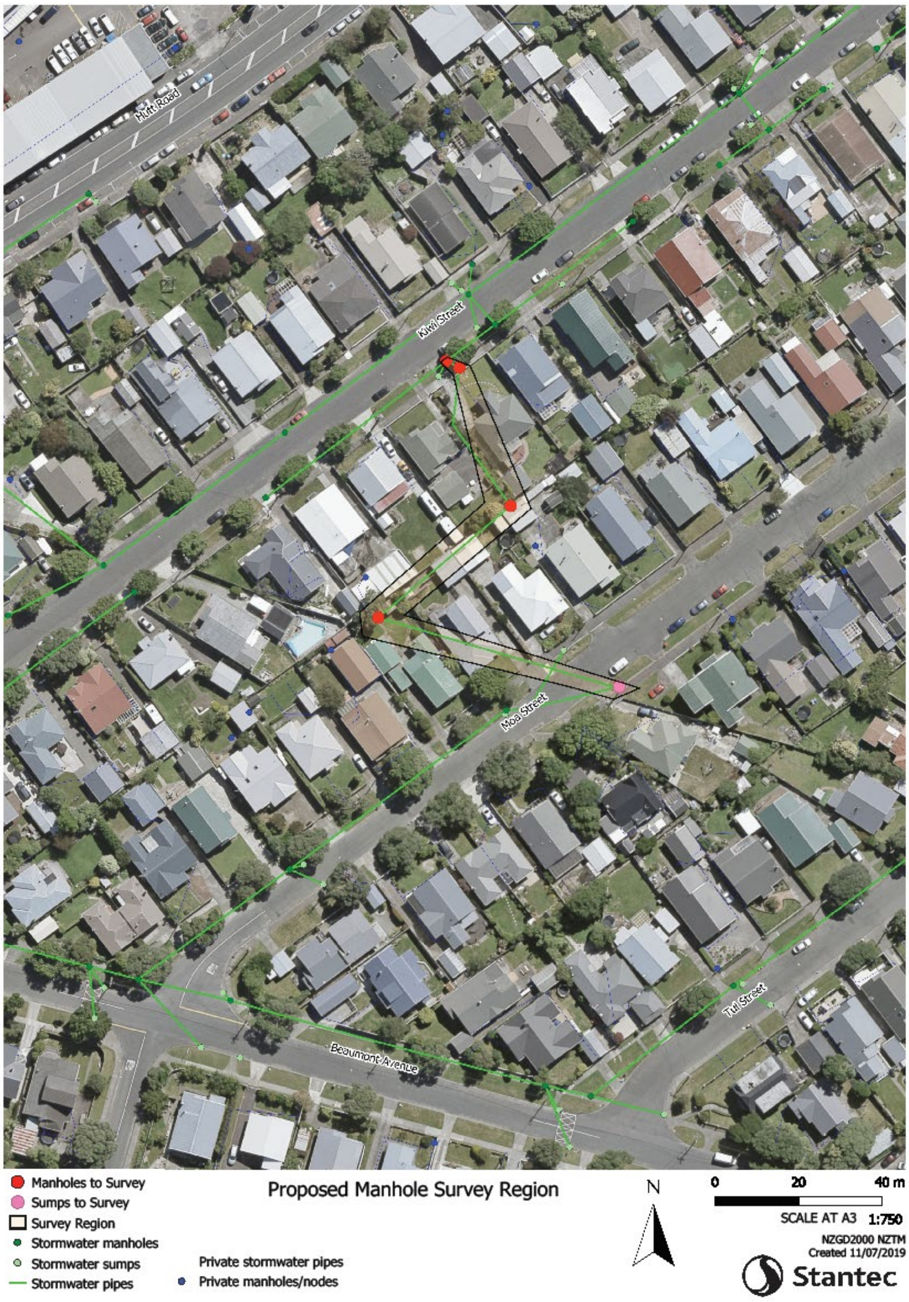


Figure D1 Survey Region 1

Figure D2 Survey Region 2

Appendix E Regional Hydrology Layers

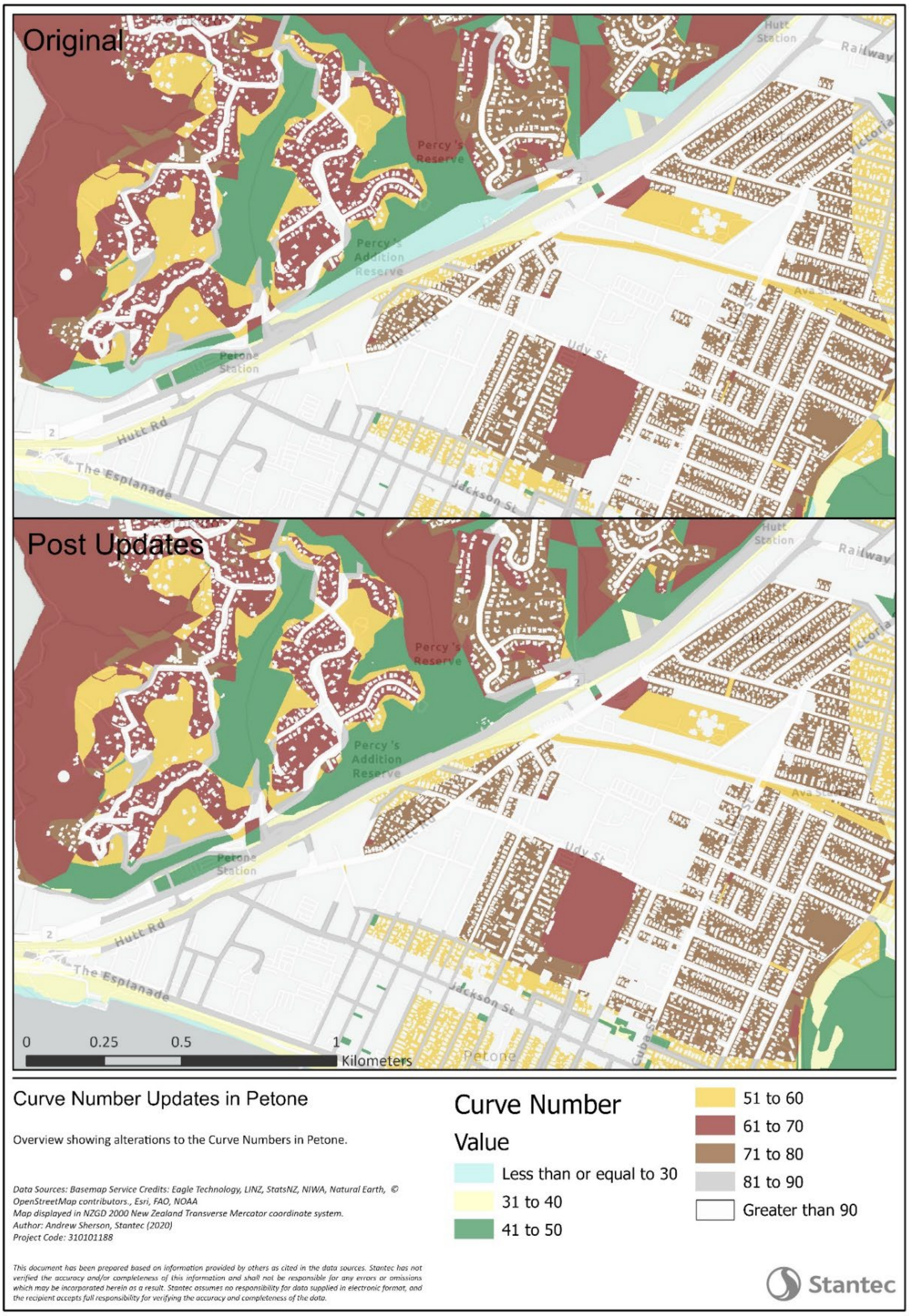


Figure E1 Curve Number updates made to the regional GIS layer

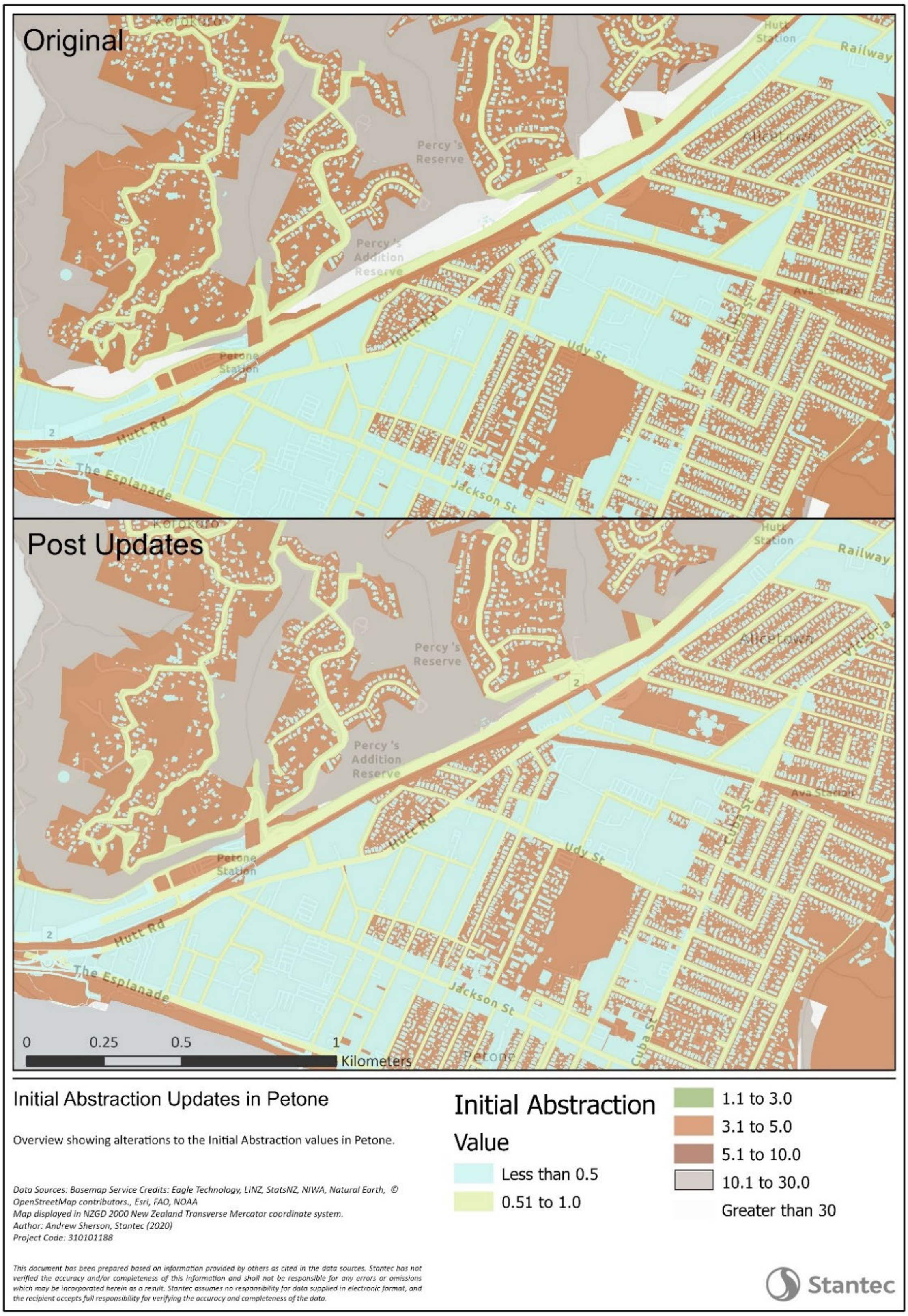


Figure E2 Initial Abstraction updates made to the regional GIS layer.

Appendix F HEC-HMS Model Checks

F.1 Introduction

Rational and Regional method calculations were conducted to check that HEC HMS model outputs were reasonable for the 100-year ARI design event. Both the Regional and Rational method were applied as they are widely recognised and align with WWL methodology, (Cardno, 2016).

A standard rainfall profile was also tested to compare with results of the nested rainfall profile previously simulated in HEC-HMS and used as input to the Petone ICM model.

F.1.1 Rational Method

A standard Rational Method calculation was conducted using the parameters listed in Table 9-1. The rainfall intensity was calculated from HIRDS V3 rainfall depths, using the same location as the ICM model.

Table F1 Subcatchment parameters used for the rational method calculation

Area (km2)	Stream length (m)	Stream slope m/m	Ramser Kirpich Tc (min)	Bransby Williams TC (min)	Average TC (min)	C	I (mm/hr for 143min 100y ARI storm)
15.5	9749	4%	79	207	143	0.6	27.9

F.1.2 NIWA Regional Method

A peak flow estimated for the Korokoro catchment was extracted from the NIWA Flood Frequency Web tool for the Reach: 9012553.

F.1.3 HEC-HMS Standard Rainfall Profile

Following the methodology described in *High Intensity Rainfall Design System, Version 4* (Carey-Smith, Henderson, & Singh, 2018) a 2-hour storm shape was created based on NIWA's rising and falling curves for the East of the North Island. This rainfall event was then applied across the HMS model.

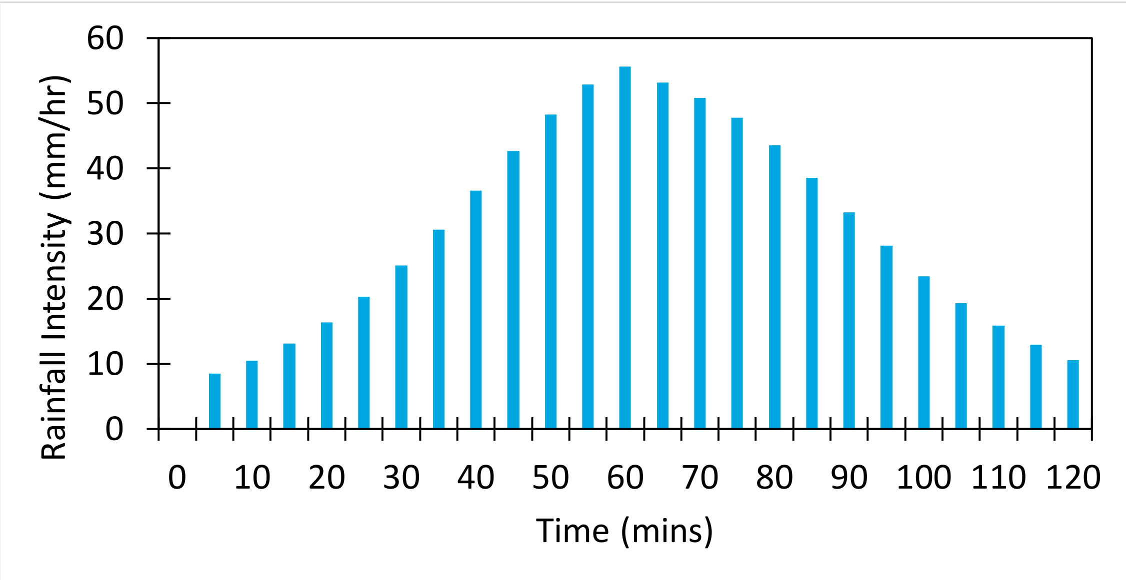


Figure F1 The 2-hour standard rainfall storm shape used in HEC-HMS.

F.2 Results

The Rational and Regional methods produce flows of 72m^3/s and 45m^3/s respectively.

HEC HMS simulations using a 2-hour standard rainfall profile gave a peak flow of 38 m^3/s.

These three peak flow values are all lower than the HEC-HMS simulation results with a nested profile (119 m^3/s) which was used as input to the Petone ICM model.

The results of the comparison are not unexpected as the use of a nested profile is known to produce relatively conservative results.

