

Belfast to Pegasus Motorway & Woodend Bypass Pre-implementation and MSQA Professional Services

Stormwater Design Report 100%

NZ Transport Agency (NZTA)

Document code: 11320-AUR-0350-PWI-SW-RPT-0002

Revision: B

Date prepared: **19 December 2025**



Document control record

Document prepared by:

Aurecon New Zealand Limited

Level 2, Iwikau Building
93 Cambridge Terrace
Christchurch 8013
New Zealand

T +64 3 366 0821

F +64 3 379 6955

E christchurch@aurecongroup.com

W aurecongroup.com

Document control						aurecon
Report title		Stormwater Design Report 100%				
Document code		11320-AUR-0350-PWI-SW-RPT-0002	Project number	11320		
Document state		PUB Published	Status code	P3 Detailed Design		
File path		https://aurecongroup.sharepoint.com/sites/528638/3_Develop/For Delivery/11320-AUR-0350-PWI-SW-RPT-0002.docx				
Rev	Date	Revision details	Author	Verifier (if required)	Approver	
A	2025-09-26	50% Design Issue – For Client Review	Y. Hooi	R. Smith	M. Flannery	
B	2025-12-19	100% Design Issue – For Client Review	Y. Hooi, J. Fleming	W. Herbert	H. Walters	
Current revision		B				

Client			
Client name	NZ Transport Agency (NZTA)	Client contact	Brendon French
Client document code	N/A	Client document revision	N/A

Approval			
Author signature		Approver signature	
Name	Yanni Hooi	Name	Haden Walters
Title	Senior Civil Engineer	Title	Design Manager

Contents

1	Introduction.....	1
1.1	Belfast to Pegasus context.....	1
1.2	Purpose of the report.....	2
1.3	Definitions and abbreviations	2
1.4	Project Objectives.....	3
2	Design process.....	4
2.1	Design development since previous stage.....	4
2.1.1	Design development to 50%	4
2.1.2	Design development to 100%	5
2.2	Requirements and Standards Compliance	5
2.2.1	Design criteria.....	5
2.2.2	Departures from Standards	6
2.2.3	Design Interfaces.....	7
2.2.4	Issues requiring resolution in IFC design stage	7
2.2.5	Design reviews	8
3	Analysis and design.....	9
3.1	Existing Drainage	9
3.2	External Stormwater Management.....	9
3.2.1	Cut-off Drains	9
3.2.2	Channel Realignment.....	10
3.2.3	Cross Culverts	11
3.3	Proposed Stormwater Quality and Quantity Management.....	15
3.3.1	Grass Lined Swale	16
3.3.2	Bioinfiltration Swale	17
3.3.3	Bioretention Swale.....	19
3.3.4	Bioinfiltration Basin	20
3.3.5	Bioretention Basin	21
3.4	Proposed Highway Stormwater Drainage Overview	22
3.4.1	Overview of External Catchments	22
3.4.2	Kaiapoi River Catchment.....	23
3.4.3	Cam River Catchment	23
3.4.4	Quarry Lakes Catchment	24
3.4.5	Mcintosh Drain Catchment	24
3.4.6	Waihora Stream Catchment	24
3.4.7	Taranaki Stream Catchment	26
3.4.8	Pipe Reticulation.....	27
3.4.9	Surface Drainage.....	27
3.5	Flood Assessment Modelling	28
3.5.1	Catchment Area Between Ohoka Road to Lineside Road	28
3.5.2	Catchment Area Between Lineside Road and Cam River	28
3.5.3	Catchment Area North of Cam River.....	29
3.6	Proposed Bridge Improvements.....	36
3.6.1	Kaiapoi River Bridge.....	36
3.6.2	Proposed Cam River Bridge.....	36
4	Next Steps.....	38

Appendices

Appendix A

Definitions and abbreviations

Appendix B

Departures

Appendix C

Stormwater and Flooding Assessment Report

Appendix D

B2P Major and Significant Culverts Memo

Appendix E

Kaiapoi River Scour Assessment Memo

Appendix F

Cam River Scour Assessment Memo

Appendix G

Flood Impact Assessment South of Cam River

Appendix H

Flood Assessment Report

Figures

Figure 1	B2P location map
Figure 2	Design documentation structure
Figure 3	Belfast to Pegasus Line Diagram
Figure 4	Typical cutoff drain profile
Figure 5	Proposed culvert locations
Figure 6	Typical detail of grass lined swale
Figure 7	Typical detail of subsoil drain
Figure 8	Typical detail of bioinfiltration swale
Figure 9	Typical detail of bioretention swale
Figure 10	Typical detail of Bioinfiltration Basin
Figure 11	Typical detail of bioretention basin and forebay
Figure 12	Overview of the external catchment
Figure 13	TUFLOW model extent boundary
Figure 14	Private buildings affected under 200 year ARI (mitigated scenario)
Figure 15	Residual impacts outside the designation but does not impact any property buildings (200 year ARI afflux level)

Tables

Table 1	Key changes to the stormwater design between 30% and 50% detailed design
Table 2	Requested Departures
Table 3	Design Interfaces
Table 4	Proposed Channel Realignment
Table 5	List of culverts to be decommissioned
Table 6	Proposed Cross Culverts
Table 7	Indicative apron dimensions for major and significant culverts (including both inlet and outlet)
Table 8	Indicative apron dimensions for minor culverts (including both inlet and outlet)
Table 9	Treatment Methodology Overview
Table 10	Stormwater treatment design parameters
Table 11	Grass Lined Swale design summary
Table 12	Bioinfiltration Swale summary for Water Quality Event (25mm rainfall)
Table 13	Bioretention Swale design summary
Table 14	Bioinfiltration basin summary
Table 15	Bioretention basin summary
Table 16	Water flow depth calculation for surface runoff at where crossfall is superelevated
Table 17	100-year and 200-year ARI Flood Levels and Differences (Post-Development vs. Pre-Development Scenario)
Table 18	100-year and 200-year ARI flood level and differences (Mitigation vs Pre-development scenario)
Table 19	Locations that observed a water level increase outside of designation boundaries
Table 20	Scour depth (m) by bridge component in the worst case SLS and ULS event scenarios
Table 21	Scour depth (m) by bridge component in the worst case SLS and ULS event scenarios
Table 22	Scour Protection Summary
Table 23	Definitions and abbreviations

Copyright © 2025 Aurecon Australasia Pty Ltd and its affiliates. "AURECON", "BRINGING IDEAS TO LIFE" and the Aurecon logos and devices are trade marks and/or registered trade marks of Aurecon Group Brand Pty Ltd and its affiliates. All rights reserved.

Disclaimer

This report (including any enclosures, data and attachments) has been prepared for the exclusive use and benefit of our client, solely for the purpose for which it is provided, and on the terms and conditions agreed with our client. Unless we provide express prior written consent, no part of this report should be reproduced, distributed or communicated to any third party. To the extent permissible by law, we do not accept any liability if this report is used or relied on by any unauthorised third party, or by the client for any unauthorised purpose, in any other contexts or without checking it is the latest revision.

1 Introduction

1.1 Belfast to Pegasus context

The State Highway 1 (SH1) Belfast to Pegasus (B2P) project is a vital initiative led by Waka Kotahi New Zealand Transport Agency (NZTA), aimed at enhancing transport capacity and connectivity in the rapidly growing northern region of Christchurch.

Recognized as a Road of National Significance (RoNS), this project is critical for supporting both local community needs and the national economy by facilitating the movement of goods and people along this essential freight route.

Overall, B2P is a 11km four-lane highway extension of SH1 north of Christchurch, running south of SH1/SH71 Lineside Road interchange and finishing just north of Pegasus illustrated in Figure 1 .



Figure 1 B2P location map

1.2 Purpose of the report

This is the Design Report for the B2P - Design Package Stormwater. This report is to be read in conjunction with the B2P Project Wide Design Report (11320-AUR-0350-PWI-GE-RPT-0005), which documents project wide issues including Safety in Design, Maintenance in Design and Sustainability and Climate Resilience.

The purpose of this Design Report is to:

- To inform NZTA of the asset they will eventually own,
- To document the design progression and analysis undertaken in detailed design; and
- To inform the independent reviewer of the analysis performed in detailed design.

This report is one of several documents forming the design reporting for B2P as outlined in following Figure 2

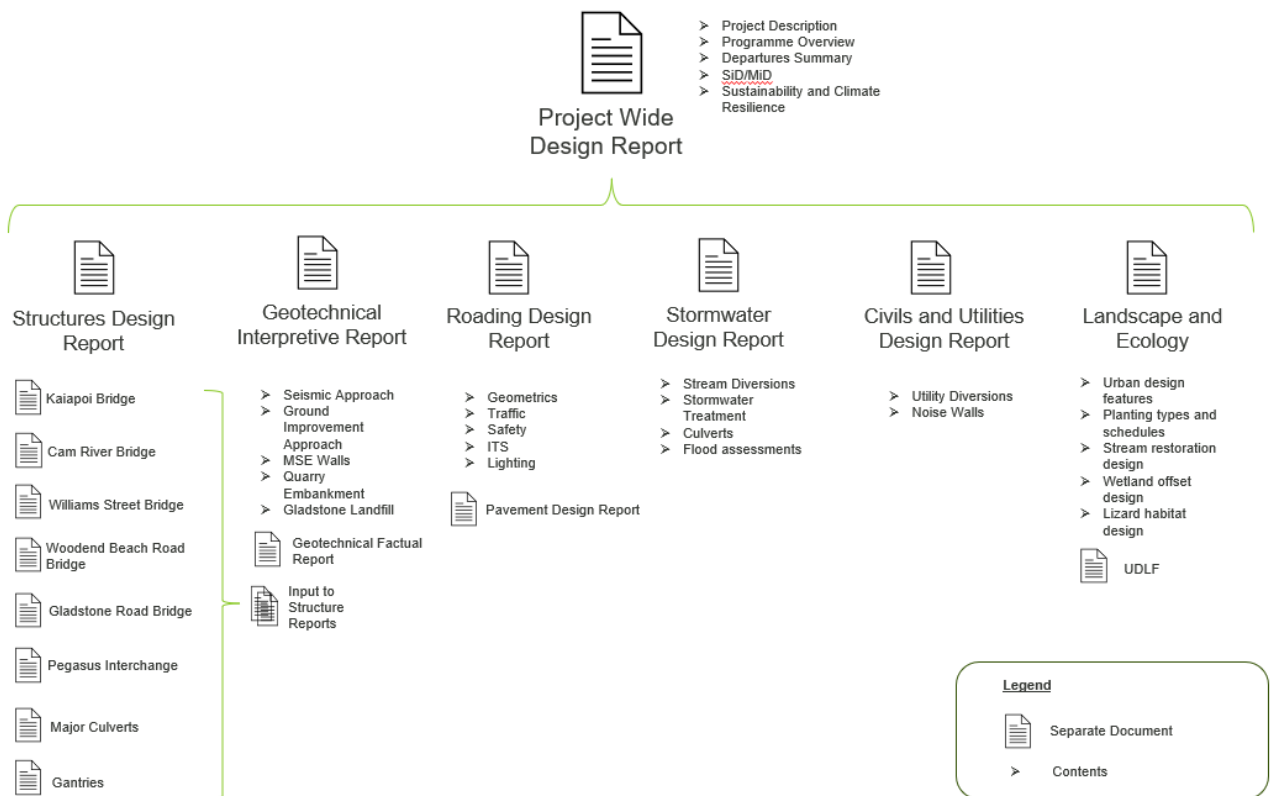


Figure 2 Design documentation structure

1.3 Definitions and abbreviations

Refer to Appendix A.

1.4 Project Objectives

The key objectives of the SH1 Belfast to Pegasus project are:

- **Improve Travel Time and Reliability:** The project aims to enhance travel time and reliability between Lineside Road and Pegasus, increasing the efficiency and productivity of the transport network. This enhancement will ensure smoother traffic flow and reduce delays, benefiting both local commuters and freight transport.
- **Reduce Severance and Improve Accessibility:** By reducing severance, the project seeks to improve accessibility to social, cultural and economic opportunities for local communities. This objective is crucial as it enables residents to connect more effectively with essential services, employment opportunities, and recreational facilities.
- **Improve Safety of the Transport Network:** A core focus of the project is to improve the safety of the transport network by minimizing exposure to crash risks and reducing the number of deaths and serious injuries. This aligns with national priorities for transport safety and aims to create a more secure environment for all road users.

In accordance with the RoNS Standardised Design, the project will incorporate best practices in roadway design. It includes a 10 km extension of SH1 featuring a 6 km bypass around Woodend, which will operate as a four-lane highway. Key upgrades will consist of grade-separated interchanges and overpasses at critical junctions, promoting optimal traffic flow and enhancing safety outcomes. At the time of writing a decision on whether the route will be tolled has not been made, and therefore for the purpose of this document it is presumed that it is not.

The SH1 Belfast to Pegasus project embodies a strategic investment in New Zealand's transport infrastructure. By focusing on enhancing connectivity, resilience, and safety, this project not only addresses current transport challenges but also lays the groundwork for a robust network that will support the region's growth and improve the overall quality of life for its residents.

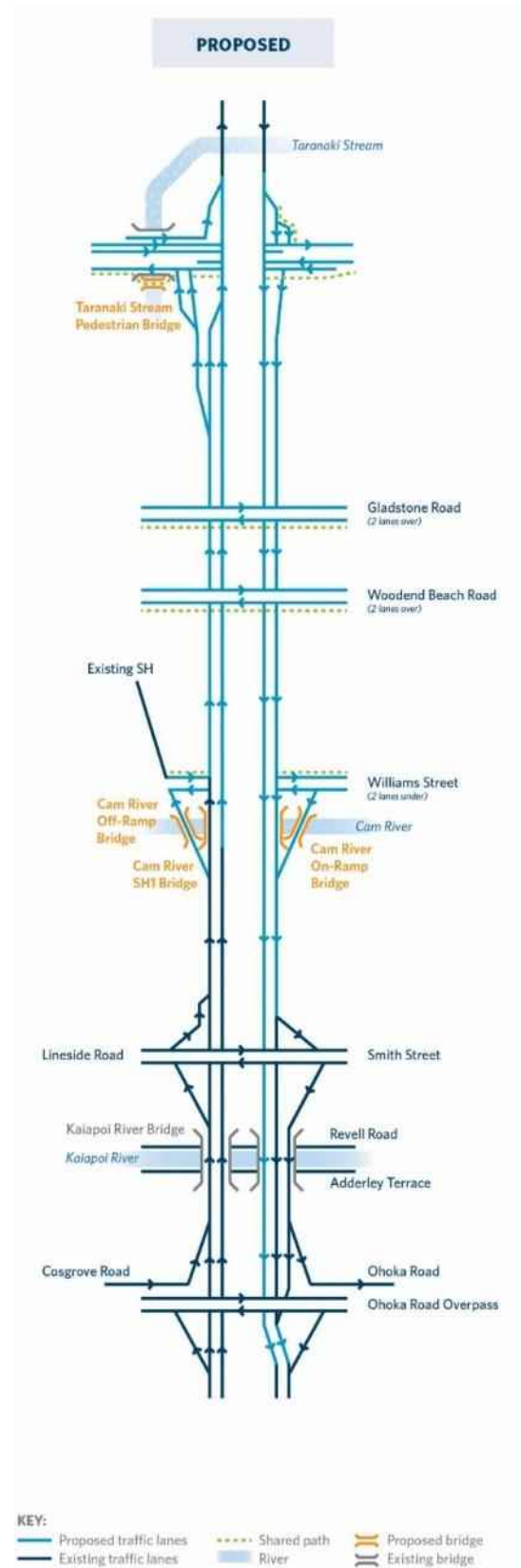


Figure 3 Belfast to Pegasus Line Diagram

2 Design process

The report and the design drawings was issued at 50% design stage. The current revision reflects detailed design development at 100% design stage, this report shall be read in conjunction with the 100% design drawings. The purpose of staged delivery of this design package is to provide stakeholders with an opportunity to comment and contribute to successful outcomes of the Project. Stakeholders are to provide comment and following the receipt of comments, the design team will endeavour to address the issues raised prior to the next delivery milestone.

The following sections summarise key changes made as part of the staged delivery programme to date.

2.1 Design development since previous stage

2.1.1 Design development to 50%

The 50% design included general design development since the:

- Design Philosophy Report (11320-AUR-0350-PWI-DM-RPT-0001) prepared on 11 July 2025,
- 30% detailed design drawings (11320-AUR-0350-PWI-RD-DRG) prepared on 2 July 2025 and
- Stormwater and Flooding Assessment (AEE) Report (11320-AUR-0350-PWI-SW-RPT-0003) prepared on 22 August 2025.

Key changes to the stormwater design between the 30% detailed design submission (consenting deliverable) and 50% detailed design submission are outlined below:

Table 1 Key changes to the stormwater design between 30% and 50% detailed design

Item	Previous design requirement	50% design requirement	Reason
Culvert cover criteria	Culverts under roads (with the potential for heavy traffic) will generally be designed with a minimum cover of 1.2m. Concrete capping will be provided for culverts/pipes with cover between 0.6 and 0.9m.	Culverts with more than 0.6m cover will not have concrete capping protection provided that the pipe strength is checked and satisfied. Culverts/pipes with cover less than 0.6m will have concrete capping protection.	Revised with departure (11320-AUR-0350-PWI-DM-DEP-0015) submitted as part of the project
Pipe cover	Target pipe cover for network drainage pipes in trafficable areas is generally 1.2m and 0.9m in non-trafficable areas	Target pipe cover for network drainage pipes in trafficable areas is generally 1.0m and 0.9m in non-trafficable areas	Revised to align with Design Philosophy report Section 6.2.3
Culvert conveyance performance criteria	Culverts to convey 10-year ARI event flow without surcharging the pipe soffit	Culverts to convey 100-year ARI event flow without surcharging more than 2m above the pipe soffit	Revised to align with latest NZTA P46 specification (Feb 2025 version)
Culvert conveyance performance criteria	Convey the 1% AEP event without increasing upstream flood levels outside the designation	Convey the 1% AEP to ensure no significant increase in flood levels at neighbouring dwellings and limit any flood level increase at neighbouring dwellings to no more than 20 mm during the 200-year ARI event	Revised to align with flood modelling requirement

2.1.2 Design development to 100%

The 100% design includes general design development since the:

- Design Philosophy Report (11320-AUR-0350-PWI-DM-RPT-0001) prepared on 11 July 2025,
- 50% detailed design drawings (11320-AUR-0350-PWI-RD-DRG) prepared on 17 Sept 2025 and
- Stormwater and Flooding Assessment (AEE) Report (11320-AUR-0350-PWI-SW-RPT-0003) prepared on 22 August 2025.

Key changes to the stormwater design between the 50% detailed design submission the latest 100% detailed design submission are outlined below:

- Woodend Beach Road culvert (No. 8) size updated from twin 2m x 1m box to twin DN1350 pipes
- Gladstone Road culvert (No. 10) size updated from twin DN1050 pipes to twin DN900 pipes
- Taranaki North Branch culvert (No. 17) size updated from 2m x 1m box culvert to DN1200 pipe
- Basin updates to layout and sizing
 - Pegasus basin separated into two basins (north and south of Bob Robertson Drive)
 - Waihora basin increased in volume to 239 m³
 - Outlet structure and sizing completed

2.2 Requirements and Standards Compliance

2.2.1 Design criteria

An overarching Design Philosophy Statement (DPS) Report (11320-AUR-0350-PWI-DM-RPT-0001 Rev B]) issued on 11 July 2025 and Stormwater and Flooding Assessment (AEE) Report issued on 5 September 2025, outline the design philosophy, design standards and design criteria to be considered during the detailed design phase to achieve the project outcomes.

Key criteria from the DPS and AEE reports are further summarised below for direct reference.

The following standards, reference documents and guidelines have been used in the development of the detailed design.

- P46 Stormwater Management and Minor Stream Diversion Design Specification (NZTA, 2025);
- NZTA Standardised Designs Solutions for Use on State Highway Roads of National Significance Version 3 (Draft), version 2 (NZTA, 2024)
- Engineering Code of Practice (Waimakariri District Council, 2009)
- Waterways, Wetlands and Drainage Guide (Christchurch City Council, 2003);
- New Zealand Fish Passage Guidelines, version 2 (NIWA, 2024)
- Constructed Waterway Design Manual (Melbourne Water, 2019)
- NZTA specification for earthworks construction (NZTA/ TNZ F1:1997)
- NZTA specification for pipe culverts construction (NZTA F3, 2010)
- NZTA specification for subsoil drainage (NZTA F2, 2013);
- Stormwater Treatment Standard for State Highway Infrastructure (NZTA, 2010);
- Technical Memorandum TM-2502: Preferred method for calculating road surface water run-off in New Zealand (NZTA, 2021);
- Bridge Manual, 3rd edition, Amendment 4 (NZTA, 2022);

- Climate Change Effects and Impacts Assessment - A Guidance Manual for Local Government in New Zealand (MfE, 2008) and Climate Change Projections for New Zealand (MfE, 2018);
- Part 630 Hydrology, National Engineering Handbook, Chapter 7 Hydrologic Soil Groups, Chapter 8 Treatment Classes, and Chapter 9 Hydrologic Soil-Cover Complexes (For use where NRCS (SCS) Curve Number Method is applied) (US NRCS);
- Hydrological Engineering Centre (HEC), HEC-9, Debris Control Structures, Evaluation and Countermeasures (US Department of Transportation (USDOT), 2005);
- HEC-11, Design of Riprap Revetment (Primarily Chapters 3 and 4 for riprap design options) (USDOT, 1989);
- HEC-14, Hydraulic Design of Energy Dissipaters for Culverts and Channels (USDOT, 2006);
- HEC-15, Design of Roadside Channels with Flexible Linings (USDOT, 2005);
- HEC-18, Evaluating Scour at Bridges (USDOT, 2012);
- HEC-20, Stream Stability at Highway Structures (FHWA, 2012)
- HEC-22, Urban Drainage Design Manual (USDOT, 2013);
- HEC-23, Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance-Third Edition, Volumes 1 and 2 (USDOT, 2009);
- National Cooperative Highway Research Program (NCHRP) 24-20, Estimation of Scour Depth at Bridge Abutments (NCHRP, 2010);
- NCHRP 24-37, Combining Individual Scour Components to Determine Total Scour (NCHRP, 2017);
- AGBT08-19, Austroads Guide to Bridge Technology (AGBT) Part 8: Hydraulic Design of Waterway Structures (Austroads, 2019)
- Australia/New Zealand Standard (AS/NZS) 3725 - Design for Installation of Buried Concrete Pipes (AS/NZS, 2007);
- Engineering Guideline, Selecting Materials for Bedding Steel Reinforced Concrete Pipe (CPAA, 2013);
- Guidance Note: Loads on Circular Precast Concrete Manholes and Access Chambers (CPAA, 2016);
- Stormwater Proprietary Device: Media-Filled Cartridge Filtration System Specification (ASM, 2020)

2.2.2 Departures from Standards

Departures from the design standards identified at this stage are highlighted in Table 2 following. The full text of the departures is provided within Appendix B of this design report.

Table 2 Requested Departures

Departure No.	Description	Status
11320-AUR-0350-PWI-DM-DEP-0010	Departure for deck drainage and water treatment at Kaiapoi River Bridge	Conditional approved by CEAG on 29/8/2025, subject to alignment with resource consent conditions
11320-AUR-0350-PWI-DM-DEP-0015	Reduction in minimum pipe cover for some culverts	Assessed as not a departure (pipe cover within specification)
11320-AUR-0350-PWI-DM-DEP-0016.pdf	Bridge abutments scour assessment methodology	Declined
11320-AUR-0350-PWI-DM-DEP-0017.pdf	Runoff flowing across road median	Accepted

Departure No.	Description	Status
11320-AUR-0350-PWI-DM-DEP-0018.pdf	Use of RCP8.5 Climate Change for Culvert Design	Declined, approach agreed with NZTA PM.

2.2.3 Design Interfaces

The following design interfaces have been coordinated with throughout the design process.

Table 3 Design Interfaces

Discipline	Interface with Stormwater Design
Geotechnical	Ground conditions to inform infiltration rates and ground improvement requirements for culverts/pipes.
Roading	Road levels, high/low points, barriers, and crossfalls critical for surface water management.
Structures	Design collaboration for major culverts and headwalls to ensure hydraulic and structural integrity.
Ecology	Provision for fish passage and lizard habitat recovery within waterway and open channel designs.
Pavement	Surface and pavement drainage to protect road integrity and tie into stormwater systems.
Survey	Existing surface and assets surveys for design inputs
Utilities & ITS	Alignment of service trenches, pits, and ITS foundations (signs/gantries) ensuring no conflict with proposed stormwater system.
Noise Barriers	Location and form of barriers, with consideration of their impacts on overland flow paths.

2.2.4 Issues requiring resolution in IFC design stage

The stormwater design has been progressed and coordinated with the roading and other disciplines designs. The process has identified potential areas for refinement to achieve cost saving and improved design performance, which will be addressed and resolved as part of the IFC design stage. Additionally, feedback from the 50% detailed design review, along with further design developments moving towards the IFC design, are summarised below.

- Finalise culvert inlet/outlet scour protection details and confirm tie ins with headwalls and channels.
- Finalise detailed designs of stream diversions with input from ecology team, while minimising impact on existing trees in key locations
- Waihora stream realignment and bioretention basin to be optimised to align to recently agreed land constraints and minimise impact on established trees
- Refine basin outlet tie ins and model optimised outlet sizing.
- Rationalise local road stormwater reticulation and access at Pegasus Boulevard, Bob Robertson Drive, Gladstone Road, Woodend Beach Road, and Williams Street
- Landscaped areas below retaining walls at Pegasus and Williams St interchanges to be designed
- Check dam locations in bioinfiltration swales to be specified and typical details to be refined

- Drainage through slots in concrete barriers to be confirmed
- Drainage of runoff from roadside noise walls / bunds to be intercepted and conveyed prior to flowing onto carriageway
- Riprap model at Cam River Bridge to be adjusted
- Details to be refined

2.2.5 Design reviews

The 50% Detailed Design was submitted to all key stakeholders and members of the project team for design review.

Feedback was received from Waimakariri District Council on the flood modelling report agreed updates incorporated into the 100% detailed design. The updated flood modelling report is also appended to this report.

Endorsement of the resource consent package, including the 30% design, was received from Whitiora.

3 Analysis and design

The drainage design is presented on the following drawings for the 100% detailed design:

- Stormwater general layouts:
 - 11320-AUR-0350-PWI-SW-DRG-0101 to 0117
- Stormwater interchange/ local road plans:
 - 11320-AUR-0350-PWI-SW-DRG-0201, 0202, 0211, 0212
- Stormwater network long sections:
 - 11320-AUR-0350-PWI-SW-DRG-0501 to 0512
- Stormwater culvert plan and sections:
 - 11320-AUR-0350-PWI-SW-DRG-0551 to 0552
- Stormwater basin plan and sections:
 - 11320-AUR-0350-PWI-SW-DRG-0601 to 0605
- Stormwater details:
 - 11320-AUR-0350-PWI-SW-DRG-1101 to 1106 and 1201 to 1211
- Stormwater major and significant culvert plan and sections:
 - 11320-AUR-0350-MJC-SW-DRG-0101 to 0901

3.1 Existing Drainage

Existing drainage within the project extents is described in Appendix C of the AEE Report (11320-AUR-0350-PWI-SW-RPT-0003 Section 3.2).

3.2 External Stormwater Management

The external stormwater management system for the SH1 Belfast to Pegasus project is designed to control, convey and divert runoff from surrounding catchments to protect the highway infrastructure and prevent environmental harm. Key elements include cut-off drains, channel realignments, and cross culverts that work together to safely manage offsite flows without mixing clean water with highway runoff. These components are sized and located in accordance with NZTA standards and account for climate change impacts to ensure long-term resilience.

3.2.1 Cut-off Drains

Cut-off drains are open channels constructed parallel to the roadway to intercept surface water from external catchments before it reaches the highway. Their purpose is to separate and divert “clean” offsite water away from the contaminated highway runoff, maintaining water quality and reducing unnecessary load on the highway stormwater management systems.

Cutoff drains are planned for low spots along the embankment that are away from discharge points or where the land is flat, such as the road section between the Lineside Road to Cam River. These drains help ensure water flows properly towards the designated discharge points. Figure 4 shows a typical design of a cutoff drain. The proposed cutoff drains have a side slope varying between 10H:1V to 2H:1V. Flatter side slopes have preferentially been provided where possible to make the drains easily traversable allowing access for maintenance. The drains will be grassed to blend in with the surrounding pasture.

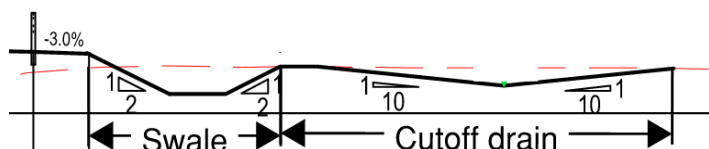


Figure 4 Typical cutoff drain profile

The proposed cutoff drains north of the Cam River have been evaluated using the TUFLOW flood model to ensure they are adequately sized to safely manage flows from a 1% AEP storm event, incorporating allowances for climate change based on RCP 8.5 projections up to the year 2100. For the cutoff drains south of the Cam River, where the proposed lane widening is minimal, the drain sizes will remain consistent with existing dimensions.

The invert of cutoff drains throughout the project area are generally planned to be at the current ground level where possible, to avoid intercepting groundwater. However, there are some locations, particularly in the Lineside Road to Cam River section on both sides of the proposed highway, where the drains may intercept groundwater due to the low lying flat terrain and high groundwater level. In this section, the new drains are replacing existing channels and structures at similar levels and slopes. This means that although the drains might intercept groundwater during times of high-water levels, they will not increase the amount of groundwater intercepted compared to the existing channels.

3.2.2 Channel Realignment

To accommodate the proposed roadway's alignment, several sections of existing streams and channels are proposed to be realigned. Realignment works involve reshaping and diverting sections of Waihora Stream and Taranaki Stream to maintain hydraulic connectivity comparable to the existing condition where the current streams are affected by the highway alignment. The realigned channels are carefully designed to match the existing channel geometry, slope, and hydraulic function and to preserve fish passage and maintain overall channel stability. Flood model testing of the designs ensures that these realignments do not result in increased flood risk or channel erosion. Revegetation and appropriate bank protection measures are included to encourage long-term ecological and geomorphic balance.

See Table 4 below for the proposed stream diversions (indicative) as part of the project scope.

Table 4 Proposed Channel Realignments

Diversion Channel ID	Project Chainage	Channel	Realignment Distance [m]	Channel Type
SD01	Woodend Beach Rd 380 to 550	Waihora Stream	170	Ephemeral
SD02	6380 to 7580	Waihora Stream	1200	Ephemeral
SD03	7820 to 8300	Waihora Stream	480	Ephemeral
SD04	9100 to 9110	Waihora Stream	11.5	Perennial at top, transitioning to Ephemeral
SD05	9100 to 9220	Waihora Stream	130	Perennial (spring fed and inter-catchment transfer from Taranaki Stream)

Diversion Channel ID	Project Chainage	Channel	Realignment Distance [m]	Channel Type
SD06	9640 to 9700	Taranaki Stream	60	Perennial
SD07	9700 to 9775	Taranaki North Branch	80	Ephemeral

At CH 9700, the existing SH1 Taranaki Stream culvert will be replaced with a new longer culvert in a new alignment to accommodate the new roadway. This involves building about 70 m of Taranaki Stream and 80 m of the northern branch channel that currently joins Taranaki Stream at the upstream end of the existing 2.2 m x 1.2 m SH1 culvert. In this section, Taranaki Stream flows continuously throughout the year. To protect the ecological values of the stream, the realigned channel will replicate the shape of the existing channel, taking into account fish passage needs as detailed in Section 3.2.3 Fish Passage.

The Waihora Stream will also be realigned in several areas to accommodate the new roadway. Between CH 9100 and 9220, where the stream is spring-fed and receiving inter-catchment transfer from Taranaki Stream via a weir and pipe, the stream flows year-round. The new channel will match the current channel's shape and slope as much as possible, considering fish passage requirements for the Waihora Stream culvert.

In other sections where the stream is poorly defined, shallow, and only flows during heavy rain events, the channel has been designed to prevent significant adverse effects during both 100-year and 200-year flood events caused by the new road and realignment. These outcomes have been confirmed through flood modelling, as explained in the flood modelling report.

The proposed diversion channel locations have been evaluated using the TUFLOW flood model to ensure they are adequately sized to safely manage flows from a 1% AEP storm event, incorporating allowances for climate change based on RCP 8.5 projections up to the year 2100. The proposed stream diversion sizes and geometry will be finalised and incorporated into the IFC detailed design.

3.2.3 Cross Culverts

In general, culverts have been proposed at locations where concentrated overland flows cross the proposed works. Culvert works also includes works to and/or replacement of existing culverts. Culverts have been designed to maintain (as much as practicably possible) the existing flow regimes.

Culvert sizes and levels have been designed based on the following hydraulic criteria:

- Convey the 1% AEP event without surcharging the pipe more than 2m above the pipe soffit;
- Convey the 1% AEP event with minimum 500mm freeboard to the road surface (i.e. outer edge line)
- Convey the 1% AEP event to ensure no significant increase in flood levels at neighbouring dwellings and limit any flood level increase at neighbouring dwellings to no more than 20 mm during the 200-year ARI event
- All culverts have a minimum cover of 0.6 m below trafficked road

Abandonment/Removal of existing Cross Culverts

Within the Project Site, where existing culverts affected by the works require decommissioning, they will be removed, or void backfilled to prevent future ground instability.

Refer to Table 5 below for a list of culverts that will be decommissioned:

Table 5 List of culverts to be decommissioned

Existing Culvert ID	Chainage (m)	Surveyed Size (mm)	Proposal
Existing 01	1610	Ø1500	Removed from ground, replaced by proposed Culvert 01
Existing 02	2350	Ø1500	Removed from ground, replaced by proposed Culvert 01
Existing 08	540 (Woodend Beach Rd)	Ø300	Removed from ground or void backfilled
Existing 10	400 (Gladstone Rd)	Box 1250 x 500	Removed from ground and backfill
Existing 14	9200	Box 4000 x 1500	Removed from ground, replaced by proposed Culvert 14
Existing 16	9700	Box 2200 x 1200	Removed from ground, replaced by proposed Culvert 16
Existing 17	9830	Oval pipe ~850 x 650	Removed from ground, replaced by proposed Culvert 17

Proposed Cross Culverts

New cross culverts are installed where the highway crosses existing waterways or drains. Culvert sizes vary by catchment and flow requirements, with some identified culverts designed to accommodate fish passage following New Zealand Fish Passage Guidelines. Culvert inlets and outlets are protected by wingwalls and riprap rock to prevent scour and erosion during high flow events.

The culverts are classified on the following basis:

- **Major** culverts are identified by the Highway Structures Design Guide (HSDG) Clause 5.6 as those with a total waterway area greater than 3.4m²
- **Significant** culverts identified are those that convey existing natural water courses, drains or flow paths and those require fish passage.
- **Minor** culverts are remaining cross drainage culverts for the proposed highway and adjacent local roads.

The structural design of culverts with a:

- Cross-sectional area of greater than 3.4 m², which are considered as major culverts, will use:
 - Specific structural design (Structural, Geotechnical and Drainage Teams) in accordance with the NZTA Bridge Manual.
- Cross-sectional area of less than 3.4 m², which are considered as minor or significant culverts, will use:
 - AS/NZS 3725 Design for installation of buried concrete pipes (The Concrete Pipe Association of Australasia “Pipe Class” software has been used for structural design of RCRRJ pipes);
 - AS/NZS 2566.1:1998 Buried flexible pipelines Part 1: Structural Design for all uPVC and PE pipes;
 - Pre-approved designs for precast box culvert units from suppliers (i.e. Hynds, Humes or other approved suppliers); and
 - Specific structural design (Structural, Geotechnical and Drainage Teams) for customised box culverts (NZS 3101 and NZS 3109).

For proprietary circular pipe and box culverts:

- All circular pipe culverts will be Reinforced Concrete Rubber Ring Jointed (RCRRJ) pipes, unless requested otherwise.

- Culverts located under roads subject to heavy traffic are generally designed with a minimum cover depth of 1.2 m. Some culverts have a reduced minimum cover of 0.6 m.
- Precast concrete wingwalls will be provided on all pipe inlets and outlets of 450mm diameter or greater. For diameters of 375mm or less, the pipe end is proposed to be cut to match the slope of the adjacent ground and surrounded with erosion protection

The culvert sizing design basis is outlined below:

- Major and significant culverts have been individually assessed to meet hydraulic and ecological standards. Design decisions draw on detailed flood modelling using TUFLOW and hydraulic analysis with HY-8, ensuring culverts can convey design flood flow events without causing upstream flooding or compromising road freeboard. The detailed assessment of major and significant culverts is documented in Appendix D B2P Major and Significant Culverts Memo (11320-AUR-0350-MJC-SW-MEM-0001).
- Minor culverts designed to convey swale runoff, are sized using HY-8, ensuring culverts can convey the swale runoff downstream while providing compliant freeboard.
- Two existing cross culverts are located between Lineside Road and Cam River (Culvert 1 and 2). The project proposes widening the existing highway at these locations, and due to the age and condition of the culverts, it is proposed to replace these culverts to achieve an appropriate asset life. These culverts fall outside the flood modelling extent; therefore, local catchment analysis was conducted using the SCS method as detailed in Appendix D B2P Major and Significant Culverts Memo Section 3.1 and 3.2.

Refer to the table below for list of proposed cross culverts

Table 6 Proposed Cross Culverts

Culvert ID	Chainage (m)	Size (mm)	Approx. Length (m)	1% AEP with climate change			Notes	Fish Passage
				Flow rate (m ³ /s)	Freeboard to outer edge (m)	Surcharge above soffit (m)		
01 Significant	1610	Ø1500	29.9	Refer to Appendix G – Flood Impact Assessment South of Cam River			Rossiter Drain, replacement of existing culvert	Y
02 Significant	2350	Ø1500	35.4				Wilson Drain, replacement of existing culvert	Y
05 Major	5000	Box 6000 x 2000	36.5	4.96	2.29	No surcharge	McIntosh Drain, new culvert, wet channel	Y
08 Major	540 (Woodend Beach Rd)	Twin Ø1350	33.7	2.61	5.85	No surcharge	Waihora Stream, new culvert, dry channel	N
10 Significant	320 (Gladstone Rd)	Twin Ø900	47.0	1.35	8.89	No surcharge	Waihora Stream, new culvert, dry channel	N
12 Minor	7980	Ø600	50.1	0.19	0.40	0.22	New culvert, dry channel	N
13 Minor	9110	Ø600	51.4	0.06	1.89	No surcharge	New culvert, dry channel	N
14 Major	9210	Box 4000 x 1500	50.2	0.04	3.02	No surcharge	Waihora Stream, wet channel	Y
15 Major	100 (Bob Robertson Dr)	Twin Ø1800	Extension only	1.62	1.23	No surcharge	Extension of ex Bob Robertson culverts, wet channel	Y (match existing)
16 Major	9700	Box 2700 x 1500	66.8	4.72	1.11	0.51	Taranaki Stream, wet channel	Y

Culvert ID	Chainage (m)	Size (mm)	Approx. Length (m)	1% AEP with climate change			Notes	Fish Passage
				Flow rate (m ³ /s)	Freeboard to outer edge (m)	Surcharge above soffit (m)		
17 Significant	9830	Ø1200	46.4	0.25	1.31	No surcharge	Taranaki Branch, existing 650x850mm brick barrel, dry channel	N

The proposed culvert locations are shown in Figure 5 below.

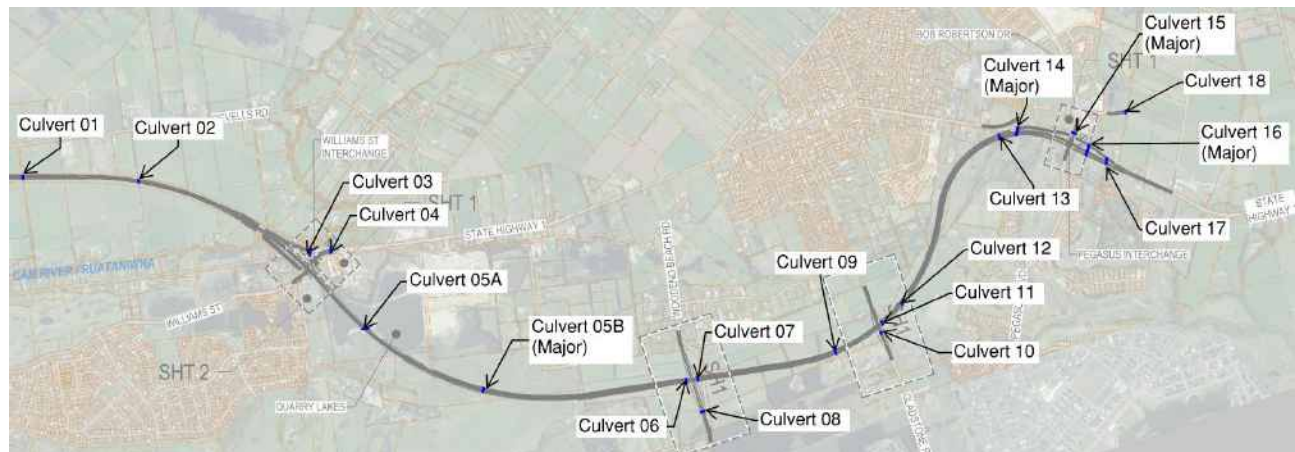


Figure 5 Proposed culvert locations

Fish Passage

The culverts that need to accommodate fish passage are listed in Table 6 above. The design for fish passage follows the New Zealand Fish Passage Guidelines v2.0 (NIWA, 2024) and the Fish Passage Guidelines for State Highways (NZTA, 2013). Each culvert is being individually reviewed to determine fish species and therefore appropriate fish passage measures, at the 100% stage this generally includes the following considerations:

- Inanga, found in all streams in the region, have been used as the key species to assess suitable water velocities within the culverts.
- Brown Trout, present in Taranaki Stream and detected via eDNA in the upper Waihora Stream, have been used alongside Inanga as a benchmark species to determine appropriate water depths in these streams.

Fish passage design basis is summarised below:

- Culvert replacements (Culverts 1 and 2): The proposed culverts match the existing culverts in size, level, and embedment. The intent is to replace them with minimal impact on existing fish passage conditions.
- Culvert extension (Culvert 15): The proposed extension is designed to match the existing culvert in size, level, and embedment. The intent is to maintain current fish passage performance.
- New culverts in wet streams (Culverts 5B, 14, and 16): The proposed culverts will be embedded approximately 0.3m below the streambed and backfilled with natural streambed material. The culvert width will be no narrower than the existing stream width and provide adequate conveyance capacity. At Culvert 5B, a defined low-flow channel will be formed in the culvert base to match the existing channel width. The intent is to provide continuous fish passage through the culverts, consistent with the natural stream conditions they connect to.

Culvert Outfall Erosion Protection

All culverts have been evaluated for erosion risk following the guidance in HEC-14 - Hydraulic Design of Energy Dissipators for Culverts and Channels. Where needed, riprap has been designed to protect against

erosion at culvert inlets and outlets. Given the low-energy site conditions, riprap aprons are sufficient for energy dissipation. At the 100% design stage, Riprap aprons are designed based on predicted culvert tie-ins, these are conservative, with refinement to follow at the IFC detailed design. Table 7 and Table 8 summarise the proposed apron dimensions for each culvert.

Table 7 Indicative apron dimensions for major and significant culverts (including both inlet and outlet)

Culvert ID	Nominal Size (mm)	D50 (mm)	Apron Length (m)	Apron Width (m)	Apron Thickness (m)
01	Ø1500	250	7.5	5	0.6
02	Ø1500	250	7.5	5	0.6
05	Box 6000 x 2000	250	10.0	12.0	0.6
08	Twin 2000 x 1000	250	10.0	14	0.6
10	Twin Ø1050	250	5.5	8.5	0.6
14	Box 4000 x 1500	250	No inlet riprap required due to low velocity. Outlet riprap to be governed by Waihora spillway design		
15	Twin Ø1800	250	9	14	0.6
			Above dimensions only apply to inlet protection. No changes proposed to existing outlet protection		
16	Box 2700 x 1500	250	13.5	15.0	0.6
17	Ø1200	250	No inlet or outlet protection required due to low velocity		

Table 8 Indicative apron dimensions for minor culverts (including both inlet and outlet)

Culvert ID	Nominal Size (mm)	D50 (mm)	Apron Length (m)	Apron Width (m)	Apron Thickness (m)
03	Ø375	250	Downstream to discharge to riprap lined swale		
04	Ø375	250	No inlet or outlet protection required due to low velocity		
06	Ø600	250	3.0	4.0	0.6
07	Twin Ø900	250	7.5	5.5	0.6
09	Ø600	250	3.0	4.0	0.6
11	Ø600	250	3.0	4.0	0.6
12	Ø600	250	3.0	4.0	0.6
13	Ø600	250	3.0	4.0	0.6

3.3 Proposed Stormwater Quality and Quantity Management

Stormwater drainage along the proposed highway includes a mix of treatment and conveyance devices to manage runoff quantity and quality. These include grass-lined swales, bioinfiltration and bioretention swales and basins, designed to treat contaminants such as sediments, heavy metals, and hydrocarbons while reducing runoff volumes through infiltration and detention. Their application depends on site constraints, soil permeability, and hydrological conditions, aiming to replicate the natural catchment behaviour as closely as possible. Table 9 summarises the stormwater treatment devices proposed across the project.

Table 9 Treatment Methodology Overview

Type	Vegetation	Primary Discharge	Secondary Discharge	Subsoil drains
Grass Lined Swale	Grass	Surface	Surface	No
Bioinfiltration Swale	Planted	Passive Infiltration	Surface	No
Bioretention Swale	Planted	Surface	Surface	As needed by grade and location – none currently proposed
Bioinfiltration Basin	Planted	Passive Infiltration	Surface	No
Bioretention Basin	Planted	Surface	Surface	As needed – none currently proposed

The hydrological parameters that have been used for the assessment and sizing of stormwater quality and quantity (hydrological mitigation for stream protection) treatment devices are summarised in Table 10.

Table 10 Stormwater treatment design parameters

Design Requirement	Rainfall
Water Quality Flow Rate	5 mm/hr intensity*
Water Quality Volume	25 mm depth**

* In accordance with the guidance provided by the Canterbury Regional Stormwater Forum

** Based on Appendix A in the NZTA Stormwater Treatment Standard for State Highway Infrastructure

3.3.1 Grass Lined Swale

Grass-lined swales are broad, shallow channels with grass surfaces that slow stormwater flow, allowing sediments to settle and filtering contaminants when sufficient residence time is achieved. They are suitable where there is enough roadside space for mowable grass batters and at locations where infiltration is limited or not required. They typically achieve over 70% removal of total suspended solids and can reduce some attached heavy metals but have limited nutrient removal capacity. Refer to Figure 6 for grass lined swale typical detail.

A grass-lined swale is proposed within the southern Kaiapoi River catchment, as detailed in Section 3.4, extending from Ohoka Road to Kaiapoi Bridge. This swale will be situated within the median strip. To enhance road safety, a gentle side slope of 5H:1V is recommended.

Grass-lined swales have been designed in accordance with the Stormwater Treatment Standard for State Highway Infrastructure (NZTA, 2010, Section 8.5.1). The design ensures that, during the water quality event (5 mm/hr, as required by the Canterbury Regional Stormwater Forum), flow depths remain below the 150 mm grass height, and that during the 10-year ARI rainfall event, water depths do not exceed the total swale depth. Where a secondary flow path is not available, water depth during the 100-year ARI rainfall event has been checked to ensure runoff stays within the swale depth with the appropriate level of freeboard to the road edge line (Table 11). Due to grass-lined swale's considerable length, the critical time of concentration is evaluated individually for each swale segment for the purpose of conveyance flow calculation under the 10-year and 100-year design event. For calculations, a Manning's n value of 0.25 is applied during the treatment event, representing the grassed swale conditions, and a reduced value of 0.03 is used during conveyance events. The swale is designed to have a maximum base width of 2 m with minimum hydraulic residence time of nine minutes. Additionally, where the swale's longitudinal slope is less than 2%, subsoil drains as shown in Figure 7 will be installed beneath the swale to improve drainage performance.

Table 11 Grass Lined Swale design summary

Swale ID	Length [m]	Catchment [m ²]	Q100, Flow Rate [m ³ /s]	Q100, Flow Depth [m]	Q100, Freeboard [m]	WQE Flow rate [m ³ /s]	WQE Flow depth [m]	WQE Average Residence Time [min]
TS01	162	1601	0.042	0.10	0.50	0.002	0.06	51
TS02	656	6931	0.126	0.11	0.49	0.009	0.09	74

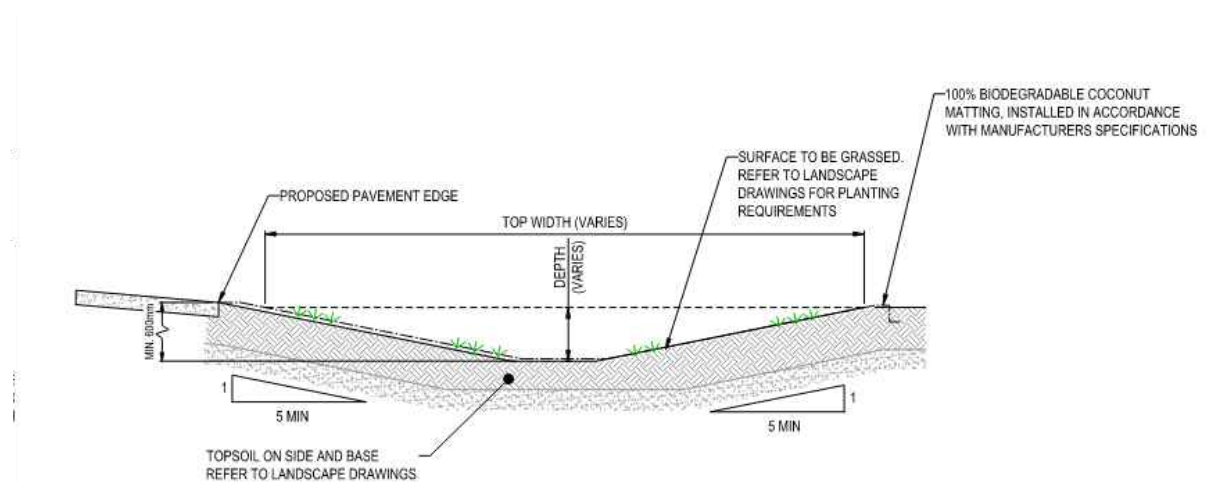


Figure 6 Typical detail of grass lined swale

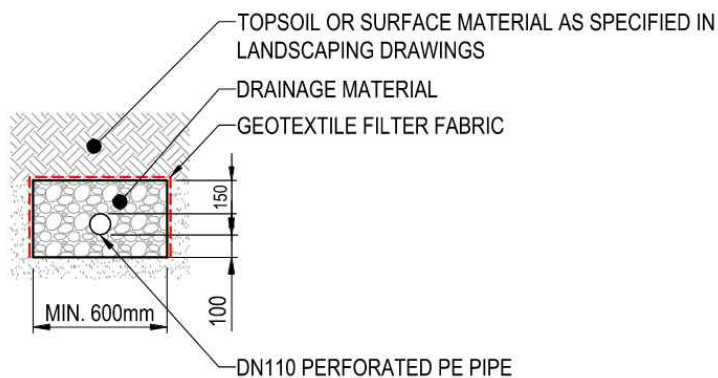


Figure 7 Typical detail of subsoil drain

3.3.2 Bioinfiltration Swale

Bioinfiltration swales are engineered channels designed to improve stormwater quality by utilising the biological and phytoremediation processes within the soil layers at their base. They are strategically located in areas suitable for infiltration-based stormwater management, relying on the local soil permeability and groundwater conditions, to promote hydraulic neutrality across the catchment. These swales effectively remove a compliant percentage of total suspended solids (TSS) and enhance the reduction of dissolved heavy metals and nutrients through active biological treatment within the soil. To promote infiltration, bioinfiltration swales incorporate 250 mm high permeable (rock / riprap) check dams that temporarily retain water over an engineered base material.

The bioinfiltration swale is engineered to achieve a minimum infiltration rate of 50 mm/hr through the bottom of the swale. Check dam spacing has been designed to impound water throughout the swale (matching crest

to toe) which optimises the swale's base area to promote infiltration immediately behind each dam. The design ensures that, during the water quality event (5 mm/hr, as required by the Canterbury Regional Stormwater Forum), flow depths remain below the 300 mm plant height, and that during the 10-year ARI rainfall event, water depths do not exceed the total swale depth. The infiltration media in the base of the swale helps with offsetting the increased runoff from the alignment into downstream catchments.

During heavy rainfall events, when inflow surpasses the swale's capacity to treat, store, and infiltrate water, the excess runoff is safely conveyed downstream through the swale system before being discharged into surface water flow paths.

The bioinfiltration swale is designed with a maximum base width of 2 m, side slopes graded at 2h:1v and is to be planted for enhanced treatment and stability. Additionally, the swales are constructed with a minimum depth of 0.6 m to accommodate pavement drainage directly into the swale system.

Refer to section 3.4 to see where bioinfiltration swales have been used throughout the project.

Table 12 Bioinfiltration Swale summary for Water Quality Event (25mm rainfall)

Swale ID	Catchment [m ²]	Length [m]	WQV [m ³]	Total Infiltration and Detention volume [m ³]	WQE Flow rate [m ³ /s]	WQE Average Residence Time [min]
TS 16	9300	560	233	443	0.012	77
TS 17	7176	479	179	379	0.009	68
TS 18A & TS 18B	9024	213	226	169	0.012	30
TS 19	8164	183	204	145	0.011	25
TS 20	9200	564	230	464	0.012	71
TS 21	13100	724	328	573	0.017	82
TS 22	7417	218	185	193	0.010	30
TS 23	5235	125	131	114	0.007	21
TS 24	4251	167	106	132	0.006	34
TS 25	5822	237	146	188	0.008	47
TS 26	5075	139	127	110	0.007	24
TS 27	5626	147	141	129	0.007	19
TS 28	13316	416	333	342	0.018	37
TS 29	11503	436	288	345	0.015	52
TS 30	8303	259	208	205	0.011	35
TS 31	1689	68	42	54	0.002	11
TS 32		63		32		
TS 33	2900	280	73	227	0.004	70
TS 34		153		81		

Refer to Figure 8 for typical detail of bioinfiltration swale.

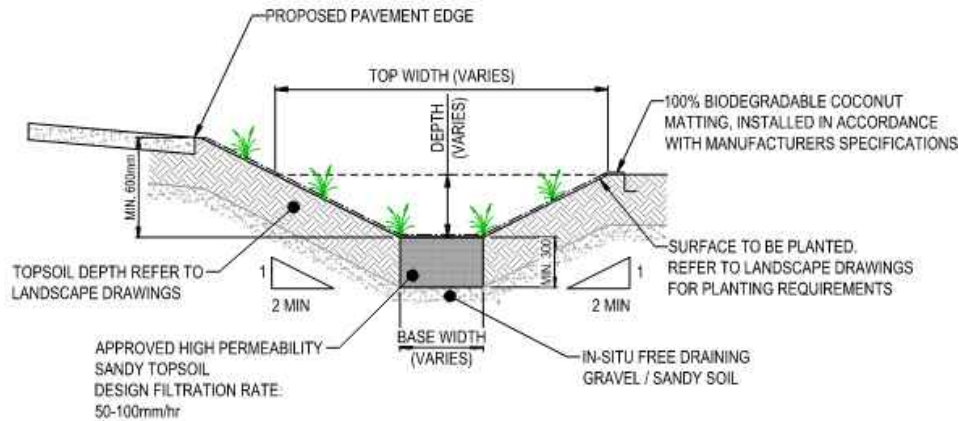


Figure 8 Typical detail of bioinfiltration swale

3.3.3 Bioretention Swale

Bioretention swales are similar to bioinfiltration swales, however they do not rely on infiltration to achieve treatment. Treatment is achieved through settlement of suspended solids and, partially, soil treatment within the vegetated topsoil layers. As bioretention swales are not intended to fully infiltrate treatment flows, they do not require an engineered base material to ensure infiltration rates, allowing water to slowly seep through their permeable check dams over time.

Bioretention swales have been designed in accordance with the Stormwater Treatment Standard for State Highway Infrastructure (NZTA, 2010, Section 8.5.1). The design ensures that, during the water quality event (5 mm/hr, as required by the Canterbury Regional Stormwater Forum), flow depths remain below the 300 mm plant height, and that during the 10-year ARI rainfall event, water depths do not exceed the total swale depth. Where a secondary flow path is not available, water depth during the 100-year ARI rainfall event has been checked to ensure runoff stays within the swale depth with the appropriate level of freeboard to the road edge line. Due to the swale's considerable length, the critical time of concentration is evaluated individually for each swale segment under the 10-year ARI and 100-year events. For calculations, a Manning's n value of 0.25 is applied during the treatment event, representing the planted swale conditions, and a reduced value of 0.1 is used during conveyance events. Bioretention swales are designed to have a maximum base width of 2 m with minimum hydraulic residence time of nine minutes under water quality event.

Refer to section 3.4 to see where bioretention swales have been used throughout the project.

Table 13 Bioretention Swale design summary

Swale ID	Length [m]	Catchment [m ²]	Q100 Flow Rate [m ³ /s]	Q100 Flow Depth [m]	Q100 Freeboard [m]	WQE Flow rate [m ³ /s]	WQE Flow depth [m]	WQE Average Residence Time [min]
TS 03	611	10530	0.135	0.18	0.42	0.014	0.16	139
TS 04	537	10469	0.134	0.17	0.43	0.014	0.16	117
TS 05	425	9197	0.135	0.17	0.43	0.012	0.15	98
TS 06	254	3513	0.064	0.14	0.46	0.005	0.11	60
TS 07	303	7968	0.144	0.17	0.43	0.011	0.13	70
TS 08 & TS 09	493	24680	0.362	0.30	0.30	0.033	0.26	121

Swale ID	Length [m]	Catchment [m ²]	Q100 Flow Rate [m ³ /s]	Q100 Flow Depth [m]	Q100 Freeboard [m]	WQE Flow rate [m ³ /s]	WQE Flow depth [m]	WQE Average Residence Time [min]
TS 10	328	4628	0.084	0.17	0.43	0.006	0.13	71
TS 10A	123	3430				0.005		
TS 11	85	1221	0.032	0.09	0.51	0.002	0.05	27
TS 12	123	2300	0.061	0.14	0.46	0.003	0.08	33
TS 13	320	9533	0.173	0.20	0.40	0.013	0.15	42
TS 14	241	6150	0.163	0.19	0.41	0.008	0.11	34
TS 15	822	20640	0.265	0.20	0.40	0.027	0.22	82

Refer to Figure 9 for typical detail of bioretention swale.

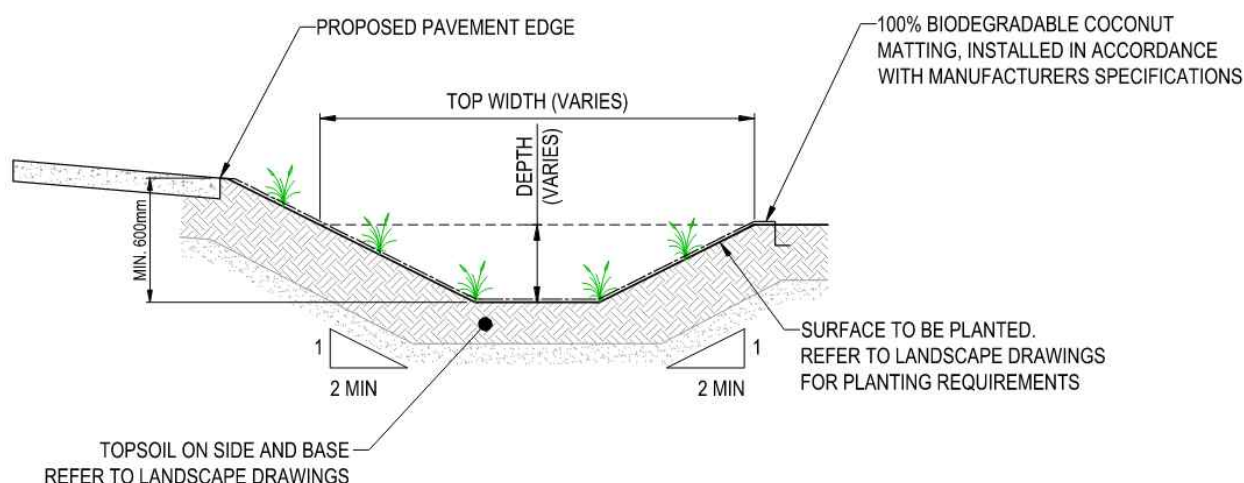


Figure 9 Typical detail of bioretention swale

3.3.4 Bioinfiltration Basin

Bioinfiltration basins are planted basins designed to improve quality by harnessing biological and phytoremediation processes within the engineered soil layers at their base, before discharging to groundwater to promote hydraulic neutrality across the catchment. These basins effectively remove a compliant percentage of total suspended solids (TSS) and enhance the reduction of dissolved heavy metals and nutrients through active biological treatment within the soil.

Bioinfiltration basins are only used along the quarry ponds embankment where local soil permeability and groundwater conditions are suitable for infiltration-based stormwater management. These basins are sized to treat at least the first flush volume from 25mm of rainfall and achieve minimum infiltration rates of 50 mm/hour (refer to the Design Philosophy Statement Report, Section 8.2.4). During heavy rainfall events, when inflow surpasses the basin's capacity to store and infiltrate water, excess runoff will be discharged directly into the quarry ponds via a riprap lined spillway.

This is sized for 15% of the main basin volume, as per the NZTA Stormwater Treatment for State Highway Infrastructure, and allows for sediment and gross pollutant reduction, before water passes through a permeable gabion wall to the main basin. The forebays are not planted and will allow for maintenance access.

Refer to Figure 10 or typical detail of bioinfiltration basin

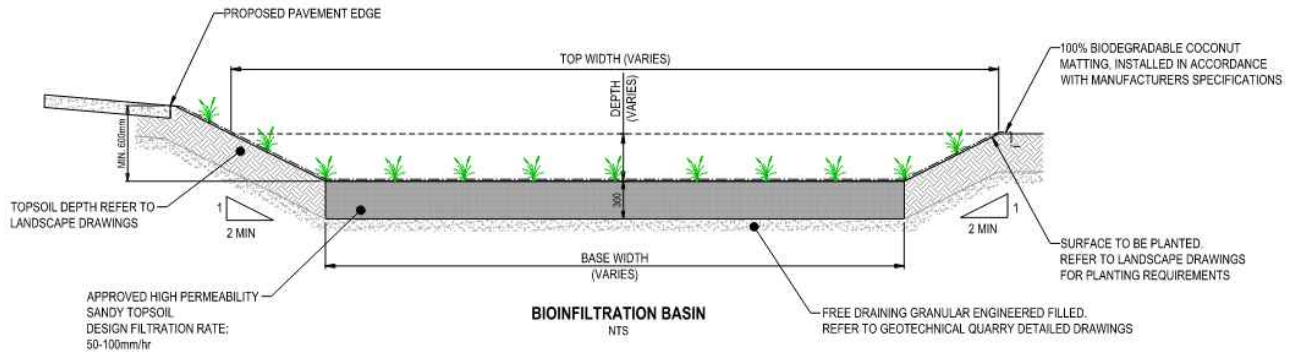


Figure 10 Typical detail of Bioinfiltration Basin

Table 14 Bioinfiltration basin summary

Basin Name	Catchment Area [m ²]	Minimum Treatment Volume Required [m ³]	Design Treatment Volume Provided [m ³]
Combined Quarry Lakes Basins	24000	540	940

3.3.5 Bioretention Basin

Bioretention basins are similar to bioinfiltration basins, however they do not rely on infiltration to achieve treatment. Bioretention basins are planted basins, where treatment is achieved through settlement of suspended solids and, partially, soil treatment within the vegetated topsoil layers. Bioretention basins are designed to provide impoundment volume for a first flush of up to 25mm of runoff to ensure treatment outcomes. As bioretention basins are not intended to fully infiltrate treatment flows, they do not require an engineered base material to ensure infiltration rates, allowing water to slowly infiltrate to ground or drain through an outlet structure to surface water.

Bioretention basins will have a sediment forebay at major inlets to the basins. This is sized for 15% of the main basin volume, as per the NZTA Stormwater Treatment for State Highway Infrastructure, and allows for sediment and gross pollutant reduction, before water passes through a permeable gabion wall to the main basin. The forebays are not planted and will allow for maintenance access.

Excess runoff over the design treatment volume will be discharged to surface water through a scruffy dome manhole outlet structure. The Waihora basin is also designed with a riprap lined spillway above the 10-year ARI storage volume, capable of discharging up to the 100-year ARI flow while achieving a minimum freeboard of 300mm to the edge of road.

Refer to Figure 11 for typical detail of bioretention basin and forebay

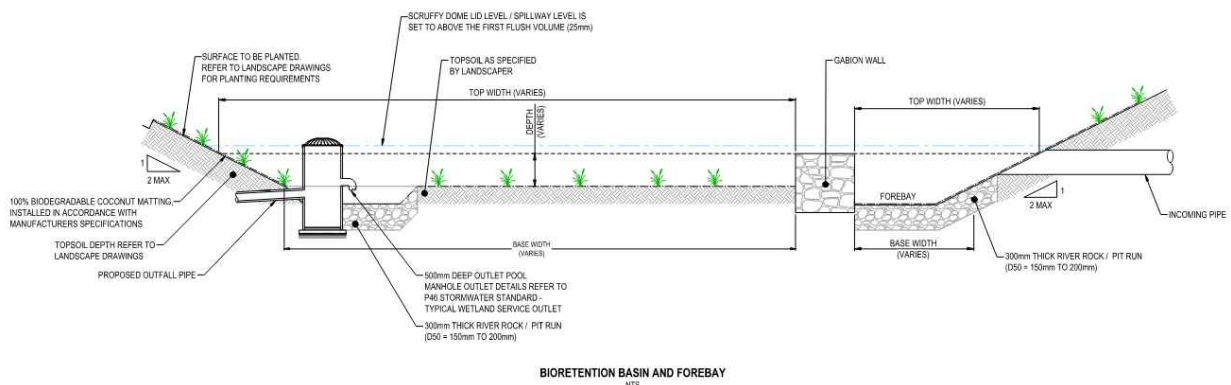


Figure 11 Typical detail of bioretention basin and forebay

Table 15 Bioretention basin summary

Basin Name	Catchment Area [m ²]	Minimum Treatment Volume Required [m ³]	Design Treatment Volume Provided (incl. forebay) [m ³]
Williams Street	21000	472	520
Waihora	10000	225	298
Pegasus South	7000	157	152*
Pegasus North	4000	90	231

*Minor adjustment to outlet level to achieve full treatment volume will be updated for IFC

3.4 Proposed Highway Stormwater Drainage Overview

3.4.1 Overview of External Catchments

The following major catchments are traversed by the Project, from South to North (refer to Figure 12). Each of the catchments is discussed in detail below.

- Kaiapoi River Catchment (CH 0 to 600)
- Cam River Catchment (CH 600 to 3600)
- Quarry Lakes Catchment (CH 3600 to 4600)
- McIntosh Drain (CH 4600 to 5800)
- Waihora Stream (CH 6100 to 9300)
- Taranaki Stream (CH 9300 to 9900)

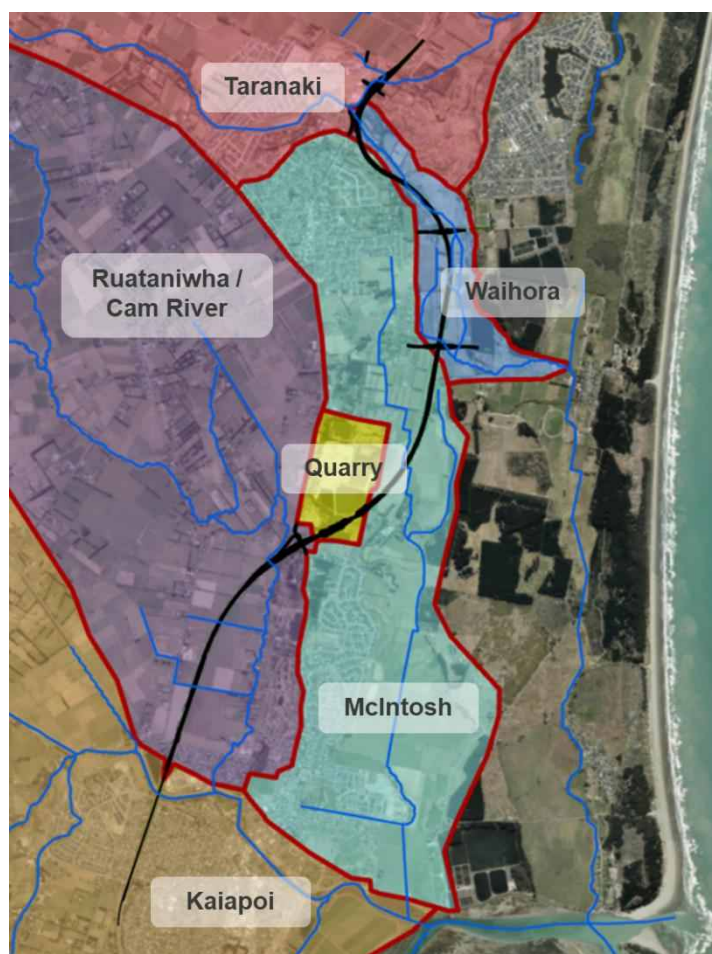


Figure 12 Overview of the external catchment

3.4.2 Kaiapoi River Catchment

South of the Kaiapoi River Bridge (CH -500 to 320)

The catchment area south of the Kaiapoi River Bridge extends from Ohoka Road to the Kaiapoi bridge, where a new southbound lane is proposed within the existing central median. Due to limited available land and the presence of steep embankments adjacent to private properties along this section, installing a swale on the roadside is impractical. An alternative proposed treatment method involves constructing kerbs and channels along the road to direct runoff into sumps and pipes equipped with proprietary treatment devices. However, this approach would require construction work within an active lane not included in the current project scope, leading to significantly higher construction and maintenance costs.

Given the limited space for a roadside treatment swale and the high costs associated with installing kerbs and channels, the design proposes a crossfall that directs runoff from the widened southbound lane into the central median which a grass lined treatment swale will be constructed to effectively manage the runoff. This swale will discharge treated stormwater into an existing Ø600 stormwater pipe at CH-380, which runs beneath the highway. The pipe conveys water to Parhams Drain on the highway's east side, from where it is pumped into the Kaiapoi River via the pump station. The invert of the existing Ø600 pipe to be surveyed and condition assessed by engineer to confirm the feasibility of connecting the swale prior to construction.

Kaiapoi River Bridge Area (CH 320 to 580)

At the Kaiapoi River Bridge, the proposed new lane is designed to slope toward the existing roadway, with runoff draining directly into the river and the adjacent rail corridor of Adderly Terrace via deck drains. Given the current bridge design, implementing water quality treatment at this site is not feasible, as retrofitting the structure to capture and redirect runoff for treatment would be impractical. A departure for deck drainage and water treatment (reference 11320-AUR-0350-PWI-DM-DEP-0010) at Kaiapoi River Bridge was conditionally approved by CEAG on 29/8/2025, subject to alignment with resource consent conditions.

Approximately 100 m north of the bridge, the new lane continues to slope toward the existing SH1 southbound lanes, allowing runoff to flow over the embankment into the adjacent existing drainage system and stormwater network alongside the Kaiapoi River stopbank. This drainage system features a broad swale that likely provides some level of treatment before the water is ultimately discharged into the Kaiapoi River.

3.4.3 Cam River Catchment

Lineside Road to Cam River (CH 0580 to 3190)

In this section, the proposed improvements expand to add an additional lane in each direction starting from CH 1400. Stormwater management here includes bioretention swales installed on both sides of the roadway, which discharge into the Cam River branch drains at CH 1610 and CH 2350. WDC requested regrading of the existing drain between CH 2350 and CH 3200 to improve the hydraulic performance of the drain from the area west of the Cam River Bridge toward the proposed Culvert 02 on Wilson Drain. The cross section of this drain is to match the existing drain and will be grass seeded to blend in with the adjacent paddock.

As indicated in Section 3.2.1, groundwater levels along this stretch are relatively high, and the drains may intercept groundwater during periods of elevated water tables. However, the new drains essentially replace existing channels and structures, maintaining similar heights and slopes. Therefore, while the drains might capture groundwater at times of high groundwater levels, they will not increase the overall volume of groundwater intercepted compared to the current drainage system.

Cam River Bridge and Williams Street Interchange (CH 3190 to 3620)

Between the Cam River Bridge and the Williams Street overpass, runoff from the mainline lanes and the southbound onramp will be collected via a stormwater network. This network will convey the stormwater to a

proposed bioretention basin located within the triangular area formed by the highway mainline, the northbound offramp, and Williams Street. The bioretention basin, sized to have a minimum volume of 520 m³, will treat the runoff before it discharges into the Cam River through a pipe proposed beneath the northbound offramp. For events which produce more runoff than the 25mm treatment volume, excess water will discharge through a Ø1050 scruffy dome outlet structure.

Runoff from the northbound offramp will be managed independently by a bioretention swale, prior to its release into the Cam River. To accommodate the new ramps, Williams Street will be realigned. South of its intersection with the northbound offramp, stormwater from Williams Street will be collected with a stormwater network draining into the bioretention basin. North of the offramp, runoff will be collected by a catchpit on the east side of the superelevated roadway, then directed beneath the roadway and discharged into the Cam River via a treatment swale.

3.4.4 Quarry Lakes Catchment

Quarry Lakes (CH 3620 to 4730)

North of the Williams Street overpass, the proposed roadway alignment enters the Quarry Lakes Catchment, where stormwater treatment will be managed by two bioinfiltration basins positioned on the north side of the roadway atop the quarry lakes hydraulic fill. This highly permeable fill material facilitates a direct connection to the lakes, enabling the effective implementation of a passive groundwater infiltration treatment system. The two basins are connected with a short swale, allowing the northernmost basin to overflow into the southern basin, both of which then overflow directly into the quarry lake via sheet flow over the bund which makes up the western side of the basins. The combined volume of the two ponds is sized to have a minimum volume of 940 m³. For the first 200 m beyond the Williams Street Interchange, runoff will be collected through a stormwater reticulation system that discharges into the southern side of the bioinfiltration basin. Throughout the rest of the catchment, runoff will either flow directly into the basin from the superelevated roadway or be conveyed via short swales before entering the treatment basins.

3.4.5 McIntosh Drain Catchment

McIntosh Drain (CH 4730 to 5800)

After passing through the Quarry Lakes, the alignment enters the McIntosh Drain Catchment, crossing McIntosh Drain at CH 5000. This catchment is predominantly undeveloped greenfield land primarily used for agriculture, with minimal existing stormwater infrastructure. Along this entire section, the roadway is superelevated to the west, necessitating stormwater management only on the west side. Runoff from the roadway will be collected in proposed planted bioretention swales ensuring effective treatment prior to discharge into McIntosh Drain.

A low-lying area near CH 4600 and CH 5320 is prone to ponding. To address this, cutoff drains are planned between CH 5320 and CH 5000, as well as between CH 4600 and CH 5000 to capture offsite flows and direct them for discharge into McIntosh Drain.

3.4.6 Waihora Stream Catchment

Beginning at CH 5800, the roadway enters the Waihora Stream Catchment, an area predominantly comprised of undeveloped greenfield land used for agriculture and featuring two significant local road crossings, Woodend Beach Road and Gladstone Road. According to Appendix C of the AEE Report, the lower reaches of Waihora Stream are characterised by broad, poorly defined channels with soils exhibiting very high infiltration rates. In contrast, the upper section consists of a spring fed perennial stream that infiltrates through high-porosity soil wetlands located downstream of the existing roadway crossing.

South of Woodend Beach Road (CH 5800 to 6400)

Starting at CH 5800, stormwater runoff from the roadway is directed northward toward Waihora Stream. In this section, due to the absence of superelevation, bioinfiltration swales are proposed on both sides of the road. These swales facilitate passive infiltration, treatment, and nominal attenuation of runoff. Excess water from the swales discharges into a cutoff drain equipped with a Ø600 culvert (Culvert 06) at CH 6300, located on the south side of the proposed Woodend Beach Road overpass. This drain flows eastward, releasing runoff into a tributary channel of Waihora Stream approximately 100 meters upstream of the main stream.

The project also includes a substantial realignment of Woodend Beach Road to enable its overpass of the proposed highway. Through this realignment, the road is elevated above the surrounding ground on both approaches as it ramps up and crosses the highway. Stormwater from the embankments is collected via stormwater reticulation pipes on both sides of the overpass. These pipes connect to bioinfiltration swales along the embankments. Runoff from the west side of Woodend Beach Road is directed into the proposed Waihora Stream diversion on the east side of Woodend Beach Road via the proposed twin Ø900 culvert (Culvert 07), eventually discharging into Waihora Stream itself via a proposed twin 2 m x 1 m box culvert system (Culvert 08), located about 200 meters east of the overpass.

Woodend Beach Road to Gladstone Road (CH 6400 to 7700)

North of Woodend Beach Road, runoff from the mainline up to CH 7100 (the crest point of the roadway) is conveyed through bioinfiltration swales on both side of the roadway. These swales discharge into Waihora Stream via the proposed twin Ø900 culvert (Culvert 07) situated on the north side of Woodend Beach Road.

At CH 7000, the roadway enters a superelevated section that slopes downward to the west, resulting in the discontinuation of the eastern bioinfiltration swale at this point. Upon reaching CH 7100, the roadway crest causes the western bioinfiltration swale to shift its flow direction northward. Runoff collected by this western swale between CH 7100 and CH 7770 (approximately 170 meters north of Gladstone Road) is directed into a proposed Ø600 culvert (Culvert 09) located beneath the mainline at CH 7400. This culvert connects to the proposed Waihora Stream diversion on the eastern side of the roadway, which eventually merges back into the existing Waihora Stream alignment at Culvert 08 on Woodend Beach Road.

At CH 7600, Gladstone Road is planned to cross over the highway via an elevated overpass. As the road ramps over the highway, stormwater runoff from the embankments on both sides is collected through proposed stormwater reticulation pipes. These pipes discharge into bioinfiltration swales located along the toes of the embankments on the northern side of Gladstone Road. On the western side, the bioinfiltration swale crosses beneath the highway through a Ø600 culvert (Culvert 11) before ultimately connecting to the downstream Waihora Stream diversion via twin Ø1050 culverts. On the eastern side, the swale directly links to the Waihora Stream diversion, which passes under Gladstone Road through a twin Ø1050 culvert (Culvert 10) situated approximately 25 meters east of the overpass.

Gladstone Road to Pegasus Interchange (CH 7700 to 9570)

Between Gladstone Road and CH 8400, the highway maintains a superelevated profile, sloping gently westward. In this stretch, a proposed bioinfiltration swale on the western side captures roadway runoff from a culvert located at CH 7980, which then discharges into the Waihora Stream diversion on the highway's eastern side. Just upstream of Gladstone Road, the realigned channel passes over a historic landfill site, designated as a HAIL area, extending approximately 100 m.

At CH 8400, the roadway superelevation shifts to slope eastward. As a result, between CH 8400 and CH 8850, runoff is collected in a bioinfiltration swale along the eastern side of the highway, which discharges into the upper reaches of the realigned Waihora Stream at CH 8270. From CH 8850 to the culvert at CH 9100, the eastward slope persists, with runoff conveyed northwards via another bioinfiltration swale into Waihora Stream.

Between CH 9100 and the Pegasus interchange at CH 9570, runoff from both the mainline and the southbound Pegasus on-ramp is directed into a stormwater reticulation network. This system channels the

flow to a proposed bioretention basin located northeast of Waihora Stream Culvert 14, a 4 m x 1.8 m box culvert, at CH 9250. Sized to hold a minimum volume of 298 m³, the basin treats the runoff before releasing it into Waihora Stream through an outlet structure equipped with an orifice drain. To accommodate runoff exceeding the 25 mm treatment volume, the outlet features a scruffy dome. Should the outlet become blocked, excess water is safely discharged via a riprap lined spillway into the downstream section of the existing Waihora Stream.

3.4.7 Taranaki Stream Catchment

Pegasus Interchange (CH 9400 to 9830)

South of the Pegasus Interchange, runoff from both the mainline of the proposed highway and the southbound on-ramp drains into Waihora Stream, as previously described. In contrast, the northbound off-ramp drains into Taranaki Stream. Runoff from this off-ramp is directed to a bioretention swale and stormwater pipe reticulation situated between the off-ramp and the mainline. This drainage system flows into a proposed bioretention basin located on the south side of Bob Robertson Drive (Pegasus Boulevard) between CH 9500 and CH 9550. Designed with a minimum capacity of 152 m³, this basin discharges west into Taranaki stream at chainage 9550 through an outlet structure fitted with an orifice drain.

There is a second bioretention basin on the north side of Bob Robertson Drive, which receives flow from Pegasus Boulevard beneath the overpass, as well as from the northbound on-ramp via a network of channels and stormwater reticulation pipes. Designed with a minimum capacity of 231 m³, this basin discharges west into Taranaki Stream at CH 9600 through an outlet structure fitted with an orifice drain. To manage runoff volumes exceeding the 25 mm treatment capacity, the outlets for both basins feature a scruffy dome.

The proposed southbound off-ramp closely follows the existing SH1 alignment, sloping eastward. Runoff from this section is collected by a series of short bioinfiltration swales, which discharge into Taranaki Stream at CH 9700 and into its tributary channel at CH 9830.

North of Pegasus Interchange (CH 9830 to 10300)

North of the Pegasus interchange, the highway transitions to the current SH1, tapering to align with the existing SH1 geometry. This section is free of superelevation, with stormwater discharged on both sides of the road. On the western side, a proposed bioretention swale extends from CH 9830 to CH 10100, collecting and treating runoff before discharging southwards into the Taranaki Branch Ø1200 culvert inlet (Culvert 17) at CH 9830.

On the eastern side, from the Culvert 17 (CH 9830) northwards to the project boundary, existing roadway runoff disperses across the golf course. The golf course effectively acts as a broad vegetated buffer, slowing and treating runoff before it drains into on-site ponds. To preserve this dispersed drainage pattern and avoid concentrating flows, no swales are proposed in this area. This approach is further justified by the minimal increase in impervious surfaces at this location.

North of CH 10100 on the western side, another proposed bioretention swale collects and treats roadway runoff, discharging into the existing SH1 roadside swales at the project boundary at CH 10300.

Garlick Street

The Project works include an extension of Garlick Street, adjacent to the Ravenswood subdivision, to tie into the existing SH1 alignment. This section of roadway runoff will be collected by proposed kerb and channel and piped reticulation, which will be discharged into the existing Ravenswood stormwater management system for treatment and attenuation.

3.4.8 Pipe Reticulation

Pipe reticulation is proposed in areas where barriers or kerb and channel are installed along the roadway to effectively convey and collect stormwater runoff. Reticulation systems are made up of various types of inlet devices and pipe networks which convey stormwater flows to discharge points.

The drainage inlet devices included are as follows:

- Catchpit manholes without sumps;
- Standard single and double catchpits;
- Scruffy dome manholes.

To consider contingency in the hydraulic capacity, blockage allowance of inlet grates is factored in by applying 20% reduction for inlets on grade and 50% for inlets located at vertical sag points. The spacing of catchpit manholes and catchpits is determined based on an assessment of the road's surface flow widths to optimise runoff collection and prevent encroachment of flows into the live lanes, in accordance with NZTA P46. All standard single and double catchpits include 450 mm sediment sump depth.

The pipe network consists of pipes and manholes, with minimum pipe diameters specified as Ø375 within the carriageway and Ø300 outside the carriageway. Stormwater pipe networks are designed to convey stormwater corresponding to a 10% AEP event without surcharging to within 500mm of the road surface level. Where secondary overland flow paths are unavailable, the stormwater pipe networks are designed to convey runoff up to 1% AEP event without surcharging onto the road surface. Target pipe cover for drainage pipes is generally 1.0 m in trafficked areas and 0.9 m in non-trafficked zones. However, where pipe cover is limited, reduced pipe cover of 0.6 m under the trafficked and non-trafficked areas with appropriate pipe structural checks has been applied. In constrained sites where achieving a minimum cover of 600mm is not feasible, suitable structural measures such as concrete surround or capping slabs have been implemented to maintain structural integrity and design life.

Typically, Class 4 reinforced concrete pipes are specified for both trafficked and non-trafficked areas. Where special conditions apply, pipe class selection has been guided by detailed structural analysis using PipeClass in accordance with AS/NZS3725.

All manholes will be circular precast units with a minimum internal diameter of 1050mm. Manhole sizing follows CPAA guidelines to ensure adequate maintenance access, with lids providing a clear opening of 600mm diameter. Manholes are strategically located at pipe junctions, changes in grade, and direction changes, with a maximum spacing of 90 m to facilitate inspection and maintenance.

3.4.9 Surface Drainage

Surface runoff at where crossfall reverses

Seven locations along the roadway have been identified where the crossfall reverses direction between lanes, increasing the risk of aquaplaning due to a combination of water accumulation and flat crossfall slopes. To ensure road safety, surface runoff depths at these sites were evaluated using the Water Flow Depth (WFD) RRL method in accordance with the NZTA TM-2502 standard.

At these locations, the road geometry has been adjusted to ensure sheet flow does not cross the median twice as super elevation transitions from one side to the other, keeping sheet flow depth less than 4mm during a 50% AEP 10 min rainfall event. The pavement is also to be surfaced with EPA7, which provides high surface permeability, further reducing surface runoff and providing an extra level of aquaplaning risk reduction.

Surface runoff where road is superelevated

Where the road is superelevated with a single consistent crossfall, fully preventing runoff from crossing the median is impractical without introducing barriers, kerb and channel, or extensive trench drains in the median. Such solutions would significantly increase construction costs and raise potential maintenance issues. Therefore, a controlled allowance for runoff crossing the median is proposed, coupled with appropriate mitigation measures to manage associated risks.

A departure (11320-AUR-0350-PWI-DM-DEP-0017) has been approved to permit runoff to cross the median. Similar WFD (RRL) assessments, following NZTA TM-2502 standards, confirm that runoff depths remain below the critical 4 mm threshold for both the 50% AEP 10-minute storm (29.4 mm/hr, based on RCP 6.0 projections through 2120), as shown in Table 16.

Table 16 Water flow depth calculation for surface runoff at where crossfall is superelevated

CHAINAGE	50% AEP (10min) = 29.4 mm/hr	
	WFD (RRL METHOD) (mm)	WFD (GALLAWAY METHOD) (mm)
Ch2775	2.0	0.6
Ch5235	2.0	0.6
Ch7395	2.0	0.6
Ch9210	2.3	0.9

3.5 Flood Assessment Modelling

3.5.1 Catchment Area Between Ohoka Road to Lineside Road

The road widening between Ohoka Road to Lineside Road is minimal, and the proposed lane crossfall directs runoff towards the swale within the median. Consequently, the risk of flooding due to the lane widening is assessed as low, and therefore, no flood modelling has been conducted in this catchment area.

3.5.2 Catchment Area Between Lineside Road and Cam River

An assessment of the potential flood impacts of the proposed road widening between Kaiapoi Bridge and the Cam River is completed using volumetric displacement assessment. The assessment is detailed in Appendix G Flood Impact Assessment South of Cam River. As the catchment is located within a hydraulically disconnected floodplain area, a "bathtub" area, the road widening will introduce additional embankment fill into the adjacent floodplain, thereby reducing the available flood storage volume within this confined area. Such a reduction in storage is expected to cause a slight increase in flood levels in the inundated zone adjacent to the road.

To quantify this impact, detailed terrain data from 1m-resolution LiDAR and bathymetry, along with flood extent information from the existing Waimakariri District Council flood model, were analysed. The volume difference between the current ground surface and the proposed road embankment was estimated to be approximately 15,820 m³.

By dividing this lost storage volume by the flooded surface area for various design flood events under the 100-year, 200-year, and 500-year ARI events, a conservative estimate of the resulting flood level rise was determined. The predicted increase in flood levels is minimal, ranging from about 3.2mm under road overtopping scenario (when flood level exceeds the minimum road crest level), to approximately 1.5 mm under the 200-year and 500-year ARI events.

While the storage reduction causes a slight elevation in flood levels within this confined floodplain, the overall impact is negligible and well within acceptable limits.

3.5.3 Catchment Area North of Cam River

For the catchment area north of Cam River, the impacts assessment was undertaken to evaluate the flood associated with the proposed Belfast to Pegasus Road development. A dedicated B2P Flood Model was created using TUFLOW modelling software with model catchment boundary **as shown in** to support this evaluation. The effect of the Project on flooding outcomes and flood modelling methodology is documented in the flood modelling report, which is included in Appendix C AEE Report's Appendix A.

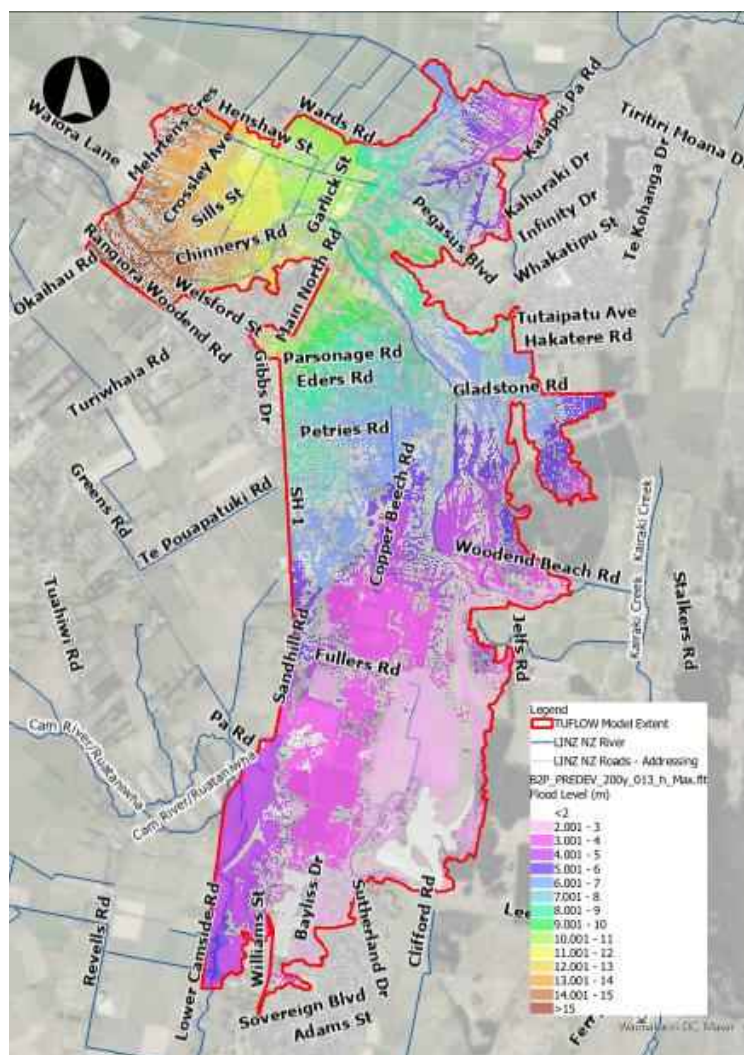


Figure 13 TUFLOW model extent boundary

The key objectives of the flood assessment were to ensure no significant increase in flood levels at neighbouring dwellings during the 100 year ARI event, to limit any flood level increase at neighbouring dwellings to no more than 20 mm during the 200-year ARI event, and to minimise the barrier effects caused by the new road on adjacent areas through mitigation strategies such as installing additional culverts, upgrading existing culverts, and providing extra channels to maintain or enhance runoff flow. Three hydrodynamic models were developed for this purpose: Pre-Development, Post-Development, and Mitigation models.

The Pre-Development model represents current site conditions, including existing culverts and overland flow paths, serving as the baseline against which changes are measured. This model was calibrated and validated against the Whangarei District Council (WDC) Flood Model, with minor differences attributed mainly to updated topographical data and areas not covered in the WDC model but included in the TUFLOW setup.

The Post-Development model incorporates the proposed road design without mitigation measures, highlighting increased flood levels and worsened impacts. and present the three impacted properties in terms of flood level and their difference between the post-development and pre-development cases at varying ARIs (100-year and 200-year). These impacts were then addressed in the Mitigation model by integrating new culverts and flow channels designed to preserve the original drainage patterns and storage capacities of the surrounding environment.

Table 17 100-year and 200-year ARI Flood Levels and Differences (Post-Development vs. Pre-Development Scenario)

Location	100-year ARI			200-year ARI		
	Pre-development flood level (m)	Post-development flood level (m)	Difference (mm)	Pre-development flood level (m)	Post-development flood level (m)	Difference (mm)
196 Woodend Beach Rd	4.13	4.33	+193	4.23	4.57	+335
1263 Main North Rd	8.28	8.31	+30	8.29	8.35	+58
1319 Main North Road	6.41	6.44	+27	6.46	6.55	+91

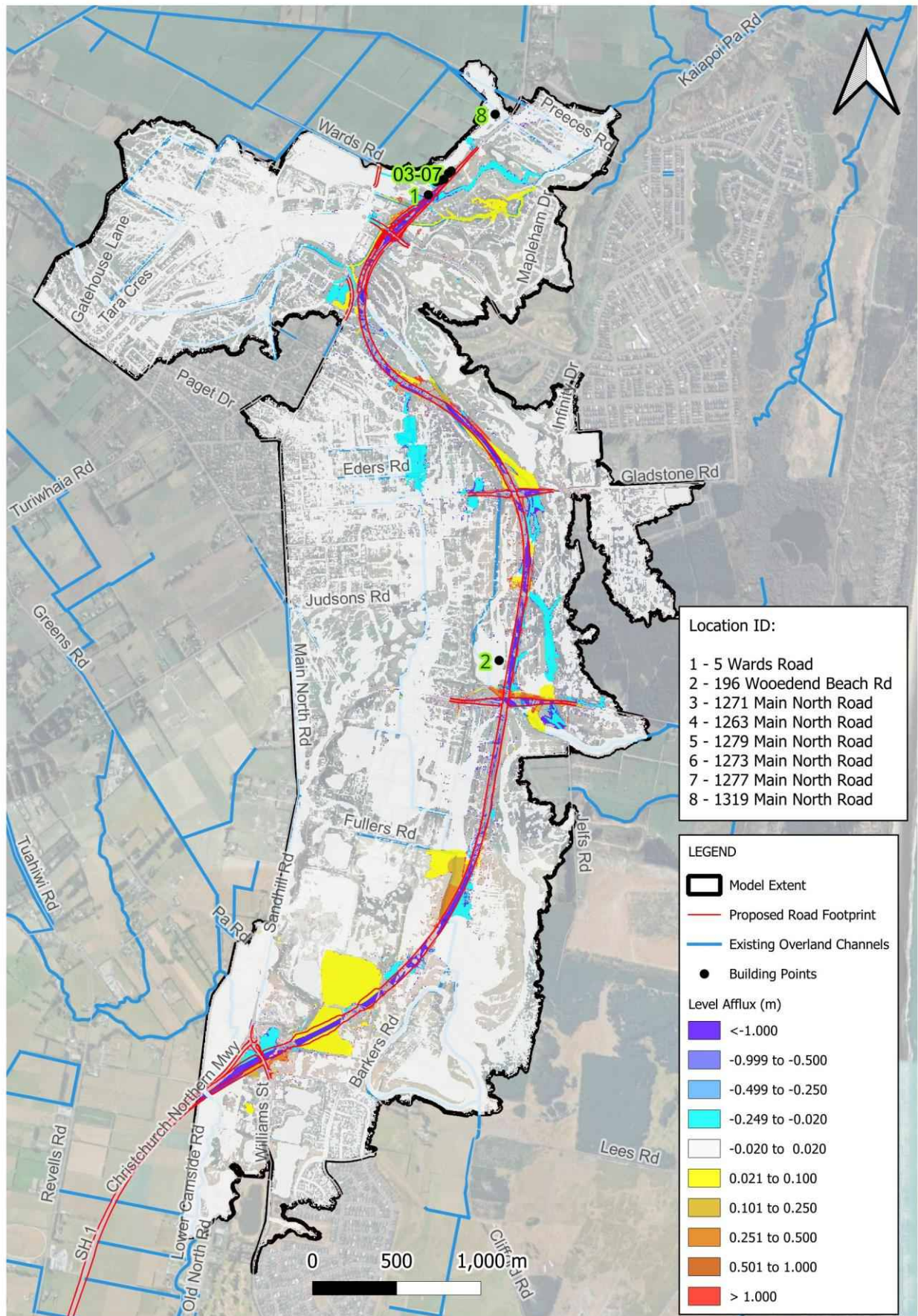


Figure 14 Private buildings affected under 200 year ARI (mitigated scenario)

To comply with regional policy and Council requirements, two principal mitigation measures were modelled, introducing additional culverts and diverting flow path channels. These proposed mitigations have proven effective at reducing flood impacts across most affected areas for both 100 year ARI and 200 year ARI design storm events. Importantly, three properties showing increased flooding in the Post-Development scenario were fully mitigated, with residual impacts limited to less than 20 mm as shown in Table 18.

Table 18 100-year and 200-year ARI flood level and differences (Mitigation vs Pre-development scenario)

Location	100-year ARI			200-year ARI		
	Pre-development flood level (m)	Post-development flood level (Mitigated) (m)	Difference (mm)	Pre-development flood level (m)	Post-development flood level (Mitigated) (m)	Difference (mm)
196 Woodend Beach Rd	4.13	4.14	+10	4.23	4.23	0
1263 Main North Rd	8.28	8.29	+10	8.29	8.29	0
1319 Main North Road	8.28	8.29	+10	8.29	8.29	0

There are still residual positive impacts at seven locations outside of the project's designation boundary as shown in and Figure 15, however, these do not affect any building structures, and mitigation is only planned if future assessments deem it necessary. Overall, the modelling confirms that with the proposed mitigation measures, the project can responsibly manage flood risks while meeting environmental and regulatory obligations. The current model indicates some impacts at the downstream of Woodend Beach Road, these impacts have been assessed as part of the refined modelling discussed above.

Table 19 Locations that observed a water level increase outside of designation boundaries

Locations	Afflux (mm)		Comments
	100-yr	200-yr	
1. Downstream Reach of Taranaki	+3	+10	Afflux maps show, at the downstream reach of the Taranaki Stream (after Mapleham Drive), minimal flood depth differences, both for the 100-year and 200-year storm events. It signifies that the additional flow, that was diverted from the original overland flow path along the golf course, was conserved and eventually discharged at the same location, maintaining both flow and flood levels
2. Mapleham Drive	+10	+70	Higher water levels were observed downstream along the Taranaki Stream, particularly near Mapleham Drive, similar to the pattern noted in item 1. However, these elevated levels remain confined within the stream channel and do not impact any nearby buildings. For example, the property at 61 Mapleham Drive, located midstream, has a floor level of approximately 6.0 meters. During the 200-year flood event, the pre-development flood level was 5.47 m, providing a freeboard of 530 mm; Under mitigation scenario flood level rises to 5.54 m, still maintaining a freeboard of 460 mm, ensuring that the property remains protected from flooding.
3. Taranaki Stream downstream of SH1	+70	+130	The increase in water depth is due to the upsizing of the Taranaki Stream culvert. However, the impacts remain within the channel and do not affect any buildings.

Locations	Afflux (mm)		Comments
	100-yr	200-yr	
4. Golf Course bounded by SH1 and Mapleham Drive	0	-110	The original overland flow path for the 200-year flood event, caused by the overtopping of SH1, no longer existed as the excess runoff is now conveyed into the Taranaki Stream via Culvert 16A. This change has resulted in a negative afflux within the golf course area bounded by SH1 and Mapleham Drive.
5. Taranaki Stream close to Harvey Norman (upstream of Bob Robertson Drive)	-200	+60	The section of the Taranaki Stream upstream of Bob Robertson Drive exhibits a negative afflux during the 100-year flood event, while a positive afflux is observed under the 200-year event. Flows during the 100-year event are conveyed efficiently due to the increased capacity of the upsized Taranaki culvert. However, during the 200-year flood, the positive afflux occurs because Culvert 16A reaches its maximum capacity, causing water to backflow all the way to upstream of Waihora Creek.
6. Upstream section of Taranaki (upstream of Garlick Street)	-90	-40	For both the 100-year and 200-year ARIs, decreased flood levels were observed in the upstream section of Taranaki Stream, primarily due to improved conveyance of the Taranaki culvert. Any increase in flood depth within the stream extends only as far as Garlick Street, beyond which additional runoff is effectively redirected southward through Waihora Creek, continuing downstream to Woodend Beach Road.
7. Garlick Street and Basin within Ravenswood residential area	+150	+70	The designed Garlick Street extension has a sag elevation of 10.4m RL, which is 0.15m higher than the existing sag level. The sag point acted as an overflow weir that connects the basins to Waihora Creek. Consequently, more water is detained in the basins with a higher weir level so that the basin storage can be better utilized. Less water is discharged to the Taranaki and Waihora streams and resulted to lower flood depths in the downstream areas. For 200 year events, basin flood level under post development is already as high as 10.41m RL, makes the raising of weir level less effective in storing water in the basins.
8. Downstream of Waihora Creek	+110	+130	Higher water levels at this location were primarily caused by the reduced channel storage due to the proposed SH1 cutting off Waihora Creek. The impacts remain contained within the channels and do not affect any buildings.
9. Floodplain North of Gladstone Road	-260	+40	During the 100-year flood event, excess runoff was efficiently conveyed through the Gladstone Road culvert (10A), resulting in a negative afflux. For the 200-year event, the design intentionally impounds more water within the existing floodplain north of Gladstone Road to attenuate flows and reduce downstream impacts, particularly near Woodend Beach Road. The impacts remain contained within the floodplain and do not affect any properties.
10. East of Copper Beach Road	+260	+180	Water ponding is due to the motorway cutting off an existing overland flow path. The ponding is localised and does not impact any buildings.

Locations	Afflux (mm)		Comments
	100-yr	200-yr	
11. Floodplain North of Woodend Beach Road	+500	+550	The Woodend Beach Road culvert (08A) was designed to impound and attenuate water before it discharges into Kairaki Creek, resulting in a positive afflux during both the 100-year and 200-year flood events. The adjacent floodplain effectively stores the excess water, ensuring that no buildings or properties are impacted.
12. Downstream of Woodend Beach Road	+130	0	Higher flood levels were observed downstream of Woodend Beach Road during the 100-year ARI event, primarily due to more efficient conveyance in the upstream sections, including the culverts at Gladstone and Woodend Beach Road. Despite this increase, no buildings were affected, and all impacts remained contained within the proposed diversion channel. For the 200-year event, the Woodend Beach Road culvert (08A) was specifically designed to limit discharge toward Kairaki Creek so that the flood level remains. The storage capacity of the floodplain north of Woodend Beach Road is well utilized.
13. Upstream of McIntosh Drain	+140	+320	The increase in flood levels is caused by the proposed SH1 cutting off the existing McIntosh Drain. The impact is largely confined to land owned by NZTA which does not include any residential properties.
14. Within the Quarry Lake.	0	+90	The increase in water levels was primarily caused by the loss of storage resulting from the infilling of the lake for the proposed bypass road. The water levels in the ponds generally represent existing groundwater levels, and the model assumes no infiltration. However, since the quarry bottom is directly connected to the groundwater, flood afflux is expected to be minimal due to the continuous exchange between surface water and groundwater

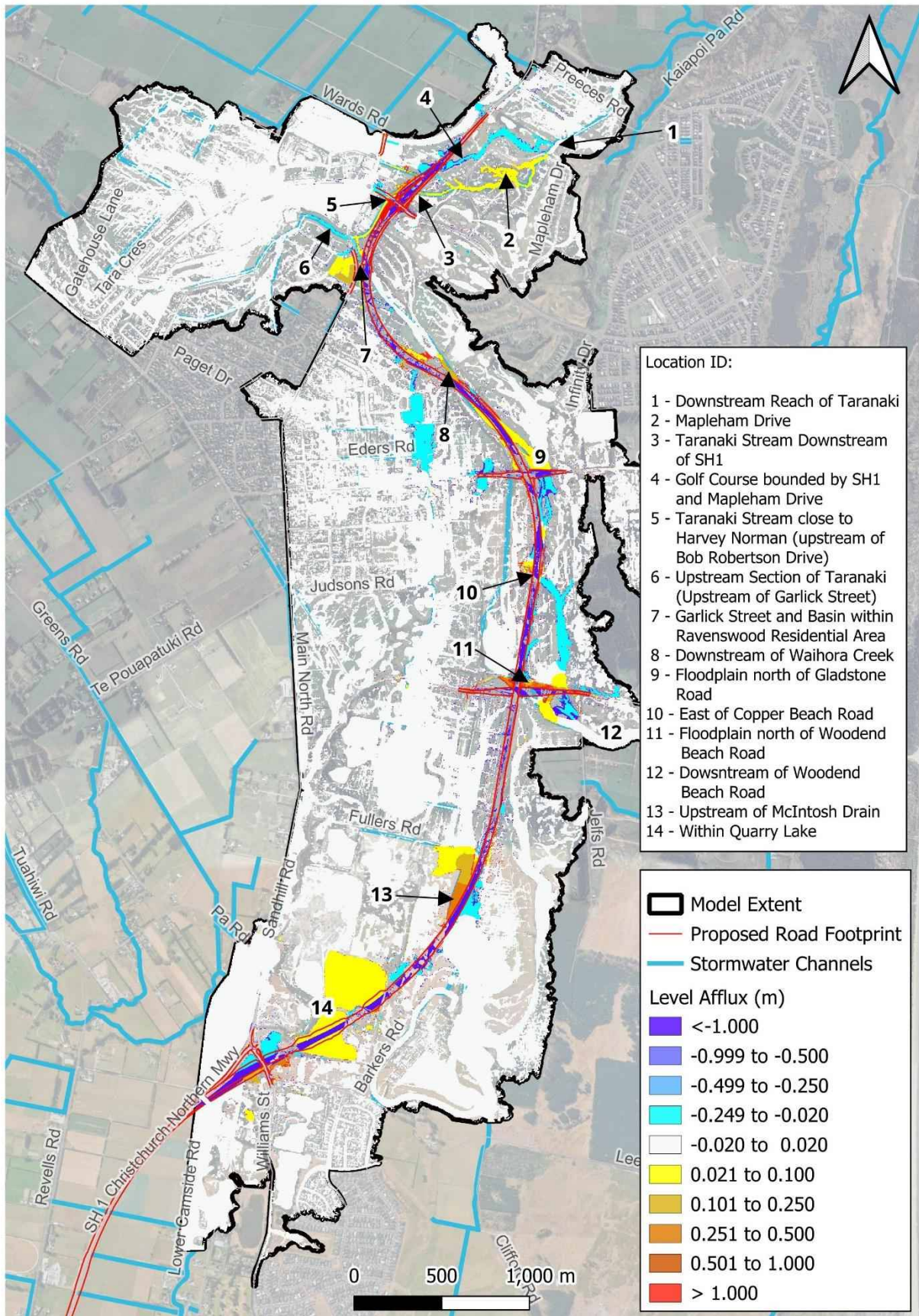


Figure 15 Residual impacts outside the designation but does not impact any property buildings (200 year ARI afflux level)

3.6 Proposed Bridge Improvements

3.6.1 Kaiapoi River Bridge

The Kaiapoi River Bridge is located within the portion of the Project with a single southbound lane widening to the inside. This structure is being maintained with its current geometry, however the structure is proposed to undergo structural strengthening to bring it into compliance with current engineering standards and to allow it to support the additional lane as part of the early works.

Hydraulic Analysis

To support the structural design of the bridge strengthening a hydraulic analysis was undertaken. This analysis is based on a HEC-RAS river model, incorporating detailed bridge design elements.

The hydraulic analysis indicates that the river is confined within the existing stopbanks for the SLS event, however the stopbanks overtop in the ULS event. Regardless, the ULS flows will not affect the bridge deck.

Scour Analysis

As part of this strengthening, a scour analysis was undertaken to determine the extent of scour that would be expected to occur in the Serviceability Limit State (SLS) event (100-year ARI) and the Ultimate Limit State (ULS) (1500-year ARI) events. The detailed assessment is documented in Appendix E Kaiapoi River Scour Assessment Memo. The preliminary scour outcomes are shown below.

Table 20 Scour depth (m) by bridge component in the worst case SLS and ULS event scenarios

Design event	Contraction	Pier 1 (Northmost)	Pier 2	Pier 3	Pier 4	Pier 5 (Southmost)	South Abutment	North Abutment
SLS	0.12	0	3.38	3.38	0	0	0	0
ULS	1.0	1.55	4.22	4.22	1.54	1.47	2.03	4.62

Although there is moderate expected pier scour in both the SLS and ULS events, the magnitude of this scour is within acceptable structural limits for both the events, based on the NZTA Bridge Manual criteria. Due to the stopbanks confining the flow in the SLS event, the bridge abutments are not predicted to scour in the SLS event, and scouring in the ULS event is not predicted to affect the bridge's stability.

Proposed Scour Protection

Based on this analysis, no additional scour protection is recommended for the Kaiapoi River Bridge.

3.6.2 Proposed Cam River Bridge

While the existing Cam River Bridge is being retained to support a northbound offramp, a new bridge is proposed to carry the proposed mainline and southbound onramp. This bridge will be located to the east of the existing bridge structures, which will share a common abutment fill slope.

Hydraulic Analysis

To support the structural design of the bridge a hydraulic analysis was undertaken. This analysis is based on a HEC-RAS river model, incorporating detailed bridge design elements. The detailed assessment is documented in Appendix F Cam River Scour Assessment Memo.

The hydraulic analysis indicates that regardless of tailwater conditions, high water levels in the Cam River are dominated by backwater effects from the Kaiapoi River impoundment area as discussed above. The

effect of this is that high flows from the Cam River do not result in the highest water levels at the Cam River Bridge, instead these levels occur when the Kaiapoi River system inundates the bridge. During these events, the flow and velocities through the Cam River Bridge become insignificant due to the 'bathtub' effect.

To understand the potential effects on the bridge, the model considered a Cam River only flood scenario, with low tailwater on the receiving Kaiapoi system. For both the SLS and ULS scenarios, these events resulted in lower water levels, but higher velocities.

Scour Analysis

The scour analysis for the Cam River Bridge was based on the Cam River only flood events, as the regional flood events created a 'bathtub' effect, reducing water velocities to near zero. The preliminary results of the scour modelling are summarised below.

