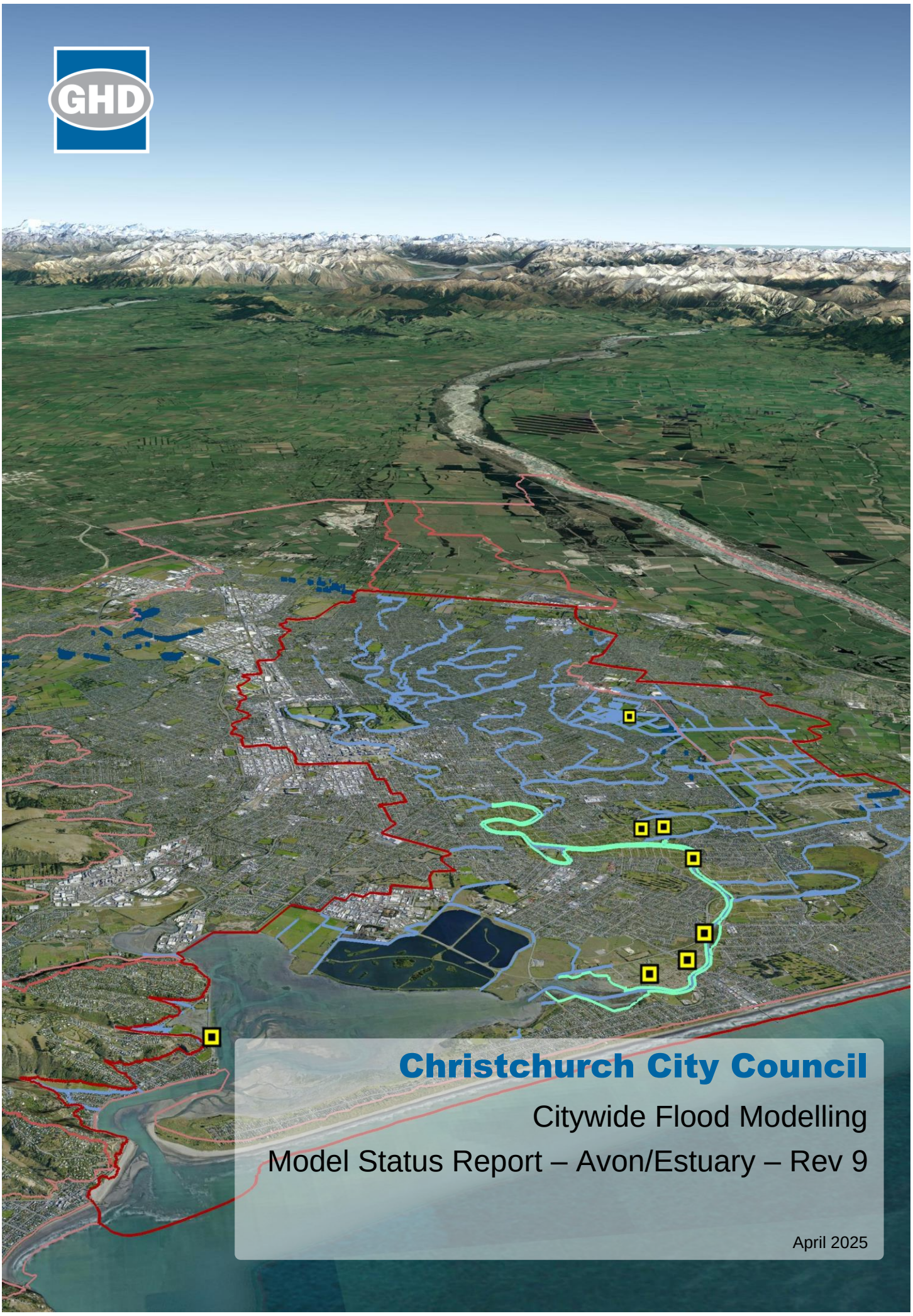


Christchurch City Council

Citywide Flood Modelling

Model Status Report – Avon/Estuary – Rev 9

April 2025

Executive Summary

The current Avon stormwater and flood model underwent a revolutionary rebuild in 2016-2018 and is now a sophisticated highly detailed model featuring:

- spatially continuous rainfall and infiltration calculations
- infiltration that responds to groundwater level (i.e. full runoff begins when storage above groundwater is filled by rainfall depth)
- detailed surface model with care taken to represent road cross sections including flow conveyance along the road gutters
- most inlets (sumps) as points of drainage into the pipe network
- all pipes of 300mm dia. and above, virtual pipes (simplified representation) for smaller pipes where required to connect the inlets
- all open channel in the CCC watercourse GIS layer
- all pump stations and flood mitigation basins
- the estuary modelled in 2D floodplain with wind setup applied
- an open ocean sea level boundary condition (well beyond the estuary mouth)

The model software is DHIv2020, 3 way coupled in MIKE Flood. The model is built with a high degree of spatial realism in consistent NZTM coordinates and Christchurch Drainage Datum.

During 2016-2018 the model was built to represent the 2014 catchment condition and was acceptably calibrated to the March 2014 flood event (and validated for the June 2013 flood event). Since 2018 the infiltration methodology has been improved from a simpler Hortons infiltration to a more realistic 'infiltration with capacity' model with a constant infiltration rate, but finite infiltration capacity (i.e. depth). At each mesh cell, once infiltration depth reaches the capacity then all further rainfall generates 100% runoff. Infiltration capacity is estimated based on depth to groundwater with soil storage parameters based on soil maps. Since this change the March 2014 calibration has been reestablished and slightly improved as a result. The monitored calibration data is limited to Gloucester St, but calibration is also significantly informed by the extensive flood extents mapping carried out by CCC following the March 2014 flood event.

There are gaps and inconsistencies in the March 2014 mapped flood extents and uncertainty remains with respect to infiltration rates in the west of Christchurch with large parts of the model dominated by theoretical parameters significantly informed by Council's WWDG document. There are also known patches in the model results where confidence is low and in extreme cases, there is a known bias in the results due to suboptimal model setup or anomalies from input data which have been noted but remain unresolved. There are a few examples where flood flows along road surfaces are artificially disrupted by modelling of open channels that cross the roads. It is not uncommon for modelled pipe networks to operate with water levels well below floodplain levels. This will be realistic in some cases where inlets are really lacking, but in other cases reflects mixed quality of input knowledge on inlets to the pipes, including a general lack of knowledge on private drainage systems conveying roof runoff directly to pipes (especially important in commercial areas).

During 2020-2022 significant model updates were completed to reflect major stormwater infrastructure projects since 2014. These Projects included Cranford Basin, Lower Dudley Creek Improvements and Diversion, improvements to six pump stations, new flood mitigation basins, as built stopbank levels and inclusion of Southshore bunds (including some minor planned work to complete this protection). Modelled floodplain levels were updated from 2018 LiDAR ground levels. The new Prestons subdivision and Northern Corridor motorway were partially represented according to the limited data which was readily available at the time. Detailed flood results in these two areas are considered poor, but the large-scale flooding patterns around them should remain reasonably valid.

During 2024 significant improvements to realism were made in the Riccarton areas around Picton Avenue and Kyle St as well as Harakeke St. These improved confidence in model predictions around Riccarton Main Drain upstream of the railway line, but were major enough to worsen calibration findings at Gloucester St. Significant extensions were also made into the Heathcote domain – a major extension near Buckleys / Linwood Ave and a minor extension near Hereford St for the purpose of

reducing boundary ponding. Another small extension was built near Moorhouse Madras for the purpose of creating an open boundary condition with eastbound outflows along Moorhouse Ave.

During 2024-2025 additional model improvements were completed, including updated future impervious input data, improvements to depth tolerance, and improvements to the Paparoa and Papanui Diversions, upstream of the Cranford Basin.

Design rain event modelling uses HIRDv4 rainfall intensities and a range of storm durations with a 'max of max' summarisation of flood risks for ARI's of 10, 50, and 200. This latest work is summarised in the metadata of the v117 post-processed model results delivered to Council on 3/31/2025 as well as the Model Scenarios tab of the Model Review Form that accompanies this submittal.

In the 10-year ARI current (circa 2020) situation, flooding is commonly restricted to the road reserves, greenspaces as well as waterways, wetlands and flood mitigation basins. Exceptional areas with worse flooding were identified as part of the LDRP097 Multihazard project (v114 of the model) and include:

- near the airport, west of Russley Road, (noting that this western area may have much higher infiltration rates than are modelled)
- St Albans, Flockton Ave area (the improvement projects are not predicted to fully resolve the 10-year flood risks here with some deep road flooding and probable floor level flooding still predicted there)
- Edgeware, Canon St southeast of Freshchoice and its small commercial precinct is also predicted to flood notably in the 10-year design event although houses in this area appear to be new so hopefully building controls will have ensured their floor levels are elevated well above the flood risks
- Southshore, flooding is predicted to be relatively minor with the new bund seeming to protect the southern residential area more than the north, although some flooding in excess of 100mm depth is predicted across a number of properties
- Moncks Bay is also modelled with significant flooding on the waterfront, apparently built up behind the low walls there (noting that modelling of these walls is difficult, that the walls are in the process of being redeveloped presently, and hillside hydrology runoff confidence is low – mainly theoretical with extrapolation of data from Bowenvale Valley)
- There are innumerable additional areas of locally notable flood risks – but too many to usefully catalogue and scattered across the City with no particular sense of order, connection or increased prevalence.

In the 50-year ARI current situation, v114 flooding patterns are generally similar but worse than the 10-year ARI, but for:

- an exception near Jefferies Road adjacent to the main railway embankment where flooding is predicted to increase dramatically (demonstrating a credibly high sensitivity to rainfall depth)

Flood results have also been prepared for future conditions out to year 2124. Modelling of future conditions is based on previously forecast spatial changes in urban housing density and population projections. For areas with capacity for greenfield development, developments are expected to fully mitigate their effects on runoff and flooding, up to 50-year ARI, with consequently minimal change in that flood risk. For areas of brownfield intensification, development mitigation is generally not expected to be practicable and is assumed not to occur, with consequently greater change in flood risks. Future modelling to date does not provide for any future infrastructure upgrades, most notably the stopbank top levels. Future modelling follows current government guidance on sea level rise and HIRDS rainfall projection with an RCP8.5 climate change scenario.

Contents

1. Introduction 1
   - 1.1 Background 1
   - 1.2 Purpose of this report 1
   - 1.3 Scope and limitations 1
     - **1.3.1** Report scope and limitations 1
     - **1.3.2** Model limitations 2
   - 1.4 Avon River/Estuary catchment 2
   - 1.5 Multi-hazard project baseline modelling 3
   - 1.6 Avon Modelling Improvements Project 4
   - 1.7 Cranford Baseline Modelling Project 4
   - 1.8 This report and other reporting 4
     - **1.8.1** Report updates for Rev 9 5
   - 1.9 Model developer 5
   - 1.10 Peer review 6
2. Modelling software 7
   - 2.1 Hydrology software 7
   - 2.2 Hydraulics software 7
   - 2.3 Software versions 7
   - 2.4 Hardware specifications 8
   - 2.5 Geographic reference systems 8
3. Model build 9
   - 3.1 MIKE 21FM 9
     - **3.1.1** Mesh extents 9
     - **3.1.2** Buildings excluded 10
     - **3.1.3** Final mesh level adjustments 10
     - **3.1.4** M21 parameters 11
     - **3.1.5** Rainfall 12
     - **3.1.6** Infiltration 12
     - **3.1.7** Impervious area 13
     - **3.1.8** Dikes 13
     - **3.1.9** Wind 13
     - **3.1.10** Mitigation basins 13
     - **3.1.11** Flow extraction lines 13
   - 3.2 MIKE Urban 14
     - **3.2.1** Pipes 15
     - **3.2.2** Manholes 15
     - **3.2.3** Open channel 16
     - **3.2.4** Pumps 16

3.2.5 Real time controls 17
3.2.6 Weirs 17
3.2.7 MIKE Urban INI file 17
3.3 MIKE 11 18
3.3.1 Network branch geometry 19
3.3.2 Network structures 20
3.3.3 Cross sections 23
3.3.4 Open channel roughness 24
3.3.5 Boundary data 24
3.3.6 HD parameters 25
3.3.7 MIKE 11.ini file 27
3.4 MIKE Flood 28
3.4.1 Lateral links 28
3.4.2 Urban links with M21 28
3.4.3 River/urban links 29
3.4.4 Standard links 29
3.5 RORB inflow boundaries 29
4. Integrated systems build 31
4.1 Paparoa Diversion 31
4.2 Winters Basin 31
4.3 Cranford Basin and PS0219 33
4.4 Shirley flow split 35
4.5 Stopbanks 35
4.6 Horseshoe Lake and Pump Station 36
4.7 Lake Kate Sheppard 37
4.8 Layout near Saint-Joseph School 38
4.9 Papanui Stream 39
4.10 Estuary 39
4.11 Lower Dudley Bypass 39
4.12 New Preston subdivision 40
4.13 Riccarton railway and Kyle St 41
4.13.1 Missing inlets 41
4.13.2 Simplifications 41
4.13.3 Invert levels at Picton Ave 41
4.13.4 Open channel 41
4.13.5 Bridge geometry correction 42
4.14 Railway ballast modelling 42
4.14.1 Cess drain 42
4.14.2 Railway mesh cells 43
5. Model calibration 43

5.1 Introduction 43
5.2 Model geometry changes 43
5.3 Open channel roughness 44
5.4 March 2014 calibration model rerun 45
5.5 Confidence in model calibration 46
5.5.1 Design model changes excluded from calibration model 47
5.5.2 Drainage system modernisations 47
5.5.3 Factors affecting model uncertainty 47
6. Design scenarios 49
6.1 Design wind parameters 49
6.2 Infiltration losses 49
6.3 Stopbanks down 50
6.4 Rainfall 51
6.5 Tidal timeseries 51
6.6 Estuary throat enlargement 52
6.7 Future earthquake settlement 53
7. Design run setup 56
7.1 Project and master model run scenarios 56
7.2 Model run durations 56
7.3 Tidal timing and pre-start 57
7.4 Hotstart level conditions 57
8. Model stability status 59
9. Design result evaluations 60
9.1 Functional model build checks 60
9.1.1 Knights Drain Basin and PS232 60
9.1.2 Tay Street Pump Station PS202 61
9.2 Flood differencing result checks 62
9.2.1 LDRP097 (V114c) minus LDRP112 (V096) models 62
9.2.2 Stopbank down minus stopbank up 64
9.2.3 2100 future minus 2020 existing scenarios 65
9.2.4 Standard result minus future EQ 66
9.3 Groundwater sensitivity 67
10. Compliance with specifications 68
10.1 Identified issues of low importance 68
10.2 Recommendations for model improvement 68
11. References 69

Tables

Table 1 Software summary 7
Table 2 Software versions 7
Table 3 Hardware platform 8
Table 4 GPU specification 8
Table 5 MIKE 21 FM parameters 11
Table 6 MIKE Urban technical summary 15
Table 7 MU pump station information 16
Table 8 Network file summary 19
Table 9 M11 dx values (values not equal to 40) 20
Table 10 Structure head loss factor 21
Table 11 Typical open cross section properties 24
Table 12 Base flow inflow boundary locations 25
Table 13 M11 computational parameters 26
Table 14 Bed resistance 26
Table 15 Quasi steady parameters 27
Table 16 Lateral link parameters 28
Table 17 Urban link parameters 29
Table 18 River/urban link parameters 29
Table 19 Standard link parameters 29
Table 20 Avon flow gauge roughness values 44
Table 21 March 2014 peak flow and water level at Gloucester Bridge 46
Table 22 Constant infiltration loss parameters 50
Table 23 Rainfall intensity with RCP8.5 climate change 51
Table 24 Sea level rises with climate change 52
Table 25 SLR v estuary throat area 52
Table 26 Standard run duration and save timesteps 56
Table 27 Extended run duration and save timesteps 56
Table 28 Tidal timing for Avon model 57
Table 29 Hotstart conditions 58
Table 30 LDRP097 V114 M21 Stability Assessment Outcome 59

Figures

Figure 1 Study area....3
Figure 2 M21 model extents (mesh topography)....9
Figure 3 Lincoln Rd flow extraction line with 200-year flood mapping....14
Figure 4 PS205 flow – level relationship....22
Figure 5 PS210 flow - level relationship....23
Figure 6 Winters Basin schematic....31
Figure 7 Cranford Basin, PS0219 and key active elements....33
Figure 8 Schematisation of Travis Swamp tidal circulation....37
Figure 9 St Joseph area – Council GIS....38
Figure 10 St Joseph area – MU model layout....38
Figure 11 Lower Dudley Bypass....40
Figure 12 Calibration time series at Gloucester St....46
Figure 13 Extent of stopbank modifications....50
Figure 14 Liquefaction settlement area of changes....54
Figure 15 Knights Drain Basin and PS232 result....60
Figure 16 Tay Street result....61
Figure 17 50ARI stopbank up V114c (LDRP097) minus V096 model results (LDRP112)....62
Figure 18 Differencing map of 2020 200ARI stopbank down minus stopbank up....64
Figure 19 Differencing map Of 2100 minus 2020 time horizon....65
Figure 20 Differencing map of standard result minus FutureEQ flood levels....66
Figure 21 85th% vs 50th% groundwater at Gloucester Street....67

Appendices

Appendix A – A3 maps, 2018 & 2022

Appendix B – Detailed stability standard methodology, 2017

Appendix C – Description of model setup requirements for updating scenarios, 2017

Appendix D – GIS raster origin and extents for DFS2 production, 2017

Appendix E – Design vs Calibration model setup memo, 2017

Appendix F – Anomalies register

Appendix G – Future improvement register

Appendix H – Runtime specification memorandum, 2016

Appendix I – Description of lateral link build process, 2017

Appendix J – Avon calibration approach memo, 2017

Appendix K – Climate change scenarios for modelling memorandum, Jacobs 2019

Appendix L – Rainfall climate change percentage, 2019

Appendix M – Avon model sensitivity to rainfall and groundwater level memo, 2021

Appendix N – Future earthquake settlement memo, T&T 2019

Appendix O – Future Impervious Report, 2024

Appendix P – Paparoa and Papanui Diversions report, 2025

Appendix Q – Investigation report on flooding near Riccarton Railway, February 2024

Appendix R – Schema report recommendations

Appendix S – Manhole headloss memo + addenda

1. Introduction

1.1 Background

The Avon/Estuary flood model originated from the LDRP044 project, which generated a new schematisation (system) for highly detailed flood modelling in Christchurch as well as specific models for the Avon/Estuary, Heathcote and Sumner catchments. The LDRP044 project resulted in a Citywide Schematisation Report and Avon Status Report (and other reports). The final version of the Avon/Estuary Model Status Report produced under LDRP044 was Rev 6.

Since LDRP044, GHD has implemented improvements to the Avon/Estuary Model, while others have completed updates on the Heathcote catchment models. A new Halswell catchment model has also been developed by others.

Following LDRP044, Avon/Estuary model improvements by GHD commenced under the LDRP097 Multihazard Project. As a part of the project, the Avon/Estuary Model Status Report was updated and was labelled as Rev 7.

The next phase of model updates by GHD occurred under the Avon River Modelling Improvements project. These updates have been incorporated into Rev 8 of the Status Report.

The most recent phase of model updates by GHD have occurred as part of the Cranford Baseline Modelling project. Model updates include improvements to the Paparoa and Papanui Diversions, improved depth tolerance settings for lateral links, and updated future impervious data. The model updates to the Paparoa and Papanui Diversions are described in Appendix P of this Rev 9 of the Status Report.

1.2 Purpose of this report

The purpose of this report is to summarise status of the Avon/Estuary flood model, which includes discussion of the model build, calibration, design setup, results, and model stability. Note that a description of the planned model build intentions is included in a separate report entitled “Model Schematisation – Avon/Estuary, Heathcote and Sumner – Rev 8”. Please refer to Section 1.5 for a more detailed outline of this report and other related reports.

To fully understand the status report, the reader should be familiar with the schema report. The status report covers exceptional cases which changed or differed from the schema intentions.

1.3 Scope and limitations

1.3.1 Report scope and limitations

This report has been prepared by GHD for Christchurch City Council and may only be used and relied on by Christchurch City Council for the purpose agreed between GHD and Christchurch City Council as set out in section 1.2 of this report.

GHD otherwise disclaims responsibility to any person other than Christchurch City Council arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no

responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD in collaboration with CCC, described throughout this report. GHD disclaims liability arising from any of the assumptions being incorrect.

GHD has prepared this report on the basis of information provided by Christchurch City Council and others who provided information to GHD (including Government authorities), which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information.

1.3.2 Model limitations

GHD has prepared the Avon River/Estuary Model ("Model") for, and for the benefit and sole use of, Christchurch City Council as part of the LDRP097 and Avon Modelling Improvements Projects, described in Sections 1.5 and 1.6, and must not be used for any other purpose or by any other person.

The Model is a representation only and does not reflect reality in every aspect. The Model contains simplified assumptions to derive a modelled outcome. The actual variables will inevitably be different to those used to prepare the Model. Accordingly, the outputs of the Model cannot be relied upon to represent actual conditions without due consideration of the inherent and expected inaccuracies. Such considerations are beyond GHD's scope.

The information, data and assumptions ("Inputs") used as inputs into the Model are from publicly available sources or provided by or on behalf of the Christchurch City Council (including possibly through stakeholder engagements). GHD has not independently verified or checked inputs beyond its agreed scope of work. GHD's scope of work does not include review or update of the Model as further Inputs becomes available.

The Model is limited by the mathematical rules and assumptions that are set out in the Report or included in the Model and by the software environment in which the Model is developed. The model is also limited by reliance on agreed methodology.

The Model is a customised model and not intended to be amended in any form or extracted to other software for amending. Any change made to the Model, other than by GHD, is undertaken on the express understanding that GHD is not responsible, and has no liability, for the changed Model including any outputs.

1.4 Avon River/Estuary catchment

The Avon River/Estuary catchment covers an area of about 146 km^2 and is approximately 25 km in length and typically 4 to 5 km wide. The main river channel (Avon) has an average channel gradient of less than 1 in 1300. The study area including significant waterways and tributaries is shown in Figure 1 below.

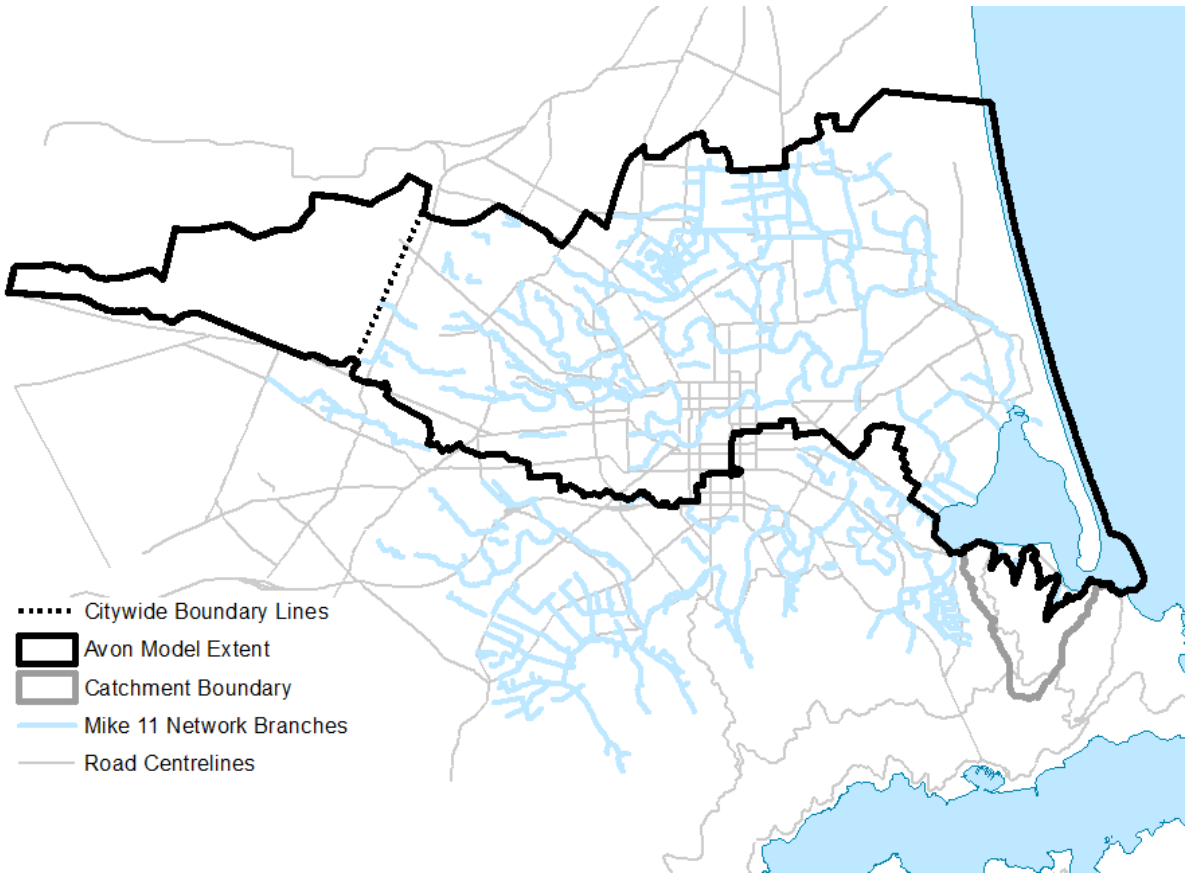


Figure 1 Study area

Note that the boundaries between the Avon and Heathcote Catchments have been recently revised. As discussed in Section 3.1.1, the Heathcote extensions have been incorporated into the model domain, and include extensions near Barbadoes St and Hereford Rd, Buckleys Rd and Linwood Ave, and the Moorhouse Ave and Madras St intersections. Figure 1 shows the Avon/Estuary Hydraulic Model Extents alongside the Avon Catchment, displaying an external hydrology catchment south of the estuary, which is not included in the hydraulic model.

The upper reaches of the Avon River are spring fed by streams originating in the northwest of the catchment. The river meanders eastward before discharging to the estuary downstream of Bridge Street.

The lower reaches, from approximately Kerrs Reach to the Estuary, are impacted by tidal flooding stopbanks have been constructed to protect the adjacent low-lying floodplain from this potential flooding source. In addition, several pump stations have been installed for flood protection purposes.

1.5 Multi-hazard project baseline modelling

In 2019, for their LDRP097 Multihazard project, GHD was engaged by the Christchurch City Council (Council) to perform Avon catchment model revisions and updates and then baseline hydraulic modelling runs to investigate the resilience of the city against future climate change (including sea level rise), earthquakes and urbanisation to assist the multi-hazard analysis. Some GHD work was also carried out updating the Sumner model but this was put on hold to better focus energy on the more urgent Avon model and the Sumner progress remains currently unreported.

As part of the LDRP097 project, the Avon/Estuary model was updated toward the 2020 existing development status and this new updated model becomes the latest master model for the Avon catchment. As part of the project, some Citywide model input datasets have also been updated or generated for the first time.

1.6 Avon Modelling Improvements Project

In 2023, GHD was engaged by the Christchurch City Council (Council) to perform improvements to the Avon/Estuary model to address modelling anomalies and produce a better, more reliable model of the system. This work effort was completed under the Council's project name "SOW070 SW Hydraulic Modelling- Avon Model" and is commonly referred to as the "Avon Modelling Improvements Project". Key issues addressed as a part of this project include significant flooding behind the Riccarton railway embankment, which is shown in the model but has not been historically noted; artificial ponding or missing inflow along the model boundaries; and missing headwalls, among other model anomalies. Additionally, the March 2014 calibration model was updated to include these significant Riccarton railway changes.

1.7 Cranford Baseline Modelling Project

In 2024, GHD was retained by the Christchurch City Council (Council) to update the Avon Catchment model to provide a more accurate understanding of existing baseline performance of the Cranford Basin, which can be used in future work to evaluate the potential impacts of the proposed Cranford Basin Stormwater Facility. The work effort included updating Paparoa and Papanui Diversions upstream of Cranford Basin, depth tolerance improvements, and providing new baseline model results.

1.8 This report and other reporting

This report summarises the built and calibrated status of the Avon/Estuary catchment model, as well as technical reporting on design event model runs subsequently carried out. Key report sections include:

- describe the modelling software
- introduce the built model
- provide details from the built model
- present model calibration
- discuss setups for design scenarios
- provide design model setup parameters
- report on model stability, and
- quality checking of results.

The report "Model Schematisation – Avon/Estuary, Heathcote and Sumner – Rev 7" provides details of the model build process and should be read in conjunction with this status report. Accordingly, the primary purpose of this Model Status Report is to document conclusions on the built and calibrated state of the model and to highlight any significant departures from the processes described in the Model Schematisation Report, including circumstances where atypical model schematisation has been adopted.

Note that the Model Schematisation Report has not been updated since the Multihazard project, however, recommended updates to the Schematisation Report are included in Appendix R.

In addition, there is a Riccarton Railway Report, written by GHD in February 2024. This project report focusses on the investigation of flooding near Riccarton Railway and Kyle St (see Appendix Q). The updated schema and status reports focus on the end product achieved. In all cases GHD have endeavoured to logically divide content between the schema, status and project reporting, while providing cross referencing to related content in the other reports to help readers.

1.8.1 Report updates for Rev 9

This report has remained almost entirely the same between Rev 8 and Rev 9. Updates incorporated into Rev 9 include:

- the Paparoa and Papanui Diversions Report included in Appendix P
- updates to the future impervious data, as described in Appendix O
- minor updates to the Executive Summary and Background (Section 1.1) to briefly discuss the work completed as part of the Cranford Baseline Modelling project
- updates to Section 3.4.1 to reflect the new depth tolerance settings, and
- updates to the anomalies register and future improvements register include in Appendix F and Appendix G, respectively.

Note that updates associated with the Cranford Baseline Modelling project resulted in minor changes to the count of certain modelling components, such as MF couples, MU weirs, and M11 structures. These updates have not been incorporated into Rev 9 of the status report.

1.9 Model developer

The Avon River/Estuary model has been developed by the Christchurch Office of GHD in partnership with AECOM and supersedes the previous Avon River flexible mesh model established by DHI. This previous model is described in the *Avon River Hydraulic Model Status Report* (DHI, April 2015).

The primary flood engineers/modellers associated with the development of the Avon model under LDRP044 project were Tim Preston, Olivier Benichou, John McArthur and Clemence Rolin.

Further improvements and updates were made to the model under the LDRP097 Multihazard project (GHD project 12506449). The primary flood engineers/modellers associated with these improvements of the model were Tim Preston, Clemence Rolin, Yanni Hooi, John McArthur and Rowan de Costa.

In 2024 and 2025, updates to the model were completed under the Avon River Modelling Improvements Project (GHD project 12625709) and the Cranford Baseline Modelling Project (GHD project 12640130). The primary flood engineers/modellers associated with these model updates were Tim Preston, Madeline Howell, Claudia Reid, William Lay, Andro Manuel Mauricio, and Greg Garrison.

Contact details are as follows:

Level 3 - 138 Victoria Street

PO Box 13468 Christchurch 8141

Phone: +64 3 378 0900

Email: chcmail@ghd.com

The current model version is V117.

1.10 Peer review

In 2021, DHI undertook a peer review on the model changes that were made under the LDRP097 Multihazard project[1].

This has resulted in a number of suggestions related to improving the representation of various model elements. Many of the recommendations were then actioned under the LDRP097 project and comments were returned to Council and DHI on all items including reasons why some items were not actioned.

[1] Review closeout email sent to DHI and Council on 11/2/2022 “Avon_model_Review.xlsx”.
GHD internal reference: G:\51\12506449\00.Document Transfer\Outgoing\20220211 Peer Review comments feedback Tim to Jo+Antoinette\Avon_model_Review.xlsx

2. Modelling software

The Avon River/Estuary model was developed using DHI's MIKE software. Table 1 summarises the MIKE software components used in the model build.

Table 1 Software summary

Process	Software	Method
Rainfall - runoff	MIKE 21 FM	Direct rain on mesh
Channel flow	MIKE 11 / MIKE URBAN	St Venant and continuity equations
Pipe flow	MIKE URBAN	St Venant and continuity equations/Colebrook White
Floodplains/roads	MIKE 21 FM	2D shallow water equations
Linking of above processes	MIKE FLOOD	

2.1 Hydrology software

Hydrology was modelled using a direct rain on mesh approach in MIKE 21 FM. This modelling approach applies a rainfall time series directly to each active mesh element in the 2D domain.

Model infiltration losses use the M21 constant infiltration with capacity method.

2.2 Hydraulics software

The Avon/Estuary model uses the hydrodynamic module of MIKE 11, the pipe flow component of MIKE URBAN and MIKE 21 Flexible Mesh (FM) software. The three software elements have been coupled together using MIKE FLOOD.

MIKE 21 FM was used to take advantage of improved run times offered by computer hardware utilising Graphical Processing Units (GPUs) incorporating additional processing capacity. The flexible mesh has been generated as an unstructured triangular mesh primarily to improve the schematisation of road elements (e.g. kerb and channel and road crown).

2.3 Software versions

Table 2 summarises the software versions used for the Avon/Estuary model development and the newest versions available at the date of this Report.

Table 2 Software versions

Model component	Software version used	Newest software version available
MIKE URBAN	v.2020 SP1	MIKE+ v.2024
MIKE 11	v.2020 SP1	MIKE+ v.2024
MIKE 21 FM	v.2020 SP1	MIKE+ v.2024
MIKE FLOOD	v.2020 SP1	MIKE+ v.2024
Note v. = Version, SP = Service Pack		

In addition, the model requires a structure licence to enable a control structure to be incorporated.

2.4 Hardware specifications

Table 3 summarises the hardware platform that was used.

Table 3 Hardware platform

Computer	Processor	Memory (GB)	GPU card	Operating system
HP Z640	2 x Intel Xeon E5-2650 v3 (10 cores 2.3 GHz)	32 - 64	NVIDIA GeForce GTX 980 Ti	Windows 10 Enterprise, 64 bit
Dell Precision 7820 Tower	2x Intel Xeon Gold 6244 (16 Cores 3.60GHz)	64	NVIDIA Quadro RTX4000	Windows 10 Enterprise, 64 bit

Table 4 summarises the graphics card specification used for modelling.

Table 4 GPU specification

GPU card	Compute capability	Memory (GB)
NVIDIA GeForce GTX 980 Ti	3.5	12

2.5 Geographic reference systems

The model has spatially correct geometry (subject to accuracy of some input data) using the NZGD 2000 New Zealand Traverse Mercator (NZTM) coordinate reference system and Christchurch Drainage Datum (CDD).

3. Model build

Summary characteristic standalone usages, data tables and the key parameters used within each model component are discussed in the following sub-sections.

3.1 MIKE 21FM

MIKE 21 FM uses a two-dimensional engine and models the Avon catchment floodplain including both manmade flood mitigation basins and natural storages. Topography is incorporated to determine overland flow paths and water depths during historic and design flood events.

The MIKE 21 (M21) component of the Avon/Estuary model utilises a triangular flexible mesh of varying sizes. Roads are included in the M21 model build with the mesh resolution providing representation of the kerb and channel and road crown features.

The mesh build process including the setting of mesh elevations is described in Section 4 of the Model Schematisation Report.

Dikes are included in the M21 model build typically to represent stopbank, seawall and mitigation basin crest levels. Basin and dike locations are shown on Map 5 in Appendix A.

3.1.1 Mesh extents

The mesh extends laterally to catchment boundaries in the north, west and south, to an open coastal underwater boundary in the east, and to a defined flat/hill boundary line south of the estuary. This extent is shown in Figure 2 below. The build incorporates approximately 1.78 million mesh elements ranging in size from 12 m^2 to 1327 m^2 and covers an area of 142 km^2.

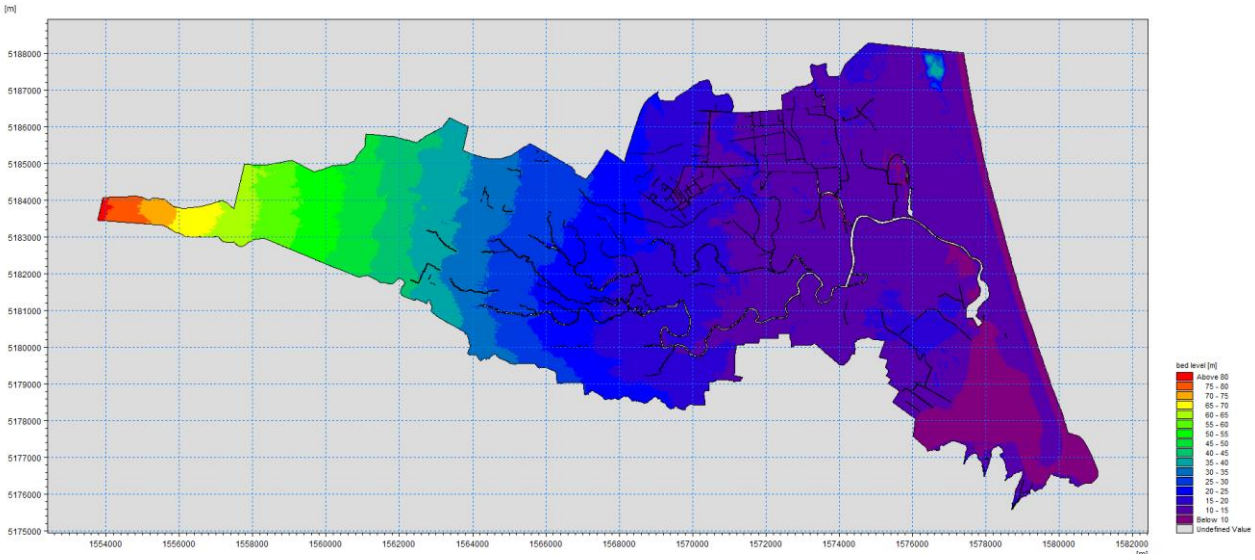


Figure 2 M21 model extents (mesh topography)

As part of the Avon Modelling Improvements Project, the mesh was extended to include areas that were previously included within the Heathcote model. These extensions were made to address locations which previously displayed artificial ponding or lacked inflows due to the closed model side boundaries. Mesh extensions include a 0.009 km^2 area north of the intersection of Barbadoes St and Hereford St and an approximately 0.62 km^2 area southeast of the intersection between Buckleys Rd and Linwood Ave. These mesh extensions were completed by clipping the Heathcote mesh and merging the clipped areas to the Avon mesh.

Small adjustments were made to some of the Heathcote elements along the boundary of the clipped mesh to align the boundary nodes from the clipped areas with the boundary nodes from the Avon mesh. These adjustments included merging or splitting existing Heathcote mesh elements. This process ensured that there were no slivers or gaps in the extended mesh.

The bathymetry values for the mesh extensions were obtained from the bathymetry layers (DFSU files) in the original Avon River/Estuary model and the Heathcote model. In areas where Heathcote mesh elements were split to allow the Heathcote clippings to merge with the Avon mesh, the z-value was taken as the average of the joined elements. In areas where Heathcote mesh elements were split, the original z-value was applied to both new elements.

Note that an approximately 2.06 km^2 area of the model east of the intersection between Moorhouse Ave and Madras St was identified as part of the Heathcote Catchment and will be added to the Heathcote model in the future. Instead of removing this area from the Avon model, an open boundary condition with a constant water level was defined in this location to allow flow to exit the model (see Section 3.1.4). The Avon mesh was also extended in this area to prevent the boundary condition from affecting hydraulic results within the Avon Catchment. The mesh and bathymetry layer were extended following the same process described above. Additionally, the z-value of mesh elements along the edge of the open boundary across Moorhouse Ave were manually lowered so that the bathymetry would not generate flow from the new open boundary condition (constant water level) into the model domain.

3.1.2 Buildings excluded

There are some building footprints that are fully outside of the mesh extents (i.e. fully within the M11 blockout) as discussed below.

One primary residential building at 59 Lake Terrace Road is outside of the mesh extents. This building is on redzone land and so is unlikely to be of future interest. Thirteen secondary buildings (generally residential separate garages or similarly small buildings in comparison to the largest building on the property) are located outside of the mesh extents. For each property the flood risk focus will be on the main building so these are also unlikely to be of future interest.

3.1.3 Final mesh level adjustments

There are three reasons why final adjustments are made to the mesh levels.

1. Firstly, there are places where the LiDAR is out of date or otherwise unsuitable for the model build purposes.
2. Secondly, MIKE Flood has some hard requirements for model initialisation at locations where 1D features are coupled to the mesh.
3. Thirdly, there are some special cases noted mainly from model stability and results inspections.

For convenience these final adjustments are made per mesh polygons using modeller discretion (rather than on the preceding ground level raster which is generated on a Citywide basis).

Cases of unsuitable LiDAR are:

- Basins newer than the LiDAR 2018 and as-builts have been used to define the mesh levels
- Christchurch Northern Corridor Motorway (NART) modification – as discussed in section 4.3 and anomaly 67.

At these locations the mesh levels are reset to manually chosen values. These adjustments are made to the basecase mesh prior to the future earthquake adjustments described in section 6.7.

The following cases of coupling have requirements for compatibility of mesh levels:

- Pipe inlet must be above ground – at M21 inlet to MU at sump locations or MU open channel, and
- Pipe outlet must be below ground – at MU outlets to M21 where pipes are discharging into M11 river or estuary.

Requirements for coupling are achieved by defining a set of thresholds as follows.

- pipe inlets are not above ground (mesh level threshold > pipe invert + 0.01m)
- pipe outlets are not below ground (mesh level threshold < pipe outlet - 0.05m.

If the mesh level does not comply with the threshold it is adjusted to meet the threshold. The adjustments therefore vary depending in the starting mesh level(s) and so need to be applied separately to each of the meshes generated by the future earthquake adjustments.

Special cases are:

- Two instances where the M11 blockout was narrower than the topography, each case resulting in a mesh cell with excessive depth and steep mesh level changes, and
- Five instances where the M21 mesh was too high to allow flow appropriately into or out of standard links to M11, resulting in mesh stability issues and/or faulty flow continuity

At these special locations the mesh levels are reset to manually chosen values. The two instances noted next to M11 blockouts were made to the basecase mesh prior to the future earthquake adjustments described in section 6.7. The five instances next to standard links were implemented to the future earthquake adjusted meshes with the coupling requirement adjustments[2].

3.1.4 M21 parameters

The key parameters within the MIKE 21 FM model are presented in Table 5.

Table 5 MIKE 21 FM parameters[3]

Parameters	Value
Simulation start date	Design events: 1/01/1900 12:00:00 a.m.
Time integration	Low order, fast algorithm
Space discretization	Low order, fast algorithm
Min time step	0.01 s
Max time	1 s
Critical CFL number	0.8
Flood and Dry>Type	flood and dry
Drying depth	0.001 m
Wetting depth	0.003 m
Eddy viscosity type	Constant eddy formulation
Eddy viscosity format	Format = Constant
Eddy viscosity value	0.1 m^2/s
Bed Resistance	Spatially varying 5 m grid
Wind Forcing>Format	Varying in time and domain (all runs have limited extent and do not vary across the domain)
Wind Forcing>Data file	Design: 10 m/s SW design wind Calibration: New Brighton pier data

[2] With hindsight it would have been more consistent to apply the special case adjustments depending on whether they are adjacent to M11 elements that are earthquake shifted or not. In all seven instances these adjustments are adjacent to unshifted M11 elements, and therefore would be best applied with the coupling requirements after the future earthquake adjustments.
[3] These parameters are recorded with rationale in the Avon model log - parameters tab

Parameters	Value
Wind Friction	Friction type = varying with wind speed; linear variation using speed; 7 m/s, friction = 0.003765; 25 m/s; friction=0.00753
Dikes>Type	Empirical Formula
Dikes>Critical_level_difference	0.01
Dikes>Coefficient	1.838
Dikes>Specification_of_crest_level	Varying in space for stopbank up scenario and Constant with 0m crest level for stopbank down scenario
Initial Conditions	Initial constant surface elevation in the Estuary and coastline. Refer to section 7.4.
Boundary Conditions	1. Time varying (tidal) surface elevation at coastal downstream boundary based on Sumner Heads design/actual data. 2. Open boundary near the Moorhouse Madras intersection was added to allow flow to exit toward the Heathcote Catchment and features a constant water level of 12.9 m. 3. All other locations closed.
Output items	1. Time-varying surface elevation, still water depth, total water depth, U velocity, V velocity, saturation and infiltration volume. 2. Last time step surface elevation, still water depth, total water depth and Infiltrated volume 3. Time-varying discharge crossing the estuary throat and winter basin

Note that as a part of the Avon Modelling Improvements project, an open boundary was added along Moorhouse Ave, west of the intersection with Barbadoes St. This boundary was added to allow flow toward the Heathcote Catchment in order to prevent artificial ponding along the Avon model boundary. The open boundary was set with a constant water level of 12.9 m, which sets the flood depths along the boundary to 100 mm. As discussed in Section 3.1.1, the z-values along the boundary were lowered to facilitate 100 mm of ponding while ensuring that the open model boundary does not introduce flow into the model domain.

3.1.5 Rainfall

As outlined in Section 2.1, the Avon catchment hydrology has been modelled as direct rain on mesh within MIKE 21 FM.

Time-varying grid (dfs2) files for design and calibration rainfall have been developed based on a citywide 100 metre spacing for calibration rainfall and 500 metre spacing for design rainfall and with units of mm/day. Design rainfall has been derived continuously in space based on HIRDS v4 rainfall depths. Hyetograph shape is based on the CCC triangular hyetograph, described in Waterways, Wetlands and Drainage Guide (CCC, 2011) with an Aerial Reduction Factor of 0.95 applied. Design rainfall (dfs2) files of 10 mins to 96 hours have been generated, with details in Schematisation Report Appendix N.

3.1.6 Infiltration

The model uses M21 constant infiltration with capacity method. The constant in time, varying in domain infiltration file uses 10m grid size and contains the following 5 different layers:

1. effective infiltration layer (mm/hr)

2. porosity (unitless)
3. depth below ground (m)
4. leakage (mm/hr), and
5. initial capacity (unitless).

Infiltration loss rates are defined spatially according to mapped soil types. The method of setting up the infiltration files for calibration and design event parameters are documented in Model Schematisation Report Appendix L.

3.1.7 Impervious area

Impervious area coverage has been determined within a Citywide process and is reported in GHD's "Future Impervious Report" 2024 found in Appendix O.

3.1.8 Dikes

Dikes have been included at 32 locations. Twenty of these represent crest levels associated with basins in the Avon catchment. Six represent stopbanks along the downstream reach of the Avon River from PS205 to Bridge Street, four represent stopbanks along the Southshore and the remaining two are associated with sea walls along Beachville Road and Moncks Bay.

It is noted here that no as-built details of the Prestons basins outlet were reviewed, and therefore the modelled outlet details are based on the existing Prestons model.

3.1.9 Wind

Design wind has been applied to the estuary area within the model. The wind speed is set to 10m/s with a wind direction from southwest to northeast. A wind friction factor is applied based on calibration outcomes.

3.1.10 Mitigation basins

There are some exceptions to the basin schematisation (schema section 4.3.4) associated with narrow basins where a basin is too narrow, a toe of bank has not been formed and therefore basin floor levels have not been accurately represented in the model. This is the case for the three Alpine View basins (south of Prestons Road).

3.1.11 Flow extraction lines

It has been observed that during large floods, substantial flows move southeast across Hagley Park. These flows continue to cross Lincoln Rd. Changes agreed between GHD/DHI/CCC are in progress to extend the Heathcote model to Lincoln Rd, which when completed will mean flood levels in this area southwest of Lincoln Road will primarily be assessed with Heathcote model results. In order to support this plan flows into the Heathcote model at this new boundary need to be generated.



Figure 3 Lincoln Rd flow extraction line with 200-year flood mapping

A similar flow extraction line captures flow out of the Moorhouse Road extension open boundary (discussed in Sections 3.1.1 and 3.1.4). This line runs along the eastern margin of Madras St and any outflow along Moorhouse crosses this flow line.

3.2 MIKE Urban

MIKE Urban has typically been used to model pipe networks and some of the smaller engineered open drains. It also models most of the pump stations.

The pipe network consisting of pipes with a diameter of 300 mm and greater has been included in the Avon catchment model with the bulk of this pipe network included in the MIKE Urban (MU) domain. In addition, as outlined in Section 6.4 of the Model Schematisation Report, some open channel reaches have been included in the MU network. The extent of the MU network is shown in Map 2 in Appendix A. This map also summarises the MIKE Urban extent relative to the MIKE 11 layout and illustrates how the number of M11 to MU links has been minimised.

Existing models, CCC as-built/design drawings, SCIRT survey data, and Council's GIS data has been used to develop the MU network. As some of the information is only available dated prior to the 2011 earthquake, such levels are adjusted to post-earthquake levels. Pre-earthquake

level adjustments to watercourse nodes and links and pipe node ground levels are described in LDRP044 Model Build Report Rev 6 Appendix J. Inflows to the pipe network are primarily from the M21 domain.

Most pump stations are functionally modelled in MU. Pump station details are tabulated in Section 3.2.4 and locations are shown on Map 5, Appendix A.

A technical summary of the MIKE Urban model content is outlined in Table 6.

Table 6 MIKE Urban technical summary

MIKE Urban general data	Count/measure
Pumps	15[4] (including the 2 dummy pumps)
Controlled pumps	1 (PS0219)
Weirs	11
Boundary outlet weirs	4
Railway ballast weirs	1
Orifices/Valves	0
Controlled weirs/gates/valve	0

3.2.1 Pipes

The modelled pipe shapes are either circular, rectangular or closed CRS. The diameters range from 0.07 m to 2.4 m. The diameter of 0.07 m is associated with an outlet at Prestons Basin. The dimensions of the rectangular pipe have widths from 0.15 m to 6.8 m, and heights from 0.15 to 3.9 m. The CRS X-Z closed type are used to model Avon.Rawhiti bridges at Rawhiti Golf Course. The hydraulic friction loss formulation for all pipes is Manning Explicit. No pipes are modelled as pressure mains.

In some instances, the link length has been increased from a minimum of 10 m to 15 – 20 m to address instability issues.

In accordance with Section 6.1 of the Model Schematisation Report, some negative graded links are included in the model.

The Avon MU model has 97 flap gates modelled as one-way pipes. They are primarily located along the lower Avon River.

These are sourced from:

- Existing models (13)
- Council's GIS (29), and
- Assumed flap gates through the Avon stopbanks as agreed with Council (55).

3.2.2 Manholes

The losses for the junctions and manholes on the 1.8 m diameter pipe system on Riccarton Main Drain between Picton Ave and Deans Ave were changed from the "MOUSE Classic" to "Weighted Inlet Energy"[5]. Refer to Appendix S analysis. There was an attempt to set junctions to zero head loss, however this did not produce sufficient head loss in the overall system.

[4] There are 13 pump stations modelled using 15 model pump items. Two of these pump stations are dummy in MU (functional in M11). Typically pump stations are modelled in aggregate rather than as individual pumps. PS219 is the exception where its three pumps are modelling individually to enable the single pump control inhibition function. Because of PS219 there are two more (15) modelled pumps than there are modelled pump stations (13)

[5] This was in accordance with the email: Critical update from Antoinette conversation - Manhole headlosses from Tim Preston on 17/05/2024.

The loss method for junctions and manholes added as part of the Heathcote mesh extensions retained their Heathcote model settings of “Weighted Inlet Energy”. Please refer to Section 3.1.1 for more discussion on the Heathcote mesh extensions.

3.2.3 Open channel

Some open channel sections have been modelled in MU where it is considered appropriate to simplify the 1D networks and reduce the number of M11-MU and/or MU-M11 links. Refer to Map 2 in Appendix A for the open channel locations.

A minimum open channel depth of 0.2 m has been used requiring modification of the actual channel geometry in some cases. These are identified with a CRS ID including “_geom_modified”.

3.2.4 Pumps

Thirteen pump stations are included in the MU model. Eleven of these are normal and functional in MU. Two exceptional pump stations are functionally modelled in M11 and “Zero Flow” dummy pump stations noting the existence of those have been included in MU for navigational convenience. Each pump station uses a Q-H relationship to represent a simplified aggregate pump station operation. Summary is provided in Table 7 below.

Table 7 MU pump station information

MUID *	Pump station description		Pump off level (m)	Max flow level (m)	Max pump flow (m^3/s)
AVON.Pump.CCCGIS.0202	PS0202	Tay Street Pump Station	12.59	12.80	2.00
AVON.Pump.CCCGIS.0203	PS0203	Wairoa Pump Station	7.57	8.17	0.51
AVON.Pump.CCCGIS.0204	PS0204	Waitaki Pump Station	8.19	9.69	0.445
AVON.Pump.CCCGIS.0207	PS0207	Evans Pump Station	7.33	8.03	0.073
AVON.Pump.CCCGIS.0209	PS0209	Celia Pump Station	8.18	9.38	0.105
AVON.Pump.CCCGIS.0219_1	PS0219	Philpotts Pump Station	12.07	12.82	1.26
AVON.Pump.CCCGIS.0219_2	PS0219	Philpotts Pump Station minor pump	12.47	12.62	0.07
AVON.Pump.CCCGIS.0219_3	PS0219	Philpotts Pump Station (Controlled Pump)	12.07	12.97	1.26
AVON.Pump.CCCGIS.0220	PS0220	Hulverstone Pump Station	8.02	8.42	0.5
AVON.Pump.CCCGIS.0229	PS0229	Blake St Pump Station	9.7	9.85	1.0
AVON.Pump.CCCGIS.0230	PS0230	Owles St Pump Station	8.902	9.752	0.8
AVON.Pump.CCCGIS.0231	PS0231	Beresford St Pump Station	8.504	9.4	0.8

MUID *	Pump station description		Pump off level (m)	Max flow level (m)	Max pump flow (m^3/s)
AVON.Pump. CCCGIS.0232	PS0232	Bexley Pump Station	8.85	9.05	0.65
AVON.PUMP CCCGIS.0205	PS0205 *	New Brighton (Horseshoe Lake) Pump Station			
AVON.Pump CCGIS.0210	PS0210 *	Locksley Pump Station			

* Not functional dummy only in MU – functionally modelled in M11 network tabulated structures (refer to Section 3.3.2)

3.2.5 Real time controls

There is one real time controlled feature in the MU model – PS0219 pump 3. This is part of the Cranford active management stormwater plan. The modelling of this controlled pump is reported in Section 4.1.

3.2.6 Weirs

There are 16 weirs in the MU model, as follows:

Eight real weirs are included. Four are associated with the Preston South Basin system. One weir is located in the Tay St project update area, along Norah St. The three remaining existing weirs are located in open channel systems modelled in MU. These three are located in:

- Papanui Stream Side Drain off Proctor Street,
- Washbourne Drain immediately upstream of the 900 mm diameter pipe system adjacent to Oakford Close, and
- Knights Drain, where flow is split between the pump station and the Knights Drain Basin (see Section 9.1.1).

Four dummy weirs are used to set Qmax outlet boundaries to the Heathcote catchment. Two of these dummy weirs are located at the bifurcation in the vicinity of Madras Street and Moorhouse Avenue (MUID's AVONHETH.Weir.Dummy.900000) and AVONHETH.Weir.Dummy.900001). The other two dummy weirs are located east of the intersection between Buckleys Rd and Linwood Ave (MUID's AVONHETH.Weir.Dummy.900012 and AVONHETH.Weir.Dummy.900013).

Another dummy weir is used to model flow through the Riccarton Railway Ballast, as described in Section 4.14.

The remaining three are also dummy weirs, located adjacent to MU outlets to M11 where the M11 invert is significantly higher than the MU invert and modelling prior to inclusion of these dummy weirs indicated substantial flow volumes were being generated into the MU system. Each weir is separated from the outlet link by a dummy manhole.

These dummy weirs have a crest level equivalent to the M11 invert level and a crest width of 10 m. The 'Weir Formula' is used, and the MUID description indicates the weir is a dummy structure.

3.2.7 MIKE Urban INI file

The following modifications to default parameters have been made in the dhiapp.ini file:

- MM21_ALLOWEDGROUNDLEVELDIFFERENCE=10 (effectively disabled here with our sump-linking approach disconnected from the node location)

- MM21_ALLOWED_OUTLET_LEVELDIFFERENCE=0.1 (default)
- CRS_EXTRAPOLATION=2 (to enable extrapolation of the MU open cross-section based on the extrapolation angle)
- CRS_EXTRAPOLATION_ANGLE=30
- CRS_EXTRAPOLATION_HEIGHT_FACTOR=29
- RESERVOIRHEIGHT=50, and
- FLOODLIMIT=100.

The minimum pipe length (BRANCH_MIN_L) has been maintained at 10 m.

3.3 MIKE 11

MIKE 11 has typically been used to model rivers, natural tributaries and most of the engineered open drainage network, with its associated culverts and sometimes longer sections of pipe or pipe networks. Two large pump stations are modelled in M11 as tabulated structures in the network (refer section 3.3.2).

The MIKE 11 component is made up from four separate files:

- Network File (.nwk11)
- Cross Section File (.xns11)
- Boundary Data File (.bnd11), and
- HD Parameters File (.hd11).

In addition, the MIKE 11.ini file provides a mechanism to adjust settings within the calculation engine of MIKE 11. The remainder of this section outlines the inputs and parameters specified for all five (5) files.

The MIKE 11 component of the Avon/Estuary model is made up of 198 branches representing the main river, tributaries and open channels, including lined drains, within the Avon catchment. In addition, as outlined in Section 5.1 of the Model Schematisation Report, some pipe lengths have been included in the M11 network as closed cross-sections. Map 2 in Appendix A summarises the MIKE 11 extent including both open channels and closed cross-sections, relative to the MIKE Urban layout.

The network includes bridges and culverts with the majority of bridges modelled as irregular culverts. In addition, two pump stations (PS205 and PS210) have been included in the MIKE 11 network with both pump stations set up and modelled as tabulated structures.

The MIKE 11 network has been established utilising available cross-section surveys undertaken in post September 2010, post February/June 2011 and 2015. This has been supplemented with timber lined drain cross section dimensional information, integrated into cross-section levels extracted from 2011 LiDAR. In addition, cross-section and structure information from existing hydraulic models has been incorporated in branches/reaches where post-earthquake survey is not available. Data sources are shown in Map 6 in Appendix A. Two pump stations are modelled as tabulated structures in M11.

Hydraulic model data sources are identified by the Cross-Section ID as discussed in Section 5.10 and summarised in Table 11 of the Model Schematisation Report. Key source models used in the Avon model build include the following:

- Aranui model
- Cranford Basin model

- Lower and Upper Dudley models including Shirley Stream
- Okeover Stream model
- Parklands model; and the
- Saint Albans Creek model
- BECA Dudley model v29, and
- Cranford Model v30.

3.3.1 Network branch geometry

A short summary of the MIKE 11 network is provided in Table 8 below.

Table 8 Network file summary

Input/parameter		Number
Network	Branches	198
	Points	8615
Structures	Weirs	77
	Culverts	540
	Bridges	3
	Pump	0
	Control structures	61*
	Tabulated structures	2
	Energy loss	12
	Hydraulic control	0

Graphical points

All graphical points within the network have been specified with x and y coordinates. Each point's chainage type is typically 'system defined' with some 'user defined' generally at the start and end chainages of branches.

Branches

All the MIKE 11 branches are defined as 'regular' branches. This is the normal branch type used within MIKE 11 and is composed of a number of calculation points defined by the cross sections specified in the cross section file. These branches describe significant waterways and smaller tributaries within the Avon catchment.

The most significant waterways are as follows:

- Avon River avon.avon
- Dudley Creek avon.dudley
- Wairarapa Stream avon.wairara
- Waimairi Stream avon.waimai
- Wai-iti Stream avon.waiiti, and
- St Albans Creek avon.stalban.

The maximum dx is 40 m with the exception of the branches tabulated below. In the few instances where there are long culvert structures in a branch, the branch dx value has been set to a larger value and cross section spacing has been used to limit actual distances.

Table 9 M11 dx values (values not equal to 40)

Branch name	Maximum dx (m)
avon.crossstr	200
avon.dudley	200
avon.dudley20	50 from ch 2006 m to 2068.7 m, 45 from ch 2068.8 m to 2415 m
avon.dudleyBP	100
avon.freesck	90 from ch 34 m to 120 m, 70 from ch 120 m to 836 m
avon.mcsaveny	160
avon.no1drain	45
avon.papanui	52
avon.paparoapipe	10000
avon.pickerng	150
avon.pynegld	25
avon.riccmain	45 from ch 2702.8 m to 3214.1 m, 80 from ch 490 m to 564.9 m
avon.shirley1	50
avon.snelldr	45
avon.stalban	120
avon.taydrn	65
avon.tilmans	65
avon.wairara	70
avonstyx.mills	10
avonstyx.watson10	25
esty.glenstr	10
esty.plsdmdrn	200
heth.heathcot	10000

There are four instances where the branch direction (based on increasing chainage) does not comply with typical low flow direction. These are:

- esty.cuthdrs0
- avonstyx.rossiter east of Buller Drain
- avon.shirley1, and
- avon.maireh.

3.3.2 Network structures

Network structures include:

- Weirs
- Culverts
- Bridges
- Pumps
- Tabulated structures
- Energy loss, and
- Control structures.

Weirs

All weirs have been modelled using the Broad Crested Weir formulation, except for two in the avon.wairara branch which have been modelled using Weir Formula 1 type. No modelled weirs

have valve regulation; therefore, depending on up and downstream conditions flow may travel in either direction. The head loss factors adopted for the modelled weirs are all default values as shown in Table 10.

Culverts

A range of culvert geometries has been used within the Avon catchment model. The structures are all “regular” type with flow passing through the culvert based on upstream and downstream conditions. The geometry type varies from rectangular and circular (pipe) structures to irregular shapes. Irregular shapes are used to represent some bridge structures with details extracted from previous post-quake models. Where the culvert is of an irregular geometry, either the cross-section file contains a closed cross-section at the relevant chainage, or the network file contains a depth-width or level–width table to define the geometry.

Manning’s ‘n’ values range between 0.013-0.07 for all culverts. Valve regulation has generally been set to ‘None’. The exceptions are value regulated culverts on the following branches:

- avon.boxoutlet ch4.9
- avon.bullerst ch9
- avon.corserst ch958
- avon.cranest2 ch107
- avon.crozir ch747
- avon.estydrbr ch485
- avon.horsehoe ch200
- avon.katshep ch960
- avon.oldlake ch555
- avon.pipeoutlet ch5
- esty.cuthbr2 ch251.77
- esty.cuthdrn0 ch805, and
- esty.plsdmdrn ch200.

Sixty two culverts have been modelled in conjunction with an overflow weir at the same location. In twenty six of these instances, the weir level has been based on the ‘top of handrail’ level.

The head loss factors adopted for the modelled culverts vary as shown in Table 10.

Table 10 Structure head loss factor

Head loss factor	Broad crested weir	Culvert	Control structures	Default
LPI	0.5	0.2-0.6	0.5	0.5
LPO	1	1	1	1
LPF	1	1	1	1
LPB	-	0	-	-/0
LNI	0.5	0.5-0.6	0.5	0.5
LNO	1	1	1	1
LNF	1	1	1	1
LNB	-	0	-	-/0

Bridges

Three bridge structures have been included within the model with details extracted from previous Avon River models including the Cranford Basin model.

They are as follows:

- Helmores Lane Bridge at chainage 7831 on the avon.avon branch
- Park Terrace Bridge (western end of Armagh Street) at chainage 9611 on the avon.avon branch, and
- The Hospital Bridge (off Cambridge Terrace) at chainage 11799 on the avon.avon branch.

The Federal Highway Administration (FHWA) WSPRO method has been used to model these bridges. This method takes into account contraction loss by the calculation of an effective length and expansion loss via experimentally based tables.

If the bridge is submerged, the Pressure Flow FHWA method is used. Two different equations are used for this method depending on whether the downstream orifice is partially or fully submerged.

If overflow occurs, the behaviour is modelled using the FHWA method. This method models overflow using a weir equation taking tailwater submergence into account.

Pumps

There are no pumps included in the .nwk file, but two pump stations have been modelled as tabulated structures in M11. Further details are provided in the Tabulated Structures section below.

Tabulated structures

Pump Station 205 and Pump Station 210 have been included in the M11 model with their operating condition modelled as Tabulated Structures. The calculation mode is set to be Q= f (h U/S, h D/S).

Pump Station 205 consists of three Archimedes screw pumps with a combined maximum flow rate of 14 m^3/s. However this combined flow rate can be significantly reduced dependent on intake (Horseshoe Lake) water levels and tailwater (Avon River) levels. Low intake levels and/or high Avon River levels can reduce pumped flow capacity to zero. Flow/downstream level/upstream relationships have been developed to reflect the screw pump discharge limitations. These relationships also take into account the pump start levels for each of the three pumps. The operating conditions used in the model are summarised in Figure 4[6].

	h D/S							
h U/S	0	10.1	10.3	10.6	10.8	11.1	11.2	11.5
0	0	0	0	0	0	0	0	0
9.3	0	0	0	0	0	0	0	0
9.5	3	3	3	3	3	3	0	0
9.9	8.5	8.5	8.5	8.5	8.5	8.5	0	0
10	14	14	14	14	14	14	0	0
100	14	14	14	14	14	14	0	0

Figure 4 PS205 flow – level relationship

Pump Station 210 consists of one small pump and two big pumps with a combined maximum flow of 1.8 m^3/s. The model includes only the two big pumps due to the instability causing by the small pump. However, it should not affect the outcome as the combined maximum flow remains unchanged. As the documented levels was based on pre-quake information, the levels were

[6] The flow-level relationship is based on information provided by Council in email "RE:PS205 Q/H relationship" dated 21/1/2020 (LDRP097).

adjusted to post-quake level. The operating conditions used in the model are summarised in Figure 5[7].

	h D/S			
h U/S	0	1	10	100
0	0	0	0	0
8.96	0	0	0	0
9.3	1.8	1.8	1.8	1.8
100	1.8	1.8	1.8	1.8

Figure 5 PS210 flow - level relationship

Energy loss

Twelve “user defined” energy loss structures have been defined on the avon.dudley10, avon.stalban, avon.dudley20, avon.shirley and avon.dudleyBP branches. These are primarily associated with grates or screens located within the channel.

Static control structures

All static control structures including gates and fence crossings have been modelled as underflow structures. The majority are fences and the head loss factors adopted for these structures are shown in Table 8. These structures use the ‘unchanged’ calculation mode.

Active control structures

There are two active control structures which are the Kate Sheppard structure discussed in Section 4.7 and Winters control gate discussed in Section 4.1.

3.3.3 Cross sections

Cross-sections include both open and closed section types. For every cross section, the following information is required:

- River (branch) name
- Chainage
- Cross Section ID (optional)
- Topo ID
- Section Type
- Radius Type
- Coordinates, and
- Resistance.

Regular branches have been used for the entire network and cross sections are defined for every branch. The Topo ID’s and key cross section properties are specified below.

Topo ID

The model typically uses the Topo ID ‘combined’. This is because branch cross-sections have been based on a number of topography sources which are identified by cross section IDs.

Cross section properties

Table 11 displays the open cross section properties specified for the Avon catchment model.

A uniform relative resistance value of 1.0 has been set within the true bank markers for all cross sections in every branch. The Manning’s n parameter has been calibrated to a global value of

[7] The flow-level relationship is based on information provided by Council in dropbox on 28/7/15 (LDRP044).

0.045. There are a number of exceptions and these are described in Section 3.3.6 “HD parameters”.

Table 11 Typical open cross section properties

Property		Value
Section type		Open
Hydraulic radius		Total area, hydraulic radius
Coordinates		Coordinates of bank markers 1 and 3
Morphological model		Not used
Resistance number	Transversal distribution	Uniform
	Resistance type	Relative resistance

Closed irregular sections have been used to represent bridges as structures and some culverts. In these instances, the radius type used has generally been Total Area, Hydraulic Radius.

Closed circular and closed rectangular sections have also been used to represent culvert and longer pipe systems. In these instances, the radius type used has generally been Resistance Radius. The location of these closed sections is shown on Map 2 in Appendix A.

3.3.4 Open channel roughness

Roughness values from the previous models were retained in many areas as follows:

1. Avon Main River: downstream from CGHS to the estuary, avon.avon chainage 7406-27135, from the Avon D14 model

   Except for avon.avon 11967-12963 (Hospital to Armagh St) where the calibrated flow gauge roughness of 0.045 was used to replace the previous DHI model values of 0.029 and 0.048 (low/ high transversal distributed value respectively)

2. Northern areas draining into Horseshoe Lake via No 2 Drain and Old No 2 Drain: from the Avon D14 model
3. St Albans Stream: avon.stalban 0-5870 from St Albans model
4. Shirley Stream: avon.shirley 0-2549 from Upper Dudley Creek model
5. Upper Dudley Creek, Mairehau Drain (along Aylesford St) and Cranford Basin: various branches all upstream of the confluence with Shirley Stream, from the Upper Dudley Creek and Cranford model(s), and
6. Lower Dudley Creek (downstream from Shirley Stream): from the Lower Dudley Creek Model.

3.3.5 Boundary data

The Avon model include closed boundaries, constant flow, point source (Rorb) and constant water level boundaries. The total number of boundary conditions is 204.

As outlined in Section 3.2 of the Schematisation Report, base flow has been applied in the MIKE 11 component of the coupled model. These flows have been included at the branch locations summarised in Table 12 below and are based on information prepared by GNS. The location of these baseflow inputs is also shown on Map 4 in Appendix A.

Table 12 Base flow inflow boundary locations

Branch name	Chainage	Constant flow value (m^3/s)
avon.avon	1615.9	0.039
avon.briggsdr 0	0	0.050
avon.dudley 10	5401	0.100
avon.fendalst	1564.2	0.170
avon.judds	7.9	0.183
avon.langes	0	0.027
avon.madras	0.8	0.062
avon.okeover	419.9	0.226
avon.riccmain	2702.8	0.050
avon.taylor	0.3	0.121
avon.waiiti	0	0.303
avon.waimai	0	0.050
avon.wairara	0	0.056

Constant flow boundaries have also been set at another 118 locations. These are primarily inflows at the beginning of the branches with a constant value of 0.01 m^3/s. The main exceptions to this are at avon.ilamstrm 0, avon.oldmill 34.5 and esty.glenstrm 198.3 where inflows are set at a constant value of 0.05 m^3/s.

A constant outflow of 0.572m^3/s has been set at branch avon.rhodes (CH 1218.9). This location is at the boundary with the Styx catchment and has been derived using a pipe full assessment.

Dummy constant water level boundaries for initialisation purposes have been specified at 50 locations. Sixteen of these boundaries are associated with links to MIKE Urban and 33 are associated with standard links to MIKE 21.

In addition at the boundary with the Styx catchment a constant water level of 14 m has been set at the end of the avonstyx.horner branch (CH 2134.4). The culvert under Prestons Road adjacent to this boundary point has a soffit level of approximately 14.5 m and upstream there is a channel invert level of approximately 13.85 m. Based on these values, a boundary level of 14 m has been chosen as it is unlikely to generate significant flows into or out of the model (i.e. to or from the Styx catchment). This fixed level boundary will be removed once the integration of the Styx and Avon models occurs.

3.3.6 HD parameters

The Hydrodynamics (HD) editor is used for setting supplementary data used for the model runs. Most of the parameters are set to default values. For many parameters global values are set for the entire model, with local values specified in particular locations if required.

The following information is included within the HD Parameters file:

- Default computational parameters
- Initial conditions
- Wind factors (not used)
- Bed resistance
- Wave approximation
- Quasi steady parameters
- Heat exchange parameters (not used)
- Stratification parameters (not used)

- Groundwater leakage (not used)
- Flood plain resistance, and
- Encroachment simulations (not used)

These parameters are summarised in the remainder of this section.

Default computational parameters

These parameters are essential for the computational scheme and have generally been set to the default values. Table 13 summarises the values adopted within the model.

Exceptions to the default values are as follows:

Delta has a value of 0.8 (default = 0.5), Delta changes the time-centring of the gravity term in the momentum equation; a high value of Delta has a dampening effect which can influence model dynamics compared to the default value of 0.5 which will produce the most accurate calculations provided the model is stable.

Inter1Max has a value of 10 (default = 20). This factor defines the maximum number of iterations permitted at each time step to obtain a solution at a structure. FroudeMax and FroudeExp have default values of -1 in both cases. These factors define the formulation for suppression of the convective momentum term.

Table 13 M11 computational parameters

Parameter	Value
Delta	0.8*
Delhs	0.01
Delh	0.1
Alpha	1
Theta	1
Eps	0.0001
Dh Node	0.01
Zeta min	0.1
Struc Fac	0
Inter1Max	10*
Nolter	1
MaxIterSteady	100
FroudeMax	-1
FroudeExp	-1
Note: * indicates non – default values	

Initial conditions

Initial conditions have been set up based on the strategy summarised in Section 7.3.

Bed resistance (roughness)

Table 14 summarises the global bed resistance settings for the MIKE 11 component. A global Manning's *n* number of 0.045 for the M11 network has been determined through calibration.

Table 14 Bed resistance

Parameter	Value
Approach	Uniform section
Resistance formula	Manning (n)
Global resistance number	0.045

Parameter	Value
Bed resistance toolbox	Not used

There are a total of 723 local values specified for the bed resistance calculations, which take precedence over the global value. 160 values are associated with closed cross-sections as discussed below. Unless modified as part of the calibration process, the values typically reflect those extracted from previous models.

Local adjustments in the HD bed resistance parameters have been made for culvert/pipe systems modelled as closed cross-sections. To take account of inlet and outlet losses, including in some instances for stability purposes, all closed cross-section roughness values have been increased from the Schematisation Report values. This has been achieved by adding 0.01 to the default.roughness value tabulated in Section 6.9 of the Schematisation Report (e.g. if pipe material is concrete with a Manning's n value of 0.013, the bed resistance parameter included in the HD file is increased to 0.023).

Wave approximation

Wave approximations have been set globally for the entire network using a High Order Fully Dynamic approximation.

Quasi steady parameters

Quasi steady simulation computational parameters and steady state options for the model are summarised in Table 15. These have not been changed from the default values.

Table 15 Quasi steady parameters

Parameter	Value
Relax	0.5
Beta_Limit	1e-008
Fac_0	2.5
Qconv_factor	0.001
Hconv_factor	0.01
Min_Hconv_In_Branch	1e-005
Q_struc_factor	0.005
H_stop	0.0001
Steady State Options	Not Used

Floodplain resistance

A default value of -99 has been set for flood plain resistance. This value indicates that the flood plain resistance should be calculated from the raw data in the cross-section data base.

3.3.7 MIKE 11.ini file

The following variables have been changed from the default value in the MIKE 11.ini file:

- HD-Variables no 3: WL_EXCEEDANCE_FACTOR (default = 4): This has been increased to **25** to improve the likelihood that calculations will continue when the water level exceeds the banks of the open cross-sections including shallow LiDAR interpolated cross-sections, and
- HD-Variables no 40: FLOATING_POINT_EXCEPTIONS (default = off): This has been changed to **on** to enable problem detection during linked modelling (e.g. M11/M21).

3.4 MIKE Flood

The three model components are coupled using MIKE FLOOD, allowing the transfer of water between the two 1D platforms (MIKE 11 and MIKE URBAN) and the 2D platform (MIKE 21 FM).

Four link types have been adopted within the Avon model in order to link the MIKE 11, MIKE 21 FM and MIKE URBAN model components. These links are as follows:

- Lateral Links: these allow a string of MIKE 21 FM cells to be laterally linked to a reach in MIKE 11 (refer to Map 3)
- Urban Links with M21: these links are used to model the interaction of water between a piped network (MIKE URBAN) and the MIKE 21 FM mesh
- River / Urban Links: these links are used to model interaction between a piped network (MIKE URBAN) and the river system (MIKE 11) (refer to Map 2), and
- Standard links: these allow one or more MIKE 21 FM elements to be linked to the upstream or downstream ends of a MIKE 11 river branch (refer to Map 3).

The following sub-sections summarise the parameters and setup used for each link type applied within the model.

Table 16 to Table 19 summarise the parameters used for each link type used within the model.

3.4.1 Lateral links

The MIKE 11 network is linked to the MIKE 21 FM grid either side of the branches. Wherever there is a lateral link, the MIKE 21 FM model mesh is blocked out to avoid double calculation.

There are 1447 sections of lateral links between the river network (MIKE 11) and floodplain (MIKE 21). All links are hydrodynamic only.

Table 16 Lateral link parameters

Parameter	Value
Number of links	1447
Coupling type	HD only
Method	Cell to cell
Structure type	Weir 1
Source	Both M21 and HGH
Depth tolerance	0.05
Weir coefficient	1.838
Manning's n	0.05 - 0.5
Smoothing factor	0.1

In the model, 1109 of the links use M21 and 338 use HGH. Refer to Map 3 in Appendix A.

71% of the links have a roughness value of 0.05, 12% have a value in the range 0.07 to 0.07, 15% have a value of 0.1, 1% have a value of 0.5 with the remainder using a value of either 0.075, 0.8, 0.15 or 0.2. The higher values have been used to improve the computational stability of these links.

3.4.2 Urban links with M21

All links between MIKE 21 FM and the piped network (MIKE URBAN) are hydrodynamic only. Generally, the links connect the MIKE 21 FM grid to manholes in MU, representing sump inlets, bubble-ups and headwalls, or to an outlet in MIKE URBAN with one instance of a link to a MU pump station (PS 209).

Table 17 Urban link parameters

Parameter	Value
Coupling type	HD only
Inlet method	Orifice equation 43 M21 to inlet links representing headwalls use the weir equation inlet method[8]
Type	M21 to inlet – >15,000 M21 to outlet - 59 pump to M21 - 1
Discharge coefficient	0.98
Qdh factor	0.1 - 0.35

3.4.3 River/urban links

There are a total of 648 River/Urban links between MIKE URBAN and MIKE 11. All links are hydrodynamic only. Seventeen of the links represent flow from a MIKE 11 water level boundary to an inlet in MIKE URBAN (refer to Map 2 in Appendix A for locations), 623 represent flow from a MIKE URBAN outlet to MIKE 11 and the remaining 8 are links between MU Pump Stations (PS 203, PS 204, PS 207, PS 220, PS 229, PS 230, PS 231 and PS 232) and the M11 network.

Table 18 River/urban link parameters

Parameter	Value
Number of links	648
Coupling type	HD only
Type	M11 water level boundary - 17, MU outlet to M11 - 623, MU pump to M11 - 8

3.4.4 Standard links

There are 33 standard links in the model. There are 4 instances where depth adjustment has not been activated. These occur at the inlet and outlet structure branches associated with Winters Basin.

Table 19 Standard link parameters

Parameter	Value
Number of links	33
Coupling type	HD only
Momentum factor	0
Depth adjustment	Yes, 4 instances of no
Exponential smoothing factor	0.3-1

3.5 RORB inflow boundaries

The hillside area (Port Hills) is excluded from the DHI models as they are now modelled in RORB and its hydrology is used to generate inflow at the edge of M11, MU and M21 models. The RORB inflows are applied into each of the three M11, MU, M21 models.

These are applied in the M11 boundary file as point source boundaries, as network loads in the MU model and as source points in the M21 model. The RORB inflows applied locations were

[8] In June-July 2024, 43 urban links representing headwalls were added to the Avon model, using the schema approach described in Appendix R - Headwalls

identified as part of LDRP097 project RORB integration modelling task (refer Schema Report Appendix K).

The item names in the RORB .dfs0 file are defined to match the model type and location therein where the RORB inflow is applied.

In order to facilitate recognition and redundant reference cross checking, this RORB item name is entered back as the boundary descriptor in all the DHI models. This is done as the boundary ID in M11, network load ID in MU and source name in M21 model.

4. Integrated systems build

The following subsections present drainage engineering features and systems which are important to the network operation and which require integrated discussion of multiple model types (i.e. cannot be described adequately under any of the single standalone model types (MU,M11,M21,MF) - i.e. preceding section 3.

4.1 Paparoa Diversion

The Paparoa Diversion detail is modelled in M11 by adoption of previous work by Opus (as supplied to GHD in Jan 2016). This work typically reflected pre quake levels so corrections to post-quake invert levels were applied per CCC survey, the diversion pipe diameter was corrected per CCC advice to 1350 mm and a headloss factor error was corrected per Opus advice. We understand these improvements were also replicated at the same time by Opus in their ongoing modelling of the area. This diversion then continues through a M11 to MU link - M11 water level boundary and on through the MU pipe network to *avon.papanui* @1002.59. This second flow split is less important to gross flooding outcomes and has not had the customised attention as that given to the Paparoa Diversion location.

4.2 Winters Basin

Winters Basin is a large flood attenuation basin connected to the south of Winters Drain (south of QEII Drive). The major elements included in the basin schematisation are shown in Figure 6.

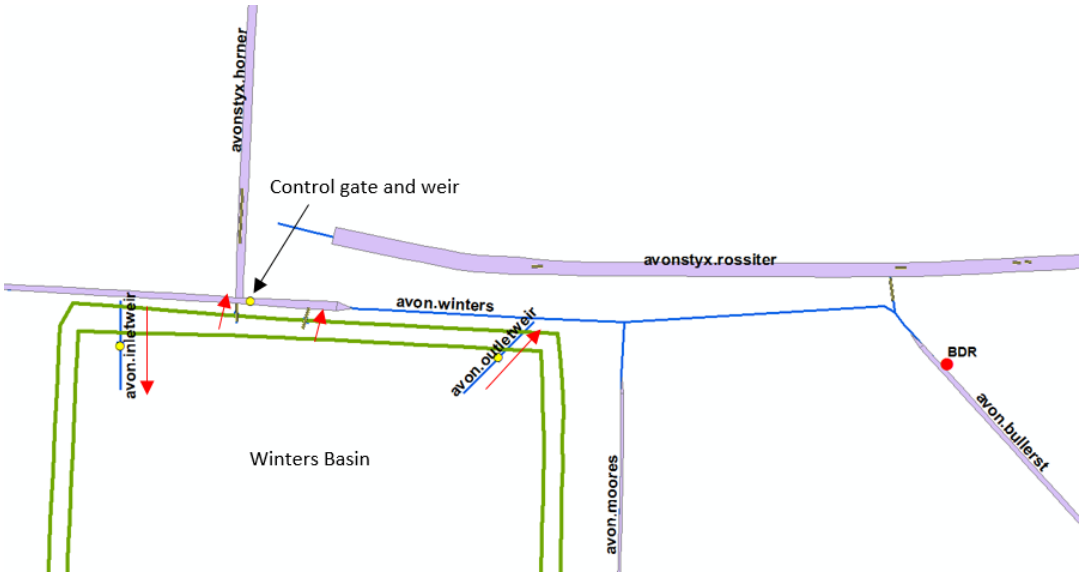


Figure 6 Winters Basin schematic

In normal conditions flow toward Winters Basin arrives from the west along *avon.winters* branch. The southern portion of *avonstyx.horner* branch can also flow south toward Winters basin[9]. In low conditions this flow continues east along Winters Drain and downstream to Bullers Stream.

On this eastern portion of Winters Drain and adjacent to Winters Basin, Winters gate[10], is normally either partly closed or fully closed and thus retains floodwater to the west, directing it north to Horners Drain, and throttling flows to the east (toward Bullers Drain).

In moderate flood conditions water levels in the western part of Winters Drain overflow into Winters Basin (above RL14.34) through a weir modelled in M11. Water is retained in the basin

[9] Flow direction in the southern portion of Horners Drain can change depending on specific conditions at the time
[10] Winter's gate is sometimes referred to as Horner's gate

until flooding subsides enough to drain back into Winters drain. There are two places for this receding flow (both M11 culverts), the first with large capacity drains to the west of Winters gate and the second with small capacity drains to the east of Winters gate.

The two low level outlets are one box culvert and one pipe, both with flap valves, serving only to drain the basin after use in a flood event. The box culvert is larger and drains north toward Horners Drain and the Styx catchment (upstream of Winters gate). The smaller pipe connects downstream of Winters gate into the eastern part of *avon.winters* and continues to *avon.bullers*. The pipe also has a throttling valve at the basin end, which is typically partly closed. The upstream cross section of this M11 culvert has been modified to mimic the shape of the estimated throttling valve opening.

In sufficiently large flood conditions, when Winters basin gets full (above RL 14.5) it will overflow into Cranford basin, modelled with the M21 dike (reverse overflow direction can also occur but is atypical). In the most extreme flood conditions Winters basin when overfull it can also overflow out of the basin to the north and back into Winters Drain east of Winters gate and flow then on to Bullers Drain (above RL 14.6). This outlet weir is modelled in M11.[11]

Model geometry for the two M11 inlet and outlet weirs are defined from survey data supplemented by understanding from previous models, site visit, photos and LiDAR[12]. The M21 dikes forming the basin perimeter are continuous across these weirs, but the dike levels follow the top of bank levels (RL 15.0) to prevent double counting of weir flows.

Outside of the basin in the M11 system Winters gate is modelled as an active controlled structure. The gate is modelled 6m downstream of the Horners drain junction (avon.winters-1740)[13]. The normal gate position is partly closed with a 200mm vertical opening. This gate is setup to automatically close, when water levels within Bullers Stream exceed the trigger point of (13.90 m) to reduce the volume discharging towards Bullers Stream, which lowers the potential for downstream flooding.[14]

The Buller Stream water level is monitored by a sensor close downstream from Lady Issac Way, modelled at Avon.Bullerst-721.02[15]. The gate returns to its normal partly closed position, once downstream levels within Bullers Stream fall below 13.70m. This reopening condition is not important for flood levels and has not been modelled.

There is a high-high level overflow across the top of Winters gate modelled at RL 14.9m for extreme flood conditions, although the lower and larger flow path through Winters basin inlet and outlet weirs should dominate and render the gate overflow capacity trivial. The top of basin bank heights around both Winters and Cranford Basins are equal at RL 15.0.[16]

[11] All features in the vicinity of Winters Basin are modelling in M11 and M21 with no use of MU elements.
[12] Sources for geometries of the M11 weirs, culverts and gate are recorded in the 'Special data sources' shapefile.
[13] Winters gate is modelled at the historic position and does not reflect the more recently As Built location downstream closer to the small outlet pipe
[14] Winters gate control structure follows the specifications defined in the Cranford run 30 model and LDRP 503 Cranford Basin Active Management Design Report
[15] This same remote water level sensor is used to inhibit PS0219 in Cranford Basin
[16] The basin top of bund and M21 weir levels are taken from LiDAR and so vary slightly around the perimeter. This closely approximates the design levels of 14.5 and 15.0 reported above.

4.3 Cranford Basin and PS0219

Cranford Basin is a large retention basin, with the only practicable outflow being pumped through PS0219 (Philpotts pump station). The basin primarily fills from the southwest. In sufficient floods The Dudley Creek catchment overflows into the Paparoa and Dudley Diversions, entering Cranford basin at a major M11 culvert under Cranford St (1.8 x 1.8m twin box culverts) into the southern corner of the basin. This culvert has no flap valve and flood levels southwest of Cranford Street are generally expected to match water level in Cranford basin.

Figure 7 shows an overview of Cranford Basin, PS0219 and key elements of the Cranford active management plan.

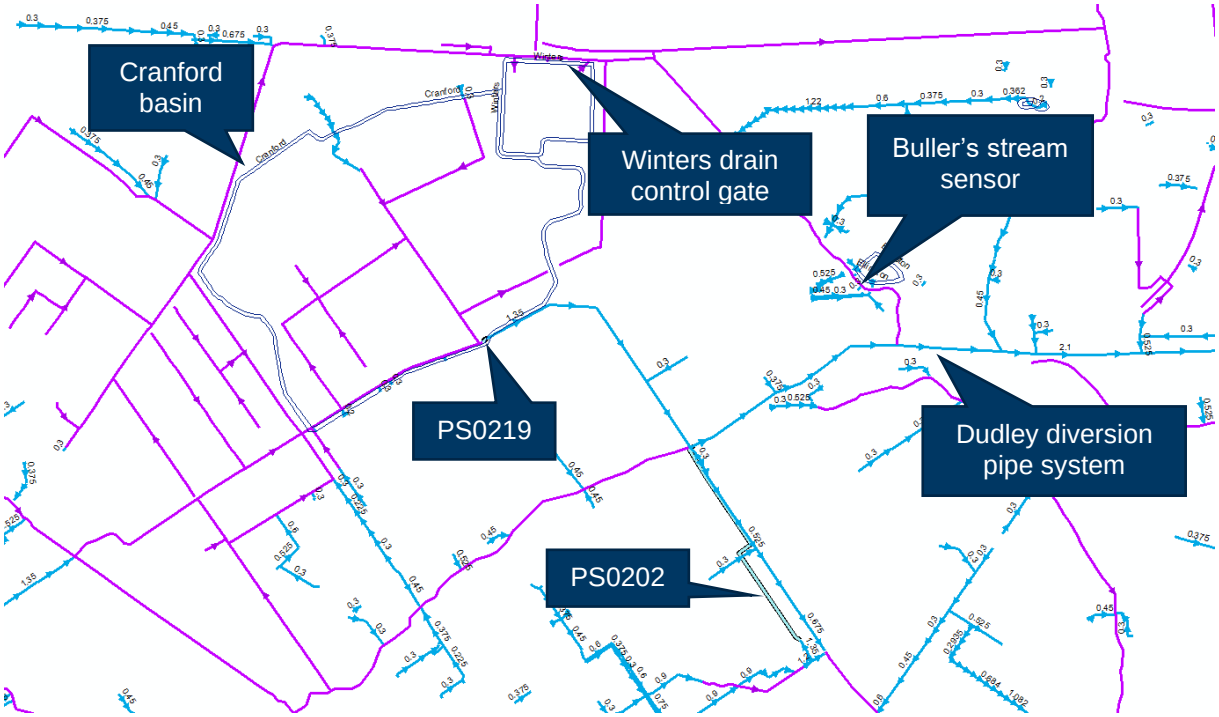


Figure 7 Cranford Basin, PS0219 and key active elements

Note: M11 shown in purple, and MU shown in light blue

In large enough rain events, after Winter's basin fills, large volumes can overflow from Winters Basin via an overflow weir (modelled in M21) into Cranford basin.

Minor flows also enter the basin through seven culverts, all modelled in MU and with flap valves (although noting that when the basin is full the marginal areas draining into the basin do not have alternative drainage and will therefore typically also flood to match the basin level).

Within the Cranford basin there are several M11 open channels which drain towards PS0219. PS0219 is a large capacity flood pump station that is the sole outlet from the Cranford basin. PS0219 is modelled in MU, as is the 1350 mm diameter discharge pipe from the pump station which is explicitly modelled.

The PS wetwell is modelled as a large box structure (6.8 x 3.9 x 10.0m) in MU and the pumps here are modelled individually to enable the active management. The pumps otherwise follow the standard MU model pump schematisation with continuously variable flow (rather than on//off flows). For stability reasons the downstream end of M11 drain avon.duddlydiv has been modified to match the wetwell geometry.

The pump station lifts the flow into the downstream gravity discharge system via a pump discharge chamber. Some surcharging can occur at the chamber and such surcharging is

provided for in the model with the standard manhole configuration allowing surcharged conditions. This surcharging is confined to the MU domain (i.e. there is no MU linking to M21 at the pump station). Partway down the discharge pipe there are several sump inlets which follow the standard model schematisation approach. These allow M21 flow into the pipe (and if suitable hydraulic conditions could ever occur would also allow reverse flow to spill out to M21).

There is no secondary outlet from the basin and in extreme high flood conditions with the basin full, additional flow will be unable to get into the basin and will drain through alternative flow paths around the basin. If peak flooding in gets close to the top of bund (15.0) then receding flood volumes could flow north via Winters Basin and out the Winters Basin outlet weir at 14.6m.

Cranford basin is part of the "Cranford active management plan", which seeks to actively increase retention of floodwaters in the Cranford basin storage and thus protect areas downstream which are subject to flooding. To achieve this, in specified conditions one of the two large pumps (AVON.Pump.CCCGIS.0219_3) is inhibited (and later uninhibited) by the water level sensors at Buller's stream and within Cranford basin.

The Bullers stream sensor is just downstream of the bridge for Lady Isaac Way and the Cranford Basin sensor is close upstream of PS0219 on Crozier Drain. Both sensors are modelled in MU with the Crozier Drain sensor at the PS0219 wetwell (MUID:Avon.Cranford.CCCGIS.219).

The Bullers stream sensor for PS0219 control is modelled at dummy node MUID:Avon.Node. Dummy.900002 connected by a dummy pipe to M11 avon.bullerst-737.961. This is within 10 m downstream of the real sensor location on Buller's stream. Note that this means there are two separate model representations of the single real Buller Stream sensor, the first in M11 to control Winters gate and the second in MU to control PS0219. The two model elements are closely connected hydraulically and should therefore always report essentially the same water level.

The control conditions and levels are as follows.[17]

Buller's Stream Water Level ()		
14	>=	Inhibit PS0219 (3)
13.8	<=	UnInhibit PS0219 (3)
Cranford basin (Crozier Dr – near the bifurcation point to Dudley Div. channel)		
14.6	>=	UnInhibit PS0219 (3) Basin high alarm (i.e. override any Bullers Stream inhibition)

The Cranford basin active management scheme also has backup controls in case of sensor or communications failure. The model only represents normal operation.

Christchurch Northern Corridor (Motorway) crosses through the Cranford basin. Modelling of this road and its associated culverts and local drainage is broadly functional but remains incomplete due to unavailable As Built information from the active construction project. Details of this are noted in the anomalies registry (entry 67). Generally, precautions were taken to leave the incomplete parts of the model build functional but also obviously not realistic.

The region around Cranford Basin has several locations where primary and secondary flows progress in different directions[18]. Flood outcomes can be highly sensitive to model schematisation at bifurcation points as well as bank spilling levels. This means that a high level of precision and detail in the model input data would be appropriate to ensure confidence that

[17] Taken from the Cranford active management plan "operation and maintenance manual".

[18] Key channel bifurcation locations include Paparoa Diversion (avon.dudley10 atCh7760) and further along the Dudley Creek Diversion scheme at avon.papanui at1002.59. The spilling flows from avon.dudley10 near Jameson Ave and performance of the inlet grating at the downstream end of avon.dudley10 at10036 (modelled clean with a M11 to MU link - M11 water level boundary type) are also crucial to flooding outcomes. The volume and depth in the basin will also be influenced by numerous features and channel overflow functions in the Winters and Cranford Basin systems.

the model function replicates reality. Within this project, this area has not had such special treatment other than as described above.

4.4 Shirley flow split

This section describes the model set up of the bifurcation at the junction of Old No.2 drain and No.2 drain immediately north of QEII Drive and the Shirley Golf Course and schematisation associated with Old No.2 drain.

A weir on No.2 drain directs low flows to the north-west side of Horseshoe Lake via Old No.2 drain (M11 branch *avon.oldno2dn*). This weir is included in the *avon.no2drn* branch as a M11 broad crested weir. Details (level and width) are based on April 2011 survey data and site photos.

Old No.2 drain discharges to Shirley Golf Course via a 1.8 m diameter pipe culvert under QEII Drive.

A flow - level gauge (ID 66624) installed in Old No.2 drain within Shirley Golf Course immediately upstream of Lake Terrace Road provides a continuous record and historic data has been used for calibration purposes. The cross-section at the gauge location is a constriction and has been included in the M11 .xns11 file based on March 2017 survey.

In general it was noted that survey of the Old No.2 branch had quality issues, particularly with regard the QEII Drive culvert and open channel sections within the Golf Course. The downstream invert of the QEII Drive culvert was dropped to provide a 0.1 m fall along the culvert length and open channels in the reach approximately 200 m downstream of QEII Drive were widened to better reflect the available channel capacity. These changes have been noted in the Avon Model Anomalies Register included in Appendix F.

4.5 Stopbanks

The Avon stopbanks system is modelled using the M11 cross sections in most places with bank marker elevations set at the stopbank crest height and HGH lateral linking type making this effective. In areas where the stopbank location is remote from the river bank the stopbank function is modelled using M21 dikes.

Stopbank topography has been erased from the M21 ground model to ensure that the stopbank function is only modelled in the above elements. Stopbank levels in the design model are mainly set at levels surveyed in Feb 2020. Some locations such as Southshore and a section of bank near Horseshoe PS0205 have been modelled with either design or As Built levels as noted in GIS data records19.

The stopbanks logical extents are defined as being continuous starting from downstream of Swanns Road to Bridge Street, although in several locations high ground means that there is no or negligible real stopbank topography. This extent includes all river banks near Porritt Park and a short length of the avon.horseshoe branch up to the PS205 weir.

In order to model stopbank down scenarios (for MPD or other purposes), the intention is to revert all lateral link types along the extent of the stopbanks from HGH to M21 type to ensure flow interchange at the low M21 ground levels. Where dikes represent the stopbanks, they would be disabled. There are 24 segments of lateral link set to HGH for stopbank function and 5 segments of dikes (all downstream from Wainoni Road and all but one on the true left bank). Where dikes are used for stopbank definition (represented in M21), the M11 bank markers are

[19] \CCC Master Models\03_Avon\07_GIS Data_GHD_fullset\Data\02.ModelInputData_Avon_M21.gdb\ CW_Dikes_SB_Lines_20210819 and CW_Dikes_SB_Points_20211006

located well inside the dike location ensuring the used part of the M11 cross section stops well short of the stopbank location.

Throughout the extent of the stopbanks, flap valves are included in all modelled pipes (both MU and M11) by assumption wherever data confirming their real existence was lacking. Flap valves inserted by assumption (as opposed to from data / knowledge) in MIKE Urban can be identified by having a description in the link attributes of “Assumed flap gate”.

4.6 Horseshoe Lake and Pump Station

Horseshoe Lake provides the downstream extent of a number of drainage systems including No.1 drain, Old No.2 drain, No.2 drain/Snellings drain and the Dudley Creek diversion. Inflows to the Lake can gravitate to the Avon River via one-way tide gates, however when high water levels in the Avon River prevent this from occurring, two pump stations provide the means to lift water from the Lake to the Avon River.

Locksley Pump Station (PS210) is the smaller of the two stations and is located on the old outlet from Horseshoe Lake to the Avon River (avon.oldlake M11 branch). The gravity outfall is flapped and is modelled as a single one-way rectangular culvert. As outlined in Section 4.1.1, the pump operation is modelled as a tabulated structure. Horseshoe Lake pump station (PS205) is the larger of the two and is located on the main Lake branch north of PS210. A barrage structure across the Lake contains 5 tide gates incorporating flap valves fitted on the River side to prevent backflow into Horseshoe Lake. These are modelled as 5 one-way rectangular culverts. A weir structure is also included to represent the walkway access over the top of the tide gates. The pump station includes 3 Archimedes screws whose pumping capacity is dependent on water levels in both the Lake and the Avon River. This pump operation is included as a tabulated structure in the M11 network.