Table F2: HEC-HMS peak flow comparison (100 yr ARI Event)

Method	Rainfall	Peak Flow (m^3/s)
HEC-HMS (nested profile)	**12hr (nested storm)**	**119***
HEC-HMS (standard profile)	2hr (East of North Island HIRDS v4 Storm (HIRDS v3 total depth)	38
Rational Method	100y ARI 2hr Storm	72
Regional Method	n/a	45

*Adopted as input to the Petone ICM model

Appendix G Data Flags

Table G1: Data flags used in ICM, table from (Wellington Water Ltd, 2017)

ID	Description
#A	Asset data
#D	System default
#G	Data from geoplan
#I	Model import
#S	System calculated
#V	CSV import
USER	User entered value (engineering judgement)
SURV	Surveyed
ASCO	From an as-constructed plan
DEF	Default values
DEM	Ground level from LiDAR Data
PCOV	Invert level based on largest pipe diameter, node ground level and a nominal amount of cover
AUTO	Automatically generated data
GIS	GIS import
GISI	Inferred in the GIS database
LL	Invert level of pipe from inlet/outlet lid level
DEP	Invert level calculated based on node depth
PIPE	Node invert level from pipe invert level or node diameters calculated form connected pipe diameter
SUMP	Calculated sump parameters
HCALC	Calculated head loss using ICM Default calculation

Appendix H Pipe Asset Data Updates

H.1 Pipe Diameter Updates - Examples

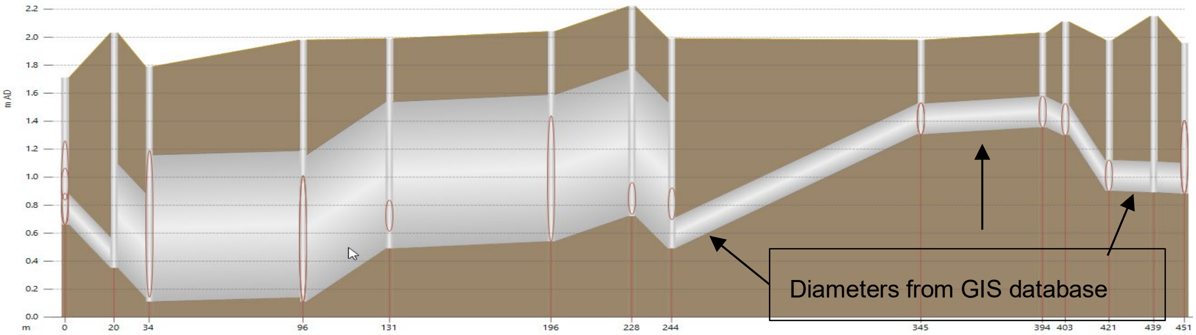


Figure H1 Example long section before diameter update. Flow direction left to right

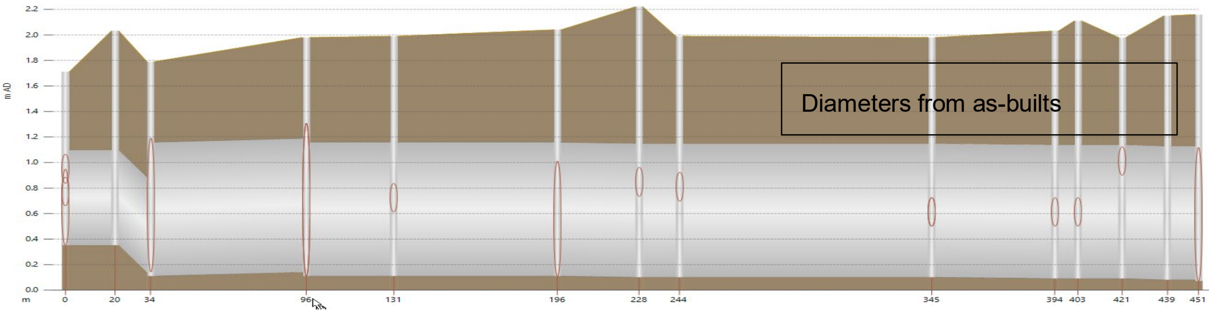


Figure H2 Example long section after diameter update. Flow direction left to right

H.2 Pipe Negative Gradients - Examples

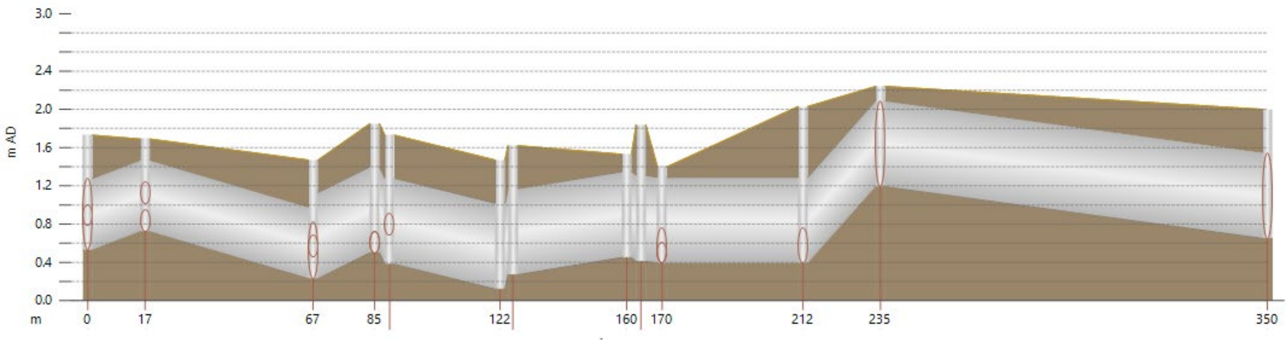


Figure H3 Example long section 2 before invert level updates. Flow direction left to right

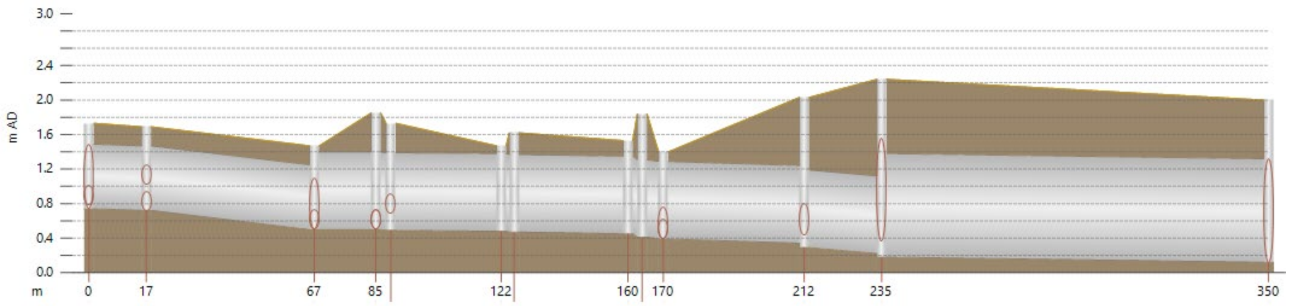


Figure H4 Example long section 2 after invert level updates. Flow direction left to right.

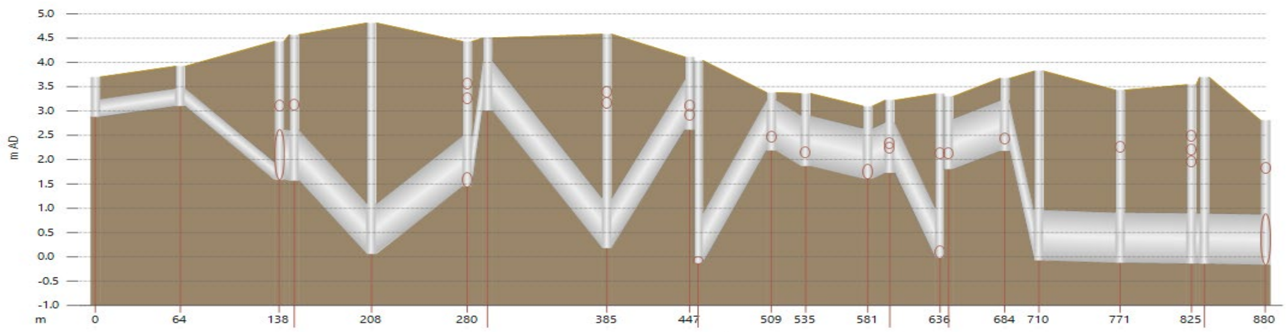


Figure H5 Example long section 3 before invert level updates. Flow direction left to right.

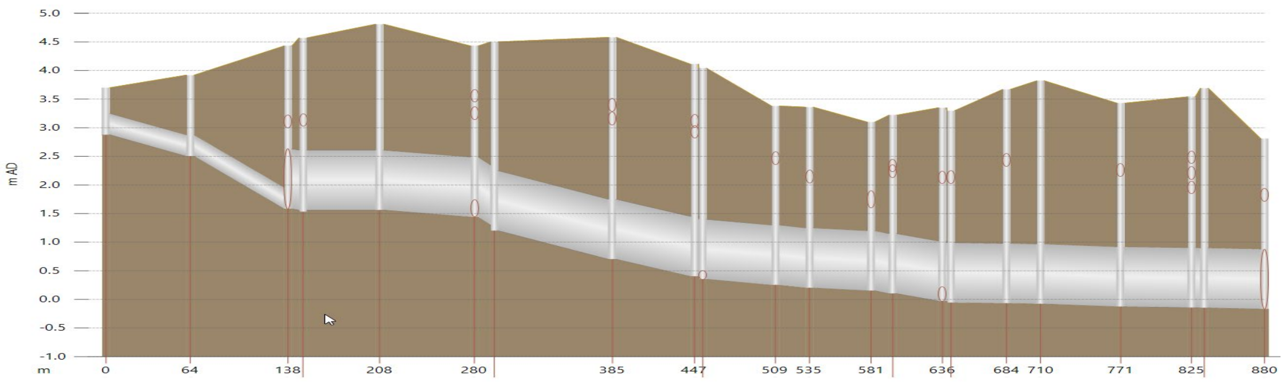


Figure H6 Example long section 3 after invert level updates. Flow direction left to right.

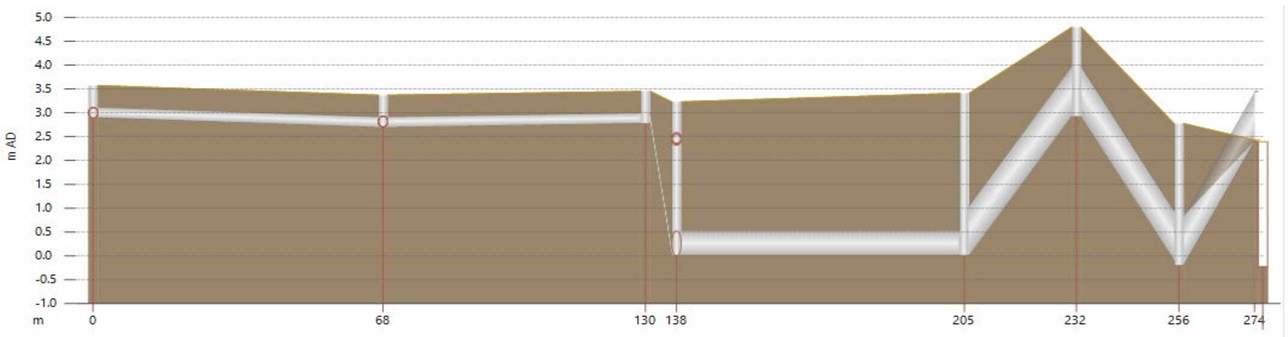


Figure H7 Example long section 4 before invert level updates. Flow direction left to right.

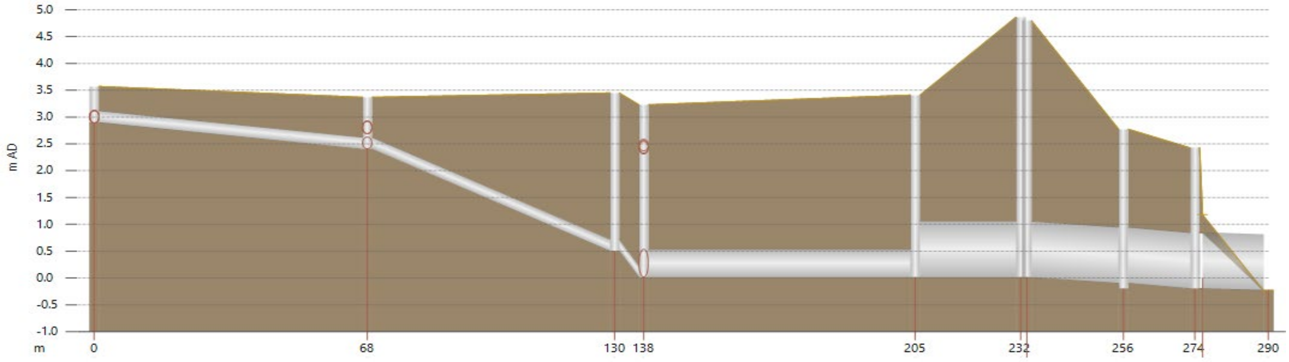


Figure H8 Example long section 4 after invert level updates. Flow direction left to right.

Appendix I Head/Discharge Curves for Sumps

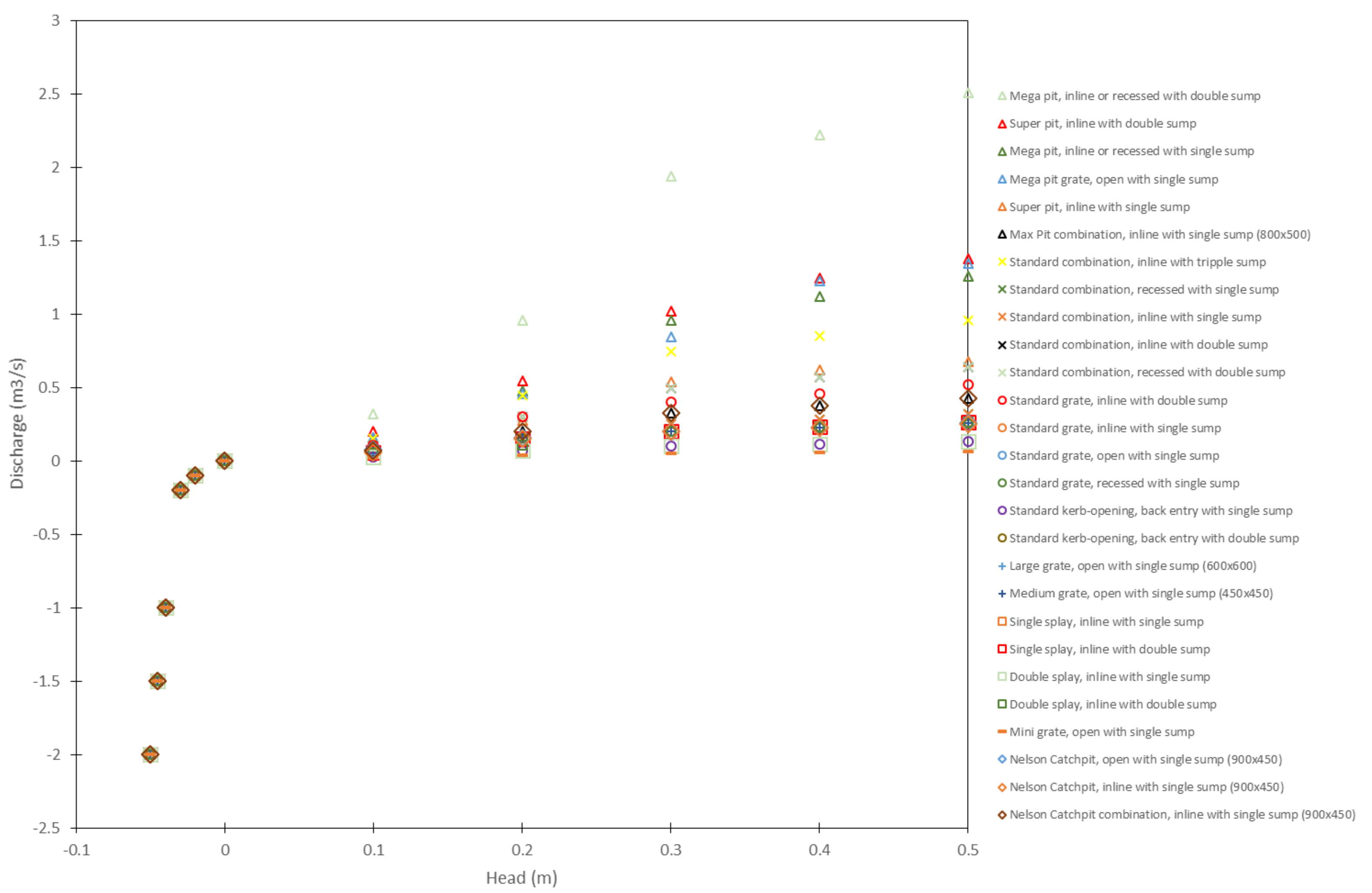


Figure I 1 Sump Head loss curves, showing the sump types used in the model. The curves were originally constructed by the WWL SW Panel .

Appendix J Petone and Hutt River Event Timing

J.1 Introduction and Objective

Stormwater flooding in Petone is affected by flood levels in the Hutt River because some runoff gravitates toward the eastern boundary of the catchment and is discharged through stormwater outlets into the Hutt River through the stop bank. Flap gates are installed on these pipes to prevent backflow from the river. When water levels in the Hutt River are higher than the hydraulic head in the stormwater pipes upstream of the flap gate, no discharge to the river can occur. Also, because the effectiveness of the flap gates can be compromised by debris, manually operated penstock valves are installed upstream of some flap gates.

There are also five pump stations on the eastern boundary of the Petone stormwater network which pump stormwater over or through the stop-bank. These are unlikely to be affected by Hutt River water levels as they are most likely high head low volume pumps with back flow prevention (although this has not been confirmed).

The purpose of this assessment is to compare the timing of hydrographs in the Hutt River with those in the Petone stormwater network during recorded flood events, to estimate when stormwater cannot gravitate to the Hutt River and the likelihood and duration of that condition. This is intended to augment the Petone Model Build Report provided to Wellington Water (dated 26/08/2019).

The comparison of event timing in Petone and the Hutt River was investigated using four methods:

1. Estimate the time difference between Petone rainfall and subsequent peak Hutt River flood levels adjacent to Petone (based on Hutt River flows at Taita Gorge plus a lag time to Petone).
2. Estimate the time difference between Petone rainfall and subsequent Petone urban flooding based on ICM model results.
3. Assess Hutt River levels relative to Petone stormwater outlet levels.
4. Assess the percent of time that Hutt River water levels have triggered closure of the Petone penstock valves based on a trigger level of 4.2m at the Birchville stage recorder.

J.2 Data

There are no flow gauges within the Petone urban area or wider catchment except for the recently reinstated Korokoro Stream at Mill Weir gauge which is not directly relevant to the assessment. In the absence of relevant flow or level data within the Petone catchment, results from Wellington Water's Petone ICM model for two recent events were used.

Rainfall data from Greater Wellington Regional Council's (GWRC) Shandon Golf Club gauge were used as it has a long, continuous record and is within the Petone urban catchment.

Hutt River flows and levels recorded at the Hutt at Taita Gorge gauge were used to estimate the timing of flood peaks at Petone as it is the closest gauge unaffected by tide.