Table 21 Scour depth (m) by bridge component in the worst case SLS and ULS event scenarios

Design event	Contraction	Pier (existing bridge)	South Abutment	North Abutment
SLS	2.52	4.39	1.89	4.32
ULS	3.10	4.74	1.89	5.92

Proposed Scour Protection

Protection against scour induced abutment failure has been designed in accordance with HEC-23, the US Federal Highway Administration's Bridge Scour and Instability Countermeasures Design Guide. Specifically, Design Guideline 14 was used in the preparation of preliminary abutment riprap revetments for the northern abutment, while Design Guideline 4 was used to remediate the bend scour at the southern abutment. Both proposed scour revetments will protect only the proposed bridge abutment and embankment. The revetment will extend upstream to the interface between the existing and proposed bridge embankments, stopping at the extent of earthworks for the new bridge. The southern abutment has a retaining wall on the downstream side due to property restrictions. The abutment revetment here will extend an extra 8m downstream and tie into the existing stopbank rather than wrapping around the abutment.

Table 22 Scour Protection Summary

Parameter Description	Design Sizing
Minimum D ₅₀ (m)	0.25m
Minimum Revetment Slope Layer Thickness (m)	0.5m
Minimum Toe Layer Thickness (m)	0.5m
Minimum Material Specific Gravity	2.65

4 Next Steps

As we progress to the IFT design stage, the focus will be on resolving the outstanding issues identified during the 100% design and advancing the overall design to IFC design. The IFC design will also incorporate any comments received from external parties on the 100% design stage.

Most changes will involve updates to tie in details, drawings, and pipe reticulation sizing.

Appendices



Appendix A

Definitions and abbreviations

Table 23 Definitions and abbreviations

Term	Definition	Term	Definition
AADT	Annual Average Daily Traffic	AASHTO	American Association of State Highway and Transportation Officials
AC20	Asphalt Concrete 20	ADT	Average Daily Traffic
AEP	Annual Exceedance Probability	AGBT	Austrroads Guide to Bridge Technology
AGPT	Austrroads Guide to Pavement Technology	AGRd	Austrroads Guide to Road Design
AP65	All Passing 65	AS/NZS	Australia/New Zealand Standard
ASD	Approach Sight Distance	B2P	Belfast to Pegasus
BM	Bridge Manual	BPO	Best Practical Option
CAT6	Category 6	CBR	California Bearing Ratio
CCC	Christchurch City Council	CCTV	Closed Circuit Television
CEAG	Chief Engineer's Advisory Group	CLB	Carriageway Loop Boxes
CLWRP	Canterbury Land and Water Regional Plan	CNC	Christchurch Northern Corridor
CNG	Cycling Network Guidance	ConOp	Concept of Operations
CoP	Engineering Code of Practice	CoPTTM	Code of Practice for Temporary Traffic Management
CPAA	Concrete Pipe Association of Australasia	CSU	Colorado State University
DPS	Design Philosophy Statement	Ecan	Environment Canterbury
EJT	Estimates Journey Time	ELV	Extra Low Voltage
EPA	Epoxy Porous Asphalt	FHWA	Federal Highway Administration
GIS	Geographic Information System	HCM	Highway Capacity Manual
HEC	Hydraulic Engineering Circular	HECRAS	Hydrologic Engineering Center's River Analysis System
HIRDS	High Intensity Rainfall Design System	HCV	Heavy Commercial Vehicle
ITS	Intelligent Transport Systems	KJB	Kerbside Junction Box
LCS	Lane and Carriageway Signs	LINZ	Land Information New Zealand
LoS	Level of Service	LPG	Liquefied Petroleum Gas
MCA	Multi Criteria Assessment	MESA	Millions of Equivalent Standard Axels
MiD	Maintenance in Design	MoE	Ministry of Environment
MOTSAM	Manual of Traffic Signs and Markings	MOU	Memorandum of Understanding
MPNZ	MainPower New Zealand	MR	Minimum Requirement
MSE	Mechanically Stabilized Earth	MTD	Manufactured Treatment Device
N/A	Not Applicable	NAASRA	National Association of Australian State Roading Authorities

NOT FOR CONSTRUCTION

Term	Definition	Term	Definition
NCHRP	National Cooperative Highway Research Program	NES	National Environmental Standards
NIWA	National Institute of Water and Atmospheric Research	NOC	Network Outcomes Contract
NUO	Network Utility Operator	NZS	New Zealand Standard
NZTA	New Zealand Transport Agency	NZTA TS	NZTA Stormwater Treatment Standard for State Highway Infrastructure
NZUAG	New Zealand Utilities Advisory Group	OD	Overdimension
ONF	One Network Framework	ONRC	One Network Road Classification
PE	Polyethylene	PSOPS	Project Scope and Outcomes Philosophy Statement
PTZ	Pan Tilt and Zoom	PWL	Permanent Water Level
PWV	Permanent Water Volume	RAP	Recycled Asphalt Pavement
RCC	Recycled Crushed Concrete	RCFJ	Reinforced Concrete Flush Jointed
RCP	Representative Concentration Pathway	RCRRJ	Reinforced Concrete Rubber Ring Jointed
RFI	Request for Information	RL	Reduced Level
RoNS	Roads of National Significance	RRL	Road Research Laboratory
RSB	Road Safety Barrier	RTS	Road and Traffic Guidelines
SCATS	Sydney Coordinated Adaptive Traffic Systems	SH1	State Highway 1
SiD	Safety in Design	SIL	Subgrade Improvement Layer
SISD	Safe Intersection Sight Distance	SLS	Serviceable Limit State
SMA	Stone Mastic Asphalt	SME	Subject Matter Expert
SSD	Stopping Sight Distance	SUP	Shared Use Path
TAN	Technical Advice Note	TBC	To be Confirmed
TCDM	Traffic Control Devices Manual	TfNSW	Transport for New South Wales
TLD	Traffic Loading Distribution	TM	Technical Memorandum
ToC	Time of Concentration	TSS	Total Suspended Solids
ULDF	Urban and Landscape Design Framework	ULDMP	Urban Design and Landscape Management Plan
ULS	Ultimate Limit State	UPS	Uninterruptible Power Supply
uPVC	unplasticized Polyvinyl Chloride	USDOT	United States Department of Transportation
VEMP	Visual Effects Management Plan	VMS	Variable Message Signs
WDC	Waimakariri District Council	WIM	Weigh-in-Motion
WTOC	Wellington Transport Operations Centre	WWDG	Waterways, Wetlands and Drainage Guide

Appendix B

Departures

- 11320-AUR-0350-PWI-DM-DEP-0015
Reduction in minimum pipe cover and use of RCP 8.5 for culvert design
- 11320-AUR-0350-PWI-DM-DEP-0016
Abutment scour assessment methodology
- 11320-AUR-0350-PWI-DM-DEP-0017
Median Drainage – Departure from NZTA P46 Section 6.1.k.

Committee Use Only

Date Received		Accepted for Committee	Yes / No
Departure Ref #			
Committee scheduled			
Notes			

REQUEST FOR TECHNICAL DEPARTURE

From:	Aurecon	Departure No:	DEP-0015
To:	Waka Kotahi	Date:	24/09/2025
Originator Name:	Yanni Hooi	Project	Belfast to Pegasus Motorway & Woodend Bypass Pre-implementation and MSQA Professional Services
Major Discipline: Other relevant disciplines:	Stormwater Design		
Subject	Reduction in minimum pipe cover		

NZTA Confirmation of Departure

NZTA Contract Rep		NZTA Technical Lead Advisor	
Request Type	Referral to Chief Engineer Advisory Group		
	Standard Departure from Op Policy		
	Urgent / Emergency Works		

1 Background

The drainage design for this project has been developed primarily in accordance with the NZTA P46 State Highway Stormwater Specification.

According to clauses 6.2.3 e) of NZTA P46, a minimum pipe cover of 1.0 metres is required beneath traffic lanes, and 0.9 metres is required in non-traffic landscaped areas. Site constraints such as topography and invert levels have, however, make it impractical to meet these cover requirements in all locations. During the 50% design stage, a review of pipe levels identified several instances where minimum cover specifications could not be fully achieved.

2 Standard or Works Requirement to be Departed from:

NZTA P46 6.2.3 e) states the following:

“The minimum pipe cover under traffic lanes is 1.0 metre and the minimum pipe cover in non-trafficable landscape areas (covered by topsoil and grass seed/vegetation) is 0.9 metres. Where these cannot be achieved, a reduced cover of no less than 0.6 metres may be approved by the Principal/Client where it can

be demonstrated with specific design that adequate structural capacity during construction and throughout the whole of life is achieved.”

3 Reason for Requested Departure (including what is requested and why, and what other options were considered)

3.1 What is requested

We request Waka Kotahi’s acceptance of the proposed design approach, by allowing a departure from the minimum pipe cover requirements, as outlined below.

Due to topographical constraints and restrictions on upstream and downstream invert levels, it is not feasible to achieve the minimum pipe cover specified by NZTA P46 at all locations across the project. Consequently, the structural capacity of the pipes has been assessed in accordance with AS/NZS 3725. This assessment confirms that the proposed rubber ring jointed reinforced concrete pipes (Class 4) in HS bedding condition are structurally adequate for HN-HO loading when a minimum pipe cover of 600mm is provided.

At the 50% design milestone, major and significant culverts requiring reduced cover are summarised in Table 1 below.

Table 1: Proposed major and significant culverts with cover less than 1.0m.

Pipe ID	Type/ Size	Approx. cover over pipe (m)
Culvert 1 (Rossiter drain)	DN1500 RCRRJ	0.70
Culvert 2 (Wilson drain)	DN1500 RCRRJ	0.88
Culvert 7 (Waihora Stream)	Twin DN900 RCRRJ	0.60
Culvert 15 (Taranaki stream)	Twin DN1800 RCRRJ	0.82
Culvert 16 (Taranaki stream)	Box 2.7m X 1.5m (WxH)	0.82
Culvert 17 (Taranaki stream north)	DN1050 RCRRJ	0.75
All stormwater pipe reticulation within the project	DN300 and DN375	0.70

Where the cover falls below 600mm due to site constraints, additional structural support in the form of a concrete capping beam is proposed. At the 50% design stage, no culvert is identified with cover less than 600mm.

For all concrete box culverts, specific structural design will be undertaken to ensure that adequate structural capacity is achieved during construction and throughout the whole of life of the asset.

3.2 Options Considered

The following are the scenarios where P46 requirements are challenging to achieve and options that have been considered:

Scenario	Options considered
Pipes or culverts are tied into existing downstream channels	NZTA P46 cover is unable to be achieved due to the need to tie into existing downstream channel invert levels. Achieving the required cover would involve deepening and flattening the downstream channel, which may adversely affect the ecological environment and require works beyond the extent of the designation. Lowering pipe below receiving channel level will

Scenario	Options considered
	increase risk of sedimentation build up and associated maintenance requirements hence not adopted.
Pipe or culverts are tied into proposed bioretention basin	NZTA P46 cover is unable to be achieved due to the depth limitations of the proposed bioretention basin. Further deepening the basin would require a significantly deeper retaining wall between the basin and the adjacent ramp and may intrude on groundwater, which presents constructability ,cost and consenting challenges.
Pipe or culverts are tied into proposed swales	NZTA P46 cover is unable to be achieved due to the proposed depth of treatment swales along the bypass road. These swales are designed within the road embankment, and further deepening would necessitate widening the embankment downstream and may intercept groundwater, which is considered uneconomical.
Pipe or culverts are tied into proposed diversion channels	NZTA P46 cover is unable to be achieved due to the elevation of the diversion channel required to convey flow from a low-lying area west of the alignment to the east. Lowering the channel to meet cover requirements would prevent tie-in to the existing downstream overland flowpath, resulting in a partially submerged channel with reduced hydraulic efficiency. Additionally, as the groundwater level are shallow in the areas, deepening the diversion channel would pose challenges during the construction phase.
Proposed catchpit lead connections	In order to utilise standard catchpit units without the need for risers / extensions, a 1.8m deep unit with 0.45m sump cannot achieve 1.0m cover at the point at which the lead leaves the catchpit. For typical catchpit leads of 0.3m / 0.375m, a cover at the pit of less than 1m is generally achieved.
Proposed pipes connect into existing stormwater pipes	There are pipes across the project which currently provide less than 1.0m cover. In order to have the proposed pipes drain via gravity, the proposed pipe covers have to be reduced to tie into the existing stormwater pipes

For pipes that have cover exceeding 600mm but less than 1.0m, a structural pipe check has been carried out using PipeClass v 2.0.23 software. A PipeClass check using cover of 600mm for a range of circular pipe sizes (including DN1050, 1500 & 1800) using H2 trench support (conservative), wet clay natural ground (conservative), HN-HO loading, skewed alignment (conservative) and internal water load (conservative), suggest Class 3 concrete pipes have sufficient capacity.

To provide extra reserve capacity and assurance, Class 4 is proposed for all locations of circular pipe that cover less than 1.0m, but more than 600mm cover. HS2 bedding support is conservatively proposed for all culverts beneath traffic lanes.

Although there are no current locations (in the 50% design stage) with less than 600mm cover, if such locations are identified in the future design phases, a concrete cap will be proposed in the design to achieve additional structural resilience.

For all concrete box culverts, specific structural design will be undertaken to ensure that adequate structural capacity is achieved during construction and throughout the whole of life of the asset.

3.3 Rationale for Request

The drainage design for this project has been developed principally following NZTA P46 State Highway Stormwater Specification (2025). However, site-specific constraints such as topography and channel invert levels have made it impractical to consistently achieve the specified minimum pipe cover requirements of 1.0 m under traffic lanes and 0.9 m in non-traffic areas, as stipulated in clause 6.2.3(e) of NZTA P46.

All pipes with reduced cover have been structurally assessed in accordance with AS/NZS 3725, confirming that rubber ring jointed reinforced concrete pipes (Class 4) with 600 mm minimum cover and HS bedding meet the required loading conditions (HN-HO). Areas with cover below 600 mm will be structurally

supported with concrete capping beams. Concrete box culverts will receive specific structural designs to ensure durability and safety through the asset's lifespan.

In summary, we request Waka Kotahi's acceptance of the proposed departure from NZTA P46 minimum pipe cover, based on the detailed technical assessments and justified constraints outlined above.

4 Benefits of the Departure

Safety:	Reduced earthworks and depth of cut due to reduced pipe cover
Environmental:	- Reduced project footprint due to reduced excavation for pipe trenching by having reduced pipe cover - Reduced carbon emissions as a result of not unnecessarily raising roadway - Reduced risk of intercepting groundwater
Performance (level of service and resilience, customer levels of service):	No impact expected
NZTA Reputation:	Improved reputation through more optimal, cost-effective design
Cost:	- Reduced cost due to reduced earthworks for pipe installation - Reduced cost by using precast concrete pipes where possible
Maintenance:	Easier maintenance due to shallower assets

5 Costs and Dis-benefits of Departure

Safety:	N/A
Environmental	N/A
Performance (level of service and resilience)	No change to level of service as demonstrated by pipe strength checks
NZTA Reputation	N/A
Cost	Capital cost and renewal costs associated with pipes requiring concrete capping (only where cover is less than 0.6m)
Maintenance	No maintenance dis-benefits as discussed with ASM

6 Residual Risks Associated with the Departure

There is a low risk of damage to pipes if they are subjected to higher loads due to reduced cover. To mitigate this risk, we have proposed concrete capping for pipes that are subject to traffic loading and have cover less than 600mm. At the 50% design stage, no culvert is identified with cover less than 600mm.

Providing Class 4 concrete pipes across the project further minimises the risk of damage in low cover situations as structural assessments suggest these will have sufficient reserve capacity. HS2 pipe bedding condition will be specified as a standard for pipes under traffic lanes.

Reduced cover pipes with appropriate specification of strength class and concrete protection details have been successfully implemented on a number of recent Waka Kotahi projects including (but not limited to) State Highway 75, Te Ahu a Turanga (Manawatū Tararua Highway), SH16 St Lukes, SH20B Short Term Improvements and SH16 Lincoln to Westgate with no condition issues reported to date.

7 Impacted parties

There are no negatively impacted parties.

8 Value for Money Summary

Strictly adhering to cover requirements in situations where reduced cover is justified, such as non-trafficked locations and low-risk areas—results in an unnecessarily high level of durability.

The preferred cover depths are 1.0 m in trafficable areas and 0.9 m in non-trafficable areas. However, due to the flat catchments and fixed downstream discharge levels, achieving these preferred depths is often not feasible. In most cases where pipe cover is less than preferred but exceeds 600 mm, concrete capping is not required, which leads to cost savings.

There may be locations with cover less than 600 mm which will require concrete capping. Such pipes will be identified in later design phases and concrete capping will be designed accordingly.

By optimising cover requirements, we can reduce construction time and costs, minimise environmental impact through less earthworks, and maintain an acceptable level of risk.

Overall, the proposed approach offers a cost-effective solution without reducing service levels or design life.

9 Attachments

- Summary of PipeClass assessment for pipes with reduced cover.
[PipeClass Attachment for DEP-0015.pdf](#)

10 Signoffs

Prepared by: Yanni Hooi	Signed: 	Date: 17/09/2025
Reviewed By: David Delagarza	Signed: 	Date: 17/09/2025
Regan Smith		
Recommended By: Matt Flannery	Signed: 	Date: 24 September 2025

Recommendations / Comments

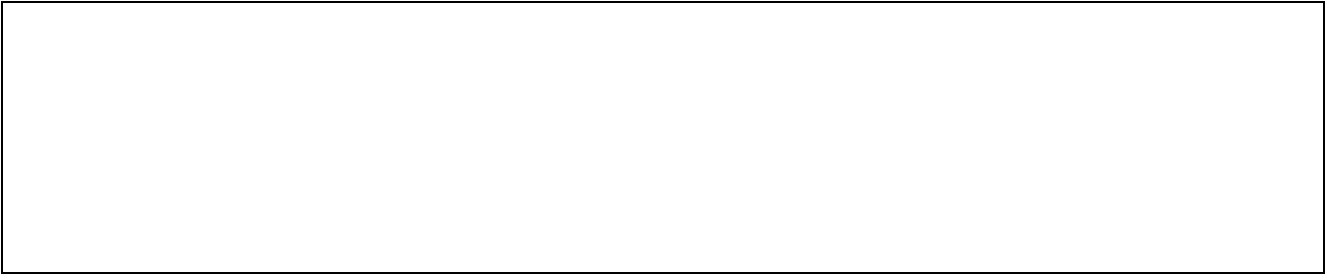
NZTA Contract Representative:

--

Principal's Agent Specialist Advisor:

--

Waka Kotahi Technical Specialist:



New Zealand Transport Agency Technical Decision:

Outcome:

Comments:

By:

Date:

New Zealand Transport Agency Authorisation to Proceed:

By:

Date:

Appendix A – Pipe Class Assessment



CPAA PipeClass Pipe Load Summary Sheet

Page 1 of 1

DESIGN OF 1500 DIA. RRJ DRAINAGE PIPE

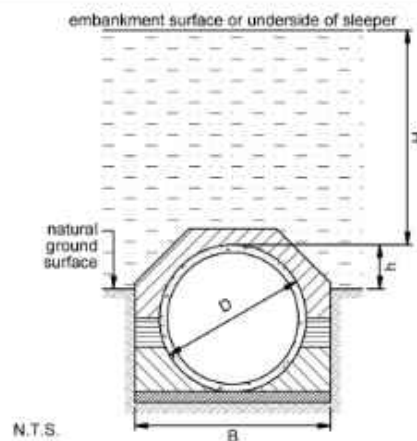
Client And Project Details

Date: 20-Aug-2025

Job number: 11320	Design: Culvert 1,2 - MIN cover
Client:	Designer: JYU
Project: B2P	Company:
Description: Significant culvert pipe strength checks	File: Pipe strength 2025.08.19.ppr

Design Parameters

Installation Condition:	embankment
Projection Type:	positive
Pipe Nominal Diameter (mm):	1500
Pipe External Diameter, D (mm):	1714
Pipeline Orientation:	skew
Soil Type:	wet clay
Soil Density (kN/m³):	20
Soil Parameter K _μ :	0.1100
Width, B (m):	2.285
Height Of Fill, H (m):	0.600
Flexible Pavement Type:	asphalt
Flexible Pavement Density (kN/m³):	21.000
Flexible Pavement Thickness (m):	0.300
Effective Height Of Fill, H _e (m):	0.915
Projection Height, h (m):	0.857
Projection Ratio, p (h/D):	0.500
Support Type:	HS2
Bedding Factor:	2.5



In Service Load Cases/Combinations Considered (controlling load case/combination highlighted)

Load Description*	Fill Height (m)	Wg/2.5	Wq/1.5	Ww/2.5	T _c	Pipe Class
earth + weight of internal water	0.915	13.3		5.6	19.0	2
HN-85%	0.915	13.3	15.5	5.6	34.5	2
HN	0.915	13.3	17.8	5.6	36.8	2
HO	0.915	13.3	30.5	5.6	49.4	2

All loads in kN/m. *Includes earth load at fill height shown.

Controlling Loads: earth + HO standard vehicle + weight of internal water

Minimum Test Load: T_c = 13.3 + 30.5 + 5.6 = 49.4 kN/m

Adopt 1500 dia. Class 2 RRJ pipe (1500/2 RRJ) in accordance with AS/NZS 4058:2007.

Design Notes:

1. A nominal pipe wall thickness of 83 mm has been assumed.

INSTALLATION OF 1500 DIA. CLASS 2 RRJ DRAINAGE PIPE

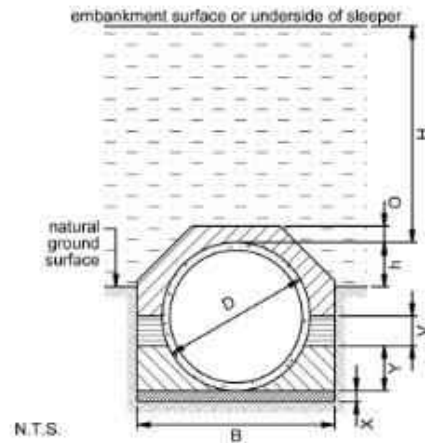
Client And Project Details

Date: 20-Aug-2025

Job number: 11320	Design: Culvert 1.2 - MIN cover
Client:	Designer: JYU
Project: B2P	Company:
Description: Significant culvert pipe strength checks	File: Pipe strength 2025.08.19.ppr

Design Parameters

Installation Condition:	embankment
Projection Type:	positive
Pipe Nominal Diameter (mm):	1500
Pipe External Diameter, D (mm):	1714
Pipeline Orientation:	skew
Width, B (m):	2.285
Height Of Fill, H (m):	0.600
Flexible Pavement Type:	asphalt
Flexible Pavement Density (kN/m3):	21.000
Flexible Pavement Thickness (m):	0.300
Effective Height Of Fill, He (m):	0.915
Projection Height, h (m):	0.857
Projection Ratio, p (h/D):	0.500
Support Type:	HS2
Excavation Volume (solid) (m³/m):	2.4



Installation Quantities

Support Zone	Depth (mm)	Quantities(m³/m)		Minimum Zone Compaction (%)	
		Solid	Loose	Density Index (for cohesionless soils)	Relative Density (standard compaction)
Bed zone	X = 150	0.343	0.412	60	-
Haunch zone	Y = 515	0.594	0.713	60	-
Side zone	V = 345	0.213	0.256	60	90
Overlay zone	O = 150	0.620	0.000	as per project specification	as per project specification

Material Grading Requirements

Sieve Size (mm)	75.0	19.0	9.5	2.36	0.60	0.30	0.15	0.075
Bed & Haunch Zones (% mass passing)	-	100	-	100-50	90-20	60-10	25-0	10-0
Side Zone (% mass passing)	100	-	100-50	100-30	50-15	-	-	20-0

Construction Equipment Requirements

No construction vehicles considered.

Design Notes:

1. All bed, haunch and side zone material passing the 0.075 mm sieve to have low plasticity (AS 1726).
2. Ordinary fill material to have no stones > 150 mm dia., and no more than 20% to be 75-150 mm.
3. For additional information refer to the project specification.
4. A nominal pipe wall thickness of 83 mm has been assumed.

DESIGN OF 1500 DIA. CLASS 2 RRJ DRAINAGE PIPE

Client And Project Details

Date: 20-Aug-2025

Job number: 11320	Design: Culvert 1,2 - MIN cover
Client:	Designer: JYU
Project: B2P	Company:
Description: Significant culvert pipe strength checks	File: Pipe strength 2025.08.19.ppr

In Service Load Cases/Combinations Considered (controlling load case/combination highlighted)

Load Description*	Fill Height (m)	Wg/2.5	Wq/1.5	Ww/2.5	Tc	Pipe Class
earth + weight of internal water	0.915	13.3		5.6	19.0	2
HN-85%	0.915	13.3	15.5	5.6	34.5	2
HN	0.915	13.3	17.8	5.6	36.8	2
HO	0.915	13.3	30.5	5.6	49.4	2

All loads in kN/m. *Includes earth load at fill height shown

weight of internal water

Working load due to water load, $Ww = 14.1 \text{ kN/m}$

earth

Height of fill, $H = 0.600 \text{ m}$

Pipeline orientation is skew

Flexible pavement type is asphalt.

Flexible pavement density = 21 kN/m^3

Flexible pavement thickness = 0.300 m

Effective height of fill, $He = 0.915 \text{ m}$

Embankment Condition, positive projection

Settlement ratio, $rs = 1.000$

Projection ratio, $p = 0.500$

Equal plane of settlement height, $He = 0.915$

Spangler coefficient, $C'e = 1.061$

Working load due to earth fill, $Wg = 33.3 \text{ kN/m}$

DESIGN OF 1500 DIA. CLASS 2 RRJ DRAINAGE PIPE**HN-85%****Axle Load Calculation**

Footprint width at top of pipe, $L1 = 3.626$ m
Footprint length at top of pipe, $L2 = 1.526$ m
Footprint area, $A = 5.533$ m²
Load on footprint = 102.0 kN

Impact factor = 1.03

Live load pressure at top of pipe, $q = 18.904$ kPa

Minimum of $L2$ and D , $S = 1.526$ m

Effective supporting length of pipe, $Le = 5.490$ m

Working load due to axle loads, Wq (axle) = 19.1 kN/m

Lane Load Calculation

Lane load design pressure (at surface) = 3.500 kPa
Lane width = 3.000 m

Impact factor = 1.03

Lane load pressure at top of pipe = 2.489 kPa

Working load due to lane load, Wq (lane) = 4.3 kN/m

Total working load due to live load, $Wq = 23.3$ kN/m

HN**Axle Load Calculation**

Footprint width at top of pipe, $L1 = 3.626$ m
Footprint length at top of pipe, $L2 = 1.526$ m
Footprint area, $A = 5.533$ m²
Load on footprint = 120.0 kN

Impact factor = 1.03

Live load pressure at top of pipe, $q = 22.240$ kPa

Minimum of $L2$ and D , $S = 1.526$ m

Effective supporting length of pipe, $Le = 5.490$ m

Working load due to axle loads, Wq (axle) = 22.4 kN/m

Lane Load Calculation

Lane load design pressure (at surface) = 3.500 kPa
Lane width = 3.000 m

Impact factor = 1.03

Lane load pressure at top of pipe = 2.489 kPa

Working load due to lane load, Wq (lane) = 4.3 kN/m

Total working load due to live load, $Wq = 26.7$ kN/m

DESIGN OF 1500 DIA. CLASS 2 RRJ DRAINAGE PIPE**HO****Axle Load Calculation**Footprint width at top of pipe, $L1 = 4.326$ mFootprint length at top of pipe, $L2 = 1.526$ mFootprint area, $A = 6.601$ m²

Load on footprint = 240.0 kN

Impact factor = 1.03

Live load pressure at top of pipe, $q = 37.283$ kPaMinimum of $L2$ and D , $S = 1.526$ mEffective supporting length of pipe, $Le = 6.190$ mWorking load due to axle loads, Wq (axle) = 39.8 kN/m**Lane Load Calculation**

Lane load design pressure (at surface) = 3,500 kPa

Lane width = 3.000 m

Impact factor = 1.03

Lane load pressure at top of pipe = 2.489 kPa

Working load due to lane load, Wq (lane) = 4.3 kN/m*Total working load due to live load, $Wq = 45.7$ kN/m*

DESIGN OF 1800 DIA. RRJ DRAINAGE PIPE

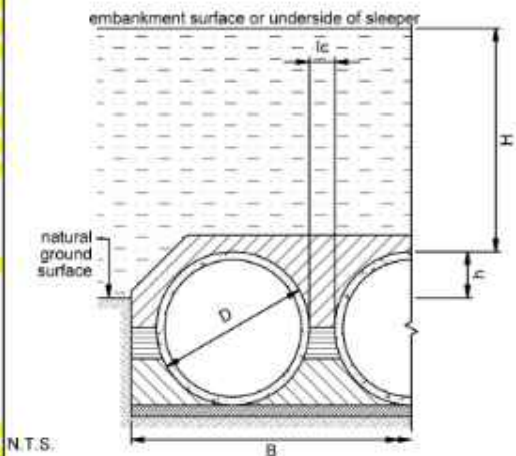
Client And Project Details

Date: 20-Aug-2025

Job number: 11320	Design: Culvert 15 - MIN cover
Client:	Designer: JYU
Project: B2P	Company:
Description: Significant culvert pipe strength checks	File: Pipe strength 2025.08.19.ppr

Design Parameters

Installation Condition:	embankment
Projection Type:	positive
Pipe Nominal Diameter (mm):	1800
Pipe External Diameter, D (mm):	2032
Number Of Barrels:	2
Barrel Spacing, Ic (mm):	1200
Pipeline Orientation:	skew
Soil Type:	wet clay
Soil Density (kN/m ³):	20
Soil Parameter K _μ :	0.1100
Width, B (m):	5.941
Height Of Fill, H (m):	0.600
Flexible Pavement Type:	asphalt
Flexible Pavement Density (kN/m ³):	21.000
Flexible Pavement Thickness (m):	0.300
Effective Height Of Fill, H _e (m):	0.915
Projection Height, h (m):	1.016
Projection Ratio, p (h/D):	0.500
Support Type:	HS2
Bedding Factor:	2.5



In Service Load Cases/Combinations Considered (controlling load case/combination highlighted)

Load Description*	Fill Height (m)	Wg/2.5	Wq/1.5	Ww/2.5	Tc	Pipe Class
earth + weight of internal water	0.915	15.6		8.0	23.6	2
HN-85%	0.915	15.6	15.3	8.0	39.0	2
HN	0.915	15.6	17.4	8.0	41.0	2
HO	0.915	15.6	29.5	8.0	53.2	2

All loads in kN/m. *Includes earth load at fill height shown.

Controlling Loads: earth + HO standard vehicle + weight of internal water

Minimum Test Load: $T_c = 15.6 + 29.5 + 8.0 = 53.2$ kN/m

Adopt 1800 dia. Class 2 RRJ pipe (1800/2 RRJ) in accordance with AS/NZS 4058:2007.

Design Notes:

1. A nominal pipe wall thickness of 95 mm has been assumed.

INSTALLATION OF 1800 DIA. CLASS 2 RRJ DRAINAGE PIPE

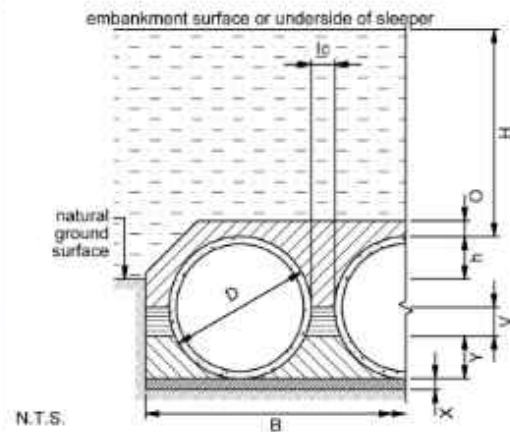
Client And Project Details

Date: 20-Aug-2025

Job number: 11320	Design: Culvert 15 - MIN cover
Client:	Designer: JYU
Project: B2P	Company:
Description: Significant culvert pipe strength checks	File: Pipe strength 2025.08.19.ppr

Design Parameters

Installation Condition:	embankment
Projection Type:	positive
Pipe Nominal Diameter (mm):	1800
Pipe External Diameter, D (mm):	2032
Number Of Barrels:	2
Barrel Spacing, I_c (mm):	1200
Pipeline Orientation:	skew
Width, B (m):	5.941
Height Of Fill, H (m):	0.600
Flexible Pavement Type:	asphalt
Flexible Pavement Density (kN/m ³):	21.000
Flexible Pavement Thickness (m):	0.300
Effective Height Of Fill, H_e (m):	0.915
Projection Height, h (m):	1.016
Projection Ratio, p (h/D):	0.500
Support Type:	HS2
Excavation Volume (solid) (m ³ /m):	7.0



Installation Quantities

Support Zone	Depth (mm)	Quantities(m ³ /m)		Minimum Zone Compaction (%)	
		Solid	Loose	Density Index (for cohesionless soils)	Relative Density (standard compaction)
Bed zone	X = 150	0.892	1.070	60	-
Haunch zone	Y = 610	1.987	2.384	60	-
Side zone	V = 410	0.815	0.978	60	90
Overlay zone	O = 150	0.777	0.000	as per project specification	as per project specification

Material Grading Requirements

Sieve Size (mm)	75.0	19.0	9.5	2.36	0.60	0.30	0.15	0.075
Bed & Haunch Zones (% mass passing)	-	100	-	100-50	90-20	60-10	25-0	10-0
Side Zone (% mass passing)	100	-	100-50	100-30	50-15	-	-	20-0

Construction Equipment Requirements

No construction vehicles considered.

Design Notes:

1. All bed, haunch and side zone material passing the 0.075 mm sieve to have low plasticity (AS 1726).
2. Ordinary fill material to have no stones > 150 mm dia., and no more than 20% to be 75-150 mm.
3. For additional information refer to the project specification.
4. A nominal pipe wall thickness of 95 mm has been assumed.

DESIGN OF 1800 DIA. CLASS 2 RRJ DRAINAGE PIPE

Client And Project Details

Date: 20-Aug-2025

Job number: 11320	Design: Culvert 15 - MIN cover
Client:	Designer: JYU
Project: B2P	Company:
Description: Significant culvert pipe strength checks	File: Pipe strength 2025.08.19.ppr

In Service Load Cases/Combinations Considered (controlling load case/combination highlighted)

Load Description*	Fill Height (m)	Wg/2.5	Wq/1.5	Ww/2.5	Tc	Pipe Class
earth + weight of internal water	0.915	15.6		8.0	23.6	2
HN-85%	0.915	15.6	15.3	8.0	39.0	2
HN	0.915	15.6	17.4	8.0	41.0	2
HO	0.915	15.6	29.5	8.0	53.2	2

All loads in kN/m. *Includes earth load at fill height shown

weight of internal water

Working load due to water load, $Ww = 20.0 \text{ kN/m}$

earth

Height of fill, $H = 0.600 \text{ m}$

Pipeline orientation is skew

Flexible pavement type is asphalt.

Flexible pavement density = 21 kN/m^3

Flexible pavement thickness = 0.300 m

Effective height of fill, $He = 0.915 \text{ m}$

Embankment Condition, positive projection

Settlement ratio, $rs = 1.000$

Projection ratio, $p = 0.500$

Equal plane of settlement height, $He = 0.915$

Spangler coefficient, $C'e = 1.051$

Working load due to earth fill, $Wg = 39.1 \text{ kN/m}$

DESIGN OF 1800 DIA. CLASS 2 RRJ DRAINAGE PIPE**HN-85%**

Axle Load Calculation

Footprint width at top of pipe, $L1 = 3.626$ mFootprint length at top of pipe, $L2 = 1.526$ mFootprint area, $A = 5.533$ m²

Load on footprint = 102.0 kN

Impact factor = 1.03

Live load pressure at top of pipe, $q = 18.904$ kPaMinimum of $L2$ and D , $S = 1.526$ mEffective supporting length of pipe, $Le = 5.836$ mWorking load due to axle loads, Wq (axle) = 17.9 kN/m

Lane Load Calculation

Lane load design pressure (at surface) = 3.500 kPa

Lane width = 3.000 m

Impact factor = 1.03

Lane load pressure at top of pipe = 2.489 kPa

Working load due to lane load, Wq (lane) = 5.1 kN/m*Total working load due to live load, $Wq = 23.0$ kN/m***HN**

Axle Load Calculation

Footprint width at top of pipe, $L1 = 3.626$ mFootprint length at top of pipe, $L2 = 1.526$ mFootprint area, $A = 5.533$ m²

Load on footprint = 120.0 kN

Impact factor = 1.03

Live load pressure at top of pipe, $q = 22.240$ kPaMinimum of $L2$ and D , $S = 1.526$ mEffective supporting length of pipe, $Le = 5.836$ mWorking load due to axle loads, Wq (axle) = 21.1 kN/m

Lane Load Calculation

Lane load design pressure (at surface) = 3.500 kPa

Lane width = 3.000 m

Impact factor = 1.03

Lane load pressure at top of pipe = 2.489 kPa

Working load due to lane load, Wq (lane) = 5.1 kN/m*Total working load due to live load, $Wq = 26.1$ kN/m*

DESIGN OF 1800 DIA. CLASS 2 RRJ DRAINAGE PIPE**HO****Axle Load Calculation**Footprint width at top of pipe, $L1 = 4.326$ mFootprint length at top of pipe, $L2 = 1.526$ mFootprint area, $A = 6.601$ m²

Load on footprint = 240.0 kN

Impact factor = 1.03

Live load pressure at top of pipe, $q = 37.283$ kPaMinimum of $L2$ and D , $S = 1.526$ mEffective supporting length of pipe, $Le = 6.536$ mWorking load due to axle loads, Wq (axle) = 37.7 kN/m**Lane Load Calculation**

Lane load design pressure (at surface) = 3.500 kPa

Lane width = 3.000 m

Impact factor = 1.03

Lane load pressure at top of pipe = 2.489 kPa

Working load due to lane load, Wq (lane) = 5.1 kN/m*Total working load due to live load, $Wq = 44.3$ kN/m*

DESIGN OF 1050 DIA. RRJ DRAINAGE PIPE

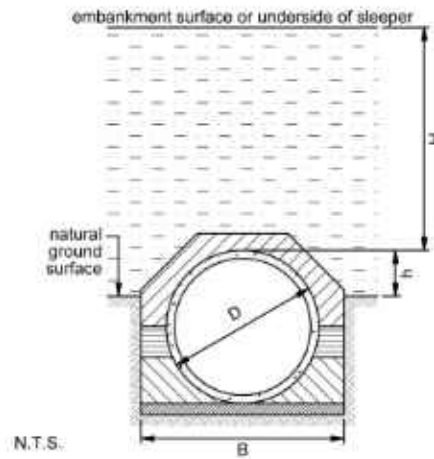
Client And Project Details

Date: 20-Aug-2025

Job number: 11320	Design: Culvert 17 - MIN cover
Client:	Designer: JYU
Project: B2P	Company:
Description: Significant culvert pipe strength checks	File: Pipe strength 2025.08.19.ppr

Design Parameters

Installation Condition:	embankment
Projection Type:	positive
Pipe Nominal Diameter (mm):	1050
Pipe External Diameter, D (mm):	1220
Pipeline Orientation:	skew
Soil Type:	wet clay
Soil Density (kN/m ³):	20
Soil Parameter K _{st} :	0.1100
Width, B (m):	1.664
Height Of Fill, H (m):	0.600
Flexible Pavement Type:	asphalt
Flexible Pavement Density (kN/m ³):	21.000
Flexible Pavement Thickness (m):	0.300
Effective Height Of Fill, H _e (m):	0.915
Projection Height, h (m):	0.610
Projection Ratio, p (h/D):	0.500
Support Type:	HS2
Bedding Factor:	2.5



In Service Load Cases/Combinations Considered (controlling load case/combination highlighted)

Load Description*	Fill Height (m)	Wg/2.5	Wq/1.5	Ww/2.5	Tc	Pipe Class
earth + weight of internal water	0.915	9.7		2.8	12.5	2
HN-85%	0.915	9.7	13.3	2.8	25.8	2
HN	0.915	9.7	15.3	2.8	27.8	2
HO	0.915	9.7	26.5	2.8	39.0	2

All loads in kN/m. *Includes earth load at fill height shown.

Controlling Loads: earth + HO standard vehicle + weight of internal water

Minimum Test Load: $T_c = 9.7 + 26.5 + 2.8 = 39.0$ kN/m

Adopt 1050 dia. Class 2 RRJ pipe (1050/2 RRJ) in accordance with AS/NZS 4058:2007.

Design Notes:

1. A nominal pipe wall thickness of 64 mm has been assumed.

INSTALLATION OF 1050 DIA. CLASS 2 RRJ DRAINAGE PIPE

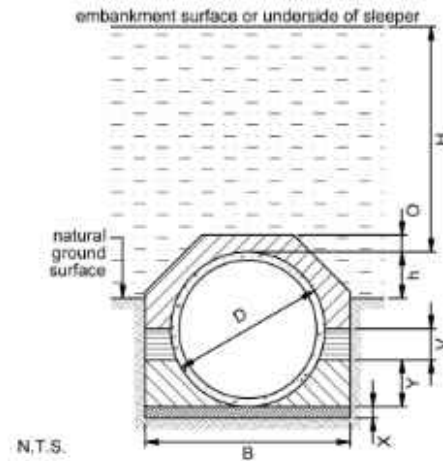
Client And Project Details

Date: 20-Aug-2025

Job number: 11320	Design: Culvert 17 - MIN cover
Client:	Designer: JYU
Project: B2P	Company:
Description: Significant culvert pipe strength checks	File: Pipe strength 2025.08.19.ppr

Design Parameters

Installation Condition:	embankment
Projection Type:	positive
Pipe Nominal Diameter (mm):	1050
Pipe External Diameter, D (mm):	1220
Pipeline Orientation:	skew
Width, B (m):	1.664
Height Of Fill, H (m):	0.600
Flexible Pavement Type:	asphalt
Flexible Pavement Density (kN/m ³):	21.000
Flexible Pavement Thickness (m):	0.300
Effective Height Of Fill, H _e (m):	0.915
Projection Height, h (m):	0.610
Projection Ratio, p (h/D):	0.500
Support Type:	HS2
Excavation Volume (solid) (m ³ /m):	1.2



Installation Quantities

Support Zone	Depth (mm)	Quantities(m ³ /m)		Minimum Zone Compaction (%)	
		Solid	Loose	Density Index (for cohesionless soils)	Relative Density (standard compaction)
Bed zone	X = 100	0.167	0.200	60	-
Haunch zone	Y = 370	0.317	0.380	60	-
Side zone	V = 240	0.115	0.138	60	90
Overlay zone	O = 150	0.413	0.000	as per project specification	as per project specification

Material Grading Requirements

Sieve Size (mm)	75.0	19.0	9.5	2.36	0.60	0.30	0.15	0.075
Bed & Haunch Zones (% mass passing)	-	100	-	100-50	90-20	60-10	25-0	10-0
Side Zone (% mass passing)	100	-	100-50	100-30	50-15	-	-	20-0

Construction Equipment Requirements

No construction vehicles considered.

Design Notes:

1. All bed, haunch and side zone material passing the 0.075 mm sieve to have low plasticity (AS 1726).
2. Ordinary fill material to have no stones > 150 mm dia., and no more than 20% to be 75-150 mm.
3. For additional information refer to the project specification.
4. A nominal pipe wall thickness of 64 mm has been assumed.

DESIGN OF 1050 DIA. CLASS 2 RRJ DRAINAGE PIPE

Client And Project Details

Date: 20-Aug-2025

Job number: 11320	Design: Culvert 17 - MIN cover
Client:	Designer: JYU
Project: B2P	Company:
Description: Significant culvert pipe strength checks	File: Pipe strength 2025.08.19.ppr

In Service Load Cases/Combinations Considered (controlling load case/combination highlighted)

Load Description*	Fill Height (m)	Wg/2.5	Wq/1.5	Ww/2.5	Tc	Pipe Class
earth + weight of internal water	0.915	9.7		2.8	12.5	2
HN-85%	0.915	9.7	13.3	2.8	25.8	2
HN	0.915	9.7	15.3	2.8	27.6	2
HO	0.915	9.7	26.5	2.8	39.0	2

All loads in kN/m. *Includes earth load at fill height shown

weight of internal water

Working load due to water load, $Ww = 7.0 \text{ kN/m}$

earth

Height of fill, $H = 0.600 \text{ m}$

Pipeline orientation is skew

Flexible pavement type is asphalt.

Flexible pavement density = 21 kN/m^3

Flexible pavement thickness = 0.300 m

Effective height of fill, $He = 0.915 \text{ m}$

Embankment Condition, positive projection

Settlement ratio, $rs = 1.000$

Projection ratio, $p = 0.500$

Equal plane of settlement height, $He = 0.915$

Spangler coefficient, $C'e = 1.087$

Working load due to earth fill, $Wg = 24.3 \text{ kN/m}$

DESIGN OF 1050 DIA. CLASS 2 RRJ DRAINAGE PIPE**HN-85%****Axle Load Calculation**

Footprint width at top of pipe, $L1 = 3.626$ m
Footprint length at top of pipe, $L2 = 1.526$ m
Footprint area, $A = 5.533$ m²
Load on footprint = 102.0 kN

Impact factor = 1.03

Live load pressure at top of pipe, $q = 18.904$ kPa

Minimum of $L2$ and D , $S = 1.220$ m

Effective supporting length of pipe, $Le = 4.953$ m

Working load due to axle loads, Wq (axle) = 16.9 kN/m

Lane Load Calculation

Lane load design pressure (at surface) = 3.500 kPa
Lane width = 3.000 m

Impact factor = 1.03

Lane load pressure at top of pipe = 2.489 kPa

Working load due to lane load, Wq (lane) = 3.0 kN/m

Total working load due to live load, $Wq = 19.9$ kN/m

HN**Axle Load Calculation**

Footprint width at top of pipe, $L1 = 3.626$ m
Footprint length at top of pipe, $L2 = 1.526$ m
Footprint area, $A = 5.533$ m²
Load on footprint = 120.0 kN

Impact factor = 1.03

Live load pressure at top of pipe, $q = 22.240$ kPa

Minimum of $L2$ and D , $S = 1.220$ m

Effective supporting length of pipe, $Le = 4.953$ m

Working load due to axle loads, Wq (axle) = 19.9 kN/m

Lane Load Calculation

Lane load design pressure (at surface) = 3.500 kPa
Lane width = 3.000 m

Impact factor = 1.03

Lane load pressure at top of pipe = 2.489 kPa

Working load due to lane load, Wq (lane) = 3.0 kN/m

Total working load due to live load, $Wq = 22.9$ kN/m

DESIGN OF 1050 DIA. CLASS 2 RRJ DRAINAGE PIPE**HO****Axle Load Calculation**Footprint width at top of pipe, $L1 = 4.326$ mFootprint length at top of pipe, $L2 = 1.526$ mFootprint area, $A = 6.601$ m²

Load on footprint = 240.0 kN

Impact factor = 1.03

Live load pressure at top of pipe, $q = 37.283$ kPaMinimum of $L2$ and D , $S = 1.220$ mEffective supporting length of pipe, $Le = 5.653$ mWorking load due to axle loads, Wq (axle) = 34.6 kN/m**Lane Load Calculation**

Lane load design pressure (at surface) = 3.500 kPa

Lane width = 3.000 m

Impact factor = 1.03

Lane load pressure at top of pipe = 2.489 kPa

Working load due to lane load, Wq (lane) = 3.0 kN/m*Total working load due to live load, $Wq = 39.7$ kN/m*

Committee Use Only

Date Received		Accepted for Committee	Yes / No
Departure Ref #			
Committee scheduled			
Notes			

REQUEST FOR TECHNICAL DEPARTURE

From:	Aurecon	Departure No:	DEP-0016
To:	Waka Kotahi	Date:	28/08/2025
Originator Name:	Yanni Hooi	Project	Belfast to Pegasus Motorway & Woodend Bypass Pre-implementation and MSQA Professional Services
Major Discipline: Other relevant disciplines:	Stormwater or Hydraulics + Bridges / Structural		
Subject	Abutment scour assessment methodology		

NZTA Confirmation of Departure

NZTA Contract Rep		NZTA Technical Lead Advisor	
Request Type	Referral to Chief Engineer Advisory Group		
	Standard Departure from Op Policy		
	Urgent / Emergency Works		

1 Background

Abutment scour is an important part of any bridge design. The two critical components of bridge design in relation to scour are the analysis of the anticipated scour depths and the design of the necessary scour mitigation systems. The first of these is to assist in the design of the structural bridge elements such as piers and abutments, with the second being to prevent damage to supporting bridge components such as roadway embankments. The focus of this departure is on the analysis of scour depths through a bridge opening.

The Waka Kotahi Bridge Manual (Third edition, Amendment 4) specifies scour estimates to be completed in accordance with the technical publication, Bridge Scour (Melville and Coleman 2000). This document has been active for over 20 years in which time new design methodologies for abutment scour analysis have been published in international scour literature.

The requested departure for the Kaiapoi Bridge and Cam River Bridge abutment scour assessment is to bring the scour estimates and corresponding designs up to date with the latest international practice. Specifically, the **National Cooperation Highway Research Program (NCHRP) 24-20 (NCHRP 2010b) methodology included in the Federal Highway Administration's (FHWA) Hydraulic Engineering Circular (HEC) 18** is the preferred and presented methodology for the abutment scour analysis on the Kaiapoi Bridge and Cam River

Bridge. Overall, using the latest guidance in the FHWA's guidance documents is intended to produce more accurate scour estimates and fit for purpose design measures.

2 Standard or Works Requirement to be Departed from:

The Bridge Manual (2.3.5) specifies the following:

*"The estimation of scour in non-cohesive bed materials should be based on **Bridge scour(16)**. However, this publication does not directly cover scour in cohesive bed materials, so for these situations **Austrroads Guide to bridge technology part 8(15)** shall be used. Note however that Austrroads Guide to bridge technology part 8(15) does not include a method for estimating local scour at piers in cohesive bed materials. In this situation **Evaluating scour at bridges(12)** provides additional guidance"*

3 Reason for Requested Departure (including what is requested and why, and what other options were considered)

3.1 What is requested

The scour assessment methodologies presented in Bridge Scour (Melville and Coleman 2000) were developed in the early 1980s and have been effectively superseded by NCHRP 24-20, released in 2001 and formally incorporated into HEC-18 in 2018. We are requesting that Waka Kotahi accept the presented abutment scour methodology, NCHRP 24-20, taken from the FHWA HEC-18 design guide in replacement of the methodologies included in Bridge Scour.

3.2 Options Considered

The fifth edition of HEC-18 was first released in 2012, 11 years after the fourth edition was published in 2001. The document states that in this time between releases, significant advances were made in the practice of estimating scour at bridges, with the NCHRP 24-20 (focused on abutment scour) being a particular area of development not previously included in the older fourth edition. Considering that Bridge Scour was published in 2001, the design team opted to implement the modernised approach detailed in the fifth edition of HEC-18. The following bullet points list the advantages that are stated in HEC-18 on the NCHRP 24-20 benefits.

- The equations are independent of the abutment effective width, a difficult parameter to estimate in traditional abutment scour approaches, which subject to significant engineering judgement.
- The equations are more physically representative of the abutment scour process, and consider the channel material's particle size, an important factor in scour analysis.
- The equations predict total scour at the abutment rather than the abutment scour component that is added to contraction scour.

3.3 Rationale for Request

- Alignment with Current Best Practice: The requested departure reflects adoption of the most recent and internationally recognised methodology for abutment scour analysis (NCHRP 24-20 from the FHWA HEC-18 fifth edition), ensuring the project applies up-to-date scientific and engineering knowledge beyond the older Bridge Scour (Melville and Coleman 2000) guidance.
- Improved Accuracy and Reliability: The NCHRP 24-20 equations offer a more physically representative model of the abutment scour process, leading to more accurate estimation of total scour depth. This improves the reliability of the scour assessment and supports more effective bridge design and mitigation.
- Consistency with Modern Engineering Standards: Since the Bridge Scour publication dates back over 20

years, the departure ensures the project aligns with recent advances and international consensus on scour analysis, enhancing the overall durability and safety of the Kaiapoi and Cam River Bridges.

- **Risk Management:** Implementing this updated methodology supports better risk assessment and management of scour-related failures, which is critical to the long-term structural integrity of bridge components and associated infrastructure.

Therefore, the rationale for this request is to improve the technical consistency, safety, and better scour assessments for the project by adopting the latest validated methodology, delivering value, and managing risk in line with current international engineering practice. This methodology has been accepted and is used internationally, including in Australia and the United States and has been approved and used on numerous recent projects within New Zealand, including SH1 Papakura to Drury and TREC rebuild work. The result will be a more accurate assessment of the abutment scour risk to bridges, based on current international engineering practice.

4 Benefits of the Departure

Safety:	There are unavoidable residual safety risks when assessing scour depths considering that all design approaches are empirical approximations of a dynamic physical process. The safety risk is that one of the possible estimation techniques fails to categorise the physical scour processes, however, with the proposed methodology we expect to provide the most resilient design available from current literature, reducing the risk of scour related safety incidents.
Environmental:	The adoption of the NCHRP 24-20 abutment scour approach is expected to provide a more resilient bridge and scour protection design. This will limit the scouring and erosion of Kaiapoi River and Cam River, a benefit for the local river environment.
Performance (level of service and resilience, customer levels of service):	No change. Both the proposed departure and the older methods in Bridge Scour would be applied to the same design conditions.
NZTA Reputation:	Minor improvements considering this departure will bring the agency in line with the most up to date assessment methodologies.
Cost:	The cost impacts have not been quantified due to the alternative abutment scour estimates specified in Bridge Scour not being considered. The adoption of NCHRP 24-20 will result in greater confidence in the scour analysis results, limiting the level of conservatism necessary in the ensuing design.
Maintenance:	Reduced maintenance frequency can be expected, considering that the scour protection is to be designed against a scour profile thought to be more physically accurate.

5 Costs and Dis-benefits of Departure

Safety:	N/A
Environmental	N/A
Performance (level of service and resilience)	No change. Both the proposed departure and the older methods in Bridge Scour would be applied to the same design conditions.
NZTA Reputation	N/A
Cost	Refer to Section 4.
Maintenance	N/A

6 Residual Risks Associated with the Departure

The requested departure does not increase the risk to the Kaiapoi and Cam River Bridge structure. The same design events and flood conditions will be used in the scour checks for the bridge abutments.

7 Impacted parties

There is no negatively impacted parties.

8 Value for Money Summary

The scour estimates produced using the proposed latest methodology are expected to be more accurate than those produced using the outdated methodology in Bridge Scour (Melville and Coleman 2000). It permits designers to undertake the assessments with a higher level of confidence, allowing for reduced conservatism in engineering judgement when assessing the scour. The final quantitative cost savings from this approach are unable to be determined, however, there is qualitative value added given that the final analysis and corresponding designs are reflective of industry best practices.

9 Attachments

No attachments included in this departure.

10 Signoffs

Prepared by: Yanni Hooi	Signed: 	Date: 27/8/2025
Reviewed By: David Delagarza	Signed: 	Date:
Regan Smith		27/8/2025
Recommended By: Matt Flannery	Signed: 	Date: 28/8/2025

Recommendations / Comments

NZTA Contract Representative:

Principal’s Agent Specialist Advisor:

Waka Kotahi Technical Specialist:

New Zealand Transport Agency Technical Decision:

Outcome:

Comments:

By:

Date:

New Zealand Transport Agency Authorisation to Proceed:

By:

Date:

Committee Use Only

Date Received		Accepted for Committee	Yes / No
Departure Ref #			
Committee scheduled			
Notes			

REQUEST FOR TECHNICAL DEPARTURE

From:	Aurecon	Departure No:	DEP-0017
To:	Waka Kotahi	Date:	28/08/2025
Originator Name:	Yanni Hooi	Project	Belfast to Pegasus Motorway & Woodend Bypass Pre-implementation and MSQA Professional Services
Major Discipline: Other relevant disciplines:	Stormwater Design		
Subject	Median Drainage – Departure from NZTA P46 Section 6.1.k.		

NZTA Confirmation of Departure

NZTA Contract Rep		NZTA Technical Lead Advisor	
Request Type	Referral to Chief Engineer Advisory Group		
	Standard Departure from Op Policy		
	Urgent / Emergency Works		

1 Background

The drainage design for this project has been developed mainly following the NZTA P46 State Highway Stormwater Specification. Clause 6.1(k) of NZTA P46 states that the surface water is not permitted to flow from one side to the other on median divided roads of four or more lanes in total. This includes preventing water from crossing gore areas and crossing the median in super elevated sections.

Site constraints such as topography, space constraints within the road corridor and maintenance made it impractical to meet this requirement in all locations. During the 50% design stage, we have identified 4 locations where road is designed with superelevation crossfall. Where it's superelevated and with space constraint, there is limited viable solutions to discharge the runoff without crossing the median.

2 Standard or Works Requirement to be Departed from:

NZTA P46 6.1 k)

“On median divided roads of four or more lanes total, surface water is not permitted to flow from one side to the other. This includes preventing water from crossing gore areas and crossing the median in super elevated sections.”

3 Reason for Requested Departure (including what is requested and why, and what other options were considered)

3.1 What is requested

We seek Waka Kotahi’s approval of the proposed design approach permitting runoff to cross the median in sections where the road is superelevated.

Below are the locations where the road is designed with superelevation and where departures from standard drainage design requirements are necessary. For road safety assurance, surface runoff depth at these sites has been evaluated using the WFD (RRL) method, confirming that runoff remains below the critical threshold of 4 mm.

Road Chainage	WFD (RRL Method) Surface runoff depth (mm)
CH2775	2.0
CH5235	2.0
CH7395	2.0
CH9210	2.3

3.2 Options Considered

In order to meet the requirement that runoff must not flow across the median from one side of the road to the other, the following options were considered but found to be impractical:

Options	Considerations
Avoid superelevated road crossfall	Roads are superelevated on curves to counteract the outward force that pushes a vehicle away from the centre of the curve. This superelevation helps vehicles navigate the curve safely and comfortably, especially at higher speeds. Without superelevation, the vehicle's tyres would need to generate more friction to counteract this force, potentially leading to skidding or loss of control, making this solution unfeasible.
Constructing median barriers or raised kerbs with catchpits installed to prevent surface runoff crossing	The limited space within the road corridor and this solution would require extensive additional stormwater reticulation and infrastructure. This along with challenges in maintaining catchpits in the median make this solution less desirable.
Constructing swale or trench drain within the median to prevent surface runoff crossing	The limited space within the road corridor, significant additional footprint width required to construct a swale or trench drain, and challenges in maintaining road swale or trench drain in the median make this solution undesirable.

Due to these physical and operational limitations at identified locations, completely preventing runoff from crossing the median is not possible without significant cost, design compromise, or maintenance issues. Therefore, a controlled allowance for runoff crossing the median is proposed, with mitigation measures to manage any associated risks.

3.3 Rationale for Request

Preventing runoff from crossing the median in superelevated areas is challenging because:

- Maintaining superelevation is critical for road safety: The road must be superelevated on curves to counteract lateral forces, ensuring safe and comfortable vehicle handling. Removing superelevation is not feasible as it increases the risk of skidding and vehicle instability, especially at higher speeds.
- Median barriers with catchpits are impractical: Limited corridor width restricts installation of median barriers combined with drainage catchpits, and maintenance access to such infrastructure within the median would be difficult.
- Swales or trench drains within the median are not viable: Space constraints and ongoing maintenance challenges also rule out swales or trench drains as effective runoff controls in the median area.

Given these constraints, a controlled and managed allowance for runoff to cross the median is proposed. This approach will include measures to mitigate any potential risks related to drainage performance and road safety, thus balancing compliance challenges with practical site realities.

In summary, the requested departure permits safe and efficient drainage management in superelevated sections without compromising the integrity of the road or excessive costs and maintenance difficulties.

4 Benefits of the Departure

Safety:	For road safety assurance, surface runoff depth at these sites has been evaluated using the WFD (RRL) method, confirming that runoff remains below the critical threshold of 4 mm. The pavement will also be surfaced with EPA7, which provides high surface permeability, further reducing surface runoff and providing an extra level of aquaplaning risk reduction Preventing installing median barrier and stormwater assets in the median avoids the associated need for maintenance in trafficked lane hence lowering maintenance safety risk.
Environmental:	Reduced project footprint due to reduced the used of concrete barrier and stormwater assets such as cesspits, grates, trench channels
Performance (level of service and resilience, customer levels of service):	No influence in road performance.
NZTA Reputation:	Improve reputation through more optimal and cost effective design
Cost:	Reduced cost by avoid installing concrete barrier and stormwater assets in the median. This also reduce cost in maintenance.
Maintenance:	Reduce the maintenance needs by avoid installing concrete barrier and stormwater assets in the median.

5 Costs and Dis-benefits of Departure

Safety:	N/A
Environmental	N/A
Performance (level of service and resilience)	No change to level of service
NZTA Reputation	N/A
Cost	N/A
Maintenance	N/A

6 Residual Risks Associated with the Departure

The residual risk associated with permitting runoff to cross the median in superelevated sections is considered low. Although this design results in a marginal increase in runoff depth downstream of the trafficked lane, Water Flow Depth (WFD) analysis confirms that runoff depths remain below the critical safety threshold of 4 mm. Additionally, open graded asphalt surfacing (EPA7) will further reduce the risk of aquaplaning. Consequently, the likelihood of aquaplaning is minimal, ensuring that traffic safety is maintained despite the controlled runoff crossing.

7 Impacted parties

There are no negatively impacted parties.

8 Value for Money Summary

The proposed departure allowing controlled runoff crossing of the median in superelevated sections provides significant value for money by delivering a safe, effective drainage solution without the need for costly and difficult to maintain central median drainage infrastructure.

By avoiding the installation of costly median barriers, raised kerbs, catchpits, swales, or trench drains, which are impractical due to additional space requirements and maintenance access difficulties, the design significantly reduces both upfront construction and long-term maintenance costs.

The design ensures road safety is not compromised, with surface runoff depths well below the critical 4 mm aquaplaning threshold, and no adverse impact on road performance or driver experience.

Eliminating median drainage infrastructure reduces maintenance risk and costs by minimising work within trafficked lanes.

In overall, this approach represents a balanced, practical solution that achieves compliance with safety objectives while optimising expenditure and maintenance efforts, thereby providing excellent value for money for all stakeholders.

9 Attachments

No attachments included in this departure.

10 Signoffs

Prepared by: Yanni Hooi	Signed: 	Date: 27/8/2025
Reviewed By: David Delagarza	Signed: 	Date: 27/8/2025
Regan Smith		
Recommended By: Matt Flannery	Signed: 	Date: 28/8/2025

Recommendations / Comments

NZTA Contract Representative:

Principal’s Agent Specialist Advisor:

Waka Kotahi Technical Specialist:

New Zealand Transport Agency Technical Decision:

Outcome:

Comments:

By:

Date:

New Zealand Transport Agency Authorisation to Proceed:

By:

Date:

I.//////////Committee Use Only

Date Received		Accepted for Committee	Yes / No
Departure Ref #			
Committee scheduled			
Notes			

REQUEST FOR TECHNICAL DEPARTURE

From:	Aurecon	Departure No:	DEP-0018 Rev B
To:	Waka Kotahi	Date:	23/10/2025
Originator Name:	Yanni Hooi	Project	Belfast to Pegasus Motorway & Woodend Bypass Pre-implementation and MSQA Professional Services
Major Discipline: Other relevant disciplines:	Stormwater Design		
Subject	Use of RCP 8.5 climate change for culvert design		

NZTA Confirmation of Departure

NZTA Contract Rep		NZTA Technical Lead Advisor	
Request Type	Referral to Chief Engineer Advisory Group		
	Standard Departure from Op Policy		
	Urgent / Emergency Works		

1 Background

The drainage design for this project has been developed primarily in accordance with the NZTA P46 State Highway Stormwater Specification.

While NZTA P46 specifies culvert sizing based on the MPD RCP 6.0 climate change scenario, most culvert catchments on this project exceed 15 hectares. Therefore, culvert sizing was informed by the TUFLOW flood model, which utilises RCP 8.5 (2081–2100) rainfall data provided by Waimakariri District Council. Although this differs from the P46 requirements, the RCP 8.5 scenario is more conservative.

2 Standard or Works Requirement to be Departed from:

NZTA P46 6.4.1 c i) states the following:

“All culverts and pipe crossings, existing and new, that cross a road or have an influence on the works must satisfy the following hydraulic conditions: i. Convey the 100-year ARI storm event flow without surcharge of the pipe more than 2 metres above the pipe soffit, whilst ensuring a minimum 500 millimetres freeboard is provided from the peak water level to the outer edge line level for the MPD RCP 6.0 scenario”

3 Reason for Requested Departure (including what is requested and why, and what other options were considered)

3.1 What is requested

We request for Waka Kotahi's acceptance of the culvert design based on RCP 8.5 (2081–2100) rainfall data in place of RCP 6.0. Although NZTA P46 requires culverts to be sized for the MPD RCP 6.0 (2110) climate change scenario, most catchment areas exceed 15 hectares. Therefore, culvert sizing has been based on the Waimakariri District Council (WDC) TUFLOW flood model, which utilises RCP 8.5 (2081–2100) rainfall data. While this differs from the P46 requirement, RCP 8.5 (2081–2100) is approximately equivalent (~1% or less difference) to the RCP 6.0 (2120) values as indicated below.

Scenario	100 yr ARI 30 min	100yr ARI 1 hr	100 yr ARI 6 hr	100 yr ARI 24 hr
RCP 8.5 (2081–2100)	55.7	39.5	15.7	6.73
RCP 6.0 (2120),	55.6	39.5	15.8	6.8
% Difference	-0.2%	0%	+0.6%	+1.0%

Note that for this assessment RCP 6.0 (2120) values have been used, which are slightly more conservative than RCP 6.0 (2110), which cannot be directly derived from current guidance. RCP 6.0 (2110) values are expected to be slightly lower, likely bringing them all below the RCP 8.5 values.

3.2 Options Considered

To ensure compliance with regional flood management, the proposed culverts have been assessed against RCP 8.5 up to 2100. To size the culverts based on the RCP 6.0 scenario up to 2120, separate hydraulic models run would need to be generated using new spatially and temporally varying rainfall data, which is a time-intensive process taking additional weeks to complete the model runs. Moreover, under P46 clause 6.4.1, if the freeboard between flood levels and the road edge is less than one metre, a sensitivity assessment using the RCP 8.5 scenario is required to evaluate potential risks to the project or adjacent properties. Since the TUFLOW flood model already uses available RCP 8.5 (2081–2100) rainfall data to meet the district council's flood assessment requirements, leveraging these model results for culvert design is more practical and efficient. This approach also satisfies the P46 sensitivity assessment requirement. Using the more conservative RCP 8.5 data provides a future-proof design and is not expected to significantly alter culvert sizing.

3.3 Rationale for Request

The hydrology of the catchments traversed by the Project is complex, consisting of a series of interconnected natural and manmade channels, with significant surface water-groundwater interactions. Catchments of this nature cannot be accurately represented using simplified rainfall runoff methodology (i.e. rational method or simple lumped catchment models).

WDC has invested significant resources into developing a comprehensive fully coupled flood model of these catchments, in consideration of these factors. This model represents the best available information for

establishing peak flows within the catchment and is the model that WDC uses for assessing flooding impacts of the overall project.

Although P46 specifies RCP 6.0 for culvert design, the project adopts the more conservative RCP 8.5 data provided by WDC through the TUFLOW model. This approach accounts for higher projected temperature increases and rainfall intensities, thereby future-proofing the design against climate change with minimal impact on culvert sizing. Leveraging existing RCP 8.5 hydraulic models also improves efficiency and satisfies P46's sensitivity assessment requirements without needing new and time-consuming model runs.

While not directly required by the Project's consent requirements, WDC undertakes assessment of flooding impacts utilising RCP 8.5 in 2100. Updating the WDC flood model to utilise RCP 6.0 rainfall would be a resource and time-consuming process, with potential costs to develop exceeding \$100,000, and developing a new model would likely exceed \$200,000 in costs.

It is furthermore noted that the hydraulic design of the culverts is generally not directly driven by the peak flows. Instead flooding mitigation requirements typically dictate the culvert sizing. Thus, modifying the design flows through the culverts would not be likely to result in reductions of culvert sizes.

In summary, based on the very minor flow differences (1% or less), high cost of updating the hydraulic model, and low likelihood of beneficial cost impacts to the project, we request Waka Kotahi's acceptance of the proposed design approach by allowing the use of RCP 8.5 climate data for culvert sizing, based on the detailed technical assessments and justified constraints outlined above.

4 Benefits of the Departure

Safety:	Future proofing the culvert size by considering higher climate change factor
Environmental:	No impact expected
Performance (level of service and resilience):	Increase design resilience by considering higher climate change factor during culvert design
NZTA Reputation:	Improved reputation through more resilient design
Cost:	No impact expected
Maintenance:	No impact expected

5 Costs and Dis-benefits of Departure

Safety:	N/A
Environmental	N/A

Performance (level of service and resilience)	• No change to level of service
NZTA Reputation	• N/A
Cost	• N/A
Maintenance	• N/A

6 Residual Risks Associated with the Departure

There are no residual risks associated with culvert design up to RCP 8.5.

7 Impacted parties

There are no negatively impacted parties.

8 Value for Money Summary

As outlined above, using more conservative RCP 8.5 climate data instead of RCP 6.0 for culvert design results in minimal changes to culvert sizing and does not affect overall costs. This approach also provides extra futureproofing against climate change with little impact on culvert size.

Overall, the proposed approach offers a cost-effective solution without reducing service levels or design life.

9 Attachments

N/A

10 Signoffs

Prepared by: Yanni Hooi	Signed: 	Date: 22/10/2025
Reviewed By: David Delagarza	Signed: 	Date: 22/10/2025
Regan Smith		
Recommended By: Matt Flannery	Signed: 	Date: 22/10/2025

Recommendations / Comments

NZTA Contract Representative:

Principal’s Agent Specialist Advisor:

Waka Kotahi Technical Specialist:

New Zealand Transport Agency Technical Decision:

Outcome:

Comments:

By:

Date:

New Zealand Transport Agency Authorisation to Proceed:

By:

Date:

Appendix C

Stormwater and Flooding Assessment Report

Belfast to Pegasus Motorway & Woodend Bypass Pre-implementation and MSQA Professional Services

Stormwater and Flooding Assessment

NZ Transport Agency (NZTA)

Document code: 11320-AUR-0350-PWI-SW-RPT-0003

Revision: D

Date prepared: 17 October 2025



Document control record

Document prepared by:

Aurecon New Zealand Limited

Level 2, Iwikau Building
93 Cambridge Terrace
Christchurch 8013
New Zealand

T +64 3 366 0821

F +64 3 379 6955

E christchurch@aurecongroup.com

W aurecongroup.com

Document control						aurecon
Report title		Stormwater and Flooding Assessment				
Document code		11320-AUR-0350-PWI-SW-RPT-0003	Project number	11320		
Document state		PUB Published	Status code	P3 Detailed Design		
File path		https://aurecongroup.sharepoint.com/sites/528638/5_WorkingFiles/Consenting/Main Works SAR — Current Working Version/Volume 3/3L - Stormwater and Flooding Assessment/11320-AUR-0350-PWI-SW-RPT-0003.docx				
Rev	Date	Revision details	Author	Verifier (if required)	Approver	
A	2025-08-22	Issue for Client Review	D. Delagarza	N. Holden	M. Flannery	
B	2025-09-05	Draft for consultation	D. Delagarza	N. Holden	M. Flannery	
C	2025-10-03	Draft for NZTA review prior to lodgement, incorporating consultation feedback	D. Delagarza	N. Holden	M. Flannery	
D	2025-10-17	Final for FTAA application lodgement	D. Delagarza	N. Holden	M. Flannery	
Current revision		D				

Client			
Client name	NZ Transport Agency (NZTA)	Client contact	Brendon French
Client document code	N/A	Client document revision	N/A

Approval			
Author signature		Approver signature	
Name	David Delagarza	Name	Matt Flannery
Title	Associate Water Resources	Title	Major Projects Director

Abbreviations And Definitions

Abbreviation	Term
AEP	Annual Exceedance Probability
CC	Climate Change
Ch.	Chainage
CIA	Cultural Impact Assessment
CMA	Coastal Marine Area
Culvert	Stormwater conduit that conveys either (or combination of) external, pervious and/or treated stormwater runoff from the upstream side of a road/motorway to the downstream outfall
DBC	Detailed Business Case
DOC	Department of Conservation
EclA.	Ecological Effects Assessment
HEC14	Hydraulic Engineering Circular 14
HEC18	Hydraulic Engineering Circular 18
HIRDS	High Intensity Rainfall Design System
HRT	Hydraulic Residence Time
Km	Kilometres
m	Metres
MCA	Multi-criteria assessment
NCHRP	National Cooperative Highway Research Program
NOR	Notice of Requirement
NUO	Network Utility Operator
NZCPS	New Zealand Coastal Policy Statement
NZTA	NZ Transport Agency Waka Kotahi
NZVD-16	New Zealand Vertical Datum 2016
Outfall	Downstream discharge location of a stormwater pipe, culvert or open channel
Project	State Highway 1 North Canterbury – Woodend Bypass Project (Belfast to Pegasus) (the construction, operation, and maintenance thereof)
RL	Relative Elevation
RoNS	Roads of National Significance
RMA	Resource Management Act 1991
SH1	State Highway 1 (note this refers to both portions of the existing roadway and the proposed motorway)
SLR	Sea level rise
SLS	Serviceability Limit State
ToC	Time of concentration
ULS	Ultimate Limit State

Contents

1	Executive Summary	1
2	Introduction.....	2
2.1	Purpose and overview of the Project.....	2
2.2	Purpose and content of this technical report.....	2
2.3	Expert witness statement	2
2.4	Applicability statement.....	2
3	Existing Environment	3
3.1	External Catchments	3
3.1.1	Overview of External Catchments	3
3.1.2	Existing Flooding – Rainfall Runoff	5
3.2	Existing Roadway Stormwater Management	11
3.2.1	Kaiapoi River Catchment (Kaiapoi Urban Area).....	11
3.2.2	Cam River Catchment	11
3.2.3	Quarry Lakes Catchment	12
3.2.4	McIntosh Drain Catchment.....	12
3.2.5	Waihora Stream Catchment	12
3.2.6	Taranaki Stream Catchment	13
4	Description of Works	15
4.1	External Stormwater Management.....	15
4.1.1	Cut-off Drains	15
4.1.2	Channel Realignment.....	15
4.1.3	Cross Culverts	16
4.2	Proposed Bridge Improvements.....	17
4.2.1	Kaiapoi River Bridge.....	17
4.2.2	Proposed Cam River Bridge.....	18
4.3	Overview of Proposed Stormwater Quality Management Methodologies.....	19
4.3.1	Grass Lined Swale	20
4.3.2	Bioinfiltration Swale	20
4.3.3	Bioretention Swale.....	20
4.3.4	Bioinfiltration Basin	21
4.3.5	Bioretention Basin	21
4.3.6	Proprietary Device	21
4.4	Proposed Motorway Stormwater Drainage Overview	21
4.4.1	Kaiapoi River Catchment.....	21
4.4.2	Cam River Catchment	22
4.4.3	Quarry Lakes Catchment	22
4.4.4	Mcintosh Drain Catchment	22
4.4.5	Waihora Stream Catchment	23
4.4.6	Taranaki Stream Catchment	24
5	Assessment of Effects.....	25
5.1	Stormwater Quality	25
5.1.1	Potential Effects.....	25
5.1.2	Proposed Mitigations	26
5.1.3	Summary and Residual Effects	26
5.2	Flooding and Stormwater Quantity.....	27

5.2.1	Potential Effects.....	27
5.2.2	Proposed Mitigations	27
5.2.3	Summary and Residual Effects	32
5.3	Erosion, Scour and Channel Stability.....	33
5.3.1	Potential Effects.....	33
5.3.2	Proposed Mitigations	34
5.3.3	Summary and Residual Effects	34
6	Conclusion.....	35

Appendices

Appendix A

Flood Modelling Report

Figures

Figure 3-1	Wider catchments interfacing with Project
Figure 3-2	McIntosh Drain Near the Motorway Crossing
Figure 3-3	Waihora Stream is well defined in the upper reaches (left) but undefined in the lower reaches (right)
Figure 3-4	B2P-Woodend flood model pre-development flood depth under 100-year ARI event
Figure 3-5	B2P-Woodend flood model pre-development flood level (mRL) under 100-year ARI event
Figure 3-6	B2P-Woodend flood model pre-development flood depth (m) under 200-year ARI event
Figure 3-7	B2P-Woodend flood model pre-development flood level (mRL) under 200-year ARI event
Figure 5-1	Private buildings affected under 200 year ARI (mitigated scenario)
Figure 5-2	Residual impacts outside the designation but does not impact any property buildings (200 year ARI afflux level)

Tables

Table 4-1	Proposed Channel Realignment
Table 4-2	Proposed Cross Culverts
Table 4-3	Scour depth (m) by bridge component in the worst case SLS and ULS event scenarios
Table 4-4	Scour depth (m) by bridge component in the worst case SLS and ULS event scenarios
Table 4-5	Scour Protection Summary
Table 4-6	Treatment Methodology Overview
Table 5-1	Expected Treatment Efficacy Overview
Table 5-2	200-year ARI flood level and differences at buildings (Mitigation vs Pre-development scenario)
Table 5-3	Locations that observed a water level increase outside of designation boundaries

Disclaimer

This report (including any enclosures, data, and attachments) has been prepared for the exclusive use and benefit of our client, solely for the purpose for which it is provided, and on the terms and conditions agreed with our client. Unless we provide express prior written consent, no part of this report should be reproduced, distributed or communicated to any third party. To the extent permissible by law, we do not accept any liability if this report is used or relied on by any unauthorised third party, or by the client for any unauthorised purpose, in any other contexts or without checking it is the latest revision.

1 Executive Summary

The New Zealand Transport Agency Waka Kotahi (NZTA) proposes to construct, operate, and maintain the State Highway 1 North Canterbury – Woodend Bypass Project (Belfast to Pegasus). The Project extends the Christchurch Northern Motorway by approximately 11 kilometres, providing a four-lane, grade-separated motorway that bypasses Woodend township and delivers improved safety, access, and reliability along this strategic transport corridor.

This technical report assesses the stormwater management and hydrological effects associated with the Project to support the resource consent application. It describes the existing stormwater and hydrological environment, evaluates the potential impacts arising from the operation of permanent stormwater infrastructure, and outlines mitigation and management measures to ensure that effects are appropriately managed.

The Project traverses a number of catchments, including the Kaiapoi River, Cam River, McIntosh Drain, Waihora Stream, and Taranaki Stream. Existing stormwater systems in these catchments are generally limited, with many discharges from the existing SH1 alignment currently untreated. Flooding is concentrated along defined channels and low-lying areas, with tidal influences present in the lower river reaches. The Project introduces earthwork, new impervious areas, realigned channels, culverts, and bridge structures that have the potential to alter flow paths, increase contaminant loads, and exacerbate flooding if not managed appropriately.

To address these potential effects, the Project incorporates a number of stormwater management measures. These include grass-lined swales, bioinfiltration and bioretention swales and basins, and proprietary treatment devices where necessary. These devices have been selected and designed in accordance with NZTA standards to provide effective contaminant removal, including suspended solids, hydrocarbons, and heavy metals. Flood modelling has informed the design of culverts, bridges, and channel diversions, demonstrating that, with mitigation, adverse flooding effects can be reduced to less than minor and confined to within the designation or existing drainage features.

Erosion and scour risks have been considered at all culvert outlets, realigned channels, and bridge crossings. Design measures such as riprap protection, energy dissipation structures, and revegetation planting will maintain channel stability and ensure long-term resilience. The assessment concludes that with the proposed mitigation, residual effects on hydrology, flooding, and stormwater quality will be less than minor.

Overall, the Project delivers enhanced stormwater management outcomes relative to the existing SH1 corridor. By adopting best practicable options for stormwater treatment, flood management, and channel stability, the Project achieves compliance with relevant standards, protects downstream environments, and ensures the resilience and functionality of the transport corridor into the future.

2 Introduction

2.1 Purpose and overview of the Project

The New Zealand Transport Agency Waka Kotahi (NZTA) proposes to construct, operate, and maintain the State Highway 1 North Canterbury – Woodend Bypass Project (Belfast to Pegasus) (the Project).

The Project is an extension of the Christchurch Northern Motorway and will provide four lanes of grade-separated motorway over an approximately 11 kilometre (km) length. The physical work commences approximately 200 metres (m) south of the Ohoka Road Overpass and extends to approximately 700 m north of the Pegasus/Ravenwood intersection, including a bypass of Woodend township.

The purpose of the Project is to provide an efficient and reliable state highway connection between Belfast and Pegasus, while delivering improved access, community safety and public health outcomes, and reduced severance through Woodend.

2.2 Purpose and content of this technical report

This report provides technical support to the Substantive Application Report (**SAR**) made by New Zealand Transport Agency Waka Kotahi (**NZTA**) under the Fast-track Approvals Act 2024 (**FTAA**).

This report assesses the stormwater management and hydrological effects resulting from the operation of permanent stormwater infrastructure and structures located within the Project Site and recommends mitigation and management measures to address any actual or potential adverse effects.

The report will:

- Describe the existing stormwater management and hydrological environment within the Project Site;
- Assess the effects of the proposed operational Project stormwater infrastructure; and
- Recommend mitigation and management measures to address potential adverse effects.
- Describe the effect of and proposed mitigations for flooding and water quantity impacts to the wider surrounding catchments

A more comprehensive background and description of the Project is contained in the SAR; please refer to the SAR for those details.

2.3 Expert witness statement

While this is not a matter before the Environment Court, the authors of this report have each read the Code of Conduct for Expert Witnesses contained in the Environment Court Practice Note 2023 ('Code'). The authors have each complied with the Code in the preparation of this report.

The data, information, facts and assumptions the authors have each considered as part of this report are set out in this report. The reasons for the conclusions of the report are also set out in this report. Unless stated otherwise, this report is within each of the authors' expertise and the authors have not omitted to consider material facts known to them that might alter or detract from the opinions expressed.

2.4 Applicability statement

We understand and agree that NZTA will submit this report as part of an application under the Fast-Track Approvals Act 2024 and the appointed panel will use this report for the purpose of assessing that application.

3 Existing Environment

The following is an overview of the existing environment, in context of stormwater and hydrology, within the Project Site and surrounding area. The motorway design chainages have been used to reference the locality of existing stormwater management devices. Refer to the design drawings for referenced motorway design chainages.

3.1 External Catchments

3.1.1 Overview of External Catchments

For the purposes of assessing stormwater effects, the following major catchments are traversed by the Project, from South to North (refer to Figure 3-1). Each of the catchments is discussed in detail below.

- Kaiapoi River Catchment (CH 0 to 600)
- Cam River Catchment (CH 600 to 3600)
- Quarry Lakes Catchment (CH 3600 to 4600)
- McIntosh Drain (CH 4600 to 5800)
- Waihora Stream (CH 6100 to 9300)
- Taranaki Stream (CH 9300 to 9900)

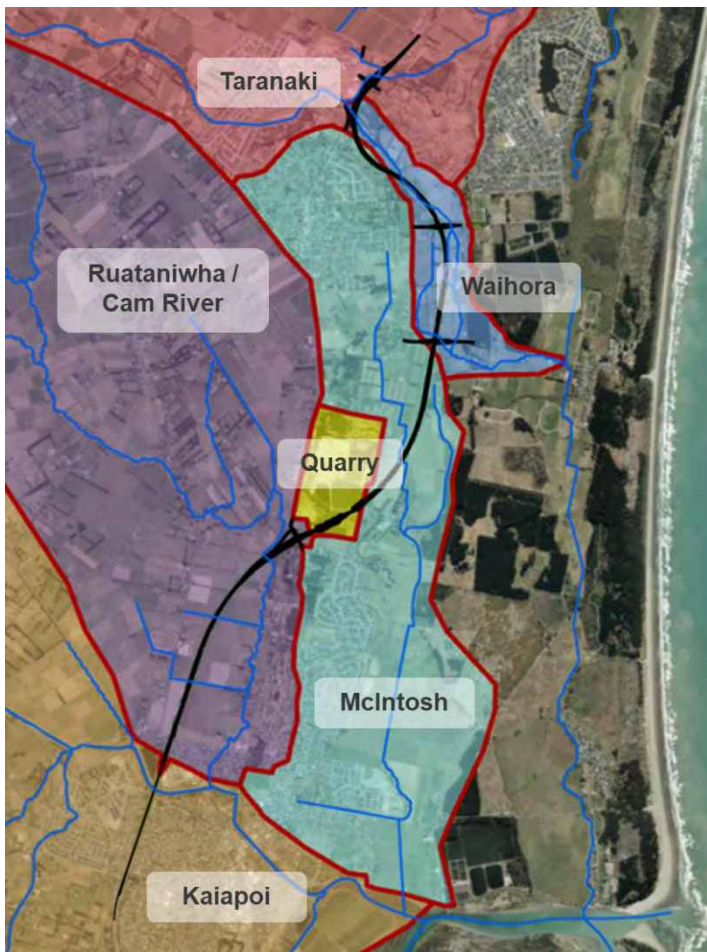


Figure 3-1 Wider catchments interfacing with Project

Kaiapoi River

The Kaiapoi River is a major tributary to the Waimakariri River, which it discharges into approximately 2km downstream of the motorway alignment. The Kaiapoi river has a catchment area of approximately 340 km² which extends across the Canterbury Plains into the foothills of the Southern Alps.

Land use within the Kaiapoi River catchment is dominated by the Kaiapoi urban environment in the lower reaches, and a mix of agricultural and large lot residential upstream of Kaiapoi.

Within the Project area, the Kaiapoi River experiences significant tidal effects, with high water levels largely driven by tidal tailwater.

Cam / Ruataniwha River

The Cam / Ruataniwha River is a perennial tributary to the Kaiapoi River (see above). Its 60km² catchment extends across the Canterbury Plains, encompassing the southern portion of Rangiora. The alignment crosses two drains Rossiters Drain and Wilsons Drain that discharge to the Cam River just south of the Cam River crossing.

The land use within this catchment is predominantly agricultural and large lot residential, however, it also incorporates suburban residential and commercial areas of Rangiora, including the Rangiora town centre.

Within the Project area, the Cam River experiences significant tidal effects, with high water levels largely driven by tidal tailwater.

Quarry Lakes

The quarry lakes are large excavations that are filled with groundwater and do not have a primary outlet. Water that is discharged into these ponds flows directly to groundwater through the gravel bottom and sides of the excavations. The land use within the quarry ponds catchment entirely consists of the quarried areas and the associated gravel processing operation.

McIntosh Drain



Figure 3-2 McIntosh Drain Near the Motorway Crossing

McIntosh Drain is a primarily manmade drain that runs north-south, parallel to the proposed motorway alignment, ultimately discharging into the Waimakariri River near its confluence with the Kaiapoi River. This drain has a catchment area of approximately 2.7km², which encompasses the southeastern portion of the Woodend township.

McIntosh Drain through the Project alignment can generally be described as a well-defined, steep sided, drain that perennially holds water.

The land use within the McIntosh Drain catchment is a mix of agricultural, large lot residential, and suburban residential land.

Waihora Stream



Figure 3-3 Waihora Stream is well defined in the upper reaches (left) but undefined in the lower reaches (right)

Waihora Stream is a distributary, or overflow channel, of the Taranaki Stream (see below). Waihora Stream flows south, parallel to the proposed roadway alignment before turning east south of Woodend Beach Road, discharging to the broad interdunal area west of Woodend Beach. This area ultimately discharges south through Kairaki Creek into the Waimakariri River near its mouth.

Surface flow into the upstream end of Waihora Stream is regulated by a weir along Taranaki Creek within the Ravenswood Subdivision and only flows in high flow events. Just downstream of the Taranaki Creek weir, Waihora Stream intercepts groundwater and the channel in this area perennially flows to the southeast, under SH1. Downstream of SH1, Waihora Stream passes through several ponds where all dry weather flow reinfilters. By the time the channel crosses Gladstone Road, 1700 metres to the southeast of SH1, Waihora Stream has become a broad, poorly defined, channel that only flows during flood flows.

Land use within the Waihora Stream catchment is almost entirely agricultural, however high flows diverted from Taranaki Creek include runoff from significant residential subdivision areas.

Taranaki Stream

Taranaki Stream is a well-defined, perennial waterway that has a catchment area of approximately 8 km². The catchment captures the easternmost portions of Rangiora, agricultural land, and the Ravenswood subdivision before crossing the existing SH1 alignment, draining to the east into the interdunal area, which ultimately discharges to the north to the Ashley River near its mouth.

3.1.2 Existing Flooding – Rainfall Runoff

A TUFLOW flood model has been developed to assess existing flood behaviour under a 100-year ARI event. The flood depth and level maps (Figure 3-4 and Figure 3-5) illustrate the primary flow paths within the Project area. Runoff is generally distributed in three directions: northward via the Taranaki Stream catchment toward Waikuku Beach, southward through the Waihora Creek catchment which then turns east toward Kairaki Creek, and through McIntosh Drain flowing south before merging with the Kaiapoi River.

The flood depth map highlights that overland flow is primarily confined to defined flow paths and low-lying areas, such as the quarry, basins within development zones, and upstream of Gladstone and Woodend Beach Roads, where culvert capacity and road embankments constrain flows. A broader floodplain extent is evident near McIntosh Drain at Fuller Road, where the channel is overtopped and floodwaters continue downstream.

Figure 3-5 and Figure 3-6 present the flood depth and level maps for the 200-year ARI event. The flood patterns are generally consistent with those observed in the 100-year ARI event, except with an increased flood depths and a broader extent of inundation across the floodplain.

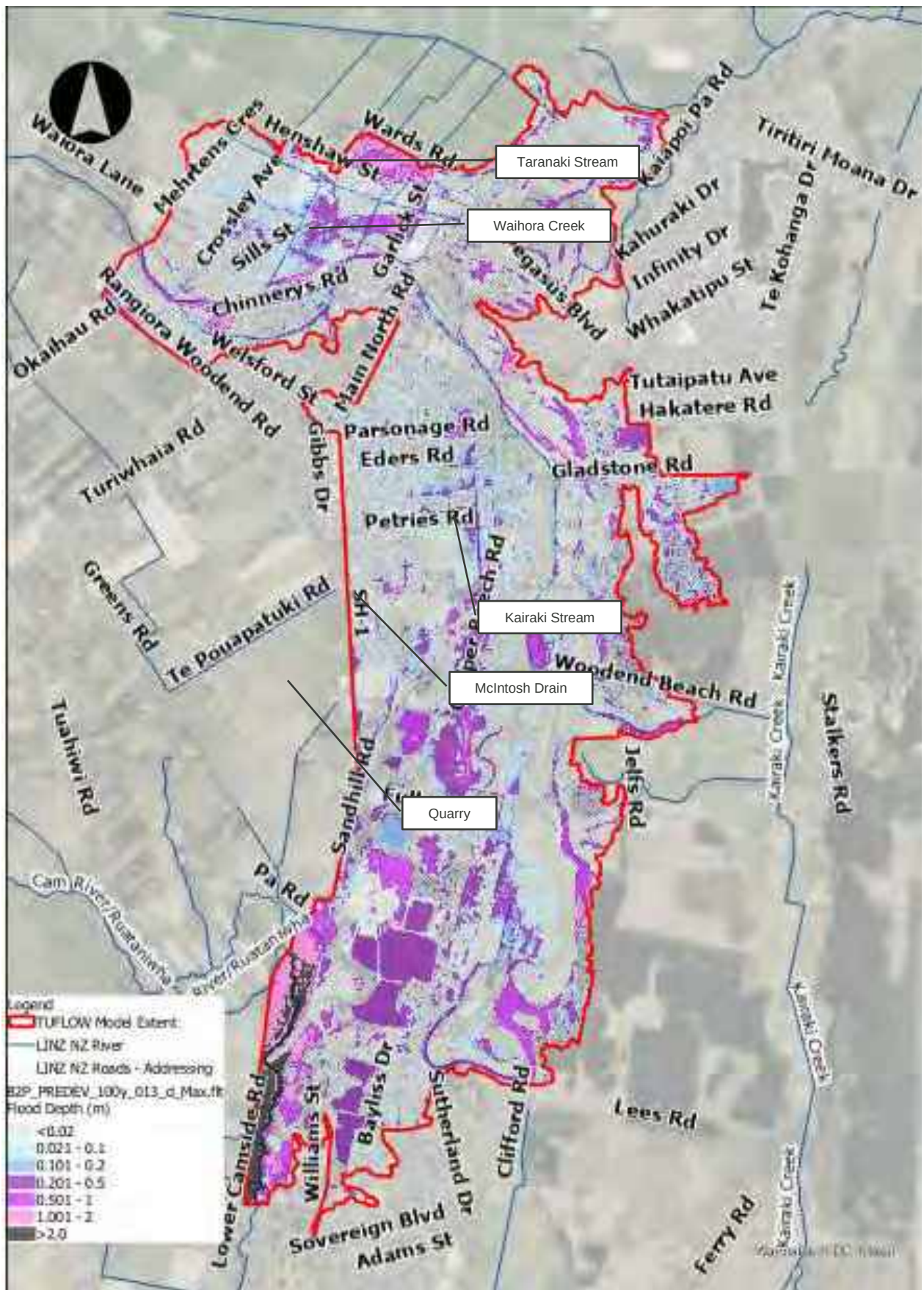


Figure 3-4 B2P-Woodend flood model pre-development flood depth under 100-year ARI event

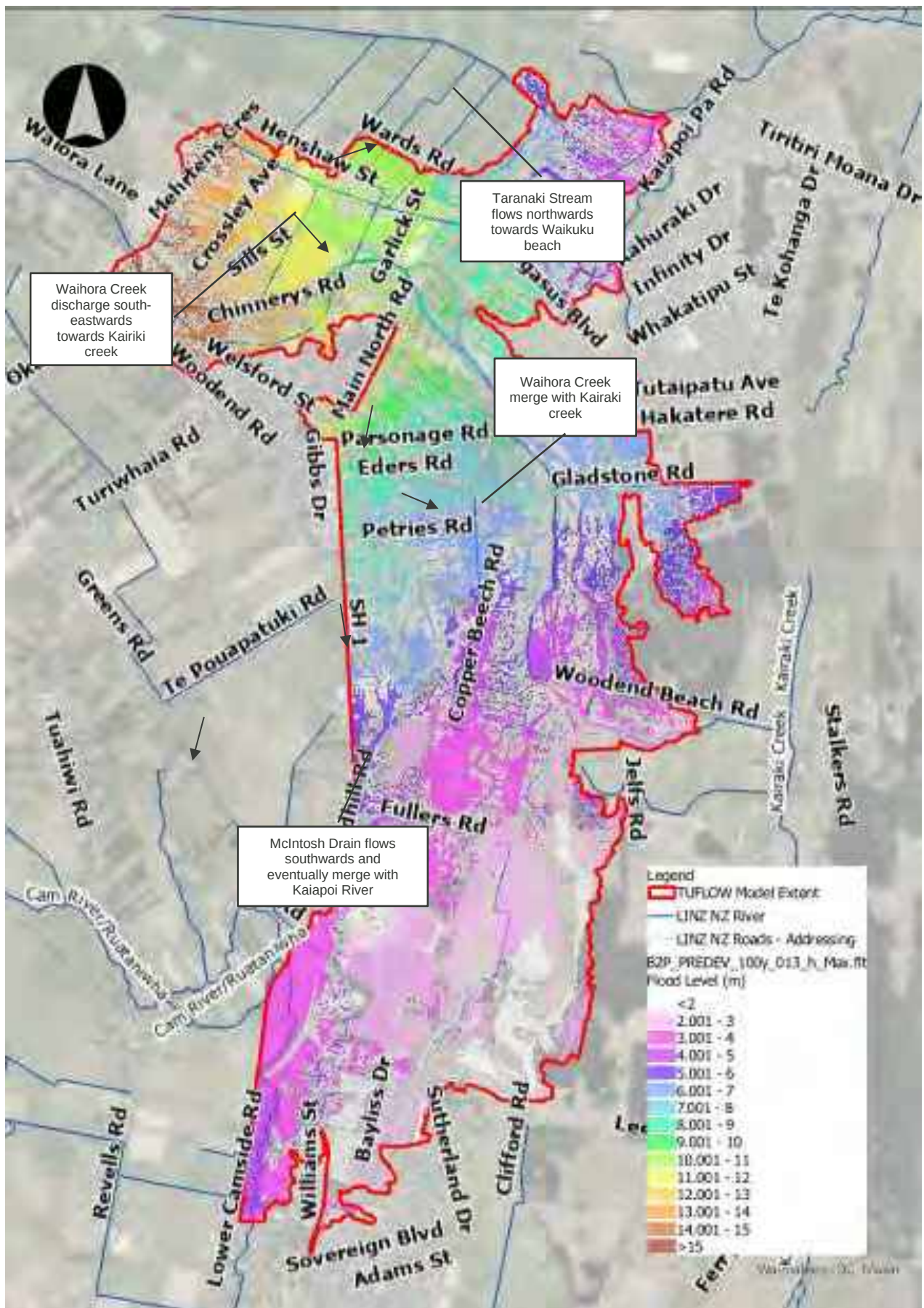


Figure 3-5 B2P-Woodend flood model pre-development flood level (mRL) under 100-year ARI event

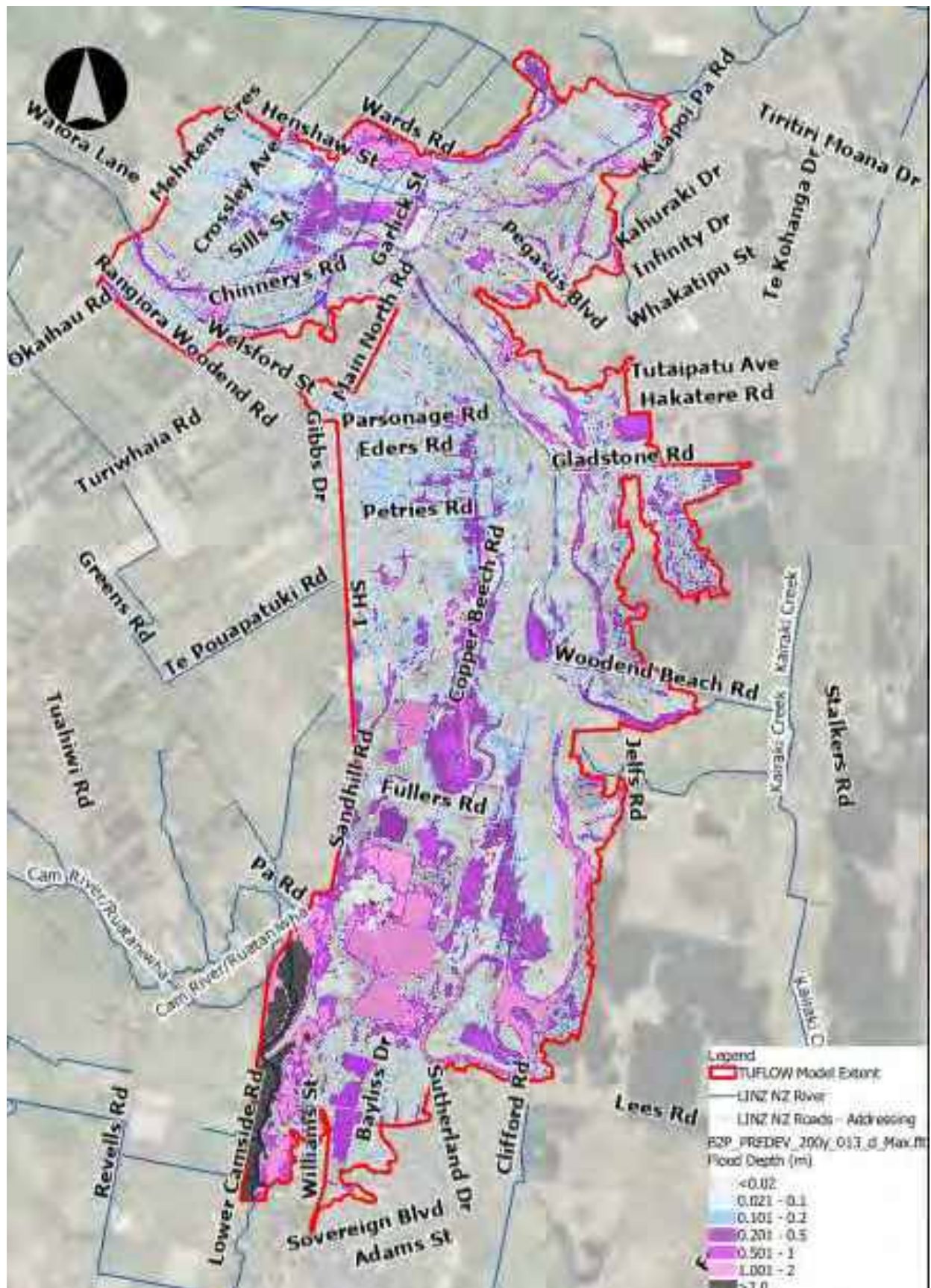


Figure 3-6 B2P-Woodend flood model pre-development flood depth (m) under 200-year ARI event

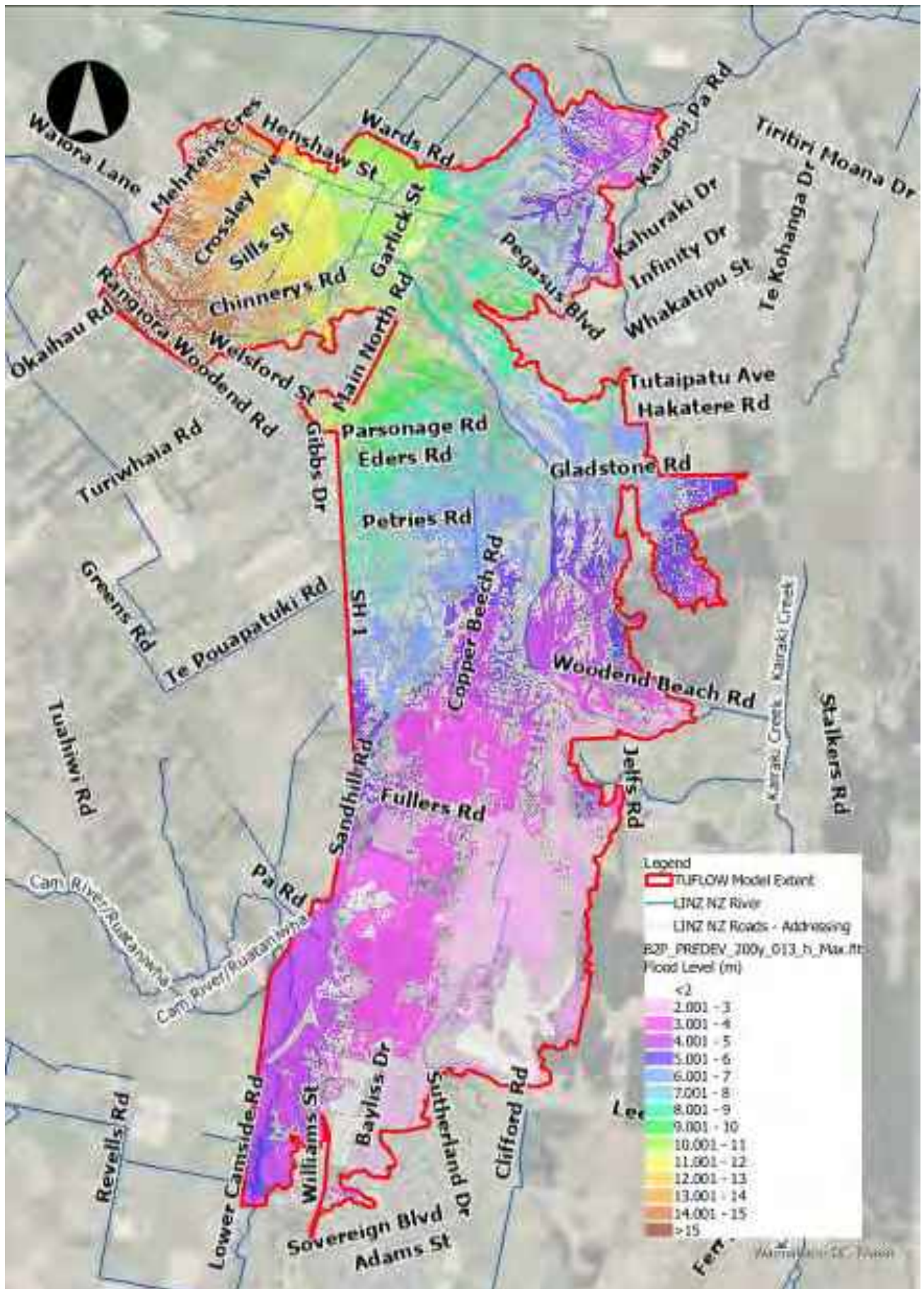


Figure 3-7 B2P-Woodend flood model pre-development flood level (mRL) under 200-year ARI event

3.2 Existing Roadway Stormwater Management

The existing roadway interfaces with the proposed motorway alignment on the north and south ends of the Project, with the central section widely deviating from the existing roadway. The stormwater management is discussed here for the portions of the existing roadway which are coincident with the proposed motorway works and where existing roads cross the proposed alignment.

3.2.1 Kaiapoi River Catchment (Kaiapoi Urban Area)

The limits of the Project begin within the Kaiapoi Urban Area near Ohoka Road. In this vicinity, only the southbound lane is affected by proposed works (a lane expansion) so the discussion below only discusses this side of the existing motorway.

South of the Kaiapoi River Bridge

Within the Kaiapoi urban area (between Ohoka Road and the Kaiapoi River), the Project interfaces with the southbound lanes of the existing motorway. In this area, the existing motorway drains to the east, over the embankment. For the southernmost 650 metres, roadway runoff flows directly to Parnhams Drain, an open drain that discharges via a stormwater pump station to the Kaiapoi River through a 1525mm diameter pipe.

For the northern 350 metres to the Kaiapoi River bridge, runoff travels over the embankment, likely entering the 1525mm pipe through catchpits and stormwater infrastructure. This area located behind private property and is inaccessible to confirm drainage patterns, however there have been no flooding complaints recorded, so this area likely has effective drainage infrastructure.

There do not appear to be any existing water quality or stormwater attenuation facilities in this reach.

Kaiapoi River Bridge Area

At the Kaiapoi River Bridge, flow from the bridge structure flows through deck drains directly to the underlying Kaiapoi River, rail corridor or Adderly Terrace.

For the first 100 metres north of the Kaiapoi River, runoff from the motorway runs down the embankment and joins a broad swale against the Kaiapoi River stopbank from where it is piped to the Kaiapoi River.

Although much of the flow from the Kaiapoi River Bridge discharges directly to the river, the swale that intercepts flow from the motorway north of the bridge is broad and flat, likely providing water quality improvement and flow attenuation prior to discharge to the Kaiapoi River.

3.2.2 Cam River Catchment

North of the Kaiapoi River Bridge, the roadway transitions into the Cam River catchment. From the Lineside Road area north, the existing roadway transitions to one lane in each direction and the proposed works impact both sides of the existing roadway.

Lineside Road Interchange

South of the Lineside Road overpass, flow from both mainline sides of the motorway drains to channels within the nose areas separating the mainline from the Lineside Road on and off ramps. These channels flow to the north where pipes on both sides of the motorway carry the flow beneath the Lineside Road overpass. These pipes, discharge into channels that parallel the motorway in the north side nose areas. Both channels flow through culverts that carry the flow under the northern on and off ramps, discharging into roadside channels that continue north.

There do not appear to be any formalised water quality or flow attenuation features to the existing stormwater system in this area. Most of the channels are relatively steep banked and narrow, limiting opportunities for both. However, the roadside channel in the northwestern quadrant, which collects runoff from the northbound lanes, is shallow, wide, and well grassed; potentially providing some attenuation and treatment.

Lineside Road to Cam River

Between Lineside Road and the Cam River, the existing roadway traverses agricultural land. The Cam River parallels the roadway to the east through this section. There are two major drains crossing the alignment, that discharge to the Cam River via 1500mm diameter culverts at CH1610 (Rossiters Drain) and 2350 (Wilsons Drain). Runoff from this portion of the roadway discharges into roadside drains, which discharge to the crossing drains.

The existing roadside drains are generally relatively steep sided, but longitudinally flat, likely providing limited water quality or stormwater attenuation benefit.

Cam River Bridge

As the roadway approaches the Cam River, it begins climbing. As a result, runoff from the entire roadway south of the Cam River is directed away from the main Cam River and into the drains described above. At the Cam River Bridge, runoff flows directly off the existing bridge into the river. North of the bridge, the roadway is superelevated to the northwest and runoff is directed into the Cam River, which parallels the west side of the roadway.

There do not appear to be any formalised stormwater treatment or attenuation facilities in this area.

Williams Street Intersection

Just north of the Cam River Bridge, the proposed motorway alignment deviates from the existing roadway alignment, veering to the northeast. The existing roadway between the Cam River and Williams Street is proposed to be utilised as an offramp for the motorway to Williams Street.

In this area, the existing roadway is superelevated with a single crossfall to the north, resulting in flow from the roadway discharging directly into the Cam River untreated and unattenuated.

3.2.3 Quarry Lakes Catchment

The existing SH1 roadway does not traverse the Quarry Lakes Area, passing to the east of it. As such, there are no existing roadway stormwater management systems in this catchment.

3.2.4 McIntosh Drain Catchment

The existing SH1 roadway does not cross McIntosh Drain, remaining well to the west of the drain. The motorway is proposed to cross McIntosh Drain approximately 1km to the east of the existing highway alignment in a location with no existing roadway infrastructure. In this area, the existing McIntosh Drain is steep sided and narrow, with minimal natural features.

3.2.5 Waihora Stream Catchment

As the proposed motorway traverses the Waihora Stream Catchment, it does not follow an existing roadway for the majority of the alignment. The only interactions with existing stormwater infrastructure are at the crossings of Woodend Beach Road, Gladstone Road, and at the northern end of the alignment, where it rejoins the existing SH1 alignment, near the Pegasus intersection.

Woodend Beach Road Area

The mainline alignment of the proposed motorway crosses Woodend Beach Road approximately 200 metres to the east of the location where Waihora Stream crosses beneath Woodend Beach Road. In this area, a channel on the north side of Woodend Beach Road collects flow from the poorly defined flowpaths that make up Waihora Stream, directing it to a 300mm culvert under Woodend Beach Road. There are no apparent water quality or stormwater attenuation features in this area.