4.7 Lake Kate Sheppard

Tidal circulation in the Kate Sheppard Lake/Travis Swamp system has been schematised with the inclusion of two flow regulation structures and a control structure, all represented in green in Figure 8 below.



Figure 8 Schematisation of Travis Swamp tidal circulation

The structure on the avon.corserst branch (Ch 958) has “only positive flow” regulation, to ensure that no flow from the Avon River discharges back into the branch. The structure on the avon.katshep branch (Ch 960) also has “only positive flow” regulation, to ensure that no flow from Travis Swamp (the avon.travissw branch) discharges back into the Kate Sheppard Lake system. In addition the model schematisation includes a control structure on the avon.katshep branch (Ch 45) which allows flow to and from the Avon River when levels in the Avon River are less than 10.6 m. When levels in the Avon River are at or above 10.6 m, the control structure closes, preventing flow in either direction. The Kate Sheppard control structure uses the ‘set equal to’ calculation mode to simulate the gate closure when the water level at chainage 22950 on the avon.avon branch reaches 10.6 m. An anti-clockwise tidal circulation is therefore induced, with occurrences based on water levels in the Avon River.

The actual connectivity between the Avon River and Kate Sheppard Lake consists of twin box culverts under New Brighton Road with stoplogs permanently installed at the river end of the culverts and an intermediate structure containing the twin control gates. The control structure is a simplification of this connectivity, representing both the culvert conveyance and gate function. In addition, the sill level of the control structure has been set at the top of stoplog level.

4.8 Layout near Saint-Joseph School

It was found necessary to make an exception to the Model Schematisation Report near St Joseph's School. Council GIS pipe flow direction in the vicinity of the school at 4 Vagues Road, Papanui could not be rationalised. Figure 9 shows the pipe layout/flow direction currently included in the Council GIS with Figure 10 presenting the MU model schematisation.

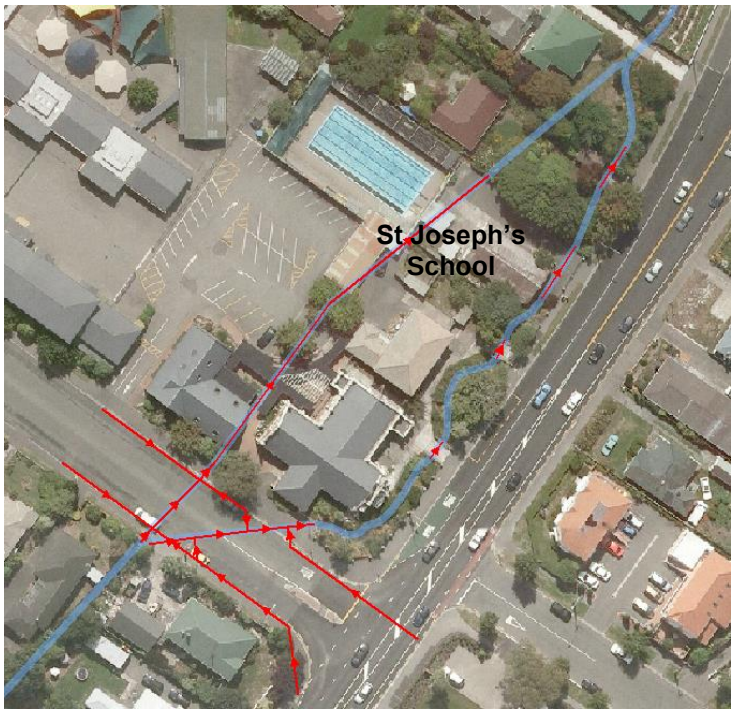


Figure 9 St Joseph area – Council GIS

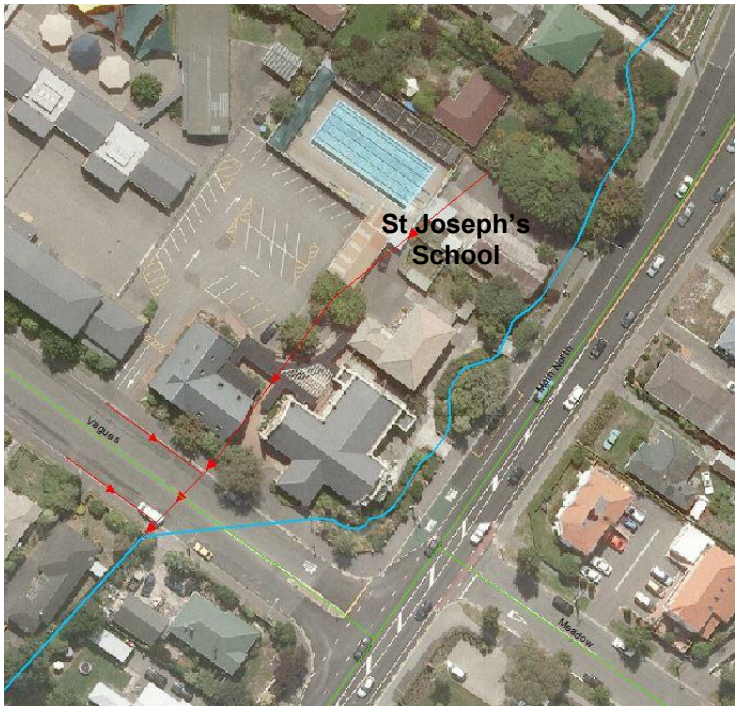


Figure 10 St Joseph area – MU model layout

The short length of waterway included as Kruses Drain in Councils GIS and on Figure 9 forming the western branch of the "loop" does not appear to be a formed drain or waterway based on the post – quake LIDAR cross-section which is the only source of information in this area. The

flow direction has therefore been reversed in the MU model build with connectivity to the M11 model on the southern side of Vagues Road as shown in Figure 10.

4.9 Papanui Stream

Council data for the Horner Street Drain system adjacent to Proctor Street connecting to Papanui Stream was initially found to be inconsistent and incomplete. This motivated survey work which was completed during the project (13/06/2016). The system has been modelled in the MU domain. The MU model schematisation includes property access structures, where the culverts have been modelled as rectangular pipe systems, in accordance with the survey information provided.

4.10 Estuary

The Estuary including McCormacks Bay has been included in the Avon model connecting to the Avon River just downstream of Bridge Street and the downstream extent of the Heathcote River at Ferry Road.

The Estuary extents are incorporated in the M21 domain with post-quake mesh levels based on bathymetric survey undertaken in March/April 2011 supplemented with bathymetric survey of the southern part of the Estuary carried out in June 2013. These surveys were provided by NIWA and included in their Delft3D hydrodynamic model of the Avon Heathcote Estuary prepared for Environment Canterbury.

As discussed in Section 4.5.1 the Sumner Heads tide is applied at the Estuary outlet open boundary. A review of tidal timing using a M21 Estuary model only indicated there was approximately a 1 hour time lag between the peak tide at the Estuary mouth and the peak tide at the M11 boundary location just downstream of Bridge Street. This lag has subsequently been applied when assessing initial conditions at Bridge Street for individual design storm events.

A nominal flow into the Estuary from the Heathcote River has also been applied as indicated on Map 4 in Appendix A.

In addition, review of historic tide level differences between Ferry Road (Heathcote) and Bridge Street (Avon) in a southwest wind situation indicated peak tide levels at Bridge Street were generally higher than at Ferry Road. On this basis, a constant design wind across from the southwest has been applied across the Estuary 2D domain for all Avon River design storm events.

4.11 Lower Dudley Bypass

The lower Dudley bypass is a link that provides a shortcut from Dudley Creek into the Avon River. The project that built the bypass also upgraded channel capacity upstream. The map extents of model update project work were defined as being the bypass, the M11 open channel east of Hills Road and south of Shirley Road plus approximately 100m of open channel upstream from the Hills Shirley Road intersection. The downstream extent on the historic channel ended at the Petrie St bridge (just below the diversion entrance).

BECA were the lead designer of this project and supplied their design model (Lower Dudley model v29). This model was never updated to formal As Built condition but based on advice from Beca and Council it was considered to be a sufficiently good representation of the As Built condition[20]. Master model updates then copied in the Beca model into the Avon master model.

[20] Tim Preston's email "Dudley creek models - conversation record and actions from Tuesday" 11/12/2020 and Kate Purton's reply 24/12/2020 and then Tim Preston email to Council "Dudley creek models - the As built WP7" 11/1/2021

The design model had roughness applied to the cross sections. This was preserved in copying and overwrites the hd11 roughness. This is an exception to the usual schematisation using hd11 roughness.

The design model did not spatially align the branch to its new form. When corrected this resulted in chainage distortions, so a number of culvert and lateral link chainages had to be readjusted. Chainage correction was pursued to downstream of Petrie Road, where a 5m wrinkle was left in the model as noted in Anomaly 90. The design model Dx = 100 was preserved.

As part of the spatial conformance the mesh blockout and lateral linking was adjusted.

The avon.dudleyBP branch has three energy loss coefficients and one weir. The weir structure is at the start of the bypass to control the level that flow starts to enter the bypass system. The three energy loss coefficients are to reflect the losses at the entrance, the bends and at the outlet of the bypass.

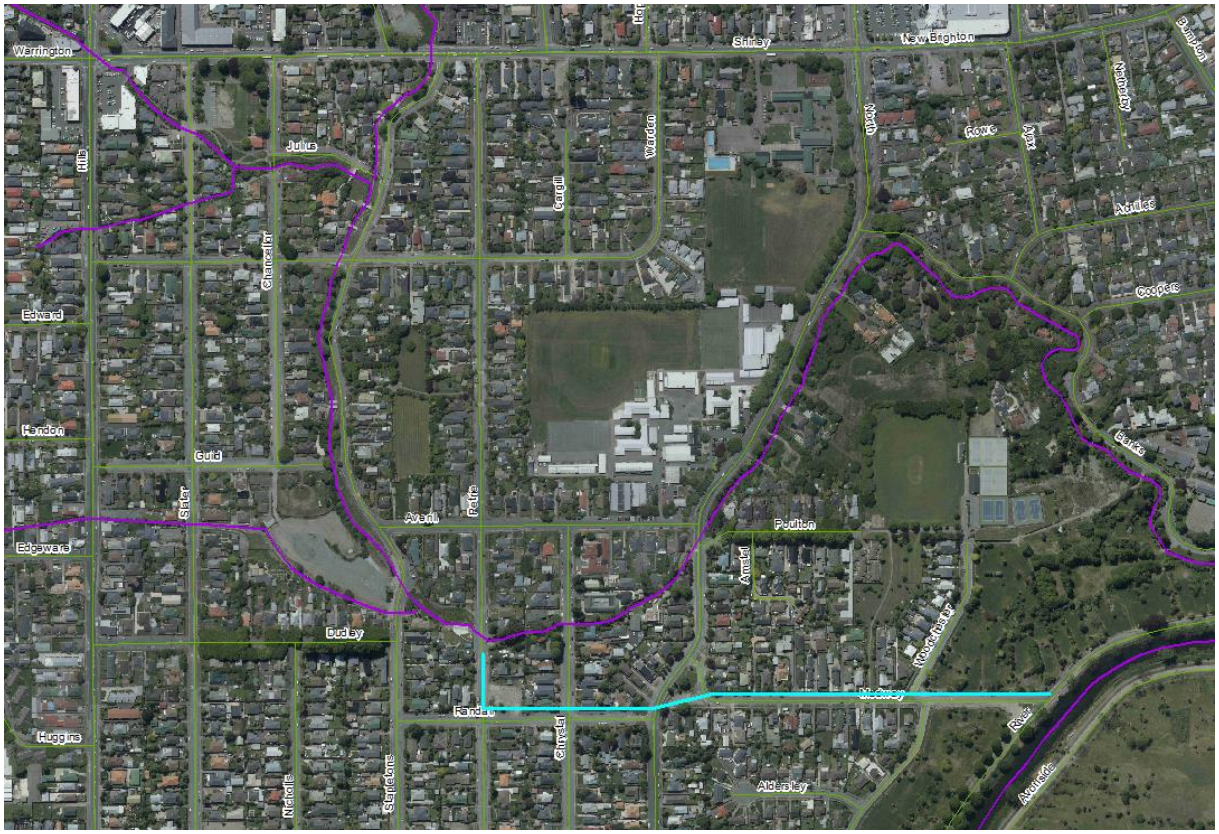


Figure 11 Lower Dudley Bypass

4.12 New Preston subdivision

The new Prestons subdivision has been partly modelled due to limitations on readily available information as recorded in Anomaly 74, Appendix F.

The subdivision new piped network is modelled in MU and model ground levels were updated to 2018 LiDAR. No model updates were done to the open channels, nor any mesh triangle revisions to take into account the new roads, basins or blocking out open channels.

In order that the pipe network can function all the MU outlets are linked using long fictional MIKE Flood urban to river linkages to Snells Drain (avon.snells-1841.6).

The Burwood hospital basin is modelled in M21 ground levels (due to the 2018 LiDAR) and with the standard top of bund M21 dike. However, no mesh triangle modifications were done so the basin geometry is only represented crudely, and the mesh does not align with the top dike.

4.13 Riccarton railway and Kyle St

Through review of as-built and CCTV data, coupled with site visits, we found several discrepancies between the CCC GIS data and the existing infrastructure. The model has been improved to reflect the existing conditions, with details as noted in model log and the Riccarton Report in Appendix Q. Strictly speaking this varies from schema approach (which does not plan for this level of detailed manual scrutiny, sump connectivity by assumption nor connecting of headwalls). The major areas of difference to the CCC GIS data, which have been corrected in the model, are the culverts under Picton Ave and part of Deans Ave where box culverts are in place whereas GIS shows circular pipes.

4.13.1 Missing inlets

Through fieldwork we identified six inlet sumps in the commercial area north of 10 Kyle St which are not present in the current GIS. We made connections for these and 15 sumps present in GIS which lack clear connectivity (and hence were not previously modelled). We have used reasonable assumptions to define and model this connectivity. We found and connected two new sumps in the current GIS from roading modernisation on Picton Ave. Full pipe capacity headwall connections were also added at the 450dia stub headwall behind the NIWA office and at the 600dia pipe end on Kiwirail land further south toward Placemakers.

4.13.2 Simplifications

In the model, pipe lengths under 10m are automatically represented by MIKE as 10m segments, which can significantly impact results, particularly in areas where several short pipes are connected in series (e.g. 3m + 2m+ 4m represented as 30m). This motivates manual simplification of existing pipes in such areas by merging pipe segments prior to incorporation as model inputs, to develop a more realistic model of the network. At the locations of Picton Ave and Deans Ave, pipes were identified that were much shorter than 10 m, i.e., less than 4 m in length. This was resolved by merging pipes and adjusting some inlet couplings.

There is a wider issue with short pipe sections as recorded in the anomalies register (item 95), with the main concern being short pipes that haven't been merged to suitable pipe lengths.

4.13.3 Invert levels at Picton Ave

An analysis of the invert levels at the headwall of the culvert across Picton Ave was conducted by comparing the as-built drawings[21] and the survey data[22]. The analysis revealed that invert levels in model links AVON.Wc.avon.riccmain2038.5, AVON.Pipe.CCCGIS.18061, and AVON.Pipe.CCCGIS.18056 needed to be updated to align with the as-built drawings. These updates were implemented through the Avon Modelling Improvements project.

4.13.4 Open channel

The open channel from Clarence St to Picton Ave was updated to align with newly available survey data. Two types of survey data were available: cross-sections of the open channel[23] and diagrams of the bridges/culverts[24].

[21] GHD reference: N:\NZ\Christchurch\Projects\51\12625709\03 Tech\1. Riccarton Railway\12625709 RiccartonDataAnalysis.xlsx
[22]GHD reference: N:\NZ\Christchurch\Water\Modelling\CCC Master Models\01_Citywide\07_GIS Data_GHD_fullset\LDRP044 Survey
[23] GHD reference: N:\global\Archive\51\2018\Christchurch\Projects\51\33275\01.Stage 1 Data\GIS\Data\09. Council Project Survey Data.gdb\Cross_Section_Survey
[24] GHD reference: N:\global\Archive\51\2018\Christchurch\Projects\51\33275\00.2Document Transfer\Incoming\20150612 Avon area A survey Mark to Kate\Christchurch City Council Area A Catchment Survey\Area A Diagrams

Adjustments were made to many cross-section widths and invert levels, as well and culvert / bridge dimensions. Some bridges required simplification due to short lengths.

4.13.5 Bridge geometry correction

In addition to the model improvements along the Riccarton Main Drain, the geometry of two bridges over the Avon River, at Harakeke St Bridge and Straven Rd Bridge, were corrected. The culvert geometry for these bridges was incorrectly schematized during the model build, resulting in artificial flow constriction within the model at these locations. The culvert geometry at these two locations was updated to reflect survey data received from Council in 2015[25].

4.14 Railway ballast modelling

Near Kyle Street at the railway embankment, coupled 1D modelling features were added to model estimated flows through the railway embankment. The maximum flow rates were 1.6 m^3/s per 100m at max 0.5m depth in accordance with the WSP memo[26] "Modelling Railway Embankments – For Discussion" dated 03/08/2023.

The key 1D modelling feature was a weir linking across the embankment between the low ground on each side. These links required a polygon definition in M21, MF linked to a dummy manhole at each end of the weir. Dummy pipe links between the manholes were also required for compliance with DHI MU2020 rules prohibiting isolated nodes. Key parameters were as follows:

1. Weir crest level 0.5m below the ground level in the centre of the tracks (lookup from LiDAR)
2. Weir width – 4m for a 200m length of M21 coverage
3. Max length of M21 polygon – 200m
4. Max vertical range of centre track ground levels within the length of a single polygon – 100mm
5. MU weir coeff – null (this setting is important to avoid disrupting the default DHI flow formulae)
6. MF Qmax of 3.2m3/s and inlet area of 3.2m2 and QdH = 0.1m
7. Dummy links with 0.010m dia and otherwise default or null values

Initially four weirs were set up along approximately 600m length of embankment, but the performance of three of these weirs was unsatisfactory (these weirs were often 'dry' and the model didn't seem to handle that condition rationally). The model was therefore reverted to the single weir at the lowest point, which generally would take most of the flow and showed rational model results.

This method is presently (as of May 2024) still experimental, with further work anticipated by DHI to develop it into a more generally suitable method that could be applied confidently to all railway embankments for a range of wet or dry conditions.

4.14.1 Cess drain

The mesh behind the NIWA office near Kyle St was lowered to better represent the drainage provided by a cess drain adjacent to the railway line in that location. This allowed water to flow from the edge of the NIWA carpark paving down to the 450mm stub pipe. The 'z_final' mesh

25 These bridges are labelled as R130 and R131 in the Christchurch City Council Area A catchment survey.
26 N:\NZ\Christchurch\Projects\51\12625709\03 Tech\1. Riccarton Railway\20240318 Railway embankment flow\20230803 WSP Rail Embankment Modelling Approach - Revision 2 TJP markups.pdf

triangle, which contained the stub pipe, was selected and the ground level was adjusted to match survey information. The ground levels of the mesh triangles between the start of the drain and the 450mm stub pipe were adjusted so that each triangle has a lower ground level than the previous one. This modification facilitates water flow into the stub pipe.

4.14.2 Railway mesh cells

The railway mesh cells were lowered back to LiDAR max of radius values (by reducing by the 150mm which had been added according to the Schema Rev7 methodology). The old rationale for the 150mm was to reflect the top of railway track level above the ballast level. We have since realised that the max of radius LiDAR levels setting often finds items higher than the base of the rail track, and that the track as a functional barrier to flow is negligible due to the highly permeable ballast.

5. Model calibration

5.1 Introduction

The Avon model was calibrated to both March 2014 and June 2013 events during the LDRP044 project. This calibration effort is outlined in detail in Rev 6 of the status report, specifically within Section 5 and Appendices E, M, and N of the report. Major aspects of the calibration sections from Rev 6 are still relevant but due to declining relevance, the contents have been removed from more recent versions of the status report. Appendix E from Rev 6 of the status report is included in this current version of the status report, and discusses the design vs. calibration model setup.

Due to the subsequent change in the infiltration approach in the M21 model, the Avon model was recalibrated to March 2014 event and the results were presented to Council as part of the LDRP097 project. The March 2014 calibration outcome from LDRP097 project is documented in “M21 Infiltration with Groundwater Capacity Report”, which can be found in Appendix L of the Model Schematisation Report. Calibration results for infiltration losses are summarised with design event infiltration losses in Section 6.2.

As a part of the current project (Avon River Modelling Improvements), the March 2024 calibration model, of the March 2014 flood event, was updated to include several of the improvements that have been incorporated into the design model since the calibration effort completed for the LDRP097 project. The results of this updated model were presented to Council[27]; however, the model was not recalibrated. The March 2024 calibration model updates and results are summarized in in Section 5.4.

5.2 Model geometry changes

There were several base model geometry changes required for calibration to the March 2014 event conditions. As described in Appendix E, blockage geometries in the main river were applied at Armagh and Colombo Street. Also described are adjustments to stopbank crest levels which were made on the true left bank of the Avon near the confluence with the Horseshoe Lake branch mainly to coincide with Council staff personal observations of stopbank overtopping locations and depths. This change also improved the level calibration at site 66611 (ALA).

[27] See Madeline Howell’s “Avon Calibration Results Update” emails to Jarad Sinni and Peter Christensen (cc Sheryl Keenan, Tim Preston, and Susan Willis) dated 12/7/2024.

In all other areas the stopbank top levels for calibration were set to design levels (11.2m in the lower reaches) and were found not to overtop in model results and from field observations.

5.3 Open channel roughness

Starting values for open channel roughness were assigned in accordance with the schematisation methodology. This is summarised as four locations with NIWA 2004 measured roughness, areas where credible previous model roughness was adopted (refer section 3.3.4) and otherwise a global starting roughness value of 0.030.

Based on initial calibration results the global default roughness was adjusted to 0.035. At the four flow gauging sites, roughness calibration was then used to determine local roughness values in order to match the measured NIWA rating curves. These outcomes were assessed collectively as presented below to determine a better global calibrated roughness value of 0.045.

Table 20 Avon flow gauge roughness values

Branch location	Chainage	Initial roughness	Final roughness	Factor change
Bullerst	0-982	0.04	0.046	1.15
Oldno2dn[28]	672-793	0.050, 0.013	0.072, 0.019	1.44
Kruses[29]	643-3281	0.040, 0.013	0.050, 0.016	1.25
Avon[30]	11967-12963	0.029	0.045	1.57
Average				1.35

The final channel roughness at Kruses Drain is abnormally high for a concrete lined drain even with protruding internal framing. This was necessary to match the NIWA rating curve and casts some doubt on that rating curve. It may indicate a significantly unclean rating condition (such as a silted channel or blocked grating downstream).

While the applicability of any of these sites for extrapolation across ungauged areas is less than good for various reasons, it was the only data that we had available for the purpose. The derived average factor of 1.35 was then applied to increase the global default roughness from 0.035 to 0.045. The calculated roughness would have been 0.047 however we felt this was stretching the credible range for typical roughness and determined the adopted value of 0.045 accordingly.

This new global roughness value of 0.045 replaced previous roughness values everywhere in M11 except for locations that were determined from previous model roughness and the at the flow gauge locations (branch reaches listed in Table 20 above). This replacement includes at the four NIWA 2004 locations (the NIWA results were dated, not well reported for our purposes and in general erratic).

[28] The two roughness values downstream of Old No2 gauge relate to the open channel and then concrete culvert (channel is unlined, pipe is concrete)
[29] The two roughness values downstream of Kruses gauge relate to the open channel and then concrete pipe (channel is concrete lined with concrete frame, pipe is concrete)
[30] Where the previous Avon model roughness was defined with two parts we found that a simpler single roughness in accordance with the model planning model schematisation succeeded in matching the NIWA rating curve

5.4 March 2014 calibration model rerun

Several improvements were incorporated into the March 2014 calibration model as part of the Avon River Modelling Improvements project. These model improvements, which are primarily located within the Riccarton area, reflect significant changes made to the design model since the previous calibration effort completed for the LDRP097 project. Not all changes to the design model since the previous calibration effort have been incorporated into the March 2014 calibration model, as some of the design model updates were determined to have minimal impact on the calibration model results. The following changes were made to the March 2014 calibration model, which are discussed elsewhere in the report:

1. Software upgrade to match the version used in the design model (see Section 2.3)
2. Updated eddy viscosity and time step to match design model run parameters (see Section 3.1.4)
3. Changed node loss type along the Riccarton Main Drain to "Weighted Inlet Energy" (see Section 3.2.2)
4. Corrected Dilworth St culvert, Deans Ave culvert, Picton Ave culvert[31], and Harakeke St Bridge[32] geometry and invert levels (see Sections 4.13, 4.13.3, and 4.13.5)
5. Added missing inlets in the Riccarton area[33] (see Section 4.13.1)
6. Made changes to open channel between Dillworth St and Picton Ave[34] (see Section 4.13.5)
7. Eliminated short pipe length on east edge of Picton Ave[35] (See Section 4.13.2)
8. Refined z-values representing the cess drain behind NIWA[36] (see Section 4.14.1)

While these changes were incorporated into the model, the model was not recalibrated following their addition. The results of the updated March 2014 calibration model (v102), including a hydrograph and water level time series at Gloucester St, are compared to the previous calibration model run (v100) and the recorded values in Figure 12. The results indicate an increase in flow and water levels at Gloucester St compared with the previous model run. These increases in flow rates and water level can be primarily attributed to capacity increases within the Riccarton Main Drain, which reduced ponding and attenuation within the Riccarton area, and increased discharge to the Avon River. Additionally, the Harakeke St Bridge geometry correction (enlargement) reduced ponding upstream of the bridge, increasing peak flow rates within the Avon River.

[31] Invert level adjustments of AVON.Pipe.CCCGIS.18061 were excluded from the calibration model updates.
[32] Straven Rd bridge geometry correction was excluded from the March 2014 Calibration model updates, since it was determined to have minimal effect on the results at Gloucester St.
[33] Approximately 23 of the missing inlets discussed in Section 4.13.1 were added to the calibration model, as these inlets were determined to have the most impact on the calibration model results.
[34] Open channel updates between Clarence St and Dillworth Ave were not included in the March 2014 Calibration model.
[35] Link simplifications of box culverts along Deans Ave were excluded from the calibration model updates.
[36] Note that AVON.Inlet.CCCGIS.21374, which is connected to the cess drain, was not modified as part of the March 2014 calibration model updates.

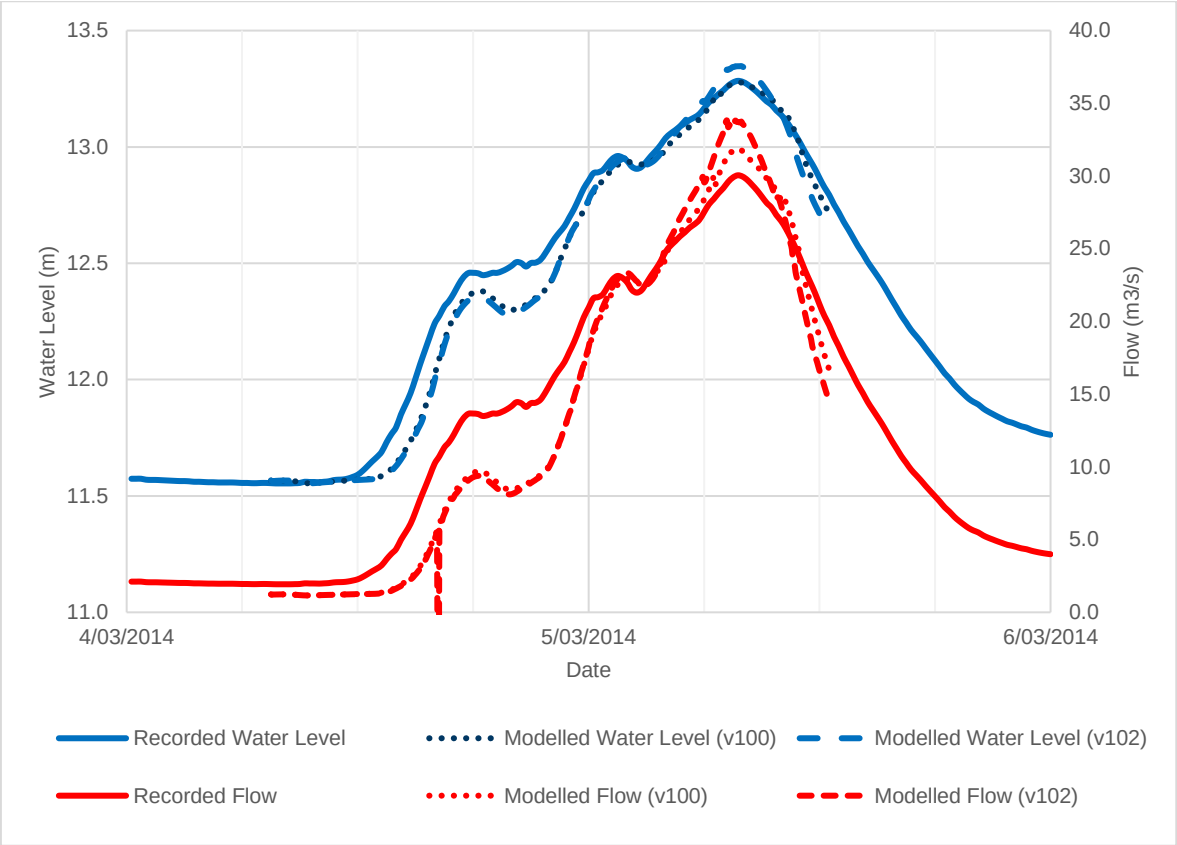


Figure 12 Calibration time series at Gloucester St

Table 21 compares peak flow rates and water levels at Gloucester St within the updated calibration model (v102), the previous calibration model (v100), and the recorded values. The results indicate a 12.7% increase in peak flow and a 0.06 m increase in water level at Gloucester Bridge, indicating that recalibration of the March 2014 Calibration model will likely be warranted as a part of future work effort. Recalibration of the March 2014 model was not included within the scope of the Avon River Modelling Improvements project.

Table 21 March 2014 peak flow and water level at Gloucester Bridge

	Peak flow (m^3/s)	% to recorded	Peak Water Level (m)	Difference to recorded (m)
Recorded	30.05	n/a	13.28	n/a
Previous calibration model (v100)	31.88	+6.1%	13.28	0.00
Updated calibration model (v102)	33.88	+12.7%	13.35	+0.06

5.5 Confidence in model calibration

As the model evolves, chronologically accumulated changes tend to diminish the confidence in model calibration. As discussed in Section 5.4, some of the significant design model updates since the LDRP097 calibration effort have been incorporated into the March 2014 calibration

model (note that the model was not recalibrated). However, not all design model updates have been incorporated into the calibration model, primarily for two reasons: (1) changes that were estimated to minimally impact the results at Gloucester St were excluded from the model updates, and (2) physical changes to the Avon Catchment and drainage network which have occurred since 2014 have been implemented in the design models but not the historic calibration models. Additionally, several future model improvements have been identified, which may impact the accuracy of results. This section summarises the main issues of this nature that are relevant to calibration.

5.5.1 Design model changes excluded from calibration model

As discussed in Section 5.4, not all design model updates since the LDRP097 calibration work effort have been incorporated into the updated March 2014 calibration model; only updates that were estimated to have a significant impact on the calibration results were included. Several updates, such as the railroad ballast modifications (see 4.14), model extensions (see Section 3.1.1), added headwalls (see Schema Report Recommendations in Appendix R- Email Chain 2 Headwalls), and changes to 1D boundary weir methodology (see Schema Report Recommendations in Appendix R- Email Chain 1 Boundary Weirs), were excluded from the calibration model updates, in addition to the exclusions discussed in Section 5.4. These design model updates are estimated to have a small impact on the calibration model results at Gloucester St.

5.5.2 Drainage system modernisations

Modernisations to the Avon Catchment and drainage infrastructure since the calibration events (June 2013 and March 2014) have been extensive. These modernisations include the Cranford Basin active management, Tay St pump station, Dudley diversion, and HSL PS205 upgrades. These modernisations limit applicability of the model calibration, since the results of the calibration model cannot be used to ensure that the model accurately simulates the new parts of the drainage system.

5.5.3 Factors affecting model uncertainty

In addition to the design model changes that have not been incorporated into the calibration model, several areas model for model improvements have been identified and have yet to be incorporated into the design or calibration model. These improvements are discussed in Section 10.2, as well as the Anomalies Register (see Appendix F), and the Future Improvements Register (see Appendix G). Some of the areas for improvement that result in general uncertainty in the model calibration are described as follows:

1. Rural vs. urban infiltration rates: While the model considers spatially varied infiltration rates based on soil types, there is no consideration of the differences in infiltration rates of pervious areas within rural areas vs. urban areas. Differences in vegetation and ground cover between urban and rural areas result in higher infiltration rates within rural areas, and lower infiltration rates within urban areas. One area of improvement for the Avon River/Estuary model would be to differentiate between rural and urban infiltration rates, which would require recalibration of infiltration parameters. This change could potentially reduce modelled flooding in rural areas, such as at the airport, while increasing modelled flooding within the city centre.
2. Roof drainage: Roof drainage is not considered within the Avon River/Estuary model, leading to uncertainty in the model results and calibration. In commercial areas, down pipes draining to underground pipe networks are not included in the model, leading to increased surface ponding and overland flow within the model. Similarly, roof drainage

from residential buildings is not well resolved, and down pipes draining to road gutters are not considered.

3. Onsite soakage: Onsite stormwater mitigation measures, such as green roofs, permeable paving, and onsite soakage are not considered within the Avon River/Estuary model. Disregarding these onsite soakage areas may lead to overestimates of stormwater runoff within the model. DHI is currently developing a methodology for including onsite soaking within the MIKE model.
4. Pipe ends: The current Avon River/Estuary model assumes that all pipe ends are plugged. In reality, while some pipes feature an end cap, other pipes daylight with the existing ground. This assumption results in uncertainty within the modelled pipe flow and surface flow.

6. Design scenarios

Design models were run under the LDRP097 project covering combinations of scenarios including existing development and a range of future time horizons with considerations including increase in rainfall intensity, sea-level rise, estuary throat enlargement and liquefaction settlement.

The selection of design run parameters and creation of the model input files representing the different scenarios are described below.

6.1 Design wind parameters

Design wind parameters are applied to both existing and future design scenarios.

Design wind speed and direction has been based in part on the March 2014 calibration event where a maximum wind speed of 15.4 m/s from the southwest was measured. This wind speed generated a peak water level difference between Ferrymead and Bridge Street in the March calibration model of 0.251 m, using the following friction factors which linearly vary with wind speed:

- Speed = 7 m/s, Friction = 0.003765, and
- Speed = 25 m/s, Friction = 0.00753.

Also relevant was finding that the peak design tide level differences between Ferrymead and Bridge Street (Ferrymead levels < Bridge Street levels) was around 50 – 60 mm based on Goring's 2011 analysis.

A preliminary evaluation was done of recorded differences in peak water level within the project data period but found there was relatively weak correlation between recorded wind in the SW component direction and the recorded difference in peak tide water levels. This analysis showed differences typically in the 50 – 200 mm range, but with some outliers at 250-350 mm.

Council agreed with GHD recommendation that the Goring difference was likely too small for the Avon River larger design flood risk purposes and generally not very applicable due to it having a quite different purpose. Knowing that some positive (non-zero) differential would be appropriate but faced with limited information as to what may be appropriate Engineering judgement was applied and a differential in excess of 100 mm was considered to be reasonable. On that basis a constant wind speed from the southwest of 10 m/s was selected and applied for all design AEP storm events.

6.2 Infiltration losses

Design infiltration losses (Hortons initial and continuing) were initially defined after the LDRP044 project calibration processes as documented in previous Rev 6 version of this status report.

During LDRP097 calibration and design infiltration losses were adapted to constant infiltration and the model calibration re-validated as reported in the "M21 Infiltration With Groundwater Capacity Report", schema report Appendix L. The resulting constant calibration and design infiltration rates for each soil type are shown in Table 22.

Table 22 Constant infiltration loss parameters

	Soils Drainage Class				
Constant Infiltration rate (75%/25% rule)	Very Poorly Drained	Poorly Drained	Imperfectly Drained	Moderately Drained	Free Draining
Calibration (mm/hr)	1.875	4.625	6	7	8.4
Design (mm/hr)	1.025	2.55	3.35	3.85	4.65

Refer to Model Schematisation Report Rev 7 Section 3.3.2 and Appendix O of this report for methodology on creating the existing and future impervious datasets.

6.3 Stopbanks down

Avon model stopbanks are modelled in both M11 and M21.



Figure 13 Extent of stopbank modifications

In order to model the stopbanks down scenario (specified as an extreme case where all the stopbanks have completely failed), the stopbanks modelled in M21 are all disabled and stopbanks modelled in M11 are set to laterally link type 'M21' meaning the river will flood directly to the fM21 floodplain regardless of the M11 cross section levels.

6.4 Rainfall

Council has adopted the NIWA's High Intensity Rainfall Design System (HIRDS) v4 as the source of rainfall depth for the current climate as specified in the Model Schematisation Report Rev 7 Section 3.3.1.

For the future design scenarios, the rainfall intensity varies with each time horizon and rainfall duration according to Jacobs's Climate Change Scenarios for Modelling Memorandum (Appendix K).

Sensitivity modelling analysis was completed on the rainfall intensity across 5th, 50th and 90th percentiles in 2019 as part of the LDRP097 project. The details of the analysis are documented in Avon/Sumner models sensitivity testing memo (Appendix M). Based on the report outcome, 50th percentile rainfall intensities were recommended to be used for future design scenario model runs.

A summary percentage of rainfall intensity increase with projected RCP8.5 temperature increase is shown in Table 23 below.

Table 23 Rainfall intensity with RCP8.5 climate change

Scenario name	RCP 8.5 temperature increase (deg)	50th percentile rainfall increase % for 50 yr ARI 24hr durations
2020	+0.0	+0.0%
2030+	+0.3	+2.7%
2060+	+1.4	+11.8%
2100+	+2.8	+23.9%
2150+	+4.6	+39.0%

Comprehensive data on the 50th percentile rainfall intensities percentage increase per time horizon, ARI and duration are tabulated in Appendix L.

In order to create rainfall files for future development scenarios, the identified climate change percentage is applied on top of the current climate rainfall files.

6.5 Tidal timeseries

Tidal boundary condition timeseries for the existing design scenario is based on the Sumner Heads tide time series developed by GHD/HKV in 2021(refer to Model Schematisation Report Rev 7 Section 7.2).

The sea level rise for future time horizons varies with each time horizon according to Jacobs's Climate Change Scenarios for Modelling Memorandum (Appendix K).

In order to create tidal files for future development scenarios, the sea level rises were simply added to the Sumner Heads tidal boundary levels.

Table 24 Sea level rises with climate change

Scenario name	RCP 8.5 temperature increase (deg)	Sea level rise (m)
2020	+0.0	+0.00
2030+	+0.3	+0.19
2060+	+1.4	+0.45
2100+	+2.8	+1.06
2150+	+4.6	+1.88

For modelling purposes, the only further required modification to this level data was timing adjustment for coincidence with flooding peaks.

6.6 Estuary throat enlargement

On the basis that rising sea levels will increase the tidal prism of the Avon-Heathcote Estuary, Jacobs provided estimates of the Estuary throat enlargement required for consistency with the increased tidal prism[37]. This estimate was developed by Jacobs using interpolated/extrapolated data from Table 3.5 of a report prepared by NIWA ('*Coastal sand budget for Southern Pegasus Bay – Stage B: Future sand budget*' dated June 2018). Jacobs estimates covered SLR scenarios of up to 1.88m.

GHD then has assessed the Estuary throat area requirement for an additional sea level rise (SLR) scenario of 2.40 m. Resulting Estuary throat areas are summarised in Table 25 below.

Table 25 SLR v estuary throat area

Modelling horizon (year)	SLR (m)	Mean sea level (m)	Estuary throat area (m^2)
2020	0	9.15	633
2060 – 2090	0.45	9.60	860
2100 – 2150	1.06	10.21	1161
2150 +	1.88	11.03	1565
2150 + (5^o temperature increase)	2.40	11.55	1821

A new bathymetry raster of the Estuary throat has been prepared for each of these scenarios. This includes new adjustment for the current post earthquake (2020) scenario as it was noted that the throat area at a mean sea level (MSL) of 9.15 m was less than 633 m^2 based on the previous Citywide (LDRP044) Estuary raster established using circa 2013 bathymetry data provided by NIWA.

The following briefly summarises the Estuary throat raster generation process:

- Establish a common upstream and downstream boundary line extent upstream of the Estuary mouth where the above throat areas are achieved at all the associated MSL's.

[37] Projected Throat Area Increase with SLR.xlsx provided 5/4/2019

- Insert additional lines between these extents and calculate the area below each SLR scenario at each of the line locations. In this instance, 11 additional lines (thus 13 in total) have been generated. The length of the cross section lines was defined to match the extent of wetted area for each respective mean sea level.
- Based on the area difference between the required Estuary throat area and the actual area under each line, calculate a constant lowering of the cross section required to achieve the throat area.
- Interpolate between the two fixed and eleven lowered cross section lines to generate new throat bathymetry patches to update existing bathymetry levels. The largest adjustments (near the centre of the throat) ranged from 1.44m for 2020 to 3.29m for 2.4m SLR.

6.7 Future earthquake settlement

Council developed a model for liquefaction-induced settlement resulting from a future design earthquake event (see T&T report in Appendix N). Within these findings, T&T recommended and Council adopted PGA=0.5g and SLR=1.0m scenario for LDRP097 flood modelling purposes[38]. These findings provided two surfaces using a 50m and 200m radius smoothing approach to avoid step changes in the data. The liquefaction settlement modelling results were limited to eastern Christchurch in extent similar to the Multihazard defined area of interest.

The liquefaction settlement was applied to the model ground levels throughout the extent of the supplied data (dark blue outline). It was also applied to the main stem of the river model as shown in Figure 14 (pink filled river extent). This 1D extent included both PS0205 and PS0210, and the first cross section upstream thereof. On the main stem of the river the 1D Liquefaction Settlement adjustment starts just east of the Madras Street (excluding Madras and Kilmore St bridges). The boundary between adjusted and not adjusted locations was abrupt with no transition zone.

[38] This approach is approved by David Cobby (Jacob) by email 10/04/2019 RE: Liquefaction-induced ground settlement model for LDRP097 project.

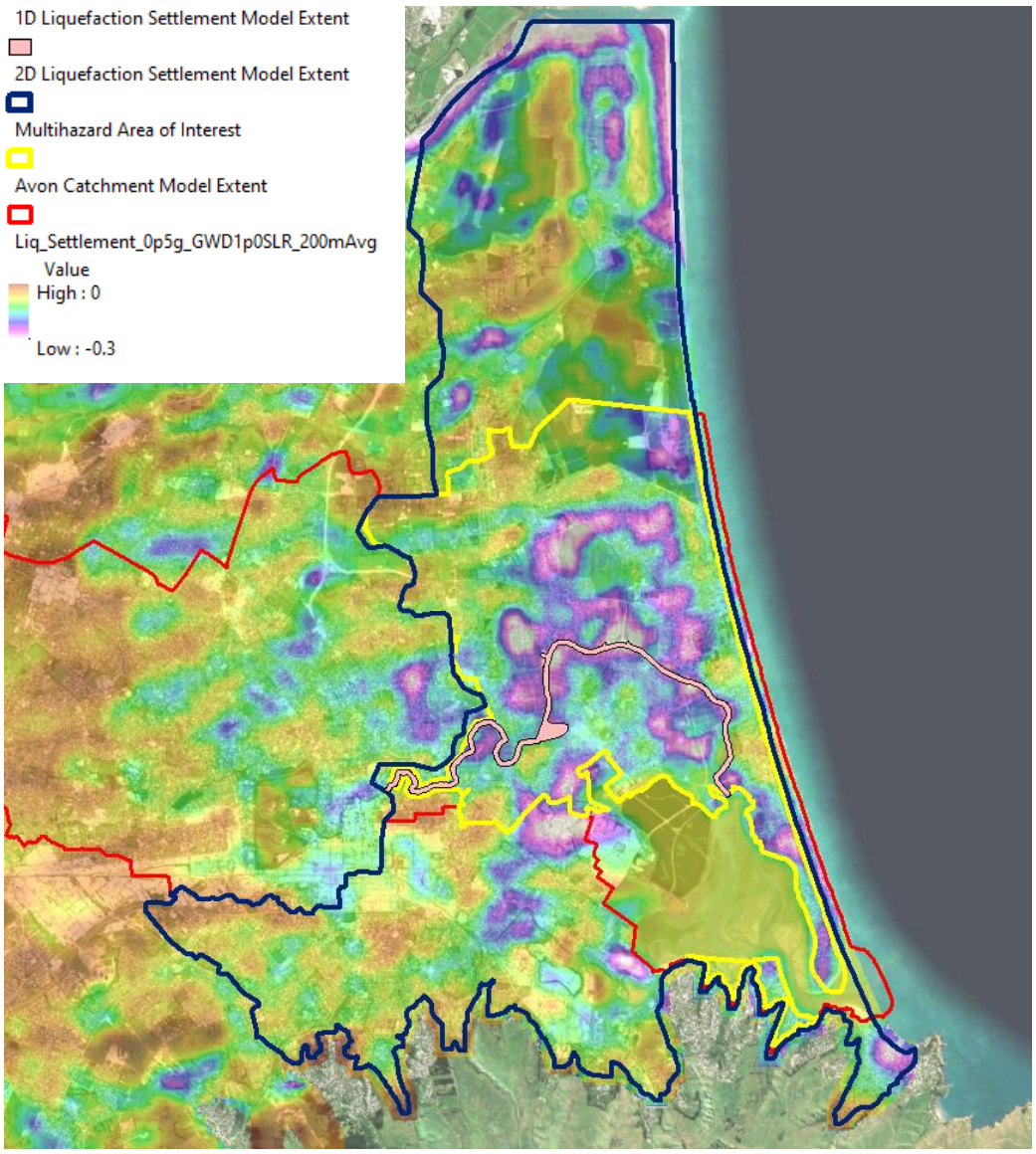


Figure 14 Liquefaction settlement area of changes

Note that the provided liquefaction settlement raster does not cover the coastal (beach and sea) extent of the Avon catchment. As the amplitude of the liquefaction settlement is small (0 – 0.3m) and the missing data area has low importance, no liquefaction settlement was applied to that area.

Future earthquake settlement was applied selectively and with differing methods to different types of model components, reflecting their importance and the modelling practicalities. T&T provided 50m radius smoothing and 200m radius smoothing liquefaction settlement raster. All M21 shifts (ground levels and dikes) were based on the 50m smoothed settlement surface. All M11 shifts (structures and cross sections) were based on the 200m smoothed settlement surface. The 50m is thought to better represent new risks of the pluvial flooding in local terrain depressions, and the 200m for the MIKE11 component will minimise risk of negative grade and model stability along the 1D components.

For M11 cross sections, the settlement at the river centreline is applied homogenously to the whole cross-section using datum shift[39]. In order to limit the risk of Q/H relationship reprocessing issue at M11 structures, the upstream and downstream bounding cross-sections of each edited structure have been assigned with the structure's shift instead of their own

[39] DHI tool "MIKE11 DatumChange.xlsm" (received 23/04/2019) was used to implement the MIKE11 cross section elevation shift

associated raster shift. In addition, in order to limit the risk of M11 junction level differences from side M11 branch to the main Avon River stem within the 1D area of interest, the last cross-section of each side branch was assigned with the closest Avon River EQ shift (instead of their own associated raster shift).

The MU pump stations invert levels, operating levels and Q/H relationship are shifted in accordance with the downstream model the pump discharges to: i.e. 50m smoothing for PS0209 which discharges to M21 and 200m smoothing for the it discharges to M11. Levels of the pipe network in MU were not changed.

7. Design run setup

7.1 Project and master model run scenarios

The project and master model scenarios that were setup and run are documented in LDRP097 Multi-hazard Baseline Modelling Report, Rev 0 (2022) Section 4.

Information associated with current procedures and requirements for setting up and running new model scenarios are included in Appendix C.

7.2 Model run durations

The run duration in terms of model clock time is important, first to ensure the model runs 'past the peak' and secondly for those interested in how quickly flooding recedes. Accordingly two specifications have been determined (new to LDRP097) with standard and extended run durations presented below in Table 26 and Table 27.

Table 26 Standard run duration and save timesteps

Storm duration (hr)	Run duration (hr)	M21 saving steps (min)	Total number of saved steps
0.5	5.75	1	90
1	5.75	2	60
2	7	5	48
3	8.5	5	72
6	10.25	10	60
8	12	10	72
9	13	10	78
12	16	10	96
18	24	15	96
24	30	15	120
36	42	15	168
48	54	15	216

Use of the extended durations for selected runs are specified in the Multihazard Project Report Table 1.

Table 27 Extended run duration and save timesteps

Storm duration (hr)	Run duration (hr)	M21 saving steps (min)	Total number of saved steps
0.5	10.75	2.5	156
1	10.75	4	105
2	13	6	100
3	15.5	7.5	104
6	18.25	10	108
8	24	10	144
9	29	10	174
12	36	10	216
18	42	15	168
24	48	15	192
36	60	15	240

7.3 Tidal timing and pre-start

The starting level at the Sumner Heads is dependent of the timing of tides which depends on design storm durations. Tidal timing is introduced in the model in order to coincide the peak tide with the hypothetical peak river outflow at Bridge St.

The methodology of defining the timing of the tides is documented in Model Schematisation Report Appendix G. For the 18hr design rainfall, the Avon showed a six-hour timing delay between the peak of rainfall (at 70% of 18hr) and the peak outflow at Bridge St when modelling with a flat median tide level. This is achieved in the modelling with a five-hour timing delay to Sumnerhead, which causes the desired six-hour delay at Bridge St. For short duration events, the 0.5hr delay schema rule is achieved at Bridge St by using timing Sumnerhead peak tide 0.5hr ahead of peak rainfall.

The initial approach to hotstart condition is to work out the coincident timing by this method and then start the model at the beginning of the design rain event. The main problem with this method is that is sometimes requires starting the model in already high flood conditions and there is no simple strategy to set those conditions appropriately.

Any simple hotstart strategy would then have significant flood volume errors at start time and static water levels could not adequately match dynamic conditions. This problem is most acute for short duration rain events where timing demands high flood levels at the start of the rainfall and there is less time during the model run for any start time volume and level discrepancies to resolve themselves hydrodynamically within the model run. The problem is compounded by the stopbanks and unanswerable questions of how much flooding will have occurred outside the stopbanks if the model starts above stopbank levels. There are also stability risks with large level discrepancies at model start time.

In order to avoid these issues, we instead start the model run near low tide prior to the peak for all short duration rain events. The models were pre-started with zero rainfall intensity and a pre-start period where the only model activity is at the rising tidal boundary. Table 28 summarises the time to Sumner peak tide for the Avon model and the pre-start periods for short duration events.

Table 28 Tidal timing for Avon model

Storm duration (hr)	Duration from rainfall start to Sumner peak tide (hrs)	Model pre-start (hrs)
0.5	-0.15	4.25
1	0.2	3.75
2	0.9	3
3	1.6	2.5
6	3.7	0.25
9	5.8	0
12	7.9	0
18	17.6	0
24	21.8	0
36	30.2	0

7.4 Hotstart level conditions

Once the timing of tides and pre-start is known, the starting tide level is then extracted from tidal time series data based on the defined starting time. Based on this tide level and the stopbanks up/down condition, hot start approaches are chosen separately for M11 and M21. For M21 the global hot start approach is preferred for economy of setup effort and is therefore used when the

flood levels are low enough that the resulting wet areas are essentially confined to the estuary and similar normally wet extents, with defined extents being used at higher starting levels.

Table 29 summarises the approaches to starting condition in the M11 and M21 models. [40]

Table 29 Hotstart conditions

Starting level at Sumner Heads (m)	Stopbank up		Stopbank down	
	M11	M21	M11	M21
<9.1	Dynamic Hotstart	Global value	Dynamic Hotstart	Global value
9.1-9.6	Dynamic Hotstart	Defined extent	Dynamic Hotstart	Global value
9.6-11.2	Dynamic Hotstart	Defined extent	Static HD11 global value using start level at Bridge Street	Global value start level at Bridge Street
>11.2	Not consider staged	Not considered	Static HD11 global value start level at Bridge Street	Global value start level at Bridge Street

Notes:

- The M11 dynamic hotstart matches the ideal starting level at Bridge Street, and
- The M21 global value and defined extent hotstarts match the ideal starting level at Sumner Heads unless noted as starting level at Bridge Street.

Because the M21 hotstart condition is always static (flat water) to match Sumner Head in most cases above the model will start with the wrong levels at the M21 side of Bridge St, but the right water levels at the M11 side of Bridge St. This results in a level discontinuity at time step zero between M11 and M21 at Bridge St.

In the stopbank down runs with starting level >9.6 a similar level discontinuity occurs at the estuary throat boundary, again due to the flat M21 starting levels. Inspection of results to date have shown that either type of discontinuity is well tolerated by model stability and the pre-start approach ensures adequate model run time for any volume irregularities to work their way out of the model.

The above flood level groupings have been chosen as follows:

- <9.1: wet area is contained within the estuary, and this is achieved with simple M21 global value
- 9.1-9.6: with stopbanks down the initial M21 flooding around the river remains minimal and we can confidently use ideal dynamic river starting conditions without concern for inconsistencies with the M21 global value level; with stopbanks up the M21 needs a “defined extent” to prevent initial flooding outside the stopbanks
- 9.6-11.2: with stopbanks down the initial M21 flooding around the river is significant and we cannot confidently use ideal dynamic river starting conditions without troubling inconsistencies with the M21 as a global value, hence we revert to static M11 global

[40] This is also described in Tim Preston email “Multihazard - Avon model initial conditions” 20/5/2021

hotstart to exactly match the M21 global level[41]; stopbanks up situation is the same as 9.1-9.6 range, and

- >11.2: stopbanks down the situation is the same as the 9.6 – 11.2 range; stopbanks up with a starting level above the 11.2m stopbank top level has not yet been considered[42].

8. Model stability status

All the 210 model runs indicated in the LDRP097 Multi-hazard Baseline Modelling Report, Rev 0 (2022) Section 4 ran to completion. 2D stability oscillation result analyses were carried out across 210 model results and delivered to Council in spatial and tabular formats. 1D model stability checks and volume balance checks were not required.

On completion of M21 stability oscillation assessment, the stability statistics are as below:

Table 30 LDRP097 V114 M21 Stability Assessment Outcome

Max amplitude (m)	Max M21 number of elements across batch runs				
	10ARI	50ARI	200ARI	500ARI	All results
< 0.05	1,768,083	1,768,083	1,768,081	1,768,046	1,768,083
0.05 - 0.10	9,474	9,457	9,512	8,817	9,512
0.10 - 0.15	4,722	4,991	5,154	297	5,154
0.15 - 0.30	294	403	463	499	499
0.30 - 0.50	92	110	107	38	110
0.50 - 1	41	49	42	7	49
1 - 2	0	0	0	0	0
2 - 5	0	0	0	0	0
>5	0	0	0	0	0

In the above table, statistics have been derived based on the maximum oscillation amplitude aggregated at each model location, from each set of individual runs within the 210 model runs.

As shown in Table 30, the model has a weak tendency to be more stable in M21 under the 10 year ARI runs and has a strong tendency toward improved reduced frequency of larger oscillations in the 500 year runs. It has the worst performance in the 50- and 200- year runs.

There is a notably higher frequency of instabilities in the 50-150mm range for the higher sea level rise scenarios and especially the longer rain durations therein. The 500 yr group of results is better because it does not include any the higher SLR setups.

Details of the stability standards assessed are presented to Council electronically in the form of XLS raw data, summary tables and graphs as well in in GIS shapefile format. These files are handed over in folder :\03_Avon\06_Calculations\.

[41] In this case, to better match total starting system volume, we start both model elements with the ideal water level at Bridge St, meaning any error in level toward the estuary throat boundary is likely to be opposite in sign to and to some extent offsetting the error upstream along the river.
[42] This would need significant assumptions defining the pre-event conditions in order to consider this

9. Design result evaluations

9.1 Functional model build checks

Functional model build checks evaluate parts of the model based on engineering understanding of design behaviour, to ensure that the model and results actually produce the expected behaviours. Functional model build checks have been carried out on all model changes that result in significant new systematic functions (as recorded in the model log). Two such checks are reported below for v114 of the Avon model, completed as part of the LDRP097 Multihazard project.

9.1.1 Knights Drain Basin and PS232

Knights basin in terms of flood flow connectivity is a dead end detention storage to the side of Knights Drain. For post flood recovery it also has a small outlet at the north end to the open drain (which is not modelled). The drainage network here is entirely modelled in MU. On the open drain immediately downstream of the connection to the basin there is a weir which creates a level drop in moderate flow conditions and helps divert flood flow into the basin. Downstream of the weir Knights Drain PS232 pumps the flow direct to the Avon River. A minor side drainage branch also connects into the pump station wetwell from the north, but this delivers immaterial flow rates.

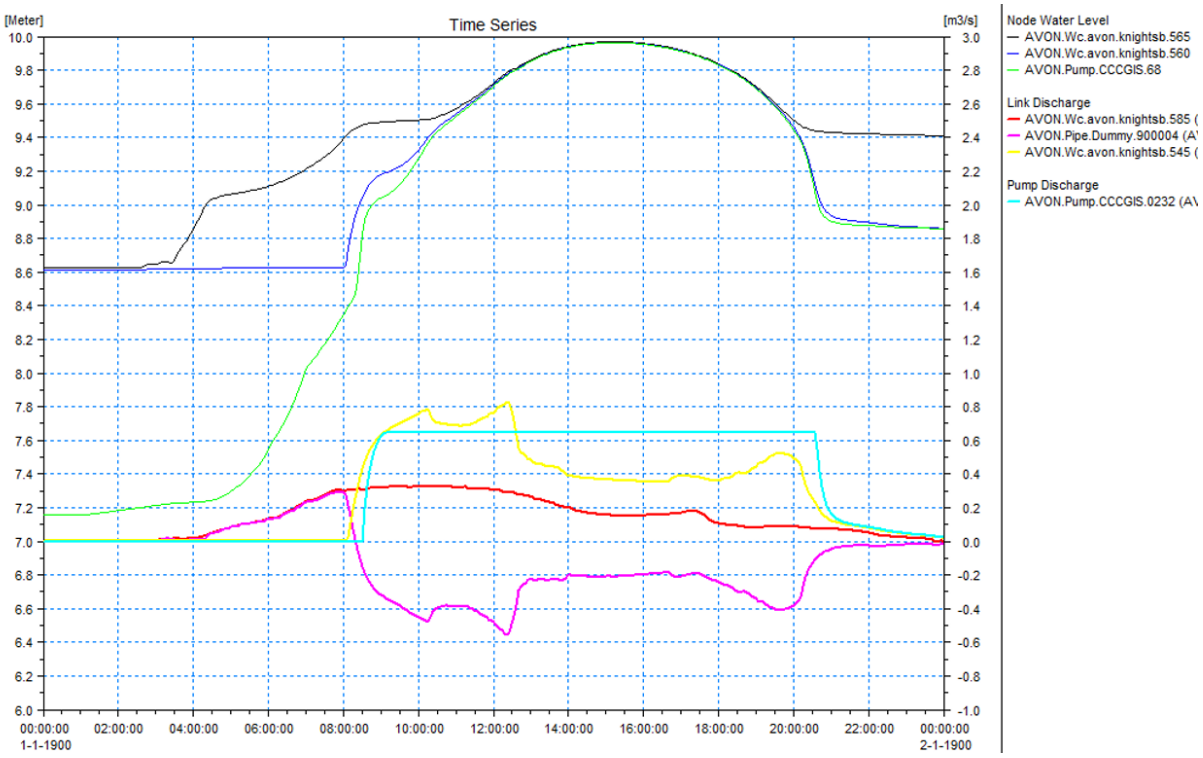


Figure 15 Knights Drain Basin and PS232 result

Figure 15 shows the key parts of the MIKE Urban system incoming to and outgoing from the Knights Drain basin and pump station. The result file is 2020, 200yr ARI, 18 hrs (V114_0SLR_200R13T_18hUp.PRF).

At the top of the figure node water levels as listed in the legend order are:

- In Knights Drain upstream of the weir where the flow splits (the same water level as is in the basin)
- In Knights Drain downstream of the weir below the flow split, and

- Knights Drain PS wetwell (PS232).

At the bottom of the figure link discharge flows as listed in the legend order are:

- Incoming flow in Knights Drain
- Diverted flow into (and out of) Knights Basin
- Knight Drain flow below the flow split weir between the basin and the pump station, and
- Knights Drain PS pumped discharge.

Notable trends are that the basin is filling from 04:00 to 08:00. At 08:00 the basin reaches the weir crest level of 9.4 m and flow begins in the drain to the pump station. Around 08:30 wetwell levels have triggered the pump station to start pumping, but levels continue to rise and by 09:00 the pump station is running at full capacity (0.65m3/s). Interestingly, from 09:00 onwards, flow reverses out of the basin, while the basin level is rising or stationary. This is associated with M21 inflows coming from the southwest, along the southern side of Pages Road and then crossing the centreline to the north prior to Anzac Drive (which is high ground). By 21:00 flooding has subsided to the point that the pump station has recovered, and the basin level has dropped close to the weir level and surface inflow to the basin (and reverse drainage flow out of the basin) has essentially stopped. By 24:00 and the end of the simulation, the basin level is materially at the weir level, and flows and pumping has ceased. Peak flood levels in this event are 9.97m, which is slightly below the start to flow crest level over Anzac Drive (10.06 m).

9.1.2 Tay Street Pump Station PS202

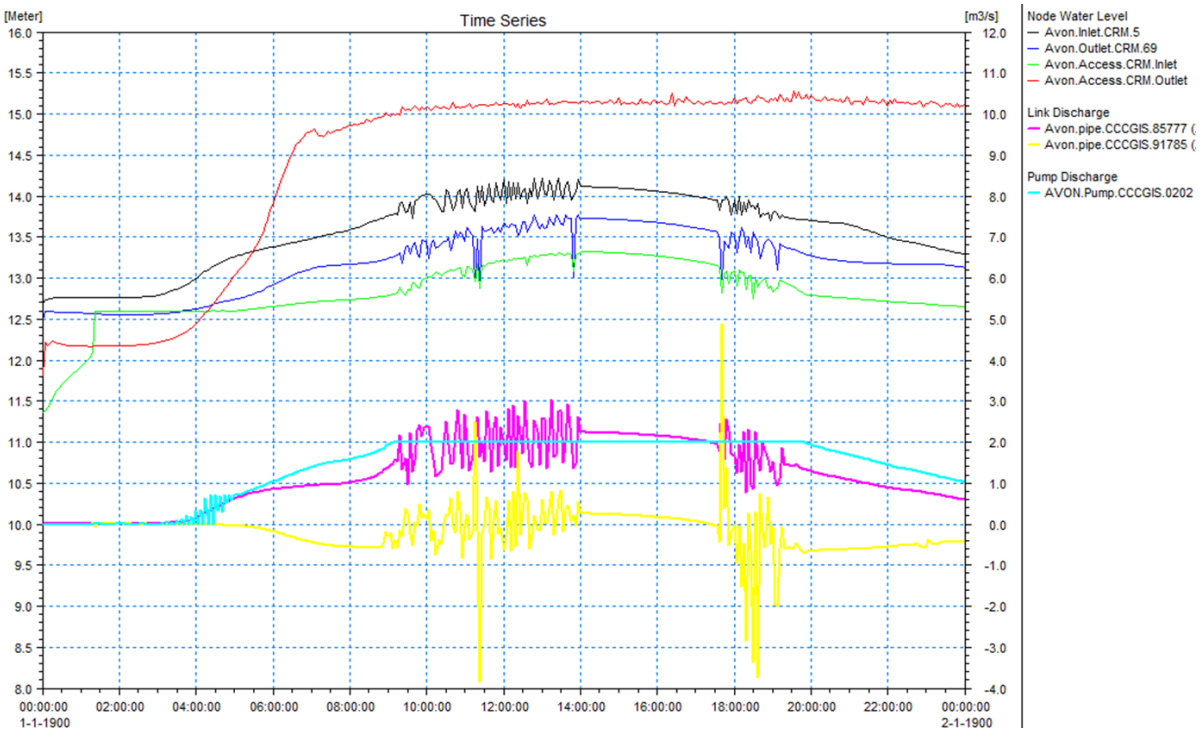


Figure 16 Tay Street result

Figure 16 shows the key parts of the MIKE Urban system incoming to and outgoing from the Tay St pump station. The result file is the same 2020, 200yr ARI, 18 hrs (V114_0SLR_200R13T_18hUp.PRF).

At the top of the figure node water levels as listed in the legend order are:

- Where Tay St Drain (open channel) from the southwest ends and flow continues into the MU pipe end

- Pipe end connected to the where Mairehau Drain opens up near 36 Kensington Street
- The wetwell inlet at Tay Street pump station
- The pump discharge outlet location into the Dudley Creek Diversion slightly downstream from Philpotts Road
- At the bottom of the figure link discharge flow as listed in the legend order are
- First pipe downstream of Tay Street Drain (open channel)
- Last pipe upstream of where Mairehau Drain opens up near 36 Kensington Street, and
- Tay Street PS pumped discharge.

Notable trends are that the pump starts to operate after four (4) hours of rainfall, flow is then drawn (negative direction) from Mairehau Drain until the PS capacity (2m3/s) is reached. At peak flow (circa 14:30 time) with the pump station capacity exceeded some positive flow returns toward Mairehau Drain. On the declining flood the sequence occurs in reverse. Level and flow instability at times is also evident but water levels are below ground level (typically at least 14.2 m) and there is minimal surface flooding at these locations.

9.2 Flood differencing result checks

9.2.1 LDRP097 (V114c) minus LDRP112 (V096) models

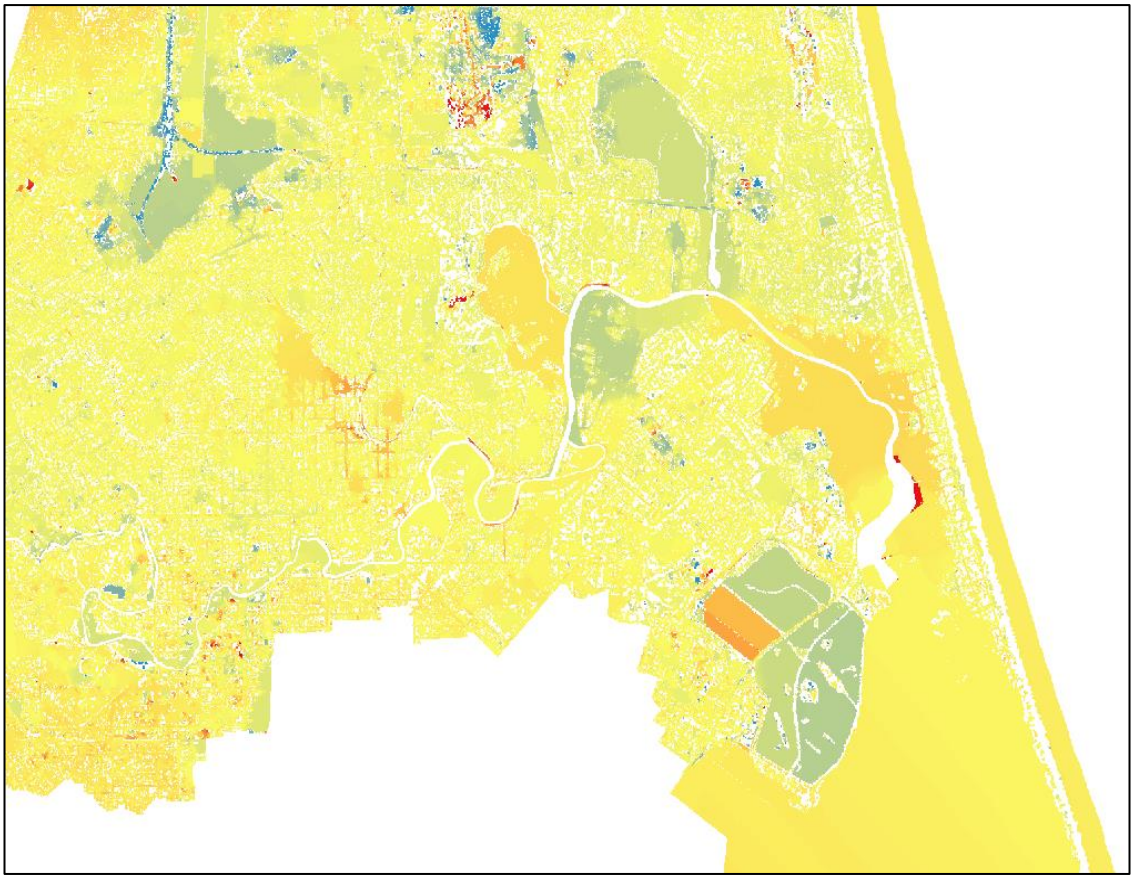


Figure 17 50ARI stopbank up V114c (LDRP097) minus V096 model results (LDRP112)

Figure 17 illustrates a lot of differences between the flood levels being used by Council to date (v96) compared to the new delivered result (v114). Both results are 50 yr stopbanks up results with approx. 0.5m SLR. The v96 result has Goring2018 tide levels with 0.5m SLR. The v114 result has HKV2020 tide levels with 0.45, SLR. The v114 result is a 2060 scenario with

associated development (unmitigated) and increased rainfall intensity. The v96 uses WWDG rainfall +16%CC and Horton infiltration, whereas v114 uses HIRDSv4 with Jacobs climate change and M21 constant infiltration. The ground levels also differ between the 2011 LiDAR and 2018 LiDAR. There are numerous features evident in the results:

- Flockton Street area is notably drier as expected with new infrastructure developed there
- Cranford Basin is wetter as expected with deliberate design to make better use of this facility for stormwater retention
- In a number of general areas including through the CBD, Avondale Road and Travis Swamp the wetter (blue) tinge is associated with the unmitigated development and possibly some rainfall influence
- In the estuary we see a 30-60mm reduction in water levels which is consistent with the 50mm difference in the SLR condition (v96 has no wind applied and was also level fixed at Bridge Street)
- Upstream of Bridge Street and up to Horseshoe Lake the dryer (orange) tinge is associated with stopbank overtopped flooding and the reduced tide levels in v114 generate an exaggerated benefit reducing stopbank overtopping and hence out of bank flood levels are much drier, and
- Finally there is noise evident in miscellaneous areas such as through the CBD and in the new Prestons subdivision area. This is associated with changes in ground levels.

9.2.2 Stopbank down minus stopbank up

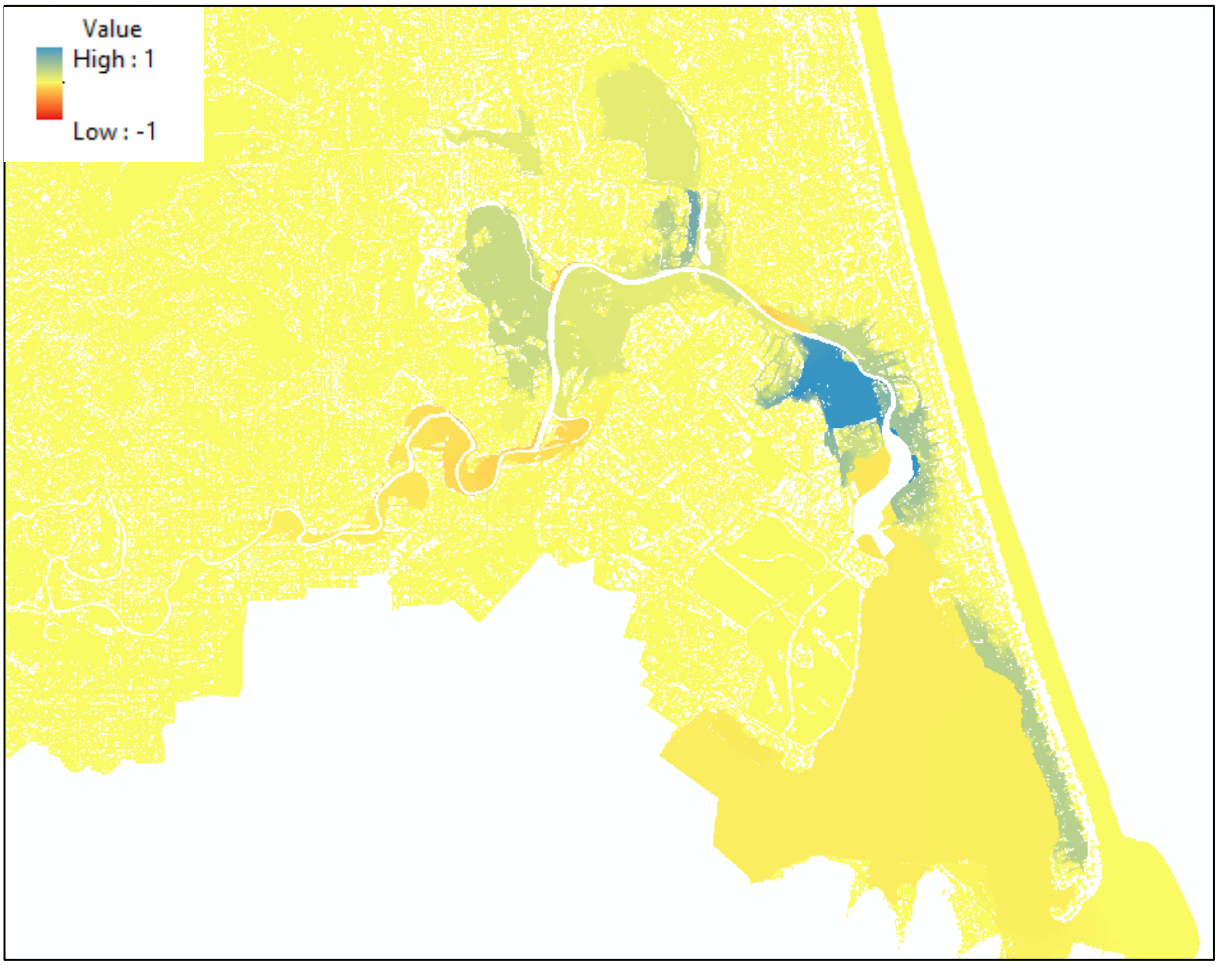


Figure 18 Differencing map of 2020 200ARI stopbank down minus stopbank up

Figure 18 shows the more than minor residual impact and benefits of the stopbanks being 'up' during the 200 year flood. In the downstream area, blue shades (wet) occur outside the stopbank along the main river stem and down to Southshore, illustrating the extra flooding, up to 1m level difference in Bexley that occurs in the stopbanks down model. In the upstream area around Dudley Creek, orange shades (dry) show where the stopbanks down produce less flooding due to the improved downstream flood channel capacity with the stopbanks down. This orange pattern (with 50mm level difference) also appears in the estuary, showing how the estuary levels benefit from the more natural stopbanks down regime with better flow attenuation, as compared to the engineering restriction with stopbanks up forcing more peak discharge through the river and estuary system.

9.2.3 2100 future minus 2020 existing scenarios

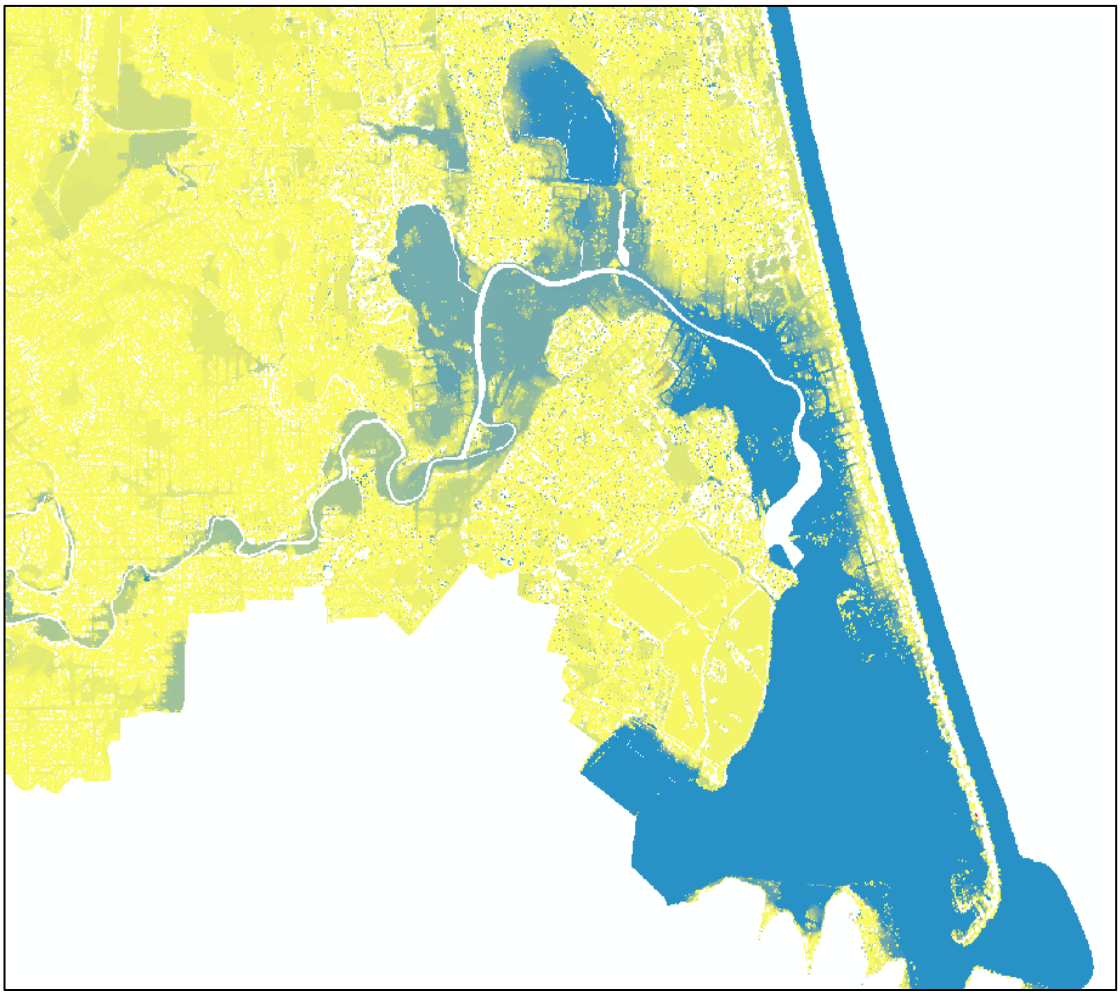


Figure 19 Differencing map Of 2100 minus 2020 time horizon

Figure 19 shows the increased flood levels predicted with all the future factors applied between 2020 and 2100 for the 50 year ARI stopbanks down scenario. The result is as expected with 1.06m SLR shown at the coast. Interestingly there is 1.09 m rise predicted in the estuary, presumably associated with the modelled amplification of the estuary throat capacity meaning that the estuary is more exposed to the peak sea level. This could also result somewhat from increased runoff and inflows at Bridge Street.

9.2.4 Standard result minus future EQ

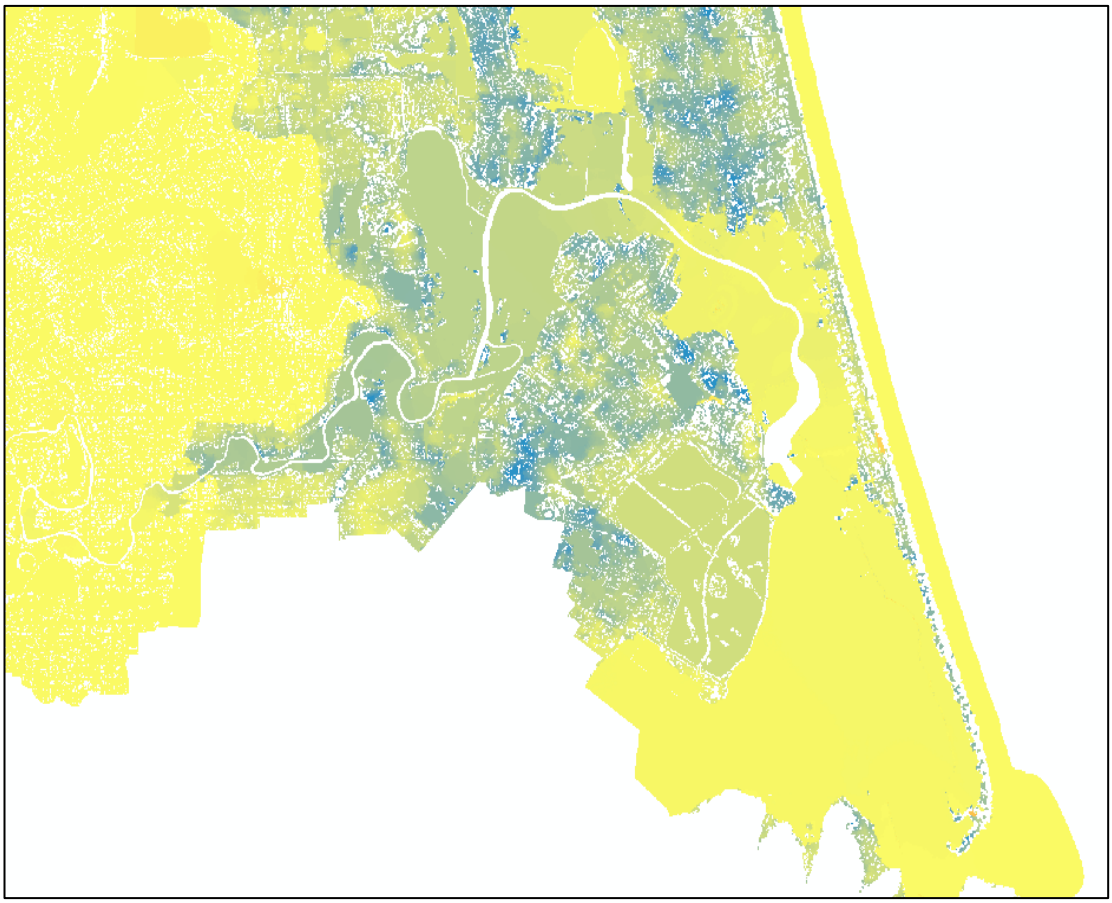


Figure 20 Differencing map of standard result minus FutureEQ flood levels

Figure 20 shows how potential land subsidence is estimated to affect flood levels for the 200yr, 2060, stopbanks down scenario. Contrary to some expectations the land subsidence is beneficial in terms of reducing flood levels (blue shows wetter in the standard result). On further consideration this makes sense as most of the wide area flooding is thin surface flooding unaffected by downstream constraints, and so the flood depths are mainly unaffected by the subsidence. This results that the flood levels drop approximately as much as the land subsides. In this illustration the darkest blue colour ranges up to 0.250m and the figure indirectly through flood levels illustrates the extent and magnitude of land subsidence. Through the Dudley and Horseshoe Lake vicinity the (similar blue) impacts of dropping the river cross sections – this impact disappears toward Wainoni and Pages Roads as the tidal influence dominates the result regardless of the river cross sections.

9.3 Groundwater sensitivity

To evaluate the sensitivity of the model (and model calibration) to assumptions on antecedent groundwater levels, twin model runs with 50% and 85% exceedance groundwater levels were run as part of the LDRP097 Multihazard project. The resulting model flows at Gloucester St flow gauge location are compared below.

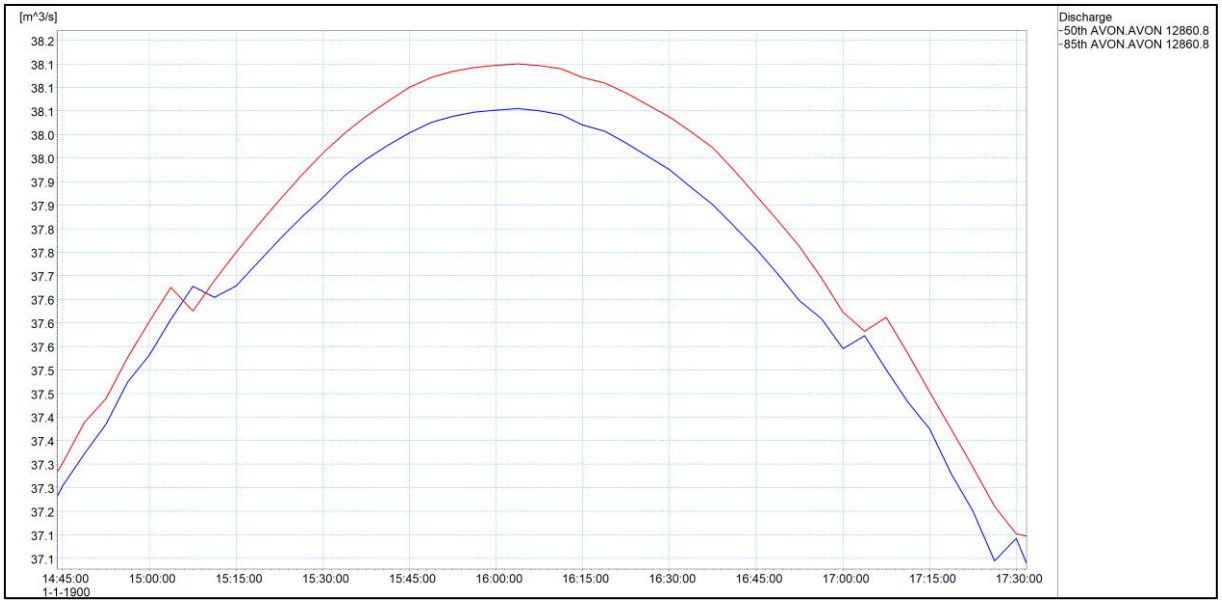


Figure 21 85th% vs 50th% groundwater at Gloucester Street

Figure 21 shows the peak flows at the Gloucester Street for 2100 time horizon 1.06 sea level rise 50 years ARI 18hrs storm scenario. The peak flow differences between the 50th percentile groundwater model results and 85th percentile model results is less than 0.2%. A similar check was done at the Bridge Street, and the differences is also found to be trivial. The results show the model has low sensitivity to the groundwater percentile in the conditions modelled.

10. Compliance with specifications

10.1 Identified issues of low importance

During the project, issues of minor importance have variously been discovered originating from input data or unexpected ramifications from the adopted model build processes. In many cases the immediate consequences were minor enough that it was agreed to be in the interests of the project (so that the project could reasonably reach a conclusion) not to address the issues through corrective actions, but rather to make a record of the issue so as to enable Council to return to address the issues in future. In the Avon catchment, these issues are recorded in the Avon Anomalies Register (attached in Appendix F).

In all instances, issues are reported to Council, typically via email. In some instances, where the better interests of the project were not obvious, Council approval was obtained confirming only to record the issue and not to take corrective action(s). Typically, fuller details of the issues and to what extent they are anomalous are available through reference to either the Council or GHD email records of the reports.

Issues include missing pipe, missing or non-standard data source flags or attributes, imperfections in model geometry, decisions not to include new local data discovered by Council staff (e.g. As-Built dwgs). None of the individual issues merit reporting in any detail.

10.2 Recommendations for model improvement

A number of different approaches have been identified which may be worth considering in any future updates of the model. Highlights are summarised below and have been extracted from the Register included in Appendix G. Item numbers (in brackets) below are in accordance with this Register:

1. The mesh build for the roads lends itself to long thin quadrilaterals rather than the current triangle build. Modifying the road schematisation in this manner could increase the minimum mesh size, improving run times and stability (Item 2)
2. To possibly improve runtimes, consideration could be given to investigating running the model using the MIKE 1D engine for both the MIKE URBAN components. This engine is optimised for parallel computing and therefore can take advantage of multiple CPUs (Item 3)
3. If assessment of Council procured radar based rainfall data indicates it is of suitable quality, use this information to improve calibration / validation modelling (Item 7), and
4. Reduce dx(max) to 20 m on the basis it could potentially improve current model stability without impacting significantly on model run times (Item 9).

11. References

1. GHD (2020) Model Schematisation – Avon/Estuary, Heathcote and Sumner – Rev 7 Report
2. GHD proposal for Triangular Mesh generation 07/07/2015
3. GNS Report (Waterways, Swamps and Vegetation of Christchurch in 1856 and Baseflow Discharge in Christchurch City Streams, May 2007).
4. Christchurch City Council (2011) Waterways, Wetlands and Drainage Guide: Part B
5. GHD (2012) Stormwater Modelling Specification for Flood Studies
6. GHD (2012) Guidelines for using Stormwater Modelling Results in Statutory Processes
7. DHI (2015) Avon River Hydraulic Model Status Report
8. GHD (2022) LDRP097 Multi-Hazard Baseline Modelling Report
9. Tonkin & Taylor (2018) Future land surface model results
10. Jacobs (2019) Climate Change Scenarios for Modelling Memorandum
11. GHD (2020) Avon model sensitivity to rainfall and groundwater level (Rev3)
12. GHD (2018) LDRP 503 Cranford Basin Active Management Design Report

Appendices

Appendix A – A3 maps, 2018 & 2022

Map 1 – Avon/Estuary Full Model Extent, 2018

Map 2 – Avon 1D Model Map, 2018

Map 3 – HGH vs M21 Map, 2018

Map 4 – Boundaries Map, 2018

Map 5 – Basins Map, 2022

Map 6 – M11 Data Sources, 2018

Paper Size A3

0 0.5 1 1.5 2 2.5

Kilometres

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

N

LEGEND

Citywide Boundary Lines

- Avon/Estuary Hydraulic Model Extent
- External Hydrology Catchment
- Mike 11 Network Branches
- Mike Urban Links

Christchurch City Council
Citywide Flood Modelling - Avon

Job Number	51-33275
Revision	0
Date	04 Dec 2018

Full Model Extent

Map 1

G:\51\33275\02.Stage 2 Model Build\GIS\Maps\Deliverables\ModelBuildReport_Figures\51_33275_Map1_ModelFullExtent.mxd

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Data source: Citywide Boundary Lines - G:\51\33275\02.Stage 2 Model Build\GIS\Data\01.Misc_Project_Data.gdb\Citywide_Boundary_lines_20171212 Created by: tpreston / yhooi

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Paper Size A3

0 0.5 1 1.5 2 2.5

Kilometers

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

N

LEGEND

- Citywide Hydraulic Model Extent
- M11 Closed Cross Section (Pipes)
- M11 Open Channels
- M11 to MU Links
- MU Pipes < 600mm Diameter
- MU Pipes >= 600mm & Rectangular Cross Sections
- MU Open Channels

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Christchurch City Council
Citywide Flood Modelling - Avon

Job Number	51-33275
Revision	0
Date	04 Dec 2018

1D Model Map

Map 2

G:\51\33275\02.Stage 2 Model Build\GIS\Maps\Deliverables\ModelBuildReport_Figures\51_33275_Map2_M11_MU_mapping for model status.mxd

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Data source: M11 Closed Cross Section (Pipes) - G:\51\33275\02.Stage 2 Model Build\GIS\Data\09.M11.gdb\PostEQ_Cross_Section_Type_Line_xns088h_20171221; M11 Open Channels - G:\51\33275\02.Stage 2 Model Build\GIS\Data\09.M11.gdb\ Avon_branch_network_ROUTE_20171211_nwk99g;
M11 to MU links - N:\NZ\Christchurch\Projects\51\33275\02.Stage 2 Model Build\GIS\Data\09.Mike_Flood_Couplings.gdb\MU_M11_River_links_20171010_V088_PostEQ; MU Pipes & Open Channels - G:\51\33275\02.Stage 2 Model Build\GIS\Data\09.MU.gdb\Avon_pipe_network_20170921; Created by: tpreston / yhooi

Paper Size A3

0 0.5 1 1.5 2 2.5

Kilometers

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

N

LEGEND

Lateral Link Level Type

HGH

M21

Standard Links - M11 to M21 links

Avon/Estuary Hydraulic Model Extent

Christchurch City Council
Citywide Flood Modelling - Avon

Job Number	51-33275
Revision	0
Date	04 Dec 2018

Lateral Link Level Type & Standard Link Location

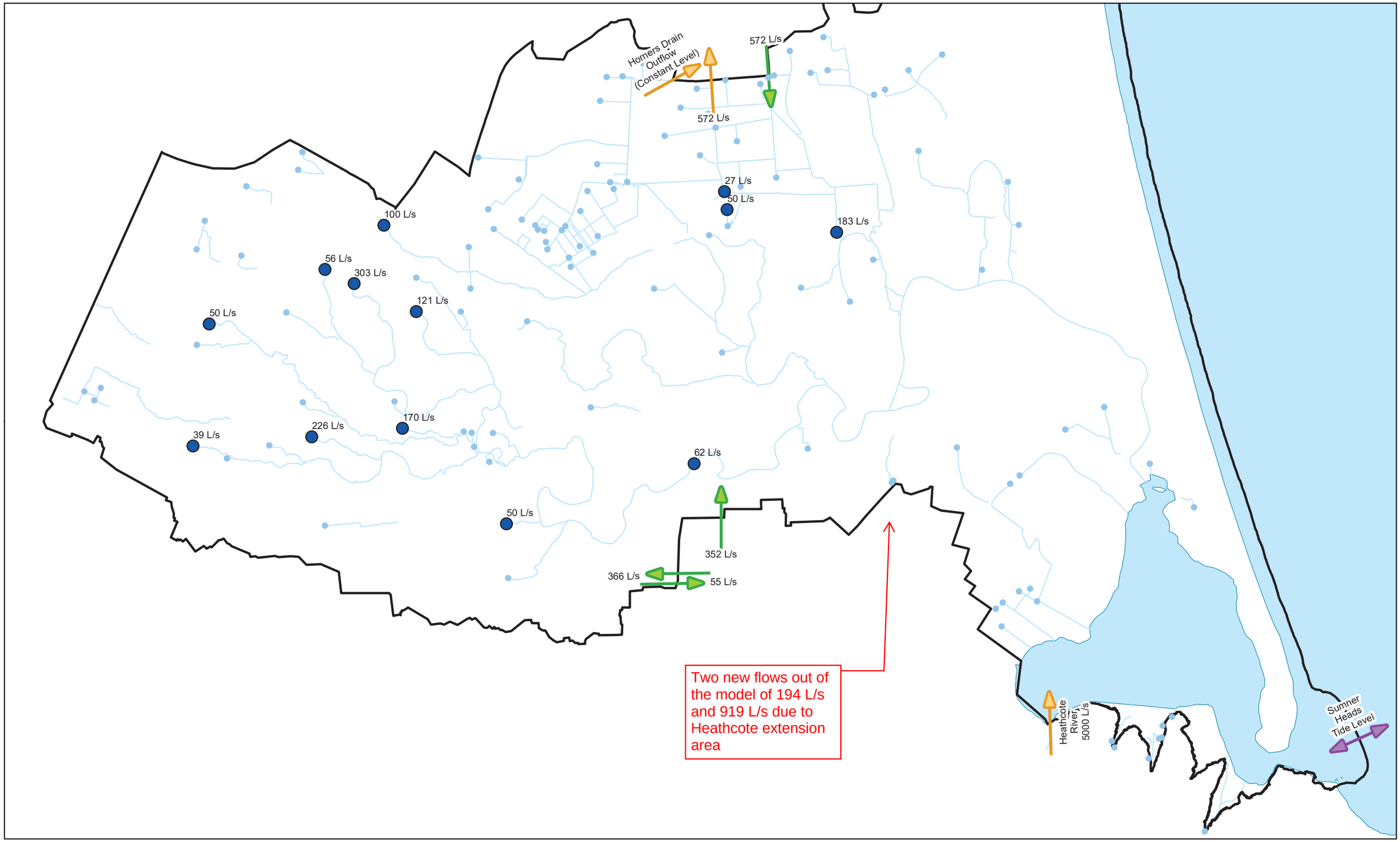
Map 3

G:\51\33275\02.Stage 2 Model Build\GIS\Maps\Deliverables\ModelBuildReport_Figures\51_33275_Map3_HGH_vs_M21.mxd

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Data source: Lateral Link Level Type - G:\51\33275\02.Stage 2 Model Build\GIS\Data\12.HGH_M21_linking_20170926.gdb\Avon_Bank_lines_for_lateral_linking_20180921_V094F; Standard Link - M11 to M21 links- G:\51\33275\02.Stage 2 Model Build\GIS\Data\09.Mike_Flood_Couplings.gdb\M11_M21_Standard_Link_Line_V094M_20180706.
Created by: tpreston / yhooi



Paper Size A3

0 0.5 1 1.5 2 2.5

Kilometers

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

N

LEGEND

Intercatchment Boundary Lines
- M11
- MU
- M21

- M11 Baseflow
- M11 Inflow Boundary <= 10L/s
- Avon/Estuary Hydraulic Model Extent
- M11 River Branches

Christchurch City Council
Citywide Flood Modelling - Avon

Job Number	51-33275
Revision	0
Date	04 Dec 2018

Model Boundary Conditions

Map 4

G:\51\33275\02.Stage 2 Model Build\GIS\Maps\Deliverables\ModelBuildReport_Figures\51_33275_Map4_Boundary_Conditions_Map.mxd

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Data source: M11 & MU InterCatchment Boundary Lines - G:\51\33275\02.Stage 2 Model Build\GIS\Maps\Deliverables\ModelBuildReport_Figures\Shapes for presentation maps\Map4.gdb\InterCatchment_Boundary_Lines_20180226_presentation;
M21 Intercatchment Boundary Lines - G:\51\33275\02.Stage 2 Model Build\GIS\Maps\Deliverables\ModelBuildReport_Figures\Shapes for presentation maps\Map4.gdb\Tidal_Boundary_Line_With_Arrow_20180504;
M11 Baseflow & M11 Inflow Boundary <= 10L/s - G:\51\33275\02.Stage 2 Model Build\GIS\Data\09.M11.gdb\M11_Boundary_Condition_V069_20170910 Created by: tpreston / yhooi

Prestons North
Prestons South
Alpine View
Burwood Hospital
Cameo Drive
Applecross Basin
Tait Basin
Cranford
Buller
Haughey
Cole Porter
Winters
Ellington Basin
Clearbrook
Birkdale
PS0219
PS0202
PS0205
PS0210
PS0220
PS0204
PS0232
PS0231
PS0203
PS0230
PS0207
Wainoni
Knights
Blakes
PS0229
PS0209

Paper Size A3

0 0.5 1 1.5 2 2.5

Kilometers

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

N

Legend

- Stopbank
- Seawall
- Mike 11 Network Branches
- MU Pump Station
- M11 Pump Station (Tabulated Structure)
- Mike 21 Structures Dikes
- Avon/Estuary Hydraulic Model Extent

GHD Christchurch City Council

Christchurch City Council
Citywide Flood Modelling - Avon

Basin, Dike and Pump Station Locations

Job Number 12506449
Revision 1
Date 30 Mar 2022

Map 5

N:\NZ\Christchurch\Projects\51\12506449\GIS\Maps\Deliverables\StatusReport\51_33275_Map5_Avon_Basin_Locations.mxd

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Created by: tpreston / rdecosta2

Paper Size A3

0 0.5 1 1.5 2 2.5

Kilometers

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

N

LEGEND

M11 Open Cross Section Sources
- Existing Model
- Survey Data
- Inferred From LiDAR
- Lined Channel / LiDAR

Avon/Estuary Hydraulic Model Extent

M11 Network Branches

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Citywide Flood Modelling - Avon

Job Number	51-33275
Revision	0
Date	04 Dec 2018

Post-Quake M11 Data Sources

Map 6

G:\51\33275\02.Stage 2 Model Build\GIS\Maps\Deliverables\ModelBuildReport_Figures\51_33275_Map6_M11_Sources_Map_10_1.mxd
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Data source: M11 Open Cross Section Sources - G:\51\33275\02.Stage 2 Model Build\GIS\Data\09.M11.gdb\PostEQ_M11_XS_Sources_V088_20171105 Created by: tpreston / yhooi

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Appendix B – Detailed stability standard methodology, 2017

B 1 - Detailed Stability Standard Methodology (Citywide)

1. The mass balance should have no more than 5% error, in accordance to CCC MIKE FLOOD Specification (DHI, March 2015). As of 13/12/2017 we are waiting on direction from DHI with respect to how the various model element mass balance errors should be evaluated.