The Hutt River level gauge at Estuary Bridge is close to Petone and was used to assess combined river and sea levels.

Flows and levels from the Hutt at Birchville gauge were used to assess the duration of time penstock valves are closed based on a Birchville trigger level that is normally used to inform their operation.

A summary of data provided by GWRC listed in Table 9-4 shows the locations of gauges except the Queens Wharf gauge which is on the southern side of Wellington Harbour. It also shows the location of where levels were exported from the Petone ICM model as discussed further in Appendix L .

Table J1: GWRC data

Gauge	Type	Gauge Number	Start	End	Rec Authority
Hutt at Birchville	Flow and Level	29818	7/9/1970	22/5/2019	GWRC
Hutt at Taita Gorge	Flow and Level	29809	17/3/1979	22/5/2019	GWRC
Hutt at Estuary Bridge	Level	29838	28/9/1976	22/5/2019	GWRC
Hutt at Shandon Golf Club	Rainfall	142813	10/11/1995	22/5/2019	GWRC
Queens Wharf	Sea Level	1438002	1/9/1994	22/5/2019	GWRC
Hutt at Taita Gorge	Flow and Level	29809	17/3/1979	22/5/2019	GWRC
Hutt at Estuary Bridge	Level	29838	28/9/1976	22/5/2019	GWRC
Hutt at Shandon Golf Club	Rainfall	142813	10/11/1995	22/5/2019	GWRC
Queens Wharf	Sea Level	1438002	1/9/1994	22/5/2019	GWRC

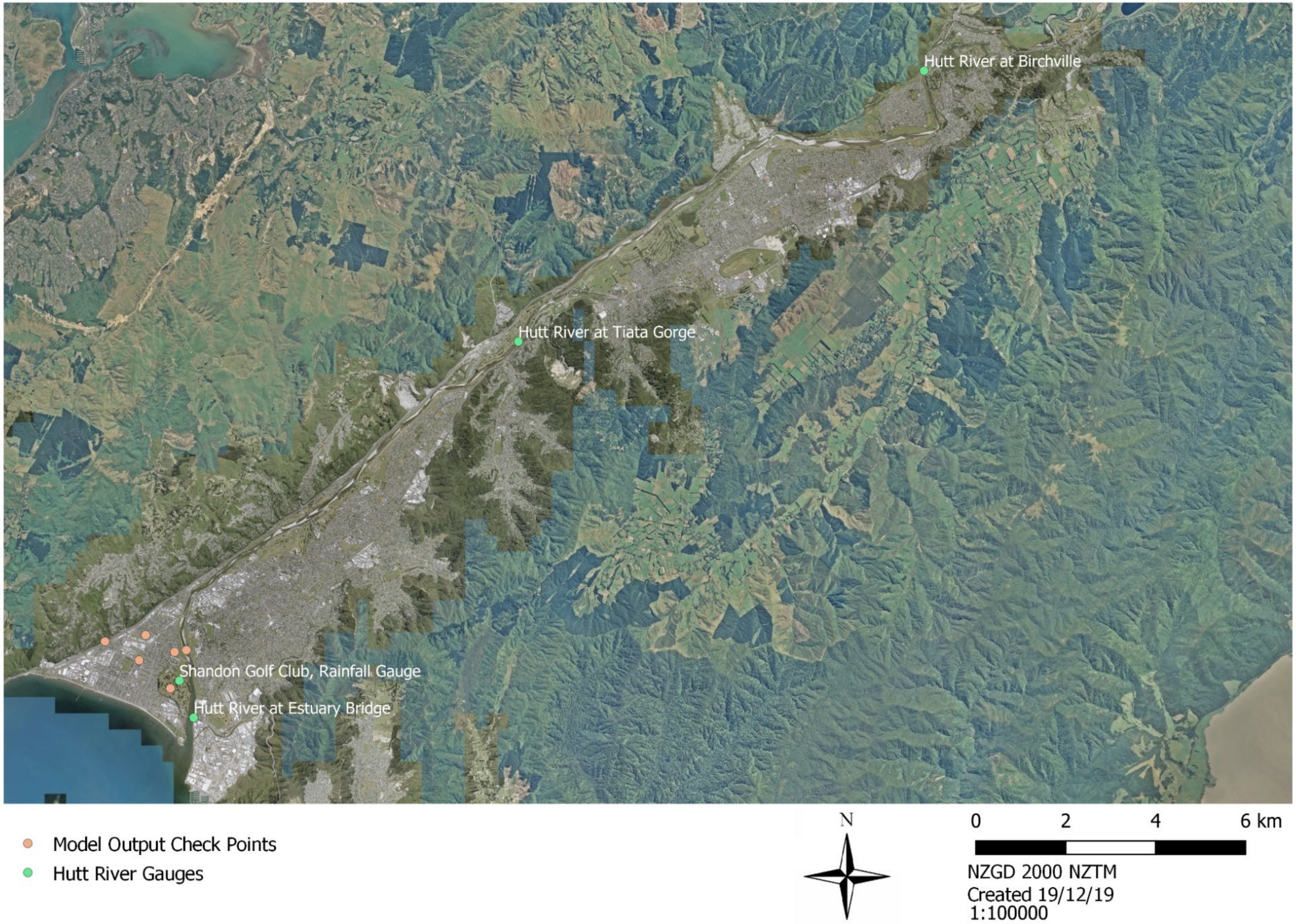


Figure J1 GWRC flow and rain gauge locations and ICM model export locations.

J.3 Method 1: Lag Between Petone Rainfall and Hutt River Water Level

To estimate the difference in time between Petone rainfall and subsequent peak Hutt River flood levels adjacent to Petone, Shandon Golf Club rainfall and Hutt River levels were compared. Figure J2 shows the Hutt at Taita Gorge stage hydrograph and Shandon Golf Club rainfall hyetograph for the period of coincident record (April 2000 to present).

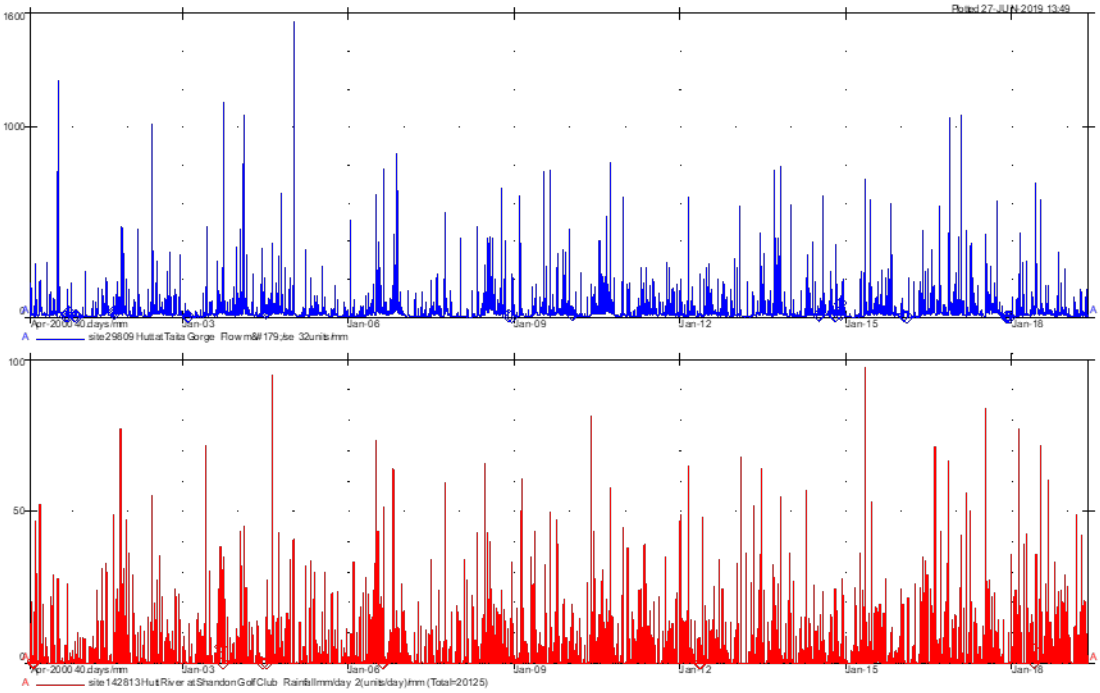


Figure J2 Hutt at Taita Gorge flow in m^3/s (top) and Shandon Golf Course rainfall in mm/day (bottom).

Ten of the largest storms recorded at the Hutt at Taita Gorge gauge were compared with Shandon recorded rainfall. Plots are included in Appendix K and a summary of the difference in time between the peak of flow at Petone (Ewen Bridge, estimated from Taita Gorge) and the peak of rainfall at the Shandon Golf Club rain gauge is listed in Table 9-5.

Thirty-five minutes' travel time was added to peak water levels to allow for the 9.5 km distance between Taita Gorge and Ewen Bridge. This estimate assumes a mean velocity of 4.5 m/s as reported for a flow at Taita Gorge of 1000 m^3/s in *Hutt River Mouth Fluvial Sediment Transport* (Opus, 2010).

Based on the assessment of the 10 floods, there is on average up to 7:47 hours/minutes between the rainfall peak within the Petone catchment and peak water level in the Hutt River at Ewen Bridge when stormwater could potentially gravitate to the Hutt River through stormwater outlets to some degree.

However, there is an average lag of about 8.5 hours between the start of the rising limb of the recorded flood and the peak for the events listed in Table 9-5. So, although there is a lag between peak rainfall and peak flow, Hutt River levels begin to rise shortly after the peak of rainfall and so low-lying culvert outlets are likely to be submerged early after the peak of rainfall in Petone.

Table J2: Lag between Petone rainfall and peak flood levels

Date	Lag between peak rainfall and peak flood Level at Taita Gorge(hh:mm)	Travel time between Taita Gorge and Ewen Bridge (hh:mm)	Total Lag (hh:mm)	Time to peak (hh:mm)
Oct-00	6:45	00:35	7:20	4.5
Jun-02	8:45	00:35	9:20	12
Oct-03	7:00	00:35	7:35	11
Feb-04	10:00	00:35	10:35	12
Jan-05	7:00	00:35	7:35	10
Aug-06	8:00	00:35	8:35	10
Jul-09	7:00	00:35	7:35	7
Sep-13	5:30	00:35	6:05	4
Nov-16	6:30	00:35	7:05	8
Feb-17	5:30	00:35	6:05	7
Average	7:12		7:47	8:36

J.4 Method 2: Lag Between Petone Rainfall and Petone Urban Flooding

To assess the lag between peak Petone rainfall and peak urban flood levels, results from the Petone model were compared with recorded rainfall for floods in 2015 and 2016. It showed the time between rainfall and peak modelled flooding varies across the catchment. Appendix L includes ICM plots for the two events at four locations. The ICM locations are shown in Figure J1 and a summary of results is listed in Table 9-6 and Table 9-7.

Table J3: ICM results - 2015 event

Location	Peak rainfall (A) hh:mm	Peak water level (B) hh:mm	Lag (B-A) hh:mm
Hutt River Bank	11:20	11:40	0:20
Petone Basin	11:20	12:40	1:20
Petone Mid	11:20	12:40	1:20
SH2	11:20	12:05	0:45
Average			0:56

Table J4: ICM results - 2016 event

Location	Peak rainfall (A) hh:mm	Peak water level (B) hh:mm	Lag (B-A) hh:mm
Hutt River Bank	10:05	10:30	0:25
Petone Basin	10:05	11:00	0:55
Petone Mid	10:05	13:00	2:55
SH2	10:05	11:00	0:55
Average			1:15

Figure J3 ICM model locations.

The average lag between peak rainfall and the peak of flooding for the four locations and two flood events is 1 hour, 5-minutes. The length of lag seems to be related to flood storage. For example, at the location labelled SH2, peak flooding occurs quickly after rainfall because runoff from the Western Hills is relatively fast and there is unlikely to be much surface ponding. Within the Petone Basin and Petone mid locations, where more extensive ponding occurs the average lag time is longer.

J.5 Method 3: Assessment of Hutt River Water Levels Relative to SW Outlets

Data from Hutt City's stormwater asset database was extracted for all outlets that had either a flap gate, or penstock valve attribute and discharged into the Hutt River from the Petone modelled catchment. Ground levels were also extracted at the location of each of the assets as no pipe invert data were available.

Figure J4 shows asset locations and Appendix M provides a list of the relevant Hutt River outlet data.

Figure J4 Petone stormwater outlet locations and IDs.

Design Floods

Following discussions with WWL, water levels in the Hutt River at Estuary Bridge were obtained and an assessment was undertaken to determine if the frequency and duration of levels at the Estuary Bridge recorder exceeded the inferred invert levels of Petone stormwater outlets. Of the 49 assets identified in Figure J4, only 10 outlets had levels below 1.5m. As can be seen in Figure J5, levels are dominated by the tide and rarely get above 1.5m at the Estuary Bridge recorder.

However, inspection of data from GWRC's MIKE 11 model showed that due to the gradient of the Hutt River channel, Estuary Bridge water level data is only relevant downstream of Estuary Bridge. At that point, the assessment was abandoned in favour of analysis of GWRC's MIKE 11 data.

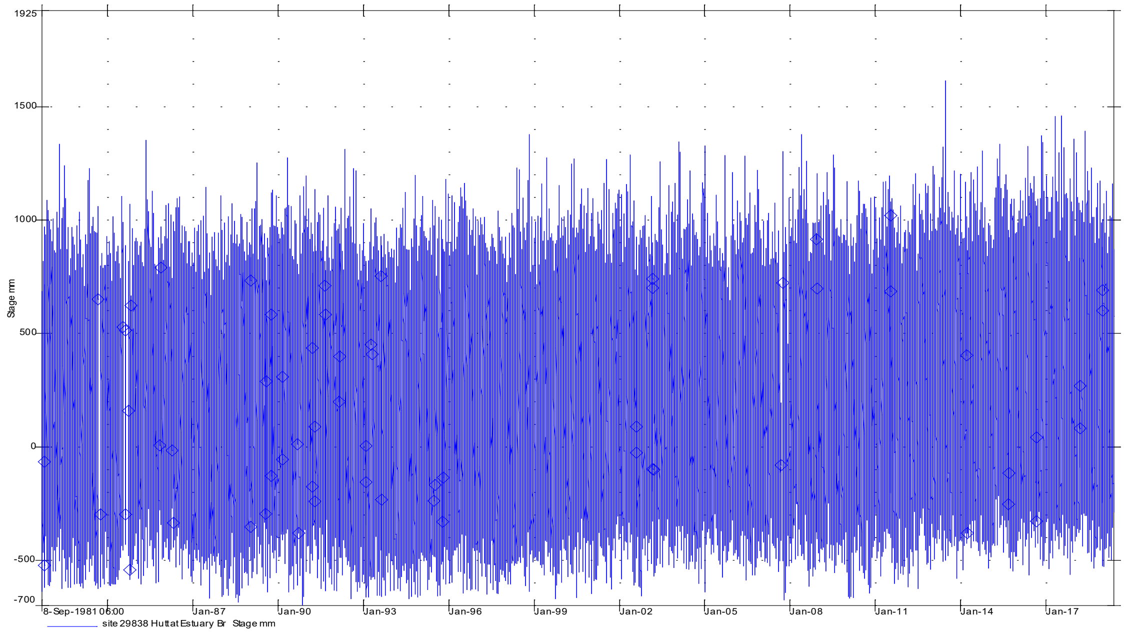


Figure J5 Stage hydrograph of the Hutt River at Estuary Bridge.

GWRC's MIKE 11 model results, for a range of ARIs, were plotted with ground level data at each outlet. The resulting long section in Figure J6 shows that maximum levels, during a 2-year ARI flood are above all but eight of the stormwater outlets. The number of inundated outlets is all but one during a 100-year ARI flood in the Hutt River.

The assessment assumes that the ground levels listed in the asset database are equivalent to outlet pipe invert levels and that the vertical datum of the asset data (Wellington Vertical Datum 1953) is equivalent to the Estuary Bridge water record (LHCC Vertical Datum). It is understood that both are based on mean sea level.

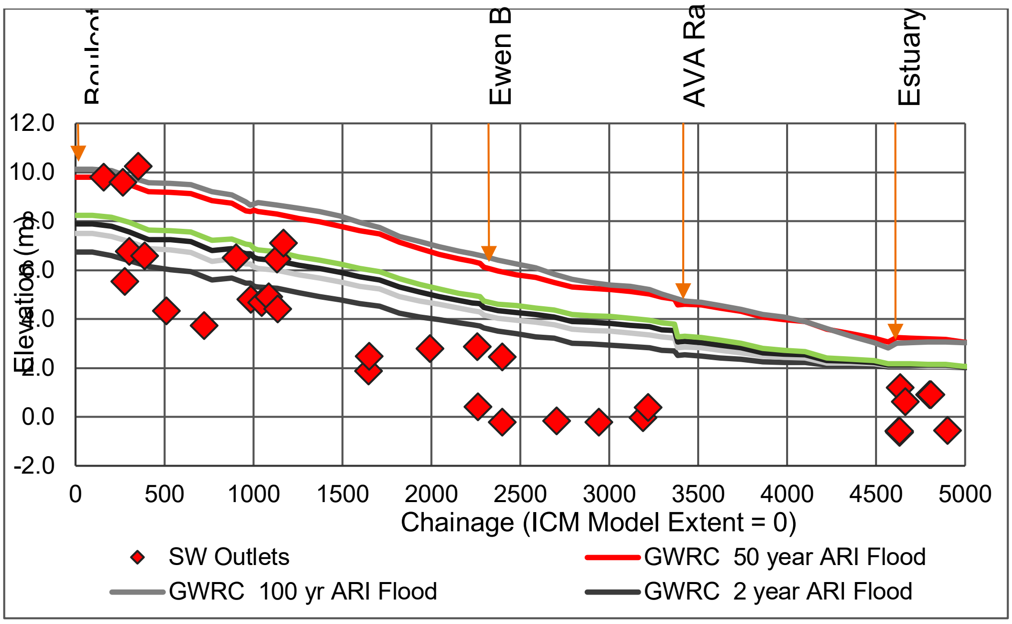


Figure J6 Hutt Riverbed long section from upstream of Melling Bridge to the Hutt River mouth.