Gladstone Road Area

The mainline alignment of the proposed motorway crosses Gladstone Road approximately 120 metres to the east of the location where Waihora Stream crosses under Gladstone Road via a 1250x500 mm box culvert. In this area, the channel upstream and downstream of the crossing is very poorly defined, consisting of a series of normally dry shallow overland flowpaths. There are no apparent water quality or stormwater attenuation features in this area.

Ravenswood Area

The proposed motorway rejoins the existing SH1 alignment at the location where Waihora Stream crosses the alignment. This location is 150 metres downstream of the location where Waihora Stream splits from Taranaki Stream, immediately intercepting spring flow.

The existing Waihora Stream culvert is a 4m x 2m box culvert under the SH1 roadway. Flow from the immediate vicinity of the Waihora culvert (approximately 200metres to the south and 100 metres to the north) discharges into roadside channels that discharge directly into Waihora Stream.

Approximately 185 metres north of the Waihora Road main crossing, a remnant Waihora branch intersects the roadway alignment. In this location, there is no culvert, so there is no split from the Taranaki Stream and all flow from the west side of the roadway is directed to the Taranaki Stream. A small portion of roadside channels on the east side of the roadway (approx. 75 metres) discharge into this branch.

There are no apparent existing water quality or stormwater attenuation features in this area.

3.2.6 Taranaki Stream Catchment

North of the Waihora Stream Crossing, the roadway enters the Taranaki stream catchment.

Pegasus Interchange

Taranaki Stream parallels the roadway to the west from the Waihora Stream Culvert to its crossing, approximately 120 metres north of the existing Pegasus roundabout. Runoff from the western side of the roadway is collected in kerb and channel, which discharges into a broad, shallow swale that parallels the roadway, discharging into Taranaki Stream just upstream of it.

On the eastern side of the roadway, runoff is collected in kerb and channel, and which is discharged directly into the golf course pond on the southeast quadrant of the intersection. This pond is the uppermost of a series of ponds throughout the Pegasus Golf Club that subsequently discharge into the Taranaki Stream.

The broad swale that collects and conveys the western side of the roadway appears to have been designed to provide swale based water quality treatment. There are no other apparent existing water quality or stormwater attenuation features treating road runoff in this area.

North of Pegasus

The final reach of the Project extends 700 metres north of the Pegasus intersection along the existing alignment of SH1. This reach is bisected by a branch of the Taranaki that passes under the roadway in a 650x850mm brick barrel culvert, 130 metres north of the main Taranaki culvert. The northbound portion of

the roadway drains to the west, where a roadside channel intercepts it, discharging into the Taranaki branch. The southbound portion of the roadway runs off across the grass verge directly onto the Pegasus Golf Club, ultimately discharging to the course's pond system..

4 Description of Works

This section is to be read in conjunction with the Stormwater Drainage Design Drawings

4.1 External Stormwater Management

4.1.1 Cut-off Drains

Where external catchments cross proposed motorway alignments, cut-off drains are proposed to intercept offsite inflow, keeping it separated from roadway runoff and conveying it to a discharge location. All cut-off drains have been designed to accommodate 1 % AEP (incl. Climate Change (CC)) flows resulting from their respective upstream catchments, based on flows from catchment flood modelling. Cutoff drains have been designed to ensure that trans-catchment diversions do not occur. Where offsite flow is redirected, it is redirected back into its historic outfall.

Throughout the Project area, cutoff drains are generally proposed at, or above the existing ground level. As a result, the majority of the cutoff drains are not expected to intercept groundwater. There are, however, several locations where proposed cutoff drains may intercept groundwater. These are located in the Lineside to Cam River section (CH 0580 to 3190) on both sides of the proposed motorway. In this section, the proposed work represents replacement in kind of existing channels and conveyance structures at similar levels and grades. As such, while the proposed drains may potentially intercept groundwater during times of high groundwater levels, the nature of the interception will not result in an increase in groundwater intercepted in comparison to the existing channels.

4.1.2 Channel Realignment

Several portions of existing channel are proposed to be realigned to accommodate the new roadway. The realignments are summarised in the table below.

Table 4-1 Proposed Channel Realignments

Project Chainage	Channel	Realignment Distance	Channel Type
9700 to 9775	Taranaki North Branch	75m	Ephemeral
9640 to 9700	Taranaki Stream	60m	Perennial
9100 to 9220	Waihora Stream	120m	Perennial (spring fed)
7820 to 8270	Waihora Stream	450m	Perennial at top, transitioning to Ephemeral
7000 to 7600	Waihora Stream	600m	Ephemeral (poorly defined)

Taranaki Realignment

At CH 9700, the existing SH1 Taranaki Stream culvert is being extended and realigned to accommodate the proposed motorway. This will require a realignment of approximately 60 metres Taranaki Stream, and 75 metres of the northern branch channel that intercepts Taranaki at the upstream end of the existing SH1 culvert.

Taranaki Stream in this reach flows perennially. To ensure that ecological benefits of the channel are not impacted by the works, the realigned channel will match the geometry of the existing channel in consideration of fish passage requirements for the Taranaki Culvert as discussed in the following section.

Waihora Stream Realignments

The Waihora Stream channel is being relocated in a number of sections to accommodate the work. At the upstream end (CH 9100 to 9220), where the channel flows perennially with spring fed water, the realigned channel will match the geometry and grade of the existing channel in consideration of fish passage requirements for the Waihora Stream Culvert

In the other reaches where Waihora Stream proposed to be realigned, the channel is poorly defined, shallow and does not carry a baseflow, only flowing during high flow events. Through this portion, the channel is designed so that in both the 100- and 200-year events, no significant adverse offsite impacts result from the proposed new roadway and channel realignment, as identified by flood modelling and outlined in the flood modelling report.

4.1.3 Cross Culverts

Abandonment/Removal of existing Cross Culverts

Within the Project Site, where culverts affected by works require decommissioning, they will be removed and/or backfilled.

Proposed Cross Culverts

Cross culverts have been sized based on NZTA criteria (P46, NZTA stormwater specification), which requires 100-year ARI capacity, considering RCP 6.0 to 2110. Additionally, the culverts have been designed in consideration of floodplain modelling in both the 100-year and 200-year ARIs to ensure that no adverse impacts occur, see flood modelling report in Appendix A. The following culverts on the alignment are proposed

Table 4-2 Proposed Cross Culverts

Culvert ID	Chainage (m)	Nominal Size (mm)	Length (m)	Notes	Fish Passage
001	1610	1500	45	Wilson Drain, Replacement of existing culvert	Y
002	2350	1500	45	Rossiter Drain, Replacement of existing culvert	Y
003	640 (ramp)	600	60	Bioretention basin outlet to Cam, new culvert, dry channel	N
004	350 (Williams St.)	600	25	New culvert, dry channel	N
005	5000	6000 X 2000	50	McIntosh Drain, new culvert, wet	Y
006	6310	600	30	New culvert, dry channel	N
007	6380	1500 x 500	40	New culvert, dry channel	N
008	540 (Woodend Beach Rd)	Twin 2000 x 1000	30	Waihora Stream, new culvert, dry channel	N
009	7400	600	30	New culvert, dry channel	N
010	320 (Gladstone Rd)	Twin 1050	55	Waihora Stream, new culvert, dry channel	N

Culvert ID	Chainage (m)	Nominal Size (mm)	Length (m)	Notes	Fish Passage
011	240 (Gladstone Rd)	600	55	New culvert, dry channel	N
012	7970	600	30	New culvert, dry channel	N
013	9110	600	65	Dry channel,	N
014	9210	4000 x 1500	70	Waihora Stream, wet	Y
015	100 (Bob Robertson Dr)	2 - . 1800 x 1000	5	Extension of ex Bob Robertson culverts, wet	Y (match existing)
016	9700	2700 x 1500	90	Taranaki Stream, wet	Y
017	9830	2000 x 1000	55	Taranaki Branch, existing 650x850mm brick barrel, dry	N

Fish Passage and Geomorphic Connectivity

Culverts identified as requiring accommodations for fish passage are identified above. Fish passage design for these culverts is based on New Zealand Fish Passage Guidelines v2.0 (Niwa, 2024) and Fish Passage Guidelines for State Highways (NZTA, 2013).

Each culvert is being individually assessed for fish passage accommodation, which generally includes the following accommodations:

- Inanga are present in all streams in the region, so will be used as the benchmark species for evaluating culvert water velocities.
- Brown Trout have been identified in Taranaki Stream and have had eDNA detected in the upper Waihora Stream, so will be used in addition to Inanga as a benchmark species for culvert water depth in these streams.

Following the fish passage guidelines will also ensure that the culverts maintain the geomorphic connectivity of the waterway, allowing the passage of sediment and debris.

Culvert Outfall Erosion Protection

All culverts have been assessed for erosion risk based on the guidelines in HEC 14 - Hydraulic Design of Energy Dissipators for Culverts and Channels. Where necessary, riprap erosion protection is proposed to ensure that inlet and outlet velocities do not result in channel scouring. Due to the low energy environment of the site, riprap will be sufficient to manage energy dissipation - extensive energy dissipation structures (e.g. plunge pools, baffle blocks, etc) will not be required.

4.2 Proposed Bridge Improvements

There are two bridges impacted by the Project, the Kaiapoi River Bridge and the Cam River Bridge. These are discussed in detail below. Both bridges were assessed for the SLS (100-year) events as well as the ULS (1500-year) event for hydraulic capacity, structural loading and bridge scour.

4.2.1 Kaiapoi River Bridge

The Kaiapoi River Bridge is located within the portion of the Project with a single southbound lane widening to the inside. This structure is being maintained with its current geometry, however the structure is proposed to undergo structural strengthening to bring it into compliance with current engineering standards and to allow it to support the additional lane as part of the early works.

Hydraulic Analysis

To support the structural design of the bridge a hydraulic analysis was undertaken. This analysis is based on a HEC-RAS river model, incorporating detailed bridge design elements.

The hydraulic analysis indicates that the river is confined within the existing stopbanks for the SLS event, however the stopbanks overtop in the ULS event. Regardless, the ULS flows will not affect the bridge deck.

Scour Analysis

As part of this strengthening, a scour analysis was undertaken to determine the extent of scour that would be expected to occur in the Serviceability Limit State (SLS) event (100-year ARI) and the Ultimate Limit State (1500 year ARI) events. The preliminary scour outcomes are shown below.

Table 4-3 Scour depth (m) by bridge component in the worst case SLS and ULS event scenarios

	Contraction	Pier 1 (Northmost)	Pier 2	Pier 3	Pier 4	Pier 5 (Southmost)	South Abutment	North Abutment
SLS	0.1	0	3.20	3.20	0	0	0	0
ULS	0.17	1.5	3.99	3.99	0.88	1.25	1.79	2.68

Although there is moderate expected pier scour in both the SLS and ULS events, the magnitude of this scour is within acceptable structural limits for both the events, based on the NZTA Bridge Manual criteria. Due to the stopbanks confining the flow in the SLS event, the bridge abutments are not predicted to scour in the SLS event, and scouring in the ULS event is not predicted to affect the bridge's stability.

Proposed Scour Protection

Based on the predicted this analysis, no additional scour protection is recommended for the Kaiapoi River Bridge.

4.2.2 Proposed Cam River Bridge

While the existing Cam River Bridge is being retained to support a northbound offramp, a new bridge is proposed to carry the proposed mainline and southbound onramp. This bridge will be located to the east of the existing bridge structures, which will share a common abutment fill slope.

Hydraulic Analysis

To support the structural design of the bridge a hydraulic analysis was undertaken. This analysis is based on a HEC-RAS river model, incorporating detailed bridge design elements.

The hydraulic analysis indicates that regardless of tailwater conditions, high water levels in the Cam River are dominated by backwater effects from the Kaiapoi River impoundment area as discussed above. The effect of this is that high flows from the Cam River do not result in the highest water levels at the Cam River Bridge, instead these levels occur when the Kaiapoi River system inundates the bridge. During these events, the flow and velocities through the Cam River Bridge become insignificant due to the 'bathtub' effect.

To understand the potential effects on the bridge, the model considered a Cam River only flood scenario, with low tailwater on the receiving Kaiapoi system. For both the SLS and ULS scenarios, these events resulted in lower water levels, but higher velocities.

Scour Analysis

The scour analysis for the Cam River Bridge was based on the Cam River only flood events, as the regional flood events created a 'bathtub' effect, reducing water velocities to near zero. The preliminary results of the scour modelling are summarised below.

Table 4-4 Scour depth (m) by bridge component in the worst case SLS and ULS event scenarios

	Contraction	South Abutment	North Abutment
SLS	1.15	1.89	3.53
ULS	2.24	1.89	4.88

The results indicate that significant abutment scour is expected to occur at the north (true left) abutment and that bend scour is expected at the south (true right) side of the channel in both the SLS and ULS events. While the proposed bridge will be structurally designed to accommodate this scour depth, both abutment slopes are at risk of failure in SLS events, requiring scour protection.

Proposed Scour Protection

Protection against scour induced abutment failure has been designed in accordance with HEC23, the US Federal Highway Administration's Bridge Scour and Instability Countermeasures Design Guide. Specifically, Design Guideline 14 was used in the preparation of preliminary abutment riprap revetments for the northern abutment, while Design Guideline 4 was used to remediate the bend scour at the southern abutment. Both proposed scour revetments will encompass both the existing and proposed bridges, allowing for increased resilience of the existing bridge structure.

Note, the data included in Table 4-5 are preliminary in nature and subject to reassessment upon finalisation of the bridge design, during future design stages. The final design will be undertaken in general accordance with the design guidance documentation referenced above and not envisaged to extend further in plan than identified in this application.

Table 4-5 Scour Protection Summary

Parameter Description	Design Sizing
Minimum D_{50}^1 (m)	0.3m
Minimum Revetment Slope Layer Thickness (m)	0.6m
Minimum Toe Layer Thickness (m)*	2.0*
Minimum Material Specific Gravity	2.65

4.3 Overview of Proposed Stormwater Quality Management Methodologies

A number of methodologies are proposed for stormwater treatment across the Project. These methodologies are outlined below.

¹ D50 is the median particle diameter or median particle size

Table 4-6 Treatment Methodology Overview

Type	Vegetation	Primary Discharge	Secondary Discharge	Subsoil drains
Grass Lined Swale	Grass	Surface	Surface	No
Bioinfiltration Swale	Planted	Passive Infiltration	Surface	No
Bioretention Swale	Planted	Surface	Surface	As needed by grade and location
Bioinfiltration Basin	Planted	Passive Infiltration	Surface	No
Bioretention Basin	Planted	Surface	Surface	As needed – Drain to stream
Proprietary Device (not part of current design)	N/A	Surface	Surface	May be included within device (e.g. proprietary raingardens)

4.3.1 Grass Lined Swale

Grass Lined Swales are intended to improve stormwater quality through sediment filtering within wide, grass bottom channels. Grass lined swales are generally utilised where there is sufficient roadside space to achieve the necessary width to accommodate the mowable grass batters and wide bases necessary to achieve the low velocities and extended residence times necessary to ensure the treatment process, and where infiltration based processes are either unnecessary or prohibitive. Where utilised on the Project, treatment swales will be designed as per the requirements in P46.

Grass Lined Swales are particularly effective at removing total suspended solids (TSS), with removal efficiencies often exceeding 80%. Swales can also remove heavy metals, especially where they are attached to sediment, however, have limited ability to remove dissolved heavy metals and nutrients.

4.3.2 Bioinfiltration Swale

Bioinfiltration Swales are channels that are intended to improve stormwater quality via bio- and phytoremediation processes within the soil layers of the base of the swale, before infiltrating the ground via high infiltration subsoils, maintaining the hydraulic neutrality by facilitating infiltration as close as possible to the point of origin. Bioinfiltration swales utilise check dams to impound water within the swales, allowing for infiltration through an engineered base material.

Bioinfiltration swales are proposed in locations where the stormwater management requires infiltration-based processes to achieve hydraulic neutrality within the catchment, as allowed by soil infiltration characteristics and groundwater levels

Bioinfiltration swales will be designed to infiltrate a minimum of 5mm/hour runoff from the roadway in accordance with Waimakariri District Council guidelines for infiltration treatment devices. During high inflows, when the hydraulic loading exceeds the ponding and infiltration capacity of the swales, runoff cascades downstream through the swale system, ultimately being discharged into surface waters.

Bioinfiltration swales exhibit very high TSS removal efficiencies and enhanced removal efficiencies of dissolved heavy metals and nutrients due to the biological treatment processes within the soil layers.

4.3.3 Bioretention Swale

Bioretention Swales are similar to bioinfiltration swales, however they do not rely on infiltration to achieve treatment. Bioretention swales are planted swales, with permeable check dams that retain water within vegetated segments. Treatment is achieved through settlement of suspended solids and, partially, soil

treatment within the vegetated topsoil layers. Bioretention swales are designed to provide impoundment volume for a first flush of up to 5mm/hr of runoff to ensure treatment outcomes. As bioretention swales are not intended to fully infiltrate treatment flows, they do not require an engineered base material to ensure infiltration rates, allowing water to slowly seep through their permeable check dams over time.

Bioretention swales exhibit high removal rates of TSS and attached heavy metals, however, have potentially limited dissolved heavy metal and nutrient treatment capability.

4.3.4 Bioinfiltration Basin

Bioinfiltration basins are basins that are intended to improve stormwater quality via bio- and phytoremediation processes within the soil layers of the bottom of the basins, before infiltrating the ground via high infiltration subsoils. Bioinfiltration basins are designed to impound the first flush of up to 25mm of runoff, allowing for slow infiltration and treatment through their vegetated, engineered base material.

Runoff in excess of the water quality volume is discharged via overflow weir to nearby surface waters.

4.3.5 Bioretention Basin

Similar to bioinfiltration basins, bioretention basins are intended to capture a first flush of up to 25mm of runoff, providing treatment through a combination of settlement and soil-based bio- and phytoremediation processes, slowly releasing water over 48-72 hours. Bioretention basins will be planted throughout in native vegetation to allow for uptake and treatment of contaminants.

4.3.6 Proprietary Device

Proprietary devices are currently not proposed on the Project but may be used if necessary to treat isolated areas. Depending on the proprietary device selected, the treatment efficacy can match or exceed the devices shown above.

4.4 Proposed Motorway Stormwater Drainage Overview

4.4.1 Kaiapoi River Catchment

South of the Kaiapoi River Bridge (CH -500 to 320)

South of the Kaiapoi River Bridge, within the Kaiapoi urban area, the proposed southbound widening to the central median will have a crossfall into the median. In this area, a grass treatment swale is proposed to collect and treat runoff from the added lane. This treatment swale will flow to an existing 600mm stormwater pipe that crosses the motorway alignment at chainage -380. This pipe discharges into Parhams Drain on the east side of the motorway, from where it is discharged through the pump station to the Kaiapoi River.

Kaiapoi River Bridge Area (CH 320 to 580)

At the Kaiapoi River Bridge, the proposed new lane will fall toward the existing roadway, where flow is discharged via deck drains directly to the underlying river, rail corridor of Adderly Terrace. Based on the existing bridge arrangement no water quality treatment is practical at this location, as the bridge cannot be easily retrofitted to capture and convey runoff to a location to treat it.

In the approximately 100 metres north of the bridge, the additional lane will continue to fall toward the existing SH1 southbound lanes, flowing over the embankment and where it will be collected by the existing stormwater network that parallels the Kaiapoi River stopbank. This system includes a broad swale that likely provides treatment prior to discharge to the Kaiapoi River.

4.4.2 Cam River Catchment

Lineside Road to Cam River (CH 0580 to 3190)

Through this section, the scope of the proposed improvements expands to include an additional lane in each direction from Ch 1400. Stormwater management in this reach consists of bioretention swales on both sides of the roadway that discharge into the Cam River branch Drains at CH1610 and 2350.

In addition to the treatment swales, cutoff drains will manage off site flows, to prevent intermingling offsite water with the roadway runoff.

Cam River Bridge and Williams Street Interchange (CH 3190 to 3620)

Between the Cam River Bridge and the William Street overpass, runoff from the mainline lanes and the southbound onramp will be collected in a stormwater network. This stormwater network will discharge into a proposed bioretention basin located in the triangle between the motorway mainline, the northbound offramp and Williams Street. Stormwater in this area will be treated by the bioretention facility prior to being released to the Cam River via a proposed pipe beneath the northbound offramp.

The northbound offramp itself will be treated by a treatment swale prior to being discharged to the Cam River.

Williams Street is proposed to be realigned to accommodate the new ramps. South of the intersection with the northbound offramp, runoff for Williams Street will be managed by a series of swales and catchpits that discharge into the bioretention basin.

North of the offramp, the flow will be collected in a treatment swale on the east side of the (superelevated) roadway before flowing beneath the roadway and being discharged into the Cam River via a cutoff drain.

4.4.3 Quarry Lakes Catchment

Quarry Lakes (CH 3620 to 4730)

North of the Williams Street overpass, the proposed roadway alignment enters the Quarry Lakes Catchment. For this catchment, stormwater treatment will be managed through 2 proposed bioinfiltration basins, located on the north side of the roadway, atop the quarry lakes hydraulic fill. This fill material provides a highly permeable connection to the lakes, allowing for installation of the passive groundwater infiltration-based treatment facility. For the initial 200 metres beyond the Williams Street Interchange, runoff will be collected within a stormwater reticulation which will discharge to the south side of the bioinfiltration basin. Within the remainder of this catchment, runoff is either discharged directly into the basin from the superelevated roadway, or via short swales.

4.4.4 McIntosh Drain Catchment

CH 4730 to 5800

After traversing the Quarry Lakes, the alignment enters the McIntosh Drain Catchment, crossing McIntosh Drain itself at CH 5000. The existing land use in this catchment entirely consists of undeveloped greenfield land, with agricultural use and very little existing stormwater infrastructure.

Throughout this entire reach, the proposed roadway is superelevated to the west, requiring stormwater management only on the west side of the roadway.

Runoff from the roadway is collected in proposed vegetated bioretention swales, providing treatment before discharge to McIntosh Drain on the west side of the roadway.

Proposed cutoff drains on both sides of the of the proposed roadway collect offsite flows, separating them from the roadway runoff and conveying them to McIntosh Drain.

4.4.5 Waihora Stream Catchment

From CH 5800, the roadway enters the Waihora Stream Catchment. This catchment is characterized by undeveloped greenfield land, with agricultural use and two major roadway crossings. As discussed above, the lower reaches of Waihora Stream consists of poorly defined broad channels, with soils with very high characteristic infiltration rates, while the upper portion consists of a spring fed perennial stream, which infiltrates via high porosity soil wetlands downstream of the existing roadway crossing.

CH 5800 to Woodend Beach Road (CH 6400)

From CH 5800, stormwater runoff from the roadway is directed to the north, to Waihora Stream. In this location, there is no superelevation, therefore bioinfiltration swales on each side of the roadway will collect runoff allowing for passive infiltration, treatment and attenuation. These swales discharge excess flow into the cutoff drain and its associated 600mm culvert at CH6300, on the south side of the proposed Woodend Beach Road overpass. This drain flows to the east, discharging into a Waihora Stream tributary channel approximately 100 metres upstream of Waihora Stream.

The Project includes a significant realignment of Woodend Beach Road to facilitate its overpass of the proposed motorway. Through this realignment, Woodend Beach Road is elevated above the ground levels on both sides as it ramps up and over the motorway. Stormwater on these embankments is collected in a stormwater reticulation on both sides of the overpass. On both sides, the reticulation ties into a bioinfiltration swale that runs along the embankments back to the motorway, ultimately discharging into Waihora Stream, which crosses Woodend Beach Road via the proposed 1500 x 750 mm culvert, approximately 200 metres east of the overpass.

Roadway runoff from the mainline north of Woodend Beach Road (up to CH 7100) flows in bioinfiltration swales on either side of the proposed motorway, which join Waihora Stream via channels and culverts on the north side of Woodend Beach Road.

Woodend Beach Road to Gladstone Road (CH 6400 to 7700)

From Woodend Beach Road north, runoff from the proposed motorway is directed into bioinfiltration swales on both sides of the roadway, discharging to the south as discussed in the previous section. At CH 7000, the roadway transitions into a superelevated section, falling to the west, so the eastern bioinfiltration swale is discontinued at this point. At CH 7100, the roadway passes a high point, thus the western bioinfiltration swale transitions to flowing north. Runoff from the western bioinfiltration swale between CH 7100 and 7770 (approx. 170 metres north of Gladstone Road) discharges to a proposed 600mm culvert beneath the mainline at CH7400. This culvert joins the proposed Waihora Stream diversion on the eastern side of the roadway, which ultimately rejoins the existing Waihora Stream alignment at CH 7000.

Gladstone Road is proposed to pass over the Motorway at Ch 7600. As Gladstone Road ramps over the motorway, runoff from the embankments on both sides is collected in proposed stormwater reticulations that ultimately discharge into bioinfiltration swales on the northern side of Gladstone Road, on both sides of the Motorway. On the western side of the motorway, this bioinfiltration swale joins the motorway's western bioinfiltration swale described above. On the eastern side of the motorway, the swale joins the proposed realignment of Waihora Stream, passing through a proposed culvert under Gladstone road approximately 50 metres east of the overpass.

Gladstone Road to Pegasus Interchange (CH 7700 to 9570)

Between Gladstone Road and CH 8400, the motorway remains superelevated, falling to the west. In this section, a proposed bioinfiltration swale on the west side of the road collects roadway runoff from the proposed culvert at CH 7980, which discharges into the realigned Waihora Stream on the east side of the

roadway. Immediately upstream (north) of Gladstone Road, this realigned channel traverses a historic landfill, that has been identified as a HAIL site for a distance of approximately 100 metres.

At CH 8400, the roadway transitions to superelevation, falling to the east. As a result, between Ch8400 and 8850, the roadway runoff is collected in a bioinfiltration swale on east side of the motorway that discharges into the beginning of the realigned section of Waihora Stream at CH 8270.

From CH 8850 to the culvert at CH 9100, the roadway remains superelevated to the east and there is a bioinfiltration swale flowing north, discharging into Waihora Stream.

Between CH 9100 and the Pegasus interchange (CH 9570), flow from the mainline and southbound Pegasus onramp discharge into a series of swales and stormwater reticulations, that are ultimately flow to a proposed bioretention basin located just northeast of the Waihora Stream Culvert at CH 9250.

4.4.6 Taranaki Stream Catchment

Pegasus Interchange Area (CH 9150 to 9830)

South of the Pegasus Interchange, the mainline of the proposed motorway and the southbound onramp both drain to Waihora Stream as described above. However, the northbound offramp flows to Taranaki Stream. Runoff from this offramp flows to a swale and stormwater pipe located between the offramp and mainline. This system discharges to the proposed bioretention basin, spanning both sides of Bob Robertson Drive (Pegasus Blvd) between CH 9500 and 9700.

This proposed bioretention basin also collects flow between from Pegasus Blvd beneath the overpass and the northbound onramp through channels and stormwater reticulation systems. This proposed bioretention basin discharges directly into the proposed Taranaki Stream Culvert at Ch 9700.

The proposed southbound offramp follows the alignment of the existing SH1 roadway, falling to the east , where runoff is collected in a series of short bioinfiltration swales that discharge into Taranaki Stream at CH 9700 and its tributary channel at CH 9830.

North of Pegasus Interchange (CH 9830 to 10300)

North of the Pegasus interchange the motorway transitions back into dual carriageway, tapering to match the existing SH1 geometry. This section has no superelevation, and stormwater is discharged to either side of the road. On the western side of the road, a proposed bioretention swale between CH 9830 and CH 10100 collects and treats runoff, discharging south, discharging to the Taranaki Branch culvert inlet at CH 9830.

On the eastern side of the road from the Taranaki Branch culvert north to the Project limits, the existing roadway runoff discharges to the golf course in a dispersed manner, with the golf course essentially forming a wide vegetated buffer, slowing and treating runoff before discharging into the golf course ponds. In order to maintain this dispersed drainage arrangement and to avoid concentrating flow, there are no swales proposed within this location. This is supported by the minimal additional impervious surfaces added at this location.

On the west side of the road from CH 10100 north, a proposed bioretention swale collects and treat roadway runoff, discharging to the existing SH1 roadside swales at the Project limits at CH 10300.

Garlick Street

The Project works include an extension of Garlick Street, adjacent to the Ravenswood subdivision, to tie into the existing SH1 alignment. This section of roadway will be collected by proposed kerb and channel, which will be discharged into the existing Ravenswood stormwater management system for treatment and attenuation.

5 Assessment of Effects

The following sections discuss the positive effects and the potential adverse impact of the Project works resulting from stormwater management.

5.1 Stormwater Quality

5.1.1 Potential Effects

Increased Surface Water Runoff Contaminant Loading

Stormwater runoff from roadways contains a wide range of contaminants introduced through both direct vehicular activity and atmospheric deposition. Among these, suspended solids, hydrocarbons, and heavy metals are of particular concern due to their prevalence, persistence, and environmental impact. Suspended solids often consist of fine particles such as silt, organic matter, tire wear debris, and dust from pavement abrasion. These solids can reduce water quality by increasing turbidity, transporting attached pollutants, and affecting aquatic habitat by smothering benthic organisms or altering stream geomorphology.

Hydrocarbons can degrade water quality through both direct and indirect contamination. When they enter aquatic environments, such as through oil spills or road runoff, they often form surface films that limit oxygen exchange, which can harm aquatic life. In addition, the breakdown of hydrocarbons by microbes can consume large amounts of dissolved oxygen, sometimes leading to low-oxygen or even oxygen-free conditions that threaten aquatic ecosystems.

Heavy metals, including zinc, copper, lead, and cadmium, are introduced through wear and corrosion of vehicle components (e.g., brake pads, tires, and engine parts), as well as from road surface degradation and atmospheric fallout. These metals can be toxic to aquatic life even at relatively low concentrations and have a tendency to accumulate in sediments, where they can pose long-term ecological risks.

When comparing stormwater runoff quality from the proposed motorway to the existing, primarily agricultural, land uses, the most pronounced increase in contaminant loading typically occurs in the concentrations of heavy metals and suspended solids. Agricultural runoff may already contain elevated levels of nutrients (e.g., nitrogen and phosphorus from fertilizers) and microbial contaminants (e.g. from livestock waste), but generally exhibits lower levels of heavy metals.

Increased Groundwater Contaminant Loading

Although there are no proposed formalised point soakage facilities (e.g. soakpits), throughout much of the Project area, stormwater discharges to ground during frequent low flow runoff events are expected, and part of the overall project's hydraulic neutrality philosophy.

Infiltration processes tend to favour the transport of dissolved pollutants over particulate-bound contaminants, as suspended solids and particulate-associated heavy metals are more likely to be retained in the upper soil profile or intercepted by vegetation and surface roughness. In contrast, dissolved species, including ionic forms of metals such as zinc, copper, and cadmium, are more mobile and can percolate through the vadose zone to reach the groundwater table. These dissolved heavy metals are of particular concern due to their potential toxicity, persistence, and ability to bioaccumulate in aquatic ecosystems if groundwater contributes to baseflow in nearby streams or wetlands.

There is a risk that the portion of the realigned Waihora Creek channel that traverses the historic landfill / HAIL site mobilises contaminants from the historic landfill. There are a series of options under consideration for the management of this site, which are discussed in detail in the Construction Methodology Statement. These options include partial or full removal of landfill materials depending on geotechnical suitability.

The risk of increased contaminant loading to groundwater is therefore an important consideration in the overall assessment and design of stormwater effects.

5.1.2 Proposed Mitigations

The primary mitigation for stormwater quality is the inclusion of stormwater treatment facilities. The Project proposes treatment for all runoff from motorway pavements within the Project extents, with limited exceptions, in accordance with the NZTA P46 stormwater specification and the NZTA stormwater treatment standard.

The proposed treatment devices are capable of removing a broad spectrum of contaminants including total suspended solids, heavy metals, hydrocarbons and nutrients. While it is noted that the efficacy of stormwater treatment devices varies widely depending on vegetation cover, contaminant loading, hydrological cycles, and seasons, the proposed devices have been selected to achieve the required NZTA treatment standards.

The general efficacy in terms of expected contaminant removal percentages of the various proposed stormwater treatment devices is shown below, these devices are discussed in detail in Section 4.3 of this AEE:

Table 5-1 Expected Treatment Efficacy Overview

Device	Total Suspended Solids (TSS)	Heavy Metals	Nutrients
Grass Lined Swale	70%	60%- 75%	20% - 30%
Bioinfiltration Swale	80%	70% - 80%	30% - 60%
Bioretention Swale	70%	60% - 75%	20% - 30%
Bioinfiltration Basin	80%	70%-80%	30% - 60%
Bioretention Basin	75%	40% - 50%	30% - 55%

Values taken from NZTA Stormwater Treatment Standard

Within the historic landfill / HAIL site north of Gladstone Road, the realigned Waihora Stream channel will be managed to preclude mobilising contaminants. Depending on the option selected for management of this site, this may include providing an impermeable liner within this portion of the channel to minimise infiltration to groundwater. Refer to the Groundwater Effects Assessment report and Contaminated Land Investigation for further information.

This analysis is generalised, covering the primary classes of contaminants present in roadway runoff and broad treatment efficiencies for various components within each class.

5.1.3 Summary and Residual Effects

Stormwater water quality treatment devices are to be installed in general accordance with NZTA P46 and NZTA stormwater treatment standard guidelines. Any future design refinements to the proposed water quality treatment devices to be in general accordance with the relevant guidelines and engineering practice. These devices provide means for promoting infiltration, settlement and the biological treatment of pollutants, all of which contribute to downstream water quality. The performance of the treatment devices will be supported through regular inspection and maintenance to ensure continued effectiveness in the long term.

Where the Project is coincident with the existing SH1 alignment (within the Cam River, Waihora Stream and Taranaki Stream catchments), there is expected to be a substantial Improvement of water quality discharging to the receiving environment. SH1 currently has no formal treatment devices within the project area and the Project proposes the installation of treatment devices to treat stormwater runoff from the vast

majority of proposed motorway impervious surfaces. Furthermore, the reduction in traffic volumes to the existing SH1 alignment north of Cam River will reduce contaminant inflows from the existing SH1 roadway.

Although the proposed treatment devices will remove a significant fraction of the contaminants generated by the proposed motorway, no treatment device is capable of full removal of all contaminants. The selected devices align with engineering practice and are intended to minimise the residual effects of contaminants on the receiving environment, reducing the effects to less than minor.

5.2 Flooding and Stormwater Quantity

5.2.1 Potential Effects

Localised Increase in Flood Depths in a Private Property

The Project proposes to place fill within flooding zones, install new bridges and culverts, realign channels and modify overland flow paths. Without appropriate design and mitigation, these activities can result in adverse impacts to private property adjacent to, and downstream of the work.

Increase in Stormwater Volumes Conveyed to Downstream Catchments

The Project also proposes to place a significant amount of new impervious area. This could result in an overall increase runoff resulting from additional impervious areas project wide resulting in an increase in the frequency, duration, and volume of runoff resulting from rainfall events.

5.2.2 Proposed Mitigations

Flood modelling has been undertaken to identify the potential impact of the proposed motorway on flooding in the vicinity of the Project. This flood modelling is intended to holistically quantify the effects of proposed earthworks, hydraulic structures, and added impervious areas. The effect of the Project on flooding outcomes adjacent to the site is documented in the flood modelling report, which is included in Appendix A.

The flood modelling analysis indicated flood levels increased in various locations due to the proposed road development, with most flooded zones confined to the vicinity of the proposed road corridor. Additionally, elevated flood levels were observed in the residential areas along Main North Road, Woodend Beach Road, and Wards Road during both the 100-year and 200-year ARI events.

Two primary measures, being the introduction of additional culverts and the diversion of flow path channels, are proposed and modelled to manage and mitigate the flood impacts. The mitigation model demonstrates that these measures effectively reduce flood impacts during both the 100-year and 200-year ARI events, with significant improvements at previously affected properties, excluding those identified for acquisition, which will be demolished.

Additional mitigation measures as discussed in Appendix A Section 2.5.1 are proposed to further reduce flood impacts, including adjustments to bypass channel levels at the north of Pegasus roundabout, upsizing culverts on Gladstone Road and Woodend Beach Road, and introducing check dams or detention basins to manage downstream effects. Overall, the current mitigation strategy demonstrates the Project's ability to meet flood management objectives.

Table 5-2 200-year ARI flood level and differences at buildings (Mitigation vs Pre-development scenario)

Point ID	Location	100-year ARI			200-year ARI		
		Pre-development flood level (m)	Post-development flood level (m)	Difference (mm)	Pre-development flood level (m)	Post-development flood level (m)	Difference (mm)
1	5 Wards Rd*	8.26	8.26	0	8.37	8.28	-90
2	196 Woodend Beach Rd	4.13	4.14	4	4.23	4.22	-13
3	1271 Main North Rd*	8.11	8.11	0	8.16	8.07	-83
4	1263 Main North Rd	8.29	8.29	0	8.29	8.29	0
5	1279 Main North Rd*	8	8	0	8.03	8.02	-10
6	1273 Main North Rd*	8.09	8.09	0	8.15	8.10	-50
7	1277 Main North Rd*	8.02	8.02	0	8.07	8.05	-20
8	1319 Main North Road	6.41	6.41	0	6.46	6.46	1

* Properties will be acquired as part of project work delineation. Flood impacts within designation have been considered acceptable.

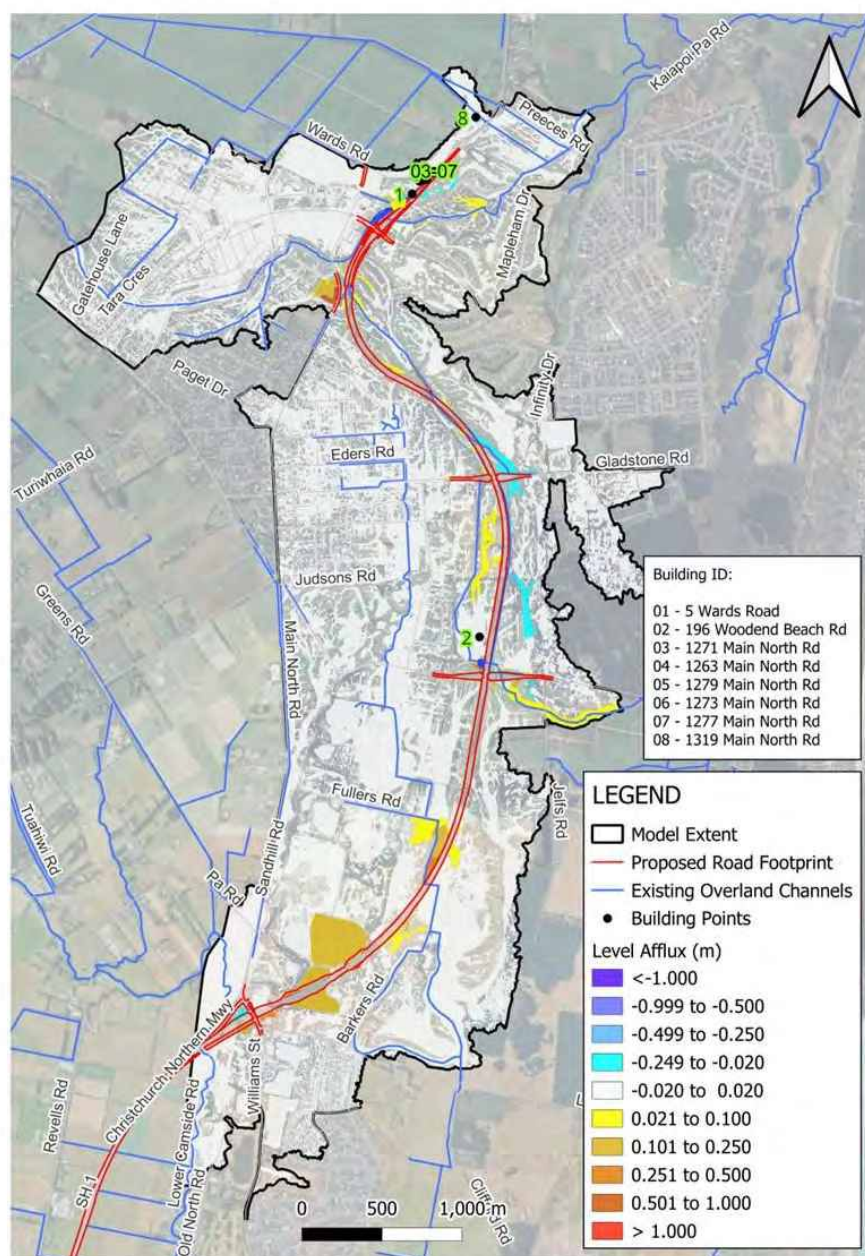


Figure 5-1 Private buildings affected under 200 year ARI (mitigated scenario)

Water level increases have been observed in the locations listed in Table 5-3 and as shown in Figure 5-2, which are outside the designation boundary. However, these increases do not affect any property buildings.

Table 5-3 Locations that observed a water level increase outside of designation boundaries

Locations	Max Afflux (mm)		Comments
	100-yr	200-yr	
1. Mapleham Dr	2	20	The increase in water depth is due to the upsizing of the Taranaki Stream culvert. However, the impacts remain within the channel and do not affect any buildings.

Locations	Max Afflux (mm)		Comments
	100-yr	200-yr	
2. Basin within Ravenswood residential area	200	670	Water depth increase is caused by raising the overflow weir from RL 10.20 m to RL 10.40 m to accommodate the proposed road (Garlick Street). The increased depth is confined within the basin and poses no risk to nearby properties.
3. Downstream of Waihora Creek	210	440	Higher water levels indicated in the model due to reduced roadway overtopping of Taranaki Stream north of Bob Robertson Drive, redirecting slightly more runoff into Waihora Creek. The impact is contained within the channel and does not affect any buildings.
4. West of Copper Beech Road	40	50	Water ponding is due to the motorway cutting off an existing overland flow path. The ponding is localised and does not impact any buildings.
5. Downstream of Woodend Beach Road	20	50	Flood levels within Waihora Creek rise slightly due to more efficient conveyance from the proposed bypass and both culverts at Gladstone Road and Woodend Beach Road have been upsized. No buildings are affected, and the impacts are confined within the channel.
6. Upstream of McIntosh Drain	130	280	For the upstream reach of McIntosh Drain, fully resolving the flood impacts remains challenging since maintaining the original flow area, spanning approximately 300 meters, will require significant structural measures (e.g. more culverts, bridges, etc.). This increase is largely confined to land owned by NZTA which does not include any residential properties.
7. Within the Quarry Lake.	0	120	Water level increases are shown within the model resulting from storage loss from the proposed bypass road infilling the lake. It is noted that the water level in the ponds generally reflects existing groundwater levels and the model assumes no infiltration. However, as the quarry bottom is directly connected to groundwater, flood afflux is expected to be minimal due to this continuous interchange with groundwater.

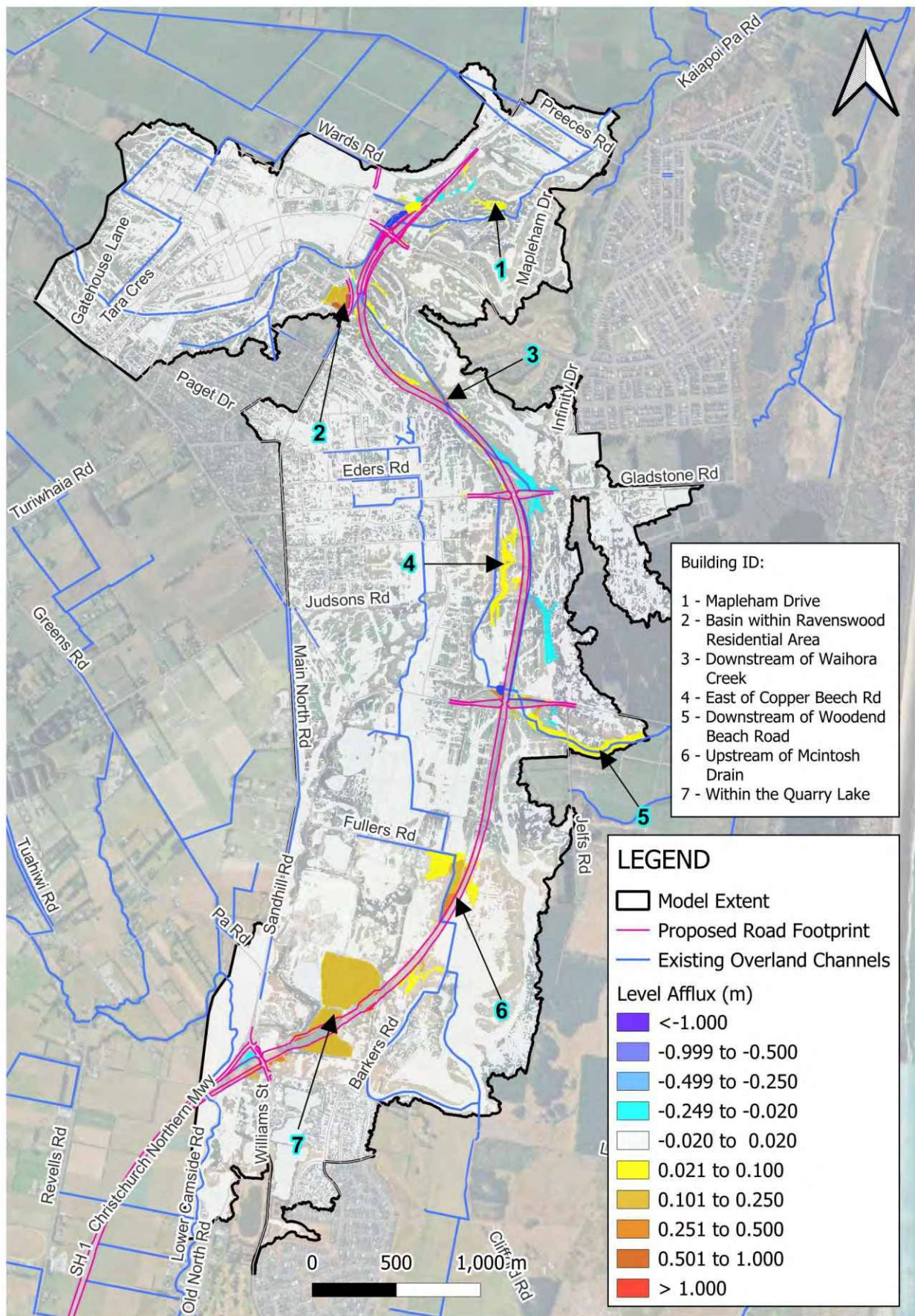


Figure 5-2 Residual impacts outside the designation but does not impact any property buildings (200 year ARI afflux level)

Within the Cam River catchment, between Lineside Road and the Cam River, the Project traverses a wide, shallow basin, that impounds water. The roadway in this section is sited below the modelled flood level. In this reach, the roadway improvements are limited to adding lanes, without increasing the existing roadway crest level. This ensures that floodwaters will move over the road in the same flow patterns as existing. As a result, the potential flood impacts of the work are related to displacement of flood storage within the impounded floodwaters. An assessment has been completed, considering the impact of the proposed roadway fill on the modelled flood levels. Based on this assessment, the calculated potential increase in flood levels within this impounded flood area is limited to 1.5mm to 3mm, considered a negligible effect.

The Project proposes the installation of swales that include features to slow and impound water, allowing for infiltration and peak flow attenuation prior to discharge to the receiving environment. In catchments where the existing hydrologic regime is dominated by infiltration (i.e. Quarry Ponds and Waihora Stream), distributed infiltration methods (i.e. bioinfiltration swales and basins) are utilised, allowing for infiltration to occur as close to the point of origin. These features will allow the proposed stormwater network to closely mimic the natural system's infiltration regime, minimising the impact of the Project. It is noted that this enhanced infiltration is not included in the flood modelling discussed above, and thus the results presented above are conservative.

5.2.3 Summary and Residual Effects

To mitigate potential adverse flooding impacts, the Project has proposed a best practicable option to mitigate the extents and depth of increased flooding on adjacent properties by undertaking an iterative flood modelling driven design approach to limit and mitigate adverse flood impacts. All impacts to buildings have been mitigated by the proposed flood management scheme. Where increases in flood levels are unavoidable, the increases are confined to existing stormwater conveyance channels and basins or are within NZTA property areas.

By incorporating swales and natural drainage features that promote primary infiltration as close as possible to the point of origin, the proposed system will mimic, to the greatest extent possible, the current hydrogeological response of the existing catchment.

Additionally, the Project proposes to increase the resilience of the overall network through replacement of existing cross culverts within the Cam River catchment and at the existing Waihora, Taranaki and Taranaki Branch culverts. This will reduce the risk of culvert failure, potentially causing adverse flooding impacts to the respective catchments.

While the increase in impervious area will locally result in increased volume of excess runoff, this has been considered in the Tuflow flood modelling, ensuring the flood carrying capacity of the waterways is not adversely affected.

Within the Cam River catchment, the proposed work consists of expanding the existing 2 lane roadway to 4 lanes and the addition of new access ramps to facilitate the Williams Street interchange. This work is expected to add approximately 3.8 ha of new impervious area to the catchment. This represents a very small percentage of the overall Cam River catchment of approximately 5,000 ha. Due to the location near the mouth of the Cam and Kaiapoi Rivers peak runoff from the site is not expected to coincide with the timing of peak flows given the extended time of concentration for these catchments.

Additionally, within the Williams Street interchange, a proposed water quality pond will have extended capture volume of approximately 2,000 m³, delaying discharge and attenuating flows from the Williams Street interchange area. Whilst this pond is not being designed to a specific attenuation outcome, it will reduce peak discharges from the most significantly developed portion of added impervious areas within the Cam River catchment.

While the design is optimised to balance the hydraulic and hydrologic effects of the work, there are unavoidable impacts resulting in impacts beyond the Project. These impacts, as documented above, are considered less than minor, resulting in a less than minor impact to the surrounding areas.

5.3 Erosion, Scour and Channel Stability

5.3.1 Potential Effects

Erosion Potential at Culvert Outfalls

There is a risk of erosion and scour at culvert and pipeline outlets during high flow events resulting from localised increased velocities, especially during the initial establishment phase. Concentrated discharges can mobilise fine sediments, undercut outlet structures, and create local depressions that may expand over time if left unmanaged.

Within the Project area, this risk is significantly reduced by the flat, low energy nature of the surroundings, where floodwater dispersal and high tailwaters typically result in floodwaters often not exceeding erosion thresholds.

Scour Potential at Bridges

Bridge scour can significantly contribute to channel erosion and instability by removing sediment from the vicinity of bridges and their approach embankments during high-flow events. This localized erosion alters flow patterns, concentrating energy at the base of manmade structures in the waterway and deepening the channel bed. As scour holes develop, they can undermine structural elements and change how water moves through the channel, often increasing flow velocity and turbulence. These changes can initiate further erosion both upstream and downstream, disrupting the channel's natural equilibrium.

Beyond the immediate vicinity of the bridge, scour can trigger broader geomorphic changes that compromise long-term channel stability. As sediment is removed and not replaced, the channel may begin to incise or shift laterally, leading to bank erosion, habitat degradation, and increased risk to nearby infrastructure. The constriction caused by bridge structures can intensify these effects by accelerating flow and altering sediment transport dynamics. Over time, these changes can set off a cycle of instability that is difficult to reverse without significant intervention.

Realignment or Modification of a Natural Watercourse

As discussed in the previous section, several reaches of the Waihora Stream and Taranaki Creek require realignment to facilitate the proposed motorway. The realignment of a natural watercourse can result in a range of potential adverse effects. It is noted that this report only considers effects related to the hydrologic and hydraulic regime of the watercourses – ecological outcomes are considered within the EclA report.

Changes to channel length, slope, and cross-sectional geometry can disrupt the natural flow regime, leading to either increased velocities and erosion potential or sediment deposition and reduced conveyance capacity. An over-steepened realigned section may cause downstream scour, while flatter sections may accumulate fine sediments, leading to channel instability.

Realigned channels may initially lack the bank stability of natural systems, especially if soils are disturbed and vegetation is not well-established. This can result in bank erosion, slumping, and increased sediment load downstream.

5.3.2 Proposed Mitigations

To mitigate against erosion at culvert and pipeline outlets, erosion protection at discharge locations is to be installed in general accordance with the HEC 14 design guidelines. These guidelines dictate the design and installation of energy dissipation structures, typically consisting of rock riprap at culvert outlets. Ongoing monitoring during the early operational period will help identify any emerging erosion features so they can be addressed promptly, ensuring the long-term stability of outlet areas and minimising sediment inputs to downstream environments.

Scour protection at the proposed Cam and Kaiapoi River bridges will be installed in general accordance with scour assessment and the abutment scour protection measures in accordance with NZTA Bridge Manual guidelines, including HEC 18 and abutment scour protection countermeasures design to be revised in accordance with relevant HEC23 design guidelines.

Where channels are realigned, the channel form will maintain the existing form in geometry and function, maintaining the overall channel geomorphic balance of the existing channels, ensuring long term channel stability. As vegetation is an important element in the long term stability of waterways, proposed planting will ensure that realigned channels are revegetated in a manner that ensures long term stability. Although the low energy of the surrounding environment will maintain low velocities throughout, in locations where hydraulic forces may be concentrated, such as at channel bends and confluences, geotechnical reinforcement linings will be provided to prevent scour and erosion before the establishment of vegetation. Riprap and rock linings are not proposed within any of the realigned channels except at bridge structures and culvert outlets.

5.3.3 Summary and Residual Effects

Overall, the risk of erosion, scour and channel instability throughout the Project area is low. This is primarily due to the low gradient nature of the catchments both upstream and downstream of the Project, and resulting low waterway velocities, and supported by the mitigations discussed above.

These measures will substantially reduce the potential for localised erosion, protect structural integrity, and improve resilience to flooding through strengthened bridge foundations and embankments. Some minor, localised erosion or sediment movement may still occur during extreme events, especially in the early establishment period; however, with ongoing monitoring and maintenance, residual effects on the erosion characteristics of the catchment are expected to be negligible.

6 Conclusion

The stormwater management and hydrological assessment for the State Highway 1 North Canterbury – Woodend Bypass Project demonstrates that the proposed infrastructure has been designed to appropriately manage the effects of proposed earthworks, increased impervious surfaces, altered flow paths, and hydraulic structures. A suite of stormwater treatment and conveyance measures, including swales, bioinfiltration and bioretention basins, and channel realignments, will ensure that the quality of runoff is managed in accordance with NZTA standards and that discharges are treated prior to entering receiving environments.

Flood modelling has confirmed that while the Project introduces changes to flow regimes, the proposed mitigation package, consisting of culverts, channel diversions, and localised storage, effectively addresses potential adverse flooding outcomes. Any residual increases in water levels are confined to existing drainage features or NZTA property and do not result in material effects on third-party assets.

Scour and erosion risks at culverts, bridges, and realigned watercourses have been addressed through targeted design measures, including riprap protection and engineered stabilisation. With these mitigations in place, the residual risk of erosion and scour is expected to be low, and long-term channel stability will be maintained.

Overall, the proposed stormwater and hydrological management framework for the Project represents the best practicable option, balancing the requirements of flood conveyance, water quality treatment, ecological considerations, and infrastructure resilience. With the recommended measures implemented and supported by appropriate monitoring and maintenance, the Project is expected to give rise to effects that are less than minor, while providing improved environmental outcomes relative to the existing SH1 corridor.

Appendices



Appendix A

Flood Modelling Report

Belfast to Pegasus Motorway & Woodend Bypass Pre-implementation and MSQA Professional Services

Flood Assessment Report

NZ Transport Agency (NZTA)

Document code: 11320-AUR-0350-PWI-SW-RPT-0001

Revision: A

Date prepared: 20 August 2025



Document control record

Document prepared by:

Aurecon New Zealand Limited

Level 2, Iwikau Building
93 Cambridge Terrace
Christchurch 8013
New Zealand

T +64 3 366 0821

F +64 3 379 6955

E christchurch@aurecongroup.com

W aurecongroup.com

Document control						aurecon
Report title		Flood Assessment Report				
Document code		11320-AUR-0350-PWI-SW-RPT-0001	Project number	11320		
Document state		PUB Published	Status code	P3 Detailed Design		
File path		https://aurecongroup.sharepoint.com/sites/528638/3_Develop/For Delivery/11320-AUR-0350-PWI-SW-RPT-0001.docx				
Rev	Date	Revision details	Author	Verifier (if required)	Approver	
A	2025-08-20	Prelim 30% design flood modelling	Y.Hooi T.De Leon E.Ragasa	S.Vaakesan	M.Flannery	
Current revision		A				

Client			
Client name	NZ Transport Agency (NZTA)	Client contact	Brendon French
Client document code	NA	Client document revision	NA

Approval			
Author signature		Approver signature	
Name	Yanni Hooi	Name	Matt Flannery
Title	Senior Water Engineer	Title	Major Projects Director

Contents

1	Introduction and Background	1
1.1	Background	1
1.2	Objectives and Scope of Assessment.....	2
1.3	Flood Model History.....	3
2	B2P Flood Model Development.....	5
2.1	B2P Pre-Development Model	6
2.1.1	Flood Model Validation	10
2.2	B2P Post-Development Flood Model	15
2.2.1	Post-Development Scenario Result Discussion	17
2.3	B2P Mitigation Flood Model	21
2.3.1	Mitigation Scenario Result Discussion	25
2.3.2	Flood Duration Impact Discussion.....	30
3	Limitation and Applicability	36
4	Conclusion.....	37

Appendices

Appendix A

B2P Flood Model Build Methodology

Appendix B

B2P Flood Model Modifications and Input Data

Appendix C

Woodend Flood Model Results

Figures

Figure 1-1 B2P Flood Model Extent with the Proposed Belfast to Pegasus Motorway and Woodend Bypass Road Alignment

Figure 1-2 South Ashley Model catchment extent

Figure 1-3 Model Extent of B2P - Woodend Flood Model

Figure 2-1 Principal River channels within the model catchment extent

Figure 2-2 Existing culverts crossing proposed road alignment

Figure 2-3 B2P Flood Model pre-development flood level under 100-year ARI

Figure 2-4 WDC South Ashley Model pre-development flood depth for 100-year ARI event

Figure 2-5 B2P-Woodend flood model pre-development flood depth for the 100-year ARI event

Figure 2-6 B2P Flood Pre-development model 100year ARI flood depth minus WDC South Ashley Model localised flood depth map under 100-year ARI event

Figure 2-7 B2P Flood Model ground level minus WDC South Ashley Model ground level

Figure 2-8 WDC Model Ground Level representing an existing 4m x 2m box culvert

Figure 2-9 Proposed B2P-Woodend design surface level

Figure 2-10 Floodwaters overtop the road at north of Bob Robertson Drive (200 year ARI Post-Development Scenario)

Figure 2-11 Impacted properties with 200yr afflux level map (post-development scenario) as background

Figure 2-12 Proposed mitigation scheme adjacent to the road alignment

Figure 2-13 Proposed culvert alignments (upper zone)

Figure 2-14 Proposed culvert alignments (upper central zone)

Figure 2-15 Proposed culvert alignments (central lower zone)

Figure 2-16 Proposed culvert alignments (lower zone)

Figure 2-17 Impacted buildings with 200yr afflux level map (Mitigation scenario) as background

Figure 2-18 Residual impacts outside the designation but does not impact any property buildings (200 year ARI afflux level)

Figure 2-19 Taranaki Stream 100-year Flood Duration Assessment

Figure 2-20 Taranaki Stream 200-year Flood Duration Assessment

Figure 2-21 Waihora Creek to Gladstone Road 100-year Flood Duration Assessment

Figure 2-22 Waihora Creek to Gladstone Road 200-year Flood Duration Assessment

Figure 2-23 Woodend Beach Road 100-year Flood Duration Assessment

Figure 2-24 Woodend Beach Road 200-year Flood Duration Assessment

Figure 2-25 McIntosh Drain 100-year Flood Duration Assessment

Figure 2-26 McIntosh Drain 200-year Flood Duration Assessment

Appendix A Figure 1 B2P Flood Model catchment delineation

Appendix A Figure 2 B2P Flood Model inflow and downstream boundaries overlaid on 100-year ARI flood depth result

Appendix A Figure 3 Boundary Conditions with Equivalent Inflow hydrographs

Appendix A Figure 4 Boundary Conditions with Equivalent Stage hydrographs

Appendix A Figure 5 Surface roughness applied in B2P Flood Models (Existing Scenario)

Appendix A Figure 6 Surface roughness applied in B2P Flood Models (Proposed Scenario)

Appendix A Figure 7 Surface roughness table extracted from 'Flood Hazard Models Update' report

Appendix A Figure 8 Existing culverts within the proposed project site

Appendix B Figure 1 Model Setup for the pre-development scenario

Appendix B Figure 2 Model Setup for the post-development scenario

Appendix B Figure 3 Model Setup for the post-development with mitigation scenario

Appendix B Figure 1 Model Setup for the pre-development scenario

Appendix B Figure 2 Model Setup for the post-development scenario

Appendix B Figure 3 Model Setup for the post-development with mitigation scenario

Tables

Table 2-1 Model scenarios and purpose

Table 2-2 Locations where large discrepancies are identified

Table 2-3 100-year and 200-year ARI Flood Levels and Differences (Post-Development vs. Pre-Development Scenario)

Table 2-4 100-year and 200-year ARI flood level and differences (Mitigation vs Pre-development scenario)

Table 2-5 Locations that observed a water level increase outside of designation boundaries

Appendix A Table 1 Comparison of WDC South Ashley Model and B2P Flood Model build

Appendix A Table 2 Water Level Downstream Boundary Conditions

Appendix A Table 3 Normal Depth Downstream Boundary Conditions

Appendix A Table 4 Horton decay rate extracted from WWDC

Appendix A Table 5 Modelled Culvert Inventory

Appendix A Table 6 List of modelled scenarios

Disclaimer

This report (including any enclosures, data and attachments) has been prepared for the exclusive use and benefit of our client, solely for the purpose for which it is provided, and on the terms and conditions agreed with our client. Unless we provide express prior written consent, no part of this report should be reproduced, distributed or communicated to any third party. To the extent permissible by law, we do not accept any liability if this report is used or relied on by any unauthorised third party, or by the client for any unauthorised purpose, in any other contexts or without checking it is the latest revision.

1 Introduction and Background

1.1 Background

Waka Kotahi has engaged Aurecon to support the resource consent applications in the proposed Belfast to Pegasus Motorway and Woodend Bypass. In line with this project, a flood assessment has been conducted to evaluate the impact of a 100-year and 200-year Average Recurrence Interval (ARI) storm event on the proposed road development.

For the purposes of the current assessment and to optimise the computation time, a cut-down catchment model tailored to the Project Site and adjacent surroundings as shown in Figure 1-1 has been built using TUFLOW modelling software. The model will be referred to as the “B2P Flood Model” throughout the rest of this report.

The purpose of this report is to outline the following:

- The methodology for building the B2P Flood Model
- A comparison of flood impacts before and after the road development



Figure 1-1 B2P Flood Model Extent with the Proposed Belfast to Pegasus Motorway and Woodend Bypass Road Alignment

1.2 Objectives and Scope of Assessment

The objectives of the current assessment include developing the B2P Flood Model to analyse flood impacts before and after development.

Based on local authority advice, the following objectives have been established for this project:

- No significant increase in flood levels at neighbouring dwellings for the 100-year ARI event.
- Any increase in flood levels at neighbouring dwellings for the 200-year ARI event should be limited to 20 mm.

- Reduce the risk of ponding/runoff buildup that will be brought about by the barrier effect of the proposed road development on adjacent surroundings. Mitigation measures will consist of provision of additional culverts across the road alignment, improvement/rehabilitation of existing culvert structures, and provision of additional channels to improve flow of runoff.

1.3 Flood Model History

Figure 1-2 shows the model extent of the Waimakariri District Council (WDC) South Ashley Flood Model. This flood model was developed in 2019 by DHI using MIKE by DHI modelling software. Detailed information with regard to model build is documented in the May 2020 “Flood Hazard Models Update” report¹.

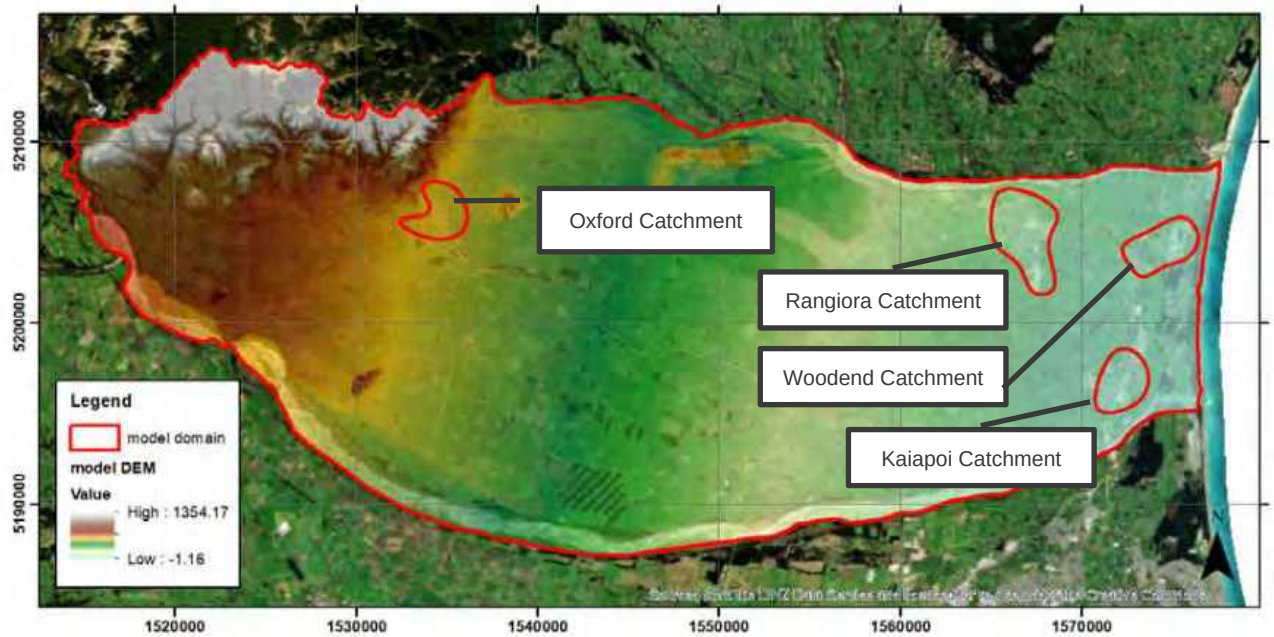


Figure 1-2 South Ashley Model catchment extent²

In this study, the B2P Flood Model has been created for the Project Site by utilising TUFLOW modelling software (Figure 1-3), using the WDC South Ashley Model as a reference.

¹ DHI (May 2020). *Flood Hazard Models Update*.

² Extracted from Flood Hazard Models Update Report.

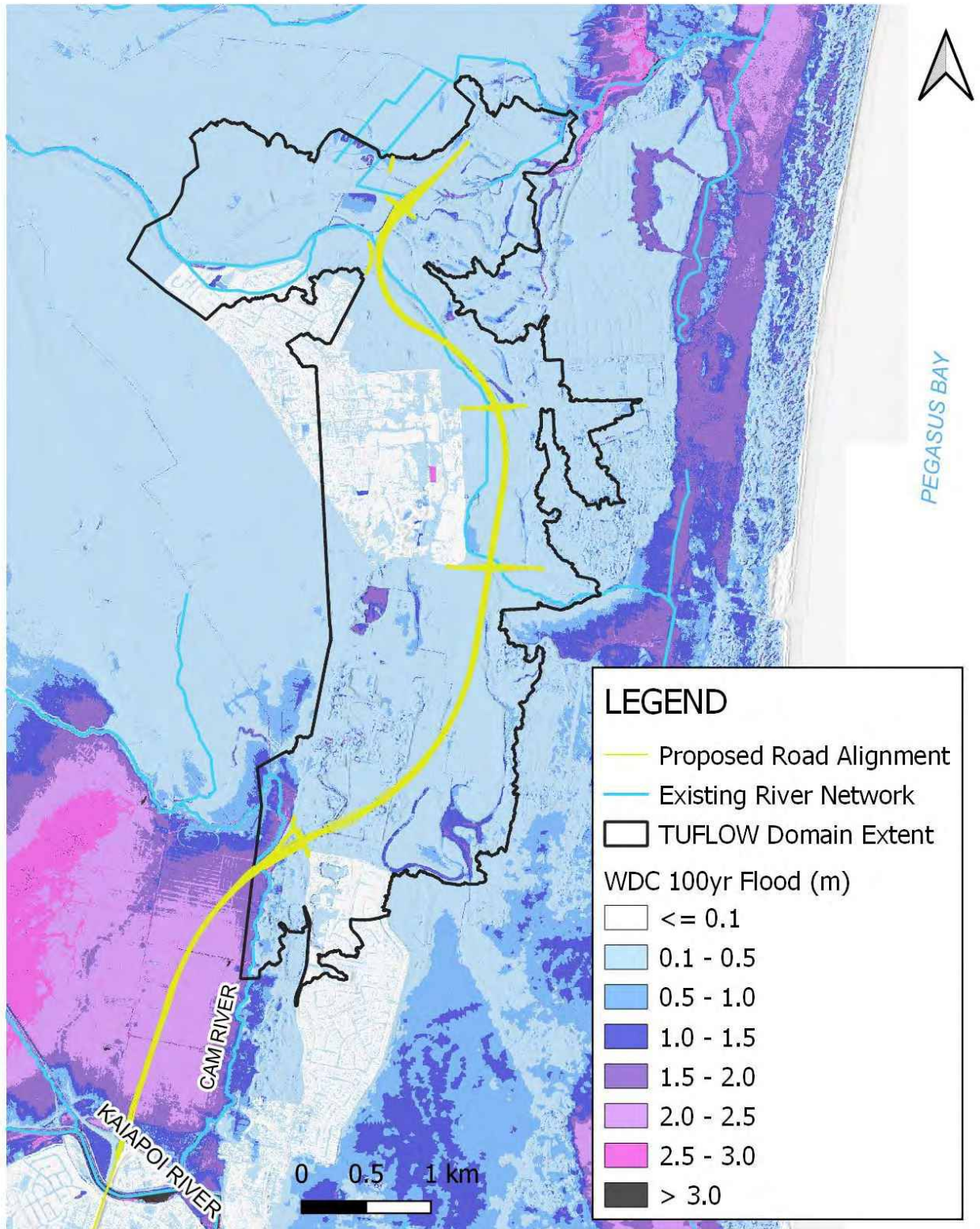


Figure 1-3 Model Extent of B2P - Woodend Flood Model

2 B2P Flood Model Development

The B2P Flood Model extent is a cut-down catchment model tailored to the Project Site and has been built using TUFLOW modelling software. Three modelling scenarios, the Pre-development, Post-development, and Mitigation scenarios were developed and assessed.

The purpose of each scenario is listed in Table 2-1. Further details of these scenarios are discussed in Appendix A. Each scenario has been run for the 100-year and 200-year ARI storm events.

Table 2-1 Model scenarios and purpose

Model Scenario	Scenario Case	Purpose
PREDEV	Existing/pre-development scenario	To understand the existing flooding conditions. This will be the baseline model for the flood impact assessment.
POSTDEV	Design/post-development scenario with Bypass Motorway included	To understand the flood impacts due to Bypass Motorway.
MITIG	Design/post-development scenario with mitigation solution included	To resolve flood impacts identified under the design scenario.

The B2P Flood Model terrain has been updated to incorporate the 2020 LiDAR Digital Elevation Model (DEM) and available surveyed data. The hydrological and hydraulic input data such as rainfall, ground infiltration, and surface roughness are extracted from the WDC South Ashley Model. The detailed model build methodology is described in Appendix A.

Note that for vertical datum, as described in Appendix A, the original flood model was carried out in Lyttleton Vertical Datum (LVD), whereas the B2P Flood Model is in New Zealand Vertical Datum 2016 (NZVD16). Levels within this memorandum are quoted in NZVD16 unless otherwise stated.

The B2P Flood Model is mainly built as a two-dimensional (2D) model, integrating open channels into the DEM, with culverts represented using a one-dimensional (1D) model. Pipe reticulation has been excluded from the B2P Flood Model, in compliance with the WDC requirements regarding the modelling of full pipe blockages.

In the B2P Flood Model, three principal river channels have been identified, as shown in Figure 2-1: the Taranaki Stream, located to the north of Bob Robertson Drive; the Waihora Creek, situated near Garlick Street; and the McIntosh Drain, which lies between Quarry Lake and Woodend Beach Road. The Taranaki Stream flows northward towards Waikuku Beach, while the Waihora Creek initially flows south, then turns east towards Kairaki Creek, close to Woodend Beach Road. The McIntosh Drain also flows south and ultimately merges with the Kaiapoi River.

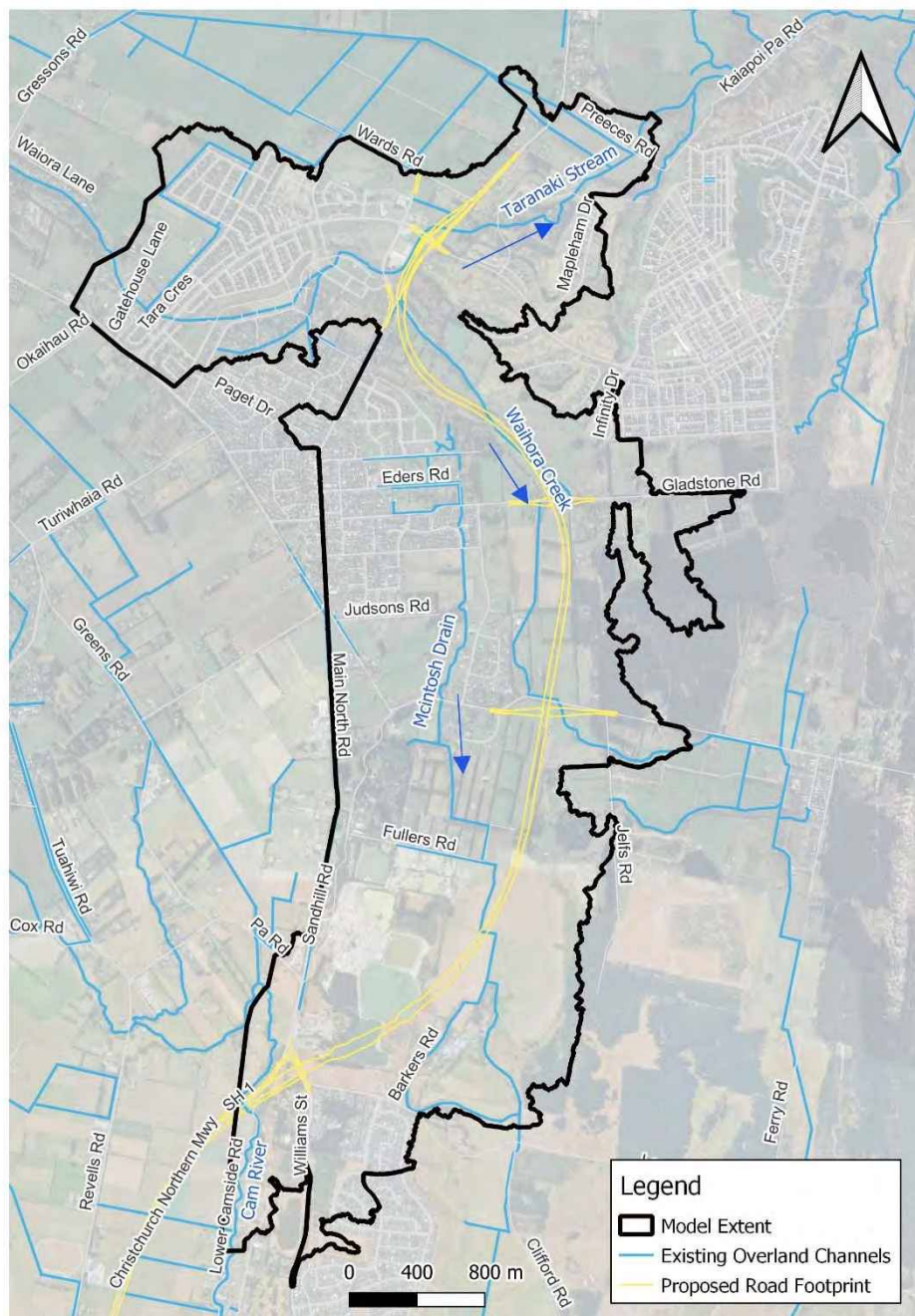


Figure 2-1 Principal River channels within the model catchment extent

A detailed account of the model development methodology is provided in Appendix A, while the updates representing various post-development and mitigation scenarios can be found in Appendix B.

2.1 B2P Pre-Development Model

The detailed model build methodology for B2P Flood Model pre-development scenario (PREDEV) is described in Appendix A Table 1.

Figure C-1 and Figure C-2 (refer to Appendix C) show the maximum flood depth and flood level of the pre-development scenario under the 100-year ARI storm event, Figures C-3 and C-4 present the maximum flood depth and flood level under the 200-year ARI storm event.

The pre-development model comprises of six existing culverts, that are crossing the project site as shown in Figure 2-2. This includes an irregular twin DN1800 culvert³ on Bob Robertson Drive, a 2.2m x 1.2m box culvert³, and a DN750⁴ circular culvert that cross State Highway 1 (SH1) discharging into the Taranaki Stream, a 4m x 1.5m box culvert³ crossing SH1 that discharges into Waihora Creek, and a 1.25m x 0.5m box culvert located on Gladstone Road and a DN600 circular pipe situated on Woodend Beach Road.

Survey shows that the existing irregular twin DN1800 was embedded by 300mm as part of the fish passage consideration, which will be maintained in the Post-Development and Mitigation scenarios. Ecological assessments have also found that fish are present along the major streams that are affected by the proposed roadway, namely McIntosh Drain, Taranaki, and Waihora Streams. Accessibility for fish movement will be preserved along these streams and will be considered in the proposed mitigation measures.

The Stormwater Network and Culverts inventory, as discussed in Appendix A, details all the culverts that have integrated the passage of fish, in all three case scenarios. Only the effective flow opening area of these culverts have been modelled, with higher surface roughness to incorporate the substrate infill within the culvert.

³ To simplify the modelling process, culverts with fish passage considerations are modelled using only their effective flow opening area.

⁴ Existing 0.85 (W) x 0.65 (H) arch culvert. For modelling simplicity, culvert modelled as DN750 circular culvert.

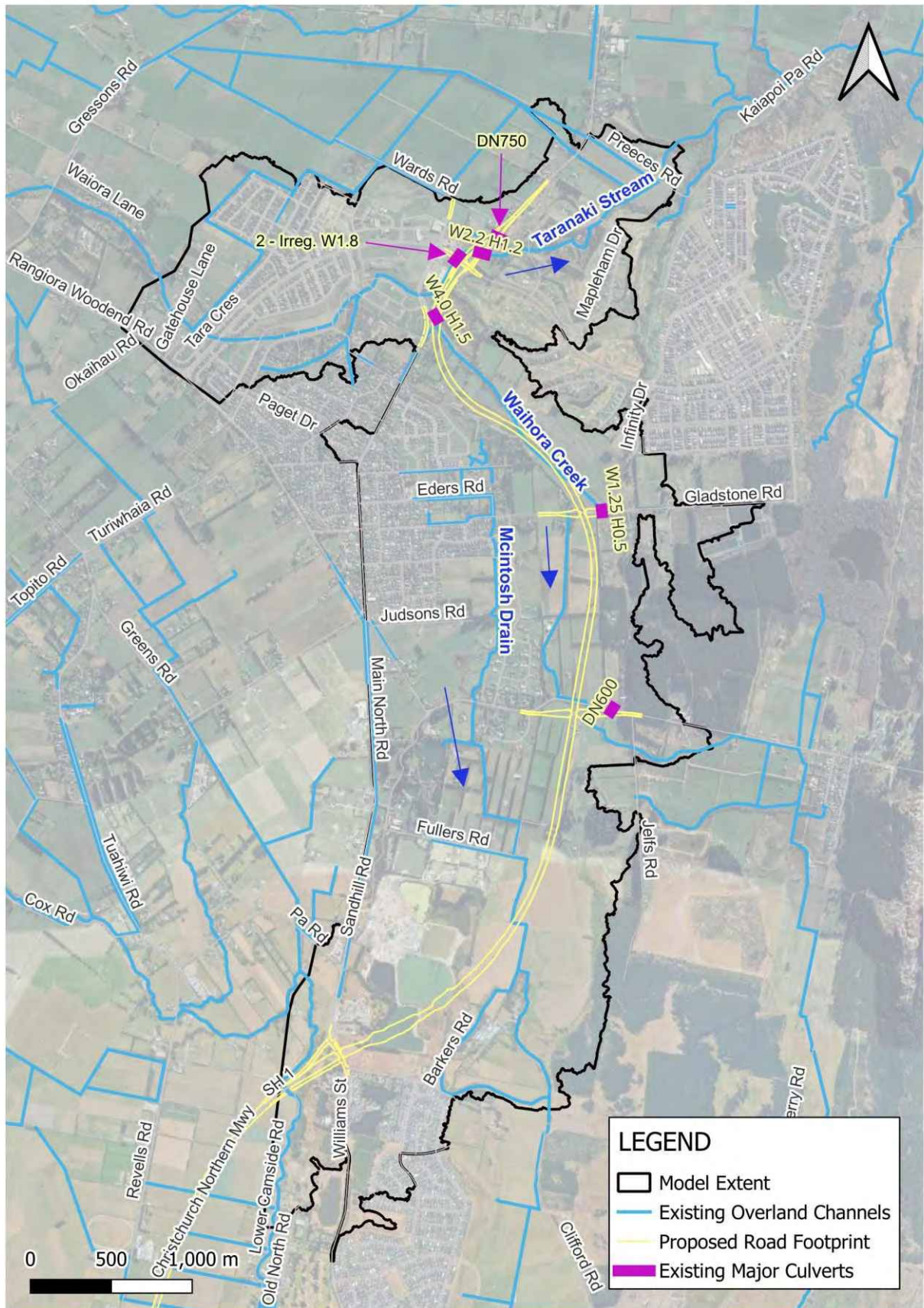


Figure 2-2 Existing culverts crossing proposed road alignment

As shown in Figure 2-3, the runoff splits into three directions:

- The Taranaki Stream catchment flow northward towards Waikuku Beach
- The Waihora Creek catchment flow towards the south and turns east towards the Kairaki Creek
- The McIntosh Drain flow south and ultimately merges with the Kaiapoi River.

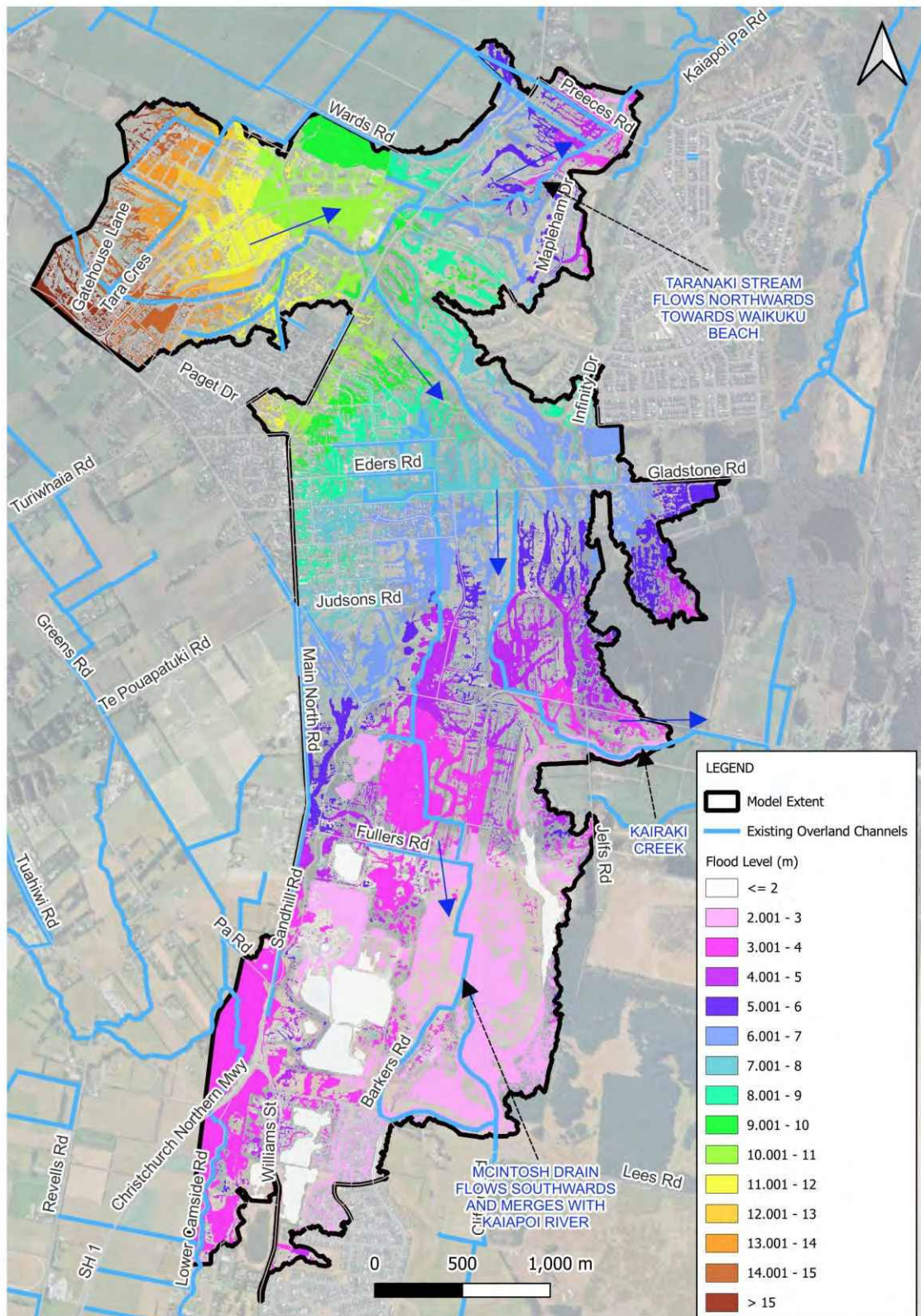


Figure 2-3 B2P Flood Model pre-development flood level under 100-year ARI

2.1.1 Flood Model Validation

Figure 2-4 shows the maximum predicted flood depths for the Existing Scenario in the WDC South Ashley Model for the 100-year ARI event. In comparison with the maximum predicted flood depths of the B2P Flood Model (refer to Figure 2-5), it can be observed that the flood extents are relatively similar, except for a few localised areas which are discussed in Table 2-2.

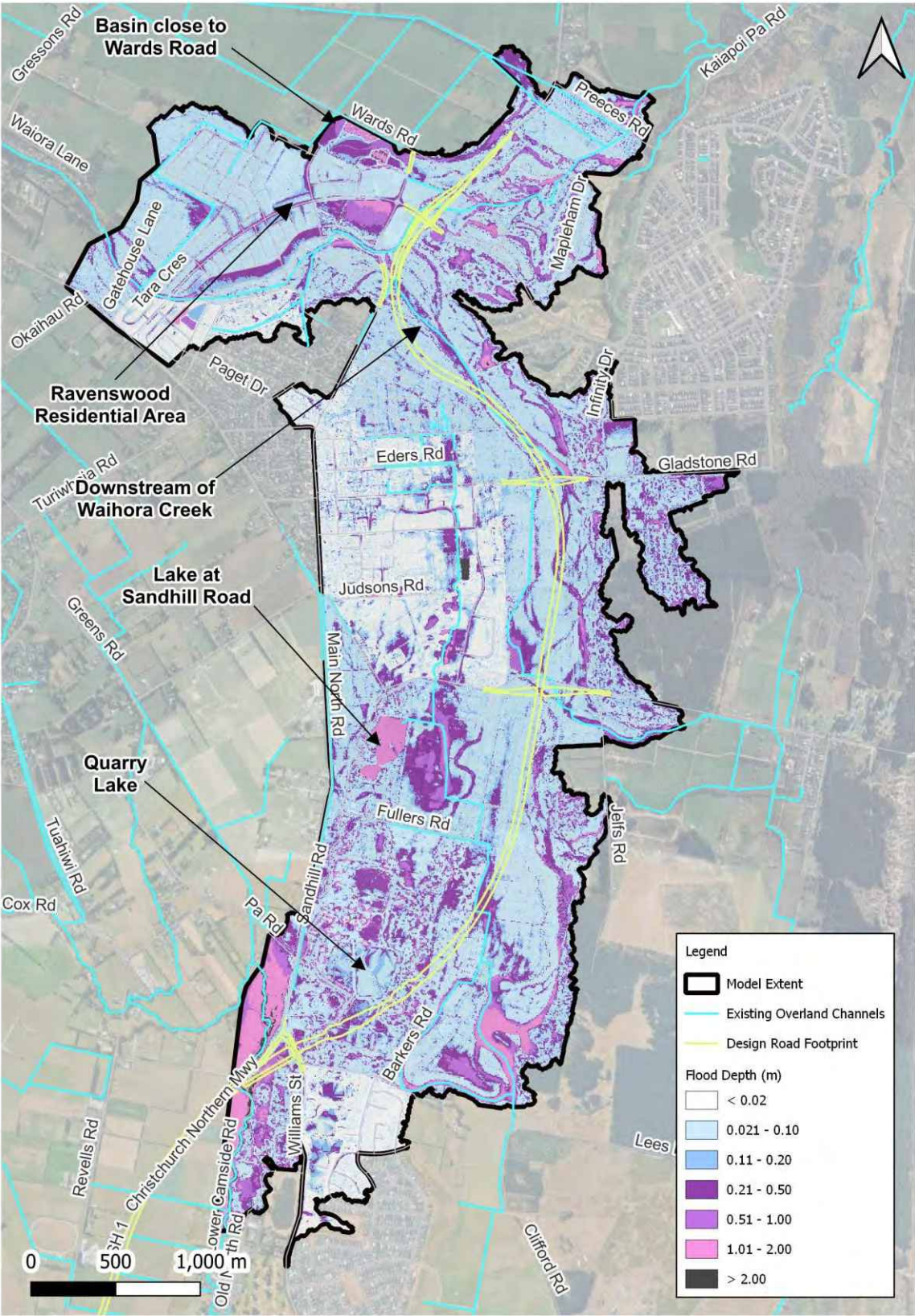


Figure 2-4 WDC South Ashley Model pre-development flood depth for 100-year ARI event

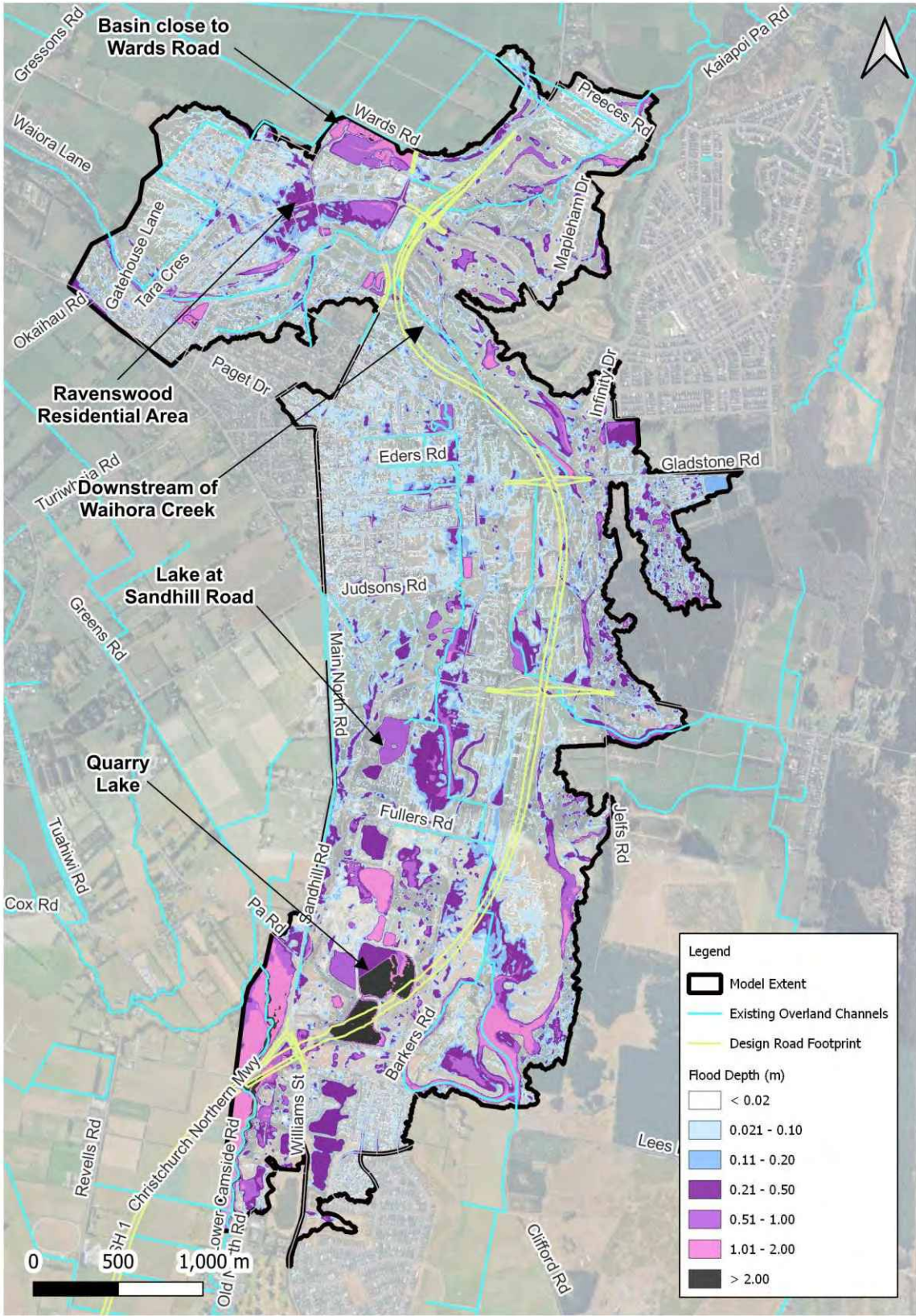


Figure 2-5 B2P-Woodend flood model pre-development flood depth for the 100-year ARI event

Figure 2-6 illustrates the differences in flood depth between the B2P Pre-development flood model and the localised flood depth map from the WDC South Ashley Model, specifically for the 100-year ARI event. Areas shaded in blue indicate regions where the flood depth is greater in the WDC South Ashley Model compared to the B2P Pre-development flood model, and vice versa.

Figure 2-7 shows the differences in ground level between the B2P Pre-development flood model and the localised flood depth map from the WDC South Ashley Model. Areas marked in blue indicate regions where the ground level is higher in the WDC South Ashley Model than in the B2P Pre-development flood model, and vice versa.

Overall, the discrepancies in results can be attributed to several factors. These include the coarser cell sizes utilised in the WDC South Ashley Model, large cross-sectional areas of open channels burnt in the road to represent cross culverts in that model, and recent ground level updates in newly developed areas. The locations where significant differences have been identified are summarised in Table 2-2.

Table 2-2 Locations where large discrepancies are identified

Locations	Reasons
Ravenswood residential area (B2P Flood Model > WDC South Ashley Model)	As shown in Figure 2-7, the Ravenswood residential area was not included in the WDC South Ashley model but has been incorporated into the B2P Flood Model. This inclusion has resulted in the increased flood depth in the affected areas.
Basin close to Wards Road (B2P Flood Model > WDC South Ashley Model)	As shown in Figure 2-7, the basin was not included in the WDC South Ashley Model but has been incorporated into the B2P Flood Model. This inclusion has resulted in increased flood depths in the affected areas.
Downstream of Waihora Creek (WDC South Ashley Model > B2P Flood Model)	Figure 2-8 shows a 40m wide trapezoidal cross-section was used in the WDC South Ashley Model to represent the existing 4m x 2m box culvert. This design facilitates a higher runoff flow downstream, leading to greater flood depths in those areas when compared to the B2P Flood Model. This has been replaced by a 1D network in the B2P Flood Model.
Lake at Sandhill Road (WDC South Ashley Model > B2P Flood Model)	As shown in Figure 2-7, variations in water levels may have resulted from the DEM which shows a higher water level in the WDC South Ashley Model compared to the B2P Flood Model.
Quarry Lake (B2P Flood Model > WDC South Ashley Model)	As shown in Figure 2-7, variations in flood depth is resulted from the ground levels, which have captured a higher water level in the B2P Flood Model compared to the WDC South Ashley Model.

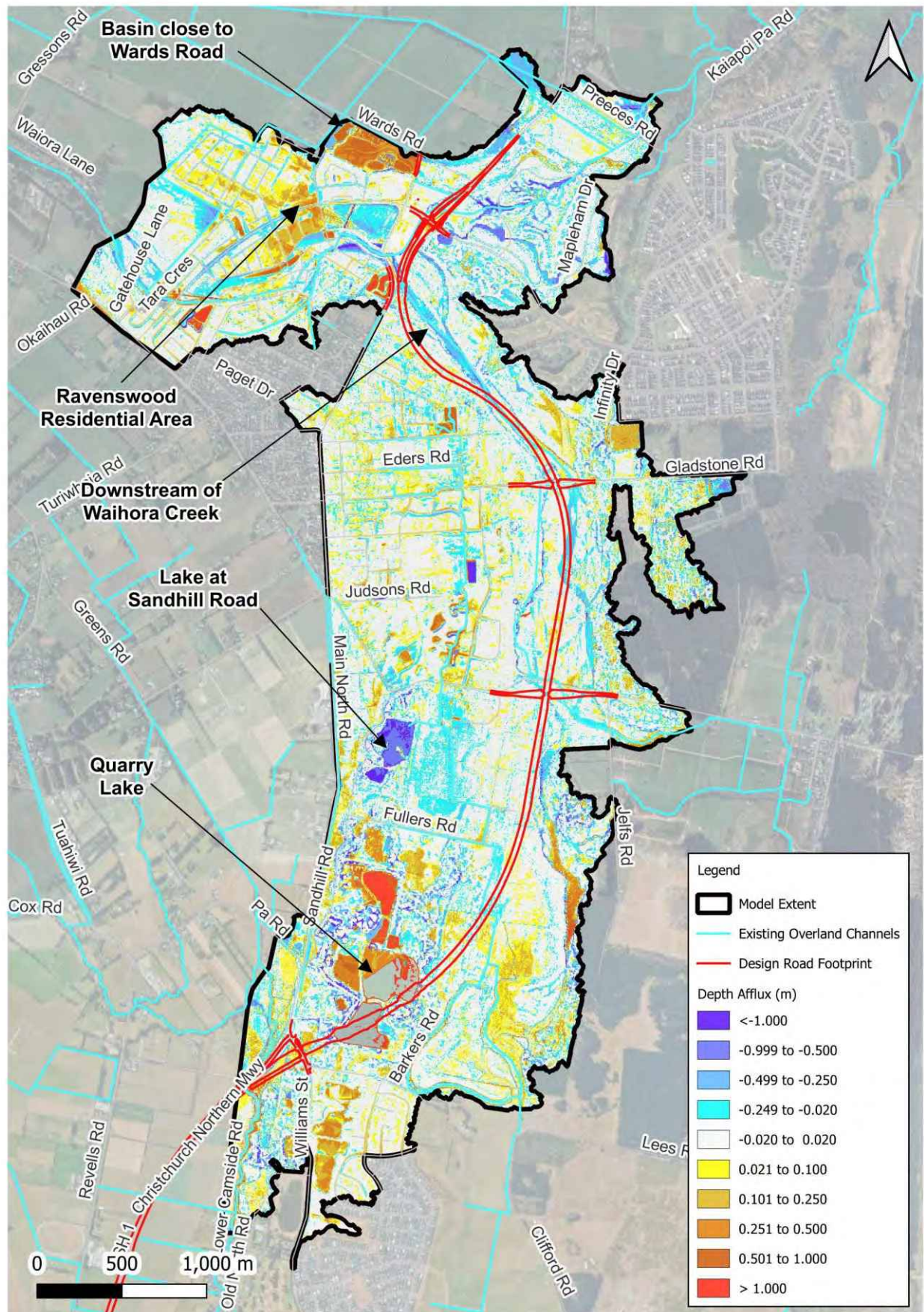


Figure 2-6 B2P Flood Pre-development model 100year ARI flood depth minus WDC South Ashley Model localised flood depth map under 100-year ARI event

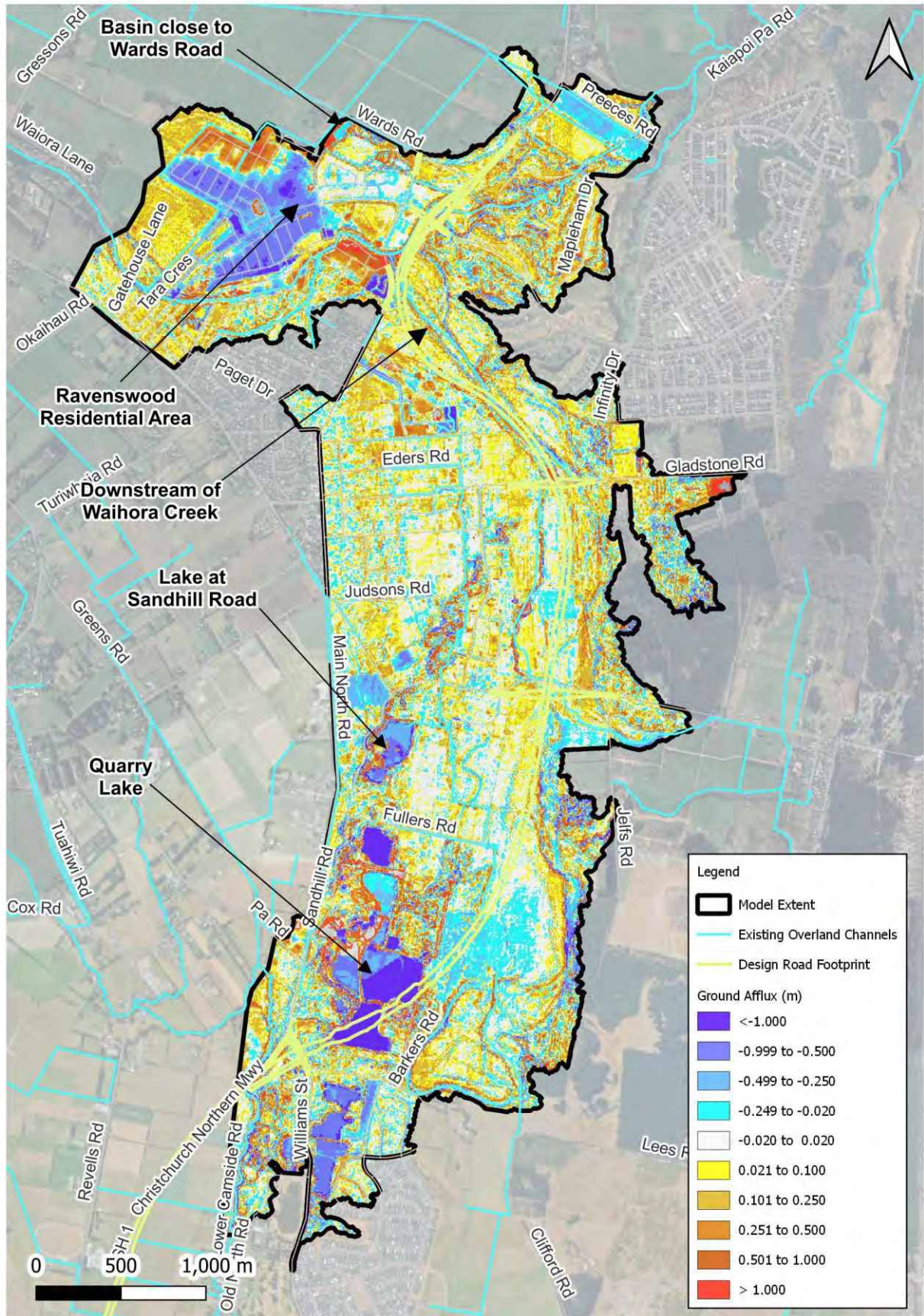


Figure 2-7 B2P Flood Model ground level minus WDC South Ashley Model ground level

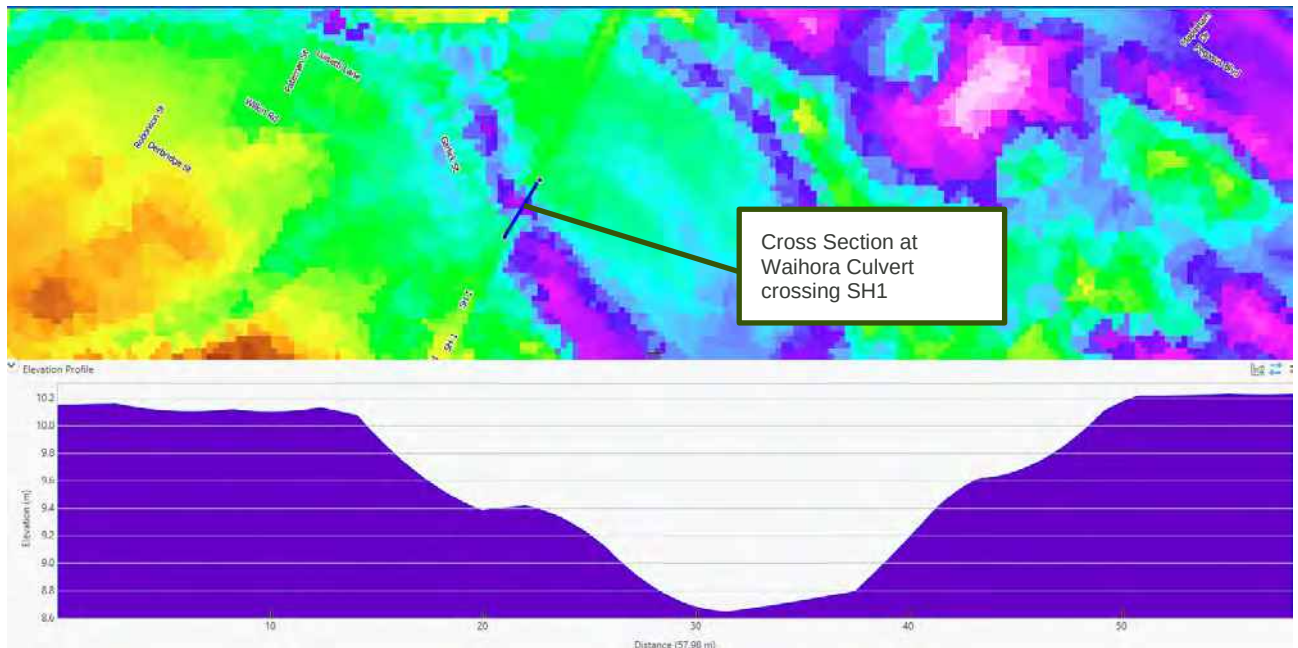


Figure 2-8 WDC Model Ground Level representing an existing 4m x 2m box culvert

2.2 B2P Post-Development Flood Model

The detailed model build methodology for B2P Flood Model post-development scenario (POSTDEV) is described in Appendix A Table 6.

Figure 2-9 illustrates the proposed design surface level for the entire road alignment. The proposed alignment generally follows the existing ground levels, with raised embankments incorporated along its length. However, the construction of these embankments alters the natural flow path, leading to the potential for water pooling behind the embankments due to the disruption of drainage. No mitigation measures have been developed in this model since the goal is to determine the effects of the bypass road and its comparison with the Pre-Development Model.

Existing culverts that fell within the designation boundary were removed, while culverts that are outside were maintained. The only culvert that was modelled with a fish passage consideration was the DN1800 culvert that crosses Bob Robertson drive, similar to the Pre-Development Model.

Comparable to the baseline flood model, the post-development flood model was developed under the assumption that the stormwater pipes are completely blocked.

Figure C-5 and Figure C-6 (refer to Appendix C) show the maximum flood depth and flood level of the post-development scenario under the 100-year ARI storm event, while Figure C-7 and Figure C-8 present the maximum flood depth and flood level under the 200-year ARI storm event.

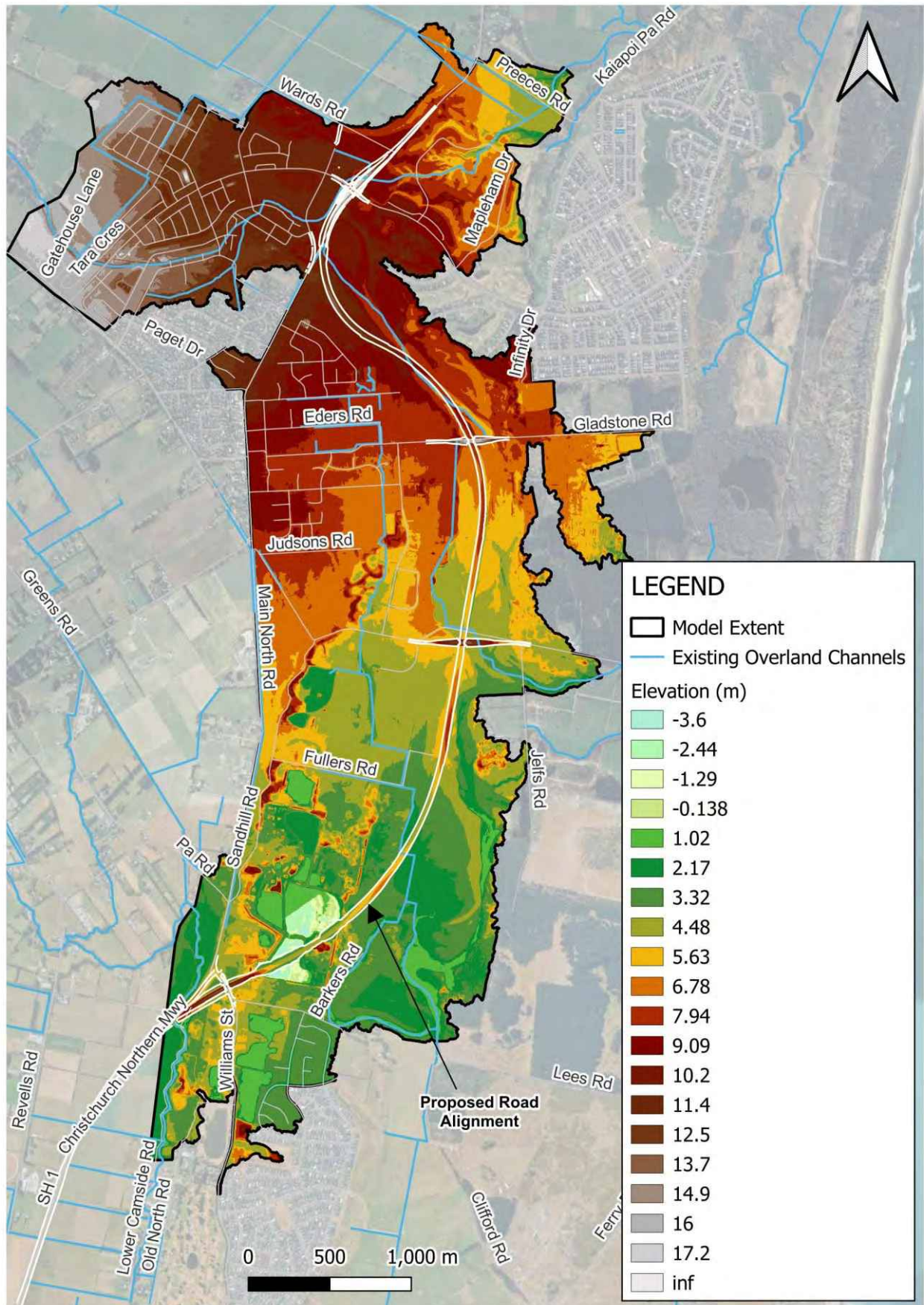


Figure 2-9 Proposed B2P-Woodend design surface level

2.2.1 Post-Development Scenario Result Discussion

Maximum flood level difference maps were developed to compare the post-development scenario with the pre-development scenario. These are presented in Figures C-9 and C-10 (see Appendix C) for the 100-year and 200-year ARI events.