2. Runs are typically evaluated from water level results saved with the standard results save time step. Oscillation max amplitude, is defined as the smaller of the value changes either side of a local extreme point, and are reported using DHI built-in tool for 1D (within MikeView) or DHI-developed tool for 2D (purpose-built tool)

3. For 1d results

   i) For MIKE 11 and MIKE URBAN stability analysis should exclude the first 25% of timesteps as they are well before the peak of the storm. (ie: for an 80 timestep result file, timestep 0 to 20 are not assessed).

   ii) Evaluate level oscillations within the MU node and M11 H-point results using the Mike View oscillation tool as described below. All sites with oscillation tool amplitude >150mm are tabulated, locations mapped (shapefile) and the 1D level timeseries data presented in XLS format. All sites have max amplitude calculated in the XLS and tabulated. All MU sites in this set that are coupled to M21 have the associated M21 timeseries results extracted from the lowest coupled cell (using “Data Extraction FM” tool with a list of named x,y coordinates) and plotted for inspection also.

   iii) Work to reduce 1D instabilities shall proceed as follows

      (a) For oscillations [150mm-300mm], then we would typically “do nothing” and report them, except for any MU sites where the associated M21 levels show level oscillations > 100mm in which case we would make one attempt to fix them and report thereafter whatever the outcome.

      (b) For oscillations >300mm, we would make an initial attempt to fix them. If not solved initially then we would typically “do nothing” and report them, except for any MU sites where the associated M21 levels show level oscillations > 100mm in which case we would make a second attempt to fix them and report thereafter whatever the outcome

4. For 2D results,

   i) Run the DHI tool on 2D water levels, defining bin values (in meters) {0, 0.05, 0.10, 0.15, 0.30, 0.50, 1, 2, >2m}, producing overall oscillation frequency summary data and a DFSU of max oscillation ‘map’ for each run

   ii) For each batch of results being evaluated

      (1) Compile the frequency summary data from each run into a single XLS table and present graphically to highlight any outliers

      (2) Convert the M21 DFSU oscillation data into a polygon geodatabase format, remove shapes with less than 50mm max oscillation and compile into a singular shape with an attribute field for each model run result. Add fields of ‘max across runs’ and ‘avg across runs’ and ‘product of max-avg’.

      (3) Attention shall be directed to runs with outlier oscillation frequencies and locations in the shape with high values in all three summary fields.

iii) No work to reduce M21 instabilities shall be undertaken unless Council's prior approval is obtained

5. The deliverables for each batch assessed are;

   (1) Transmittal note, covering email, readme file or similar defining the model runs being evaluated and tabulating both their long and abbreviated file names

   (2) 1D spreadsheets – one for each model run - that Council can play with (drop down menu for 1D timeseries graphs)

   (3) 1D shapefile, per batch of model runs being evaluated, with a separate field for each model run and a summary count of occurrences across the batch

   (4) 2D spreadsheet per batch assessed with XLS and graphical 2D statistics

   (5) 2D shapefile per batch with summary statistics on each mesh tringle

6. File / field naming: The abbreviated 10 characters naming convention defined below should be used throughout the reporting including; drop down XLS functions, shapefile field names and individual XLS file names. The file names for a batch of the 1D spreadsheets should share a common batch name prefix of not more than 12 char and the abbreviated run name as suffix.

   (1) Std Format: ###_##[T]e##h – where [T] means optional character

   (2) eg: 079_10e12h translates as Model version 079, 10% exceedance probability rainfall, 12 hour storm duration

   (3) eg: 083_20Te01h translates as Model version 083, 20% exceedance probability rainfall [T]idal run, 1 hour storm duration

   (4) In both AEP and storm duration the 0.5 values (0.5% AEP or 0.5 hr) will be represented as "00".

B 2 - Detailed Volume Balance Methodology (Citywide)

DHI confirm that the automated volume balance summaries reported by M11, M21 and MU DHI2016 version when running 3-way coupled models are incorrect. The DHI reported volume balances are contained in the model run logs. Variables below which are underlined have wording from those logs and values should be taken directly from the logs. Variables which are newly defined herein are shown in *italics*.

Errors that contributed to incorrect volume balance are as follows:

1. M11 and M21 include correction values in their calculations which should be excluded.
2. MU includes "Water generated in empty parts of the system" in the calculation which should be excluded.
3. MU does not account inflow and outflow from M11 water level boundaries which should be included.
4. It is recommended to make the above corrections before calculating an overall (Mike Flood) volume balance result

Volume balance for M11

Corrected total inflow =
C: Total inflow
- Lateral correction

Total change of volume in M11 =
B: Final volume in model area
- A: Initial volume in model area

Total inflows and runoff from M11 =
Lateral sources inflow
+ Open boundaries inflow

Corrected M11 Continuity balance =

Total change of volume in M11
- Corrected total inflow
+ D: Total outflow

Volume balance for M21

Corrected total volume from source =
C: Total volume from source
+ MIKE 11 outflow correction
+ MIKE Urban outflow correction

Total change of volume in M21 =
B: Final volume in model area
- A: Initial volume in model area

Total inflows and runoff from M21 =
D: Total volume from precipitation/evaporation
+ E: Total volume from boundaries

Corrected M21 Continuity balance =
Total change of volume in M21
- Corrected Total volume from source
- Total inflows and runoff from M21

Volume balance for MU

Total change of volume in MU =
2. End volume in Pipes, Manholes and Structures
- 1. Start volume in Pipes, Manholes and Structures

Corrected MU Total inflow volume =
3. Total inflow volume
+ C: Total inflow; MIKE URBAN CS boundary outflow (M11)

Corrected MU Total diverted volume =
4. Total diverted volume
+ B: Final volume in model area; MIKE URBAN CS boundary inflow (M11)

Corrected MU Continuity Balance =
Total change of volume in MU
- (*Corrected MU Total inflow volume - Corrected MU Total diverted volume*)

Corrected MU Continuity Balance (check) =
Continuity Balance
+ 5.Water generated in empty parts of the system
- C: Total inflow; MIKE URBAN CS boundary outflow (M11)
+ B: Final volume in model area; MIKE URBAN CS boundary inflow (M11)

Volume balance for MIKE Flood is calculated by

Corrected MF Continuity Balance =
Total change of volume in M11
+ Total change of volume in M21
+ Total change of volume in MU
- Total inflows and runoff from M11
- Total inflows and runoff from M21
- Total inflows and runoff *Specified inflows; Boundary from MU*
+ Total outflow Open boundaries outflow from M11

Notes:

1. MF Continuity Balance is not the same as the sum of the three individual model continuity balances
2. Some of the above calculations are specifically designed for this project and may not apply well to other projects.

Mass balance errors in principle are calculated from Continuity Balance ÷ the minimum of Total Inflow or Total Outflow.

Mass balance error for M11 =
Corrected M11 Continuity balance
÷

MIN(Corrected total inflow, D: Total outflow*)*

Mass balance error for M21 =
Corrected M21 Continuity balance
÷
D: Total volume from precipitation/evaporation

Mass balance error for MU =
Corrected MU Continuity Balance
÷
MIN(Corrected MU Total inflow volume, Corrected MU Total diverted volume)

Mass balance error for MIKE Flood =
Corrected MF Continuity Balance
÷
D: Total volume from precipitation/evaporation (M21)

GHD Ref: "N:\NZ\Christchurch\Water Team\33275\00.2Document Transfer\Outgoing\00_Outgoing_2018\20180212_Revised (rev4) Mass balance template ClemR to MarkP\20180209_ Volume balance_02AEP_18hr_Proposed CFM template_rev4.xlsx"

B 3 - Using the Mike View Oscillation Tool

This tool evaluates and provides summary report on instabilities in the Mike Urban and Mike 11 model results.

How to Access:

Open Mike Urban or Mike 11 model results (limit result timesteps to load as specified above).

While the 'Horizontal Plan' is active, Go to 'Tools' -> 'Compute' -> 'Instability'

Tool settings

'Data type' - Node level (Mike Urban) and Water Level (Mike 11)

(Note the flow data is not required to be assessed due the limited practicality, high level of effort required and the lower importance of flows in relation to flood levels)

'Calculation period' - All Data (control the data period for analysis when loading the result as above)

Select 'Oscillations' option

'Limit for number of extremes' – set to zero

'Duration of time interval' field – leave as default

'Min. difference for extremes' – ie: set to 0.15 or 0.30m or otherwise in accordance with the general specification

B 4 - Introducing the Mike 21FM Oscillation Tool

The M21FM oscillation tool was developed by DHI in Auckland for the Citywide modelling project with the latest revision on 13/12/2017. The tool name is "DHI M21 oscillation tool" [DHI.NZSOL.DFSU.UI.exe and DHI.NZSOL.DFSU.CLI.exe]. Its function is to inspect a time varying M21 DFSU of level or depth data assess each mesh cell separately and evaluate max oscillation for each cell over the time.duration of the result. It then produces a polygon output

with max oscillation amplitude and from this polygon output assesses the frequency of user defined amplitude bins across the extent of the mesh domain.

One executable operates with a user interface (UI) and the other tool operates from the command line interface (CLI). If the command line interface tool is run with no arguments, it reports on usage instructions.

Appendix C – Description of model setup requirements for updating scenarios, 2017

The Citywide model setups are organised based on the model file structure appended to the Model Schematisation Report. The following requirements need particular attention when setting up and running a new scenario:

1. MU naming and filing :

   i) The MU database (*.mdb) name in any particular setup matches the name of the most current MU model version (master MU model), typically with a catchment name, status ~~(preEQ or postEQ)~~ and date stamp (refer to File naming convention appended to the Model Schematisation Report). The *.mdb is saved in the ~~"Setup"~~ "Data" folder.

   ii) After pre-processing of the master MU model (creation of a *.mex file with the same filename as the MDB), the *.mex is copied and renamed to be consistent with each particular design run scenario. When the renamed copy of the master *.mex file is created, the saving timestep is also adjusted to each particular scenario by adjusting the parameter "SaveStep_HrMiSec" within the mex file.The *.mex is saved in the "Result" folder, in order to save the MU model results (*.PRF) into the "results" folder The *PRF file is named after the *.mex file and will therefore carry the name of each particular design run

   iii) In the MIKE FLOOD model setup (*.couple file), the path to the *.mex file is set to the "result" folder.

2. File paths are always relative and model setups function properly when moved to different root file path or across a different network structure, as long as the three key folders (01 Data, 02 Setup, 03 Results) are always copied together. This was tested internally by GHD by moving from a network path to a local path (C: drive).

3. Attention should be given to the maximum complete path length, in particular for MU and M11 components. The MU *.mex maximum complete path length, including file name and extension, has been identified to be 130 characters or less (the Mike 11 limitation has not been confirmed in such precision but some error message can occur if the path is too long). Usage of a mapped network drive shortcut helps achieving shorter fixed paths for the M11 part, however wasn't effective for MU. To conform to the MU requirement, we had to abbreviate various stages of the full path including the result file name (refer to file naming convention)

4. ~~Mike11 2D output. The model setup includes generation of a M11 raster output. Specifications, including file path and output names, are defined in the HD file (*.hd11). This is the reason why each model setup has a copy of the HD file saved next to the M11 simulation file (*.sim11). For a new setup, one should not forget to update the M11 2D output in the HD file.~~

Appendix D – GIS raster origin and extents for DFS2 production, 2017

Tim Preston

From: Tim Preston
Sent: Wednesday, 5 July 2017 3:35 p.m.
To: Passier, Mark (Mark.Passier@aecom.com); 'sally.williams@aecom.com'
Cc: 'Clemence Rolin (Clemence.Rolin@ghd.com)'; Jeff Tuck; Yanni Hooi; John McArthur
Subject: GIS raster origins and extents

CompleteRepository: 5133275
Description: Citywide Flood Modelling (LDRP044)
JobNo: 33275
OperatingCentre: 51
RepoEmail: 5133275@ghd.com
RepoType: Job

Hi Mark / Sally,

We would ask that AECOM match the following approach so that the project raster grids are compatible and can readily be overlaid and integrated. (We will include this in the Avon 'build/calibrate' report as an appendix item).

======
In order to generate a tidy final 2m grid size raster product, our GIS raster extents (for both M21 and M11) are rounded to the nearest 100 m (while encompassing the entire results area).

Note that to achieve this using DHI DFSU to DFS2 tools and the M11 Raster save at runtime process (both of which define cell-centroid origin rather than extents origin) the DHI origins need to be defined to the nearest 101 m and the number of elements in each direction defined to the nearest count of 50.

For the Avon catchment GHD has thus defined the following for DHI DFS2 production:

- Cell size: [2, 2]
- Origin M21: [1553701, 5173601]
- Number of Elements: [13700, 7350]

The resulting GIS raster extents of Avon catchment for M21+M11 integrated are;

- Top: 5188300
- Left: 1553700
- Right: 1581100
- Bottom: 5173600

(Note the the extents of the M11 only intermediate raster are reduced as required only to fit around the M11 result extents).

Regards

Tim Preston
Senior Water Industry Engineer

GHD
T: +64 3 3780913 | Mob: +64 27 6414301 | V: 510913 | E: tim.preston@ghd.com
Level 3, 138 Victoria St, Christchurch 8011, New Zealand | www.ghd.com
WATER | ENERGY & RESOURCES | ENVIRONMENT | PROPERTY & BUILDINGS | TRANSPORTATION

Please consider our environment before printing this email

Appendix E – Design vs Calibration model setup memo, 2017

Memorandum

22 December 2017

To	John McArthur		
Copy to	Tim Preston		
From	Jenna Stuart	Tel	VOIP 510998
Subject	Avon Model - Calibration Geometry	Job no.	51/33275/03

1 Purpose

This memo outlines the atypical differences in model inputs between the 'calibration' events (March 2014 calibration and June 2013 validation) and post-quake design runs (ED2014), for enabling quick translation between the three setups.

2 Mike 11 Model Differences

The March 2014 geometry differs to the design model in that it is intended to replicate anecdotal spilling and flooding near Horseshoe Lake and observed blockages at the Armagh Street and Colombo Street bridges at the time of the event, as described further in the following sections.

The June 2013 geometry matches the ED2014 model.

2.1 M11 Bank Markers / Lateral Links

March 2014 calibration work resulted in estimated bank levels below an approximate peak water level of 10.9, as listed in Table 1. Design runs and June 2013 are specified to have stopbank levels of 11.2 universally. In the 2D side of the lateral links, Mike21 mesh elements are identical between calibration runs and the design model.

Table 1: Bank Marker Level Differences between Calibration and Design Events

Branch	XS Chainage	Bank Marker	Mar2014 Event	ED2014
AVON.AVON	21018.16	1	10.930	11.200
AVON.AVON	21160.79	1	10.838	11.200
AVON.HORSESHOE	212.85	3	10.000	11.200
AVON.HORSESHOE	249.23	3	10.700	11.200
AVON.HORSESHOE	280.67	3	10.900	11.200

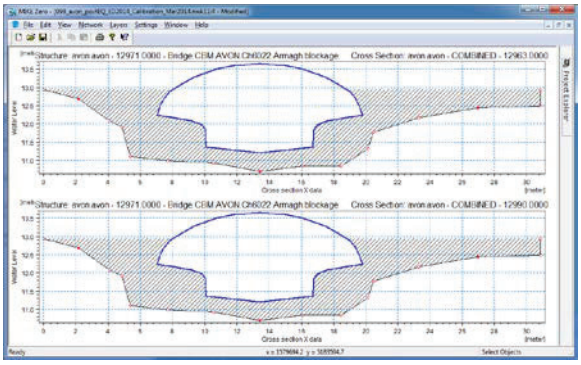
GHD Limited
Level 3 138 Victoria Street Christchurch Central 8013 PO Box 13468 Christchurch 8141 New Zealand
T 64 3 378 0900 **F** 64 9 370 8001 **E** chcmail@ghd.com **W** www.ghd.com

2.2 M11 Structures

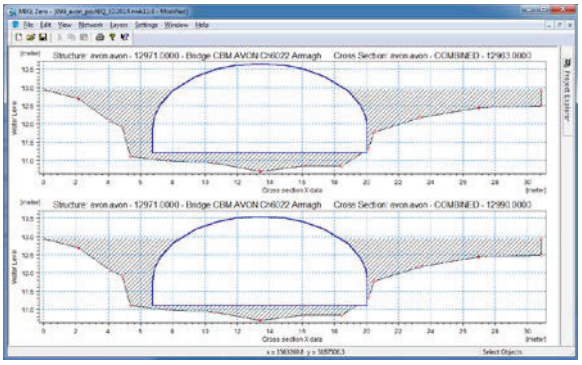
The bridge blockages applied in the March 2014 calibration event derive from files provided by DHI (N:\NZ\Christchurch\Water Team\33275\00.2Document Transfer\Incoming\20170126 DHI Avon calibration res11 and setup_ Antoinette to Tim). Differences between this and the 'unblocked' bridges (i.e. design model) are illustrated below. Refer to the Avon Model Log and email from John McArthur to Yanni Hooi on Monday 15 May 2017, 1:31 p.m. "Colombo street blockage correction" for further information.

Bridge.CBM.AVON.Ch6022.Armagh

March 2014 Validation Event

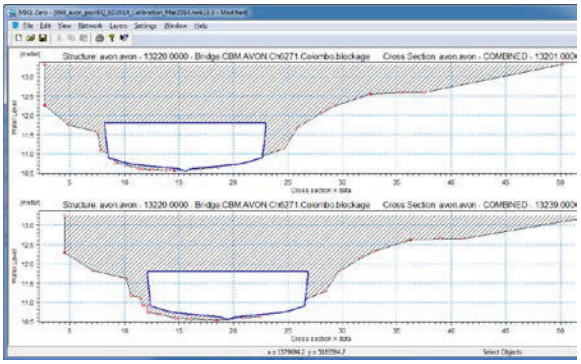


ED2014 Model

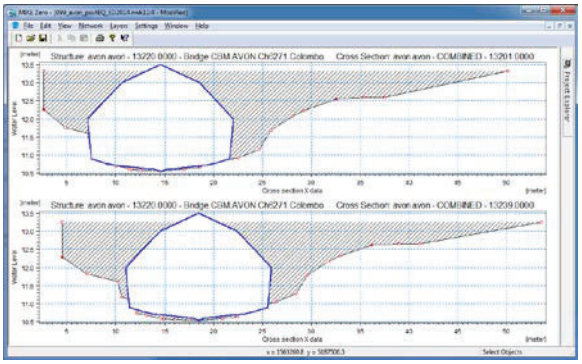


Bridge.CBM.AVON.Ch6271.Colombo

March 2014 Validation Event



ED2014 Model



Controllable or operable structure settings do not change between calibration/validation events and post-quake design runs. This includes pump station setup, such as PS205, Horners control gate and the outlet pipe from Winters Basin.

2.3 M11 Boundary Conditions

The only variation between design and calibration Mike11 boundary conditions is the inclusion of the Styx-Horner gauge data for calibration/validation event levels, which replaces a constant level of 14 m used in design runs.

Table 2: Water Level Boundary Condition at avonstyx.horner 2134.4

June 2013 Event	March 2014 Event	ED2014 Model
Time series[1]: 66471_styx_horner2013.dfs0	Time series[1]: 66471_styx_horner2014.dfs0	Constant Level: 14 m

3 Mike 21 Model Differences

3.1 M21 Infiltration

The infiltration files differ for each event. The following list indicates the current infiltration files to use for each scenario:

- **March 2014 Event:** Soil_infiltration20170421_for2014 _pervious_CROPPED.dfs2
- **June 2013 Event:** June2013_60pc_Infiltration_ Pervious_AVON_Cropped.dfs2
- **ED2014 Design Runs:**
    - Infiltration_Avon_postEQ_ED2014_ 01to04hr_10m_mmday.dfs2
    - Infiltration_Avon_postEQ_ED2014_ 06to18hr_10m_mmday.dfs2
    - Infiltration_Avon_postEQ_ED2014_ 24to48hr_10m_mmday.dfs2

3.2 M21 Wind

The table below illustrates the file names for wind inputs for each of the events:

Table 3: Wind Conditions

ED2014 Model,	June 2013 Event	March 2014 Event
Wind_Estuary_10ms_SW_225.dfs2	Wind_XY_2013_timed.dfs2	Wind_XY_2014_timed.dfs2

The dfs2 files derived for the two historic events are based on recorded wind speed and direction at the New Brighton Pier wind station (37654).

4 Event-specific Data

In addition to the abovementioned differences, the usual event-specific data (i.e. rainfall, tide, initial conditions/hot-start, simulation periods etc.) differs between each scenario, but have been excluded from this memo for brevity.

Kind Regards,

Jenna Stuart
Water Resources Engineer

[1] Located in N:\NZ\Christchurch\DHI Models\33275\03.Stage 3 Calibration\Avon\01Data\M11\C_BND

Appendix F – Anomalies register

Avon Anomalies Register

Item No.	Date registered	Defined by who	Issue Name Descriptive	Risk for floor levels 3-high, 2-med, 1-low, 0-zero	Risk comment (TJP Nov2021)	Model component (MU, M11, M21)	References (docs, emails)	Resolution	Current status	Date Resolved	Reported to Council reference	Council acknowledgement	Model reference for location	Mapped to GIS polygon	Backup Information?
1	5/12/2016	Chamila	SwPipe.73931 missing 300mm	1	A typical non-descript shortish length of 300mm pipe with alternative flow paths	MU	Email from TJP 20/9/2016 3:56pm. Presented tech meeting 20161129. Closed out 20161205 TJP.	Do nothing - leave it out	Closed		email from TJP 29/11/16 7:30am	NA	Council's GIS CCC GIS ID SwPipe.73931	Yes	YES
2	5/12/2016	Chamila	Mt Pleasant causeway six pipes, need to transfer from M11 to MU no data source attributes	0	Attributes issue only - no impact on results	MU	Email from TJP 29/9/2016 1:52pm. Presented tech meeting 20161129. Closed out 20161205 TJP.	Do nothing - leave attribution incomplete	Closed		email from TJP 29/11/16 7:30am	NA	GHD MikeUrban Model, MUID: ESTY.Pipe.CCCGIS.1733 ESTY.Pipe.CCCGIS.1714 ESTY.Pipe.CCCGIS.776 ESTY.Pipe.CCCGIS.2821 ESTY.Pipe.CCCGIS.52112 ESTY.Pipe.CCCGIS.2866	Yes	YES
3	5/12/2016	Clemence	Mt Pleasant two road crossing pipes toward Shag Rock	1	A very localised issue with ponded flow not getting across the road into the estuary due to omission of pipes	MU	Email from TJP 17/11/2016 10:15am. Presented tech meeting 20161129. Closed out 20161205 1:25pm TJP email .	Do nothing - leave them out	Closed		email from TJP 29/11/16 7:30am	NA	Council's GIS CCC GIS ID SwPipe.151, SwPipe.81684	Yes	YES
4	5/12/2016	Clemence	Greenhaven Drive Drain open drain no survey data	1	A somewhat redundant piece of drainage, between Snellings and No2 drains, now at least half overtaken by new drains as part of Prestons development	M11/MU	Email from TJP 17/11/2016 10:15am. Presented and closed out tech meeting 20161129.	Do nothing - drain to remain as M21 representation only	ClosedApproved		email from TJP 29/11/16 7:30am "suggested agenda and"	Granted during tech meeting 20161129	Council's GIS WcWaterCourse: CCC GIS ID Greenhaven Drive Drain	Yes	YES
5	5/12/2016	Clemence	Proctor Grant missing watercourse data	2	In some flood types local flood level might be exaggeraged by 400mm as a result of this anomaly	M11	VPR 50	Do nothing	ClosedApproved		VPR50 submitted by Helen email 28/11/2016 "Grants Road / Proctor Street"	VO50 issued approved Catherine Campbell 10/1/2017	GHD Mike11 Model: AVON.PAPANUI Chainage 566m	Yes	YES
6	5/12/2016	Clemence	Warwick Street Drain alignment	1	Pipe is route is not followed, should be a non issue for results, but haven't checked the correct hydraulic length, so might be a minor issue.	MU	Email from TJP 17/11/2016 10:15am. Presented tech meeting 20161129.	Length will be corrected, alignment left as is	ClosedApproved		email from TJP 5/12/16 4:00pm	TomP email 7/12/16 8:55am plus Peter C in email chain below	GHD MikeUrban Model, MUID: AVON.Pipe.CCCGIS.8087	Yes	YES
7	5/12/2016	Clemence	Fendalton Drain alignment	1	Pipe is route is not followed, should be a non issue for results, but haven't checked the correct hydraulic length, so might be a minor issue.	MU	Email from TJP 17/11/2016 10:15am. Presented tech meeting 20161129.	Length will be corrected, alignment left as is	ClosedApproved		email from TJP 5/12/16 4:00pm	TomP email 7/12/16 8:55am plus Peter C in email chain below	GHD MikeUrban Model, MUID: AVON.Pipe.CRM.13_M	Yes	YES
8	14/12/2016	Clemence	Data source incorrect	0	Attributes issue only - no impact on results	MU	Manhole IL data source shows SCIRT but should be "Inferred" as confirmed by AECOM during meeting 29/11/16.	Do nothing	Closed		email from TJP 29/11/16 7:30am "suggested agenda and", item d1	NA	GHD MikeUrban Model, MUID: AVON.Inlet.CCCGIS.42571	Yes	YES
9	2/2/2017	Tim Preston	Shirley Str 675 dia pipe GIS ID 16470 - missing from MU and M11 models	1	A key cross connection feature, seems to have been fixed by GHD to GIS records, but not cross checked against other potentially better geometry definitions, noting a weir is present to keep DWF in the channel	M11/MU	Email from TJP 20/9/2016 3:56pm, item 4. MarkP reply 2/22017 10:35am.	Added the pipe into the M11 model based on GIS data mainly inferred levels and with some As Built and photo evidence of associated bifurcation weirs	Resolved	Date unknown, found 'done' on 20250401 TJP, probable date done early 2017 during Oli era when logging was less robust. Model results 'today' show the major portion of flood flows discharging out of Shirley Stream here into MU pipe network thru M11 closed XS 675dia.	email from TJP 2/2/17 12:45pm "MU data source and accuracy"	NA	New branch "avon.brigdiv", only 24m long	Yes	YES
13	31/3/2017	Tim Preston	Monavale pipe missing feed to small southern branch	0	More of a cosmetic data completion question - likely small pipe in a much larger river system - trivial to flood levels as noted in the registration email	M11	TimP email 31/3/2017 5:55pm	Do nothing	Closed		TimP email 31/3/2017 5:55pm	email TomP 3/4/17 2:57pm	Avon11 branch. Chainage 34m	Yes	YES
14	31/3/2017	Tim Preston	Strowan cross connection weir	2	At low flood levels, these two pipes can share flow using the weir system, but as the weir is not modelled then the model can't share flow until there is higher flooding levels to share flow via M21. Result in calibration was higher flood levels south of here and lower ones north. If connected better they wold equalise and mitigate flooding in the south. At major flood levels the model will be good	M11/MU	ChrisM 9/3/17 14:38 supplying As Built weir drawings	Weir will not be deliberatly represented in modelling. Model might replicate its function to some extent by accident.	ClosedApproved		email from TimP 10/3/17 8:48am	email PeterC 10/3/2017 9:10am	MU pipe GIS_ID 18338 under railway near Strowan Rd	Yes	YES
19	4/4/2017	Tim Preston	Styx 01 cross section	2	Fictional cross sections could wrong by a lot, with impacts upstream and downstream, negative or positive depending on the type of wrongness	MU	Tim's email 4/4/17 9:40am "Styx 01 cross section - anomaly registration approval request"	Do nothing	ClosedApproved		Tim's email 4/4/17 9:40am "Styx 01 cross section - anomaly registration approval request"	TomP email 13/4/2017 12:40pm	Refer email screenshot several locations in Cranford overlap area	Yes	YES
21	4/4/2017	Tim Preston	Pedestrian roads left out of mesh geometry eg: Cashel St	1	Model is coarse in these areas and lacks finesse and detail, but probably still close to good on average	M21	TimP email 4/4/2017 12:30pm	Do nothing	Closed		TimP email 4/4/2017 12:30pm	NA	Cashel St and High St pedestrian malls	Yes	YES
22	4/4/2017	Tim Preston	Bullers gauge -220mm anomaly	1	Likely immaterial as Buller gauge was largely disregarded in the model calibration. It could imply a model geometry error, which could be material, but note that the channel has since been rebuilt and not remodelled so the issue is now very minor historical	M11	TimP email 31/3 9:12am 'workshop actions', items 6 + A.	Force adjust gauge in short term note for future review with NIWA data generally 17/1/2022 TJP comment that since Buillers gauge is now demolished and the old gauge wasn't used, that this issue has become irrelevant	Resolved	17/1/2022	TimP email 31/3 9:12am 'workshop actions', items 6 + A.	TimP email 31/3 9:12am 'workshop actions', items 6 + A.	Buller Creek	Yes	YES
24	21/4/2017	Tim Preston	Old No2 Drain geometry	2	Poor quality survey data in a widespread area of open channel is likely to impact model results in the area noticably.	M11	Tech meeting minutes 20/4/2017 and TimP email 21/4/2017 9am	Some improvements as noted in the email, other issues no action for now	ClosedApproved		TimP email 21/4/2017 9am "Old No2 drain anomalies"	TomP email 20181113 "Old No2 drain anomalies"	Branch: avon.oldno2dn	Yes	YES
25	3/5/2017	Tim Preston	Avon CBD stopbank XS	1	Main issue is XS too wide (skew not factored in) so the model river capacity is overstated locally. This may have been corrected - refer abbreviated model log records Oli 13/4/2017 NWK section and prior to that in the XNS section. The Gloucester bridge skew issue is also mentioned in the discussion but seems to have been fixed anyway according to Anom#16 records.	M11	email ex Oli 3/5/17 "avon gloucester" and same date "QA notes today"	Recommended do nothing	ClosedApproved		TimP email 20180925_1025 "avon gloucester XS"	TomP email 20181130 "avon gloucester XS U/S anomaly #25"	avon.avon XS at chainage 12816	Yes	YES
26	5/4/2017	Tim Preston	Moorhouse Ave pipe disconnect	1	Data confusion results in the model being wrong wrt specific assets functioning, but would be very close to the same in performance	MU	MarkP email 5/4/17	conclusion SCIRT survey was confused, straight through is abandoned dogleg is current CCC GIS is good, model should be revised later but is still 'good enough'.	ClosedApproved		TimP email 20180925_1025 "Moorhouse Colombo Street - anom #26"	TomP email 20181113 "Moorhouse Colombo Street - anom #26"		Yes	YES
27	1/5/2017	Tim Preston	Paparoa diversion inconsistencies in model vs various input data version and esp vs Opus DT model	2	Difficult to assess given unquantified differences between model and best information, but the flow spilt is a reasonably significant site, capable of influencing flow volume and level significantly in both downstream directions. Email notes 20170504 indicate decent confidence at the split itself.	M11	TimP email 20170504 1722 "Upper Dudley Creek Diversion" also various points noted 20170505 1349 "Avon Calibration recommendation preview - Flockton part"	Major model revisions undertaken, with conclusions reported 1/4/25 as a new appendix to the Status Report. Major improvements, not perfect but subtantially resolved	Resolved	1/4/2025	TimP email 20180925_1220 "Upper Dudley Creek - Paparoa Diversion - Anom #27"	NA	avon.dudley chainage 7765	Yes	YES

Avon Anomalies Register

Item No.	Date registered	Defined by who	Issue Name Descriptive	Risk for floor levels 3-high, 2-med, 1-low, 0-zero	Risk comment (TJP Nov2021)	Model component (MU, M11, M21)	References (docs, emails)	Resolution	Current status	Date Resolved	Reported to Council reference	Council acknowledgement	Model reference for location	Mapped to GIS polygon	Backup Information?
30	16/5/2017	Jeff Tuck	New subdivision areas which only some model input data includes	2	Typical issue with slow input data currency, model then (and still as of Dec2021 part updates) lacks Lady Isaac Retirement village in general other than per the LiDAR levels update, basins, roads and pipes generally absent from the model in terms of current flood risks model will be lousy in this vicinity, probably a minor impact beyond the new development extent	MU	TimP email 20180925_0755 "Avon anomaly #30 - West of Ellington Basin" Refer also Email: Jeff Tuck; MU-M11 Anomalies; 20170516_1812, second part of email (first part was corrected)	Do nothing	Closed		TimP email 20180925_0755 "#30 - West of Ellington Basin"	NA	Buller Stream - Lady Isaac Rd MUID AVON.Access.CCCGIS.19506 (node) (West side)	Yes	YES
31	25/5/2017	Tim Preston	M11 branches (or parts) with negative flow directions.	0	Cosmetic and modelling convenience only	M11	TimP email 20181002_1358 "M11 branches - flow direction - Avon anom #31" and John's email chain therein	Do nothing	Closed		TimP email 20181002_1358 "M11 branches - flow direction - Avon anom #31"	NA	esty.cuthdrso avon.shirley1 avonstyx.rossiter (west part of)	Yes	YES
32	21/8/2017	Tim Preston	Upper Wairarapa stream M11 invert levels pre quake	0	Discrepancy identified in pre-quake model only, now irrelevant	M11	TimP email 20170818 6pm and Jenna's email in the chain below	Do nothing as it impacted only the pre-quake model which is now abandoned (long ago)	Resolved	1/4/2025	TimP email 20170818 6pm	NA	AVON.WAIRARA at chainage 0-450m	Yes	YES
33	31/8/2017	Tim Preston	Dudley lower M11 oversized cross sections not coordinated with M21	2	Significant extent of double counted M11+M21 volume and capacity, probably modest flood level impact but could be notable in some types of flood condition	M11	TimP email 20170831 7pm	Resolved within LDRP097 12506449 scope. Some proximate or subsequent new anomalies remain - refer Anom's 64/90, but the primary issue for Anom 33 was sorted. [Tim Preston note 310/7/2023 referencing our model results].	Resolved	22/7/2023	TimP email 20170831 7pm	Tech meeting 20170907 minutes	Various along the lower Dudley area down to Avon River	Yes	YES
34	5/9/2017	Tim Preston	Survey point error Lake Kate Sheppard	0	Poor quality survey data in M11 bank marker level but will not affect model as we use M21 lateral linking	M11	TimP email 20170905 5pm	Do nothing	Closed		TimP email 20170905 5pm	NA	avon.katshep, chainage 646, left (west) bank	Yes	YES
37	13/9/2017	Tim Preston	Travis swamp bridge width	1	Similar to above a general concern but no specific risks or actionable points presently known. Would need to get the CCC surveyor reports on requested QA findings to assess properly. Not sure whether the requested QA was even done or where to find it in CCC if it was.	M11	TimP email 20170913-1117, "Travis Swamp bridge survey data incident". Refer also anomaly #44	Do nothing	Closed		TimP email 20180924_1655 "Bridge survey data - width vs length - Avon anom #37"	NA	avon.travissw Ch 979.56	Yes	YES
38	13/9/2017	Tim Preston	Urban open channel material errors	0	More of an administrative concern with model functional impact predictibly minor as roughness remains near enough correct	MU	John McA email to TimP 20170913-1305, noted two locations where input data and model show natural channel but parts of these are known to be timber lined	Do nothing	Closed		TimP email 20180924_1520 "MU channel materials - Avon anom #38"	NA	AVONSTYX.Access.CCCGIS.80 - 100, AVON.Wc.avon.knightsb.391 - 585	Yes	YES
39	20/9/2017	Tim Preston	Pipe missing from MU model	1	Missing pipe but likely that M21 will deliver flow well enough the same. Checked CFM results and no signs of suspicion, 10yr result is dry, 50yr result is drowned. Seems ok. Watercourse is M11 modelled	MU	TimP email to CCC 20170920-1315 "Missing Pipe anomaly report"	Do nothing	Closed		TimP email to CCC 20170920-1315	NA	MUID AVON.Outlet.CCCGIS.16485	Yes	YES
40	20/11/2017	Clem Rolin	Open channel missing from M11 model at Angela Stream branch into Travis Swamp	0	Email records only likely hdyraulic relevance is during low flow conditions, otherwise cosmetic/administrative inconsistency only	M11	TimP email to CCC 20180925_0805 "large outlet to Travis Swamp" incl Clem chain input email to TimP 20/11/2017 5:09 p.m.	Do nothing	Closed		TimP email 20180925_0805 "large outlet to Travis Swamp"	NA	AVON.Outlet.CCCGIS.284	Yes	YES
42	19/12/2017	Catherine Campbell	Hereford St Pipe diameter and invert level data incorrect	1	Model diameter too small in places, but Opus assessment shows capacity is ample even with the unrealistic small size, but their evaluation relies on an unconservative exaggerated slope and lowered downstream invert level. If all corrected it would seem unlikely to influence flood risk results	MU	Catherine's email (fwd from Ben Pascoe source) 20171219-1306	Do nothing	Closed		NA - Council reported to GHD	NA - Council reported to GHD	tbc	Yes	YES
43	18/1/2018	Tim Preston	Pleasant Point Domain Drain, downstream pipe ommitted from M11 model due to bad GIS input data	1	Bad data but reasonable resulting model approximation to reality - unlikely to drive risks as the area affected is mainly tidal risk anyway	M11	TimP email to CCC 20180118-1645	Do nothing	Closed		TimP email to CCC 20180118-1645	NA	esty.plsdmdrn, chainage 115m to 287m	Yes	YES
47	21/2/2018	Tim Preston	High frequency of MU open channels with 0.2m depth	0	This was essentially resolved by email "Query regards to 0.2m CRS in MU - Resolved" 20180221, no issue	MU	TJP email 20180221-1424 "Query regards to 0.2m CRS in MU"	Do nothing	Resolved	21/2/2018	TimP email 20180924_1320 "MU open channel depths - Avon anom #47"	NA	Various MU open channel links - refer email list	Yes	YES
49	23/2/2018	Tim Preston	Lake Kate Sheppard control structure model geometry is too big due to survey data error.	2	Model flow path is several times bigger than best understanding. Actual tidal in and out flows through here will be much more limited in reality. Upstream Lake levels in the model will be less protected from king tide events. Impact could be variable depending on the type of event. Hard to anticipate this without fixing it and running model.	M11	Refer VPR64 28/9/2017 (not approved) and TJP email 20170822-1707 and tech meeting minutes 20170824.	Model geogmetry updated to match new understanding. Actioned Yanni	Resolved	20/8/2019	TimP email 20180924_1220 "Lake Kate Sheppard query - anomaly item #49"	VO64 Catherine email 20180627 "Agreement on VPR64"	avon.katshep, chainage 45	Yes	YES
50	6/3/2018	Tim Preston	Provisional item: Overhang survey cross section errors. General risk that survey field work and/or data processing did not put cross section points in correct sequence. Specific instance found by incident at No1 drain. GHD recommended that CCC manually review any survey work where point sequence was modified after field survey by office processing. CCC declined.	1	Expected rare risk, probable hydraulic impact where realised is likely minor, geography unknown and would need CCC survey details to discover. Almost nil risk.	M11	Preston email 20180301-1230 No1drain problem; Catherine email 20180306-1606 enclosing one off correction from surveyors but there is the broader question of how many more locations to ask.	Do nothing	Closed		Preston email 20180301-1230 No1drain problem	Tech meeting 20180320	avon.no1drain	Yes	YES
51	15/6/2018	Jenna Stuart	2x updated CRS found to have inexact extensions upon review	0	A pedantic report but declared as no material bearing on results	MU	Yanni email to Jenna 20180502-1424 "MU open channel setup". Subsequently decided to register as anomalies rather than make corrections.	Do nothing	Closed		Tim email 20180924_1210 "MU open channel depths - Avon anomalies #51"	NA	CRS ID: LiDAR2011.6050, LiDAR2011.17079	Yes	YES
52	18/6/2018	Jenna Stuart	31x river links found to be generating flow (due to misaligned inverts).	1	Numerous cases were found where MU invert was below M11, a large fraction of these might be real, but DHI software can generate water in these cases. A 1 hr 'dry' model run was completed to identify which of the locations did generate water. 31 were found with >1m3 generated in 1 hr (>0.3 l/s minor). 3 of these had >100m3 generated ie: >30 l/s but these were fixed. Remaining 27 locations will still have between 0.3 - 30 l/s of water generation in 1D network, generation is likely to be overwhelmed and not happen when flood flows occur. Risks to flood levels are very minor.	MU-M11	As instructed during tech meeting 20180605_1530. Concluded with Tim email 20180621-1134 "MU-M11 link dummy weir selection"	3x locations dummy weirs implemented in MU; recommend survey associated 3 links; recommend update St. Albans dummy weir (Webb St bypass) with dimensions per Opus Dudley Ttibutaries updated model; remaining locations 'do nothing'.	PartResolved		Tim email 20180621-1134 "MU-M11 link dummy weir selection"	NA	Various MU d/s end locations refer email attachments	Yes	YES

Avon Anomalies Register

Item No.	Date registered	Defined by who	Issue Name Descriptive	Risk for floor levels 3-high, 2-med, 1-low, 0-zero	Risk comment (TJP Nov2021)	Model component (MU, M11, M21)	References (docs, emails)	Resolution	Current status	Date Resolved	Reported to Council reference	Council acknowledgement	Model reference for location	Mapped to GIS polygon	Backup Information?
53	15/8/2018	Tim Preston	avon.mckenzie survey levels and bank marker type anomaly	1	Model here is pretty broken, could cause exaggerated flooding in some situations, limited results inspection suggests it doesn't and likely won't, but we haven't checked everything.	MF	Preston email 20180921_0930 "Lateral link HGH M21 McKenzie", noting that survey levels and LiDAR levels are inconsistent by at least 0.5m and the right bank marker remains as HGH type but both should be M21 type.	Fixed by Clem 23/1/2018	Resolved	23/1/2018	TimP email to CCC 20180924_1125 "Lateral link HGH M21 Avon.McKenzie - anomaly #53"	NA	avon.mkenzie branch - all chainages,	Yes	YES
54	19/10/2018	Tim Preston	Dudley diversion at Philpotts	2	The known result impacts of level would only rate a 1, but the qualities of the anomaly are more major, the area in the scheme of the Avon is big/important, and the risks of major impacts on water levels in events not yet inspected are appreciable	MF	Plan geometry imperfection and Invert level circumstances leads to anomalous water level model results in low to medium flow conditions	Invert levels adjusted 14/1/2022 Yanni	Resolved	14/1/2022	TimP email to CCC 20181019_1800 "Dudley diversion at Philpotts - Anomaly 54"	TomP email 20181113 "Dudley diversion at Philpotts - Anomaly 54"	AVON.DUDLEY10 chainage 10036 (d/s end)	Yes	YES
56	6/11/2018	Tim Preston	Anomalies with M11 raster generation	0	Bascially just reports on a possible way that instabilities might be evident in the (in this case M11 raster) results. It is a subset of the stability issue overall and if that subject is well addressed generally then this anomaly has only cosmetic interest	M11	Preston email 20181107_1340 "Anomalies M11 result mapping - Anom 56"	M11 raster mapping was abandoned years ago so this anomaly is no longer relevant (TJP 20250401)	Resolved	1/4/2025	Preston email 20181107_1340 "Anomalies M11 result mapping - Anom 56"	NA	avon.avon near Anzac Drive and Lake Kate Shepherd	Yes	YES
57	13/11/2018	Tim Preston	Anomalies with estuary water level gaugings - a datum discrepency between Bridge St and Ferrymead is suspected	1	This anomaly has been substantially superseed by HKV 2021 data review and adjustments. It would have otherwise played only a minor part in the Citywide model setup as this data was really only used for the estuary wind speed estimate which loosely influences the design model results around Bridge St especially. But the simplicity of estuary and wind modelling is more important than this apparent data issue.	M21	Refer also anomaly #23 TJP email 20170517_1340 "Design Wind Across Estuary recommendation" N:\NZ\Christchurch\Projects\51\33275\03.Stage 3 Calibration\Technical\20170427 Estuary Tides and Wind\20170508 Estuary and Sea Water Levels.xlsx	Do nothing	Closed		Preston email 20181113 "Avon anomaly item #57 - estuary recorded water levels"	NA	Across Bridge St and Ferrymead gauge sites	Yes	YES
63	27/4/2021	Tim Preston	Waitaki St PS has been inactive for some years and is due to be decommissioned soon. There are likely implications for local drainage.	2	Impact will be that for 2020 model results there will be some areas helped by the model active PS that in reality would flood more. Impact generally minor given the redzone environment. If Knights Drain PS capacity is exceeded and material flooding builds up around then Waitaki PS will fictionally remove it, so some risk.	MU	12506449 email subject: "Waitaki St PS" 22/4/21, Preston/Golden/Jan.Lenselink	Switched off by Rowan, set start levels to abstract high value	Resolved	20/12/2021	12506449 email subject: "Waitaki St PS" 22/4/21, Preston/Golden/Jan.Lenselink	12506449 email subject: "Waitaki St PS" 22/4/21, Preston/Golden/Jan.Lenselink		Yes	YES
74	24/9/2021	Rowan De Costa	Prestons subdivision model incomplete MU pipes model is generally done. New and modified open channels (including mesh blockout locations) have not been updated. Road mesh has not been formed, nor Burwood Hospital basin mesh, (top and toe line definition). There are approx 20 fictional MU to M11 couplings used to connect the MU outlets to the old Snells Drain alignment.	2	Low flow conditions should work ok, and mesh levels are generally good to reflect extra high flood conditions, but there could be significant distortions in results for intermediate flood conditions.	All	Refer to https://projects.ghd.com/oc/NewZealand1/ldrp097baselinemodel/Delivery/Documents/20210705_LDRP097%20Baseline%20Modelling_VO11B_Signed.PDF	Future model build once good open channel data is obtained. Not LiDAR was considered unsuitable due to ponded water and lack of reasonable invert identification.	ApprovedClosed		As expected in scoping of VO11 (12506449)			Yes	YES
75	22/9/2021	Rowan De Costa	At the south corner of Cranford Basin, various data is contradictory, like the rest of Cranford area there is evidence that the local drainage 1D model is outdated and inconsistent with the ground surface.	2	In the localised area up to 150m upstream (south east) of the crux at the bund, the 1D model is outdated and inconsistent with current ground levels. Impacts on the basin levels are predicatbly trivial, but there are likely noticable impacts around the local drainage catchment upstream, hard to anticipate.	M11, MU	Rowan email "Cranford basin water course layer" 20210916 Chris Treddenick suppled prelim as builts	Future clarify actual pipe and channel details and update model	Closed		TimP email to Tyler 12506449 "South Cranford basin" 20220209		South corner of Cranford basin, avon.cranest2 chainage 107.	Yes	YES
79	8/10/2021	Rowan De Costa / Tim Preston	The invert level of timber lined channel Avon.Snelldr, along Prestons road, looks likely to be lower than in the model, and if so model predicted flood levels will be overstated.	2	This is an area with increased importance due to subdivision and the old rural data (LiDAR approach) is really not good enough. Open channel capacity could be substantially choked compared to reality and impact results locally at least.	M11	Refer emails 12506449 "Prestons Road invert gap anomaly" TimP 20201011 and "QA anomoly (Avon.Snelldr invert levels)" sent by Rowan.D 09/10/2021 2:16 pm Refer also anom item 73	Recommended for future Council investigation to get good level data	Closed		Refer email 12506449 "Prestons Road invert gap anomaly" TimP 20211011		Avon.Snelldr chainage 702~892	Yes	YES
81	16/11/2021	Rowan De Costa	Single pipe in Prestons area has its DS invert level well above ground level. This is causing the mesh element to be raised by 2.5m.	2	This is a minor upstream area, which remains still only partly modelled due to the new subdivision and incomplete data yet, but locally this anomaly will change flood behaviour a bit as the pipe is effectively disabled.	MU+M21	TimP email to Tyler 12506449 "Waitakiri Sq pipe above ground" 20220210	The MU inverts have been corrected to new GIS levels by Yanni 14/1/2022 (reduced d/s by 5m, reduced u/s by 1m). The next time the mesh levels are regenerated the M21 levels will be fixed. (Not corrected immediately since we have too many copies of the mesh at present). The pipe remains dysfunctional due to the high M21 ground level downstream.	Resolved	1/3/2024	TimP email to Tyler 12506449 "Waitakiri Sq pipe above ground" 20220210	Not required	AVON.Pipe.CCCGIS.68685 Intersection of Bluestone Dr, and Waitkiri Square, Prestons	Yes	YES
82	14/1/2022	Tim Preston	Lateral link mass balance loss, flood levels be low by 300mm+, resolved May2021, 50mm impact extent, refer GHD memo dated 23/12/21	3	Has three M21 cells with flood level exageration of more than 300mm	MF	Memo "Mass balance error corrections and validation" dated 23/12/21, published 12/1/21 (Preston sent from 12506449 project scope)	Model edits to insert cross sections so as to eliminate the problem	Resolved	1/5/2021	Memo "Mass balance error corrections and validation" dated 23/12/21, published 12/1/21 (Preston sent from 12506449 project scope)	Not required	AVON.DUDLEY 230-270	Mapped around 50mm impact extent	
83	14/1/2022	Tim Preston	Lateral link mass balance loss, flood levels be low by 50mm+, resolved May2021, 50mm impact extent, refer GHD memo dated 23/12/21	1	Has three M21 cells with flood level exageration of more than 50mm	MF	Memo "Mass balance error corrections and validation" dated 23/12/21, published 12/1/21 (Preston sent from 12506449 project scope)	Model edits to insert cross sections so as to eliminate the problem	Resolved	1/5/2021	Memo "Mass balance error corrections and validation" dated 23/12/21, published 12/1/21 (Preston sent from 12506449 project scope)	Not required	AVON.WINTERS11 119	Mapped around 50mm impact extent	
84	14/1/2022	Tim Preston	Lateral link mass balance loss, flood levels be low by 200mm+, resolved May2021, 50mm impact extent, refer GHD memo dated 23/12/21	3	Has three M21 cells with flood level exageration of more than 200mm	MF	Memo "Mass balance error corrections and validation" dated 23/12/21, published 12/1/21 (Preston sent from 12506449 project scope)	Model edits to insert cross sections so as to eliminate the problem	Resolved	1/5/2021	Memo "Mass balance error corrections and validation" dated 23/12/21, published 12/1/21 (Preston sent from 12506449 project scope)	Not required	AVON.FREESCK 0-34	Mapped around 50mm impact extent	
85	14/1/2022	Tim Preston	Lateral link mass balance loss, flood levels be low by 200mm+, resolved May2021, 50mm impact extent, refer GHD memo dated 23/12/21	3	Has three M21 cells with flood level exageration of more than 200mm	MF	Memo "Mass balance error corrections and validation" dated 23/12/21, published 12/1/21 (Preston sent from 12506449 project scope)	Model edits to insert cross sections so as to eliminate the problem	Resolved	1/5/2021	Memo "Mass balance error corrections and validation" dated 23/12/21, published 12/1/21 (Preston sent from 12506449 project scope)	Not required	AVON.CORSER2 10	Mapped around 50mm impact extent	

Avon Anomalies Register

Item No.	Date registered	Defined by who	Issue Name Descriptive	Risk for floor levels 3-high, 2-med, 1-low, 0-zero	Risk comment (TJP Nov2021)	Model component (MU, M11, M21)	References (docs, emails)	Resolution	Current status	Date Resolved	Reported to Council reference	Council acknowledgement	Model reference for location	Mapped to GIS polygon	Backup Information?
86	18/1/2022	Tim Preston	Buller Creek downstream end (ch xxx) connects directly into the side of a the MU Dudley Diversion pipe. This is questionable modelling practice and the model results are known to be less than good for stability. Antoinette noted this to Tim 14/1/2022. Alternative would be to move the couple upstream by inserting a short length of MU on the Buller branch, and then having a common end to end M11 to MU couple.	2	Hard to know what the impacts of the results instability here might have until they are fixed. But the system is biggish and the potential certainly more than minor, positive or negative equally likely, just highish risk.	MF			Prelim					Yes	
92	14/7/2022	Tim Preston	In the vicinity of 72 Langdons Road, some pipe assets including 750 dia SwPipeID 41376 and 41432, have been removed and superseeded by new assets. Both pipes remain in the model overstating the pipe network capacity here.	2	This overstated capacity will be significant locally for some events but larger flood events will still be driven by performance at the smaller railway culvert (800dia), so flood levels upstream from here will not be drastically affected	MU	Tim's email reply to Sheryl "Anomalies" 12/7/2022 on 12555628. Refer also items 19+38.	Update the model to match current GIS. May be worth checking the Styx model data source for similar issues	Resolved	21/2/2024	Tim's email reply to Sheryl "Anomalies" 12/7/2022 on 12555628.		eg: AVONSTYX.Pipe.CCCGIS.41376	Yes	
98	25/7/2024	Madeline Howell	Lack of mesh definition along Buckleys Rd, Barbadoes St, and Moorhouse Ave. These areas were formerly model boundaries, but the mesh has been extended to include these areas.	1	The model is likely underestimating the conveyance capacity of the roads, which can be major flow paths	M21	GHD 12625709 From: Madeline Howell to Tim Preseton Sent: 25/7/2024 (US Time) Subject: "Avon Anomaly #98- Buckleys Rd, Barbadoes St, and Moorhouse Ave"		Closed					Yes	
99	30/7/2024	Madeline Howell	There are a few locations where there is a low flow pipe under an open channel. The channel has capacity to convey flood flow, but is typically not included in the model. There is no schema that attempts to model open channel flow on top of pipes.	2	There may be artificial constriction of flow upstream of the pipes, which could be artificially increasing ponding upstream and lead to artificial attenuation of flows.	M11	GHD 12601181 From: Tim P to Madeline Howell & CCC Sent: 30/7/2024 (US Time) Subject: "RE: Culvert Conveyance Analysis-Summary of Results"		Closed					Yes	
101	12/3/2025	Tim Preston	The Syd Bradely Road / Russley Road underpass is not modelled.	2	Based on topography this ommission would likely alter the balance of flows between Wairarapa and Waimairi	MU	Email "Syd Bradely Road / Russley Road underpass - Avon" 20250312 in job 12601181		Closed					Yes	
103	25/3/2025	Tim Preston	At the Bellvue Ave pedestrian underpass, the model build was copied from Opus on instruction from CCC (by Jenna Stuart in 2017), but it has an unrealistic 6m w x 1m h geometry. The reality is arched armco with something like 2m x 2m (refer google street view). Proper dimensional information would be preferred to build the model more correctly here.	2	Current model is too big vs reality and upstream water levels in bigger flood flows will be understated, and downstream water levels overstated	M11	Tim's email "Bellvue underpass - anomaly #103" 20250325 (filed 12640130)		Closed					Yes	
104	1/4/2025	Tim Preston	At Avon.ilamstrm ch 1266 to 1701, invert levels are exceedingly deep and do not appear realistic.	1	There are some stability risks in this area, possibly tied to the negative slope at the d/s end. Channel capacity here	M11	None, refer model longsection. Resolution here to audit model data sources against other benchmark truth (incl LiDAR) and make		Closed		Not reported		Avon.ilamstrm ch 1266 to 1701	Yes	

Appendix G – Future improvement register

Future Improvements Register

Item No.	Date registered	Defined by who	Issue Name Descriptive	Description	Model component (MU, M11, M21, MF)	References (docs, emails)	Completion Date
1	8/3/2017	Tim Preston	DFSU infiltration	Infiltration patterns from our DSF2 spatially in the area of road edges are not ideal. DHI peer review comments highlight this. Opportunity to tidy this up using DFSU infiltration but technology has arrived too late to impliment now.	M21	From Antoinette 8/3/2017 9:51am From Tim 16/2/2017 10:32pm	
2	3/3/2017	Tim Preston	Road mesh quadralaterals	The road geometry lends itself naturally to long thin quadraterals. Might help with min cell size and stability.	M21	From Tim 3/3/2017 9:19am	
3	15/2/2017	DHI	Mike 1D	DHI suggested circa Feb 2017 that this might speed up runtimes a lot. Raised twice with CCC in Feb tech meetings.	MF	Tech minutes 9/2 and 23/2	
~~4~~	~~28/8/2017~~	~~Tim Preston~~	~~M21 Rail priorities vs Road~~	~~Noted from inspection of vicinity of Riccarton Road / Fendalton Road that giving the roads priority in the mesh build results in 'gutters' cutting through the rail lines. Better approach would be to give rail priority given they are often forming an embankment.~~	~~M21~~	~~TimP email example screen shots to John McA 2070828 4:50pm~~	~~2021 completed VO11 12506449 (we revised the Z4 levels~~
5	30/10/2017	Tim Preston	Private bridges	Review the data prepared by GHD showing likely private bridge locations and for areas of model built from LiDAR add roughness or some similar impediment to approximate the impact of the bridges (mainly M11 also some MU)	M11	N:\NZ\Christchurch\Projects\51\33275\01.Stage 1 Data\GIS\Data\01.Misc_Project_Data.gdb\Private_Bridges_20150715	
6	20/11/2017	Tim Preston / Clem Rolin	Mike Urban downstream end of pipe	Opportunity to understand better the typology of downstream end of pipes, be it bubble up pipe or actual outlet, and to adjust the model schematisation accordingly, especially with regard to MikeFlood coupling parameters. This is required due to lack of detail regarding outlet/ground connectivity in Council's current GIS.	MU	none	
7	18/1/2018	Tim Preston	Use radar rainfall data for future calibration or recalibrations	Council are procuring radar based rainfall data Citywide. The quality needs assessment but if good it will likely improve calibration outcomes.	M21	Tech minutes 20180118	
8	8/2/2018	Tim Preston	Simplified seawall limitations	The simplified seawalls around Moncks Bay have been found to retain significant water on the terrestrial side, suggesting that the wall openings (not modelled) may be significant and motivating interest in adding detail of them to the model as 1D elements or M21 structures	M21	Clem's email to TimP 20180131-1724 "RE: Seawalls QA - add to model future improvements"	
9	1/5/2018	DHI	Reduce M11 dx value	Dx values generally within the range of 1 -4 times the typical M21 mesh element side length. Reduce dx_max to 20m on the basis it provides a value which is approximately mid this range and consequently should improve current model stability.	M11	DHI review recommendation 16/4/2018 and discussed tech meeting 20180501	
10	1/5/2018	CCC	Additional detailed channel survey	Improve channel definition in areas where channel representation is important and LiDAR information is of poor quality. Setup screening system to identify variability of invert levels as an indicator of cross section quality. Also could use 'depth, width and steep sided characteristics to ID areas where LiDAR would be expected to perform badly (eg: Brookside Tce). Both could motivate 'areas of concern' and from this Council could overlay their own priorities wrt to understanding of flood risks. It would also be feasible to run a study comparison of LiDAR vs survey for various types of channel geometry and characterise how good LiDAR is and whether any bias correction rules could be developed to improve it at low cost. This could also be applied to lined channels where vertical position of the shape is taken from LiDAR.	M11	Raised by Tom Parsons and discussed in tech meeting 20180501 after his review of schema report	
11	1/5/2018	CCC	Add momentum factor to standard link at Bridge St	Refer to assessment in Avon Model Build report, chapter 8, to understand significance of this simplification. Regardless of practical significance there is some motivation to include for it in principal.	MF	Raised by Tom Parsons and discussed in tech meeting 20180501 from his review of schema report	
12	15/5/2018	CCC	Further consideration to MU manhole headloss setup	This could include theoretical review and sensitivity to options. DHI's Schema Report review memo dated 16/4/2018 suggests that alternatives may be worth considering.		DHI review recommendation 16/4/2018 and discussed tech meeting 20180515	

Future Improvements Register

Item No.	Date registered	Defined by who	Issue Name Descriptive	Description	Model component (MU, M11, M21, MF)	References (docs, emails)	Completion Date
13	17/7/2018	DHI	Avon.pynegld branch	Branch cross-section geometry has been derived from LiDAR and is of poor quality. A lateral link roughness value of 0.5 has been applied for link stability purposes which could create erroneous water levels at the links. Obtaining detailed channel survey and updating the model geometry in this branch could improve stability and reduce this high link roughness value requirement, providing more accurate flood level results.	M11	DHI review recommendation 22/6/2018	
14	30/8/2018	DHI	Refine model representation at PS219 to reflect capacity loss with high tailwater levels	Similar in principal to what was done at PS205. In bigger floods it is probable that this station capacity is reduced by high tailwater. This may be important to future performance and flood risks. In Mar14 calibration tailwater level reaches 17.1m with pump flow rate of 2.6 m3/s.	MU	Refer DHI review comments 29/8/2018. "N:\NZ\Christchurch\Projects\51\33275\02.Stage 2 Model Build\Technical\20160217 DHI peer review\44800767 - LDRP44 Peer Review - Avon 20180829.xlsx" "N:\NZ\Christchurch\Water Team\33275\00.2Document Transfer\Incoming\20180913 SKM report on Cranford esp PS219 TomP to TimP\Post earthquake operations of the Dudley Creek Diversion Investigation Rev 0 SKM September 2013.PDF"	
~~15~~	~~30/8/2018~~	~~DHI~~	~~Replace Horton infiltation with the new M21 constant infiltration with capacity option~~	~~This would imply a totally new recalibration but would be great for predicting impacts of grounwater level changes~~	~~M21~~	~~"N:\NZ\Christchurch\Projects\51\33275\02.Stage 2 Model Build\Technical\20160217 DHI peer review\44800767 - LDRP44 Peer Review - Avon 20180829.xlsx"~~	year 2020
16	11/10/2018	DHI	Improve baseflow representation	The major criticism is that where there are constant inflows at the model boundary (eg: inflow at Marshlands Road), that these have an unsatisfactory affect on model baseflows and that this should be improved.	M11	Build\Technical\20160217 DHI peer review\44800767 - LDRP44 Peer Review - Avon 20181011.xlsx Tab: QA-QC Model Build Report	
17	9/11/2018	Tom Parsons	Ponding on the boundary near Madras Moorhouse	Refer to assessment in Avon Model Status report, section 8.2 desirable to improve circumstances here through model integration, improved 1D boundary flows and/or opening up the M21 boundary	M21	TomP email comments on status report received 18/10/2018	
18	16/3/2020	Tim Preston	Lateral link start and end chainage sensitive to sub-millimetre chainage position	Refer to Tim's email within Clem's email in the reference field.	MF	Clem email 20200316_1456 "Avon Model Future Improvements Register #18 - RE: Lateral link chainages query?" (Project Repository 12506449)	
19	18/1/2021	Tim Preston	Inclusion of FIG waves	As of Jan 2021, all recent modelling with Goring 2018 has included FIG waves into peak sea level at Sumner Heads. Question raised by GHD in the JPM report as to whether this should continue since the estuary mouth is likely to filter FIG waves so they would not exist (or be greatly diminished amplitude) inside the estuary. Council decision 18/1/2021 is to continue with historic approach for now. Options for future modelling to be considered include	M21	Refer JPM report recommendations. Also Tim Preston email 18/1/2021 "FIG waves" 12506449. Also Tim Preston email 11/1/2021 "FIG waves - Antoinette's comment positive" 12506449. And more recently.. Tyler email "Sumner Heads model control levels" 20211221_1544 (GHD12506449)" and incl Jo Golden's email "FIG waves"	
20	11/2/2021	Tim Preston	Pilot work to evaluate the benefits of tidal tripled (rather than paired) model runs	A potential scope was submitted as a variation proposal to CCC on 4/2/21. This work was subsequently declined, but Council asked in a subsequent meeting 11/2 to preserve reference to this scoping in the FI register with a view that it might be done in future	M21	Email 4/2/2021, Tim to Jo Golden "Multihazard VPR#10 draft" PDF attachment, GHD j/n 12506449	
21	15/5/2021	Tim Preston	Revisit tidal timing	Now that we are running longer runs, we have more options and more to consider. Current timing rules were influenced by historically short run durations. Now that we are running longer, the current durations are not consistent with the Citywide principles used to establish them and should logically be updated. More advanced principles might also be considered.	M21	Email 15/5/2021, Tim to Tyler "VPR14 extended run-out time and tidal timing" GHD j/n 12506449	
22	22/6/2021	Tim Preston	Upstream ends of M11	In several locations we have flow paths defined in M21 transitioning into M11 defined channels. At the transition we have used M21-M11 standard links to connect the two flow paths. These tend to have high M21 invert levels and some justify Z5 level adjustments. It	MF	Was noted while writing VO11b	
23	12/7/2021	Tim Preston	Basin names with GIS IDs	Tyler pointed out that Council GIS "" has been significantly updated since 2015/16 and now has probably useful GIS ID numbers. He requested that the Schema be changed to define the M21 dike element names as "AVON.BASIN.CCCGIS.753.BURWOODHOSPITAL" similar to the	M21	Emails: From Tyler "Basins to be added and removed from the Avon model" 26/3/2021 and chain below	
24	19/8/2021	Tim Preston	Modelling of private sumps only connected by private pipes (separate GIS layer) and esp those connected to M21	Issue noted with respect to Prestons subdivision and an area near 218 Main South Road being studied by floor levels team.	MF	Email from Tim 20/8/2021 (12506449) "Potential future improvement - private sumps / laterals" and the chain of several underlying emails	

Future Improvements Register

Item No.	Date registered	Defined by who	Issue Name Descriptive	Description	Model component (MU, M11, M21, MF)	References (docs, emails)	Completion Date
25	1/12/2021	Tim Preston	Modelling of low crossings like near 546 Madras St is not connecting M21 well to the M11 system	Current model retains water on M21 at an exaggerated flood level and fails to connect well to the M11 drainage. Issue applies Citywide (not limited to Avon model).	MF	TimP email "Citywide road crossing drainage modelling (Madras St example)" 20211201 (j/n 12555628) Refer two other examples 233 Edgeware Road, 34 Webb St as per	
26	31/1/2022	Tim Preston	Option to check for residual impacts from the initial conditions	with a view to satisfaction or if not then putting more effort into better initial conditions	M11/M21	Tyler email "Avon model initial conditions - a better option" 20220131_1917 , jn12506449	
27	13/2/2022	Tim Preston	Rename the MU open channel nodes and links to remove the second (doubled up) .Avon.	This is cosmetic but remains a point of confusion when building new model, so worth tidying it up.	MU	None	
28	16/2/2022	Rowan.D	Strategy of 1D modelling for 'all watercourses' should be refined. In some cases 1D modelling is inappropriate and	Example: Esty.lovetts channel modelled in M11, however, depth is so small that it is a real nuisance on exceedance levels. Channel is in farm area of low importance and also very low coastal so impacted heavily by 2.4m SLR.	All	Tim email "Esty.lovetts FutureImprovement #28" 20220216, jn12506449	
29	9/3/2022	Tim Preston	General improvements - eg: stability, data quality, currency and calibration	All areas of the model can be 'done better' with widespread opportunities for post quake channel survey and better validation of all sorts of input data (including survey data). Innumerable areas with previously reported stability problems can be addressed. Input data	All	Tim's email "Dudley Diversion instability - requesting approval to improve" 16/2/22 and Tylers reply 17/2/22	
30	9/3/2022	Tim Preston	DHI peer review comments not actioned	Some of DHI peer review suggestions were not actioned due to scope constraints and priorities. These may be actioned later. Key points were; +Be more careful with past the peaks testing and summary reporting +Stability improvements especially to reduce high velocities +Cranford and Prestons areas are only partly updated	All	Tim's email "Avon_model_Review.xlsx" 11/2/2022 and especially the attached xls	
31	31/3/2022	Tim Preston	Future build improvements for stability resilience	Enact action item 4 from Tim's email "New modelling issue - No2 drain result - meeting decisions" 20220331, being a series of build method improvements	M11	Tim's email "New modelling issue - No2 drain result - meeting decisions" 20220331 Tim's email "No2 drain result -positive findings" 24/3/2022	
32	31/3/2022	Tim Preston	Future result QA improvements to find anomalies such as unstable result jumps	Enact the various alternative option items 4, 5, 6, 9 from Tim's email "No2 drain result - positive findings" 24/3/2022, using pattern recognition for 1D and 2D results, looking for lumps and step changes	All	Tim's email "New modelling issue - No2 drain result - meeting decisions" 20220331 Refer 3rd trial in N:\NZ\Christchurch\Projects\51\12601181\05.GHD	
33	16/5/2022	Tim Preston	From M21 stability QA noted a cluster of bad results either side of M21 dikes, in a stopbanks down result. Motivates	Suspect that there is a numerical problem when water levels are bathtub and the dikes are small and have a realistically negligable effect on flow and levels. The numerical scheme would probably work fine if the dikes were gone, but not so well with the dikes in the	M21	Tim's email "Future Improvement #33" 16/5/2022 (j/n 12506449) 20230412 Email: Theme 3 - Existing modelling concerns - Meeting Minutes 12/04/2023. From Tim Preston.	20240823 (see Model Build Log "m21fm" tab row 36)
34	16/5/2022	Tim Preston	From M21 stability QA noted regions of quite bad results around the estuary. Amplitude locally is quite consistent (not	Suspect that there is a numerical convergence problem when depths are higher, and wind shear has a small impact on water surface slopes. Difficult to converge on the equilibrium water level. Speculation but results show bad enough to motivate investigation and a do	M21	Tim's email "Future Improvement #34 - estuary stability" 16/5/2022 (j/n 12506449)	
35	4/11/2022	Tim Preston	Improve modelling of road surfaces that are currently missing from the mesh	Sheryl raised this general issue. Some acute areas of concern are Marine Parade where in places the unformed model road, with large mesh elements and high ground around, is resulting in large 'dam' type model geometry blocking the road. Similar concerns along Buckleys Road, model boundary in general and near other priority features in principle. Various actions are possible, including customised local approach to mesh levels, improved	M21	Sheryl briefed Tim verbally on screen	
36	4/8/2023	Tim Preston	Addition of private laterals and gully trap inlets into the models	Some areas rely significantly on private lateral drainage to prevent quite gross flooding. Eg: the depressed region south of Nederland Ave in Burwood. Modelling to date excludes these due to a combination of lack of deliberate consideration to include or not, private pipe layer	MU	Meeting minutes 4/8/2023 11am, per Tim's email "1000 yr+ model runs and private pipes - meeting record" that evening (GHD j/n 12601181)	
37	16/8/2023	Tim Preston	Review the model inlet capacities for the 'large' inlets	Review the model inlet capacities for the 'large' inlets that are supposed to be 'unlimited' and rely on other constraints like d/s pipe entry and diameter. Check that actual flows are not constrained by the inlet capacity as intended.	MF	Schema section 8.2.1 Email "Styx Model Build" from Tim 20230816	
38	17/8/2023	Tim Preston	Model mesh levels on river banks are sometimes distorted too low	Discrepant bank levels between the floodplain and river models are sometime gross due to misalignment of the blockout extents and the floodplain model levels including significant 'invert' levels – these can locally exaggerate the apparent flood extents (and floodplain	M21	Refer Heatmaps generated circa 2018 LDRP044 that show areas of discrepancy where M21 levels are low wrt M11 bank marker levels	
39	26/3/2025	Tim Preston	Noted a while ago that practically speaking it would be simpler if we uniformly hotstarted all our normal		M11	Refer operational help doc re simpler Refer Tim's email 20250326 "Understanding model baseflows" - Cranford project	

Appendix H – Runtime specification memorandum, 2016

Memorandum

25/5/2016

To	Tom Parsons		
Copy to	Peter Kinley, Greg Whyte, Chris Maguire, Iris Brookland, Graham Harrington, Peter Christensen		
From	Tim Preston	Tel	027-6414301
Subject	Citywide Flood Modelling - Proposed technical specification for model runtimes	Job no.	51/33275/

Hi Tom,

At the technical meeting 19/6, you asked GHD to prepare our recommendations in terms of improving the CCC specification for model runtimes. This memo is produced to meet that objective and we would recommend that the following specifications be adopted for the Citywide Flood Modelling project:

This memo is herewith updated (with changes markedup) to reflect expectations associated with a rain on grid hydrology approach. The recommendation specifies runtime requirements for draft (common) model runs. There is a general expectation that on occasions either during or on conclusion of some modelling work that higher specification (slower) model run(s) will be required either as a progress check or as a final design confirmation. Runtime requirements for such runs are not specified because they are much less common and will actually be quite varied according to the modelling need.

For closely related sets of (similar) model runs, evidence of satisfactory stability and accuracy with low order calculations may only need to be thoroughly checked for one selected model run with confidence in the similar model runs inferred. The check run should verify acceptable water balance result, stability through checking velocities and CFL numbers as well as quantifying 2D grid max water level differences between low order and high order calculations. The method of checking results shall be included in reporting associated with the runs and the extent to which suggest inferred confidence is used as a QA tool is at the discretion of the senior modelling person involved.

Table 1 Run specification and runtime parameters

Runtime Parameter Description	Draft model runs	Final design / or check model runs
1. Precision	Double	Double
2. Order of computation engine (both in time and space)	Low order	Low order*
3. Auto water balance	Off	On (discretionary)
4. Courant number	0.9	0.8

GHD Limited
Level 3 138 Victoria Street Christchurch Central 8013 PO Box 13468 Christchurch 8141 New Zealand
T 64 3 378 0900 **F** 64 9 370 8001 **E** chcmail@ghd.com **W** www.ghd.com

Runtime Parameter Description	Draft model runs	Final design / or check model runs
5. Model stability (2D)	Qualitative inspection of 2D results and sample timeseries results	Qualitative inspection
6. Maximum values calculated at runtime	No	Discretionary
7. Data saving fields	Depth, level, speed, CFL	Discretionary
8. Data saving timesteps	10 minute	Discretionary
9. Rainfall type	Single event	Multiple events batched
10. Rain event duration	Per river catchment critical duration 18 hrs and 24 hrs for Avon and Heathcote	Up to 72 hours
11. Flood event	50yr ARI, climate change rainfall and sea level rise	Optional
12. Stopbanks	Included in modelling	Optional
13. Computer specification	Refer below (dual GPU)	Dual or quad GPU
14. Geography	Single river catchment	Single river or Citywide catchment
15. Runtime requirement	7 hours	Not required

Computer specification: A suitable computer is a 16 core Dell Precision T7610 with Intel Xeon processors, 32 GB memory and running 64-bit Windows 7 operating system, with 2 Nvidia GeForce GTX Titan GPUs, with a solid state local hard drive (mostly adopted from DHI specification Mar 2015, issued as App C to the RFP)

GHD Discussion / rationale:

1. For models in Christchurch with small vertical extents (eg: 100m) that single precision 8 sig figs, produces with a minimum vertical resolution for numerical calculations of 0.01mm, however rain on grid requires double precision.
2. This relates to the number of terms used in the numerical approximation (computation). . With rain on grid we propose to use low order for draft and final runs supported by evidence that the differences between low and high order are not significant
3. Auto water balance slows down the runtimes significantly and should be used sufficiently to derive confidence in the model stability, but is not required for most model runs
4. The higher 0.9 courant number will accelerate runtimes but increase risks of model instability, reducing the number to 0.8 for final runs should improve outcomes in item 5 below.
5. 2D instabilities are generally identified by high CFL numbers and unreasonably high velocities. To our knowledge there is no way to specify what is considered acceptable, but in general small amounts (both in time and space) of instability that do not appear to affect overall results of interest are considered acceptable. This needs to be judged instinctively by a senior modelling person, in conjunction with ‘reasonableness’ of the overall results and Council may want to review this.
6. Processing of maximum values post runtime is usually preferred.

7. With FM software it is tedious to post process depth into level (or vica versa), hence specification to produce both and velocity is required in order to assess stability. Sometime other parameters may be required for specific purposes, but generally three should be sufficient for most purposes.
8. Recommendation is consistent with DHI specification and strikes a balance between speed and results files sizes vs the need for resolution driven by the critical use to inspect areas of potential instability
9. Once the critical duration is established for a typical design purpose, a single event should be sufficient for most design runs. Running a 'final proof' using the full suite of storm durations would be recommended for final design runs.
10. Recommendation is essentially following current practice of 24, 18, 48 hrs for the Heathcote, Avon and Styx models respectively. Recommendation to adopt river specific standards will advantage the Avon/estuary model which is likely to be the 'heaviest' in terms of detail and runtime requirements.
11. The 50 year flood event with climate change rainfall and sea level rise will be the most common design model run undertaken. Larger events may be expected to take longer to run.
12. Stopbanks included will be the most common design model run undertaken. Including the stopbanks will reduce wetted cells and be of some benefit to runtimes.
13. Due to the additional computational demand of rain on grid we anticipate dual GPU processing will be required to achieve runtimes. We have appended a requirement for solid state local hard drives as these improve results saving performance (prerogative being that results are saved locally to maximise speed).
14. This specification would apply to any of the major CCC river catchments (Styx, Avon, Heathcote or Halswell) individually but not the aggregate Citywide model.
15. Seven hours, includes for model initialisation time and runtime, but excludes any hotstart runtime. This time allows for two runs to be completed per working day (one daytime, one overnight).

Regards

Tim Preston
National Technical Leader Water Planning

Appendix I – Description of lateral link build process, 2017

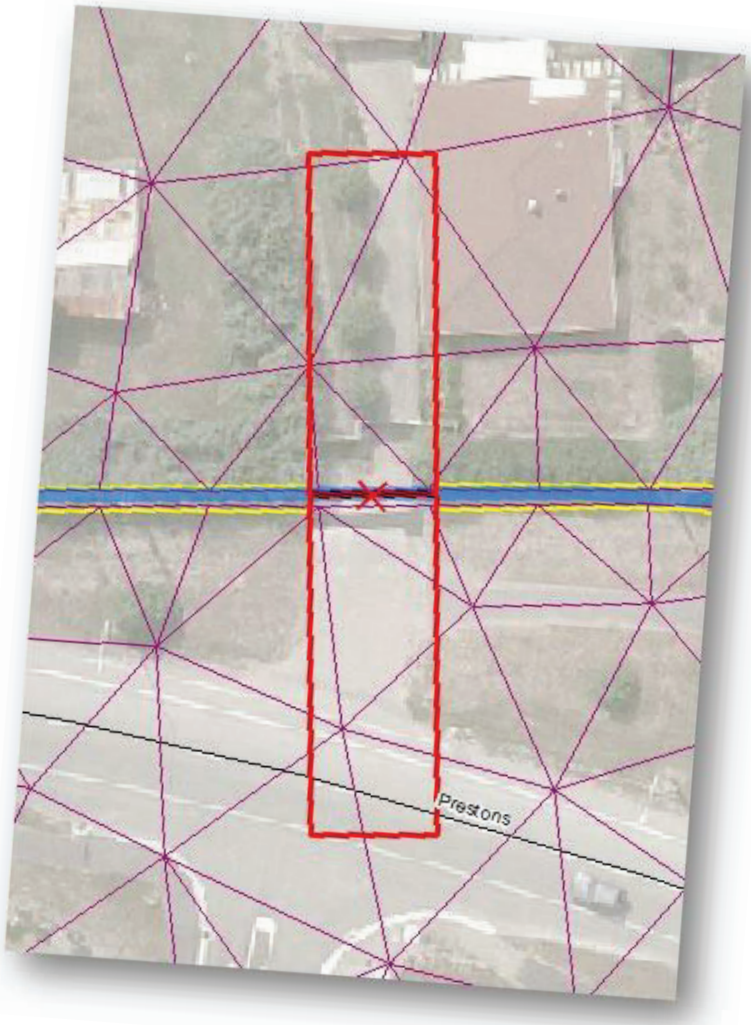
M21-M11 Lateral links

Lateral linking lines were formed from the M11 bank marker lines. The bank lines were cleaned up to remove small kinks and create a data set with a low resolution of vertices, while maintaining proximity to the appropriate side of the M11 blockout. In GIS, the lines were assigned to a M11 branch, side of river (left/right), and the upstream and downstream chainage based on closest proximity to the model calculation points (H-points). These simplified lines were then split into batches and run through a scripting tool developed by the Project Team, to create a lateral link couple file (SHP2MFLLFM_Inputs.py).The tool works in two stages; firstly a low density line is created with a small offset into the mesh area, and secondly the line is co-ordinated with the mesh cells.

Due to variable input geometries, buffer distance and line simplification approaches have evolved during the linking process and are not consistent across the model build.

A further improvement to the lateral linking has been done to avoid extreme cases of lateral link skewness (M11 H-points linked to a mesh element some distance away) and associated lateral link connections crossing structures. This has involved splitting lateral link lines at all M11 structures in the Avon model and has been undertaken using an automated process in GIS, as described below:

1. The lateral link lines are originally continuous along both sides of the mesh block out area, in accordance with the Schematisation Report section 8.1.1.
2. The M11 structures have been exported from the model to GIS as points,
3. For each structure, a buffer equal to the structure length has then been created from the structure point,
4. For each structure, the M11 centreline that intersects with the buffer has been extracted and used to represent the structure alignment (lines).
5. For each structure, a buffer of the structures alignment has then been created to represent the “footprint” of the structure. The buffer width has been set to a large value to ensure that the footprint will intersect the M11 blockout and more importantly the original lateral link line alignment. The buffer width is set to 25 m generally and 35 m on the Avon River downstream of Madras Street, where the mesh blockout is larger.
6. The original lateral link lines have then been deleted where they intersect with the structure “footprint”
7. The snapshot below illustrates the process :
   - Structure point (cross), line, and buffer in red.
   - M11 centreline in blue
   - Resulting lateral link line extents in yellow.


Prestons

Appendix J – Avon calibration approach memo, 2017

Memorandum

22 February 2017

To	Tom Parsons, Peter Christensen		
Copy to	Olivier Benichou, Mark Passier, Sally Williams, Mel Collett, Clemence Rolin, Catherine Campbell		
From	Tim Preston / John McArthur	Tel	03-3780913
Subject	Avon Calibration Approach	Job no.	51/33275/03

This memorandum summarises GHD's proposed approach to achieving a suitable calibration of the post-quake Avon River catchment flood model for the March 2014 calibration event (and also covers data for the June 2013 validation event). This version supersedes the previous version dated 14/2/17 and changes since that version are tracked herein.

1 Calibration Methodology

The approach outlined below is the initial plan 'all going smoothly' and so it may change in response to anomalous findings and learning during calibration.

1. Antecedent conditions: Evaluation of evapotranspiration, soil moisture deficit and groundwater levels and previous model calibration work (by others) confirm that it was reasonably dry prior to the March 2014 event and reasonably wet prior to the June 2013 event. (This is acknowledged as being contrary to rainfall records and river baseflow values at Gloucester St). We will account for antecedent conditions based on DHI previous calibration experience as described later in this memo.
2. Rating curve checks: We will compare the flow gauging site rating curves (from sites in Table 1 with 2014 flow data – four total) with the flow/level relationship from the model results, with an expectation of locally adjusting reasonable model channel roughness values to achieve a reasonable consistency.
3. Boundary condition setups: The Sumner Heads level will be assigned to the open sea boundary. The New Brighton Pier wind data will be assigned over the estuary. At Ferrymead bridge and Horners drain, level data will be assigned as level boundaries. Model flows associated with these two level boundaries will be reported and in the case of Horners drain compared with the flow data, however these flows will be sensitive to minor model details and a good fit is not expected until the adjacent models are calibrated and integrated with the Avon model.
4. Hydrology: We will base the hydrology calibration on the five soil drainage classes provided in the Landcare S-map data (Sep 2016) and a sixth Citywide project defined 'hillside' classification applied to any Port Hills areas (refer Figure 2 attached). Our initial goal will be to achieve March 2014 calibration with consistent parameters across these soil types citywide. Starting values for these soil types are tabulated below. We expect however (partly based on the DHI calibration experience) that it will prove necessary to have a second set of soil parameters applied to the northern catchment area (ie: areas north of the Avon and draining to lower Dudley Creek, PS210 or Horseshoe Lake) as discussed below. The hillside soil classification will remain uncalibrated in the Avon model calibration work, with calibration parameters to be adopted from the Bowenvale catchment calibration once it is

GHD Limited
Level 3 138 Victoria Street Christchurch Central 8013 PO Box 13468 Christchurch 8141 New Zealand
T 64 3 378 0900 **F** 64 9 370 8001 **E** chcmail@ghd.com **W** www.ghd.com

complete. The remaining smaller pump station flow estimates will only be used for validation for both the March 2014 and the June 2013 events. Hydrology adjustments for the Gloucester St gauge will also be extrapolated onto all Lower Avon catchment areas (other than the northern areas). We will typically adjust initial and final infiltration rates by an equal ratio to match flow hydrographs and reasonably preserve the relative infiltration characteristics across the soil drainage classifications. The intent is a single sensitivity run to quantify effect of adjustments and then one 'adjusted' run to confirm outcomes as expected (calibration runs take 18 hrs to complete). We may consider strategic separate adjustment of previousness, initial, final or decay infiltration rates in order to improve peak flow and runoff shape but only if there is a reasonably clear opportunity and benefit. Wetting losses are set to zero (due to the rain on grid method and because there are some effective losses associated with its wetting depth function) and so this will not be adjusted during calibration.

5. Hydraulic: All level gauge sites will be considered as three collective subgroups.
    a. Gloucester St, Horseshoe Lake and Bridge St main river level gauging sites will each be used to calibrate channel roughness downstream of the gauge. Bridge St level gauging will be used to calibrate channel capacity downstream of Bridge St and into the estuary. Gloucester St roughness calibration will be extrapolated upstream through Hagley Park to the Fendalton Road bridge.
    b. Pump station level sites will only be used for validation for both the March 2014 and the June 2013 events (except Horseshoe Lake water level, which will be used in conjunction with the PS205 flow rate to calibrate hydrology for the lake catchment as described above).
    c. A collective citywide approach will be used to adjust all remaining M11 channel roughness values based on consideration of levels at the minor stream level gauges (five – Bullers, Shirley, Dudley, Snellings and Kruses) to achieve an equal incidence of overs/unders across the level sites. Results from the three above sites which have flow-gauging and rating curve assessments described above will be preserved locally at these gauges distinct from this citywide roughness adjustment. The Mike Urban model in whole and the Mike11 culverts and pipes will not be adjusted during calibration. Further details are discussed later in the memo.
6. Low order: All calibration runs (and final modelling) for the project will use the low order computational approach (in accordance with the revised specification approved as part of the rain on grid variation 43 20/7/2016)
7. Flood extents: The March 2014 flood extents will be used for calibration in terms of the main river stem in the CBD and downstream, and in Cranford and Flockton basin areas. Outside of these main areas the flood extents will be used for validation and will be reported with comparison with the calibrated model results, once the calibration is 70% complete. We do not plan to make calibration adjustments to improve ‘fit’ to the flood extents, but if Council inspection of calibration results identify reasonable opportunities to improve model calibration in validation areas we will discuss and agree any actions with Council at the time. We understand that these flood extent maps have been carefully prepared with due consideration to all surveyed flood levels and photographic information and that further reference to this detailed input is not of value except for local understanding of ‘in stream’ water levels such as at Colombo St.
8. Aerial photos: The March 2014 flood event aerial photos will be qualitatively referred to in order to understand mitigation basin behaviour as the mapped flood extents often exclude the mitigation basins (eg: Winters Road Basin).
9. Western catchments (airport): We verify (and calibrate if needed to ensure) that western catchment areas generate zero or negligible flow rates at key observation points as has been anecdotally reported by Council officers. (Note that during Stage 4 priority should be given to the 50yr results to

inspect the same negligible flow condition and further hydrology adjustments to the western catchments may be motivated if these results show material runoff flow rates).

Table 1 – Expected soil infiltration rates

Soil classification	Initial infiltration (mm/hr)	Final infiltration (mm/hr)	Decay time (hrs)
Well drained *	12.5	5	36
Moderately well drained	10	3.5	24
Imperfectly drained *	7.5	2.5	12
Poorly drained	5	1	1.5
Very poorly drained *	2.5	1	1.5
Hillside	2.5	1	1.5

10.Note *: Numbers from WWDG table 21-12.

1.1 Northern soils calibration area

While we hope not need it, there are numerous imperfections in input data and model simplifications which may mean that the model cannot achieve a good calibration fit at Gloucester St and the northern area with any single set of reasonable soil infiltration parameters. If this is the case we will adopt a different set of parameters in this northern area. The main imperfections would be unmodelled groundwater levels, soils infiltration variability within drainage class and whether the Hortons represents actual soil infiltration behaviour. DHI's previous calibration experience suggests this outcome may be likely.

1.2 Further discussion of collective small channel roughness

For the five level gauging sites outside the main river channel, the information from these sites with respect to channel roughness is realistically applicable to perhaps 100-200m downstream and a short distance upstream of the gauging sites. The frequency with which channel roughness changes materially along the length of channels is similarly short with frequent changes in vegetation patterns, bank construction and man-made obstructions such as fences. We therefore do not consider that extrapolation of the level gauge site specific calibration results to a wider area such as several km downstream of the gauge to be good practice.

The alternative which we are pursuing is to take an average of these gauge site results to be indicative of the whole of Citywide channel roughness collectively. We consider this to be the best practice use of the scarce available calibration data.

1.3 Proposed Infiltration Adjustments for Design Events

Because the March 2014 event had unusually dry antecedent conditions we propose to adjust the calibrated infiltration parameters to create the design infiltration parameters. From DHI's most recent D14 model calibration work, DHI estimated that final infiltration rates for the March event at Gloucester St were 8mm/hr but adopted 5mm/hr for the design infiltration. This rate was applied to the whole Avon catchment except the northern areas (ie: areas north of the Avon and draining to lower Dudley Creek, PS210 or Horseshoe Lake). Once we have calibrated the model for the March 2014 event we will estimate all the upper and lower Avon design initial and final infiltration rates as being 62% (5 / 8^{ths}) of

the March 2014 infiltration rates. DHI's work on the northern areas suggested that infiltration rates did not change with antecedent conditions and so we will retain at 100% the March 2014 calibrated infiltration rates for design events in this area.

We will run the June 2013 validation run using the above design infiltration parameters. Based on DHI's calibration experience we would then expect that the June 2013 validation model would somewhat underestimate flows and levels as the June antecedent conditions were wetter than design (2 mm/hr vs 5mm/hr final infiltration from DHI final infiltration rates at Gloucester St). Once our results are presented Council may wish to consider and request additional June calibration work.

2 Available Information

There are a number of continuous recording level gauges in the catchment, some with derived flows based on relatively regular rating updates. There are also several pump stations with recorded wetwell level data and flows derived from pump capacity and pump operating records. Those relevant to the Avon model with data for the June 2013 and March 2014 events are tabulated below with locations shown on attached Figure 1.

Table 2 – Flow / Level Gauges

ID	Site Name	Data Type	2013	2014
66470	Bullers Drain at QEII Drive	Flow and Level	Good	Good
66611	Avon River at PS205	Level	Good	Peaks truncated
66624	No.2 Drain, Shirley Golf Course	Flow and Level	Good	Good
66634	Avon River at Bridge Street	Level	Good	Good
66635	Horseshoe Lake at PS205	Level	Good	Good
66636	Dudley Creek at Aylesford Street	Level	Nil	Good
66643	Snellings Drain at Mairehau Road	Level *	Good – flow also for 2013	Good
66649	Kruses Drain at Grimseys Road	Flow and Level	Good	Good
966602	Avon River at Gloucester Street Bridge	Flow and Level	Good	Offset 140mm level

Raw data quality confirmed John McArthur.

In addition, it is intended to utilise information from the following gauge types as boundary conditions in the calibration process. Their locations are also shown on attached Figure 1.

Table 3 – Calibration Boundary Sites

ID	Site Name	Data Type	2013	2014
66471	Horners Drain at Prestons Road	Flow and Level	Good	Good
325715	New Brighton Pier Aws	Wind	Good	Good
966699	Sumner Head	Level (sea)	Good	Good
66605	Heathcote River at Ferrymead Bridge	Level	Good	Good

Raw data quality confirmed John McArthur

There are also a number of pump stations with flow and level information, which may be useful for calibration and general validation purposes. These are tabulated below with locations shown on attached Figure 1.

Table 4 – Pump Stations

Pump Station Name	Available Information
PS0203 Wairoa SW	Flow and Level
PS0204 Waitaki SW	Flow only
PS0205 New Brighton SW	Flow only
PS0210 Locksley SW	Flow only
PS0219 Philpotts SW	Flow and Level
PS0220 Hulverstone SW	Flow and Level

Regards



Tim Preston / John McArthur

Attached: Figure 1 – Avon Calibration Data Map

Figure 2 – Citywide Soils Drainage Map

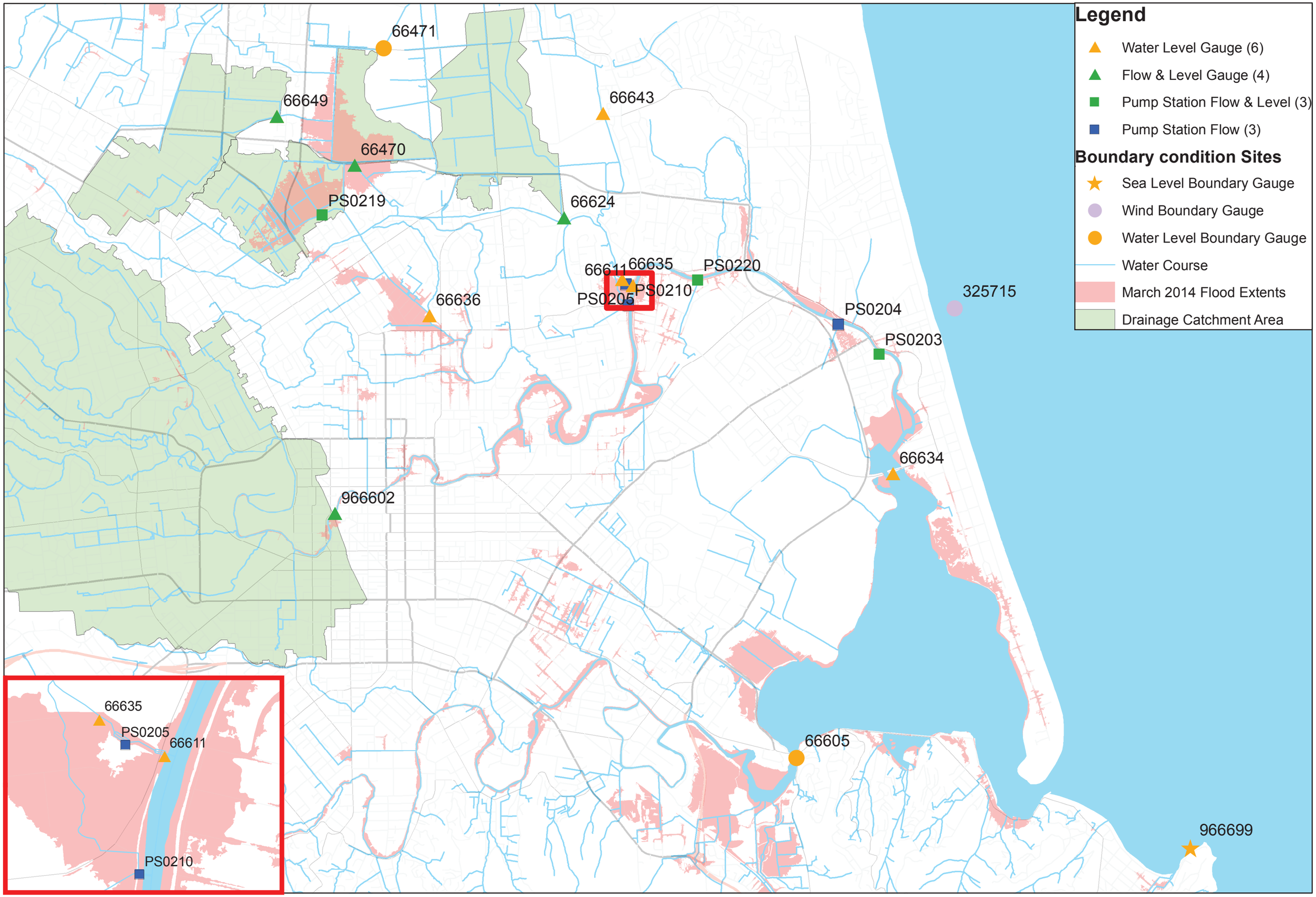
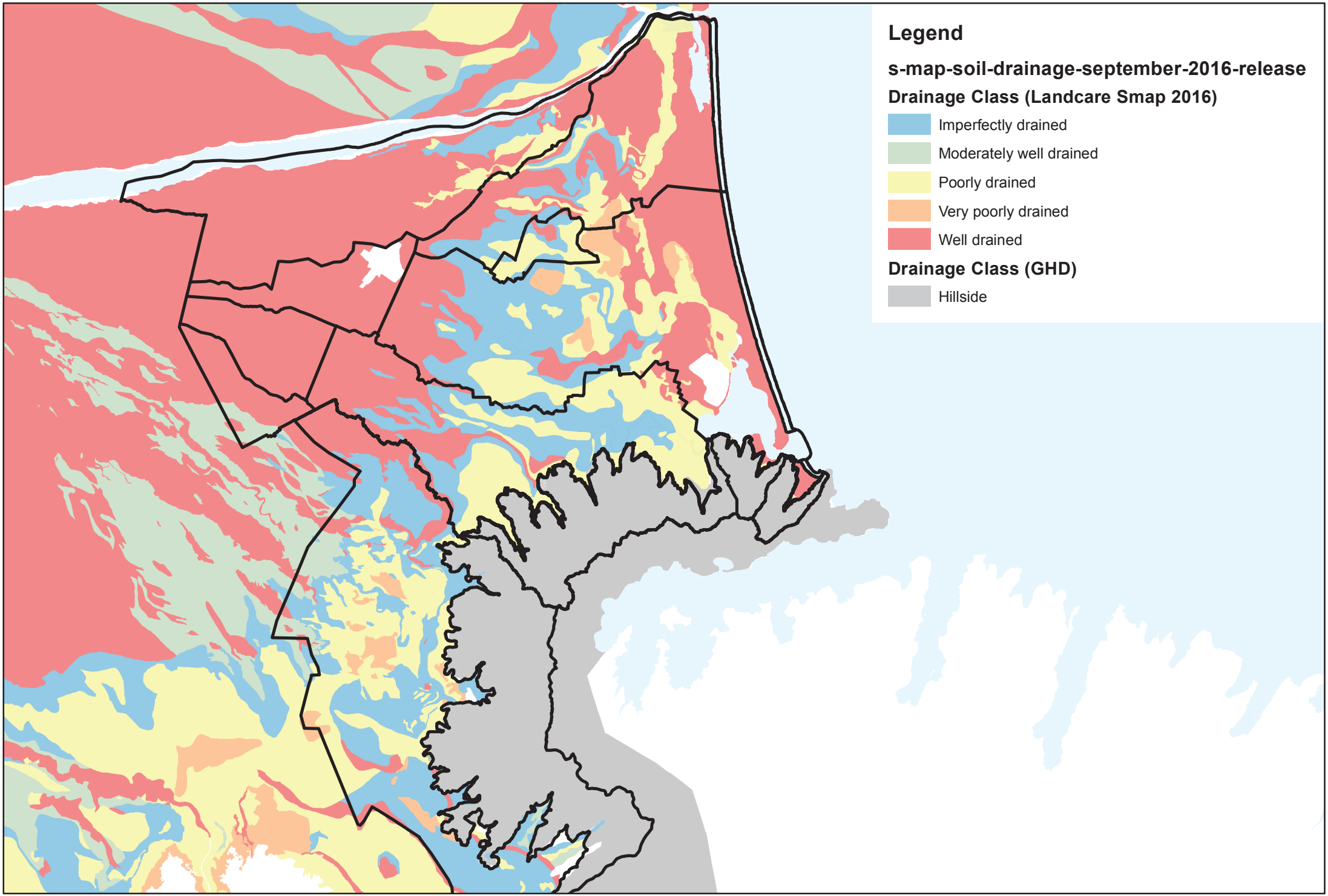


Fig.1 Avon Calibration Data Map



Data source: N:\NZ\Christchurch\Projects\51\33275\03.Stage 3 Calibration\GIS\Maps\Mxd\Soil_type_Citywide.mxd

Fig. 2 Citywide Soil Drainage Map

Appendix K – Climate change scenarios for modelling memorandum, Jacobs 2019[43]

[43] GHD internal reference: G:\51\12506449\00.Document Transfer\Incoming\20190321Updated climate scenario_Jacobs David Cobby to Clem\IZ084400-NC-MEM-0001 Climate Change Scenarios for Modelling Rev1.pdf

JACOBS

Memorandum

Level 2, Wynn Williams Building
47 Hereford Street
Christchurch Central
PO Box 1147, Christchurch 8140
New Zealand
T +64 3 940 4900
F +64 3 940 4901

Subject	**Climate Change Scenarios for Modelling**	**Project Name**	Christchurch Multi-hazard Analysis
Attention	Tom Parsons, Jo Golden	**Project No.**	IZ084400
From	Dr David Cobby	**Reviewed by**	Craig Clifton
Date	13 March 2019	**Approved by**	Vince Vella-Brincatt
Copies to	**Ian Wiseman**		

1. Introduction

This memo updates *Modelling Climate Change Impacts*, which was previously issued to Council on the 15th May 2017 for the LDRP110 Heathcote Floodplain Management project. Rainfall and sea level are both projected to vary with increasing temperature, and this methodology ties together changes in both to projections of temperature rise, which can be attributed to timescales based on different emissions scenarios as published in the MfE (2017) Coastal Hazards and Climate Change[1] and MfE (2018) Climate Change Projections for New Zealand[2]. This memo is being issued to inform upcoming flood modelling in the LDRP097 Multi-Hazards Study, which is planning to run 10, 50 and 200 year ARI probability events for a range of durations (to be determined) for a number of climate change scenarios. The final model run schedule has yet to be determined, but the principles in this memo will be applied.