Observed Floods

Figure J7 and Figure J8. show stage hydrographs at four Hutt River locations during flood events that occurred on 15/5/2015 and 15/11/2016. These locations are also marked in Figure J6.. The recorded stage hydrograph at Taita Gorge is also superimposed on Figure J7and.Figure J8 The hydrographs are simulated results from the ICM model.

- *Hutt at Boulcott:* The Hutt at Boulcott location is at the upper boundary of the model. From Figure J6., it can be seen that the three culverts that discharge through the stopbank into the Hutt River have inverts between 9.6m and 10.2m and the peak of the 2015 and 2016 floods were 7.6m and 7.5m respectively (from Figure J7 and Figure J8). So, stormwater could have gravitated through these outlets even during the peak of both storms.
- *Hutt at Ewen Bridge:* The three culverts adjacent to Ewen Bridge are between 2.5m and 2.9m, significantly lower than the 4.2m and 4.7m simulated for the 2015 and 2016 events and so are submerged for all but the early part of the flood. Another 6 culverts located between Ewen and Ava bridges all have inverts between 0.4 and – 0.2m and so are also submerged throughout both floods.
- *Hutt at Ava Rail Bridge:* All culverts located on the right bank of the Hutt River near Ava Rail Bridge are low lying with a maximum invert of 0.4m. These are fully submerged for the duration of the 2015 and 2016 simulated hydrographs.
- *Hutt at Estuary Bridge:* Of the five culverts near the Estuary Bridge, three are above Hutt River water levels during the lower part of the tidal cycle.

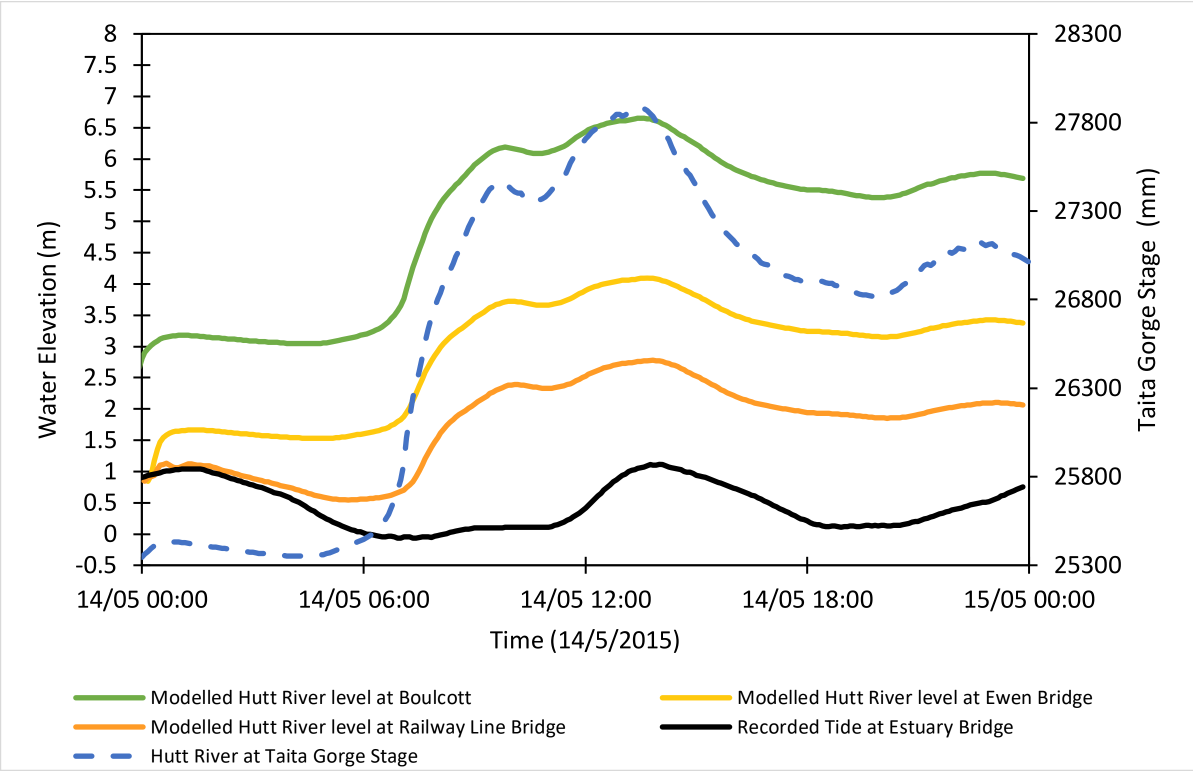


Figure J7 2015 event stage hydrographs at 5 Hutt River Locations.

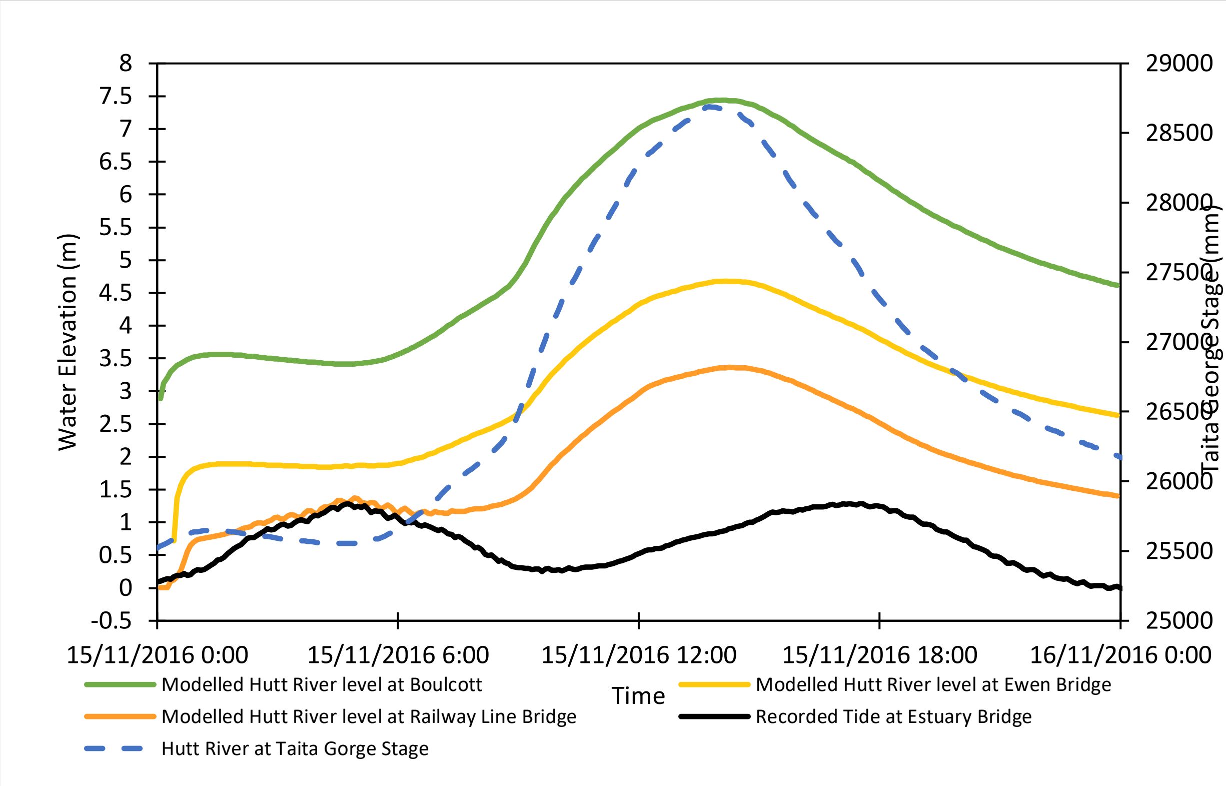


Figure J8 2016 event stage hydrographs at 5 Hutt River Locations

In summary, based on the long section of the Hutt River and MIKE 11 design flood results shown in Figure J6, all but eight of the culverts that discharge from the Petone modelled catchment into the Hutt River are submerged during a 2-year ARI modelled flood. This number reduces to one culvert not submerged during a 100-year ARI event.

Modelled hydrographs from two observed floods show that at Boulcott's near the upstream boundary of the model, half the stormwater outlets are submerged for part of both floods. At Ewen and Ava bridges, all culverts are submerged and at Estuary Bridge, higher level culvert outlets are not submerged only during the low part of the tidal cycle.

J.6 Method 4: Timing of Operation of Penstock Valves

To prevent backflow from the Hutt River into the Petone stormwater system penstock valves are installed at various locations upstream of stormwater outlets.

Information in *Hutt River Lower - Hutt Penstock Valve Flood Control Procedures*, (Wellington Water, 17/02/2018), reports penstocks are manually closed when river stage levels at Birchville reach 4.2m. Following discussion with WWL an assessment of how often the trigger level is reached was undertaken.

Figure J9**.** is a plot of Birchville water level since 1970 and shows there have been several significant changes in riverbed levels. The stage level trigger of 4.2m at Birchville is related in some way to subsequent flows and water levels in the Hutt River at Petone. Therefore, changes in the stage-flow rating will have a significant impact on that relationship.

Assuming the current trigger level of 4.2m is based on the current Birchville rating, the percentage of time a stage value of 4.2m is equalled or exceed was estimated for the period since the last major rating shift in 1998. As shown in Figure J10, water levels at the Birchville gauge exceeded 4.2m for 0.12% of the time since 1998, That is a total of 222 hours over 21.3 years or on average 10.4 hours/ year.

Figure J11**.** shows a level of 4.2 has been exceeded 48 times between 1998 and 2019 which is on average 2.3 times per year.

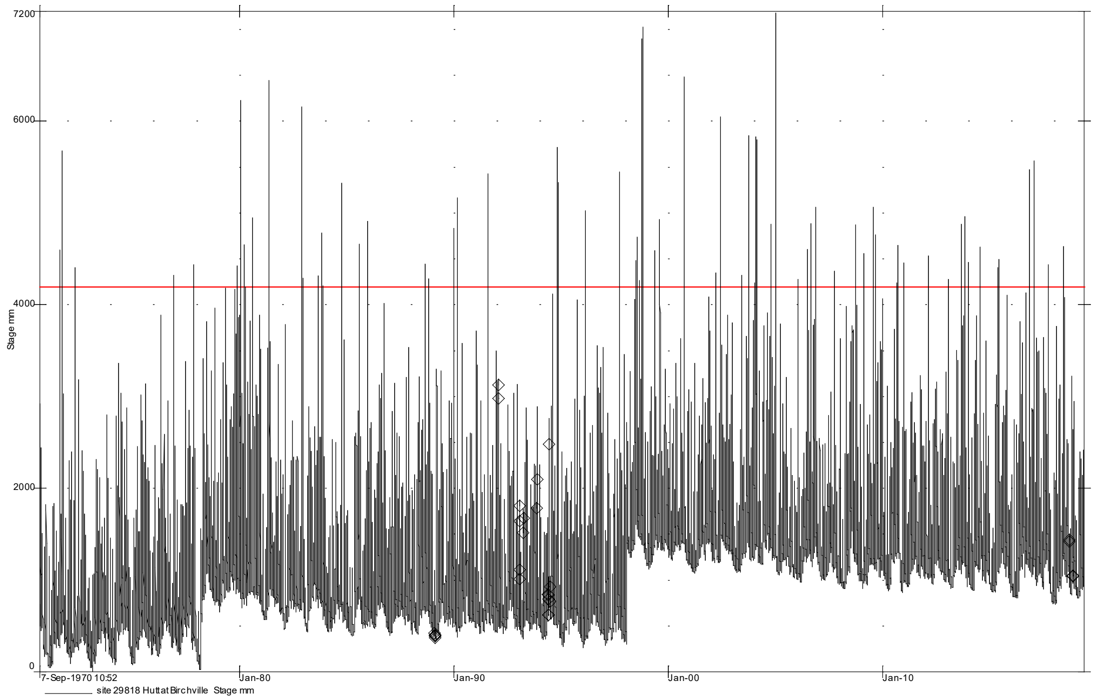


Figure J9 Stage record of the Hutt River at Birchville. the 4.2m trigger level is in red.

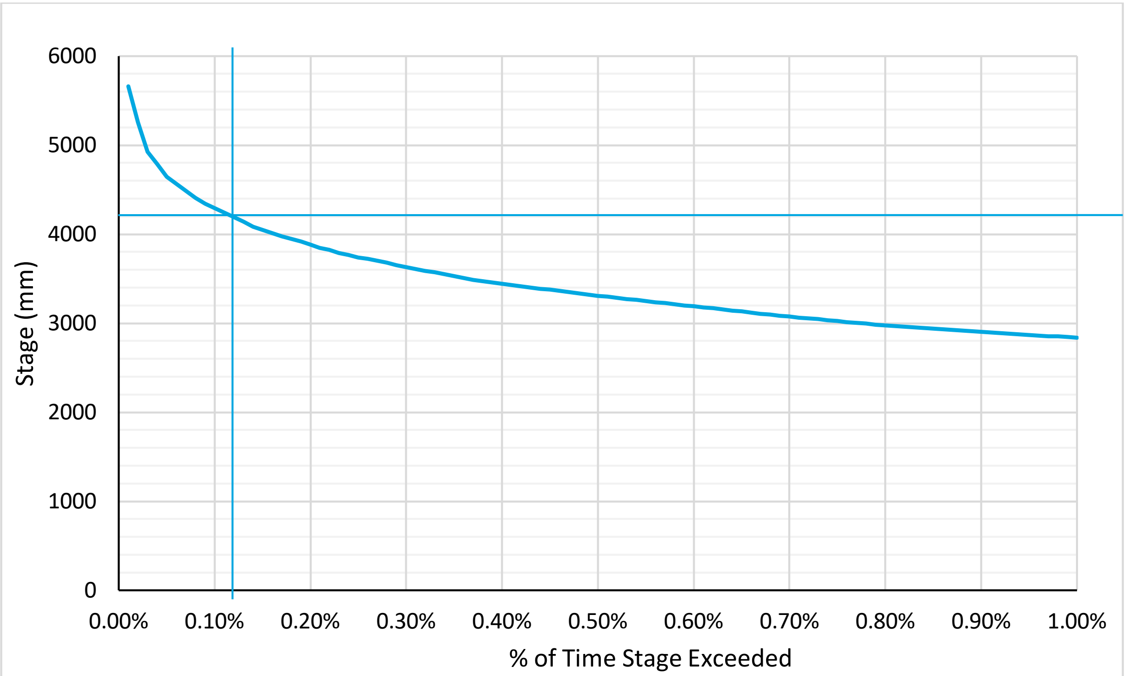


Figure J10 Percent of time 4.2m is equalled or exceeded (from 1998) in the Hutt River at Birchville.

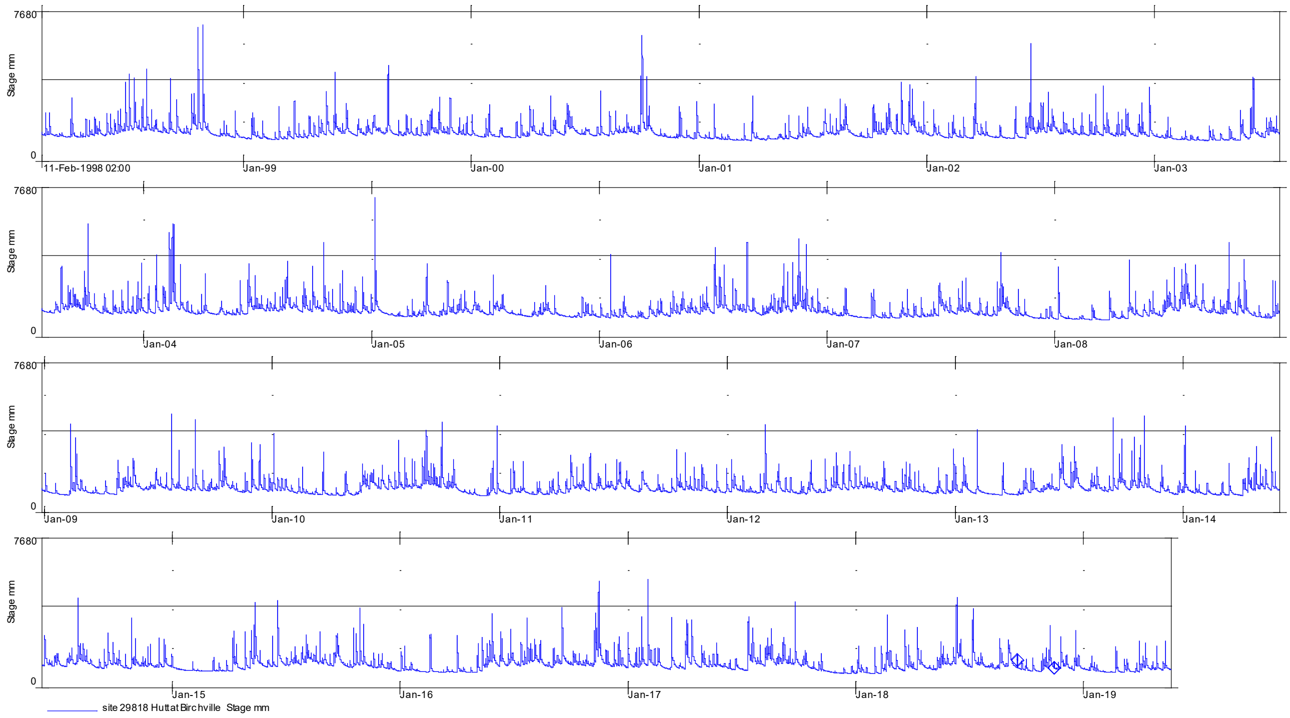


Figure J11 Events greater than 4200mm from 1998 in the Hutt River at Birchville

J.7 Conclusions

Method 1: Lag Between Petone Rainfall and Hutt River Water Level

Based on the 10 largest recorded floods, there is on average a lag of over 7 hours between the peak of rainfall in Petone and peak flood levels in the Hutt River at Ewen Bridge. However, the rising limb of the flood hydrograph typically begins on average about 8.5 hours before the peak. This means that outfalls may be submerged and so prevented from draining by gravity early in the event as levels begin to rise in the Hutt River.