Based on the Figures C-9, C-10, the proposed state highway remains dry under both the 100-year and 200-year ARI scenarios, with the exception of the road section north of Bob Robertson Drive, where floodwaters overtop the road as shown in Figure 2-10. The road overtopping is primarily due to a low existing road level and undersized existing culverts. To address this issue, the 2.2m x 1.2m culverts at Taranaki stream and DN750 at north of Taranaki stream are proposed to be upsized. This will allow more runoff to be conveyed downstream through culverts instead of overtopping the road.

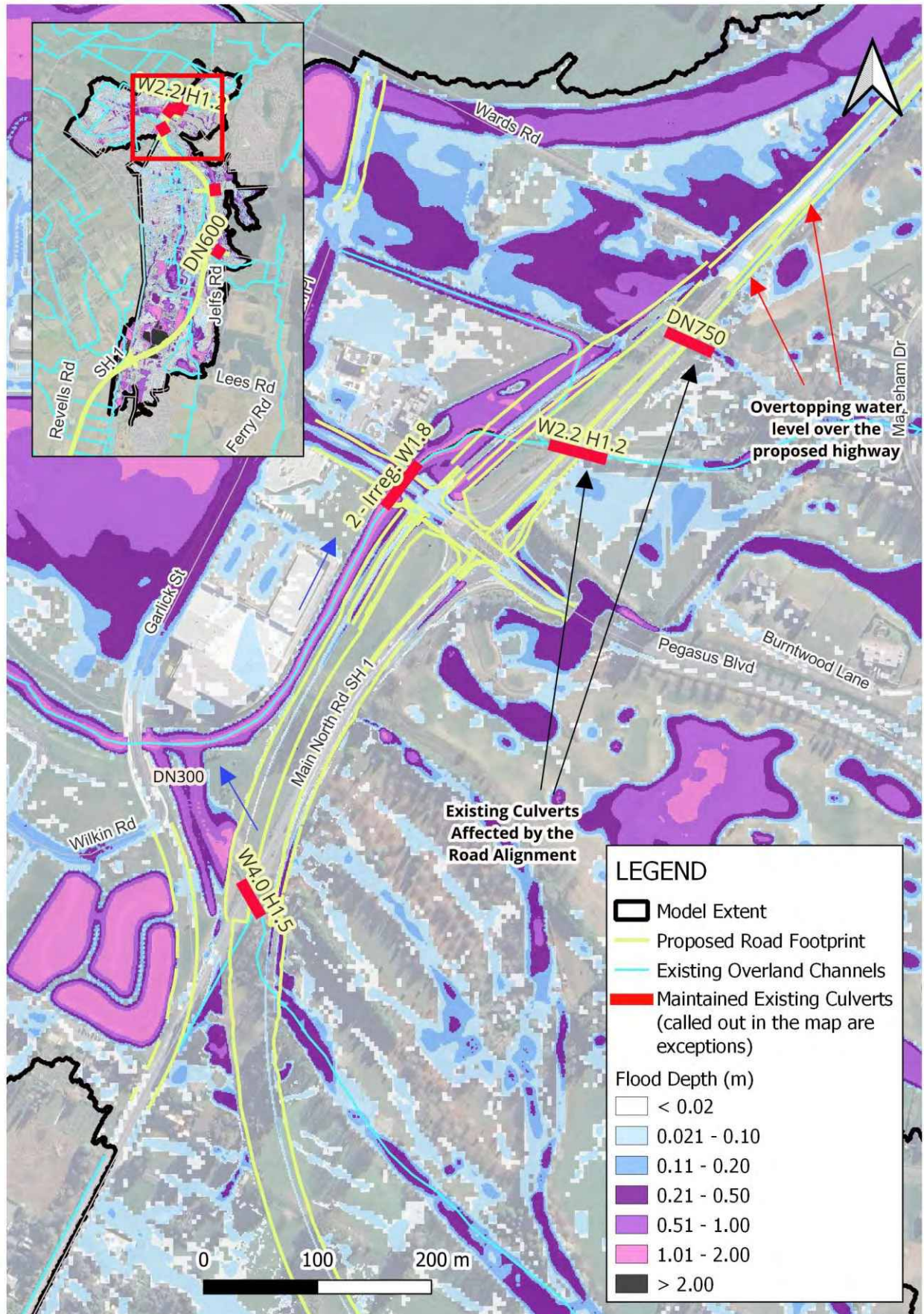


Figure 2-10 Floodwaters overtop the road at north of Bob Robertson Drive (200 year ARI Post-Development Scenario)

Figures C-9 and Figures C-10 indicate that most flooded zones are confined adjacent to the proposed road corridor. However, elevated flood levels are observed in the residential area along Main North Road, Woodend Beach Road and Wards Road during both the 100-year and 200-year ARI events.

The increase in flood level is attributed to the impact of road embankment towards the natural flow of runoff, such that existing channels are blocked by the road embankment that eventually contributed to the runoff buildup. Flood depth, flood levels, and their differences at the eight impacted properties (as shown in Figure 2-11 are presented in Table 2-3).

The following tables present the findings in terms of flood level and their difference between the post-development and pre-development cases at varying ARIs (100 and 200-year):

Table 2-3 100-year and 200-year ARI Flood Levels and Differences (Post-Development vs. Pre-Development Scenario)

Point ID	Location	100-year ARI			200-year ARI		
		Pre-development flood level (m)	Post-development flood level (m)	Difference (mm)	Pre-development flood level (m)	Post-development flood level (m)	Difference (mm)
1	5 Wards Rd*	8.28	8.42	140	8.37	8.5	137
2	196 Woodend Beach Rd	4.13	4.33	193	4.23	4.57	335
3	1271 Main North Rd*	8.11	8.29	183	8.16	8.33	171
4	1263 Main North Rd	8.28	8.31	30	8.29	8.35	58
5	1279 Main North Rd*	8.01	8.13	120	8.03	8.15	114
6	1273 Main North Rd*	8.10	8.24	140	8.15	8.28	130
7	1277 Main North Rd*	8.02	8.18	160	8.07	8.21	142
8	1319 Main North Road	6.41	6.44	27	6.46	6.55	91

*Property will be acquired as part of project work delineation. Flood impacts within designation have been considered acceptable

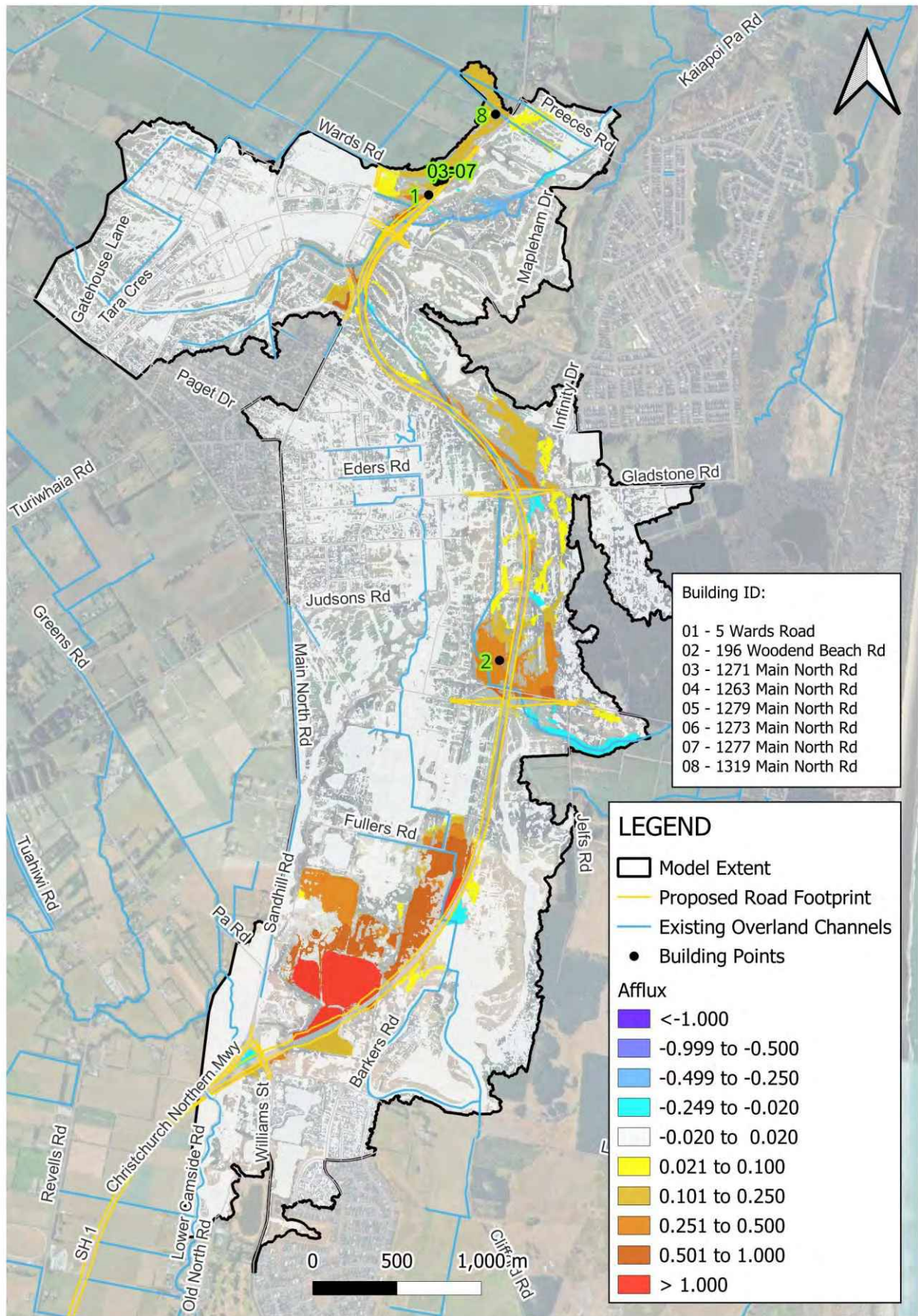


Figure 2-11 Impacted properties with 200yr afflux level map (post-development scenario) as background

These findings indicate that the increases exceeded the project objectives, which require that no flood level increase affecting neighbouring properties under the 100-year ARI event, and that any increase for the 200-year ARI event should not exceed 20 mm.

Of the eight impacted properties, five will be acquired as part of the project work delineation. Flood impacts within the designated area have been considered acceptable, if the flooding does not overtop the road.

Given the increase in flooding at the above three locations, flood mitigation measures are necessary to reduce impacts on these zones. The following solutions will be considered during detailed design:

- Provision of adequate culverts across the design road alignment.
- Divert impounded runoff through additional channel alignments.

2.3 B2P Mitigation Flood Model

The methodology for the B2P Flood Model post-development scenario with mitigation solution is outlined in Appendix A Table 6. The key components of the mitigation solution include:

- Provision of culvert crossings:
 - Culvert structures shall be added along critical areas beside the road alignment to avoid runoff build-up. These structures are incorporated in the flood model to properly represent the mitigation solution once the road embankment is constructed.
 - Culverts sizing at Taranaki Stream, Waihora Creek and McIntosh Drain include the 300mm embedment depth for fish passage consideration.
 - The design invert levels for the culvert structures are based on the provided survey data, such that the defined upstream and downstream invert levels are enough to have a minimum adequate slope to convey runoff without overflowing at the road level.
 - Where road widening is proposed at Bob Robertson Drive, existing culverts are to be extended or replaced with like-to-like sizes to avoid introducing flood impacts downstream.
- Provision of flow path channel diversions:
 - Additional channels are accounted for the model to ensure smooth flow behaviour of impounded runoff from upstream to downstream.
 - The design slopes of channels are established to match the natural topography to permit the flow of runoff without causing impounding upstream.

Figure 2-12, Figure 2-13, Figure 2-14, Figure 2-15, and Figure 2-16 show the locations where culverts and flow paths diversions are proposed as part of mitigation solutions. Some major flow changes due to proposed mitigating structures are shown in the following figures. Blue arrowheads represent the flow direction of runoff.

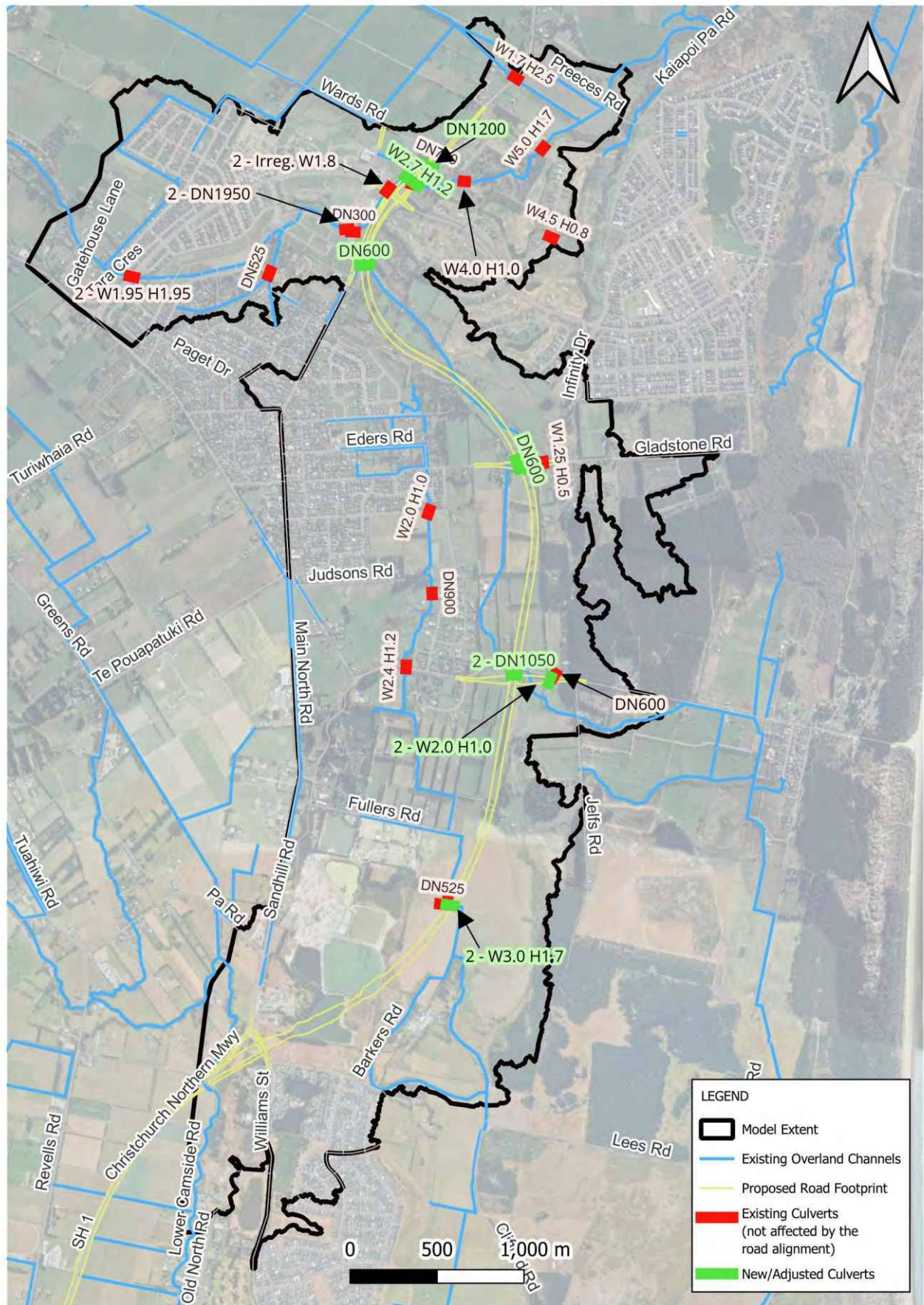


Figure 2-12 Proposed mitigation scheme adjacent to the road alignment

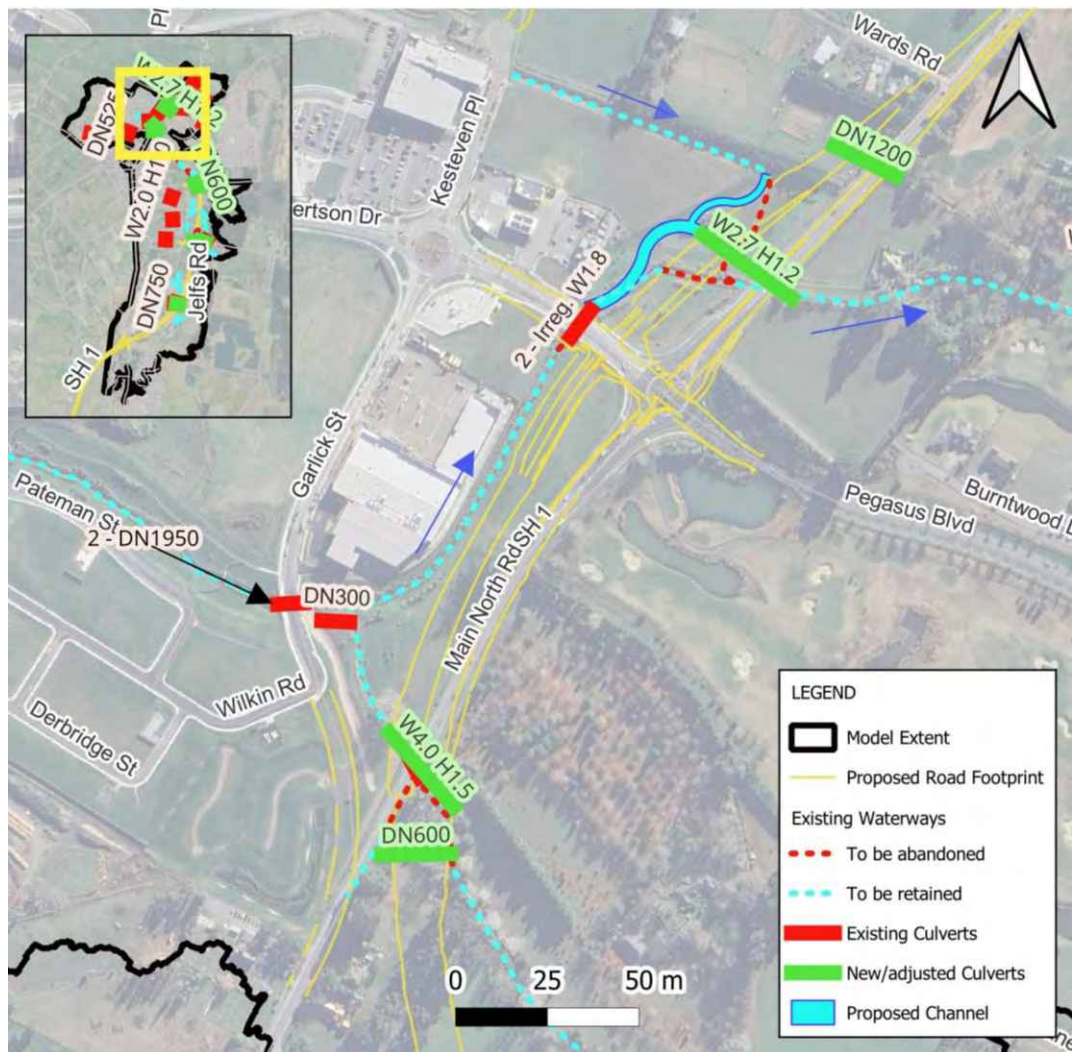


Figure 2-13 Proposed culvert alignments (upper zone)

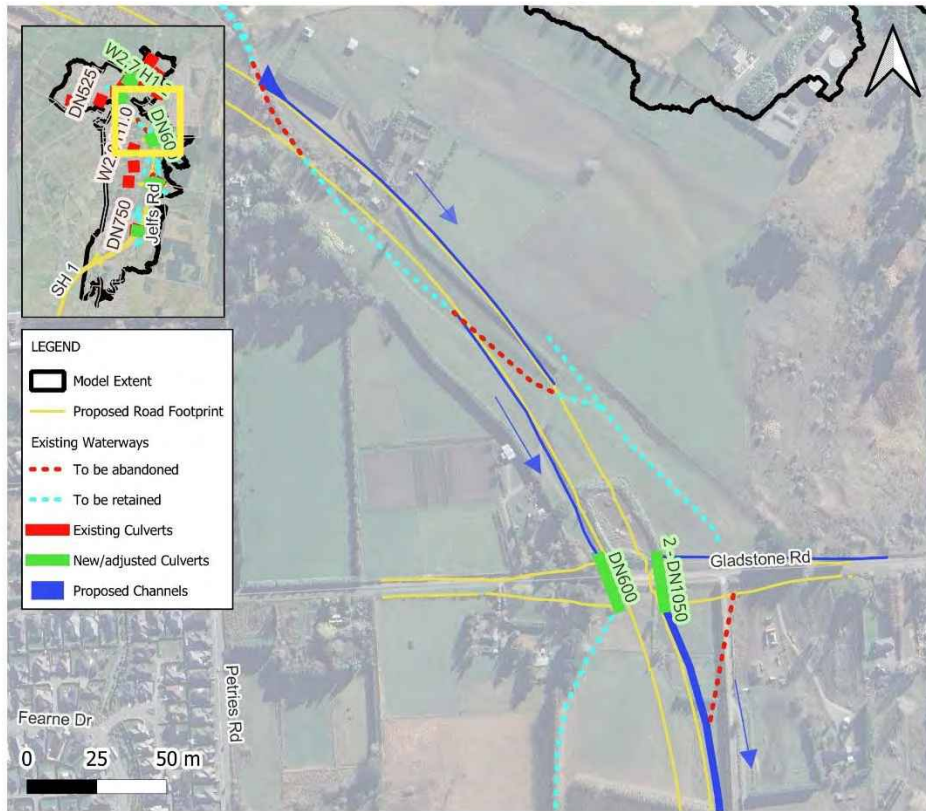


Figure 2-14 Proposed culvert alignments (upper central zone)

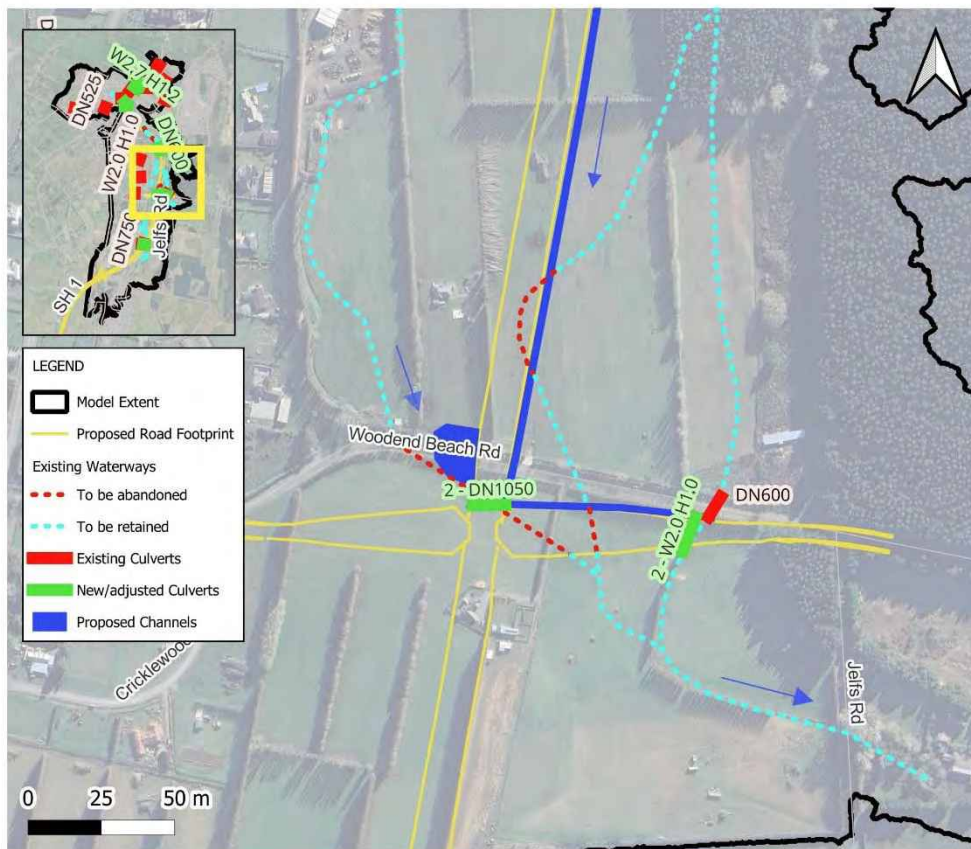


Figure 2-15 Proposed culvert alignments (central lower zone)

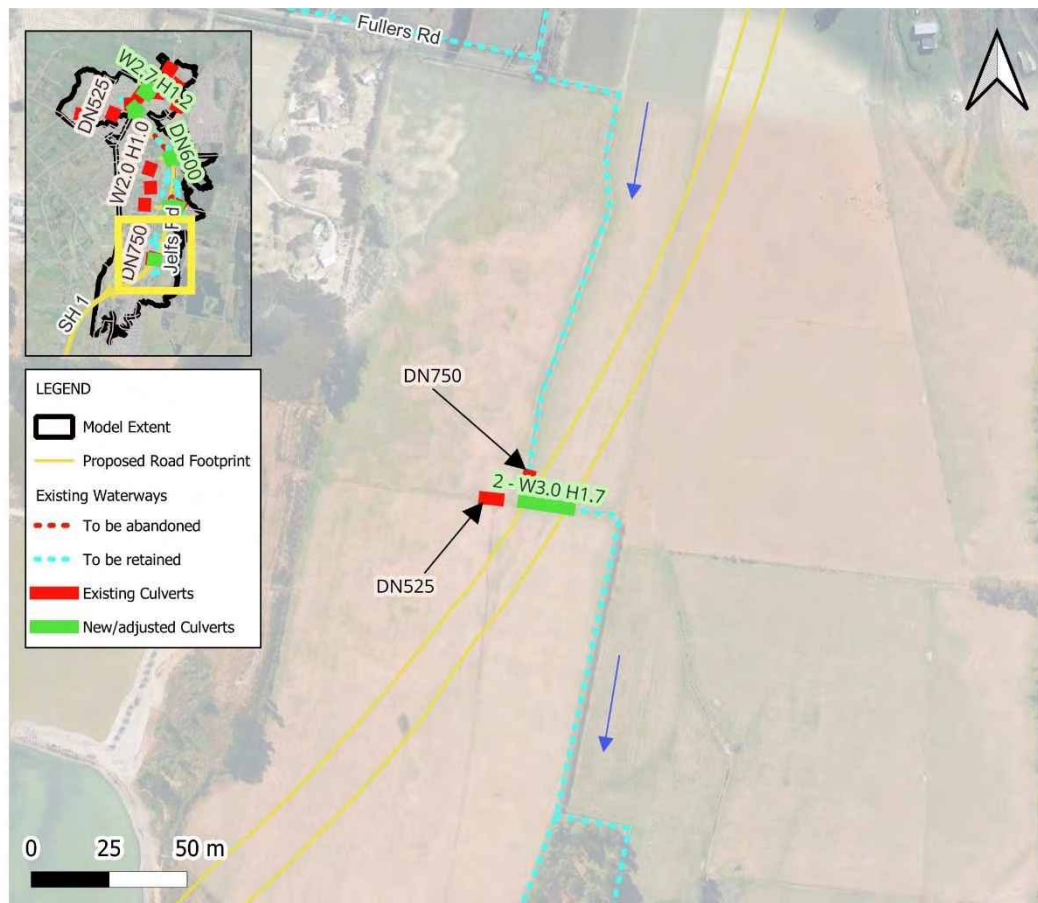


Figure 2-16 Proposed culvert alignments (lower zone)

2.3.1 Mitigation Scenario Result Discussion

Figures C-11 and C-12 (refer to Appendix C) illustrate the maximum flood depth and flood level for the mitigation scenario under a 100-year ARI storm event, while Figures C-13 and C-14 represent these parameters for the 200-year ARI storm event.

Maximum flood level difference maps, comparing the mitigation and pre-development scenarios, are shown in Figures C-15 and C-16 for the 100-year, and 200-year ARI events, respectively. Based on these figures, with the mitigation measures implemented, the proposed state highway remains dry under both the 100-year and 200-year ARI scenarios.

Table 2-4 100-year and 200-year ARI flood level and differences (Mitigation vs Pre-development scenario)

Point ID	Location	100-year ARI			200-year ARI		
		Pre-development flood level (m)	Post-development flood level (m)	Difference (mm)	Pre-development flood level (m)	Post-development flood level (m)	Difference (mm)
1	5 Wards Rd*	8.26	8.26	0	8.37	8.28	-90
2	196 Woodend Beach Rd	4.13	4.14	4	4.23	4.22	-13
3	1271 Main North Rd*	8.11	8.11	0	8.16	8.07	-83
4	1263 Main North Rd	8.29	8.29	0	8.29	8.29	0
5	1279 Main North Rd*	8	8	0	8.03	8.02	-10
6	1273 Main North Rd*	8.09	8.09	0	8.15	8.10	-50
7	1277 Main North Rd*	8.02	8.02	0	8.07	8.05	-20
8	1319 Main North Road	6.41	6.41	0	6.46	6.46	1

* Properties will be acquired as part of project work delineation. Flood impacts within designation have been considered acceptable.

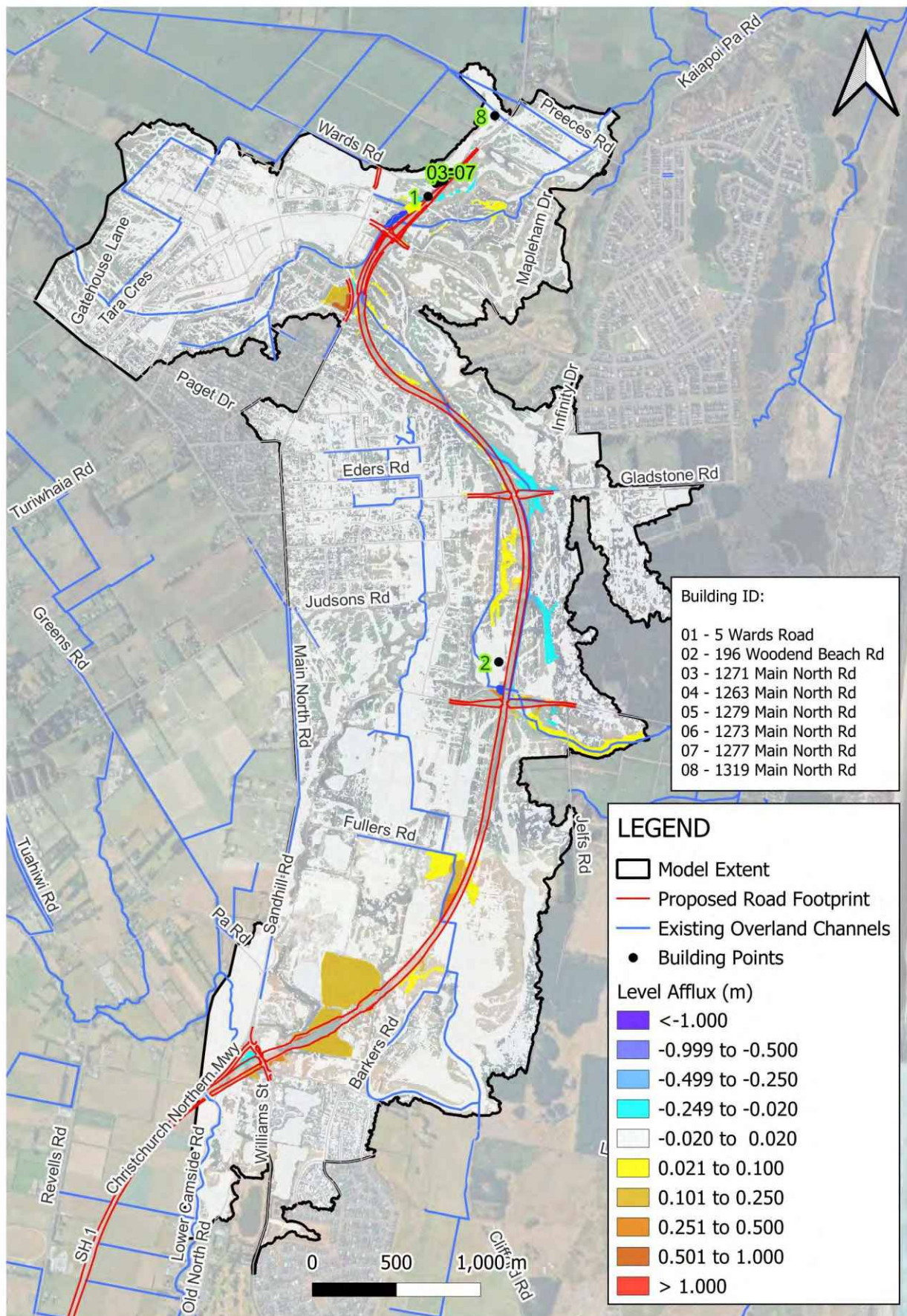


Figure 2-17 Impacted buildings with 200yr afflux level map (Mitigation scenario) as background

Based on the results presented in Table 2-4 and illustrated in Appendix C, the mitigation model has successfully reduced flood impacts across most areas, including at previously affected buildings (excluding the five buildings identified for acquisition as part of project delineation) during the 100-year and 200-year ARI event.

Water level increases have been observed in the locations listed in Table 2-5 and as shown in Figure 2-18, which are outside the designation boundary. However, these increases do not affect any property buildings.

Table 2-5 Locations that observed a water level increase outside of designation boundaries

Locations	Max Afflux (mm)		Comments
	100-yr	200-yr	
1. Mapleham Dr	2	20	The increase in water depth is due to the upsizing of the Taranaki Stream culvert. However, the impacts remain within the channel and do not affect any buildings.
2. Basin within Ravenswood residential area	200	670	Water depth increase is caused by raising the overflow weir from RL 10.20 m to RL 10.40 m to accommodate the proposed road (Garlick Street). The increased depth is confined within the basin and poses no risk to nearby properties.
3. Downstream of Waihora Creek	210	440	Higher water levels indicated in the model due to reduced roadway overtopping of Taranaki Stream north of Bob Robertson Drive, redirecting slightly more runoff into Waihora Creek. The impact is contained within the channel and does not affect any buildings.
4. West of Copper Beech Road	40	50	Water ponding is due to the motorway cutting off an existing overland flow path. The ponding is localised and does not impact any buildings.
5. Downstream of Woodend Beach Road	20	50	Flood levels within Waihora Creek rise slightly due to more efficient conveyance from the proposed bypass and both culverts at Gladstone Road and Woodend Beach Road have been upsized. No buildings are affected, and the impacts are confined within the channel.
6. Upstream of McIntosh Drain	130	280	For the upstream reach of McIntosh Drain, fully resolving the flood impacts remains challenging since maintaining the original flow area, spanning approximately 300 meters, will require significant structural measures (e.g. more culverts, bridges, etc.). This increase is largely confined to land owned by NZTA which does not include any residential properties.
7. Within the Quarry Lake.	0	120	Water level increases are shown within the model resulting from storage loss from the proposed bypass road infilling the lake. It is noted that the water level in the ponds generally reflects existing groundwater levels and the model assumes no infiltration. However, as the quarry bottom is directly connected to groundwater, flood afflux is expected to be minimal due to this continuous interchange with groundwater.

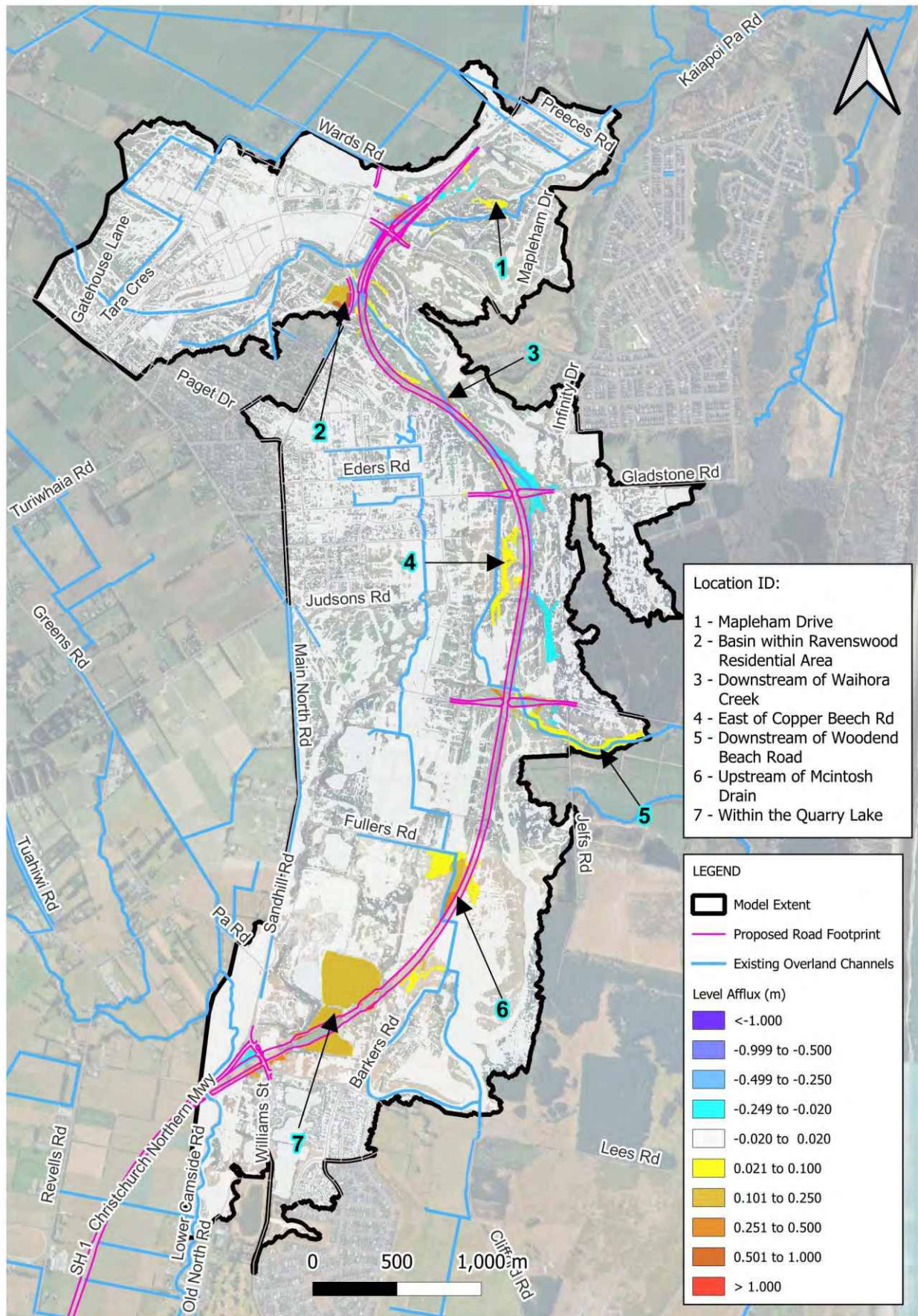


Figure 2-18 Residual impacts outside the designation but does not impact any property buildings (200 year ARI afflux level)

2.3.2 Flood Duration Impact Discussion

Flood hydrographs from the Pre-Development and Mitigation Scenarios were compared to determine if a prolonged flood event was observed, specifically along major waterways that intersect the proposed bypass road. Four locations were identified, which have also been the site of most mitigation measures with the goal of maintaining existing overland flow paths:

- Taranaki Stream (Figure 2-19 and Figure 2-20)
- Waihora Creek to Gladstone Road (Figure 2-21 and Figure 2-22)
- Woodend Beach Road (Figure 2-23 and Figure 2-24)
- McIntosh Drain (Figure 2-25 and Figure 2-26)

Taranaki Stream

Under the 100-year ARI event, hydrographs at both the upstream and downstream ends of Taranaki Stream (Figure 2-19) show minimal difference between the pre-development and mitigation scenarios.

Under the 200-year ARI event, the upstream hydrograph (Figure 2-20) shows a slight reduction in peak flow due to the proposed road reducing storage. The new Taranaki Stream culvert acts as a flow constriction, causing water to back up and divert via Waihora Creek. This is confirmed by the afflux maps (Figure C-16), which show increased flood levels upstream of Taranaki Stream and downstream of Waihora Creek.

Despite these local affluxes, flood extents remain unchanged and no increased flood risk to is expected to nearby properties.

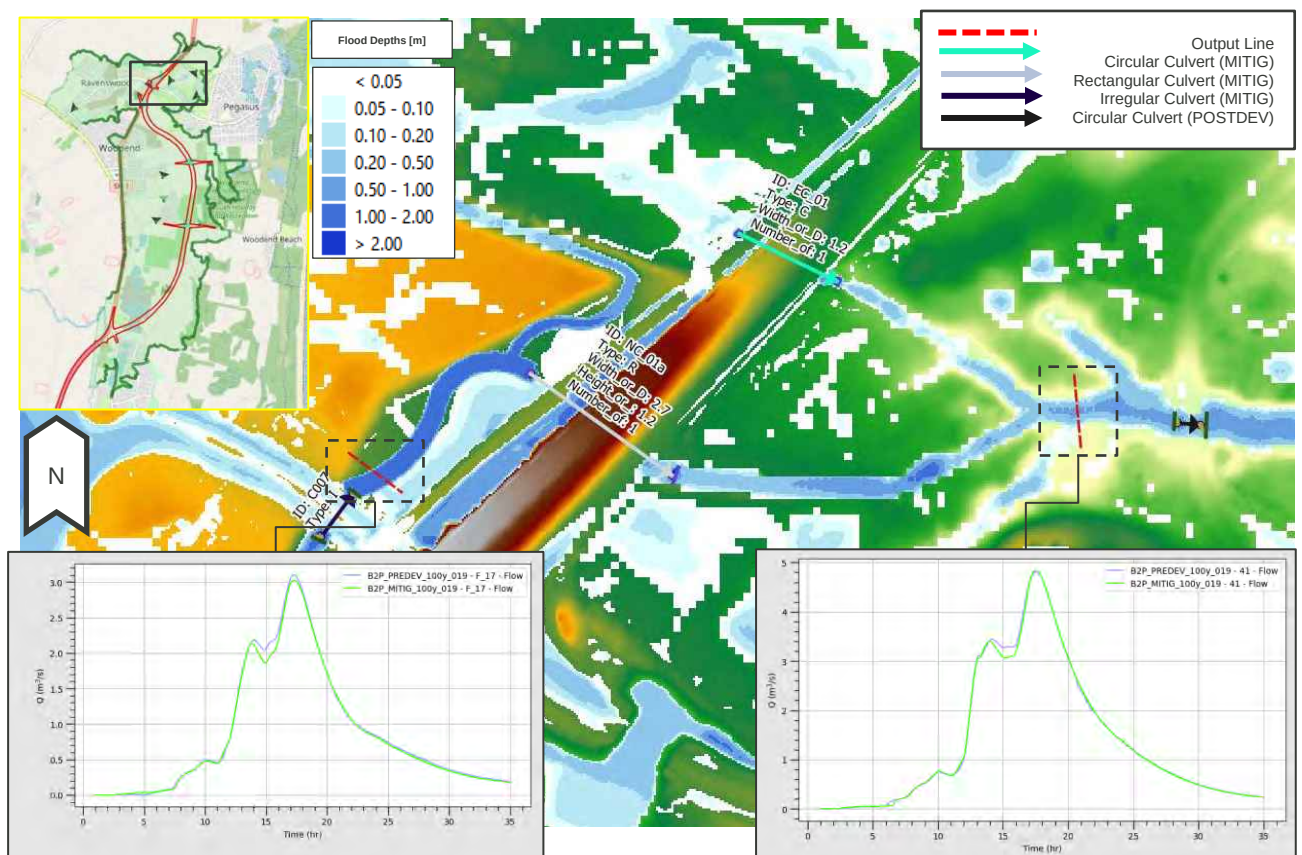


Figure 2-19 Taranaki Stream 100-year Flood Duration Assessment

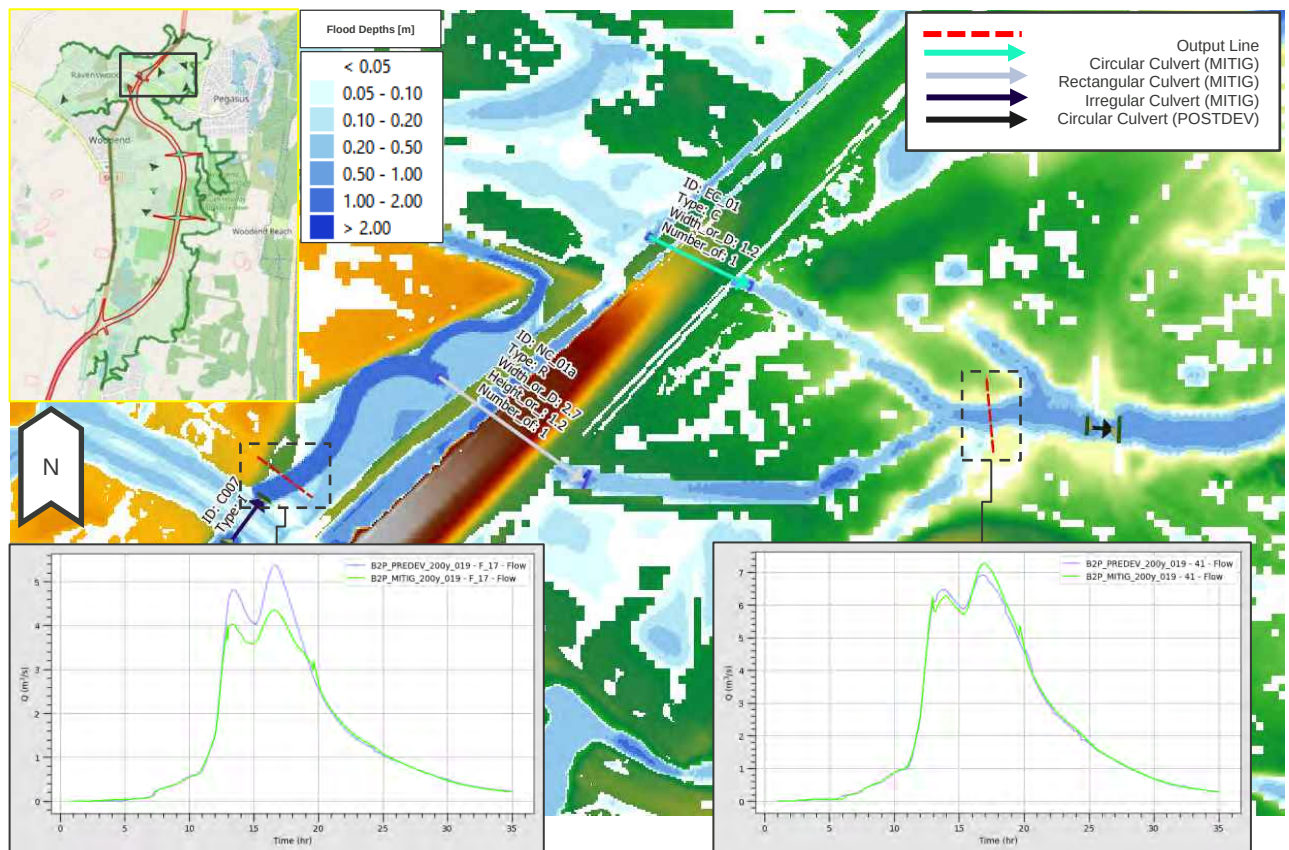


Figure 2-20 Taranaki Stream 200-year Flood Duration Assessment

Waihora Creek to Gladstone Road

The overland flow path connecting Waihora Creek to Gladstone Road functions as a key drainage route, conveying runoff through to Woodend Beach Road and ultimately eastward. During the 100-year ARI event (Figure 2-21) upstream hydrographs near Waihora Creek show similar dual-peak patterns for both pre-development and mitigation scenarios. At the downstream location near Gladstone Road, the pre-development peak is lower due to the undersized existing culvert, which causes runoff to back up and pond upstream before overtopping. In contrast, the mitigation scenario includes a larger culvert, allowing runoff to convey more efficiently downstream without ponding.

In the 200-year ARI event (Figure 2-22) the upstream hydrograph also shows a dual-peak pattern. The mitigated scenario has a lower first peak as more flow is initially conveyed to Taranaki Stream through the larger proposed culvert. Once the Taranaki Stream culvert reaches capacity, excess flow backs up and diverts into Waihora Creek, resulting in a higher second peak in the mitigation scenario.

Despite these local affluxes, flood extents remain unchanged and no increased flood risk to is expected to nearby properties.

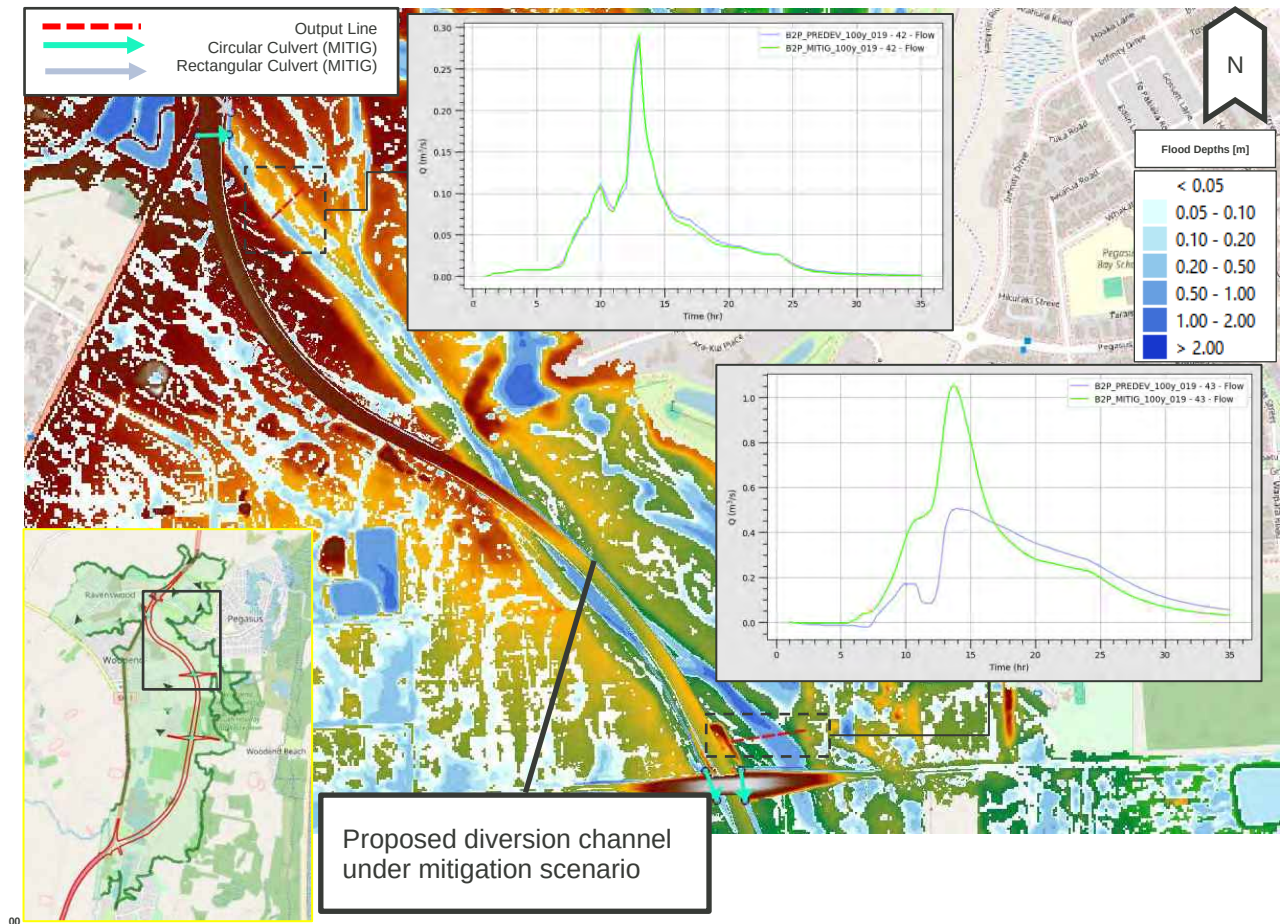


Figure 2-21 Waihora Creek to Gladstone Road 100-year Flood Duration Assessment

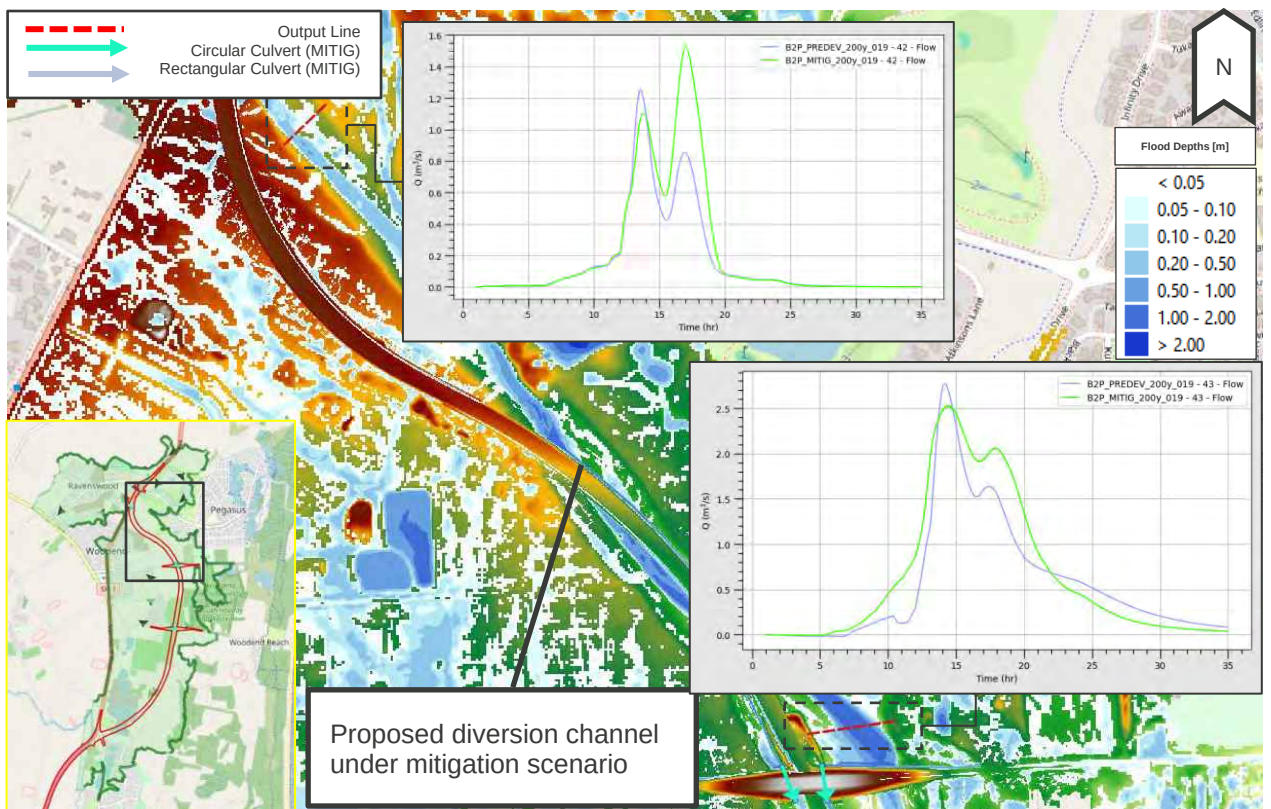


Figure 2-22 Waihora Creek to Gladstone Road 200-year Flood Duration Assessment

Woodend Beach Road

Similar to the situation at Gladstone Road, the proposed interchange at Woodend Beach Road functions as an impounding structure, temporarily storing runoff from the proposed eastern waterways of the bypass.

For the 100-year ARI event (Figure 2-23), the hydrographs at both the upstream and downstream of Woodend Beach Road show higher peaks in the mitigation scenario. This is due to the proposed larger culvert, which enables greater flow conveyance downstream. In the existing scenario, runoff ponds behind the road before overtopping downstream.

For the 200-year ARI event (Figure 2-24), the hydrograph patterns are similar, with the mitigation scenario again allowing slightly more runoff to travel downstream as a result of the culvert upsizing.

A positive afflux is observed downstream (Figure C-16). However, flood extents remain unchanged, and there is no increased flood risk to nearby properties.

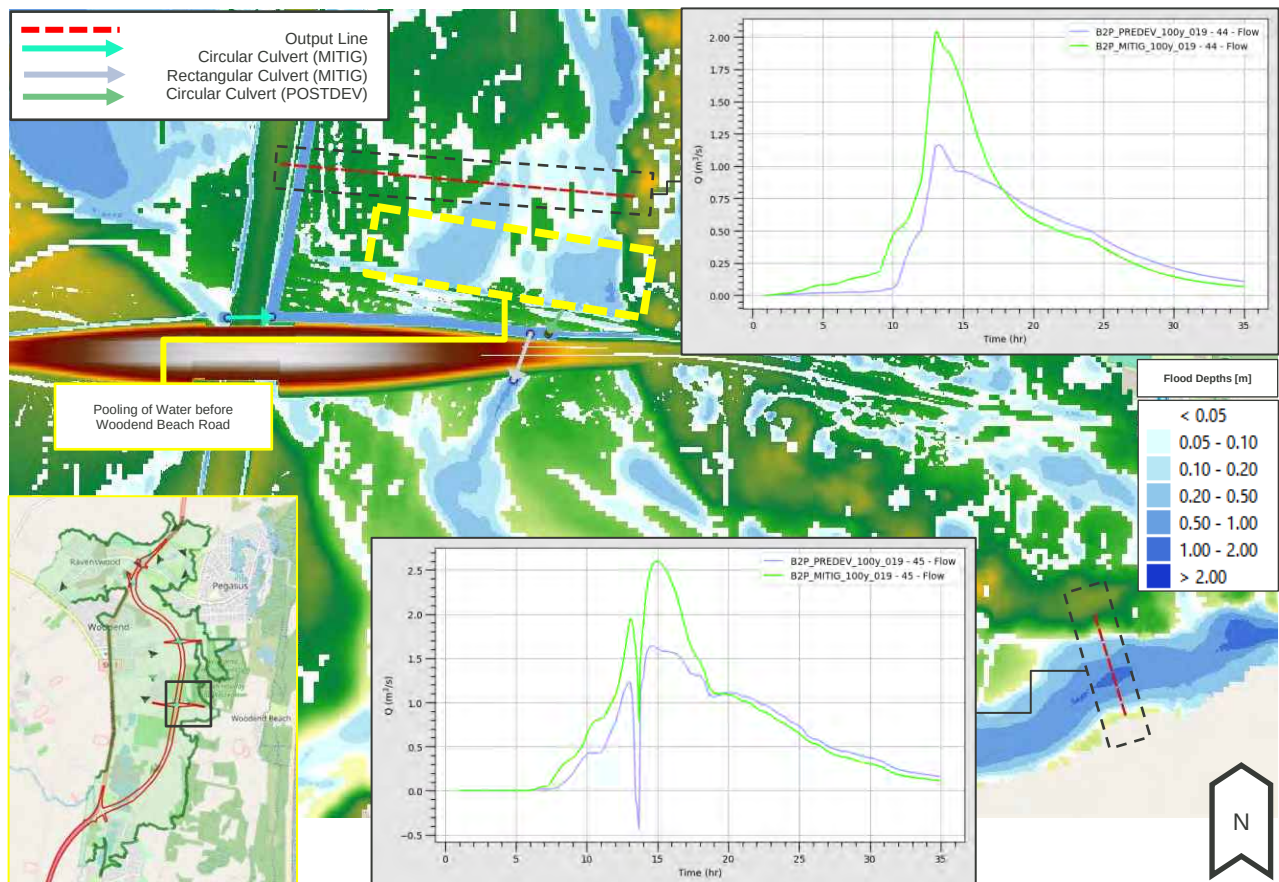


Figure 2-23 Woodend Beach Road 100-year Flood Duration Assessment

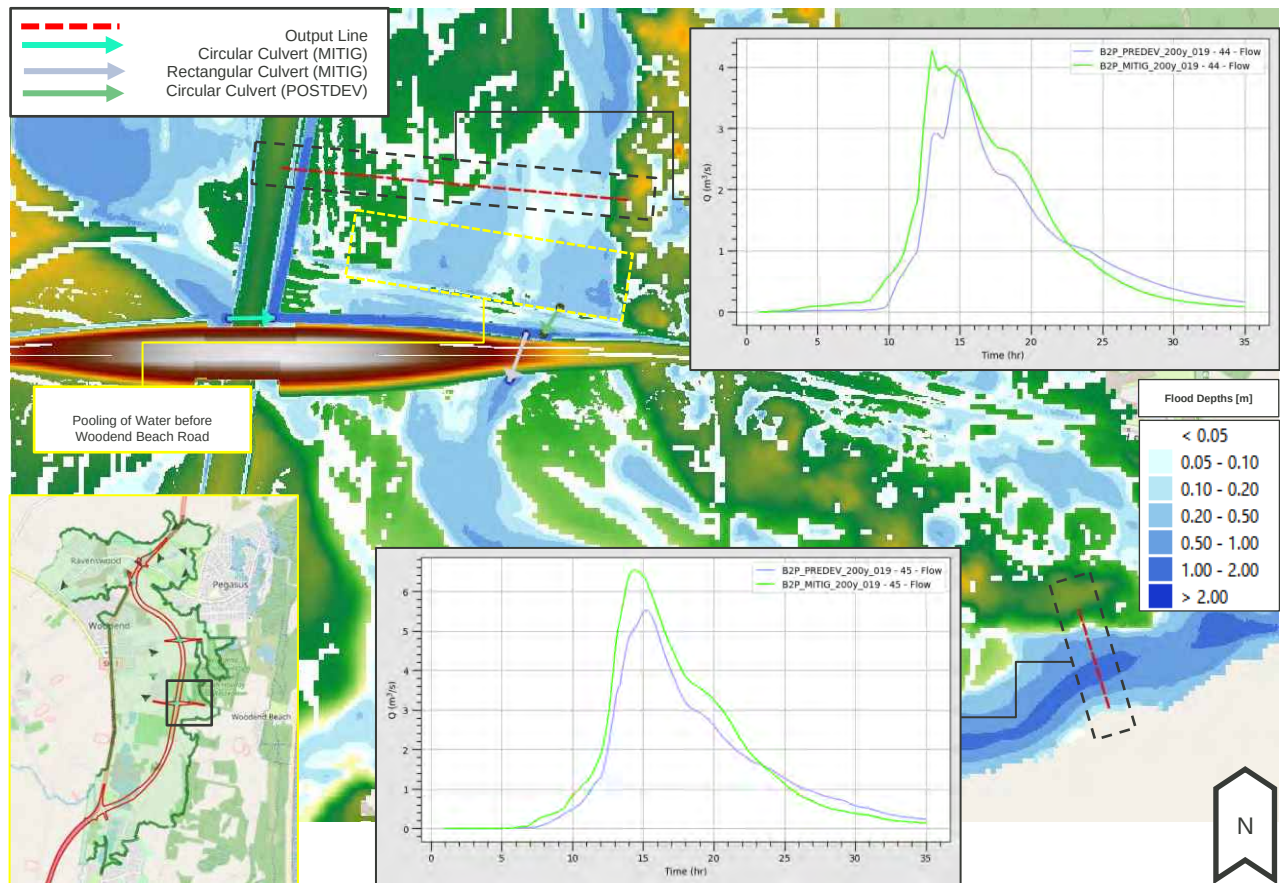


Figure 2-24 Woodend Beach Road 200-year Flood Duration Assessment

McIntosh Drain

The hydrographs for both the 100-year and 200-year ARI events (Figure 2-25 and Figure 2-26) show similar flow patterns between the pre-development and mitigation scenarios, with the pre-development scenario exhibiting a slightly higher peak. This is due to the wider overland flow path available under existing conditions, which becomes more constrained in the mitigation scenario as flows are directed through the proposed bypass and rely on the new proposed culvert for conveyance.

Overall, the drainage function is maintained. A positive afflux is observed upstream of McIntosh Drain (Figure C-15 and C-16), attributed to the raised elevation of the proposed road and a reduced cross-sectional flow area, which limits downstream conveyance. A wider flow path may be required to match pre-development levels. However, as there are no vulnerable assets within the affected floodplain, no increase in flood risk is anticipated.

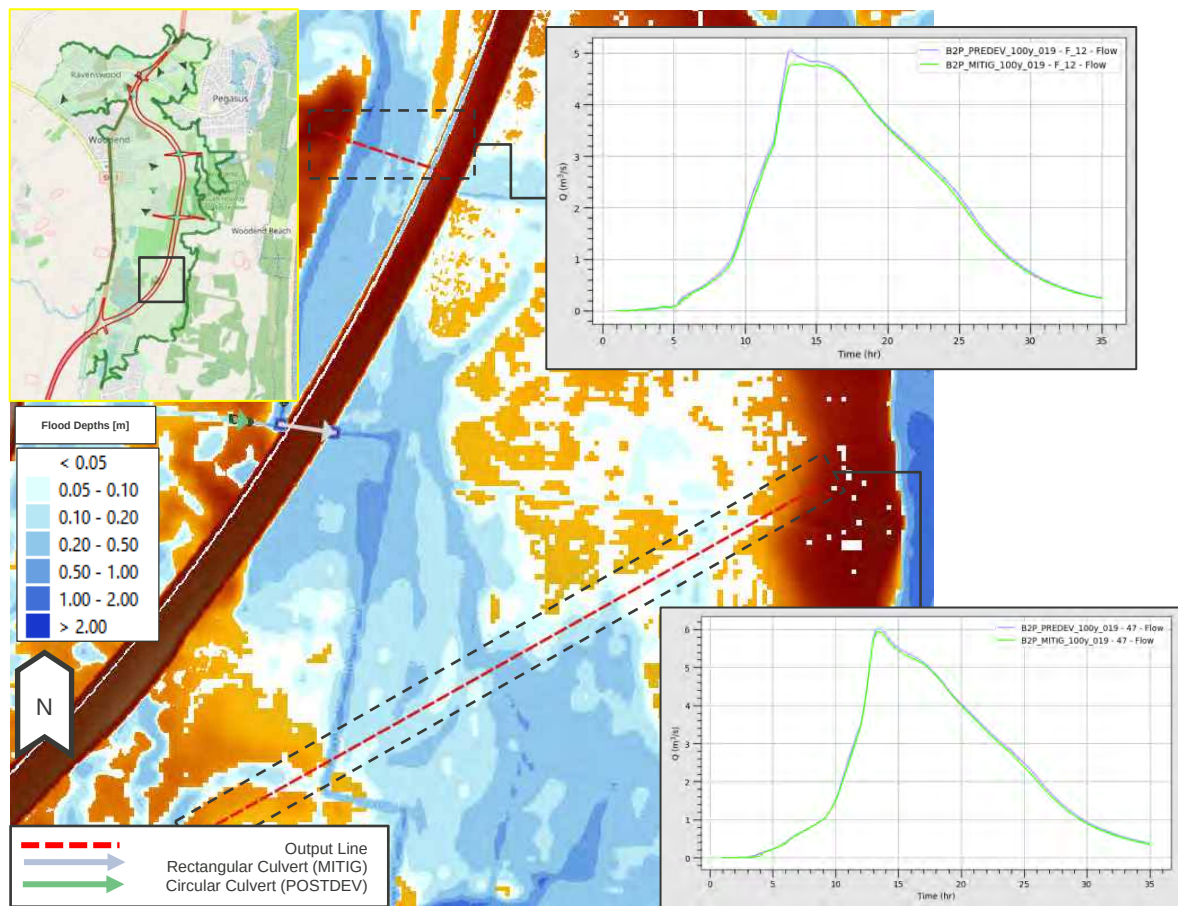


Figure 2-25 McIntosh Drain 100-year Flood Duration Assessment

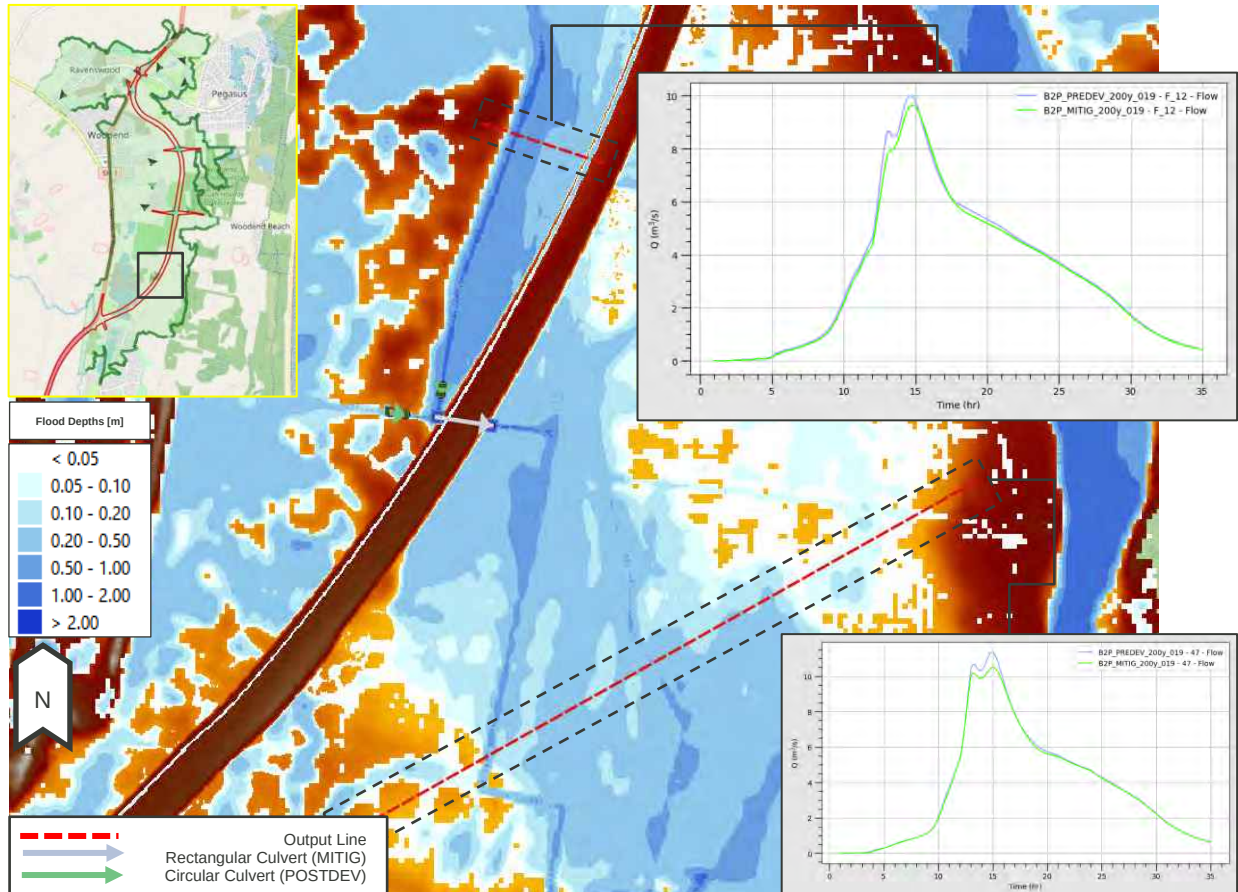


Figure 2-26 McIntosh Drain 200-year Flood Duration Assessment

3 Limitation and Applicability

The limitations and applicability of the study include the following:

- As with any model, this model still includes assumptions and simplifications of the complex real-world processes. On a broad scale, the model build provides a reasonable representation of flood depths and extents. The model, therefore, is deemed fit for purpose for broad-scale planning measures and for testing of flood impacts from proposed stormwater design and flood mitigation options.
- The B2P Flood Model has been developed using the hydraulic and hydrological data from WDC South Ashley Model. With this, all assumptions and limitations from the WDC model also apply. The 'Flood Hazard Models Update' report⁵ can be referred to for further information.
- Terrain data:
 - a. The B2P Flood Model is a snapshot of the catchment when the LiDAR was flown in 2020 and does not include new developments and future development. It may, therefore, not be applicable where flows from newly developed areas impact on results.
 - b. The existing basin surrounding the Ravenswood development area has not been accurately represented due to a lack of survey data and was not captured in the 2020 LiDAR data.
- It should be noted that localised ponding depths may vary significantly where overland flow is obstructed or blocked (e.g. by obstacles, debris, solid fences, retaining walls, sheds, etc.). Results could also be impacted in places which are not necessarily fully resolved by the 2-m grid used in this study.

⁵ DHI (May 2020). *Flood Hazard Models Update*.

4 Conclusion

This assessment has been carried out to investigate the flood impacts associated with the proposed Belfast to Pegasus Road development. A B2P Flood Model was developed using TUFLOW modelling software for this assessment. The following are the objectives of the flood assessment:

- No significant increase in flood levels at neighbouring dwellings for the 100-year ARI event.
- Any increase in flood levels at neighbouring dwellings for the 200-year ARI event should be limited to 20 mm.
- Ensure that the risks associated with the barrier effect induced by the proposed road development on the adjacent areas are reduced through various mitigation measures such as additional culverts, improvement/rehabilitation of existing culverts, and provision of additional channels to maintain or improve flow of runoff.

Three models have been developed, namely Pre-Development, Post-Development, and Mitigation models. The Pre-Development model assessed the current conditions of the model domain, including existing culverts and overland flow paths. It was considered as the base line model where flows, flood levels, and extents will be maintained through the proposed mitigation measures. The model was calibrated and validated with the WDC Flood Model in terms of flood levels and extents. The observed variations were attributed to mainly topographical differences and areas that were not incorporated in the WDC model but were updated and included in the TUFLOW model. The Post-Development model accounts for the proposed roadway, without the mitigation measures and shows the exacerbation of impacts and flood levels. These impacts were reduced in the Mitigation model through the installation of new culverts and channels to maintain the original drainage line and storage capacities of the surrounding areas.

The assessment findings indicate a flood level increase in various locations due to the proposed road development, with most impacted zones confined to the vicinity of the proposed road corridor. Additionally, elevated flood levels were observed in the residential areas along Main North Road and Woodend Beach Road, during both the 100-year and 200-year ARI events.

To ensure compliance with the regional policy statement and council requirements, two primary measures were proposed and modelled to reduce the flood impacts: the introduction of additional culverts and the diversion of flow path channels. The mitigation measures have demonstrated that effective reduction in flood impacts can be achieved across most areas during both the 100- and 200-year ARI events. The three impacted properties in the post-development model have been fully resolved in the mitigation model, with impacts less than 20mm.

Positive affluxes are still observed in seven areas outside of the designation boundary; however, these increases do not affect any property buildings, with mitigation measures only considered if deemed necessary.

Appendices



Appendix A

B2P Flood Model Build Methodology

B2P Flood Model Development

Appendix A Table 1 provides an overview of the model build methodology of the B2P Flood Model for pre-development scenario (PREDEV) and summarises the hydrological and hydraulic parameters used. This also compares the parameters used in the B2P Flood model with the WDC South Ashley Model.

Appendix A Table 1 Comparison of WDC South Ashley Model and B2P Flood Model build

Parameter	WDC South Ashley Model	B2P Flood Model (PREDEV)
Software version	MIKE FLOOD	TUFLOW-2023-03-AC-iSP-w64
Projection	NZGD 2000 New Zealand Transverse Mercator	NZGD 2000 New Zealand Transverse Mercator
Vertical datum	Lyttelton Vertical Datum (LVD)	New Zealand Vertical Datum 2016 (NZVD16)
Computational and simulation parameters	- Minimum timestep: 1e-05s - Maximum timestep: 1s - Simulation time: 56hr - Results save timestep: 5min	1d timestep: 0.1s 2d timestep: 0.5s - Simulation time: 35hr - Time series output interval: 60s - Map out interval: 300s
Catchment delineation	Model extent covering Waimakariri district	- Project extent covering Pegasus in the north and Kaiapoi in the south. - Refer to Appendix A Catchment Delineation Section
Topography	- Model grid element size: 5-m rectangular flexible mesh grid 5-m LiDAR DEM 2014 from WDC South Ashley Model	- Model grid element size: 4-m rectangular grid with sub-grid sampling 1-m LiDAR DEM 2020 from LINZ server - Surveyed LiDAR for Kaiapoi Market Square pre-development
Hydrology		
Rainfall	24-hr nested storm profile as derived from HIRDS v4 for the RCP 8.5 (2081-2100) climate change scenario.	24-hr nested storm profile for 100-year and 200-year ARI as extracted from WDC South Ashley Model
Initial Condition	Catchment is assumed dry initially	Catchment is assumed dry initially
Boundary Condition	- Constant water level downstream boundary of 1.4m RL at the confluence of Kaiapoi River and Waimakariri River - Free outflow for the remaining areas downstream of the model	- Inflow boundaries (QT) applied upstream of the pipe network as extracted from WDC South Ashley Model - Water level boundaries (HT) applied downstream as extracted from WDC South Ashley Model - Refer to Appendix A Boundary Condition Section
Ground infiltration	- Infiltration method: MIKE21 infiltration and leakage module - Parameters such as initial infiltration, leakage, porosity, and initial saturation were applied	- Infiltration Method: TUFLOW Horton - Parameters such as initial loss, initial loss rate, final loss rate, porosity, and initial moisture were applied as extracted from WDC South Ashley Model - Exponential decay rate was applied as extracted from Waterways, Wetlands, and Drainage Guide of the Christchurch City Council - Refer to Appendix A Ground Infiltration Section

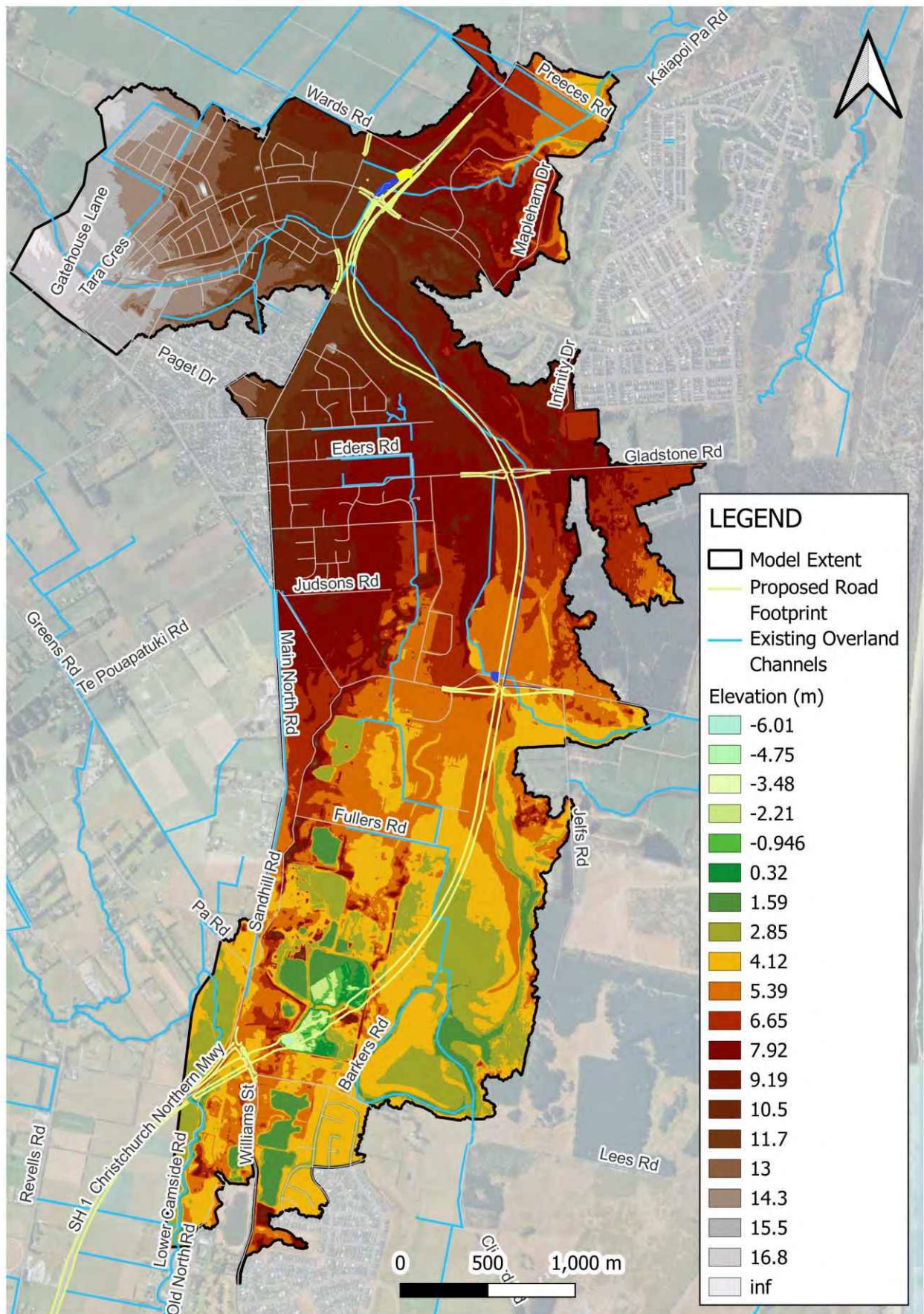
Parameter	WDC South Ashley Model	B2P Flood Model (PREDEV)
Surface roughness	Manning's surface roughness 'M' based on LCDB v4.1 database shapefiles	- Manning's surface roughness 'n' as extracted from WDC South Ashley Model - Refer to Appendix A Surface Roughness Section
Stormwater network		
Pipe network & Culverts	- Stormwater asset data covering the Waimakariri District - Culverts are not defined in the model. Where flow path is blocked by existing road, existing road level has been lowered down to provide overland flow path	- Models were run with the assumption of 100% pipe blockage. - For culvert modelling, refer to Section Stormwater Network and Culverts under Appendix A

Terrain

The topographic data used in the B2P Flood Model comprises of an on-going survey data collection and the 2020 LiDAR Digital Elevation Model (DEM) from LINZ. The survey covers the entire length of the designation of the proposed roadway bypass, including newly developed communities near the project site that were not covered in the 2020 LiDAR DEM. Once the survey data has been finalized, a new surface will be used to re-run the flood models. The Flood Mitigation Scenario Model will also be re-run with the finalised proposed roadway surface to analyse the effectivity of the measures with the ultimate design.

Catchment Delineation

The catchment was delineated using the 1m DEM LiDAR 2020 to determine the B2P Flood Model extent as shown in Appendix A Figure 1. The model extent is simplified by defining the catchment boundary covering the road alignment mostly in the towns of Pegasus, Woodend and Kaiapoi. Upper catchments were just represented as inflow boundary condition lines to avoid longer model runtime.



Appendix A Figure 1 B2P Flood Model catchment delineation

Hydrology

Rainfall

Rainfall was ingested into the model domain using a set of 50m x 50m resolution time varying 24-hour nested rainfall grid which was extracted from the WDC Flood Model. The rainfall grids have an hourly timestep with the first observation of rainfall starting at hour 1. As stated in Appendix A Table 1, the dataset was derived from HIRDS and incorporates a climate change scenario of RCP 8.5 for the period of 2081-2100.

Boundary Condition

The WDC Flood Model was the basis of all the boundary condition locations, where major drainage lines cross the extents of the model domain. Upstream flow BCs (QT) and downstream water level BCs (HT) were extracted also from the WDC Flood Model. Open boundaries, as represented by the slope, were calculated from the topographical data used in the TUFLOW model.

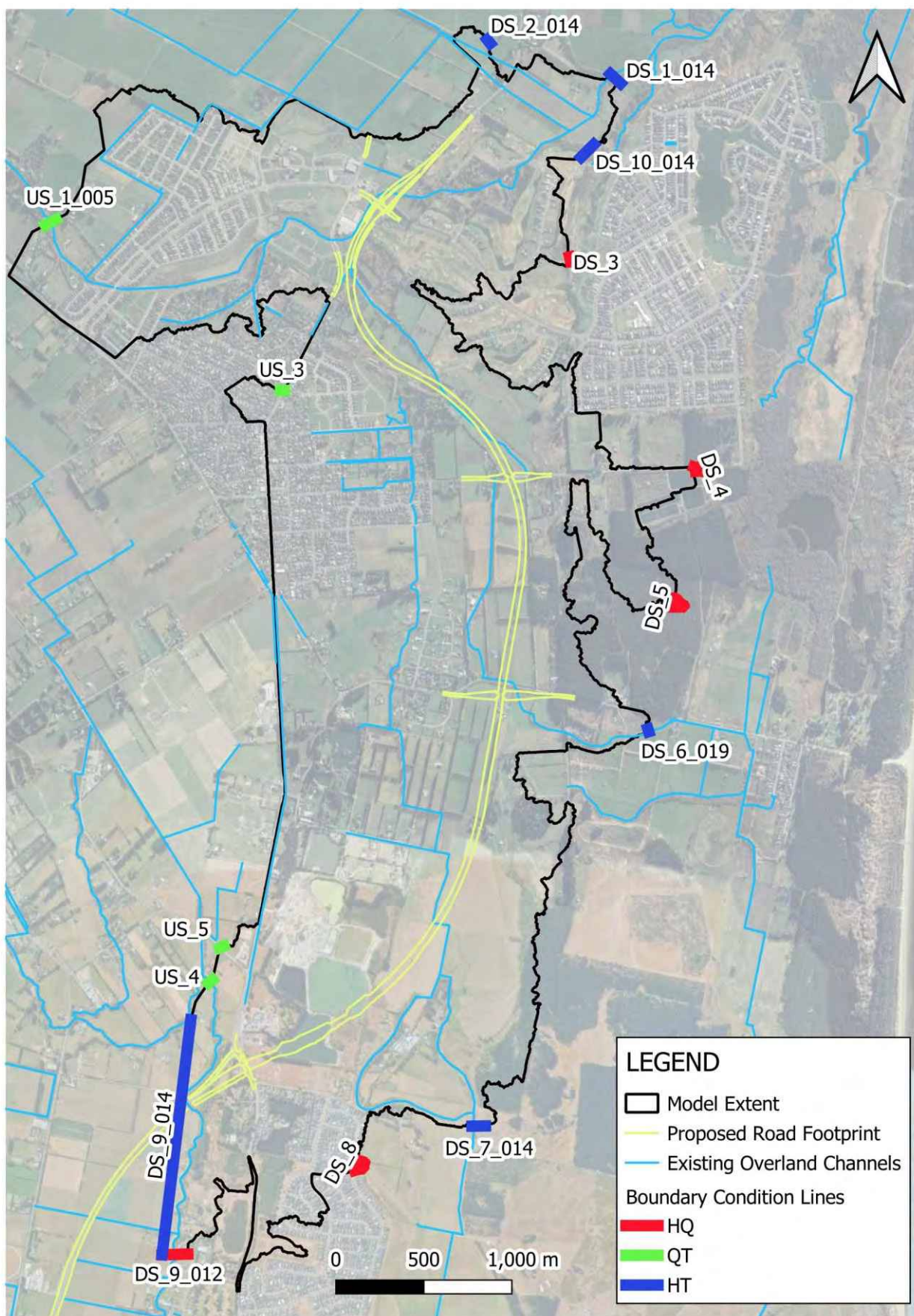
Flow hydrographs were used as an upstream boundary condition (US_1_005, US_3, US_4, US_5) along major drainage lines (e.g. rivers and streams) which are connected to upstream catchments and have been the source of water for the model domain. These are represented as a Flow vs. Time (QT) input with locations shown in Appendix A Figure 2. The 100- and 200-year ARI flow events are shown in Appendix A Figure 3.

Water level boundaries, Head vs Time (HT), are applied downstream of the model (DS_1_014, DS_2_014 and DS_10_014) at Waikuku to the north and at the western side of the Cam River (DS_9_014), as shown in Appendix A Table 2 and Appendix A Figure 4. These boundaries are influenced by other sources of water, specifically for areas near river or stream junctions that were not considered in the model domain. The same approach was applied to the floodplain at the southwestern boundary of the model (near Cam River) to simplify the movement of water since the WDC model shows that the area is a catch basin of water from different locations, apart from the Cam River.

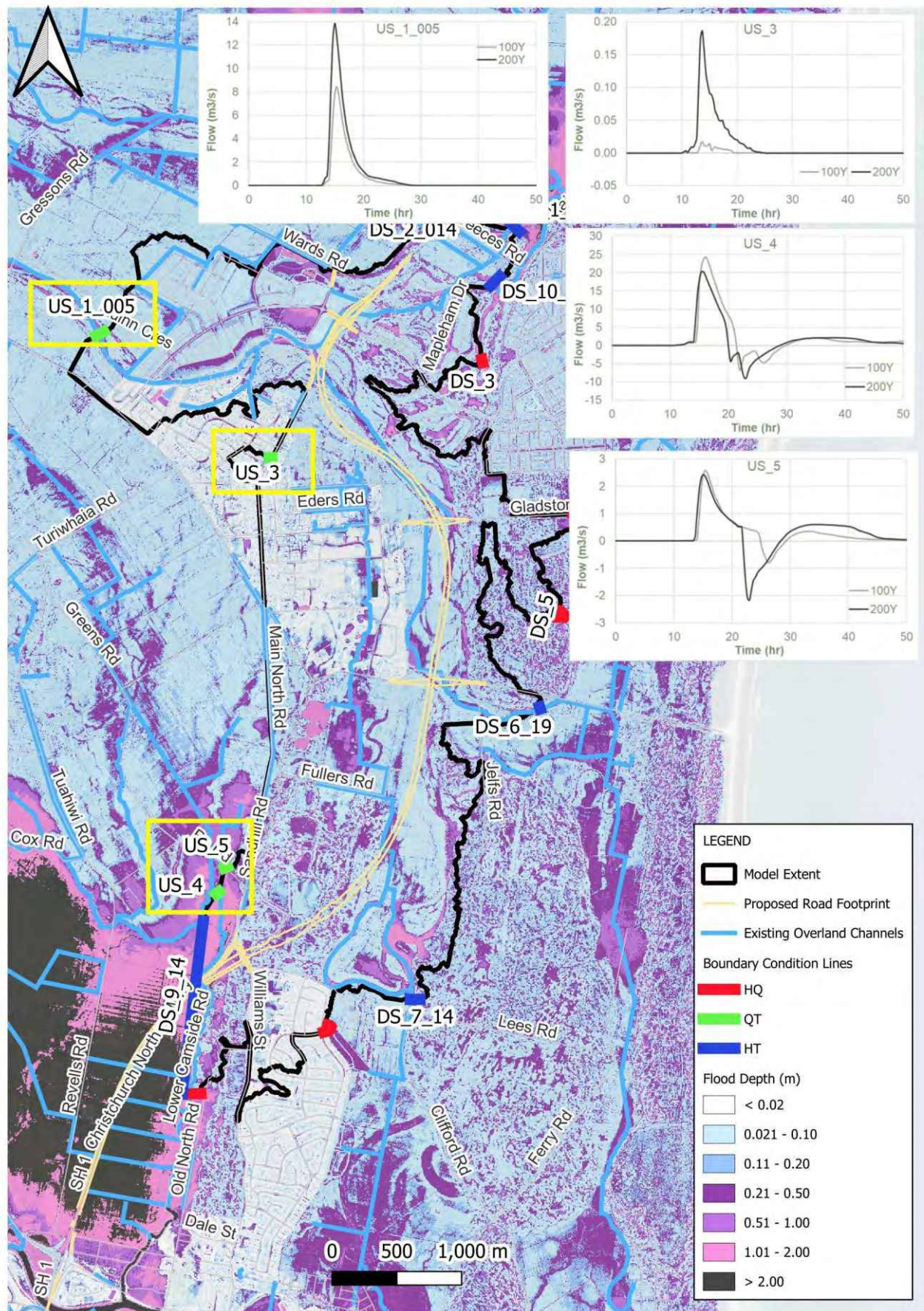
Water level boundaries Appendix A Figure 4 were also established at a major overland flow path discharging towards the east, near Woodend Beach Road (DS_6_019), and at a drain located east of quarry lake (DS_7_014). Both flow paths connect to a small drainage channel that can constrict flow in rare storm events, consequently funnelling water and producing tailwater conditions.

Free flow boundaries, as shown in

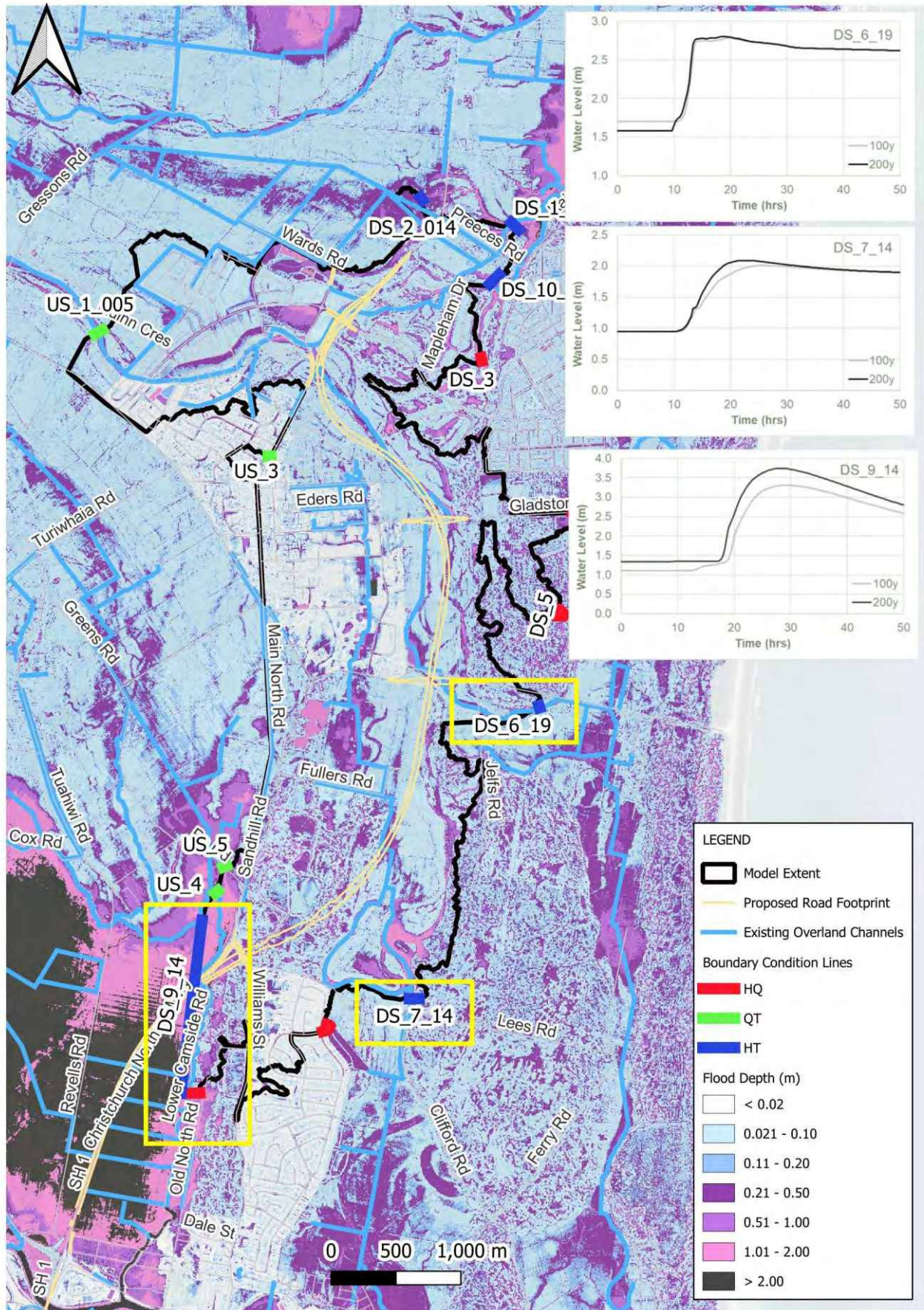
(HQ), have been placed at the downstream cross section of the Cam River (DS_9_012), and a few other streams that eventually discharges towards the east (DS_3, DS_4, DS_5, DS_8). These areas drain water freely out of the model domain.



Appendix A Figure 2 B2P Flood Model inflow and downstream boundaries overlaid on 100-year ARI flood depth result



Appendix A Figure 3 Boundary Conditions with Equivalent Inflow hydrographs



Appendix A Figure 4 Boundary Conditions with Equivalent Stage hydrographs

Appendix A Table 2 Water Level Downstream Boundary Conditions

BC Line	100-year Level (m)	200-year Level (m)
DS_1_014	2.93	2.94
DS_2_014	5.31	5.31
DS_10_014	2.93	2.94
DS_6_019	Varying Water Level (Appendix A Figure 4)	
DS_7_014		
DS_9_014		

Appendix A Table 3 Normal Depth Downstream Boundary Conditions

BC Line	Slope
DS_3	0.0036
DS_4	0.0041
DS_5	0.0041
DS_8	0.0008
DS_9_012	0.0008

Ground Infiltration

The soils infiltration losses are applied to the B2P Flood Model using the parameters of the Horton method. Parameters such as the dry loss, initial loss rate, final loss rate, porosity, and initial moisture were extracted from WDC South Ashley Model. Detailed information on ground infiltration methodology can be referred to Section 3.2.2 of 'Flood Hazard Models Update' report.

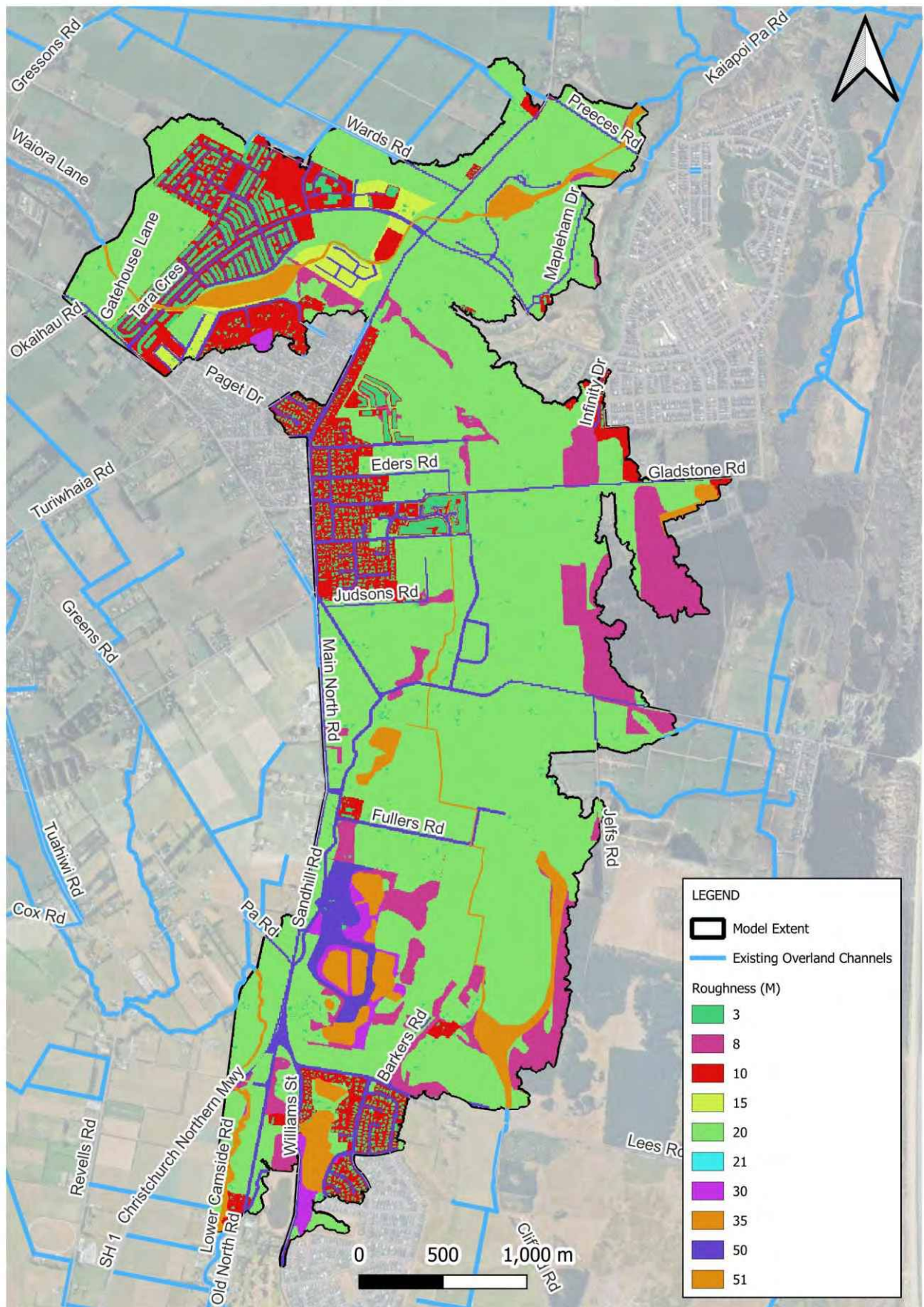
The exponential decay rate (k) parameter was determined based on the Waterways, Wetlands, and Drainage Guidelines (WWDG) of the Christchurch City Council (CCC), as detailed in Appendix A Table 4 decay rate was identified based on the initial infiltration rate extracted from WDC South Ashley Model.

Appendix A Table 4 Horton decay rate extracted from WWDG

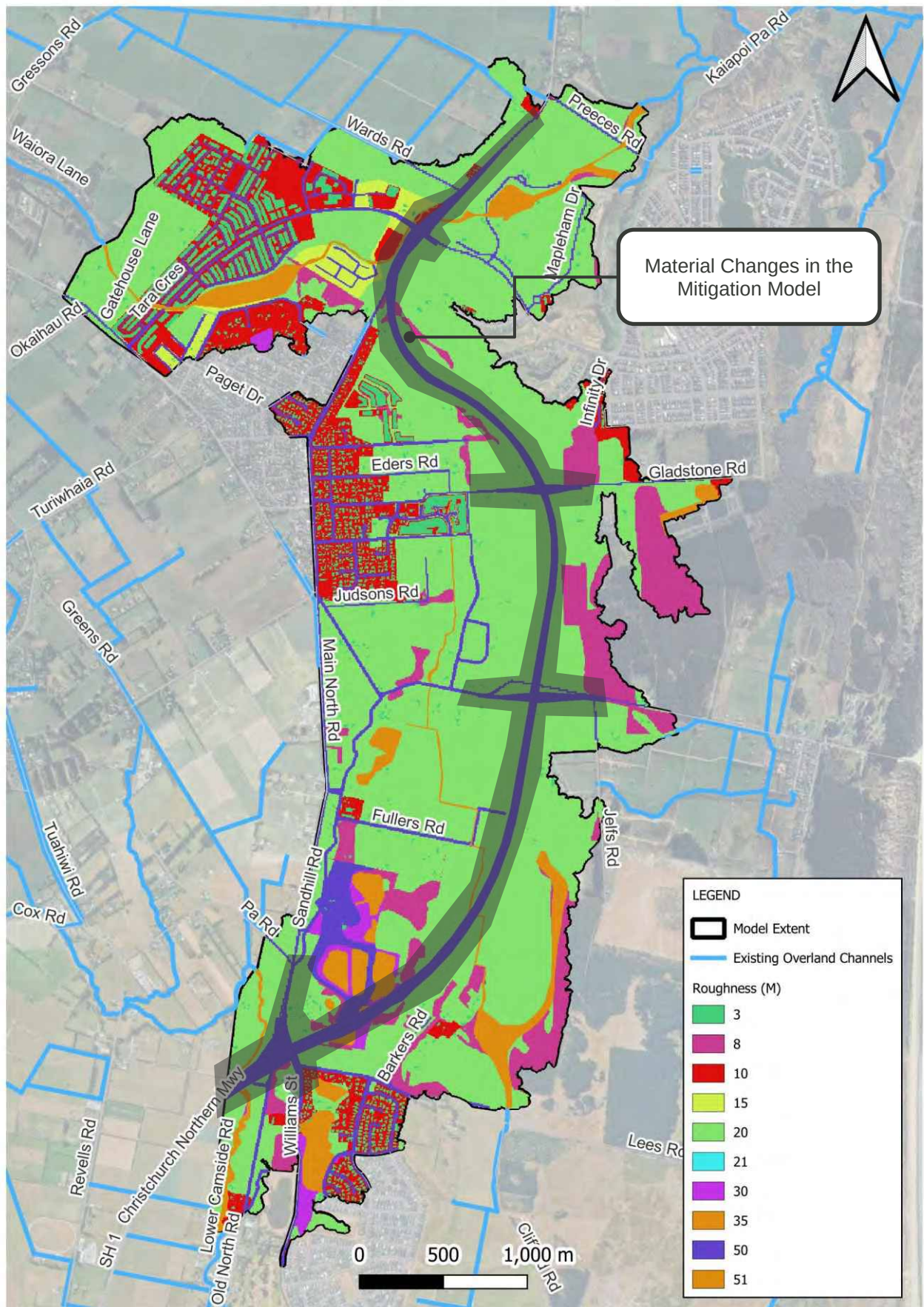
Initial infiltration rate (mm/hr)	Horton decay rate, k	Infiltration type
0 – 5	0.0015	Poor
5 – 10	0.0001	Moderate
10 – 15	0.00003	Free

Surface Roughness

The surface roughness as shown in Appendix A Figure 5 is applied to the B2P Flood Model. The surface roughness information is extracted from WDC South Ashley Model (refer to Appendix A Figure 7) A few changes to the surface roughness have been done in the Mitigation Scenario, to account for the proposed measures, such as the adjustment of the Taranaki Stream near Bob Robertson Drive (refer Appendix A Figure 6).



Appendix A Figure 5 Surface roughness applied in B2P Flood Models (Existing Scenario)



Appendix A Figure 6 Surface roughness applied in B2P Flood Models (Proposed Scenario)

Table 3-2: MIKE 21 roughness

Name_2012	Manning's M	Manning's n
Broadleaved Indigenous Hardwoods	8	0.125
Buildings	3	0.33
Built-up Area (settlement)	10	0.1
Deciduous Hardwoods	8	0.125
Depleted Grassland	50	0.02
Estuarine Open Water	35	0.029
Exotic Forest	8	0.125
Fernland	8	0.125
Flaxland	8	0.125
Forest - Harvested	8	0.125
Gorse and/or Broom	8	0.125
Gravel or Rock	50	0.02
Herbaceous Freshwater Vegetation	10	0.1
Herbaceous Saline Vegetation	10	0.1
High Producing Exotic Grassland	20	0.05
Indigenous Forest	8	0.125
Lake or Pond	35	0.029
Landslide	50	0.02
Low Producing Grassland	10	0.1
Manuka and/or Kanuka	8	0.125
Matagouri or Grey Scrub	10	0.1
Mixed Exotic Shrubland	20	0.05
Orchard, Vineyard or other Perennial Crop	20	0.05
River	35	0.029
Road	50	0.02
Sand or Gravel	50	0.02
Short-rotation Cropland	20	0.05
Sub Alpine Shrubland	20	0.05
Surface Mine or Dump	50	0.02
Tall Tussock Grassland	10	0.1
Transport Infrastructure	10	0.1
Urban Parkland/Open Space	20	0.05

Appendix A Figure 7 Surface roughness table extracted from 'Flood Hazard Models Update' report

Stormwater Network and Culverts

Appendix A Figure 8 shows the existing stormwater culverts along the major rivers and channels around the proposed development. For the B2P flood modelling purposes, urban pipe networks were assumed to be at full capacity during the 100-yr and 200-yr storm events, essentially not providing any storage and conveyance functionalities.

Geometric details of the pipes were extracted from the WDC database and were cross-referenced with the on-going topographical and drainage inventory survey along the designation of the proposed roadway. For areas that were not covered by the survey, details from the WDC database were considered absolute and were used whenever available since a few culverts lacked geometric details, such as the sizes and invert elevations.