2. Sea Level Rise Increments

Table 10 in MfE (2017) contains decadal projections of sea level rise above the 1986-2005 baseline[3], which are reproduced in Table 2.1. From these, we have selected the increments in Table 2.2 for the flood modelling, to achieve a balance between sufficient resolution to inform decisions, and budgetary constraints. Based on the different Representative Concentration Pathways (RCP) emissions scenarios, Table 2.2 indicates the earliest and latest decades within which the SLR increment is anticipated, based on MfE (2017).

[1] Ministry for the Environment 2017. Coastal Hazards and Climate Change. Guidance for Local Government.
[2] Ministry for the Environment 2018. Climate Change Projections for New Zealand: Atmosphere Projections Based on Simulations from the IPCC Fifth Assessment, 2nd Edition. Wellington: Ministry for the Environment.
[3] These increments are recommended for use by local government across New Zealand, including in a number of studies linked to LDRP097, and therefore will be referred to here. However, we note that the recent update of tidal levels for Christchurch supersedes the 2005 baseline, and therefore all increments for 2020 in Table 2.1 could be set to zero, and subsequent decadal increments updated. Instead, we have retained the recognisable increment values but will consider that these increments could occur in slightly different decades than in MfE (2017). Tide level update reported in Mulgor (2018) Extreme Sea Levels at Christchurch Sites: EV1 Analysis. 24 Jul 2018.

Table 2.1: Sea level rise increments for different emissions scenarios (taken from MfE, 2017). Yellow highlighting indicates values to be used in flood modelling

NZ SLR scenario Year	NZ RCP2.6 M (median) [m]	NZ RCP4.5 M (median) [m]	NZ RCP8.5 M (median) [m]	NZ RCP8.5 H+ (83rd percentile) [m]
2020	0.08	0.08	0.09	0.11
2030	0.13	0.13	0.15	0.18
2040	0.18	0.19	0.21	0.27
2050	0.23	0.24	0.28	0.37
2060	0.27	0.3	0.36	0.48
2070	0.32	0.36	0.45	0.61
2080	0.37	0.42	0.55	0.75
2090	0.42	0.49	0.67	0.9
2100	0.46	0.55	0.79	1.05
2110	0.51	0.61	0.93	1.2
2120	0.55	0.67	1.06	1.36
2130	0.6	0.74	1.18	1.52
2140	0.65	0.81	1.29	1.69
2150	0.69	0.88	1.41	1.88

Table 2.2: Climate change increments proposed (not yet confirmed) for the baseline flood modelling

Sea Level Rise (m)	Earliest Decade Anticipated	Latest Decade Anticipated	Justification
+0.00	2020		Today's Risk
+0.19	2030	2040	Lifetime of key existing assets (e.g. Avon temporary stopbanks) and Long Term Plan
+0.45	2060	2090	50-year time horizon
+1.06	2100	2150+	100-year time horizon
+1.88	2150	2150+	SLR scenario for long-term land use planning and stress-testing pathways

3. Rainfall Increments

Table 13 of MfE (2018) contains projected percentage increases in rainfall events, for various storm frequencies and durations, which are reproduced in Table 3.1.

Table 14 of MfE (2018) gives the temperature increases which can be used together with the values in Table 3.1 to estimate increases in rainfall. Values can be identified at the same approximate times indicated for the sea level rise scenarios. The temperature increases are reproduced in Table 3.2, from which we select the RCP8.5 projections, to tie in with the majority of the sea level rise increments in Table 2.1. These RCP8.5 values are graphed in Figure 3.1 to enable estimation of temperature increase for a given year from 2020 onwards, by interpolation up to 2120 and extrapolation out to 2150. To constrain the extrapolation out to 2150 to a reasonable value, a linear (rather than polynomial) relationship has been fitted to the available data.

Table 3.1: Percentage change in rainfall depth from 1 degree increase in temperature (reproduced from MfE, 2018). Durations do not reflect those to be used in this study.

ARI (Years)	2	5	10	20	30	50	100
Duration (Hours)							
1	12.2	12.8	13.1	13.3	13.4	13.5	13.6
2	11.7	12.3	12.6	12.8	12.9	13	13.1
6	9.8	10.5	10.8	11.1	11.2	11.3	11.5
12	8.5	9.2	9.5	9.7	9.8	9.9	10.1
24	7.2	7.8	8.1	8.2	8.3	8.4	8.6
48	6.1	6.7	7	7.2	7.3	7.4	7.5
72	5.5	6.2	6.5	6.6	6.7	6.8	6.9
96	5.1	5.7	6	6.2	6.3	6.4	6.5
120	4.8	5.4	5.7	5.8	5.9	6	6.1

Table 3.2: Temperature increase recommended to estimate rainfall increases (reproduced from MfE, 2018). Yellow highlighting indicates the values to be used to interpolate and extrapolate the change in temperature.

	2020 - 2030	2031 - 2050	2081 - 2100	2101 - 2120
RCP 2.6	0	0.59	0.59	0.59
RCP 4.5	0	0.74	1.21	1.44
RCP 8.5	0	0.85	2.58	3.13

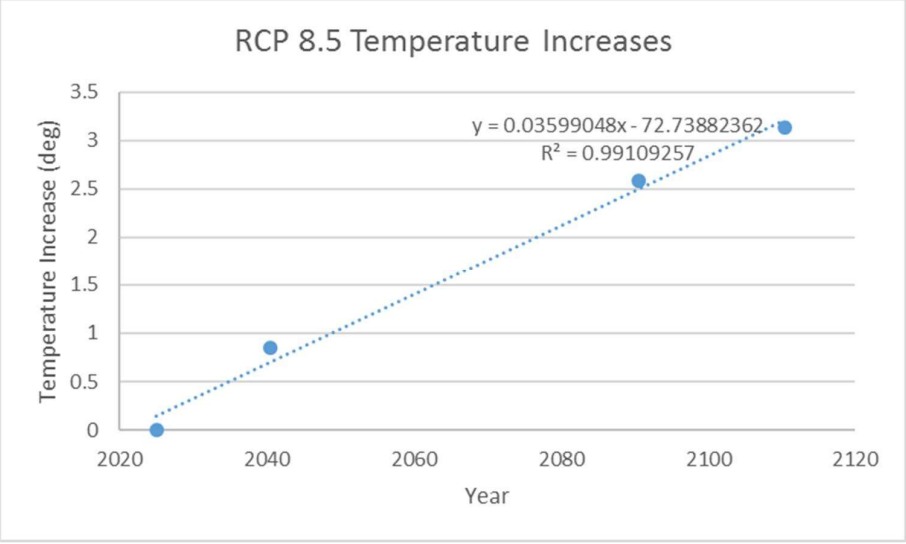


Figure 3.1: Plot of recommended temperature increases to estimate rainfall increases for the RCP 8.5 scenario in Table 3.2

Using the estimated temperature increases, we can use Table 3.1 to estimate the percentage increase in rainfall totals which match the timing of the sea level rise increases. For consistency with timing of sea level rise increases in Table 2.2, the temperature increase at the earliest anticipated decade is taken. For example, for a SLR of +0.19m which could occur in the 2030s or 2040s, the

temperature increase at 2030 is used. An example for an illustrative +0.85 deg increase is shown in Table 3.3. Note that the event durations not included in Table 3.1 have been calculated through linear interpolation, the 200 year probability event has been calculated through linear extrapolation, and these values can be identified in the tables by their orange row and column headers.

Table 3.3: Percentage increase in rainfall totals for illustrative +0.85 deg (orange row and column headers denote interpolated or extrapolated values)

ARI (Years)	10	50	200
Duration (Hours)			
2	10.7	11.1	11.3
3	10.3	10.7	11.0
6	9.2	9.6	10.1
9	8.6	9.0	9.5
12	8.1	8.4	8.9
18	7.5	7.8	8.3
24	6.9	7.1	7.7
30	6.7	6.9	7.4
36	6.4	6.7	7.1
48	6.0	6.3	6.5

4. Summary

Based on the latest guidance on impacts of climate change on sea level rise and rainfall, Table 4.1 summarises the proposed (not yet confirmed) modelling scenarios, where sea level and rainfall total increase together with projected temperature. Proposed flood modelling will run 10, 50 and 200 year ARI probability events for a range of durations (to be determined) for the different sea level rise scenarios, for which comparable data to Table 4.1 is contained in an accompanying spreadsheet.

This memo is being issued to inform upcoming flood modelling in the LDRP097 Multi-Hazards Study, which is planning to run 10, 50 and 200 year ARI probability events for a range of durations (to be determined) for a number of climate change scenarios. The final model run schedule has yet to be determined by Council, but the principles in this memo will be applied.

Table 4.1: Summary climate change scenarios proposed (not yet confirmed) for flood modelling

Sea Level Rise (m)	Earliest Decade Anticipated	Latest Decade Anticipated	Temperature Increase (deg)	% Rainfall Increase for 50y ARI 24h Duration
+0.00	2020		+0.0	+0.0%
+0.19	2030	2040	+0.3	+2.7%
+0.45	2060	2090	+1.4	+11.8%
+1.06	2100	2150+	+2.8	+23.9%
+1.88	2150	2150+	+4.6	+39.0%

5. Important Note About Your Report

The sole purpose of this report and the associated services performed by Jacobs is to summarise the proposed (not yet confirmed) modelling scenarios, where sea level and rainfall total increase together with projected temperature for the LDRP97 Multi Hazard project. The scope of services, as described in this report, was developed between Jacobs and Christchurch City Council ('the Client').

In preparing this report, Jacobs has relied upon, and presumed accurate, any information (or confirmation of the absence thereof) provided by the Client and/or from other sources. Except as otherwise stated in the report, Jacobs has not attempted to verify the accuracy or completeness of any such information. If the information is subsequently determined to be false, inaccurate or incomplete then it is possible that our observations and conclusions as expressed in this report may change.

Jacobs derived the data in this report from information sourced from the Client and/or other sources at the time or times outlined in this report. The passage of time, manifestation of latent conditions or impacts of future events may require further examination of the project and subsequent data analysis, and re-evaluation of the data, findings, observations and conclusions expressed in this report. Jacobs has prepared this report in accordance with the usual care and thoroughness of the consulting profession, for the sole purpose described above and by reference to applicable standards, guidelines, procedures and practices at the date of issue of this report. For the reasons outlined above, however, no other warranty or guarantee, whether expressed or implied, is made as to the data, observations and findings expressed in this report, to the extent permitted by law.

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This report has been prepared on behalf of, and for the exclusive use of, the Client, and is subject to, and issued in accordance with, the provisions of the contract between Jacobs and the Client. Jacobs accepts no liability or responsibility whatsoever for, or in respect of, any use of, or reliance upon, this report by any third party.

Appendix L – Rainfall climate change percentage, 2019[44]

"20190804_12506449_CALC_RAIN_Perc increase of rainfall intensity.xlsx" GHD 4/8/2019

[44] GHD internal reference: https://projects.ghd.com/oc/NewZealand1/ldrp097baselinemodel/Delivery/Documents/20190804_12506449_CALC_RAIN_Perc%20increase%20of%20rainfall%20intensity.xlsx

Scenarios					ARI															
Scenario	SLR (m)	Duration	Duration (hrs)	Temperature rise (°C):	1.58	2	3	5	7	10	13	20	30	40	50	60	80	100	200	250	500
2030	0.19	10m	0.17	0.3	4.03	4.06	4.13	4.25	4.31	4.35	4.37	4.41	4.45	4.46	4.48	4.49	4.50	4.51	4.58	4.61	4.77
2030	0.19	20m	0.33	0.3	4.01	4.03	4.10	4.23	4.28	4.32	4.34	4.39	4.42	4.44	4.45	4.46	4.47	4.48	4.55	4.58	4.74
2030	0.19	30m	0.5	0.3	3.98	4.01	4.07	4.20	4.26	4.30	4.32	4.36	4.39	4.41	4.43	4.43	4.44	4.46	4.52	4.55	4.72
2030	0.19	1	1	0.3	3.90	3.93	3.99	4.12	4.18	4.22	4.24	4.28	4.31	4.33	4.34	4.35	4.36	4.38	4.44	4.47	4.63
2030	0.19	2	2	0.3	3.74	3.77	3.83	3.96	4.02	4.06	4.07	4.12	4.15	4.17	4.18	4.19	4.20	4.22	4.28	4.31	4.47
2030	0.19	3	3	0.3	3.58	3.61	3.68	3.81	3.87	3.91	3.93	3.98	4.02	4.03	4.05	4.06	4.07	4.09	4.17	4.21	4.41
2030	0.19	6	6	0.3	3.12	3.15	3.23	3.38	3.44	3.48	3.50	3.57	3.60	3.62	3.64	3.65	3.68	3.70	3.83	3.89	4.22
2030	0.19	9	9	0.3	2.91	2.94	3.02	3.17	3.23	3.27	3.29	3.35	3.38	3.40	3.41	3.42	3.45	3.48	3.60	3.67	3.99
2030	0.19	12	12	0.3	2.70	2.74	2.81	2.96	3.02	3.06	3.08	3.12	3.15	3.17	3.19	3.20	3.22	3.25	3.38	3.44	3.77
2030	0.19	18	18	0.3	2.50	2.53	2.60	2.74	2.79	2.83	2.85	2.88	2.91	2.93	2.94	2.96	2.98	3.01	3.14	3.20	3.52
2030	0.19	24	24	0.3	2.29	2.32	2.38	2.51	2.57	2.61	2.62	2.64	2.67	2.69	2.70	2.72	2.74	2.77	2.90	2.96	3.28
2030	0.19	30	30	0.3	2.20	2.23	2.29	2.42	2.48	2.52	2.53	2.56	2.59	2.61	2.62	2.63	2.66	2.68	2.79	2.85	3.13
2030	0.19	36	36	0.3	2.11	2.14	2.20	2.33	2.39	2.43	2.44	2.48	2.51	2.53	2.54	2.55	2.57	2.59	2.69	2.74	2.98
2030	0.19	48	48	0.3	1.94	1.96	2.03	2.16	2.21	2.25	2.27	2.32	2.35	2.37	2.38	2.39	2.40	2.41	2.48	2.51	2.67
2030	0.19	60	60	0.3	1.84	1.87	1.94	2.08	2.13	2.17	2.19	2.22	2.25	2.27	2.29	2.29	2.30	2.32	2.38	2.41	2.57
2030	0.19	72	72	0.3	1.74	1.77	1.85	2.00	2.05	2.09	2.10	2.12	2.16	2.17	2.19	2.20	2.21	2.22	2.29	2.32	2.48
2030	0.19	96	96	0.3	1.61	1.64	1.71	1.83	1.89	1.93	1.95	2.00	2.03	2.04	2.06	2.07	2.08	2.09	2.16	2.19	2.35
2030	0.19	120	120	0.3	1.52	1.54	1.61	1.74	1.80	1.83	1.84	1.87	1.90	1.92	1.93	1.94	1.95	1.96	2.03	2.06	2.22
2060	0.45	10m	0.17	1.4	17.57	17.68	17.96	18.52	18.78	18.94	19.03	19.22	19.36	19.44	19.51	19.53	19.59	19.65	19.93	20.07	20.77
2060	0.45	20m	0.33	1.4	17.45	17.57	17.85	18.41	18.66	18.83	18.91	19.11	19.25	19.32	19.39	19.42	19.47	19.53	19.81	19.95	20.65
2060	0.45	30m	0.5	1.4	17.33	17.45	17.73	18.29	18.54	18.71	18.79	18.99	19.13	19.20	19.27	19.30	19.36	19.41	19.69	19.83	20.53
2060	0.45	1	1	1.4	16.98	17.10	17.38	17.94	18.19	18.36	18.44	18.64	18.78	18.85	18.92	18.95	19.01	19.06	19.34	19.48	20.18
2060	0.45	2	2	1.4	16.28	16.40	16.68	17.24	17.49	17.66	17.74	17.94	18.08	18.15	18.22	18.25	18.30	18.36	18.64	18.78	19.48
2060	0.45	3	3	1.4	15.61	15.73	16.02	16.61	16.86	17.03	17.12	17.34	17.48	17.55	17.62	17.66	17.73	17.80	18.15	18.33	19.20
2060	0.45	6	6	1.4	13.60	13.74	14.06	14.72	14.97	15.14	15.26	15.56	15.70	15.77	15.84	15.89	16.01	16.12	16.68	16.96	18.36
2060	0.45	9	9	1.4	12.69	12.82	13.15	13.81	14.06	14.23	14.33	14.58	14.72	14.79	14.86	14.91	15.02	15.14	15.70	15.98	17.38
2060	0.45	12	12	1.4	11.78	11.91	12.24	12.89	13.15	13.31	13.40	13.60	13.74	13.81	13.88	13.93	14.04	14.16	14.72	15.00	16.40
2060	0.45	18	18	1.4	10.87	11.00	11.31	11.91	12.17	12.33	12.40	12.54	12.68	12.75	12.82	12.88	12.99	13.10	13.67	13.95	15.35
2060	0.45	24	24	1.4	9.97	10.09	10.37	10.93	11.18	11.35	11.39	11.49	11.63	11.70	11.77	11.83	11.94	12.05	12.61	12.89	14.30
2060	0.45	30	30	1.4	9.59	9.71	9.99	10.55	10.80	10.97	11.02	11.14	11.28	11.35	11.42	11.47	11.57	11.67	12.16	12.40	13.63
2060	0.45	36	36	1.4	9.20	9.32	9.60	10.16	10.41	10.58	10.64	10.79	10.93	11.00	11.07	11.11	11.20	11.28	11.70	11.91	12.96
2060	0.45	48	48	1.4	8.43	8.55	8.83	9.39	9.64	9.81	9.90	10.09	10.23	10.30	10.37	10.40	10.46	10.51	10.79	10.93	11.63
2060	0.45	60	60	1.4	8.00	8.13	8.43	9.04	9.29	9.46	9.52	9.67	9.81	9.88	9.95	9.98	10.04	10.09	10.37	10.51	11.21
2060	0.45	72	72	1.4	7.57	7.71	8.04	8.69	8.94	9.11	9.15	9.25	9.39	9.46	9.53	9.56	9.61	9.67	9.95	10.09	10.79
2060	0.45	96	96	1.4	7.03	7.15	7.43	7.99	8.24	8.41	8.49	8.69	8.83	8.90	8.97	9.00	9.05	9.11	9.39	9.53	10.23
2060	0.45	120	120	1.4	6.61	6.73	7.01	7.57	7.82	7.99	8.03	8.13	8.27	8.34	8.41	8.44	8.49	8.55	8.83	8.97	9.67
2100	1.06	10m	0.17	2.8	35.61	35.85	36.41	37.55	38.06	38.40	38.57	38.97	39.26	39.40	39.54	39.60	39.71	39.82	40.39	40.68	42.10
2100	1.06	20m	0.33	2.8	35.37	35.61	36.18	37.31	37.83	38.17	38.34	38.73	39.02	39.16	39.30	39.36	39.47	39.59	40.16	40.44	41.86
2100	1.06	30m	0.5	2.8	35.13	35.37	35.94	37.08	37.59	37.93	38.10	38.50	38.78	38.92	39.07	39.12	39.24	39.35	39.92	40.20	41.62
2100	1.06	1	1	2.8	34.42	34.66	35.23	36.37	36.88	37.22	37.39	37.79	38.07	38.21	38.36	38.41	38.53	38.64	39.21	39.49	40.91
2100	1.06	2	2	2.8	33.00	33.24	33.81	34.95	35.46	35.80	35.97	36.37	36.65	36.79	36.94	36.99	37.11	37.22	37.79	38.07	39.49
2100	1.06	3	3	2.8	31.64	31.89	32.48	33.67	34.18	34.52	34.71	35.16	35.44	35.59	35.73	35.80	35.94	36.08	36.79	37.15	38.92
2100	1.06	6	6	2.8	27.57	27.84	28.51	29.83	30.34	30.68	30.94	31.54	31.82	31.96	32.11	32.22	32.45	32.67	33.81	34.38	37.22
2100	1.06	9	9	2.8	25.72	26.00	26.66	27.99	28.50	28.84	29.05	29.55	29.83	29.97	30.12	30.23	30.46	30.68	31.82	32.39	35.23
2100	1.06	12	12	2.8	23.87	24.15	24.81	26.14	26.65	26.99	27.16	27.56	27.84	27.99	28.13	28.24	28.47	28.70	29.83	30.40	33.24
2100	1.06	18	18	2.8	22.04	22.30	22.92	24.15	24.66	25.00	25.13	25.43	25.71	25.85	26.00	26.11	26.34	26.57	27.70	28.27	31.11
2100	1.06	24	24	2.8	20.22	20.46	21.02	22.16	22.67	23.01	23.10	23.30	23.58	23.72	23.87	23.98	24.21	24.43	25.57	26.14	28.98
2100	1.06	30	30	2.8	19.44	19.68	20.24	21.38	21.89	22.23	22.34	22.59	22.87	23.01	23.16	23.26	23.45	23.65	24.65	25.14	27.63
2100	1.06	36	36	2.8	18.66	18.89	19.46	20.60	21.11	21.45	21.58	21.88	22.16	22.30	22.45	22.53	22.70	22.87	23.72	24.15	26.28
2100	1.06	48	48	2.8	17.09	17.33	17.90	19.04	19.55	19.89	20.06	20.46	20.74	20.88	21.02	21.08	21.20	21.31	21.88	22.16	23.58
2100	1.06	60	60	2.8	16.22	16.48	17.09	18.33	18.84	19.18	19.31	19.60	19.89	20.03	20.17	20.23	20.34	20.46	21.02	21.31	22.73
2100	1.06	72	72	2.8	15.35	15.63	16.29	17.62	18.13	18.47	18.55	18.75	19.04	19.18	19.32	19.38	19.49	19.60	20.17	20.46	21.88
2100	1.06	96	96	2.8	14.25	14.49	15.06	16.19	16.71	17.05	17.22	17.62	17.90	18.04	18.18	18.24	18.35	18.47	19.04	19.32	20.74
2100	1.06	120	120	2.8	13.40	13.64	14.21	15.34	15.85	16.19	16.28	16.48	16.76	16.91	17.05	17.10	17.22	17.33	17.90	18.18	19.60
2150	1.88	10m	0.17	4.6	58.16	58.55	59.48	61.33	62.17	62.73	63.01	63.66	64.12	64.35	64.58	64.68	64.86	65.05	65.98	66.44	68.76
2150	1.88	20m	0.33	4.6	57.77	58.16	59.09	60.95	61.78	62.34	62.62	63.27	63.73	63.96	64.20	64.29	64.47	64.66	65.59	66.05	68.37
2150	1.88	30m	0.5	4.6	57.39	57.78	58.70	60.56	61.40	61.95	62.23	62.88	63.35	63.58	63.81	63.90	64.09	64.27	65.20	65.67	67.99
2150	1.88	1	1	4.6	56.23	56.62	57.54	59.40	60.24	60.79	61.07	61.72	62.19	62.42	62.65	62.74	62.93	63.11	64.04	64.51	66.83
2150	1.88	2	2	4.6	53.91	54.30	55.22	57.08	57.92	58.47	58.75	59.40	59.87	60.10	60.33	60.42	60.61	60.79	61.72	62.19	64.51
2150	1.88	3	3	4.6	51.69	52.09	53.06	54.99	55.83	56.38	56.70	57.43	57.89	58.12	58.36	58.47	58.70	58.94	60.10	60.68	63.58
2150	1.88	6	6	4.6	45.02	45.48	46.56	48.73	49.56	50.12	50.54	51.51	51.98	52.21	52.44	52.63	53.00	53.37	55.22	56.15	60.79
2150	1.88	9	9	4.6	42.01	42.46	43.55	45.71	46.55	47.10	47.45	48.26	48.73	48.96	49.19	49.38	49.75	50.12	51.98	52.90	57.54

Scenarios					ARI															
Scenario	SLR (m)	Duration	Duration (hrs)	Temperature rise (°C):	1.58	2	3	5	7	10	13	20	30	40	50	60	80	100	200	250	500
2150	1.88	12	12	4.6	38.99	39.45	40.53	42.69	43.53	44.09	44.37	45.01	45.48	45.71	45.94	46.13	46.50	46.87	48.73	49.66	54.30
2150	1.88	18	18	4.6	36.01	36.43	37.44	39.45	40.28	40.84	41.05	41.53	42.00	42.23	42.46	42.65	43.02	43.39	45.25	46.18	50.82
2150	1.88	24	24	4.6	33.02	33.41	34.34	36.20	37.03	37.59	37.73	38.05	38.52	38.75	38.98	39.17	39.54	39.91	41.77	42.69	47.34
2150	1.88	30	30	4.6	31.75	32.14	33.07	34.92	35.76	36.31	36.49	36.89	37.36	37.59	37.82	37.98	38.31	38.63	40.26	41.07	45.13
2150	1.88	36	36	4.6	30.47	30.86	31.79	33.65	34.48	35.04	35.25	35.73	36.20	36.43	36.66	36.80	37.08	37.36	38.75	39.45	42.93
2150	1.88	48	48	4.6	27.92	28.31	29.24	31.09	31.93	32.48	32.76	33.41	33.88	34.11	34.34	34.43	34.62	34.81	35.73	36.20	38.52
2150	1.88	60	60	4.6	26.49	26.92	27.92	29.93	30.77	31.32	31.53	32.02	32.48	32.72	32.95	33.04	33.23	33.41	34.34	34.81	37.13
2150	1.88	72	72	4.6	25.07	25.52	26.61	28.77	29.61	30.16	30.30	30.63	31.09	31.32	31.56	31.65	31.84	32.02	32.95	33.41	35.73
2150	1.88	96	96	4.6	23.28	23.67	24.60	26.45	27.29	27.84	28.12	28.77	29.24	29.47	29.70	29.79	29.98	30.16	31.09	31.56	33.88
2150	1.88	120	120	4.6	21.89	22.28	23.20	25.06	25.90	26.45	26.59	26.92	27.38	27.61	27.84	27.94	28.12	28.31	29.24	29.70	32.02
2150P	2.4	10m	0.17	5.0	62.66	63.08	64.08	66.08	66.98	67.58	67.88	68.58	69.08	69.33	69.58	69.68	69.88	70.08	71.08	71.58	74.08
2150P	2.4	20m	0.33	5.0	62.25	62.67	63.67	65.67	66.57	67.17	67.47	68.17	68.67	68.92	69.17	69.27	69.47	69.67	70.67	71.17	73.67
2150P	2.4	30m	0.5	5.0	61.83	62.25	63.25	65.25	66.15	66.75	67.05	67.75	68.25	68.50	68.75	68.85	69.05	69.25	70.25	70.75	73.25
2150P	2.4	1	1	5.0	60.58	61.00	62.00	64.00	64.90	65.50	65.80	66.50	67.00	67.25	67.50	67.60	67.80	68.00	69.00	69.50	72.00
2150P	2.4	2	2	5.0	58.08	58.50	59.50	61.50	62.40	63.00	63.30	64.00	64.50	64.75	65.00	65.10	65.30	65.50	66.50	67.00	69.50
2150P	2.4	3	3	5.0	55.69	56.13	57.17	59.25	60.15	60.75	61.09	61.88	62.38	62.63	62.88	63.00	63.25	63.50	64.75	65.38	68.50
2150P	2.4	6	6	5.0	48.51	49.00	50.17	52.50	53.40	54.00	54.45	55.50	56.00	56.25	56.50	56.70	57.10	57.50	59.50	60.50	65.50
2150P	2.4	9	9	5.0	45.26	45.75	46.92	49.25	50.15	50.75	51.13	52.00	52.50	52.75	53.00	53.20	53.60	54.00	56.00	57.00	62.00
2150P	2.4	12	12	5.0	42.01	42.50	43.67	46.00	46.90	47.50	47.80	48.50	49.00	49.25	49.50	49.70	50.10	50.50	52.50	53.50	58.50
2150P	2.4	18	18	5.0	38.80	39.25	40.33	42.50	43.40	44.00	44.23	44.75	45.25	45.50	45.75	45.95	46.35	46.75	48.75	49.75	54.75
2150P	2.4	24	24	5.0	35.58	36.00	37.00	39.00	39.90	40.50	40.65	41.00	41.50	41.75	42.00	42.20	42.60	43.00	45.00	46.00	51.00
2150P	2.4	30	30	5.0	34.21	34.63	35.63	37.63	38.53	39.13	39.31	39.75	40.25	40.50	40.75	40.93	41.28	41.63	43.38	44.25	48.63
2150P	2.4	36	36	5.0	32.83	33.25	34.25	36.25	37.15	37.75	37.98	38.50	39.00	39.25	39.50	39.65	39.95	40.25	41.75	42.50	46.25
2150P	2.4	48	48	5.0	30.08	30.50	31.50	33.50	34.40	35.00	35.30	36.00	36.50	36.75	37.00	37.10	37.30	37.50	38.50	39.00	41.50
2150P	2.4	60	60	5.0	28.55	29.00	30.08	32.25	33.15	33.75	33.98	34.50	35.00	35.25	35.50	35.60	35.80	36.00	37.00	37.50	40.00
2150P	2.4	72	72	5.0	27.01	27.50	28.67	31.00	31.90	32.50	32.65	33.00	33.50	33.75	34.00	34.10	34.30	34.50	35.50	36.00	38.50
2150P	2.4	96	96	5.0	25.08	25.50	26.50	28.50	29.40	30.00	30.30	31.00	31.50	31.75	32.00	32.10	32.30	32.50	33.50	34.00	36.50
2150P	2.4	120	120	5.0	23.58	24.00	25.00	27.00	27.90	28.50	28.65	29.00	29.50	29.75	30.00	30.10	30.30	30.50	31.50	32.00	34.50

Appendix M – Avon model sensitivity to rainfall and groundwater level memo, 2021[45]

[45] GHD internal reference: https://projects.ghd.com/oc/NewZealand1/ldrp097baselinemodel/Delivery/Documents/20201123_12506449_MEMO_Rainfall%20and%20groundwater%20sensitivity_Rev3.pdf

Memorandum

23 November 2020

To	Jo Golden		
Copy to	Tom Parsons		
From	Clemence Rolin, Tim Preston	Tel	
Subject	Avon model sensitivity to rainfall and groundwater level (Rev3)	Job no.	1250/6449/

1 Introduction

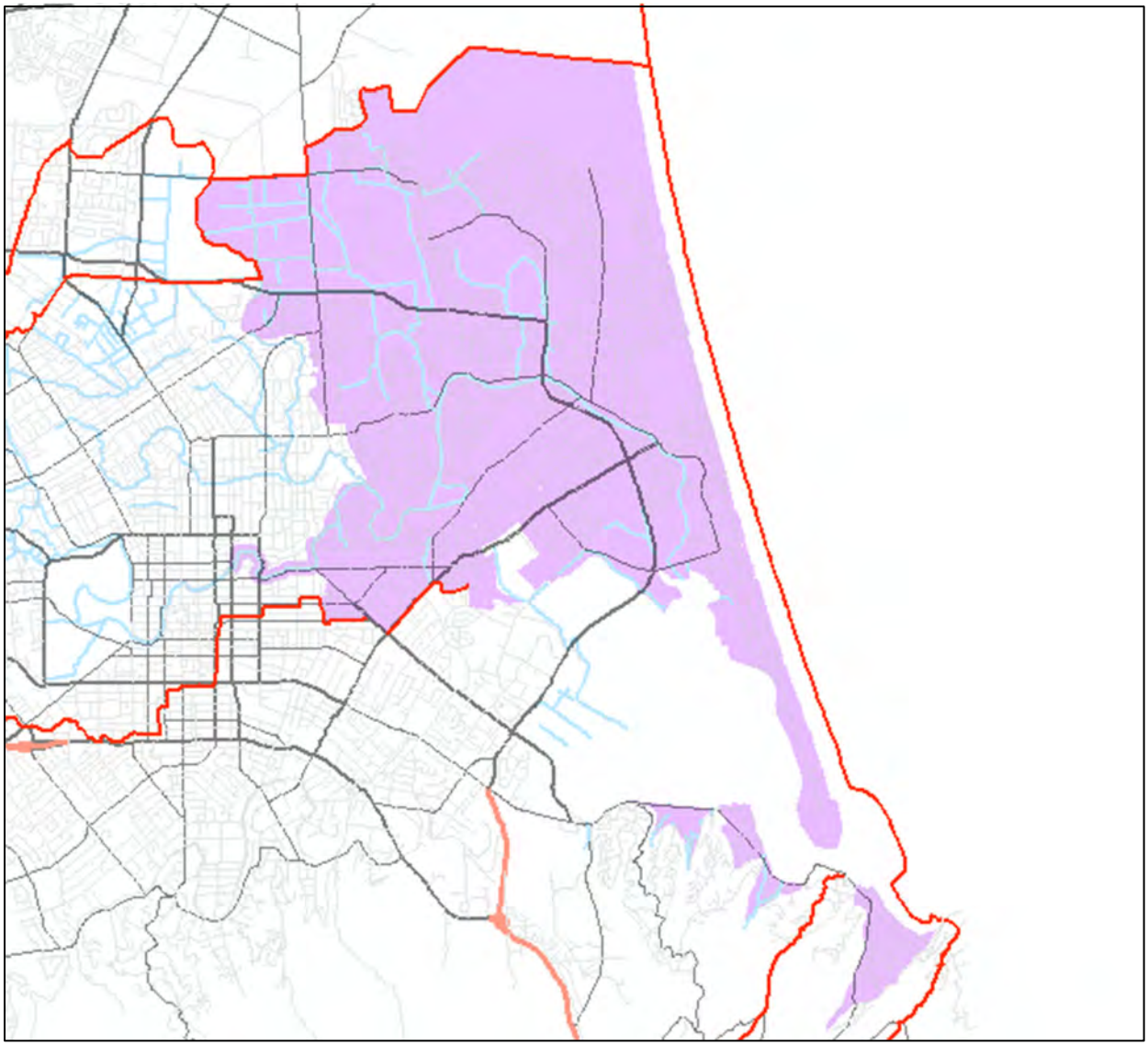
This memo on model sensitivity is a precursor to the baseline modelling work associated to Council's wider Multi-Hazard study to better understand flood hazard in Christchurch, as outlined in Jacobs August 2019 memo[1] . It will be appended to GHD's final report for LDRP97 Baseline modelling work.

The Multi-Hazard study project focuses on regional and local flooding, surface flooding and impact of future groundwater levels, for a project-specific 2020 existing development (ED2020) model, and a range of future development and scenarios (increased rainfall intensity, sea level rise, groundwater level rise, future earthquake). The sensitivity testing has utilised the Avon model as this catchment has been prioritised within the Multi-hazard investigation to align with Council's LTP infrastructure programme. On the Avon catchment, the project area of interest is on the eastern, downstream, flat catchment (refer to Figure 1).

This document outlines the results from the Avon/Estuary model sensitivity to groundwater and rainfall intensity increase. It will assist Council's decision on the groundwater surfaces and rainfall inputs into the model.

[1] JACOBS, Christchurch Multi-hazard Analysis, Climate Change Scenarios for Modelling. 08 August 2019

Figure 1 : Project area of interest for Avon/Estuary and Sumner model (purple)



2 Revision 3 Changes

This document replaces the previous memo from 10/12/2019 “Avon/Sumner models sensitivity testing memo” (rev2).

GHD’s rev2 memo submitted to Council 10/12/2020 included rainfall sensitivity testing for both Avon and Sumner catchments. The Sumner results have been removed from this memo as described in section 2.1 below. Another localised defect, which does not affect our conclusions, has also been identified on the rain-on-mesh rainfall and is described in section 2.2

2.1 Sumner rainfall sensitivity results

Subsequent to rev2 memo submission, it was found out that the Sumner RORB rainfall files used for the sensitivity testing were actually identical (both the nominally 5th percentile and 95th percentile rainfall were the same as the 50th percentile rainfall file). Accordingly, the results differencing presented on the Sumner catchment in the previous memo (rev2) where driven by hillside runoff were incorrect and would have misrepresented zero sensitivity as the inputs had not changed. The inputs and results correctly included different rainfall percentiles of the rain on mesh component (on the flat) and so results dependant only on the flat land runoff were valid. The hillside catchments cover a majority of the interesting flooding in the Sumner catchment, and the results should therefore not be relied on.

The sensitivity results from the Avon catchment have been enough to support resulting decisions and the Sumner model is being updated and further improved so there is no interest to produce and present the corrected results.

2.2 Defect in the Mt Pleasant/ Redcliffs/ Moncks Bay area (Avon model results)

Until recently GHD’s approach to RORB hillside hydrology modelling had been to delete rain-on-mesh rainfall from the hillside to avoid generating mesh rainfall in areas that would double-count RORB generated rainfall. However, from results inspection we found that the DHI computation process was still putting rain on the hillside and we understand that it was extrapolating rainfall data to fill in the null values and thus creating full depth rainfall on the hillside mesh.

In the technical work presented in this memo, the above issue results in approximately double-counted rainfall depth on the hillside catchments. In the Avon model these are limited to the “estuary” catchments of Mount Pleasant / Redcliffs / Moncks Bay area.

We have since fixed this by reducing the M21 mesh extents for both Sumner and Avon model catchments to eliminate the hillside mesh areas. This eliminates the risk of repeating this technical issue and removes the need to delete the rainfall from the hillsides for future mode runs[2].

GHD conclusions and recommendations in this memo are driven by flood level differences in the Avon river catchment (not the Estuary catchments), so our conclusion and recommendation remain unaffected by this issue.

[2] This was all documented to Council by email from Tim Preston Thu 23/07/2020 2:33 PM “RE: Chch hillside rainfall - Avon/Sumn changes”

3 Rainfall Sensitivity (Avon/Estuary model)

3.1 Summary Rainfall Sensitivity Methodology

The methodology can be found in Appendix C.

The sensitivity tests were based on the 2100+ time horizon model for the 2%AEP rainfall (all durations) with 1.06m SLR, no future earthquake and stopbanks down scenario, and are comparing the 5th (low), 50th (median) and 95th (high) rainfall percentiles.

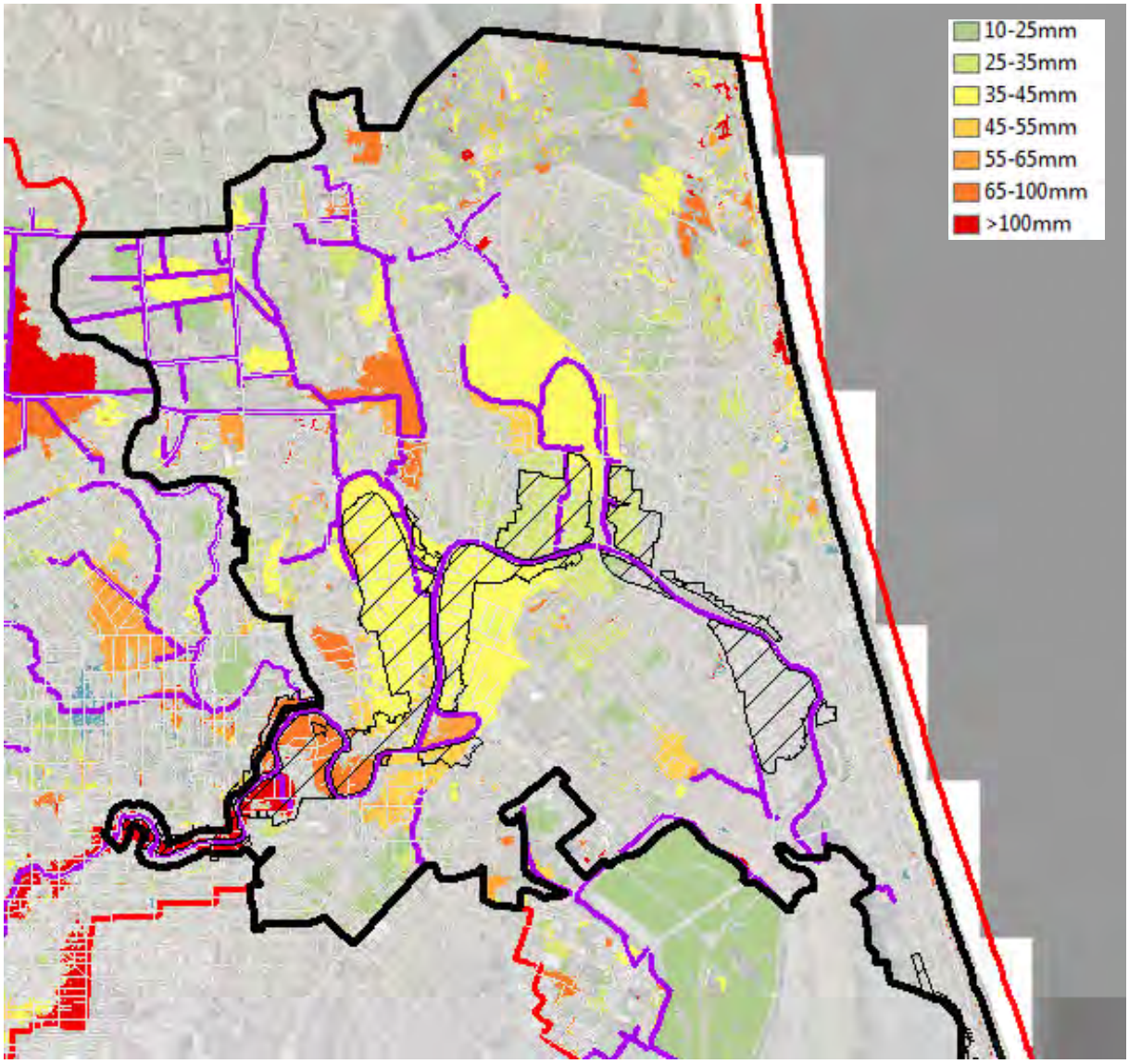
3.2 Rainfall Sensitivity Results

In the section below, differences in flood levels in the various areas are given between the 95th (high) and the 50th (median) rainfall percentiles that have a higher response to the differences between the the 50th (median) and the 5th (low) rainfall percentiles.

This section should be read in conjunction with the flood difference maps in Appendix A (general colour ramp 10-400mm) and with Figure 2 below (refined resolution 10-100mm over the project area).

Within the project extent of the Avon catchment, the changes are mostly observed in (but not limited to) areas that are not built-up (either red zone or “wet” system like Horseshoe Lake and Travis Wetland. Outside the Avon project area of interest, the differences do affect built-up / residential areas.

Figure 2 : Difference in max of max flood level between the 95th (high) and 50th (median) rainfall intensity.



The red zone is shown in black hatched and the project area in bold black
G:\51\12506449\GIS\Maps\Working\wkg_Clem\20191128_12506449_WKG_Results_RainfallSensitivity_95ile_50ile.mxd

3.2.1 Horseshoe Lake / Travis swamp/Avondale system

The results show an increased flooding in these areas of +45mm (+39mm) in Horseshoe lake, +39mm (+34mm) in Travis wetland and +36mm (+31mm) in Avondale on the true right bank of the Avon river. The first result is difference from 95th rainfall percentile minus 50th rainfall percentile (and results given in bracket are 50th rainfall percentile minus 5th rainfall percentile).

3.2.2 Moncks Bay

In Moncks Bay, there is an increased flooding of +37mm (+15mm), located at the junction of the Rifle Range Waterway and Rifle Range Drain Overflow (called esty.riflew and esty.rifle in the model)

3.2.3 Other location of interest outside the project area of interest

Cranford

In the Cranford basin (south of QEII), a typical flood level increase of +78mm (+83mm) is shown in the results (note that in the area, the medium/low differencing is higher than the high/median difference)

Flockton area

In the Flockton area, a typical flood level increase of +59mm (+36mm) is shown in the results.

Kyle Street either side of railway

Additional ponding is shown in the results either side of the railway embankment. On the upstream side (west) of the railway +26mm (+20mm) and on the downstream side (east) of the railway +64mm (+58mm) (note that in all scenarios 5th/50th/95th rainfall percentiles, the railway embankment is overtopping resulting in increased flood levels downstream).

Gloucester Street

The Avon river fringes in the CBD also show an increased flood depth of up to +144mm (+142mm) in the vicinities of Gloucester Street bridge.

3.3 Rainfall Recommendation

There is no particular surprise in the rainfall sensitivity and the default recommendation remains to use the 50th rainfall percentile as it will generate the best unbiased estimate of future flood risks.

Council may wish to consider using the 95th rainfall percentile for their residual risks model run (2.4m SLR).

4 Groundwater Sensitivity (Avon/Estuary model)

4.1 Summary Groundwater Sensitivity Methodology

The methodology can be found in Appendix C.

The sensitivity tests were based on the 2100+ time horizon model for the 2%AEP rainfall (all durations) with 1.06m SLR, no future earthquake and stopbank down scenario, and are comparing the 85th percentile groundwater condition to the 50th percentile groundwater condition. Groundwater condition includes for predicted groundwater levels as well as likely capillary rise and soil effective porosity variations as detailed in Appendix C.

4.2 Pre-assessment of Avon Groundwater Sensitivity Results

GHD's previous work (refer May 2019 memo - Appendix D) explored input data with a view to understand likely impacts on runoff and flood risk associated with variable groundwater levels. The key finding from this earlier work looked at 85th percentile groundwater levels and compared the current situation with a future 1m SLR scenario, with a nominally 1% AEP (100 yr ARI) rain event (150-200mm of rainfall depth).

The key findings of this earlier work are presented in Appendix D, Map 8 and discussed in section 3.2 of Appendix D. This highlighted five areas where increased runoff would have potential to accumulate and could cause material differences in flood level. Map 4 also presents results associated with a 75-100mm rainfall event depth.

The sensitivity work presented herein uses the 2% AEP and 1.06m SLR scenario with stopbanks down and nil anthropogenic changes after 2019. This is broadly in line with the scenario for the previous work described in Appendix D. The event 50th percentile rainfall depths for 3hr, 18hr and 36hr results are 54, 119 and 149mm respectively (based on Anzac Drive bridge location with peak intensities of 857, 316 and 199 mm/day). The areas noted in the May 2019 work are discussed further as follows.

4.2.1 East of Travis Wetland

(Parklands, North New Brighton, Beach Rd). The May memo anticipated greater runoff from this area. In the 2% AEP event the pipe network capacity is likely to be exceeded and overland flow paths activated in most areas for either groundwater scenario. While this flow path activation would occur in a smaller event when the groundwater level is higher this study only looks at a single 2%AEP rain event. The results from this study are therefore likely to feature areas which are poorly served by overland flow paths and do not have their surface storage capacity exhausted by the 2%AEP event.

The conditions east and west in this area are expected to be quite different. In the west the differences in runoff flows are not expected to make detectible differences to flows or levels in the main river stem. In the east however flow drains through pipe systems under the sand dunes which have nil overland flow capacity. The storage behind the sand dunes is expected to fill to a higher level with increase GWL.

The sand dune pipe outlets do not have flap gates in the model (and we understand not in reality) so some results in low lying areas will be drowned out by sea level flooding.

4.2.2 North of Horseshoe Lake (Burwood and part of Shirley Golf course)

The main piped inflows into Horseshoe Lake from the north are through two large culverts under Lake Terrace Drive (on No2 Drain and Old No2 Drain branches. Increased timeseries flows in these two culverts (and their corresponding culverts under QEII Drive) from the 18 hour event will be expected, and some material increases in flood depth and levels will also be expected.

To some extent then increased runoff flows and flow levels associated with elevated groundwater in this area are likely to be drowned out by tide levels.

4.2.3 West of Hagley Park

The May 2019 work (Maps 4 & 8) shows some areas west and northwest of Hagley Park (Riccarton, Fendalton, Ilam, Burnside, Strowan) at risk of saturation from the 100 – 200mm rain event with the 85th percentile groundwater levels.

We anticipate modestly increased runoff into these areas during the final parts of the long duration rain events, and accordingly increased flood levels. Subtle increases in local overland flow path flooding similar to west Parklands above may also be anticipated.

4.2.4 Southshore

This area showed some significant expected increase in saturation and runoff from the 100mm rainfall depth event (typically longer duration event), however the prevalent critical duration for fluvial flooding in the area is less than one hour, so the increased runoff in the longer larger events is unlikely to produce any material increase in max of max flood levels.

4.2.5 Summary

Discernible results in terms of notable flood differences between 50th and 85th percentile groundwater level sensitivity runs are anticipated in the following areas.

- Eastern areas of Parklands that drain through the sand dunes
- North of Horseshoe Lake
- West of Hagley Park

Within these areas, locations with longer critical durations (12-24 hours) are expected to show the largest changes.

4.3 Actual Groundwater Sensitivity Results

Actual model sensitivity results are presented in Appendices A (flood difference maps) and B (timeseries graphs). The results reflect all the above discussed prior expectations.

Around Parklands in addition to response associated with the main sand dune formation the model results also respond similarly to secondary topography features further west such as Burwood Landfill and south of there.

The area north of Horseshoe Lake responds again as expected although only upstream (North of QEII Dr) and east on No2 Drain channel. Flooding appears to remain in-channel on the western Old No2 Drain meaning there is no apparent sensitivity response on the floodplain.

West of Hagley Park areas with 10-30mm of sensitivity response are common and concentrated on areas of ponded model flooding such as Kyle Street and Cross Stream. Around Gloucester St, floodplain differences of 40-50mm are common along the lower lying river margins.

Other positive result areas are generally well known ponded flooding areas such as Travis Wetland, Cranford Basin and the Flockton St area. The model flooded area of boundary ponding near Moorhouse / Madras also features in the sensitivity.

4.4 Interpretation of Groundwater Sensitivity Results

Results are generally as expected. The results appear credible and reinforce confidence that the model methodology is functioning as expecting.

Visible highlights on the sensitivity map are typically 25-50mm change in depth. Generally, areas with more than 50mm sensitivity are rare so the magnitude of response to the tested sensitivity variables is toward the low end of expectations and not highly significant.

There are some minor areas in the difference result (small spatially) showing more than 100mm sensitivity results. These are likely to have similar causes to other areas of sensitivity, but with more locally acute circumstances (e.g.: Moorhouse / Madras) and in some cases may represent secondary ponding areas which are only activated after a primary ponding area becomes full and overflows. None appear to have any strategic or particular interest.

There are some minor areas in the difference result (small spatially and vertically) showing negative sensitivity results. These are not of any strategic interest and are likely caused by minor model instabilities and/or localised flood plain areas where peak flood levels might not be realised within the specified model run durations.

Interestingly, the areas showing sensitivity to the groundwater levels are also showing sensitivity to change to rainfall intensity. Those are typically in low lying ground areas where groundwater level is likely to rise closer to the surface, and also where additional runoff from increased rainfall intensity will pond.

4.5 Groundwater Recommendation

The outcome of the groundwater sensitivity test shows that the model flood results typically have low responsiveness to initial groundwater conditions, with much of the sensitivity limited to areas already dedicated to floodplain storage.

As part of the wider Multi-Hazard Analysis NIWA[3] investigated the statistical coincidence of various extreme weather parameters (Smart, et al 2018). It was found that there was a correlation between antecedent rainfall (i.e. the proxy used for ground saturation) and river flows.

[3] NIWA, Coincident Inclement Weather Study for climate change planning. Christchurch City Council. May 2018

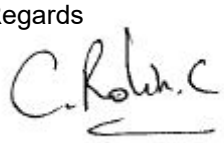
Therefore the ‘wet groundwater’ scenario (85th percentile) is recommended for use within the baseline modelling scenarios as this reflects a wet antecedent condition.

5 Conclusion

The sensitivity of flood risks in the Avon/Estuary catchment to variable rainfall intensities and variable groundwater conditions has been tested by computer modelling study. This showed a positive response to rainfall intensity changes and a low response to groundwater level changes.

GHD’s best recommendation is that Council adopt 50th percentile rainfall and 85th percentile groundwater levels for future baseline modelling for the LDRP097 Multihazard project. We also acknowledge that Council’s better understanding of context and especially the whole Multihazard project scope and objectives needs to be considered in making a final decision.

Regards



Clemence Rolin

Senior Water Modeller

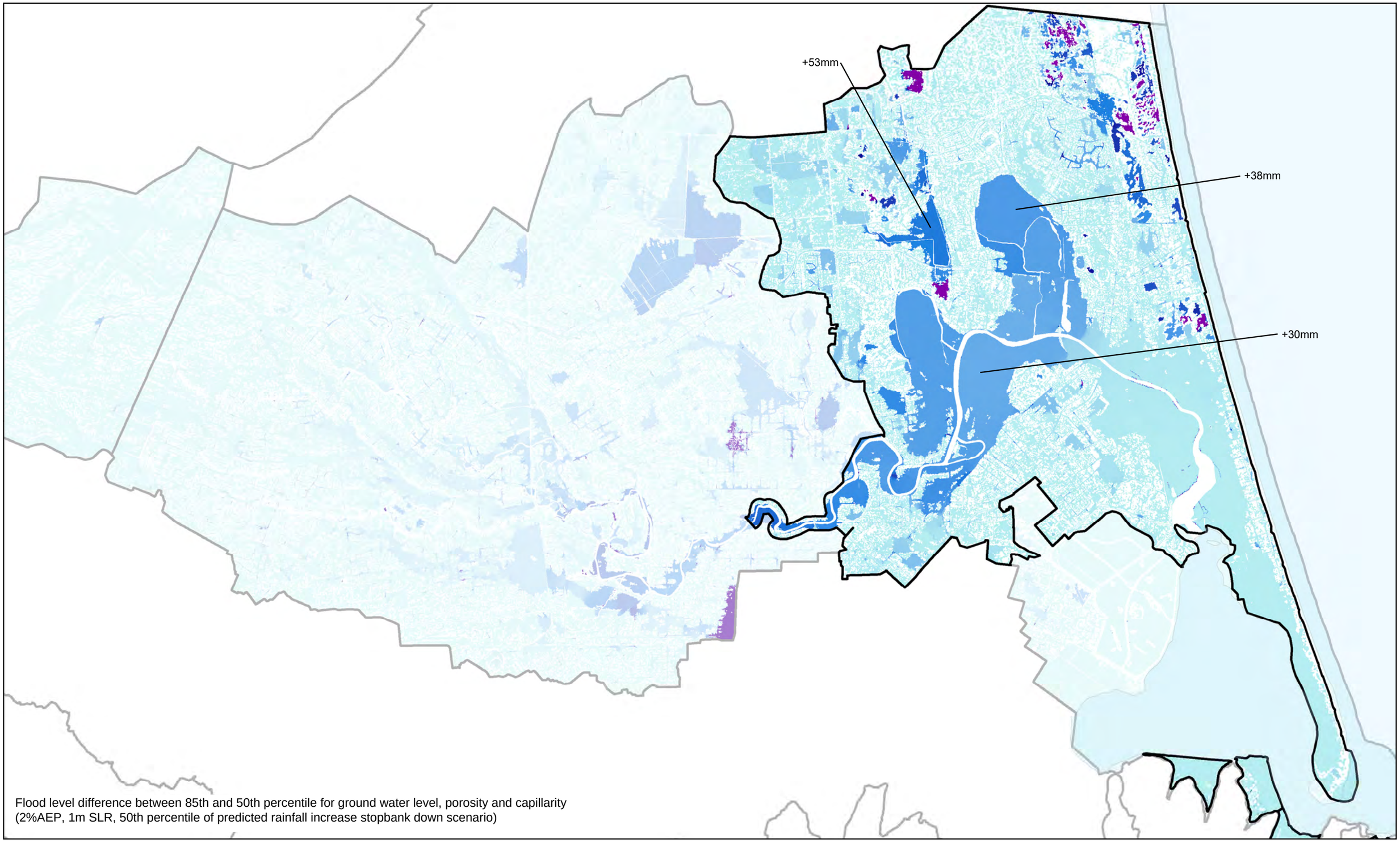
QA Signoff: Tim Preston

APPENDICES

A. Avon/Summer flood result difference maps (three differences vs the common 50yr result)
B. 1D Timeseries flow graphs (Gloucester St, No2 Drain, Parklands outfall)
C. Scope of Sensitivity Work (detailed methodology)
D. GHD memo “LDRP115 - Study of Ground Water Level Changes on Flooding - Summary of Study Outcomes” 20 May 2019

Appendices

Appendix A - Avon flood result difference maps



Flood level difference between 85th and 50th percentile for ground water level, porosity and capillarity
(2%AEP, 1m SLR, 50th percentile of predicted rainfall increase stopbank down scenario)

Paper Size A3

0 290 580 1,160 1,740 2,320

Metres

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

N

LDRP97 AOI

Catchment Boundary

>0.1m

High : 0.1m

Low : 0

Christchurch City Council
LDRP097 Baseline Modelling

Job Number 12506449
Revision A
Date 28 Nov 2019

Model sensitivity to groundwater

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Data source: GHD, AVON_R50G85_Minus_R50G50_L, 27/11/2019. Created by:mmarshall

Level 3, 138 Victoria Street, PO Box 13468, Christchurch 8141, New Zealand **T** 64 3 378 0918 **W** www.ghd.com

Map 1

Flood level difference between 95th and 50th percentile of predicted rainfall increase (2%AEP, 1m SLR, groundwater 85th percentile, stopbank down scenario)

Paper Size A3

0 345 690 1,380 2,070 2,760

Metres

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

N

LDRP97 AOI
Catchment Boundary
Avon M11 Network
RedZone
Railway

Difference in Max of Max flood levels:

10-25mm
25-50mm
50-100mm
100-200mm
200-400mm
>400mm

Note: Results outside AOI are 70% transparent

Christchurch City Council
LDRP097 Baseline Modelling

Job Number 12506449
Revision C
Date 10 Dec 2019

Model sensitivity to rainfall
Difference 95th percentile - 50th percentile

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Data source: GHD, AVON_R95G85_Minus_R50G85_L, SUMN_R95G85_Minus_R50G85, 03/12/2019. Created by:crolin

Level 3, 138 Victoria Street, PO Box 13468, Christchurch 8141, New Zealand T 64 3 378 0918 W www.ghd.com

Flood level difference between 50th and 5th percentile of predicted rainfall increase (2%AEP, 1m SLR, 85th percentile groundwater, stopbank down scenario)

Paper Size A3

0 345 690 1,380 2,070 2,760

Metres

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

N

LDRP97 AOI
Catchment Boundary
Avon M11 Network
RedZone
Railway

Difference in Max of Max flood levels:

10-25mm
25-50mm
50-100mm
100-200mm
200-400mm
>400mm

Note: Results outside AOI are 70% transparent

Christchurch City Council
LDRP097 Baseline Modelling

Job Number 12506449
Revision B
Date 09 Dec 2019

Model sensitivity to rainfall
Difference 50th percentile - 5th percentile

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Data source: GHD, AVON_R50G85_Minus_R5G85_L, SUMN_R50G85_Minus_R5G85, 03/12/2019. Created by:mmarshall

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Map 3

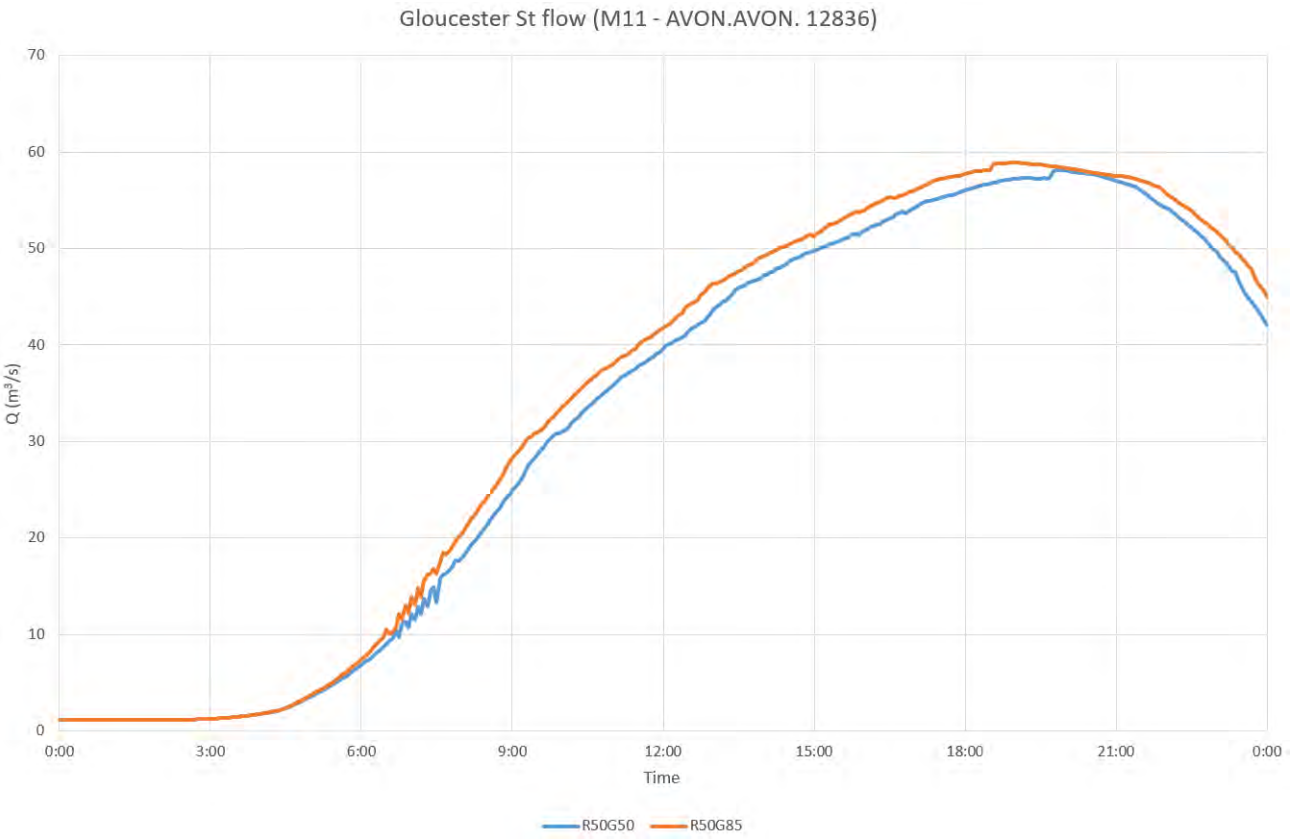
Appendix B – 1D Timeseries flow graphs

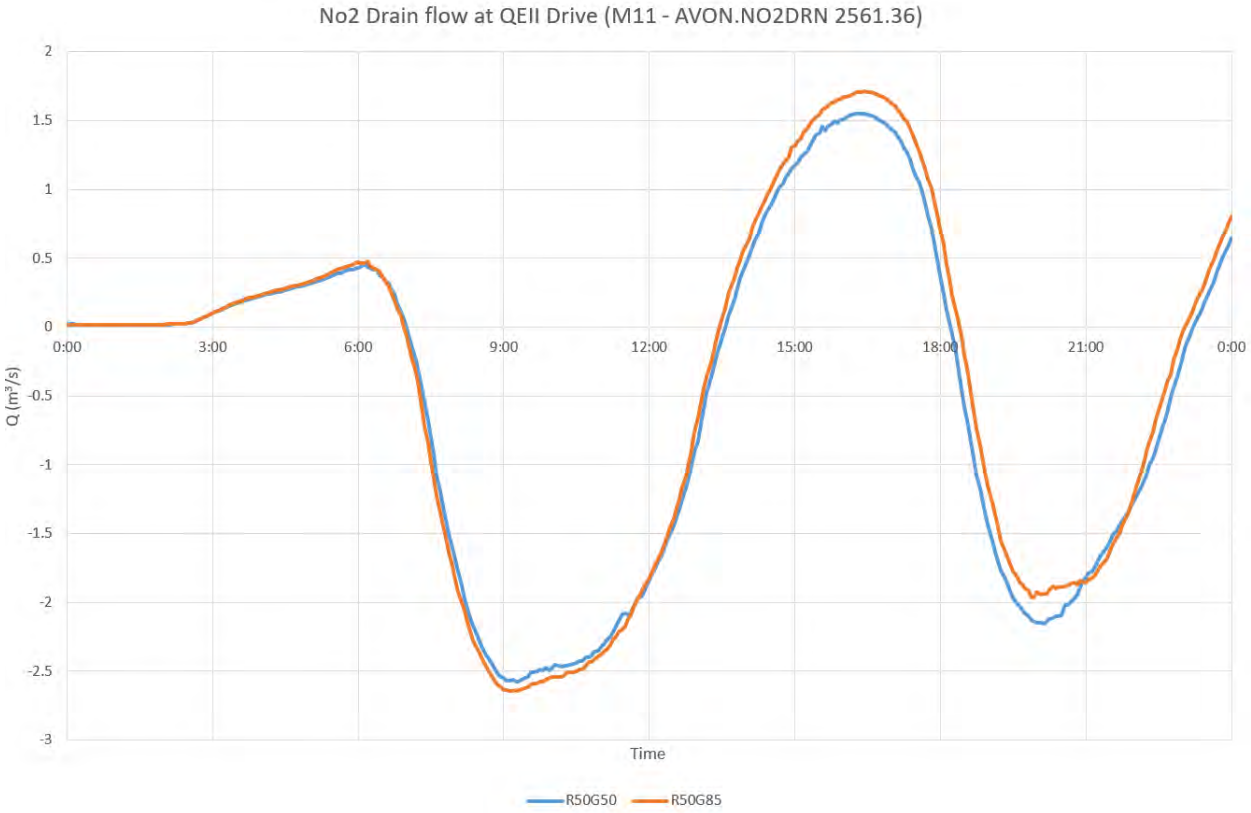
Groundwater sensitivity results (for the 2%AEP, 18hr pluvial event)

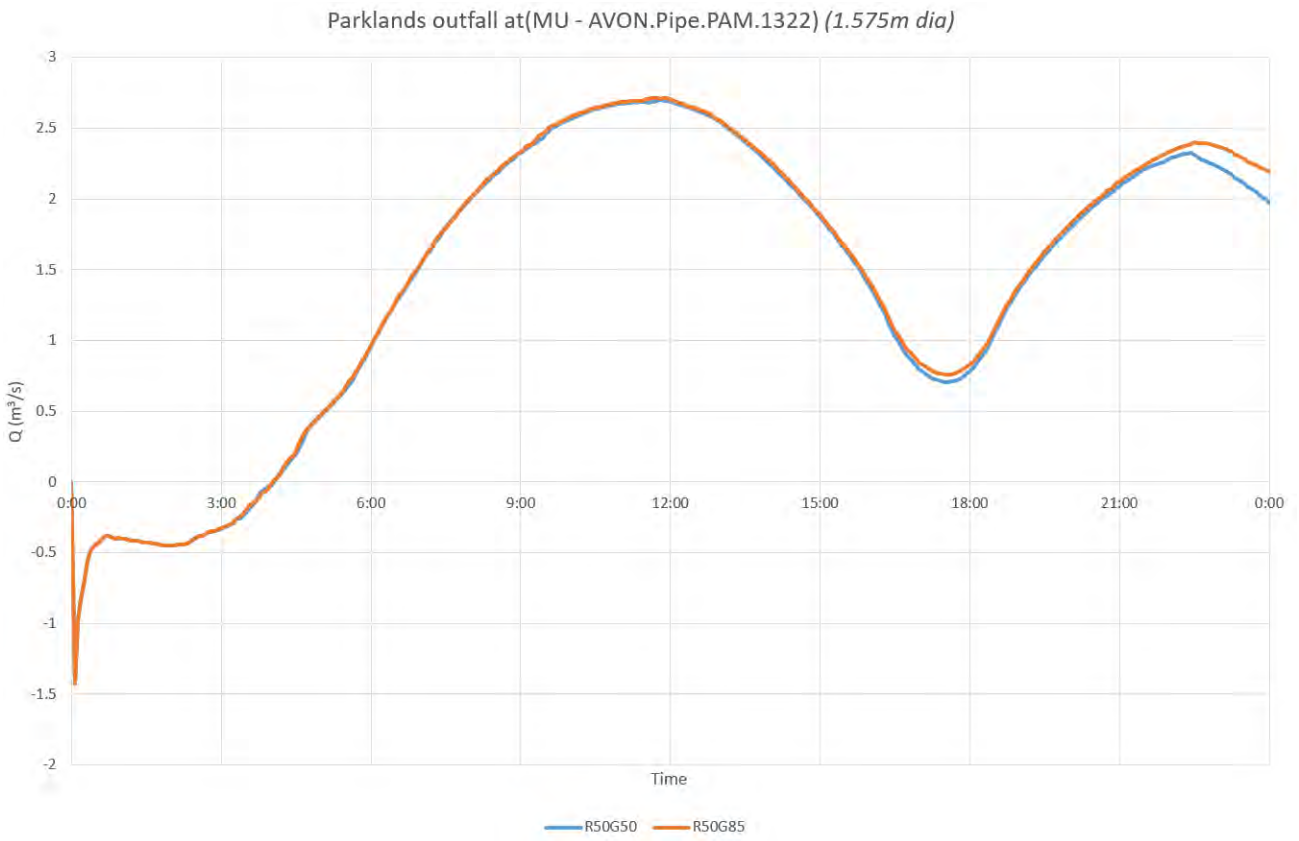
Gloucester St flow (M11 – AVON.AVON 12836)

No2 Drain flow at QEII Drive (M11 – AVON.NO2DRN 2561.36)

Parklands outfall flow at (MU – AVON.Pipe.PAM.1322*) (1.575m dia)*


Gloucester St flow (M11 - AVON.AVON. 12836)
70
60
50
40
30
20
10
0
Q (m³/s)
0:00
3:00
6:00
9:00
12:00
15:00
18:00
21:00
0:00
Time
R50G50
R50G85


No2 Drain flow at QEII Drive (M11 - AVON.NO2DRN 2561.36)
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
-2.5
-3
Q (m³/s)
0:00
3:00
6:00
9:00
12:00
15:00
18:00
21:00
0:00
Time
R50G50
R50G85


Parklands outfall at(MU - AVON.Pipe.PAM.1322) (1.575m dia)
3
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
Q (m³/s)
0:00
3:00
6:00
9:00
12:00
15:00
18:00
21:00
0:00
Time
R50G50
R50G85

Appendix C - Content Scope of Sensitivity Work (detailed methodology)

Council instructed GHD to undertake sensitivity testing on the Avon and Sumner model prior to commencing model result production. The models are tested for their sensitivity to change in groundwater (GW) level (Avon model only) and rainfall increase (Avon and Sumner models). The tests will be done on the 2100+ time horizon model for the 2%AEP rainfall (all durations) with 1.06m SLR, no future earthquake and stopbank down scenario[4].

Model sensitivity to Groundwater (AVON model only)

Background

As part of the Multi hazard project, the infiltration approach of the Avon and Sumner models has been changed from Horton's decay to M21 Infiltration module using constant infiltration with capacity[5]. The infiltration capacity is defined by:

- the depth to GW at the start of the simulation
    - depth to GW includes an allowance for capillary rise as described in "Approach" below
- the (constant) rate of surface infiltration
- the soil (effective) porosity

For design events, the primary intention was to use the GW level equal to the 85th percentile (data produced by Aqualinc), but Council would like to understand the model sensitivity to the initial GW level by comparing model results obtained with the 85t^{h} percentile to results obtained with the 50th percentile.

Input data:

- Aqualinc's 50th and 85th percentile groundwater level surfaces received 09/09/2019.
- 50th percentile rainfall generated by GHD
- Soil map for porosity/capillarity analysis : McCahon et al 1991 "The earthquake hazard in Christchurch: A detailed evaluation, Research report to the EQC"
    - Figure 7.7, Soil Type Map 0-2m depth
- "Unsaturated Zone Hydrology" (textbook), Gary Guymon, Irvine CA, 1994, ISBN 0-130369083
    - Effective Porosity: figure 2.6, page 24
    - Capillary rise: table 5.1, page 85
- https://agupubs.onlinelibrary.wiley.com/doi/full/10.1029/2006WR004855 - describes subsoils draining down to equilibrium moisture content after saturation

Approach

GHD will:

- run 2 model batches (5 durations per batch) for the Avon model with the 50th percentile rainfall,
    - with the 50th percentiles GW depth, capillarity rise (within vadose zone) and porosity

[4] Infrastructure upgrades in the model are limited to the project area of interest as set out in Figure 1 of the covering memo (GHD, Avon models sensitivity to rainfall and groundwater level, November 2020).

[5] The sensitivity runs have used infiltration parameters derived from the March 2014 calibration event. This re-calibration / validation and the resulting parameters will be described in the final project report

- o with the 85th percentiles GW depth, capillarity rise (within vadose zone) and porosity

These setups require calculation of groundwater depth from level, selection of capillary rise parameters, porosity and infiltration rate and modelling methodology for applying these.

Initially Engineering parameters for capillary rise and porosity were derived from references as follows.The 50th and 85th percentile porosity per soil type are shown in Table 1 below.

Table 1 : capillarity and porosity per soil type used in the GW sensitivity testing

Soils classification (from soil map)	Description	Capillary rise (m) 50th percentile*	Capillary rise (m) 85th percentile (inferred)	Effective Porosity (Fig 2.6)	Model porosity 50% GW (inferred)	Model porosity 85% GW (inferred)
C	Fine gravel	0.025	0.050	15%	14%	13%
M	Medium sand	0.246	0.492	22%	15%	10%
F	Silt - <0.05mm	2.000	4.000	15%	7.5%	2.5%
P	Peat	1.000	2.000	20%	15%	10%
X	Fill	0.065	0.130	20%	15%	10%
W	Water (n/a)	-	-	0%	0%	0%

1. The soils classification was defined by the soils map input data
2. GHD related these classifications to likely soil descriptions to enable them to be related to the other references
3. 50th percentile capillary rise was then determined directly from Guymon table 5.1
4. 85th percentile capillary rise was inferred as being twice the above values based on understanding that the subsoil moisture will likely be higher than equilibrium levels in an 85th percentile case due to recent antecedent pre-wetting and/or also representing a slightly more cautious approach to the soil type classification
5. The selection and application of the above porosity parameters is described further below
6. Reference values for peat and fill were not found in the literature and have been assumed. Spatially there are only small pockets of fill but there are larger areas locally significant of peat.

Groundwater Depth:

It was decided to apply the capillary rise distance into the modelling by elevating the groundwater surface. This is consistent with understanding the subsoil will be saturated in this capillary zone and will not provide storage capacity for infiltration which is consistent with the then modelled function.

Groundwater depth was primarily calculated from the input level data using the specific model bathymetry (M21 ground levels) ensuring that the effective model groundwater surface accurately matched the input data in level terms. The capillary rise distance was then subtracted from this depth.

There were two exceptions to the above which were also applied:

1. If the groundwater level without capillary rise was above ground level then groundwater depth was set to zero (not negative) and capillary rise was also set to zero.
2. If the capillary rise distance reached into the topsoil soil (which was defined as the top 200mm below ground) then any capillary rise in this topsoil zone was nullified

Effective Porosity

Effective porosity is understood to represent the void spaces in soils which are reasonably available for water movement and wetting / drying in typical field conditions. It differs from (total) porosity by the

exclusion of unconnected or dead end pores and water held by molecular forces and surface-tension forces.

Effective porosity is defined as a dried soil. The exact specifications for drying are not standardised but in effect any moisture content present in the effective porosity space further reduces the storage capacity for infiltration in the model function.

Effective porosity has been taken from the input data Fig 2.6. Model porosity has then been reduced by prudent assumption of the impact of likely antecedent moisture within that effective porosity. Topsoil was not considered separately in terms of porosity as the model setup does not facilitate that, so model porosity is uniform at all depths below ground within the model groundwater depth.

The above derived variables were processed from GIS form into model ready DFS2 input data (using a 10m grid resolution).

Electronic Deliverables

- “Max of max” flood levels generated for each batch
- Difference between the two “max of max” flood levels
- Graph showing the peak flow and level at Gloucester Street and Bridge Street for the 2% 18hr runs associated to 85th percentile and 50th percentile.

Model sensitivity to Rainfall intensity (AVON and SUMNER model)

Background

For design events, the primary intention was to use the 50th percentile increase, but Council would like to understand the model sensitivity to the rainfall percentile by comparing model results obtained with the 5th (low), 50th (median) and 95th (high) rainfall percentile.

The percentage rainfall increase was generated by Jacobs[6] based on different emissions scenarios as published in the MfE (2017) Coastal Hazards and Climate Change[7] and MfE (2018) Climate Change Projections for New Zealand[8]. The percentage increase varies with time horizon (associated to predicted temperature increase), AEP, rainfall duration and percentile. The tables below summarise the percentage increase applied to the base case rainfall data (based on WWDG, with no climate change increment, and aerial reduction factor of 0.95[9]) for the sensitivity testing.

Table 2: % change factor by duration and rainfall increase percentile

Percentage Change Factor for the 2% AEP event*				
Scenario	e	Low (5th)	Median (50th)	High (95th)
2100	3	27.6%	35.7%	53.2%
	6	24.2%	32.1%	48.3%
	9	22.4%	30.1%	45.6%
	09T	20.6%	28.0%	41.6%
	12	20.7%	28.1%	42.9%
	18	17.6%	26.0%	39.2%
	18T	15.8%	24.2%	36.8%
	36	12.1%	22.4%	34.0%

* By default, all modelled events are "2% AEP rain- 20% AEP tide" combination except tidal event (09T and 18T) which are "20% AEP rain-2% AEP tide" combination

Table 3 : Rainfall intensity and accumulated depth by duration and rainfall increase percentile

Rainfall intensity (and accumulated depth) for the 2% AEP events *				
Scenario	Duration (hrs)	Low (5th)	Median (50th)	High (95th)
2100	3	17mm/hr (51mm)	18.1mm/hr (54.3mm)	20.4mm/hr (61.3mm)
	6	11.6mm/hr (69.8mm)	12.4mm/hr (74.3mm)	13.9mm/hr (83.4mm)
	9	9.3mm/hr (84mm)	9.9mm/hr (89.3mm)	11.1mm/hr (99.9mm)
	09T	5.4mm/hr (48.2mm)	5.7mm/hr (51.2mm)	6.3mm/hr (56.6mm)
	12	7.9mm/hr (95.4mm)	8.4mm/hr (101.2mm)	9.4mm/hr (112.9mm)
	18	6.3mm/hr (113.5mm)	6.8mm/hr (121.6mm)	7.5mm/hr (134.3mm)
	18T	3.6mm/hr (65.1mm)	3.9mm/hr (69.9mm)	4.3mm/hr (76.9mm)
	36	3.9mm/hr (140.8mm)	4.3mm/hr (153.7mm)	4.7mm/hr (168.2mm)

Note: Intensity and depth have been calculated for where the P24 is 50mm

[6] JACOBS, Christchurch Multi-hazard Analysis, Climate Change Scenarios for Modelling. 08 August 2019.

[7] Ministry for the Environment 2017. Coastal Hazards and Climate Change. Guidance for Local Government.

[8] Ministry for the Environment 2018. Climate Change Projections for New Zealand: Atmosphere Projections Based on Simulations from the IPCC Fifth Assessment, 2nd Edition. Wellington: Ministry for the Environment.

[9] GHD, Citywide Flood Modelling (LDRP044) Model build report – Avon/Estuary – rev 5. December 2018.

Approach

GHD will:

- run 2 model batches (5 durations) for the Avon model with the 85th percentile GW surface,
    - with the 5th percentile rainfall
    - with the 95th percentile rainfall

Electronic Deliverables

- max of max level results of the 2% AEP 1.06m SLR with low, median and high % increase.
- up to 3 rasters of level differencing to compare the results to each other.

Appendix D - GHD memo “LDRP115 - Study of Ground Water Level Changes on Flooding - Summary of Study Outcomes” 20 May 2019

Memorandum

20 May 2019

To	Midhun Joseph (CCC)		
Copy to	Tom Parsons (CCC), Jo Golden (CCC), John McArthur (GHD)		
From	Tim Preston	Tel	03 378 0913
Subject	LDRP115 - Study of Ground Water Level Changes on Flooding - Summary of Study Outcomes	Job no.	51/37994/

1 Background

Council's future approach to flood modelling is to consider changes to rainfall, sea level rise and groundwater levels and how they vary as a result of projected temperature change.

This study has been undertaken to review future depth to groundwater projections, to broadly assess the potential impact a significant reduction in groundwater could have on surface runoff and to establish a methodology associated with modifying existing flood model parameters to simulate groundwater depth reductions.

Depth to groundwater projections have been established by Aqualinc and included in their Report (C17054, dated 29/01/2018); *LDRP45: Impacts of earthquakes and sea level rise on shallow groundwater levels* (herein referred to as Rutter, 2018). These projections include an 85th percentile depth (base case) and changes due to predicted sea level increases of 0.19 m, 0.4 m and 1 m respectively.

2 Sea Level Rise Impact on Groundwater Depth

Table 1 summarises the reduction in depth to groundwater due to various sea level rise (SLR) scenarios at selected locations shown on Figure 1. This information has been extracted from Rutter, 2018.

Table 1 – Reduction in Depth to Groundwater v SLR

Area	0.19 m SLR	0.4 m SLR	1 m SLR
Avondale	0.08	0.18	0.45
South New Brighton	0.19	0.40	1.00
Linwood	0.16	0.34	0.86
QEII (Clare Park)	0.10	0.20	0.51
Flockton	0.01	0.02	0.05

GHD Limited
Level 3 138 Victoria Street Christchurch Central 8013 PO Box 13468 Christchurch 8141 New Zealand
T 64 3 378 0900 **F** 64 9 370 8001 **E** chcmail@ghd.com **W** www.ghd.com

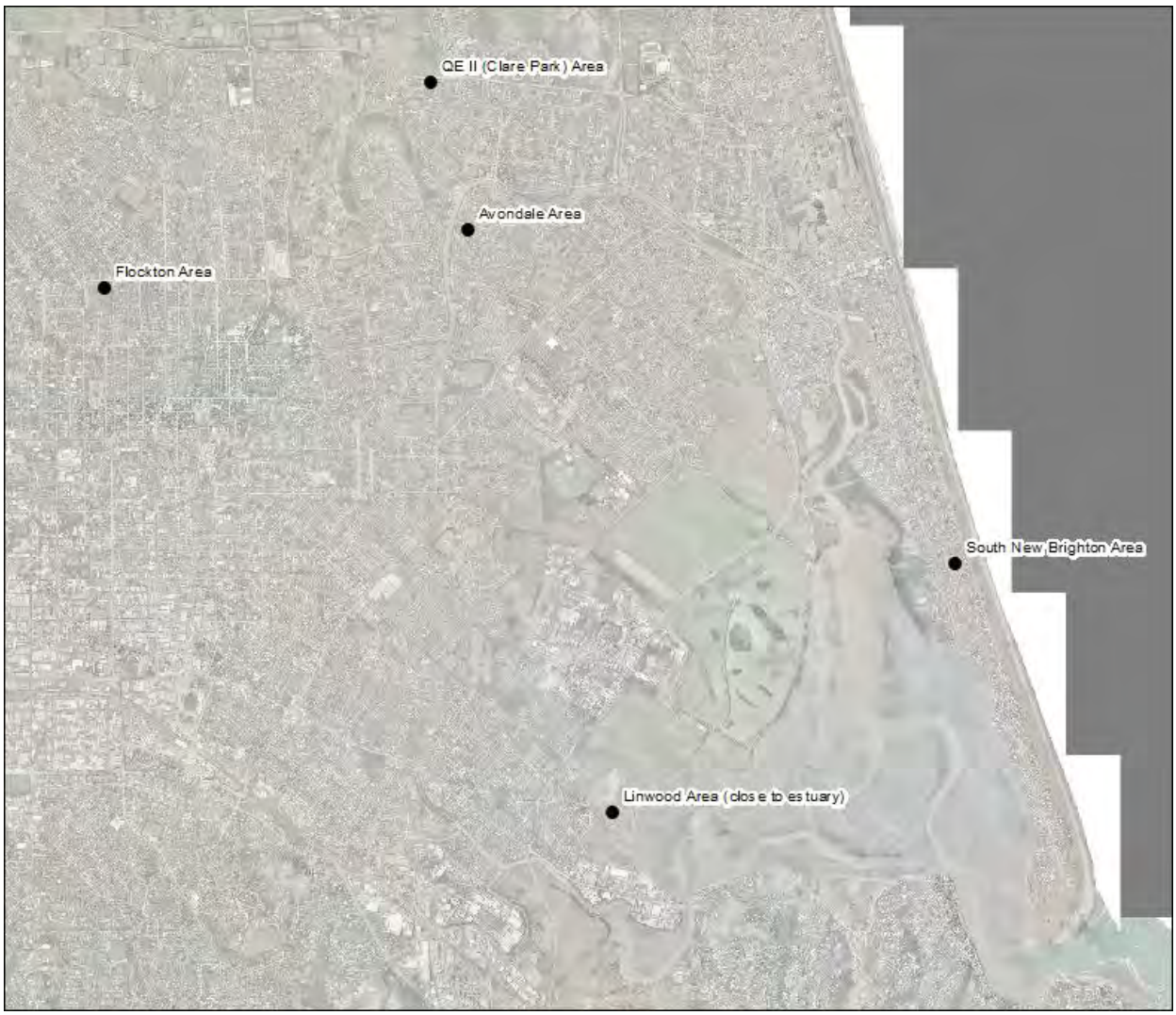


Figure 1 – Map of Locations in Table 1

The Aqualinc groundwater modelling predicts the main areas where groundwater depths are impacted by SLR are adjacent to coastal and/or estuarine environments with the South New Brighton groundwater depth reduction the same as the SLR and the Linwood groundwater depth reduction approximately 85% of the SLR. Further inland but adjacent to tidal reaches, the groundwater depth reduction is approximately 40 to 50% of the SLR at the coast. The various SLR scenarios have little impact on Flockton Basin or other areas more distant from the coast.

3 Likely Impact on Runoff Characteristics

Existing flood models associated with Christchurch river catchments typically manage infiltration using the Horton's equation which includes an initial infiltration rate, a final infiltration rate and a decay rate based on the time for the initial infiltration rate to reduce to the final value. Christchurch City Council's Waterways, Wetlands and Drainage Guide provides typical values for standard soil infiltration types. A range is provided for the initial infiltration rate to take account of antecedent conditions.

Typical design storm rainfall depths are tabulated below. These include the current 16% allowance for climate change.

Table 2 – Typical Christchurch Design Storm Rainfall Depths

Total Rainfall Depth (mm) - P24 = 50					
Duration (hr)	**ARI (years)**				
	10	50	100	200	500
2	28	40	46	52	61
6	49	70	80	90	106
12	69	98	112	127	149
24	97	139	158	180	210
48	120	171	195	222	259

More frequent events (75-100 mm)

Less frequent events (150-200 mm)

To assess the likely impact of the SLR scenarios of 0.4 m and 1.0 m on the extent/location of areas which may become saturated, the following approach has been taken.

i. Conceptually we recognise the main interest in large flood generating rain events have depths in the range 150 -200 mm of rainfall depth, however for this study the impacts of rising groundwater level are expected to be more predominant for medium and smaller flood events. Consequently we have evaluated both a 75-100 mm rainfall depth range to represent more frequent events and 150-200 mm to represent less frequent events. Based on soil effective porosity values in the range of 0.15 to 0.20, such events would fill the soil porosity in areas where depths to groundwater are less than 0.5 m.

ii. We have also tested sensitivity to rainfall depth (in terms of additional areas that may become saturated) by evaluating depth to groundwater of 0.25 m (essentially half the above rainfall i.e. 38 – 50 mm).

iii. Accordingly, areas with depth to groundwater less than 0.25 m and 0.5 m have been mapped for each of the base case (85th percentile) and 0.4 m and 1 m SLR cases. The difference in these extents highlights new locations where the soil saturation may be expected to occur as a result of the higher groundwater levels.

iv. Soil types within the study area have been given a rating based on the impact the soil type has on infiltration capacity and associated increase in runoff that would occur once soil saturation is reached. Very poorly drained soils have been given a rating of 0 (as saturation has only minor impact on already high runoff properties) and well drained soils a rating of 1. Poorly drained, imperfectly drained and moderately drained soil types have been given values of 0.25, 0.5 and 0.75 respectively.

v. The fraction impervious within the study area have been given a rating based on the impact the imperviousness has on infiltration capacity and associated increase in runoff that would occur once soil saturation is reached. On this basis an area that is 100% impervious has a rating of 0 (nil change with saturation) and an area that is 0% impervious a rating value of 1. The rating for

impervious fractions between these two extremes is 1 – fraction impervious (e.g. an area that is 80% impervious would have a rating value of 0.2).

vi. Overall impact rating maps were then created by multiplication of the above two sensitivity ratings in the mapped extents of new locations where soil saturation may be expected to occur, for each of four scenarios.

3.1 Assessment of more frequent events (75-100 mm)

Attached Maps 1 and 2 show the soil type and impervious information respectively used in the assessment. Map 3 shows the new locations where soil saturation may be expected to occur for the 0.5 m depth to groundwater and 1 m SLR scenario in frequent events.

Map 4 provides the outcome of the 0.5 m depth to groundwater and 1 m SLR scenario assessment and demonstrates that the reduction in groundwater depth due to a 1 m SLR and associated increase in surface runoff will be localised particularly in the Brighton/South Shore area. Maps 5, 6 and 7 provide the remaining of results covering four scenarios of 0.25 and 0.5 m depth to groundwater and 0.4 m and 1 m SLR.

A visual inspection of the maps indicate that for the majority of the Avon River catchment, that locations where increased runoff in frequent events would be expected are modest and are located in areas which while likely to have local significance are unlikely to influence large scale flood risks.

The relationship between increased runoff and increased flood level is not simple. Essentially the area shown on Map 4 drains either to the lower Avon or the ocean, both of which are tidally dominated and neither are likely to show an increased flood level due to increased runoff only.

Comparison across Maps 4 and 5 show as expected that there is significantly more area affected by the larger 75-100 mm rain event (0.50 m depth to groundwater) than by a 38-50 mm rain event (0.25 m depth to groundwater). Close comparison shows however that there are also some areas affected by the smaller rain event that are unaffected by the larger rain event. The interpretation here is that where the initial (pre rain) groundwater level difference with SLR is less severe, that some areas would become saturated by the larger rain event regardless of the groundwater level change (thus there is a no change (white) result on the map. The smaller rain event though may only achieve saturation with the increased initial groundwater level hence producing a positive (black) result on the map. The main effect of the larger rain event is clearly more responsiveness of runoff to groundwater level rise, but there is a secondary effect of reduced responsiveness in some areas.

Comparison of Maps 4 and 6 show the sensitivity to different sea level rise conditions. As expected the impact of a 1 m SLR scenario on areas prone to increased runoff and saturation are much greater than those for 0.4 m SLR.

3.2 Assessment of less frequent events (150-200 mm)

The above work has illustrated a trend of ‘more saturation impact from bigger rain events’ which in turn has pointed to a gap in the analysis of 100 yr ARI and larger rain events. Accordingly an additional assessment was completed with a 1 m depth to groundwater, with 1 m SLR (Map 8). The 1 m depth to groundwater corresponds with 150-200 mm of rainfall depth and 100-200 yr ARI at the 24 hr storm duration.

These new results, compared to the closest comparison being Map 4, have the following points of interest;

1. There are significant additional areas impacted east of Porritt Park. Understanding that much of this area is pumped now with more to be pumped in future, there will be implications for pumping capacity and storage (or implications for flood risks if the pumping solution is considered a constant)

2. There is an additional area impacted east of Travis swamp (Beach Rd). This area has higher elevation and generally reasonable overland flow drainage which means we would expect less impact on flood levels once those flow paths (generally roads) are activated. However anywhere in the area that might be sensitive to overland flow is likely to see more flow more often.
3. The area north of Horseshoe Lake shows additional impacts although fairly minor generally in relation to the relatively large Horseshoe Lake catchment area and generally has good topography for drainage also
4. The area west of Hagley Park is also starting to show up a bit more – presumably this would get more important if we looked at even bigger rain events but remains of low interest in this analysis.
5. Southshore shows less sensitivity to the larger event – most of it is so shallow currently that it is saturated by a large event 'today' and those areas can't get any more saturated as a result of higher future groundwater levels.

These result areas collectively, and with particular emphasis on the area east of Porritt Park, indicate likely impacts on flood levels from increased runoff in the larger events with elevated groundwater. These additional areas are modest with respect to Christchurch as a whole, but may justify modelling to develop a further understanding, irrespective of whether current drainage systems remain or increasingly pumped drainage is developed in future.

3.3 Overall Assessment

In areas predominantly drained by pipe networks and pump stations behind stopbanks, then increased storage flooding upstream of these constraints may be expected. In areas predominantly drained by overland flow paths, particularly road surfaces, after surface depression storages have been filled then increased runoff flow is unlikely to materially increase flood levels for frequent events. For the larger ARI events such as 200 year, the latter condition predominates most of the area highlighted in this study, as discussed above.

This suggests that groundwater level changes are unlikely to produce material or widespread changes to flood levels for such events. The impact of groundwater levels on flood risk are most likely to be significant for the small ARI events (such as 10 year ARI) and for longer storm durations.

4 Potential Modelling Approaches

Two methods for modelling groundwater level change have been identified conceptually and are described below.

4.1 M21 Infiltration

A recent feature of the DHI M21 software is the ability to model a 'constant infiltration with capacity' method using M21 rain on mesh. This method would allow a reasonably fine (say 10m) grid to be established across the model domain, and in each grid cell a constant infiltration rate and capacity would be chosen at model setup. During runtime when the depth of infiltration reaches this capacity then infiltration stops and there is 100% runoff which is maintained for the remainder of the rainfall duration.

The existing soils map and knowledge of infiltration from previous model calibration will be used to set the infiltration rate. Knowledge of the depth to groundwater and effective porosity will be used to set the capacity. Using different initial groundwater levels (surfaces) the capacity will adjust and the model event response will change accordingly.