Method 2: Lag Between Petone Rainfall and Petone Urban Flooding

An assessment of floods in 2015 and 2016 showed on average a 1-hour, 5-minute lag between peak rainfall and peak flooding based on results of ICM modelling at four locations. The duration of lag at each location is related to the extent of storage (ponding) and varies between 20 minutes and 2 hours 55 minutes. In the low-lying areas of the Petone basin it takes longer to reach peak water level. Therefore, for areas with less storage and a faster time to peak, there is a short period when water could theoretically gravitate to the Hutt River.

Method 3: Assessment of Hutt River Levels Relative to SW Outlets

Design Floods: Water level recorded at Estuary Bridge is only relevant to stormwater outlets downstream of the Estuary Bridge. This is because levels upstream of the bridge are affected by rising riverbed levels.

Between chainage 1200 (just downstream of Melling Bridge) and the mouth of the river all outlets are inundated during a 2-year ARI flood. Upstream of chainage 1200 most 7 out of 15 culverts are submerged.

During a 100-year ARI flood all but one outlet is submerged between Boulcott's and the mouth of the river.

Observed Floods: Modelled hydrographs from two observed floods show that at Boulcott's near the upstream boundary of the model, half the stormwater outlets are submerged for part of both floods. At Ewen and Ava bridges, all culverts are submerged during these two floods. At Estuary Bridge, only the higher-level outlets are not submerged during the lower part of the tidal cycle.

Method 4: Timing of Operation of Penstock Valves

Since 1998 water levels at Birchville have exceeded a trigger level of 4.2m at Hutt at Birchville recorder on average 2.3 times per year for a total of 10.4 hours (on average) a year. A water level of 4.2m at Birchville is the trigger level when penstock valves that outlet to the Hutt River at Petone are closed.

J.8 Summary and Recommendations

Hutt River water levels have a significant effect on the operation of the Petone stormwater network because as levels in the river rise above stormwater outlet levels, drainage by gravity to the Hutt River is prevented.

Several sources of data were assessed to test if there is a lag between flooding in Petone and flooding in the Hutt River. Results show that by the time flooding occurs in Petone, levels in the Hutt River will have begun to rise and will have submerged several stormwater outlets.

It is therefore recommended that future stormwater modelling of Petone and Eastern Lower Hutt represent Hutt River boundary conditions with dynamic hydrographs extracted from GWRC's MIKE 11 model. This will ensure Hutt River flood peaks occur at an appropriate time during the modelled event. Sensitivity testing should be undertaken to determine the impact of varying the timing of the peak by approximately two hours.

Appendix K Event Timing - Rainfall and Hutt River Flows

A summary of the comparison between rainfall at Shandon Golf Course and Hutt River at Taita Gorge is listed in Table 9-5.

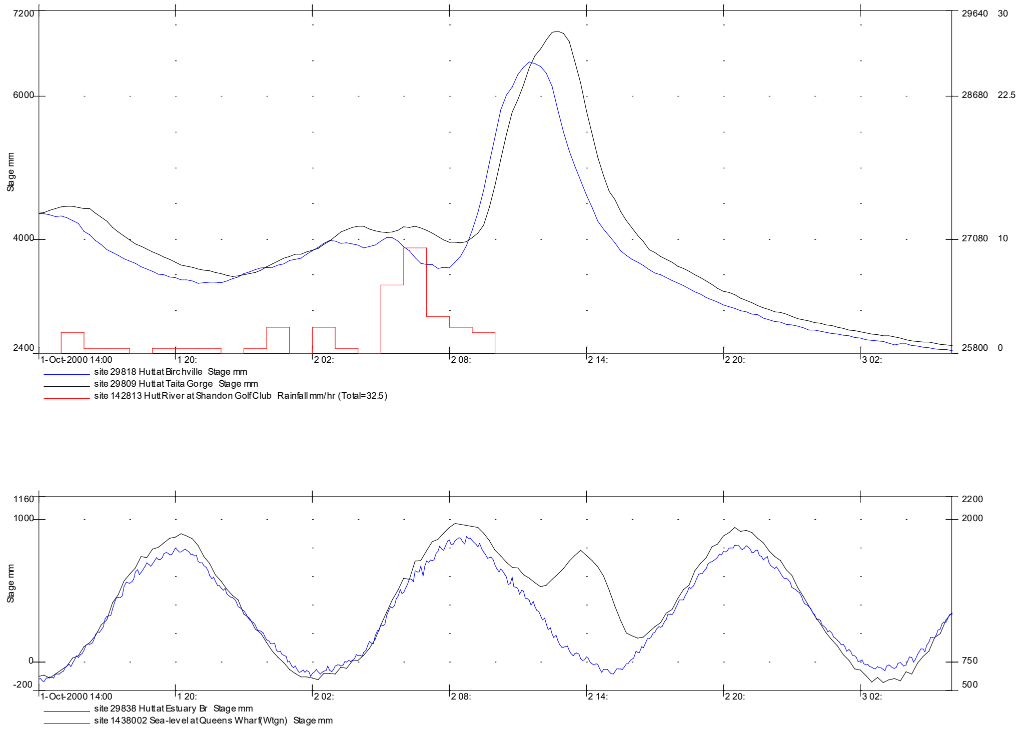


Figure K1 2000 event.

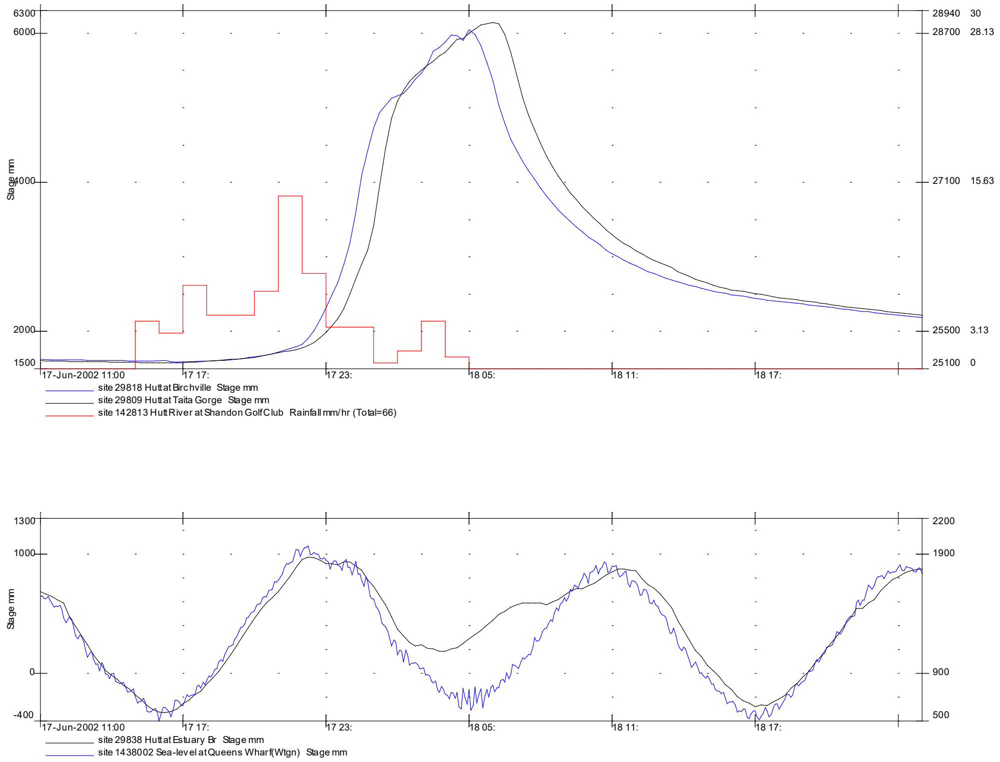
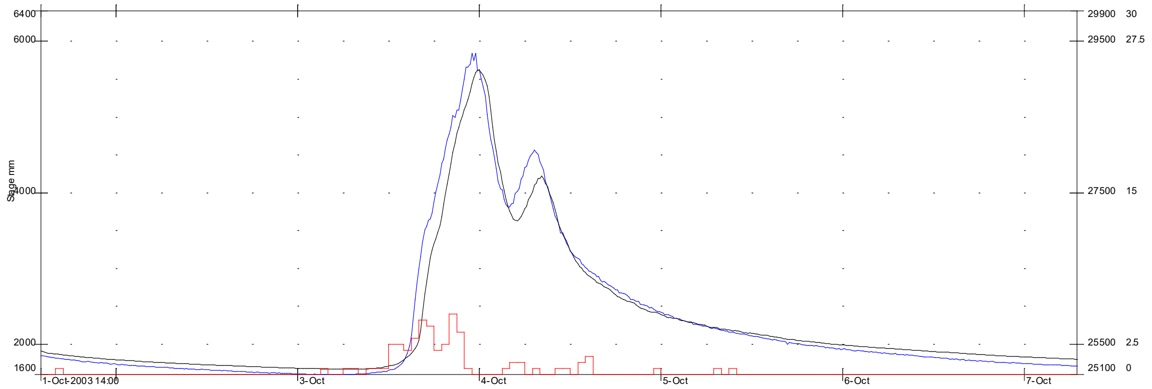


Figure K2 2002 event.



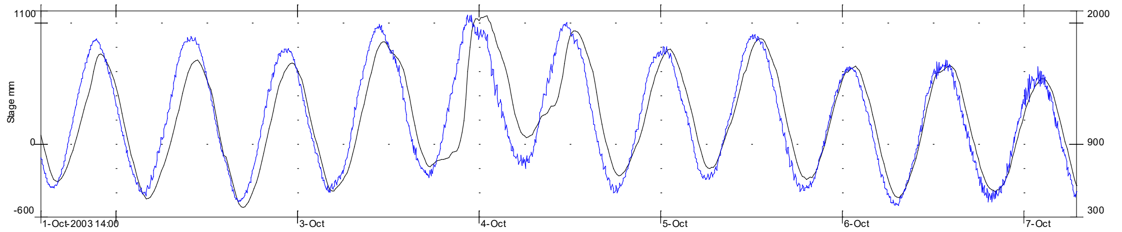
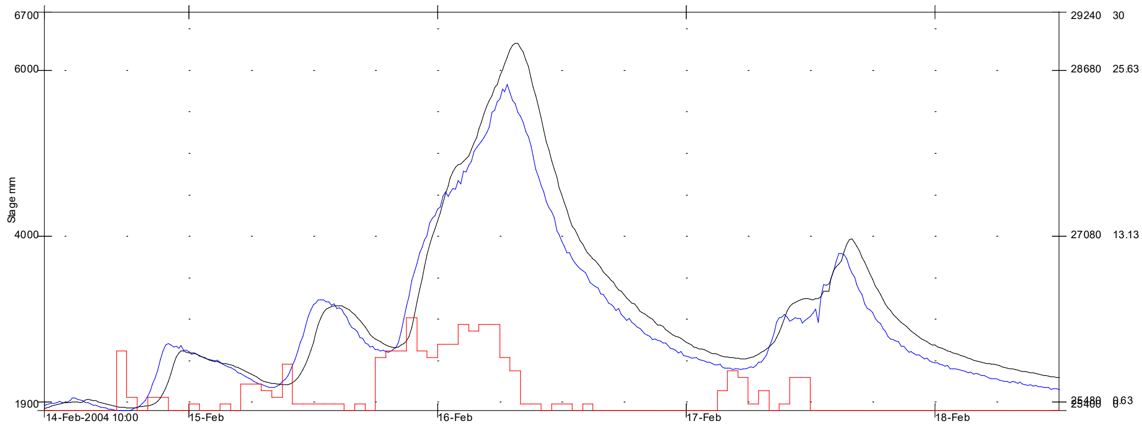


Figure K3 2003 event.



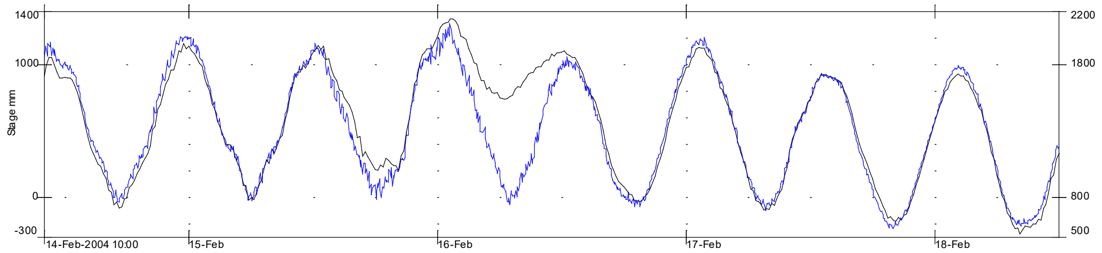


Figure K4 2004 event.

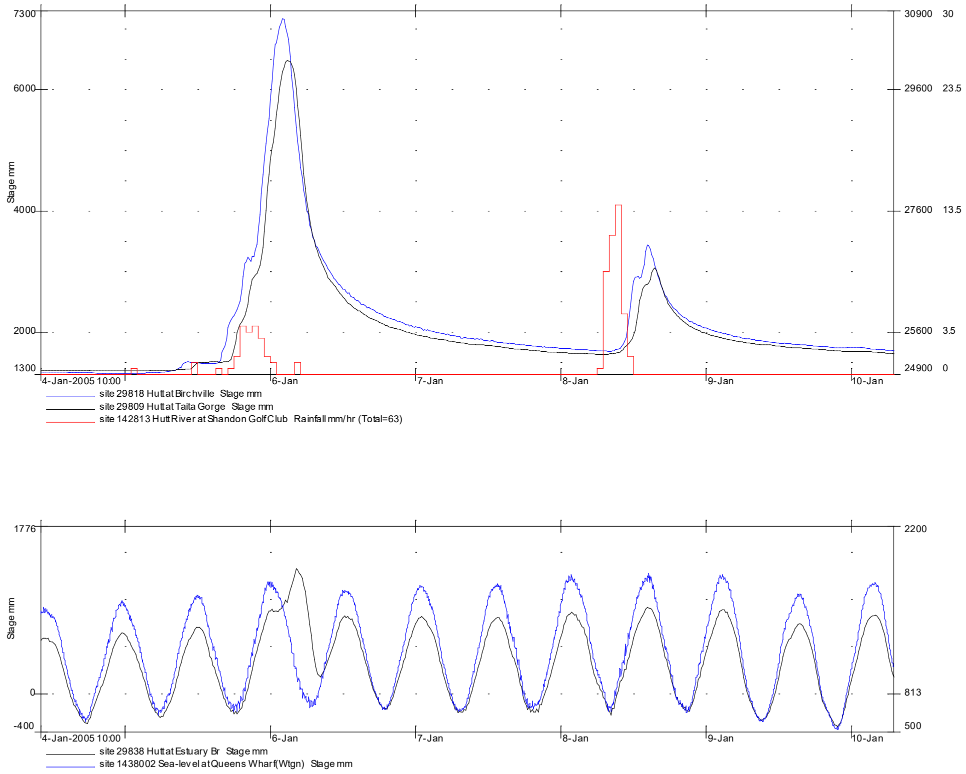


Figure K5 2005 event.