Appendix A Table 5 shows the inventory of all the culverts modelled in the B2P Flood Model. It indicates which culvert were either new, maintained, adjusted, or removed in the Mitigation Scenario:

- Maintained – culverts were preserved from the Pre-Development Scenario Model since these were outside of the Proposed Roadway Designation and are not affected by the development.
- Adjusted – geometrical properties and/or alignment changes to the culvert. IDs of the adjusted culverts have been renamed in the Mitigation Model Scenario.
- Removed – decommissioned culverts due to changes of the overland flow path and have been affected by the Proposed Roadway Designation
- New Culvert – completely new culverts as part of the proposed mitigation measures

NOT FOR CONSTRUCTION

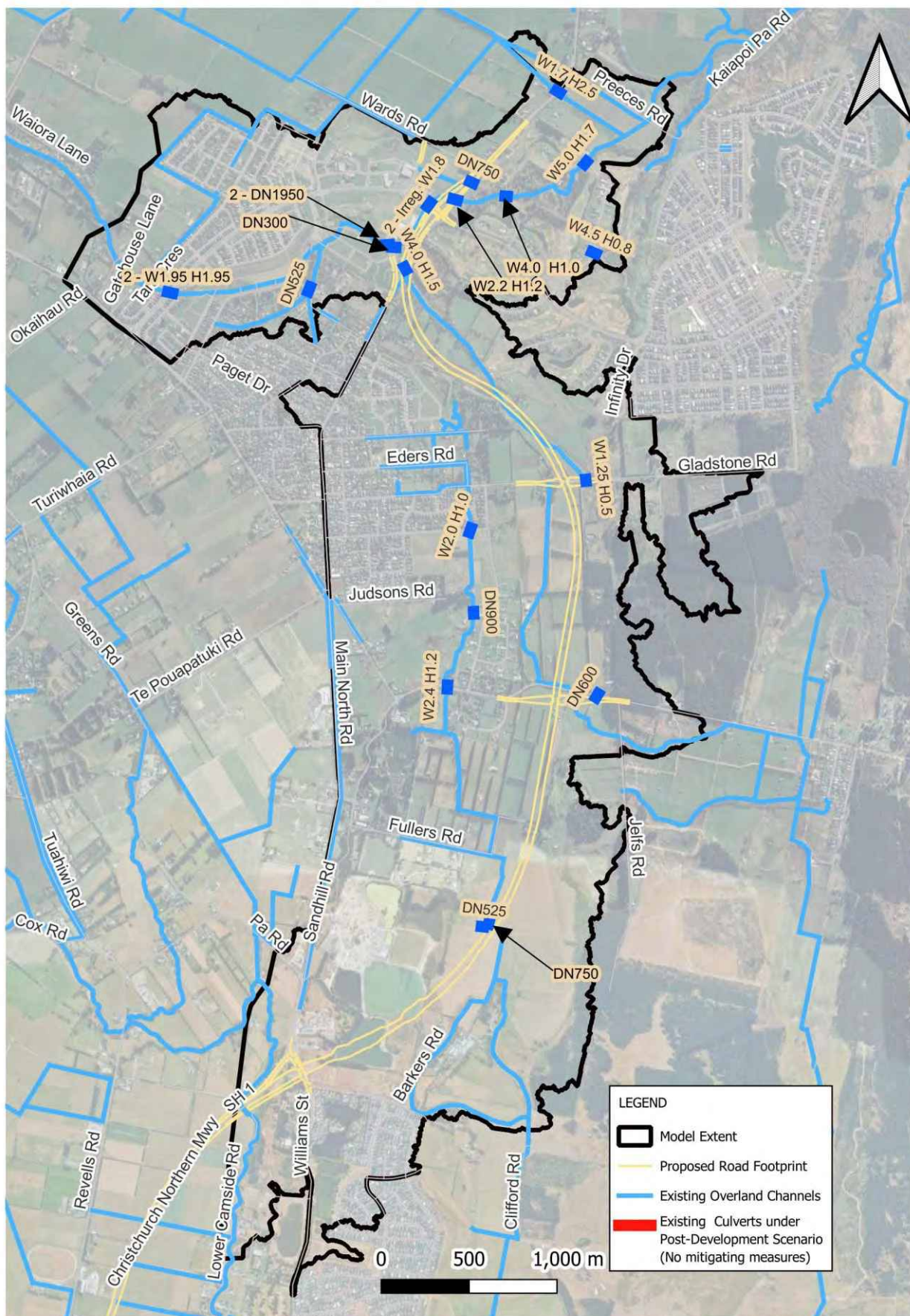
The table also details the culverts that are present in each model scenario (Pre-Development, Post-Development, or Mitigation).

Appendix A Table 5 Modelled Culvert Inventory

ID	Size	No. of Barrels	Roughness (Manning's n)	Source	Dummy Geometry [Y/N]	Fish Passage [Y/N]	Model Scenario Present	MITIG Scenario Status
C001	DN525	1	0.013	Stormwater_Pipes -6254-SW007089	N	N	PREDEV, POSTDEV, MITIG	Maintained
C002	DN900	1	0.013	Stormwater_Pipes -4248-SW022584	N	N	PREDEV, POSTDEV, MITIG	Maintained
C003	DN1950	2	0.013	Survey	N	N	PREDEV, POSTDEV, MITIG	Maintained
C004	DN600	1	0.013	Survey	N	N	PREDEV, POSTDEV, MITIG	Maintained
C005	DN750	1	0.013	Survey	N	N	PREDEV, POSTDEV, MITIG	Maintained
C006	DN525	1	0.013	Survey	N	N	PREDEV, POSTDEV, MITIG	Maintained
C007	DN1800	2	0.0203	Survey	N	Y [300mm Embedment]	PREDEV, POSTDEV, MITIG	Maintained
C008	DN300	1	0.013	Survey	N	N	PREDEV, POSTDEV, MITIG	Maintained
EC_01	DN750	1	0.013	Survey	N	N	PREDEV, MITIG	Adjusted (MITIG ID: EC_01)
R001	4.0m X 1.5m	1	0.013	Survey	N	N	PREDEV, MITIG	Adjusted (MITIG ID: NC_02)
R002	2.0m X 1.0m	1	0.013	-	Y	N	PREDEV, POSTDEV, MITIG	Maintained
R003	2.4m X 1.2m	1	0.013	-	Y	N	PREDEV, POSTDEV, MITIG	Maintained
R004	5.0m X 1.7m	1	0.013	-	Y	N	PREDEV, POSTDEV, MITIG	Maintained
R005	4.0m X 1.0m	1	0.013	-	Y	N	PREDEV, POSTDEV, MITIG	Maintained
R006	4.5m X 0.8m	1	0.013	-	Y	N	PREDEV, POSTDEV, MITIG	Maintained
R007	1.95m X 1.95m	2	0.013	-	Y	N	PREDEV, POSTDEV, MITIG	Maintained
R008	1.25m X 0.5m	1	0.013	Survey	N	N	PREDEV, MITIG	Removed

NOT FOR CONSTRUCTION

ID	Size	No. of Barrels	Roughness (Manning's n)	Source	Dummy Geometry [Y/N]	Fish Passage [Y/N]	Model Scenario Present	MITIG Scenario Status
R009	2.2m X 1.2m	1	0.013	Survey	N	N	PREDEV, MITIG	Adjusted (MITIG ID: NC_01a)
R010	1.7m X 2.5m	1	0.013	Survey	N	N	PREDEV, POSTDEV, MITIG	Maintained
NC_01a	2.7m X 1.2m	1	0.0233	-	-	Y [300mm Embedment]	MITIG	New Culvert
NC_02	4.0m X 1.5m	1	0.026	-	-	Y [300mm Embedment]	MITIG	New Culvert
NC_04	DN600	1	0.013	-	-	N	MITIG	New Culvert
NC_05	DN1050	2	0.013	-	-	N	MITIG	New Culvert
NC_06	DN1050	2	0.013	-	-	N	MITIG	New Culvert
NC_06b	2.0m X 1.0m	2	0.013	-	-	N	MITIG	New Culvert
NC_07	3.0m X 1.7m	2	0.0233	-	-	Y [300mm Embedment]	MITIG	New Culvert
NC_08	DN600	1	0.013	-	-	N	MITIG	New Culvert



Appendix A Figure 8 Existing culverts within the proposed project site

Flood Modelling Scenarios

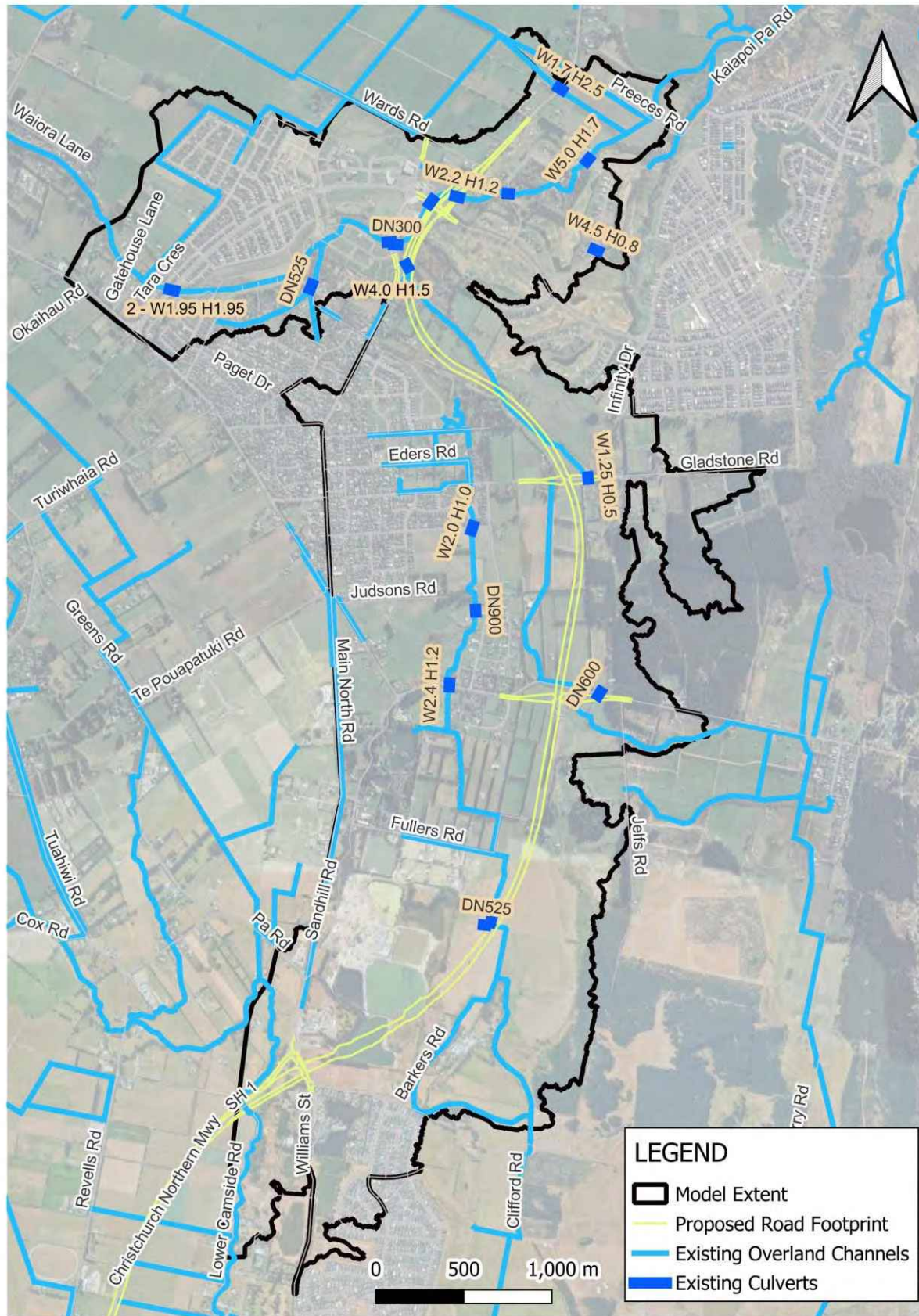
Appendix A Table 6 summarises the baseline and design scenarios that were modelled in the study for the flood impact assessment of the road development. This includes the details and corresponding changes in each modelled scenario, as well as figures for reference.

Appendix A Table 6 List of modelled scenarios

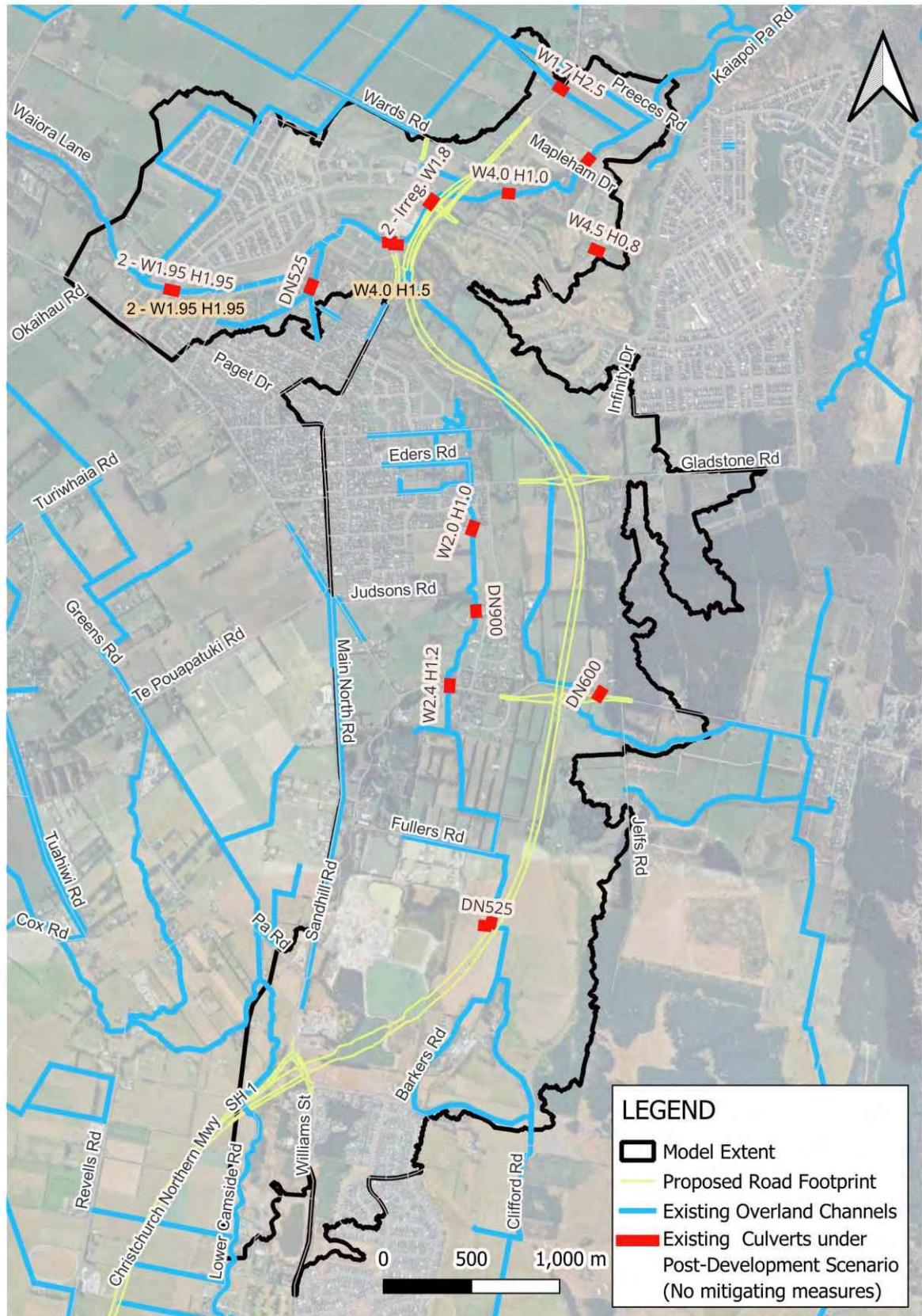
Item	Figure	Modelled Scenarios
1	Figure B-1	PREDEV: Existing/pre-development scenario • Baseline model for flood impact assessment • Refer to Table A-11 for B2P Flood Model setup
2	Figure B-2	POSTDEV: Design/post-development scenario with proposed B2P road alignment included • EXST with the following changes: - o included design surface of proposed B2P road development - o modifications on surface roughness to represent proposed B2P road alignment.
3	Figure B-3	MITIG: Design/post-development scenario with mitigation solution implemented • DES with the following changes: - o Propose additional culverts - o Propose additional flow path channels.

Appendix B

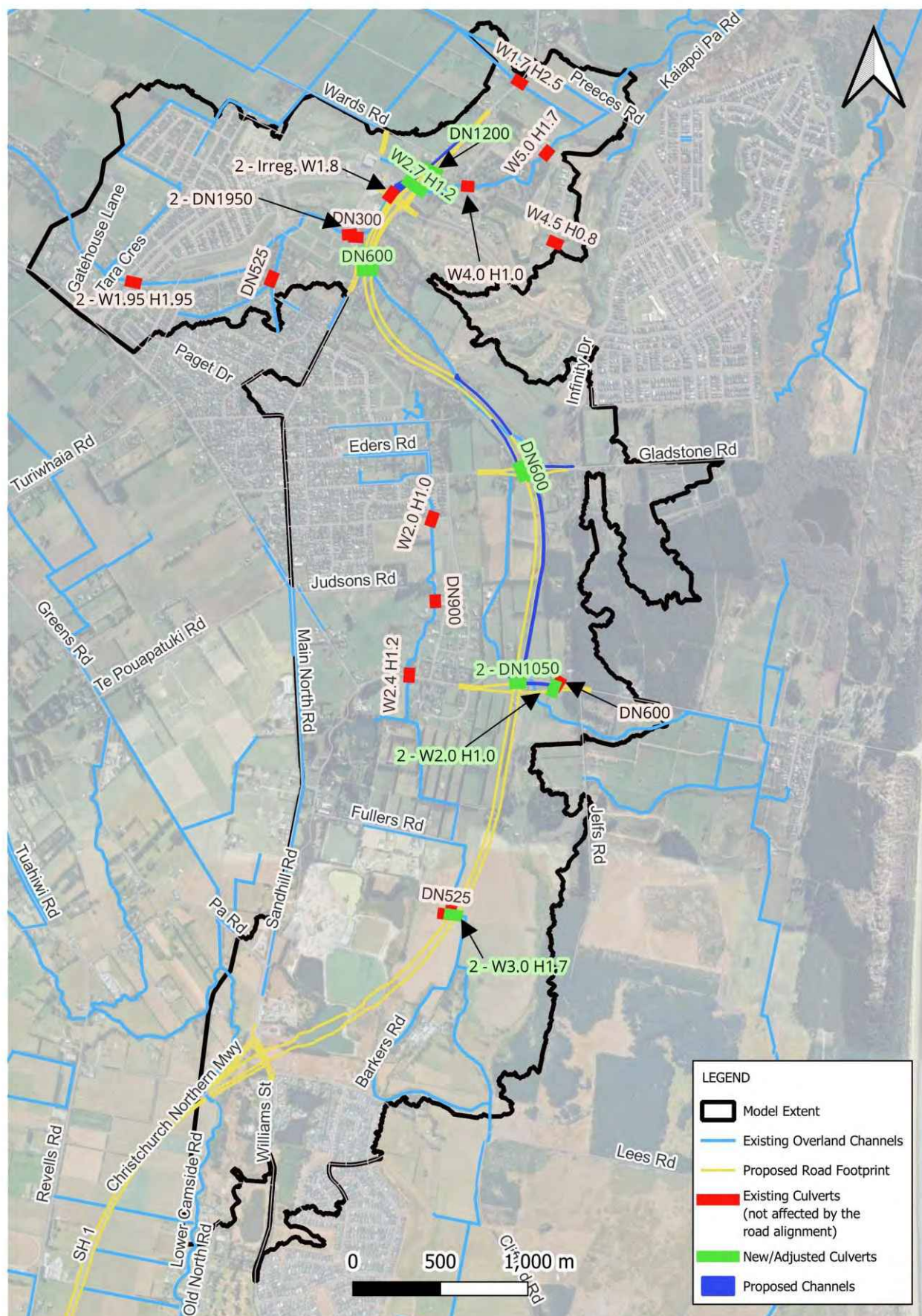
B2P Flood Model Modifications and Input Data



Appendix B Figure 1 Model Setup for the pre-development scenario



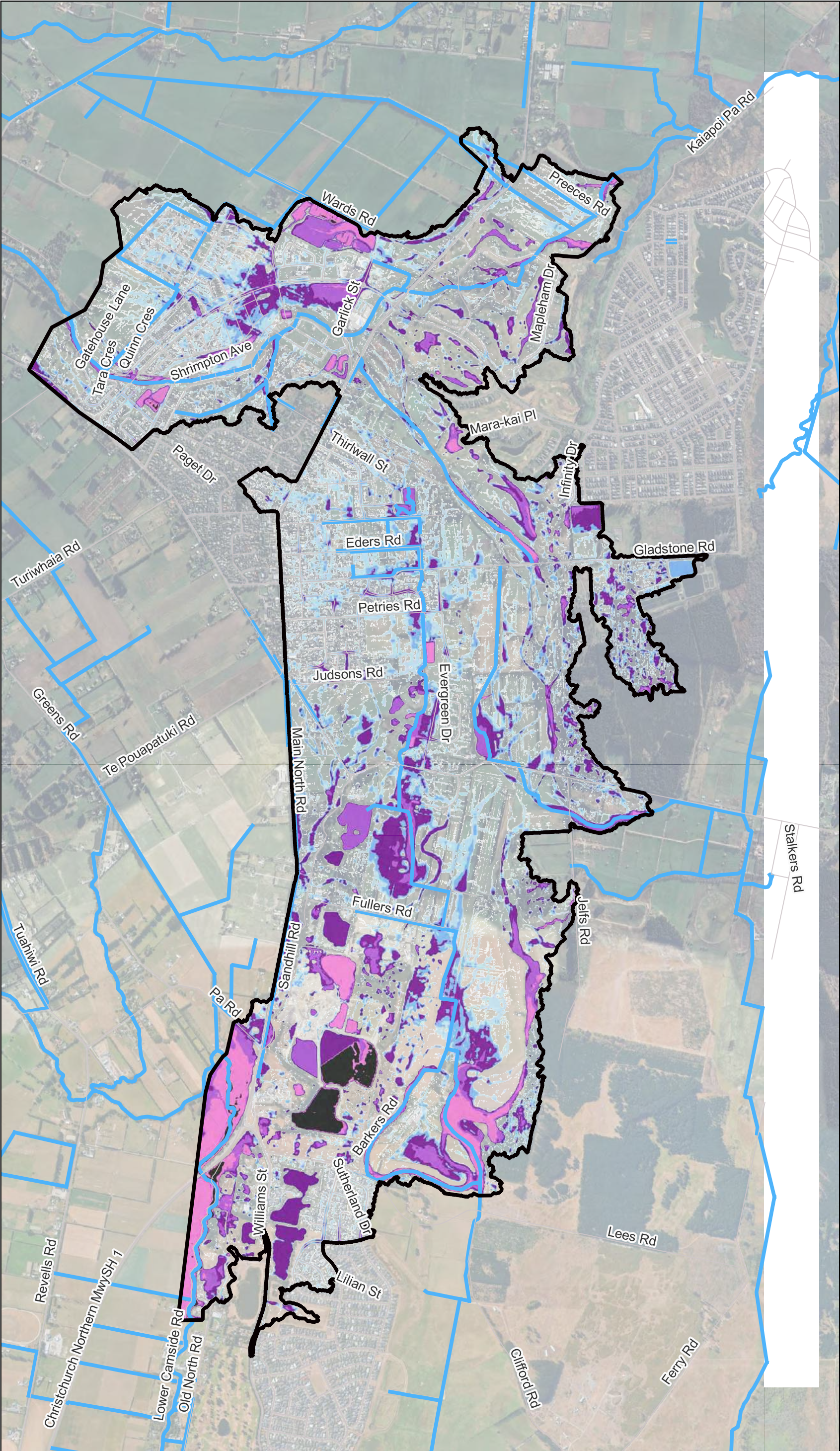
Appendix B Figure 2 Model Setup for the post-development scenario



Appendix C

Woodend Flood Model Results

Item	Figure	Modelled Scenarios
1	Figure C-1	Pre-development Model (PREDEV) Flood Depth - 100yr ARI Storm
2	Figure C-2	Pre-development Model (PREDEV) Flood Level - 100yr ARI Storm
3	Figure C-3	Pre-development Model (PREDEV) Flood Depth – 200yr ARI Storm
4	Figure C-4	Pre-development Model (PREDEV) Flood Level – 200yr ARI Storm
5	Figure C-5	Post-development Model (POSTDEV) Flood Depth - 100yr ARI Storm
6	Figure C-6	Post-development Model (POSTDEV) Flood Level - 100yr ARI Storm
7	Figure C-7	Post-development Model (POSTDEV) Flood Depth – 200yr ARI Storm
8	Figure C-8	Post-development Model (POSTDEV) Flood Level – 200yr ARI Storm
9	Figure C-9	Flood Level Difference – Post-development (POSTDEV) minus Pre-development (PREDEV) - 100yr ARI Storm
10	Figure C-10	Flood Level Difference – Post-development (POSTDEV) minus Pre-development (PREDEV) – 200yr ARI Storm
11	Figure C-11	Post-development Model (MITIG) Flood Depth – 100yr ARI Storm
12	Figure C-12	Post-development Model (MITIG) Flood Level – 100yr ARI Storm
13	Figure C-13	Post-development Model (MITIG) Flood Depth – 200yr ARI Storm
14	Figure C-14	Post-development Model (MITIG) Flood Level – 200yr ARI Storm
15	Figure C-15	Flood Level Difference – Post-development (MITIG) minus Pre-development (PREDEV) – 100yr ARI Storm
16	Figure C-16	Flood Level Difference – Post-development (MITIG) minus Pre-development (PREDEV – 200yr ARI Storm

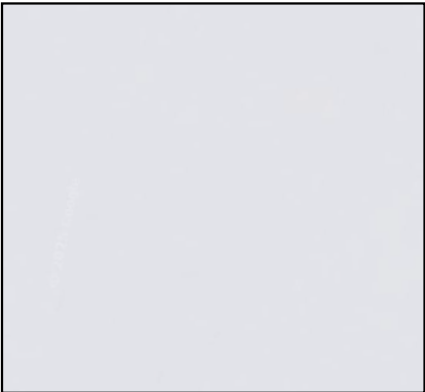


LEGEND

Model Extent

Flood Depth (m)

- < 0.02
- 0.021 - 0.10
- 0.11 - 0.20
- 0.21 - 0.50
- 0.51 - 1.00
- 1.01 - 2.00
- > 2.00



Notes:

Hydraulic TUFLOW Model

Model Cell Size:
4m x 4m SGS with Quad Tree

LIDAR Cell Size:
1m x 1m

Output Grids Cell Size:
1m x 1m

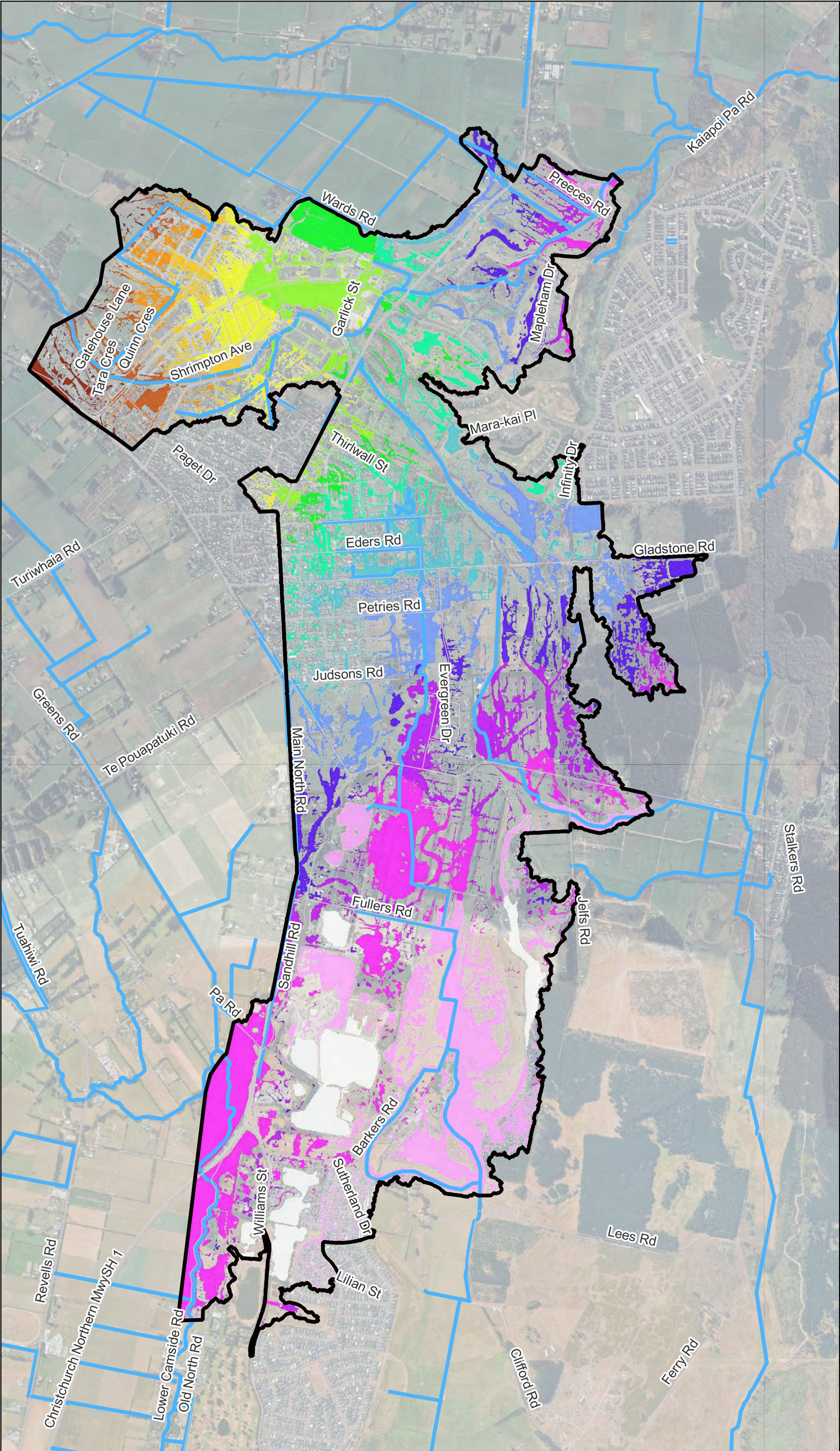
Run Scenarios:

PREDEV - Existing topography

POSTDEV- Existing topography
+ Proposed Road
Embankment

MITIG -Post-development
scenario + mitigating measures
(add'l culverts, channels etc.)

Water Level Differencing Notes:
The water level afflux is not
displayed on the map when
either the pre-development or
mitigated/post-development
scenarios indicate dry
conditions.



LEGEND

Model Extent

Roads

Existing Overland Channels

Flood Level (m)

<= 2

2.001 - 3

3.001 - 4

4.001 - 5

5.001 - 6

6.001 - 7

7.001 - 8

8.001 - 9

9.001 - 10

10.001 - 11

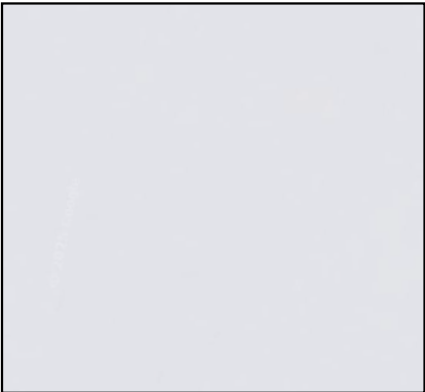
11.001 - 12

12.001 - 13

13.001 - 14

14.001 - 15

> 15



Notes:

Hydraulic TUFLOW Model

Model Cell Size:
4m x 4m SGS with Quad Tree

LIDAR Cell Size:
1m x 1m

Output Grids Cell Size:
1m x 1m

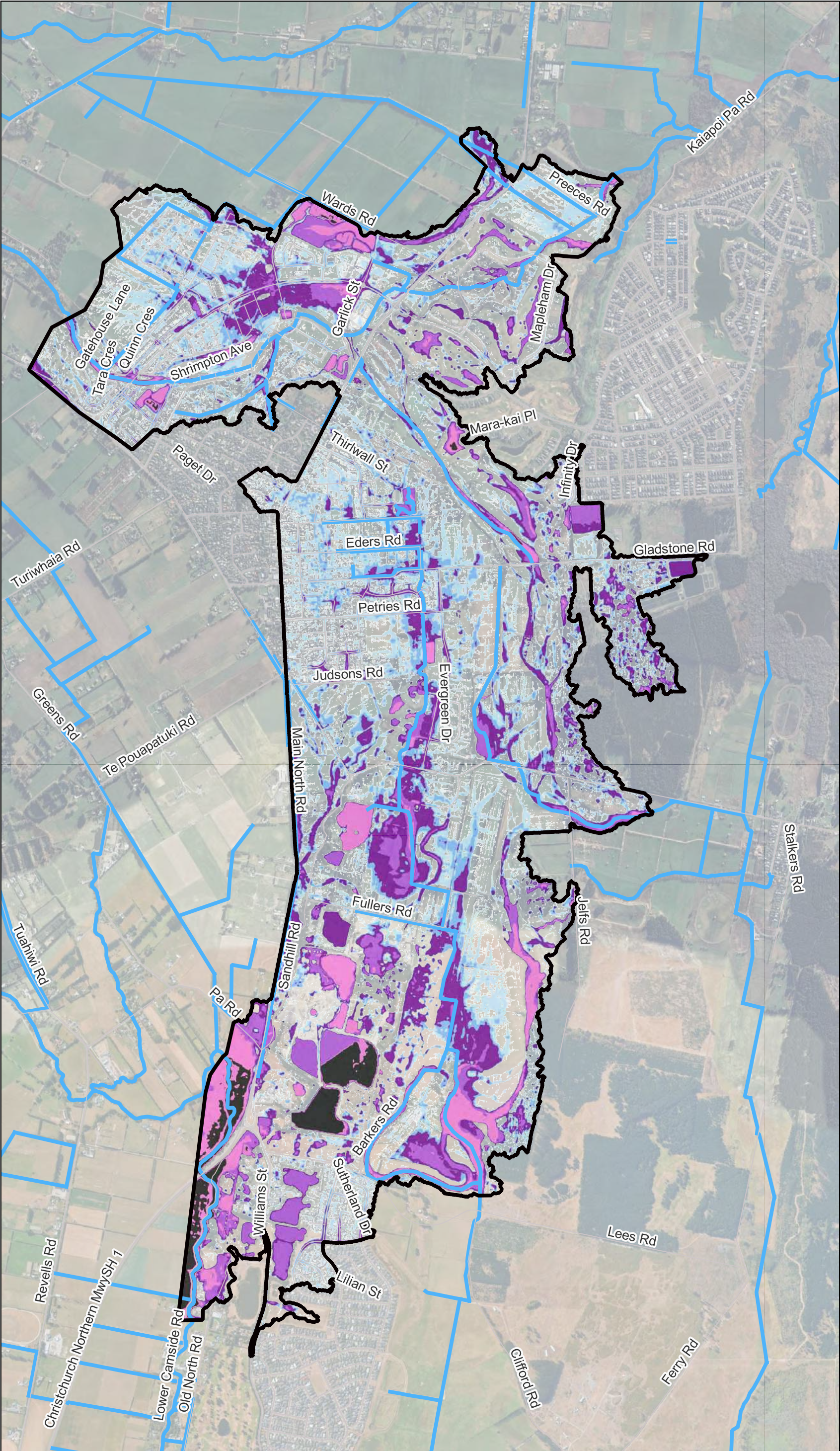
Run Scenarios:

PREDEV - Existing topography

POSTDEV- Existing topography
+ Proposed Road
Embankment

MITIG -Post-development
scenario + mitigating measures
(add'l culverts, channels etc.)

Water Level Differencing Notes:
The water level afflux is not
displayed on the map when
either the pre-development or
mitigated/post-development
scenarios indicate dry
conditions.



LEGEND

Model Extent 

Roads 

Existing Overland Channels 

Flood Depth (m)

< 0.02 

0.021 - 0.10 

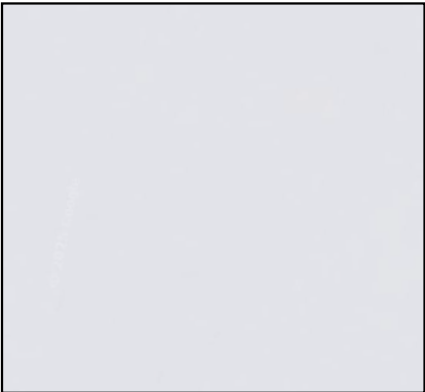
0.11 - 0.20 

0.21 - 0.50 

0.51 - 1.00 

1.01 - 2.00 

> 2.00 



Notes:

Hydraulic TUFLOW Model

Model Cell Size:
4m x 4m SGS with Quad Tree

LIDAR Cell Size:
1m x 1m

Output Grids Cell Size:
1m x 1m

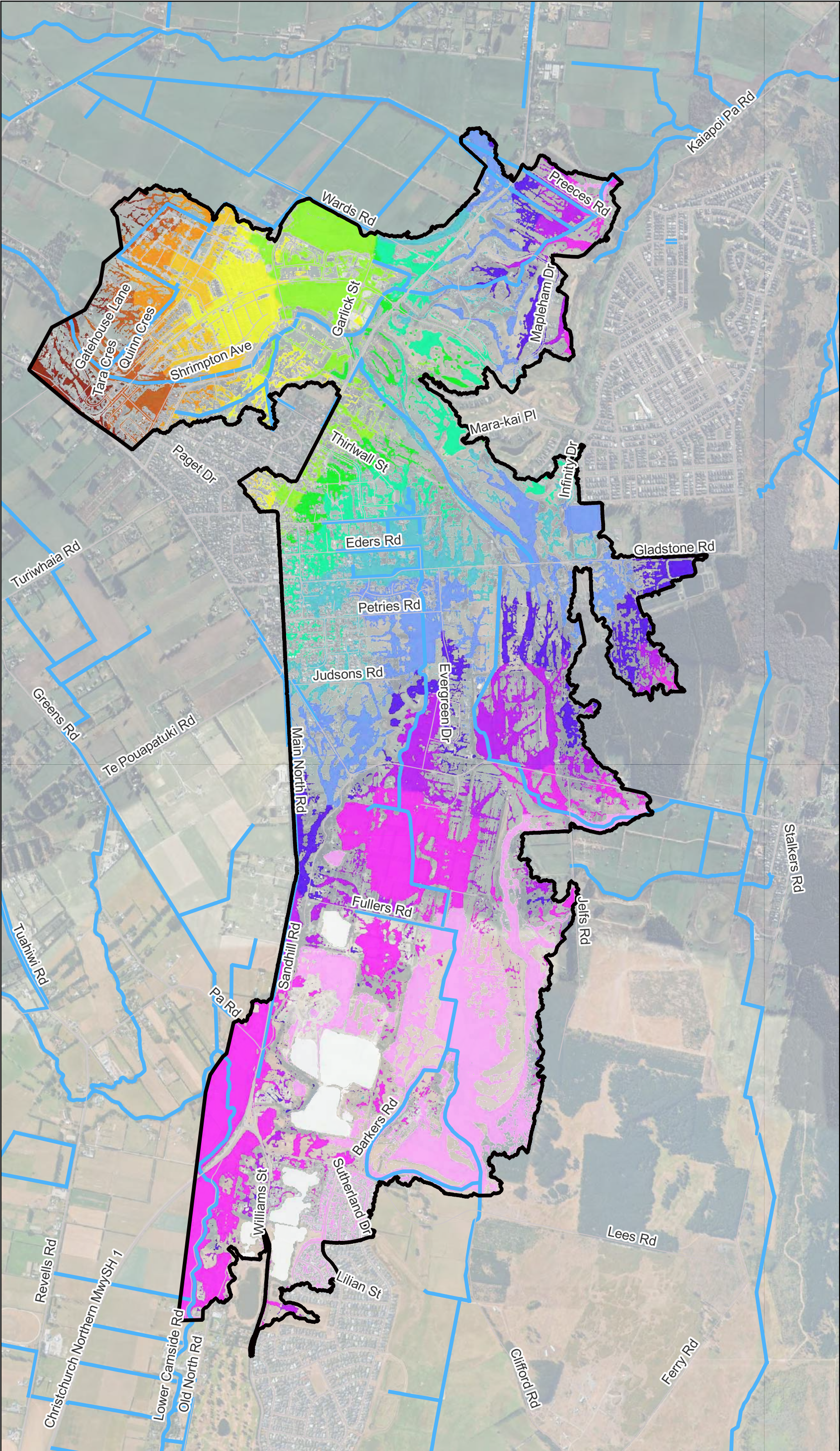
Run Scenarios:

PREDEV - Existing topography

POSTDEV- Existing topography
+ Proposed Road
Embankment

MITIG -Post-development
scenario + mitigating measures
(add'l culverts, channels etc.)

Water Level Differencing Notes:
The water level afflux is not
displayed on the map when
either the pre-development or
mitigated/post-development
scenarios indicate dry
conditions.



LEGEND

Model Extent

Roads

Existing Overland Channels

Flood Level (m)

<= 2

2.001 - 3

3.001 - 4

4.001 - 5

5.001 - 6

6.001 - 7

7.001 - 8

8.001 - 9

9.001 - 10

10.001 - 11

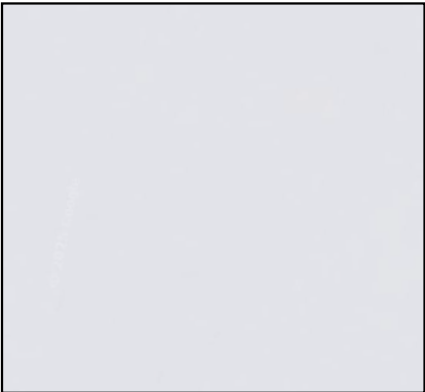
11.001 - 12

12.001 - 13

13.001 - 14

14.001 - 15

> 15



Notes:

Hydraulic TUFLOW Model

Model Cell Size:
4m x 4m SGS with Quad Tree

LIDAR Cell Size:
1m x 1m

Output Grids Cell Size:
1m x 1m

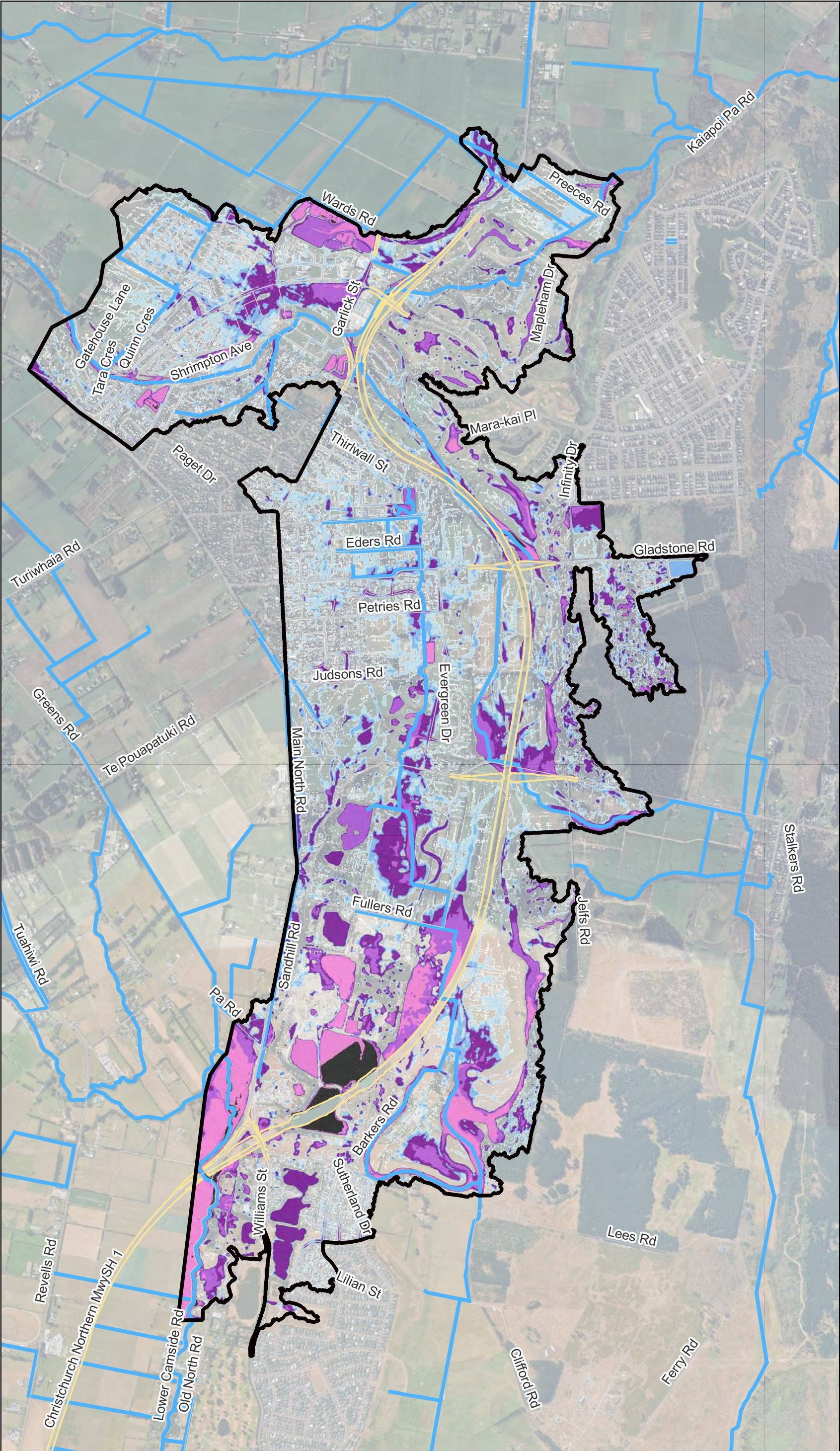
Run Scenarios:

PREDEV - Existing topography

POSTDEV- Existing topography
+ Proposed Road
Embankment

MITIG -Post-development
scenario + mitigating measures
(add'l culverts, channels etc.)

Water Level Differencing Notes:
The water level afflux is not
displayed on the map when
either the pre-development or
mitigated/post-development
scenarios indicate dry
conditions.



LEGEND

Model Extent 

Roads 

Proposed Road Footprint 

Existing Overland Channels 

Flood Depth (m)

< 0.02 

0.021 - 0.10 

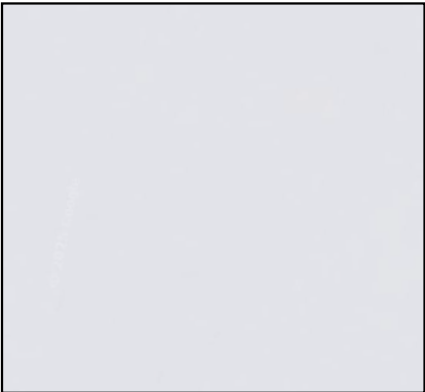
0.11 - 0.20 

0.21 - 0.50 

0.51 - 1.00 

1.01 - 2.00 

> 2.00 



Notes:

Hydraulic TUFLOW Model

Model Cell Size:
4m x 4m SGS with Quad Tree

LIDAR Cell Size:
1m x 1m

Output Grids Cell Size:
1m x 1m

Run Scenarios:

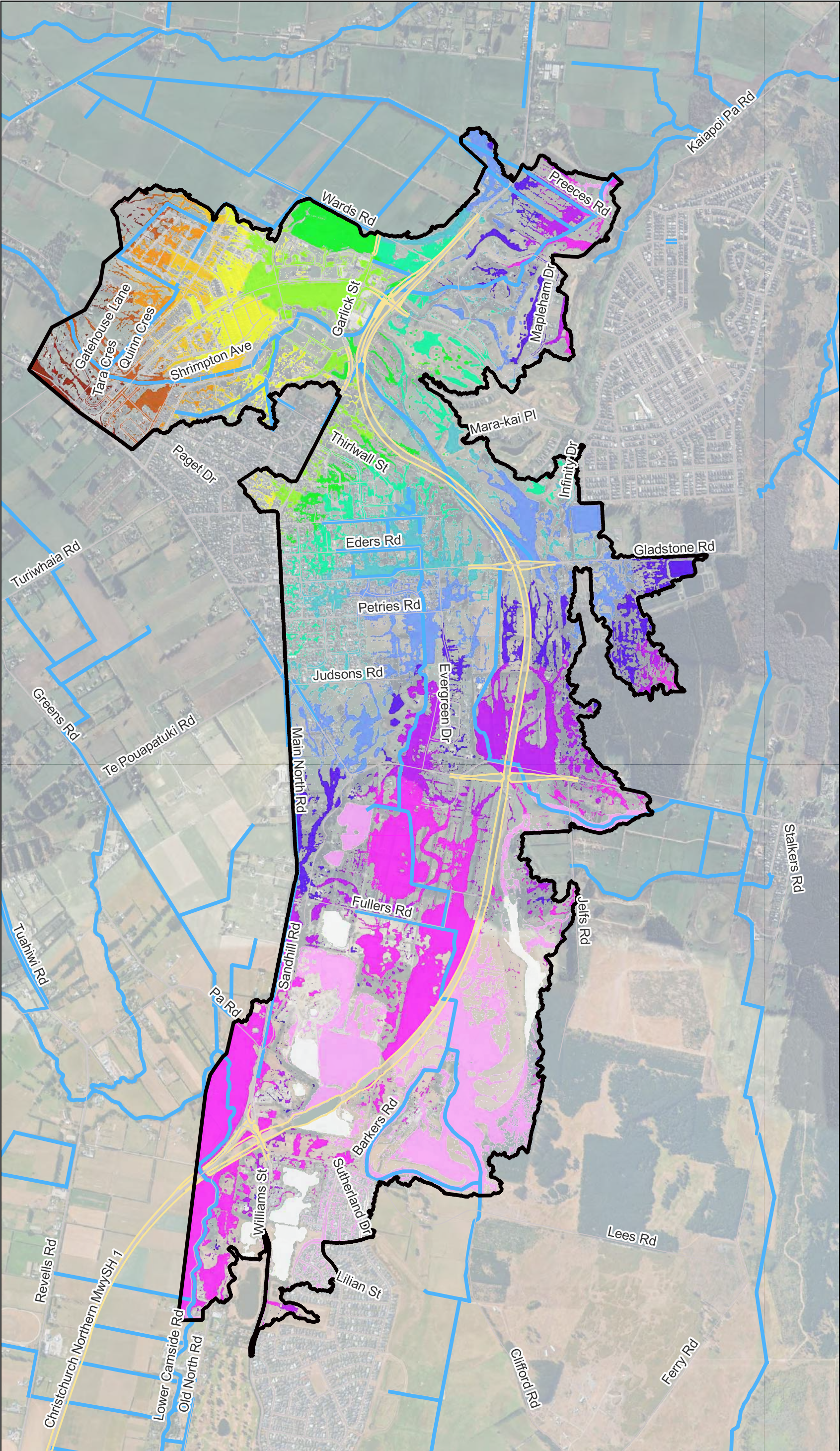
PREDEV - Existing topography

POSTDEV- Existing topography
+ Proposed Road
Embankment

MITIG -Post-development
scenario + mitigating measures
(add'l culverts, channels etc.)

Water Level Differencing Notes:
The water level afflux is not
displayed on the map when
either the pre-development or
mitigated/post-development
scenarios indicate dry
conditions.





LEGEND

Model Extent

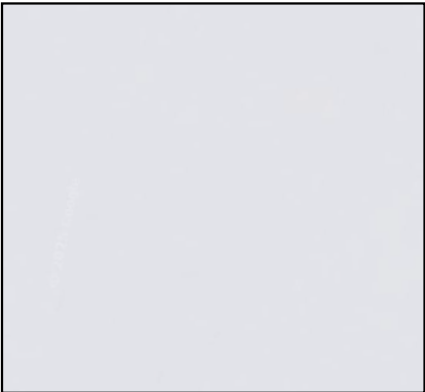
Roads

Proposed Road Footprint

Existing Overland Channels

Flood Level (m)

- <= 2
- 2.001 - 3
- 3.001 - 4
- 4.001 - 5
- 5.001 - 6
- 6.001 - 7
- 7.001 - 8
- 8.001 - 9
- 9.001 - 10
- 10.001 - 11
- 11.001 - 12
- 12.001 - 13
- 13.001 - 14
- 14.001 - 15
- > 15



Notes:

Hydraulic TUFLOW Model

Model Cell Size:
4m x 4m SGS with Quad Tree

LIDAR Cell Size:
1m x 1m

Output Grids Cell Size:
1m x 1m

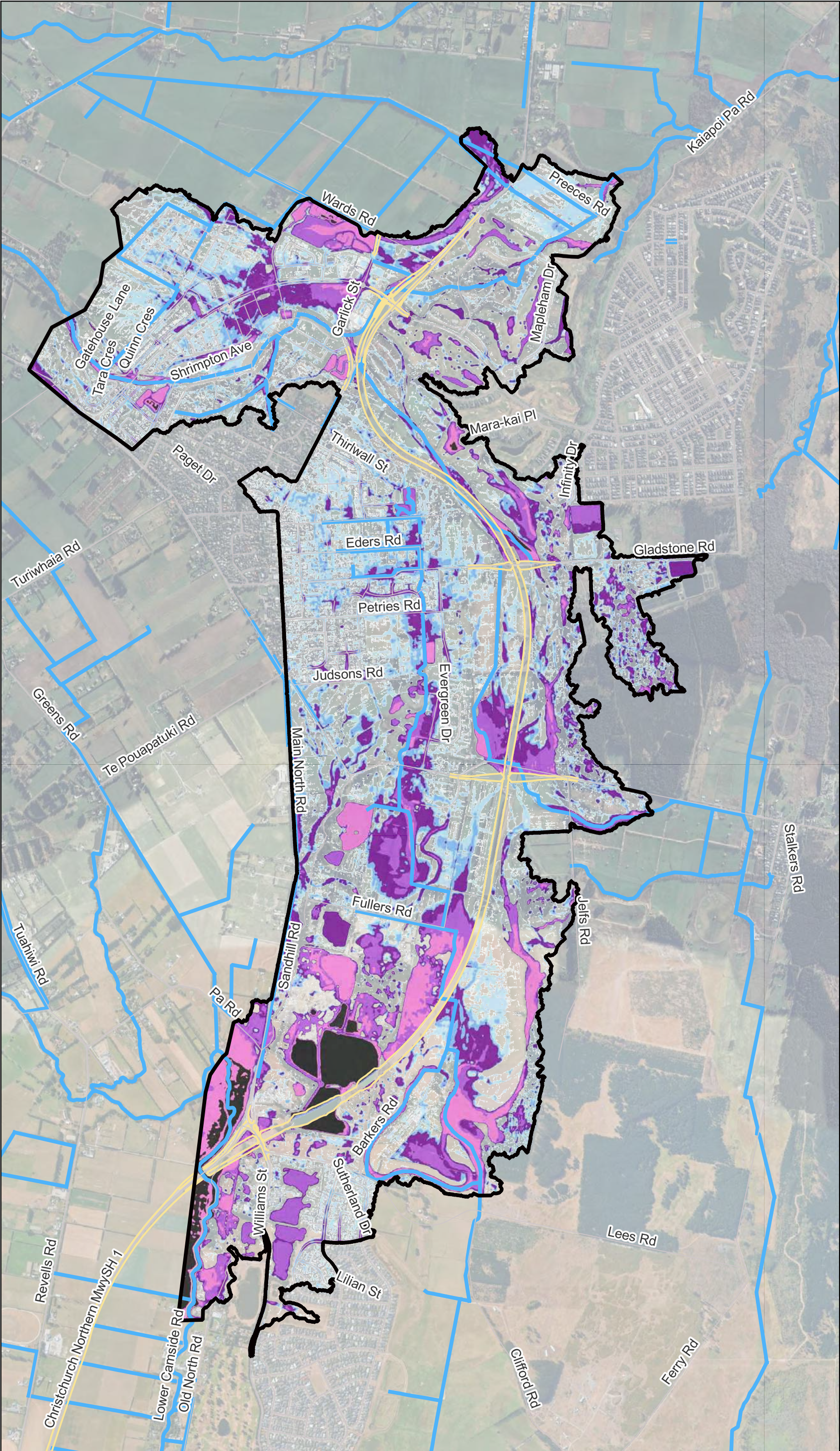
Run Scenarios:

PREDEV - Existing topography

POSTDEV- Existing topography
+ Proposed Road
Embankment

MITIG -Post-development
scenario + mitigating measures
(add'l culverts, channels etc.)

Water Level Differencing Notes:
The water level afflux is not
displayed on the map when
either the pre-development or
mitigated/post-development
scenarios indicate dry
conditions.



LEGEND

Model Extent

Roads

Proposed Road Footprint

Existing Overland Channels

Flood Depth (m)

< 0.02

0.021 - 0.10

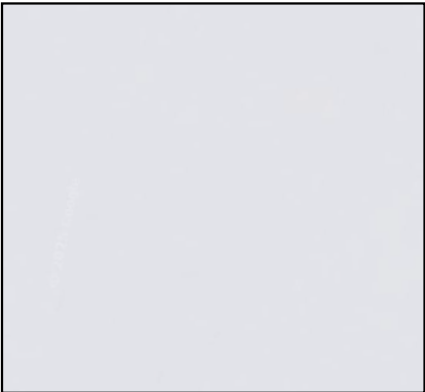
0.11 - 0.20

0.21 - 0.50

0.51 - 1.00

1.01 - 2.00

> 2.00



Notes:

Hydraulic TUFLOW Model

Model Cell Size:
4m x 4m SGS with Quad Tree

LIDAR Cell Size:
1m x 1m

Output Grids Cell Size:
1m x 1m

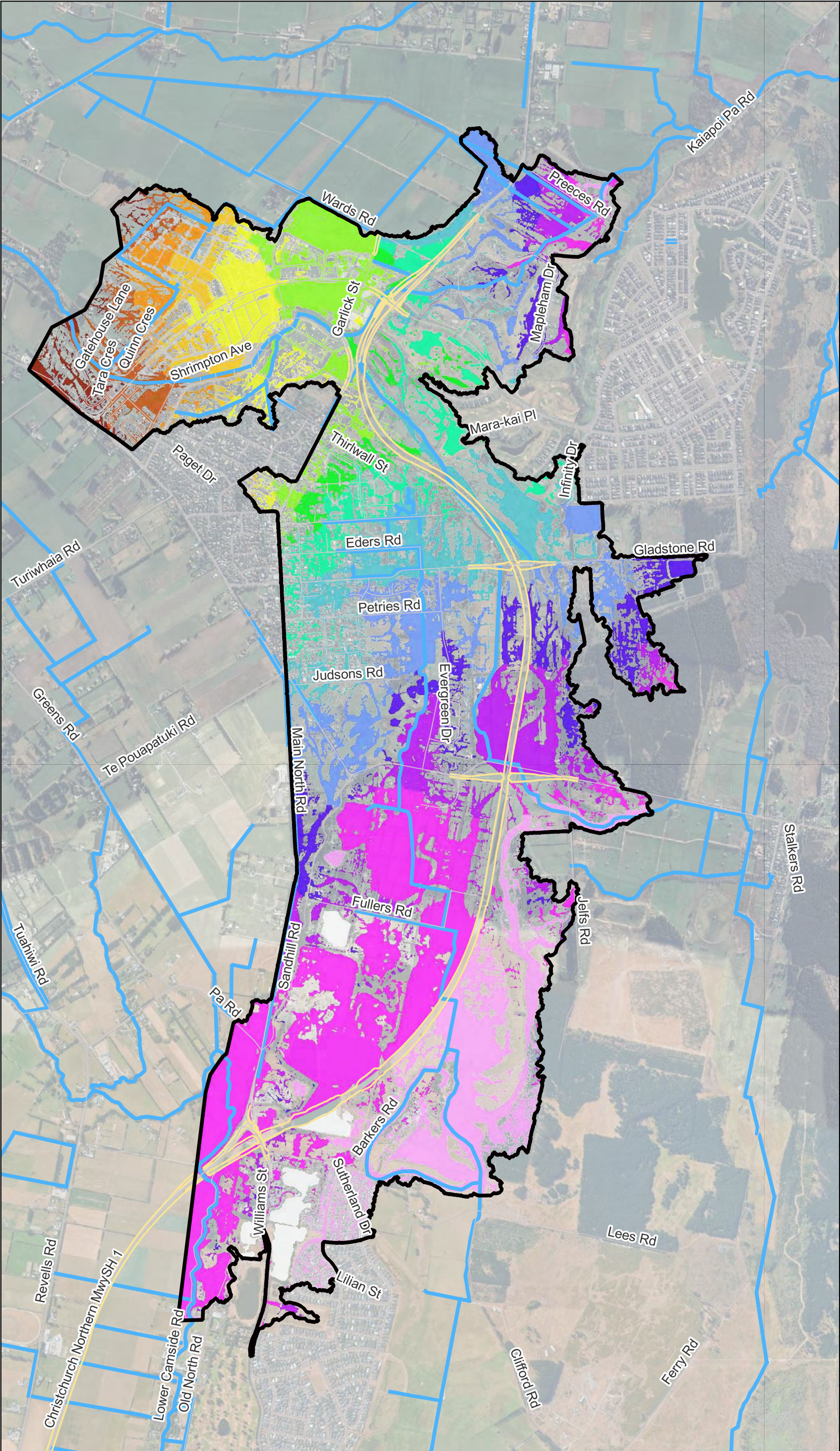
Run Scenarios:

PREDEV - Existing topography

POSTDEV- Existing topography
+ Proposed Road
Embankment

MITIG -Post-development
scenario + mitigating measures
(add'l culverts, channels etc.)

Water Level Differencing Notes:
The water level afflux is not
displayed on the map when
either the pre-development or
mitigated/post-development
scenarios indicate dry
conditions.



LEGEND

Model Extent

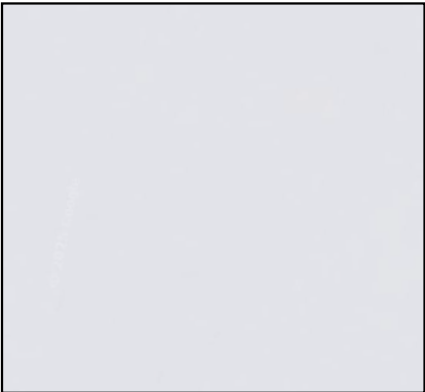
Roads

Proposed Road Footprint

Existing Overland Channels

Flood Level (m)

- <= 2
- 2.001 - 3
- 3.001 - 4
- 4.001 - 5
- 5.001 - 6
- 6.001 - 7
- 7.001 - 8
- 8.001 - 9
- 9.001 - 10
- 10.001 - 11
- 11.001 - 12
- 12.001 - 13
- 13.001 - 14
- 14.001 - 15
- > 15



Notes:

Hydraulic TUFLOW Model

Model Cell Size:
4m x 4m SGS with Quad Tree

LIDAR Cell Size:
1m x 1m

Output Grids Cell Size:
1m x 1m

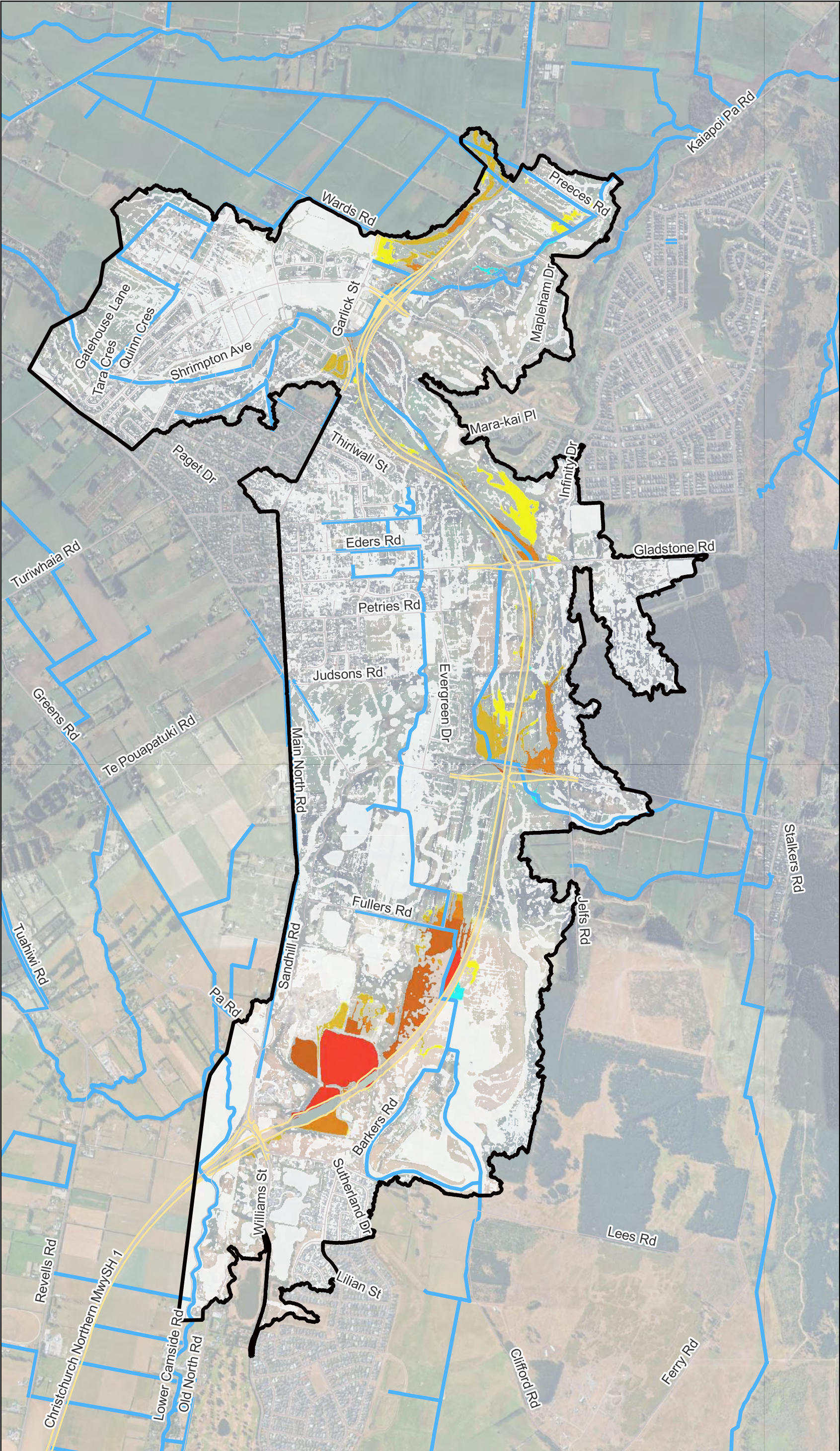
Run Scenarios:

PREDEV - Existing topography

POSTDEV- Existing topography
+ Proposed Road
Embankment

MITIG -Post-development
scenario + mitigating measures
(add'l culverts, channels etc.)

Water Level Differencing Notes:
The water level afflux is not
displayed on the map when
either the pre-development or
mitigated/post-development
scenarios indicate dry
conditions.



LEGEND

Model Extent 

Roads 

Proposed Road Footprint 

Existing Overland Channels 

Afflux (m)

<-1.000 

-0.999 to -0.500 

-0.499 to -0.250 

-0.249 to -0.020 

-0.020 to 0.020 

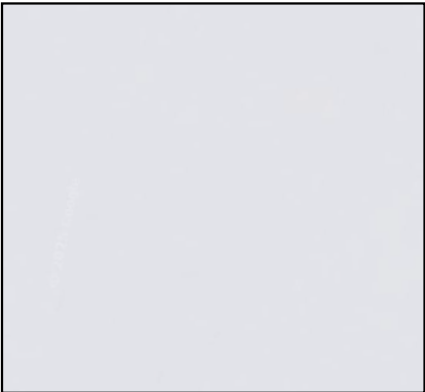
0.021 to 0.100 

0.101 to 0.250 

0.251 to 0.500 

0.501 to 1.000 

> 1.000 



Notes:

Hydraulic TUFLOW Model

Model Cell Size:
4m x 4m SGS with Quad Tree

LIDAR Cell Size:
1m x 1m

Output Grids Cell Size:
1m x 1m

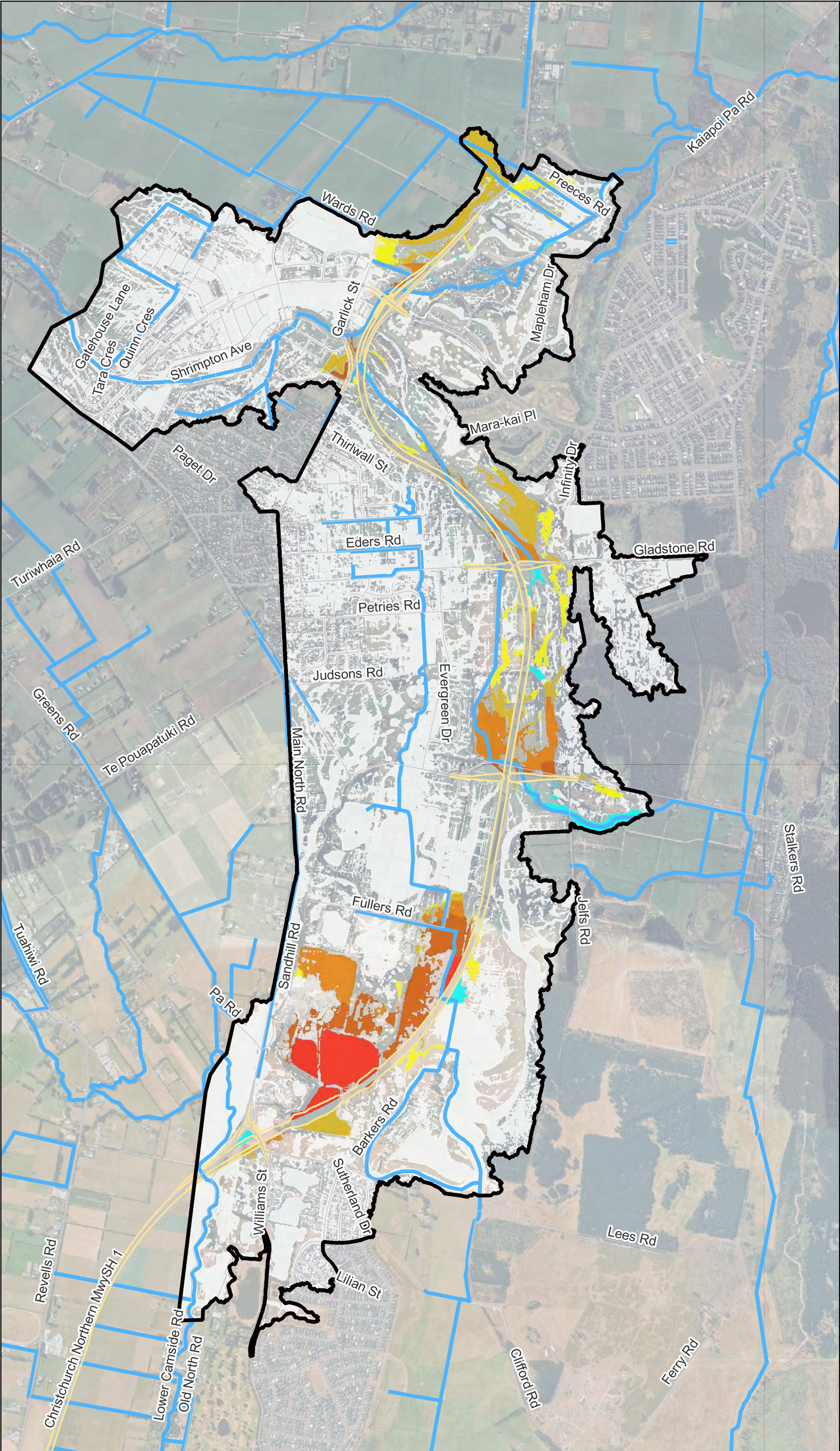
Run Scenarios:

PREDEV - Existing topography

POSTDEV- Existing topography
+ Proposed Road
Embankment

MITIG -Post-development
scenario + mitigating measures
(add'l culverts, channels etc.)

Water Level Differencing Notes:
The water level afflux is not
displayed on the map when
either the pre-development or
mitigated/post-development
scenarios indicate dry
conditions.



LEGEND

Model Extent

Roads

Proposed Road Footprint

Existing Overland Channels

Afflux (m)

<-1.000

-0.999 to -0.500

-0.499 to -0.250

-0.249 to -0.020

-0.020 to 0.020

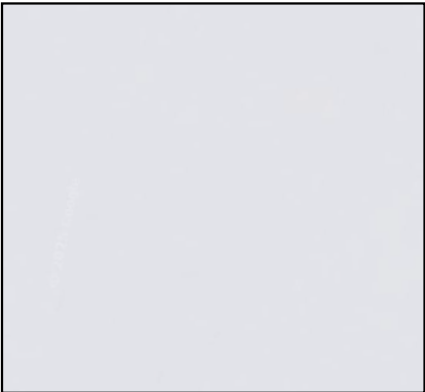
0.021 to 0.100

0.101 to 0.250

0.251 to 0.500

0.501 to 1.000

> 1.000



Notes:

Hydraulic TUFLOW Model

Model Cell Size:
4m x 4m SGS with Quad Tree

LIDAR Cell Size:
1m x 1m

Output Grids Cell Size:
1m x 1m

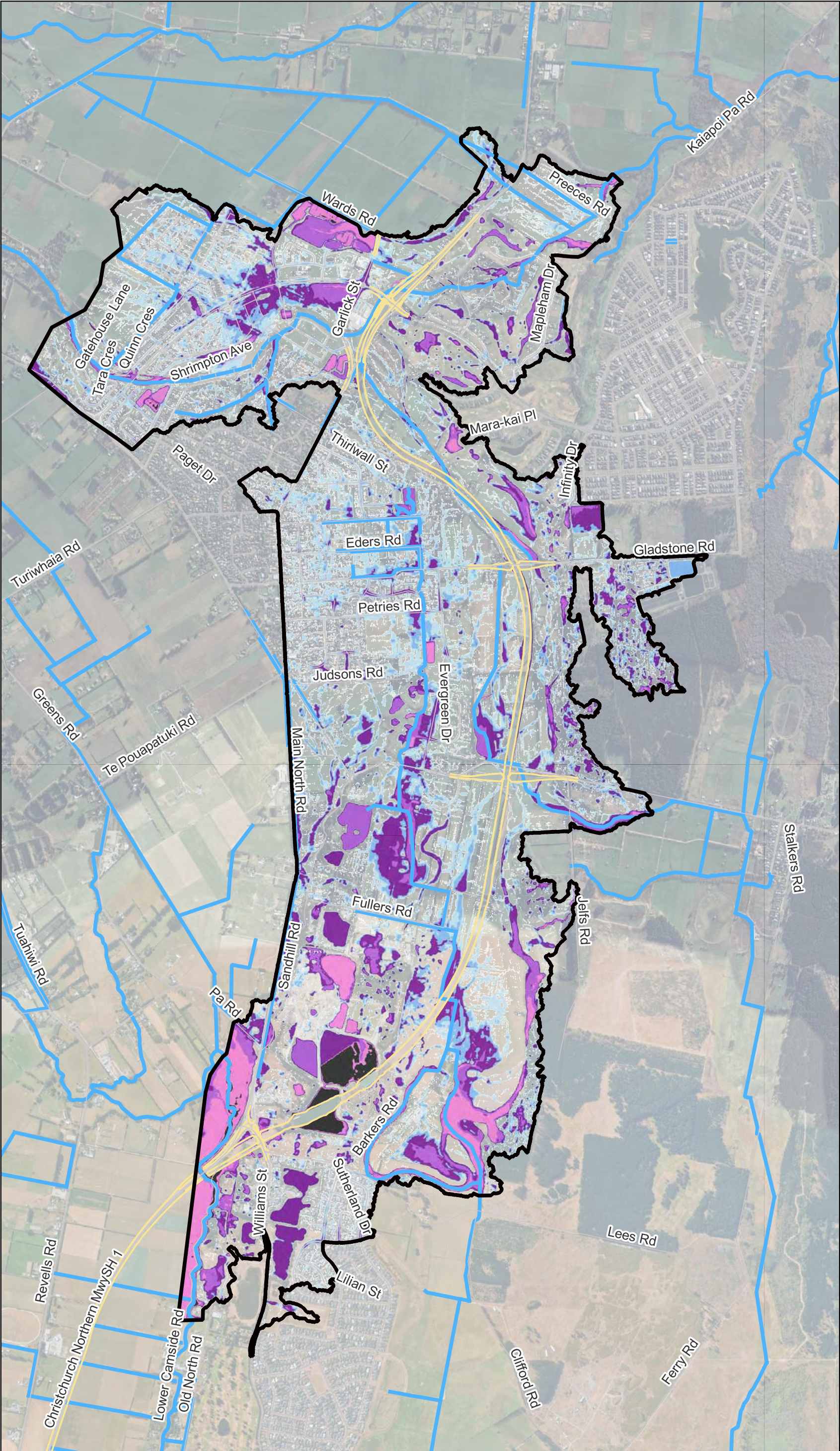
Run Scenarios:

PREDEV - Existing topography

POSTDEV- Existing topography
+ Proposed Road
Embankment

MITIG -Post-development
scenario + mitigating measures
(add'l culverts, channels etc.)

Water Level Differencing Notes:
The water level afflux is not
displayed on the map when
either the pre-development or
mitigated/post-development
scenarios indicate dry
conditions.



LEGEND

Model Extent

Roads

Proposed Road Footprint

Existing Overland Channels

Flood Depth (m)

< 0.02

0.021 - 0.10

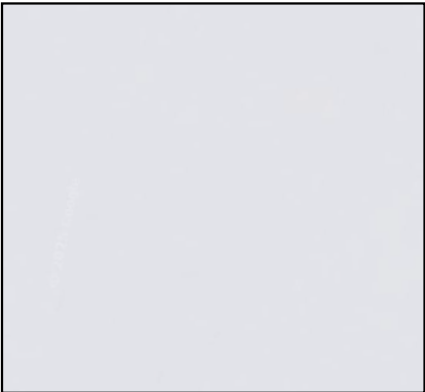
0.11 - 0.20

0.21 - 0.50

0.51 - 1.00

1.01 - 2.00

> 2.00



Notes:

Hydraulic TUFLOW Model

Model Cell Size:
4m x 4m SGS with Quad Tree

LIDAR Cell Size:
1m x 1m

Output Grids Cell Size:
1m x 1m

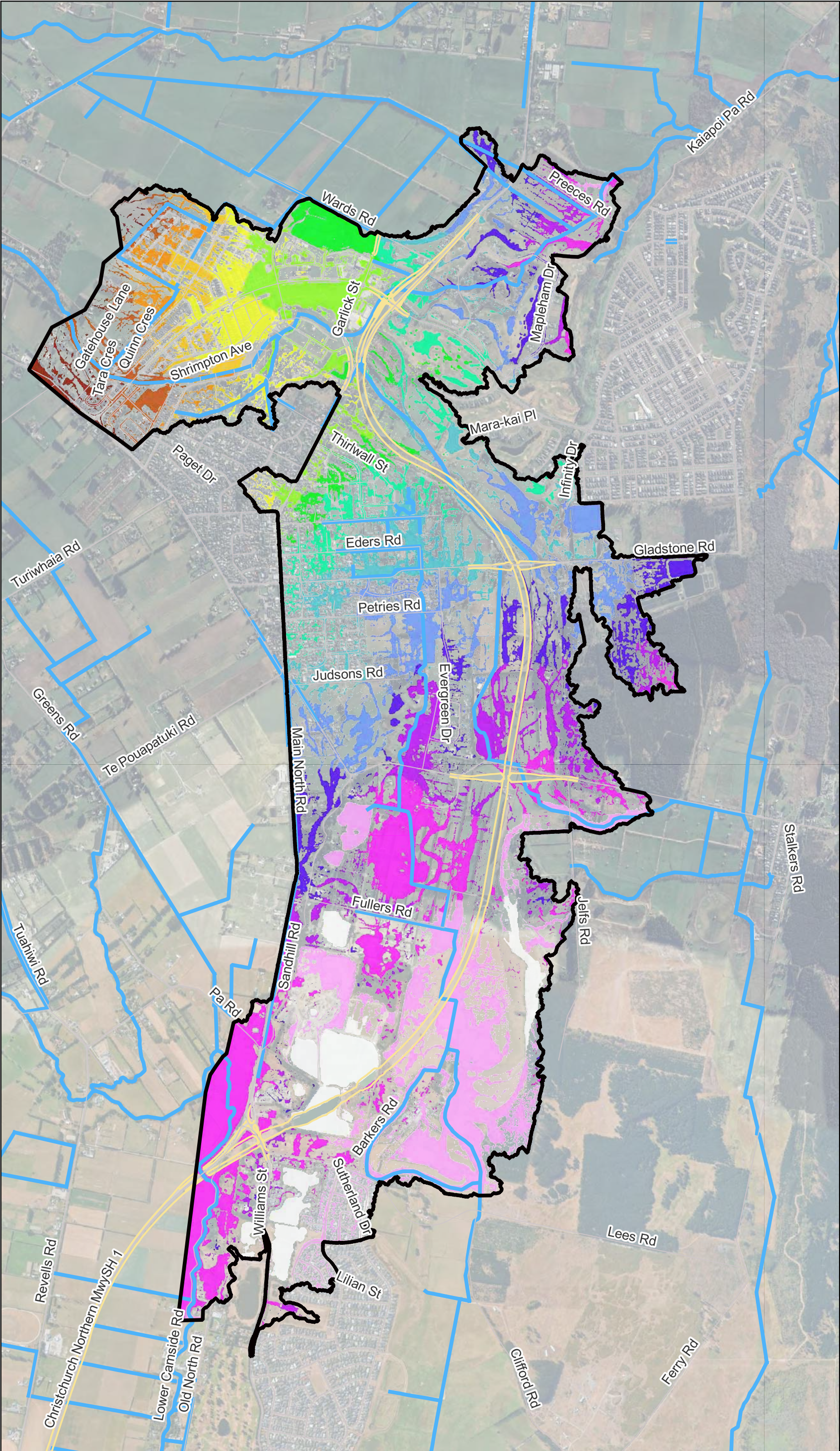
Run Scenarios:

PREDEV - Existing topography

POSTDEV- Existing topography
+ Proposed Road
Embankment

MITIG -Post-development
scenario + mitigating measures
(add'l culverts, channels etc.)

Water Level Differencing Notes:
The water level afflux is not
displayed on the map when
either the pre-development or
mitigated/post-development
scenarios indicate dry
conditions.



LEGEND

Model Extent

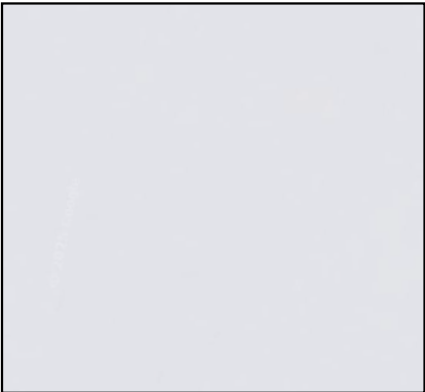
Roads

Proposed Road Footprint

Existing Overland Channels

Flood Level (m)

- <= 2
- 2.001 - 3
- 3.001 - 4
- 4.001 - 5
- 5.001 - 6
- 6.001 - 7
- 7.001 - 8
- 8.001 - 9
- 9.001 - 10
- 10.001 - 11
- 11.001 - 12
- 12.001 - 13
- 13.001 - 14
- 14.001 - 15
- > 15



Notes:

Hydraulic TUFLOW Model

Model Cell Size:
4m x 4m SGS with Quad Tree

LIDAR Cell Size:
1m x 1m

Output Grids Cell Size:
1m x 1m

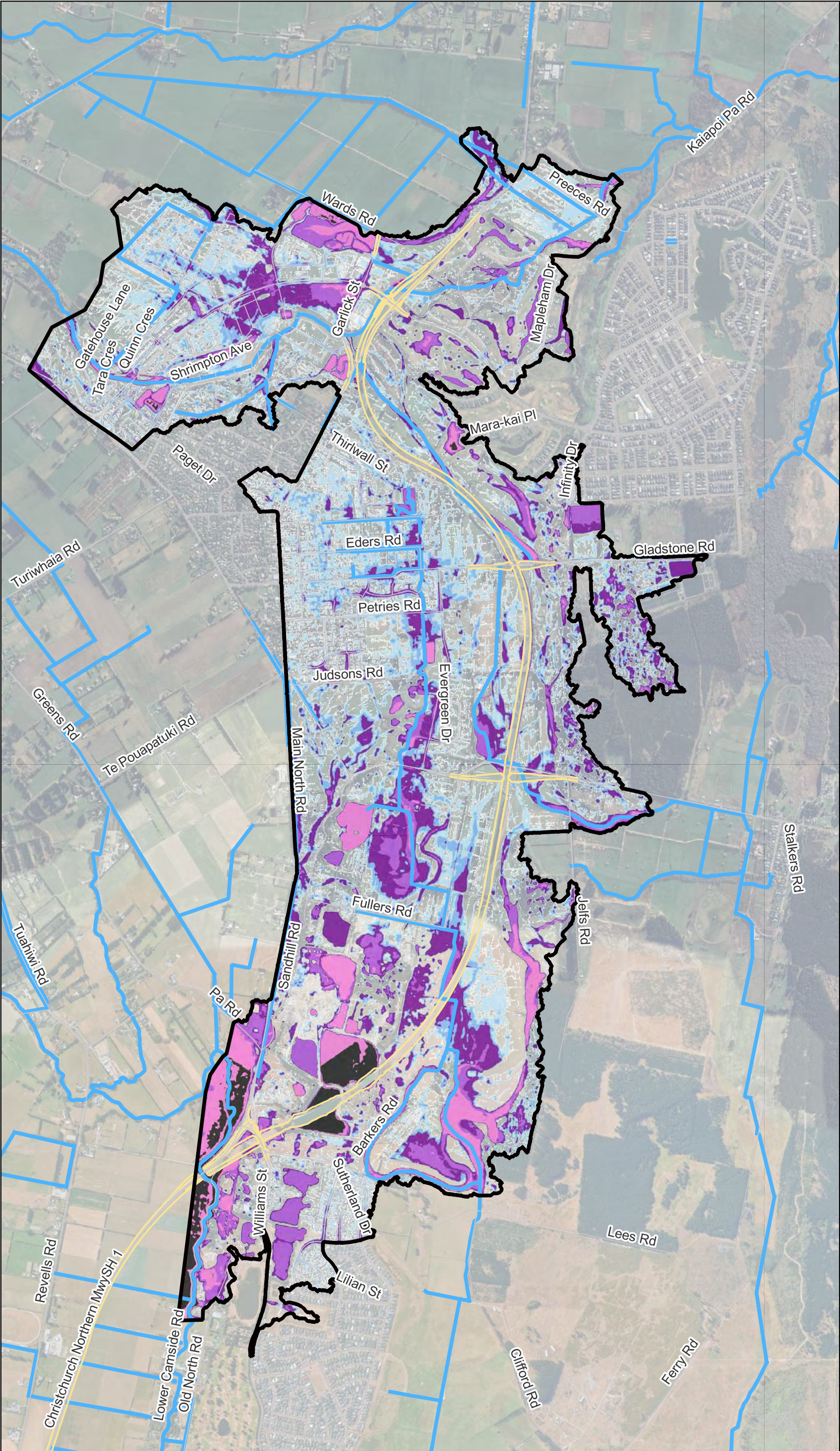
Run Scenarios:

PREDEV - Existing topography

POSTDEV- Existing topography
+ Proposed Road
Embankment

MITIG -Post-development
scenario + mitigating measures
(add'l culverts, channels etc.)

Water Level Differencing Notes:
The water level afflux is not
displayed on the map when
either the pre-development or
mitigated/post-development
scenarios indicate dry
conditions.



LEGEND

Model Extent

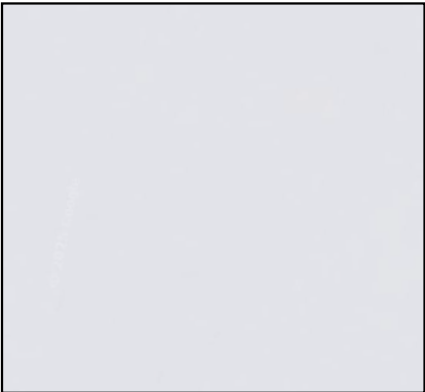
Roads

Proposed Road Footprint

Existing Overland Channels

Flood Depth (m)

- < 0.02
- 0.021 - 0.10
- 0.11 - 0.20
- 0.21 - 0.50
- 0.51 - 1.00
- 1.01 - 2.00
- > 2.00



Notes:

Hydraulic TUFLOW Model

Model Cell Size:
4m x 4m SGS with Quad Tree

LIDAR Cell Size:
1m x 1m

Output Grids Cell Size:
1m x 1m

Run Scenarios:

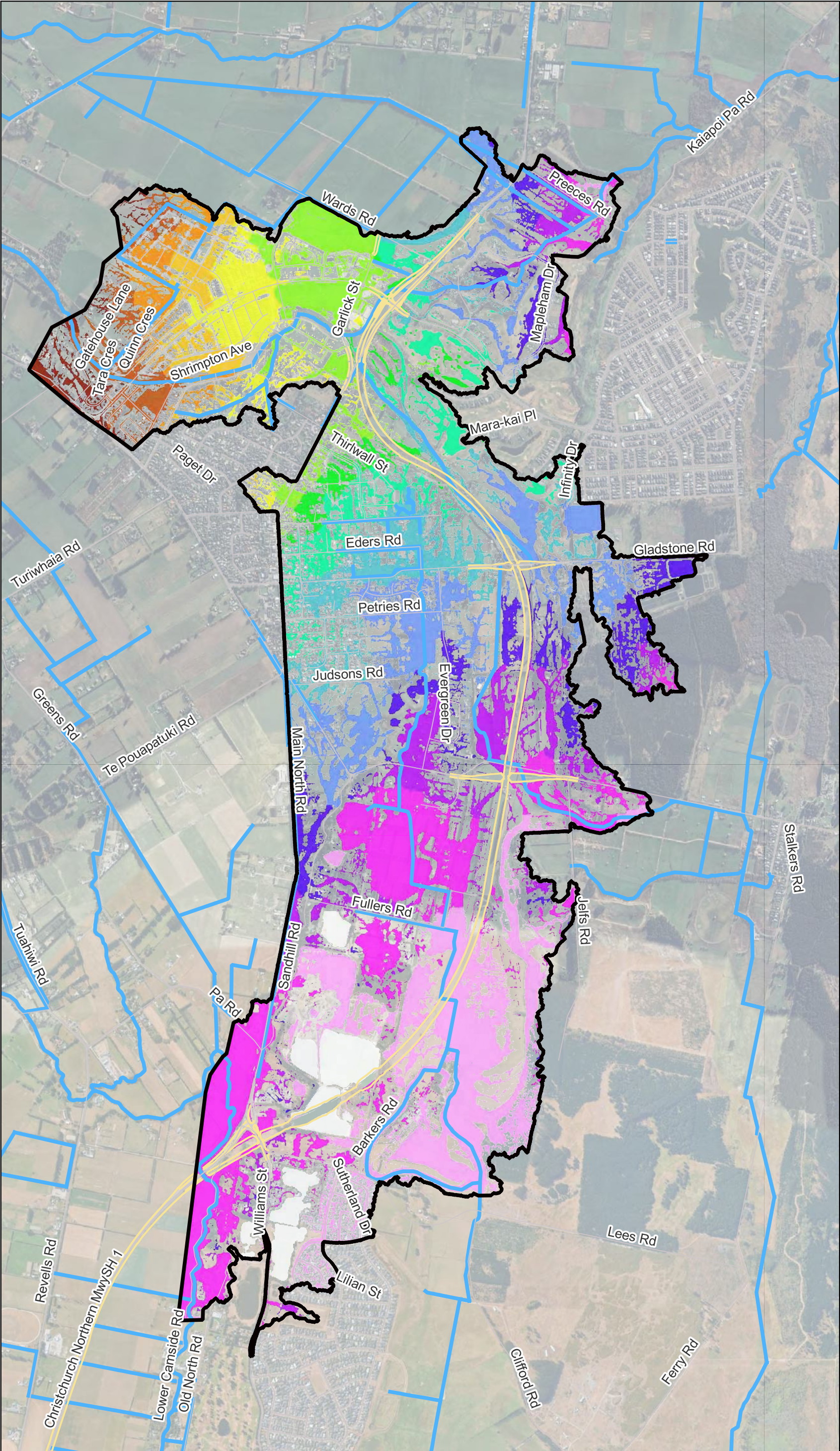
PREDEV - Existing topography

POSTDEV- Existing topography
+ Proposed Road
Embankment

MITIG -Post-development
scenario + mitigating measures
(add'l culverts, channels etc.)

Water Level Differencing Notes:
The water level afflux is not
displayed on the map when
either the pre-development or
mitigated/post-development
scenarios indicate dry
conditions.





LEGEND

Model Extent

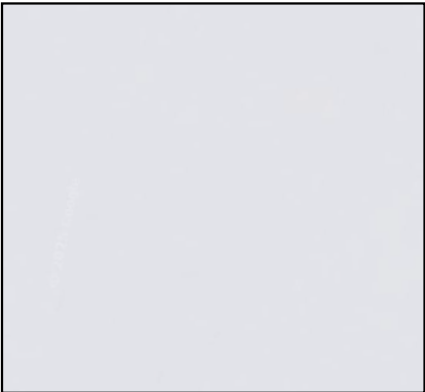
Roads

Proposed Road Footprint

Existing Overland Channels

Flood Level (m)

- <= 2
- 2.001 - 3
- 3.001 - 4
- 4.001 - 5
- 5.001 - 6
- 6.001 - 7
- 7.001 - 8
- 8.001 - 9
- 9.001 - 10
- 10.001 - 11
- 11.001 - 12
- 12.001 - 13
- 13.001 - 14
- 14.001 - 15
- > 15



Notes:

Hydraulic TUFLOW Model

Model Cell Size:
4m x 4m SGS with Quad Tree

LIDAR Cell Size:
1m x 1m

Output Grids Cell Size:
1m x 1m

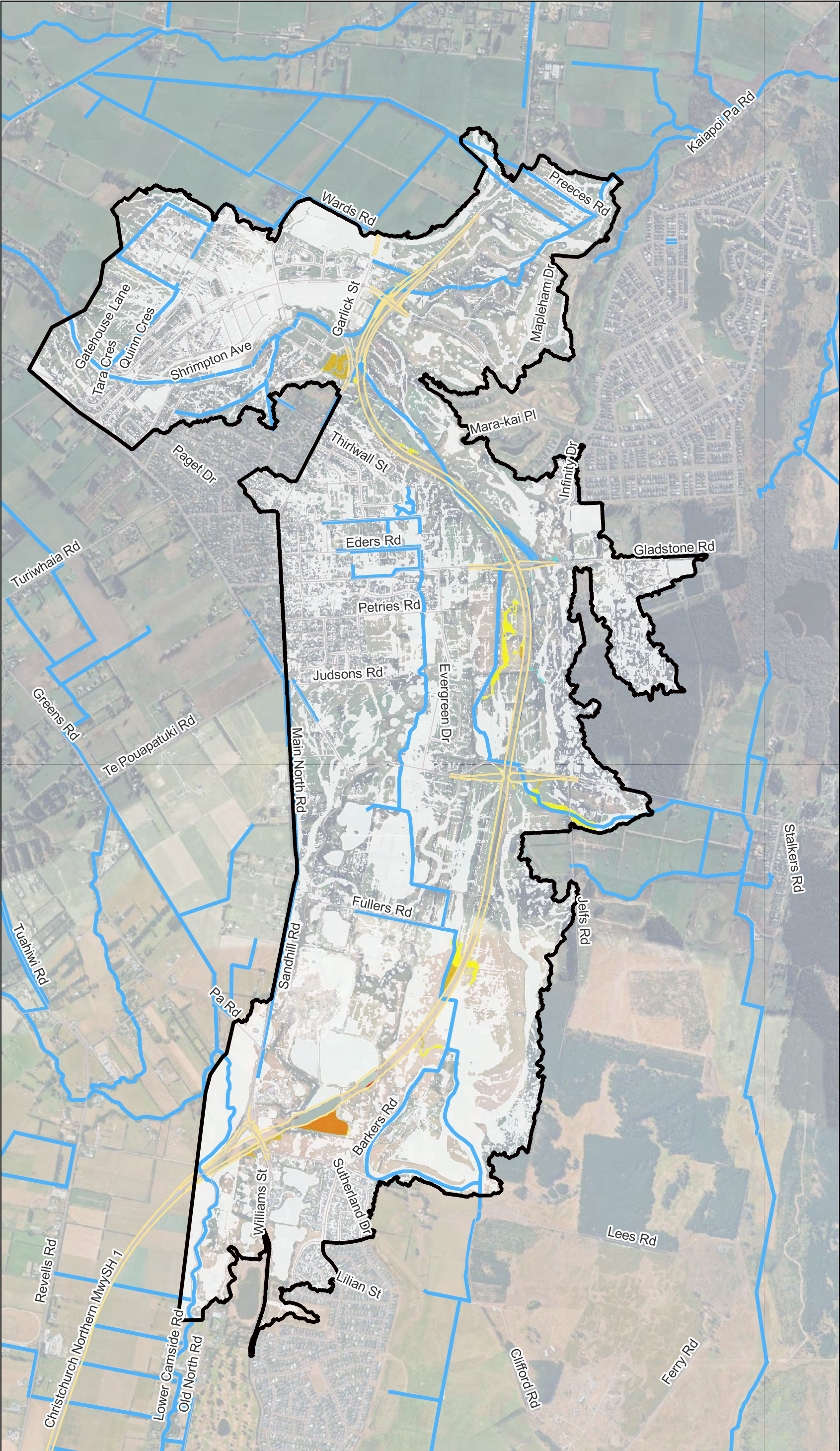
Run Scenarios:

PREDEV - Existing topography

POSTDEV- Existing topography
+ Proposed Road
Embankment

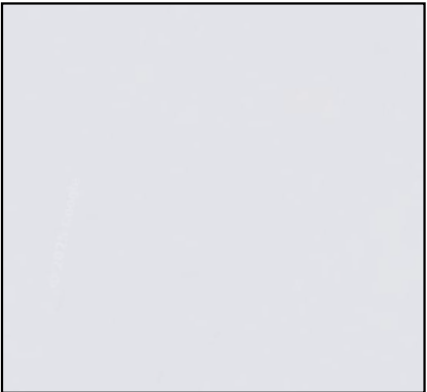
MITIG -Post-development
scenario + mitigating measures
(add'l culverts, channels etc.)

Water Level Differencing Notes:
The water level afflux is not
displayed on the map when
either the pre-development or
mitigated/post-development
scenarios indicate dry
conditions.



LEGEND

- Model Extent
- Roads
- Proposed Road Footprint
- Existing Overland Channels
- Afflux (m)
 - <-1.000
 - 0.999 to -0.500
 - 0.499 to -0.250
 - 0.249 to -0.020
 - 0.020 to 0.020
 - 0.021 to 0.100
 - 0.101 to 0.250
 - 0.251 to 0.500
 - 0.501 to 1.000
 - > 1.000



Notes:
Hydraulic TUFLOW Model

Model Cell Size:
4m x 4m SGS with Quad Tree

LIDAR Cell Size:
1m x 1m

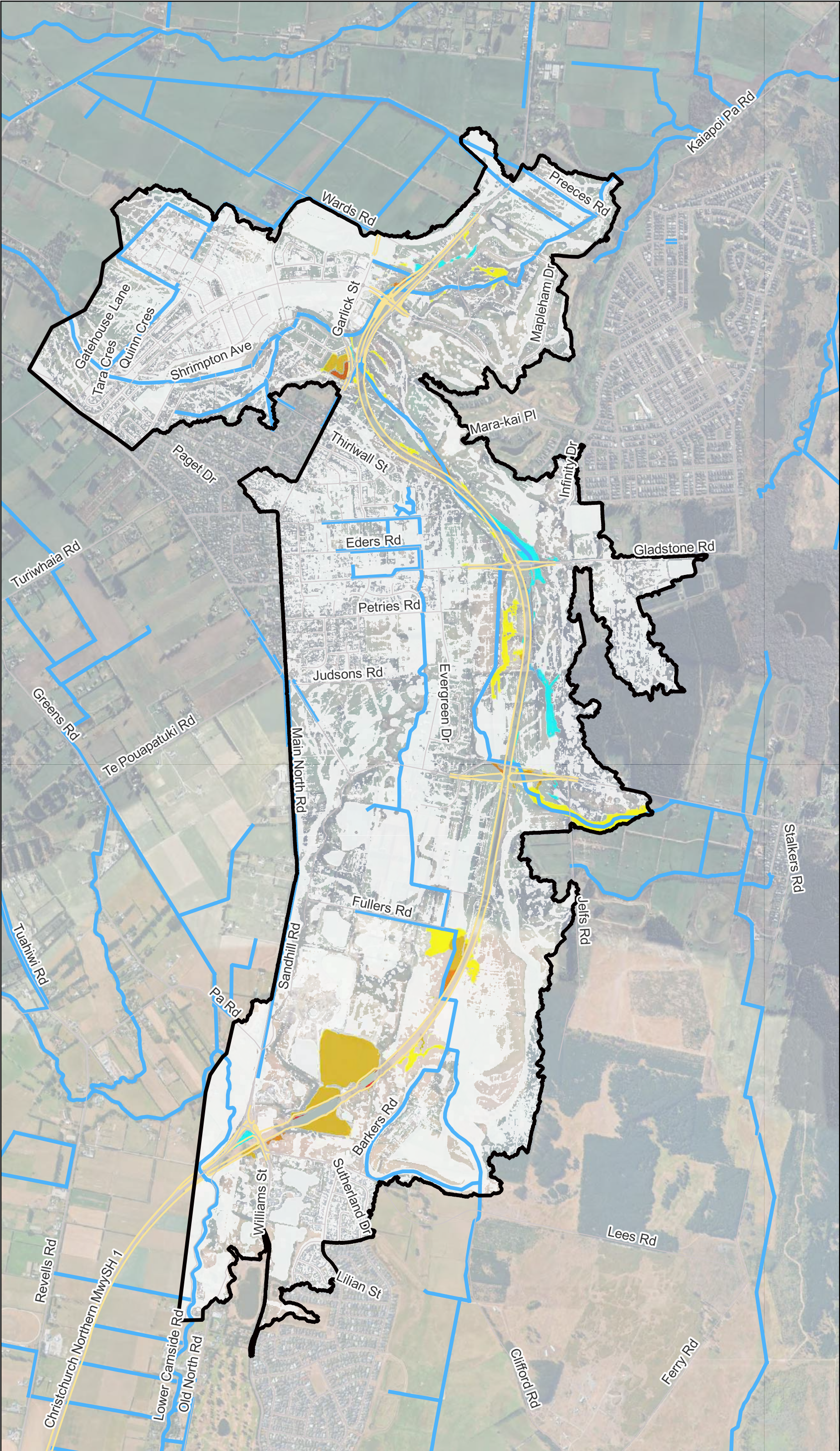
Output Grids Cell Size:
1m x 1m

Run Scenarios:
PREDEV - Existing topography

POSTDEV- Existing topography
+ Proposed Road
Embankment

MITIG -Post-development
scenario + mitigating measures
(add'l culverts, channels etc.)

Water Level Differencing Notes:
The water level afflux is not
displayed on the map when
either the pre-development or
mitigated/post-development
scenarios indicate dry
conditions.



LEGEND

Model Extent

Roads

Proposed Road Footprint

Existing Overland Channels

Afflux (m)

<-1.000

-0.999 to -0.500

-0.499 to -0.250

-0.249 to -0.020

-0.020 to 0.020

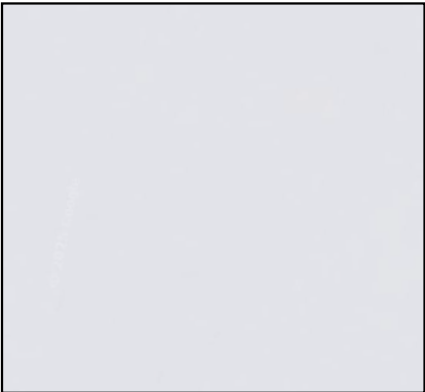
0.021 to 0.100

0.101 to 0.250

0.251 to 0.500

0.501 to 1.000

> 1.000



Notes:

Hydraulic TUFLOW Model

Model Cell Size:
4m x 4m SGS with Quad Tree

LIDAR Cell Size:
1m x 1m

Output Grids Cell Size:
1m x 1m

Run Scenarios:

PREDEV - Existing topography

POSTDEV- Existing topography
+ Proposed Road
Embankment

MITIG -Post-development
scenario + mitigating measures
(add'l culverts, channels etc.)

Water Level Differencing Notes:
The water level afflux is not
displayed on the map when
either the pre-development or
mitigated/post-development
scenarios indicate dry
conditions.

Document prepared by

Aurecon New Zealand Limited

Level 2, Iwikau Building
93 Cambridge Terrace
Christchurch 8013
New Zealand

T +64 3 366 0821

F +64 3 379 6955

E christchurch@aurecongroup.com

W aurecongroup.com

Document prepared by

Aurecon New Zealand Limited

Level 2, Iwikau Building
93 Cambridge Terrace
Christchurch 8013
New Zealand

T +64 3 366 0821

F +64 3 379 6955

E christchurch@aurecongroup.com

W aurecongroup.com

Appendix D

B2P Major and Significant Culverts Memo

Memorandum

To	Brendon French, NZTA	From	Regan Smith
Copy	Bronwyn Leek, NZTA Matt Flannery, Aurecon Katherine Eveleigh, Aurecon Haden Walters, Aurecon	Reference	11320
Date	2025-12-19	Pages (including this page)	13
Subject	B2P Major and Significant Culverts (100% Detailed Design)		

1 Introduction

This memorandum describes the design of the proposed major and significant culverts as part of the SH1 Belfast to Pegasus project. It summarises the existing culvert, stream conditions and sets out the high-level proposed culvert arrangements, including type, size, invert levels, gradients and embedment. Major Culverts are identified by the Highway Structures Design Guide (HSDG) Clause 5.6 as those with a total waterway area greater than 3.4m²; significant culverts identified are those that convey significant water courses, drains or flow paths and those requiring fish passage design.

The designs have been developed to meet hydraulic conveyance requirements and provide effective fish passage in accordance with project requirements and relevant guidelines. For each culvert, this memorandum provides a summary of expected performance, focusing on conveyance capacity under design flows and fish passage suitability. The basis of design is also outlined, drawing on background information from the existing site conditions and informing the proposed sizing and configuration.

The major and significant culvert locations are shown in Figure 1 below and summarised in Table 1:

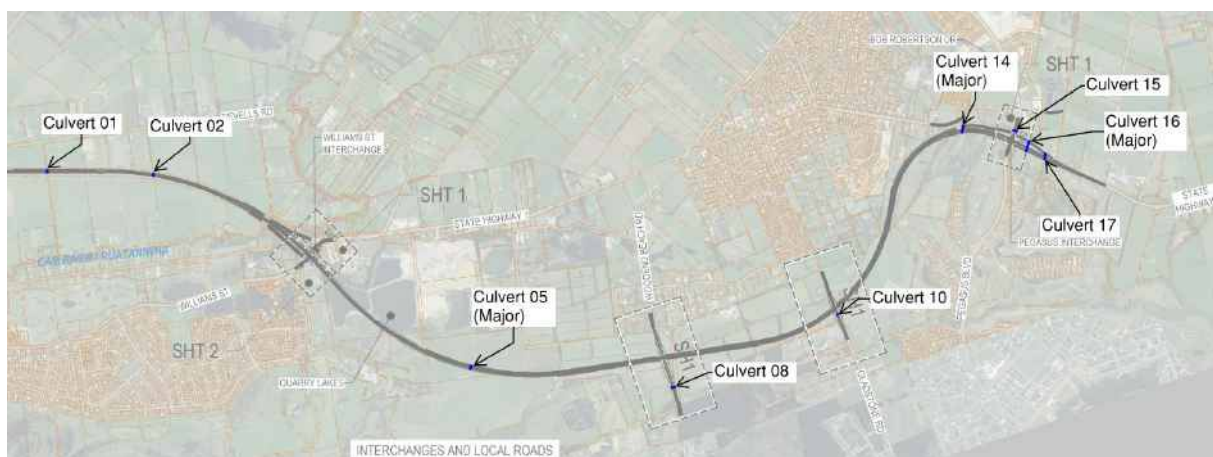


Figure 1 Major and significant culvert locations

Table 1 Proposed major and significant culvert summary

Culvert ID	Project Chainage	Stream name	Proposed Culvert	Approx. total Waterway Area (m ²)
01	Mainline CH1610	Rossiter drain	DN1500 Concrete (embed 0.3m)	1.5
02	Mainline CH2350	Wilsons drain	DN1500 Concrete (embed 0.3m)	1.5
05 (Major)	Mainline CH5000	Mcintosh drain	Box 6m x 2m (WxH) (embed 0.3m, low flow 0.2m)	9.2
08	Woodend Beach Road CH540	Waihora Stream	2x DN1350 Concrete	2.9
10	Gladstone Road CH360	Waihora Stream	2x DN900 Concrete *	1.3
14 (Major)	Mainline CH9210	Waihora Stream	Box 4m X 1.8m (WxH) (embed 0.3m)	6
15 (Major)	Bob Robertson Drive CH100	Taranaki Stream	2x DN1800 Concrete (embed 0.8m)	2.9
16 (Major)	Mainline CH9700	Taranaki Stream	Box 2.7m X 1.5m (WxH) (embed 0.3m)	3.2
17	Mainline CH9830	Taranaki Stream north branch	DN1200 Concrete (0.4m submerged inlet)	1.1

* Exact culvert arrangement may subject to further refinement in IFC design.

Note: This memorandum is in preliminary form, prepared as part of the 50% design stage. The proposed culverts are subject to change based on updated survey data and detailed hydraulic checks.

2 Culvert Design Approach

2.1 Stormwater

Culverts for this project were designed with consideration of existing infrastructure, where present. In most cases, culverts are located near existing culvert crossings where overland flows intersect the proposed works. Culvert works will also include upgrades, extension, and/or replacement of existing culverts. The design aims to facilitate cross-catchment drainage to maintain, as practicable, the natural flow regime.

Flood modelling was undertaken in TUFLOW north of Cam River to assess the potential impact of the proposed motorway on flooding in the project vicinity. This modelling covers the majority of significant and major culverts within the project. The effects on flooding adjacent to the site are documented in the flood modelling report. Culverts within the flood model are sized iteratively based on the flood modelling results, with local hydraulics cross-verified using HY-8 1D analysis.

Two existing cross culverts are located between Lineside Road and Cam River (Culvert 1 and 2). The project proposes widening the existing motorway at these locations, requiring replacement of these culverts to achieve a combined asset lifespan of 100 years, in accordance with NZTA P46. These culverts fall outside the flood modelling extent; therefore, local catchment analysis was conducted using the SCS method. Refer to Section 3.1 and 3.2 for details.

Overall, culvert sizes and levels will be designed based on the following hydraulic criteria and constraints:

- Convey the 10% AEP event without surcharging;
- Convey the 1% AEP event with minimum 500mm freeboard to the road surface (i.e. edge of seal) and without the headwater depth exceeding 2m above the pipe soffit;
- Convey the 1% AEP event without increasing upstream flood levels outside the designation; and

In accordance with NZTA P46, fish passage will also be provided for proposed culverts in accordance with the National Environmental Standards for Freshwater Management (this is aligned with ECan's requirements). For existing culverts requiring extension, an assessment shall be undertaken to determine the benefits and practicalities of enabling / retrofitting for fish passage.