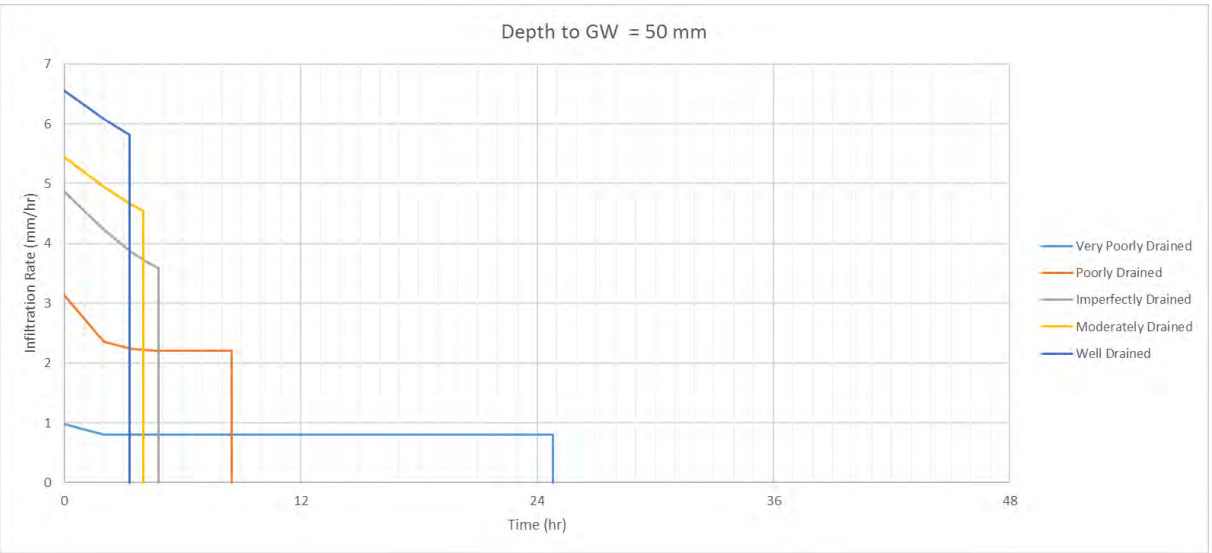
While on an individual grid cell the change to 100% runoff (soil saturation) will be sudden, any cluster of cells will be different so the timing to reach 100% runoff will vary and the model generated runoff from

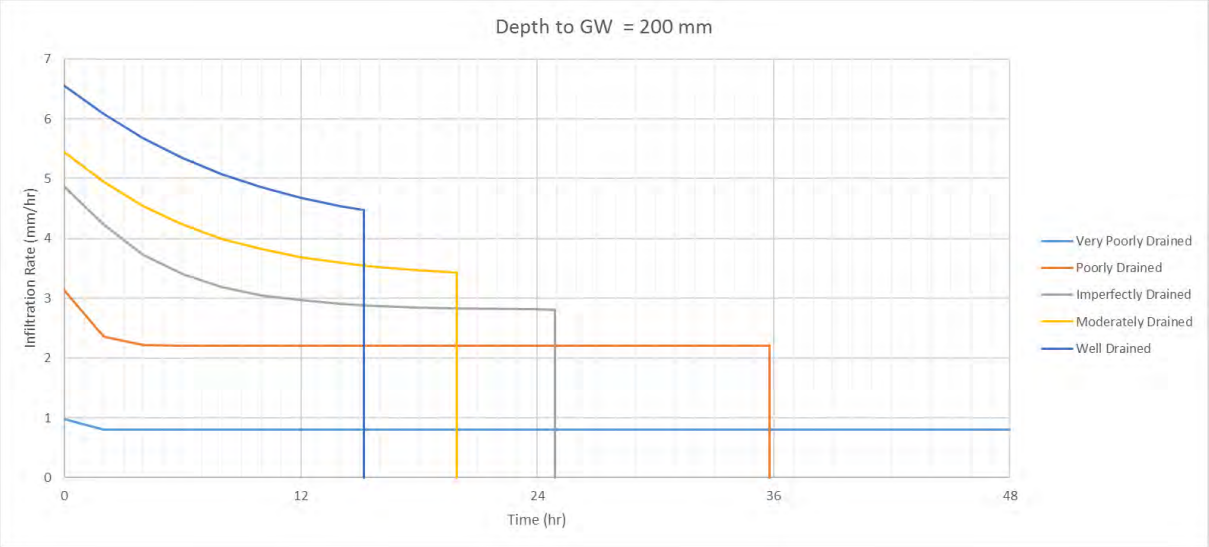
anything other than a micro catchment area will be smoothly varying and potentially quite similar to a Horton decay curve.

4.2 Truncated Hortons

A simplified modification to existing time varying infiltration curves is another approach. This approach ignores the rainfall intensity and would work best when rainfall intensity exceeds infiltration rates. The approach would involve calculating the depth of infiltration required to fill soil porosity (saturation) at every raster cell across the study area, then using the soil infiltration curve to determine the time at which this saturation could occur and then setting infiltration to zero after that time.

Examples of potential modified infiltration curves are provided below for two different depths to groundwater.





This approach neglects rainfall intensity and is only properly applicable for large high intensity rainfall which is always in excess of infiltration capacity. This limitation will produce increasingly biased results (too much runoff) as smaller rain events are modelled.

4.3 Method Comparison

Key advantages of the M21 infiltration method include

1. Responds appropriately during smaller low rainfall events – will not saturate prematurely and overestimate runoff as the truncated Hortons method would
2. Software supported method not reliant on complex bespoke GHD calculations and programming to produce model inputs. Model input data remains more closely related to the source groundwater level data and can readily be inspected and checked. Methodology much more transferable to other modellers.
3. Arguably more technically credible than truncated Hortons which is more of an abstract combination of macro catchment Horton's decay, and micro catchment saturation volume concepts

Key advantages of the truncated Hortons method include

1. Avoids the need for selection of a constant infiltration rate (the current initial / final rates can simply be preserved)
2. Probably has a lesser risk of disrupting model calibration outcomes although both methods have significant risks
3. Once the new input data is created its format is similar to the existing modelling approach and therefore there is 'less change' and lower risk of unforeseen consequences from change. Consultation with DHI has established that the M21 Infiltration method has been widely used globally and that they have no reason to anticipate any difficulties with its use. In terms of the complex iterative hydraulic computations (and stability risks) there is no material difference as both approaches are simple non-iterative methods to determine model input flows. The only expected risks to computational stability would be associated with any difference in the input flows, which should be nearly identical if both approaches are calibrated to the same outcome.

As of April 2019, GHD's recommendation that the M21 Infiltration method be the preferred approach, has been accepted by Council for the next phase of groundwater level modelling under LDRP097 baseline modelling.

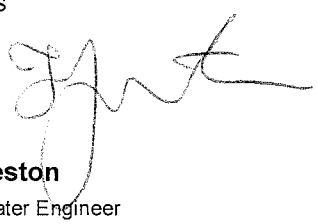
5 Conclusion

Initial work (Maps 4 – 7) have demonstrated sensitivity of soil saturation with differing sea level rise assumptions and differing rainfall event depths. As expected there are greater saturation extents with higher sea level rise scenarios, and also generally greater saturation extents with larger rainfall depths.

Based on the assessment of the largest considered sea level rise and rainfall depth evaluation (1 m SLR/ 1 m groundwater depth or 200 mm rainfall depth) scenario there are some notable areas where the higher groundwater levels associated with 1 m SLR would be expected to cause additional ground saturation and higher runoff in a large 200 mm rain event in areas where flood levels are likely to be sensitive to the additional runoff flow and volume.

Although the GIS approach used demonstrates the potential increase in area that could be fully saturated, it does not quantify the impact this may have on runoff and associated flood levels. The areas of predicted impact are modest with respect to Christchurch as a whole, but this study methodology is also simplified and has only assessed a small range of potentially interesting scenarios. Further evaluation using more sophisticated methods involving flood modelling would be of interest.

Regards



Tim Preston
Senior Water Engineer

Attachments:
Map 1 – Soil Drainage Map
Map 2 – Impervious Map
Map 3 – New locations where soil saturation may be expected to occur
Map 4 – 0.50 m groundwater, 1 m Sea Level Rise Impact Map
Map 5 – 0.25 m groundwater, 1 m Sea Level Rise Impact Map
Map 6 – 0.50 m groundwater, 0.4 m Sea Level Rise Impact Map
Map 7 – 0.25 m groundwater, 0.4 m Sea Level Rise Impact Map
Map 8 – 1.00 m groundwater, 1 m Sea Level Rise Impact Map

Paper Size A3

0 0.5 1 1.5 2 2.5

Kilometres

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

N

LEGEND

Soil Drainage Classification

- Very Poorly Drained
- Poorly Drained
- Imperfectly Drained
- Moderately Drained
- Well Drained
- Water
- Avon Catchment Boundary

Christchurch City Council
Study of Ground Water Level Changes on Flooding

Job Number	51-37994
Revision	A
Date	10 Aug 2018

Soil Drainage Map - Avon Catchment Map 1

\\ghdnet\ghd\NZ\Christchurch\Projects\51\37994\GIS\Maps\Working\Deliverable_Maps\51_37994_Map1_SoilDrainageMap.mxd

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Kilometres

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

N

LEGEND

Impervious Map
100% Impervious
0% Impervious

Avon Catchment Boundary

Christchurch City Council
Study of Ground Water Level Changes on Flooding

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Impervious Map

Map 2

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Kilometres

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

N

LEGEND

Depth to groundwater less than 0.5m

- Base Case (85th Percentile)
- With 1m Sea Level Rise
- Avon Catchment Boundary

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Study of Ground Water Level Changes on Flooding

New locations where soil saturation may be expected to occur

Job Number	51-37994
Revision	A
Date	10 Aug 2018

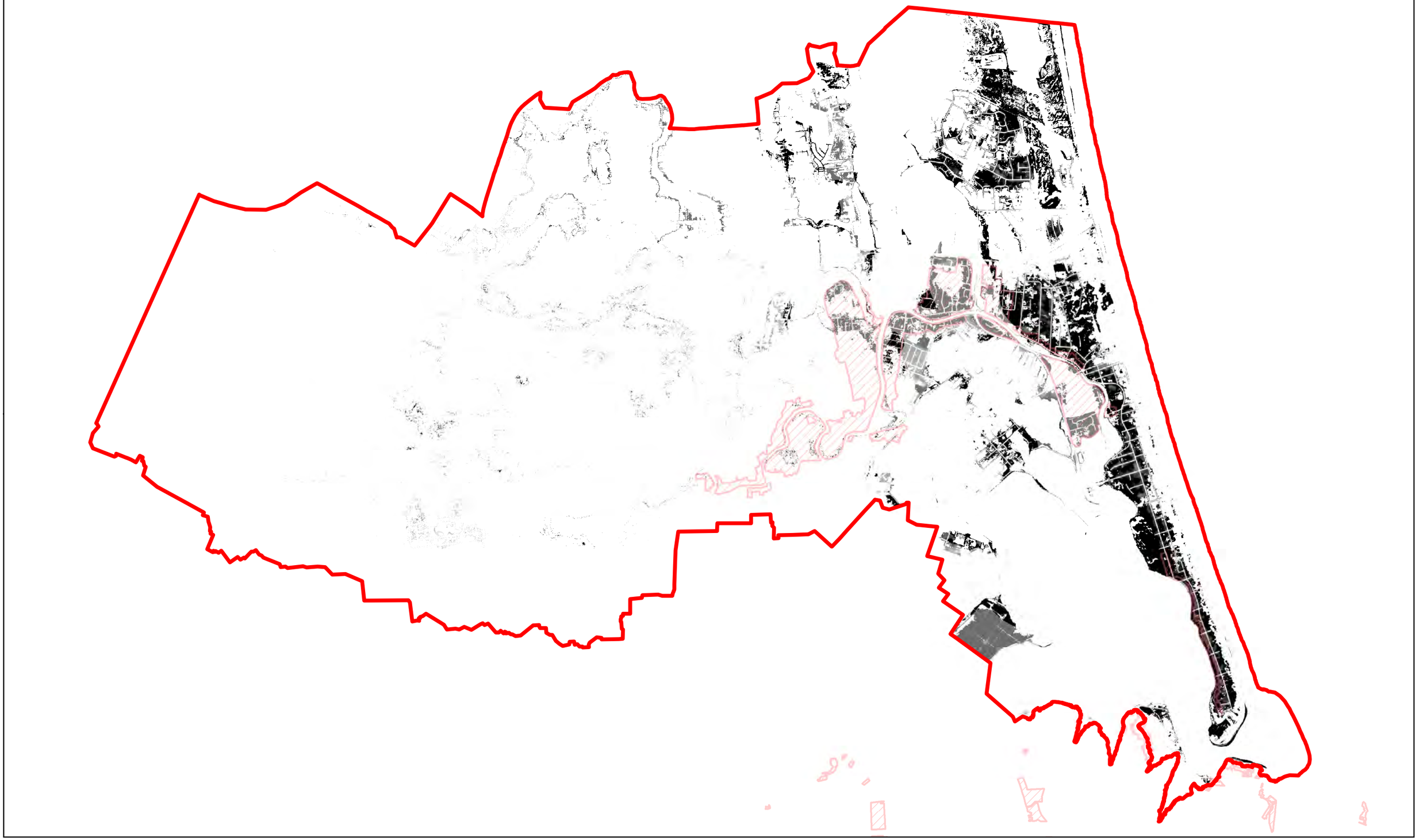
Map 3

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Kilometres

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

N

LEGEND

Sea Level Rise Impact

Value

High : 1

Low : 0

Avon Catchment Boundary

Redzone Land

Christchurch City Council

Study of Ground Water Level Changes on Flooding

Sea Level Rise Impact Map - Avon Catchment (1m SLR with depth to groundwater less than 0.5m)

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Revision B
Date 01 Feb 2019

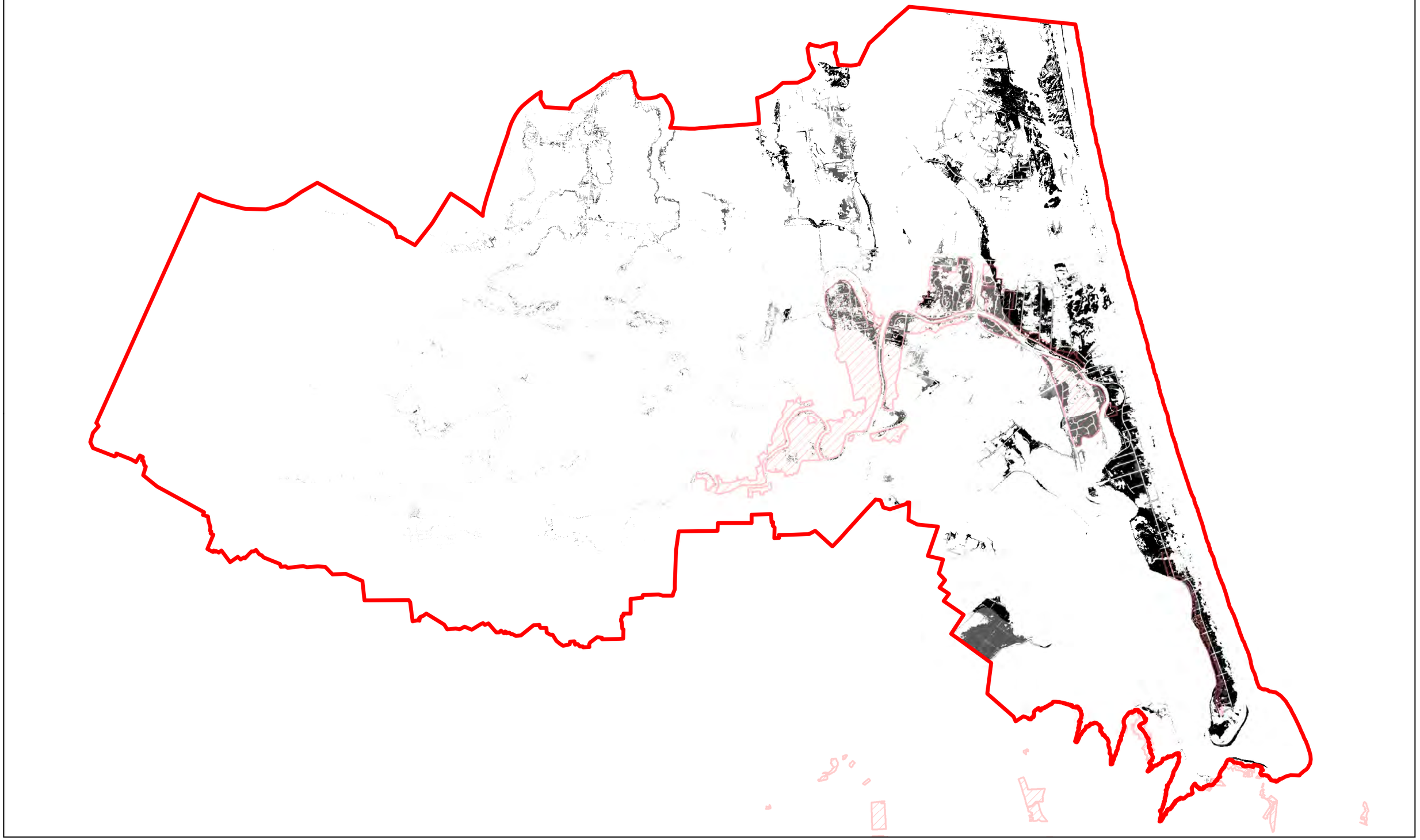
Map 4

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Kilometres

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

N

LEGEND

Sea Level Rise Impact

Value

High : 1

Low : 0

Avon Catchment Boundary

Redzone Land

Christchurch City Council

Study of Ground Water Level Changes on Flooding

Sea Level Rise Impact Map - Avon Catchment (1m SLR with depth to groundwater less than 0.25m)

Job Number	51-37994
Revision	A
Date	01 Feb 2019

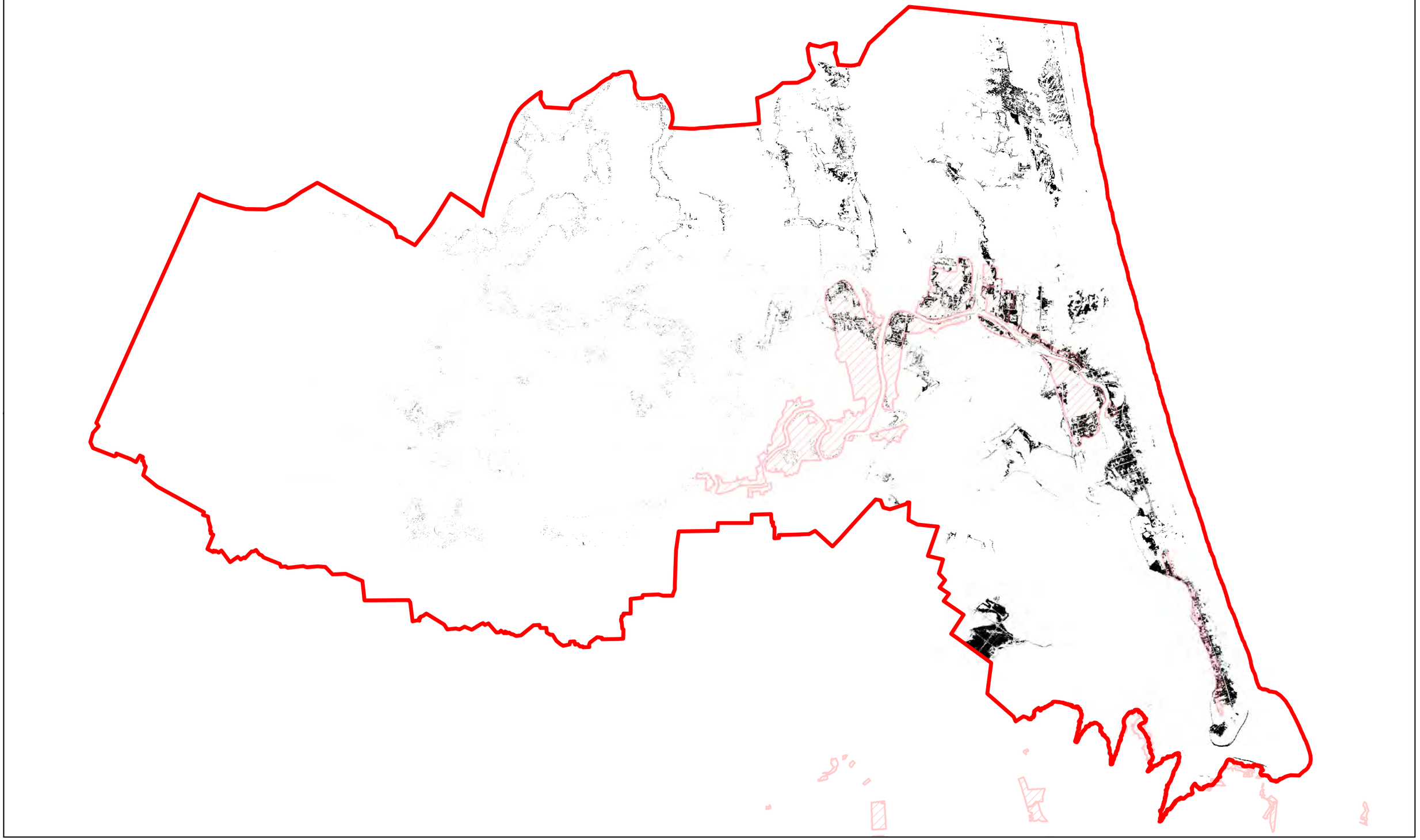
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Kilometres

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

N

LEGEND

Sea Level Rise Impact

Value

High : 1

Low : 0

Avon Catchment Boundary

Redzone Land

GHD Christchurch City Council

Christchurch City Council
Study of Ground Water Level Changes on Flooding

Sea Level Rise Impact Map - Avon Catchment (0.4m SLR with depth to groundwater less than 0.5m)

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Date	01 Feb 2019

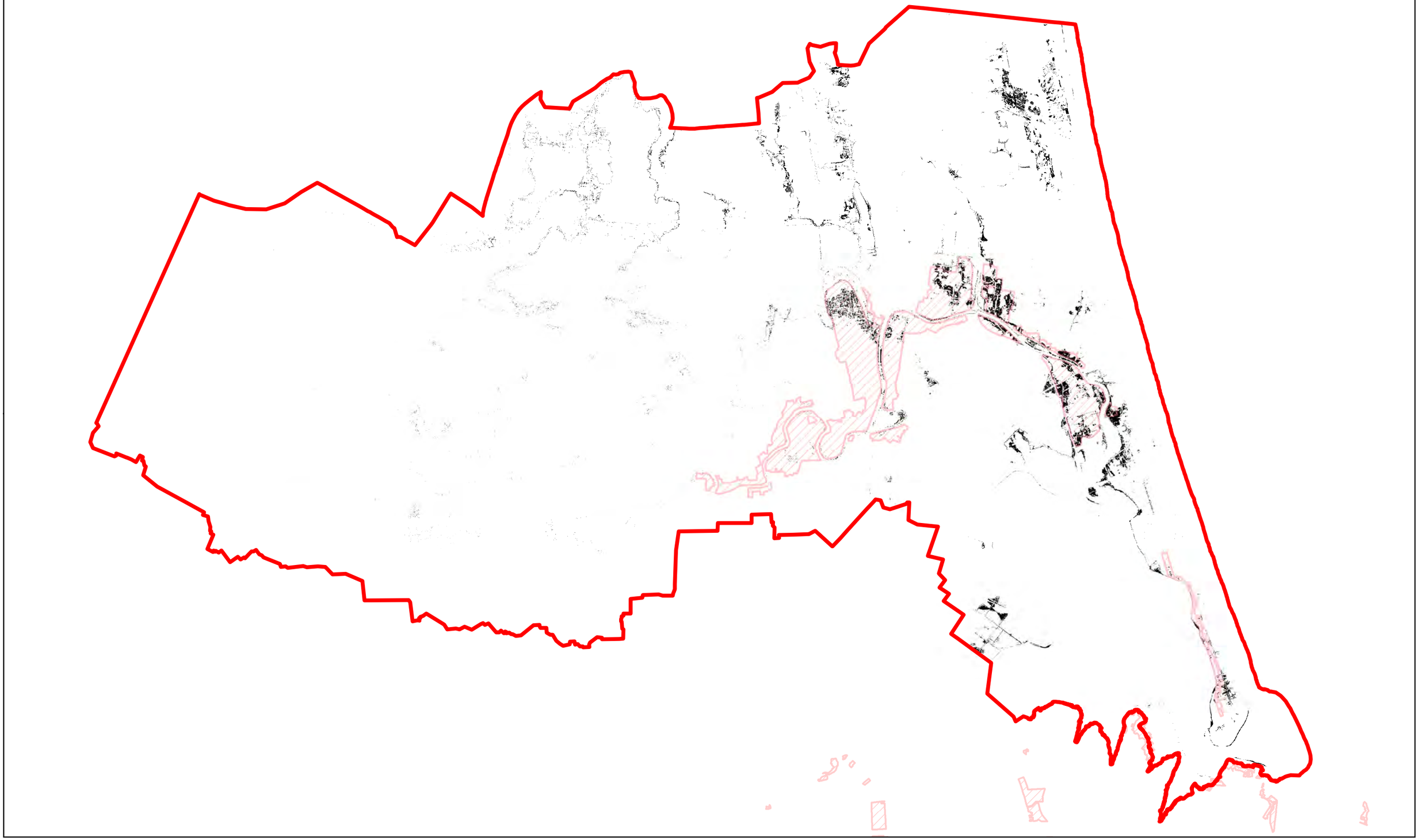
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Kilometres

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

N

LEGEND

Sea Level Rise Impact

Value

High : 1

Low : 0

Avon Catchment Boundary

Redzone Land

Christchurch City Council

Study of Ground Water Level Changes on Flooding

Sea Level Rise Impact Map - Avon Catchment (0.4m SLR with depth to groundwater less than 0.25m)

Map 7

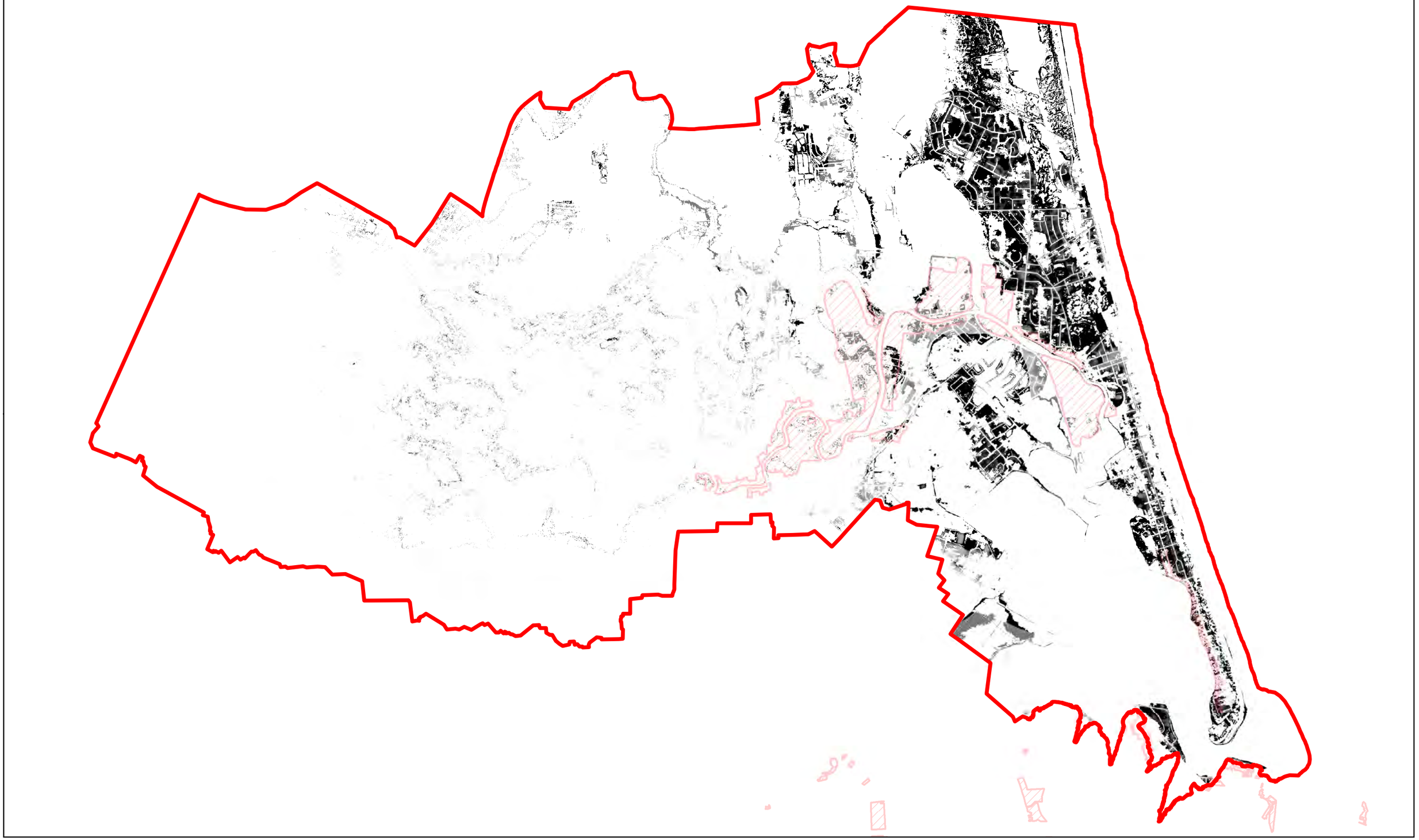
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Revision	A
Date	01 Feb 2019

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Kilometres

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

N

LEGEND

Sea Level Rise Impact

Value

High : 1

Low : 0

Avon Catchment Boundary

Redzone Land

Christchurch City Council

Study of Ground Water Level Changes on Flooding

Sea Level Rise Impact Map - Avon Catchment (1m SLR with depth to groundwater less than 1m)

Job Number	51-37994
Revision	A
Date	20 Feb 2019

Map 8

\\ghdnet\ghd\NZ\Christchurch\Projects\51\37994\GIS\Maps\Working\Deliverable_Maps\51_37994_Map4_SeaLevelRiseImpactMap.mxd

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Appendix N – Future earthquake settlement memo, T&T 2019[46]

“Future land surface model results” memo Tonkin & Taylor 8/4/2019

[46] GHD internal reference: "G:\51\12506449\00.Document Transfer\Incoming\20190409 MIKE Jacka T&T Liquefaction induced ground settlement To Tim\2019-04-08_ResultsMemo.pdf"

Tonkin+Taylor

Memo

To:	**LDRP multihazards team**	**Job No:**	**1000273**
From:	**Mike Jacka**	**Date:**	**8 April 2019**
Subject:	**Future land surface model results**		

We have completed our model for liquefaction-induced settlement resulting from a future earthquake event. I have attached summary maps and charts of the settlements predicted by the model, for various groundwater and earthquake scenarios.

The GIS data files can be downloaded from the download link provided with this email.

We have provided results for two earthquake scenarios, a magnitude 6 earthquake causing either 0.3g or 0.5g of ground acceleration. Our model is based on a uniform level of ground shaking across the entire city - this is unlikely to be the case in an actual earthquake event, where shaking will be stronger closer to the fault. However, this assumption appears to provide results which are reasonable for the LDRP multihazard flood modelling purposes, as once liquefaction is triggered the predicted settlements are not overly sensitive to input parameters (as discussed further below).

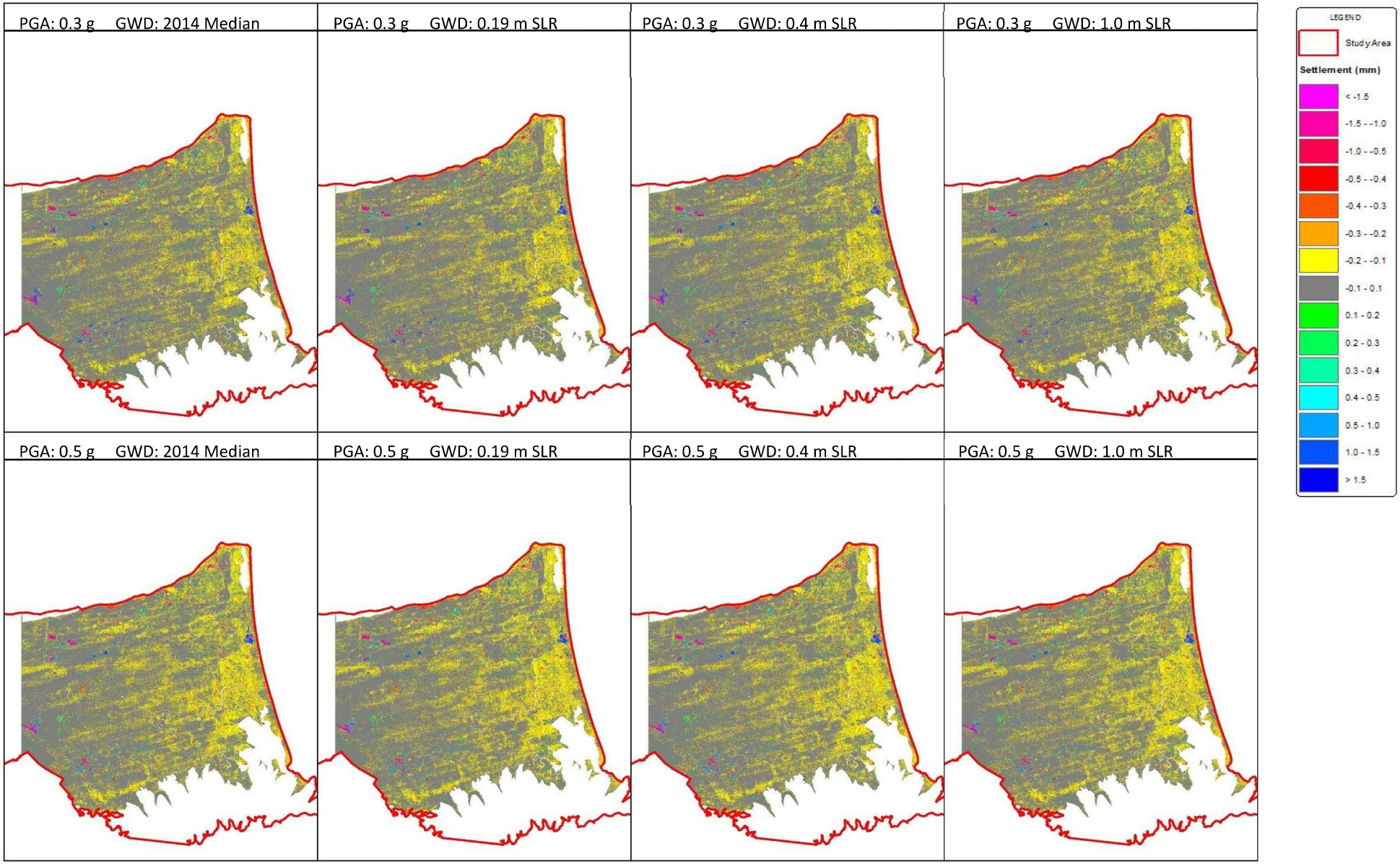
The modelled range of 0.3g to 0.5g (magnitude 6 equivalent) encompasses the strength of shaking that was felt across much of Christchurch City over the course of the various Canterbury Earthquakes. Areas in the northwest of the city (e.g. Burnside to Belfast) had maximum shaking levels of about 0.25g to 0.30g, while areas in the southeast (e.g. Woolston to South New Brighton) had maximum shaking levels of 0.50g or more.

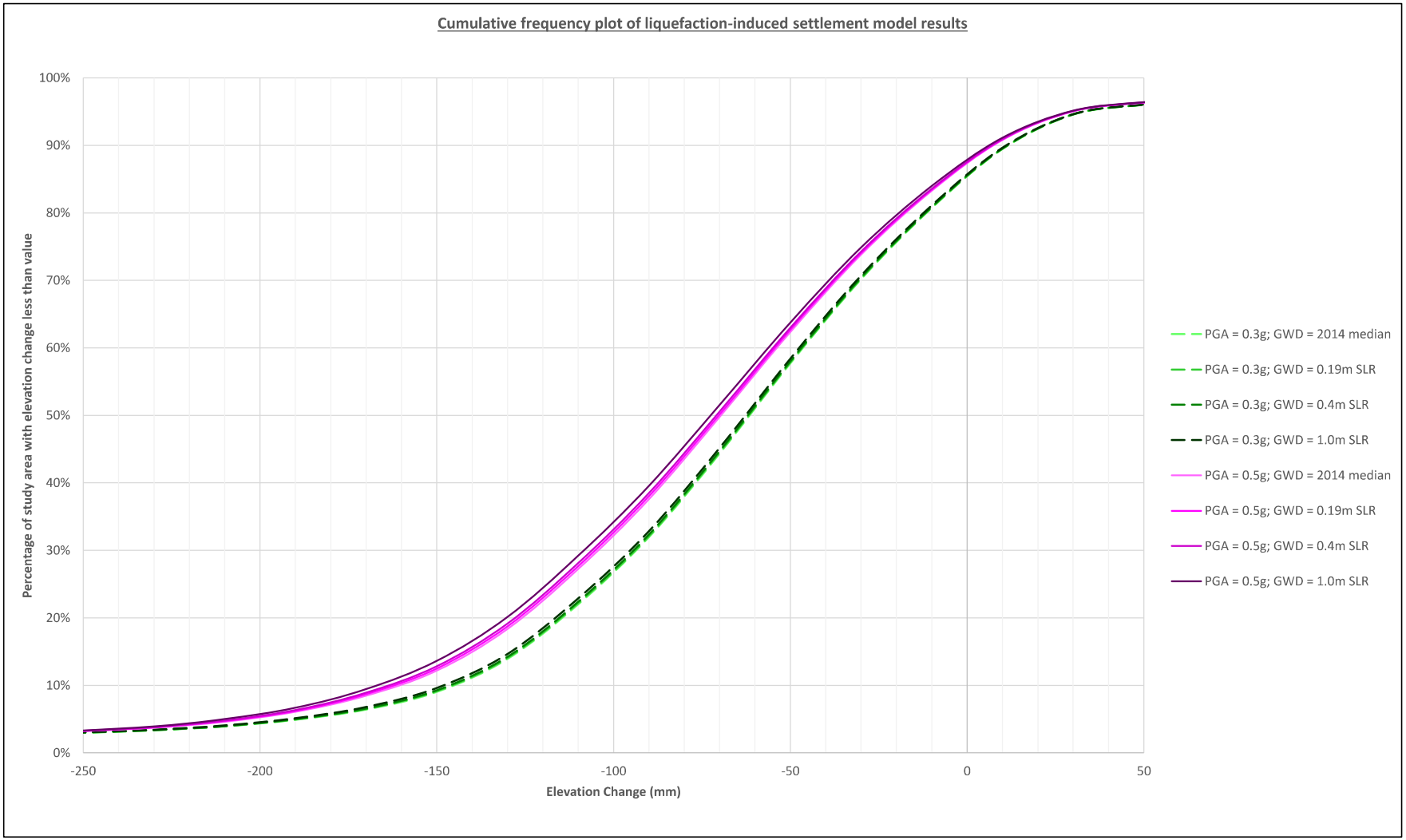
We have also provided results for four groundwater scenarios – our current-day median groundwater surface, plus groundwater effects from sea level rise of 0.19m, 0.4m and 1.0m. The change in groundwater levels across the city for these scenarios has been taken from the LDRP45 groundwater models prepared by Aqualinc (received on 13th March 2019).

As you can see from the attached summary, our settlement model is not particularly sensitive to the exact groundwater level and ground acceleration modelled. Therefore it would appear that for LDRP's flood modelling purposes, it would be appropriate to simply adopt just one of the scenarios provided (e.g. the scenario with PGA=0.5g and SLR=1.0m).

You will notice from the attached maps that various noise artefacts from the original lidar surveys are reflected in the model, such as the east-west flight path lines. Our settlement model included adjustments to some correct some of the more significant sources of error in the per-event LiDAR surveys, (e.g. large areas of subdivision earthworks or gross lidar measurement shifts) but this did not extend to smoothing out these flight path lines or localised extreme changes in elevation. You may wish to consider whether any smoothing of the model results would be appropriate before incorporating into your flood model.

Summary maps showing results of liquefaction-induced settlement model




Cumulative frequency plot of liquefaction-induced settlement model results
Percentage of study area with elevation change less than value
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
-250
-200
-150
-100
-50
0
50
Elevation Change (mm)
PGA = 0.3g; GWD = 2014 median
PGA = 0.3g; GWD = 0.19m SLR
PGA = 0.3g; GWD = 0.4m SLR
PGA = 0.3g; GWD = 1.0m SLR
PGA = 0.5g; GWD = 2014 median
PGA = 0.5g; GWD = 0.19m SLR
PGA = 0.5g; GWD = 0.4m SLR
PGA = 0.5g; GWD = 1.0m SLR

Appendix O – Future Impervious Report, 2024[47]

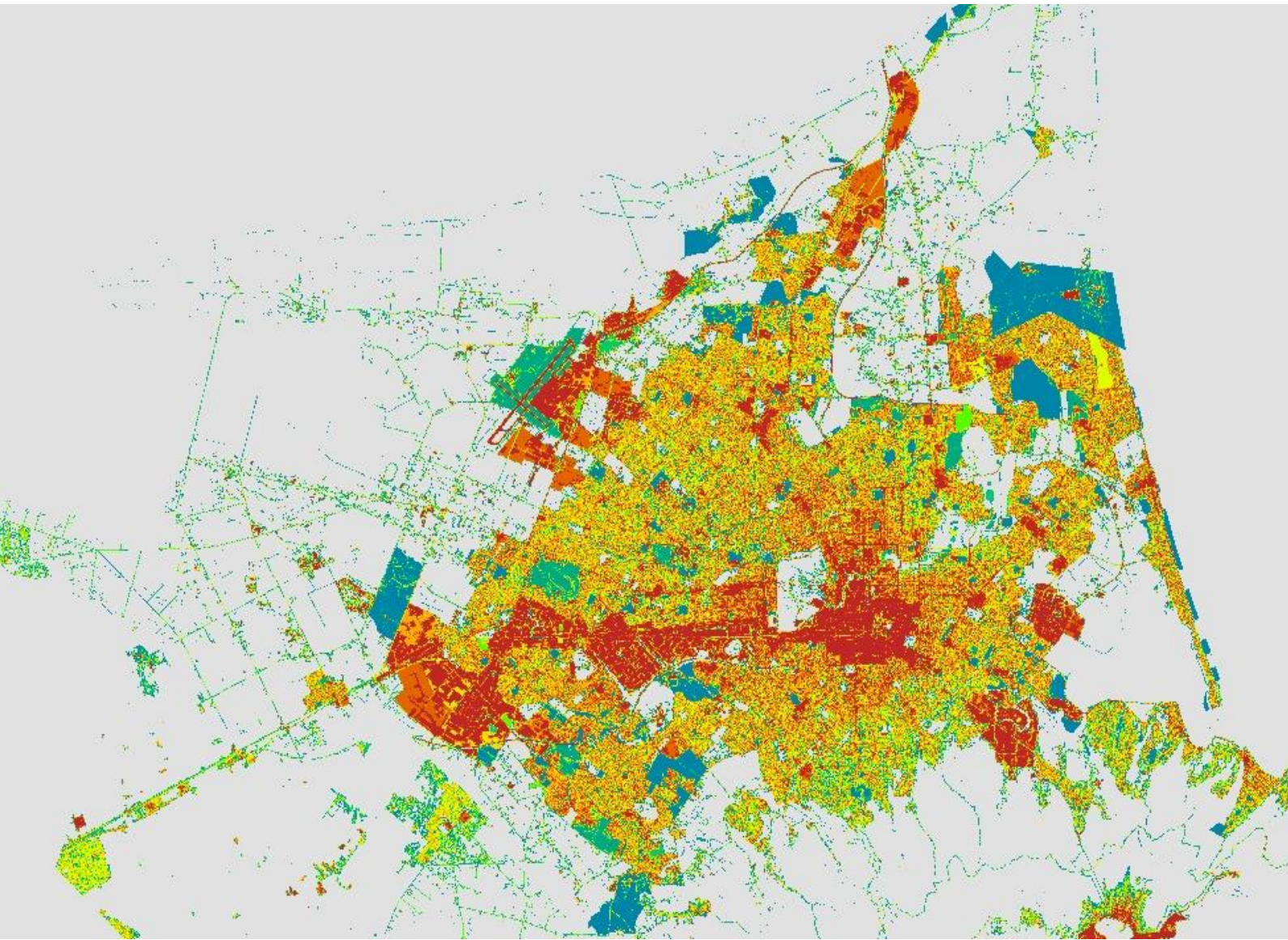
[47] GHD internal reference: \\projectsportal.ghd.com@SSL\DavWWWRoot\sites\pp02_10\cccsow192futureimper\ProjectDocs\20241030_FutureImperviousReport.pdf

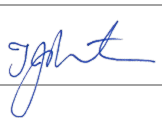
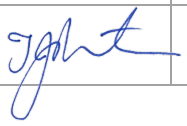
Future Impervious Report

Christchurch City Council

30 October 2024

→ **The Power of Commitment**



Project name		CCC SoW192 Future Impervious Modelling					
Document title		Future Impervious Report					
Project number		12651364					
File name		20241028_FutureImperviousReport.docx					
Status Code	Revision	Author	Reviewer		Approved for issue		
			Name	Signature	Name	Signature	Date
S4	0	Jenna, Clemence, Chamila, Tim	Tim Preston, Chamila Haegoda		Tim Preston		19/01/2022
S4	1	Madeline Howell	Tim Preston		Tim Preston		30/10/2024

GHD Limited 227
Contact: Madeline Howell, Water Resource Engineer | GHD
138 Victoria Street, Level 3
Christchurch Central, Canterbury 8013, New Zealand
T +64 3 378 0900 | **F** +64 3 377 8575 | **E** chcmail@ghd.com | **ghd.com**

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Contents

1. Introduction **1**
1.1 Context and Purpose of this Report 1
1.2 Scope and Limitations 1
2. Input Data **2**
3. Typical Percent Impervious and Household Density by Zone **4**
4. Meshblock Methodology **7**
4.1 General Meshblock Approach 7
4.2 Supporting Analyses 8
4.2.1 Park and Greenspace Extents 8
4.2.2 Greenspace Case Studies 9
4.2.3 Transportation Corridors 9
4.3 Summary Data for Meshblock Calculations 10
5. Future Residential Imperviousness **10**
5.1 Calculation Overview 10
5.2 Residential Brownfield/Greenfield Fraction 10
5.3 Predicted Brownfield Imperviousness (Equation 1) 11
5.3.1 Brownfield Percent Impervious Limits 11
5.4 Predicted Greenfield Residential Imperviousness (Equation 2) 12
5.4.1 Primary Greenfield Calculation 12
5.4.2 Alternative Greenfield Calculation 12
5.4.3 Lower Limit to Greenfield Calculation 13
5.5 Combined Greenfield/Brownfield calculations 13
5.5.1 Unmitigated and Mitigated Outputs 13
5.5.2 Unmitigated impervious surface (200-yr ARI) 13
5.5.3 Mitigated impervious surface (10-yr and 50-yr ARI) 13
6. Future Non-Residential Imperviousness **14**
6.1 Road Reserve 14
6.2 Commercial 14
6.3 Red Zones 14
7. Generating Impervious Rasters **14**
7.1 Generating Impervious Rasters from Meshblock Results 14
7.2 Incorporating Feasible Dwelling Capacity 15
8. Modelling Scenario Selection **15**
9. Digital Deliverables **16**
10. Summary **16**

Table index

Table 1 - District Plan household density and percent impervious assumptions 6
Table 2 – Greenfield impervious values based on household density 12

Figure index

Figure 1- Extents of impervious input data and analysed meshblocks 4
Figure 2 - Meshblock future impervious calculation overview 8

Attachments

Attachment 1: Schedule of Adopted Household Densities and Percentage Impervious for District Plan Zones

Attachment 2: Waterways, Wetlands and Drainage Guide (WWDG) Recommendations

Attachment 3: Impervious Raster Results Maps

1) 2023 Impervious Raster
2) 2103 Non-Mitigated Impervious Raster
3) 2103 Non-Mitigated Impervious Raster Considering Feasible Dwelling Capacity

Attachment 4: Meshblock Results Maps

1) 2103 Household Growth Map
2) 2023 Impervious Values
3) 2043 Non-Mitigated Impervious Values
4) 2073 Non-Mitigated Impervious Values
5) 2103 Non-Mitigated Impervious Values
6) 2103 Mitigated Impervious Values
7) 2103 Non-Mitigated vs. 2023 Impervious Values
8) 2103 Mitigated vs. 2023 Impervious Values
9) 2103 Non-Mitigated vs. 2103 Mitigated Impervious Values

1. Introduction

1.1 Context and Purpose of this Report

In 2021, GHD completed a future impervious analysis to develop impervious data corresponding with future development scenarios. This analysis was documented in the GHD's *Methodology for Future Horizons Imperviousness* report dated January 2022. This future impervious data was developed to assess the impact of future development within Christchurch's flood modelling catchments.

Since the 2021 work, the Christchurch City Council (CCC) has updated household growth projections. Additionally, new 2023 high-resolution impervious data is available across Christchurch. GHD were thus asked to update the future impervious analysis given the new household projections and current (2023) impervious data. The methodology used in the current analysis is largely the same as the 2021 analysis, with adaptations as required to adjust for changes in the nature of the input data, including a more detailed analysis of red zone development.

The purpose of this report is to outline the methodology and results of the future impervious analysis, which was completed to generate impervious rasters for Christchurch's flood modelling catchments based on future times of 2043, 2073, and 2103[1]. These future times were analysed per direction of CCC. This report is an update of the *Methodology for Future Horizons Imperviousness* dated January 2022 and describes:

1. The input data used in the future impervious analysis
2. The future impervious analysis methodology carried out at the meshblock resolution
3. The methodology to map the meshblock future impervious results into a finer (2 m) raster data.

1.2 Scope and Limitations

GHD was retained by CCC to assemble comprehensive current (2023) impervious data and generate future impervious rasters for 2043, 2073, and 2103, using methodology from GHD's *Methodology for Future Horizons Imperviousness* report dated January 2022, with adaptations as required to adjust for changes in the nature of the input data.

This report has been prepared by GHD for Christchurch City Council and may only be used and relied on by Christchurch City Council for the purpose agreed between GHD and Christchurch City Council as set out in the current section of this report, as well as the "SoW192 Future Impervious Conditions Modelling Support" Statement of Work dated 13/8/2024.

GHD otherwise disclaims responsibility to any person other than Christchurch City Council arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report. GHD disclaims liability arising from any of the assumptions being incorrect.

GHD has prepared this report on the basis of information provided by Christchurch City Council and others who provided information to GHD (including Government authorities and other consultants), which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information.

[1] Dates were instructed by CCC. 2103 was the most distant growth forecast available, 2073 serves the building code 50-yr future interests, and 2043 was considered an interested intermediate date.

Accessibility of documents

If this report is required to be accessible in any other format, this can be provided by GHD upon request and at an additional cost if necessary.

2. Input Data

A variety of information from various sources has been used in this analysis. This input data is summarised below:

- Household Projections: Household (HH) projections used in the analysis were prepared by the CCC Monitoring and Research Team on June 24, 2024. This data includes meshblock based household count projections throughout Christchurch, for various years ranging from 2018 to 2103. Based on discussions with CCC, the future impervious analysis was completed for the years of 2043, 2073, and 2103. Note that growth outside of these data extents (i.e.: outside the CCC TLA) was agreed with CCC to be evaluated as zero.
- Feasible Dwelling Capacity[2] (FDC): CCC have also analysed economically feasible dwelling redevelopment capacity in the city. This work identified existing parcels which have potential to be redeveloped (increasing household counts) profitably given today's prices and economic circumstances. The main difference between the analysis completed by a Senior Policy Planner in the City Planning Team and the household projections defined above, is that the FDC analysis only considers potential capacity, it does not consider demand (e.g. growth), which would be needed for these areas to be redeveloped, nor the willingness of landowners to sell. Accordingly, the FDC analysis is not associated with any specific timeline, other than today. For the purposes of this study, GHD asked the City Planning Team to increase the profit margin in the analysis, which reduced the number of economically feasible parcels, resulting in new HH numbers. The results of this revised analysis were provided to GHD on September 11, 2024, with 17,264 parcels supporting a net increase of 97,029 new households. Upon consultant with the CCC, the FDC data was incorporated into the future impervious raster for 2103, as discussed in Section 7.2.
- 2018 Meshblocks: 2018 meshblock boundaries used in the analysis were downloaded from the Statistics New Zealand (Stats NZ) website. This meshblock data was clipped to only include meshblocks that are within the study area, defined as the current citywide catchment areas plus a 1 km buffer. Note that any meshblock partially within the catchment area was included in the analysis.[3]
- 2023 Impervious Data: The 2023 impervious data used in the analysis was based on the sources listed below, in order of importance. Figure 1 displays the extents of the meshblocks included in the analysis as well the impervious input data. The 2023 impervious was clipped to reduce the GIS processing times.[4]
    1. 2023 Lynker/DHI Raster: A raster containing 2023 impervious data for Christchurch was received from DHI on August 4, 2024. DHI created this raster based on polygon data created by Lynker Analytics in April 2024. This data consisted of polygons that classified the land cover using the following eight categories: water, bare earth/gravel/sand, grass, scrub/shrub, high vegetation, road, building, and other impervious. The land cover classification was completed by Lynker Analytics using Christchurch 0.075 m aerial imagery from 2023.[5] DHI created a 2 m raster from this dataset by defining areas with water, grass, scrub/shrub, and high vegetation as pervious.

[2] Please refer to Peter Christensen's email to Tim Preston entitled "PC14 IHP recommendations - filtered feasible capacity outputs" and "Feasibility figures" dated 3/9/2024 and 11/9/2024, respectively. "N:\NZ\Christchurch\Projects\51\12651364\Document Transfer\Incoming\20240903 PC14 IHP Recommendations Peter to Tim\ReadMe.txt" and "N:\NZ\Christchurch\Projects\51\12651364\Document Transfer\Incoming\20240911 Revised PC14 Scallon growth data PeterC to TimP\Readme.txt"

[3] Note the 2023 (and future impervious) raster was subsequently clipped to reduce processing times, so that twelve of the meshblocks (six mainland and six oceanic/inlet/water) were not fully covered by the raster

[4] The clipped extents include the eastern extremity at 1,582,000 (just beyond Sumner) and the southern extremity at 5,151,000 (near the middle of Lake Ellesmere—to contain Kaituna Valley Catchment)

[5] Please refer to Lynker Analytics' *Data Quality and Accuracy Report: Christchurch City Council Machine Learning Surface* dated April 29, 2024 for more information: N:\NZ\Christchurch\Projects\51\12651364\Document Transfer\Incoming\20240509 Lynker impervious Jarad to Tim\Christchurch Council Impervious Land Cover Data Quality and Accuracy Report_V2.docx

Areas with bare earth/gravel/sand land cover were defined as pervious or impervious by DHI based on the CCC District Plan Zone.[6] All other areas were defined by DHI as impervious.

2. Halswell Impervious Data: The 2023 Lynker impervious data is limited to the Christchurch territorial area, while the Heathcote and Halswell catchment boundaries extend south and west of the city. Thus, additional impervious data was needed to supplement the 2023 impervious data set. A 2 m impervious raster extension for the Halswell catchment was developed by Beca in September 2024 based on LINZ building, CCC building footprints, LINZ NZ roads, and LINZ NZ railway centrelines.[7]
3. 2020 CCC Impervious Data: 2020 impervious data, which was derived by CCC GIS from 2020 imaging data, was used to supplement both the Lynker and Halswell impervious data.[8]
4. Pervious Assumption: Any area outside of the 2023 Lynker data, Halswell impervious data, and 2020 impervious data was assumed to be entirely pervious. These areas are primarily rural and are expected to be largely pervious.

- Roads: Road reserve extents were defined CCC GIS street catchment polygons downloaded by GHD on September 4, 2024.
- District Plan: The current CCC District Plan (DP) was downloaded from the CCC Open Data Portal on July 26, 2024. Note that the District Plan contains zones with both the operative and notified legal statuses. Based on discussions with CCC, only the operative DP Zones were used in the analysis.[9] Planning zones for the Selwyn DP were downloaded on July 26, 2024 from Canterbury Map Open Data Portal. The CCC and Selwyn DP zones were used together with various guidance and assumptions on associated developed impervious values and household densities, as discussed in Section 0.[10][11]
- Red Zones: Red zones within the analysis were defined as CCC DP Zones "Specific Purpose (Flat Land Recovery)" or "Specific Purpose (Otakaro Avon River Corridor)".
- Roads and Buildings in OARC Red Zone: Within the Otakaro Avon River Corridor (OARC), shapefiles were provided by CCC on July 26, 2024, which define existing roads that will remain in service, existing roads to be removed, future roads, existing buildings, and future buildings. These layers were used to define the future imperviousness within the OARC, as discussed in Section 6.3.

[6] Please refer to Antionette's emails to Tim Preston, Jarad Sinni and Peter Christensen (cc Susan Willis and Madeline Howell) "Future Impervious Analysis" 29/7/2024 and 5/8/2024. Impervious raster and csv with the gravel classification are saved here: N:\NZ\Christchurch\Projects\51\12651364\Document Transfer\Incoming\20240805 Infiltration Layers Antionette to Tim

[7] Please refer to Danika Ferguson's email to Tim Preston (cc Madeline Howell, Susan Willis, Stephen Holder, Jarad Sinni, Peter Christensen, and Elliot Tuck) "Halswell impervious – request" 13/8/2024 for detailed discussion regarding the methodology used to create this dataset.

[8] This dataset is the raw impervious data from CCC satellite data. It does not include any modifications to the dataset that were completed prior to previous modelling, for example editing the impervious values within the road reserves.

[9] Please refer to Peter Christen's email to Tim Preston (cc Madeline Howell, Susan Willis, Gemma Pullan, and Jarad Sinni) "Future Impervious – operative and notified D.P. inquiry?" 18/9/2024

[10] Note that the CCC District Plan contains some internal overlapping zones. A similar issue was noted in the Selwyn District Plan. These overlapping areas were either in areas where no growth was expected (e.g. Redzones or outside of extents of HH projections) or generally <300 m^2 and deemed insignificant. Thus, these areas were not cleaned up in the analysis.

[11] The CCC District Plan and Selwyn District Plan contain some data gaps. These data gaps were generally small and not included in the analysis. However, a significant (17,800 m^2) gap was noted within Meshblock 4000477. The calculations were corrected to include this area, which was assumed to be Residential Medium Density based on adjacent areas.

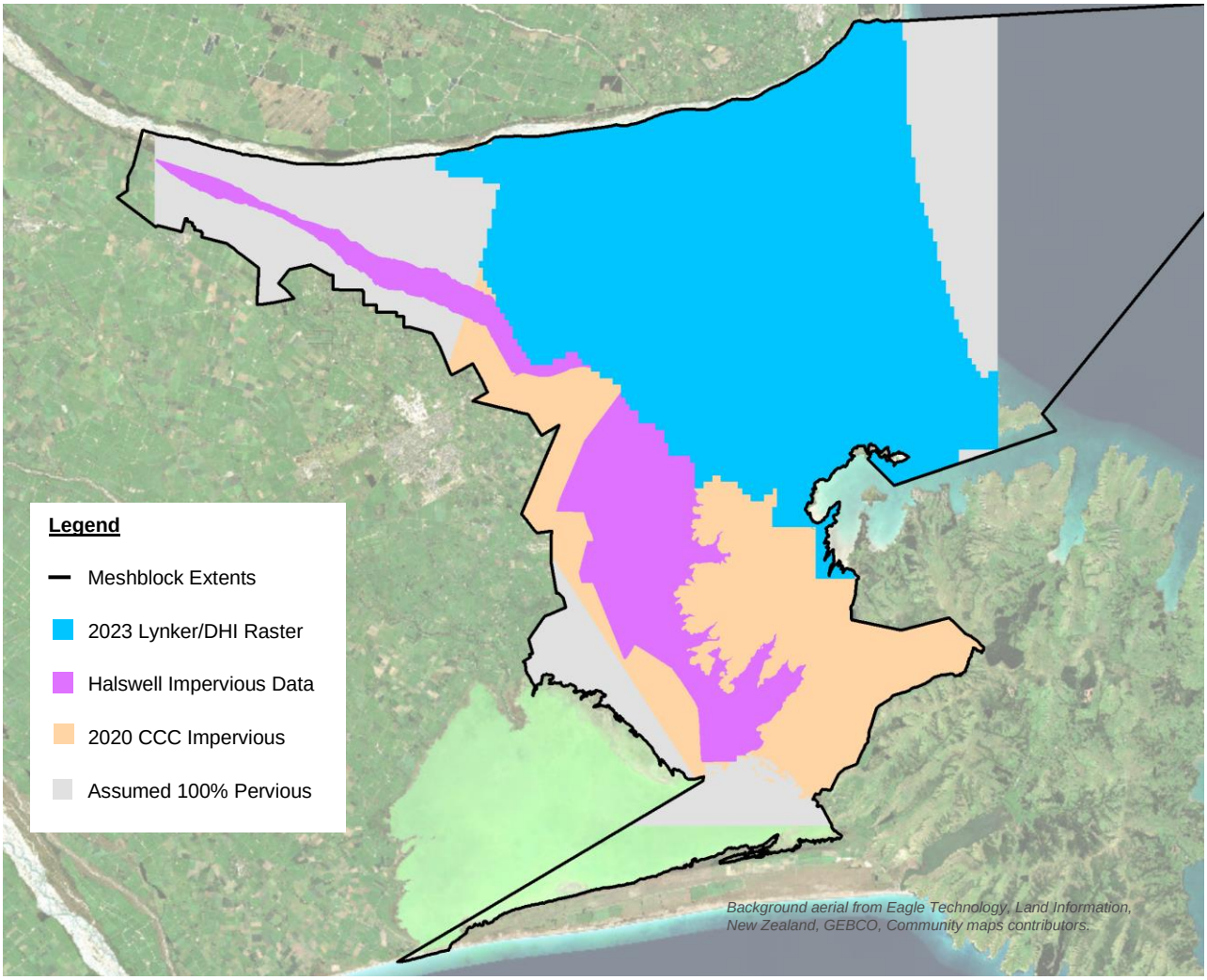


Figure 1- Extents of impervious input data and analysed meshblocks

3. Typical Percent Impervious and Household Density by Zone

Typical developed impervious values and household densities, consistent with District Plan (DP) zonings, are key components of the future calculations. The 2023 percent impervious values and household counts were examined in five groups of residential zones to determine appropriate fully developed impervious values and household densities. The grouped residential zones included the following:

- Central City (CBD), which included Commercial Central Mixed Use, Commercial Central South Frame, and Residential Central City zones.
- Residential Medium Density (RMD), which included Residential Visitor Accommodation zone.
- Residential New Neighbourhood (RNN)
- Residential Suburban Density Transition (RSDT)
- Residential Suburban (RS), which included Residential Hills, Residential Banks Peninsula, Residential Small Settlement, and Residential Large Lot.

The spatial approach used in this analysis follows the current District planning approach which applies zonings to land parcels and treats roading land separately.

The spatial distribution of existing percent impervious values for each zone group was determined from the 2023 Lynker/DHI impervious raster[12].

The spatial distribution of household densities was also calculated for each zone group. These household densities were calculated using two methods described below:

1. Determining the number of parcels dividing by the total parcel area within each zone group.
2. Taking the 2023 household count (from the CCC HH projections spreadsheet) and dividing this household count by the total parcel area within each spatial area.

The maximum household density from the two calculation methods above was used to analyse the District Plan household density, as method 1 allows for consideration of parcels that have yet to be developed and method 2 considers that there may be multiple households within one parcel.

The spatial distribution data was studied carefully in collaboration with CCC staff to exclude atypical values, to identify the reasonable range of values and to pick a typical central value from within the range. Ranges were typically near normal distribution with low skew; therefore, average, median and mode were within margin for error. The average values are noted below.

The resulting typical household densities and percent impervious values for each of these residential zones is summarized in Table 1 along with a brief justification.[13] These values were selected in consultation with CCC. Note that impervious and household density filters were applied when analysing the 2023 impervious values and household densities for the different zone groups to omit outliers, including zones residential areas that are not fully developed.

The assumptions outlined in Table 1 were used to assign fully developed impervious values and household densities for residential, mixed-use zones, and rural zones[14] within the CCC and Selwyn District Plans. Attachment 1 includes a list of all zones used in the analysis along with the assumed impervious values and household densities. Note that the values in Table 1 differ than the impervious values and household densities utilized in GHD's 2022 analysis, which were primarily based on the CCC's Waterways, Wetlands and Drainage Guide (WWDG). GHD recommends updating the impervious values within the WWDG, given the results of the impervious analysis outlined in Table 1. Please refer to Attachment 2 for GHD's recommendations.

Fully developed impervious value calculations also considered commercial, industrial, open space, and specific purpose zones, as noted in Attachment 1. The percentage impervious values for these zones were determined using the CCC's percentage impervious and HH density spreadsheet[15] developed by Brian Norton and received on 18/07/2019 as well as GHD's engineering judgement (in consultation with CCC) where no other data source was available.

[12] To analyse the 2023 impervious values, the study area needed to be divided into moderately sized polygons to aggregate the high-resolution impervious data. We used the 2018 meshblock polygons for this division, but the use of meshblocks here is not critical, and similar results would have been achieved with a 500x500 m polygon grid, for example.
[13] A more detailed version of this table is saved here: N:\NZ\Christchurch\Projects\51\12651364\Tech\20240927 Variation GIS Prework_MH\20241008_Prework_Conclusions.xlsx
[14] Not including Rural Quarry zones, which were assumed to have household densities of 0.
[15] *2016 Ch21_Runoff_Coefts.xlsm*

Table 1 - District Plan household density and percent impervious assumptions

	Household Density (HH/ha)[16]	Percent Impervious	Justification
Central City (CBD)	60	80%	Data with household densities < 25 HH/ha was excluded from the analysis, which yielded an average household density of 57 HH/ha (rounded to 60) and percent impervious value of 80%.[1]
Residential Medium Density (RMD)	30	75%	The data was filtered to exclude areas with impervious values <55% and > 88% to omit outliers. This filter led to a weighted average household density of 30 HH/ha and percent impervious value of 69%. Several developed neighbourhoods[2] featured a weighted average impervious value of 70%. An impervious value of 75% was selected as a conservative value to account for future development.[3]
Residential New Neighbourhood (RNN)	20	70%	Several of the RNN-zoned areas have yet to be developed. Inspection of two representative neighbourhoods[4] indicated that impervious values of <60% typically involved undeveloped lots. Thus, a 60% impervious filter was applied to the data, which yielded a weighted average household density of 19 HH/ha (rounded to 20) and impervious value of 70%.
Residential Suburban Density Transition (RSDT)	22.5	65%	A weighted average household density of 21 HH/ha and impervious value of 63% was noted (no filters applied). These values are very similar to the assumptions used in GHD's 2021-2022 analysis (22.5 HH/ha and 65% impervious), so the 2021-2022 assumptions were not changed.
Residential Suburban (RS)	15	60%	Data was filtered to remove areas with <40% impervious values (not fully developed) and > 80% impervious values (outliers). This filter considered 82% of the total area and led to a weighted average household density of 15 HH/ha and impervious value of 58% (rounded to 60%).

Notes:

1. *Note that only 13% of the Residential Central City zone exhibited an impervious value > 80%, so 80% is a conservative assumption.*
2. *Neighbourhoods include Addington, Sydenham, Phillipstown, Linwood, Merivale, Edgeware, St Albans, and Richmond.*
3. *Note also that this is similar to the weighted average impervious value of Riccarton (76%).*
4. *Representative neighbourhoods included Wigram and Prestons.*

[16] Note this density is per hectare of land ownership parcels (excludes transport/road reserve areas) and the percent impervious is similarly defined within the land ownership parcels.

4. Meshblock Methodology

4.1 General Meshblock Approach

The future impervious calculations are based on 2018 meshblocks. Within the meshblocks, subareas are classified sequentially, with road reserve defined first, then redzone and then commercial. All other areas are nominally defined as 'residential', noting that this includes an amalgamation of different zones, most and notably green space parks and reserves as well as significant wet areas such as the estuary. Please refer to Section 4.2 for justification of this approach.

The spatial definition of the road reserve is based on CCC street catchment data as discussed in Section 2. Commercial zones were defined following the same process as GHD's 2021-2022 analysis:

1. All areas defined as commercial or industrial within the CCC current District Plan were included.
2. All areas defined as "Specific Purpose (Hospital)" within the CCC current District Plan were included.
3. GHD custom defined certain special areas (within zones "Specific Purpose (Styx Mill Road Transfer)", "Specific Purpose (Airport)", and "Specific Purpose (Lyttleton Port)") as commercial, where that seemed appropriate given current land use / impervious patterns in those zones. Assigning commercial to those entire zones was considered to be an unrealistic overestimate for these zones.
4. All areas defined as "Business" within Selwyn District Plan were included, except "Business Type 3", which is Lincoln University.
5. Additional commercial areas not included in the CCC current District Plan were included as agreed with CCC.[17]

Once the meshblocks were split into the four subareas, the future impervious value for each subarea was calculated separately. These calculations are described in further detail in Sections 0 and 6, with the overall process summarized in Figure 2. A combined weighted average result for the meshblock is calculated but used only for the purposes of visual QA checks on the meshblock polygons. The generation of the final model input data (raster format) is described in Section 7.

[17] Refer to Tyler McMillan's email entitled "RE: LDRP97 Multihazard baseline modelling - MPD commercial?" and dated 27/07/2020, which defines these areas. Also refer to Peter Christensen's email entitled "FW: LDRP97 Multihazard baseline modelling - MPD commercial?" and dated 16/7/2024, which confirms that these commercial areas should still be included in the analysis.

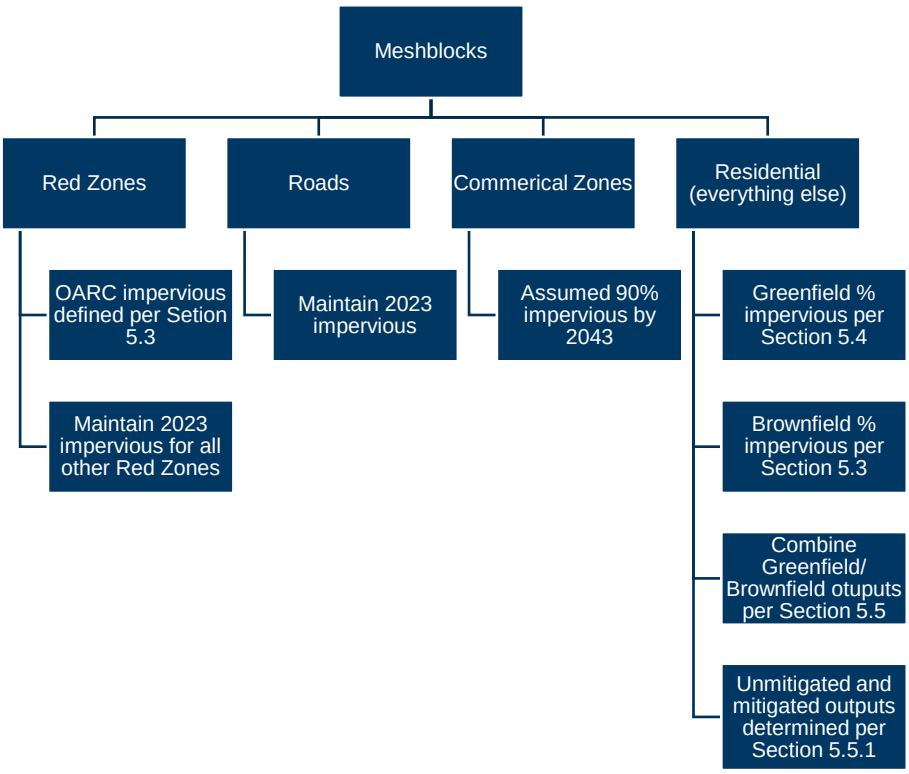


Figure 2 - Meshblock future impervious calculation overview

4.2 Supporting Analyses

In the previous work, greenspace was not treated separately for the following reasons:

- To minimise unnecessary complexity.
- It was recognised by GHD as being minor within any larger context of City or suburb.
- Some greenspace is not amenable to spatial definition (e.g.: school fields and drainage reserves could be treated as greenspace but are not zoned greenspace).
- It is not impossible for development to occur in greenspace in the most general future view.

Within the scope of this work (without delaying the core progress) CCC asked GHD to take a closer look at justifying this approach. Analyses included the following:

1. Analysing the spatial extents of parks and greenspace.
2. Analysing the average impervious values and spatial extents of transportation corridors.
3. Case studies of two meshblocks with greenspace and residential areas to determine the difference in resulting impervious value when the residential and greenspace zones are separated.

The results of the analyses suggest that including greenspace in residential zones and disregarding future roads are expected have minimal impact on the future impervious results. These analyses are discussed in more detail in the following sections.

Notwithstanding the above considerations, GHD recommends that a simple zoning-based definition of greenspace be used to create a fifth subarea type and thus keep it apart from the “residential and other” in future revisions. Any secondary greenspace areas such as school fields and drainage reserve not captured in the simple definition of greenspace would continue to be included in ‘residential and other’.

4.2.1 Park and Greenspace Extents

The spatial extents of the parks and greenspace was analysed. Large parks (with areas > 50,000 m^2) were excluded from the analysis as they do not feature in typical urban growth areas which was our focus. They are

often also an entire meshblock anyway, and typically have no growth forecast in them. Analysis of the “open space” (excluding > 50,000 m^2), “residential”, and “mixed use” District Plan zones indicated that the “open space” land use accounted for 6% of the total area, while the residential areas (residential and mixed-use zones) account for the remaining 94% of the total. Additionally, discussions with Brian Norton of the CCC indicated that a typical park size within a 20-hectare greenfield development would be approximately 0.3 hectares (3,000 m^2), which is approximately 1.5% of the developed area. However, there are additional greenspaces within Greenfields developments (such as drainage reserves) and sometimes regional parks, which may increase typical open space area to 0.6 to 1.0 hectares (i.e. 3-5%). Since parks and greenspace represent a small fraction of the residential area, treating these areas in aggregate with residential in the future impervious calculations is expected to have limited impacts on the results.

4.2.2 Greenspace Case Studies

Hand calculations for two meshblocks containing areas of Burnside Park were completed to examine the difference between amalgamating parks and residential areas vs. separating these zones. These hand calculations were completed following the calculation approach outlined in Section 0 and assuming a 50% growth in household count.

The first examined meshblock was 2535400, which contains Burnside Bowling Club. The residential zoning accounts for approximately 67% of the total land parcel area (not including streets), while the bowling club and parks greenspace accounts for the remaining 33%. To make up a more typical case study the starting impervious value was assumed to be 10% for the park and 60% for the residential areas (aligning with the residential suburban zone), which results in a weighted average impervious value of 43% (current). The starting household density was 16.5 HH/ha density within the residential zone (only).

When the residential and open space areas are treated in aggregate (our methodology), the resulting future impervious meshblock value for the 50% household growth is 57%. If the residential and park areas had been treated separately, the resulting weighted average future impervious meshblock value is 50% (10% within the park and 70% within the residential zone).

The second examined meshblock was 2518400, in which Burnside Park accounts from approximately 96% of the area (not considering roads) and the residential areas account for 4%. The 2023 impervious value for the ‘residential and other’ areas in this meshblock is 8%. That comes from a 6% impervious value within the parks and 59% residential impervious value. The 2023 household density is approximately 14.4 HH/ha within the residential zone. When residential and park areas are combined, the resulting future impervious meshblock value was 17%, compared with a 9% future impervious meshblock value when treated separately (6% impervious in parks and 70% impervious in the residential area).

The results of this analysis indicate that combining the parks and residential areas is a conservative approach, which results in higher impervious values than treating these areas separately. The difference in resulting impervious values between the two approaches, however, is relatively small (7-8%) and therefore expected to have a small impact on the overall results. A 50% growth in household count is also higher than the average; analysis of 2103 vs. 2023 household projections indicates an overall household growth of 26%.

Additionally, these modest effects are further diluted by the small spatial extents of parks. Therefore, the difference in resulting future impervious values when amalgamating parks and residential areas vs. treating them separately is expected to be smaller in a more typical meshblock.

4.2.3 Transportation Corridors

While the calculation approach separates existing roads from residential areas, it does not consider new roads (transportation corridors) within Greenfields development area. The average 2023 impervious value of existing street catchments was analysed to approximate the impervious values of future transportation corridors. The average 2023 impervious value of the entire street catchment layer is approximately 68%. Spatial selection was utilized to remove roads within the OARC as well as remote/rural areas from the analysis. This filtered approach yielded an average impervious value of 76%. Since the impervious values of the roads are generally within the range of the impervious values for the fully developed residential zones, the effect of the roads on the overall residential future impervious values is expected to be minimal, justifying the approach of disregarding future roads

int the analysis. Note also that the spatial extents of roads are expected to be relatively small within new developments (10-20% of the total area).

4.3 Summary Data for Meshblock Calculations

Within the (four) subzones in each meshblock, actual and district planning data was tabulated into excel for the calculations. District planning data for typical impervious percentage and household density was based on zonings in accordance with Section 0. Where meshblocks had a single uniform zoning, this was as simple assignment, but where they spanned multiple zones, the area weighted average value was calculated for the meshblock.

Current actual data on impervious percentage was extracted from the 2023 impervious data. Current actual data on household density was taken as the meshblocks' 2023 HH count (from the CCC HH projection data) divided by the residential meshblock area.

5. Future Residential Imperviousness

5.1 Calculation Overview

For the "residential and other" subarea of the meshblock (hereafter referred to as 'residential meshblock'), the methodology begins by defining rules to characterise residential meshblocks as currently being fully Greenfields or fully Brownfields, or more generally having a mix of Greenfields and Brownfields. This assignment of fraction Brownfield to each residential meshblock is an attribute on the residential meshblock's subarea and is not geographically delineated within the residential meshblock. Two separate calculations are then done for each residential meshblock, first as if the residential meshblock was fully Greenfields and second as if it was fully Brownfields.

The unmitigated residential meshblock impervious result is then a fractional combination of the two results. It is assumed that residential Greenfield developments will typically mitigate runoff up to 50-yr ARI. Therefore, the mitigated residential meshblock impervious result is then a fractional combination of the new brownfield result and the existing (no change) greenfield impervious.

5.2 Residential Brownfield/Greenfield Fraction

The brownfield/greenfield fraction for each residential portion of meshblock is determined from the **maximum** value of the household density ratio and the 2023 impervious ratio described in Equations A and B below.

- **Equation A** is based on the 2023 Household (HH) density, compared to a typical future residential household density of 15 household per hectare.[18] Equation A is typically relevant for residential areas.

$$\textbf{Equation A}: \ \%\textbf{Brownfield} = \textbf{2023 HH density} \div \textbf{15}$$

Where:

%Brownfield	= fraction Brownfield of the meshblock (limited to 100%)
2023 HH density	= 2023 household density, taken as the meshblocks' 2023 HH count (from the CCC HH projection data) divided by the residential meshblock area

Example: If a meshblock has an existing (2023) household density of 7.5HH/ha, then it is considered at least 50% Brownfield (from Equation A).

[18] This value was selected in GHD's 2021-2022 future impervious analysis from visual inspection of Citywide 2020 residential household density and is consistent with the predominant Residential Suburban zoning (previously Living 1).

- **Equation B** is based on the 2023 percentage impervious, compared to a typical residential impervious of 60%[19].

  Equation B is more typically relevant to non-residential areas where the percentage impervious is driven by commercial or business developments with low or null household-count per meshblock (where Equation A would yield an erroneously low percentage Brownfield).

$$\textbf{Equation B}: \%\textbf{Brownfield} = \textbf{ED2020}\ \%\ \textbf{Imp} \div \textbf{60}\%$$

Example: If a meshblock has an existing (ED2020) percentage impervious of 45%, then it is considered at least 75% Brownfield (from Equation B).

The final brownfield percentage is the larger (maximum of) the results from Equations A and B.

The complimentary fraction Greenfield is calculated as: $\%\text{Greenfield} = 100\% - \%\text{Brownfield}$

Note that this is a simple tabular calculation per meshblock and there is no geographical split of Brownfield and Greenfield fractions within each meshblock.

5.3 Predicted Brownfield Imperviousness (Equation 1)

Equation 1 estimates the future impervious value for Brownfield meshblocks (or Brownfield parts of each meshblock) in proportion to the increase in household counts from the CCC's household projection data.

$$\textbf{Equation 1 (Brownfield): YYYY "Brownfield" Impervious} = \textbf{2023 Impervious} * \left(\frac{\textbf{YYYY HH}}{\textbf{2023 HH}}\right)$$

Where:

YYYY = Analysis year: 2043, 2073, or 2103.

2023 Impervious = 2023 average percent impervious

YYYY HH = Estimated household count for the analysis year

2023 HH = 2023 household count

Notes:

- if 2023 HH = 0, or if YYYY HH < 2023 HH, then it the ratio of YYYY HH/2023 HH is set to 1 and the result is no change in between current and future Brownfield percent impervious from this interim estimation process. This ensures that the Brownfield percent impervious outcome is never less than the existing 2023 percent impervious.

5.3.1 Brownfield Percent Impervious Limits

In zones with high growth ratios, equation 1 was found to overestimate the future impervious values, including results exceeding 100%. An upper limit on Brownfield impervious value was therefore defined slightly higher than the District Plan typical zone impervious. This limit was set at 75% of the District Plan zoning derived pervious value (as defined in Section 0).[20]

Example: if the district plan zone weighted impervious is 80% (20% pervious), the Brownfield impervious is limited to 85% (15% pervious). This 15% pervious is calculated as 75% of the 20% DP pervious.

[19] This threshold value was derived in GHD's 2021-2022 future impervious analysis from visual inspection of citywide 2020 percentage impervious and is midway between the two prevalent Residential zoning guidance values (50% and 70%).

[20] In rare cases, the 2023 impervious data from residential class area is higher than the above credible upper limit on Brownfield impervious value, in which case the credible upper limit on Brownfield impervious value was increased to match the 2023 impervious data from residential class area.

5.4 Predicted Greenfield Residential Imperviousness (Equation 2)

The following two methods were used for predicting residential impervious from future development in greenfield areas.

5.4.1 Primary Greenfield Calculation

Where the predicted development is within the District Plan's limits on household counts (and hence density), then the District Plan fully residential impervious is used to identify the projected future impervious (Equation 2).

Equation 2: If YYYY HH < District HH then

$$(\textbf{Greenfield}): \textbf{YYYY Greenfield Impervious} = \textbf{District Plan Impervious} * \left(\frac{\textbf{YYYY HH}}{\textbf{District HH}}\right)$$

Where:

YYYY = Analysis year: 2043, 2073, or 2103.

District Plan Impervious = Impervious value derived from District Plan zonings in a fully developed state (Section 0)

District HH = House count projected by the District Plan zonings in a fully developed state

YYYY HH = Projected household count for the analysis year

5.4.2 Alternative Greenfield Calculation

Where the predicted development exceeds the District Plan's limits on household counts (and hence density), then the following alternative calculation method was applied.

For meshblocks where the household projections exceed the District Plan's land use designation[21] (YYYY HH > District HH), it is understood that the District Plan would need to change to achieve the predicted development. Equation 2 would generate a poor estimate for impervious value and was prone to over-estimation.

Therefore, where YYYY HH > District HH, Greenfield impervious values are therefore determined by interpolation using the HH density / Impervious value relationship defined in Table 2, based on the analysis year's predicted household density in each meshblock. The resulting value was checked against the District Plan impervious value, and the larger impervious value was selected, to ensure the alternative calculation method did not underpredict the future impervious value.[22]

Table 2 – Greenfield impervious values based on household density

HH density	Impervious	Background
0	10%	Minimum Imperviousness of 10% (typical of open space, etc.)
15	60%	Matching "Residential Suburban" zoning
30	75%	Matching District Plan "Residential Medium Density" zoning
60	80%	Matches Central City zones
300+	90%	Capped to 90% Impervious

[21] Note that in the HH projection data, material household growth was often predicted in meshblocks dominated by commercial zoning. It is not clear whether such growth is plausible within commercial zoning or implies anticipated zone change to residential, or is more simply an error, but such zones will result in high impervious anyway due to the commercial zoning.

[22] The District Plan impervious value exceeded the alternative Greenfields calculation method results in approximately 1% of meshblocks in the 2103 calculations. For example, Table 2 underpredicted the percent impervious values of meshblocks with RNN, due to the zone's higher impervious value in relation to household density per Table 1. The District Plan impervious value was thus selected as the future impervious value in these cases.

5.4.3 Lower Limit to Greenfield Calculation

Whether the primary or alternate greenfield calculation was used, the predicted Greenfield developed meshblock impervious percentage should be equal or greater than the 2023 impervious value. If the above result is lower than 2023 impervious value, then it is replaced with 2023 impervious percentage.

5.5 Combined Greenfield/Brownfield calculations

The respective greenfield and brownfield calculation results for any meshblock are then combined in proportion to the percentage greenfield and brownfield previously assessed for that meshblock. There are two versions of output required: unmitigated and mitigated outputs.

5.5.1 Unmitigated and Mitigated Outputs

The Christchurch City Council requires that any development of Greenfield areas mitigate increased storm runoff up to the 50-year storm event. To simulate the effects of these mitigation devices in the future impervious data, two approaches are used. For the 10- and 50-year storm events, the greenfield fraction of each meshblock Greenfield areas that are planned to be developed retain their existing impervious values, thereby simulating the effect of the mitigation devices. For the 200-year storm event, the Greenfield areas are treated as developed, thereby simulating no mitigation devices present. This is a conservative approach, but as stated previously, the mitigation devices are only designed to mitigate up to the 50-year event. Anything larger than the 50-year event would be expected to overflow the mitigation systems and result in close to unmitigated runoff.

As a result, there are two versions of the 2043, 2073, and 2103 impervious surfaces:

1. The mitigated future impervious surface represents the 10- and 50-year storm events.
2. The unmitigated future impervious surface represents the 200-year storm event.

In all cases (10/50/200yr), the impact of brownfield development is reflected in the impervious surfaces.

5.5.2 Unmitigated impervious surface (200-yr ARI)

The combined future unmitigated impervious (YYYY Imp (unmitigated)) values are calculated using Equation 3a below for each meshblock for the given analysis year (YYYY):

Equation 3 a - Unmitigated Impervious value:

$$\text{YYYY Imp (unmitigated)} = (\%\text{Brownfield} \times \text{YYYY Brownfield Impervious}) + (\%\text{Greenfield} \times \text{YYYY Greenfield Impervious})$$

5.5.3 Mitigated impervious surface (10-yr and 50-yr ARI)

The combined future mitigated impervious values (YYYY Imp (mitigated)) are calculated using Equations 3b below for each meshblock for the given analysis year (YYYY):

Equation 3 b - Mitigated Impervious value:

$$\text{YYYY Imp (mitigated)} = (\%\text{Brownfield} \times \text{YYYY Brownfield Impervious}) + (\%\text{Greenfield} \times \text{2023 Impervious})$$

6. Future Non-Residential Imperviousness

6.1 Road Reserve

The impervious values within the road reserve were maintained in future horizons (2043/2073/2103).

6.2 Commercial

The 2023 commercial impervious values for each meshblock were determined from the 2023 impervious data described in Section 0. The commercial impervious value for 2043, 2073 and 2103 was set to 90%.[23] This approach assumes full build-out of commercial areas by 2043, consistent with discussions with CCC during the previous 2021 work.

6.3 Red Zones

The approach for determining the red zone future impervious values was decided in consultation with CCC. The general approach was to preserve 2023 impervious values throughout 2043, 2073 and 2103.

The future impervious approach in some parts of the OARC (part of the red zone) was different as summarised below:[24]

- Buildings: Existing and proposed buildings within OARC were both defined as 100% impervious, assuming development of buildings by 2043. Existing buildings were assumed to remain and are reflected in the 2023 impervious data. As discussed in Section 2, GHD received shapefiles with the proposed building outlines from CCC. Thes polygons were buffered to maintain confidentiality of the proposed building outlines. An average percent impervious value was applied to these areas.
- Roads: Roads were treated as 100% impervious within the OARC red zone. As discussed in Section 2, GHD received shapefiles outlining existing roads, proposed roads, and existing roads to be removed. Therefore, the future state was taken as the existing roads + proposed roads – roads to be removed. The road polygons provided reflected curbline extents rather than road reserve extents. Note also that the impervious values of the existing roads were not revised, as these are reflected in the 2023 impervious raster.

7. Generating Impervious Rasters

7.1 Generating Impervious Rasters from Meshblock Results

The percent impervious outcomes for the subareas (roads, red zones, commercial, and residential) were applied to the separated four subareas for each meshblock. As discussed in Section 6, the impervious areas within the street catchments and the red zone areas (other than within the OARC) were not altered in the future impervious rasters. The OARC impervious was modified within the future rasters as described in Section 6.3.

To maintain the spatial resolution of the 2023 impervious raster (2 m), changes in impervious values for the residential and commercial zones were applied as scaling factors to the 2023 impervious raster. Scaling factors were applied to the pervious values (not impervious) to ensure that impervious results would never exceed 100%. The same pervious commercial scaling factors approach were applied to all future horizons (2043/2073/2103) to

[23] Note that if the 2023 impervious value exceeded 90%, this 2023 impervious value was retained in future impervious layers (2043/2073/2103)
[24] Refer to Tim Preston's email to Tom Parsons, Jarad Sinni, Peter Christensen, and Madeline Howell dated 26/7/2024 and entitled "Future Impervious – Avon River corridor redzone" for discussion of how to treat buildings and roads within the OARC.

achieve 90% impervious within the commercial zones (see Section 6.2). Different residential scaling factors were applied for each analysis year (2043/2073/2103) as well as for the non-mitigated and mitigated conditions.

The results of the analysis yielded six rasters to account for three future horizons (2043/2073/2103) as well as the two mitigation scenarios (mitigated and non-mitigated). Please refer to Attachment 3 and Attachment 4 for results maps including weighted average impervious values, change in impervious values, and impervious rasters.[25]

7.2 Incorporating Feasible Dwelling Capacity

The 2103 mitigated and non-mitigated rasters were modified to consider the outputs of the feasible dwelling capacity. The FDC analysis identified developed parcels with additional capacity for intensification (see Section 2 for a description of input data). These FDC parcels are primarily zoned as "Residentials Suburban" and could be developed to more closely resemble the "Residential Medium Density" zone. The percent impervious values within the 2103 rasters were modified within these parcels to 75% impervious, consistent with Residential Medium Density zoning. Note that the 2023 impervious values were retained in parcels where 2023 impervious value exceeded 75% to avoid reductions in impervious percentage.[26] Raster results incorporating FDC are included in Attachment 3.

8. Modelling Scenario Selection

CCC's policy is to anticipate 100 years into the future for their 200-year ARI District Plan modelling. The above process generated 2103-time horizon data which is the nearest practicable time horizon for growth planning and expect future impervious values. Because results have been generated with and without incorporation of the feasible dwelling analysis, a choice was required as to which data should be selected for model runs. This modelling scenario selection was completed in consultation with the CCC.

Incorporation of results of the feasible dwelling capacity for the 2103 raster was debated. CCC raised concerns that combining the 2103 household predictions with the feasible dwelling analysis would double-count growth, leading to a conservative scenario. On the other hand, a precautionary approach to imperviousness is also warranted, so that the CCC can plan for flooding implications due to future growth, instead of mitigating the flood impacts after development has occurred.

Incorporating the results of the feasible dwelling analysis (see Section 7.2) are expected to increase the imperviousness across Christchurch by approximately 2%. The FDC parcels make up approximately 9% of the urban area[27] and signify a 19% increase in imperviousness across these parcels when adjusted to 75% impervious in 2103.[28]

Discussions led by Tim Preston with four Senior Stormwater Planning Engineers collectively assembled from the Stormwater and Asset Planning Team reached a unanimous decision[29] to include the feasible dwelling analysis results in the 2103 raster (per Section 7.2). Some key points pertaining to this decision are summarized as follows:[30]

- Intensification and redevelopment in the next 100-years is expected to impact more than 9% of the urban area, suggesting that inclusion of the FDC parcels may be unconservative.
- The CCC is already noting intensification within the residential suburban and residential suburban density transition zones, which is likely underrepresented in the household growth model but is captured in the feasible dwelling analysis.

[25] Focal statistics (with a 10 m radius) were applied to the rasters in the results maps to allow for better visualization of results.
[26] Approximately 5% of parcels displayed 2023 impervious values > 75% and were therefore not altered.
[27] This spatial extent was calculated by taking the total area of the parcels that were modified to 75% divided by the total area of the meshblocks, filtering out >0.1 km^2
[28] The 2103 weighted average impervious value across the FDC parcels is 56% when filtering out impervious values below 30%. This weighted average well with the mode of the frequency distribution. Refer to Madeline's email "FIMP discussion – John or not?" 25/10/2024.
[29] 25/10/2024
[30] Refer to Tim Preston's email "Future impervious options for modelling – meeting notes" 25/10/2024 as well as Tim Preston's email (and prior emails on the same chain) "Future impervious modelling" 24/10/2024.

- CCC has observed trends toward increases in impervious coverage with lifestyle changes independent of growth (i.e.: larger new houses, with more pavement / pool and less garden, even though old and new were single household family dwellings), which is not accounted for in the current future impervious methodology.
- The growth model used to generate the household growth predictions for the future impervious analysis assumes distributed growth, while actual growth patterns tend to be clustered. Only considering predicted distributed growth could lead to underpredicting local flooding impacts of future clustered developments.

In summary, the 2103 non-mitigated raster considering the feasible dwelling capacity was selected for the 200-year ARI.

9. Digital Deliverables

The following digital deliverables (2 m rasters) have been submitted to the CCC:

1- 2023 impervious raster

2- 2043 non-mitigated impervious raster

3- 2043 mitigated impervious raster

4- 2073 non-mitigated impervious raster

5- 2073 mitigated impervious raster

6- 2103 non-mitigated impervious raster

7- 2103 mitigated impervious raster

8- 2103 non-mitigated impervious raster considering feasible dwelling capacity

9- 2103 mitigated impervious raster considering feasible dwelling capacity

10. Summary

GHD completed a future impervious analysis to develop impervious data corresponding to future scenarios (2043/2073/2103). This analysis method was largely the same as a previous analysis completed by GHD in 2022 and outlined in GHD's *Methodology for Future Horizons Imperviousness* report. The current analysis utilizes updated current impervious values (2023), new household projections, updated assumptions on District Plan household densities and percent impervious values, and parcels identified with capacity for additional densification. The results of the analysis yielded 2 m impervious rasters for 2043, 2073, and 2103 for both non-mitigated and mitigated Greenfields development, which can be used as an input to hydraulic modelling to assess the impact of future development in Christchurch. After consultation with CCC, GHD recommends utilizing the 2103 non-mitigated raster with the FDC parcels for the 200-year ARI.