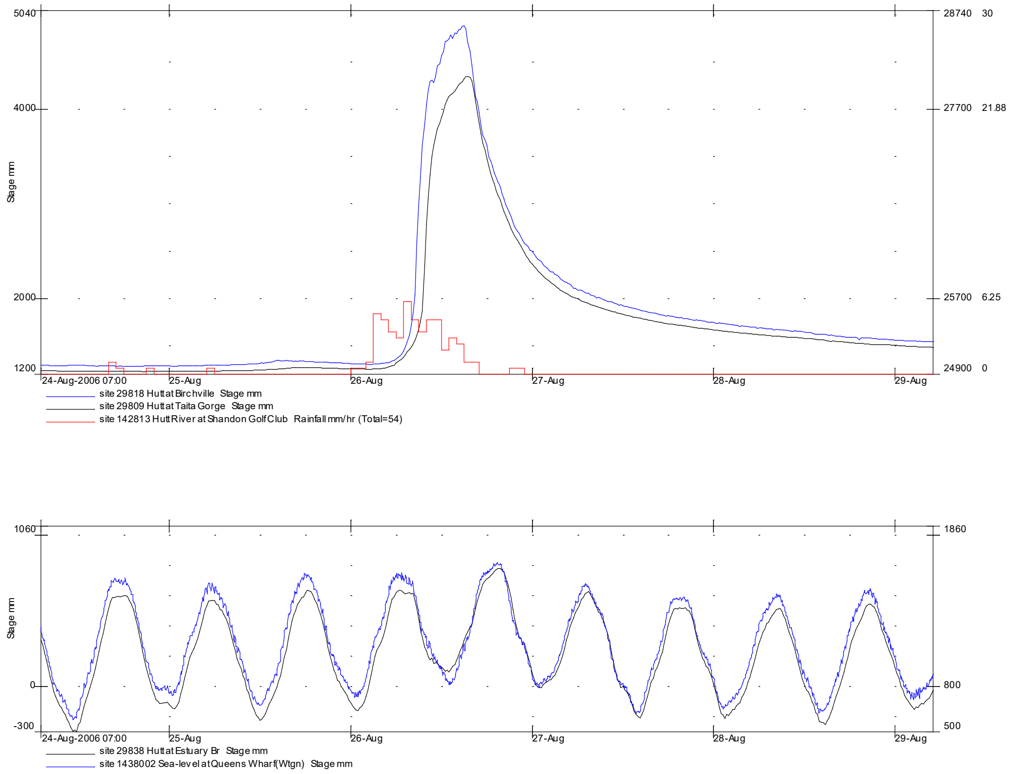


Figure K6 2006 event.

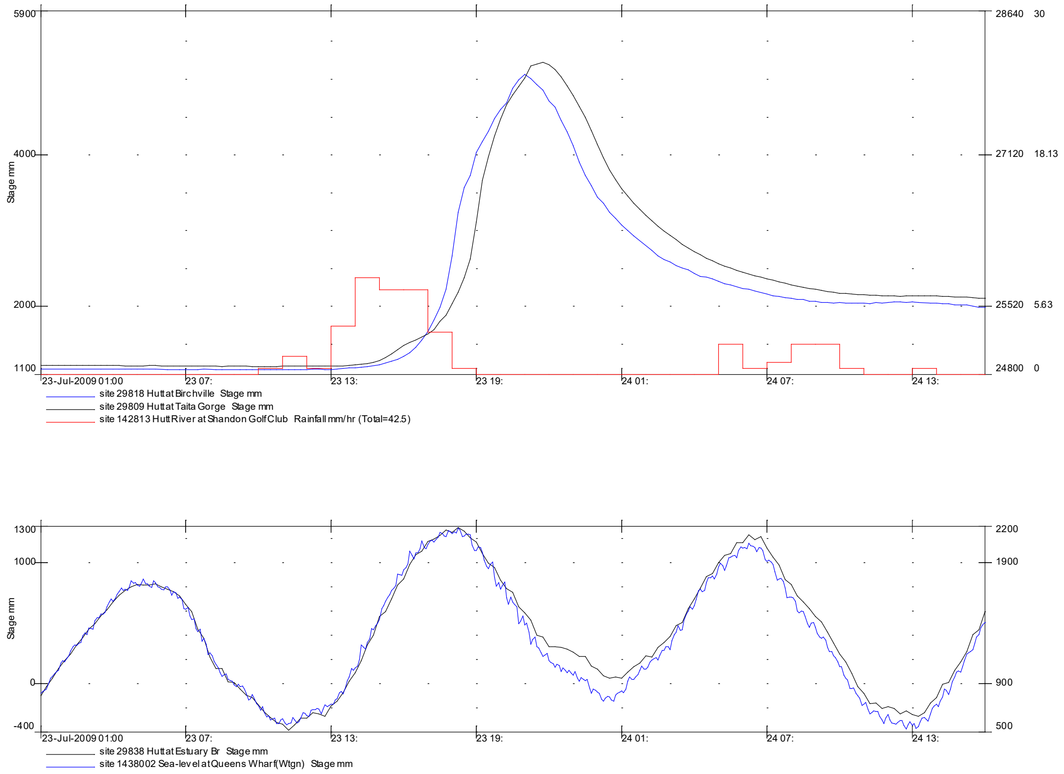


Figure K7 2009 event.

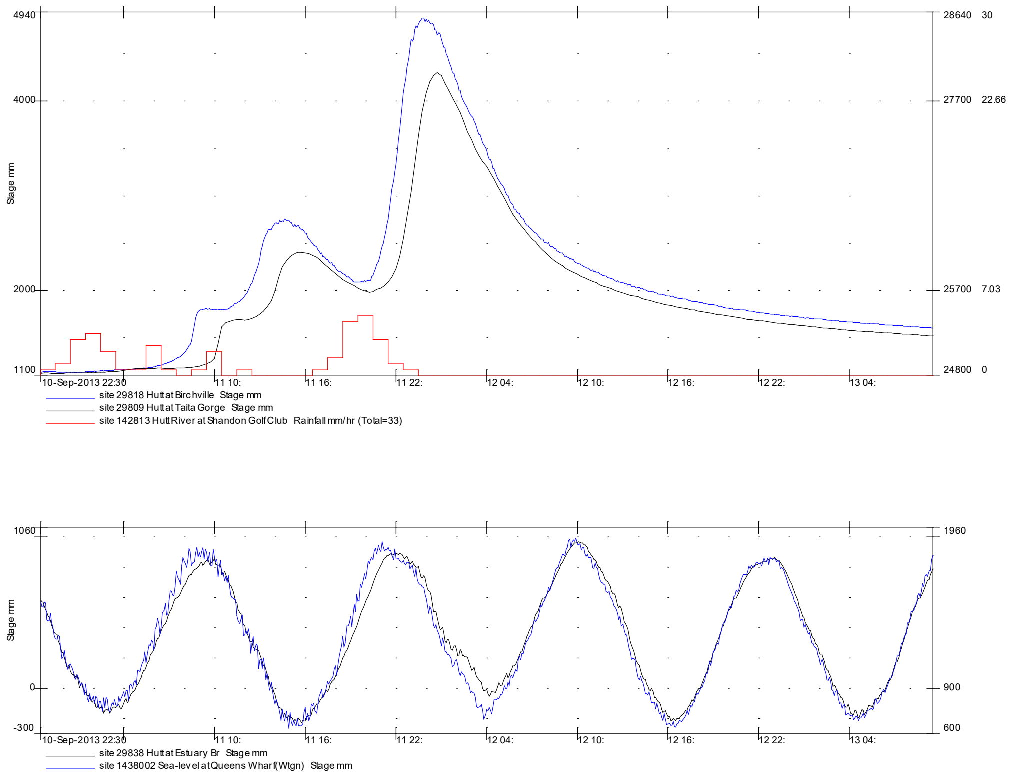


Figure K8 2013 event.

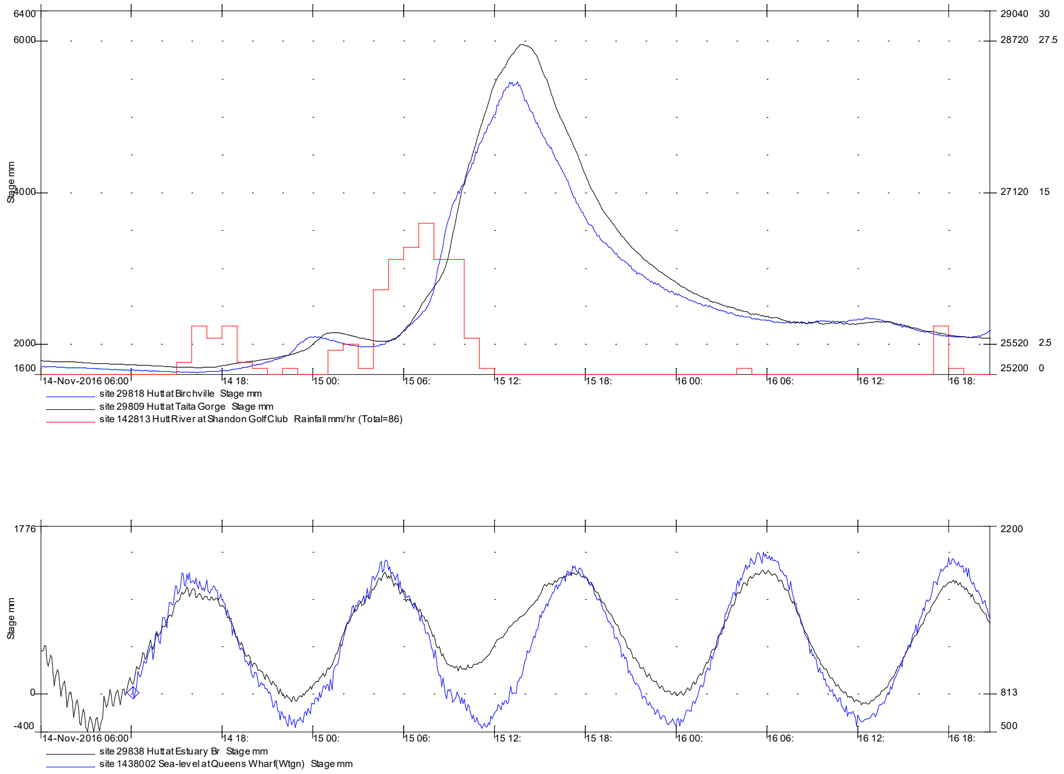


Figure K9 2016 event

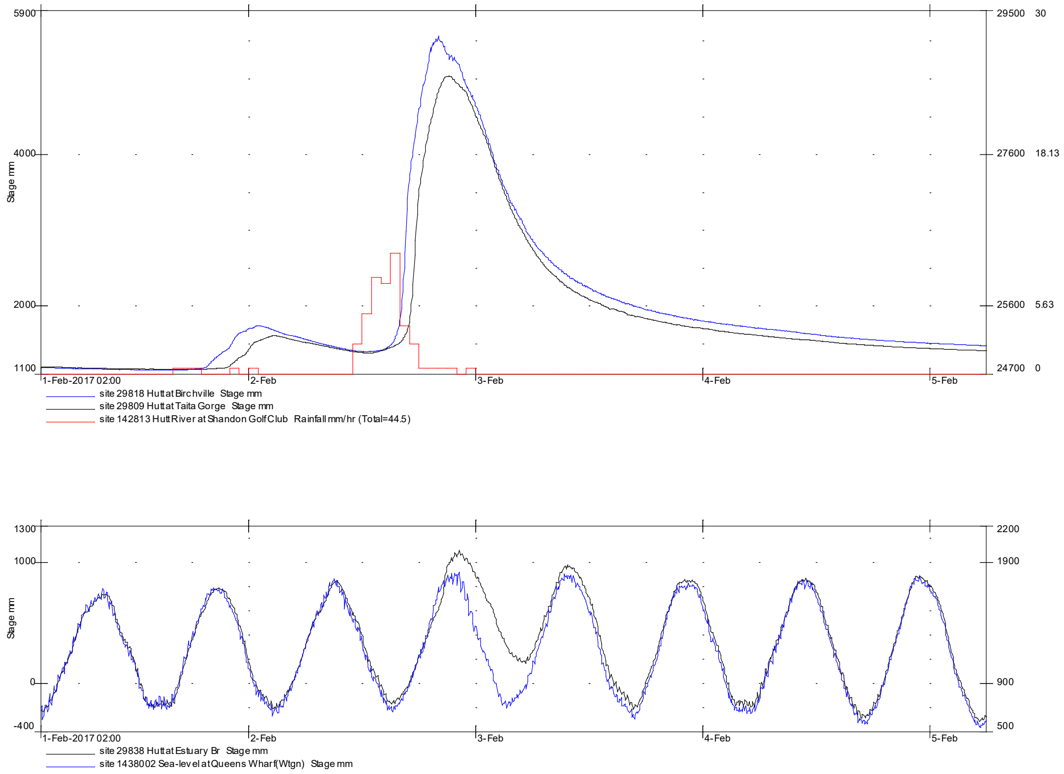


Figure K10 2017 event.

Appendix L Event Timing - Petone ICM Output

L.1 2015 Event

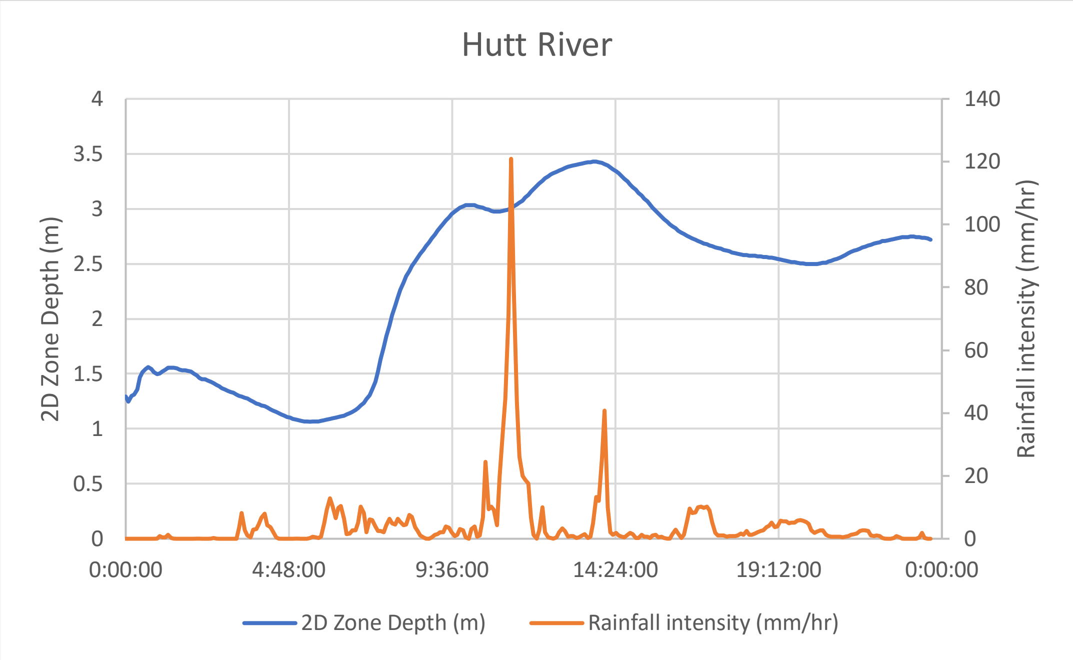


Figure L1 Water depth over time in the 2015 rainfall event at the "Hutt River" site,

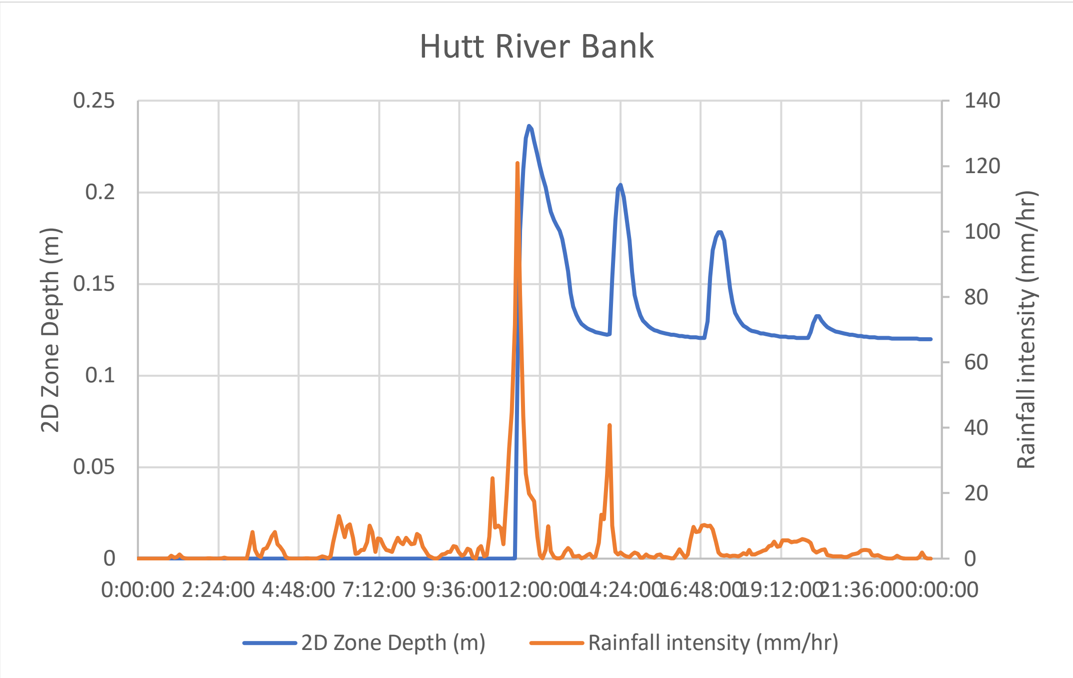


Figure L2 Water depth over time in the 2015 rainfall event at the "Hutt River Bank" site.