NZTA P46 requires the minimum pipe cover under traffic lanes to be 1.0m to provide sufficient load distribution and structural protection. It is noted that several culverts cannot achieve this minimum pipe cover. See indicative pipe cover provided in Section 7.1. It is anticipated that a departure or other form of approval will be sought during the detailed design process.

A detailed discussion of the hydraulic design approach for individual culverts is provided below in Section 3.

2.2 Structural

The HSDG defines Major Culverts as culverts or multiple culverts whose total waterway area exceeds 3.4m². There are four culverts proposed that meet the definition of major culverts in this project:

- Culvert 5 for McIntosh Drain
- Culvert 14 for Waihora Stream
- Culvert 15 Bob Robertson Drive (Existing)
- Culvert 16 for Taranaki Stream

The structural design of single barrel culverts greater than 3.4m² in cross-sectional area shall be in accordance with the NZTA Bridge Manual.

Structural design approach is provided in 11320-AUR-0350-MJC-ST-RPT-0001.

3 Basis of Design

3.1 Culvert 1 (Rossiter Drain)

3.1.1 Existing crossing

- The existing culvert is a DN1500 concrete pipe with headwalls, approximately 61m long.
- The culvert has a negligible gradient of less than 0.1%.
- Up to 200mm of streambed substrate fills the bottom of both the inlet and outlet.
- The culvert connects the upstream and downstream sections of the WDC-maintained Rossiter Drain. The downstream section is effectively flat, with an average gradient of less than 0.1%.
- Rossiter Drain outfalls to the Cam River through a flap gate, which in turn discharges to the Kaiapoi River. There is a flood gate on the Cam River just upstream of the confluence with the Kaiapoi River.
- The upstream catchment includes a very low lying large “bathtub” storage area. The “bathtub” area includes more than 600,000m³ of volume below 1.2mRL (level approx. 600mm freeboard to SH1). This storage is larger than the full 100-year ARI 24-hour runoff volume expected from the local catchment between SH71 and Cam River. Rossiter Drain is one of several interlinked council-maintained drains that drain this area.

3.1.2 Proposed crossing

- In order to maintain the existing flood condition, is proposed that the culvert be replaced with the same size, like-for-like pipe, consisting of a DN1500 concrete pipe, with headwalls, approximately 22m long.
- The proposed gradient is approx. 0.3% to improve self-cleansing and constructability.
- The culvert connects an artificial drainage channel used to drain the upstream “bathtub” area to the Cam River. There is also a secondary “bathtub” area with significant storage available below the road level on low lying land downstream of the culvert between SH1 and the Cam River.
- The culvert's hydraulic capacity is highly sensitive to its tailwater condition. Based on WDC modelling, the design future tide level downstream of the culvert (accounting for climate change and sea level rise) is 1.4mRL, only 0.4m below the critical road level. Therefore, the required 0.5m of freeboard theoretically cannot be achieved in the event of local catchment flooding coinciding with future high tidal levels. However, the significant storage within the upstream and downstream catchment, along with flap gates outlets to the Cam River and flood gate management of the Cam River outlet mean local catchment flooding alone is unlikely to result in flooding that impacts SH1's freeboard.
- The culvert will be embedded approx. 300mm below the existing stream bed level to match existing.

- The typical low-flow channel width of Rossiter Drain in the vicinity of the culvert is approximately 1.2m. With embedment, the proposed culvert will provide a similar low flow width. Flow conditions will be maintained, and fish passage performance is expected to remain unchanged.
- The 85th percentile groundwater level is approximately RL 1.0m (T&T, June 2025 contours), about 1.2m above the streambed level. High groundwater levels in this area are expected to sustain baseflow through the stream and culvert.

3.2 Culvert 2 (Wilsons Drain)

3.2.1 Existing crossing

- The existing culvert is a DN1500 concrete pipe with headwalls, approximately 30m long.
- The culvert has a gradient of approx. 0.5%.
- Approx. 50-100mm of streambed substrate fills the invert of the culvert at both the inlet and outlet.
- The culvert connects the upstream and downstream sections of the WDC-maintained Wilson Drain. The downstream section is effectively flat, with an average gradient of less than 0.1%.
- The culvert and drain area within the same wider bathtub catchment and downstream Cam River out-fall as the Rossiter drain culvert above.

3.2.2 Proposed crossing

- In order to maintain the existing flood condition, is proposed that the culvert be replaced with the same size, like-for-like pipe, consisting of a DN1500 concrete pipe, with headwalls, approximately 30m long.
- The proposed gradient is approx. 0.3% to improve self-cleansing and constructability.
- The culvert and drain hydraulic performance and tailwater constraints are linked with Rossiter Drain as above, with a major “bathtub” area with significant storage upstream and downstream of the culvert. Local catchment flooding alone is unlikely to generate sufficient volume to have a significant impact on SH1’s freeboard. It is proposed that the culvert be upgraded with the same size, like-for-like pipe.
- The culvert will be embedded approx. 300mm below the existing stream bed level to match existing.
- The typical Wilsons Drain channel width is approximately 1.2m. With embedment, the proposed culvert will provide a similar low flow width. Flow conditions will be maintained, and fish passage performance is expected to remain unchanged.
- The 85th percentile groundwater level is approximately RL 1.0m (T&T, June 2025 contours), about 1.4m above the streambed level. High groundwater levels in this area are expected to sustain baseflow through the stream and culvert.

3.3 Culvert 5 (McIntosh Drain)

3.3.1 Existing crossing

- Culvert 5 is under the new alignment. There is no existing crossing
- The existing drain at the location of the proposed culvert is a narrow and incised excavated manmade drain with a negligible gradient and a broad flood plain.

3.3.2 Proposed crossing

- The proposed culvert is a Single 6m x 2m (WxH, indicative) concrete box culvert, approximately 36.5m long with headwalls. The main flowable area required is 6m x 1.7m (WxH).
- The proposed gradient is 0.3% for self-cleansing and constructability.
- The proposed culvert sizing is based on wider catchment flood modelling using TUFLOW. It is designed to convey overland flows of 4.96m³/s and 9.39m³/s for the 100-year and 200-year ARI events, respectively, across the proposed road. This capacity is required to limit adverse impacts to post-development flood levels on the private land upstream of the culvert and road alignment.
- The proposed culvert will include a formed low-flow channel for fish passage. The formed channel in the culvert will have approx. width 2m, embedded 300mm below stream tie in level and backfilled with void filled material (D₅₀ 150mm) resistant to scour in the design flood event (100-year ARI event). A 300 mm deep low-flow channel will be formed above the substrate infill, below the main flow area.
- Detailed cross-section design is presented on 11320-AUR-0350-MJC-ST-DRG-0102.
- In low flow conditions (water depth ≤ 0.3m), water will flow only in the cutout channel. This maintains depth for fish passage. Above this, water will start to fill the main culvert barrel, which provides capacity for the overland flow path.
- The typical low flow channel width is approximately 2.1 - 3.1m (avg. 2.3m). With embedment, the proposed culvert will provide a similar low flow width within the cutout channel. Flow conditions will be largely maintained, and fish passage performance is expected to largely remain unchanged.
- The 85th percentile groundwater level is approximately RL 1.25m (T&T, June 2025 contours), about 0.15m above the streambed level. High groundwater levels in this area are expected to sustain baseflow through the stream and culvert.

3.4 Culvert 8 (Waihora Stream at Woodend Beach Rd)

3.4.1 Existing crossing

- The existing culvert is a DN300 PVC pipe, approximately 13.5 m long
- The culvert has a negligible gradient.
- Sediment was observed in the pipe.
- The culvert connects the upstream and downstream sections of the Waihora Stream overland flow path. The downstream section has a gradient of about 0.3%.
- Waihora Stream is ephemeral at this point, so this is usually a dry culvert.
- The existing roadway has a low point of approximately 3.8mRL at the culvert crossing location, where the existing culvert has about 0.5 m of cover. Pre-development flood modelling indicates that

the roadway is overtopped during both the 100-year and 200-year ARI events along a significant section, due to the insufficient capacity of the existing DN300 pipe.

3.4.2 Proposed crossing

- The proposal is to replace the existing culvert with twin DN1350 concrete pipes with bespoke headwalls, approximately 33.6m long.
- The proposed gradient is 0.3% for self-cleansing and constructability.
- The proposed culvert sizing is based on wider catchment flood modelling using TUFLOW. It is designed to convey the Waihora Stream overland flows of 2.61m³/s and 5.16m³/s for the 100-year and 200-year ARI events, respectively, across the proposed road. This capacity is required to prevent significant impoundment on private land and to meet the flow conveyance requirement post-development. The new Woodend Beach Road alignment is considerably higher, and the new culvert will prevent any overtopping of the Woodend Beach Road during the 200-year ARI event.
- The proposed culvert is in an ephemeral watercourse that is only expected to convey flow during flood events. The 85th percentile groundwater level is approximately RL 2.75m (T&T, June 2025 contours), about 0.5-1m below the existing ground level at the proposed culvert crossing location. Fish passage is not required for this crossing.

3.5 Culvert 10 (Waihora Stream at Gladstone Rd)

3.5.1 Existing crossing

- The existing culvert is a 1.25 x 0.5 m (WxH) box culvert with headwalls, approximately 7.5m long
- The culvert has a negligible gradient.
- Gravel and cobbles were observed in the pipe.
- The culvert connects the upstream and downstream sections of the Waihora Stream overland flow path. The downstream section has a negligible gradient.
- Waihora Stream is ephemeral at this point, so this is usually a dry culvert.
- The existing roadway is at approx. 6.4mRL at the culvert crossing location, where the existing culvert has about 0.3m of cover. Pre-development flood modelling indicates that the roadway is overtopped during the 200-year ARI events along a significant section, due to the insufficient capacity of the existing box culvert.

3.5.2 Proposed crossing

- The proposal is to replace the existing culvert with twin DN900 concrete pipes with headwalls, approximately 47.5m long.
- The proposed gradient is 0.4% for self-cleansing and constructability.
- The proposed culvert sizing is based on wider catchment flood modelling using TUFLOW. It is designed to convey the Waihora Stream overland flows of 1.35m³/s and 2.47m³/s for the 100-year and 200-year ARI events, respectively, across the proposed road. This capacity is required to prevent significant impoundment on private land and to meet the flow conveyance requirement post-development. The new Gladstone Road alignment is considerably higher, and the new culvert will prevent any overtopping of the Gladstone Road during the 200-year ARI event.

- The proposed culvert is in an ephemeral watercourse that is only expected to convey flow during an flood events. The 85th percentile groundwater level is approximately RL 5.0m (T&T, June 2025 contours), about 0.5-1m below the existing ground level at the proposed culvert crossing location. Fish passage is not required for this crossing.

3.6 Culvert 14 (Waihora Stream at SH1)

3.6.1 Existing crossing

- The existing culvert is a 4m x 1.5m box culvert with headwalls, approximately 14m long.
- The culvert has a negligible gradient
- Approx. 300mm of streambed substrate is present at the inlet and outlet
- The culvert connects the upstream and downstream sections of Waihora Stream. The downstream section has a negligible gradient.
- Waihora Stream flows are supplemented/ augmented by a subsurface pipe intake from Taranaki stream, as part of Ravenswood development. Fish identified in Waihora Stream via eDNA (T&T). It is intended to provide embedment for fish passage as part of the culvert replacement work.

3.6.2 Proposed crossing

- Replacement with a 4m x 1.8m concrete box with headwalls, approximately 50.2m long.
- Proposed gradient is 0.3%, to improve self-cleaning and constructability, while minimising work required around tie-ins.
- The culvert connects the upstream end of Waihora Stream and crosses the proposed SH1 alignment. Waihora Stream currently receives a baseflow from a DN300 bypass pipe from the Taranaki Stream, installed as part of the Ravenswood development. Current flood modelling estimates that the culvert will convey 0.04m³/s and 1.46m³/s during the 100-year and 200-year ARI events, respectively. However, in the extreme event of a blockage or breakout of the Taranaki Stream, or an overflow from Ravenswood stormwater attenuation system, significant overland flow could reach this culvert. Therefore, it is conservatively proposed to retain the culvert at the same hydraulic capacity as the existing structure.
- The culvert will be embedded 300 mm (or 2x D₅₀) and backfilled with material resistant to scour in the design flood event (100-year ARI).
- Detailed cross-section design is presented on 11320-AUR-0350-MJC-ST-DRG-0132.
- The typical low-flow channel width is approximately 4m. With embedment, the proposed culvert will provide a similar low flow width. Flow conditions will largely be maintained, and fish passage performance is expected to largely remain unchanged.
- The 85th percentile groundwater level is approximately RL 8.75m (T&T, June 2025 contours), about 0.6 m above the streambed level. High groundwater levels in this area are expected to sustain baseflow through the stream and culvert.

3.7 Culvert 15 (Taranaki Stream US – Bob Robertson Drive)

3.7.1 Existing crossing

- The existing culvert is a twin 1800mm diameter concrete pipe with headwalls, approximately 25m long.
- The culvert has a negligible gradient.
- Approx. 800mm embedment depth with hard substrate.
- The culvert connects two sections of the Taranaki Stream diversion, downstream of Ravenswood residential and commercial development.
- The downstream stream diversion section has a shallow gradient of about 0.2%

3.7.2 Proposed crossing

- Extension of the twin 1800mm diameter concrete pipe further upstream for road widening, proposed extension section is approximately 10.6m long.
- It is intended that the extension section uses the same pipe (size, class and joint detail) as per the existing culvert, the joint between the existing pipe and the proposed pipe will be confirmed on site.
- The new section of pipe culvert will be embedded approx. 0.5m to 0.8m to match the existing, and backfilled similar material as the existing section.
- Overall as part of the culvert extension, the flow conditions are expected to be largely maintained, and fish passage performance is expected to remain largely unchanged.
- The 85th percentile groundwater level is approximately RL 8.75 m (T&T, June 2025 contours), about 1 m above the streambed level. High groundwater levels in this area are expected to sustain baseflow through the stream and culvert.

3.8 Culvert 16 (Taranaki Stream DS – SH1)

3.8.1 Existing crossing

- The existing culvert is a 2.2 x 1.2 m (WxH) concrete box with headwalls, approximately 22m long.
- The culvert has a negligible gradient.
- Approx. 350 mm of streambed substrate is present at both the inlet and outlet.
- The culvert connects two sections of the Taranaki Stream diversion, from the outlet of culvert 15 to the downstream section, which ties back into the original stream alignment further downstream.
- The downstream section has a gradient of approximately 0.3%

3.8.2 Proposed crossing

- Replacement with a 2.7 x 1.5m (WxH) box culvert with headwalls, approximately 66.8 m long
- This culvert will be considered as a major culvert regardless of whether the waterway area is larger than the NZTA Bridge Manual dictates, due to the surrounding retaining wall structure.
- Proposed gradient is 0.58% to improve self-cleaning and constructability, while minimising work required around tie-ins.

- The culvert will be embedded 300mm (or 2x D₅₀) and backfilled with material resistant to scour in the design flood event (100-year ARI).
- Detailed cross-section design is presented on 11320-AUR-0350-MJC-ST-DRG-0152.
- The typical low-flow stream channel width is approximately 3m. With embedment, the proposed culvert will provide a similar low flow width. Flow conditions will be largely maintained, and fish passage performance is expected to remain largely unchanged.
- The 85th percentile groundwater level is approximately RL 8.0m (T&T, June 2025 contours), about 0.7 m above the streambed level. High groundwater levels in this area are expected to sustain baseflow through the stream and culvert.

3.9 Culvert 17 (Wards Rd North)

3.9.1 Existing crossing

- The existing culvert is a DN650-850 oval (squashed) Brick Barrel pipe with headwalls, approximately 16m long
- The culvert has a gradient of 1.2%.
- Approx. 150mm of streambed substrate is present at both the inlet and outlet.
- The culvert connects swales on either side of SH1 at the northern extent of the project.

3.9.2 Proposed crossing

- Replacement with a DN1200 concrete pipe culvert with headwalls, approximately 46.4m long.
- Proposed gradient is 0.6%.
- The culvert will be submerged by approx. 400mm to achieve a minimum cover below the carriageway.

4 Major Box Culvert Structural Design

The four culverts that classify as major culverts, in the HSDG, are the McIntosh Drain, Waihora Stream, Taranaki Stream (Downstream), and Taranaki Stream (Upstream) culverts. These major culverts are designed to convey water below the motorway, supporting the fill, road pavement, and state highway traffic above.

Structural design detail is provided in 11320-AUR-0350-MJC-ST-RPT-0001.

5 Other Major Culverts

Culvert 15, Taranaki Stream (Upstream) that goes under Bob Robertson Drive is also classified as a major culvert as per the HSDG. This is an existing twin 1800mm diameter concrete pipe culvert that will be extended further upstream as part of this project. The headwall for this structure will be similar to that of the existing structure's. Waimakariri District Council will be consulted to ensure visual similarity with the intention of the original design.

Within the scheme design, a timber footbridge was to run parallel to the culvert. This bridge is proposed to be removed from the scope. Additional width allowance will be made on Bob Robertson Drive, within the scope of the culvert extension, to allow foot traffic to the south side of the road.

6 References

6.1 Related reports

- Aurecon (2025 July). Belfast to Pegasus Motorway & Woodend Bypass Pre-implementation and MSQA Professional Services - Design Philosophy Statement (Rev B)
[\Aurecon Group\11320 - Belfast to Pegasus Motorway & Woodend Bypass Pre-implementation and MSQA Professional Services - 3 Develop\For Delivery\ 11320-AUR-0350-PWI-DM-RPT-0001.docx](#)
- Aurecon (2025 June). Belfast to Pegasus Motorway & Woodend Bypass Pre-implementation and MSQA Professional Services - Stormwater and Flooding Assessment A01.01
[\Aurecon Group\11320 - Belfast to Pegasus Motorway & Woodend Bypass Pre-implementation and MSQA Professional Services - 3 Develop\For Delivery\ 11320-AUR-0350-PWI-SW-RPT-0003.docx](#)

6.2 Referenced surveys and models

- Carriageway road design model, dated 22 July 2025
[\Users\Public\A\ACCDocs\Aurecon AU NZTA 01\528638 - B2P and Woodend Bypass\Project Files\5 Working Files\Stormwater\Civil3D\Output\2025.07.22\D-RR-COMB.tif](#)
- Existing channels and culverts survey from July 2025
[\Aurecon Group\11320 - Belfast to Pegasus Motorway & Woodend Bypass Pre-implementation and MSQA Professional Services - 4 Exchange\Shared\11320-AUR-0350-PWI-SU-M3M-0050\[A07\].dwg](#)
- Digital Elevation Model obtained from LINZ Data Service in March 2025
[\Users\Public\A\ACCDocs\Aurecon AU NZTA 01\528638 - B2P and Woodend Bypass\Project Files\5 Working Files\Stormwater\ARCGIS_01_ProjectInformation\2DSurfaceLINZ_DEM_NZVD_NZTM.tif](#)
- TUFLOW flood model - Case 019 dated 05 Aug 2025
[\Aurecon Group\11320 - Belfast to Pegasus Motorway & Woodend Bypass Pre-implementation and MSQA Professional Services - Stormwater\01 Modelling\02 TUFLOW\TUFLOW\results\019\MITIG](#)

7 Appendices

7.1 Proposed culvert summary

A summary of the proposed significant and major culverts is provided in Table 2 below.

Table 2 Proposed significant culvert summary

Culvert ID	Project Chainage	Stream name	Total catchment approximate (ha)	Existing Culvert	Proposed Culvert	Culvert invert US (mRL)	Culvert invert DS (mRL)	Length (m)	Grade	Propose embedment (mm)	Road level - critical edge of seal approx. (mRL)	Cover over the pipe approx. (m)
01	Mainline CH1610	Rossiter drain	53.5	DN1500 Concrete	DN1500 Concrete	-0.58	-0.65	22	0.33%	300	1.77	0.75
02	Mainline CH2350	Wilson drain	63.1	DN1500 Concrete	DN1500 Concrete	-0.50	-0.60	30	0.33%	300	1.88	0.88
05 (Major)	Mainline CH5000	Mcintosh drain	207.4	N/A	Box 6m x 2.2m (WxH) (see sketch in Section 3.3)	0.75	0.62	25	0.50%	300	4.50	1.26
08	Woodend Beach Road CH540	Waihora Stream	-	DN300 PVC	Twin Box 2m x 1m (WxH)	2.70 (Exact TBC)	2.60 (Exact TBC)	35	0.30%	0	6.90	3.00
10	Gladstone Road CH360	Waihora Stream	-	Box 1.25m X 0.5m (WxH)	Twin DN1050 Concrete	5.30 (Exact TBC)	5.12 (Exact TBC)	60	0.30%	0	13.9	7.45
14 (Major)	Mainline CH9210	Waihora Stream	15.4	Box 4m X 1.5m (WxH)	Box 4m X 1.8m (WxH) (see sketch in Section 3.6)	8.00	7.86	47	0.30%	300	11.60	1.60
15 (Major)	Bob Robertson Drive CH100	Taranaki Stream	448.9	2x DN1800 Concrete	2x DN1800 Concrete	7.02	6.90	30	0.30%	800	9.74	0.82
16 (Major)	Mainline CH9700	Taranaki Stream	452.3	Box 2.2m X 1.2m (WxH)	Box 2.7m X 1.5m (WxH) (see sketch in Section 3.8)	6.95	6.76	65	0.30%	300	9.27	0.82
17	Mainline CH9830	Taranaki Stream north branch	-	DN650-850 Oval Pipe	Box 2m x 1m (WxH)	6.47	6.28	55	0.35%	400	8.50	0.73

7.2 Conveyance performance

Conveyance performance for 100-year ARI with climate change RCP 6.0 up to 2120 is summarised in Table 3 below:

Table 3 Conveyance performance summary

Culvert ID	Stream name	Design flow ^a (m3/s)	HY-8 (1D) - headwater level ^b (mRL)	Road level – u/s edge of seal (mRL)	Freeboard to edge of seal (mRL)	Soffit Level – u/s (mRL)	Comment
01	Rossiter stream	8.16 (SCS method)	N/A	1.77	N/A	0.88	<u>Local catchment rainfall and ponding:</u> A full 100-year 24-hour runoff volume can be stored in the 'bathtub' 1km west of SH1 and along the western side of SH1. Assuming no water discharges from the 'bathtub', the water ponding level is up to approx. 1.2m, this is 0.6m below the critical SH1 edge of road level of 1.8m.
02	Wilson stream	11.61 (SCS method)	N/A	2.17	N/A	1.0	<u>Culvert conveyance capacity during a high tidal event:</u> Upstream flooding is affected by tidal conditions. Design future tide level (accounting for climate change and sea level rise is 1.4mRL, 0.4m below critical road level. 1D hydraulic assessment is undertaken assuming the maximum tidal level of 1.4m. In this tailwater condition, Culvert 1 and Culvert 2 can convey a combined flow of approx. 5 cms, before the road overtops at its low point (1.8m).
05 (Major)	Mcintosh drain	5.06	2.25	4.49	2.31	3.05	NZTA BM Table 2.5, 0.3m freeboard to soffit for Culverts > 6m2, is satisfied.
08	Woodend Beach Rd east crossing	2.53	3.56	4.50 (Motorway) 6.90 (Woodend Beach Rd)	0.94 (Motorway) 3.34 (Woodend Beach Rd)	3.70	Compliant with NZTA P46 requirement of >500mm freeboard to edge of lane
10	Gladstone Rd east crossing	1.09	5.89	7.90 (Motorway) 13.90 (Gladstone Rd)	1.01 (Motorway) 8.01 (Gladstone Rd)	6.35	Compliant with NZTA P46 requirement of >500mm freeboard to edge of lane
14 (Major)	Waihora stream	0.03	8.53	11.58	3.05	9.8	NZTA BM Table 2.5, 0.3m freeboard to soffit for Culverts > 6m2, is satisfied.
15 (Major)	Taranaki stream	2.09	8.74	9.38 (Offramp) 9.95 (Bob Robertson Rd)	0.64 (Offramp) 1.21 (Bob Robertson Rd)	8.82	Compliant with NZTA P46 requirement of >500mm freeboard to edge of lane
16 (Major)	Taranaki stream	4.35	8.55	15.19 (Motorway) 9.40 (Onramp)	6.64 (Motorway) 0.85 (Onramp)	8.45	Compliant with NZTA P46 requirement of >500mm freeboard to edge of lane
17	Wards Rd north	0.45	7.56	9.99 (Motorway) 8.95 (Onramp)	2.43 (Motorway) 1.39 (Onramp)	7.67	Compliant with NZTA P46 requirement of >500mm freeboard to edge of lane

- Design flows are taken from TUFLOW flood modelling results where available. For Culverts 1 and 2, the design flows are calculated from the upstream catchment using the SCS method.
- Headwater levels are calculated in HY-8, using flow and tailwater elevation data from TUFLOW. For Culverts 1 and 2, upstream catchment storage characteristics and tidal influences are discussed; detailed hydraulic behaviour can only be assessed through 2D flood modelling.

Appendix E

Kaiapoi River Scour Assessment Memo

Memorandum

To	Brendon French, NZTA	From	Jamie Fleming, Aurecon Yanni Hooi, Aurecon
Copy	Bronwyn Leek, NZTA Matt Flannery, Aurecon Katherine Eveleigh, Aurecon Haden Walters, Aurecon	Reference	11320
Date	2025-09-26	Pages (including this page)	11
Subject	Kaiapoi Bridge Scour Assessment Memo		

1 Introduction

This memorandum summarises the HEC-RAS model schematisation, results, and scour assessment findings for the Kaiapoi Bridge.

The following scenarios have been assessed to identify the worst-case conditions for water level and potential scouring:

- Scenario 1: High Flow + Open Boundary
- Scenario 2: High Flow + High Tide (Combination AEP as specified by WDC at a 1:10 ratio)
- Scenario 3: High Tide + Low Flow (Combination AEP as specified by WDC at a 1:10 ratio)

Each scenario was modelled using the 100-year ARI and 1500-year ARI events to assess both the Serviceability Limit State (SLS) and Ultimate Limit State (ULS).

The HEC-RAS model was developed to determine the critical water levels and velocities under each scenario that could contribute to scouring. Model schematisation and results are presented in Section 2 of this memorandum. The scour assessment was conducted in accordance with Bridge Scour (Melville and Coleman, 2000) and HEC-18, the US Federal Highway Administrations bridge scour guidelines and findings are summarised in Section 3.

1.1 Related Reports

The HEC-RAS model input data was extracted from the existing Waimakariri District and Urban Mike Flood Model and Jacob Phase 2 Coastal Inundation Model.

The related reports are listed below:

DHI. (2020, May). Flood Hazard Models Update District and Urban and Mike Flood Models.
https://www.waimakariri.govt.nz/__data/assets/pdf_file/0008/140111/district-flooding-map-2019-update-DHI-report.pdf

Jacobs. (2020, March). Phase 2 Coastal Inundation Modelling Final Study Report.
https://www.waimakariri.govt.nz/__data/assets/pdf_file/0009/140112/Phase-2-Coastal-Inundation-Modelling-report-including-all-Appendices-Jacobs-Final-peer-reviewed.pdf

2 HECRAS Model Build

The hydraulic model used for the hydraulic assessment of the Kaiapoi River Bridge can be summarised as a one-dimensional flow model, using a steady state flow analysis.

2.1 Plans



Figure 1. 1-D model geometry plan

2.2 Topography

The digital terrain was taken from the Canterbury 2020/2021 1m LiDAR DEM, obtained from LINZ Data Service in March 2025, supplemented by bathymetry with extents 100m either side of the bridge, scanned by Elliot Sinclair on 03/04/2025.

2.3 1-D Model Geometry

The cross sections were drawn into HEC-RAS at approximately 30-40m intervals for the main channel reach and either side of the bridge. Bank lines were traced on the existing flood banks adjacent to the river, with ineffective flow areas outside the bank lines and below the level of the flood banks (Figure 2).

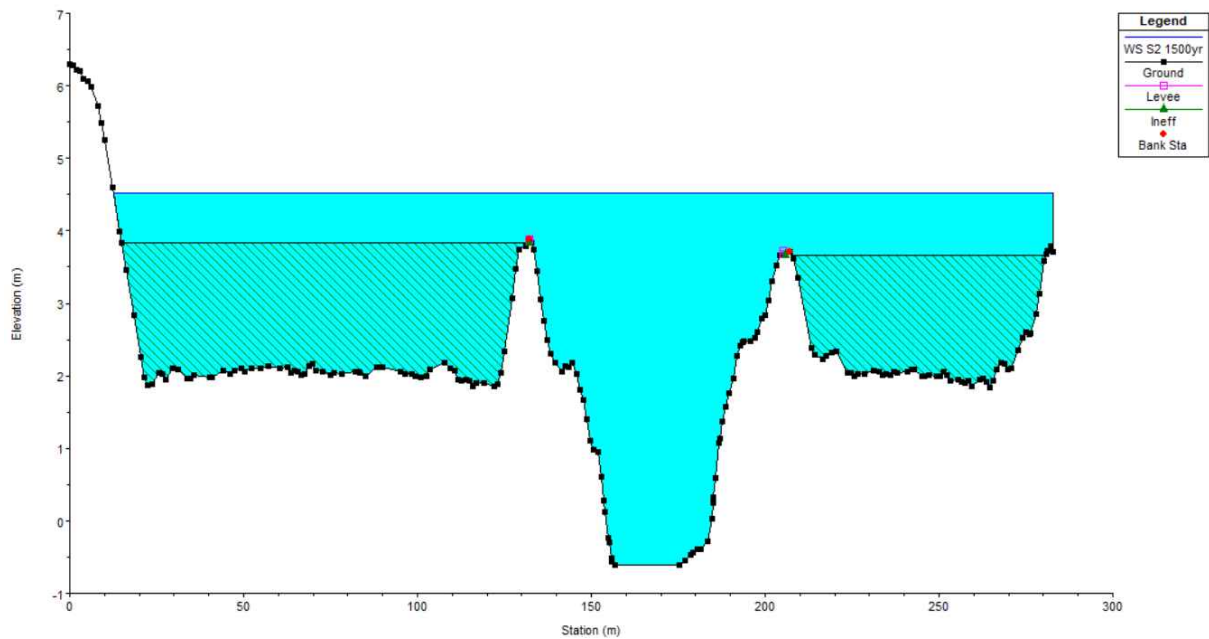


Figure 2. Example cross-section showing ineffective flow areas behind stopbanks. This cross-section is upstream of Bathymetry extents

The bridge data was manually input to the bridge geometry editor based on the original design drawings (Christchurch Northern Motor Kaiapoi River Bridge General Elevation and Plan of Bridge, 7/11/1968)

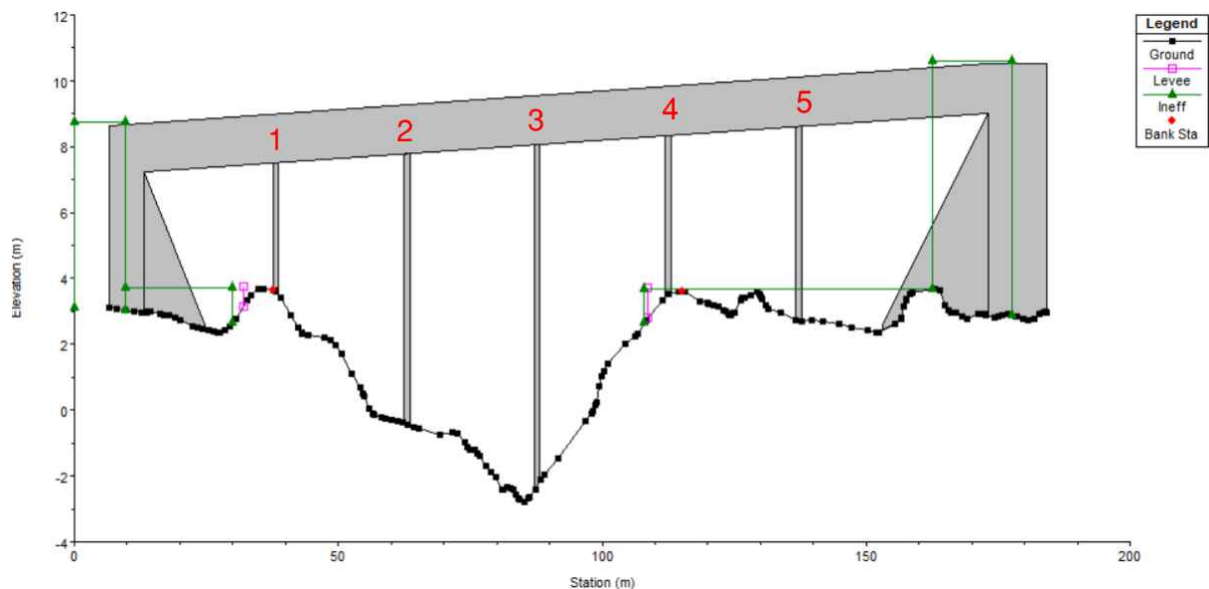


Figure 3. Upstream bridge cross-section

Due to the projection of the cross-sections either side of the bridge, the levees and ineffective flow areas are slightly offset in the internal bridge cross-sections.

2.4 Boundaries

Three hydraulic scenarios were considered for each of the SLS and ULS events to determine the worst-case scenario in each event. These scenarios were considered to estimate how the water levels in the downstream environment impacted flow hydraulics such as velocity and depth, as these variables are important for scour assessment.

The flow boundaries are obtained from NIWA River Flood Statistics Map and scaled up by 22%. This is the difference between historical and projected rainfall in 2100 under RCP8.5 conditions from HIRDS. Tailwater levels were extracted or interpolated from Jacobs Phase 2 Coastal Inundation Modelling Final Study Report (2020).

Table 1. Input river flows and boundary conditions for each of the ARI events and scenarios.

	Upstream Boundary (Flow)	Downstream Boundary (Tailwater level)
Scenario 1: High Flow + Low Tide	100yr ARI: 348m ³ /s 1500yr ARI: 646m ³ /s	Open Boundary
Scenario 2: High Flow + High Tide	100yr ARI: 348m ³ /s 1500yr ARI: 646m ³ /s	10yr Tide: 3.09m 150yr Tide: 3.72m
Scenario 3: Low Flow + High Tide	10yr ARI: 196m ³ /s 150yr ARI: 361m ³ /s	100yr Tide + 1m SLR: 3.05m 1500yr Tide + 1m SLR: 3.41m

3 HECRAS Model Result

3.1 Long Section Outputs

Figure 4 represents the output of river profiles of Kaiapoi River next to the existing bridge at different run scenarios. Each model scenario presents the resulting water level profiles for both ARI 100 and ARI 1500.

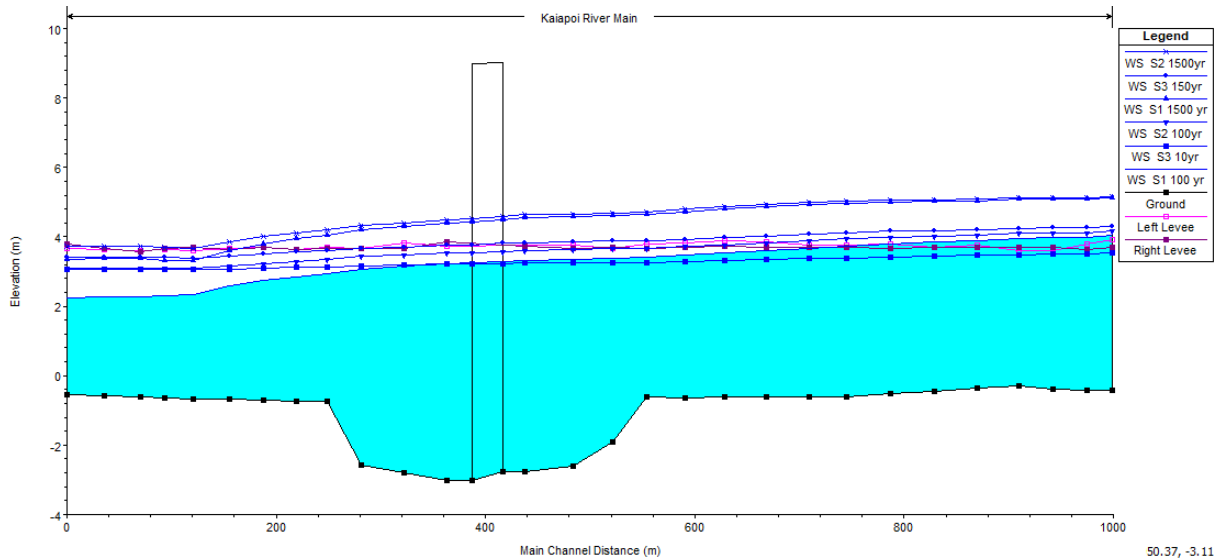


Figure 4. Kaiapoi River long-section profile. Bathymetry extent can be clearly seen approximately 100m up and downstream of bridge

The summary of output water levels at the bridge is listed below. Scenarios 1 and 2 produced similar water levels for the 1500yr ARI event, while Scenario 3 has a much lower water level. This indicates that, for larger flood events, the upstream flow condition is the dominant driver rather than tail water conditions. In contrast, in the 100yr ARI event, scenarios 1 and 3 produce similar results, while scenario 2 has much higher water levels, suggesting tail-water conditions are also important factor for the critical water level at the bridge.

Table 2. Output water levels at bridge

Kaiapoi Bridge Section	Bridge Water Level (m)		
	Scenario 1	Scenario 2	Scenario 3
100yr US	3.23	3.58	3.29
100yr DS	3.24	3.55	3.21
1500yr US	4.49	4.58	3.81
1500yr DS	4.42	4.52	3.77

3.2 Bridge Cross-Section Outputs

The existing geometry of the Kaiapoi River Bridge is incorporated in the model using provided as-built plans and LiDAR information to represent the structure. Figure 5 shows the HEC-RAS water level outputs in cross-section. The bridge sections are modelled and presented facing downstream with input ineffective areas on both sides of the river behind the flood banks. The water levels are the same values previously summarised in Table 2.

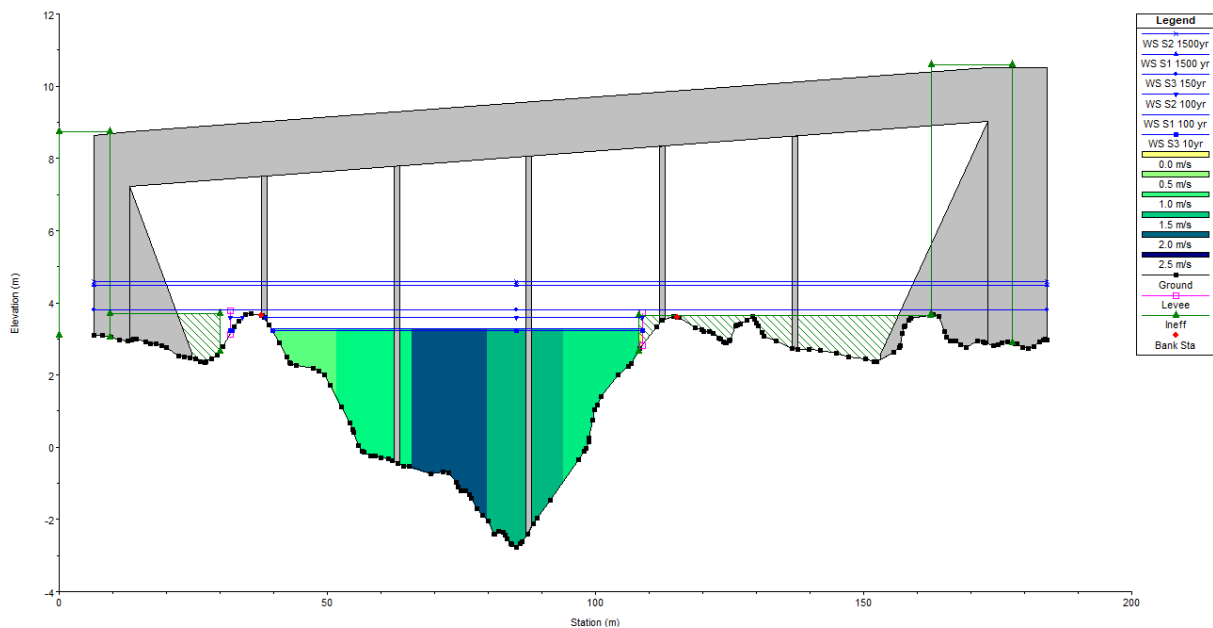


Figure 5. HEC-RAS results in the bridge cross-section

4 Scour Assessment

4.1 Input Data and Assumptions

- Hydraulic inputs of flow, velocity, depth, and flow width were extracted from the HEC-RAS model outputs.
- Geotechnical input of D_{50} was retrieved from the bed material rating curve.
- Bore logs from the bridge construction (1966) give the bed material layers.
- There is approximately 1m of silt buildup on the channel bed which has accumulated in the last 60 years since the channel was excavated. It is assumed this will all flush away in a significant flood event. Scour depths are calculated from the excavated channel invert.
- The scour equations used are empirical equations derived using real world flume studies. These equations do not directly compute the physical processes which result in scour. The equations represent the current best practices for computing scour, however due to their empirical foundation, the actual scour may differ in real world conditions.
- The scour analysis does not account for long term aggradation or degradation (general scour). Due to the surrounding town and stop banks, it is assumed general scour will be minimal.

4.2 Calculations

Scour depths were calculated using procedures outlines in Bridge Scour (Melville and Coleman, 2000) and HEC-18, the US Federal Highway Administrations bridge scour guidelines.

Although the HEC-RAS outputs were used to assess the scour condition, the scour assessment itself was completed using an Excel spreadsheet, based on the guidelines above.

The general scour calculation is summarised in Equation 1:

$$y_s = y_2 - y_o \quad (1)$$

Where y_0 and y_2 represent water depth before and after a scouring event has occurred respectively. The output from the hydraulic model were used in estimating y_0 and the following sections summarise the calculation of y_2 .

4.2.1 Contraction Scour

Contraction scour is the channel erosion attributed to the narrowing of the whole channel upstream of the bridge opening and through the bridge opening. Contraction scour is to be considered across the width of the channel which is capable of transporting material. Both live bed and clear water scour were considered in this assessment.

4.2.1.1 Live Bed

Live bed contraction scour is characterised as the flow regime where bed material is being mobilised upstream of the bridge section.

The trigger for live bed contraction scour is when the upstream flow is inducing sufficient bed shear stress on the channel to mobilise bed material. Under live bed conditions, scouring of a given channel cross section will occur until an equilibrium is reached where the transport of mobilised sediment out of the section equals that coming in.

The live bed equation is given in Equation 2:

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_2}{W_1}\right)^{k_1} \quad (2)$$

Where:

y is the average flow depth through the section, m

Q is the flow through the cross section, m³/s

W is the bottom width of the channel that is transporting bed material, m

Subscripts 1 and 2 refer to the upstream cross section and contracted cross section respectively.

4.2.1.1 Clear Water

Clear water contraction scour is characterised as the flow regime where there is either no bed material being mobilised upstream of the contracted section, or that the mobilised material is being transported at less than the capacity of the flow.

Clear water scour is considered the maximum theoretical scour depth for a given flow and bed material.

The clear water equation is given as Equation 3:

$$y_2 = \left[\frac{K_u Q^2}{D_m^{2/3} W^2} \right]^{3/7} \quad (3)$$

Where:

y_2 is the average equilibrium depth in the contracted section after scour, m

Q is the discharge through the section, m³/s

D_m is the diameter of the smallest non-transportable particle in the bed material ($1.25D_{50}$) in the contracted section, m

W is the bottom width of the channel less pier widths, m

$K_u = 0.025$ (SI constant)

4.2.2 Pier Scour

Pier scour is the channel erosion attributed to the presence of obstructions (piers) in the river flow. Flow velocities increase to move around bridge piers causing the current in these locations to have higher capacity for sediment mobilisation. Pier scour is only to be considered at the base of bridge piers.

Pier scour is dependent on pier shape and size, spacing of columns, and if the pier geometry is complex then every component will contribute to pier scour. See Figure 6 for a sketch of scour contributing pier components. Equation 4 defines the total pier scour from the various components:

$$y_s = y_{s \text{ pier}} + y_{s \text{ pc}} + y_{s \text{ pg}} \quad (4)$$

Where:

$y_{s \text{ pier}}$ = scour component for the pier stem in the flow, m

$y_{s \text{ pc}}$ = scour component for the pile cap in the flow, m

$y_{s \text{ pg}}$ = scour component for the pile group in the flow, m

Each of the scour components are calculated from the basic pier equation (Equation 5), using adjusted depths and velocities as shown in Figure 6.

$$\frac{y_s}{y_1} = 2.0K_1K_2K_3 \left(\frac{a}{y_1} \right)^{0.65} Fr_1^{0.43} K_{comp} \quad (5)$$

Where:

y_1 is flow depth directly upstream of the pier, m

K_1 , K_2 , and K_3 are correction factors for pier nose shape, flow attack angle, and bed condition

a is pier width, m

Fr_1 is Froude Number directly upstream of the pier

K_{comp} is a correction factor depending on the scour component of the pier

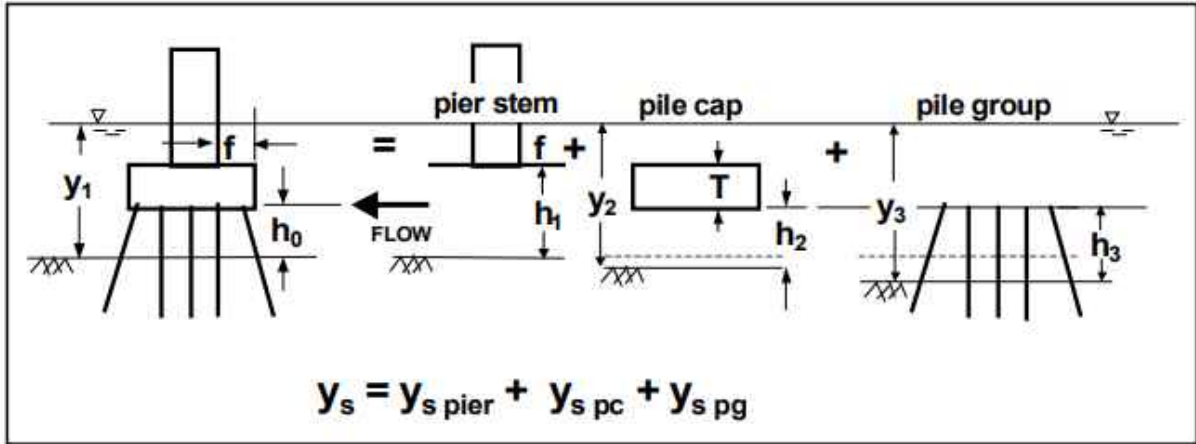


Figure 6. Definition sketch for scour contributing components of a pier with complex geometry

The pier stem component should use an effective pier width, a_d^* (Equation 6), which accounts for a floating debris raft trapped against the pier.

$$a_d^* = \frac{K_{d1}TW(L/y)^{K_{d2}} + (y - K_{d1}T)a}{y} \text{ for } L/y > 1 \quad (6a)$$

$$a_d^* = \frac{K_{d1}TW + (y - K_{d1}T)a}{y} \text{ for } L/y \leq 1 \quad (6b)$$

Where:

K_{d1} , K_{d2} are constants depending on the shape of the debris raft

T is the raft thickness

L is the upstream raft length

W is the raft width normal to flow direction

y is approach flow depth

For a conservative approach, the debris raft was modelled as rectangular.

4.2.3 Abutment Scour

Abutment scour is local scour caused by flow that returns from portions of the floodplain that are cut off by raised embankments. NCHRP (2010) developed a method of calculating abutment scour by considering it as a form of contraction scour focusses at the end of an embankment. The flow is more concentrated close to the abutment, so the contraction scour is greater than the average across the channel.

The abutment scour equations for live bed and clear water conditions are:

$$y_{max} = \alpha_A y_c \text{ or } y_{max} = \alpha_B y_c \quad (7)$$

$$y_s = y_{max} - y_0 \quad (8)$$

Where:

y_{max} is the maximum flow depth resulting from abutment scour, m

α_A is an amplification for live bed conditions

α_B is an amplification for clear water conditions

4.3 Results

The results from the scour calculations have been summarised in the following tables:

Table 3. Contraction scour (m) in the main channel.

	S1 ARI100	S2 ARI100	S3 ARI100	S1 ARI1500	S2 ARI1500	S3 ARI1500
Scour Condition	Clear-Water	Clear-Water	Clear-Water	Live-Bed	Live-Bed	Clear-Water
Live Bed Scour depth	0.12	0.05	0	0.25	0.25	0.04
Clear Water Scour depth	-0.16	-0.51	-1.7	1.0	0.86	-0.63

For contraction scour, the worst-case scenario was used in each case, even though this was the opposite of what was used for the critical velocity check. This conservative approach ensures the design accommodates the maximum potential scour depth..

Table 4. Pier scour (m).

	S1 ARI100	S2 ARI100	S3 ARI100	S1 ARI1500	S2 ARI1500	S3 ARI1500
Pier 1	0	0	0	1.55	1.55	0
Pier 2	3.38	3.37	2.68	4.22	4.20	3.41
Pier 3	3.38	3.37	2.68	4.22	4.20	3.41
Pier 4	0	0	0	1.54	1.54	0
Pier 5	0	0	0	1.47	1.47	0

Piers 1, 4, and 5 are only engaged during high flow Scenarios in the 1500yr ARI event, resulting in zero scour in all other cases.

Table 5. Abutment scour (m) based on live bed contraction scour of flow behind the stop banks.

	S1 ARI100	S2 ARI100	S3 ARI100	S1 ARI1500	S2 ARI1500	S3 ARI1500
Left	0	0	0	4.60	4.62	0
Right	0	0	0	1.18	1.23	0

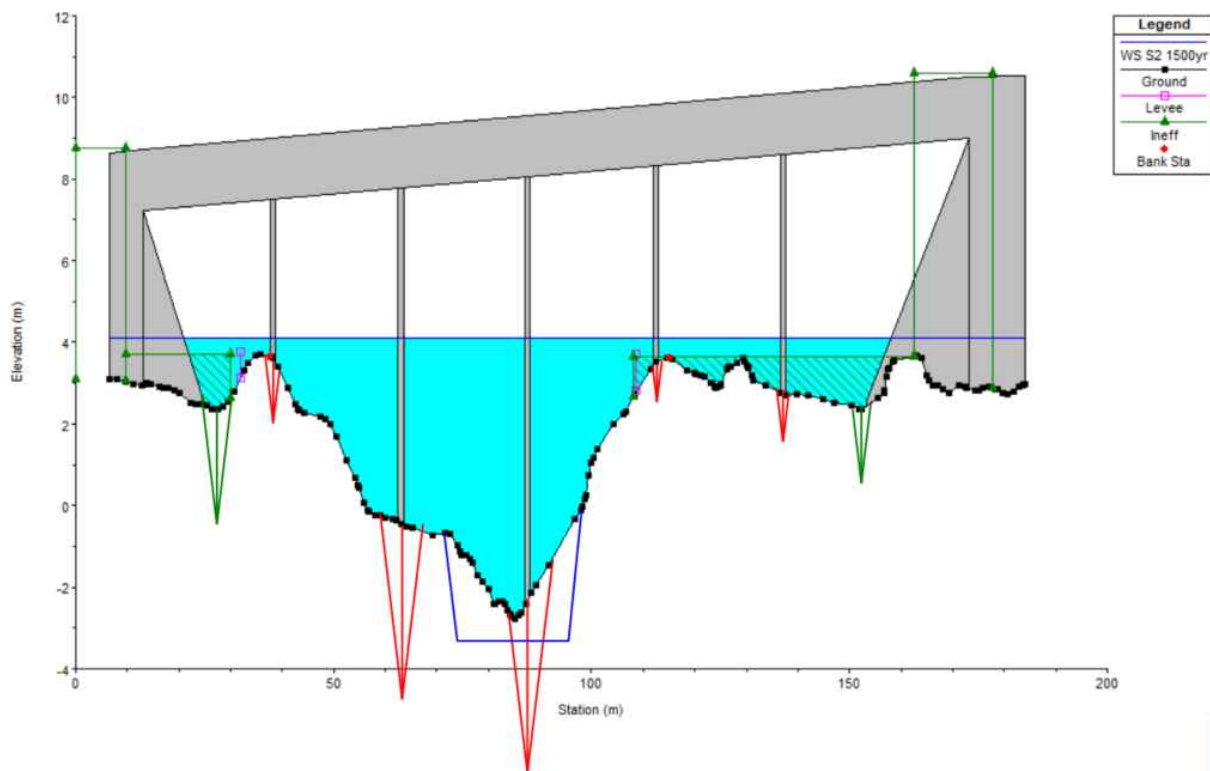


Figure 7. Graphical visualisation of scour depths in cross-section for Scenario 1, ARI 1500 years (worst case).

4.4 Riprap Protection Measures

No scour protection is required at the abutments as there is no scour in the SLS (100-year ARI) flood event. The 2.68 m and 1.79 m of scour in the ULS flood event (1500-year ARI) is not enough to affect the stability of the bridge, so this is an acceptable level of scour in this event

No scour protection is required at the base of the piers. Scour protection is not required in the SLS flood event, and the scour depths in the ULS event does not affect the stability of the bridge.

5 Conclusion

The deepest scour will likely occur around and behind piers 2 and 3 in the main channel, at a predicted depth of 4.1m under the most extreme flow conditions in the 1500yr ARI events.

Significant scour is also likely at the base of both embankments, where the overbank flows are constricted under the bridge. The abutment scour will only occur during larger events when floodwater overtops the flood banks. While the roads at the base of each abutment may offer some protection, the gravel road at the left abutment remains vulnerable to erosion.

In both ARI events, Scenarios 1 and 2 show minimal difference, indicating tidal tailwater conditions have limited influence on the overall scour outcomes.

Appendix F

Cam River Scour Assessment Memo

Memorandum

To	Brendon French, NZTA	From	Jamie Fleming, Aurecon Yanni Hooi, Aurecon
Copy	Bronwyn Leek, NZTA Matt Flannery, Aurecon Katherine Eveleigh, Aurecon Haden Walters, Aurecon	Reference	11320
Date	2025-12-18	Pages (including this page)	16
Subject	Cam River Bridge Scour Assessment Memo		

1 Introduction

This memorandum summarises the HEC-RAS model schematisation, results, and scour assessment findings for the Cam River Bridge.

The following scenarios have been assessed to identify the worst-case conditions for water level and potential scouring:

- Scenario 1: High Flow + Open Boundary
- Scenario 2: High Flow + High Tide (Combination AEP as specified by WDC at a 1:10 ratio)
- Scenario 3: High Tide + Low Flow (Combination AEP as specified by WDC at a 1:10 ratio)
- Scenario 4: High Flow + WDC Bathtub

Each scenario was modelled using the 100-year ARI and 1500-year ARI events to assess both the Serviceability Limit State (SLS) and Ultimate Limit State (ULS).

The HEC-RAS model was developed to determine the critical water levels and velocities under each scenario that could contribute to scouring. Model schematisation and results are presented in Section 2 of this memorandum. The scour assessment was conducted in accordance with Bridge Scour (Melville and Coleman, 2000) and HEC-18, the US Federal Highway Administrations bridge scour guidelines and findings are summarised in Section 3.

1.1 Related Reports

The HEC-RAS model input data was extracted from the existing Waimakariri District and Urban Mike Flood Model and Jacob Phase 2 Coastal Inundation Model.

The related reports are listed below:

DHI. (2020, May). Flood Hazard Models Update District and Urban and Mike Flood Models.
https://www.waimakariri.govt.nz/__data/assets/pdf_file/0008/140111/district-flooding-map-2019-update-DHI-report.pdf

Jacobs. (2020, March). Phase 2 Coastal Inundation Modelling Final Study Report.
https://www.waimakariri.govt.nz/__data/assets/pdf_file/0009/140112/Phase-2-Coastal-Inundation-Modelling-report-including-all-Appendices-Jacobs-Final-peer-reviewed.pdf

2 HECRAS Model Build

The hydraulic assessment of the Cam River Bridge was done using one-dimensional HECRAS model using a steady state flow analysis.

2.1 Plans



Figure 1 shows the plan of the Cam River Bridge HECRAS model. The model contains a river branch, cross sections, riverbank lines, and the existing and new Cam River bridges.

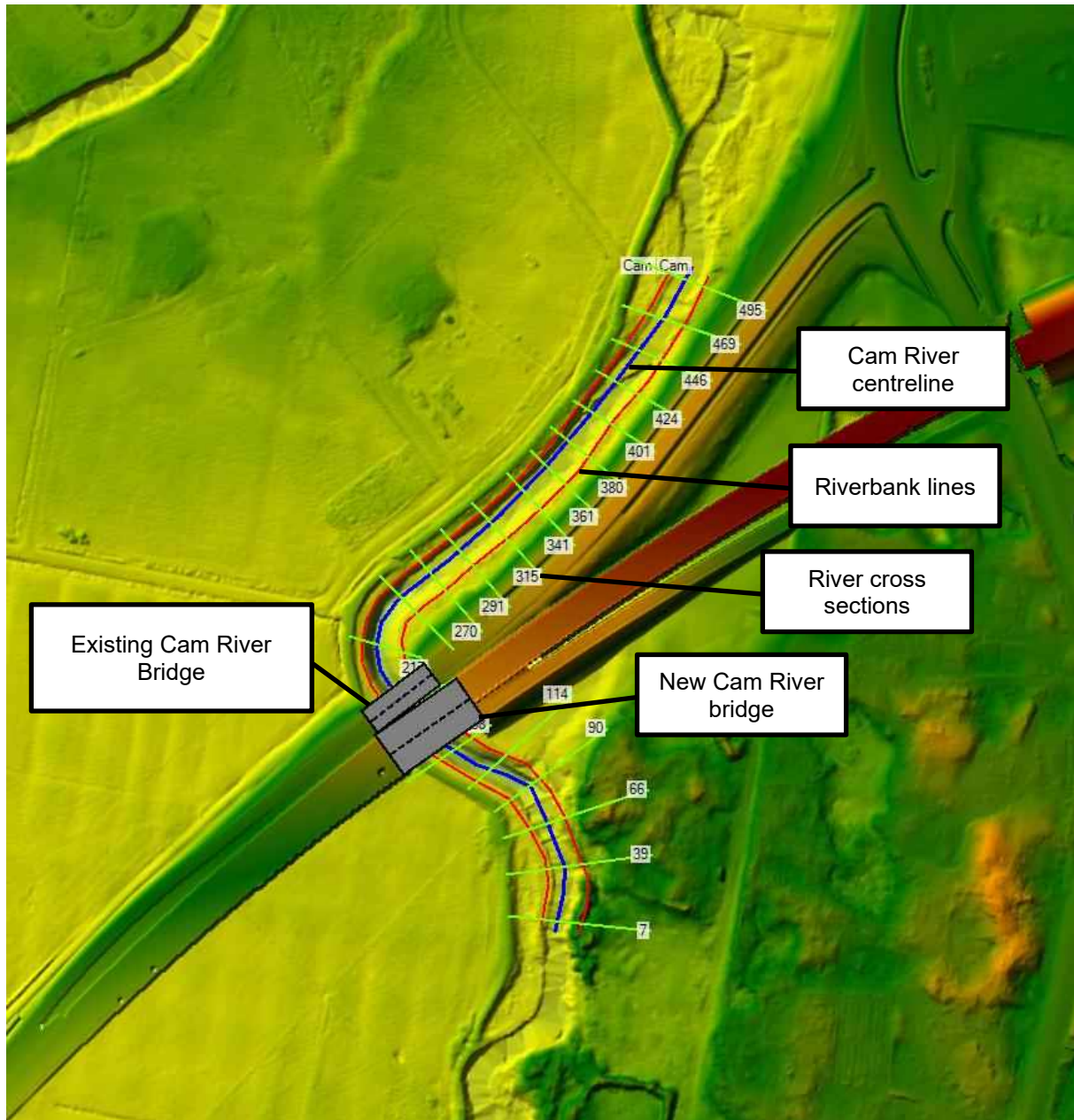


Figure 1. 1-D model geometry plan

2.2 Topography

The digital terrain was taken from the Canterbury 2020/2021 1m LiDAR DEM, obtained from LINZ Data Service in March 2025, supplemented by bathymetry with extents 100m either side of the bridge, scanned by [Elliot Sinclair on 09/05/2025](#).

2.3 1-D Model Geometry

The cross sections were drawn into HEC-RAS at approximately 20-30m intervals for the main channel reach and either side of the bridge. Bank lines were traced along the riverbanks adjacent to the river,

with ineffective flow areas behind the stopbank and close to the end of the road embankment (Figure 2).

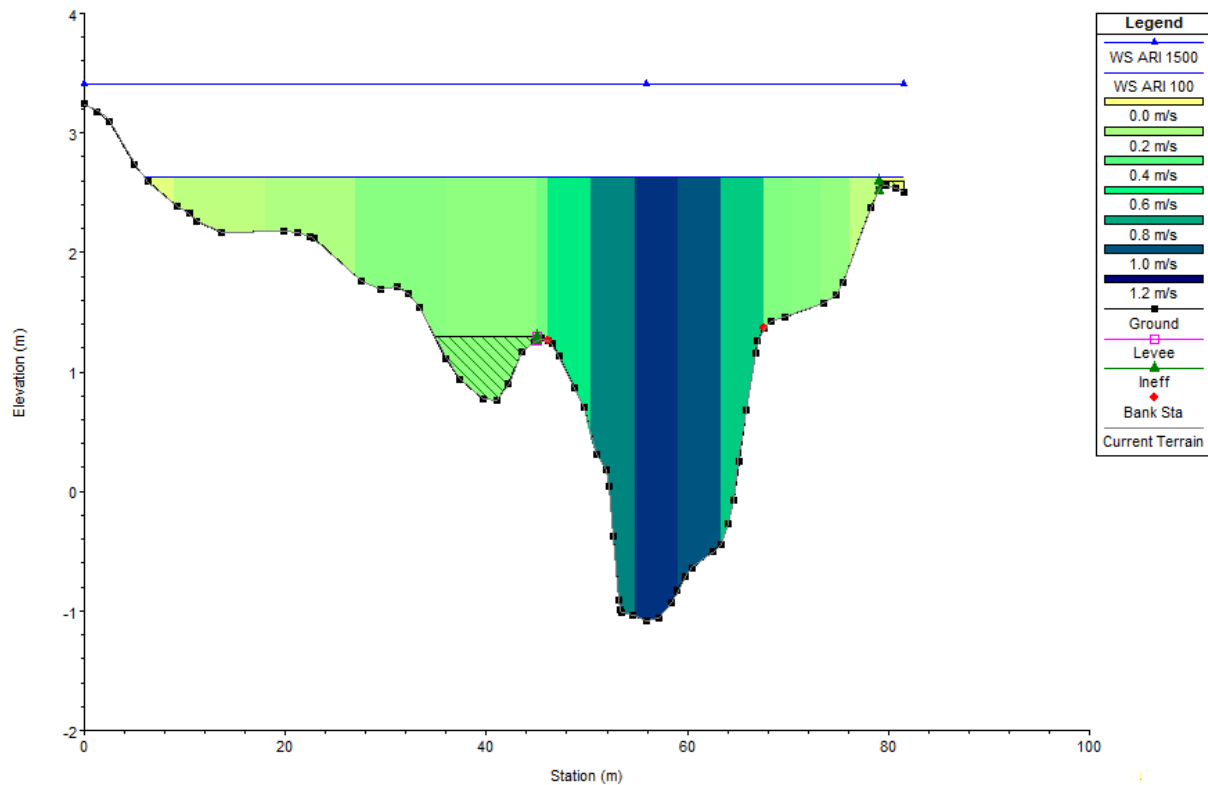


Figure 2. Example cross section showing ineffective flow areas behind stopbanks.

2.4 Boundaries

Four hydraulic scenarios were considered for each of the SLS and ULS events to determine the worst-case scenario in each event. These scenarios were considered to estimate how the water levels in the downstream environment impacted flow hydraulics such as velocity and depth, as these variables are important for assessing scour.

The flows are obtained from NIWA River Flood Statistics Map, then scaled up 22%, based on the projected difference in rainfall between historical data and the year 2100 using RCP 8.5, obtained from NIWA High Intensity Rainfall Design System (HIRDS). Tide levels were extracted or interpolated from Jacobs, 2020.

Table 1. Input river flows and boundary conditions for each of the ARI events and scenarios.

	US Boundary (Flow)	DS Boundary (WSE/Tide)
Scenario 1: High Flow + Low Tide	100yr ARI: 65m ³ /s 1500yr ARI: 123m ³ /s	Open Boundary
Scenario 2: High Flow + High Tide	100yr ARI: 65m ³ /s 1500yr ARI: 123m ³ /s	10yr Tide: 3.09m 150yr Tide: 3.72m

	US Boundary (Flow)	DS Boundary (WSE/Tide)
Scenario 3: Low Flow + High Tide	100yr ARI: 36.42m ³ /s 1500yr ARI: 67.76m ³ /s	100yr Tide + 1m SLR: 3.05m 1500yr Tide + 1m SLR: 3.41m
Scenario 4: WDC Bathtub + WDC Water Level	100yr ARI: 65m ³ /s 1500yr ARI: 123m ³ /s	3.308m

3 HECRAS Model Result

3.1 Long Section Outputs

The following figures represent the output river profiles of Cam River next to the existing and new bridges at different run scenarios. Each model scenario presents the resulting water level profiles for both ARI 100 and ARI 1500. The dip in riverbed level approx. 100m either side of the bridges shows the bathymetry survey extent

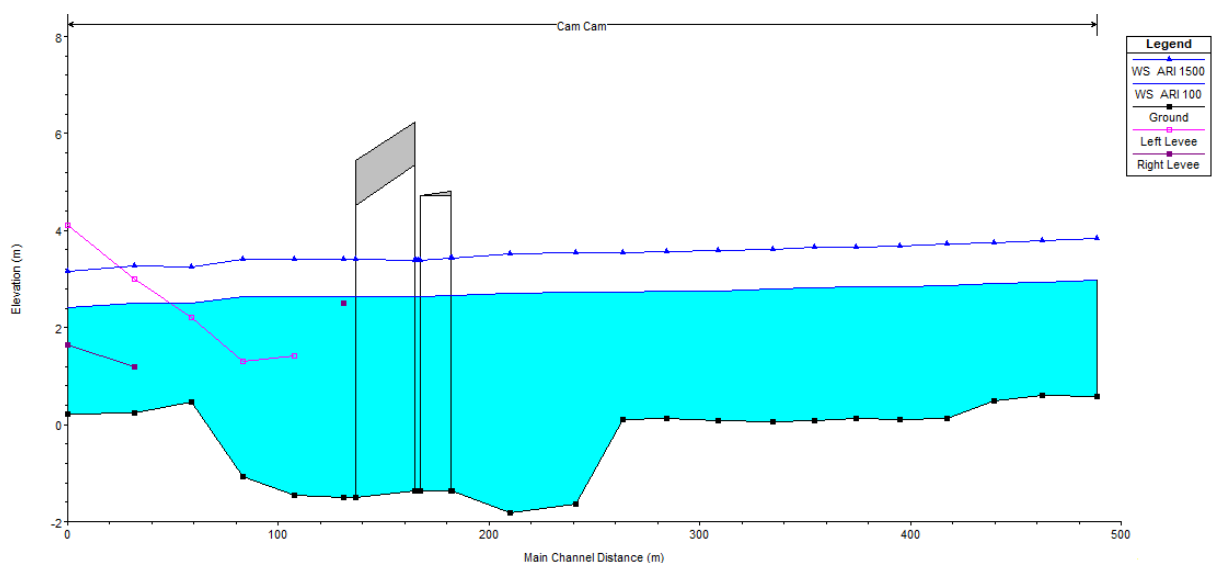


Figure 3. Cam river long profile (Scenario 1 – high flow low tide)

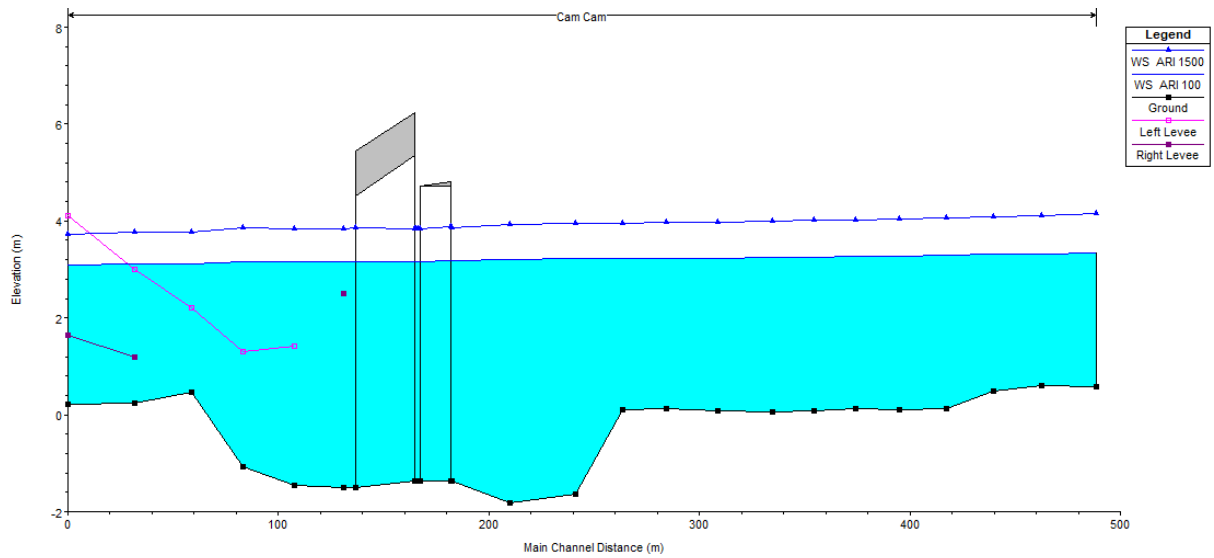


Figure 4. Cam river long profile (Scenario 2 – high flow high tide)

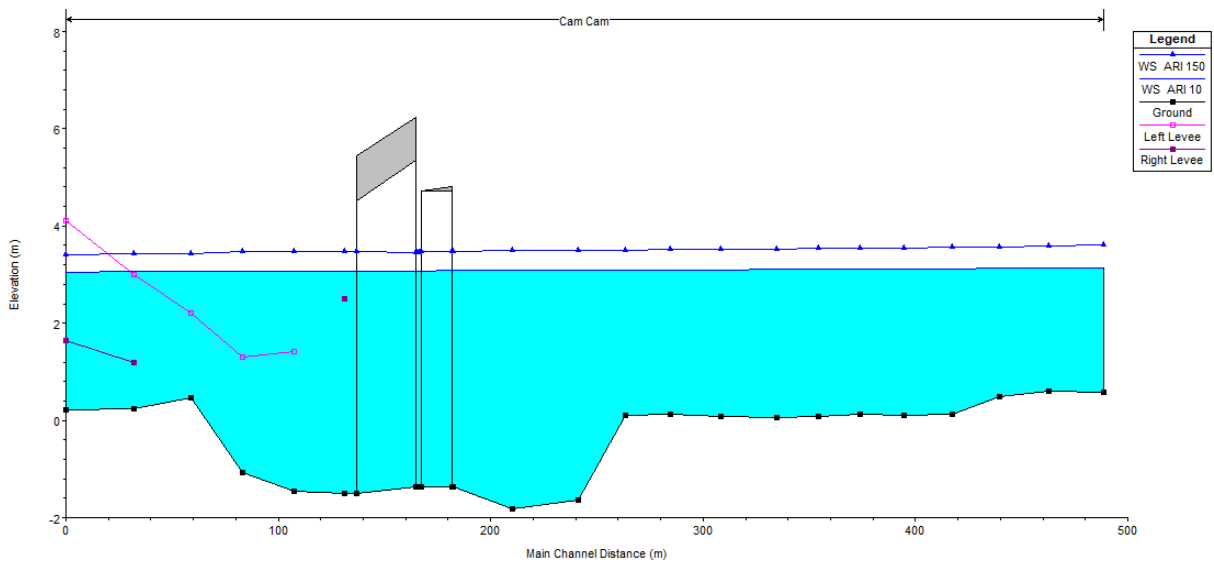


Figure 5. Cam river long profile (Scenario 3 – extreme tide)

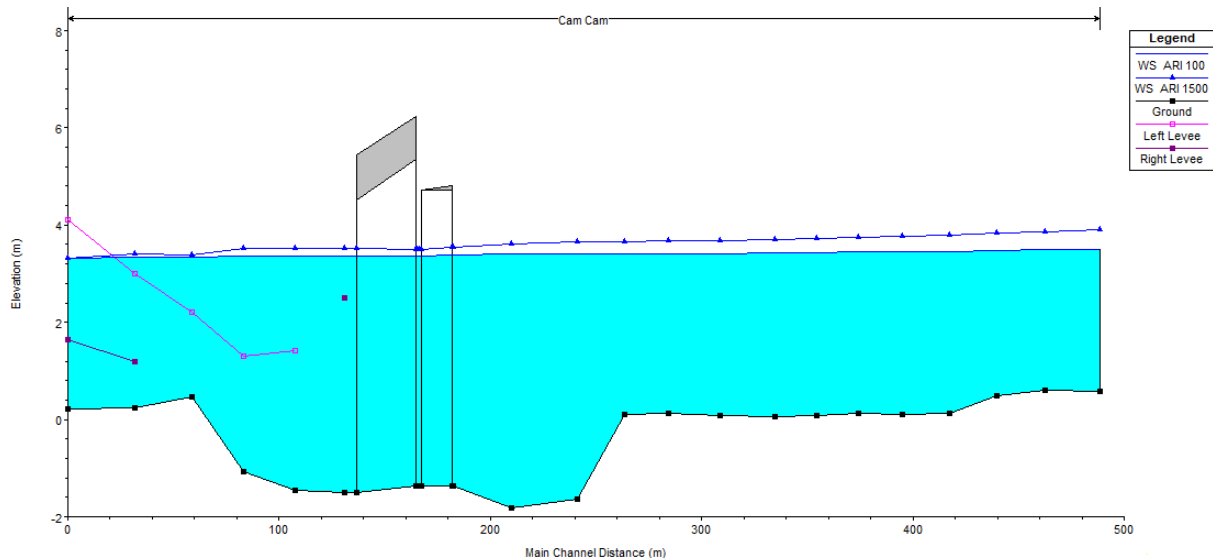


Figure 6. Cam river long profile (Scenario 4 - WDC bathtub scenario)

The summary of output water levels at the bridge section is listed below. US values are upstream of the old bridge, while DS values are downstream of the new bridge. For NIWA100, the water level generally increases from scenario 1 to 4, however, scenario 3 is about 0.1m lower than scenario 2. As for NIWA 1500, the water level under Scenario 2 is the highest. The water levels for scenarios 1, 3, and 4 increases respectively.

Table 2. Output Bridge Water Levels per Scenario

Cam Bridge Section	Bridge Water Level (m)			
	Sce 1	Sce 2	Sce 3	Sce 4
NIWA 100 US	2.66	3.18	3.08	3.38
NIWA 100 DS	2.64	3.17	3.08	3.37
NIWA 1500 US	3.44	3.87	3.48	3.55
NIWA 1500 DS	3.40	3.85	3.47	3.51

3.2 Bridge Cross Section

The geometry of the proposed Cam bridge is incorporated in the model using design plans to accurately represent the structure across the segment of Cam River. The following figures present the output water levels across the bridge per scenario. The bridge sections are modelled and presented facing downstream with input ineffective areas both sides of the river. The water levels are the same values previously summarised in Table 2.

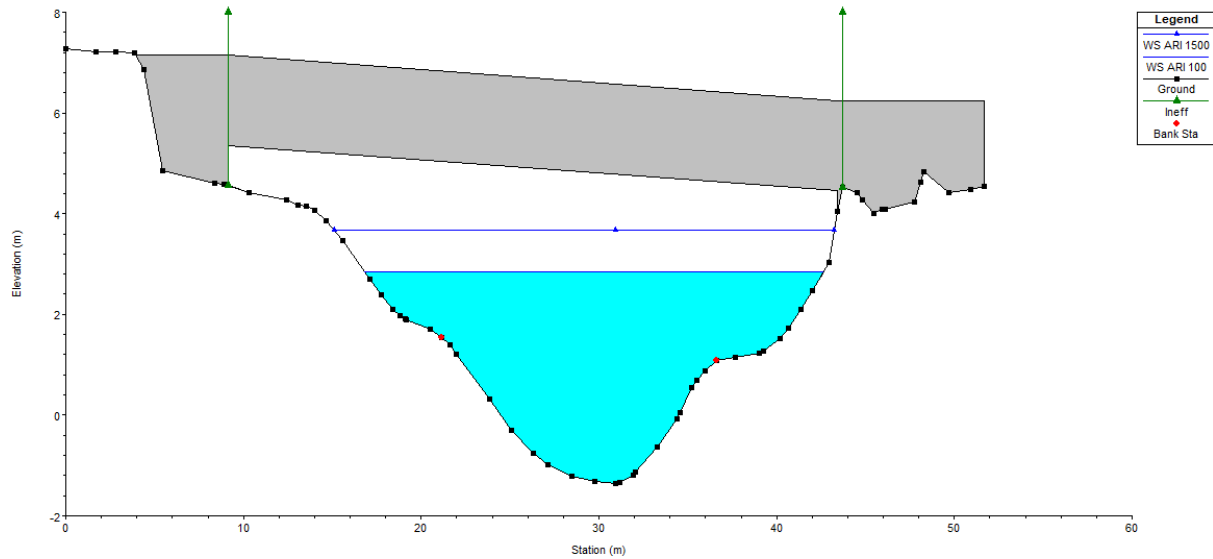


Figure 7. New bridge section (Scenario 1 – Open boundary)

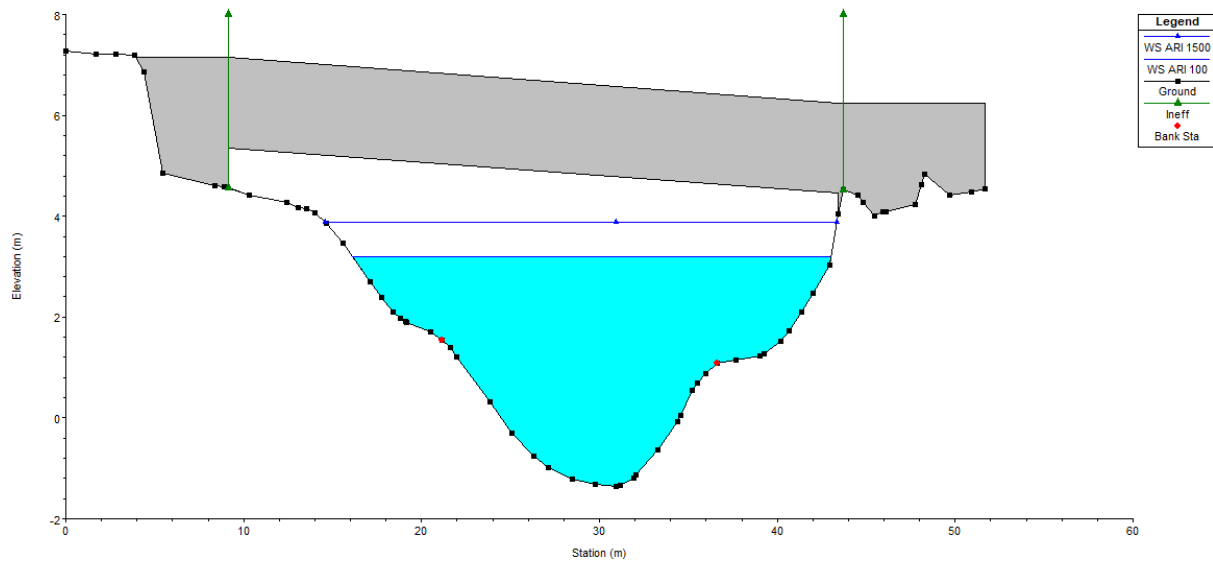


Figure 8. New bridge section (Scenario 2 – high flow high tide)

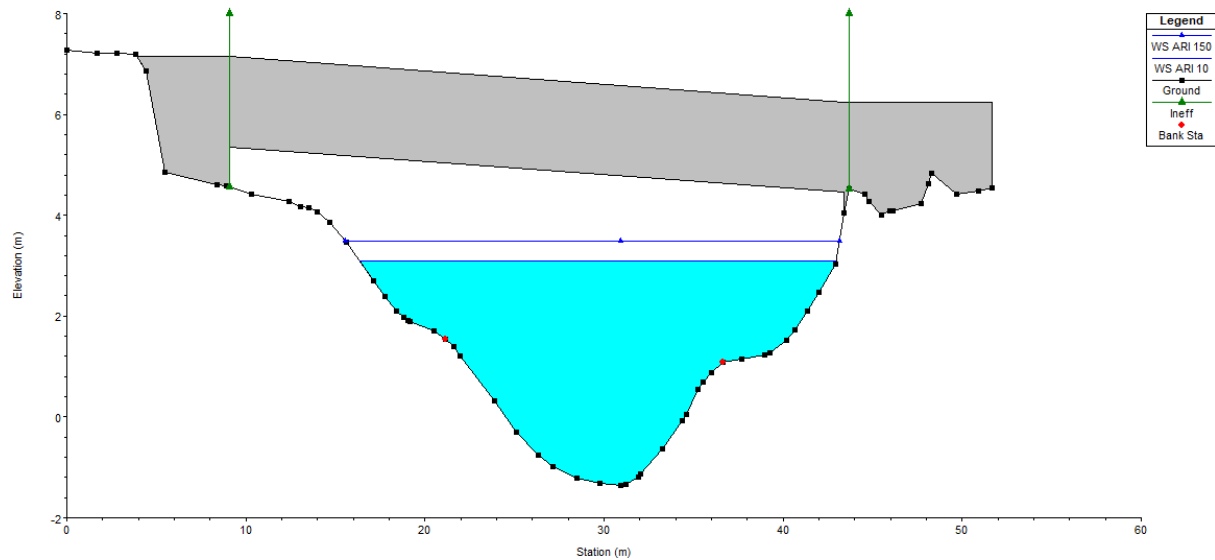


Figure 9. New bridge section (Scenario 3 - extreme tide)

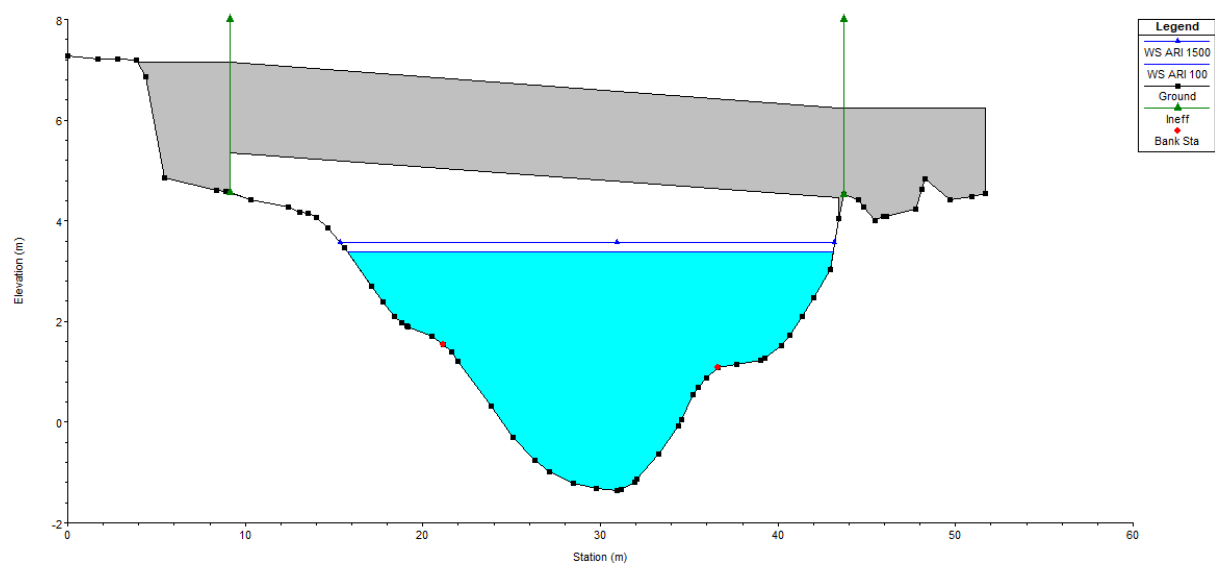


Figure 10. New bridge section (Scenario 4 – WDC bathtub scenario)

4 Scour Assessment

4.1 Input Data and Assumptions

- Hydraulic inputs of flow, velocity, depth, and flow width were extracted from the HEC-RAS model outputs.
- Geotechnical input of D_{50} was retrieved from the bed material rating curve.
- Bore logs from the bridge construction (1966) give the bed material layers.
- There is approximately 1m of silt buildup on the channel bed which has accumulated in the 60 years since the channel was excavated. It is assumed this will all flush away in a significant flood event. Scour depths are calculated from the excavated channel invert.
- The scour equations used are empirical equations derived using real world flume studies. These equations do not directly compute the physical processes which result in scour. The equations represent the current best practices for computing scour, however due to their empirical foundation, the actual scour may differ in real world conditions.
- The scour analysis does not account for long term aggradation or degradation (general scour). Due to the surrounding town and stop banks, it is assumed general scour will be minimal.

4.2 Calculations

Scour depths were calculated using procedures outlines in Bridge Scour (Melville and Coleman, 2000) and HEC-18, the US Federal Highway Administrations bridge scour guidelines.

Although the HEC-RAS outputs were used to assess the scour condition, the scour assessment itself was completed using an Excel spreadsheet, based on the guidelines above.

The general scour calculation is summarised in Equation 1:

$$y_s = y_2 - y_0 \quad (1)$$

Where y_0 and y_2 represent water depth before and after a scouring event has occurred respectively. The output from the hydraulic model were used in estimating y_0 and the following sections summarise the calculation of y_2 .

4.2.1 Contraction Scour

Contraction scour is the channel erosion attributed to the narrowing of the whole channel upstream of the bridge opening and through the bridge opening. Contraction scour is to be considered across the width of the channel which is capable of transporting material. Both live bed and clear water scour were considered in this assessment.

4.2.1.1 Live Bed

Live bed contraction scour is characterised as the flow regime where bed material is being mobilised upstream of the bridge section.

The trigger for live bed contraction scour is when the upstream flow is inducing sufficient bed shear stress on the channel to mobilise bed material. Under live bed conditions, scouring of a given channel cross section will occur until an equilibrium is reached where the transport of mobilised sediment out of the section equals that coming in.

The live bed equation is given in Equation 2:

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_2}{W_1}\right)^{k_1} \quad (2)$$

Where:

y is the average flow depth through the section, m

Q is the flow through the cross section, m³/s

W is the bottom width of the channel that is transporting bed material, m

Subscripts 1 and 2 refer to the upstream cross section and contracted cross section respectively.

4.2.1.1 Clear Water

Clear water contraction scour is characterized as the flow regime where there is either no bed material being mobilised upstream of the contracted section, or that the mobilised material is being transported at less than the capacity of the flow.

Clear water scour is considered the maximum theoretical scour depth for a given flow and bed material.

The clear water equation is given as Equation 3:

$$y_2 = \left[\frac{K_u Q^2}{D_m^{2/3} W^2} \right]^{3/7} \quad (3)$$

Where:

y₂ is the average equilibrium depth in the contracted section after scour, m

Q is the discharge through the section, m³/s

D_m is the diameter of the smallest non-transportable particle in the bed material (1.25D₅₀) in the contracted section, m

W is the bottom width of the channel less pier widths, m

K_u = 0.025 (SI constant)

4.2.1.2 Cohesive Materials

When the material D₅₀ is less than approximately 2mm, the material becomes cohesive, and the above equations no longer hold true. Scour in cohesive materials is very dependent on the critical shear stress of the material. The only reliable way of determining critical shear stress for silt and clay particles is to perform materials testing.

Figure 11 below shows the relationship between critical shear stress and mean particle size. For cohesive materials (D₅₀ < 2mm), there are two equations giving the likely bounding envelope of critical shear stress. Materials may have critical shear stresses much greater than the lower bound, meaning there is potential for little to no scour in these materials, even in extreme flood events.

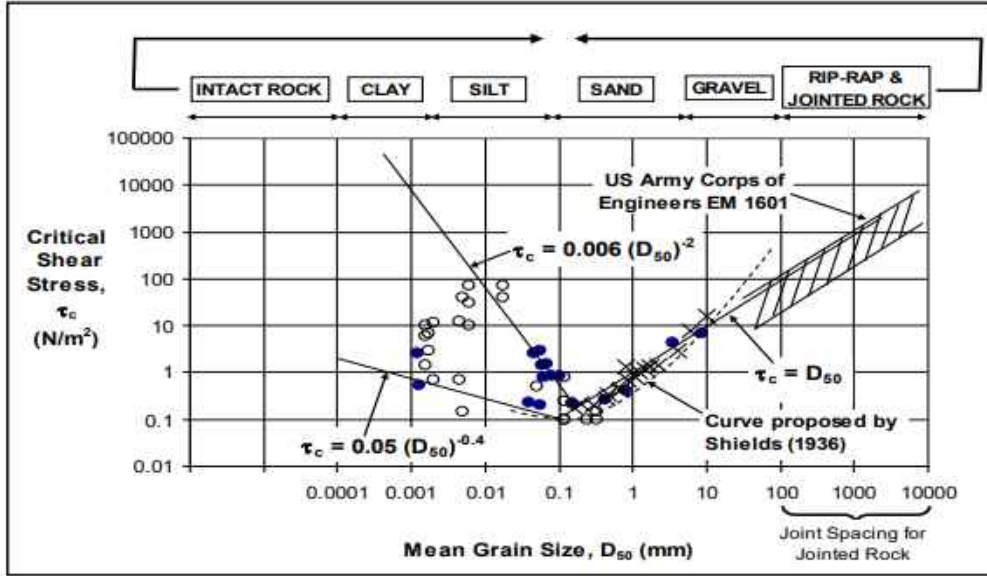


Figure 11. Critical shear stress relationship with D_{50} (Briaud et al. 2011)

Equation 4 defines the initial shear stress exerted on the bed material by flow. If this exceeds the critical shear stress of the material, then scour will occur.

$$\tau = \gamma \left(\frac{V_2 n}{K_u} \right)^2 y_0^{-1/3} \quad (4)$$

Where:

$\gamma = 9810 \text{ Nm}^{-3}$ is the specific weight of water

V_2 is the flow velocity through the contracted section

n is Manning n

$K_u = 1$ in S.I. units

If scour occurs, the depth of contraction scour can be calculated with Equation 5:

$$y_{s-ult} = 0.94 y_1 \left(\frac{1.83 V_2}{\sqrt{g y_1}} - \frac{K_u \sqrt{\frac{\tau_c}{\rho_w}}}{g n y_1^{1/3}} \right) \quad (5)$$

Where:

τ_c is the critical shear stress

ρ_w is density of water

$g = 9.81 \text{ ms}^{-2}$ is acceleration due to gravity

4.2.2 Pier Scour

Pier scour is the channel erosion attributed to the presence of obstructions (piers) in the river flow. Flow velocities increase to move around bridge piers causing the current in these locations to have higher capacity for sediment mobilisation. Pier scour is only to be considered at the base of bridge piers.

The proposed Cam River bridge structure does not have any piers, so pier scour was not included in this analysis

4.2.3 Abutment Scour

Abutment scour is local scour caused by flow that returns from portions of the floodplain that are cut off by raised embankments. NCHRP (2010) developed a method of calculating abutment scour by considering it as a form of contraction scour focusses at the end of an embankment. The flow is more concentrated close to the abutment, so the contraction scour is greater than the average across the channel.

The abutment scour equations for live bed and clear water conditions are:

$$y_{max} = \alpha_A y_c \text{ or } y_{max} = \alpha_B y_c \quad (8)$$

$$y_s = y_{max} - y_0 \quad (9)$$

Where:

y_{max} is the maximum flow depth resulting from abutment scour, m

α_A is an amplification for live bed conditions

α_B is an amplification for clear water conditions

4.2.4 Bend Scour

Bend scour is local scour caused by higher velocity flow around the outside of a bend than in the straight sections of a channel. The flow depth on the outside of a bend is usually greater than the average depth of a straight channel, while the depth on the inside of the bend is usually less.

$$\frac{Y_{bs}}{Y_u} = 1.8 - 0.051(r_c/W) + 0.0084(W/Y_u) \text{ for } 1.5 < r_c/W < 10 \text{ and } 20 < W/Y_u < 125 \quad (10a)$$

$$\frac{Y_{bs}}{Y_u} = 2.07 - 0.19 \ln[(r_c/W) - 2] \text{ for } r_c/W > 2 \quad (10b)$$

Where:

Y_{bs} is the scour depth at the bend

Y_u is the average channel depth upstream of the bend

r_c is the centreline radius of the bend.

It is recommended to adopt values of $r_c/W=1.5$ when $r_c/W < 1.5$, and $W/Y_u=20$ for values of $W/Y_u < 20$.

4.3 Results

From geotechnical bore logs, the top layer of bed material of the Cam River is predominantly made up silty-clay, which is a cohesive material. As specified in section 4.2.1.2, the only reliable way of determining critical shear stress for cohesive materials is to perform materials testing. No laboratory testing of this material was done, so the D_{50} of this material is assumed to be $2\mu\text{m}$ ($2 \times 10^{-6}\text{m}$). This is

on the upper end of particle sizes defined as clay, at the boundary with silt (refer to Figure 11). The critical shear stress was assumed to be the lower bound value for this particle size, leading to clear water scour occurring.

Table 2. Clear-water contraction scour (m) for Cam River Bridge.

	Scenario 1	Scenario 2	Scenario 3	Scenario 4
100yr ARI	1.55	1.35	0.73	2.16
1500yr ARI	2.50	2.27	1.33	2.44

Table 3. North abutment scour (m) for Cam River Bridge.

	Scenario 1	Scenario 2	Scenario 3	Scenario 4
100yr ARI	3.04	3.98	3.18	4.32
1500yr ARI	5.44	5.92	4.38	5.62

For bend scour, both 100 and 1500 year ARI events for all scenarios have the value $W/Y_u < 20$, so 20 was the adopted value. They also all have $r_b/W < 1.5$, so 1.5 was the adopted value. All events therefore have a bend scour of:

$$Y_{bs} = 1.89 \text{ m}$$

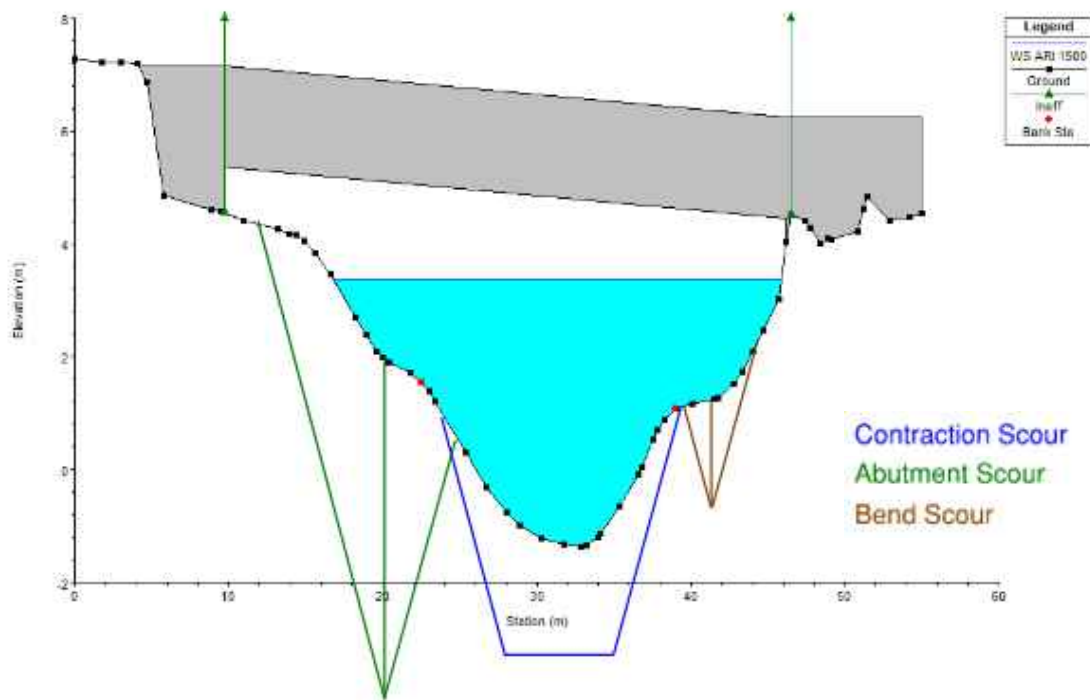


Figure 12. Graphical visualisation of scour depths in cross-section for the worst case scenario in each scour mode

4.4 Riprap Protection Measures

Scour protection is required at the abutments to prevent scour in the SLS (100-year ARI) flood event. The minimum D_{50} was calculated using HEC23 Design Guidelines 4 and 14, giving 40 mm and 120 mm for the bend scour and abutment scour respectively. To prevent people being able to easily carry away the riprap, a minimum D_{50} of 250 mm was chosen.

The riprap will extend up to 3.25 mRL (600 mm above SLS riprap design flood level) and have a 3.2 m wide apron around the toe of the abutment, starting from where the embankment slope intersects the existing ground. The revetment riprap will be placed on top of the existing bank, while the apron will be excavated into the riverbed, so the top surface will be inline with the existing ground level. In front of the south abutment, the apron will be buried at least 300mm deep below the level of the proposed future cycleway.

The riprap protection will extend at least 8m downstream on the south bank, following and built up to the top of the existing flood bank. The flood bank is not to be excavated into. The riprap protection will wrap around the downstream side of the north embankment, extending at least 8m back from the bridge abutment.

The riprap protection will end at the interface between the proposed and existing bridge abutments. Both the revetment and apron will thicken to $4 \cdot D_{50}$ at this interface to provide a strong upstream anchor to the riprap.

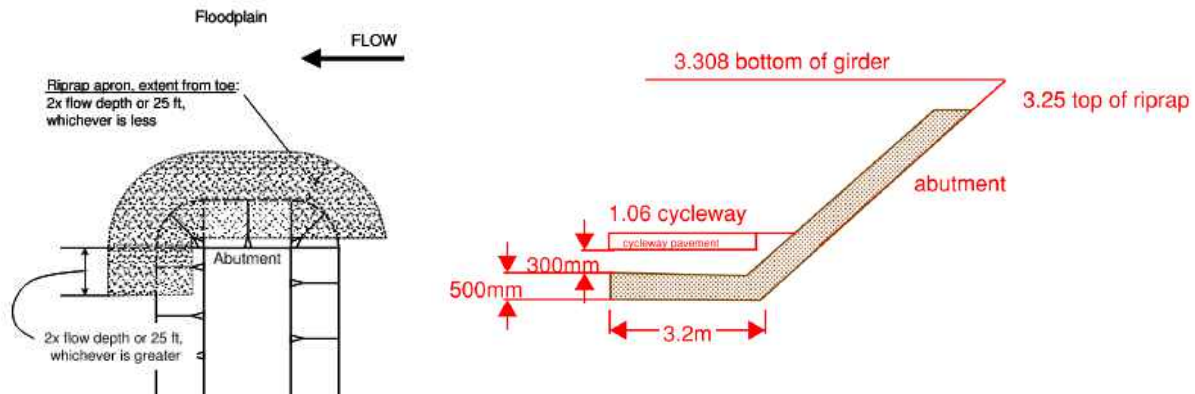


Figure 13. Sketch of riprap layout around south abutment. Image on left is from Lagasse et al. 2006

Additional scour protection is not required for the ULS event scour as the scour does not affect the stability of the bridge.

5 Conclusion

The deepest scour will likely occur around the north abutment, at a predicted depth of 5.92 m in the worst case– Scenario 2, joint probability high flow and high tide.

Contraction scour of predicted depths up to 2.50 m are expected across the main channel width, with an additional 1.89 m on the right side of the river going around the bend.

Scenarios 1 and 3 have similar scour depths to each other, as do scenarios 2 and 4. This indicates the high flow and high tide conditions have a similar influence on overall scour outcomes.

Appendix G

Flood Impact Assessment South of Cam River

Memorandum

To	Brendon French, NZTA	From	Eldrin Ragasa, Aurecon Yanni Hooi, Aurecon
Copy	Bronwyn Leek, NZTA Matt Flannery, Aurecon Katherine Eveleigh, Aurecon Haden Walters, Aurecon	Reference	11320
Date	2025-09-26	Pages (including this page)	6
Subject	Flood Impact Assessment South of Cam River		

1 Introduction

This memorandum presents an assessment of the potential impact of proposed road widening on flood levels within a hydraulically disconnected floodplain area, a "bathtub" area. The assessment aims to quantify the change in flood storage capacity and the resulting rise in flood levels for a range of design flood events. The analysis has been carried out to support design decision-making and to ensure that potential flood effects are understood and appropriately managed within the project scope.

1.1 Related Reports

The flood depth and level rasters were provided by the council. These are utilised in GIS to extract flood levels for volume computations.

The related reports are listed below:

DHI. (2020, May). Flood Hazard Models Update District and Urban and Mike Flood Models.
https://www.waimakariri.govt.nz/__data/assets/pdf_file/0008/140111/district-flooding-map-2019-update-DHI-report.pdf

Jacobs. (2020, March). Phase 2 Coastal Inundation Modelling Final Study Report.
https://www.waimakariri.govt.nz/__data/assets/pdf_file/0009/140112/Phase-2-Coastal-Inundation-Modelling-report-including-all-Appendices-Jacobs-Final-peer-reviewed.pdf

2 Analysis Setup

The assessment was undertaken to estimate the potential rise in flood levels within a hydraulically disconnected floodplain area, "bathtub" area, as a result of proposed road widening. The methodology is as follows:

■ Estimation of Lost Storage Volume

The volume of flood storage displaced by the proposed road embankment was calculated by determining the sum of ground level differences on a 1 m x 1 m grid within the footprint of the proposed road works.

■ Delineation of Existing Floodplain Extent

The existing flood extents for the 100-year, 200-year, and 500-year ARI events were delineated using outputs from the WDC existing flood model. These extents represent the surface area available for flood storage under different flood events.

■ Flood Level Rise Calculation

The rise in flood level was estimated by dividing the lost storage volume by the surface area of the inundated floodplain (bathtub area). The resulting value provides an approximation of the increase in flood level attributable to the proposed road embankment.

This method provides a conservative estimate of flood level changes and assumes uniform distribution of displaced volume across the affected floodplain. The approach is suitable where hydraulic connectivity is limited, and the affected area behaves as a confined storage basin.

2.1 Topography

The existing topography was established using the Canterbury 2020/2021 1-meter LiDAR Digital Elevation Model (DEM) obtained from the LINZ Data Service in March 2025. This dataset was further enhanced by incorporating the latest survey data collected as part of the project. The proposed road surface design was provided by the roading team during the 50% detailed design phase.

2.2 Proposed Road Width Extent

The bathtub floodplain area under assessment extends from the Kaiapoi Bridge to the Cam River. The proposed road alignment introduces a wider carriageway compared to the existing road (refer to Figure 2). For the purpose of this assessment, the evaluation area covers the full span from Kaiapoi Bridge to the Cam River.

The proposed road widening will result in additional embankment fill encroaching into the adjacent floodplain. This fill effectively reduces the available flood storage volume within the hydraulically disconnected floodplain area. The reduction in flood storage is expected to cause a corresponding increase in flood levels within the inundated zone adjacent to the road. The typical cross-section illustrated in Figure 3 shows the reduction in storage cross-section area.

To quantify this impact, the volume difference between the existing ground surface and the proposed road embankment has been calculated using ArcPro tool. This volume, based on the extent shown in Figure 2, is used to estimate the equivalent rise in flood level within the defined bathtub area.

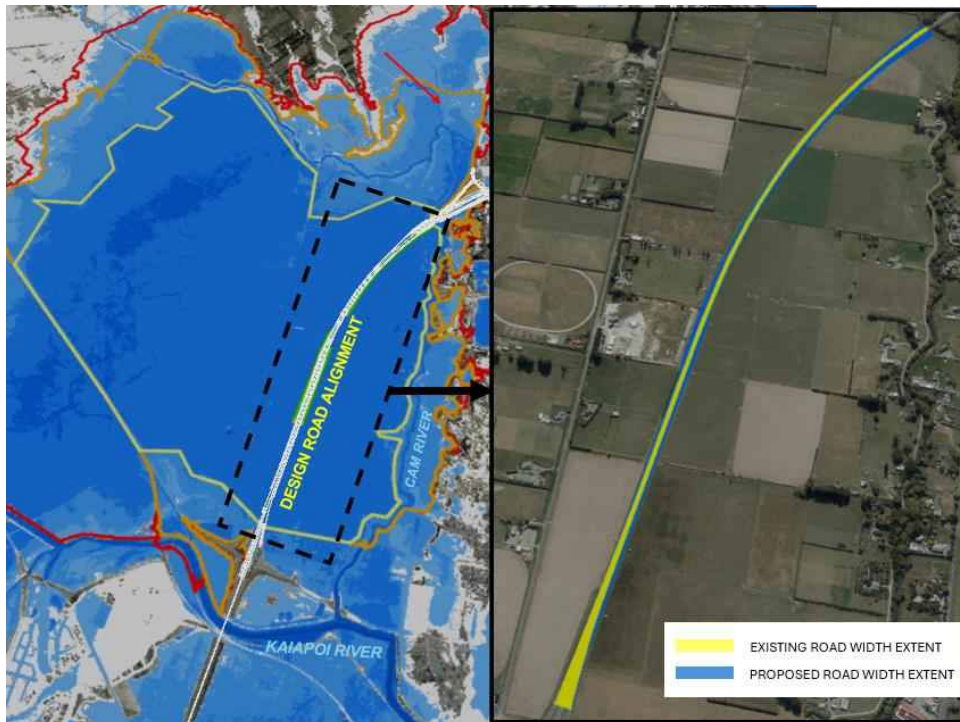


Figure 1. Width extent of the proposed road segment adjacent to Cam River

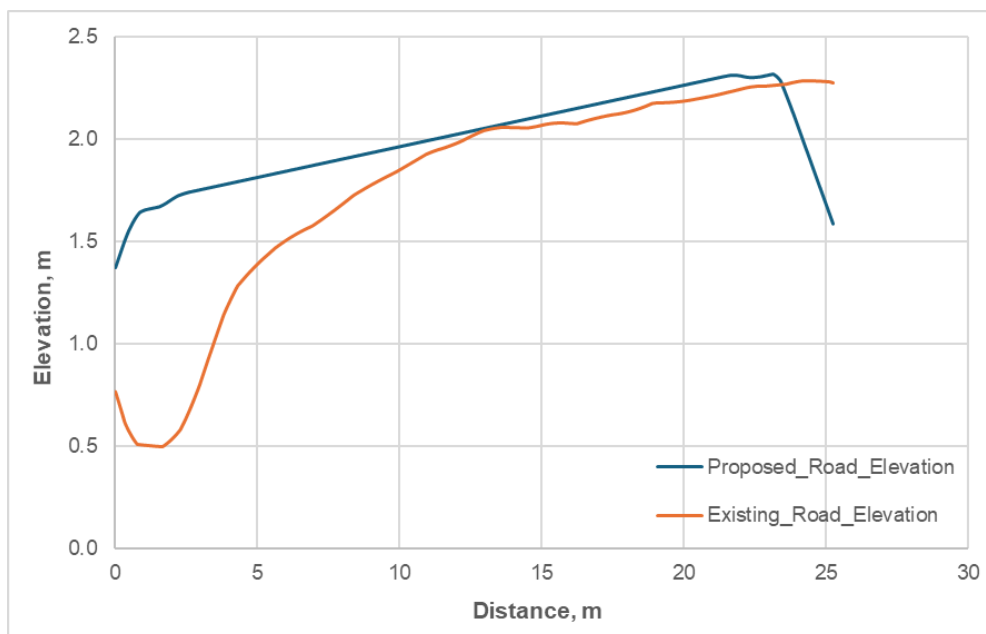


Figure 2. Comparison between typical road sections for the existing and proposed road extents

2.3 Bathtub Boundary

Using flood depth raster for the 100-year, 200-year, and 500-year ARI events, flood extent boundaries were delineated for the inundated area located adjacent to the road alignment, where the floodplain

exhibits a “bathtub” behaviour. These inundation areas were subsequently used in the calculation of flood level increase resulting from the proposed road embankment works.

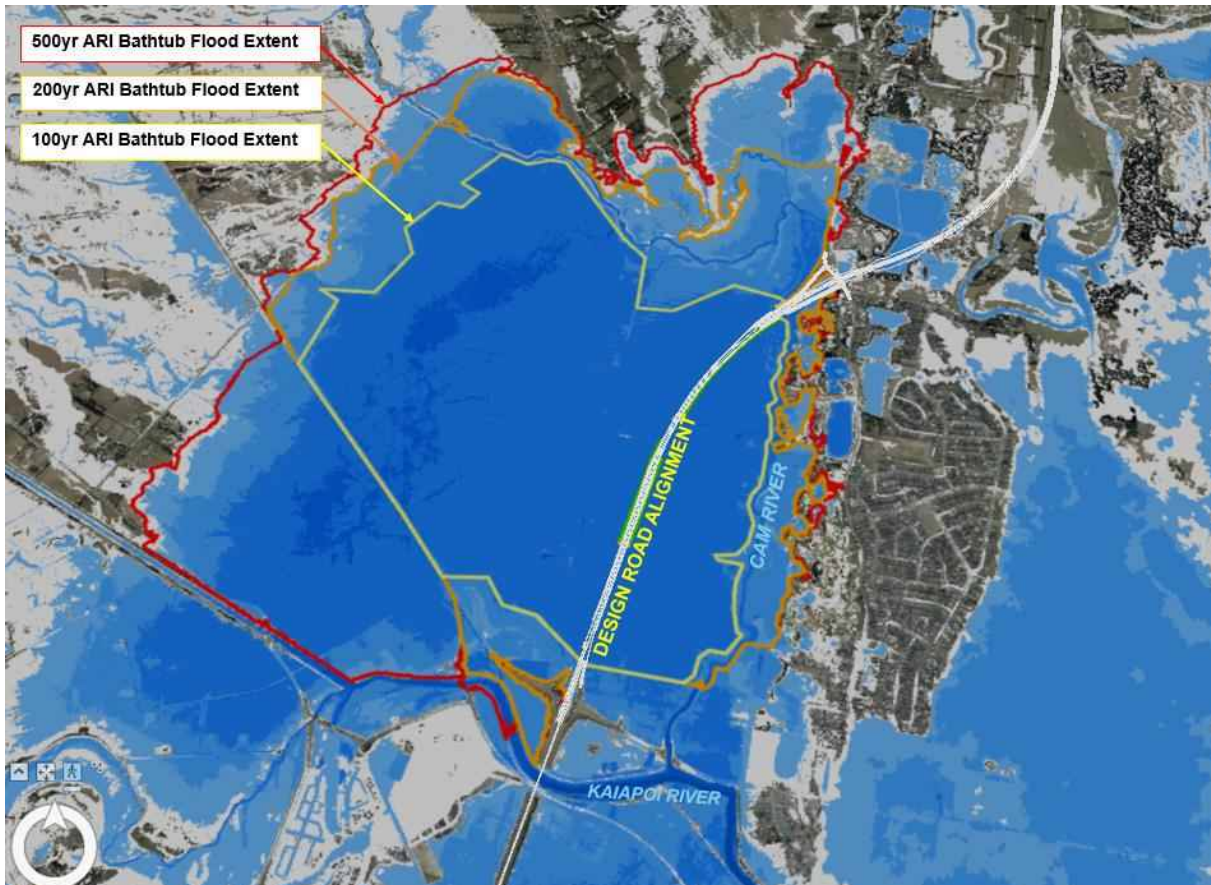


Figure 3. Established area boundaries for pre-development scenario at the bathtub zone in south road alignment

As presented in Figure 1, the following table shows the area extent at the bathtub zone and the corresponding flood levels for each ARI.