Attachments

Attachment 1

Schedule of Adopted Household Densities and Percentage Impervious for District Plan Zones

Schedule of Adopted Household Densities and Percentage Impervious for District Plan Zones

District Plan Zone Name	Assumed Zone Density (HH/ha)	Assumed Zone % Impervious	District Plan	Source	Comment
Residential Central City (CBD)					
Commercial Central City Mixed Use	60	80	CCC	5	
Commercial Central City (South Frame) Mixed Use	60	80	CCC	5	
Residential Central City	60	80	CCC	5	
Residential Medium Density					
Residential Medium Density	30	75	CCC	5	
Residential Visitor Accommodation Zone	30	75	CCC	5	
Residential Guest Accommodation	30	75	CCC	5	A
Residential New Neighbourhood	20	70	CCC	5	
Rural Banks Peninsula	20	70	CCC	5	
Rural Port Hills	20	70	CCC	5	
Rural Templeton	20	70	CCC	5	
Rural Urban Fringe	20	70	CCC	5	
Rural Waimakariri	20	70	CCC	5	
Living 3	20	70	Selwyn	5	
Living X	20	70	Selwyn	5	
Living Z	20	70	Selwyn	5	
Living MD1	20	70	Selwyn	5	
Residential Suburban Density Transition					
Residential Suburban Density Transition	22.5	65	CCC	5	
Residential Suburban					
Residential Banks Peninsula	15	60	CCC	5	
Residential Hills	15	60	CCC	5	
Residential Large Lot	15	60	CCC	5	
Residential Suburban	15	60	CCC	5	
Residential Small Settlement	15	60	CCC	5	
Papakainga Kainga Nohoanga	15	60	CCC	5	
Existing Devlopment Area: Rocklands	15	60	Selwyn	5	B
Existing Development Area: Devine Acres	15	60	Selwyn	5	B
Existing Development Area: Johnsons Road	15	60	Selwyn	5	B
Existing Development Area: Johnsons Road	15	60	Selwyn	5	B
Existing Development Area: Jowers Road	15	60	Selwyn	5	B
Existing Development Area: Kingcraft Drive	15	60	Selwyn	5	B
Living 1	15	60	Selwyn	5	B
Living 1A	15	60	Selwyn	5	B
Living 1A1	15	60	Selwyn	5	B
Living 1A2	15	60	Selwyn	5	B
Living 1A3	15	60	Selwyn	5	B
Living 1A4	15	60	Selwyn	5	B
Living 1A5	15	60	Selwyn	5	B
Living 1A6	15	60	Selwyn	5	B
Living 1B	15	60	Selwyn	5	B
Living 2	15	60	Selwyn	5	B
Living 2A	15	60	Selwyn	5	B
Living 2A (Blakes Road)	15	60	Selwyn	5	B
Living WM	15	60	Selwyn	5	B
Living WM	15	60	Selwyn	5	B
Living West Melton South	15	60	Selwyn	5	B
Living West Melton (South)	15	60	Selwyn	5	B
Rural Zone	15	60	Selwyn	5	B
Inner Plains	15	60	Selwyn	5	B

Schedule of Adopted Household Densities and Percentage Impervious for District Plan Zones

District Plan Zone Name	Assumed Zone Density (HH/ha)	Assumed Zone % Impervious	District Plan	Source	Comment
Residential Suburban					
Outer Plains	15	60	Selwyn	5	B
Port Hills	15	60	Selwyn	5	B
Commercial/Industrial					
Commercial Banks Peninsula	0	90	CCC	1	
Commercial Core	0	90	CCC	1	
Commercial Local	0	90	CCC	1	
Commercial Mixed Use	0	90	CCC	1	
Commercial Office	0	90	CCC	1	
Commercial Retail Park	0	90	CCC	1	
Commercial Central City Business	0	90	CCC	1	
Industrial General	0	90	CCC	1	
Industrial Heavy	0	90	CCC	1	
Industrial Park	0	90	CCC	1	
Business 1	0	90	Selwyn	3	
Business 1B	0	90	Selwyn	3	
Business 3	0	30	Selwyn	3	C
Key Activity Centre	0	70	Selwyn	3	D
Open Space/Non-Residential Rural					
Avon River Precinct (Te Papa Otakaro)	0	10	CCC	2	
Open Space Coastal	0	10	CCC	2	
Open Space Community Parks	0	10	CCC	2	
Open Space Community Parks or Rural Quarry (Templeton)	0	10	CCC	2	
Open Space Metropolitan Facilities	0	60	CCC	3	E
Open Space McLeans Island	0	10	CCC	2	
Open Space Natural	0	10	CCC	2	
Open Space Water and Margins	0	10	CCC	2	
Rural Quarry	0	30	CCC	2	
Rural Quarry or Open Space Community Parks (Templeton)	0	30	CCC	2	
Specific Purpose					
Specific Purpose (Airport)	0	50	CCC	3	
Specific Purpose (Burwood Landfill & Resource Recovery Park)	0	50	CCC	2	
Specific Purpose (Cemetery)	0	10	CCC	2	
Specific Purpose (Hospital)	0	90	CCC	2	
Specific Purpose (Lyttelton Port)	0	90	CCC	2	
Specific Purpose (Flat Land Recovery)	0	5	CCC	4	F
Specific Purpose (Nga Hau e Wha)	0	60	CCC	2	
Specific Purpose (Otakaro Avon River Corridor)	0	5	CCC	4	F
Specific Purpose (Golf Resort)	0	20	CCC	2	
Specific Purpose (Ruapuna Motorsport)	0	20	CCC	3	G
Specific Purpose (School)	0	40	CCC	2	
Special Purpose - School	0	40	CCC	2	
Specific Purpose (Styx Mill Road Transfer Station)	0	90	CCC	3	
Specific Purpose (Tertiary Education)	0	50	CCC	2	
Specific Purpose (Defence Wigram)	0	20	CCC	3	G
Transport					
Transport	0	91	CCC	2	

Comments:
A. Assumed equivalent to RVA
B. No household growth was modelled for these areas
C. Lincoln University
D. Lincoln Commercial Centre
E. Highly variable
F. Red Zone development will be taken into account separately
G. Significant portion of greenspace surrounding track

References:
1) CCC
2016 Ch21_Runoff_Coefts.xlsx
% impervious from "2016 main" tab, Table 21-7.
Interim recommendation, received from Jo Golden, 18/07/2019 16:09 "FW: LDRP 97 Wastewater 2068 Population Predictions"
N:\NZ\Christchurch\Projects\51\12506449\00.Document Transfer\Incoming\20190718 District plan runoff coeff

2) GHD (2015)
20151014 CityPlanZones Impervious percentage.xls
"G:\51\12506449\02.ModelBuild\Technical\MPD" G.Garrison (supervised by J. McArthur)

3) GHD (2019)
:0190909_12506449_CALC_MPD_data model.xlsx
Tim's email: 14/04/2020 4:54 PM - "MPD actions" (\\51\12506449\02.ModelBuild\Technical\MPD\20200429 - District Plan Perc impervious background - tims email.pdf)

Tim's email: 29/04/2020 5:53 PM "RE: Conversation record 9/8/2019 - MPD - 21 confirmed plus"

4) GHD (2024a)
Madeline's email: 23/07/2024 "Future Impervious- Discussion with Tom" project number 12601181, which defines the percent impervious for red zones outside of roadways. Note that houses and roadways will be dealt with separately, per Tim's email 26/7/2024 "Future Impervious - Avon River corridor redzone"

5) GHD (2024b)
N:\NZ\Christchurch\Projects\51\12651364\Tech\20240927 Variation GIS Prework_MH\20241008_Prework_Conclusions.xlsx
Included in Attachment 2 of this memo

Attachment 2

Waterways, Wetlands and Drainage Recommendations

Attachment 2: WWDG Recommendations

Waterways, Wetlands and Drainage Guide Recommendations

As part of GHD's future impervious analysis, GHD investigated the impervious values of various residential zones within the Christchurch District Plan[1]. This investigation yielded typical percent impervious values for these residential zones that differ from the Waterways, Wetlands and Drainage Guide (WWDG), motivating consideration of updates to the WWDG. GHD recommends the following:

1. Clearly redefine the applicable spatial extents of the percentage impervious values and runoff coefficients with roads and parks separated from residential zones.
2. Update the percentage impervious values for residential zones reported in Table 21-6 of the WWDG, to align with the values presented in Table 1 (especially changes in RS and RMD), along with corresponding Rational Method runoff coefficients provided in Table 21-5.

Table 1: Recommended impervious percentage updates for WWDG Table 21-6.

District Zone	WWDG % Impervious	Recommended % Impervious
Residential Suburban	50%	60%
Residential Suburban Density Transition	65%	65%
Residential New Neighbourhood	70%	70%
Residential Medium Density	80%	75%
Road reserve	NA	75%[2]
Parks	NA	10%

Spatial Extent Updates

The spatial extents of the percentage impervious values and corresponding Rational Method runoff coefficients (reported in Tables 21-6 and 21-5 of the WWDG, respectively) are not well-defined in the WWDG. Based on discussions with the Christchurch City Council, GHD understands the percent impervious values are aggregate values in the scale of suburbs or large subdivisions and considering residential areas, roads, and parks in aggregate.

GHD recommends separating parks and roads from residential areas when presenting the percent impervious values and runoff coefficients in the WWDG. This approach would require the WWDG user to assess the spatial extents (% area) of roads, parks, and residential areas for the case being evaluated. The recommended percent impervious values provided in Table 1 above align with this approach (i.e. residential percent impervious values do not include roads or parks). Given that roads account for approximately 10-20% of development area, separating the roads from the residential areas would increase the accuracy of runoff calculations. Greenspace within developments is estimated to contribute to 3-5% of the total area[3] and would have a lesser impact on the overall percent impervious and associated runoff coefficients, however, separating parks and greenspace would improve clarity and consistency of the % impervious values given. If the recommended WWDG separation of parks and roads from residential areas is not undertaken, GHD recommends defining the current applicable spatial extents (i.e. in gross) of the percent impervious values and runoff coefficients.

[1] Refer to Section 3 of GHD's *Future Impervious Technical Report* dated October 2024
[2] Refer Section 4.2.3 of GHD's *Future Impervious Technical Report*
[3] Refer to Section 4.2.1 of GHD's *Future Impervious Technical Report*

Attachment 3

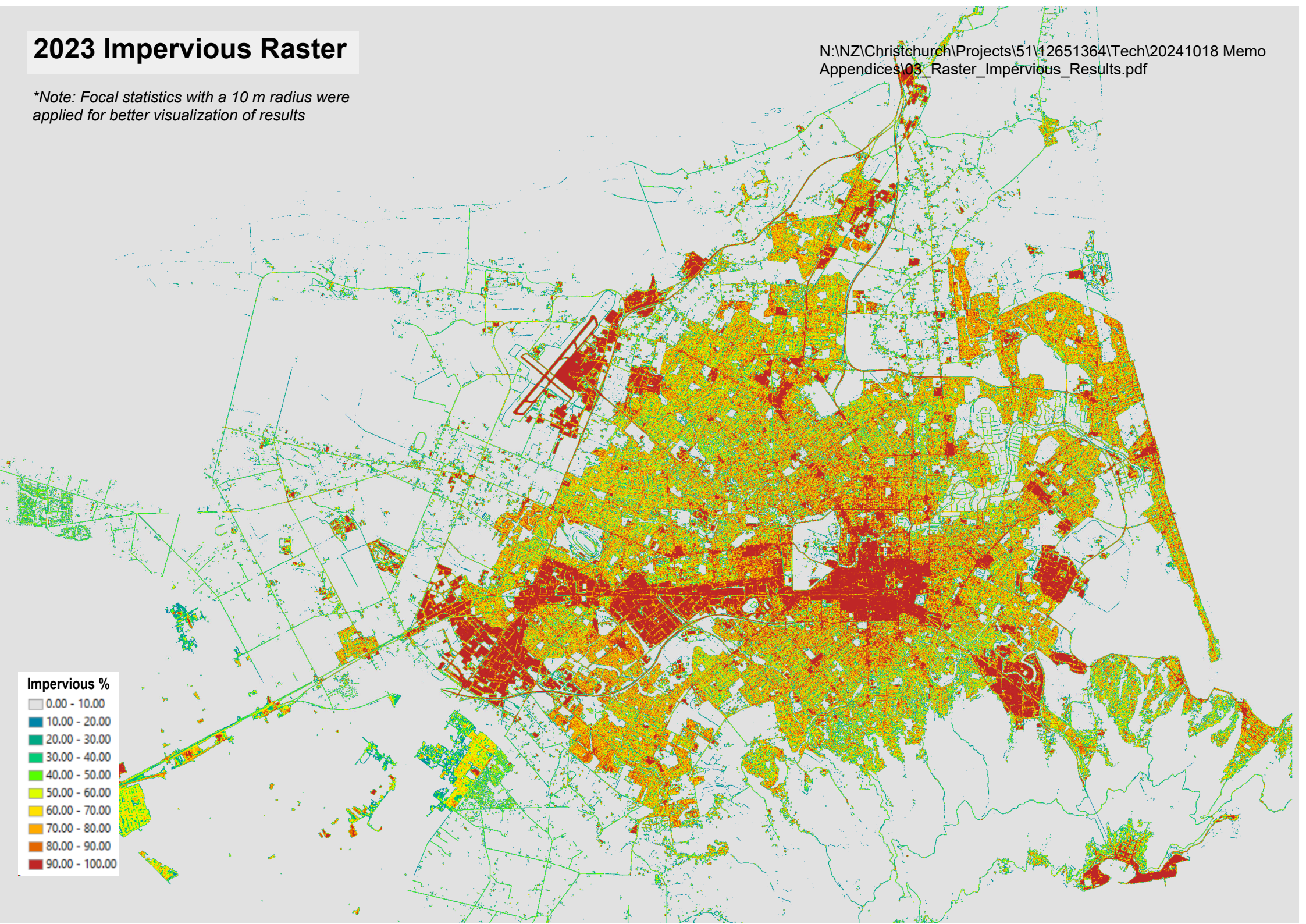
Impervious Raster Results Maps

1) 2023 Impervious Raster
2) 2103 Non-Mitigated Impervious Raster
3) 2103 Non-Mitigated Impervious Raster Considering Feasible Dwelling Capacity

2023 Impervious Raster

**Note: Focal statistics with a 10 m radius were applied for better visualization of results*

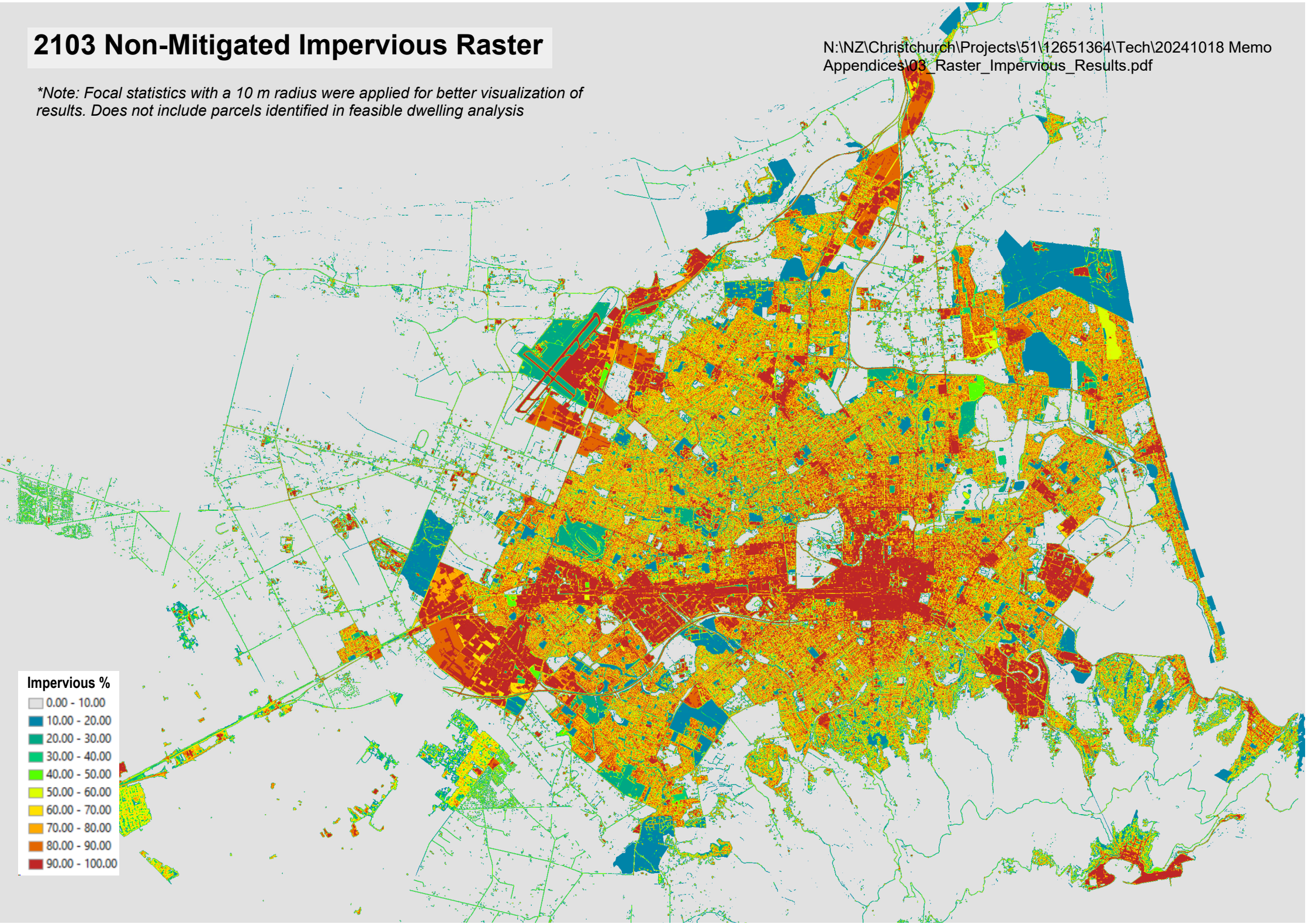
N:\NZ\Christchurch\Projects\51\12651364\Tech\20241018 Memo Appendices\03_Raster_Impervious_Results.pdf



2103 Non-Mitigated Impervious Raster

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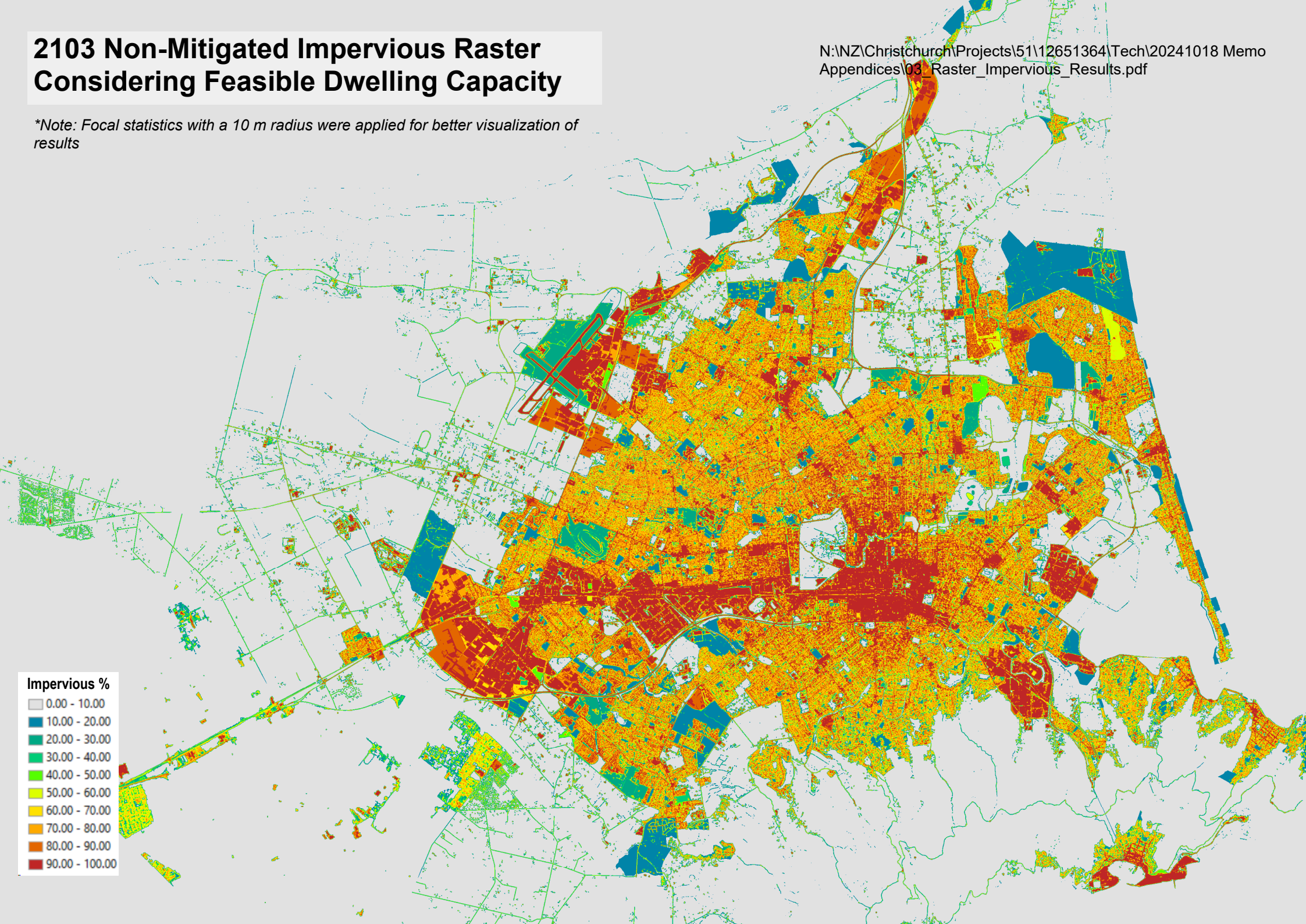
**Note: Focal statistics with a 10 m radius were applied for better visualization of results. Does not include parcels identified in feasible dwelling analysis*



2103 Non-Mitigated Impervious Raster Considering Feasible Dwelling Capacity

N:\NZ\Christchurch\Projects\51\12651364\Tech\20241018 Memo Appendices\03_Raster_Impervious_Results.pdf

**Note: Focal statistics with a 10 m radius were applied for better visualization of results*



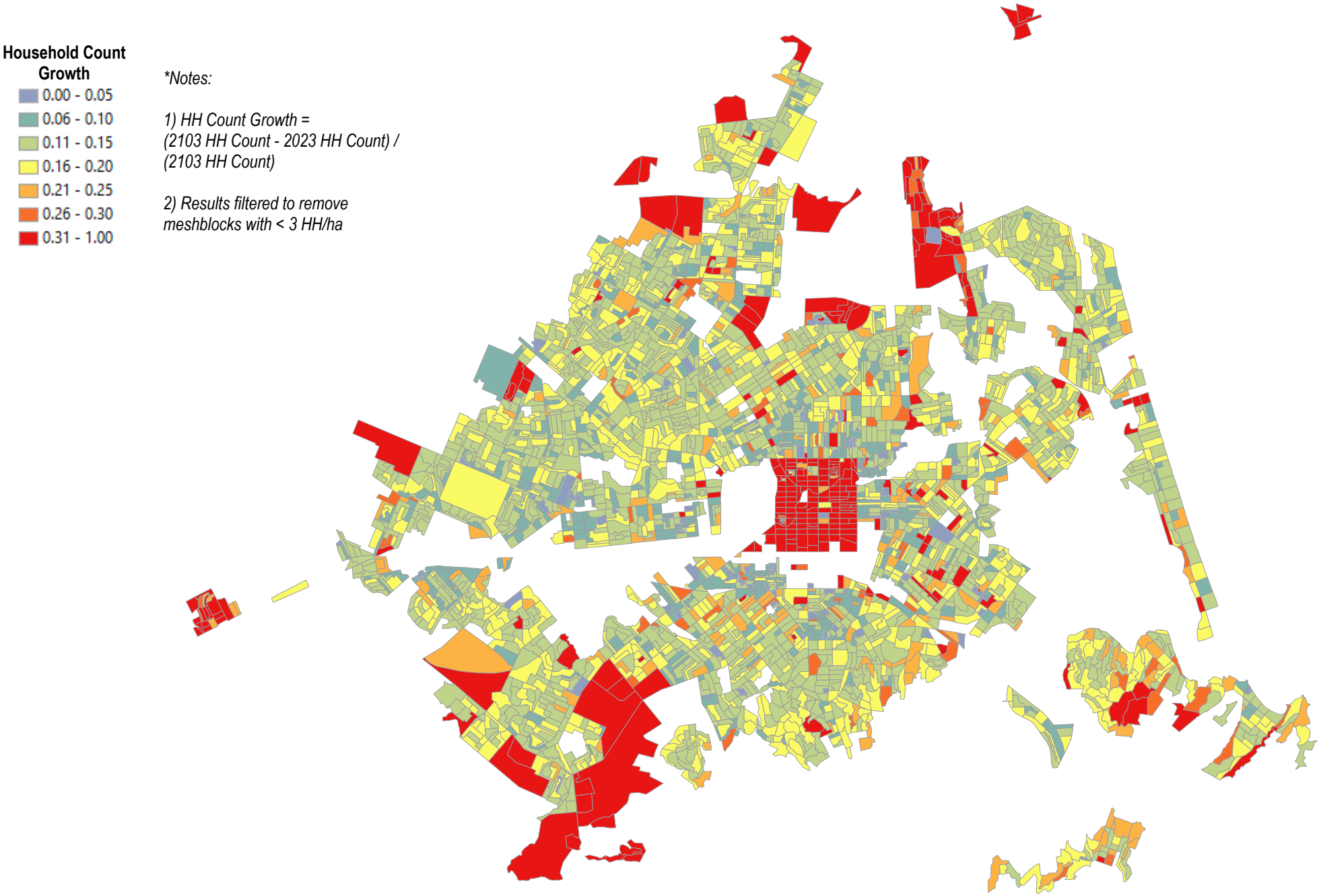
Attachment 4

Meshblock Impervious Results Maps

1) 2103 Household Growth Map
2) 2023 Impervious Values
3) 2043 Non-Mitigated Impervious Values
4) 2073 Non-Mitigated Impervious Values
5) 2103 Non-Mitigated Impervious Values
6) 2103 Mitigated Impervious Values
7) 2103 Non-Mitigated vs. 2023 Impervious Values
8) 2103 Mitigated vs. 2023 Impervious Values
9) 2103 Non-Mitigated vs. 2103 Mitigated Impervious Values

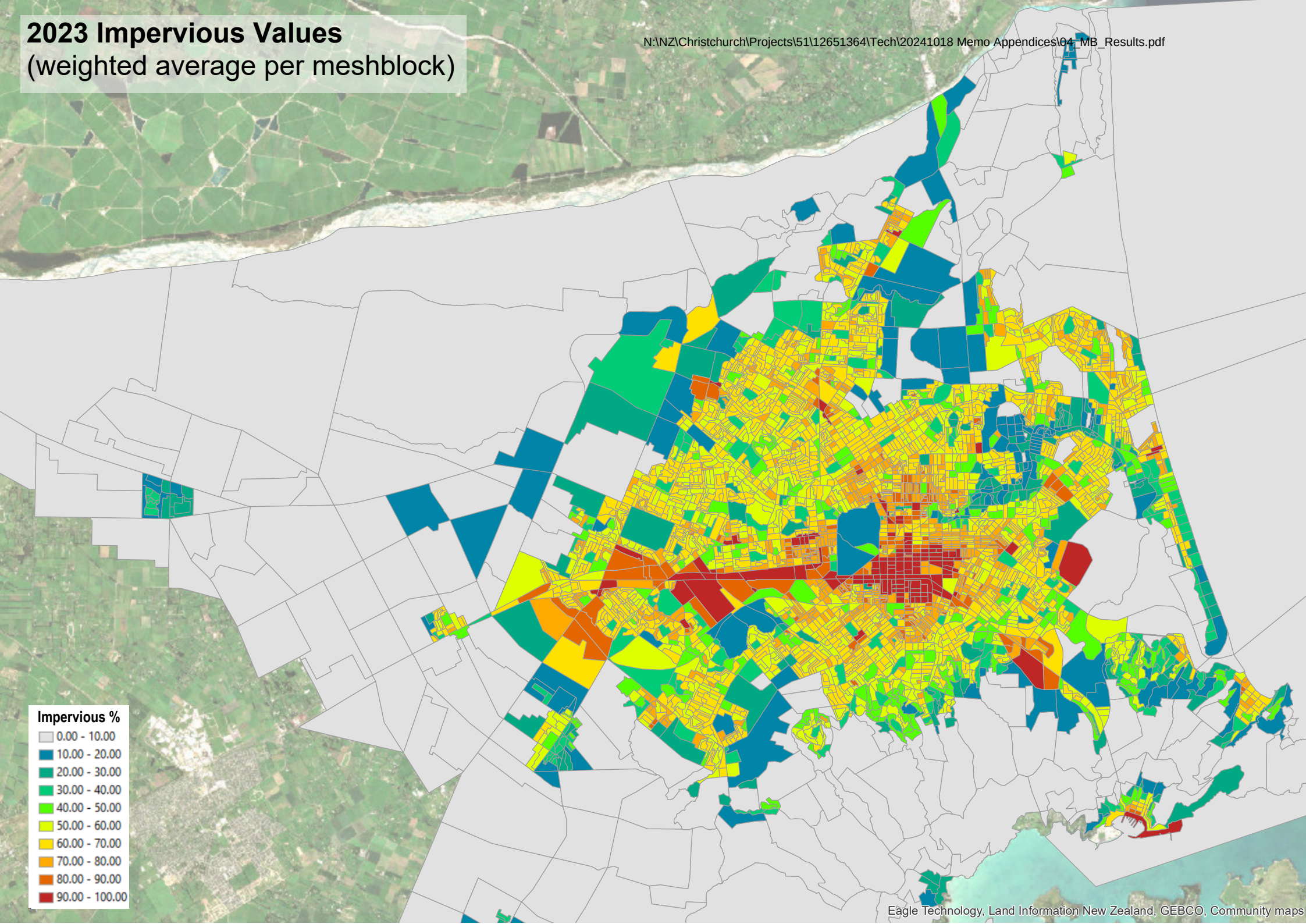
2103 vs. 2023 Household Growth

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2023 Impervious Values
(weighted average per meshblock)

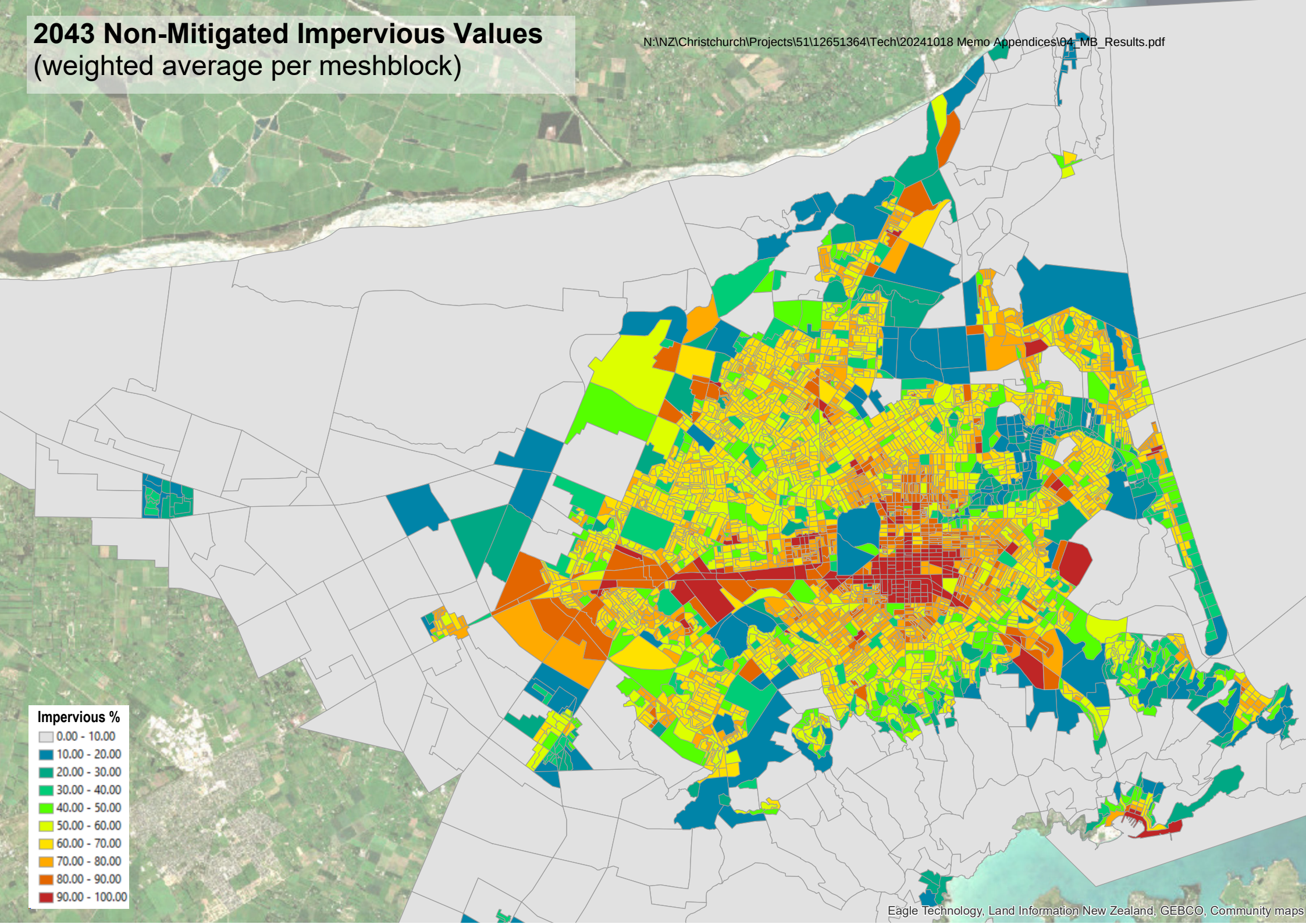
N:\NZ\Christchurch\Projects\51\12651364\Tech\20241018 Memo Appendices\04_MB_Results.pdf



Eagle Technology, Land Information New Zealand, GEBCO, Community maps

2043 Non-Mitigated Impervious Values
(weighted average per meshblock)

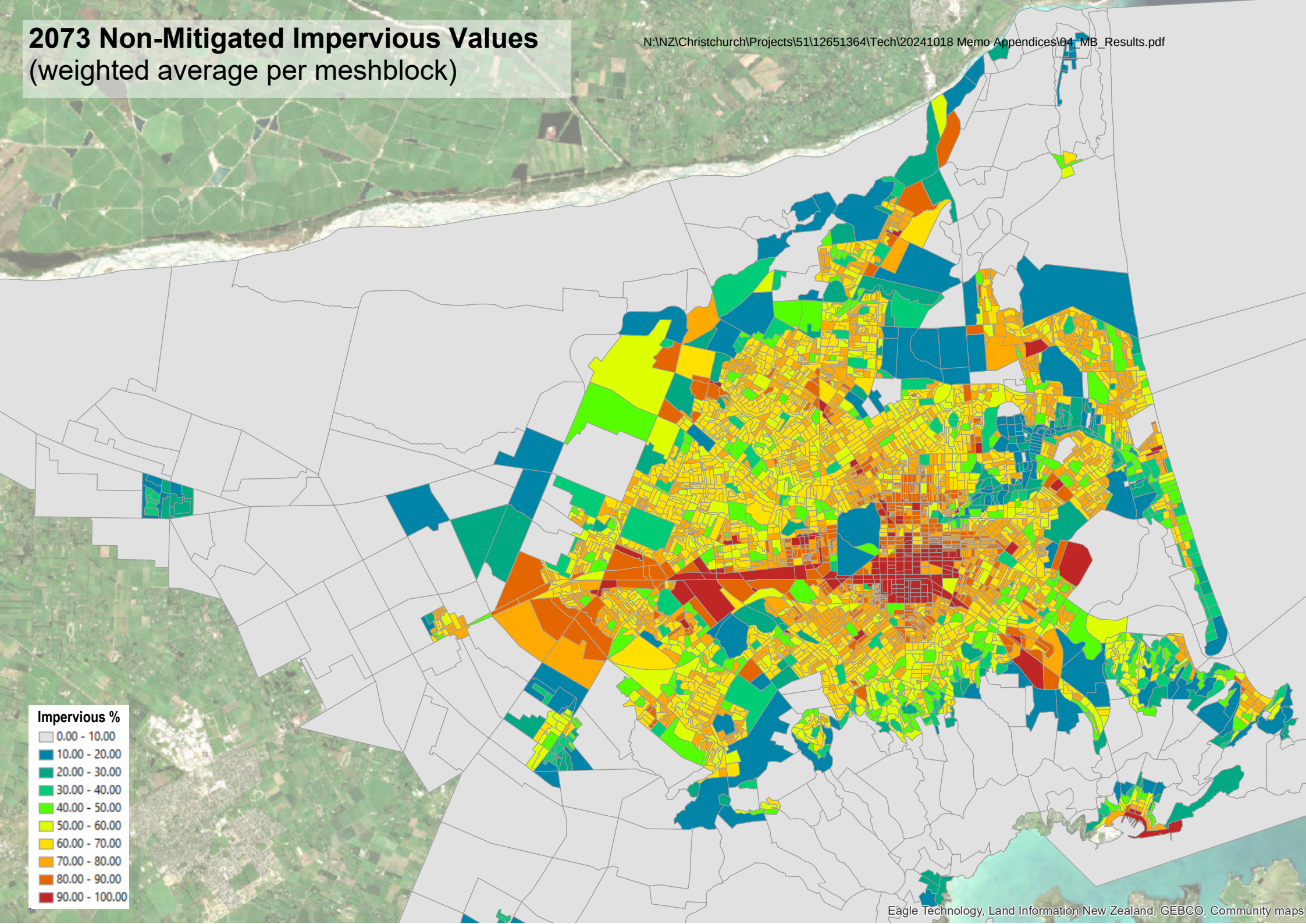
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Eagle Technology, Land Information New Zealand, GEBCO, Community maps

2073 Non-Mitigated Impervious Values
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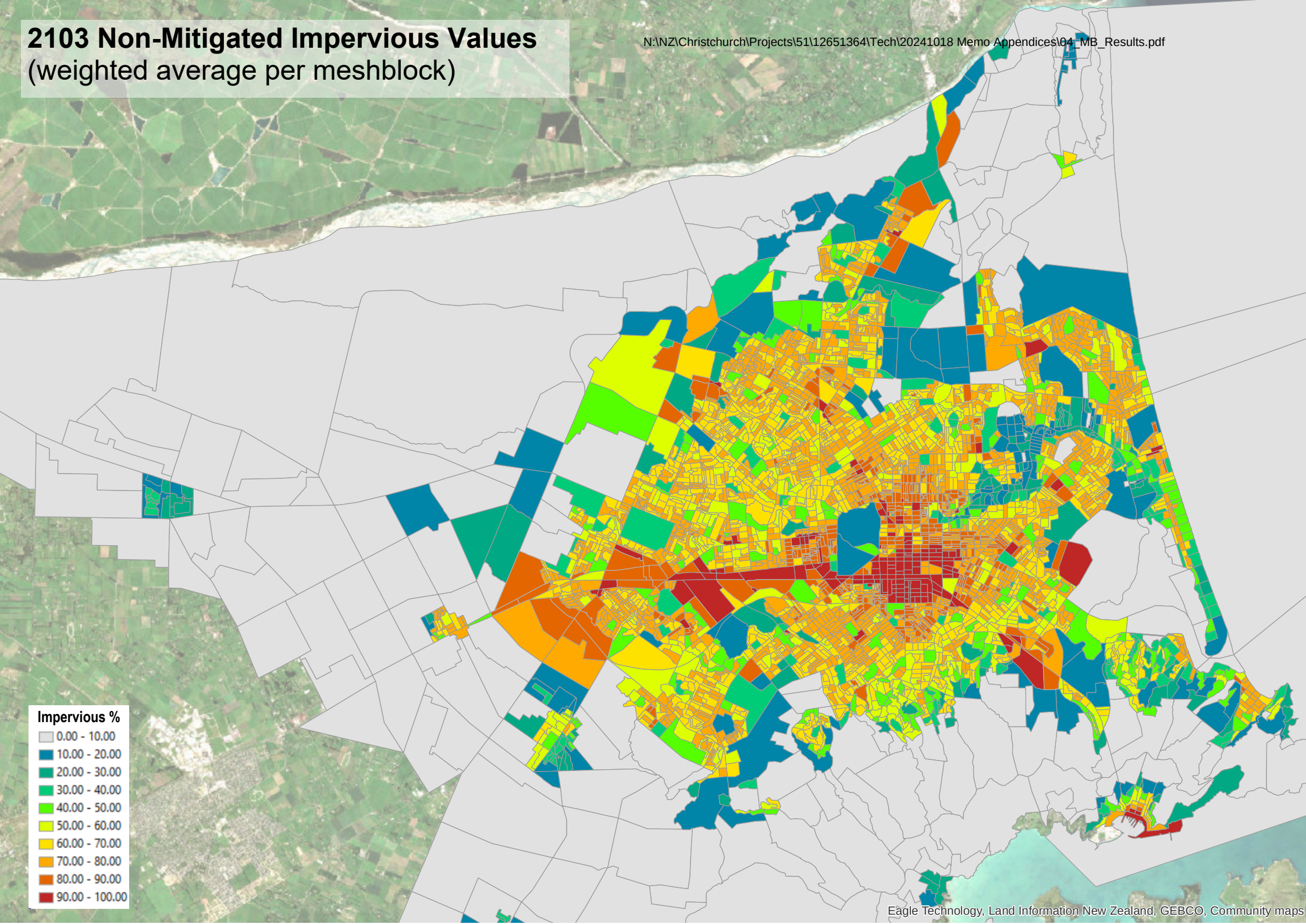
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2103 Non-Mitigated Impervious Values
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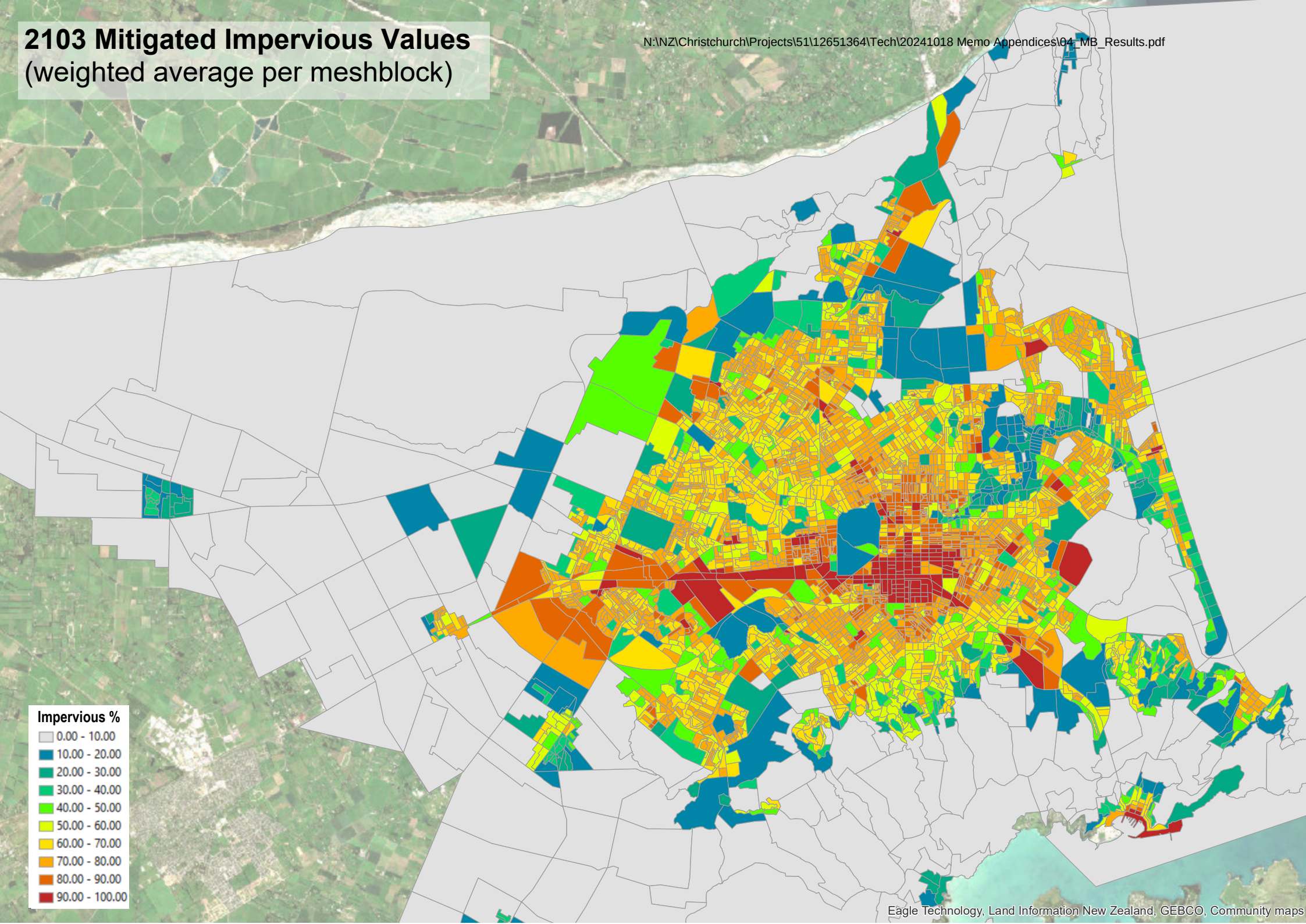
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2103 Mitigated Impervious Values
(weighted average per meshblock)

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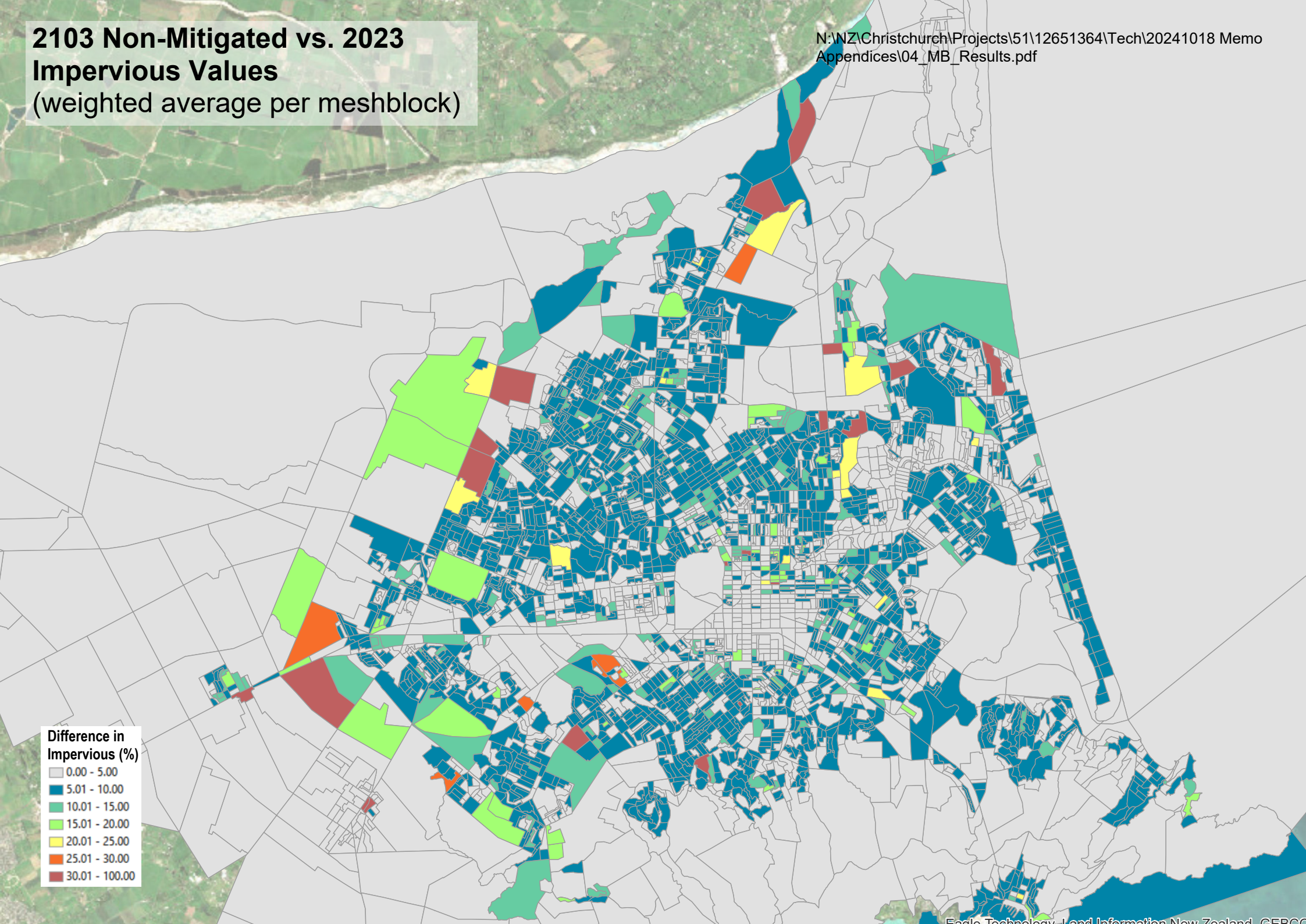


Eagle Technology, Land Information New Zealand, GEBCO, Community maps

2103 Non-Mitigated vs. 2023 Impervious Values

(weighted average per meshblock)

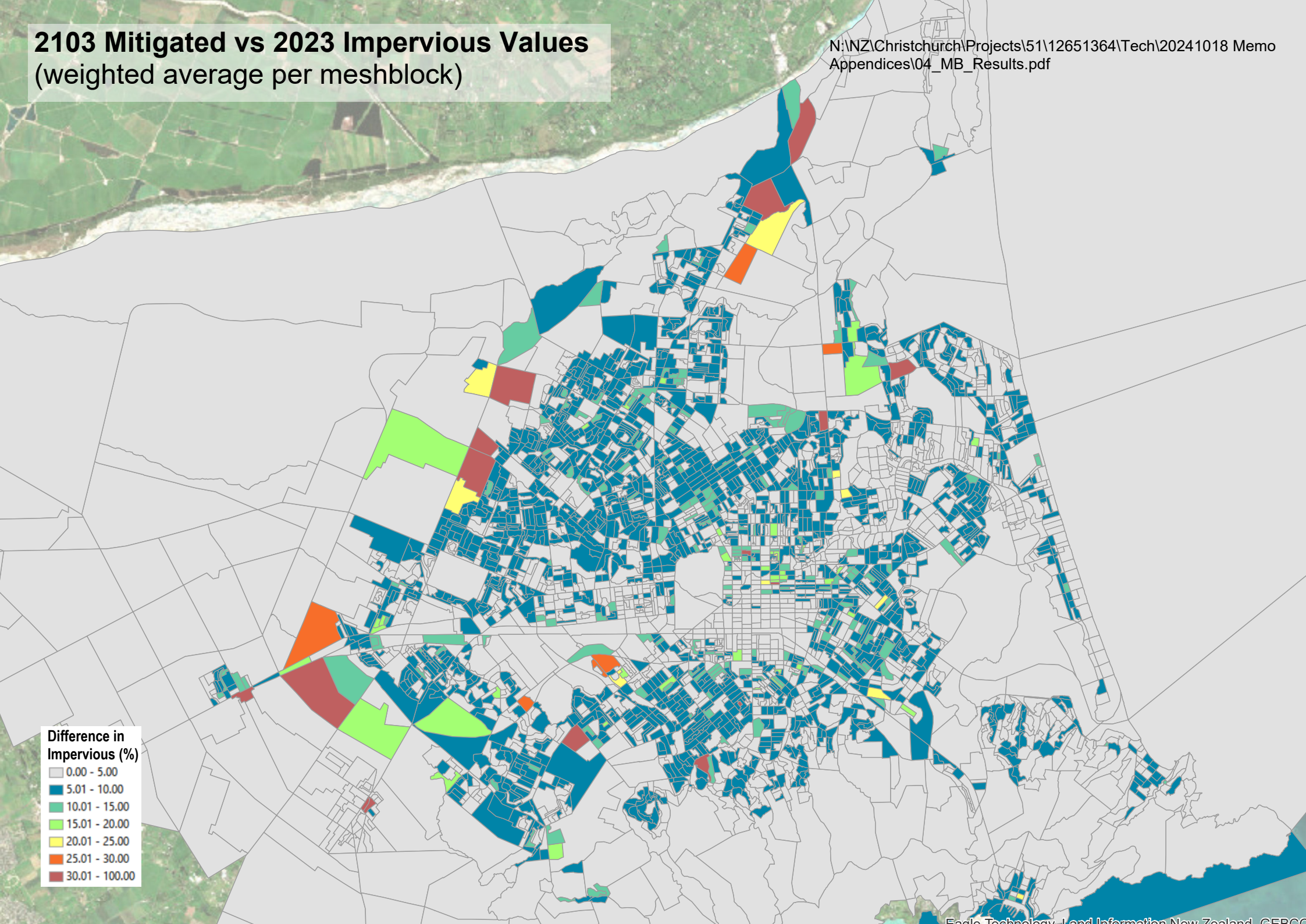
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Eagle Technology, Land Information New Zealand, GEBCO

2103 Mitigated vs 2023 Impervious Values
(weighted average per meshblock)

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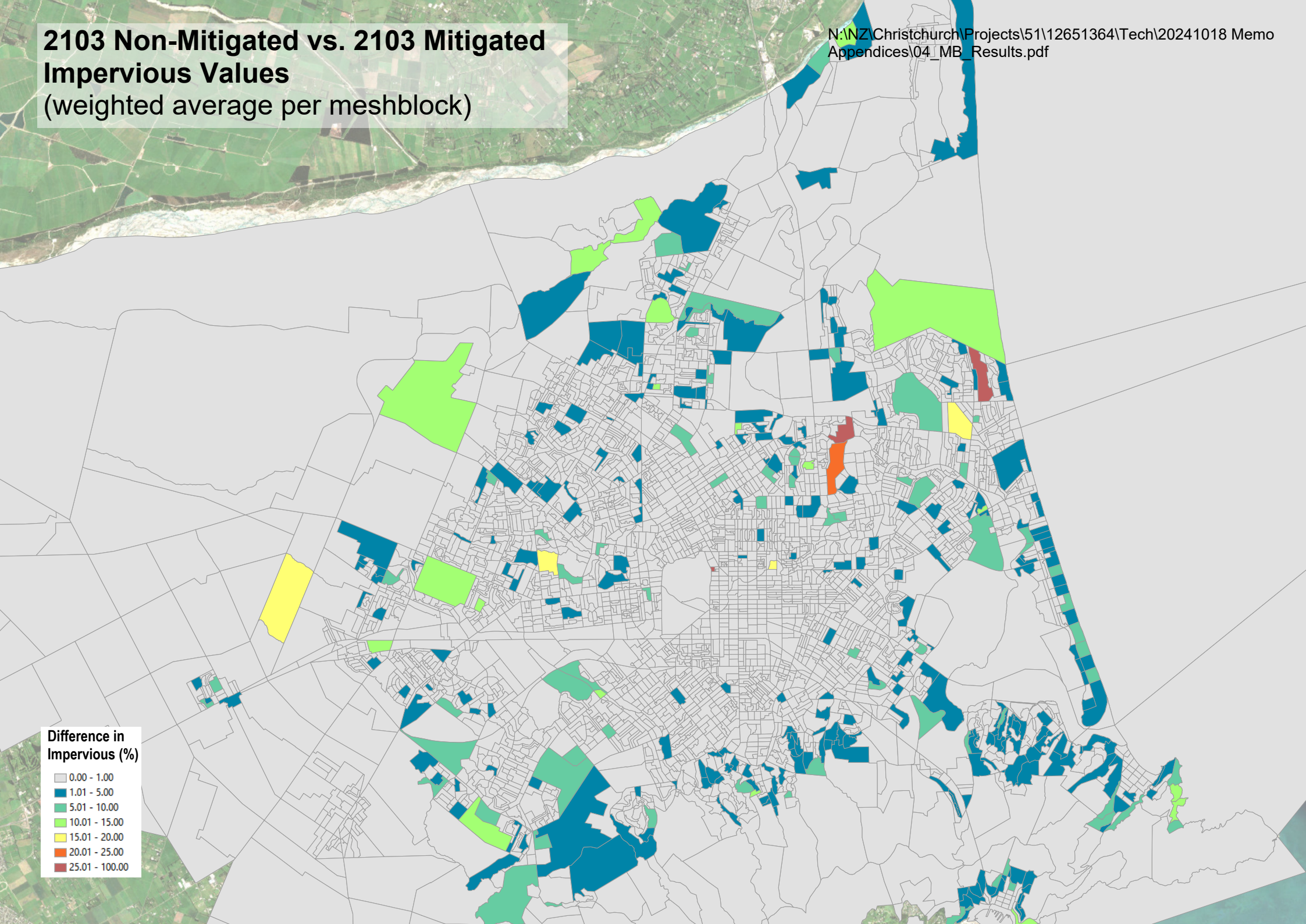


Eagle Technology, Land Information New Zealand, GEBCO

2103 Non-Mitigated vs. 2103 Mitigated Impervious Values

(weighted average per meshblock)

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Appendix P – Paparoa and Papanui Diversions report, 2025[48]

[48] GHD internal reference: \\projectsportal.ghd.com@SSL\DavWWWRoot\sites\pp02_10\ccccranfordswtreatme\ProjectDocs\Stage 1 - Design philosophy and Gap analysis\20250319 Diversion Status Report\Papanui_Papaora_Status_Report.docx

Paparoa and Papanui Diversions – Avon Model Status Report Addenda

04 April 2025

To	Peter Christensen, Noelle Evans (Christchurch City Council)	**Contact No.**	+64 03 378 0900
Copy to	Matt Jackson (Christchurch City Council), Roanna Purcaru (GHD), Helen Barclay (GHD)		
From	Madeline Howell, Tim Preston	**Project No.**	12640130
Project Name	CCC-Cranford SW Treatment - Stage 1		
Subject	Papanui and Paparoa Diversions Status Report: Avon River Hydraulic Model		

1. Introduction

Christchurch City Council (Council) engaged GHD to update the Avon Catchment model to provide a more accurate understanding of existing baseline performance of the Cranford Basin, which can be used in the future to evaluate the potential impacts of the proposed Cranford Basin Stormwater Facility. The work effort carried out included updating the model representation of the Paparoa and Papanui Diversions upstream of Cranford Basin, depth tolerance improvements, and providing new baseline model results.

Prior to this work effort, the Paparoa Street Diversion geometry (from upper Dudley Creek) had been predominantly inherited from “Upper Dudley” modelling completed circa 2014, with adjustments to post-earthquake levels based on recent survey work. However numerous inconsistencies in geometry were left unanswered (anomaly 27) and positioning of stoplogs and model flow split performance was not validated.

The geometry at the Papanui Diversion, which feeds into the Cranford Basin, was also not refined prior to this work effort. The geometry did not match as-built information and disregarded internal weirs designed to control baseflows and low flood flows. Disregarding these weirs was consistent with the principle that the model purpose is only for representing higher flood flow conditions in which the internal weirs will be drowned out and having only a minor influence on split flow rates. However, it is important for the Cranford Basin baseline modelling that the two outlet flow systems are modelled well including at low flows, to realistically determine the split of all flows at this diversion.

1.1 Purpose of this report

The purpose of this report is to describe the Paparoa Street Diversion and Papanui Diversion geometry updates within the Avon Catchment hydraulic model and to compare model results with hand calculations of the diversion hydraulics. This report will be appended to a new issue of the Avon Model Status Report (Rev 9).

1.2 Limitations

This report has been prepared by GHD for Christchurch City Council and may only be used and relied on by Christchurch City Council for the purpose agreed between GHD and Christchurch City Council as set out in section 1.1 of this report.

GHD otherwise disclaims responsibility to any person other than Christchurch City Council arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

GHD has prepared this report on the basis of information provided by Christchurch City Council and others who provided information to GHD (including Government authorities), which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report. GHD disclaims liability arising from any of the assumptions being incorrect.

Accessibility of documents

If this report is required to be accessible in any other format, this can be provided by GHD upon request and at an additional cost if necessary.

GHD has prepared the Avon Catchment MIKE Model ("Model") for, and for the benefit and sole use of, Christchurch City Council to support evaluation of flood hazards through the Avon Catchment and baseline performance of the Cranford Basin and must not be used for any other purpose or by any other person.

The Model is a representation only and does not reflect reality in every aspect. The Model contains simplified assumptions to derive a modelled outcome. The actual variables will inevitably be different to those used to prepare the Model. Accordingly, the outputs of the Model cannot be relied upon to represent actual conditions without due consideration of the inherent and expected inaccuracies. Such considerations are beyond GHD's scope.

The information, data and assumptions ("Inputs") used as inputs into the Model are from publicly available sources or provided by or on behalf of the Christchurch City Council, or by other consultants. GHD has not independently verified or checked Inputs beyond its agreed scope of work. GHD's scope of work does not include review or update of the Model as further Inputs becomes available.

The Model is limited by the mathematical rules and assumptions that are set out in the Report or included in the Model and by the software environment in which the Model is developed.

The Model is a customised model and not intended to be amended in any form or extracted to other software for amending. Any change made to the Model, other than by GHD, is undertaken on the express understanding that GHD is not responsible, and has no liability, for the changed Model including any outputs.

2. Diversion Structure Operational Setup Realities

2.1 Paparoa Diversion

On inquiry, Council records did not show how the Paparoa Diversion is configured presently. To proceed with the modelling, Council asked GHD to suggest how it should be setup, with the intention that they would then field validate this setup (including the likelihood of making physical changes to match this). Accordingly, GHD suggested the future operational setup should largely match the as-built condition (as detailed below), and Council accepted this.[1] This setup is reflected in the hydraulic modelling. It is not in scope for GHD to follow Council's work to field validate the physical condition and adjust if necessary to ensure it matches this agreed intended setup.

The Paparoa Diversion future operational setup is as follows:

– Objective: 100-150 L/s baseflow in Dudley Creek before starting to spillover to the diversion
– Earthquake shift: drop by 160 mm[2]
– Main stoplogs:

[1] Refer to Helen Barclay's email "Cranford – modelling of structures" 10/12/2024 and Peter Christensen's response 11/12/2024.
[2] This shift was determined based on the average difference between 2014 survey levels and design levels at four locations within the Paparoa diversion. Please refer to Stephen Bensberg's 1/5/2017 email "Paparoa diversion operation in Mar 2014?"

    - 250 mm low flow opening below
    - Top level is 100% full per as-built drawings, to match adjacent timber levels (100 mm above top of concrete)
- Low weir: remove the timbers and revert to bare concrete (location shown on the as-built drawings as level 17.95 m)
    - This means the top of the low weir will coincide with the bottom of the main opening (both 250 mm above the floor level)
- The high concrete weir retains its timber edging, and the top of the stop logs should match this level (shown on as-built drawings as 900 mm above floor level)
- The as-built drawing floor level is 17.70 m, and the current floor level, considering the post-earthquake (PostEQ) shift, is 17.54 m.

As-built drawings for the Paparoa Diversion structure are included in Appendix A and have been marked up to reflect this setup.

2.2 Papanui Diversion

Similar to the Paparoa Diversion, GHD suggested a future operational setup to Council. The future operational setup for the Papanui Diversion, which was accepted by Council[3] and is modelled, is summarized below:

- Objective: 50-100 L/s of baseflow in Papanui Creek before starting to spillover to the diversion
- Earthquake shift: drop by 130 mm[4]
- Stoplogs: per as-built drawings[5]
    - Perpendicular to incoming flow in Papanui Creek – 150 mm timber (on the top of concrete at 15.95 m as shown on the as-built drawings)
    - Parallel to incoming flow in Papanui Creek – 200 mm timber (on the top of concrete at 15.95 m as shown on the as-built drawings)
- Outlet manhole sluice gate: fully open
- Along Papanui Creek, the incoming concrete floor level is 0.05 m above the invert of the outgoing 450 mm diameter pipe (referencing levels 15.95 m and 15.90 m on the as-built drawings, with the assumption that the 450 mm pipe was installed with its invert level matching the previous timber lined drain invert and that its earthquake shift is the same as measured upstream at the structure[6])
- High level weir between crossing systems per as-built drawings (shown on the as-built drawings as 16.20 m)

Marked up as-built drawings reflecting this operational setup are included in Appendix B.

Similar to the Paparoa Diversion, it is not in GHD's scope to validate the current field condition, and it is Council's responsibility to ensure the field condition matches the as-built condition (and thus the model).

[3] Refer to Helen Barclay's email "Cranford – modelling of structures" 10/12/2024 and Peter Christensen's response 11/12/2024.
[4] This shift was selected based on comparing 2014 survey data at four locations within the Papanui Drain to the 1989 design levels. Refer to 9/12/2024 email "UDCD Structure level changes over time 2014 survey.xls" from Stephen Bensberg to Alexis Guidon.
[5] Photographic records suggest that the timber weirs at the Papanui Diversion may be missing. It is Council's responsibility to ensure the field condition matches the as-built condition.
[6] The design team is expecting that the invert level of the 450 mm diameter pipe may not be material to their design considerations at flow conditions important to the design. They are planning to evaluate this assumption after looking at the 3 mm/hr and 5 mm/hr model results. This evaluation might (or might not) motivate a survey request to validate the pipe invert level.

3. Performance Calculations

The reader may benefit from flipping ahead to schematics Figure 3 and Figure 4 in Section 4, Appendix A, and Appendix B to identify physical components of the Paparoa and Papanui Diversions. The following section describes hydraulic calculations completed for the Paparoa and Papanui Diversions for model setup validation.

3.1 Paparoa Diversion Rating Curves

Hand calculations were completed to develop rating curves for the Paparoa Diversion for model setup validation. The model results were compared to these rating curves to assess the model performance (refer section 5.1). The rating curves were developed by determining the flow through Dudley Creek and through the diversion for a range of upstream water levels. Table 1 summarizes the different flow conditions considered in the rating curve calculations for flow over/under the stop logs and the calculation approach. The rating curve calculations were completed assuming a free outfall (i.e. no backwater) for condition 1, and normal depth downstream of the diversion for conditions 2-4.

Table 1 Summary of analysed flow conditions for the Dudley outflow rating curve.

Condition	Description	Range of y_1 and y_3	Range of Dudley Outflow	Calculation Summary
1	Low flow under stop logs (y_2<250 mm), without submergence due to backwater conditions downstream	y_1 < 400 mm and y_3 < 250 mm	0 to 0.3 m^3/s	Assuming critical flow[7] immediately upstream of the stop logs (at y_2), calculated y_1 by applying energy balance across the contraction.
2	Orifice flow through stop logs (250 mm < y_2 < 900 mm)	400 mm < y_1 < 900 mm OR y_3 > 250 mm and y_1 < 900 mm	0.3 to 0.6 m^3/s	Calculated flow through the orifice using the standard orifice equation and considering submergence.
3	Weir flow over stop logs/high weir and orifice flow through the 250 mm opening	900 mm < y_1 < 1130 mm	0.6 to 1.0 m^3/s	Calculated flow through the orifice using the standard orifice equation and flow over the stop logs using the standard weir equation. Considered submergence of both the orifice and weir.
4	Orifice flow over the stop logs/high weir[8] and orifice flow through the low flow opening	y_1 > 1130 mm	> 1.0 m^3/s	Calculate the flow under and over the stop logs using the standard orifice equation, considering submergence.

Notes:

- *y_1 is equal to the flow depth immediately upstream of the concrete weir diversion structure.*
- *y_2 is equal to the flow depth within the diversion structure, immediately upstream of the stoplogs.*
- *y_3 is equal to the flow depth immediately downstream of the diversion structure.*
- *Approximate range of flow corresponds with the Dudley outflow rating curve.*

Flow into the diversion pipe was calculated by determining the capacity of the low weirs at lower flow rates (Q< 3 m^3/s), and the pipe capacity at higher flow rates (Q> 3 m^3/s). The flow capacity of the low weirs was calculated considering head loss through the diversion structure before the weirs. This head loss results in y_2 being lower than y_1[9], resulting in less flow over the parallel weir.[10] The weir flow was estimated by assuming no flow over the parallel weir for y_1 = 400 mm and that y_1=y_2 (full flow over the parallel weir) at y_1>= 700 mm. Between y_1 = 400 mm and y_2 = 700 mm, parallel weir flow was estimated to be between the 0 and full weir flow.

Pipe flow calculations were completed assuming a tailwater level equal to the downstream soffit of the diversion pipe[11]. The corresponding upstream water level was calculated by applying energy balance along the pipe.

[7] In free-flowing downstream conditions, critical flow will be caused by the sudden contraction in channel width associated with the low flow coming against the concrete weir.
[8] Where the water level exceeds the concrete lid of the diversion structure (at 1130 mm).
[9] Refer to as-builts in Appendix A for depiction of y1 and y2. y1 is equal to the flow depth immediately upstream of the concrete weir diversion structure, whereas y2 is equal to the flow depth within the diversion structure, immediately upstream of the stoplogs.
[10] Refer to Figure 3 for an illustration of the parallel low weir vs. the perpendicular low weir.
[11] This is likely to be a reasonable approximation to real conditions given that the pipe is only a flow constraint above 3 m^3/s, when it will be full or near full downstream, and that the flow area opens up into Papanui Drain structure at the downstream end of this pipe.

Calculations considered acceleration and entrance losses as flow enters the pipe, friction losses throughout the pipe, and exit losses. Pipe flow was compared to weir capacity for a range of upstream water levels and flow rates and was determined to limit the flow through the diversion above approximately 3 m^3/s (at $y_1 = y_2 = 1$ m depth). The resulting rating curve for the Paparoa Diversion is included in Figure 1. An Excel spreadsheet with the rating curve calculations accompanies this report.[12]

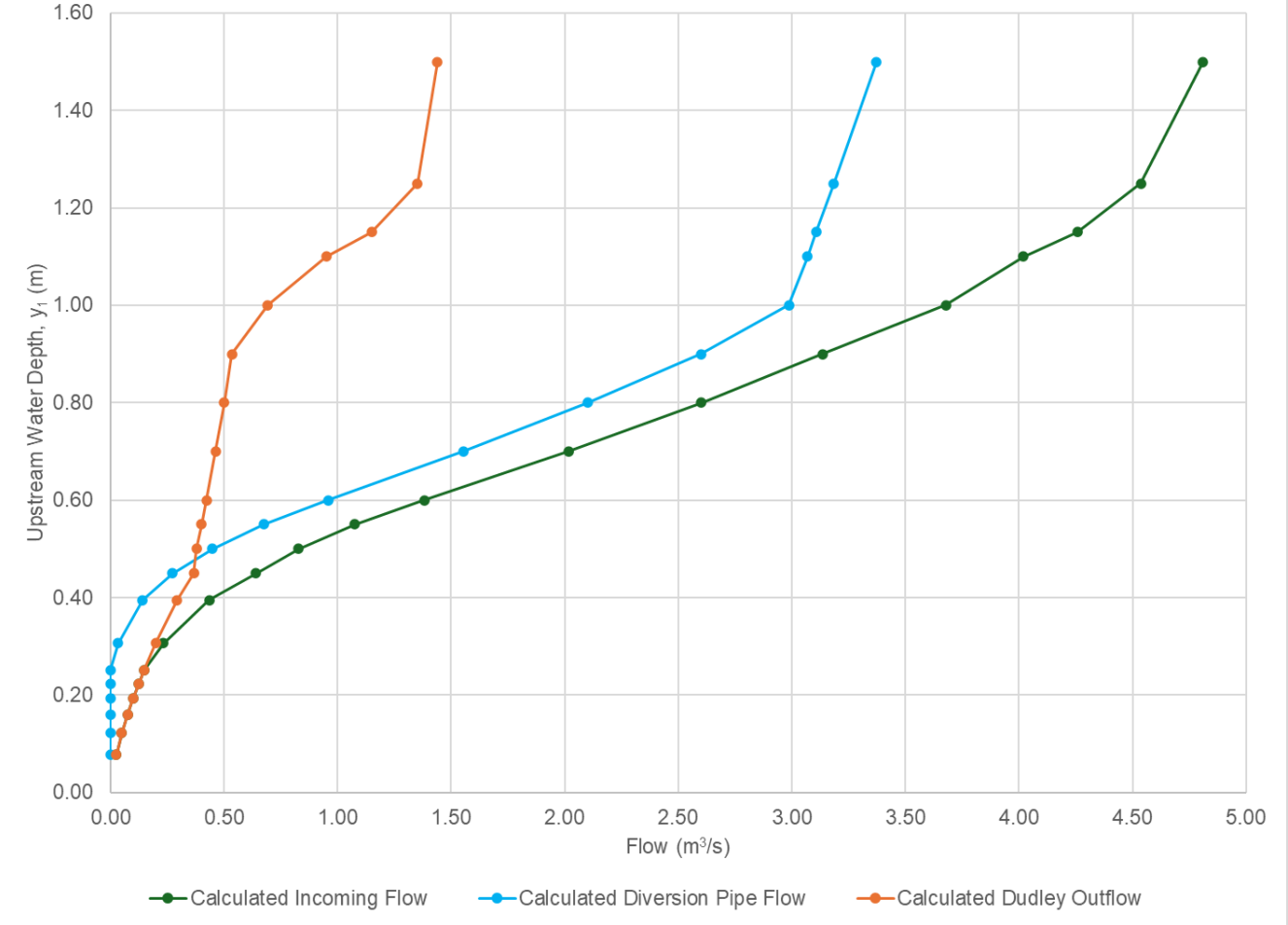


Figure 1 Paparoa Diversion rating curves

3.2 Paparoa Diversion Tabulated Structure

The Paparoa Diversion performance calculations, which did not consider potential backwater conditions, were extended to create a tabulated structure to represent flow under and over the stoplogs in the model (see Section 4.1 for discussion of model setup). The tabulated structure was created using the same calculation approach as the rating curve, summarized in Table 1 for the four analysed flow conditions. The tabulated structure considers both upstream and downstream water levels. Thus, flow under and over the stoplogs was calculated for a matrix of upstream and downstream water levels, with upstream and downstream water depths ranging from 0 to 2 m.

Please refer to Appendix C for detailed calculations and the resulting tabulated structure.[13] Note that the rating curve was developed for downstream normal depth, so result into various columns of the tabulated structure. Rating curve values extend approximately diagonally through the tabulated structure from the zero origin to approximately y_1=2 m / y_3=0.4 m / Q=2.3 m^3/s.

The tabulated structure in Appendix C includes 0 m^3/s flow when the upstream and downstream water levels are equal.

[12] N:\NZ\Christchurch\Projects\51\12640130\Tech\Hydro\20241211 Paparoa Papanui Flow Calcs MHowell\20250326_Paparoa_Diversion_RatingCurve.xlsx
[13] N:\NZ\Christchurch\Projects\51\12640130\Tech\Modelling\20241220 Paparoa Tabulated Structure MHowell\20241220_Paparoa_Diversion_TabulatedStructures.xlsx

The tabulated structure also includes backflow over and under the stoplogs (and high weir) when the downstream water level exceeds the upstream water level. This reverse flow was estimated by assuming it is the opposite flow value of the positive flow result for the same head difference. For example, flow through the tabulated structure at y_1= 1 m and y_3 = 0.5 m is 0.569 m^3/s, so the flow for y_1= 0.5 m and y_3 =1 m was set to -0.569 m^3/s. Minimal reverse flow is expected through the tabulated structure in the model, so this assumption is not expected to significantly impact model results.

3.3 Papanui Diversion

Hand calculations were also completed for the Papanui Diversion to validate model performance. Flow that continues down Papanui Creek was calculated by assuming an absence of backwater, which means the flow will be at normal depth within the downstream 450 mm pipe.[14] Upstream from there, acceleration and head loss into the 450 mm pipe entrance was considered and energy balance was applied (considering friction, entrance and exit head losses) through the 600 mm pipe to determine the water level at the upstream end of the 600 mm pipe. This approach assumes negligible incoming velocity in the box upstream of the 600 mm pipe.

Flow over the low diversion weirs was calculated using the standard weir equation, neglecting downstream submergence.[15] Low weir calculations were completed assuming that the water level upstream of the perpendicular weir is equal[16] to the water level upstream of the parallel weir.

The results of these hydraulic calculations are included in Table 2. These results indicate a maximum flow rate of approximately 27 L/s within Papanui Creek before it starts spilling over the lowest low weir (upstream water level=15.97). The incoming Papanui Drain flow rate at the Papanui Diversion prior to the water level overtopping at the crest level of the higher diversion weir (into the southern box of the culvert) is estimated to be approximately 150 L/s.

Flow calculations were also completed for a higher upstream water level of 16.16 m for reference. Note that at this water level, the high weir would overtop. Flow over this high weir was not estimated, as it depends not only on the water levels within the Papanui Drain at the diversion structure, but also on the water levels within the Dudley Diversion, upstream of the south barrel of the diversion culvert. It is also possible that the water levels within the Dudley Diversion exceed water levels in Papanui Drain, in which case the high weir will convey flow toward the Papanui Drain. Therefore, flow calculations for water levels above 16.07 m are left as inconclusive, and no model validation is attempted above this water level condition.

Table 2 Papanui Diversion performance calculation results

Papanui Drain Upstream Water Level (m)	Flow (m^3/s)			
	450 mm Pipe	Low Weirs	High Weir	Inflow
15.82	0.000	0.000	0.000	0.000
15.97	0.027	0.000	0.000	0.023
16.07	0.053	0.097	0.000	0.150
16.16	0.074	0.331	unknown	unknown

In addition to the results presented in Table 2, the 450 mm pipe and low weir calculations were used to develop rating curves that compares upstream water level within Papanui Drain to flow over the low weirs and in the 450 mm pipe. This rating curve is included in Figure 2. This rating curve does not consider impacts of downstream submergence, so the calculated flow rates at the higher upstream water levels are likely overestimates. Additionally, this rating curve is approximate above the upper weir level of 16.07 m.

Excel spreadsheets that include the Papanui performance calculations are included with this report,[17] and as-built drawings are included in Appendix B.

[14] Note the uncertainty in 450 mm pipe invert levels due to lack of survey data, which would affect the normal depth in this pipe, and consequently, the flow split between the 600 mm pipe and the diversion weirs.
[15] This is likely a good assumption upstream Papanui Drain water levels are below the high weir level (16.07), since normal depth within the culvert at this flow rate is well below the low weir elevations. When water levels exceed the high weir, this is likely a poor assumption, and the rating curve likely overpredicts low weir flow.
[16] This is a poor assumption for a minor range of flows at the lowest 'start to flow' condition, which is hydraulically intractable for hand calculations, however that rapidly becomes a good assumption as flows increase and backwater from the 450 mm pipe takes effect
[17] N:\NZ\Christchurch\Projects\51\12640130\Tech\Hydro\20241211 Paparoa Papanui Flow Calcs MHowell\20250321_Papanui_Flow_Calcs.xlsx

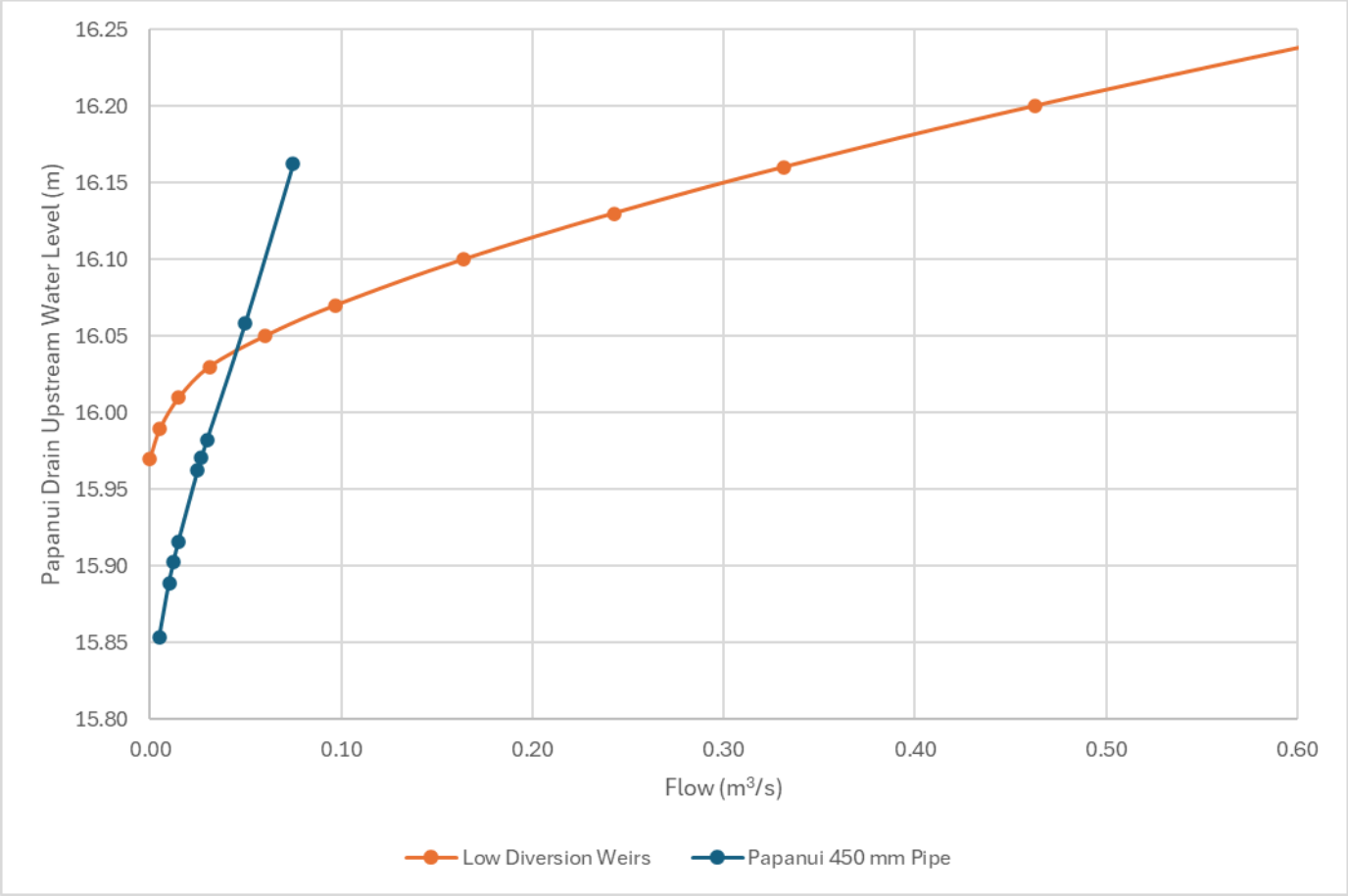


Figure 2 ***Rating curves for the Papanui Diversion low weirs and 450 mm pipe.***

4. Model Setup

4.1 Paparoa Diversion

A schematic representing the model set up for the Paparoa Diversion is included in Figure 3. Figure 3 shows that Dudley Creek and the Paparoa Diversion structures were modelled in MIKE 11 (M11), while the 1350 mm diversion pipe was modelled in MIKE Urban (MU). The Paparoa Diversion is modelled with a flow split between the avon.dudley10 branch (chainage 7760) and the avon.paparoapipe branch (chainage 0). The low diversion weir was modelled as a broad crested weir within the avon.paparoapipe branch[18] (chainage 15), with widths and levels obtained from the as-built drawings. The flow under and over the stop logs and over the high weir were modelled in the avon.dudley10 branch with a tabulated structure that considers both upstream and downstream water levels.

Invert levels and geometry for the Paparoa Diversion, weirs, upstream culvert, and downstream culvert, were obtained from as-built drawings included in Appendix A but dropped by 160 mm (to represent the post-earthquake shift in levels, as discussed in Section 2.

[18] The M11 branch has significant fictional (large) geometry to perform within constraints of the modelling software, including a fictional length of 30 m for stability reasons. The only desired hydraulic behaviour of this branch is to represent the two low weirs and connect into the MU pipe.
While the low weir was modelled in M11 as one structure, it was calculated in two parts in the performance calculations due to different upstream water levels at low to moderate flows.
The broad crested weir at avon.paparoapipe chainage 15 is modelled with varying widths; at level 17.79 m the width matches the width of the low weirs, whereas at level 18.45 m the width widens to represent both the low and high weirs. At water levels above 18.45 m, the 1350 mm diversion pipe is expected to control the hydraulics, so the wider width is not expected to significantly impact model results but may impact model stability.

This model configuration simplified the complex water levels at y_1 and y_2, representing both with a single water level and not representing the water level differences at low and medium flows. At higher flood flows the model works better with real water levels by then drowning out the low weirs and flow contraction below 250 mm depth.

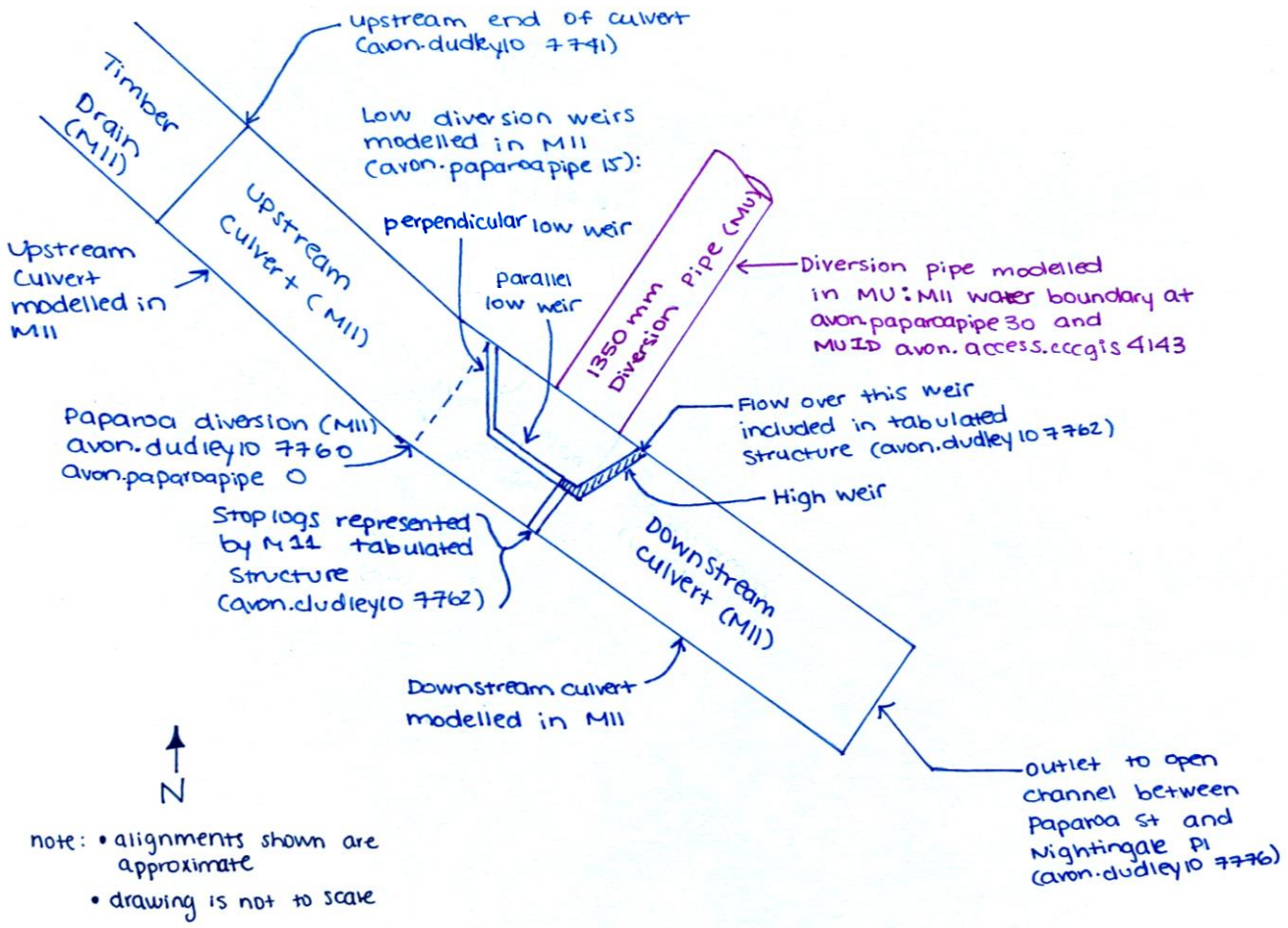


Figure 3 ***Schematic of Paparoa Diversion model setup.***

4.2 Papanui Diversion

A schematic representing the model setup of the Papanui Diversion is shown in Figure 4. Geometry for the Papanui diversion, including the Papanui Drain at the diversion, diversion weirs, and culverts, was obtained from as-built drawings included in Appendix B. Note that an earthquake drop of 130 mm was applied to all as-built levels, as discussed in Section 2.2.

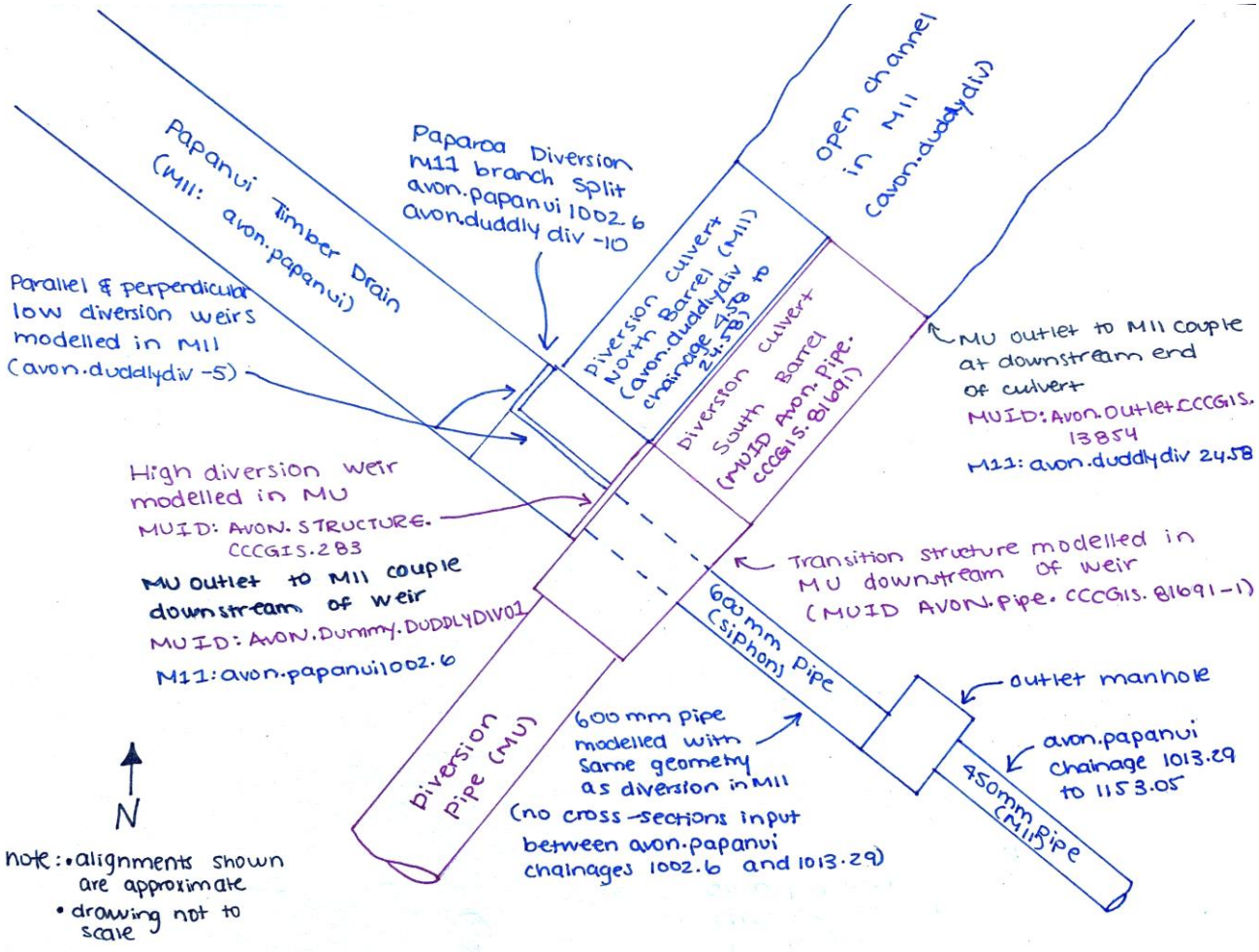


Figure 4 ***Schematic of Papanui Diversion model setup.***

As shown in Figure 4, the Papanui Drain upstream, downstream, and through the diversion was modelled in M11 (avon.papanui branch). The avon.papanui branch features a closed cross-section at the diversion location (avon.papanui 1002.6) with geometry from the as-built drawings. The invert level at this location was set to 15.5 m, which is a fictional value selected to be lower than the downstream 450 mm pipe (to make sure the 450 mm pipe controls hydraulics) and high enough to be within the 0.5 m deep fictional slot of the 450 mm section at chainage 1013[19]. The next cross-section in the avon.papanui branch is located at the upstream end of the 450 mm pipe, with a closed circular cross-section geometry. The invert level and cross-sectional geometry of the 600 mm pipe was therefore not represented in the model; the area is instead modelled as if transitional between the cross sections at 1002.6 and 1013.29[20] to avoid model instabilities. The invert of the downstream 450 mm pipe is significantly higher than the 600 m pipe invert, therefore, the 450 mm pipe is expected to control the hydraulics. The diameter of the 450 mm pipe was determined from Council GIS data. The upstream invert level was assumed to be equal to the timber drain level shown on the as-built drawings, earthquake shifted, which was later replaced by the 450 mm diameter pipe. The downstream invert level was estimated based on previous model geometry for this pipe, applying the same EQ shift as the diversion structure, as survey data was not available for the pipe. The uncertainty in the 450 mm pipe invert levels leads to uncertainty in the modelled flow split at this location.

Figure 4 shows that the lower diversion weirs along with the north barrel of the diversion culvert and the open channel were modelled in M11. Geometry and invert levels for the weirs and culvert were obtained from the as-built drawings (EQ shifted). Note that the low diversion weirs were modelled as broad crested weirs within M11.

Figure 4 shows that the incoming diversion pipe, upper diversion weir, and south culvert barrel were modelled in MU. Geometry and invert levels for these components were obtained from as-built drawings (EQ-shifted). The

[19] The invert was modelled to be within this fictional slot to avoid flow generation in the model.
[20] There could be more realistic geometry trialed, but there is natural tension between realistic geometry and model stability.

upper diversion weir conveys flow between the incoming diversion pipe and incoming Papanui Drain. A modelled dummy MU link is used to contain this weir. The dummy pipe was included for model practical requirements and stability, and its outlet is connected to the Papanui Diversion through a MU outlet to M11 couple. The south diversion culvert barrel discharges through a second MU outlet to M11 couple to the open channel.

5. Evaluation of Model Results

5.1 Modelled peak flow conditions

The modelled peak flow conditions at the Paparoa and Papanui Diversions were analysed to check for flow conservation at the diversions and are discussed in the following section. Analysis of results was completed for the 2023 50-year 12-hour storm event.

In the peaks of this 12-hr event, water level in the Paparoa Diversion reaches depth a 0.95 m (level of 18.49 m, which is marginally above the high weir at 18.44), but water levels fluctuate with the high weir level, exceeding it (and then receding) in four separate periods within this event. Model stability here is evidently imperfect. The high weir is effective at holding the peak water levels around 18.5 m, but this level is nearly coincident with the real life (and modelled) instability of the 1350 mm diameter diversion pipe reaching its soffit levels. The protracted conjunction of these conditions, results in moderately noisy model results near peak flows. Peak inflows to and outflows from the Paparoa Diversion (avon.dudley10 7760) are summarized in **Error! Reference source not found.**.

Table 3 Modelled peak inflows and outflows at the Paparoa Diversion

Flow (m^3/s)	Dudley Creek	1350 mm Diversion Pipe	450 mm Pipe (MU)
Diversion Structure Inflow	2.80	N/A	0.15
Diversion Structure Outflows	0.45	2.55	N/A

Outflow from the Paparoa Diversion to the diversion pipe peaks very briefly at 2.60 m^3/s just prior to the first overtopping of the high weir, but the highest sustained outflow rate is around 2.0 m^3/s. Timeseries flow patterns in M11 at avon.paparoapipe 25 and in MU immediately downstream suggest significant failure to conserve flows in the model.[21] The problem may be resolved further downstream in the MU system, but this is recommended for further investigation. Flow conservation (total inflow = total outflow) should also be investigated at the diversion (avon.dudley10 7760) considering secondary inflows from a 450 mm dia MU pipe running from Papanui Road along Paparoa Street.

In the Papanui Diversion, the water level reaches 16.59 m (well above the high weir at 16.07 m). Flow over the high weir always goes from the 1350 diversion pipe into Papanui Drain and the highest flow over the weir is 0.45 m^3/s. Peak inflows and outflows to the Papanui Drain and the 1350 diversion pipe at the Papanui Diversion are summarized in **Error! Reference source not found.**.

Table 4 Papanui Drain and the Diversion Pipe peak inflows and outflows at the Papanui Diversion

	Papanui Drain	Diversion Pipe
Upstream inflow	1.90	2.08
High weir flow	-0.45[22]	0.45
Box-culvert outflow	1.55	2.44
450 mm pipe outflow	0.09	N/A

Note that the Diversion Pipe flows peak much later in the event than the Papanui Drain peak, which is why the tabulated peak inflows exceed peak outflows.

Flow conservation along the 1350 diversion pipe is also recommended for investigation. In the later hours of the model result (after time 12 hrs, with rainfall stopped and local inflows negligible), the M11 flows at

[21] i.e. degeneration of 5 m^3/s over 4 hrs in M11 but apparent regeneration of flows in downstream MU adjacent
[22] Outflow from the diversion pipe over the high weir into Papanui Drain

avon.paparoapipe chainage 5 are reasonably stable at 2.3 m^3/s whereas the MU flows downstream trend toward 2.05 m^3/s (flow degeneration). This flow condition is steady and sustained for approximately 3 hours at the end of the model result and the difference in flow rates is unexplained. Earlier in the model result, associated with timing of peak rainfall, local sump inflows (approx. 0.2 m^3/s at peak) evidently create somewhat higher flow rates downstream than upstream, as expected.

5.2 Paparoa Diversion

The model results for the 2023 50-year Average Recurrence Interval (ARI) 12-hour storm event[23] at the Paparoa Diversion are compared to the rating curves in Figure 5.[24]

The model results indicate that flow continuing down Dudley Creek (in red), match the calculated rating curves closely at $y_1 < 0.4$ m. For $y_1 > 0.4$ m, the model results show less flow under (and over) the stoplogs than predicted by the rating curve, due to backwater conditions downstream. As discussed in Section 3.1, the rating curve for flow under/over the stoplogs assumes normal depth downstream conditions, whereas model results show higher downstream water levels than the normal depth, likely due to the constrained channel dimensions between Paparoa Street and Nightingale Place.

The modelled flow over the lower diversion weirs (in blue) is higher than predicted for upstream water depths of less than approximately 0.6 m. For $y_1 <= 0.6$ m, the rating curve was developed considering head loss as flow contracts through the diversion structure below depth 0.25 m, causing $y_2 < y_1$. The model does not consider the difference between y_1 and y_2, with y_1 applied at both low weirs resulting in higher flow over the parallel low weir. For $y_1 > 0.6$ m, the model results in less flow over the low weirs than calculated. Differences between the rating curve and modelled results may be partly due to the different calculation approaches. The rating curve was developed using the standard weir equation, whereas M11 calculates flow across broad-crested weirs by assuming critical depth at the weir and applying head loss factors between upstream and downstream cross-sections.

Model results show backwater effects at the low diversion weirs at lower flow rates and water levels than predicted by the performance calculations. Model results start to show backwater conditions on the low weirs start at approximately y_1= 0.8 m and Q = 1.9 m^3/s. The hand calculations estimated the backwater condition at approximately y_1= 0.9 m and Q = 2.6 m^3/s. The differences between the low diversion weirs rating curve and the model results for y_1> 0.8 m are likely caused by head loss at twelve modelled manholes along the diversion pipe. From previous analyses at Riccarton and in Heathcote we are aware that the model is likely significantly overpredicting head loss (underpredicting flow) throughout this pipe due to these manhole head losses. The rating curve does not consider any manhole head loss, so likely marginally underpredicts head loss across this pipe. Work is ongoing in the Heathcote catchment to determine a better comprehensive modelling approach to node head loss. Site specific improvements to node head loss along this pipeline could reasonably be achieved ahead of this comprehensive work if Council seeks better outcomes.

The modelling incoming flow rates (in green) include inflow from Dudley Creek[25] and a 450 mm pipe modelled in MU[26], which outflows to Dudley Creek at the Paparoa Diversion. The calculated flow rates for each water surface elevation were taken as the sum of the calculated Dudley Creek outflow and the calculated Diversion Pipe outflow. The differences between the modelled and calculated results are the result of differences between modelled and calculated Dudley Creek flow and Diversion Pipe flow, discussed above. The modelled incoming flow rates were compared to the modelled outflow (Diversion Pipe outflow plus Dudley Creek outflow) to assess flow continuity. Results indicated more inflow than outflow was modelled near the peak of the storm event[27], with differences of up to approximately 0.36 m^3/s. It is recommended that this flow continuity be investigated further.

[23] N:\NZ\Christchurch\Water\Modelling\12640130_Cranford\AVON\03Reslt\2023\50ARI\v117c_50R07T_12hrUp\M11\v117c_Avon_2023_0p00SLR_50R07T_12hr_Up.res11 and v117c_Avon_2023_0p00SLR_50R07T_12hr_Up.prf

[24] Upstream water surface elevation taken at avon.dudley10 7762, diversion flow taken at avon.paparoapipe 15, and dudley creek downstream flow taken at avon.dudley10 7759.7. Figure 5 includes the results from the start of the simulation up to the peak of the storm event at Paparoa Diversion (1/1/1900 8:32). Receding limb model results have been excluded for clarity due to instabilities +/- 50 mm.