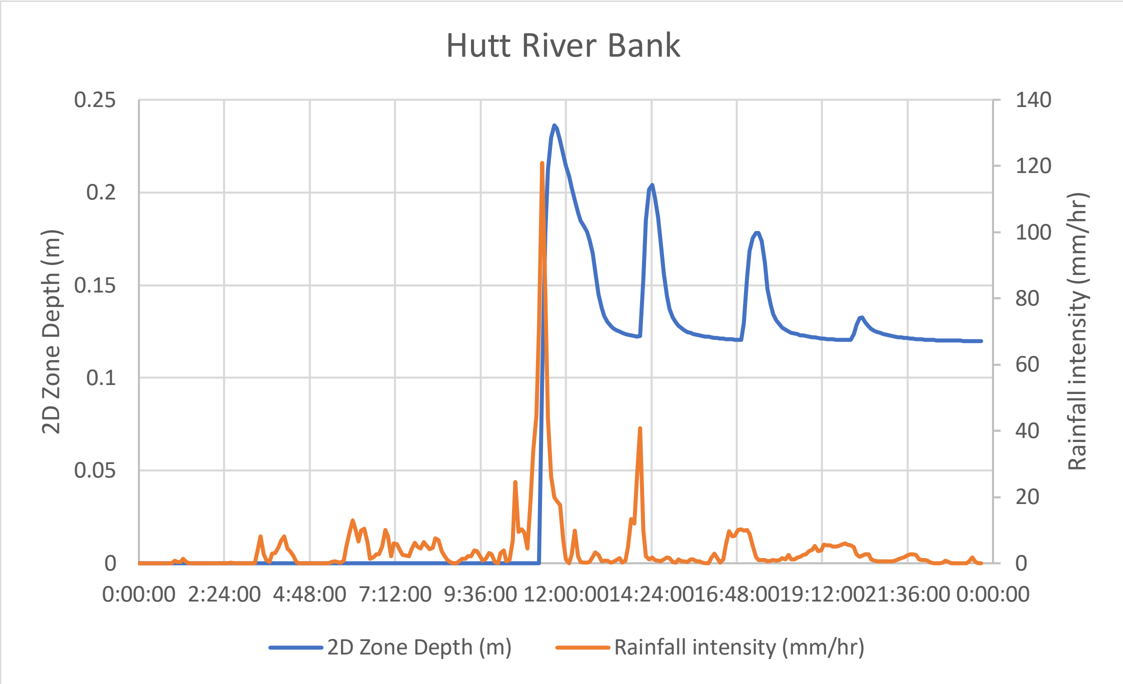


Figure L 3 Water depth over time in the 2015 rainfall event at the "Petone Basin" site.

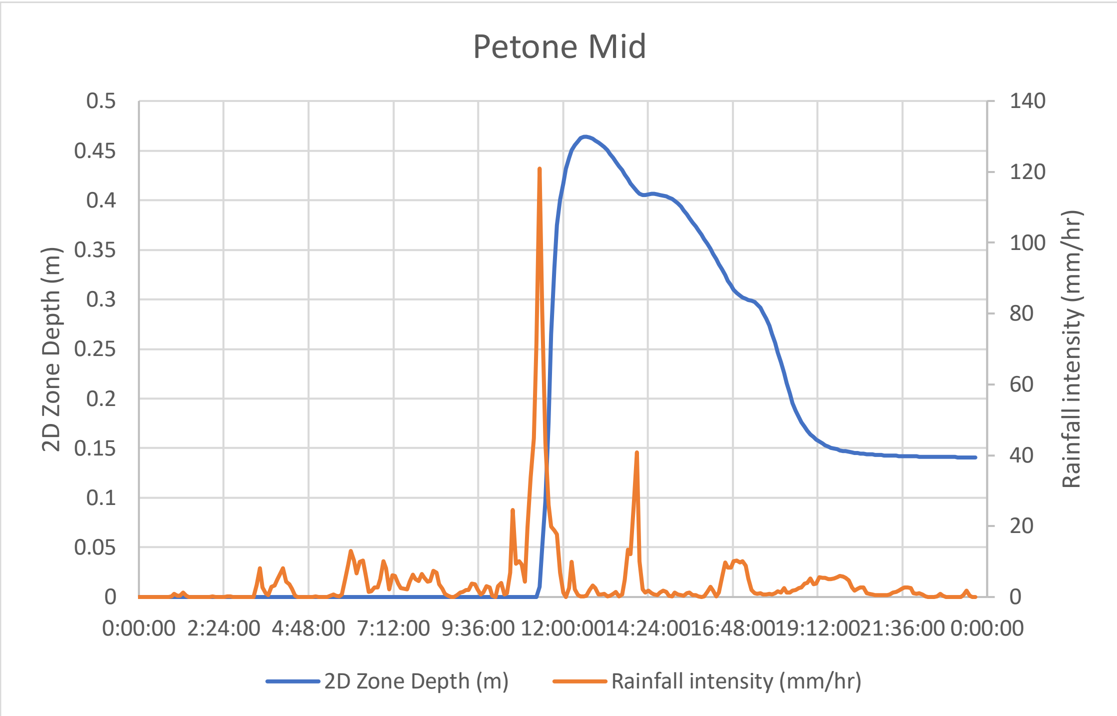


Figure L 4 Water depth over time in the 2015 rainfall event at the "Petone Mid" site.

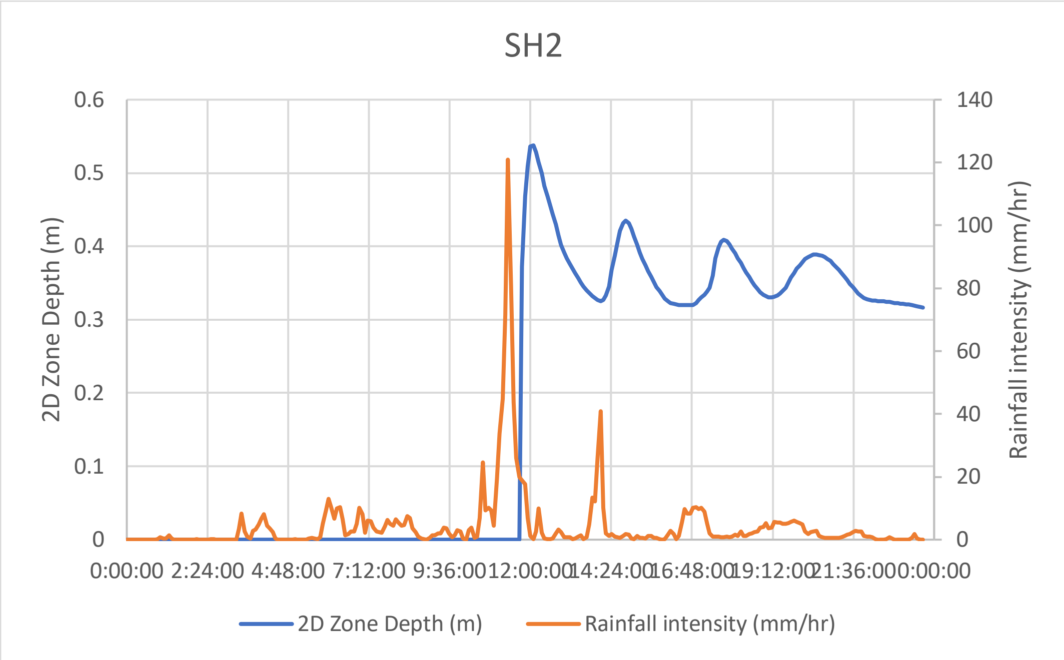


Figure L5 Water depth over time in the 2015 rainfall event at the "SH2" site.

L.2 November 2016 Event

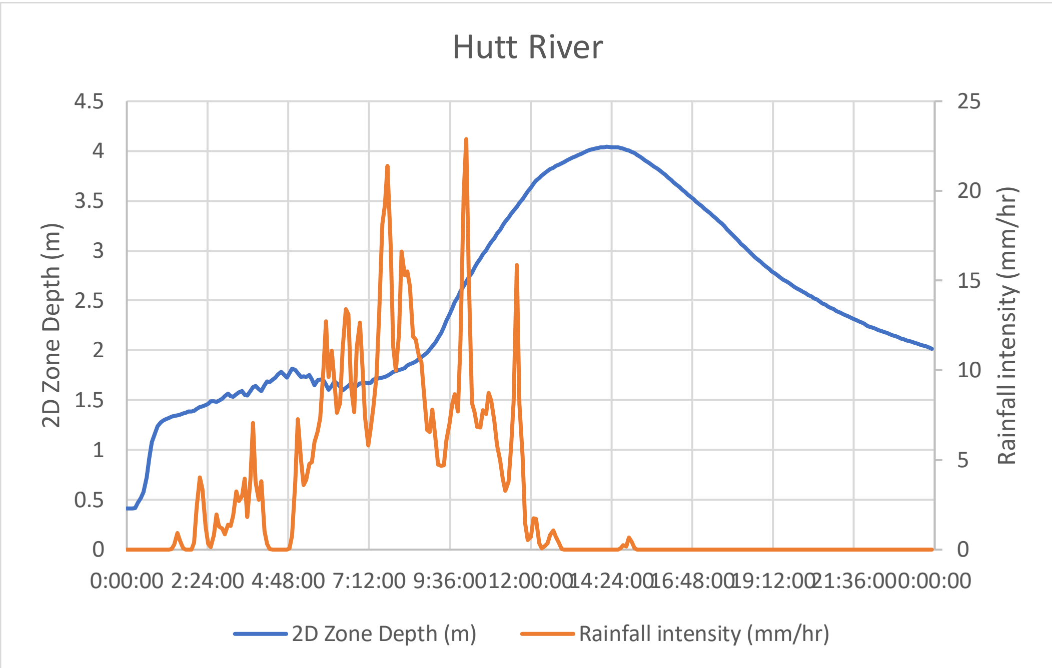


Figure L6 Water depth over time in the 2016 rainfall event at the "Hutt River" site.

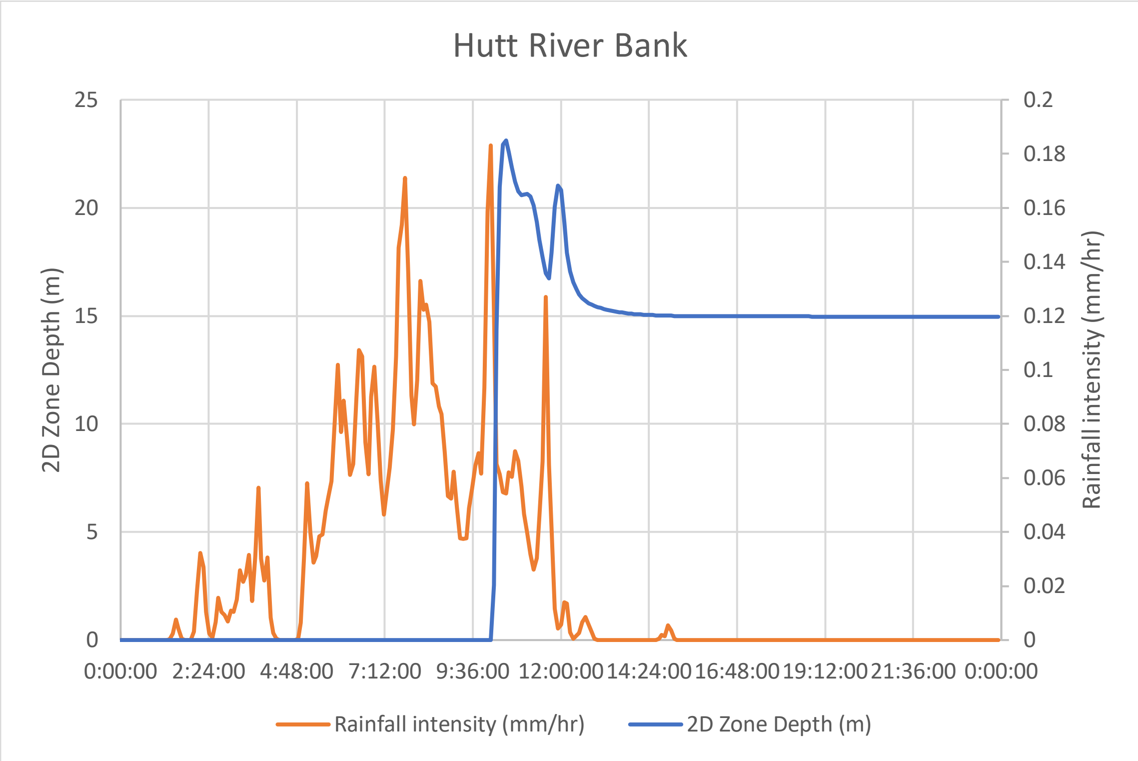


Figure L 7 Water depth over time in the 2016 rainfall event at the "Hutt River Bank" site.

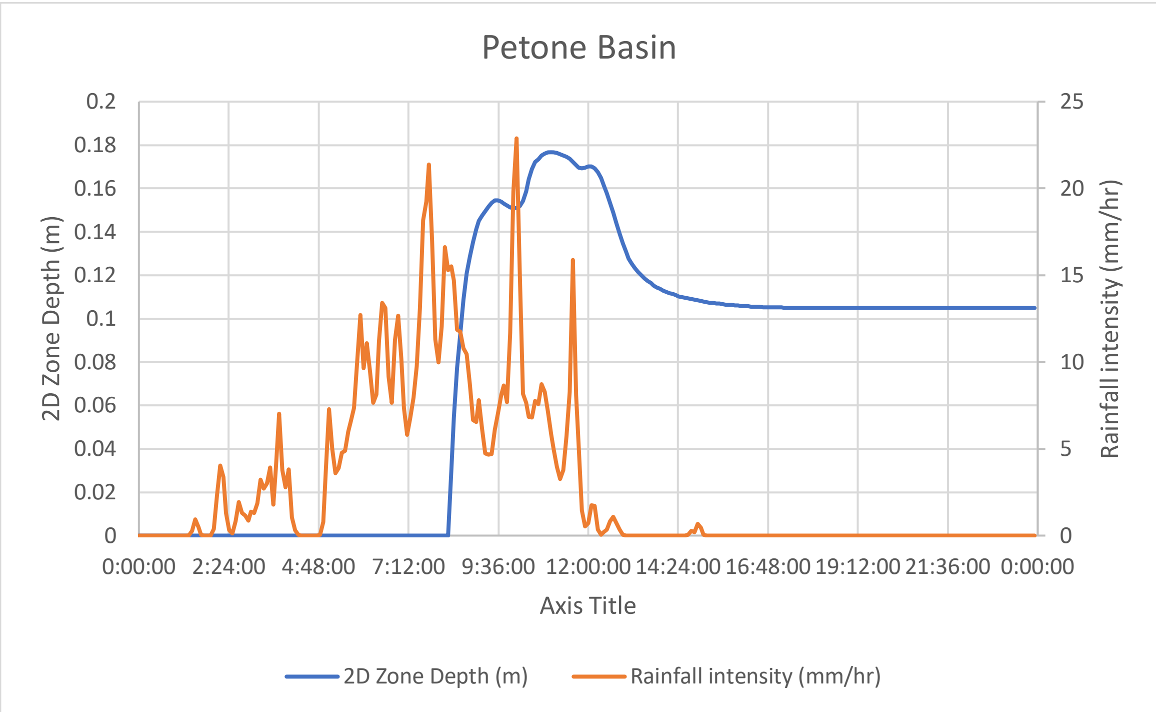


Figure L8 Water depth over time in the 2016 rainfall event at the "Petone Basin" site.

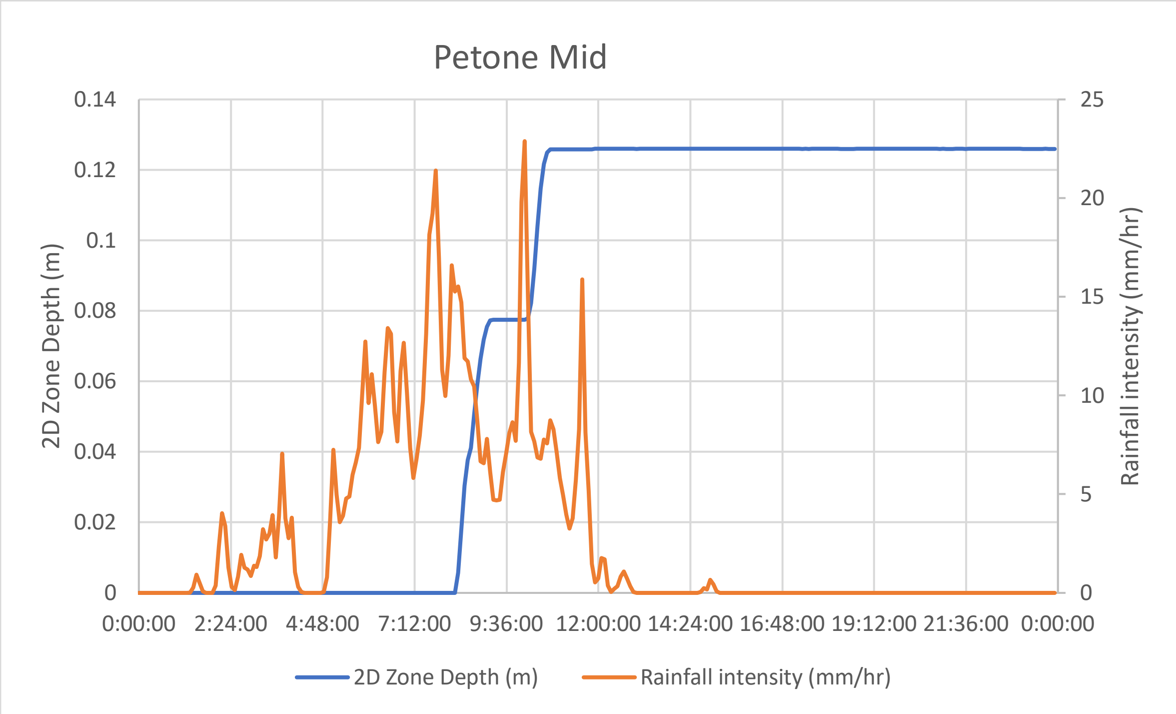


Figure L9 Water depth over time in the 2016 rainfall event at the "Petone Mid" site.