Table 1. Flood Levels and Extents

Scenario	Flood Level (Pre-development), m	Bathtub Extent, m2
Overtopping case	1.790	4,693,956
ARI 100 year	3.309	7,173,697
ARI 200 year	3.747	10,007,52
ARI 500 year	4.080	10,007,522

3 Results

3.1 Volume Offset Computation

The volume between the existing and proposed road levels are calculated using the surface volume computation tool in ArcPro. Given the terrain model resolution (at 1 meter by 1 meter per cell), the computed volume difference per cell is added to approximately quantify the total volume difference for the entire road span. The output raster is converted into point features to simply represent the computed volume difference values into each point (see Figure 4). Adding the volume values of all point features, the total volume difference is equal to 15,820m³.



3.2 Water Level Increase Computation

The rise in flood level was estimated by dividing the lost storage volume (15,820 m³) by the surface area of the inundated floodplain (bathtub area). The resulting value provides an approximation of the increase in flood level attributable to the proposed road embankment as shown in Figure 1. The resulting flood level increase values are tabulated as follows:

Table 2. Flood Levels and Extents

Scenario	Flood Level (Pre-development), m	Bathtub Extent, m ²	Flood Level Increase (Post-development), m
Overtopping case (Flood level = road crest level)	1.790	4,693,956	0.0032
ARI 100 year	3.309	7,173,697	0.0021
ARI 200 year	3.747	10,007,522	0.0015

Scenario	Flood Level (Pre-development), m	Bathtub Extent, m ²	Flood Level Increase (Post-development), m
ARI 500 year	4.080	10,007,522	0.0015

4 Conclusion

The analysis indicates a minor increase in flood levels within the bathtub area due to the proposed road embankment. By dividing the displaced storage volume (15,820 m³) by the inundated surface area for each ARI event, the estimated rise in flood levels was calculated. The resulting increases range from approximately 0.0032 m under the overtopping scenario (when flood level exceed the minimum road crest level) to 0.0015 m for the 200 year and 500 year ARI events. These results suggest that the proposed road widening will have a minimal impact on flood levels within the affected floodplain extent.

Appendix H

Flood Assessment Report

Belfast to Pegasus Motorway & Woodend Bypass Pre-implementation and MSQA Professional Services

Flood Assessment Report

NZ Transport Agency (NZTA)

Document code: 11320-AUR-0350-PWI-SW-RPT-0001

Revision: B

Date prepared: **19 December 2025**



Document control record

Document prepared by:

Aurecon New Zealand Limited

Level 2, Iwikau Building
93 Cambridge Terrace
Christchurch 8013
New Zealand

T +64 3 366 0821

F +64 3 379 6955

E christchurch@aurecongroup.com

W aurecongroup.com

Document control						aurecon
Report title		Flood Assessment Report				
Document code		11320-AUR-0350-PWI-SW-RPT-0001	Project number		11320	
Document state		PUB Published	Status code		P3 Detailed Design	
File path		https://aurecongroup.sharepoint.com/sites/528638/5_WorkingFiles/Stormwater/03_Reporting/TUFLOW Flood Model Report/11320-AUR-0350-PWI-SW-RPT-0001 -100%DD.docx				
Rev	Date	Revision details	Author	Verifier (if required)	Approver	
A	2025-08-20	Prelim 30% design flood modelling	Y. Hooi T. De Leon E. Ragasa	S. Vaakesan	M. Flannery	
B	2025-12-19	Prelim 100% design flood modelling	Y. Hooi T. De Leon E. Ragasa N. Zhang	D. Delagarza	H. Walters	
Current revision		B				

Client			
Client name	NZ Transport Agency (NZTA)	Client contact	Brendon French
Client document code	NA	Client document revision	NA

Approval			
Author signature		Approver signature	
Name	Yanni Hooi	Name	Haden Walters
Title	Senior Water Engineer	Title	Design Manager

Contents

1	Introduction and Background	1
1.1	Background	1
1.2	Objectives and Scope of Assessment.....	2
1.3	Flood Model History.....	3
2	B2P Flood Model Development.....	5
2.1	B2P Pre-Development Model	6
2.1.1	Flood Model Validation	10
2.2	B2P Post-Development Flood Model	15
2.2.1	Post-Development Scenario Result Discussion	17
2.3	B2P Mitigation Flood Model	21
2.3.1	Mitigation Scenario Result Discussion	26
2.3.2	Flood Impact Discussion	33
3	Limitation and Applicability	48
4	Conclusion	49

Appendices

Appendix A

B2P Flood Model Build Methodology

Appendix B

B2P Flood Model Modifications and Input Data

Appendix C

Woodend Flood Model Results

Figures

- Figure 1-1 B2P Flood Model Extent with the Proposed Belfast to Pegasus Motorway and Woodend Bypass Road Alignment
- Figure 1-2 South Ashley Model catchment extent
- Figure 1-3 Model Extent of B2P - Woodend Flood Model
- Figure 2-1 Principal River channels within the model catchment extent
- Figure 2-2 Existing culverts crossing proposed road alignment
- Figure 2-3 B2P Flood Model pre-development flood level under 100-year ARI
- Figure 2-4 WDC South Ashley Model pre-development flood depth for 100-year ARI event
- Figure 2-5 B2P-Woodend flood model pre-development flood depth for the 100-year ARI event
- Figure 2-6 B2P Flood Pre-development model 100year ARI flood depth minus WDC South Ashley Model localised flood depth map under 100-year ARI event
- Figure 2-7 B2P Flood Model ground level minus WDC South Ashley Model ground level
- Figure 2-8 WDC Model Ground Level representing an existing 4m x 2m box culvert
- Figure 2-9 Proposed B2P-Woodend design surface level
- Figure 2-10 Floodwaters overtop the road at north of Bob Robertson Drive (200 year ARI Post-Development Scenario)
- Figure 2-11 Impacted properties with 200yr afflux level map (post-development scenario) as background
- Figure 2-12 Proposed mitigation scheme adjacent to the road alignment
- Figure 2-13 Proposed culvert alignments (upper zone)
- Figure 2-14 Proposed culvert alignments (upper central zone)
- Figure 2-15 Proposed culvert alignments (lower central zone)
- Figure 2-16 Proposed culvert alignments (lower zone)
- Figure 2-17 Impacted buildings with 200yr afflux level map (Mitigation scenario) as background
- Figure 2-18 Residual impacts outside the designation (upper zone) but does not impact any property buildings (200 year ARI afflux level)
- Figure 2-19 Residual impacts outside the designation (central zone) but does not impact any property buildings (200 year ARI afflux level)
- Figure 2-20 Residual impacts outside the designation (lower zone) but does not impact any property buildings (200 year ARI afflux level)
- Figure 2-21 Taranaki Stream 100-year Flood Duration Assessment
- Figure 2-22 Taranaki Stream 200-year Flood Duration Assessment
- Figure 2-23 Bathtub 1 extent (Downstream of Taranaki Stream)
- Figure 2-24 Bathtub 2 extent (Downstream of Taranaki Stream)
- Figure 2-25 Bathtubs downstream of Taranaki Stream
- Figure 2-26 Flood Extents results from the Volumetric Assessment (Brown– Pre-development Scenario; Green – Mitigation Scenario)
- Figure 2-27 Flood Extents results from the Volumetric Assessment (Brown– Pre-development Scenario; Green – Mitigation Scenario)
- Figure 2-28 Waihora Creek to Gladstone Road 100-year Flood Duration Assessment
- Figure 2-29 Waihora Creek to Gladstone Road 200-year Flood Duration Assessment
- Figure 2-30 Woodend Beach Road 100-year Flood Duration Assessment
- Figure 2-31 Woodend Beach Road 200-year Flood Duration Assessment
- Figure 2-32 Bathtub Assessment using the WDC Flood Model Results
- Figure 2-33 Woodend Beach and Stalkers Road Flood Levels
- Figure 2-34 McIntosh Drain 100-year Flood Duration Assessment
- Figure 2-35 McIntosh Drain 200-year Flood Duration Assessment
- Appendix A Figure 1 B2P Flood Model catchment delineation
- Appendix A Figure 2 B2P Flood Model inflow and downstream boundaries
- Appendix A Figure 3 Boundary Conditions with Equivalent Inflow hydrographs
- Appendix A Figure 4 Boundary Conditions with Equivalent Stage hydrographs
- Appendix A Figure 5 Surface roughness applied in B2P Flood Models (Existing Scenario)
- Appendix A Figure 6 Surface roughness applied in B2P Flood Models (Proposed Scenario)

NOT FOR CONSTRUCTION

Appendix A Figure 7 Surface roughness table extracted from 'Flood Hazard Models Update' report
Appendix A Figure 8 Existing culverts within the proposed project site
Appendix B Figure 1 Model Setup for the pre-development scenario
Appendix B Figure 2 Model Setup for the post-development scenario
Appendix B Figure 3 Model Setup for the post-development with mitigation scenario
Appendix B Figure 1 Model Setup for the pre-development scenario
Appendix B Figure 2 Model Setup for the post-development scenario
Appendix B Figure 3 Model Setup for the post-development with mitigation scenario

Tables

Table 2-1 Model scenarios and purpose
Table 2-2 Locations where large discrepancies are identified
Table 2-3 100-year and 200-year ARI Flood Levels and Differences (Post-Development vs. Pre-Development Scenario)
Table 2-4 100-year and 200-year ARI flood level and differences (Mitigation vs Pre-development scenario)
Table 2-5 Locations with Flood Level Differences
Table 2-6 Ravenswood Basin Weir Level Change and Modelled Flood Level Change
Table 2-7 Properties Freeboard Changes
Table 2-8 Total Volumetric Water Difference - Waikuku Beach
Table 2-9 Total Volumetric Water Difference – Woodend Beach Road and Stalkers Road
Appendix A Table 1 Comparison of WDC South Ashley Model and B2P Flood Model build
Appendix A Table 2 Water Level Downstream Boundary Conditions
Appendix A Table 3 Normal Depth Downstream Boundary Conditions
Appendix A Table 4 Horton decay rate extracted from WWDG
Appendix A Table 5 Modelled Culvert Inventory
Appendix A Table 6 List of modelled scenarios

Copyright © 2025 Aurecon New Zealand and its affiliates. "AURECON", "BRINGING IDEAS TO LIFE" and the Aurecon logos and devices are trade marks and/or registered trade marks of Aurecon Group Brand Pty Ltd and its affiliates. All rights reserved.

Disclaimer

This report (including any enclosures, data and attachments) has been prepared for the exclusive use and benefit of our client, solely for the purpose for which it is provided, and on the terms and conditions agreed with our client. Unless we provide express prior written consent, no part of this report should be reproduced, distributed or communicated to any third party. To the extent permissible by law, we do not accept any liability if this report is used or relied on by any unauthorised third party, or by the client for any unauthorised purpose, in any other contexts or without checking it is the latest revision.

1 Introduction and Background

1.1 Background

Waka Kotahi has engaged Aurecon to support the resource consent applications in the proposed Belfast to Pegasus Motorway and Woodend Bypass. In line with this project, a flood assessment has been conducted to evaluate the impact of a 100-year and 200-year Average Recurrence Interval (ARI) storm event on the proposed road development.

For the purposes of the current assessment and to optimise the computation time, a cut-down catchment model tailored to the Project Site and adjacent surroundings as shown in Figure 1-1 has been built using TUFLOW modelling software. The model will be referred to as the “B2P Flood Model” throughout the rest of this report.

The purpose of this report is to outline the following:

- The methodology for building the B2P Flood Model
- A comparison of flood impacts before and after the road development



Figure 1-1 B2P Flood Model Extent with the Proposed Belfast to Pegasus Motorway and Woodend Bypass Road Alignment

1.2 Objectives and Scope of Assessment

The objectives of the current assessment include developing the B2P Flood Model to analyse flood impacts before and after development.

Based on local authority advice, the following objectives have been established for this project:

- No significant increase in flood levels at neighbouring dwellings for the 100-year ARI event.
- Any increase in flood levels at neighbouring dwellings for the 200-year ARI event should be limited to 20 mm.

- Reduce the risk of ponding/runoff buildup that will be brought about by the barrier effect of the proposed road development on adjacent surroundings. Mitigation measures will consist of provision of additional culverts across the road alignment, improvement/rehabilitation of existing culvert structures, and provision of additional channels to improve flow of runoff.

1.3 Flood Model History

Figure 1-2 shows the model extent of the Waimakariri District Council (WDC) South Ashley Flood Model. This flood model was developed in 2019 by DHI using MIKE by DHI modelling software. Detailed information with regard to model build is documented in the May 2020 “Flood Hazard Models Update” report¹.

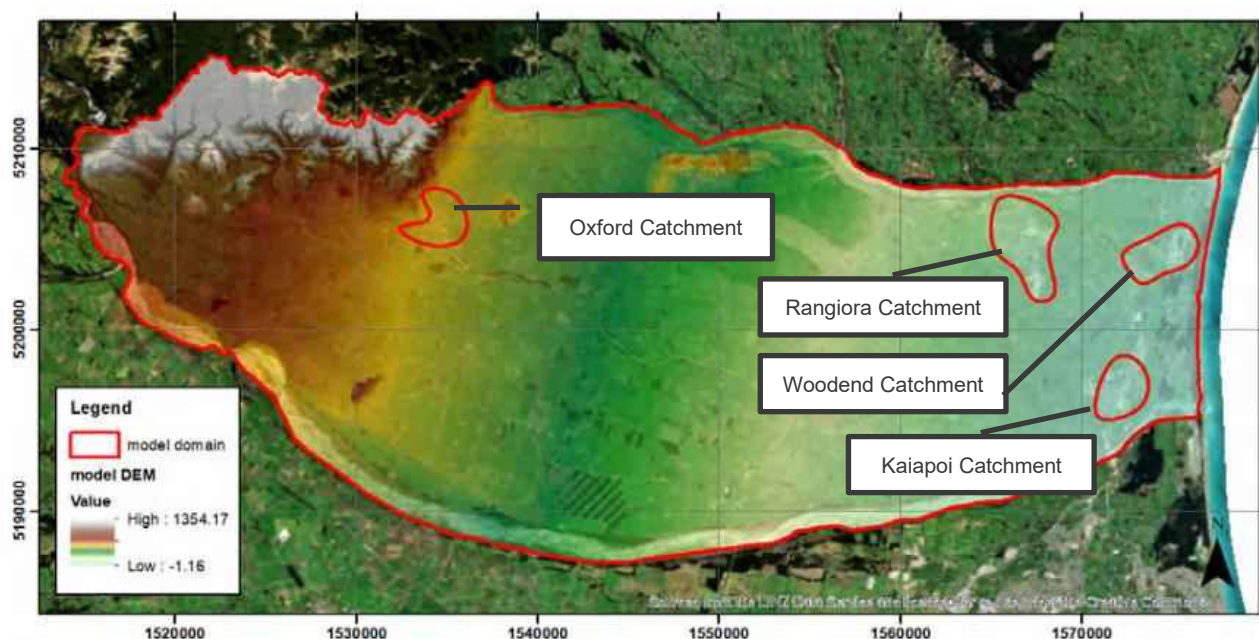


Figure 1-2 South Ashley Model catchment extent²

In this study, the B2P Flood Model has been created for the Project Site by utilising TUFLOW modelling software (Figure 1-3), using the WDC South Ashley Model as a reference.

¹ DHI (May 2020). *Flood Hazard Models Update*.

² Extracted from Flood Hazard Models Update Report.

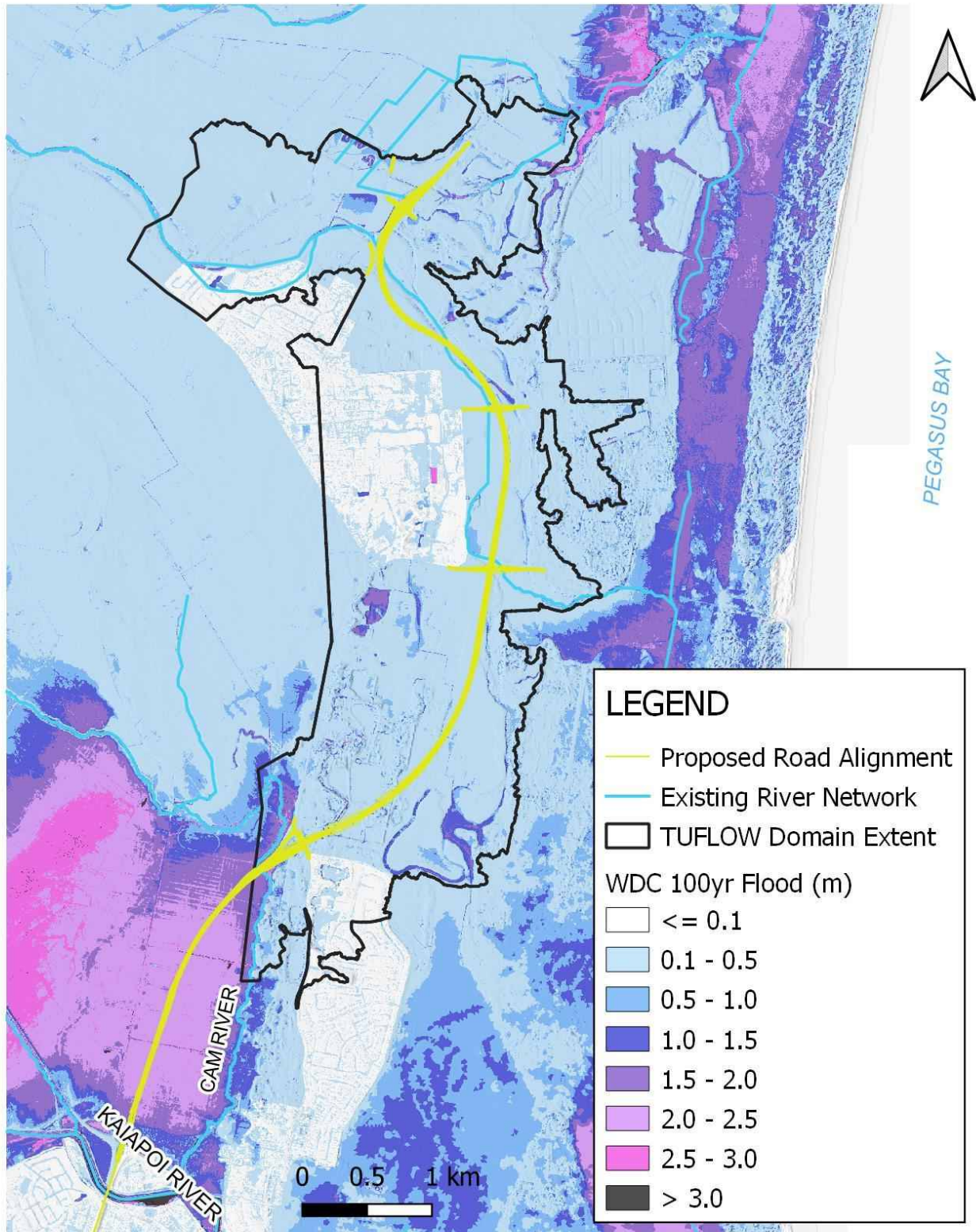


Figure 1-3 Model Extent of B2P - Woodend Flood Model

2 B2P Flood Model Development

The B2P Flood Model extent is a cut-down catchment model tailored to the Project Site and has been built using TUFLOW modelling software. Three modelling scenarios, the Pre-development, Post-development, and Mitigation scenarios were developed and assessed.

The purpose of each scenario is listed in Table 2-1. Further details of these scenarios are discussed in Appendix A. Each scenario has been run for the 100-year and 200-year ARI storm events.

Table 2-1 Model scenarios and purpose

Model Scenario	Scenario Case	Purpose
PREDEV	Existing/pre-development scenario	To understand the existing flooding conditions. This will be the baseline model for the flood impact assessment.
POSTDEV	Design/post-development scenario with Bypass Motorway included	To understand the potential flood impacts due to Bypass Motorway if no mitigation measure are undertaken.
MITIG	Design/post-development scenario with the Bypass Motorway and mitigation solutions included	To resolve flood impacts identified under the design scenario.

The B2P Flood Model terrain has been updated to incorporate the 2020 LiDAR Digital Elevation Model (DEM) and available surveyed data. The hydrological and hydraulic input data such as rainfall, ground infiltration, and surface roughness are extracted from the WDC South Ashley Model. The detailed model build methodology is described in Appendix A.

Note that for vertical datum, as described in Appendix A, the original flood model was carried out in Lyttleton Vertical Datum (LVD), whereas the B2P Flood Model is in New Zealand Vertical Datum 2016 (NZVD16). Levels within this memorandum are quoted in NZVD16 unless otherwise stated.

The B2P Flood Model is mainly built as a two-dimensional (2D) model, integrating open channels into the DEM, with culverts represented using a one-dimensional (1D) model. Pipe reticulation has been excluded from the B2P Flood Model, in compliance with the WDC requirements regarding the modelling of full pipe blockages.

In the B2P Flood Model, three principal river channels have been identified, as shown in Figure 2-1: Taranaki Stream, located to the north of Bob Robertson Drive; Waihora Creek, situated near Garlick Street; and McIntosh Drain, which lies between Quarry Lake and Woodend Beach Road. Taranaki Stream flows northward towards Waikuku Beach, while the Waihora Creek initially flows south, then turns east towards Kairaki Creek, close to Woodend Beach Road. McIntosh Drain also flows south and ultimately merges with the Kaiapoi River.

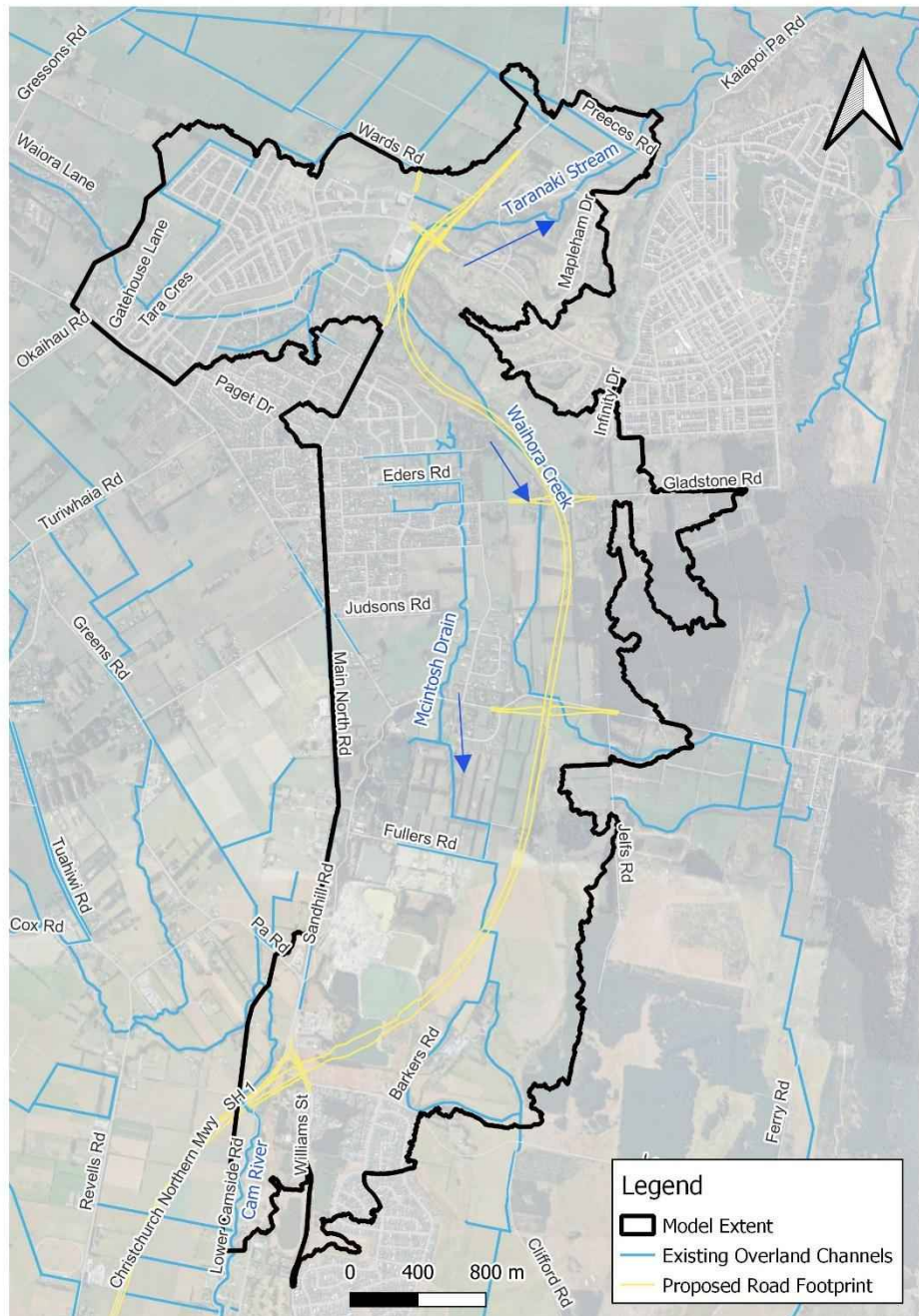


Figure 2-1 Principal River channels within the model catchment extent

A detailed description of the model development methodology is provided in Appendix A, while the updates representing various post-development and mitigation scenarios can be found in Appendix B.

2.1 B2P Pre-Development Model

The detailed model build methodology for B2P Flood Model pre-development scenario (PREDEV) is described in Appendix A Table 1.

Figure C-1 and Figure C-2 (refer to Appendix C) show the maximum flood depth and flood level of the pre-development scenario under the 100-year ARI storm event, Figures C-3 and C-4 present the maximum flood depth and flood level under the 200-year ARI storm event.

The pre-development model comprises of six existing culverts, that are crossing the project site as shown in Figure 2-2. This includes an irregular twin DN1800 culvert³ on Bob Robertson Drive, a 2.2m x 1.2m box culvert³, and a DN750⁴ circular culvert that cross State Highway 1 (SH1) discharging into the Taranaki Stream, a 4m x 1.5m box culvert³ crossing SH1 that discharges into Waihora Creek, and a 1.25m x 0.5m box culvert located on Gladstone Road and a DN300 circular pipe situated on Woodend Beach Road.

Survey shows that the existing irregular twin DN1800 was embedded by 300mm as part of the fish passage consideration, which will be maintained in the Post-Development and Mitigation scenarios. Ecological assessments have also found that fish are present along the major streams that are affected by the proposed roadway, namely McIntosh Drain, Taranaki, and Waihora Streams. Accessibility for fish movement will be preserved along these streams and will be considered in the proposed mitigation measures.

The Stormwater Network and Culverts inventory, as discussed in Appendix A, details all the culverts that have integrated the passage of fish, in all three case scenarios. Only the effective flow opening area of these culverts have been modelled, with higher surface roughness to incorporate the substrate infill within the culvert.

³ To simplify the modelling process, culverts with fish passage considerations are modelled using only their effective flow opening area.

⁴ Existing 0.85 (W) x 0.65 (H) arch culvert. For modelling simplicity, culvert modelled as DN750 circular culvert.

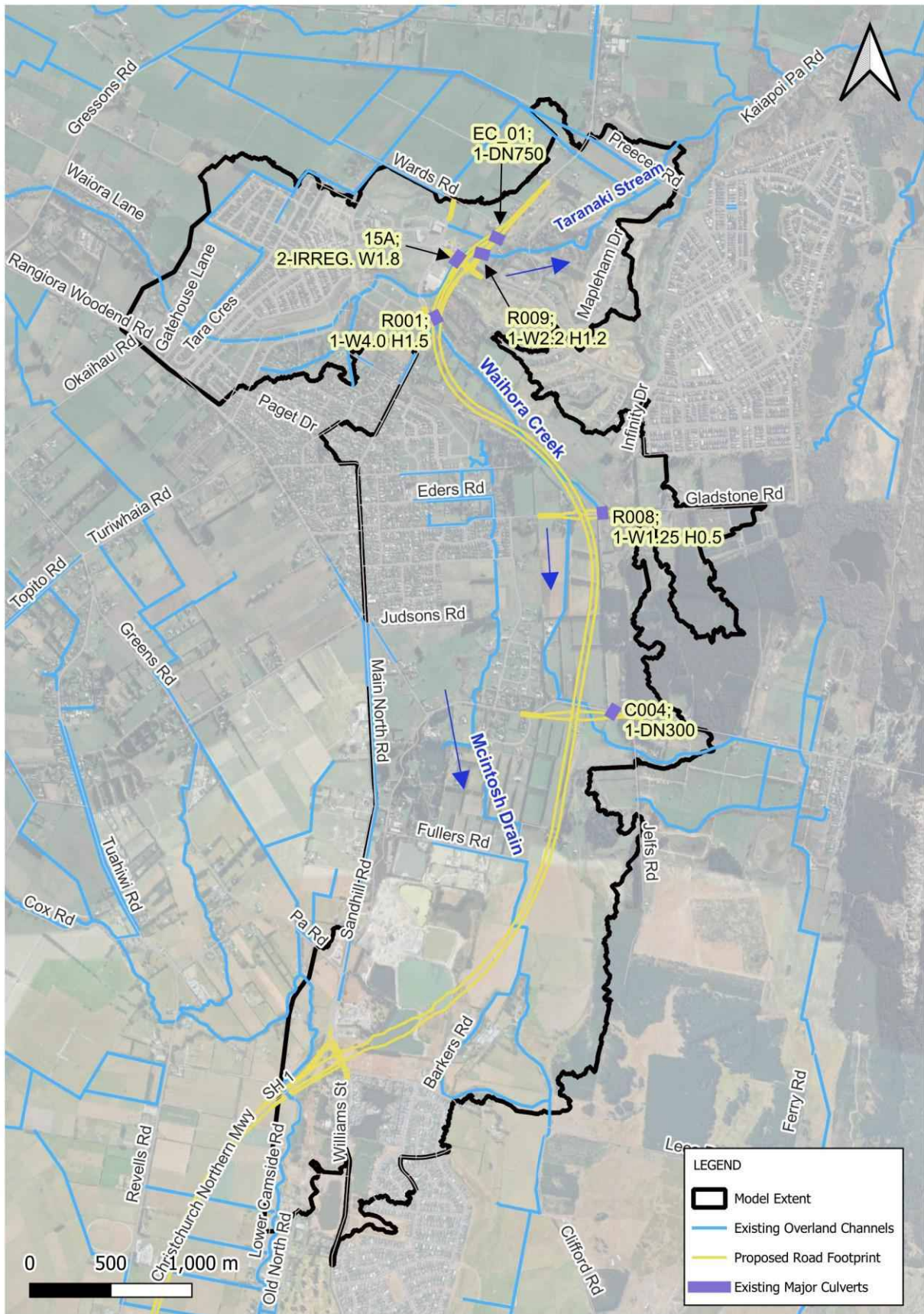


Figure 2-2 Existing culverts crossing proposed road alignment

As shown in Figure 2-3, the runoff splits into three directions:

- The Taranaki Stream catchment flow northward towards Waikuku Beach
- The Waihora Creek catchment flow towards the south and turns east towards the Kairaki Creek
- The McIntosh Drain flow south and ultimately merges with the Kaiapoi River.

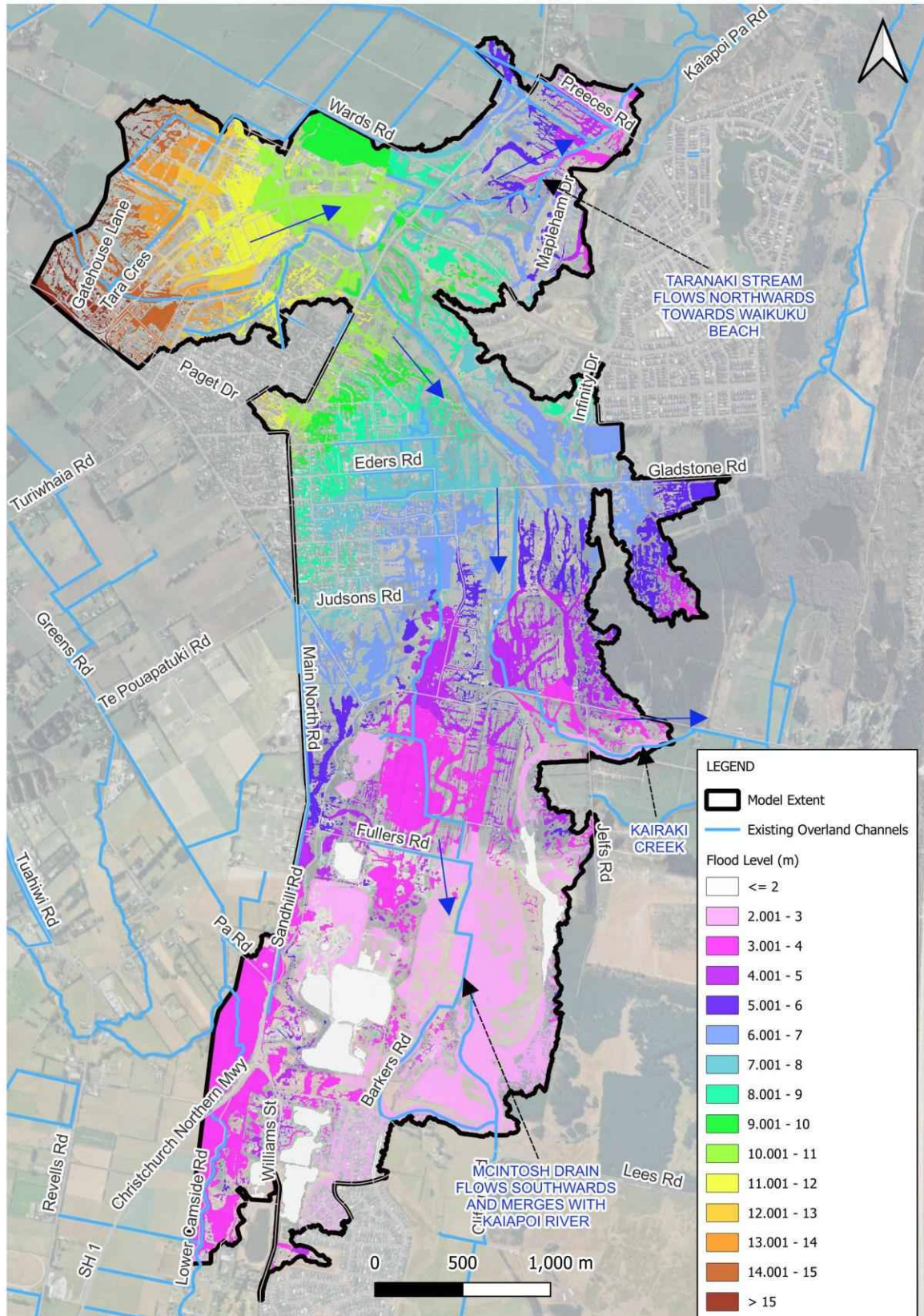


Figure 2-3 B2P Flood Model pre-development flood level under 100-year ARI

2.1.1 Flood Model Validation

Figure 2-4 shows the maximum predicted flood depths for the Existing Scenario in the WDC South Ashley Model for the 100-year ARI event. In comparison with the maximum predicted flood depths of the B2P Flood Model (refer to Figure 2-5), it can be observed that the flood extents are relatively similar, except for a few localised areas which are discussed in Table 2-2.

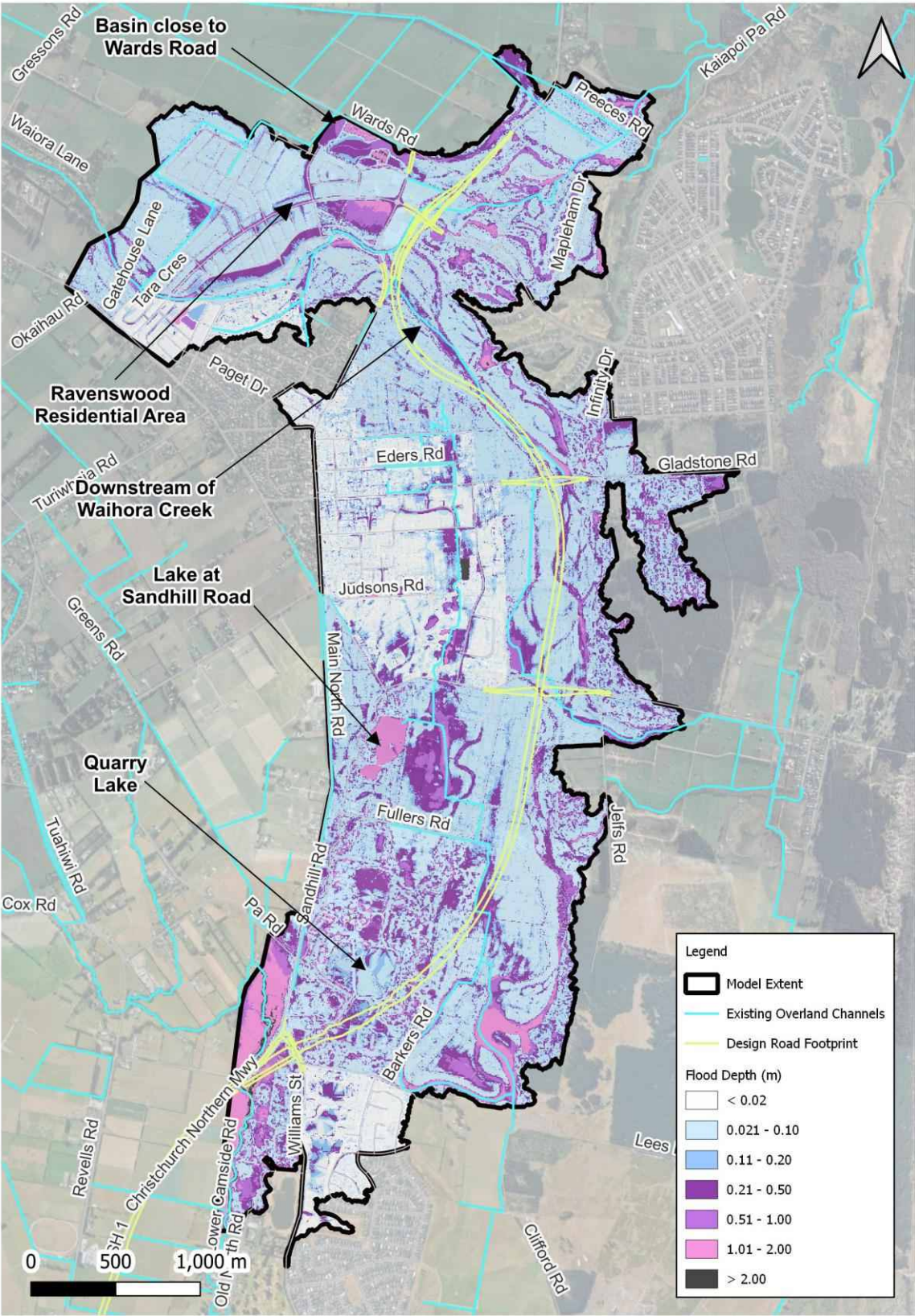


Figure 2-4 WDC South Ashley Model pre-development flood depth for 100-year ARI event

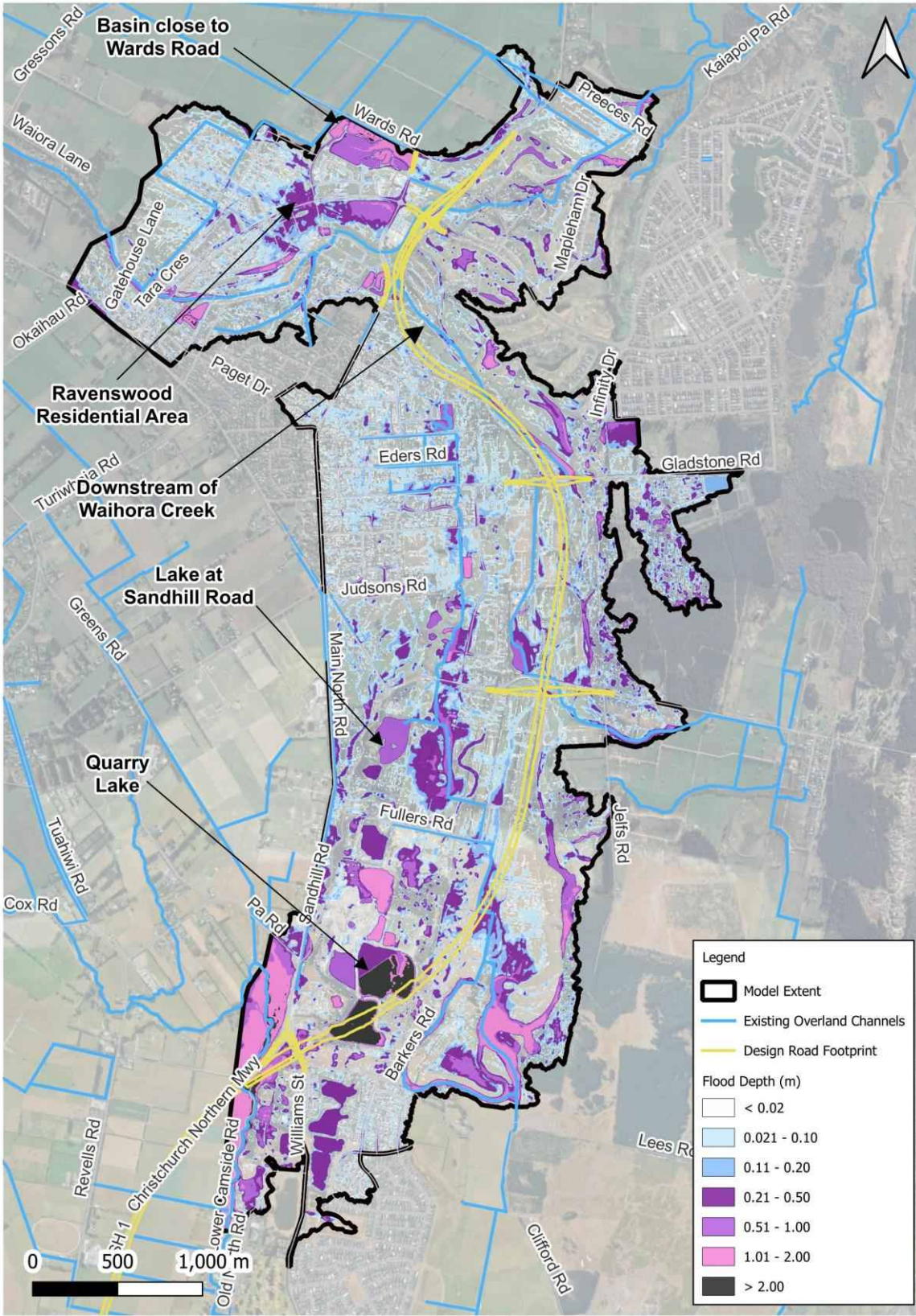


Figure 2-5 B2P-Woodend flood model pre-development flood depth for the 100-year ARI event

Figure 2-6 illustrates the differences in flood depth between the B2P Pre-development flood model and the localised flood depth map from the WDC South Ashley Model, specifically for the 100-year ARI event. Areas shaded in cooler colours indicate regions where the flood depth is greater in the WDC South Ashley Model compared to the B2P Pre-development flood model, while warmer colours show the opposite.

Figure 2-7 shows the differences in ground level between the B2P Pre-development flood model and the localised flood depth map from the WDC South Ashley Model. Areas marked in blue indicate regions where the ground level is higher in the WDC South Ashley Model than in the B2P Pre-development flood model, and vice versa.

Overall, the discrepancies in results can be attributed to several factors. These include the coarser cell sizes utilised in the WDC South Ashley Model, large cross-sectional areas of open channels burnt in the road to represent cross culverts in that model, and recent ground level updates in newly developed areas. The locations where significant differences have been identified are summarised in Table 2-2.

Table 2-2 Locations where large discrepancies are identified

Locations	Reasons
Ravenswood residential area (B2P Flood Model > WDC South Ashley Model)	As shown in Figure 2-6 and Figure 2-7, discrepancies were observed in the Ravenswood residential area since the 2020 LiDAR DEM did not reflect its full development, which resulted to higher flood depths in the B2P Flood model, due to its original land form. On the other hand, the WDC South Ashley model already integrated the current development (only west side of the development area) with higher topographic levels which consequently yielded lower flood depths. These discrepancies produced a positive afflux between the B2P Flood Model and the WDC South Ashley Model.
Basin near to Wards Road (B2P Flood Model > WDC South Ashley Model)	As shown in Figure 2-7, the basin was not included in the WDC South Ashley Model but has been incorporated into the B2P Flood Model. This inclusion has resulted in increased flood depths in the affected areas.
Downstream of Waihora Creek (WDC South Ashley Model > B2P Flood Model)	Figure 2-8 shows a 40m wide trapezoidal cross-section was used in the WDC South Ashley Model to represent the existing 4m x 2m box culvert. This design facilitates a higher runoff flow downstream, leading to greater flood depths in those areas when compared to the B2P Flood Model. This has been replaced by a 1D network in the B2P Flood Model.
Lake at Sandhill Road (WDC South Ashley Model > B2P Flood Model)	As shown in Figure 2-7, variations in flood depths may have resulted from the DEM which have captures a higher water levels during the LiDAR data collection in the WDC South Ashley Model compared to the B2P Flood Model.
Quarry Lake (B2P Flood Model > WDC South Ashley Model)	As shown in Figure 2-7, variations in flood depth resulted from the ground levels, which have captured a higher water level in the B2P Flood Model compared to the WDC South Ashley Model.

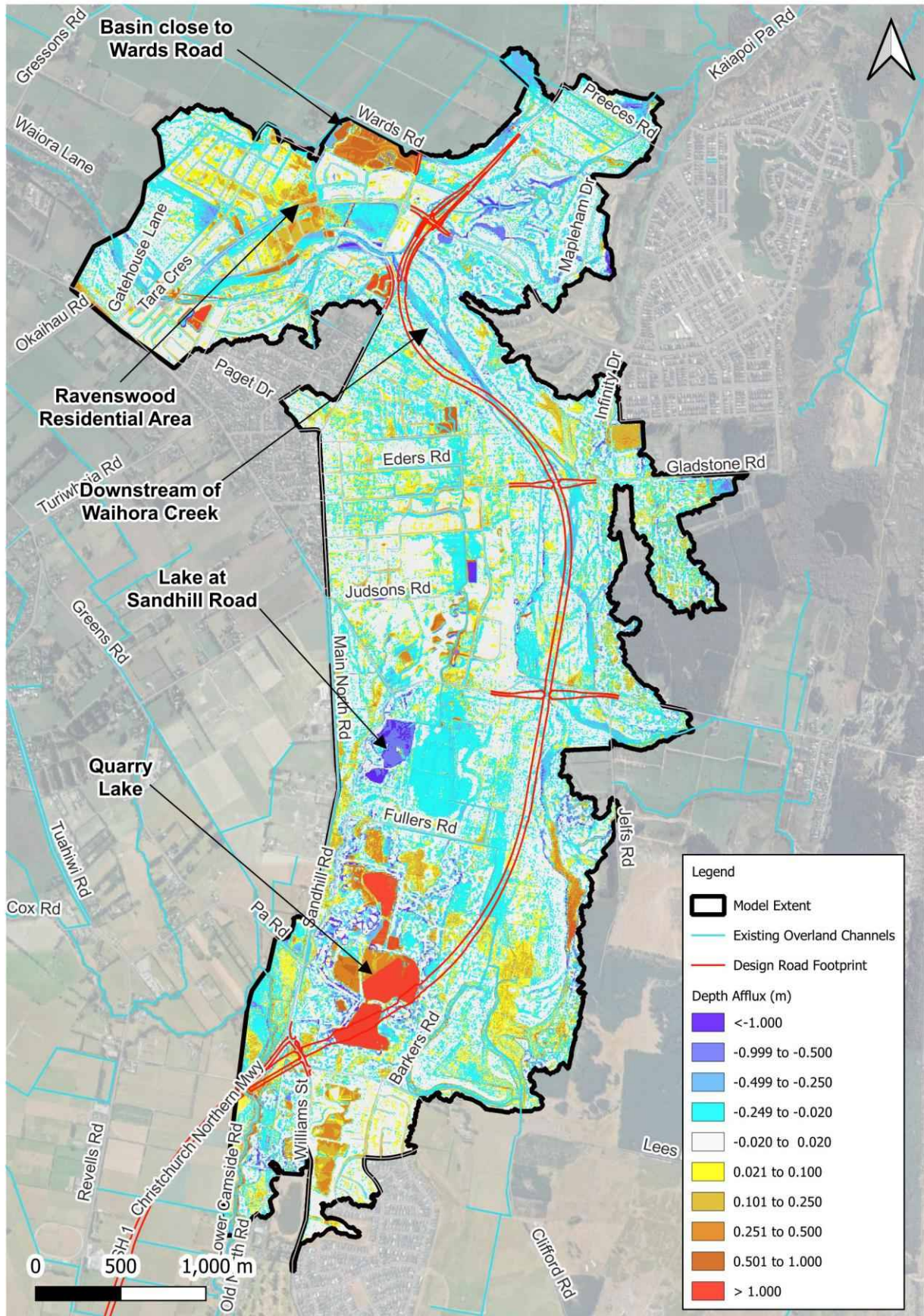


Figure 2-6 B2P Flood Pre-development model 100year ARI flood depth minus WDC South Ashley Model localised flood depth map under 100-year ARI event

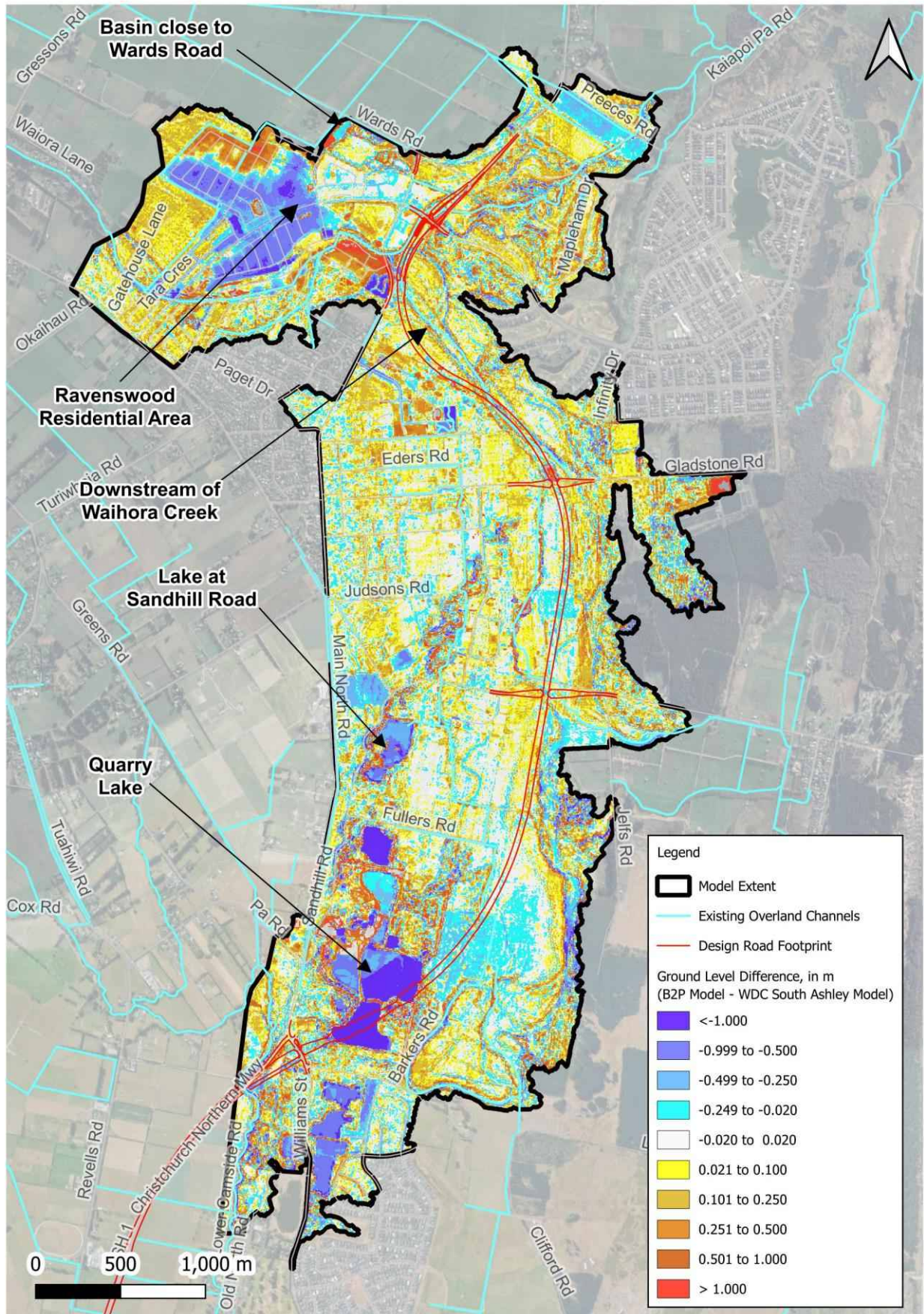


Figure 2-7 B2P Flood Model ground level minus WDC South Ashley Model ground level

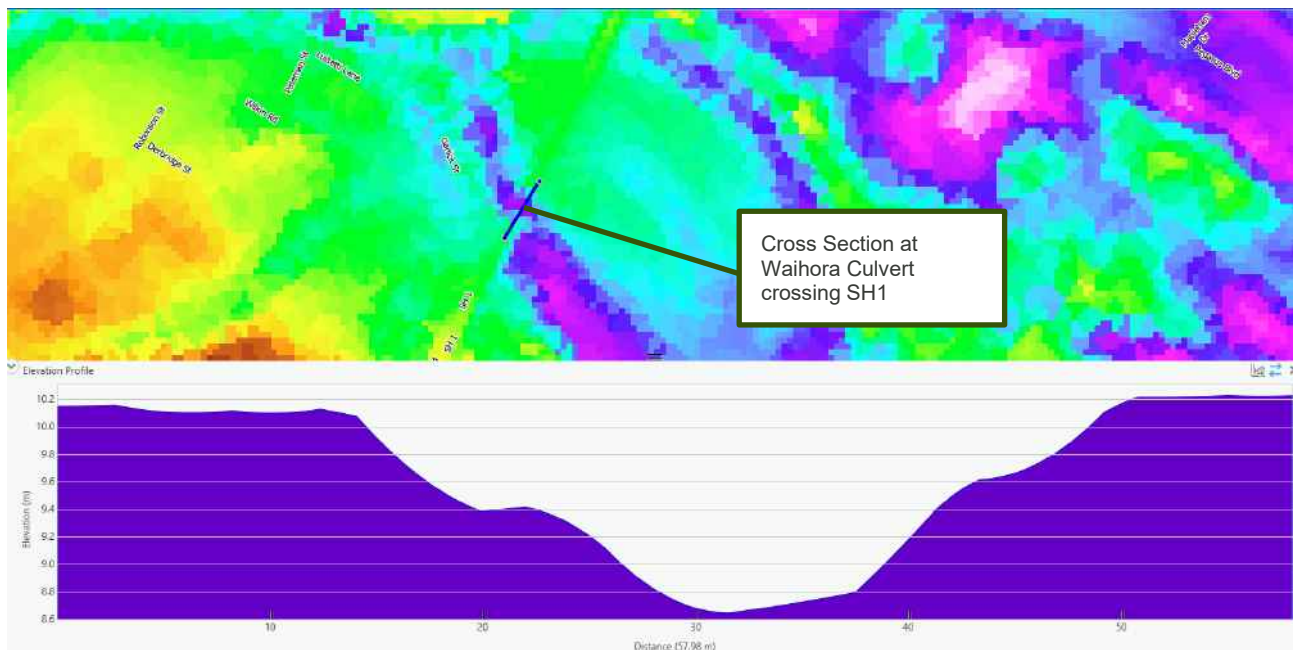


Figure 2-8 WDC Model Ground Level representing an existing 4m x 2m box culvert

2.2 B2P Post-Development Flood Model

The detailed model build methodology for B2P Flood Model post-development scenario (POSTDEV) is described in Appendix A Table 6.

Figure 2-9 illustrates the proposed design surface level for the entire road alignment. The proposed alignment generally follows the existing ground levels, with raised embankments incorporated along its length. However, the construction of these embankments alters the natural flow path, leading to the potential for water pooling behind the embankments due to the disruption of drainage. No mitigation measures have been developed in this model since the goal is to determine the effects of the bypass road and its comparison with the Pre-Development Model.

Existing culverts that fell within the designation boundary were removed, while culverts that are outside were maintained. The only culvert that was modelled with a fish passage consideration was the DN1800 culvert that crosses Bob Robertson drive, similar to the Pre-Development Model.

Comparable to the baseline flood model, the post-development flood model was developed under the assumption that any stormwater pipes are completely blocked.

Figure C-5 and Figure C-6 (refer to Appendix C) show the maximum flood depth and flood level of the post-development scenario under the 100-year ARI storm event, while Figure C-7 and Figure C-8 present the maximum flood depth and flood level under the 200-year ARI storm event.

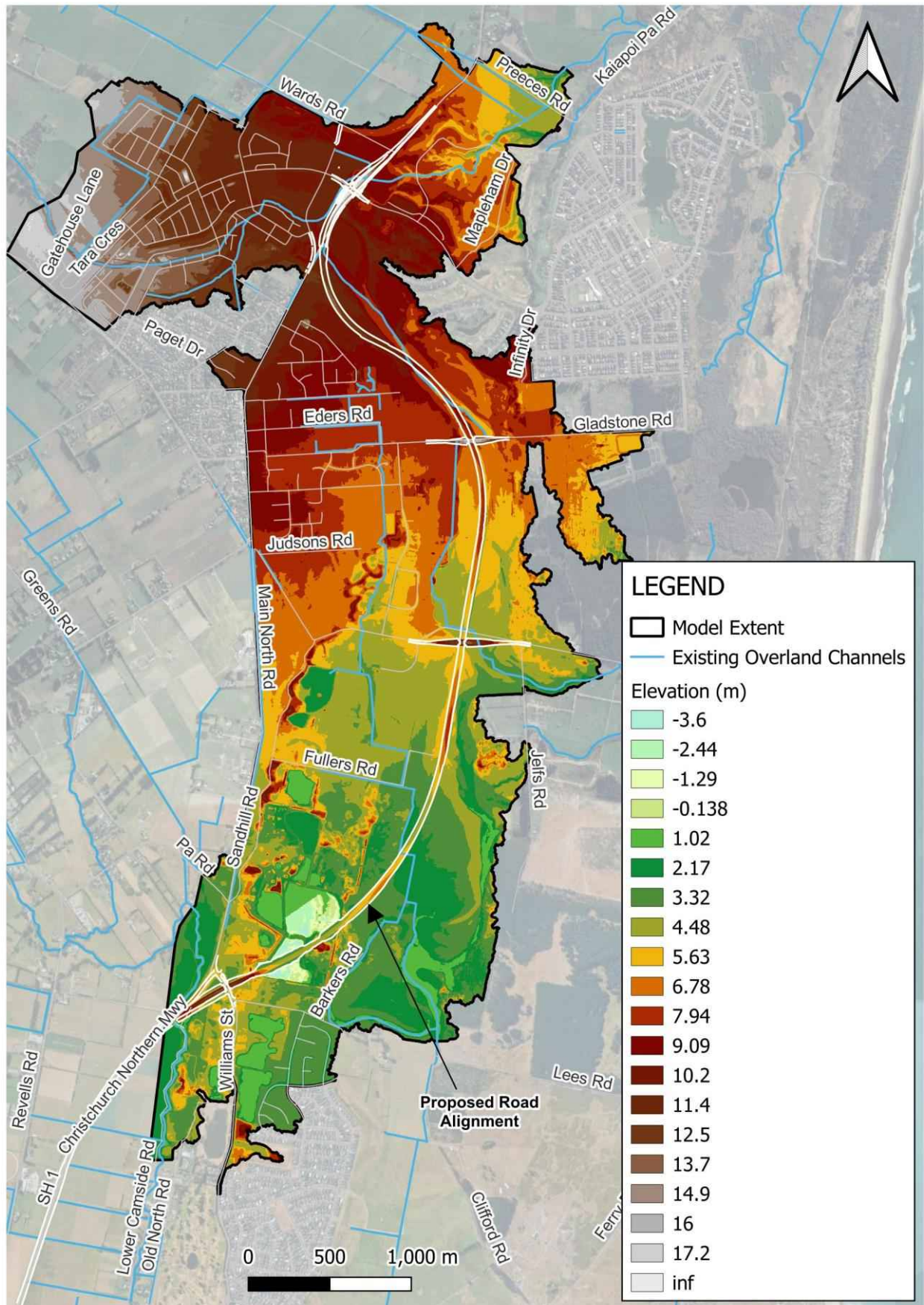


Figure 2-9 Proposed B2P-Woodend design surface level

2.2.1 Post-Development Scenario Result Discussion

Maximum flood level difference maps were developed to compare the post-development scenario with the pre-development scenario. These are presented in Figures C-9 and C-10 (see Appendix C) for the 100-year and 200-year ARI events.

Based on the Figures C-9, C-10, the proposed state highway remains dry under both the 100-year and 200-year ARI scenarios, with the exception of the road section north of Bob Robertson Drive, where floodwaters overtop the road as shown in Figure 2-10. The road overtopping is primarily due to a low existing road level and undersized existing culverts. To address this issue, the 2.2m x 1.2m culverts at Taranaki stream and DN750 at north of Taranaki stream are proposed to be upsized. This will allow more runoff to be conveyed downstream through culverts instead of overtopping the road.

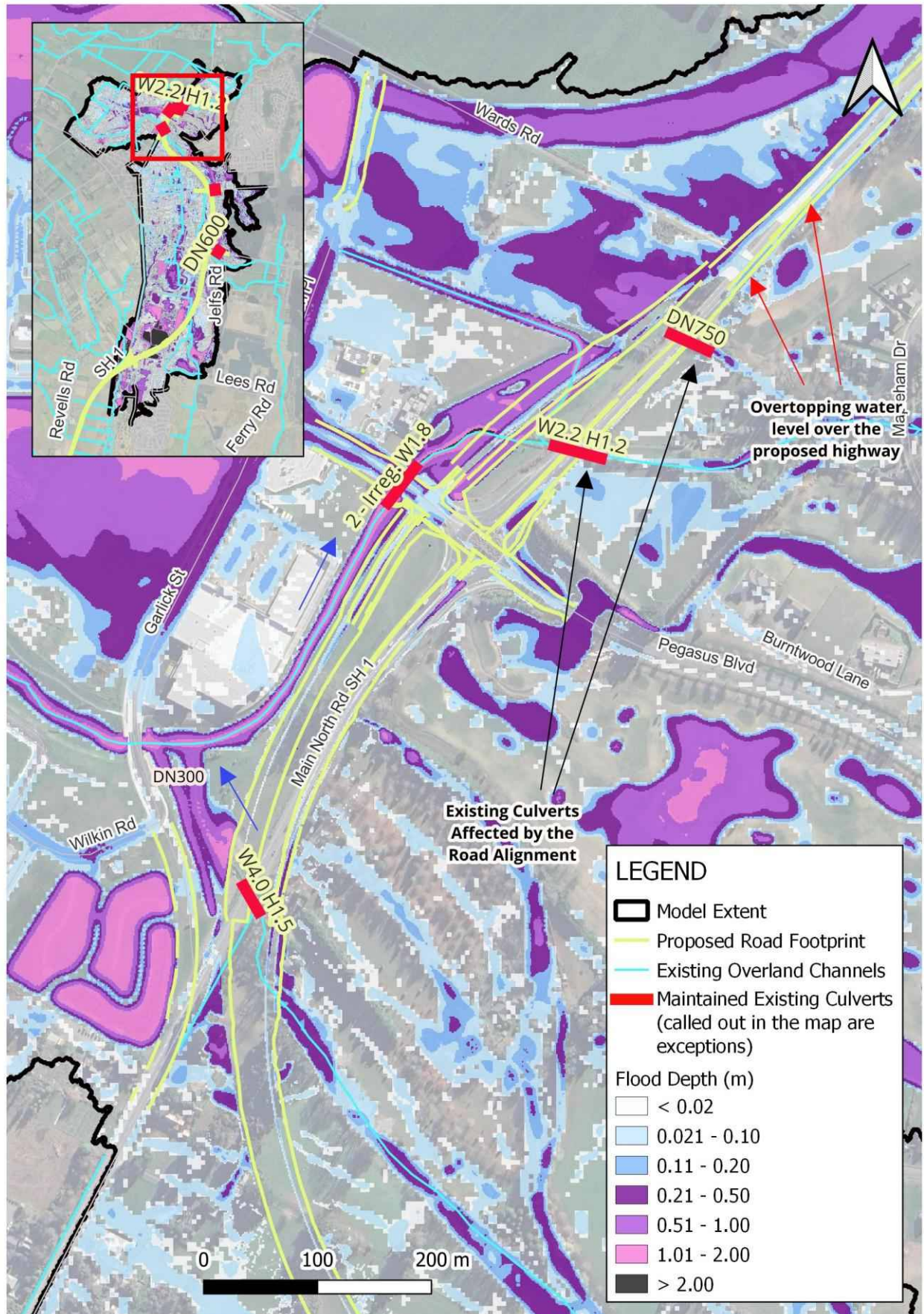


Figure 2-10 Floodwaters overtop the road at north of Bob Robertson Drive (200 year ARI Post-Development Scenario)

Figures C-9 and Figures C-10 indicate that most flooded zones are confined adjacent to the proposed road corridor. However, elevated flood levels are observed in the residential area along Main North Road, Woodend Beach Road and Wards Road during both the 100-year and 200-year ARI events.

The increase in flood level is attributed to the impact of road embankment towards the natural flow of runoff, such that existing channels are blocked by the road embankment that eventually contributed to the runoff buildup. Flood depth, flood levels, and their differences at the eight impacted properties (as shown in Figure 2-11 are presented in Table -3).

The following tables present the findings in terms of flood level and their difference between the post-development and pre-development cases at varying ARIs (100 and 200-year):

Table 2-3 100-year and 200-year ARI Flood Levels and Differences (Post-Development vs. Pre-Development Scenario)

Point ID	Location	100-year ARI			200-year ARI		
		Pre-development flood level (mRL)	Post-development flood level (mRL)	Difference (mm)	Pre-development flood level (mRL)	Post-development flood level (mRL)	Difference (mm)
1	5 Wards Rd*	8.28	8.42	140	8.37	8.5	137
2	196 Woodend Beach Rd	4.13	4.33	193	4.23	4.57	335
3	1271 Main North Rd*	8.11	8.29	183	8.16	8.33	171
4	1263 Main North Rd	8.28	8.31	30	8.29	8.35	58
5	1279 Main North Rd*	8.01	8.13	120	8.03	8.15	114
6	1273 Main North Rd*	8.10	8.24	140	8.15	8.28	130
7	1277 Main North Rd*	8.02	8.18	160	8.07	8.21	142
8	1319 Main North Road	6.41	6.44	27	6.46	6.55	91

*Property will be acquired as part of project work delineation. Flood impacts within designation have been considered acceptable

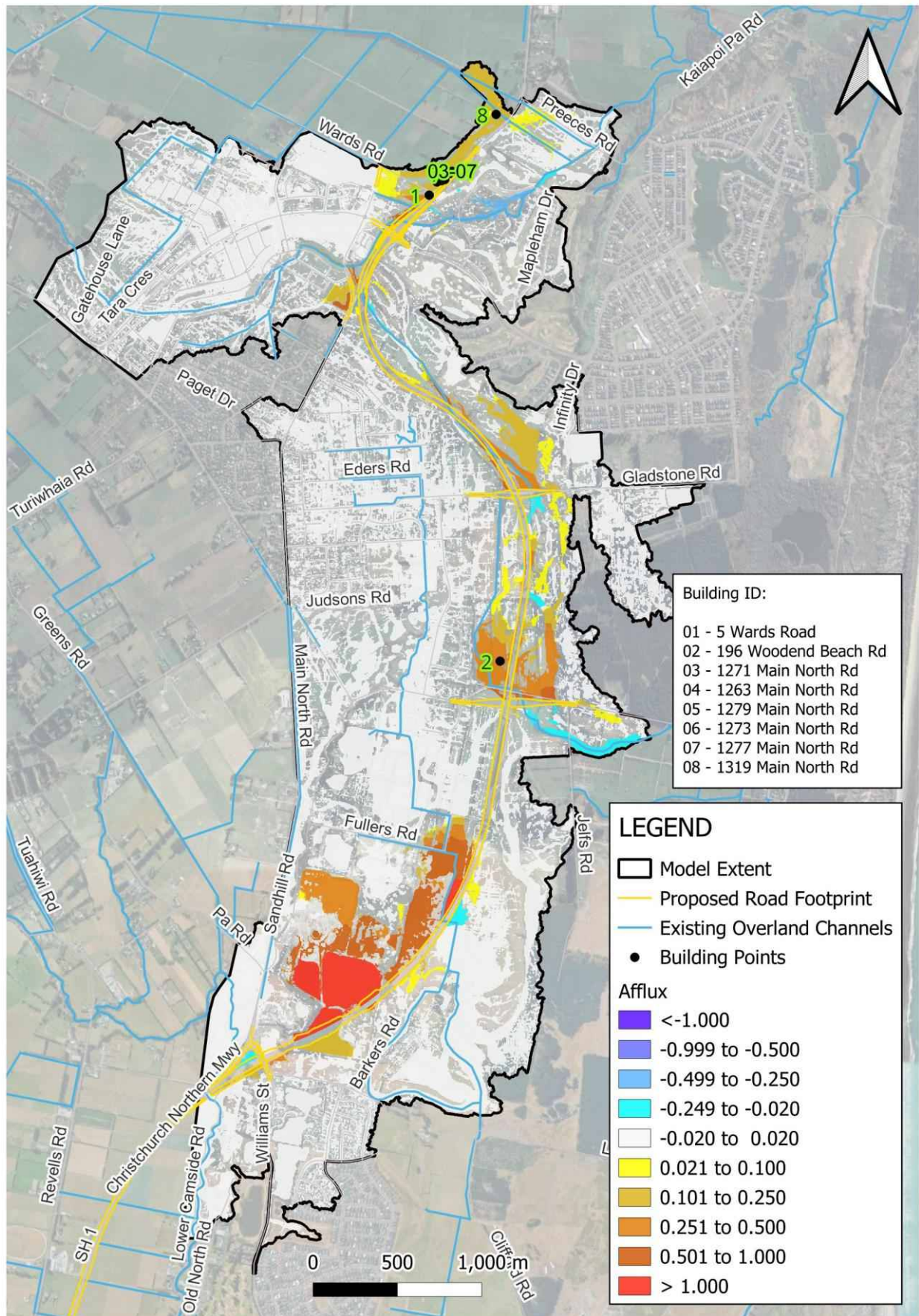


Figure 2-11 Impacted properties with 200yr afflux level map (post-development scenario) as background

These findings indicate that the increases exceeded the project objectives, which require that no flood level increase affecting neighbouring properties under the 100-year ARI event, and that any increase for the 200-year ARI event should not exceed 20 mm.

Of the eight impacted properties, five will be acquired as part of the project work delineation. Flood impacts within the designated area have been considered acceptable, if the flooding does not overtop the road.

Given the increase in flooding at the above three locations, flood mitigation measures are necessary to reduce impacts on these zones. The following solutions will be considered during detailed design:

- Provision of adequate culverts across the design road alignment.
- Divert impounded runoff through additional channel alignments.

2.3 B2P Mitigation Flood Model

The methodology for the B2P Flood Model post-development scenario with mitigation solution is outlined in Appendix A Table 6. The key components of the mitigation solution include:

- Provision of Culvert Crossings:
 - Culvert structures have been added along critical areas beside the road alignment to avoid runoff build-up and ponding issues. These structures are incorporated in the flood model to properly represent the mitigation solution once the road embankment is constructed.
 - Culverts have been added to connect the proposed treatment swales and cut-off drains to the designated discharge points across the proposed motorway.
 - Culverts sizing at Taranaki Stream, Waihora Creek and McIntosh Drain include the 300mm embedment depth to accommodate fish passage. Only the flowable portions of the culvert cross-sectional area are represented in the flood model, and the average channel roughness has also been accounted for.
 - The design invert levels for the culvert structures are based on the provided survey data, for best stream channel tie in at upstream and downstream end and achieve a minimum adequate slope to convey runoff without overflowing at the road level.
 - Where road widening is proposed at Bob Robertson Drive, existing culverts are to be extended or replaced with like-to-like sizes to avoid introducing flood impacts downstream.
- Provision of flow path Diversion Channels (DIV. CHAN.):
 - Additional channels are included for the model to ensure smooth flow behaviour of impounded runoff from upstream to downstream.
 - The design slopes of the channels are established to match the natural topography and to permit the flow of runoff, preventing any impounding upstream.
 - Channels are designed to maintain and replicate original overland flow paths and permanent streams that have been affected by the development.
- Provision for Treatment Swales (TS), and Cut-off Drains (CD)
 - Treatment swales are designed to collect and treat stormwater runoff generated from the road corridor while conveying it towards designated discharge locations. Bio-retention and bio-infiltration basins are also proposed to provide treatment of runoff adjacent to the proposed road before discharging to the receiving environment.
 - Cut-off Drains are proposed to intercept water from external overland catchments and convey it to the designated discharge points, best maintaining hydraulic neutrality and avoiding inter-catchment transfer.

- Both systems are designed to direct runoff to the discharge locations such as permanent streams and overland flow paths.
- Treatment swale, treatment basins and cut off drains are represented in the flood model through the design surface model.
- Raising the weir level at Garlick Street to 10.40 m to utilise the existing basin volume:
 - Raising the weir level at Garlick Street to 10.40 m increases the storage capacity of the existing basin, allowing it to hold a greater volume of runoff. This enhancement helps to reduce the amount of runoff discharging downstream, thereby managing flood risks more effectively.

Figure 2-12, Figure 2-13, Figure 2-14, Figure 2-15, and Figure 2-16 show the locations where culverts and designed waterways are proposed as part of the mitigation solutions. Some major flow changes due to proposed mitigating structures are shown in the following figures. Blue arrowheads represent the flow direction of runoff.

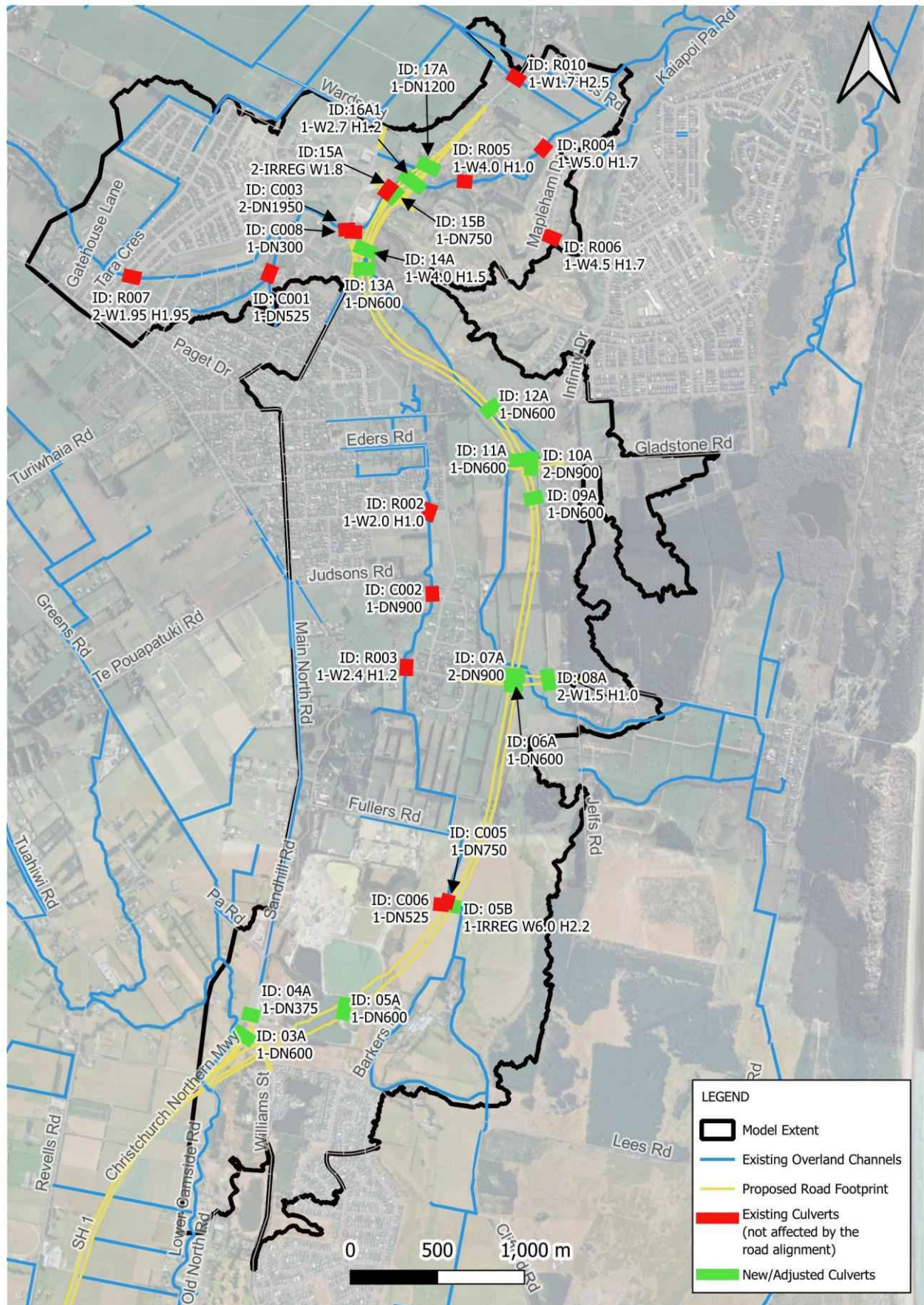


Figure 2-12 Proposed mitigation scheme adjacent to the road alignment

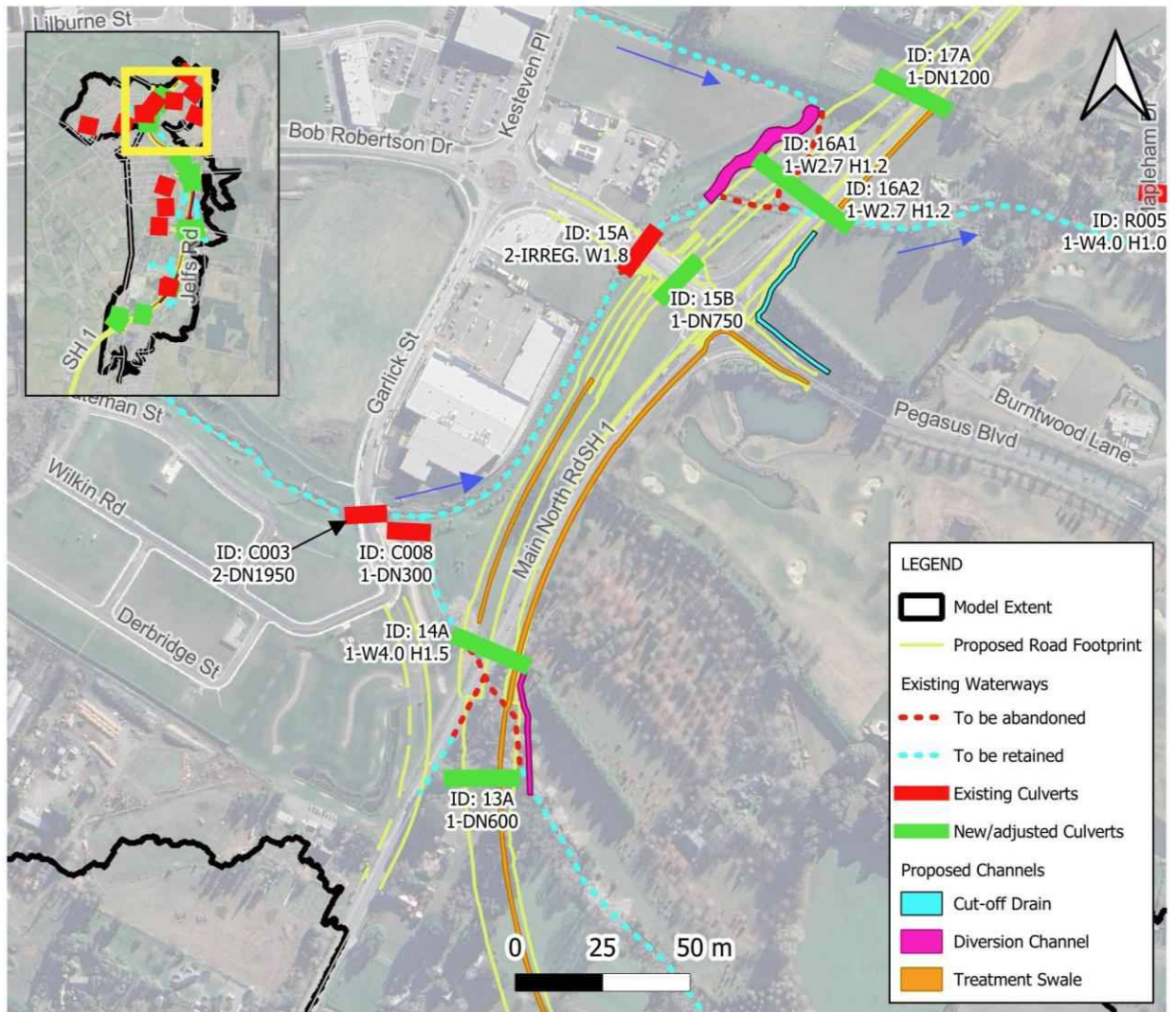


Figure 2-13 Proposed culvert alignments (upper zone)

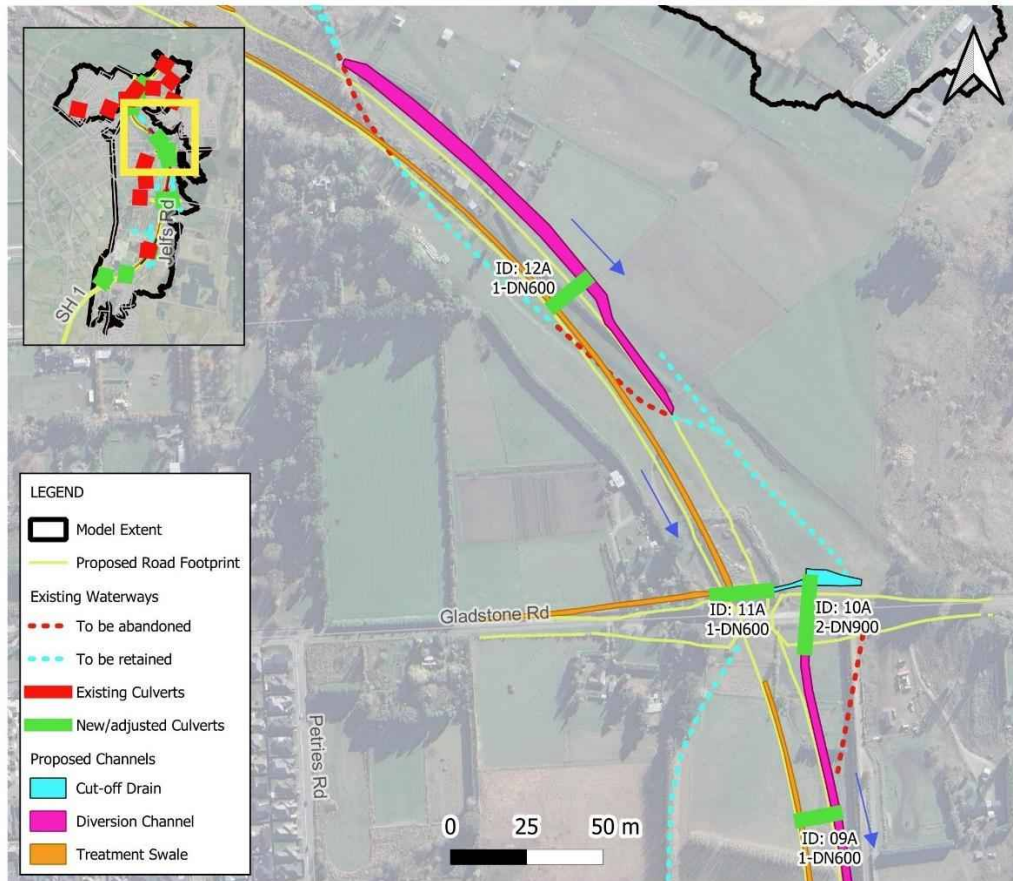


Figure 2-14 Proposed culvert alignments (upper central zone)

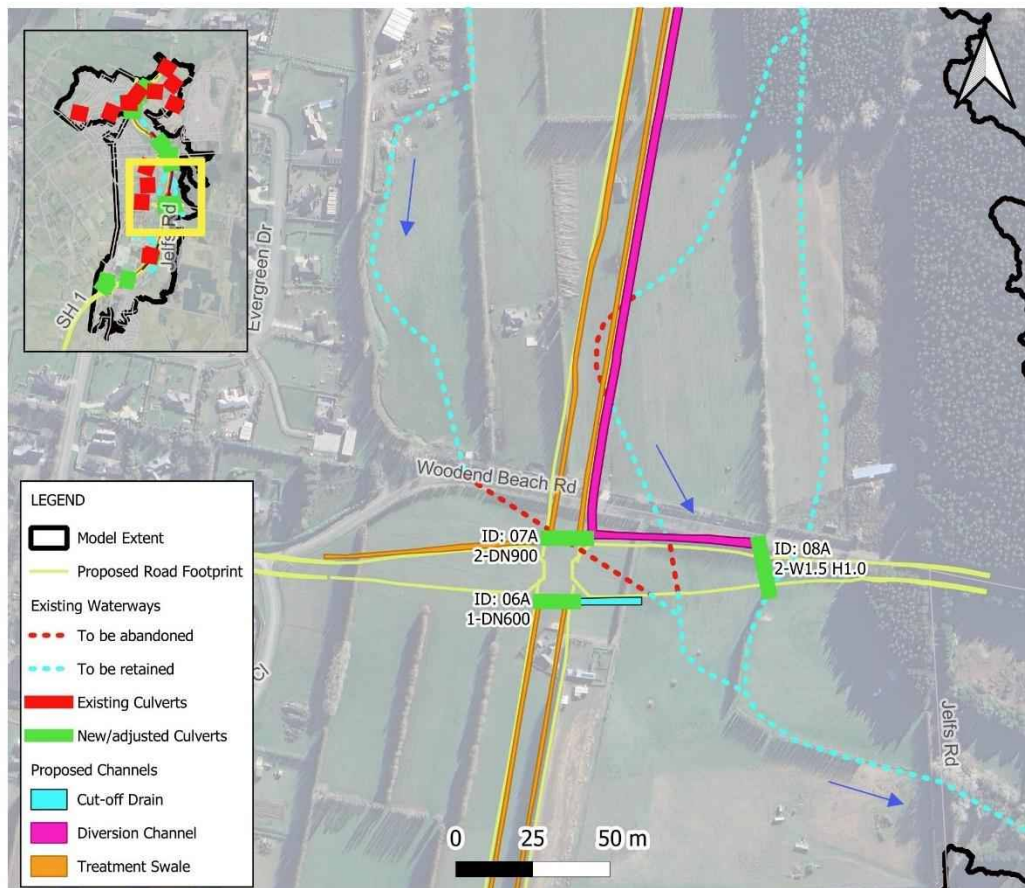


Figure 2-15 Proposed culvert alignments (lower central zone)

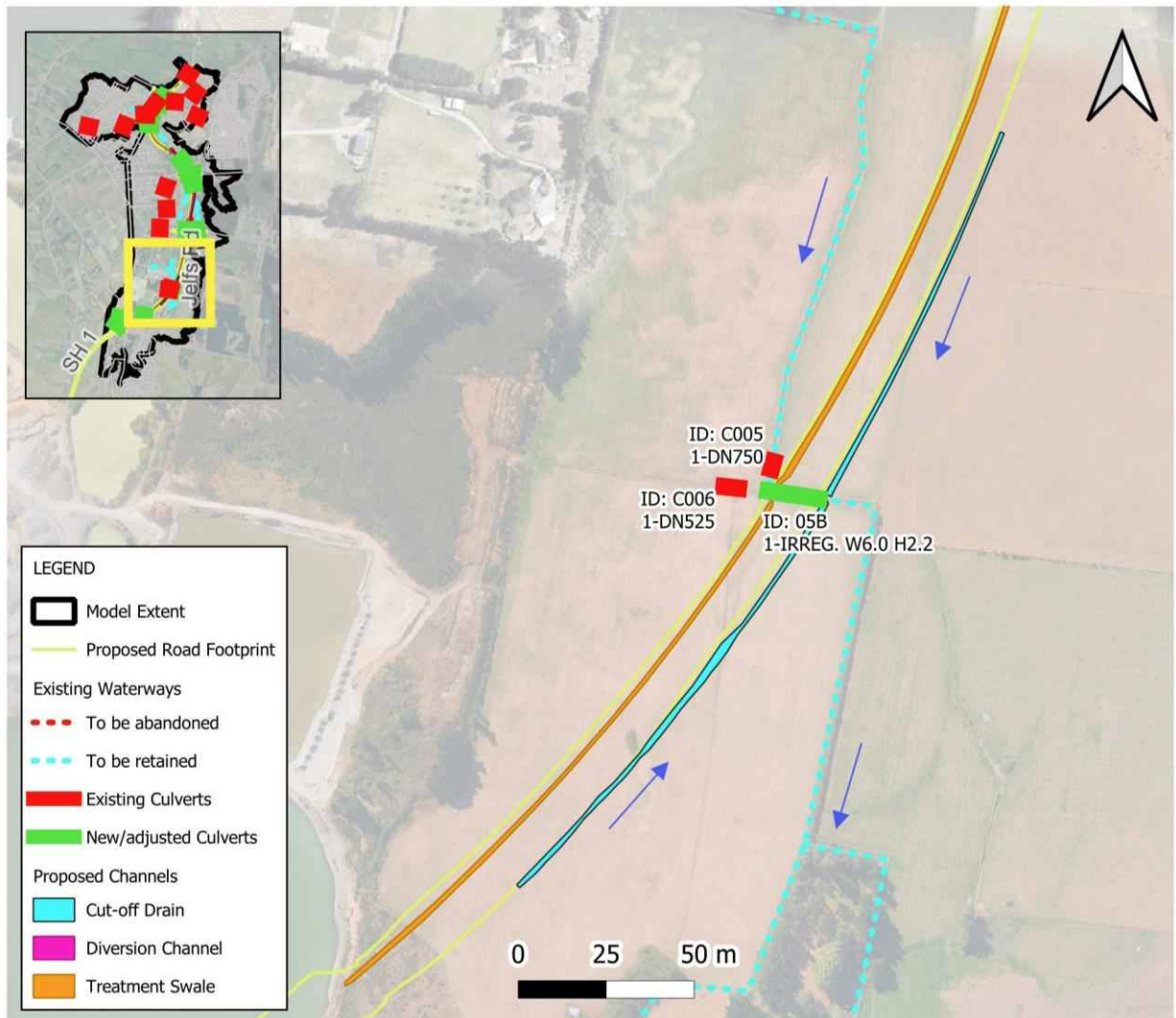


Figure 2-16 Proposed culvert alignments (lower zone)

2.3.1 Mitigation Scenario Result Discussion

Figures C-11 and C-12 (refer to Appendix C) illustrate the maximum flood depth and flood level for the mitigation scenario under a 100-year ARI storm event, while Figures C-13 and C-14 represent these parameters for the 200-year ARI storm event.

Maximum flood level difference maps, comparing the mitigation and pre-development scenarios, are shown in Figures C-15 and C-16 for the 100-year, and 200-year ARI events, respectively. Based on these figures, with the mitigation measures implemented, the proposed state highway remains dry under both the 100-year and 200-year ARI scenarios.

Table 2-4 100-year and 200-year ARI flood level and differences (Mitigation vs Pre-development scenario)

Point ID	Location	100-year ARI			200-year ARI		
		Pre-development flood level (mRL)	Post-development flood level (mRL)	Difference (mm)	Pre-development flood level (mRL)	Post-development flood level (mRL)	Difference (mm)
1	5 Wards Rd*	8.06	7.64	-420	8.36	7.92	-440
2	196 Woodend Beach Rd	4.13	4.14	10	4.23	4.23	0
3	1271 Main North Rd*	8.11	8.06	-50	8.15	8.09	-60
4	1263 Main North Rd	8.28	8.29	10	8.29	8.29	0
5	1279 Main North Rd*	8.01	8.01	0	8.03	8.01	-20
6	1273 Main North Rd*	8.10	8.11	10	8.14	8.11	-30
7	1277 Main North Rd*	8.02	8.01	-10	8.06	8.03	-30
8	1319 Main North Road	8.28	8.29	10	8.29	8.29	0

* Properties will be acquired as part of project work delineation. Flood impacts within designation have been considered acceptable.

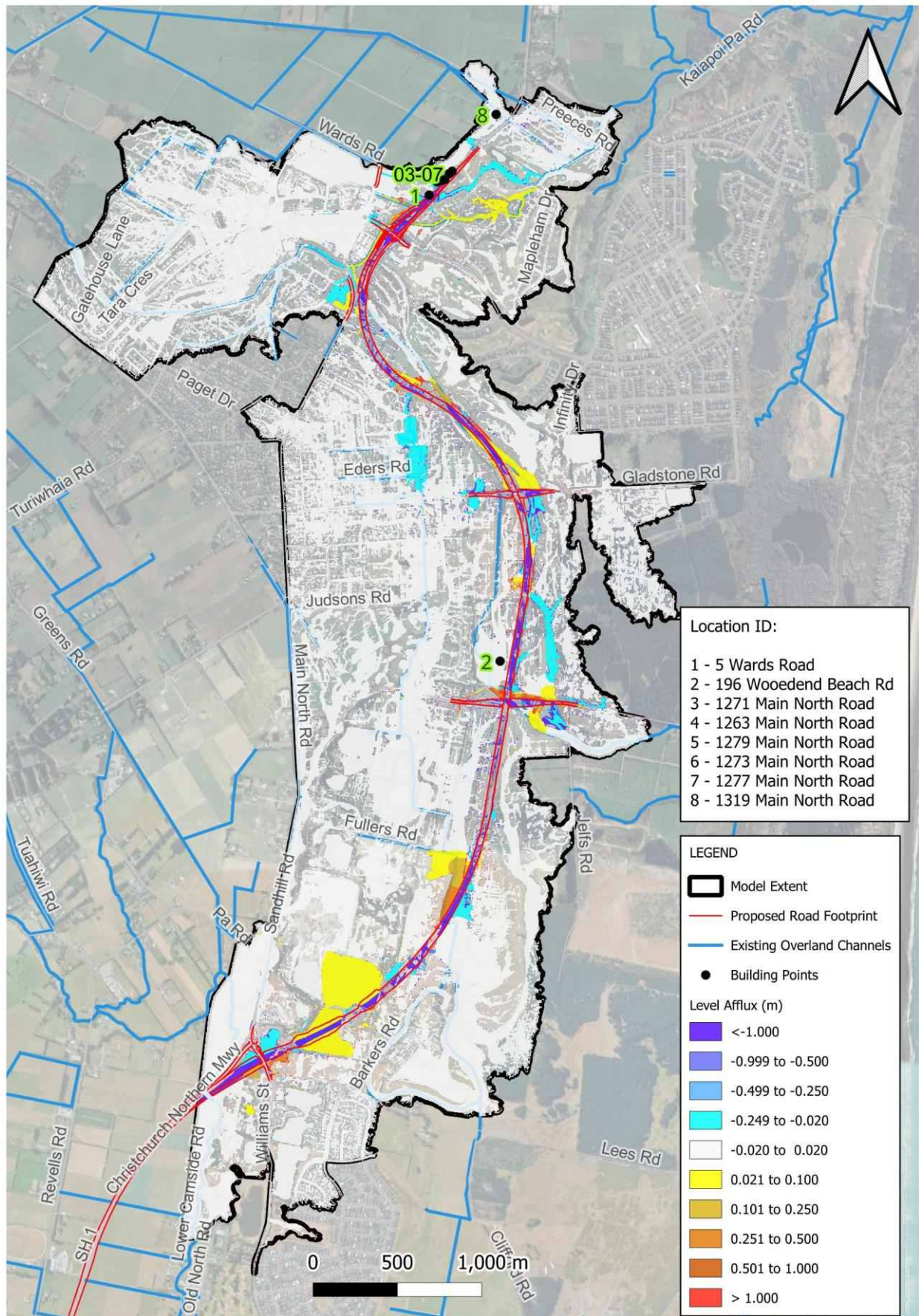


Figure 2-17 Impacted buildings with 200yr afflux level map (Mitigation scenario) as background

Based on the results presented in Table 2-4 and illustrated in Appendix C, the mitigation model has successfully reduced flood impacts across most areas, including previously affected buildings (excluding the five buildings identified for acquisition as part of project delineation) during the 100-year and 200-year ARI event.

Water level increases have been observed in the locations listed in Table 2-5 and as shown in Figure 2-18, Figure 2-19, and Figure 2-20. However, these increases do not affect any property buildings.

Table 2-5 Locations with Flood Level Differences

Locations	Afflux (mm)		Comments
	100-yr	200-yr	
1. Downstream Reach of Taranaki	3	10	Afflux maps show, at the downstream reach of the Taranaki Stream (after Mapleham Drive), minimal flood depth differences, both for the 100-year and 200-year storm events. It signifies that the additional flow, that was diverted from the original overland flow path along the golf course, was conserved and eventually discharged at the same location, maintaining both flow (Figure 2-21 and Figure 2-22) and flood levels
2. Mapleham Drive	10	70	Higher water levels were observed downstream along the Taranaki Stream, particularly near Mapleham Drive, similar to the pattern noted in item 1. However, these elevated levels remain confined within the stream channel and do not impact any nearby buildings. For example, the property at 61 Mapleham Drive, located midstream, has a floor level of approximately 6.0 meters RL. During the 200-year flood event, the pre-development flood level was 5.47 mRL, providing a freeboard of 530 mm; Under mitigation scenario flood level rises to 5.54 mRL, still maintaining a freeboard of 460 mm, ensuring that the property remains protected from flooding.
3. Taranaki Stream downstream of SH1	70	130	The increase in water depth is due to the upsizing of the Taranaki Stream culvert. However, the impacts remain within the channel and do not affect any buildings.
4. Golf Course bounded by SH1 and Mapleham Drive	0	-110	The original overland flow path for the 200-year flood event, caused by the overtopping of SH1 (see Appendix C Figures C-2 and C-4), no longer existed as the excess runoff is now conveyed into the Taranaki Stream via Culvert 16A. This change has resulted in a negative afflux within the golf course area bounded by SH1 and Mapleham Drive.
5. Taranaki Stream close to Harvey Norman (upstream of Bob Robertson Drive)	-200	60	The section of the Taranaki Stream upstream of Bob Robertson Drive exhibits a negative afflux during the 100-year flood event, while a positive afflux is observed under the 200-year event. Flows during the 100-year event are conveyed efficiently due to the increased capacity of the upsized Taranaki culvert. However, during the 200-year flood, the positive afflux occur because culvert 16A reaches its maximum capacity, inducing tailwater upstream of Waihora Creek.
6. Upstream section of Taranaki (upstream of Garlick Street)	-90	-40	For both the 100-year and 200-year ARIs, decreased flood levels were observed in the upstream section of Taranaki Stream, primarily due to improved conveyance of the Taranaki culvert. Any increase in flood depth within the stream extends only as far as Garlick Street (see Figure C-16), beyond which additional runoff is effectively redirected

NOT FOR CONSTRUCTION

Locations	Afflux (mm)		Comments
	100-yr	200-yr	
			southward through Waihora Creek, continuing downstream to Woodend Beach Road.
7. Garlick Street and Basin within Ravenswood residential area	150	70	The designed Garlick Street extension has a sag elevation of 10.4m RL, which is 0.15m higher than the existing sag level but is consistent with the consented weir level. The sag point acts as an overflow weir that connects the basins to Waihora Creek. Consequently, more water is detained in the basins with a higher weir level so that the basin storage can be better utilized. Less water is discharged to the Taranaki and Waihora streams and resulted to lower flood depths in the downstream areas. For 200 year events, basin flood level under post development is already as high as 10.41m RL, makes the raising of weir level less effective in storing water in the basins.
8. Downstream of Waihora Creek	110	130	Higher water levels at this location were primarily caused by the reduced channel storage due to the proposed SH1 cutting off Waihora Creek. The impacts remain contained within the channels and do not affect any buildings.
9. Floodplain North of Gladstone Road	-260	40	During the 100-year flood event, excess runoff was efficiently conveyed through the Gladstone Road culvert (10A), resulting in a negative afflux. For the 200-year event, the design intentionally impounds more water within the existing floodplain north of Gladstone Road to attenuate flows and reduce downstream impacts, particularly near Woodend Beach Road. The impacts remain contained within the floodplain and do not affect any properties.
10. East of Copper Beach Road	260	180	Water ponding is due to the motorway cutting off an existing overland flow path. The ponding is localised and does not impact any buildings.
11. Floodplain North of Woodend Beach Road	500	550	The Woodend Beach Road culvert (08A) was designed to impound and attenuate water before it discharges into Kairaki Creek, resulting in a positive afflux during both the 100-year and 200-year flood events. The adjacent floodplain effectively stores the excess water, ensuring that no buildings or properties are impacted.
12. Downstream of Woodend Beach Road	130	0	Higher flood levels were observed downstream of Woodend Beach Road during the 100-year ARI event, primarily due to more efficient conveyance in the upstream sections, including the culverts at Gladstone and Woodend Beach Road. Despite this increase, no buildings were affected, and all impacts remained contained within the proposed diversion channel. For the 200-year event, the Woodend Beach Road culvert (08A) was specifically designed to limit discharge toward Kairaki Creek so that the flood level remains. The storage capacity of the floodplain north of Woodend Beach Road is well utilized.
13. Upstream of McIntosh Drain	140	320	The increase in flood levels is caused by the proposed SH1 cutting off the existing McIntosh Drain. The impact is largely confined to land owned by NZTA which does not include any residential properties.

Locations	Afflux (mm)		Comments
	100-yr	200-yr	
14. Within the Quarry Lake.	0	90	The increase in water levels was primarily caused by the loss of storage resulting from the infilling of the lake for the proposed bypass road. The water levels in the ponds generally represent existing groundwater levels, and the model assumes no infiltration. However, since the quarry bottom is directly connected to the groundwater, flood afflux is expected to be minimal due to the continuous exchange between surface water and groundwater

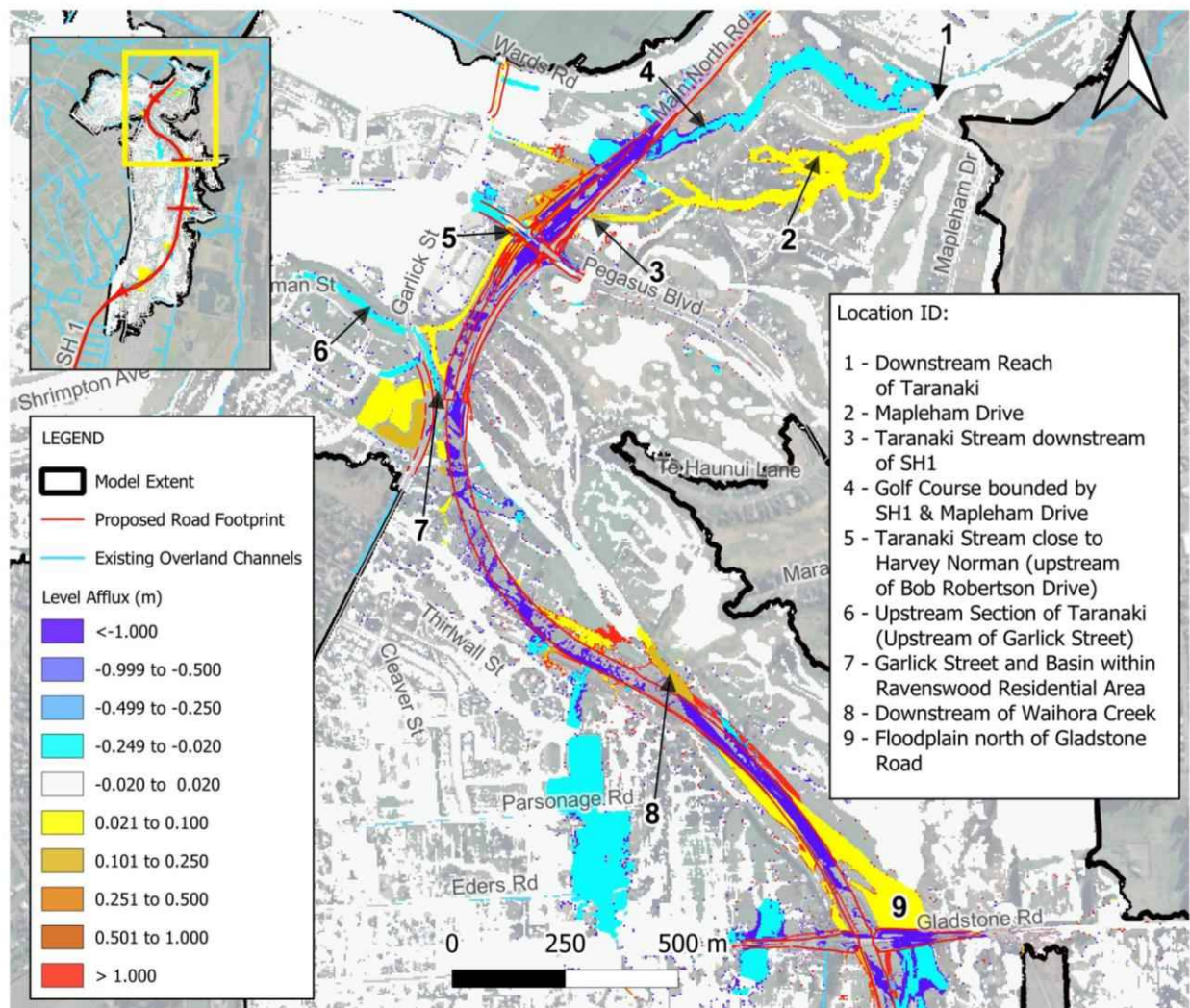


Figure 2-18 Residual impacts outside the designation (upper zone) but does not impact any property buildings (200 year ARI afflux level)

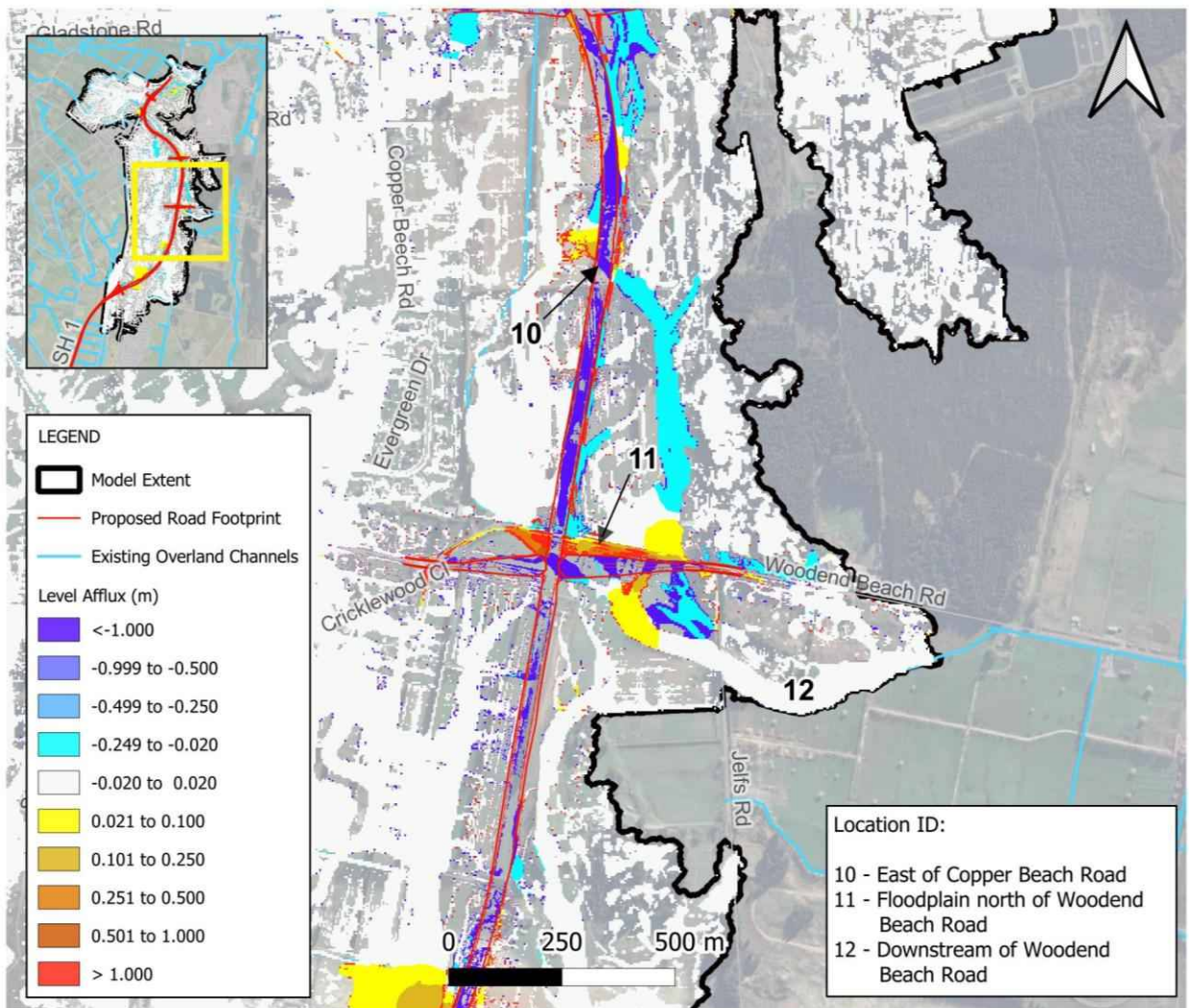


Figure 2-19 Residual impacts outside the designation (central zone) but does not impact any property buildings (200 year ARI afflux level)

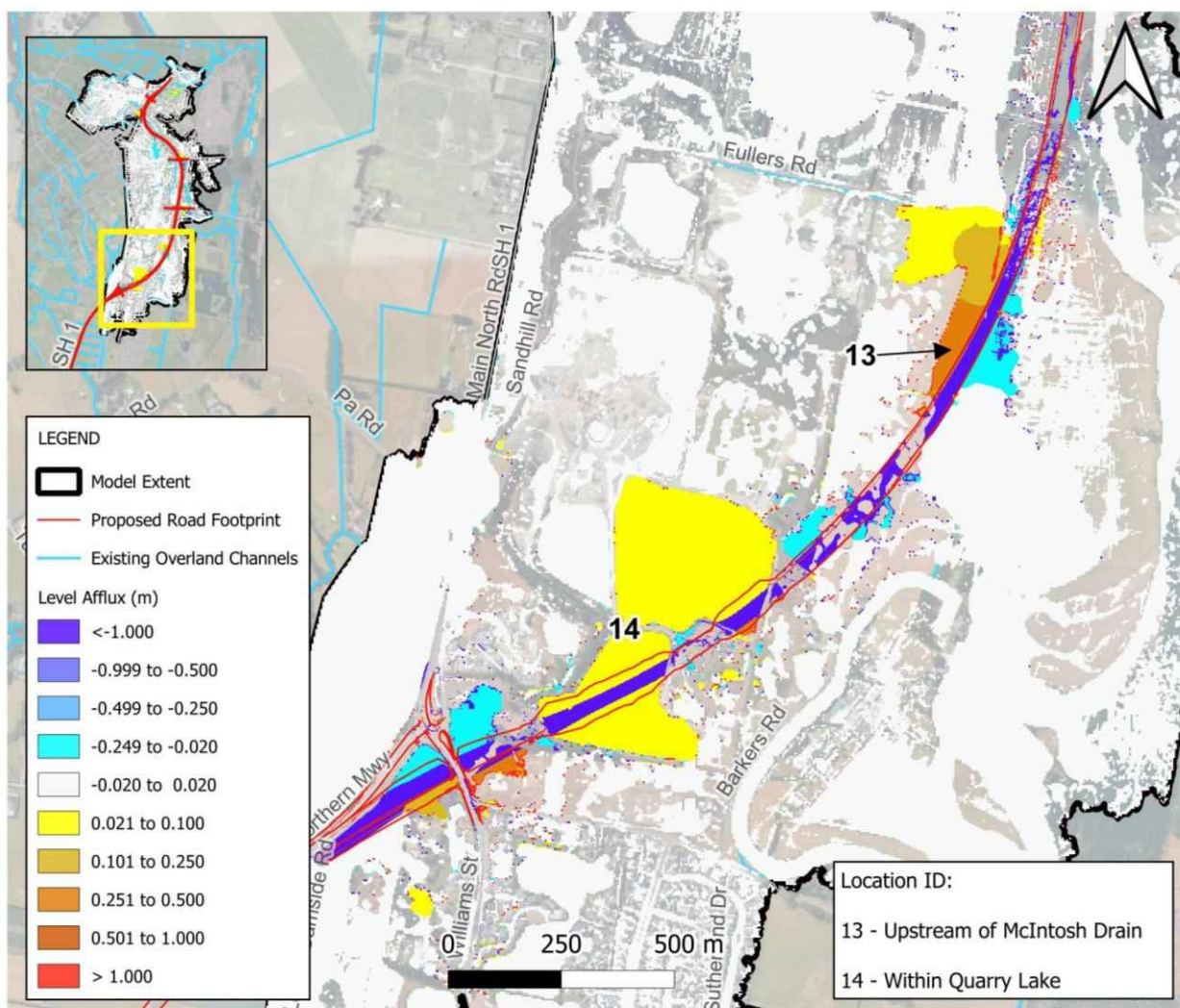


Figure 2-20 Residual impacts outside the designation (lower zone) but does not impact any property buildings (200 year ARI afflux level)

2.3.2 Flood Impact Discussion

Flood impact assessment between the Pre-Development and Mitigation Scenarios were done to determine if a prolonged flood event was observed and if there are possible negative risks that might not have been accounted for due to the limited extent of the flood model. The assessment focuses on major waterways that intersect the proposed bypass road and areas that these waterways directly discharge to. Four locations were identified, which have also been the site of most mitigation measures with the goal of maintaining existing overland flow paths:

- Ravenswood Existing Basin
- Downstream of Taranaki Stream
- Waikuku Beach
- Waihora Creek to Gladstone Road
- Woodend Beach Road
- Woodend Beach and Stalkers Road
- McIntosh Drain

Ravenswood Existing Basin

In the B2P Mitigation Flood Model, the road level at the sag of Garlick Street has been raised to 10.40m RL. Model results indicate higher water levels within the Ravenswood basin and wetland, primarily due to the increased weir elevation at Garlick Street; a slight reduction in downstream flood levels for the 100-year ARI event and a minor increase in downstream flood levels for the 200-year ARI event.