[25] AVON.DUDLEY10 7759.7

[26] AVON.Pipe.CCCGIS.58078

[27] Starting at around 1/1/1900 7:00

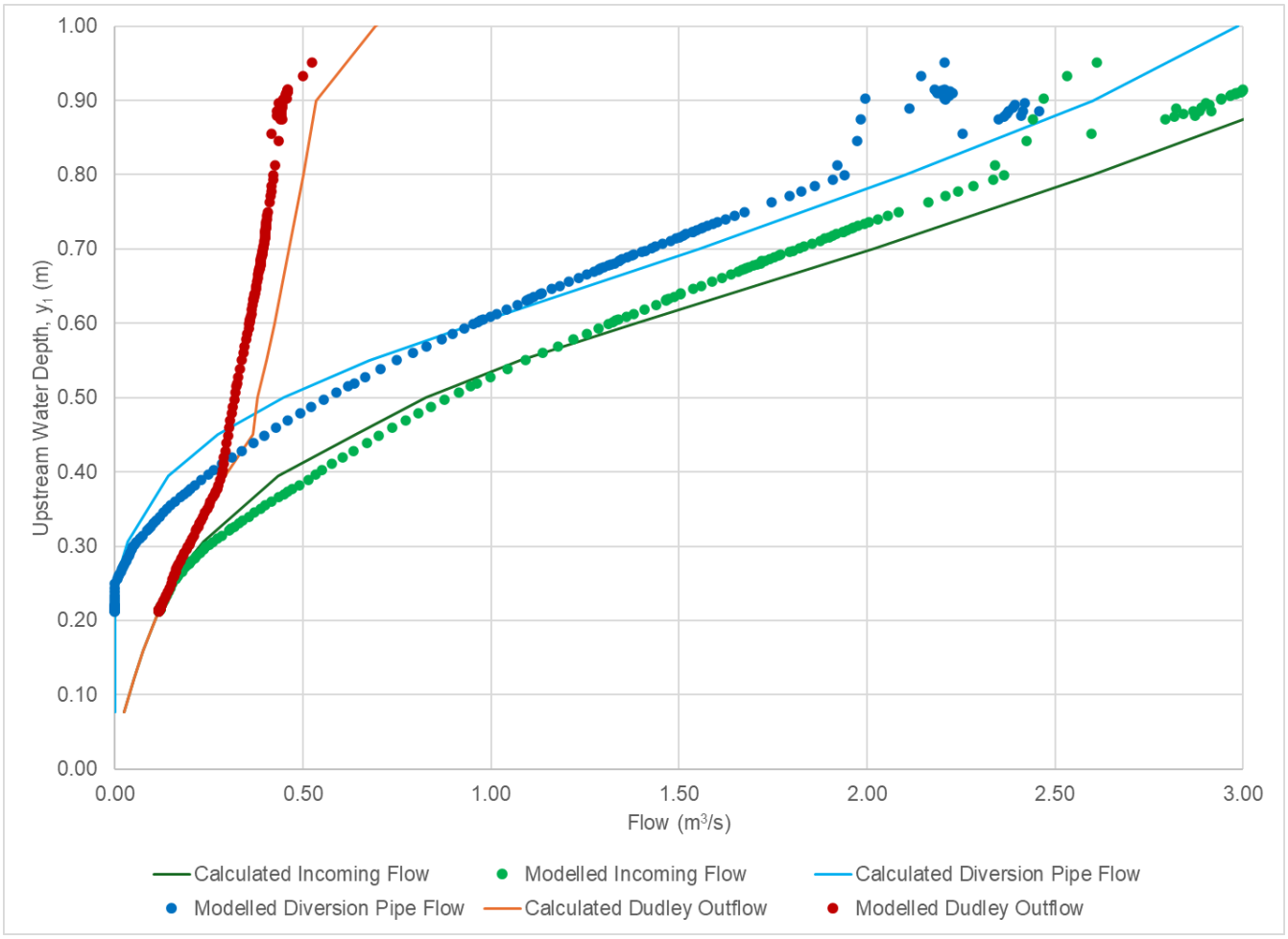


Figure 5 ***Comparison of model results for the 2023, 50 ARI, 12-hour storm event to developed rating curves at the Paparoa Diversion***

5.3 Papanui Diversion

A comparison of the calculated flow rates (discussed in Section 3.3) to the modelled flow rates[28] at the Papanui Diversion are included in Table 5. The results of these hand calculations indicate a maximum flow rate of approximately 27 L/s would be expected within Papanui Creek before it starts spilling into the diversion. Model results indicate a lower maximum flow rate of approximately 18 L/s. The hand calculations estimated a maximum flow rate of approximately 150 L/s within the Papanui Drain prior to the water level overtopping the crest level of the higher diversion weir (into the southern box of the culvert), compared with the modelled 137 L/s.

The results within Table 5 show lower modelled flow rates within the 450 mm pipe than calculated. These differences may partly be attributed to the fact that the 600 mm pipe (siphon) was approximated in the M11 model with transitional closed-cross-section geometry, between the rectangular geometry at the upstream end of the Papanui Diversion and the 450 mm pipe geometry. Additionally, the model does not capture the low invert levels of the 600 mm pipe; the model includes a fictional invert level of 15.5 m at the upstream end of the diversion and slopes linearly to the 450 mm pipe invert level of 15.77 m. This configuration was modelled because the low invert levels and small cross-section of the 600 mm pipe were found to lead to model instabilities that prevented the model from running. This more stable configuration will overestimate the head loss associated with the 600 mm pipe, as the model will overpredict velocities and friction losses within at least the downstream portion of this transitional cross-section geometry as model invert levels are linear trending toward the 450 mm pipe invert level, rather than being 1.3 m below this invert.

[28] N:\NZ\Christchurch\Water\Modelling\12640130_Cranford\AVON\03Reslt\2023\50ARI\v117c_50R07T_12hrUp\M11\v117c_Avon_2023_0p00SLR_50R07T_12hr_Up.res11 and N:\NZ\Christchurch\Water\Modelling\12640130_Cranford\AVON\02Setup\2023\50ARI\v117c_50R07T_12hrUp\MFLOOD\MU\v117c_0SLR_50R7T_12hUp.PRF

Table 5 Comparison of calculated vs. modelled flow rates at the Papanui Diversion

Papanui Drain Upstream Water Level (m)	Flow (m^3/s)			
	450 mm Pipe	Low Weirs	High Weir	Inflow
Calculated				
15.82	0.000	0.000	0.000	0.000
15.97	0.027	0.000	0.000	0.027
16.07	0.053	0.097	0.000	0.150
16.16	0.074	0.331	N/A	N/A
Modelled				
15.82	0.000	0.000	0.000	0
15.97	0.018	0.000	0.000	0.018
16.07	0.038	0.099	0.000	0.137
16.16	0.057	0.300	0.096	0.453

In addition to the tabulated comparison in Table 5, the model results[29] for the low weirs and 450 mm pipe were compared to the rating curves developed in Section 3.3. This comparison is included in Figure 6, and indicate that the model results in less flow through the 450 mm pipe than predicted, due to the higher model head loss discussed previously.

The low weir model results closely match the calculated results at lower flow rates ($Q<0.42$ m^3/s) The differences between the model results and rating curve at these lower flow rates can be attributed to the different calculation approaches. Like the Paparoa Diversion low weirs, the rating curve calculations are based on the standard weir equation, whereas M11 models the weir assuming critical depth at the weir and applying head loss between upstream and downstream cross-sections. At higher flow rates ($Q>0.42$ m^3/s), the model results show less flow over the diversion weirs due to backwater submergence, which was not considered in the hand calculations.

[29] N:\NZ\Christchurch\Water\Modelling\12640130_Cranford\AVON\03Reslt\2023\50ARI\v117c_50R07T_12hrUp\M11\v117c_Avon_2023_0p00SLR_50R07T_12hr_Up.res11

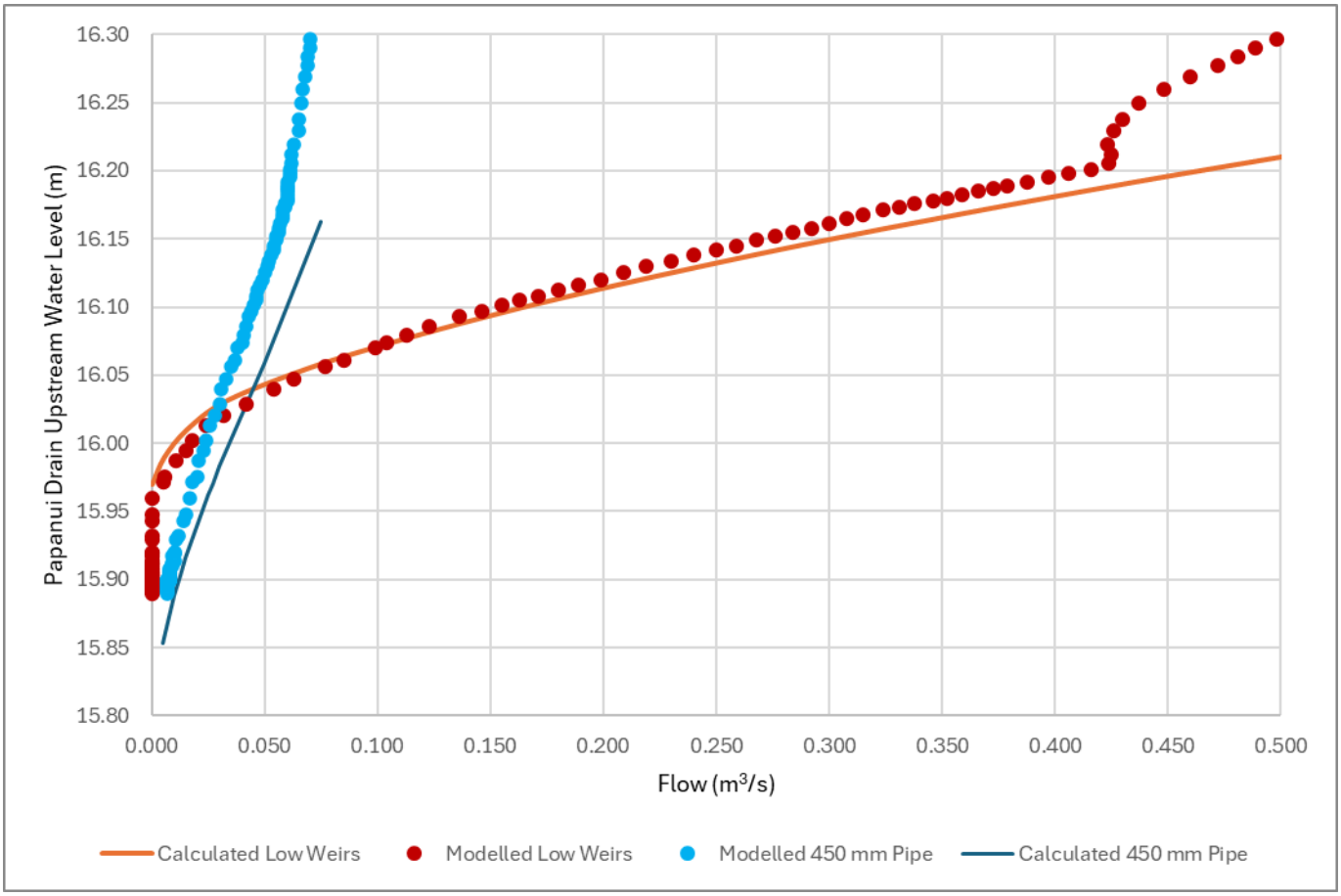


Figure 6 ***Comparison of modelled vs. calculated low weirs and 450 mm pipe flow at the Papanui Diversion***

Key points of the Cranford project team review will be whether the flow split into the 450 mm diameter pipe is modelled sufficiently well, with the next potential improvements likely to be conversion to MU modelling. Also of interest may be the sensitivity of both calculation and model to the significant (perhaps +/- 100 mm) uncertainty in the invert level of the 450 mm diameter pipe. This uncertainty is due to a lack of as-built drawings or survey records for the pipe.

6. Future Model Improvements

6.1 Paparoa Diversion

Future improvements to the hydraulic modelling of the Paparoa Diversion may include the following:

– Modify the cross-section geometry of avon.dudley10 chainage 7759.4 to match avon.paparoa pipe chainage 0 (at the Paparoa Diversion flow split) to help create a more robust model. The modelled geometry is similar, but the avona.paparoapipe chainage 0 is represented with a closed irregular section[30] compared to the avon.papanui chainage 7759.4, which is modelled with a closed rectangular section.

– Investigate flow continuity at the following locations, as model results suggest failure in flow conservation (see Section 5.1 for further discussion):

- At the Paparoa Diversion (avon.dudley10 7760) considering M11 and MU inflows.
- Within the 1350 diversion pipe in M11 avon.paparoapipe branch
- Within the 1350 diversion pipe in MU, downstream of the avon.paparoapipe branch.

[30] The bottom of the section is skewed to improve model stability.

– Model results show significant instability at the 1350 diversion pipe (avon.paparoapipe branch). Flow bifurcations are tricky areas for model stability and this instability is probably also associated with the water levels reaching the pipe soffit (see Section 5.1). Further investigation of model stability in this area is recommended, such as analysing flow continuity at various depth and flow regimes and investigating model results from other runs. A potential factor that may be leading to instabilities is the rapid reduction in M11 conveyance as flow converges from the unrealistically large rectangular cross-sections around the low weir to the 1350 mm closed pipe (between avon.papaorapipe chainages 20 and 30). A potential stability improvement may include lengthening the distance between the rectangular and 1350 mm closed cross-sections. Another potential stability improvement could include matching invert levels on either side of the MU-M11 coupling.
– Modelling the Paparoa Diversion low weir (avon.paparoapipe 15) with only the 3.11m width of the low weirs (and deleting the extra width representing the high weir) would improve model realism and may improve model stability.
– The hand calculations for the low weirs could be revised to more accurately reflect the broad crested weirs by varying the weir coefficient with headwater and tailwater levels, considering the ratio of the headwater level to the thickness of the weirs. The actual concrete wall breadth of 230 mm is significant relative to flow depths of interest.
– Remove cross-section at avon.duddlydiv chainage 0 to create longer dx and a more robust model.[31]
– Improve the geometry at the upstream end of the culvert upstream of the Paparoa Diversion.
    - Upstream of the culvert, there is a segment of the timber drain that is covered. The covered timber drain geometry was unavailable for this area, so this area was modelled using the same geometry as the culvert.
    - Geometry updates for this area would likely require additional survey.
    - Given the piecemeal history of the entire culvert, a continuous internal 3D survey would be warranted
– Improve the culvert geometry downstream of the Paparoa Diversion, which is currently modelled based on the as-built plans and surveyed invert levels.
    - Based on photos of the culvert exit, the culvert cross-section geometry does not match the as-built plans. Photos of the culvert exit show side benching at bottom of the culvert, which is not shown on the Paparoa Diversion as-built drawings. As-built plans for the culvert downstream of the diversion have not been found.
    - Additional internal culvert measurements and/or survey data would likely be required to improve the geometry in this area.
– Refine the M11 roughness within the Nightingale Pl Bridge over Dudley Creek. These roughness values are currently set to 0.040, consistent with the roughness in the open channel, and were outside the scope of work to address. This may not be hugely significant other than for largest flood flows, as the bridge flow area is quite large and open cross sections upstream are quite constricted.
– Reconsider manhole head loss along the Paparoa Diversion pipe, which may include considering manhole losses in the rating curve calculations and significantly lowering head losses (and increasing flow capacity) in the model.

6.2 Papanui Diversion

Future improvements to the hydraulic modelling of the Papanui Diversion could include changing the branch definition around the diversion:

– The Papanui branch could be defined continuously from upstream of the diversion into the diversion, following the flood flow capacity, rather than following the natural alignment.
– The 450 mm pipe (and potentially the 600 mm pipe) would then become a side branch in MU.
– The current M11 flow split would be removed, replaced with a new MU outlet to M11 coupling.

[31] This cross-section is left over from the previous modelling and could be deleted to potentially improve stability.

These edits were not implemented into the model because it would have required substantial model edits, which are outside of the scope of the current project. A more radical change would convert the entire upstream portion of the Papanui Drain to MU. A benefit of the MU configuration is that MU can handle abrupt changes in invert levels, so the 600 mm pipe geometry could be included more realistically in the model. This configuration would also simplify the dummy link in MU.

An additional model improvement could be to modify the cross-section geometry of avon.papanui chainage 1002.6 to match avon.duddly10 chainage -10 (at the Papanui Diversion flow split) to help create a more robust model. The modelled geometry is almost the same, but the avon.duddlydiv chainage -10 is represented with a closed irregular section[32] compared to the avon.papanui chainage 1002.6, which is modelled with a closed rectangular section.

Finally, if the Papanui Drain is left in M11, the weir type for the low flow weirs in the Papanui Diversion could be changed from broad crested to Weir Type 1, which may improve the match between the model results and calculated rating curve. Weir Type 1 in M11 uses the standard weir expression for sharp crested weirs, reduced for submergence according to the Villemonte formula.

7. Appendices and Digital Files

PDF Appendices:

Appendix A: Paparoa Diversion As-Built Drawings

Appendix B: Papanui Diversion As-Built Drawings

Appendix C: Paparoa Diversion Tabulated Structure

Digital Files[33]

1. Paparoa Diversion Rating Curve Calculations and model result comparisons
   a. 20250326_Paparoa_Diversion_RatingCurve.xlsx
2. Paparoa Diversion Tabulated Structure Calculations
   a. 20241220_Paparoa_Diversion_TabulatedStructures.xlsx
3. Papanui Diversion Rating Curve Calculations and model result comparisons
   a. 20250321_Papanui_Flow_Calcs.xlsx

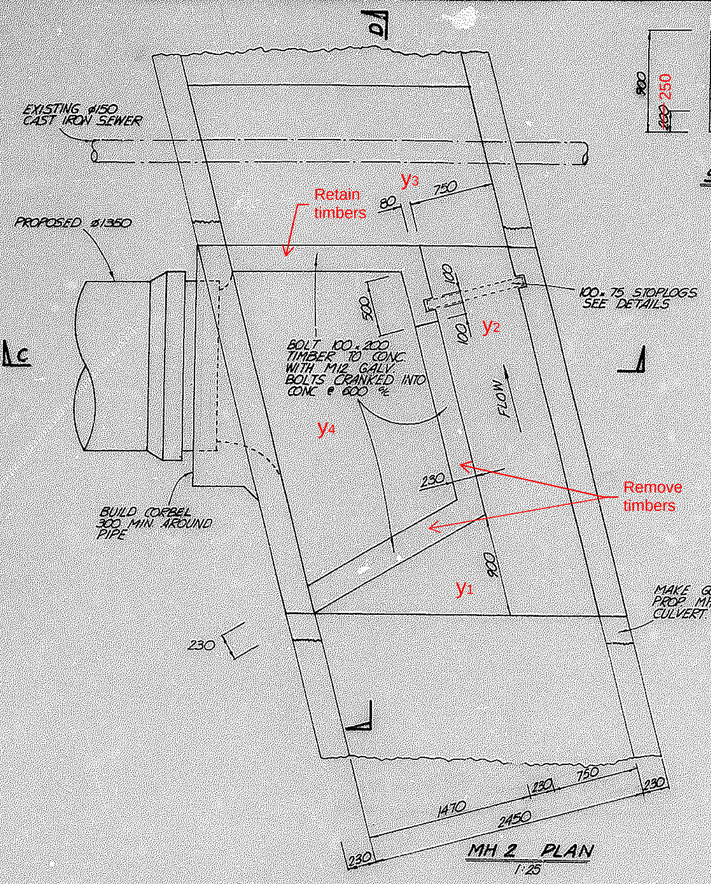
[32] The bottom of the section is skewed by 10 mm to improve model stability.
[33] Included with Cranford Delivery package 2/4/2025.

Appendices

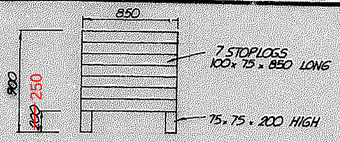
Appendix A

Paparoa Diversion As-Built Drawings

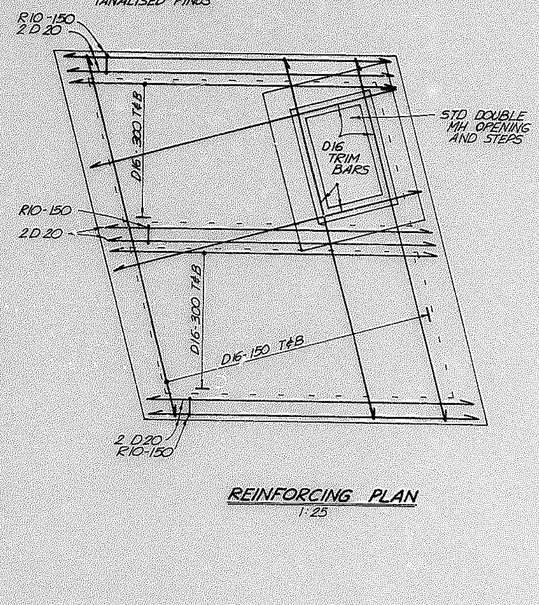
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Patent Number 137041



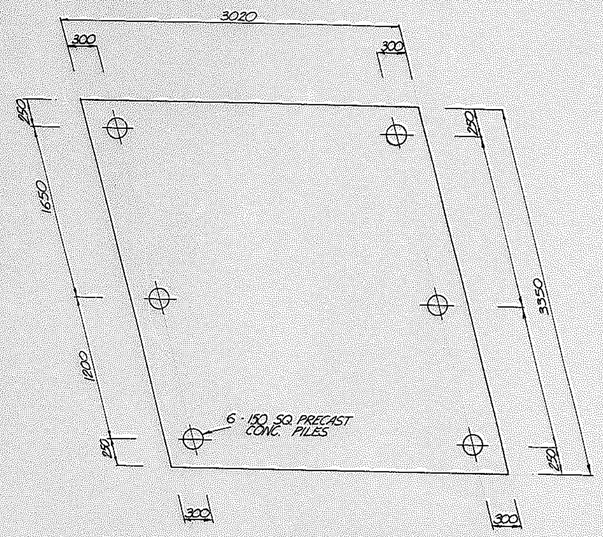
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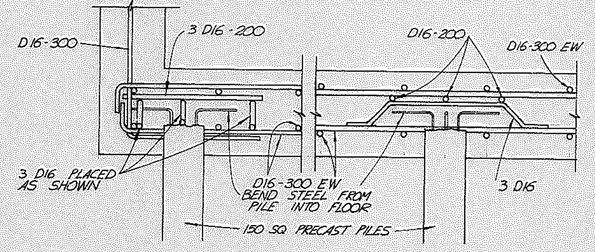
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1:25



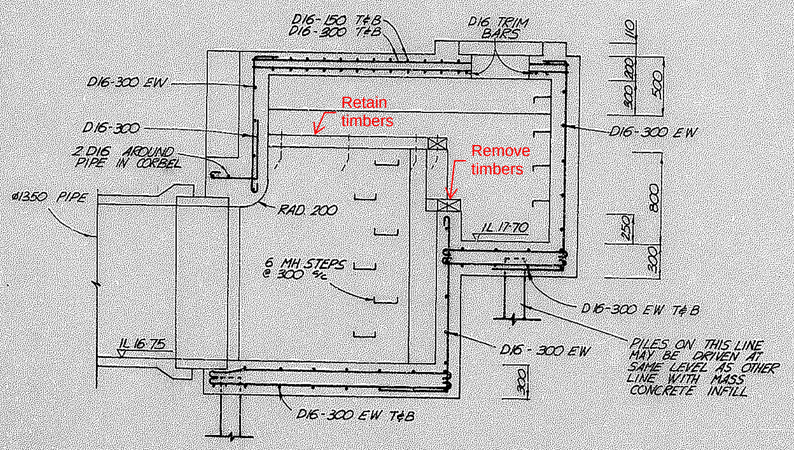
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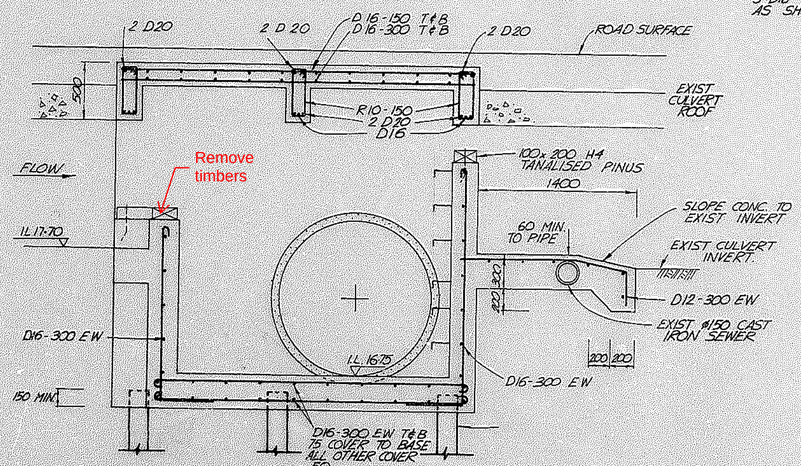
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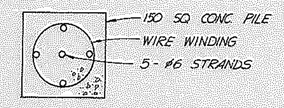
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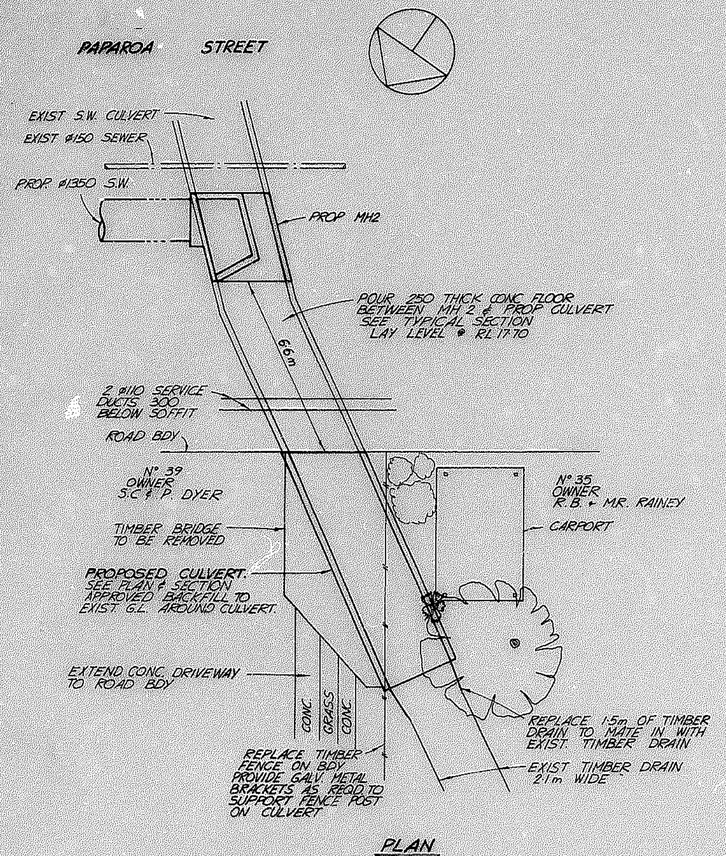


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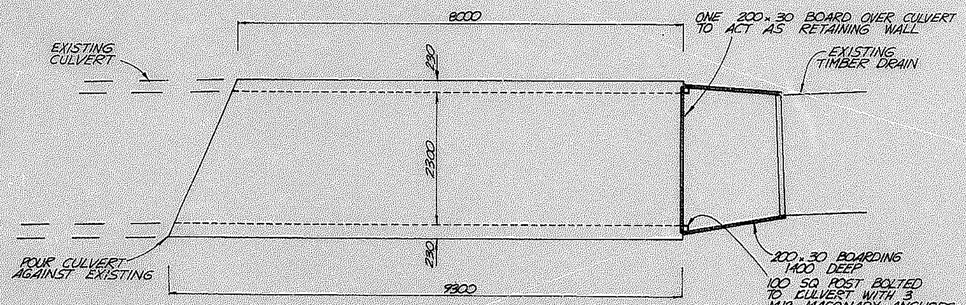


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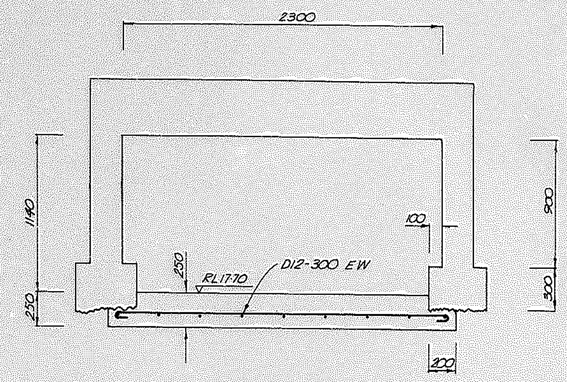
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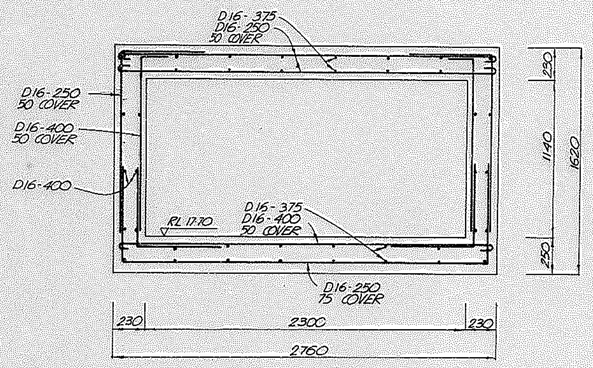
PLAN
1:100



PROPOSED CULVERT (PLAN)
1:50



EXIST. CULVERT TYPICAL SECTION
1:20
NOTE - FLOOR TO BE POURED IN ALTERNATE BAYS OF 2.2m.



PROP CULVERT TYPICAL SECTION
1:20

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				SURVEY L.B.						PLAN 1:100 1:50	DRAWN	S.D. BENSBERG	25-1-89	Christchurch Drainage Board	UPPER DUDLEY CREEK DIVERSION	89/2046	11550
				F.B.						LONG. SECTN HOR.	TRACED				STAGE 2 - PAPAROA STREET		
				CONSTR.N E.B.						VERT.	CHECKED					J.H.S. Nº.	SHEET	REVISION
AMENDMENT	INIT.	DATE	APPD.	L.B.					DATUM 9.943m BELOW M.S.L.	DETAILS 1:20	DESIGN. SUPERVD.		30/1/89	APPROVED		D50-0016	6 OF 6	
											RECOMMENDED		1/89	N.P. Hunt 30-1-89 ENGINEER TO THE BOARD			

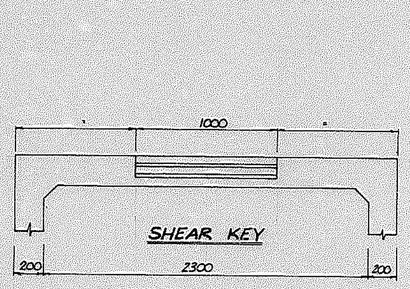
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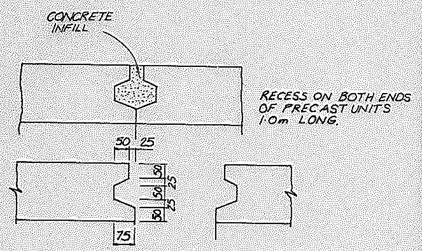
SELF-ADHESIVE PLAN HANGER
Re-Order Filemaster "D"
Patent Number 137941

6-8

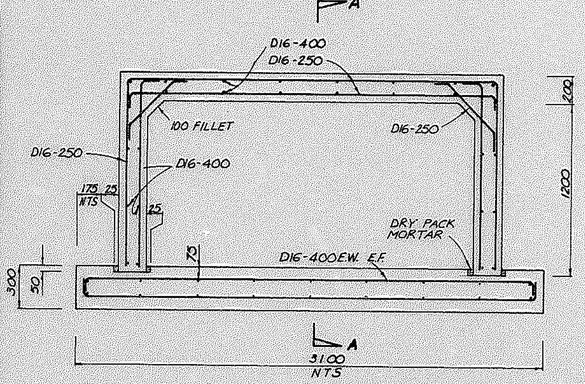
20X

11550-7
Re-Order Filemaster "D"
SELF-ADHESIVE PLAN HANGER
Patent Number 137641

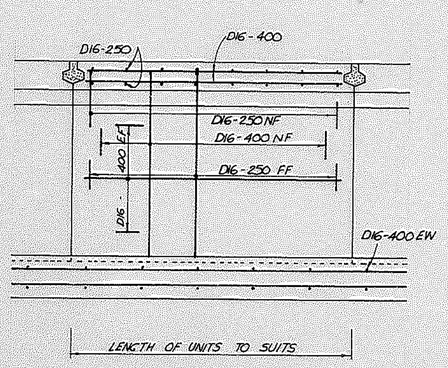




SCALE 1:10



PRECAST CULVERT DETAILS
1:20



SECTION A-A
1:20

				BOOK	PAGE	INIT'L	DATE	ORIGIN OF LEVELS	ORIGINAL SCALES		NAME	DATE
			SURVEY L.B.						PLAN	DRAWN	S.D.BENSBERG	3-8-89
			F.B.						LONG' SECT' HOR.	TRACED		
			CONSTR' F.B.						VERT.	CHECKED		
AMENDMENT	INIT'	DATE	APPD.	L.B.				DATUM: 9.943m BELOW M.S.L.	DETAILS AS SHOWN.	DESIGNED		2-1-89
										DESGN. SUPERVD.		2-1-89
										RECOMMENDED	P.D. Adams	9/89

Christchurch Drainage Board

APPROVED H.P. Hunt 18.9.89 ENGINEER TO THE BOARD

UPPER DUDLEY CREEK DIVERSION
STAGE 2 - PAPAROA STREET
PRECAST CULVERT UNITS

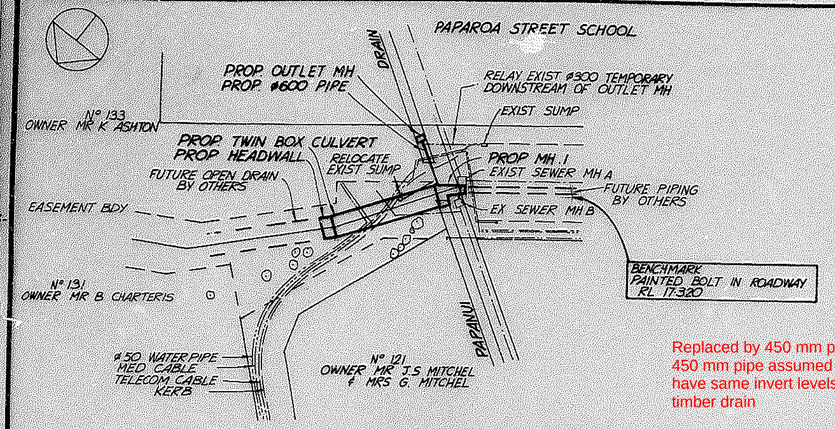
CONTRACT No.	PLAN No.
	11550

J.H.S. No.	SHEET	REVISION
	7 OF	

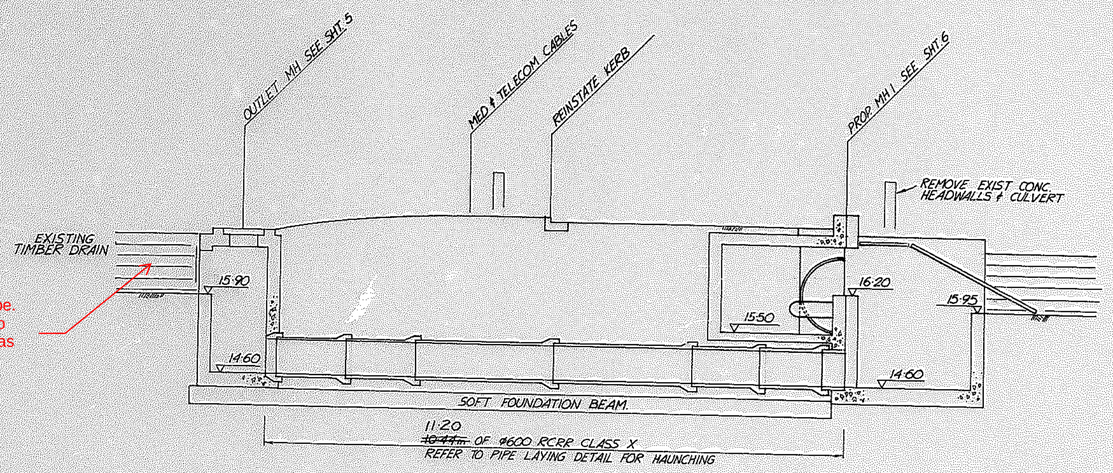
7-8
20X

Appendix B

Papanui Diversion As-Built Drawings

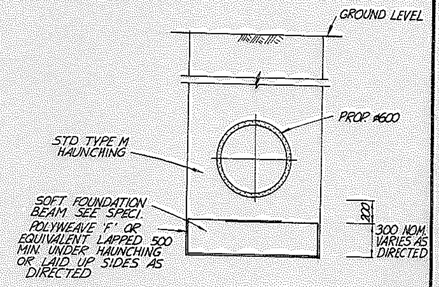


SITE PLAN
1:500

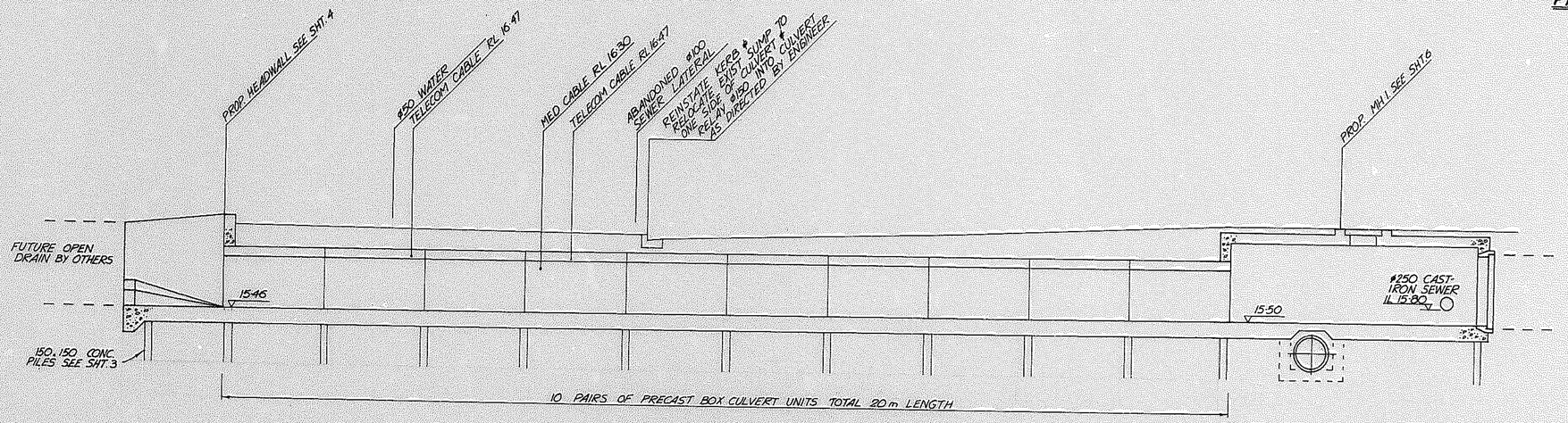


Replaced by 450 mm pipe. 450 mm pipe assumed to have same invert levels as timber drain

LONGSECTION THRU ⌀600 PIPING
1:50



PIPE LAYING DETAIL
~~1:20~~ 1:25



LONGSECTION THRU TWIN BOX CULVERT
1:50

AMENDMENT	INIT.	DATE	APPD

	BOOK	PAGE	INIT.	DATE
SURVEY L.B.				
F.B.				
CONSTR.N E.B.				
L.B.				

ORIGIN OF LEVELS: DATUM: 9.963m BELOW M.S.L.

ORIGINAL SCALES: PLAN ..... 1:500; LONG. SECT.N HOR.; VERT.; DETAILS 1:50; 1:20

	NAME	DATE
DRAWN	S.D. BENSBERG	1-9-88
TRACED		
CHECKED		1-9-88
DESIGNED		6-9-88
DESGN. SUPERVD.		6-9-88
RECOMMENDED		9/88

Christchurch Drainage Board

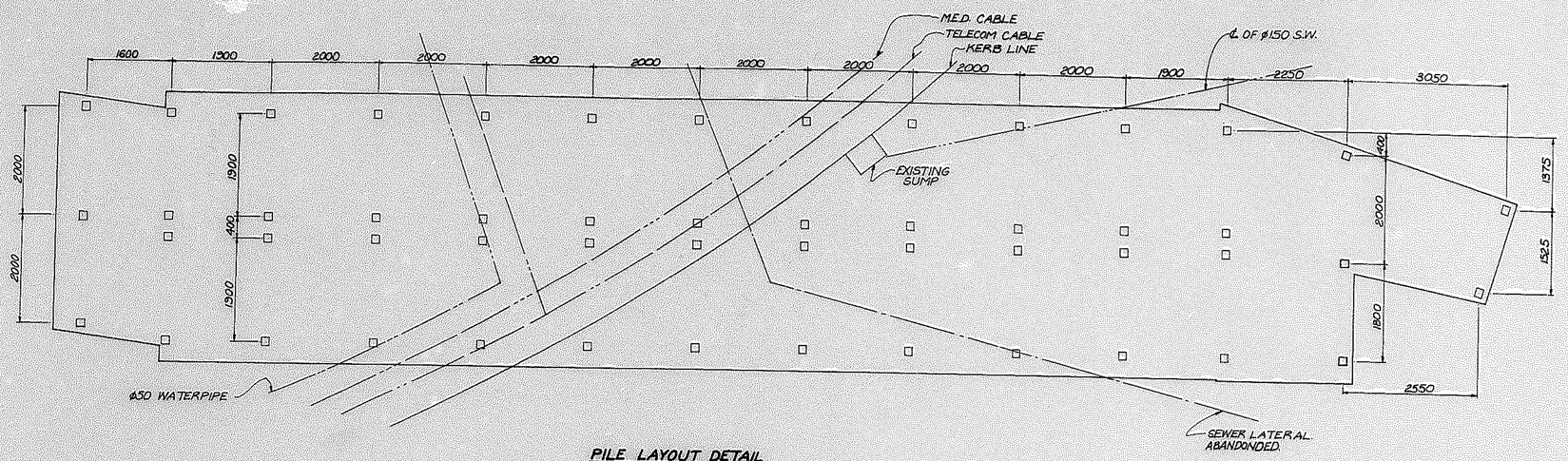
APPROVED 6-9-88 ENGINEER TO THE BOARD

UPPER DUDLEY CREEK DIVERSION FARM BRIDGES & PAPANUI DRAIN DIVERSION STRUCTURE

CONTRACT No.	PLAN No.
88/2032	11441B
J.H.S. No.	SHEET	REVISION
050-0016	2 OF 6	

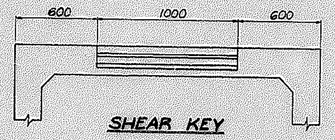
2 of 6

20X

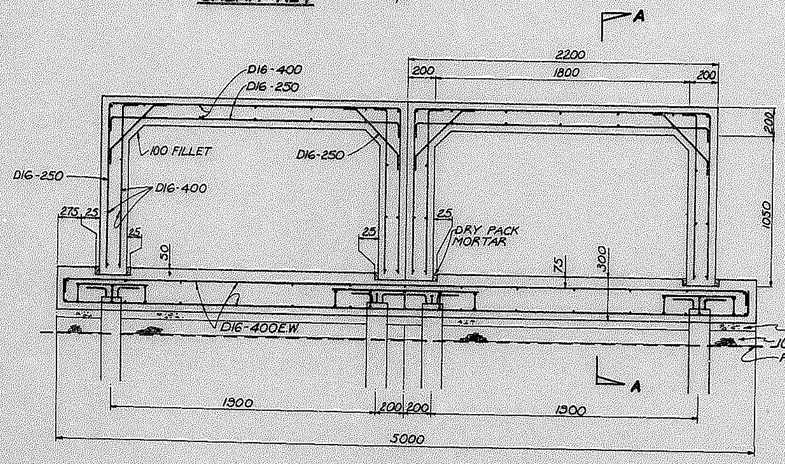


PILE LAYOUT DETAIL
SCALE 1:50

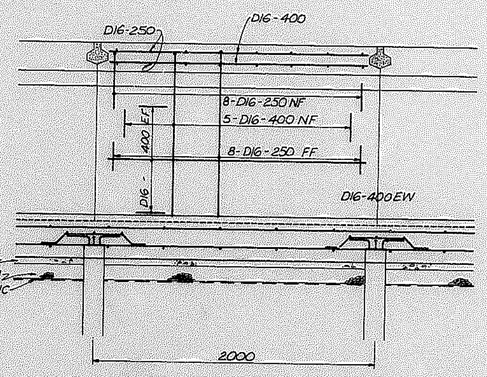
NOTE:
OUTSIDE WALL STEEL FROM END CULVERT UNITS TO EXTEND 600mm INTO HEADWALL & M.H.I.



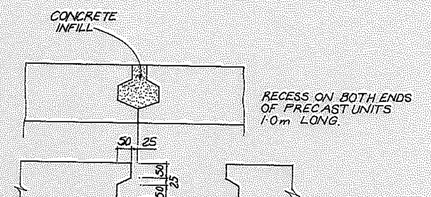
SHEAR KEY



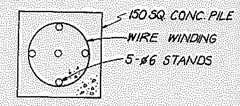
TWIN BOX CULVERT DETAILS
SCALE 1:20



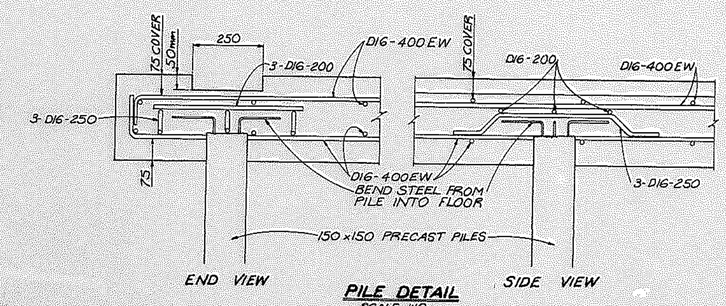
SECTION A-A
SCALE 1:20



UNIT JOINT DETAIL
1:10



PRECAST PILE SECTION
1:5



PILE DETAIL
SCALE 1:10

				BOOK	PAGE	INIT'	DATE	ORIGIN OF LEVELS
			SURVEY L.B.					
			F.B.					
			CONSTR^N E.B.					
AMENDMENT	INIT'	DATE	APP'D.	L.B.				DATUM: 9.643m BELOW M.S.L.

ORIGINAL SCALES
PLAN .....
LONG' SECT^N HOR.
VERT.
DETAILS ..... AS SHOWN

	NAME	DATE
DRAWN	[illegible]	5.9.88
TRACED	[illegible]	5.9.88
CHECKED	[illegible]	"
DESIGNED	[illegible]	5.9.88
DESGN. SUPERVD.	[illegible]	6.9.88
RECOMMENDED	[illegible]	9/88

Christchurch Drainage Board
APPROVED [illegible] 6.9.88
ENGINEER TO THE BOARD

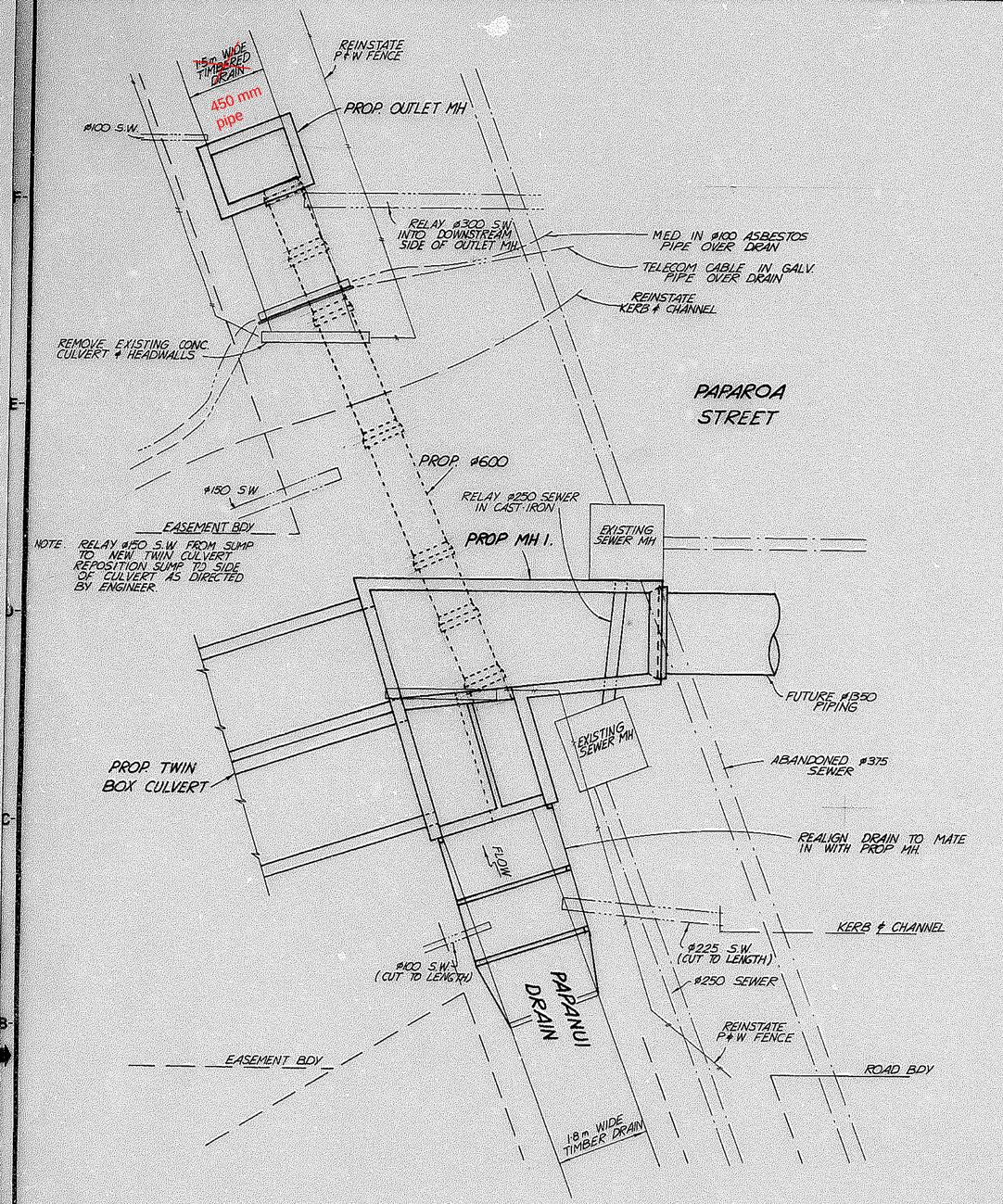
UPPER DUDLEY CREEK DIVERSION FARM BRIDGES & PAPANUI DRAIN DIVERSION STRUCTURE

CONTRACT No.	PLAN No.
88/2032	11441B
J.H.S. No.	SHEET	REVISION
D50-0016	3 OF 6	

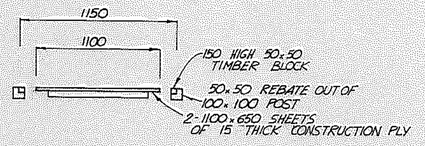
3 of 6

20x

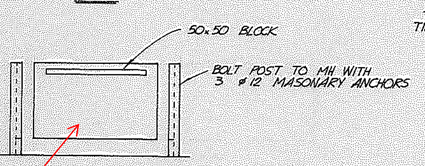
Re-Order Filemaster "D"
Sellotape 1230 Double Side Tissue
SELF-ADHESIVE PLAN HANGER
Patent Number 137844



PLAN
1:50



PLAN

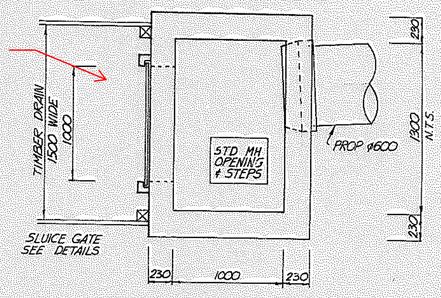


ELEVATION

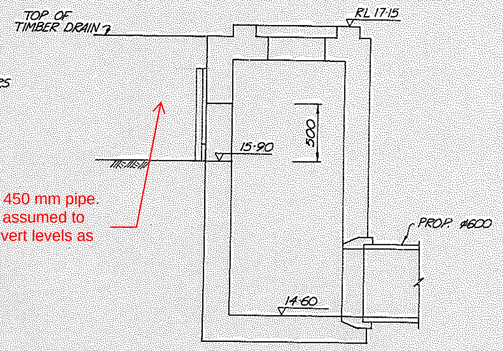
Fully open sluice gate

SLUICE GATE DETAILS
1:25

Replaced by 450 mm pipe.



PLAN



SECTION

Replaced by 450 mm pipe. 450 mm pipe assumed to have same invert levels as timber drain.

OUTLET MH DETAILS
1:25

AMENDMENT	INIT.	DATE	APPD.

	BOOK	PAGE	INIT.	DATE
SURVEY L.B.				
F.B.				
CONSTR.N E.B.				
L.B.				

ORIGIN OF LEVELS
DATUM: 9.643m BELOW M.S.L.

ORIGINAL SCALES
PLAN 1:50
LONG. SECT.N HOR.
VERT.
DETAILS 1:20

	NAME	DATE
DRAWN	S.D. BENSEMER	30.5.88
TRACED		
CHECKED		5.9.88
DESIGNED		6.9.88
DESGN. SUPERVD.		6.9.88
RECOMMENDED		9/88

Christchurch Drainage Board
APPROVED
ENGINEER TO THE BOARD

UPPER DUDLEY CREEK DIVERSION FARM BRIDGES & PAPANUI DRAIN DIVERSION STRUCTURE

CONTRACT No.	PLAN No.
88/2032	11441B

J.H.S. No.	SHEET	REVISION
D50-0016	5 OF 6	

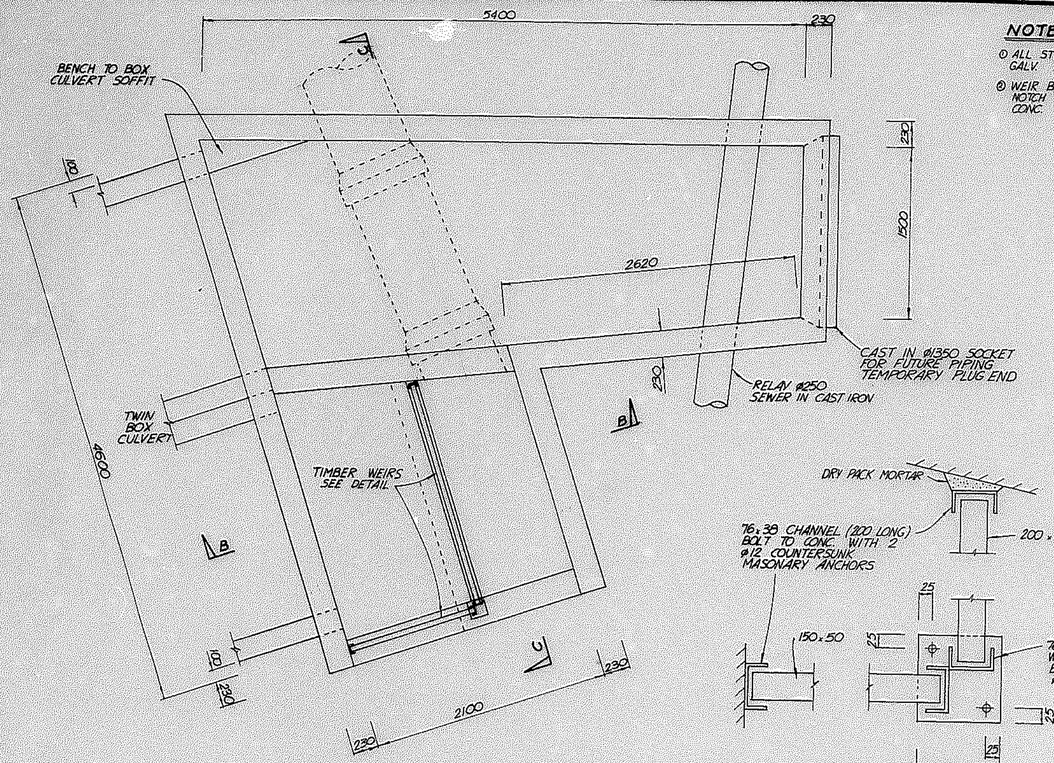
5 of 6

20X

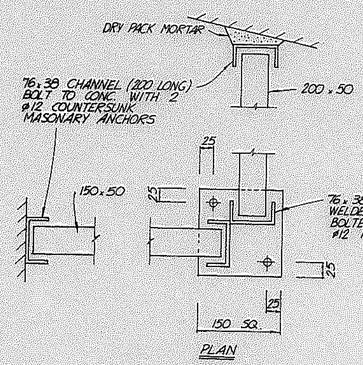
11418/6
Re-Order Remaster "D"
Sellotape 1230 Double Side Tissue
SELF-ADHESIVE PLAN HANGER
Patent Number 137241

NOTES

1. ALL STEEL WORK TO BE HOT DIP GALV. AFTER FABRICATION.
2. WEIR BOARDS TO BE H.5 TREATED NOTCH TO PROVIDE NEAT FIT TO CONC. SURFACE

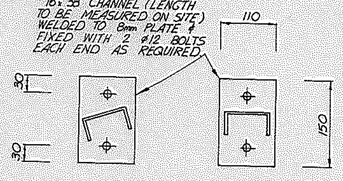


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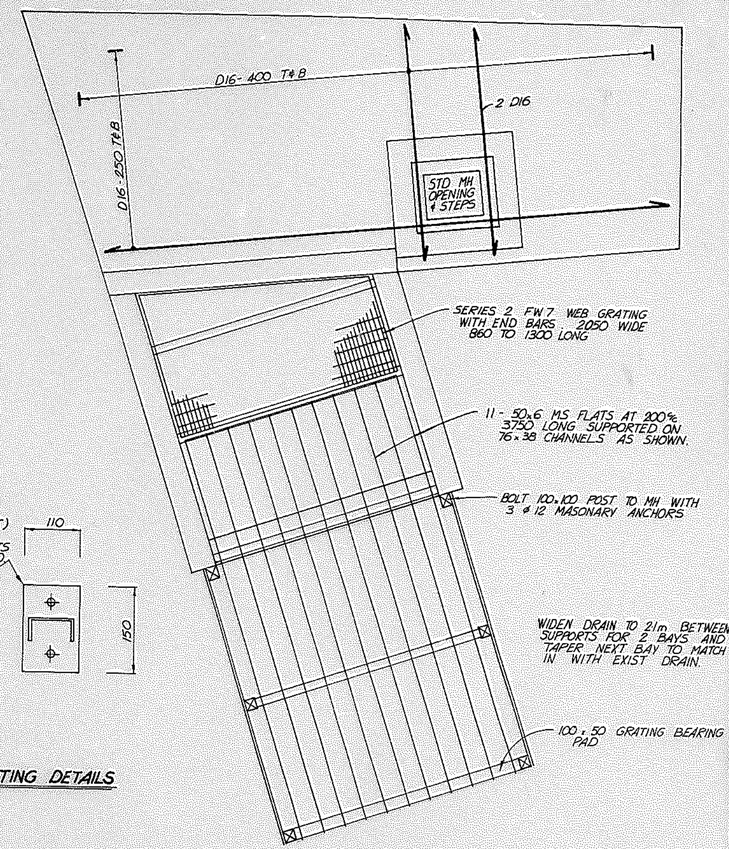


PLAN

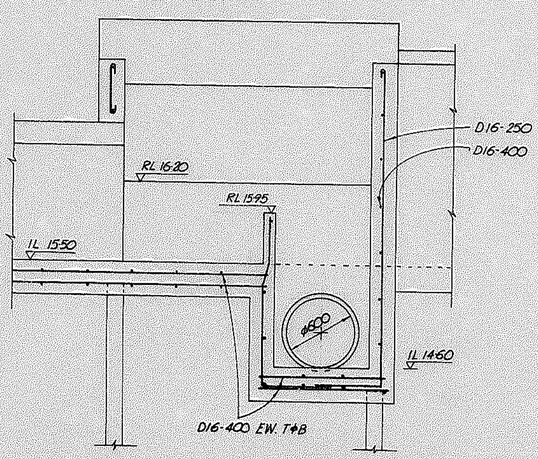
WEIR DETAILS
1:5



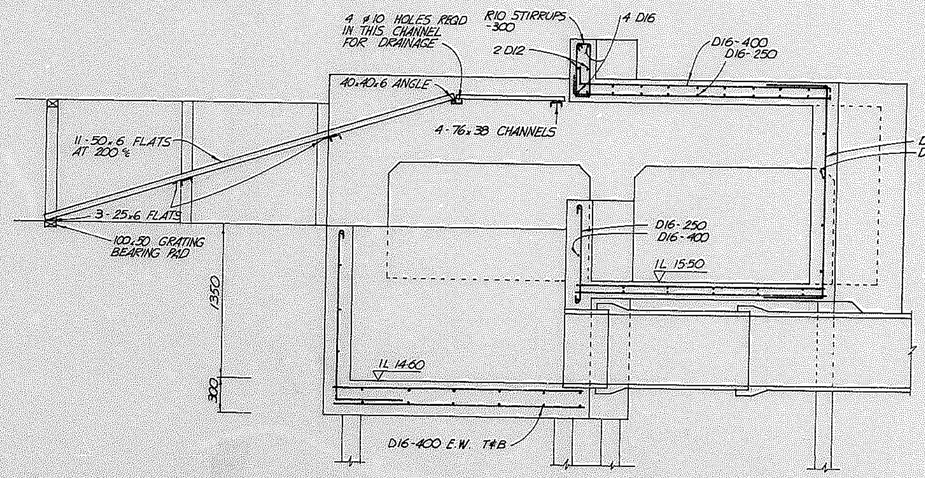
GRATING MOUNTING DETAILS
1:5



TOP SLAB & GRATING PLAN
1:25



SECTION B-B



SECTION C-C

AMENDMENT	INIT'S	DATE	APP'D

	BOOK	PAGE	INIT'S	DATE
SURVEY L.B.				
F.B.				
CONSTR^N L.B.				
L.B.				

ORIGIN OF LEVELS: DATUM 9.043m BELOW M.S.L.

ORIGINAL SCALES: PLAN ..... 1:25; LONG^L SECT^N HOR.; VERT.; DETAILS ..... 1:25 & 1:5

	NAME	DATE
DRAWN	S.D. BENSBERG	5.9.88
TRACED		5.9.88
CHECKED	R.T. Hutchinson	5.9.88
DESIGNED	Young	6.9.88
DESGN. SUPERVD.	Anderson	6.9.88
RECOMMENDED	R.D. Adams	9/88

Christchurch Drainage Board
APPROVED H.P. Hunt 6.9.88
ENGINEER TO THE BOARD

UPPER DUDLEY CREEK DIVERSION FARM BRIDGES & PAPANUI DRAIN DIVERSION STRUCTURE

CONTRACT No.	PLAN No.
88/2032	11441B
J.H.S. No.	SHEET 6 OF 6
	REVISION

6 of 6
20x

Appendix C

Paparoa Diversion Tabulated Structure

Tabulated Structure:

y3 (across) / y1 (down)	0.000	0.050	0.100	0.200	0.300	0.400	0.500	0.600	0.7	0.8	0.9	1	1.1	1.2	1.3	1.4	1.6	1.8	2
0	0.000	-0.013	-0.038	-0.105	-0.259	-0.325	-0.380	-0.427	-0.470	-0.509	-0.546	-0.711	-0.983	-1.300	-1.488	-1.648	-1.922	-2.157	-2.366
0.05	0.013	0.000	-0.038	-0.105	-0.259	-0.325	-0.380	-0.427	-0.470	-0.509	-0.546	-0.711	-0.983	-1.300	-1.488	-1.648	-1.922	-2.157	-2.366
0.10	0.038	0.038	0.000	-0.105	-0.259	-0.325	-0.380	-0.427	-0.470	-0.509	-0.546	-0.711	-0.983	-1.300	-1.488	-1.648	-1.922	-2.157	-2.366
0.20	0.105	0.105	0.105	0.000	-0.196	-0.277	-0.340	-0.392	-0.438	-0.480	-0.519	-0.686	-0.959	-1.277	-1.466	-1.628	-1.903	-2.138	-2.349
0.30	0.195	0.195	0.195	0.195	0.000	-0.196	-0.277	-0.340	-0.392	-0.438	-0.480	-0.650	-0.925	-1.246	-1.436	-1.599	-1.876	-2.113	-2.325
0.4	0.294	0.294	0.294	0.294	0.196	0.000	-0.196	-0.277	-0.340	-0.392	-0.438	-0.611	-0.889	-1.212	-1.404	-1.568	-1.848	-2.088	-2.301
0.5	0.380	0.380	0.380	0.340	0.277	0.196	0.000	-0.196	-0.277	-0.340	-0.392	-0.569	-0.851	-1.176	-1.370	-1.536	-1.819	-2.061	-2.276
0.6	0.427	0.427	0.427	0.392	0.340	0.277	0.196	0.000	-0.196	-0.277	-0.340	-0.523	-0.809	-1.138	-1.335	-1.503	-1.789	-2.033	-2.250
0.7	0.470	0.470	0.470	0.438	0.392	0.340	0.277	0.196	0.000	-0.196	-0.277	-0.471	-0.762	-1.096	-1.296	-1.467	-1.757	-2.004	-2.224
0.8	0.509	0.509	0.509	0.480	0.438	0.392	0.340	0.277	0.196	0.000	-0.196	-0.408	-0.710	-1.049	-1.254	-1.429	-1.723	-1.974	-2.196
0.9	0.546	0.546	0.546	0.519	0.480	0.438	0.392	0.340	0.277	0.196	0.000	-0.327	-0.648	-0.997	-1.208	-1.387	-1.688	-1.942	-2.167
1	0.711	0.711	0.711	0.686	0.650	0.611	0.569	0.523	0.471	0.408	0.327	0.000	-0.509	-0.935	-1.155	-1.340	-1.649	-1.909	-2.137
1.1	0.983	0.983	0.983	0.959	0.925	0.889	0.851	0.809	0.762	0.710	0.648	0.509	0.000	-0.679	-0.961	-1.177	-1.519	-1.797	-2.038
1.2	1.300	1.300	1.300	1.277	1.246	1.212	1.176	1.138	1.096	1.049	0.997	0.935	0.679	0.000	-0.679	-0.961	-1.359	-1.664	-1.921
1.3	1.488	1.488	1.488	1.466	1.436	1.404	1.370	1.335	1.296	1.254	1.208	1.155	0.961	0.679	0.000	-0.679	-1.177	-1.519	-1.797
1.4	1.648	1.648	1.648	1.628	1.599	1.568	1.536	1.503	1.467	1.429	1.387	1.340	1.177	0.961	0.679	0.000	-0.961	-1.359	-1.664
1.6	1.922	1.922	1.922	1.903	1.876	1.848	1.819	1.789	1.757	1.723	1.688	1.649	1.519	1.359	1.177	0.961	0.000	-0.961	-1.359
1.8	2.157	2.157	2.157	2.138	2.113	2.088	2.061	2.033	2.004	1.974	1.942	1.909	1.797	1.664	1.519	1.359	0.961	0.000	-0.961
2	2.366	2.366	2.366	2.349	2.325	2.301	2.276	2.250	2.224	2.196	2.167	2.137	2.038	1.921	1.797	1.664	1.359	0.961	0.000

Legend

Condition	Description	Valid y1 and y3 range
1	Low flow through stop logs (y2<250 mm), without submergence due to backwater conditions downstream	y1 < 400 mm and y3 < 250 mm
2	Orifice flow through stop logs (250 mm < y2 < 900 mm)	400 mm < y1 < 900 mm OR y3 > 250 mm and y1 < 900 mm
3	Weir flow over stop logs + orifice flow through 250 mm opening (y1=y2>900 mm)	900 mm < y1 < 1130 mm
4	Orifice flow over the stop logs + orifice flow through the low flow opening	y1 > 1130 mm

Notes:

- Reverse flow (y3 > y1) estimated based on results for positive flow. Assumed to be opposite flow value of positive flow result for same head difference

M11 Input:

Same as tabulated structure above, but y3 and y1 converted to RL (postEQ shift of 160 mm)

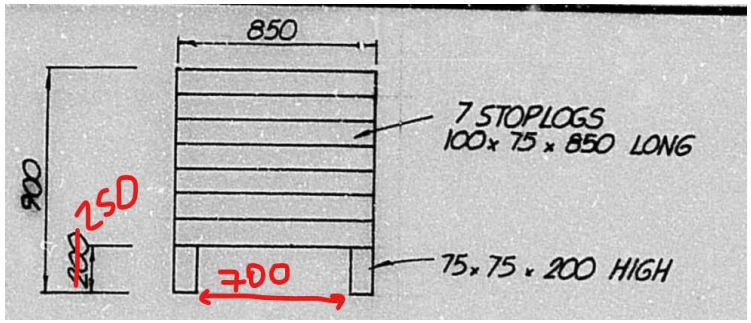
	h D/S																		
h U/S	17.54	17.59	17.64	17.74	17.84	17.94	18.04	18.14	18.24	18.34	18.44	18.54	18.64	18.74	18.84	18.94	19.14	19.34	19.54
17.54	0.000	-0.013	-0.038	-0.105	-0.259	-0.325	-0.380	-0.427	-0.470	-0.509	-0.546	-0.711	-0.983	-1.300	-1.488	-1.648	-1.922	-2.157	-2.366
17.59	0.013	0.000	-0.038	-0.105	-0.259	-0.325	-0.380	-0.427	-0.470	-0.509	-0.546	-0.711	-0.983	-1.300	-1.488	-1.648	-1.922	-2.157	-2.366
17.64	0.038	0.038	0.000	-0.105	-0.259	-0.325	-0.380	-0.427	-0.470	-0.509	-0.546	-0.711	-0.983	-1.300	-1.488	-1.648	-1.922	-2.157	-2.366
17.74	0.105	0.105	0.105	0.000	-0.196	-0.277	-0.340	-0.392	-0.438	-0.480	-0.519	-0.686	-0.959	-1.277	-1.466	-1.628	-1.903	-2.138	-2.349
17.84	0.195	0.195	0.195	0.195	0.000	-0.196	-0.277	-0.340	-0.392	-0.438	-0.480	-0.650	-0.925	-1.246	-1.436	-1.599	-1.876	-2.113	-2.325
17.94	0.294	0.294	0.294	0.294	0.196	0.000	-0.196	-0.277	-0.340	-0.392	-0.438	-0.611	-0.889	-1.212	-1.404	-1.568	-1.848	-2.088	-2.301
18.04	0.380	0.380	0.380	0.340	0.277	0.196	0.000	-0.196	-0.277	-0.340	-0.392	-0.569	-0.851	-1.176	-1.370	-1.536	-1.819	-2.061	-2.276
18.14	0.427	0.427	0.427	0.392	0.340	0.277	0.196	0.000	-0.196	-0.277	-0.340	-0.523	-0.809	-1.138	-1.335	-1.503	-1.789	-2.033	-2.250
18.24	0.470	0.470	0.470	0.438	0.392	0.340	0.277	0.196	0.000	-0.196	-0.277	-0.471	-0.762	-1.096	-1.296	-1.467	-1.757	-2.004	-2.224
18.34	0.509	0.509	0.509	0.480	0.438	0.392	0.340	0.277	0.196	0.000	-0.196	-0.408	-0.710	-1.049	-1.254	-1.429	-1.723	-1.974	-2.196
18.44	0.546	0.546	0.546	0.519	0.480	0.438	0.392	0.340	0.277	0.196	0.000	-0.327	-0.648	-0.997	-1.208	-1.387	-1.688	-1.942	-2.167
18.54	0.711	0.711	0.711	0.686	0.650	0.611	0.569	0.523	0.471	0.408	0.327	0.000	-0.509	-0.935	-1.155	-1.340	-1.649	-1.909	-2.137
18.64	0.983	0.983	0.983	0.959	0.925	0.889	0.851	0.809	0.762	0.710	0.648	0.509	0.000	-0.679	-0.961	-1.177	-1.519	-1.797	-2.038
18.74	1.300	1.300	1.300	1.277	1.246	1.212	1.176	1.138	1.096	1.049	0.997	0.935	0.679	0.000	-0.679	-0.961	-1.359	-1.664	-1.921
18.84	1.488	1.488	1.488	1.466	1.436	1.404	1.370	1.335	1.296	1.254	1.208	1.155	0.961	0.679	0.000	-0.679	-1.177	-1.519	-1.797
18.94	1.648	1.648	1.648	1.628	1.599	1.568	1.536	1.503	1.467	1.429	1.387	1.340	1.177	0.961	0.679	0.000	-0.961	-1.359	-1.664
19.14	1.922	1.922	1.922	1.903	1.876	1.848	1.819	1.789	1.757	1.723	1.688	1.649	1.519	1.359	1.177	0.961	0.000	-0.961	-1.359
19.34	2.157	2.157	2.157	2.138	2.113	2.088	2.061	2.033	2.004	1.974	1.942	1.909	1.797	1.664	1.519	1.359	0.961	0.000	-0.961
19.54	2.366	2.366	2.366	2.349	2.325	2.301	2.276	2.250	2.224	2.196	2.167	2.137	2.038	1.921	1.797	1.664	1.359	0.961	0.000

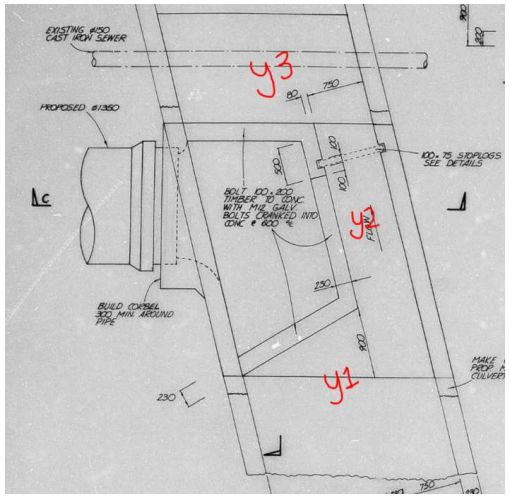
Condition 1: Low flow through stop logs (y2<250 mm), without submergence due to backwater conditions downstream

This condition analyzes the flow through the 250 mm stop log opening before it becomes orifice flow (i.e. y2<250 mm)

Process Overview and Assumptions:

Calculate y1 considering contraction losses between y2 and y1
Assume critical depth at y2
y1 was then calculated by applying energy balance across the contraction
These calculations were repeated up until y2 = 250 mm (hits the top of the opening).
Calculations in which y1 > 250 mm are very approximate because they do not consider the fact that flow is not conserved across the weir
Calculate normal depth downstream using Manning's equation





Equations

Critical depth in a rectangular channel:

$$y_c = \sqrt[3]{\frac{q^2}{g}}$$

Where
yc=critical depth
q= flow per channel width = Q/W
W= channel width

Energy Balance:

$$y_1 + z_1 + \frac{V_1^2}{2g} = y_2 + z_2 + \frac{V_2^2}{2g} + h_L$$

Where
z= invert elevation, assumed to be approximately equal at both locations
V= velocity, equal to Q/A
A= flow area, taken as y*W, where W=channel width
hL= head loss across the contraction, taken as K*V2^2/2g
*note that a similar loss was estimated across expansion from y2 to y3
K= contraction loss coefficient

Condition 1: Low flow through stop logs (y2<250 mm), without submergence due to backwater conditions downstream

Inputs:

W2=	0.75	m
K=	0.2	
g=	9.81	m3/s
W1=	2.45	m
b=	2.45	m
n=	0.013	
S=	0.0047	m/m

Steps:

1. Select a desired flow rate for the calculations, Q
2. Calculate y2 assuming critical depth at y2
3. Calculate V2 by taking V2=Q/A2, where A2=y2*W2
4. Calculate hL as K*V2^2/2g
5. Estimate (guess) y1
6. Calculate A1 where A1=y1*W1
7. Calculate V1
8. Calculate y1 by applying the energy balance equation, neglecting differences in z values
9. Calculate difference in estimated vs. calculated y1 values
10. Use Goal Seek function in excel to change estimated y1 until y1 est - y1 calc = 0
11. Assuming a similar expansion head loss to the contraction head loss (0.2 contraction coefficient), calculate energy (y3 + z3 + V3^2/2g) at y3, neglecting the vertical drop between y2 and y3. In other words, E3= V2^2/2*g - hL
12. Assume this is the max energy that can occur at point 3. If y3 conditions lead to E3 above this value, then y3 would impact y2 and decrease flow through the opening (start to backwater)

Calculations:

y2	V2	Q (m3/s)	q2 (m2/s)	yc2 (m)	y1 (est)	A1	V1	hL	y1 (calc)	y1 est -y1 calc	y1-y2	E3 max
0.031	0.55	0.013	0.02	0.031	0.05	0.12	0.11	0.003	0.05	0.001	0.02	0.044
0.063	0.79	0.038	0.05	0.063	0.100	0.25	0.15	0.006	0.10	0.000	0.04	0.095
0.126	1.11	0.11	0.14	0.126	0.20	0.49	0.21	0.013	0.20	0.001	0.07	0.189
0.190	1.37	0.195	0.26	0.190	0.30	0.74	0.27	0.019	0.30	-0.001	0.11	0.285
0.250	1.57	0.294	0.39	0.250	0.40	0.97	0.30	0.025	0.40	0.000	0.15	0.375

Results for Tabulated Structure:

Assume that above flow rates are only valid if E3 is less than E3 max
In all of the conditions below, this condition was met
Note that once y3 >= 250 mm, it is assumed that the stop log opening is submerged and starts acting like an orfiice (conditoin 2)

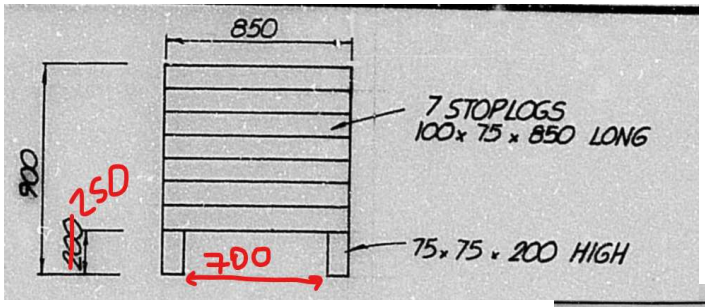
Condition 1: Low flow through stop logs (y2<250 mm), without submergence due to backwater conditions downstream

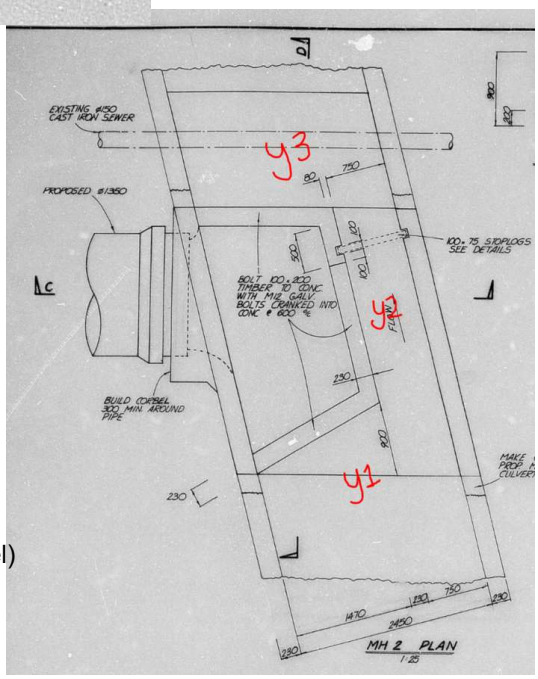
y1	y3	E3	Q	E max	E3 < Emax?
0.05	0	0	0.013	0.044	YES
0.10	0	0	0.038	0.095	YES
0.20	0	0	0.11	0.189	YES
0.30	0	0	0.195	0.285	YES
0.4	0	0	0.294	0.375	YES
0.1	0.05	0.055	0.038	0.095	YES
0.20	0.05	0.087	0.105	0.189	YES
0.30	0.05	0.179	0.195	0.285	YES
0.4	0.05	0.344	0.294	0.375	YES
0.2	0.1	0.109	0.105	0.189	YES
0.3	0.1	0.132	0.195	0.285	YES
0.4	0.1	0.173	0.294	0.375	YES
0.3	0.2	0.208	0.195	0.285	YES
0.4	0.2	0.218	0.294	0.375	YES

Condition 2: Orifice flow through stop logs (250 mm < y2 < 900 mm)

Process Overview and Assumptions:

Solved for flow rate through the orifice using the orifice equation and considering submergence
Assumed y2 is approximately equal to y1
This condition is assumed to be valid when:
1. 900 mm > y1 > 400 mm and y3 < normal depth ("Calc_Cond1" calculations indicate that y2 < 250 mm when y1 < 400 mm without submerged tailwater conditions)
2. y1 < 400 mm but y3 > 250 mm. This assumes y2 becomes influenced by downstream conditions and starts behaving like an orifice
3. For reverse (negative flow) when y3 > 250 mm and y1 < y3. y2 and y1 are approximated as equal for these conditions





Equations:

Orifice Equation

$$Q = CoAg\sqrt{2gd}$$

Where:

Q=	Flow through the orifice
Co=	Orifice discharge coefficient
Ag=	Opening area
g=	Gravitational acceleration
d=	Head above greater of the centroid or downstream water surface elevation
y3=	Water level above invert opening (equal to d + centroid level)
W=	Orifice width
H=	Height of orifice

Inputs:

Co=	0.8		Since the orifice leads to small contraction in width, but more significant vertical contraction
g=	9.81	m/s2	
Orifice width	0.7	m	
Orifice height	0.25	m	
Orifice centroid	0.125	m	
Orifice area	0.175	m2	

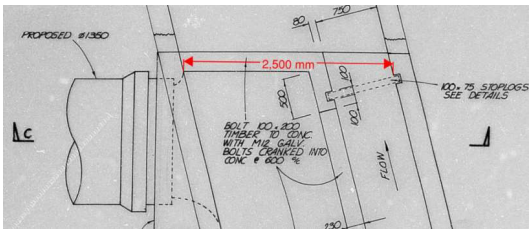
Steps:

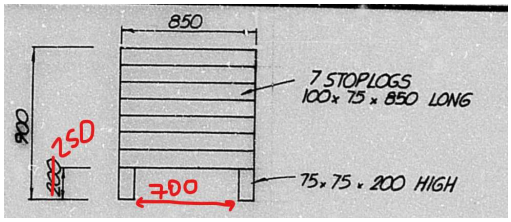
1. Select y1 and y3
2. Calculate flow using the equation above

Condition 2: Orifice flow through stop logs (250 mm < y2 < 900 mm)

y2=y1 (m)	y3 (m)	Orifice Q
0.3	0	0.259
0.4	0	0.325
0.5	0.000	0.380
0.6	0.000	0.427
0.7	0.000	0.470
0.8	0.000	0.509
0.9	0.000	0.546
0.3	0.050	0.259
0.4	0.050	0.325
0.5	0.050	0.380
0.6	0.050	0.427
0.7	0.050	0.470
0.8	0.050	0.509
0.9	0.050	0.546
0.3	0.100	0.259
0.4	0.100	0.325
0.5	0.100	0.380
0.6	0.100	0.427
0.7	0.100	0.470
0.8	0.100	0.509
0.9	0.100	0.546
0.3	0.200	0.196
0.4	0.200	0.277
0.5	0.200	0.340
0.6	0.200	0.392
0.7	0.200	0.438
0.8	0.200	0.480
0.9	0.200	0.519
0.4	0.300	0.196
0.5	0.300	0.277
0.6	0.300	0.340
0.7	0.300	0.392
0.8	0.300	0.438
0.9	0.300	0.480
0.5	0.400	0.196
0.6	0.400	0.277
0.7	0.400	0.340
0.8	0.400	0.392
0.9	0.400	0.438
0.5	0.500	0.000
0.6	0.500	0.196
0.7	0.500	0.277
0.8	0.500	0.340
0.9	0.500	0.392
0.6	0.600	0.000
0.7	0.600	0.196
0.8	0.600	0.277
0.9	0.600	0.340
0.7	0.700	0.000
0.8	0.700	0.196
0.9	0.700	0.277
0.8	0.800	0.000
0.9	0.800	0.196
0.9	0.900	0.000

Condition 3: Weir flow over stop logs + orifice flow through 250 mm opening (y1=y2>900 mm)





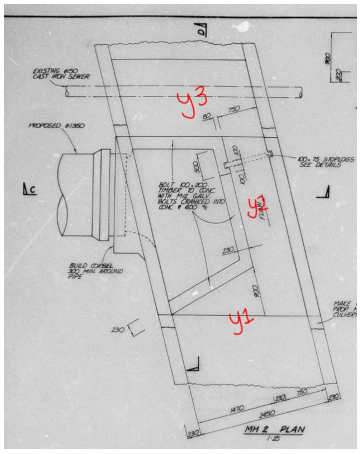
Equations

Orifice equation

$$Q = CoAg\sqrt{2gd}$$

Where:

Q=	Flow through the orifice
Co=	Orifice discharge coefficient
Ag=	Opening area
g=	Gravitational acceleration
d=	Head above greater of the centroid or downstream water surface elevation



Free Weir Flow Equation:

$$Q = C_W L H^{1.5}$$

Where

Q=	Flow over the weir (m3/s)
L=	Weir length (m)
H=	Head above the weir (m)
Cw=	Weir Coefficient

Submerged Weir Flow Equation

For submerged weir flow, use the Villemonte (1947) equation (which was developed for thin-crested weirs)

$$K = \frac{Q_{submerged}}{Q_{free}} = \left[1 - \left(\frac{h2}{h1}\right)^n\right]^{0.385}$$

Where

Q submerg	Flow over the weir considering submergence (m3/s)
Q free=	Free flow over the weir
n=	1.5 for rectangular channel
h2=	downstream water level above weir crest (m)
h1=	upstream water level above weir crest (m)

Inputs:

Co=	0.8	Since the orifice leads to small contraction in width	
g=	9.81	m/s2	
Orifice width	0.7	m	
Orifice height	0.25	m	
Orifice centroid	0.125	m	
Orifice area	0.175	m2	
Cw=	3	Imperial	
Cw=	1.656	Metric	
Weir level=	0.9	m	*above culvert invert
L=	2.5	m	

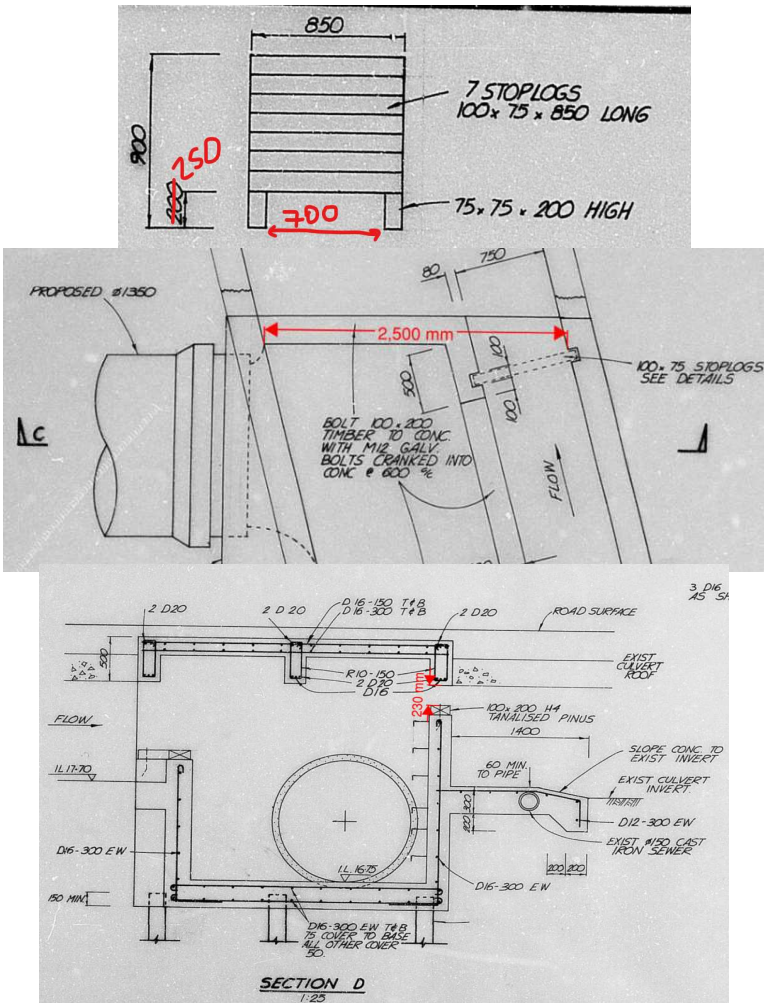
Steps:

1. Select y1 (assume equal to y2) and y3
2. Calculate free flow over the weir using the free flow weir equation
3. Calculate h2 given y3
4. calcualte h1 given y1
5. Calculate K using the Villamonte Equation. For h2<weir level (900 mm), set K=1
6. Calculate submerged weir flow by multiplying K by free weir flow
7. Calculate the total flow as the sum of orifice flow + weir flow

Condition 3: Weir flow over stop logs + orifice flow through 250 mm opening (y1=y2>900 mm)

y2 (m)	Free Weir Q	y3 (m)	h2	h1	K	Submerged Weir Q	Orifice Q (submerged)	Total Calculated Flow (cms)
0.95	0.05	0	-0.9	0.05	1	0.046	0.563	0.610
1	0.13	0	-0.9	0.1	1	0.131	0.580	0.711
1.05	0.24	0	-0.9	0.15	1	0.241	0.596	0.837
1.1	0.37	0	-0.9	0.2	1	0.370	0.612	0.983
0.95	0.046	0.05	-0.85	0.05	1	0.046	0.563	0.610
1	0.131	0.05	-0.85	0.1	1	0.131	0.580	0.711
1.05	0.241	0.05	-0.85	0.15	1	0.241	0.596	0.837
1.1	0.370	0.05	-0.85	0.2	1	0.370	0.612	0.983
0.95	0.05	0.1	-0.8	0.05	1	0.046	0.563	0.610
1	0.13	0.1	-0.8	0.1	1	0.131	0.580	0.711
1.05	0.24	0.1	-0.8	0.15	1	0.241	0.596	0.837
1.1	0.37	0.1	-0.8	0.2	1	0.370	0.612	0.983
0.95	0.046	0.2	-0.7	0.05	1	0.046	0.537	0.583
1	0.131	0.2	-0.7	0.1	1	0.131	0.555	0.686
1.05	0.241	0.2	-0.7	0.15	1	0.241	0.572	0.812
1.1	0.370	0.2	-0.7	0.2	1	0.370	0.588	0.959
0.95	0.05	0.3	-0.6	0.05	1	0.046	0.500	0.546
1	0.13	0.3	-0.6	0.1	1	0.131	0.519	0.650
1.05	0.24	0.3	-0.6	0.15	1	0.241	0.537	0.778
1.1	0.37	0.3	-0.6	0.2	1	0.370	0.555	0.925
0.95	0.046	0.4	-0.5	0.05	1	0.046	0.460	0.506
1	0.131	0.4	-0.5	0.1	1	0.131	0.480	0.611
1.05	0.241	0.4	-0.5	0.15	1	0.241	0.500	0.740
1.1	0.370	0.4	-0.5	0.2	1	0.370	0.519	0.889
0.95	0.05	0.5	-0.4	0.05	1	0.046	0.416	0.462
1	0.13	0.5	-0.4	0.1	1	0.131	0.438	0.569
1.05	0.24	0.5	-0.4	0.15	1	0.241	0.460	0.700
1.1	0.37	0.5	-0.4	0.2	1	0.370	0.480	0.851
0.95	0.046	0.6	-0.3	0.05	1	0.046	0.367	0.413
1	0.131	0.6	-0.3	0.1	1	0.131	0.392	0.523
1.05	0.241	0.6	-0.3	0.15	1	0.241	0.416	0.657
1.1	0.370	0.6	-0.3	0.2	1	0.370	0.438	0.809
0.95	0.05	0.7	-0.2	0.05	1	0.046	0.310	0.356
1	0.13	0.7	-0.2	0.1	1	0.131	0.340	0.471
1.05	0.24	0.7	-0.2	0.15	1	0.241	0.367	0.607
1.1	0.37	0.7	-0.2	0.2	1	0.370	0.392	0.762
0.95	0.046	0.8	-0.1	0.05	1	0.046	0.240	0.286
1	0.131	0.8	-0.1	0.1	1	0.131	0.277	0.408
1.05	0.241	0.8	-0.1	0.15	1	0.241	0.310	0.551
1.1	0.370	0.8	-0.1	0.2	1	0.370	0.340	0.710
0.95	0.05	0.9	0	0.05	1	0.046	0.139	0.185
1	0.13	0.9	0	0.1	1	0.131	0.196	0.327
1.05	0.24	0.9	0	0.15	1	0.241	0.240	0.481
1.1	0.37	0.9	0	0.2	1	0.370	0.277	0.648
1	0.131	1	0.1	0.1	0	0.000	0.000	0.000
1.05	0.241	1	0.1	0.15	0.74	0.178	0.139	0.316
1.1	0.370	1	0.1	0.2	0.85	0.313	0.196	0.509

Condition 4: Orifice flow over the stop logs + orifice flow through the low flow opening



Orifice equation

$$Q = CoAg\sqrt{2gd}$$

Where:

Q=	Flow through the orifice
Co=	Orifice discharge coefficient
Ag=	Opening area
g=	Gravitational acceleration
d=	Head above greater of the centroid or downstream water surface elevation

Inputs:

Low Flow Orifice (250 mm stop log opening)

Co=	0.8	Since the orifice leads to small contraction in width
g=	9.81	m/s2
Orifice width	0.7	m
Orifice height	0.25	m
Orifice centroid	0.125	m
Orifice area	0.175	m2

Condition 4: Orifice flow over the stop logs + orifice flow through the low flow opening

High Orifice (flow over stop logs)

Co=	0.6	*typical orifice coefficient
Orifice Width	2.5	m
Orifice Height	0.23	m
Orifice Area	0.575	m2
Centroid	1.015	

Steps:

1. Select y1 (assume equal to y2) and y3
2. Calculate flow over the high orifice using the orifice equation and high orifice inputs
3. Calculate flow over the low orifice using the orifice equation and low orifice inputs
4. Combine flow from both orifices to determine the total flow

y2 (m)	y1 (m)	High Orifice Q	y3	Low Orifice Q	Total Orifice Q
1.15	1.15	0.56	0.000	0.628	1.189
1.2	1.2	0.66	0.000	0.643	1.300
1.3	1.3	0.82	0.000	0.672	1.488
1.4	1.4	0.95	0.000	0.700	1.648
1.5	1.5	1.06	0.000	0.727	1.791
1.6	1.6	1.17	0.000	0.753	1.922
1.7	1.7	1.26	0.000	0.778	2.043
1.8	1.8	1.35	0.000	0.803	2.157
1.9	1.9	1.44	0.000	0.826	2.264
2	2	1.52	0.000	0.849	2.366
2.1	2.1	1.59	0.000	0.871	2.463
1.15	1.15	0.56	0.050	0.628	1.189
1.2	1.2	0.66	0.050	0.643	1.300
1.3	1.3	0.82	0.050	0.672	1.488
1.4	1.4	0.95	0.050	0.700	1.648
1.5	1.5	1.06	0.050	0.727	1.791
1.6	1.6	1.17	0.050	0.753	1.922
1.7	1.7	1.26	0.050	0.778	2.043
1.8	1.8	1.35	0.050	0.803	2.157
1.9	1.9	1.44	0.050	0.826	2.264
2	2	1.52	0.050	0.849	2.366
2.1	2.1	1.59	0.050	0.871	2.463
1.15	1.15	0.56	0.100	0.628	1.189
1.2	1.2	0.66	0.100	0.643	1.300
1.3	1.3	0.82	0.100	0.672	1.488
1.4	1.4	0.95	0.100	0.700	1.648
1.5	1.5	1.06	0.100	0.727	1.791
1.6	1.6	1.17	0.100	0.753	1.922
1.7	1.7	1.26	0.100	0.778	2.043
1.8	1.8	1.35	0.100	0.803	2.157
1.9	1.9	1.44	0.100	0.826	2.264
2	2	1.52	0.100	0.849	2.366
2.1	2.1	1.59	0.100	0.871	2.463

Condition 4: Orifice flow over the stop logs + orifice flow through the low flow opening

y2 (m)	y1 (m)	High Orifice Q	y3	Low Orifice Q	Total Orifice Q
1.15	1.15	0.56	0.200	0.604	1.166
1.2	1.2	0.66	0.200	0.620	1.277
1.3	1.3	0.82	0.200	0.650	1.466
1.4	1.4	0.95	0.200	0.679	1.628
1.5	1.5	1.06	0.200	0.707	1.771
1.6	1.6	1.17	0.200	0.734	1.903
1.7	1.7	1.26	0.200	0.759	2.024
1.8	1.8	1.35	0.200	0.784	2.138
1.9	1.9	1.44	0.200	0.809	2.246
2	2	1.52	0.200	0.832	2.349
2.1	2.1	1.59	0.200	0.855	2.447
1.15	1.15	0.56	0.300	0.572	1.133
1.2	1.2	0.66	0.300	0.588	1.246
1.3	1.3	0.82	0.300	0.620	1.436
1.4	1.4	0.95	0.300	0.650	1.599
1.5	1.5	1.06	0.300	0.679	1.744
1.6	1.6	1.17	0.300	0.707	1.876
1.7	1.7	1.26	0.300	0.734	1.999
1.8	1.8	1.35	0.300	0.759	2.113
1.9	1.9	1.44	0.300	0.784	2.222
2	2	1.52	0.300	0.809	2.325
2.1	2.1	1.59	0.300	0.832	2.424
1.15	1.15	0.56	0.400	0.537	1.099
1.2	1.2	0.66	0.400	0.555	1.212
1.3	1.3	0.82	0.400	0.588	1.404
1.4	1.4	0.95	0.400	0.620	1.568
1.5	1.5	1.06	0.400	0.650	1.715
1.6	1.6	1.17	0.400	0.679	1.848
1.7	1.7	1.26	0.400	0.707	1.972
1.8	1.8	1.35	0.400	0.734	2.088
1.9	1.9	1.44	0.400	0.759	2.197
2	2	1.52	0.400	0.784	2.301
2.1	2.1	1.59	0.400	0.809	2.400
1.15	1.15	0.56	0.500	0.500	1.061
1.2	1.2	0.66	0.500	0.519	1.176
1.3	1.3	0.82	0.500	0.555	1.370
1.4	1.4	0.95	0.500	0.588	1.536
1.5	1.5	1.06	0.500	0.620	1.684
1.6	1.6	1.17	0.500	0.650	1.819
1.7	1.7	1.26	0.500	0.679	1.944
1.8	1.8	1.35	0.500	0.707	2.061
1.9	1.9	1.44	0.500	0.734	2.171
2	2	1.52	0.500	0.759	2.276
2.1	2.1	1.59	0.500	0.784	2.376
1.15	1.15	0.56	0.600	0.460	1.021
1.2	1.2	0.66	0.600	0.480	1.138
1.3	1.3	0.82	0.600	0.519	1.335
1.4	1.4	0.95	0.600	0.555	1.503
1.5	1.5	1.06	0.600	0.588	1.653
1.6	1.6	1.17	0.600	0.620	1.789
1.7	1.7	1.26	0.600	0.650	1.915

Condition 4: Orifice flow over the stop logs + orifice flow through the low flow opening

y2 (m)	y1 (m)	High Orifice Q	y3	Low Orifice Q	Total Orifice Q
1.8	1.8	1.35	0.600	0.679	2.033
1.9	1.9	1.44	0.600	0.707	2.145
2	2	1.52	0.600	0.734	2.250
2.1	2.1	1.59	0.600	0.759	2.351
1.15	1.15	0.56	0.700	0.416	0.977
1.2	1.2	0.66	0.700	0.438	1.096
1.3	1.3	0.82	0.700	0.480	1.296
1.4	1.4	0.95	0.700	0.519	1.467
1.5	1.5	1.06	0.700	0.555	1.619
1.6	1.6	1.17	0.700	0.588	1.757
1.7	1.7	1.26	0.700	0.620	1.885
1.8	1.8	1.35	0.700	0.650	2.004
1.9	1.9	1.44	0.700	0.679	2.117
2	2	1.52	0.700	0.707	2.224
2.1	2.1	1.59	0.700	0.734	2.326
1.15	1.15	0.56	0.800	0.367	0.928
1.2	1.2	0.66	0.800	0.392	1.049
1.3	1.3	0.82	0.800	0.438	1.254
1.4	1.4	0.95	0.800	0.480	1.429
1.5	1.5	1.06	0.800	0.519	1.583
1.6	1.6	1.17	0.800	0.555	1.723
1.7	1.7	1.26	0.800	0.588	1.853
1.8	1.8	1.35	0.800	0.620	1.974
1.9	1.9	1.44	0.800	0.650	2.088
2	2	1.52	0.800	0.679	2.196
2.1	2.1	1.59	0.800	0.707	2.299
1.15	1.15	0.56	0.900	0.310	0.872
1.2	1.2	0.66	0.900	0.340	0.997
1.3	1.3	0.82	0.900	0.392	1.208
1.4	1.4	0.95	0.900	0.438	1.387
1.5	1.5	1.06	0.900	0.480	1.545
1.6	1.6	1.17	0.900	0.519	1.688
1.7	1.7	1.26	0.900	0.555	1.819
1.8	1.8	1.35	0.900	0.588	1.942
1.9	1.9	1.44	0.900	0.620	2.058
2	2	1.52	0.900	0.650	2.167
2.1	2.1	1.59	0.900	0.679	2.271
1.15	1.15	0.56	1.000	0.240	0.802
1.2	1.2	0.66	1.000	0.277	0.935
1.3	1.3	0.82	1.000	0.340	1.155
1.4	1.4	0.95	1.000	0.392	1.340
1.5	1.5	1.06	1.000	0.438	1.503
1.6	1.6	1.17	1.000	0.480	1.649
1.7	1.7	1.26	1.000	0.519	1.784
1.8	1.8	1.35	1.000	0.555	1.909
1.9	1.9	1.44	1.000	0.588	2.026
2	2	1.52	1.000	0.620	2.137
2.1	2.1	1.59	1.000	0.650	2.242
1.2	1.2	0.48	1.100	0.196	0.679
1.3	1.3	0.68	1.100	0.277	0.961
1.4	1.4	0.84	1.100	0.340	1.177

Condition 4: Orifice flow over the stop logs + orifice flow through the low flow opening

y2 (m)	y1 (m)	High Orifice Q	y3	Low Orifice Q	Total Orifice Q
1.6	1.6	1.08	1.100	0.438	1.519
1.8	1.8	1.28	1.100	0.519	1.797
2	2	1.45	1.100	0.588	2.038
1.2	1.2	0.00	1.200	0.000	0.000
1.3	1.3	0.48	1.200	0.196	0.679
1.4	1.4	0.68	1.200	0.277	0.961
1.6	1.6	0.97	1.200	0.392	1.359
1.8	1.8	1.18	1.200	0.480	1.664
2	2	1.37	1.200	0.555	1.921
1.3	1.3	0.00	1.300	0.000	0.000
1.4	1.4	0.48	1.300	0.196	0.679
1.6	1.6	0.84	1.300	0.340	1.177
1.8	1.8	1.08	1.300	0.438	1.519
2	2	1.28	1.300	0.519	1.797
1.4	1.4	0.00	1.400	0.000	0.000
1.5	1.5	0.48	1.400	0.196	0.679
1.6	1.6	0.68	1.400	0.277	0.961
1.7	1.7	0.84	1.400	0.340	1.177
1.8	1.8	0.97	1.400	0.392	1.359
1.9	1.9	1.08	1.400	0.438	1.519
2	2	1.18	1.400	0.480	1.664
2.1	2.1	1.28	1.400	0.519	1.797
1.6	1.6	0.00	1.600	0.000	0.000
1.7	1.7	0.48	1.600	0.196	0.679
1.8	1.8	0.68	1.600	0.277	0.961
1.9	1.9	0.84	1.600	0.340	1.177
2	2	0.97	1.600	0.392	1.359
2.1	2.1	1.08	1.600	0.438	1.519
1.8	1.8	0.00	1.800	0.000	0.000
1.9	1.9	0.48	1.800	0.196	0.679
2	2	0.68	1.800	0.277	0.961
2.1	2.1	0.84	1.800	0.340	1.177
2	2	0.00	2.000	0.000	0.000
2.1	2.1	0.48	2.000	0.196	0.679

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S4	0	Madeline Howell	Tim Preston		Roanna Purcaru		4/4/2025

GHD
138 Victoria Street, Level 3
Christchurch Central, Canterbury 8013, New Zealand
T +64 3 378 0900 | **F** +64 3 377 8575 | **E** chcmail@ghd.com | **ghd.com**

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Appendix Q – Investigation report on flooding near Riccarton Railway, February 2024

Riccarton Railway

Investigation on flooding near Riccarton Railway- Interim Report

Christchurch City Council

February 21, 2024

→ **The Power of Commitment**



Project name		Avon River Modelling Improvements					
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S3	A	Sarah Millerick	Estelle Boivin Tim Preston	On file	Tim Preston	On file	21-2-24

GHD
Contact: Tim Preston, Project Director |
138 Victoria Street,
Christchurch, 8013, New Zealand
T 03 378 0900 | **E** chcmail@ghd.com

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Contents

1. Introduction **1**
1.1 Purpose of this report 1
1.2 Expected readership 1
2. Scope and limitations **1**
2.1 Scope of work 1
2.2 Limitations 2
2.3 Description of network 2
3. Methodology **4**
3.1 Desktop assessment 4
3.1.1 Site visit 4
3.1.2 Items not performed 5
4. Desktop Study Findings **5**
4.1 General 5
4.2 Monitoring Data Investigation 6
4.2.1 Existing Monitoring Data 6
4.2.2 Additional monitoring sites 6
4.3 Pipe capacity 7
4.4 Historical Flooding 9
4.5 Inlets Points 10
4.6 New GIS Information 10
4.7 Channel overflow at Waimairi Road 10
4.8 Flow Balance in the Upper Riccarton Vicinity 11
4.9 Main Pipe Geometry 11
4.10 Free draining infiltration rate 13
4.11 WSP report 13
4.12 Updates recommended for GIS data 13
5. Model Upgrades Performed and Future Recommendations **13**

Table index

Table 1 Dimensions of the Picton Ave Culvert 12
Table 2 Culvert Dimensions for the Model 12

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Figure index

Figure 1	The investigated area for flooding in the model, located within the boundaries defined by Deans Ave, Riccarton Road, Clarence Street, and Blenheim Road.	3
Figure 2	Flooding depth in area of interest, for a 1 in 10 year - 18hour rainfall event in 2020.	5
Figure 3	Monitoring Data at Riccarton Drain Deans Ave and Riccarton Ave.	6
Figure 4	Other for available monitoring data for upper Avon River catchment.	7
Figure 5	Comparison of pipe water level and floodplain water level, where the circular icons show the max pipe WL.	8
Figure 6	Long sectional view of Riccarton SW Main (route 1), approx. location between Picton Ave Manhole3511 and the Railway Tracks Manhole3859.	8
Figure 7	Flow severity in the pipes for a 1 in 10 year, 18 hour rainfall event.	9
Figure 8	Flow extraction lines.	11

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1. Introduction

The current Avon model results have issues at the eastern portion of Riccarton, in the vicinity of Kyle Street. Currently, the model predicts serious flooding behind the railway embankment, however there is no historical evidence of this occurring. This is resulting in minimal confidence in, and uncertain use of the model results for this area.