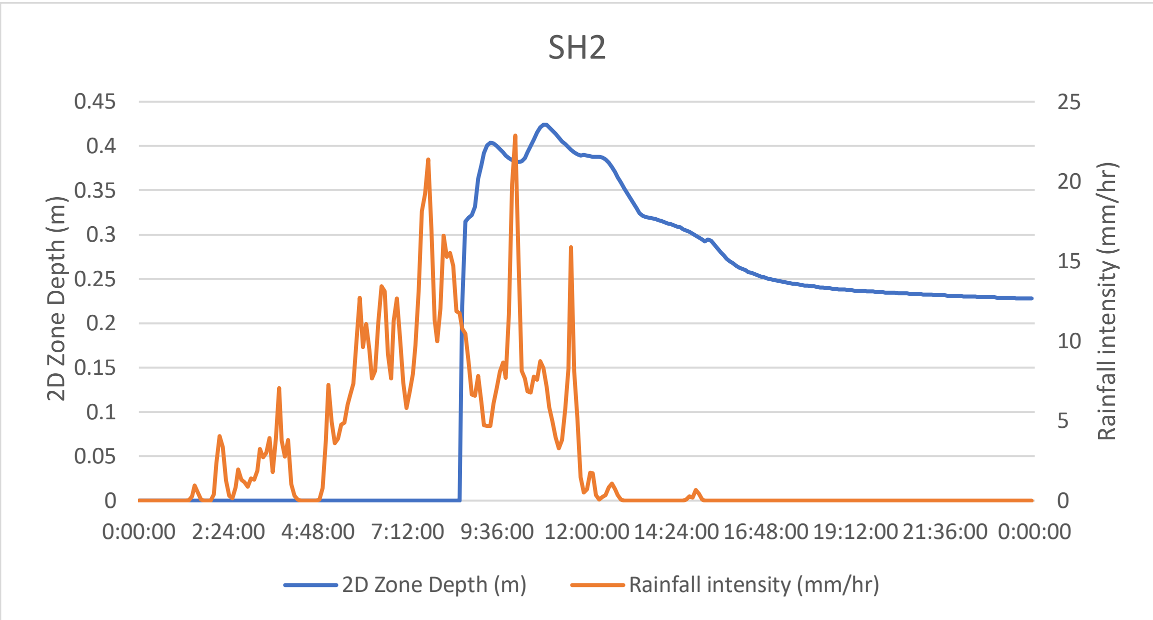


Figure L 10 Water depth over time in the 2016 rainfall event at the "SH2" site.

Appendix M Stormwater Outlet Asset Data

	Short ID	Outlet ID	Outlet Ground Level (m) RL	US Pipe Diameter mm	Has Flap Valve?	Has Penstock Valve?	Notes
	1	670342R01186	9.8	300	Yes	No	Pipe is single pipe under SH2. SH2 acts as stop bank
	2	1760489.919x5437432.579y	20.0	225	Yes	No	Assumed. May not exist
	3	1760412.131x5437421.202y	37.8	225	Yes	No	Assumed. May not exist
	4	670011R00663	33.2	1050	Yes	No	Assumed. May not exist
	5	670337R01186	9.6	300	Yes	No	Pipe is single pipe under SH2 where next upstream node is a sump. SH2 acts as stop bank
	6	670010R00663	5.5	1050	Yes	No	Pipe is single pipe under SH2 where next upstream node is a sump. SH2 acts as stop bank
	7	670336R01186	6.8	375	Yes	No	Pipe is single pipe under SH2 where next upstream node is a sump. SH2 acts as stop bank
	8	670335R01186	10.2	300	Yes	No	Pipe is single pipe under SH2 where next upstream node is a sump. SH2 acts as stop bank
	9	670334R01186	6.6	375	Yes	No	Pipe is single pipe under SH2 where next upstream node is a sump. SH2 acts as stop bank
	10	670311R01186	4.3	1800	Yes	Yes x2	Two pipes drain to same outlet
	11	n1760102X5437057Y	3.7	300	Yes	No	No need for penstock. Closest upstream manhole in centre of SH2
	12	670004R00462	6.5	1200	Yes	No	Manhole upstream of outlet is in an area where there is no defined stop bank. It has a higher elevation anyway
	13	670008R00462	4.8	600	Yes	Yes	
	14	670003R00462	4.7	300	Yes	Yes	
	15	670007R00462	4.9	225	Yes	No	Pipe is single pipe under SH2 where next upstream node is a sump. SH2 acts as stop bank
	16	670002R00462	6.4	225	Yes	No	Pipe is single pipe under SH2 where next upstream node is a sump. SH2 acts as stop bank
	17	670001R00462	4.4	1800	Yes	No	Next upstream node is an inlet in the Western Hills
	18	670006R00462	7.1	225	Yes	No	Pipe is a pipe under SH2 where next upstream node is a sump. SH2 acts as stop bank
	19	670006R00462	7.1	825	Yes	Yes	
	20	670094R00185	1.9	300	No	yes	As built shows no flap valve
	21	620001R00185	2.5	675	Yes	Yes	
	22	670018R00145	2.8	1200	Yes	Yes	
	23	670019R00145	2.9	450	Yes	No- Has upstream flap valve though	Two outfalls connected to Marsden Pump station
	24	670015R00145	0.4	525	Yes	No- Has upstream flap valve though	Two outfalls connected to Marsden Pump station
	25	PS Tama-St emergency-outlet	2.5	375	Yes	No- Has upstream flap valve though	Two outfalls connected to Tama St Pump station
	26	670043R00109	-0.2	450	Yes	No- Has upstream flap valve though	Two outfalls connected to Tama st Pump station
★	27	**670036R00109**	-0.2	1050	No	Yes	As built shows no flap valve
★	28	**670001R00118_c**	-0.2	1050	Has U/S flap valve	Yes	As built shows there is a flap valve at manhole upstream of the extended outlet, as well as an upstream penstock
★	29	**670004R00121**	0.0	750	Yes	Yes	
	30	670005R00121	0.4	1050	Yes	No- Has upstream flap valve though	Te Mome Pump Station Outlet
★	31	670035R01071	-0.6	225	Yes	No	Small pipeline with no room or need of penstock
	32	620003R01071	-0.6	950 X 940 Rectangle	Yes	No	Outlet from Dead Arm into Hutt River
	33	620002R01071	-0.6	950 X 940 Rectangle	Yes	No	Outlet from Dead Arm into Hutt River
	34	620001R01071	-0.6	950 X 940 Rectangle	Yes	No	Outlet from Dead Arm into Hutt River
★	35	670036R01071	1.2	600	Yes	No	Outlet past in an area with no stop bank. Assumed no penstocks
★	36	670036R01071	1.2	525	Yes	No	Outlet past in an area with no stop bank. Assumed no penstocks
★	37	670011R01012	0.6	225	Yes	No	Upstream node is a sump
★	38	670036R01028	0.9	150	Yes	No	Outlet past in an area with no stop bank. Assumed no penstocks
★	39	670035R01028	0.9	225	Yes	No	Upstream node is a sump
★	40	670034R01028	-0.6	375	no	Yes	Assumed no flap valve
	41	670010R01032	0.0	1200	Yes	No	Drains to dead arm
	42	610032R01028	1.8	225	Yes	No	Drains to dead arm. Outlet for William st Pump station
	43	670010R01032	0.0	1200	Yes	No	Drains to dead arm
	44	670004R01056	-0.4	750	Yes	No	Drains to dead arm
	45	670003R01056	0.1	450	Yes	No	Drains to dead arm
	46	670001R01014	0.9	450	Yes	No	Drains to dead arm. 2 pipes connected to one outfall
	47	670019R01018	-0.1	300	Yes	No	Drains to dead arm
	48	670001R01010	-0.3	225	Yes	No	Drains to dead arm
	49	670018R01002	0.4	600	Yes	No	Drains to dead arm

Figure M 1 Hutt River outlet information in model.

Appendix N Flow Distribution - Birchville

```
~~~ NIWA Tideda ~~~ NIWA                                      21-OCT-2019 16:55
~~~ PDIST ~~~
Source is PETONE 1.MTD Site 29818 Hutt at Birchville
From  3-Feb-1998 13:00:00 to 22-May-2019 09:15:00
Stage mm

DISTRIBUTION                PERCENTAGE OF TIME "VALUE" IS EQUALLED OR EXCEEDED

 DAYS  HHMMSS  "VALUE"   %  0        20        40        60        80        1
                  7181      *MAXIMUM |         |         |         |
    0  22354      6988 .001 *        |         |         |         |
    0  34838      6732 .003 *        |         |         |         |
    0  22529      6476 .005 *        |         |         |         |
    0  35224      6220 .007 *        |         |         |         |
    0  34135      5964 .009 *        |         |         |         |
    0  64706      5708 .012 *        |         |         |         |
    0  90332      5452 .017 *        |         |         |         |
    0 104935      5196 .023 *        |         |         |         |
    0 125255      4940 .030 *        |         |         |         |
    1  75305      4684 .047 *        |         |         |         |
    2  14216      4428 .074 *        |         |         |         |
    3  74818      4172  .12 *        |         |         |         |
    4  32105      3916  .17 *        |         |         |         |
    6 192646      3660  .26 *        |         |         |         |
    9  34326      3404  .37 *        |         |         |         |
   12 185909      3148  .54 *        |         |         |         |
   17 222851      2892  .77 *        |         |         |         |
   31 151030      2636  1.2 |*       |         |         |         |
   63 220738      2380  2.0 |*       |         |         |         |
  120 220032      2124  3.6 | *      |         |         |         |
  236 202506      1868  6.6 |  *     |         |         |         |
  654  93430      1612  15. |       *|         |         |         |
 1882 115430      1356  39. |        |         *         |         |
 3015 202119      1100  78. |        |         |         |        *|
 1558 201150       844  98. |        |         |         |         |        *
  152 222101       741 100. |MINIMUM |         |         |         |
     0 030000=GAPS
  7777 201500=TOTAL         MEAN=    1345.57            STD.DEV.=    373.915

This table uses   26 classes.
Percentages in the printplot table are accurate only to the
precision shown (e.g. 5.0% is 5.0% +/- 0.05%)

Exceedance percentiles

Source is PETONE 1.MTD Site 29818 Hutt at Birchville
From  3-Feb-1998 13:00:00 to 22-May-2019 09:15:00
Stage mm

          0      1      2      3      4      5      6      7      8      9

  0    7181   2728   2380   2200   2073   1976   1905   1846   1798   1760
 10    1730   1701   1676   1653   1631   1612   1594   1577   1562   1548
 20    1534   1520   1508   1497   1486   1476   1466   1456   1446   1437
 30    1429   1420   1412   1404   1396   1388   1381   1372   1365   1358
 40    1350   1343   1336   1330   1322   1315   1308   1301   1295   1288
 50    1282   1275   1269   1263   1258   1252   1246   1240   1234   1228
 60    1222   1216   1209   1203   1197   1190   1183   1177   1170   1163
 70    1156   1149   1142   1136   1129   1121   1114   1107   1100   1092
 80    1085   1076   1068   1058   1049   1039   1029   1018   1007    996
 90     984    972    959    948    935    920    905    884    845    819
100     741
```

Figure N 1 TIDEDA Hutt River flow distribution export.

Appendix O Useful As-builts

Appendix P DEM modifications

Minor edits were made to the DEM to fill missing information and/ or correct slight inconsistencies. These edits included the following:

- An area of Wellington harbour seabed was added to the 2D area and ground model. This was done so that an appropriate tidal boundary condition could be set far enough away from Petone foreshore to allow a tidal boundary to be modelled. The addition to the DEM was constructed from bathymetric survey points, sourced from New Zealand navigation/ hydrographic chart NZ4633, using a contour interpolation process to determine elevation between points.
- The “Dead Arm” of the Hutt River, a small oxbow water body adjacent to the Hutt River, was burned into the DEM to lower the ground surface to allow the surveyed stormwater outlets to discharge effectively, as many were initially below the DEM ground level. For efficiency, a flat channel bottom was used for each section of the Dead Arm, using inverts from the survey of the surrounding outfalls. (Refer Appendix E for details of the survey carried out as part of the model build)
- After adding harbour bathymetry to the DEM if was found that a small patch (0.001km2) was required to fill a small triangular gap between the harbour and the original DEM along SH2. This was to prevent model instability and was constructed by interpolating elevations from surrounding known elevations.
- In the UMM model build process and following the Regional Stormwater Hydraulic Modelling Specifications (Capacity Infrastructure Services Ltd, 2013), gradient lines were added to alter the DEM around outfalls that had surveyed or known inverts in the GIS database below the ground level of the DEM. These gradient lines would burn a channel into the DEM to allow the outlets or inlets to function properly. Some further alterations were completed in ICM following the same process where they were missed in the UMM process.
- A small patch was added around node 670001R00762. Like Alteration 3, above, the patch was interpolated from surrounding known elevations. It was created to fill a small hole in the DEM which caused significant unrealistic water depths of around 30m.
- Three building floor levels, including the Ulrich Aluminium building floor level, were burned into the DEM to represent the three buildings built over the Korokoro Stream along Cornish Street. A floor level was chosen instead of the default roof level, picked up in the LiDAR, to allow water to interact with the buildings. Unlike the rest of the large buildings in the model which were voided out of the 2D mesh, these buildings were included in the 2D mesh to allow water in the Korokoro
- Stream to rise above the building floor level without causing the model to become unstable. In addition, by including these buildings in the mesh their flooding history can be modelled and compared to observed events. To allow water to interact with the buildings a high roughness zone with a Manning’s n of 0.5 was selected.

Appendix Q Rapid Flood Hazard Results

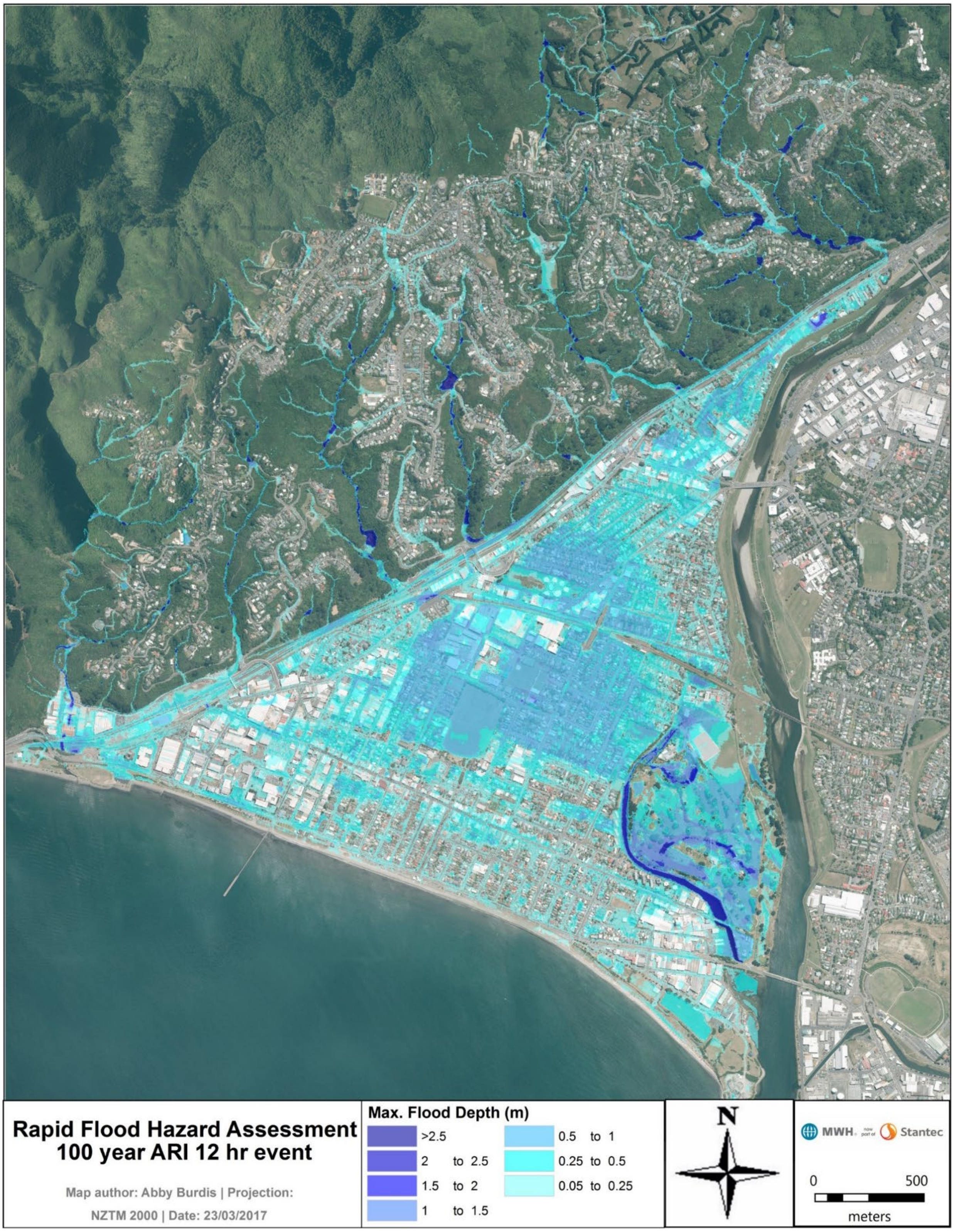


Figure Q-9-1 Rapid Flood Hazard Assesment Results for the 100yr ARI 12hr nested storm

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