In the Mitigation Model, the property adjacent to the existing Ravenswood basin showed a reduction in freeboard due to the increase of flood level in the basin. However, it still maintained a minimum of 0.527 m freeboard under the 100-year ARI event and 0.478 m under the 200-year ARI event. Ravenswood Harvey Norman showed an increase of freeboard (0.592m) for the 100-year event due to the reduction in flood level downstream of the weir, and a reduction of freeboard (0.054m) for the 200-year event due to the increase in flood level downstream of the weir.

Table 2-6 summarizes weir level change and modelled flood level change for Ravenswood Basin between pre-development model and mitigation model. Table 2-7 summarises freeboard changes of the properties between pre-development model and mitigation model.

Table 2-6 Ravenswood Basin Weir Level Change and Modelled Flood Level Change

Scenario	Weir Level	Weir Length	Modelled flood level in the basin (m RL)	
			100-year ARI	200-year ARI
Pre-development	10.19 m	8.0 m	10.28	10.41
Mitigation	10.40 m	10.0 m	10.43	10.48

Table 2-7 Properties Freeboard Changes

Scenario	Freeboard of the property adjacent to the basin (m)		Freeboard of Ravenswood Harvey Norman (m)	
	100-year ARI	200-year ARI	100-year ARI	200-year ARI
Pre-development	0.68	0.54	1.69	1.76
Mitigation	0.53	0.48	2.28	1.71

Downstream of Taranaki Stream

To assess the potential downstream effects, hydrographs were extracted at two locations within the overland flow path (Lines 34 and 42, as shown in Figure 2-21 and Figure 2-22).

As illustrated in Figure 2-21 and Figure 2-22, the differences in discharge between the pre-development and mitigation scenarios under the 100-year ARI event are marginal, which is consistent with the afflux map as shown in Appendix C Figure C-15 (with no impacts shown further downstream).

Under the 200-year ARI event, the peak discharge at Line 34 increases by approximately 1 m³/s due to additional flow being diverted from the north (overland flowpath within golf course) toward the Taranaki culverts. However, at Line 42, where the northern overland flow branch merges with the main Taranaki flow path at the downstream, the difference becomes less significant, at around 0.25 m³/s or 3%.



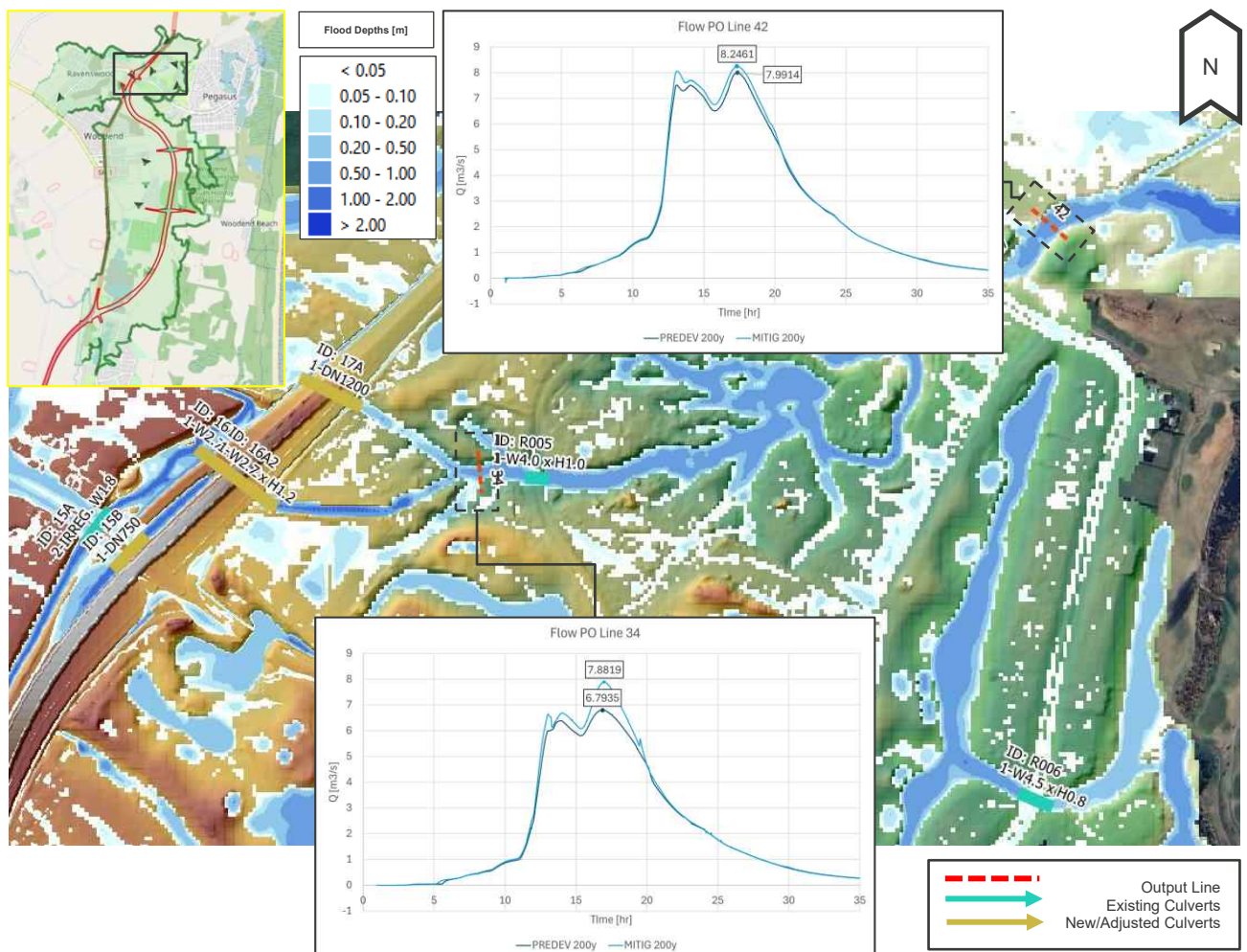


Figure 2-22 Taranaki Stream 200-year Flood Duration Assessment

Waikuku Beach

A volumetric assessment for Waikuku Beach, which is within the interdunal area at the downstream portion of the Taranaki Stream, where runoff ultimately reaches Pegasus Bay, was done to ensure no aggravation of impacts was observed in this area. Pre- and post-development volumes were calculated from the hydrographs at the very downstream end of the model and were subtracted to determine the potential amount of water that would be conveyed towards Waikuku Beach. Two ponding areas were established, based on the WDC Flood Model extents, which are connected by the Taranaki Stream (Figure 2-25). Details and calculations of the volumetric assessment are shown in Table 2-8

For a more conservative approach, relative levels of 3.20 mRL and 1.5 mRL were established for ponding areas 1 (Figure 2-23) and 2 (Figure 2-24), respectively, which are based on the flood extents of the 200-year storm event of the WDC model. Additionally, the two areas were assumed to be hydraulically separated with no outflow; Whereas in reality, flow exchange between the two areas would result to lower flood levels and smaller flood extents.

Table 2-8 Total Volumetric Water Difference - Waikuku Beach

Parameter	PREDEV	MITIG	DIFFERENCE
100-year ARI Total Volume [m³]	191,016	193,913	2,897
200-year ARI Total Volume [m³]	320,922	330,826	9,904
Ponding Area 1 [ha]	104		
Ponding Area 2 [ha]	170		
100-year ARI			

Parameter	PREDEV	MITIG	DIFFERENCE
Ponding Area 1			
Depth Increase [mm]		2.79	
Assumed Water Level [mRL]		3.20	
Final Water Level [mRL]		3.202	
Ponding Area 2			
Depth Increase [mm]		1.71	
Assumed Water Level [mRL]		1.5	
Final Water Level [mRL]		1.502	
200-year ARI			
Ponding Area 1			
Depth Increase [mm]		9.53	
Assumed Water Level [mRL]		3.2	
Final Water Level [mRL]		3.209	
Ponding Area 2			
Depth Increase [mm]		5.83	
Assumed Water Level [mRL]		1.5	
Final Water Level [mRL]		1.506	

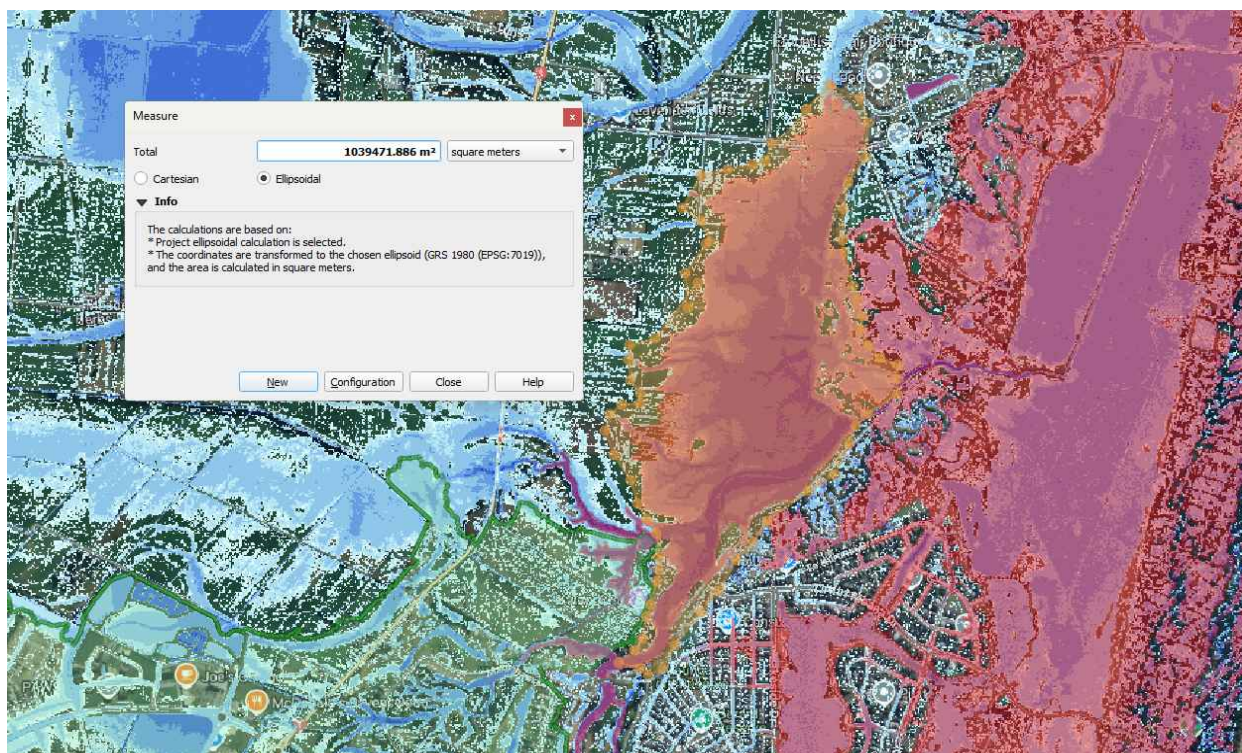


Figure 2-23 Bathtub 1 extent (Downstream of Taranaki Stream)

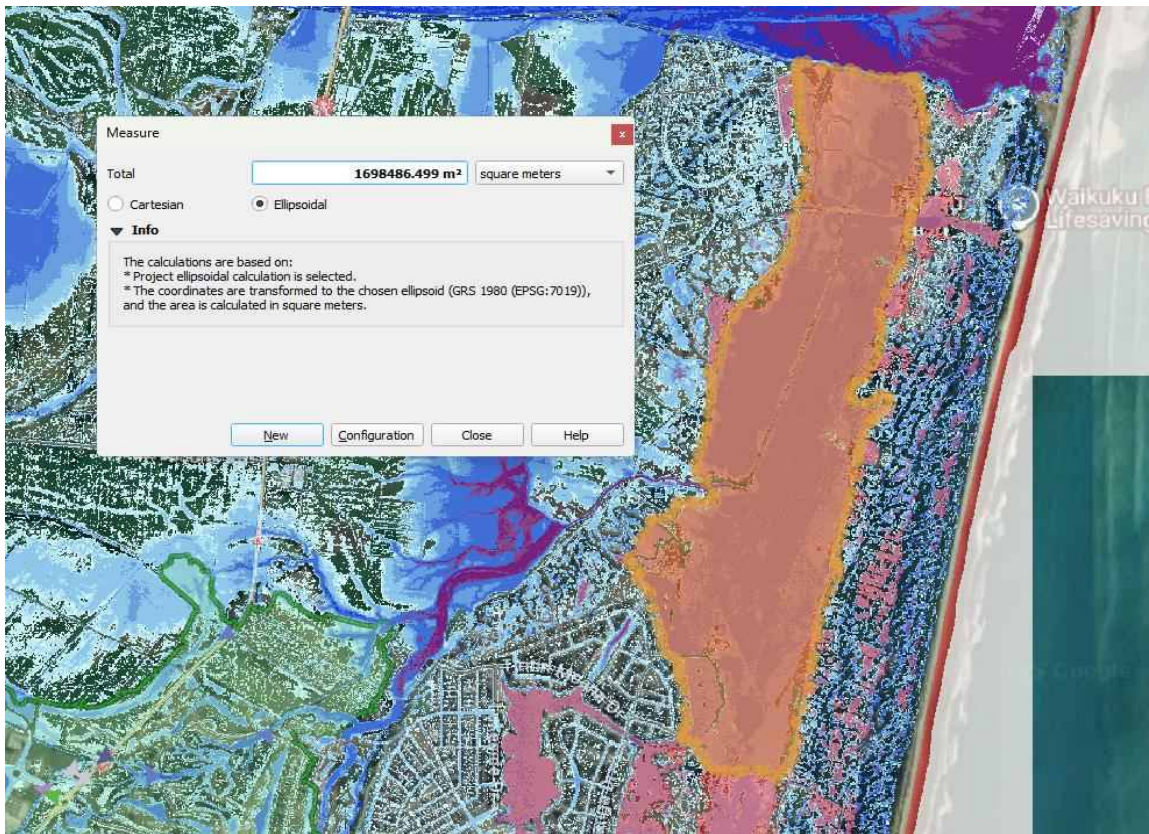


Figure 2-24 Bathtub 2 extent (Downstream of Taranaki Stream)

Figure 2-26 and Figure 2-27 show the post-development flood extents for the 200-year storm event. It is evident from the pictures that any excess runoff stored within the ponding areas results in a negligible increase of extents, moreover, no additional vulnerable entities are affected. The extents were contained within the existing flooded areas, with no houses or structures affected.

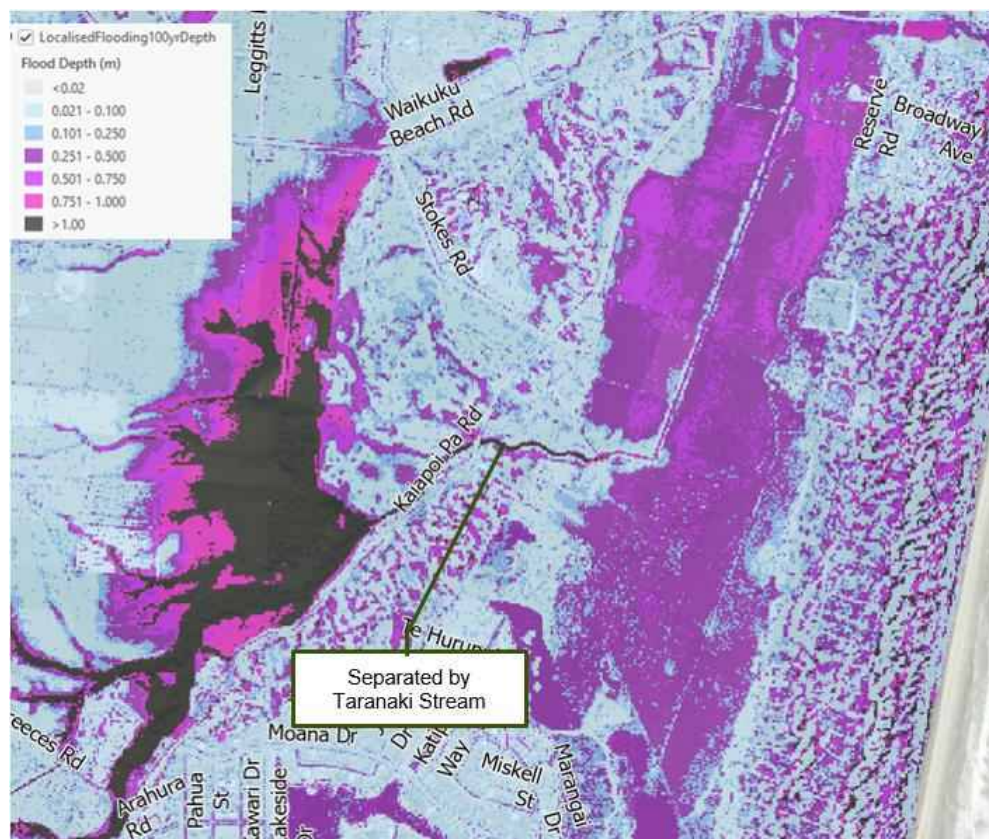


Figure 2-25 Bathtubs downstream of Taranaki Stream



Figure 2-26 Flood Extents results from the Volumetric Assessment (Brown– Pre-development Scenario; Green – Mitigation Scenario)

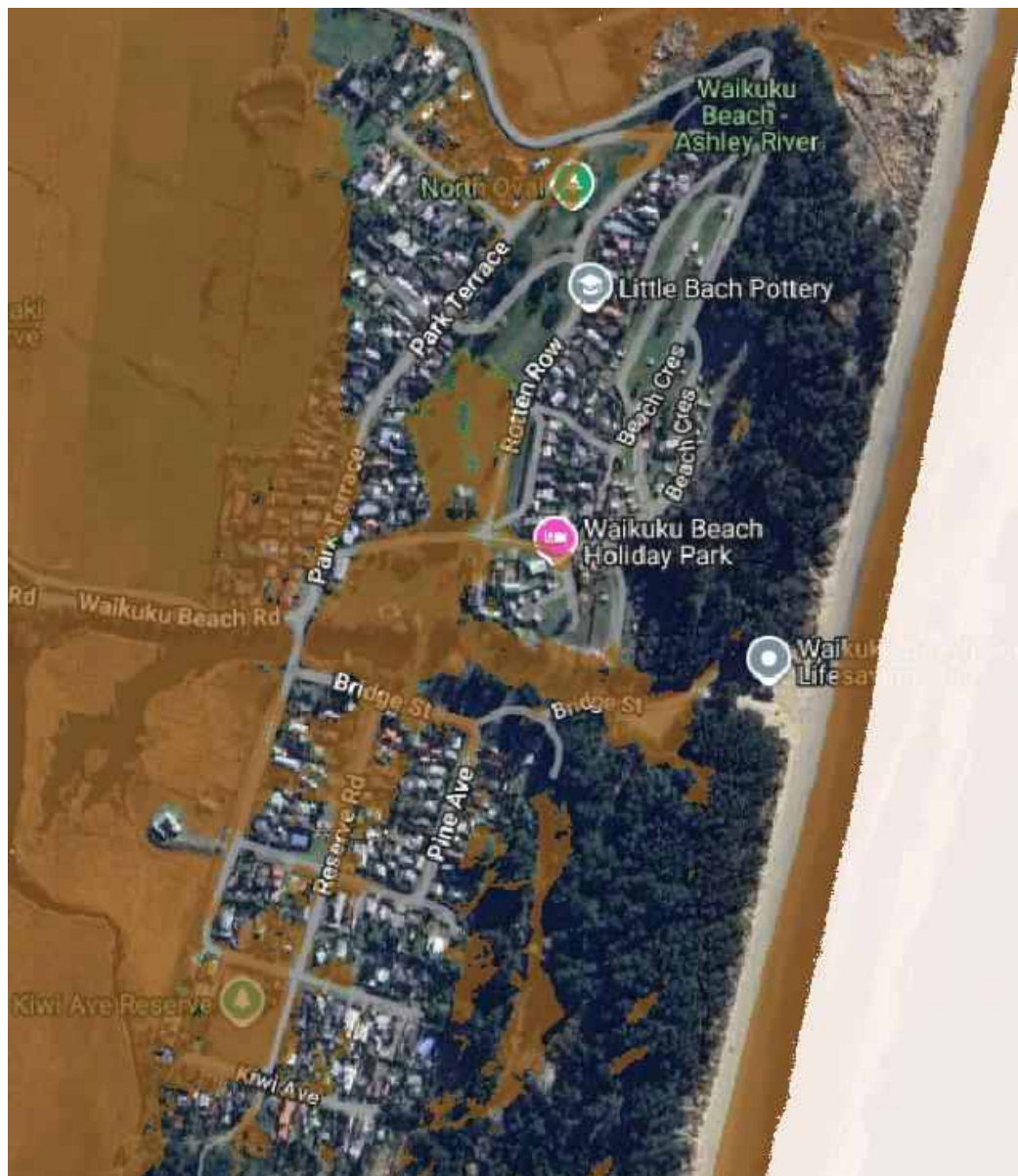


Figure 2-27 Flood Extents results from the Volumetric Assessment (Brown– Pre-development Scenario; Green – Mitigation Scenario)

Waihora Creek to Gladstone Road

The overland flow path connecting Waihora Creek to Gladstone Road functions as a key drainage route, conveying runoff through to Woodend Beach Road and ultimately eastward.

During the 100-year ARI event (Figure 2-28), the upstream hydrograph near the Waihora Culvert shows a 0.08 m³/s increase in the mitigation scenario compared to the pre-development scenario resulting from the disposal of excess runoff coming from the treatment swales and cut-off drains (refer to Section 2.3). This excess runoff was translated to the downstream section, near Gladstone Road, where a larger hydrograph was observed in the mitigation scenario. The downstream section is showing a much bigger volume of flow, primarily due to the proposed SH1 cutting off Waihora Creek thus channel storage is reduced. The proposed diversion channel doesn't make up for the lost channel storage thus more water is passed downstream.

In the 200-year ARI event (Figure 2-29) the upstream hydrograph shows a dual-peak pattern. The mitigated scenario has a lower first peak as more flow is initially conveyed to Taranaki Stream through the larger proposed culvert. Once the Taranaki Stream culvert reaches capacity, excess flow backs up and diverts into Waihora Creek, resulting in a higher second peak in the mitigation scenario (as discussed also in Table 2-5). The downstream hydrograph for the mitigated scenario signifies the intended storage of excess runoff in the

flood plain, north of Gladstone Road, by limiting the flow through culvert 10A in the 200-year ARI event (i.e. flattened hydrograph). Figure C-16 also shows the positive afflux caused by the design storage.

Despite these local affluxes, flood extents remain unchanged and no increased flood risk to is expected to nearby properties.

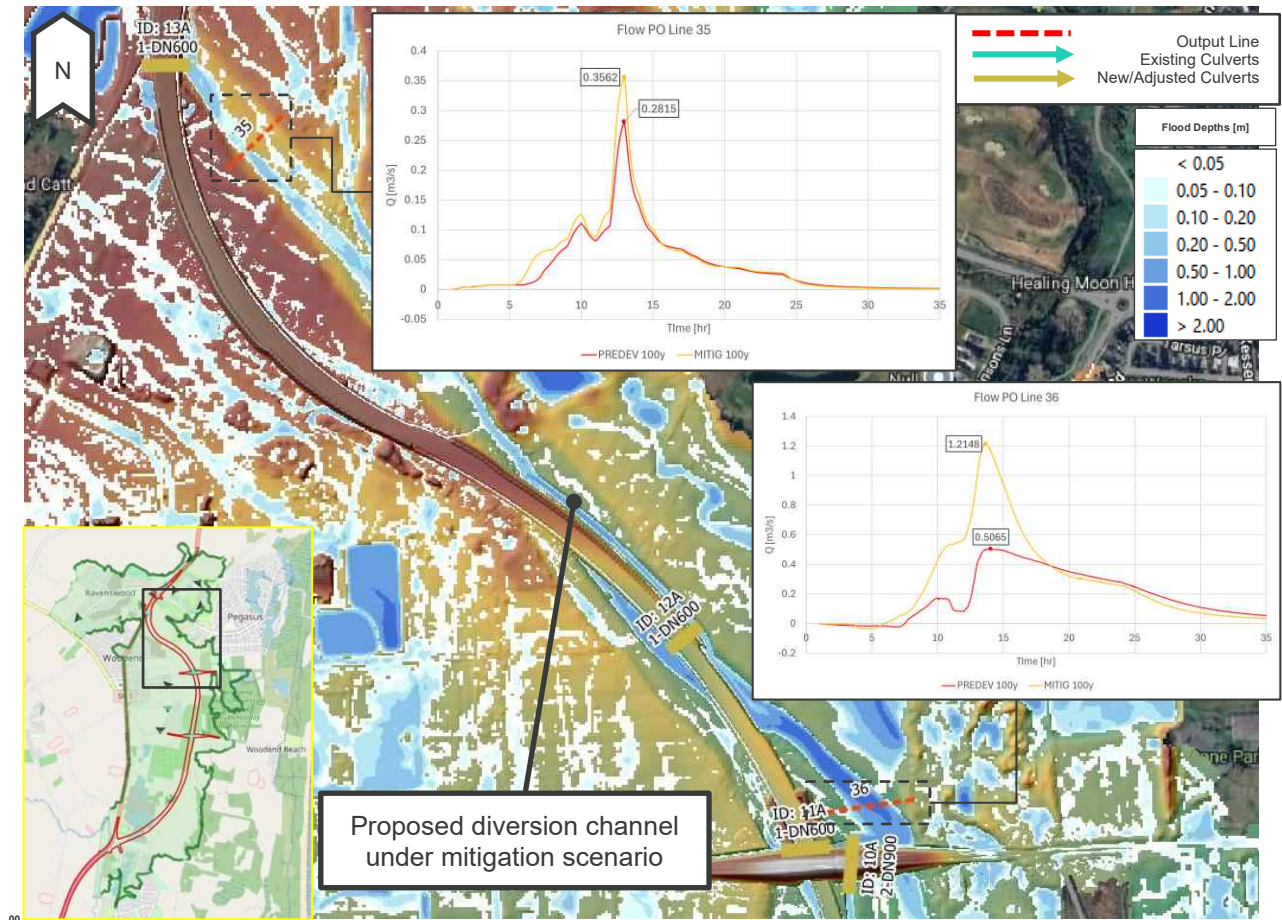


Figure 2-28 Waihora Creek to Gladstone Road 100-year Flood Duration Assessment

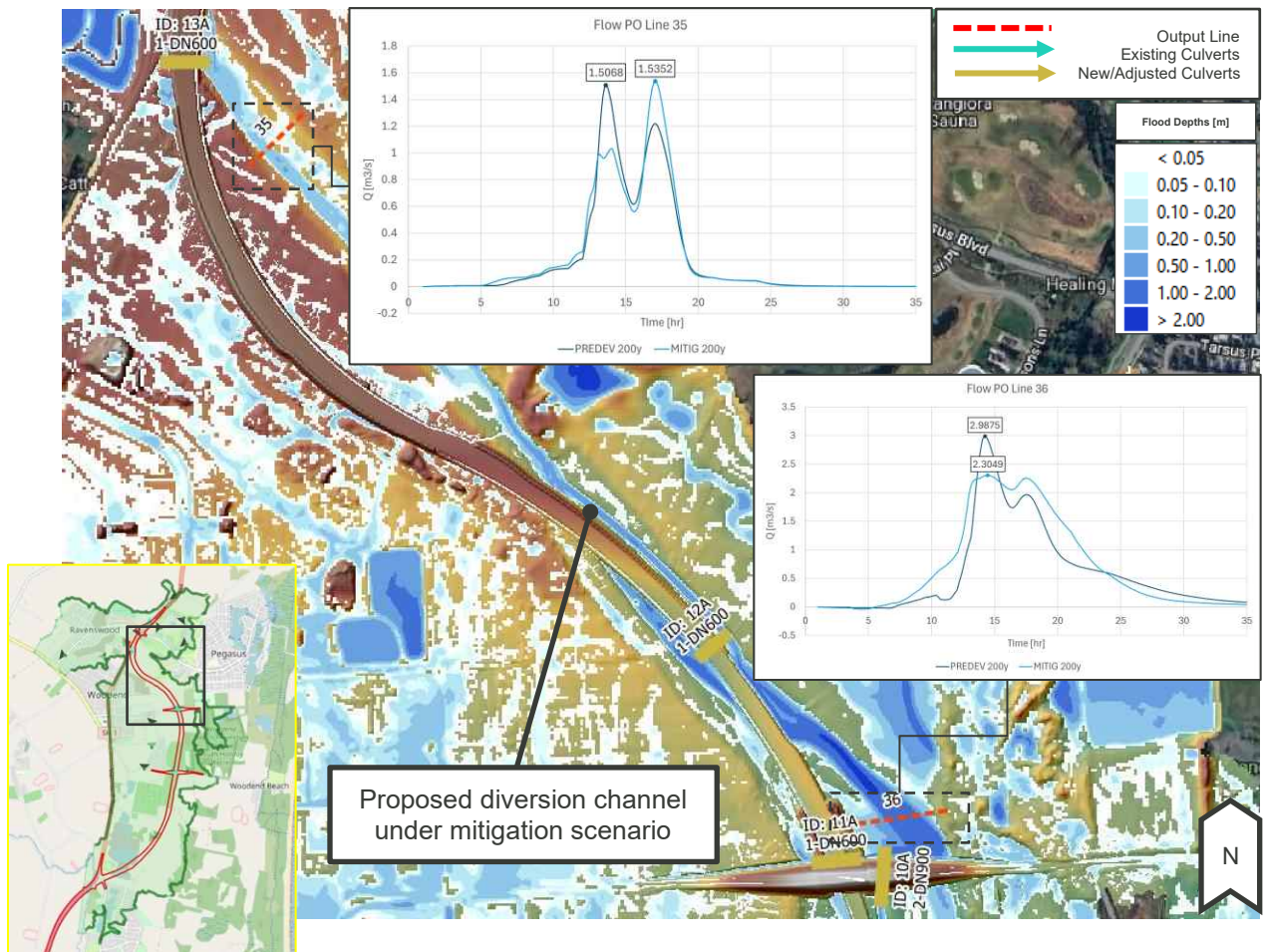


Figure 2-29 Waihora Creek to Gladstone Road 200-year Flood Duration Assessment

Woodend Beach Road

Similar to the situation at Gladstone Road, the proposed interchange at Woodend Beach Road functions as an impounding structure, temporarily storing runoff from the proposed eastern waterways of the bypass.

For the 100-year ARI event (Figure 2-30), the hydrographs at both the upstream and downstream of Woodend Beach Road show higher peaks in the mitigation scenario. This is due to the proposed larger culvert, which enables greater flow conveyed downstream. In the existing scenario, runoff ponds behind the road before overtopping downstream. Positive afflux was also observed at the downstream area but was contained within the existing flood plain and overland flow path (Figure C-15).

For the 200-year ARI event (Figure 2-31), the higher peak between the pre-development and mitigation scenario hydrographs at the upstream and downstream sections switched which indicates a constricted flow through the proposed culvert 08A in the mitigation scenario, with the intension of utilizing the storage behind Woodend Beach Road. This was effective in controlling the afflux at the downstream section with pre-development levels maintained, as shown in Figure C-16. Although, it should be noted that the total volume of water for the mitigation scenario in the downstream hydrograph was still higher than the pre-development scenario hydrograph.

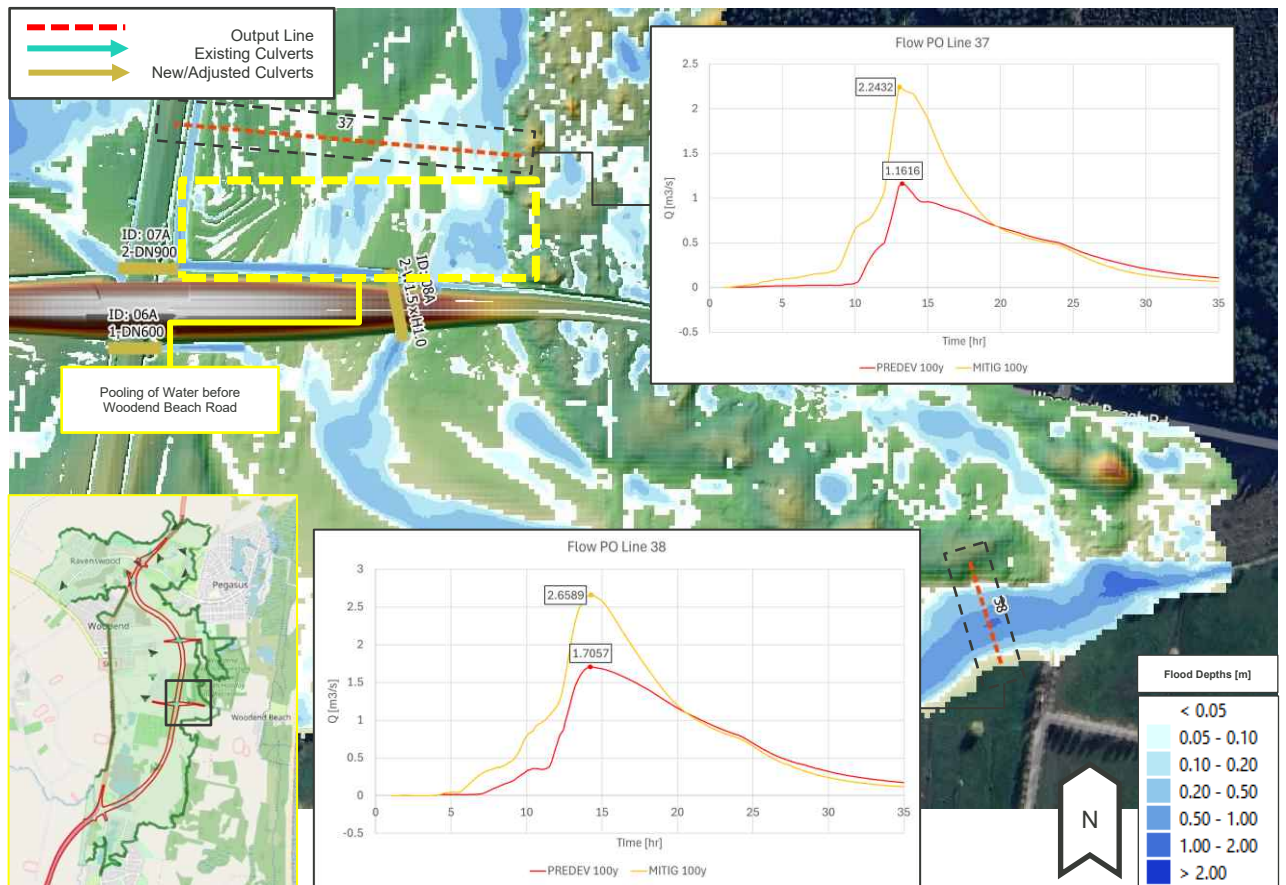


Figure 2-30 Woodend Beach Road 100-year Flood Duration Assessment

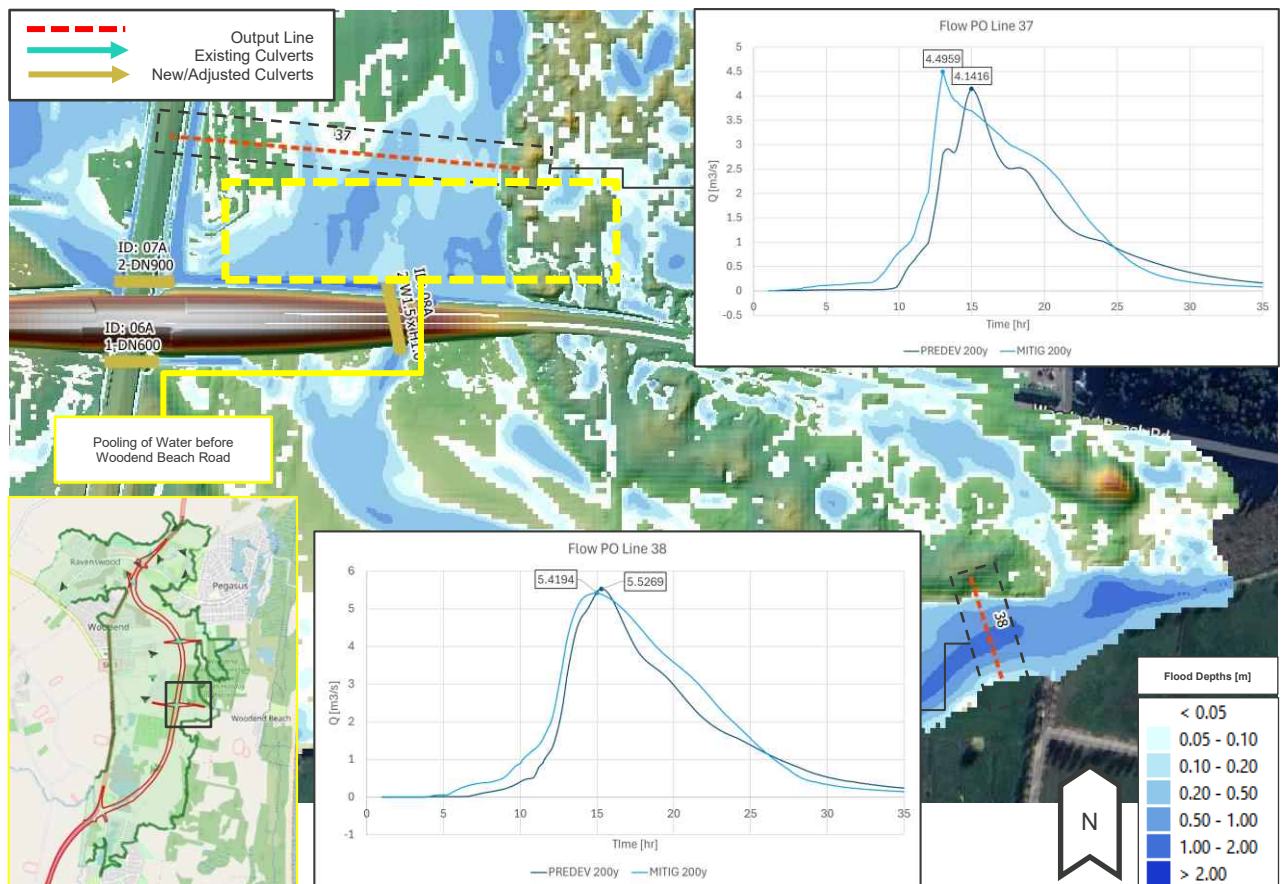


Figure 2-31 Woodend Beach Road 200-year Flood Duration Assessment

Woodend Beach and Stalkers Road

A volumetric assessment at the interdunal ponding areas further downstream of Waihora Creek (outside of the model domain) was done to quantify any potential impacts in this area, specifically in Woodend Beach, and communities near Stalkers Road. Table 2-9 shows the calculated total volume difference between the two scenarios.

Table 2-9 Total Volumetric Water Difference – Woodend Beach Road and Stalkers Road

Parameter	PREDEV	MITIG	DIFFERENCE
100-year ARI Total Volume [m³]	75,399	96,706	21,306.78
200-year ARI Total Volume [m³]	182,601	212,238	29,636.82
Ponding Area [ha]	106.28		
100-year ARI			
Depth Increase [mm]	20.00		
Assumed Water Level [mRL]	1.20		
Final Water Level [mRL]	1.22		
200-year ARI			
Depth Increase [mm]	27.88		
Assumed Water Level [mRL]	1.20		
Final Water Level [mRL]	1.2278		

A conservative bathtub storage assessment was done to establish the water level that corresponds to the 200-year flood extents of the WDC model. An assumed relative level of 1.20mRL was determined that matched the flood extents best and has become the basis for the total ponding area, which is approximately 106.28 ha. (Figure 2-32). The volume difference and the ponding area translated to a potential maximum 27.88mm increase. Pre- and post-development flood extents were overlaid (Figure 2-33) using the calculated water levels and showed minimal increase to the extents with no impacts to buildings and properties. The assessment is considered conservative as it assumes that no increased water discharges from the ponding area due to higher inflows.

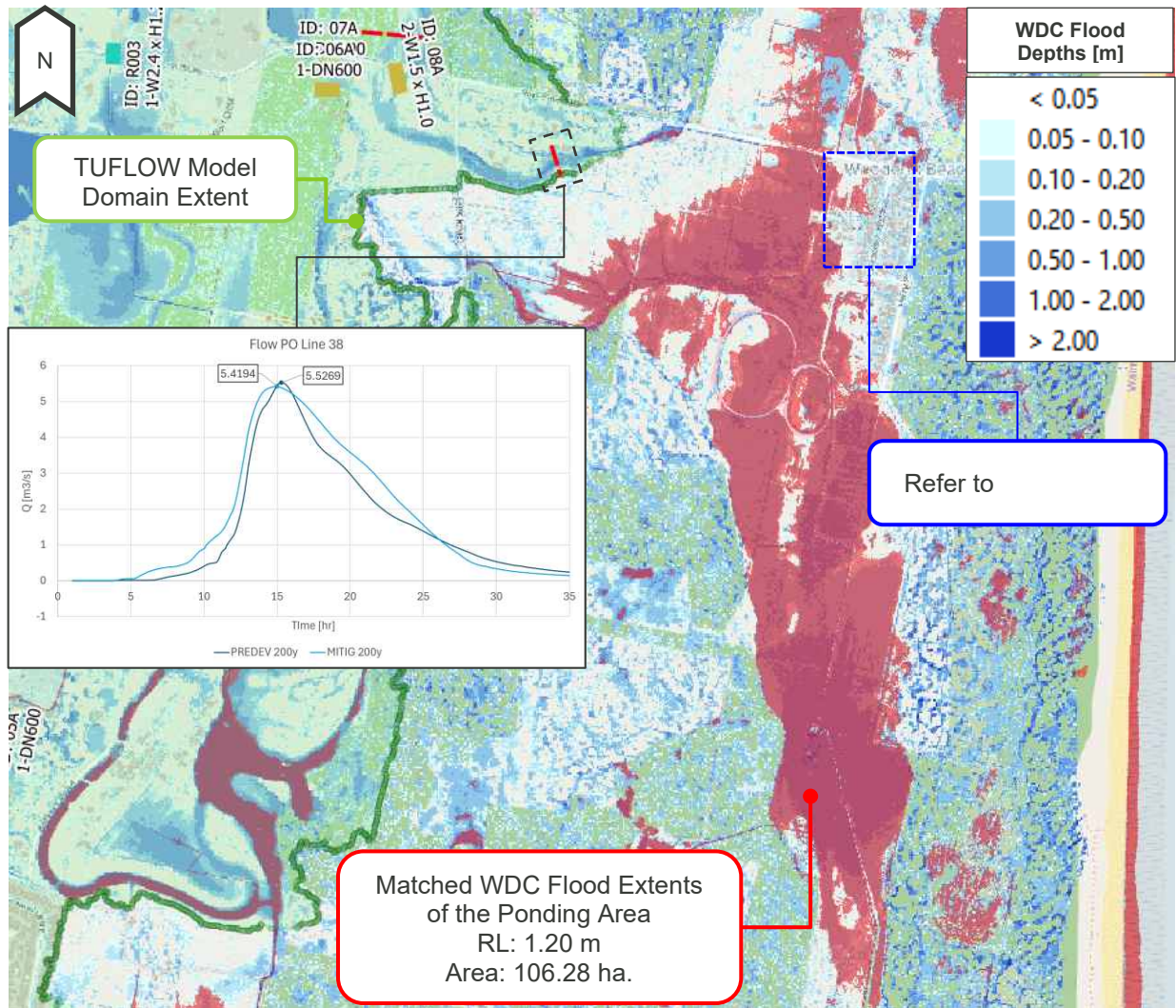


Figure 2-32 Bathtub Assessment using the WDC Flood Model Results



Figure 2-33 Woodend Beach and Stalkers Road Flood Levels

McIntosh Drain

The hydrographs for both the 100-year and 200-year ARI events (Figure 2-34 and Figure 2-35) show similar flow patterns between the pre-development and mitigation scenarios, with the pre-development scenario exhibiting a slightly higher peak. This is due to the wider overland flow path available under existing conditions (Figure 2-3), which becomes more constrained in the mitigation scenario as flows are directed through the proposed bypass and the proposed culvert.

Overall, the drainage function is maintained. A positive afflux is observed upstream of McIntosh Drain (Figure C-15 and C-16), attributed to the raised elevation of the proposed road and a reduced cross-sectional flow area, which limits downstream conveyance. A wider flow path may be required to match pre-development levels. However, as there are no vulnerable assets within the affected floodplain, no increase in flood risk is anticipated.

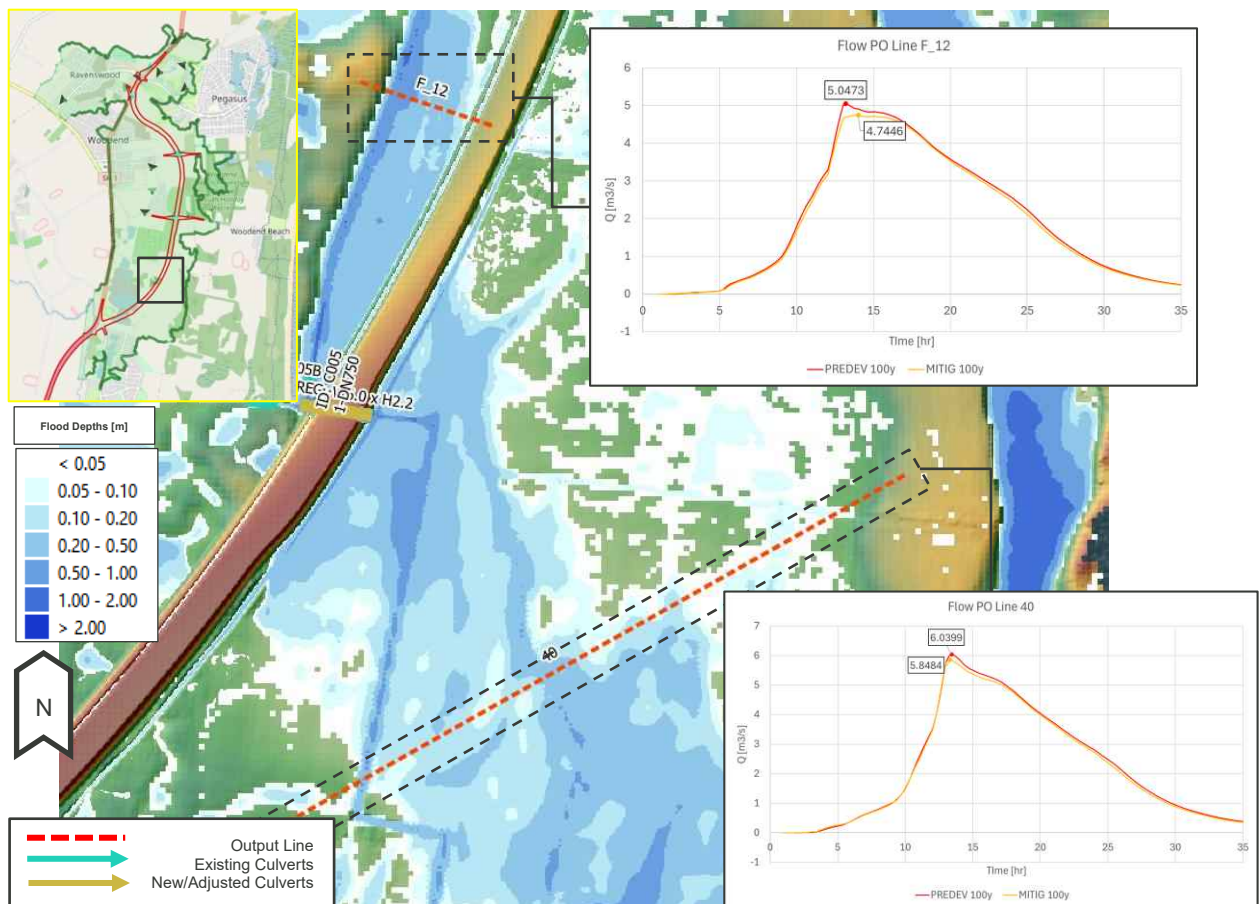


Figure 2-34 McIntosh Drain 100-year Flood Duration Assessment

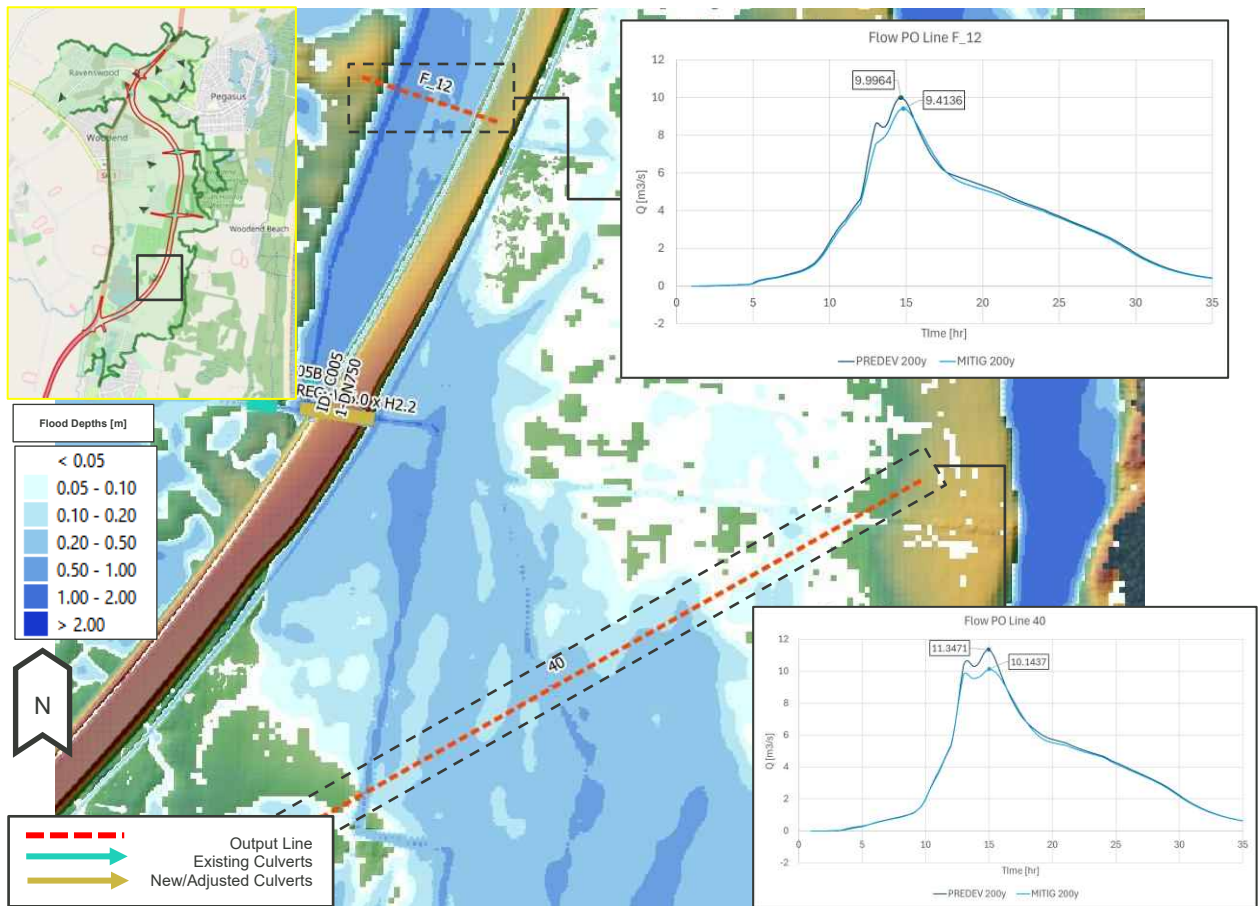


Figure 2-35 McIntosh Drain 200-year Flood Duration Assessment

3 Limitation and Applicability

The limitations and applicability of the study include the following:

- As with any model, this model still includes assumptions and simplifications of the complex real-world processes. On a broad scale, the model build provides a reasonable representation of flood depths and extents. The model, therefore, is deemed fit for purpose for broad-scale planning measures and for testing of flood impacts from proposed stormwater design and flood mitigation options.
- The B2P Flood Model has been developed using the hydraulic and hydrological data from WDC South Ashley Model. With this, all assumptions and limitations from the WDC model also apply. The 'Flood Hazard Models Update' report⁵ can be referred to for further information.
- Terrain data:
 - a. The B2P Flood Model is a snapshot of the catchment when the LiDAR was flown in 2020 and does not include new developments and future development. It may, therefore, not be applicable where flows from newly developed areas impact on results.
 - b. The existing basin surrounding the Ravenswood development area has not been accurately represented due to a lack of survey data and was not captured in the 2020 LiDAR data.
- It should be noted that localised ponding depths may vary significantly where overland flow is obstructed or blocked (e.g. by obstacles, debris, solid fences, retaining walls, sheds, etc.). Results could also be impacted in places which are not necessarily fully resolved by the 2-m grid used in this study.

⁵ DHI (May 2020). *Flood Hazard Models Update*.

4 Conclusion

This assessment has been carried out to investigate the flood impacts associated with the proposed Belfast to Pegasus Road development. A B2P Flood Model was developed using TUFLOW modelling software for this assessment. The following are the objectives of the flood assessment:

- No significant increase in flood levels at neighbouring dwellings for the 100-year ARI event.
- Any increase in flood levels at neighbouring dwellings for the 200-year ARI event should be limited to 20 mm.
- Ensure that the risks associated with the barrier effect induced by the proposed road development on the adjacent areas are reduced through various mitigation measures such as additional culverts, improvement/rehabilitation of existing culverts, and provision of additional channels to maintain or improve flow of runoff.

Three models have been developed, namely Pre-Development, Post-Development, and Mitigation models. The Pre-Development model assessed the current conditions of the model domain, including existing culverts and overland flow paths. It was considered as the base line model where flows, flood levels, and extents will be maintained through the proposed mitigation measures. The model was calibrated and validated with the WDC Flood Model in terms of flood levels and extents. The observed variations were attributed to mainly topographical differences and areas between the WDC and B2P Flood Models. The computational grid resolution was also seen as a factor to the differences of results. The Post-Development model accounts for the proposed roadway, without the mitigation measures and shows the exacerbation of impacts and flood levels. These impacts were reduced in the Mitigation model through the installation of new culverts and channels to maintain the original drainage line and storage capacities of the surrounding areas.

The assessment findings indicate a flood level increase in various locations due to the proposed road development, with most impacted zones confined to the vicinity of the proposed road corridor. Additionally, elevated flood levels were observed in the residential areas along Main North Road and Woodend Beach Road, during both the 100-year and 200-year ARI events.

To ensure compliance with the regional policy statement and council requirements, two primary measures were proposed and modelled to reduce the flood impacts: the introduction of additional culverts and the diversion of flow path channels, including drainage channels, treatment swales, and cut-off drains. The mitigation measures have demonstrated effective reduction in flood impacts across most areas during both the 100- and 200-year ARI events. The three impacted properties in the post-development model have been fully resolved in the mitigation model, with impacts of less than 20mm afflux.

Positive affluxes are still observed in areas outside of the designation boundary; but some of these areas were intended to be flooded and it was ensured that no property will be affected. A conservative analysis was conducted for areas outside of the model domain to ensure that no risk is exacerbated. It showed that a marginal increase in the hazard is imminent, but due to the absence of buildings and properties, it is expected that the overall risk is maintained from the pre-development scenario.

Appendices



Appendix A

B2P Flood Model Build Methodology

B2P Flood Model Development

Appendix A Table 1 provides an overview of the model build methodology of the B2P Flood Model for pre-development scenario (PREDEV) and summarises the hydrological and hydraulic parameters used. This also compares the parameters used in the B2P Flood model with the WDC South Ashley Model.

Appendix A Table 1 Comparison of WDC South Ashley Model and B2P Flood Model build

Parameter	WDC South Ashley Model	B2P Flood Model (PREDEV)
Software version	MIKE FLOOD	TUFLOW-2023-03-AC-iSP-w64
Projection	NZGD 2000 New Zealand Transverse Mercator	NZGD 2000 New Zealand Transverse Mercator
Vertical datum	Lyttelton Vertical Datum (LVD)	New Zealand Vertical Datum 2016 (NZVD16)
Computational and simulation parameters	- Minimum timestep: 1e-05s - Maximum timestep: 1s - Simulation time: 56hr - Results save timestep: 5min	1d timestep: 0.1s 2d timestep: 0.5s - Simulation time: 35hr - Time series output interval: 60s - Map out interval: 300s
Catchment delineation	Model extent covering Waimakariri district	- Project extent covering Pegasus in the north and Kaiapoi in the south. - Refer to Appendix A Catchment Delineation Section
Topography	- Model grid element size: 5-m rectangular flexible mesh grid 5-m LiDAR DEM 2014 from WDC South Ashley Model	- Model grid element size: 4-m rectangular grid with sub-grid sampling 1-m LiDAR DEM 2020 from LINZ server - Surveyed LiDAR for Kaiapoi Market Square pre-development
Hydrology		
Rainfall	24-hr nested storm profile as derived from HIRDS v4 for the RCP 8.5 (2081-2100) climate change scenario.	24-hr nested storm profile for 100-year and 200-year ARI as extracted from WDC South Ashley Model
Initial Condition	Catchment is assumed dry initially	Catchment is assumed dry initially
Boundary Condition	- Constant water level downstream boundary of 1.4m RL at the confluence of Kaiapoi River and Waimakariri River - Free outflow for the remaining areas downstream of the model	- Inflow boundaries (QT) applied upstream of the pipe network as extracted from WDC South Ashley Model - Water level boundaries (HT) applied downstream as extracted from WDC South Ashley Model - Refer to Appendix A Boundary Condition Section
Ground infiltration	- Infiltration method: MIKE21 infiltration and leakage module - Parameters such as initial infiltration, leakage, porosity, and initial saturation were applied	- Infiltration Method: TUFLOW Horton - Parameters such as initial loss, initial loss rate, final loss rate, porosity, and initial moisture were applied as extracted from WDC South Ashley Model - Exponential decay rate was applied as extracted from Waterways, Wetlands, and Drainage Guide of the Christchurch City Council - Refer to Appendix A Ground Infiltration Section

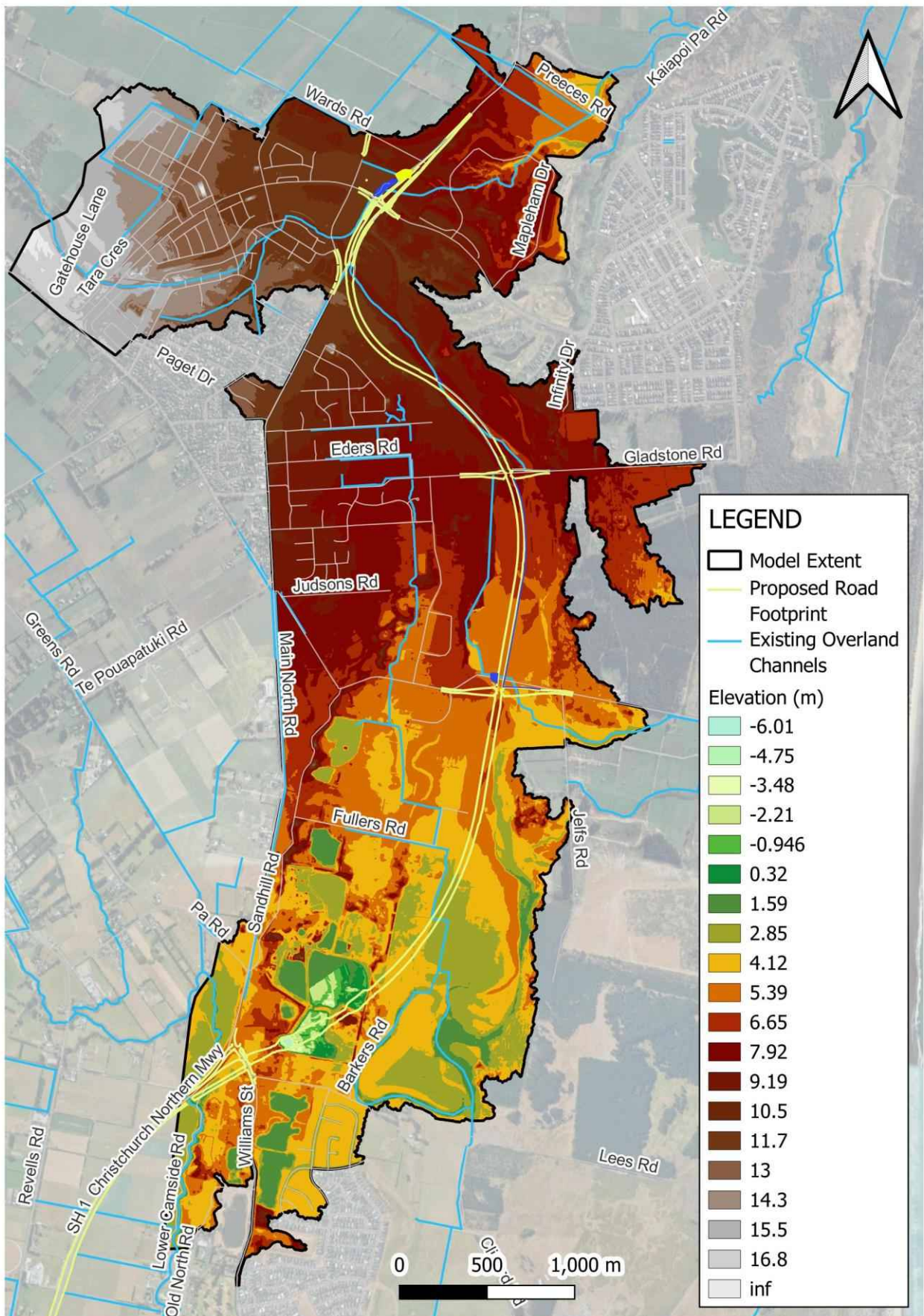
Parameter	WDC South Ashley Model	B2P Flood Model (PREDEV)
Surface roughness	Manning's surface roughness 'M' based on LCDB v4.1 database shapefiles	- Manning's surface roughness 'n' as extracted from WDC South Ashley Model - Refer to Appendix A Surface Roughness Section
Stormwater network		
Pipe network & Culverts	- Stormwater asset data covering the Waimakariri District - Culverts are not defined in the model. Where flow path is blocked by existing road, existing road level has been lowered down to provide overland flow path	- Models were run with the assumption of 100% pipe blockage. - For culvert modelling, refer to Section Stormwater Network and Culverts under Appendix A

Terrain

The topographic data used in the B2P Flood Model comprises of the collected survey data and the 2020 LiDAR Digital Elevation Model (DEM) from LINZ. The survey covers the entire length of the designation of the proposed roadway bypass, including newly developed communities near the project site that were not covered in the 2020 LiDAR DEM. The Flood Mitigation Scenario Model was re-ran with the finalised proposed roadway surface to analyse the effectivity of the measures with the ultimate design.

Catchment Delineation

The catchment was delineated using the 1m DEM LiDAR 2020 to determine the B2P Flood Model extent as shown in Appendix A Figure 1. The model extent is simplified by defining the catchment boundary covering the road alignment mostly in the towns of Pegasus, Woodend and Kaiapoi. Upper catchments were just represented as inflow boundary condition lines to avoid longer model runtime.



Appendix A Figure 1 B2P Flood Model catchment delineation

Hydrology

Rainfall

Rainfall was ingested into the model domain using a set of 50m x 50m resolution time varying 24-hour nested rainfall grid which was extracted from the WDC Flood Model. The rainfall grids have an hourly timestep with the first observation of rainfall starting at hour 1. As stated in Appendix A Table 1, the dataset was derived from HIRDS and incorporates a climate change scenario of RCP 8.5 for the period of 2081-2100.

Boundary Condition

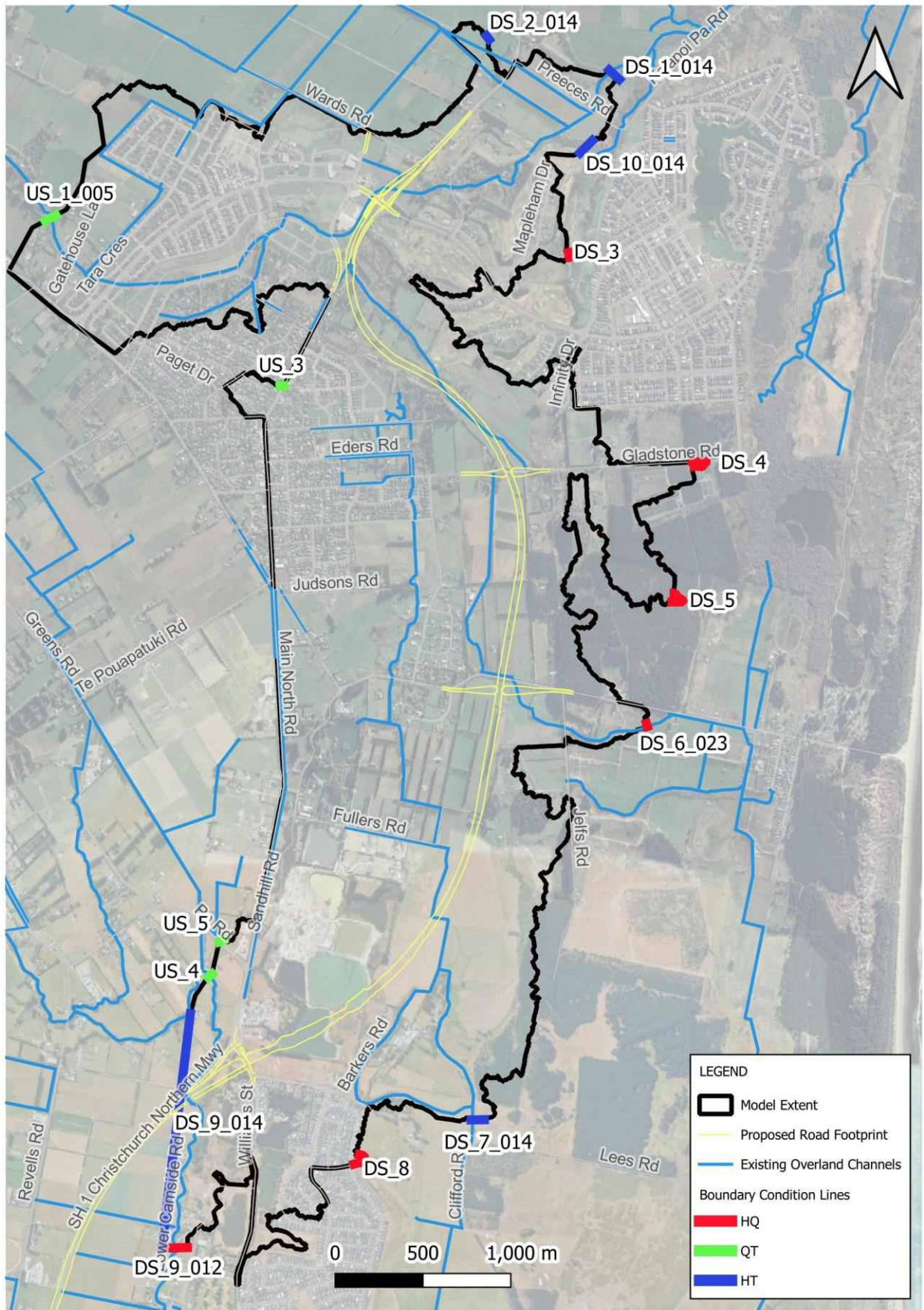
The WDC Flood Model was the basis of all the boundary condition locations, where major drainage lines cross the extents of the model domain. Upstream flow BCs (QT) and downstream water level BCs (HT) were extracted also from the WDC Flood Model. Open boundaries, as represented by the slope, were calculated from the topographical data used in the TUFLOW model.

Flow hydrographs were used as an upstream boundary condition (US_1_005, US_3, US_4, US_5) along major drainage lines (e.g. rivers and streams) which are connected to upstream catchments and have been the source of water for the model domain. These are represented as a Flow vs. Time (QT) input with locations shown in Appendix A Figure 2. The 100- and 200-year ARI flow events are shown in Appendix A Figure 3.

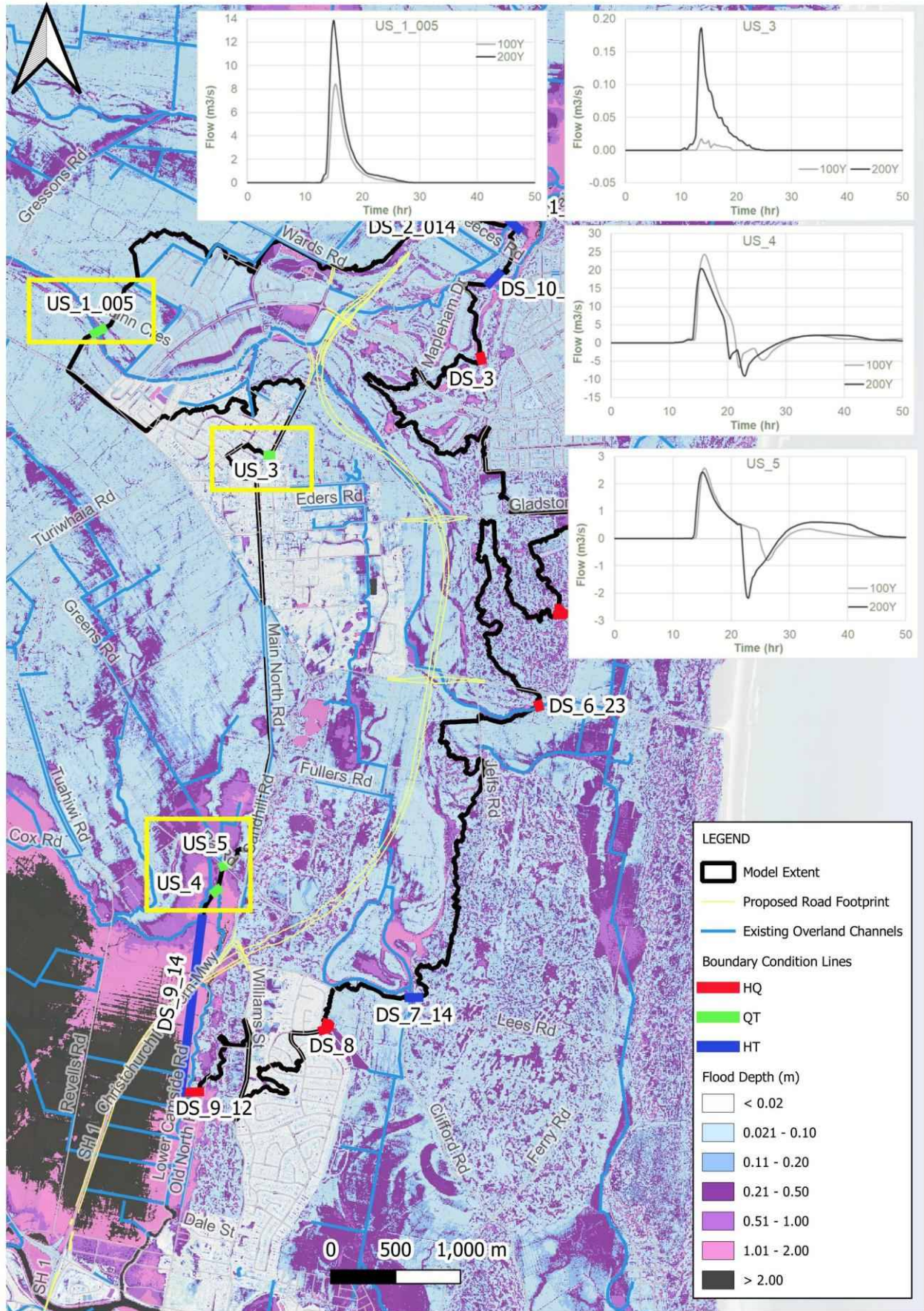
Water level boundaries, Head vs Time (HT), are applied downstream of the model (DS_1_014, DS_2_014 and DS_10_014) at Waikuku to the north and at the western side of the Cam River (DS_9_014), as shown in Appendix A Table 2 and Appendix A Figure 4. These boundaries are influenced by other sources of water, specifically for areas near river or stream junctions that were not considered in the model domain. The same approach was applied to the floodplain at the southwestern boundary of the model (near Cam River) to simplify the movement of water since the WDC model shows that the area is a catch basin of water from different locations, apart from the Cam River.

A water level boundary Appendix A Figure 4 was also established at a major overland flow at a drain located east of quarry lake (DS_7_014). The flow paths connect to a small drainage channel that can constrict flow in rare storm events, consequently funnelling water and producing tailwater conditions.

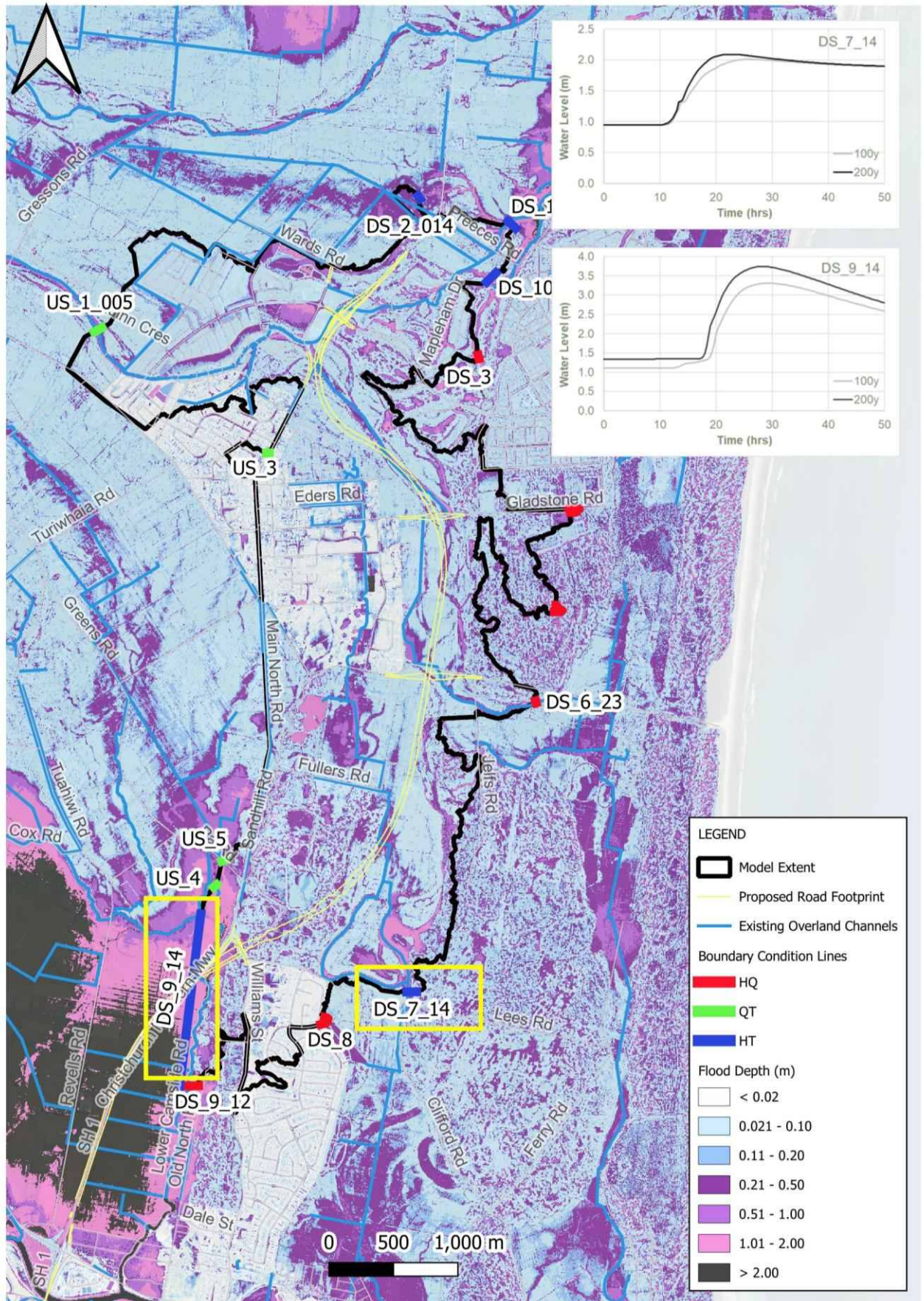
Free flow boundaries, as shown in (HQ), have been placed at the downstream cross section of the Cam River (DS_9_012), and a few other streams that eventually discharges towards the east (DS_3, DS_4, DS_5, DS_8), and specifically the downstream end of the Waihora Creek (DS_6_023). These areas drain water freely out of the model domain.



Appendix A Figure 2 B2P Flood Model inflow and downstream boundaries



Appendix A Figure 3 Boundary Conditions with Equivalent Inflow hydrographs



Appendix A Figure 4 Boundary Conditions with Equivalent Stage hydrographs

Appendix A Table 2 Water Level Downstream Boundary Conditions

BC Line	100-year Level (m)	200-year Level (m)
DS_1_014	2.93	2.94
DS_2_014	5.31	5.31
DS_10_014	2.93	2.94
DS_7_014	Varying Water Level (Appendix A Figure 4)	
DS_9_014		

Appendix A Table 3 Normal Depth Downstream Boundary Conditions

BC Line	Slope
DS_3	0.0036
DS_4	0.0041
DS_5	0.0041
DS_6_023	0.0017
DS_8	0.0008
DS_9_012	0.0008

Ground Infiltration

The soils infiltration losses are applied to the B2P Flood Model using the parameters of the Horton method. Parameters such as the dry loss, initial loss rate, final loss rate, porosity, and initial moisture were extracted from WDC South Ashley Model. Detailed information on ground infiltration methodology can be referred to Section 3.2.2 of 'Flood Hazard Models Update' report.

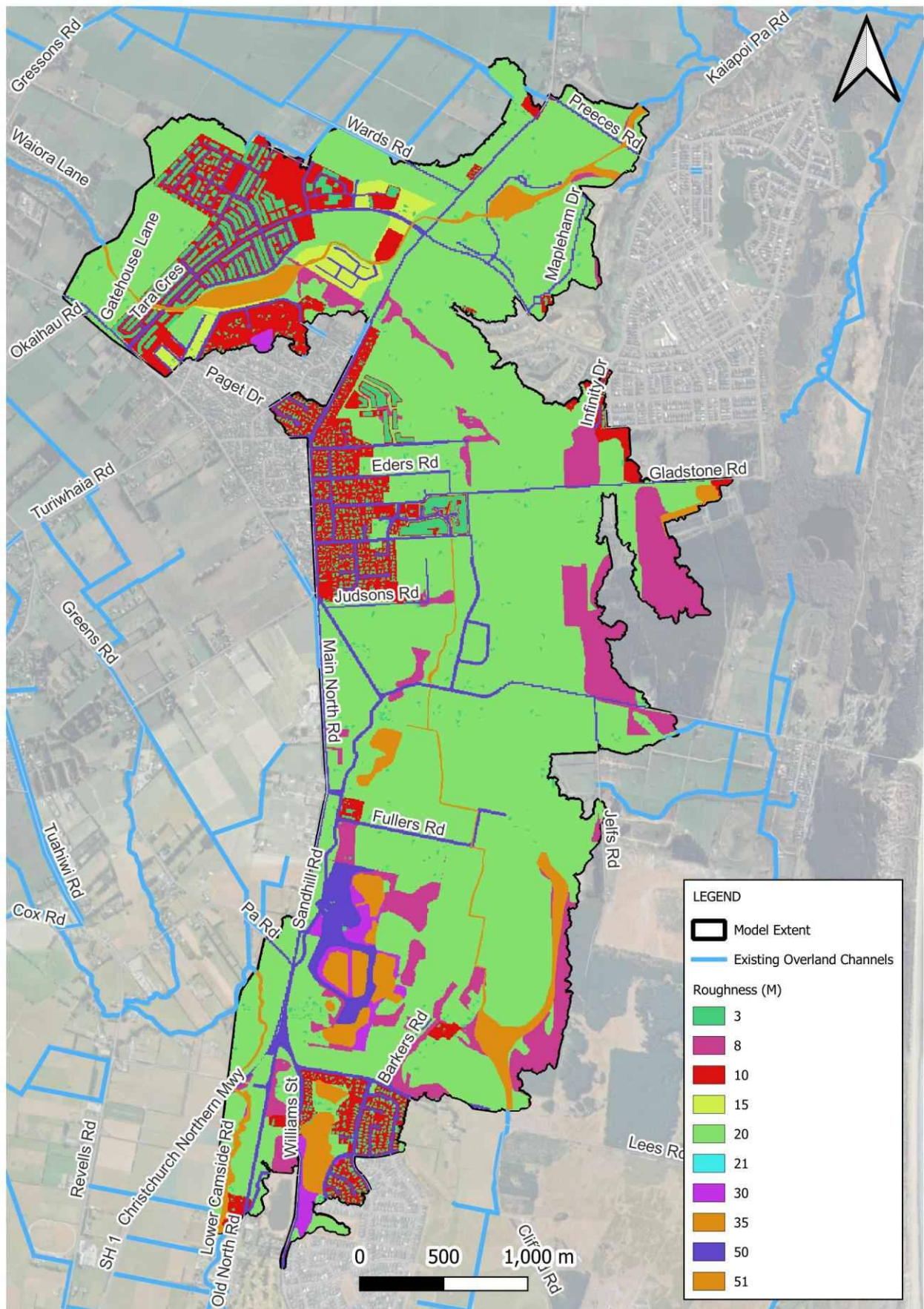
The exponential decay rate (k) parameter was determined based on the Waterways, Wetlands, and Drainage Guidelines (WWDG) of the Christchurch City Council (CCC), as detailed in Appendix A Table 4 decay rate was identified based on the initial infiltration rate extracted from WDC South Ashley Model.

Appendix A Table 4 Horton decay rate extracted from WWDG

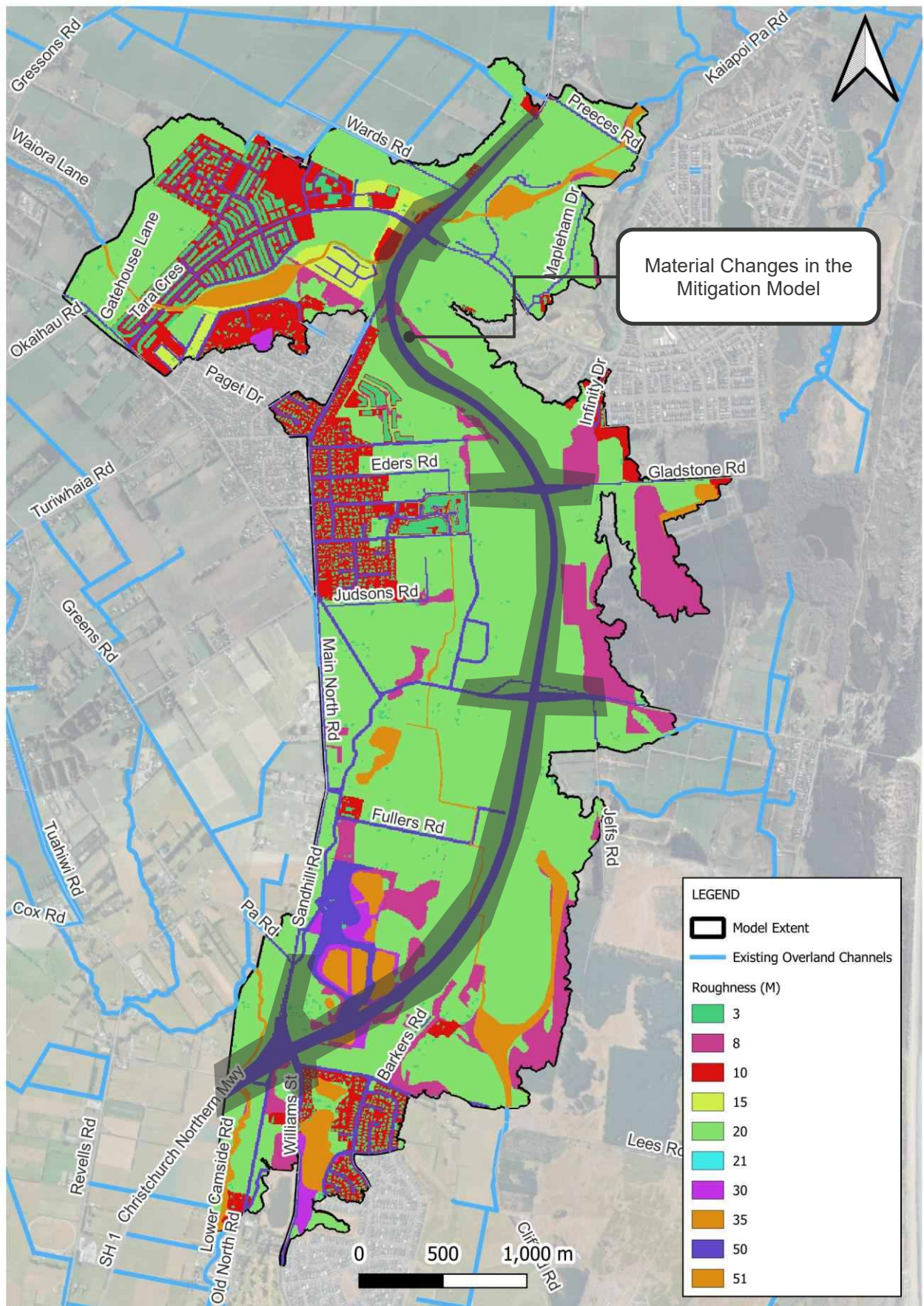
Initial infiltration rate (mm/hr)	Horton decay rate, k	Infiltration type
0 – 5	0.0015	Poor
5 – 10	0.0001	Moderate
10 – 15	0.00003	Free

Surface Roughness

The surface roughness as shown in Appendix A Figure 5 is applied to the B2P Flood Model. The surface roughness information is extracted from WDC South Ashley Model (refer to Appendix A Figure 7) A few changes to the surface roughness have been done in the Mitigation Scenario, to account for the proposed measures, such as the adjustment of the Taranaki Stream near Bob Robertson Drive (refer Appendix A Figure 6).



Appendix A Figure 5 Surface roughness applied in B2P Flood Models (Existing Scenario)



Appendix A Figure 6 Surface roughness applied in B2P Flood Models (Proposed Scenario)

Table 3-2: MIKE 21 roughness

Name_2012	Manning's M	Manning's n
Broadleaved Indigenous Hardwoods	8	0.125
Buildings	3	0.33
Built-up Area (settlement)	10	0.1
Deciduous Hardwoods	8	0.125
Depleted Grassland	50	0.02
Estuarine Open Water	35	0.029
Exotic Forest	8	0.125
Fernland	8	0.125
Flaxland	8	0.125
Forest - Harvested	8	0.125
Gorse and/or Broom	8	0.125
Gravel or Rock	50	0.02
Herbaceous Freshwater Vegetation	10	0.1
Herbaceous Saline Vegetation	10	0.1
High Producing Exotic Grassland	20	0.05
Indigenous Forest	8	0.125
Lake or Pond	35	0.029
Landslide	50	0.02
Low Producing Grassland	10	0.1
Manuka and/or Kanuka	8	0.125
Matagouri or Grey Scrub	10	0.1
Mixed Exotic Shrubland	20	0.05
Orchard, Vineyard or other Perennial Crop	20	0.05
River	35	0.029
Road	50	0.02
Sand or Gravel	50	0.02
Short-rotation Cropland	20	0.05
Sub Alpine Shrubland	20	0.05
Surface Mine or Dump	50	0.02
Tall Tussock Grassland	10	0.1
Transport Infrastructure	10	0.1
Urban Parkland/Open Space	20	0.05

Appendix A Figure 7 Surface roughness table extracted from 'Flood Hazard Models Update' report

Stormwater Network and Culverts

Appendix A Figure 8 shows the existing stormwater culverts along the major rivers and channels around the proposed development. For the B2P flood modelling purposes, urban pipe networks were assumed to be at full capacity during the 100-yr and 200-yr storm events, essentially not providing any storage and conveyance functionalities.

Geometric details of the pipes were extracted from the WDC database and were cross-referenced with the on-going topographical and drainage inventory survey along the designation of the proposed roadway. For areas that were not covered by the survey, details from the WDC database were considered absolute and were used whenever available since a few culverts lacked geometric details, such as the sizes and invert elevations. Details for culverts that did not have any data, both from the WDC database and survey, were assumed intuitively using satellite imagery and topographic information.

Appendix A Table 5 shows the inventory of all the culverts modelled in the B2P Flood Model. It indicates which culvert were either new, maintained, adjusted, or removed in the Mitigation Scenario:

- Maintained – culverts were preserved from the Pre-Development Scenario Model since these were outside of the Proposed Roadway Designation and are not affected by the development.
- Adjusted – geometrical properties and/or alignment changes to the culvert. IDs of the adjusted culverts have been renamed in the Mitigation Model Scenario.
- Removed – decommissioned culverts due to changes of the overland flow path and have been affected by the Proposed Roadway Designation
- New Culvert – completely new culverts as part of the proposed mitigation measures

NOT FOR CONSTRUCTION

The table also details the culverts that are present in each model scenario (Pre-Development, Post-Development, or Mitigation).

Appendix A Table 5 Modelled Culvert Inventory

ID	Size	No. of Barr els	Roughn ess (Mannin g's n)	Source	Dumm y Geome try [Y/N]	Fish Passage [Y/N]	Model Scenario Present	MITIG Scenario Status
C001	DN525	1	0.013	Stormwater_Pipes - 6254-SW007089	N	N	PREDEV, POSTDEV, MITIG	Maintained
C002	DN900	1	0.013	Stormwater_Pipes - 4248-SW022584	N	N	PREDEV, POSTDEV, MITIG	Maintained
C003	DN1950	2	0.013	Survey	N	N	PREDEV, POSTDEV, MITIG	Maintained
C004	DN300	1	0.013	Survey	N	N	PREDEV, POSTDEV, MITIG	Maintained
C005	DN750	1	0.013	Survey	N	N	PREDEV, POSTDEV, MITIG	Maintained
C006	DN525	1	0.013	Survey	N	N	PREDEV, POSTDEV, MITIG	Maintained
15A	Embedded Pipe with Fish Passage	2	0.0203	Survey	N	Y [300mm Embedme nt]	PREDEV, POSTDEV, MITIG	Maintained
C008	DN300	1	0.013	Survey	N	N	PREDEV, POSTDEV, MITIG	Maintained
EC_01	DN750	1	0.013	Survey	N	N	PREDEV, MITIG	Adjusted (MITIG ID: 17A)
R001	4.0m X 1.5m	1	0.013	Survey	N	N	PREDEV, MITIG	Adjusted (MITIG ID: 14A)
R002	2.0m X 1.0m	1	0.013	Stormwater_Pipes - 6636-SW025220	-	N	PREDEV, POSTDEV, MITIG	Maintained
R003	2.4m X 1.2m	1	0.013	Stormwater_Pipes - 4726-SW007089	-	N	PREDEV, POSTDEV, MITIG	Maintained
R004	5.0m X 1.7m	1	0.013	-	-	N	PREDEV, POSTDEV, MITIG	Maintained
R005	4.0m X 1.0m	1	0.013	-	-	N	PREDEV, POSTDEV, MITIG	Maintained
R006	4.5m X 0.8m	1	0.013	-	-	N	PREDEV, POSTDEV, MITIG	Maintained
R007	1.95m X 1.95m	2	0.013	Stormwater_Pipes - 8475-SW031268	-	N	PREDEV, POSTDEV, MITIG	Maintained
R008	1.25m X 0.5m	1	0.013	Survey	N	N	PREDEV, MITIG	Removed
R009	2.2m X 1.2m	1	0.013	Survey	N	N	PREDEV, MITIG	Adjusted (MITIG ID: 16A)
R010	1.7m X 2.5m	1	0.013	Survey	N	N	PREDEV, POSTDEV, MITIG	Maintained

NOT FOR CONSTRUCTION

ID	Size	No. of Barr els	Roughn ess (Mannin g's n)	Source	Dumm y Geome try [Y/N]	Fish Passage [Y/N]	Model Scenario Present	MITIG Scenario Status
03A	DN699	1	0.013	-	-	N	MITIG	New Culvert
04A	DN375	1	0.013	-	-	N	MITIG	New Culvert
05A	DN600	1	0.013	-	-	N	MITIG	New Culvert
05B	6.0m x 2.2m	1	0.017	-	-	Y [300mm Embedme nt]	MITIG	New Culvert
06A	DN600	1	0.013	-	-	N	MITIG	New Culvert
07A	DN900	2	0.013	-	-	N	MITIG	New Culvert
08A	1.5m x 1m	2	0.013	-	-	N	MITIG	New Culvert
09A	DN600	1	0.013	-	-	N	MITIG	New Culvert
10A	DN900	2	0.013	-	-	N	MITIG	New Culvert
11A	DN600	1	0.013	-	-	N	MITIG	New Culvert
12A	DN600	1	0.013	-	-	N	MITIG	New Culvert
13A	DN600	1	0.013	-	-	N	MITIG	New Culvert
14A	4.0m x 1.5m	1	0.026	-	-	Y [300mm Embedme nt]	MITIG	Adjusted (PREDEV ID: R001)
15B	DN750	1	0.013	-	-	N	MITIG	New Culvert
16A	2.7m x 1.2m	1	0.0233	-	-	Y [300mm Embedme nt]	MITIG	Adjusted (PREDEV ID: R009)
17A	DN1200	1	0.013	-	-	N	MITIG	Adjusted (PREDEV ID: EC_01)



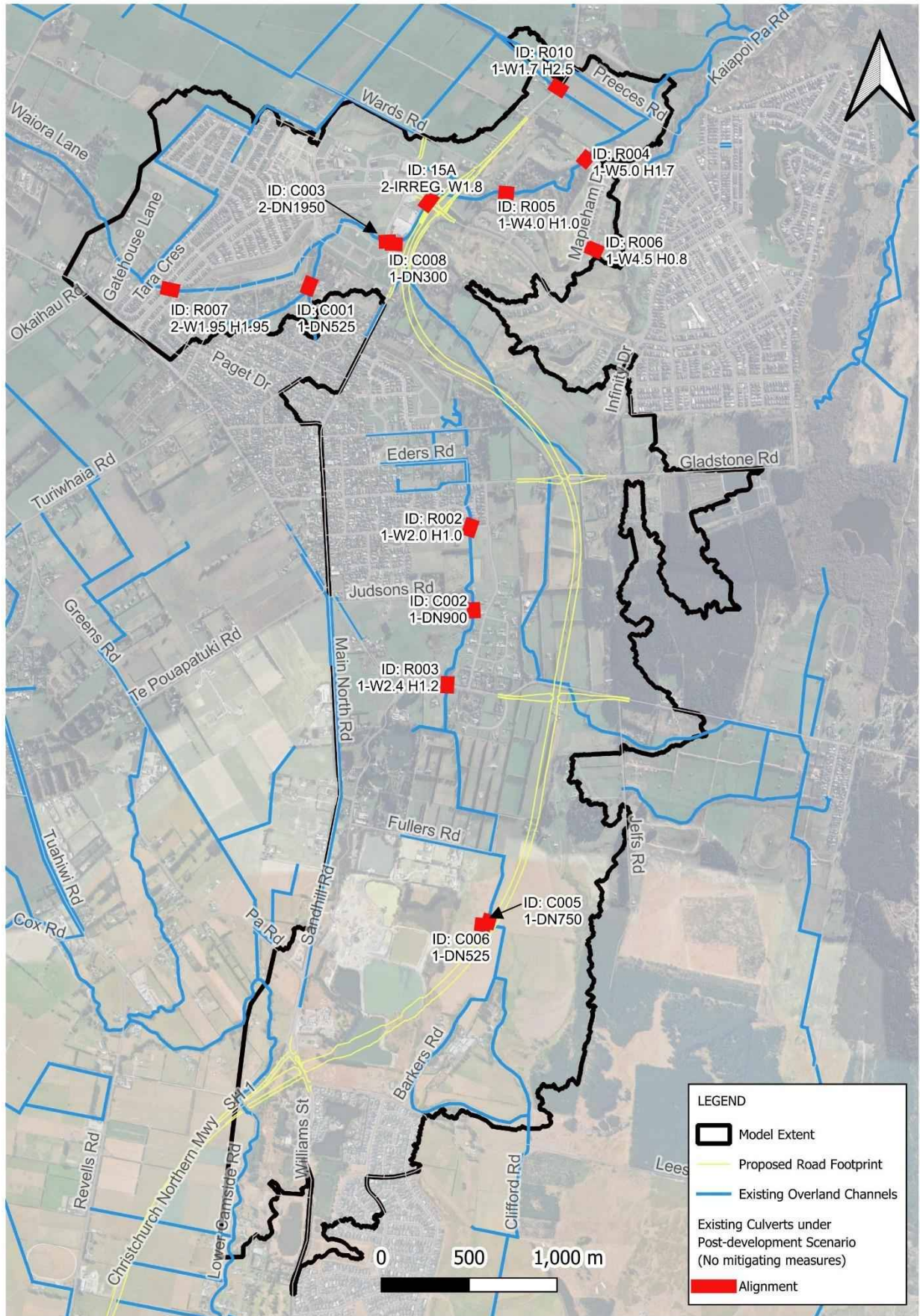
Flood Modelling Scenarios

Appendix A Table 6 summarises the baseline and design scenarios that were modelled in the study for the flood impact assessment of the road development. This includes the details and corresponding changes in each modelled scenario, as well as figures for reference.

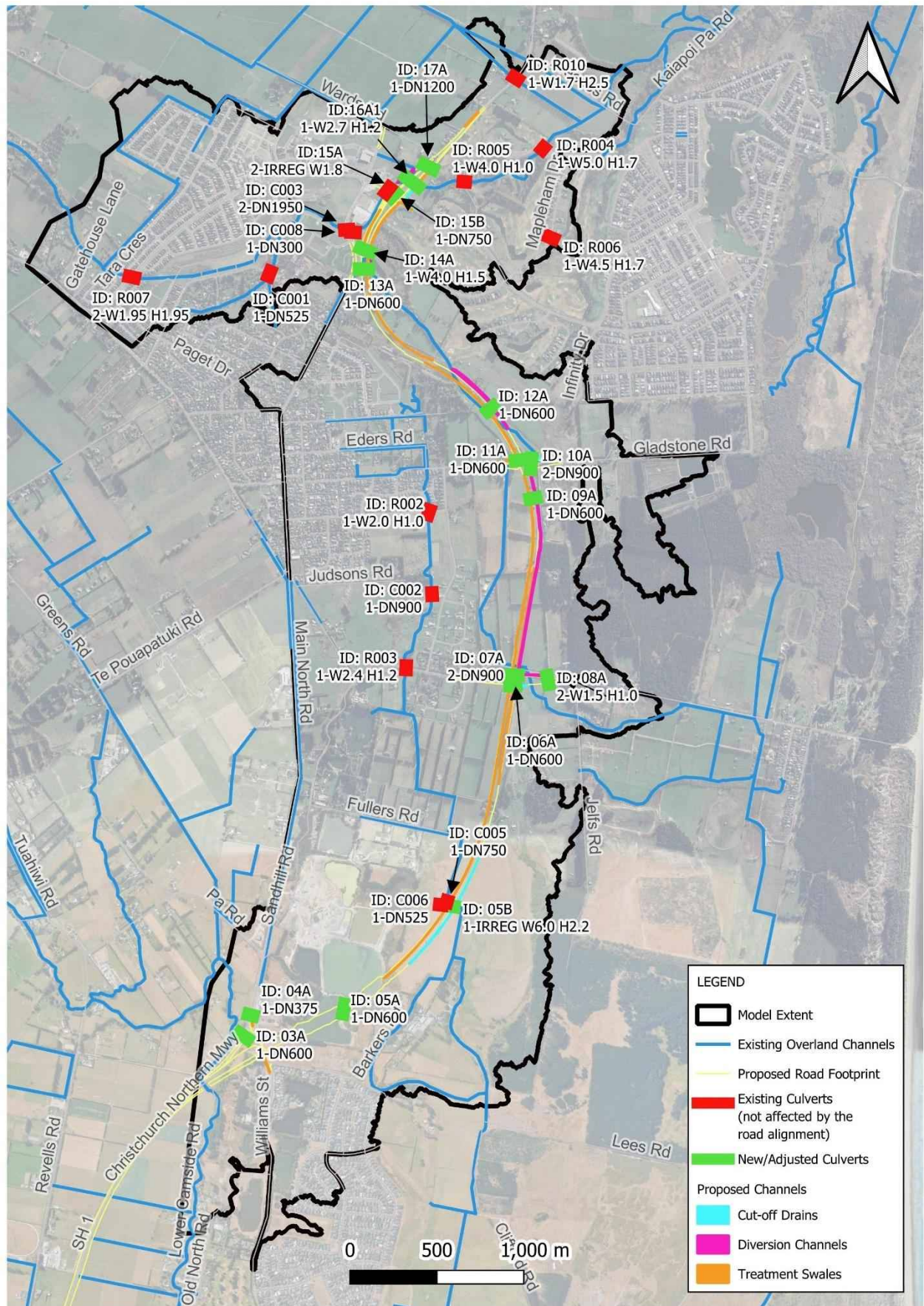
Appendix A Table 6 List of modelled scenarios

Item	Figure	Modelled Scenarios
1	Figure B-1	PREDEV: Existing/pre-development scenario - Baseline model for flood impact assessment - Refer to Table A-11 for B2P Flood Model setup
2	Figure B-2	POSTDEV: Design/post-development scenario with proposed B2P road alignment included - PREDEV with the following changes: - included design surface of proposed B2P road development - modifications on surface roughness to represent proposed B2P road alignment.
3	Figure B-3	MITIG: Design/post-development scenario with mitigation solution implemented - POSTDEV with the following changes: - Propose additional culverts - Propose additional flow path channels.





Appendix B Figure 2 Model Setup for the post-development scenario



Appendix B Figure 3 Model Setup for the post-development with mitigation scenario

Appendix C

Woodend Flood Model Results

Item	Figure	Modelled Scenarios
1	Figure C-1	Pre-development Model (PREDEV) Flood Depth - 100yr ARI Storm
2	Figure C-2	Pre-development Model (PREDEV) Flood Level - 100yr ARI Storm
3	Figure C-3	Pre-development Model (PREDEV) Flood Depth – 200yr ARI Storm
4	Figure C-4	Pre-development Model (PREDEV) Flood Level – 200yr ARI Storm
5	Figure C-5	Post-development Model (POSTDEV) Flood Depth - 100yr ARI Storm
6	Figure C-6	Post-development Model (POSTDEV) Flood Level - 100yr ARI Storm
7	Figure C-7	Post-development Model (POSTDEV) Flood Depth – 200yr ARI Storm
8	Figure C-8	Post-development Model (POSTDEV) Flood Level – 200yr ARI Storm
9	Figure C-9	Flood Level Difference – Post-development (POSTDEV) minus Pre-development (PREDEV) - 100yr ARI Storm
10	Figure C-10	Flood Level Difference – Post-development (POSTDEV) minus Pre-development (PREDEV) – 200yr ARI Storm
11	Figure C-11	Post-development Model (MITIG) Flood Depth – 100yr ARI Storm
12	Figure C-12	Post-development Model (MITIG) Flood Level – 100yr ARI Storm
13	Figure C-13	Post-development Model (MITIG) Flood Depth – 200yr ARI Storm
14	Figure C-14	Post-development Model (MITIG) Flood Level – 200yr ARI Storm
15	Figure C-15	Flood Level Difference – Post-development (MITIG) minus Pre-development (PREDEV) – 100yr ARI Storm
16	Figure C-16	Flood Level Difference – Post-development (MITIG) minus Pre-development (PREDEV – 200yr ARI Storm

Document prepared by

Aurecon New Zealand Limited

Level 2, Iwikau Building
93 Cambridge Terrace
Christchurch 8013
New Zealand

T +64 3 366 0821

F +64 3 379 6955

E christchurch@aurecongroup.com

W aurecongroup.com

Document prepared by

Aurecon New Zealand Limited

Level 2, Iwikau Building
93 Cambridge Terrace
Christchurch 8013
New Zealand

T +64 3 366 0821

F +64 3 379 6955

E christchurch@aurecongroup.com

W aurecongroup.com