The main emphasis of this memo was focused on moderate flood occurrences that peak well below the railway crest level (10-year ARI model result and the March 2014 calibration event). This scope excludes determining how to improve modelling of the most severe flooding, where the water overflows the crest level of the railway embankment. This was being addressed separately within the working group.[1]

The key variables for this area of interest have been identified as the following:

- Capacity (including inlets) of the Riccarton Main Drain (route 1, see Figure 2 below)
- Hydrology, infiltration, and flow rates in the upstream western catchments
- Capacity of the upstream river channel near Waimairi Road

Investigations in this memo were prioritised based on low cost and high expected learning value tasks first, as well as an early start on long lead time items like new flow monitoring recommendations.

1.1 Purpose of this report

Investigation of the stormwater network on site against model predictions. This is to identify possible causes of the model limitations.

1.2 Expected readership

The reader of this report is expected to have background knowledge of the existing model limitations in simulating the observed historical responses in Riccarton for various rainfall events including: the 1 in 10yr 18hr, 1 in 50yr 3hr and 1 in 50yr 18hr rain event.

2. Scope and limitations

2.1 Scope of work

Investigations were prioritised based on low cost and high expected learning value tasks first, as well as an early start on long lead time items like new flow monitoring recommendations. The methodology to identify the Riccarton Railway issues required the following:

- Desktop Investigations and validation of the model pipe hydraulic and inlet capacities at location of flooding
- Site visit validation of assets
- Investigation and validation of model geometry at the predicted channel overflow location (Waimairi Road)
- A review of the free draining infiltration rate knowledge

[1] "Modelling Rail Embankments - For Discussion", MEMO, 2 June 2023, Mark Groves WSP

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The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report. GHD disclaims liability arising from any of the assumptions being incorrect.

Accessibility of documents

If this report is required to be accessible in any other format, this can be provided by GHD upon request and at an additional cost if necessary.

The opinions, conclusions and any recommendations in this report based on information obtained from site visits performed in the Riccarton Vicinity were completed to focus on the Riccarton flood results at this stage of the project.

Investigations undertaken in respect of this report are constrained by the particular site conditions, such as the location of infrastructure, services and vegetation. As a result, not all relevant site features and conditions may have been identified in this report.

2.3 Description of network

The scope of this investigation encompasses the network within the Avon catchment, with a specific focus on stormwater flooding and infrastructure located within the boundaries defined by Deans Ave, Riccarton Road, Clarence Street, and Blenheim Road. There is an exception for this scope, this is the investigation on predicted channel overflows upstream from the flooding (Waimairi Road).

The following Figure 2 depicts the network area investigated.

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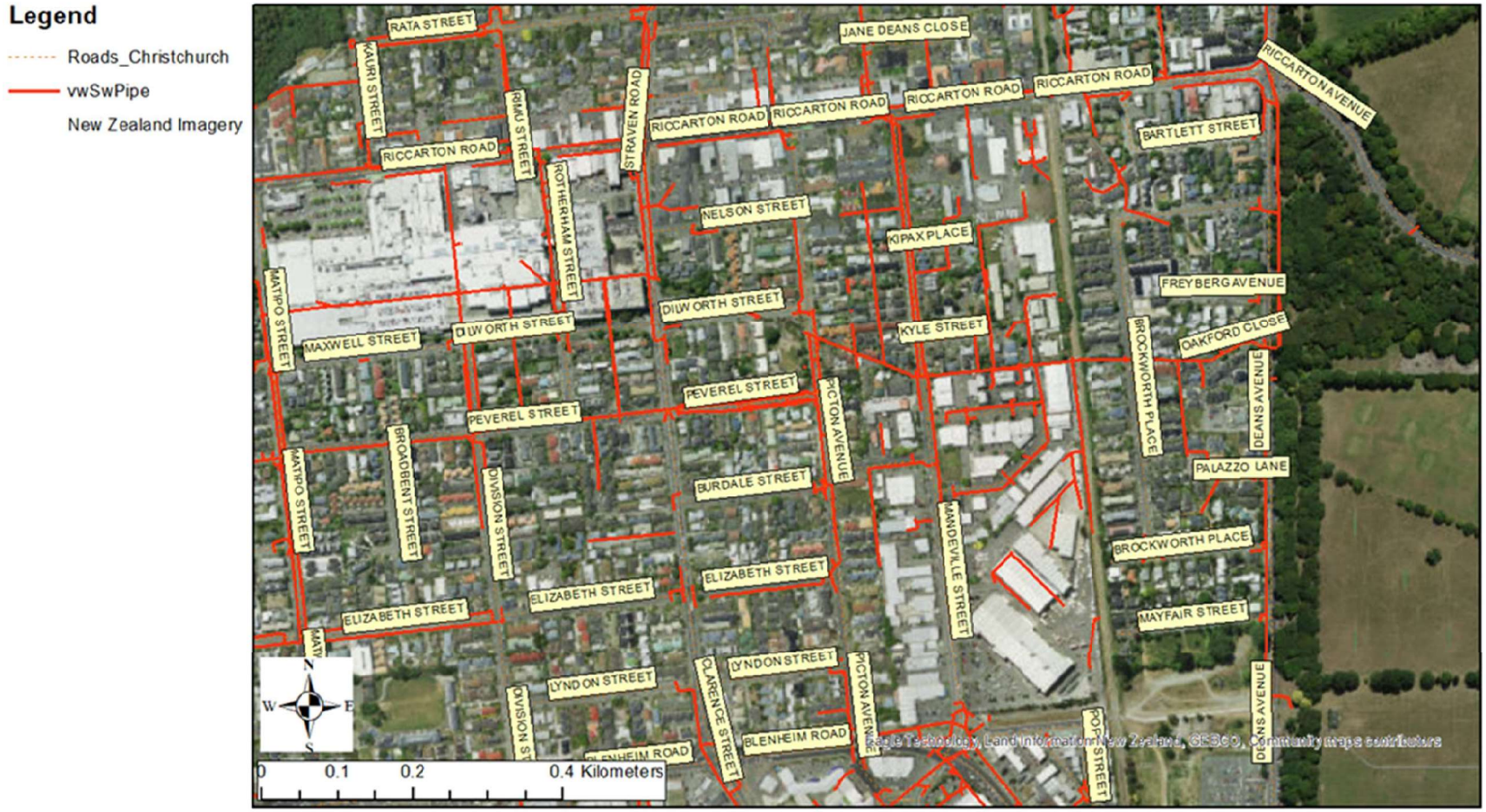


Figure 1 *The investigated area for flooding in the model, located within the boundaries defined by Deans Ave, Riccarton Road, Clarence Street, and Blenheim Road.*

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3. Methodology

The initial analysis consisted of analysing the model results for the 1 in 10 yr, 18 hr rainfall occurrence in the ED2020 model. Furthermore, the 1-in-50 yr, 3 hr and 18 hr events for ED2020 were explored to investigate channel flow in the upper reaches of Riccarton. The calibration event from March 2014 was also analysed as part of this study.

3.1 Desktop assessment

An initial desktop assessment was completed to assess the model results, this included the following:

- A model results inspection of pipe capacity, points of high head loss, and variation with different rainfall events including the March 2014 calibration event. This was done in both ArcGIS and Mike View for the model data.
- Review of the available monitoring data to determine if more flow data should be recommended and to prepare for the validation of the 2017 rainfall event model. Aquarius was used to review rainfall data and water level at the discharge point of Riccarton Main Drain (Hagley Park Avon River).
- Review of aerial photos and LiDAR to validate inlet locations. Arial photos were reviewed using GIS and Google Earth Pro.
- A comparison with current GIS and older GIS used to build the model and records within the model geometry.
- Review of as-builts for the main Riccarton SW pipe (route 1, see Figure 2 below) that discharges to Hagley Park.
- Follow up with NIWA regarding history of flooding at the location of Kyle Street, to validate if flooding has occurred in the Riccarton vicinity.
- A preview of existing CCTV footage.
- A review of Christchurch City Council (CCC) spatial photo database via SmartMaps.
- A review of CCC flood complaint records in the Riccarton area. To identify records of flooding in the Riccarton vicinity.
- Investigation at the predicted channel overflow location (Waimairi Road).
- A review of the free draining infiltration rate knowledge
- A review of “Addington Brook & Riccarton Stream Stormwater Treatment - Gap Analysis Report, June 2020 by WSP”.

3.1.1 Site visit

A site visit was performed in the Riccarton vicinity to validate findings from desktop study.

- The site visit included inspections of inlets, missing culverts, missing culvert diameters and downpipes on industrial buildings.

Limitations:

- Unable to determine diameter of SW Main as both accesses had square culverts before the pipe.
- Unable to access area near railway to inspect pipe inlets at railway, due to railway being fenced off for safety reasons
- When measuring culvert diameters, measurements have some uncertainty due to limitations in access.

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3.1.2 Items not performed

- Investigation and validation of model geometry at the predicted channel overflow location (Waimairi Road). This was not performed due to a preliminary assessment of the area showing that the flooding in Riccarton Vicinity is likely not a direct result of overflow from the river or pipe in Waimairi Area.

4. Desktop Study Findings

4.1 General

The model data for the 1-in-10 yr, 1hr rainfall event in ED2020 showed flooding occurring on the floodplain in the Riccarton vicinity.

There is a prominent stormwater pipe located in the centre of the area of flooding to which many of the smaller pipes connect (route 1 to Hagley in Figure 2). This main pipe, which we'll refer to as the "Riccarton SW Main" for the purpose of this report, channels stormwater from Riccarton into the stream at Hagley Park.

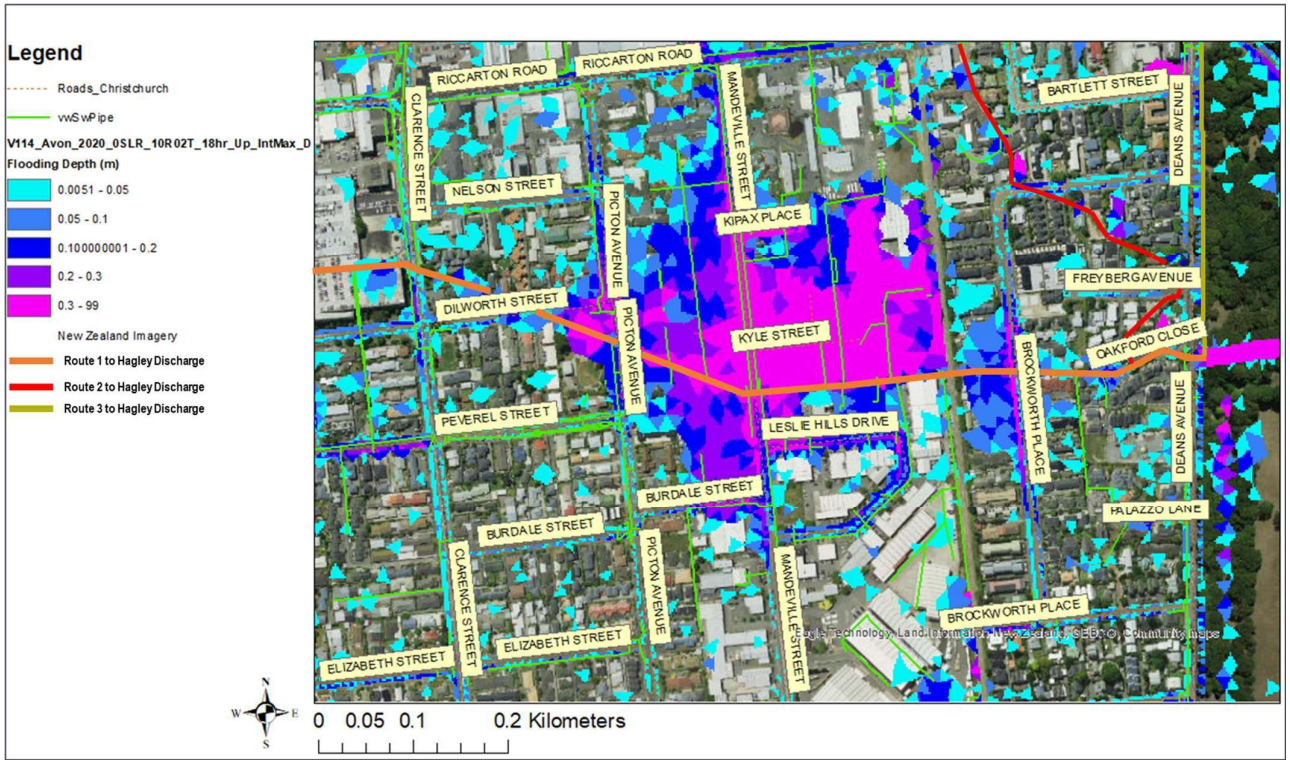


Figure 2 Flooding depth in area of interest, for a 1 in 10 year - 18hour rainfall event in 2020.

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4.2 Monitoring Data Investigation

4.2.1 Existing Monitoring Data

There is flow monitoring data available for Riccarton Drain (NIWA), monitored at both Deans Ave and Riccarton Ave (see Figure 3). While Riccarton Ave's data spans 2016 to 2022, Deans Ave's extends from 2021 to present. The Deans Ave discharge data is considered more reliable with the Riccarton Ave site suspected of backwater influence. There is additional monitoring data (5-minute intervals) in the upper reaches of Avon.

It is recommended that the preferred event for validating or calibrating the Riccarton Main Drain catchment should be with the 26th July 2022 rainfall event. The preferred event for validating or calibrating the upper Avon River catchment (wider range of monitoring locations north the Riccarton catchment which have data between 2016 to 2018 should be calibrated with the 22nd July 2017 event. This data should then be used to validate the model[2].

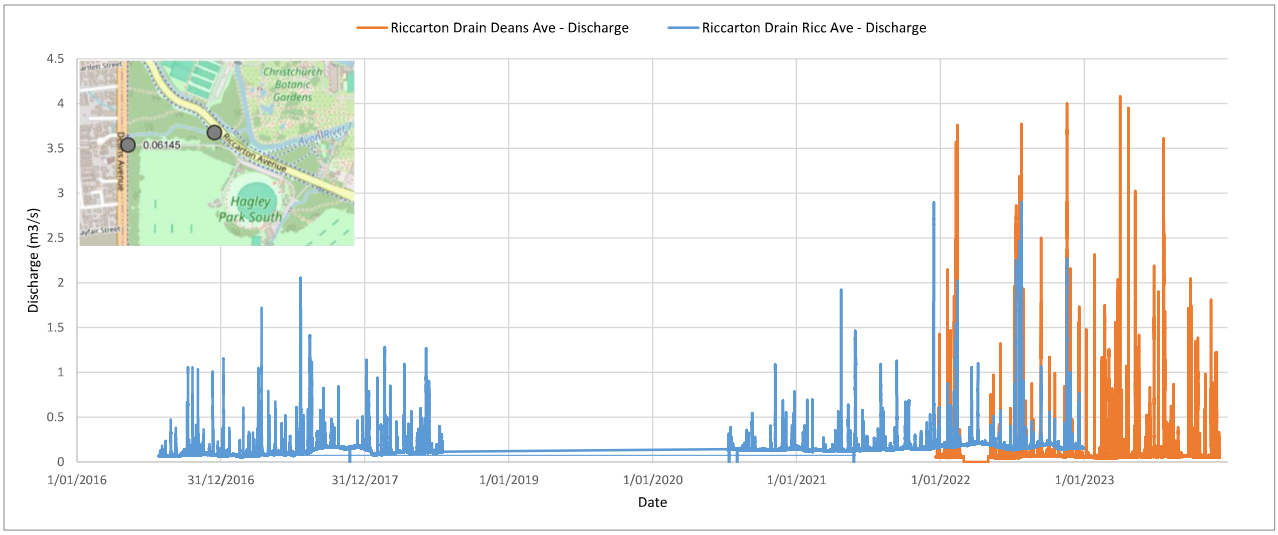


Figure 3 Monitoring Data at Riccarton Drain Deans Ave and Riccarton Ave.

4.2.2 Additional monitoring sites

Once results from the updated model are received, additional monitoring sites may be recommended to validate the results from the updated model. Theise monitoring sites would need to identify locations that would pick up the early stages of the flooding which is occurring in the model. Possible locations are Picton Ave or Clarence Park identified in . As of Feb 2024, improvements in the model are expected to make substantial changes in the results to the extent that monitoring may no longer be motivated, or if motivated that new model results could substantially change our understanding of system performance and conclusions as to where monitoring would be most desirable.

[2] There are two rainfall events which peak at a flow greater than 3.5 m^3/s which would work for validating the model. The dates for these events are 26th July 2022 and 12th February 2022. The 26th July 2022 event seems to have the best set of data (less noisy than the 12th).

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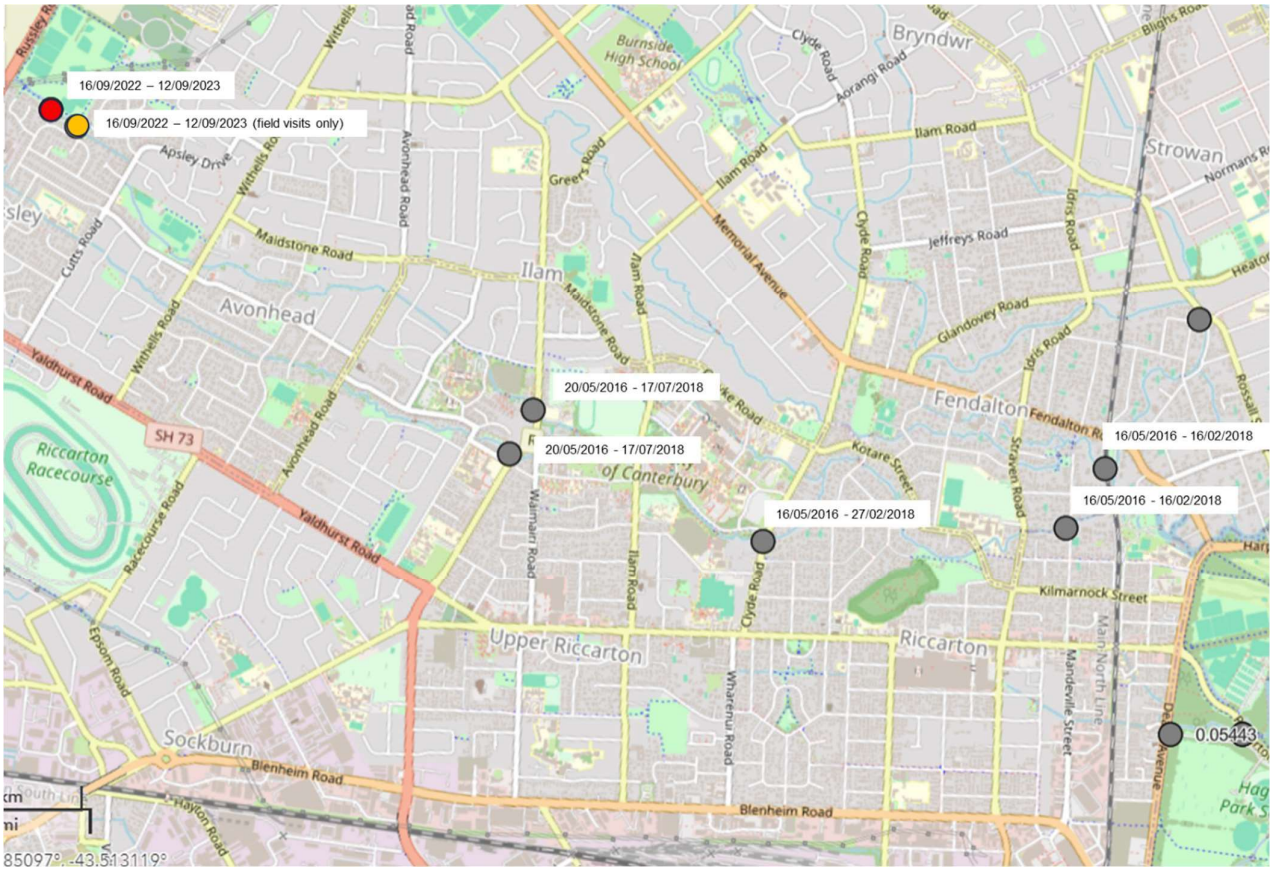


Figure 4 *Other for available monitoring data for upper Avon River catchment.*

4.3 Pipe capacity

Investigations using MIKE11 and ArcGIS on pipe capacity, floodplain, and water levels were conducted for a 1 in 10yr, 18hr rainfall event in ED2020. Results indicated a significant level difference between the floodplain and pipe water levels, particularly near the flooding area towards the railway.

Most of the SW pipes from Picton Ave to the Railway linked to the Riccarton SW Main, are filled to their maximum water level (WL). However, the WL is typically below the manhole tops, indicating unused network capacity. This suggests an unexpected interaction between the pipe (Mike Urban) and the floodplain (Mike 21), especially given the higher floodplain levels despite available network capacity.

Furthermore, many of the pipes discharging into the Riccarton SW Main have a lower than expected flow rates, suggesting that the Riccarton SW Main could potentially handle a higher flowrate.

The following Figure 5 depicts the difference between the max water level on floodplain and in the pipe. The water level in the pipe is represented by circular markers, while the floodplain is depicted by a solid colour on the map. A clear contrast in colour between the two signifies a clear difference in the water levels of the floodplain and the pipe, as shown in legend.

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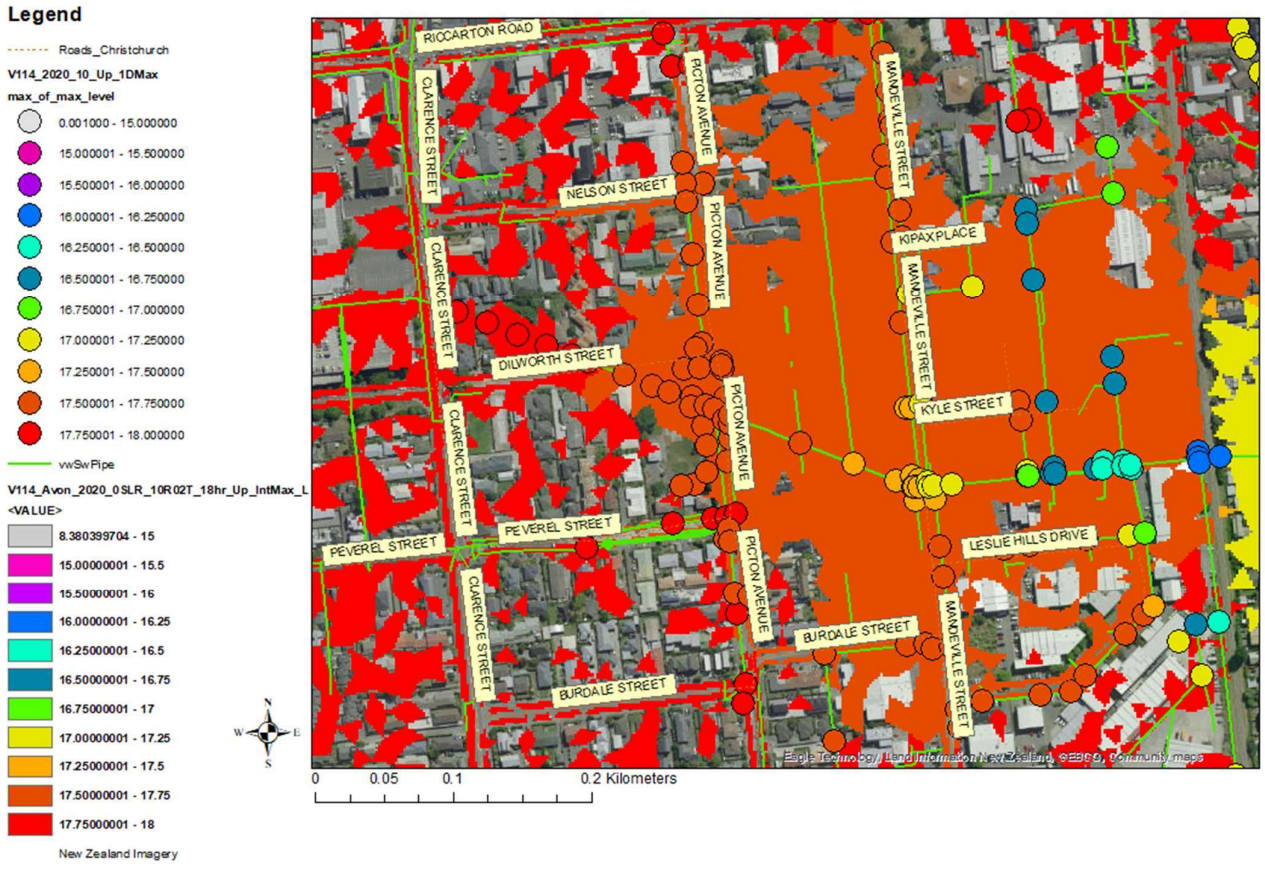


Figure 5 *Comparison of pipe water level and floodplain water level, where the circular icons show the max pipe WL.*

The following Figure 6 displays the WL inside the Riccarton SW Main (route 1) from the manhole at the Picton Ave interception (ID 3511) to the manhole following Riccarton Railway (ID3859). The Riccarton SW Main WL is shown to be full across the entire stretch studied, however, the manholes within the pipe are not all at full WL. Head loss is observed within pipe, shown by the gradual decline in WL as the pipe passes each manhole or junction point modelled.

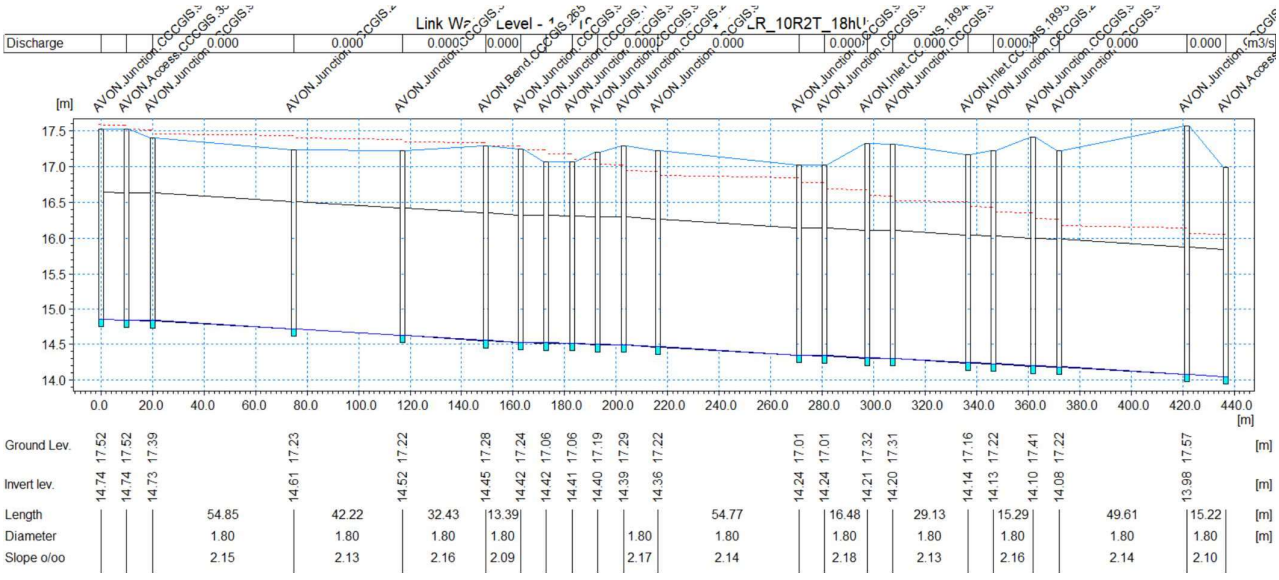


Figure 6 *Long sectional view of Riccarton SW Main (route 1), approx. location between Picton Ave Manhole3511 and the Railway Tracks Manhole3859.*

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Riccarton SW Main (route 1) flow was analysed in multiple places across the pipe and in most places depicted a bell-shaped curve over the 18-hour rainfall event.

When analysing the flow rates in each pipe discharging into the Riccarton SW Main, it was observed that some of them were not discharging at the expected rate given their size.

The following Figure 7 depicts the flow in the pipes. The chart defines the flow in four categories:

- Low flow — Flow is < 50% of cross sectional area
- Expected flow — Flow is > 50% of cross sectional area & < cross sectional area
- High flow — Flow is > cross sectional area
- Zero flow — Flow is < 0.001 m^3/s

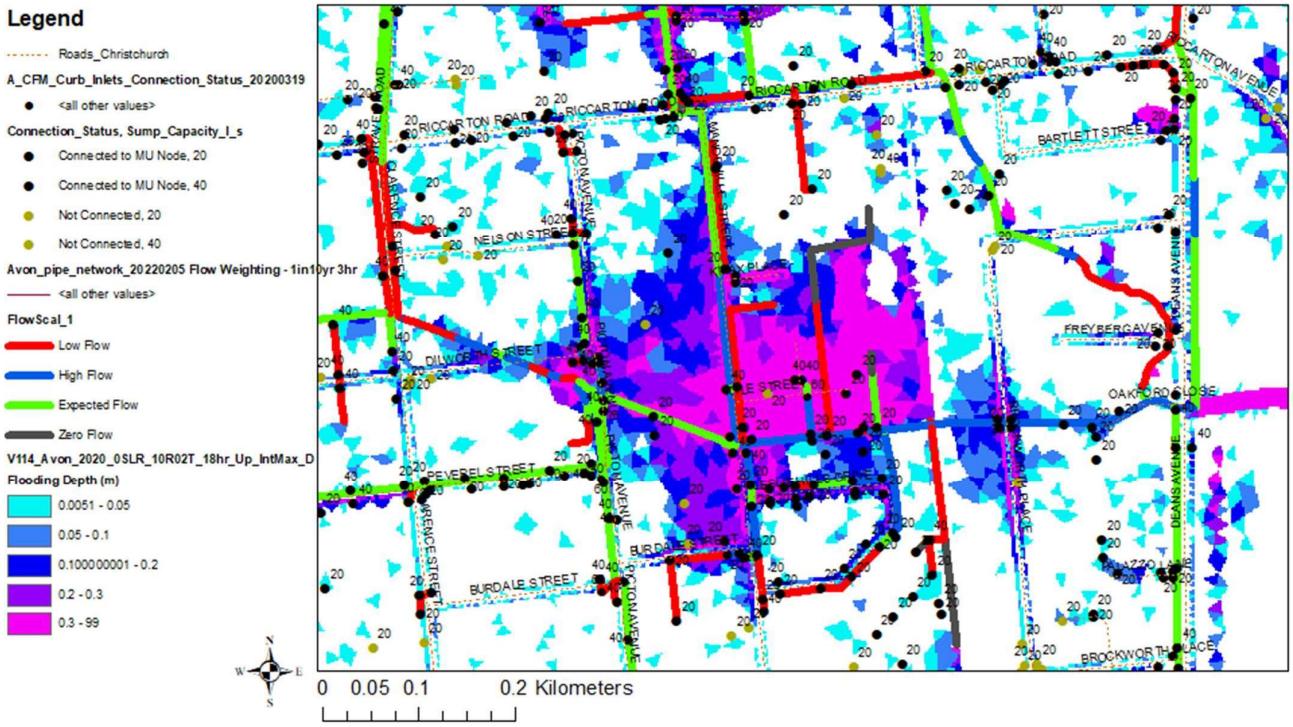


Figure 7 *Flow severity in the pipes for a 1 in 10 year, 18 hour rainfall event.*

For areas where the flow is low, but flooding is occurring in floodplain, this is a sign that there are not enough inlets on the pipe. It should be noted that the low flow in route 2 (route is displayed in Figure 2) is occurring in an open channel area so can be ignored.

4.4 Historical Flooding

The March 2014 model was analysed, and it appears to show the similar flooding occurring as shown the 1 in 10 yr, 18 hr rain event. The flooding is shown to occur in the Riccarton vicinity, which did not occur historically with evidence of this is outlined below:

- GHD contacted NIWA for information on the history of flooding at their site location (Kyle Street). NIWA confirmed they have not had any flooding (apart from localised flooding caused by blocked drains) since August 1992. [4]
- The CCC flood complaints tickets (both active and inactive) were checked in Smart Map, these showed no history of flooding due to rainfall events in the area (aside from a few tickets due to blocked drains).
- The CCC Snap Send Solve Layer in Smart Map was checked and had some flooding photos in the Riccarton vicinity, but they did not appear to be due to rainfall events rather blocked drains.

[4] GHD email to Tim Preston: "Understanding of flooding around the NIWA office", 30/110/2023 9:15 AM.

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4.5 Inlets Points

An examination of the curb inlets was performed, using the online and offline CCC GIS, Google Earth Street View and LiDAR. This was undertaken to identify any inlets that were absent or inaccurately modelled in the model. In addition to this, missing curb inlets near Kyle Street and downpipes were investigated during the site visit. The as-builts for the Riccarton drain were also analysed for inlets. The following is a summary of the findings:

- We identified six inlets in commercial area north of 10 Kyle St that were not in current GIS (some of these we have assumed to connect up to an identified stormwater pipe, approximately 290m in length, that appeared to have no inlets, and having no flow within the model).
- We identified 15 other sumps in GIS that do not have clear connectivity (and hence were not modelled), These were connected up to the model with assumptions made to define and model this connectivity.
- We found two new sumps in the current GIS from roading modernisation on Picton Ave. These have been added to the model.
- During the site visit it was discovered that near Placemakers many of the industrial buildings have down pipes from the roof. These are not included in the model although some of these are shown on the online GIS. Most industrial buildings had one at each corner and one between each corner.
- During the site visit, it was discovered that a private drain exists at NIWA which is connected to a water treatment system (for the boat wash water). This water is released manually to the Council stormwater network after being tested and treated as required. NIWA confirmed they would not release this water during a flood event.
- The as-builts of the Riccarton SW Main and LiDAR showed an open drain discharging to Pipe ID18031 near the Niwa Office and Railway site. This 450 mm diameter pipe discharges into the Riccarton SW Main and could hold a considerable flow. This is currently modelled as just a pipe with no inlet and no flow, and the open drain is not modelled. The model was updated to add a full capacity inlet at the 450 pipe end.
- The LiDAR also indicates an open drain at the 600 mm diameter pipe end on KiwiRail land further south toward Placemakers (SWPipeID26414). The model does not have an inlet at this location. The model was updated to add a full capacity inlet to the 600 pipe end.

4.6 New GIS Information

A comparison between online 3-Waters Network Asset Map and the ArcMap GIS (2015) for the Christchurch network was performed, the following items were found for which the ArcMap GIS does not have:

- The online GIS includes building roof drainage in the “Stormwater (not Council owned)” layer, this is not shown on the offline AVON private pipe network.
- Two new Picton Ave sump/inlets on the online GIS

4.7 Channel overflow at Waimairi Road

Previous study of the modelling issue has shown high sensitivity of flood results to predicted channel overflows at the upper reaches of the Avon River (approximately 3km upstream from the area of flooding, near Waimairi Road). Due to the Avon River’s reverse topography, there is a possibility that the out-of-channel flows would cross into the Riccarton Main Drain catchment and create flooding around Kyle Street.

A preliminary analysis was conducted on the overflows at the upper reaches of the Avon River (Waimairi Road) for two ED2020 design rainfall events. The first event, a 1 in 10yr rainfall event lasting 18hrs, showed no signs of the river overflowing. This led to a further analysis of a more intense 1 in 50yr, 3hr rain event to determine if any overflow from the Avon River was contributing to the flow.

The river flow near Peer St and Waimairi Road was examined to ascertain if water was exiting the river. Simultaneously, the flow within the pipes was also analysed. The findings revealed that:

– Both the river and pipes experienced an increased flow during the flood event. However, there was no significant evidence of a substantial amount of flow leaving the river or pipes at the Waimairi location.
– There were indications that the flow into the river within the pipes was slowing down due to the river reaching its full capacity.

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Therefore, it was concluded that the pooling observed in the Riccarton vicinity is likely not a direct result of overflow from the river or pipes, suggesting that other factors may be responsible.

4.8 Flow Balance in the Upper Riccarton Vicinity

Flow extraction lines were setup as shown in Figure 8, to extract data from the saved model results. This showed considerable flow through Flow Extraction Line 9 which was not evident further upstream on the floodplain. Flow Extraction Line 10 (which appears to feed directly into Flow Extraction Line B) only shows a peak flow of 0.5 m^3/s.

It is likely some of this flow is derived from the rainfall occurring in the approximately 1 km^2 area. There are three main pipes in this area discharging water into the river. Based off the model one of these pipes has an expected flow and two of the pipes have a high flow (no pipes are overflowing or at the top of manhole).

The source of the high flow through Extraction Line 9 was investigated, and a flow balance was performed on the area of interest (inside flow extraction lines 9, 10 and B - approximately 1 km^2 area) at peak water level in the floodplain (2.25 hours into the rain event). By totalling inflows from rainfall and floodplain, and removing outflows through pipes and floodplain, the infiltration rate over this area was estimated at 0.6 m^3/s. This is 12% of the inflow from rainfall which is lower than expected. This inconsistency could mean that there is an issue with the way the model is functioning (potential artificial flow generation) or that the floodplain flow extraction processes are producing misleading timeseries data. This finding could motivate further investigation. It is not presently considered of sufficient impact to pursue.

It also noted that flow extraction lines 11 and 12 as labelled in the map were observed to have an unusual flow pattern. This pattern was characterized by two distinct peak events. The first peak, which was smaller, occurred within a 3-hour timeframe. Following this, a second, larger peak was observed after the initial 3-hour period, which demonstrated a higher flow throughput. The higher throughput is likely due to flooding from upstream locations tracking its way to the location. This was not further investigated.

In summary, there seems to be a significant amount of modelled water flow in the area that is apparently not consistent with expectations. If new model results continue to show excessive flooding then attention on this area will increase.

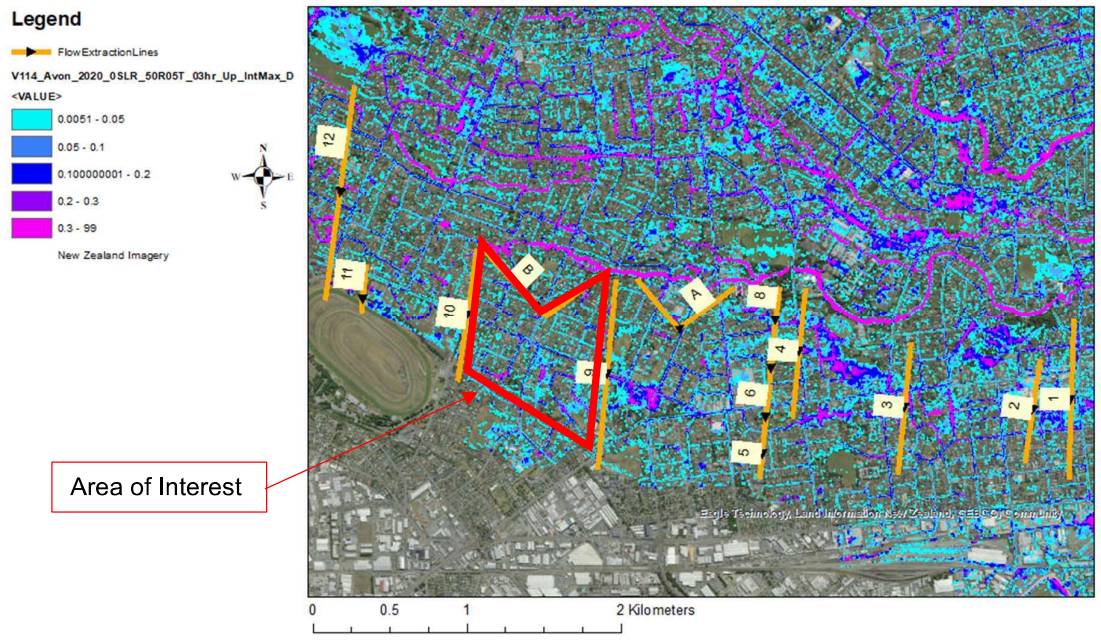


Figure 8 Flow extraction lines.

4.9 Main Pipe Geometry

The model geometry was analysed and cross checked with as built/design drawings, online GIS, site visits and CCTV. The following findings were made:

Picton Ave Box Culvert

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– The dimensions for Picton Ave Culvert were determined from both site visit investigations and as built/design drawing analysis. The culvert is a box culvert with curved roof thru to the manhole about ¾ of the way across Picton Ave where the newer circular pipe starts. The culvert has concrete footing (thickening) on the bottom inside of each wall. It was determined that the upstream and downstream height vary. The dimensions for the pipe are as summarised below.

Table 1 Dimensions of the Picton Ave Culvert

Parameter	Dimensions (mm)	Uncertainty (mm) (+/-)
Width of culvert, footing to footing	1750	50
Width of culvert, wall to wall	1975	50
Upstream Height at wall (open end) PipeID18061	1625	25
Downstream Height at wall (manhole end) PipeID18056	1800	100
Additional height at centre (both ends)	125	50
Height of footing	450	50

The culvert in the model was updated to match the following dimensions:

Table 2 Culvert Dimensions for the Model

Parameter	Dimensions (mm)
Width of Culvert	1850 (balance between the 1750 and 1975)
Height of Culvert (open end – PipeID18061)	1700 (balance between the 1625 and 1750)
Height of Culvert (manhole end – PipeID18056)	1875 (a balance between 1800 and 1925)

–

– The box culvert changes to a circular 1800 pipe at manhole ID3511, this is shown in both the As builts and CCTV footage.

Main Pipe

– After checking the as-built information for the Riccarton SW Main, the diameter of the main appears to stay at 1.8m diameter between Picton Ave and Deans Ave.

– Initially, 33 manholes were modelled in the pipe section between Picton Ave Culvert and Deans Ave Culvert. However, it was found that only 8 of these were true manholes, while the other remaining 25 were direct entry and junction points or inlets along the pipe. This discrepancy will be leading to inaccuracies of the model's calculations of head loss. This may be part of the reason there is evidence of head loss in the pipes as outlined in section 5.3. The extra head loss may be causing a decrease in the flow predicted within the model. The model has been updated to reflect the truth. While these manholes do provide extra capacity to the pipes, it's important to understand that the effect of head loss on the flow within the pipes is likely to outweigh the benefits of this additional capacity. The additional head loss could be reducing the flow modelled in the pipes. The model has been updated to incorporate site investigation results.

Deans Ave Box Culvert

– At Deans Ave, the Riccarton SW Main discharges into a box culvert that subsequently flows into a stream at Hagley. The current GIS data indicates the box culvert dimensions as 2.4 m by 2.4 m. The as-built records show the dimensions to be 2440 x 2540m (8ft x 8ft 4inch). However, during a site visit, the actual measurements were 2.4 m (±50mm) wide and 2.2 m (±50mm) high. Notably, there was a layer of gravel in the culvert's base. Our interpretation is that the gravel layer (approx. 200-300mm) was installed in more recent times for environmental purposes. For modelling purposes, the critical dimension is the 2.2m open flow path, the model was updated to reflect dimensions of 2.4 meters by 2.2 meters.

– There was a pipe seen to be crossing the inside of the Deans Ave culvert with an unknown diameter (approximated 300mm to 500mm), this is not included in the model. It is not expected to have a significant impact on the hydraulic performance.

– The GIS (and model) shows the pipe to change from a circular 2.4 pipe to a 2.4m x 2.4m box culvert at junction ID2359, however, the CCTV and as-builts shows the pipe to change to box culvert at manhole ID4000. The model should be updated to match the CCTV footage findings.

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4.10 Free draining infiltration rate

The existing model knowledge on free-draining infiltration rates is primarily informed by WWDG, the origins, applicability and confidence of which are uncertain. It is likely that free-draining rural/agricultural areas will have higher infiltration rates than urban greenspace areas for instance.

Investigations to understand what informed the WWDG and to synthesise this with new knowledge from more recent model calibration and related work is ongoing. To date we understand from the authors of WWDG that significant work did inform that document, but very little of that work has been rediscovered yet. Recent Heathcote calibration work resulted in low infiltration rates in the free draining areas, arguably below the credible range.

Next step: The existing data on free-draining infiltration rates raises questions. Specifically, the free draining infiltration rate in the Avon model still has high uncertainty, with plausible values ranging perhaps from 5-20 mm/hr. Investigations continue and will be reported later. A model sensitivity run to determine this importance of this uncertainty would be useful to understand the benefit of further work on this.

4.11 WSP report

GHD reviewed "Addington Brook & Riccarton Stream Stormwater Treatment - Gap Analysis Report, June 2020 by WSP". This was discussed in further detail with Christchurch City Council (Mark Groves and Peter Christensen) on 4th December 2023.

A summary of this review is outlined below:

- The time period studied had only small rainfall events and the catchment response to those is likely muted compared to flood response due to combinations of storage, soakage and evaporation.
- The Avon model is not expected to model small rainfall events well.
- The model version used by WSP (LDPR044 version with externally calculated Horton's infiltration) has only one function to represent these initial losses which is the meshes wetting depth.
- WSP conclude variously that the runoff observed from the flow monitoring is unreasonably low with respect to simple impervious area (ie: assuming full event infiltration on all pervious surfaces), albeit there being significantly questionable confidence in the flow monitoring.

4.12 Updates recommended for GIS data

- We identified six inlets in commercial area north of 10 Kyle St that are not in current GIS. These need to be added.
- Update Picton Ave Culvert to be a box culvert with the correct dimensions as outlined in section 4.9.
- Upstream parts of Deans Ave Culvert to be updated to be a box culvert (currently shown as circular).
- Open drains were identified to be missing at the 450mm diameter stub headwall behind NIWA (SwHeadWallID 9042) and at the 600mm diameter pipe end on KiwiRail land further south toward Placemakers (SWInletID 21374).

5. Model Upgrades Performed and Future Recommendations

Through fieldwork, as built and CCTV reviews we found errors in the CCC GIS data. The model has been modified to fix these errors. Details were noted in model log and deviations from the schema approach (which does not allow assumptions nor connect headwalls or pipe ends) were noted in a new internal draft version of the Model Status report (to be issued later as Rev8).

- Updates were made to the culverts under Picton Ave and part of Deans Ave where the reality is a box culvert and the GIS shows circular pipe.
- The dimensions for the box culverts at Picton Ave and Deans Ave were updated.

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- We identified six inlets in commercial area north of 10 Kyle St that were not in current GIS and added these to the model. We made connections for these and 15 sumps in GIS that do not have clear connectivity (and hence were not modelled before now). We used reasonable assumptions to define and model this connectivity.
- We found two new sumps in the current GIS from roading modernisation on Picton Ave and added these to the model.
- Full pipe capacity connections were also added at the 450 mm diameter stub headwall behind NIWA and at the 600 mm diameter pipe end on KiwiRail land further south toward Placemakers.
- Initially, 33 manholes were modelled in the pipe section between Picton Ave Culvert and Deans Ave Culvert. However, it was found that only eight of these were true manholes, while the other remaining 25 were junction points or inlets along the pipe. The model has been updated to reflect the truth.

In addition to the listed above model upgrades, the following items should be further investigated in future work:

- Main pipe invert levels review and corrections.
- Calculation of Infiltration Sensitivity to Runoff Calculation (eg: double and quadruple the class 5 infiltration rates).
- Follow the WSP progress on catchment calibration.
- Validation or calibration of the model with the use of the available rainfall and monitoring data.
- Following model fixes, recommendation of additional monitoring locations.

The following items are not proposed to be further investigated, as they are not high priority items, unless new findings trigger increased interest:

- Investigation of downpipes on industrial buildings and connecting them up to the model (which logically requires new methods to model buildings).
- The low infiltration rate of 0.6 m^3/s occurring near Avonhead Road/ Racecourse Road in the model.
- The unusual floodplain flow characterized by two distinct peak events in the upper reaches of Avon.

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Appendix R – Schema report recommendations

Boundary Weirs - attachment
The methodology for the boundary weir has been updated from what is in the schema report and is recommended to be added to the next version of the schema report. Previously boundary weirs were modelled with a constant QH at the full pipe flow. It had been changed to vary the Q from 0 at the pipe invert to full flow at the top of the pipe (full flow was not changed). Refer to the new methodology in email chain 1 attached.

Headwalls - attachment
The headwall methodology is recommended updated in the next version of the schema report. The methodology was updated from what is currently in the schema report when adding headwalls into the Riccarton area. The updated methodology is outlined in bullet point 7 on page 2 of email chain 2 attached.

Railway Crest Levels
An additional suggestion which has not yet been discussed or agreed but warrants future consideration is revision of Schema Rev 7 section 4.4.3 to remove the 150mm addition to LiDAR ground levels to account for railway crest levels. Avon Model v115 has already incorporated this change (see section 4.14 of this report). This is a subset of a wider discussion about railway embankments, and removal of the 150mm is a likely ‘step in the right direction’ but well short of a complete solution.

Appendix R - Email Chain 1 Boundary Weirs

Claudia Reid

From: Antoinette Tan <ant@dhigroup.com>
Sent: Monday, 31 July 2023 12:29 pm
To: Tim Preston
Cc: Elliot Tuck; peter.christensen@ccc.govt.nz; Sinni, Jarad; Groves, Mark
Subject: 1D boundary weirs

OperatingCentre: 51
JobNo: 12601181
CompleteRepository: 12601181
RepoEmail: 12601181@ghd.com
Description: SW hydraulic modelling working group
RepoType: Project

Hi Tim,

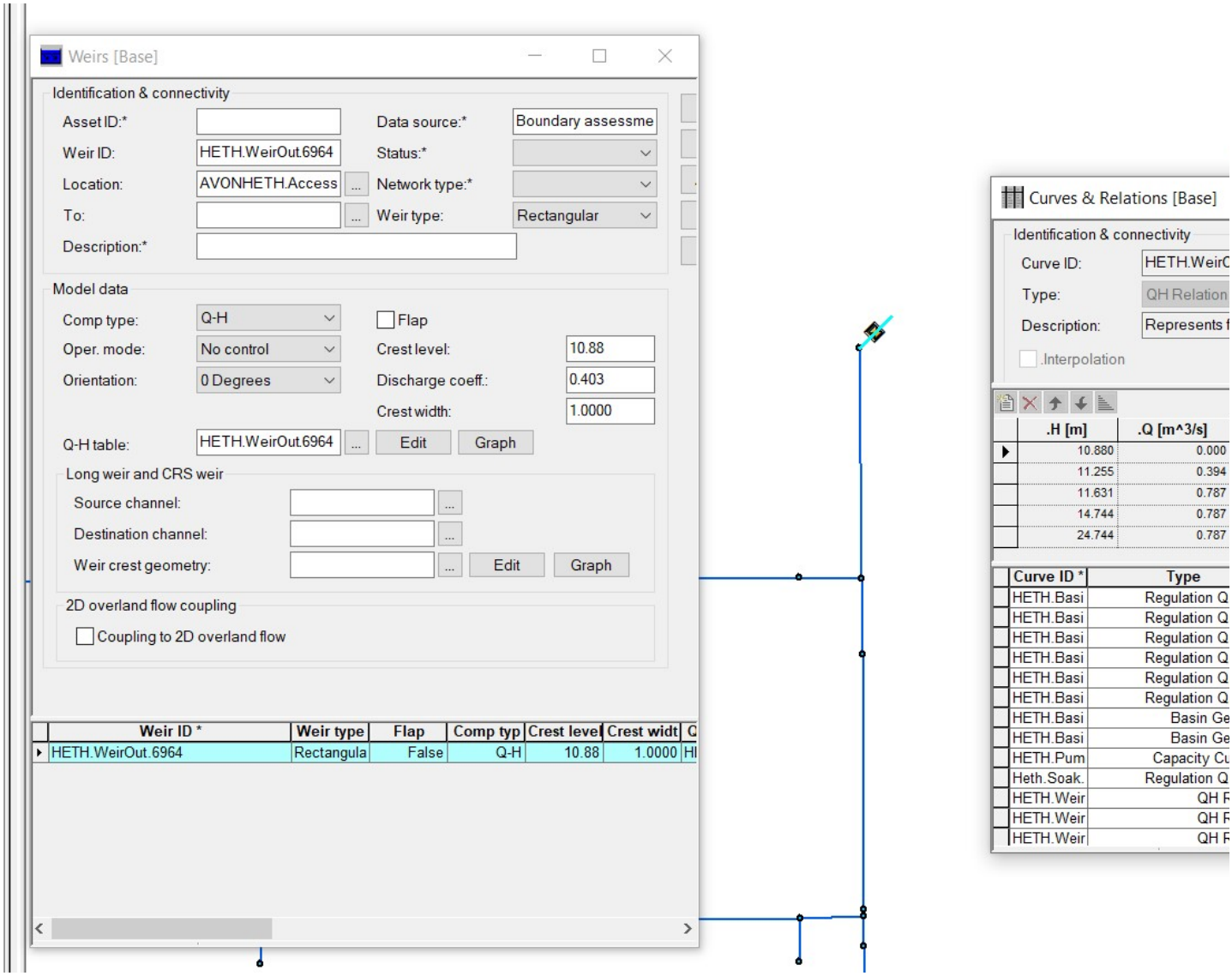
Just to wrap up our investigation into the 1D boundaries. I have tested the latest scenario you suggested, and I think this will do the trick. Please confirm that you think this is wrapped up now, and we can file for update to the Schema v2, and the upcoming model updates.

So where we have an MIKE urban pipe(s) exiting the model – i.e. flowing out:

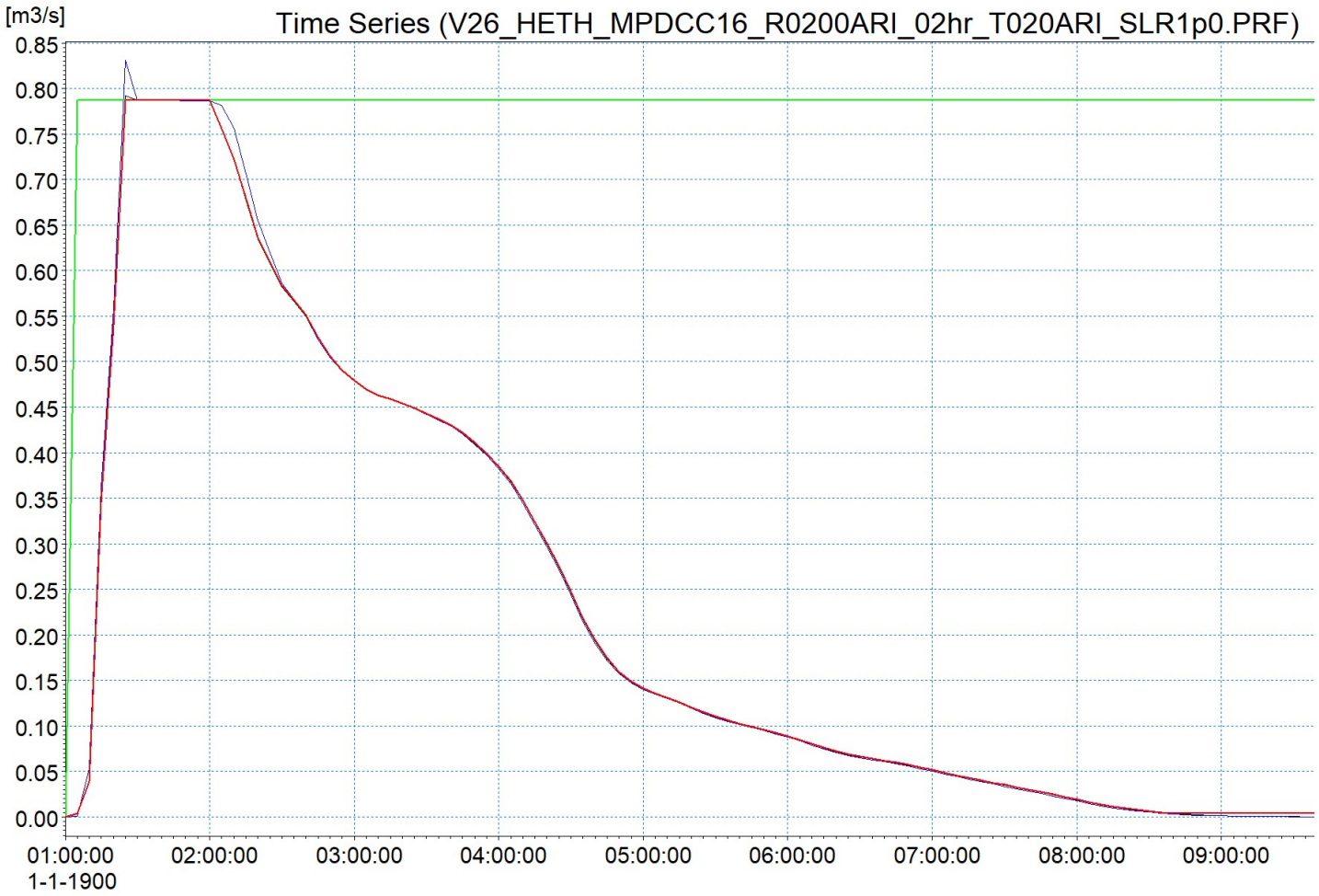
1. Add a weir structure at the outlet location
2. The weir should have a regulation setup so that he flow is governed by a QH relationship, the weir crest level should be set to the invert of the upstream pipe.
3. This relationship should start at 0 at the pipe invert, and linearly ramp up to the pipe full rate at the top of the pipe.
4. No water level boundary needs to be set at the outlet node. (where these have been setup in existing models these can be deleted)
5. The weir outflow will effectively act as a free outflow condition, where the weir discharge can be extracted directly for use in the adjacent model.

An example from the Heathcote model:

The weir is setup as shown in the screenshot.



The results show that using the new method the discharge matches almost exactly to the upstream pipe discharge, while using the old method the discharge was stuck at the pipe full flowrate.



Kind Regards

Antoinette Tan
Team Leader: Flooding
Urban Drainage - NZ

The expert in **WATER ENVIRONMENTS**

Work Days Mon-Wed, Fri

DHI Water & Environment Ltd.
1st Floor
Unit 9, Cowlishaw Mews
296 Montreal Street
Christchurch 8013

PO Box 26131
Christchurch 8148
New Zealand

Tel: +64 3 355 5403
Mobile: +64 21 035 1717

ant@dhigroup.com

Appendix R - Email Chain 2 Headwalls

Claudia Reid

From: Tim Preston
Sent: Tuesday, February 20, 2024 5:09 PM
To: Antoinette Taylor (InTouch)
Cc: Christensen, Peter; Jarad Sinni
Subject: RE: Various issues - record of GHD/DHI meeting today

CompleteRepository: 12601181
Description: SW hydraulic modelling working group
JobNo: 12601181
OperatingCentre: 51
RepoEmail: 12601181@ghd.com
RepoType: Project

Antoinette (cc: Peter, Jarad)

Notes below. Actions highlighted Tim Antoinette, Jarad/Peter old content that remains relevant / future is coloured grey.

1. Communications to others
    a. Peter delegated to Jarad, Tim to followup with Jarad on intentions and progress
2. Alternative mesh
    a. Antoinette expects that having a large quad adjacent to two half sized quads will not work as desired and violates DHI mesh expectations for simpler mesh forms (ie: triangles and quads only, all faces having a simple 1:1 connection between two adjacent cells)
    b. She has trialled this with the DHI mesh merger process - it did not work right but there are plenty of other mesh building processes that could make a difference
    c. Tim has made progress with Quadtree mesh built - detail is sitting with Peter expecting to be circulated more widely for comment and discussion tomorrow
    d. Antoinette yet to ask around for a technical specification of the DHI mesh expectations to confirm specifics, also to ask for evidence of testing sensitivity of model results to grid rotation (in the FM quads, high resolution context)
    e. Antoniette noted that large mesh sizes (in terms of cell counts, circa 20,000,000 may be approaching bounds of desktop GPU computation, limited by the available memory on the GPU cards) – this implies for example that a 1m grid on the Avon model would probably not be able to run on common desktop GPU, but 2m grid probably ok, 4m grid should be ok
    f. GHD still have the interest to try to test run the Avon model on grid, but yet to be defined, proposed or agreed
3. 2Jul2017 new calibration rainfall data (Motts)
    a. eta this week (with the programme still tbc - Motts)
    b. Immediate motivation to compare the 22July2017 event rainfall depths vs Metservice
    c. Then consider further motivation for Heathcote model revalidation and/or calibration – which would include some new work to generate Rorb outputs
4. Generation of CSV-DFS0 timeseries data, from PRF / RES11 / RES1D (for the side boundaries)
    a. Andres tool to extract results to CSV has been shared with T+T already (needs Iron Python and presumably Mike I-O with it)
    b. Extension of tool to save into DFS0 format is an obvious next step which Andre can do readily
    c. Andres enhanced tool was issued yesterday
5. Depth Tolerance
    a. Agreed first step is a wider trial with reduced tolerance (say 0.025m)
    b. Other possible subsequent steps in Antoinette memo are hopefully not required
    c. Compared Avon / Heath and it seems like Avon is the better ‘test bed’ for this trial – it might have (seems to have) more demanding circumstances, but this work could be led by DHI or GHD
    d. General intent would be to test run a single model scenario and then move onto wider scenarios, meanwhile monitoring stability results in the hope that precursor of problems if any might show up before we get run completion failures

e. Antoinette view that this would need some new funding for DHI to progress – seems like 50/50 call between DHI or GHD action, CCC preference for DHI to action this
f. Antoinette to put forward funding proposal to CCC for continuing this DT trial work

6. Stability - M11-MU stability issue - freq in Heathcote?
   a. Heathcote has 9 instances of M11 flow into MU (vs Avon has 16 instances)
   b. In both Avon and Heathcote there are probably 70% which could/should be eliminated by redefining MU/M11 extents
   c. Heathcote results have similar problems but considerably less so than Avon – this suggests probable room for optimism that Avon results can be practicably improved somewhat without radical change (ie: geometric improvements)
   d. GHD to check some of the Avon timeseries at a couple of key M11-MU locations and check the prevalence of stability over time and in particular around the peaks
   e. Further general stability summarisation ideas were discussed as follows
      i. Per batch (of durations) we can focus on the critical duration only (and the resulting max of batch is as good as that critical duration) – Antoinette to trial this
      ii. Per each timeseries we could check whether the 'max-oscillation' top level equals the max of timeseries – if so it confirms the relevance of the instability, but if not it doesn't mean much as there is likely to be secondary instabilities – hence we both thought this idea was a bit weak
      iii. Tim expressed ideal of a smoothed timeseries and then differencing from the actual timeseries, in which case the amplitude of this difference could be found and particularly the difference between the raw result max and the smoothed max would be a strong measure of significance, but there is a fair bit of development needed for that and neither had appetite to push for this
      iv. Another approach could limit the time window of interest to say after 50% of the rainfall duration, this would eliminate hotstart stability issues, but it would need to be paired with some checks to ensure the max of run did not occur within this first 50%, and/or a more general credibility check that the timing of max-of-run was within reasonable bounds (after the rainfall peak at least), and similar across a local neighbourhood – again too hard basket now but might be worth it

7. Headwalls feedback
   a. Antoinette has found circa 40 issues in Heathcote, qualitatively similar to the Avon issues
   b. General agreement on need to add MU-M21 couples to the model where appropriate
      i. At present 'where appropriate relies on manual modeller review of all the GIS headwalls, some of which can be ignored as already modelled or not required through spatial filters
   c. Agreed on couple type = inlet / weir
      i. Change all the coupled mesh levels to 10mm above pipe invert (for the right weir formulation, to help reasonably ensure local mesh flows into the coupling, and to comply with requirements on inlet couples)
      ii. Weir width to equal pipe diameter (or rect width)
      iii. Qmax = 2m/s times pipe cross sectional area
      iv. DischCoeff = 0.98
      v. This should be an addenda to the Schema report – Jarad to note for that SOW scope

      Also: GHD added since
      QdH = 0.1
      Inlet area = 1m2
   d. Atnoinette also to note scoping needed to address pipe ends – another motivation for Tim to ask CCC GIS about Data dictionary, data model etc

8. Lynker Imperv, - what 'model ready' format do we prefer?
   a. Agreed on 1m raster input as the 'model ready' form of this data – Tim to advise Lynker (done)

9. Modelling roofs - esp large commercials
   a. Various methods, some software version dependant, more options in Mike Plus than v2020
   b. A main challenge is routing roof runoff from 2D to 1D
   c. Current approaches are somewhat experimental / pilot / don't have public tools for model build at large data scales
   d. In Mike Plus there are a M21 infrastructure functions (Antoinette to fwd link to the manual)
   e. In v2020 routing options include forming the roof space with a central depression, or using dikes for containment and then using an inlet coupling to the pipe network (or similar to kerb)
   f. Tauranga experimenting with this
   g. Agreed something here is a logical part of Schema 2.0 and all pipes
   h. Significant related implications on needs for mesh resolution
   i. Antoinette raised the related concern with fences, but so far no-one has practicable modelling methods and real world data is mainly absent
   j. General agreement that this is inevitable in the next year or three, but that the genre remains perhaps a bit immature today, so maybe not ready yet
   k. Auckland (Hansol) has no current or intended modelling of buildings, so it seems too early for Chch now and perhaps Schema 2.0, but still feels like inevitable progress in years ahead

Appendix S – Manhole headloss memo + addenda

Report

18 July 2024

To	Peter Christensen	**Contact No.**	+64 3 378 0973
Copy to	Jarad Sinni, Tim Preston, Russell Knipe, Claudia Reid,	**Email**	Russell.knipe@ghd.com
From	Russell Knipe	**Project No.**	12601181
Project Name	SW hydraulic modelling SoW081		
Subject	Investigation of Manhole Minor Loss K Factors on Riccarton Main Drain		

1. Introduction

Council's flood models evaluate headloss in manholes using regimes in the hydraulic software. In the DHI models to date, methods have been determined by DHI recommendation, with an initially recommended method being 'MouseClassic k=0.5' and a more recently recommended method being 'WeightedInletEnergy k=0.25'. Until recently the Avon model retained the MouseClassic headloss while the Heathcote model had moved to the WeightedInletEnergy. In both models these headlosses are calculated at every node without discrimination between manhole nodes and junctions or other types of node that should have materially nil headloss. The Heathcote model of node headloss is supported somewhat by success in model calibration with particular attention to getting the correct flow rate along Tennyson St and calibration flood extents upstream of there (near Barrington Mall).

1.1 Purpose of this Report

In order to get more realism (and or confidence) in modelled manhole headlosses, Council would like to have headloss at manholes and other similar structural features but not at junctions where there is negligible interruption to continuous flow with constant velocity. Aligning the models with supporting Engineering evaluation of realistic manhole headlosses (independent of model calculations or calibration) will also help either improving (or validating) the model realism. This report presents that independent engineering evaluation of manhole headloss for a specific set of locations and hydraulic conditions and compares these with the model results from both MouseClassic and WeightedInletEnergy. The report does not provide modelling recommendations but rather is proposed as a basis for further testing, evaluation, recommendations and decisions for future modelling approaches by others.

The Engineering evaluation is focused on the 1.8m dia pipe system on Riccarton Main Drain, between Picton Ave and Deans Ave. The manholes in this system are well described in As Built records, showing benching to mid-pipe and relatively small manholes with limited flow expansion and contraction through the manhole. These manholes are mid-range in terms of quality and headloss performance. The general 'norms' of other manholes in Christchurch are unknown, but it is likely to include both higher quality manholes with better benching and/or flow transition details and lesser quality manholes with no benching and more abrupt flow transitions. This evaluation then has partial relevance to a 'whole of Christchurch' perspective, but only to the extent (ie: assumption) that the manholes in the study might be representative of typical Christchurch manholes.

1.2 Scope and limitations

This evaluation considers only hydraulic conditions when the pipes are running full. This is both to reduce complexity and variables in the study, but also noting that this is the predominant and important hydraulic condition that is widespread during flood events. The evaluation only considers manholes with two pipes (Qin = Qout simplicity). It considers a variety of bend angles in the manholes but does not consider more complex 3-way junction manholes.

The Engineering evaluation relies on published Engineering guidance. The guidance used does not match the geometry of the manholes studied very well and significant Engineering inference is required to reach a conclusion accordingly. This inference is subjective and has been determined through the collaborative opinions of Tim Preston, John McArthur and Russell Knipe. It is expected that other Engineers may have different interpretations of the same data, and noted that our estimation of confidence (range) in our conclusions is qualitative not quantitative.

This report: has been prepared by GHD for Christchurch City Council (CCC) and may only be used and relied on by CCC for the purpose agreed between GHD and CCC of better understanding the merits of various methods of modelling headloss in stormwater manholes (described in section 1.1).

GHD otherwise disclaims responsibility to any person other than CCC arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible. The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report, especially in section 1.1. GHD disclaims liability arising from any of the assumptions being incorrect.

2. Geometry of Manholes

The as built drawings show that the width of the manholes is slightly larger than that of the pipe and the average length of the manhole approximately 80% of the diameter of the pipe. The as built drawings also show the manholes have benching halfway up the pipe. Refer to Figure 1 for a typical manhole example used in this study.

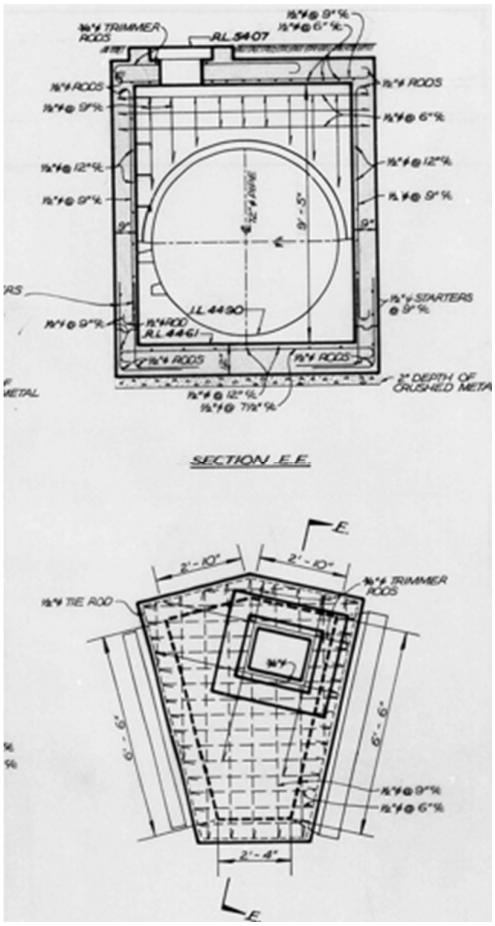


Figure 1 *As built drawing for manhole AVON.Access.CCCGIS.3964. This is typical of the manholes on this section of pipe.*

3. Method

This study compares modelled headloss with other industry methods of calculating K factors and headloss through manholes. The Queensland Urban Drainage Manual (QUDM) was chosen to calculate the minor loss K for specific manhole arrangements, i.e. 10 degree bends, 45 degree bends etc. QUDM utilises Hare charts and Cade & Thompson charts for calculating minor losses in both square and circular manholes. In order to estimate a “lower bound” for minor loss in a pressurised (pipe) system, AS2200 Chart 14 was also used for K values for bends.

Headloss through the manholes were extracted from two model versions. An older model V115b which has the older ‘MouseClassic k=0.5’ manhole headloss method and a more recent model which V115c uses ‘WeightedInletEnergy k=0.25’.

An Engineering evaluation was then performed comparing the K values from the QUDM and AS2200 standard and the K values from the model (model K factors being back calculated from the model headloss result). As part of this evaluation the accuracy of the QUDM and AS2200 was assessed by comparing the geometry of the typical manholes used in the QUDM and AS2200 standards and the As-built drawing showing the manholes in reality.

3.1 Calculating K from Model Headloss

Data was extracted from models V115b and V115c in three[1] locations for manholes located from Picton Ave to Deans Ave. All manholes had simple singular inflows, and 1.8m dia pipe incoming and outgoing, but with varying angular direction changes, nominally 0°, 10° and 30° as highlighted below. The manhole IDs and both nominal and actual bend angles are shown in Appendix A.



Figure 2 Three manholes selected for model headloss analyses

To calculate the head loss through the manholes data was extracted for the water level in the manhole and the water level in the pipe immediately downstream of the manhole for each time step[2]. The flow through the manhole for each time step was also extracted from the model. The various Q/deltaH points were inspected on scatter plots and a high flow value (above pipe full) was chosen to back-calculate the K value for each manhole. K values were not calculated where the pipe was not flowing full. Along the length of the drain, manholes further downstream toward Deans Ave and Hagley Park were noted as flowing full less often than the upstream manholes.

The minor loss equation was rearranged and used for the back-calculation of K[3]:

$$K = \Delta H \frac{2g}{V^2}$$

Velocity (V) in that equation is calculated from flow/area in the outgoing pipe (while noting in our cases incoming velocity is the same as outgoing velocity). The resulting model K-factors for each of the two model setups are shown in Appendix A.

3.2 Estimation of K from QUDM

QUDM provides charts for calculating minor loss K values[4] for various manhole configurations based on multiple sets of experiments with different physical characteristics. The experiments that informed the Hare Charts were conducted using square manholes while the Cade & Thompson charts used round manholes. These experiments were conducted with a manhole (pit) that was approximately twice the width of the downstream pipe. These charts are considered primarily applicable to road longitudinal drainage, where

[1] Originally data from eight locations was evaluated. MH3991 was too far downstream and didn't reach pipe full condition. Tee MH's 3511 4000 3985 915 were also briefly looked at but these were set aside due to the extra degree of difficulty in evaluating losses in manholes with two incoming pipes

[2] The HGL in the pipe immediately upstream of the manhole is identical to the water surface elevation in the manhole

[3] Note that this calculated K-value is quite different from the model setup parameters (K = 0.25, 0.50)

[4] There are two different types of K values that the QUDM charts calculate. These are the Kw value which is used to determine water level inside the manhole and Ku which represents the change in the hydraulic grade line (HGL). This evaluation used the Ku value as this is what can be compared to the headloss found in the model. In our cases upstream and downstream velocities are equal and therefore change in HGL equals change in water level (change in water level is what the model results give)

pipes of typical sizes DN375 to DN600 are installed in standard pits of sizes 1050-1200. Both charts assume no benching. The Hare charts can allow for grate inflow but all our analyses used zero grate inflow.

Accordingly, the types of manholes encountered in the Riccarton Main are expected to have lower losses than the QUDM manholes. For the purposes of comparison, both styles are charts were used.

QUDM Table 7.16.4 (pdf pg 172/391) describes the expected reduction in K value for benching within a manhole. For the half height benching expected in the Riccarton manholes, a reduction of 20-30% could be expected for a manhole with a single pipe in and single pipe out.

Quoting QUDM Appendices section A2.2.1 (pdf pg 320/391) "*In the usual range (0.8 < Du/Do < 1.0) Ku and Kw will decrease by up to 0.4 as B/Do is reduced from 2 to 1 (except as noted in A2.2.2 below for intersection on the downstream face)*". We find the meaning from this somewhat debatable with two likely interpretations being;

1. If B/Do is close to 1, then multiply the Ku and Kw values by (1 – 0.4) ie: a 0.4 proportional reduction.
2. If B/Do is close to 1, then subtract an adjustment from the Ku and Kw values ie: a 0.4 subtraction reduction, but some initial k values are less than 0.4 so then some smaller subtraction would needed.

The percentage reductions in Table 7.16.4 would be analogous to interpretation 1 above. However, the presentation and terminology in Table 7.16.4 is completely different to A.2.2.1, and this difference would suggest that it does not mean analogous to interpretation 1 but more likely interpretation 2.

We do not expect to resolve this doubt, but the net effect of either interpretation for the manholes in our case with B/Do approx. 1.1, in either case will be a further significant reduction in Ku from the starting values.

The combination of reduction for benching and reduction for B/Do motivate reductions to the Cade & Thompson values as shown in Appendix A, columns "2/3*Cade and Thompson" and "1/2*Cade and Thompson".

The Hare Charts produced a wide range of k-values for each angle assessed, as there may be as many as four relevant charts for a given angle through a manhole. When the pipes are flowing full and a pit has a water surface level above the top of the pipes, the flow of water out of the pipe acts as a "buoyant jet". How this jet interacts with the rest of the pit significantly impacts the expected headloss. If the jet discharges directly into the downstream pipe the losses will be low, however the losses will be magnified greatly if the jet impinges on one of the walls and needs to reflect into the downstream pipe. Because of this wide range in results from Hare and because our manholes didn't match any of the Hare geometries, these results were not considered very useful in guidance for this study.

3.3 Estimation of K from AS2200

AS2200 Chart 14 provides minor losses for various valves and fittings used in the water industry. Included in this chart are minor losses for mitred bends and swept bends at various angles. No manholes are expected to have losses this low, however they form a useful lower bound comparison to the QUDM charts when considering K values. These K values are tabulated in Appendix A.

A subjective factor of x2 was applied to these K factors to derive guidance for lower bound values that might be more reasonable to expect in the open manhole situations being assessed here.

4. Results

A reasonable range (upper, median, lower) for the manhole k-values was then produced based on the GHD derived guidance values and GHD subjective inference as to which values were more credible in relation to the real manhole geometry. The reasonable k-values from the Engineering evaluation and from the model results are presented in Figure 3 and tabulated in Appendix A.

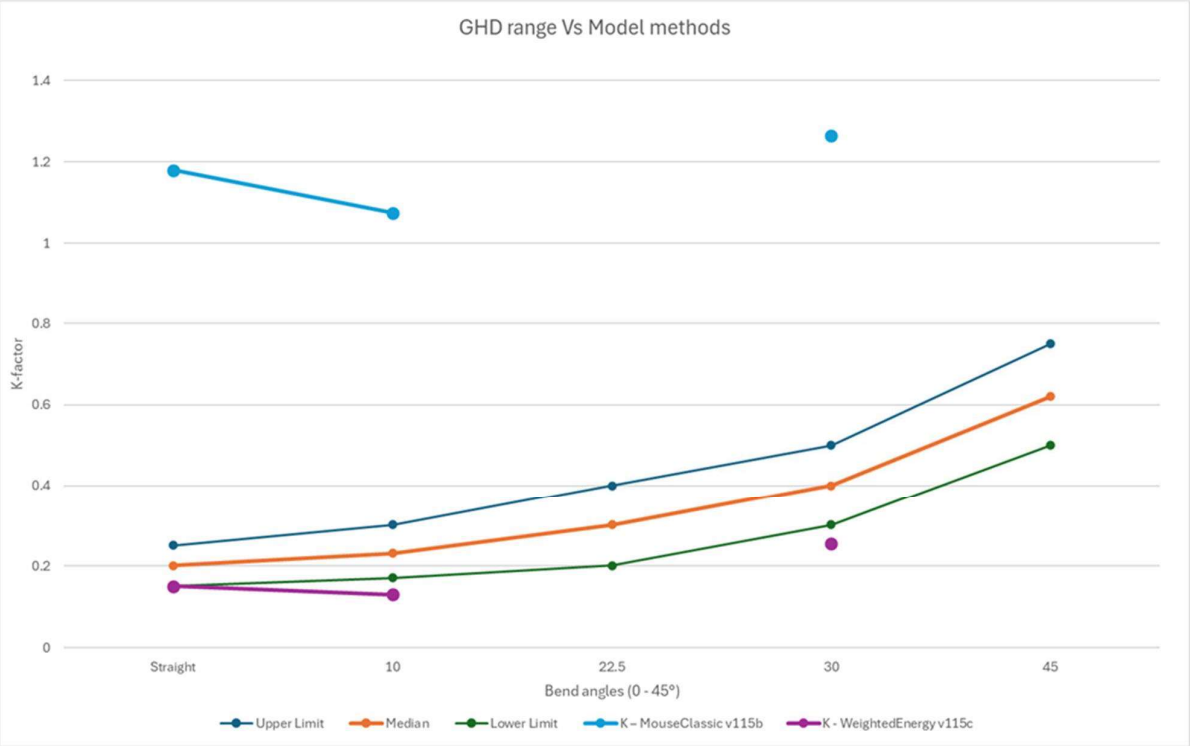


Figure 3 ***GHD reasonable value range k-factors compared to models***

The reasonable range range presented here is not absolute max/min but rather means that we would consider it unlikely that the truth is really outside this range. The k-values calculated from the Mouse classic model were well outside the range that GHD has determined to be reasonable. The WeightInletEnergy model manholes produced k-values below the reasonable range, but close to it. The tendency to the modelled headlosses to drop with minor bend angles, is noted as anomalous.

5. Conclusions

Neither of the two current DHI model methods provides a good calculation of manhole headloss for the manholes studied here. The WeightedEnergy method understates the likely real headloss but at or slightly below the low margin of the reasonable range. The MouseClassic method estimates headloss which is well in excess of reasonable range estimates for these manholes.

Further work is recommended to;

1. assess better headloss modelling options for the manholes in this study area
    a. part of this assessment should consider why model headloss for straight manholes are higher than for manholes with and slight angular deviations
2. understand typical manhole geometries and estimated k-factors across the range of situations common in Christchurch. Consideration of results could be stratified by age of construction, pipe diameter and perhaps geographic clusters such as by historic TLA boundaries
3. extend this study to include 3-way “T”-manholes with two inlets

Project name		SW hydraulic modelling SoW081					
Document title		Investigation of Manhole Minor Loss K Factor on Riccarton Pipework					
Project number		12601181					
File name		12601181 REP Manhole Headlosses.docx					
Status Code	Revision	Author	Reviewer		Approved for issue		
			Name	Signature	Name	Signature	Date
			Name	Signature	Name	Signature	Date
[Status code]	1	Russell Knipe, Tim Preston	John McArthur		Tim Preston		19/7/24
[Status code]							
[Status code]							
[Status code]							
[Status code]							

GHD Limited
Contact: Russell Knipe
Water Engineer | GHD
138 Victoria Street, Level 3
Christchurch Central, Canterbury 8013, New Zealand
T +64 3 378 0900 | **F** +64 3 377 8575 | **E** chcmail@ghd.com | **ghd.com**

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Appendix A

Table of K Value Results

Investigation of Manhole Minor Loss K Factors on Riccarton Pipework Main Drain
Appendix A

Type	QUDM			AS2200	GHD Derived Guide Values			Reasonable Range for Manhole K-values			Model Results	
	K – Hare for the Worst Flow Path	K – round manhole (Cade and Thompson Chart)	K - Hare for the Optimal Flow Path	K – Mitred Bend	2/3*Cade and Thompson	1/2*Cade and Thompson	2x mitred bend	Upper Limit	Median	Lower Limit	K – MouseClassic v115b	K - WeightedEnergy v115c
Straight	N/A	0.3	0.2	N/A	0.2	0.15	N/A	0.25	0.2	0.15	1.18	0.15
10	N/A	0.40	N/A	0.075	0.27	0.20	0.15	0.30	0.23	0.17	1.07	0.13
22.5	1.35	0.60	0.30	0.1	0.40	0.30	0.2	0.40	0.30	0.2	N/A	N/A
30	N/A	0.70	N/A	N/A	0.47	0.35	N/A	0.50	0.40	0.3	1.26	0.25
45	1.70	1.10	0.65	0.25	0.73	0.55	0.5	0.75	0.62	0.5	N/A	N/A
90[4]	1.8	1.8	1.8	1.2[5]	na	na	na	1.8	1.6	1.4	N/A	N/A

Notes:
The Hare optimal flow path is when the jet discharges directly into the downstream pipe.
The Hare worst flow path is when the jet impinges on one of the walls and need to reflect into the downstream pipe.
There is no worst case for straight, nor a best case for 90deg
No manhole that could be approximated as 45° or 90° change was found in the study area (manhole 3991 was the closest but only bent 34.6°)
AS2200 only offers a 90-degree short radius elbow
The 90° bend condition was an exception and treated differently in terms of derived guide values and reasonable range

Manhole Type	QUDM Cade and Thompson Chart	QUDM Hare Chart for the Best Flow Path	QUDM Hare Chart for the Worst Flow Path	GIS / Model node MUID	Actual model bend angle (deg)	GIS / Model d/s pipe MUID
Straight	A2-35	A2-4	NA	3910	0.5°	18022
10	A2-35	NA	NA	3859	5.9°	18029
22.5	A2-35	A2-5	A2-8	NA		
30	A2-35	NA	NA	3964	28.4°	18016
45	A2-35	A2-6	A2-12	NA		
90	A2-35	NA	A2-28	NA		

Claudia Reid

From: Claudia Reid
Sent: Wednesday, 24 July 2024 8:48 am
To: Tim Preston; Jarad Sinni; Christensen, Peter
Cc: Antoinette Taylor (InTouch); John McArthur; Russell Knipe; Madeline Howell; Susan Willis; Elliot Tuck; Avinash Chakravarthy; Chusit Apirumanekul; Tony Trueman
Subject: Manhole headloss memo addenda

CompleteRepository: 12601181
Description: SW hydraulic modelling working group
JobNo: 12601181
OperatingCentre: 51
RepoEmail: 12601181@ghd.com
RepoType: Project

Hi all,

The manhole K-factors from the model showed a lower than expected value for the 10-degree bend. After comparing diameters of the three manholes in the model it was found that the manhole with a 10-degree bend has a smaller diameter than the other two manholes (2.3 m compared with 3 m). This explains reduced K-factor for this manhole.

This message has been approved by Tim.

Regards,

Claudia Reid

BE(Hons) (Civil)
Graduate Water Engineer

GHD
Proudly employee-owned | ghd.com
138 Victoria Street, PO Box 13468, Christchurch 8141, New Zealand
D +64 3 363 0819 **E** claudia.reid@ghd.com

From: Tim Preston <Tim.Preston@ghd.com>
Sent: Friday, July 19, 2024 2:06 PM
To: Jarad Sinni <Jarad.Sinni@ccc.govt.nz>; Christensen, Peter <Peter.Christensen@ccc.govt.nz>
Cc: Antoinette Taylor (InTouch) <ant@dhigroup.com>; John McArthur <John.McArthur@ghd.com>; Russell Knipe <Russell.Knipe@ghd.com>; Madeline Howell <Madeline.Howell@ghd.com>; Susan Willis <Susan.Willis@ghd.com>; Elliot Tuck <Elliot.Tuck@beca.com>; Avinash Chakravarthy <avinash.chakravarthy@awa.kiwi>; Chusit Apirumanekul <chusit.apirumanekul@awa.kiwi>; Tony Trueman <tony.trueman@awa.kiwi>
Subject: Manhole headloss memo

Hi Jarad / Peter (cc DHI, BECA, AWA)

Please find attached our report assessing some realistic manhole headloss from a pure Engineering perspective and then comparison of that with two versions of the DHI model headloss methods.

There is no direct implication of further actions required by anyone, but I would recommend at least 15 minutes review to familiarise with the contents. There is perhaps a soft implication toward DHI in terms of 'potential to reconsider the recommend method of manhole headloss'.

I have recommended Jarad put on the agenda for our next coordination meeting. There is plenty of potential for further work in this area, but whether and work would meet Council criteria and priorities for funding and if so, who should do what, is unclear. I expect Council will benefit accordingly from some collective discussion and your opinions on 'what next, if anything'.

To date this has a DHI software focus, but there is clear an analogous Tuflow question that could be explored in parallel.

Regards

Tim Preston
BE Civil, MComm
Senior Water Engineer

My typical working hours are
Mon-Fri 9:30am-6:00 pm
I am typically on secondment in CCC offices Tuesdays and Fridays
For communication about CCC secondment work and calendar appointments @CCC during these hours, please use tim.preston@ccc.govt.nz
I generally monitor both GHD and CCC email addresses and Teams communications throughout the day
---

GHD
Proudly employee owned | ghd.com
Level 3, 138 Victoria St, Christchurch 8013, New Zealand
D +64 3 3780913 **M** +64 27 6414301 **E** tim.preston@ghd.com

The Power of Commitment

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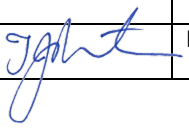
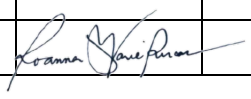
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GHD
Level 3/138 Victoria Street, Christchurch 8013
T: 64 3 378 0900 F: 64 9 370 8001 E: chcmail@ghd.com

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https://projectsportal.ghd.com/sites/pp02_10/ccccranfordswtreatme/ProjectDocs/Stage 1 - Design philosophy and Gap analysis/Avon Status Report Rev 9/CFM Avon Model Status Report_20250331_Rev9.docx

Document Status

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		Name	Signature	Name	Signature	Date
5	T. Preston	J. McArthur		T. Preston		4/12/2018
6	R. De Costa	T. Preston		T. Preston		16/11/2020
7	Y.Hooi T.Preston R. De Costa	T. Preston		T. Preston		20/7/2022
8	C. Reid M. Howell	S. Willis		T. Preston		6/9/2024
9	M. Howell	T. Preston		R. Purcaru		4/4/2025

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