Summary Report

Allansford Flood Investigation and SWMS

Warrnambool City Council

7 May 2025

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Cover Photo: Sandstone cliff bank of Hopkins River, Allansford

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ACKNOWLEDGEMENT OF COUNTRY

The Board and employees of Water Technology acknowledge and respect the Aboriginal and Torres Strait Islander Peoples as the Traditional Custodians of Country throughout Australia. We specifically acknowledge the Traditional Custodians of the land on which our offices reside and where we undertake our work.

We respect the knowledge, skills and lived experiences of Aboriginal and Torres Strait Islander Peoples, who we continue to learn from and collaborate with. We also extend our respect to all First Nations Peoples, their cultures and to their Elders, past and present.

Artwork by Maurice Goolagong 2023. This piece was commissioned by Water Technology and visualises the important connections we have to water, and the cultural significance of journeys taken by traditional custodians of our land to meeting places, where communities connect with each other around waterways.

The symbolism in the artwork includes:

- *Seven circles representing each of the States and Territories in Australia where we do our work*
- *Blue dots between each circle representing the waterways that connect us*
- *The animals that rely on healthy waterways for their home*
- *Black and white dots representing all the different communities that we visit in our work*
- *Hands that are for the people we help on our journey*

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1 INTRODUCTION

1.1 Overview

Water Technology has been commissioned by Warrnambool City Council (Council) to undertake the Allansford Flood Investigation and Stormwater Management Strategy (FI & SWMS). The investigation area covers the Hopkins River and local stormwater catchment in the township of Allansford, located east of Warrnambool, as shown in Figure 1-1 below. Key objectives and study outputs are outlined in section 1.2 below.

The Allansford Strategic Framework Plan identifies the need to balance growth within the township against flooding and stormwater constraints. The Strategic Framework Plan was adopted by Council on 3rd May 2021 with the implementation of the Strategic Framework Plan to include a planning scheme amendment to incorporate the Plan and its recommendations into the scheme. The outputs of the Flood Investigation and Stormwater Management Strategy will likely form part of the planning scheme amendment. In addition, Allansford is identified in the Glenelg Hopkins Regional Floodplain Management Strategy (RFMS) as a priority location for which an improvement in flood information is required. The preferred management action for Allansford in the RFMS is to *identify flood prone areas through structure plans for Logans Beach and Allansford and introduce planning controls*.

The Allansford Flood Investigation and SWMS defines the 1% AEP inundation extents in Allansford, considering both riverine flooding and stormwater inundation under present day and projected climate change scenarios.

In addition to modelling current conditions inundation, the development of the Stormwater Management Strategy considers an ultimate development scenario including preferred options for stormwater conveyance and treatment, both in the context of future development and climate change. The strategy includes cost estimates of preferred options and will form part of the overall strategic planning for Allansford.

This report is one of a series documenting the outcomes of the Allansford Flood Investigation and Stormwater Management Strategy Study. Each reporting stage is shown below:

- R01 - Data Review and Validation
- R02 – Hydrologic and Hydraulic Modelling Report
- R03 – Stormwater Management Strategy
- **R04 – Summary Report – This Report**

R01 presents the relevant data as collated, assessed and validated prior to use in the study. Key data inputs such as topographic LiDAR datasets were verified as part of this report.

R02 details the riverine hydrologic and hydraulic modelling undertaken as part of the study. The report describes the model builds, calibration process, and key modelling outputs for the existing conditions and projected flooding conditions under climate change scenarios.

R03 presents modelling and results of rain on grid modelling of the local catchment, considering a fully developed scenario for Allansford. Stormwater network and water quality treatment infrastructure to effectively convey and treat flows originating within and upstream of the township will be proposed

This report provides a summary of the study, highlighting key findings and recommendations. It is intended as a less technical overview, with more detailed information available in the accompanying study reports. The hydrology and hydraulics sections of this report summarise highly technical modelling information and may be of limited relevance to non-technical readers.

1.2 Study Objectives and Outputs

The key objectives of the study as provided in the project brief are:

Objective 1

Update knowledge and data around impacts of climate change, specifically an increase in frequency of extreme rainfall events to enable more effective planning for a worsening flood risk profile for Allansford.

Objective 2

Amendment of flood related land use and development controls in the Warrnambool planning scheme.

Objective 3

Provision of flood mapping. The new mapping outputs must be of sufficient resolution to enable flood risk management planning at the building envelope scale (i.e. be of suitable resolution and rigour for a planning scheme amendment).

Objective 4

Preparation of a Stormwater Management Strategy for Allansford, which addresses, at a level of detail appropriate to the objectives of the Framework Plan, the arrangements for collecting, conveying, storing, and discharging stormwater from the existing and planned development and achieving water quality improvements consistent with established WSUD principles.

The key outputs which will allow the study to achieve the above objectives are:

- Detailed reporting summarising the available data, community consultation, site inspection findings and survey undertaken.
- Updated flood frequency analysis for the Hopkins River at appropriate locations.
- Development and calibration of hydrologic and hydraulic models.
- Design event modelling of the 20%, 10%, 1% AEP and Probable Maximum Flood (PMF) events in accordance with Australian Rainfall and Runoff 2019.
- Assessment and modelling of the impact of increased rainfall intensity associated with climate change under Representative Concentration Pathway (RCP) 8.5 to the years 2050 and 2100.*
- Development of the Allansford Stormwater Management Strategy, considering conveyance and treatment of stormwater generated in the local catchment and cost estimation of preferred mitigation/development options.

* Note ARR version v4.2 was released during the project, which required this output to be altered.

1.3 Study Area

Allansford is a relatively small but growing rural settlement located approximately 10km east of Warrnambool. The town is located on the southern side of the Princes Highway. While the area has been occupied by traditional owners for thousands of years, the township was established by European settlement in the 1800s.

The Hopkins River runs through Allansford and has an upstream catchment area of approximately 8,700 km^2, extending from Ararat in the north, Ballarat in the northeast, through to Warrnambool in the south. The main tributaries to the Hopkins River are Fiery Creek and Mt Emu Creek. The river is recognised for its ecological, community and cultural value. Hopkins Falls, approximately 6km north of Allansford, is a popular lookout spot particularly when the river is in flood and is the closest stream gauging station to Allansford.

Current understanding of flood risk for Allansford is largely based on the observed impacts of the 2011 flood event on the Hopkins River. No formal flood study has been undertaken and the intelligence gathered during the 2011 event has guided responses to more recent events such as those of October 2022. Riverine floodwater backing into the stormwater system during flood events has the potential to cause inundation particularly if local rainfall is occurring simultaneously. During the 2011 and 2022 flood events, the main stormwater drain was blocked with sandbags and other materials and pumps set up to allow local stormwater drainage to the river.

One of the objectives of the project is to update the Warrnambool Planning Scheme within the township of Allansford as part of delivering the Allansford Strategic Framework Plan. Thus, the township boundary of Allansford is the focus area of this study. Flood models developed for the study extent beyond the focus area.

Stormwater drainage of the local catchment relies almost exclusively on a single outfall, known as the Main Open Drain and Tooram Road North outfall (the open drain flows into the outfall). Another historic outfall, referred to as the Tooram Road South outfall, remains functional however catchment alterations such as earthworks over many years have resulted in the south outfall draining a very small portion of the town.

The study area is shown in Figure 1-1 below. The local stormwater catchment and key features therein are shown in Figure 1-2 below.

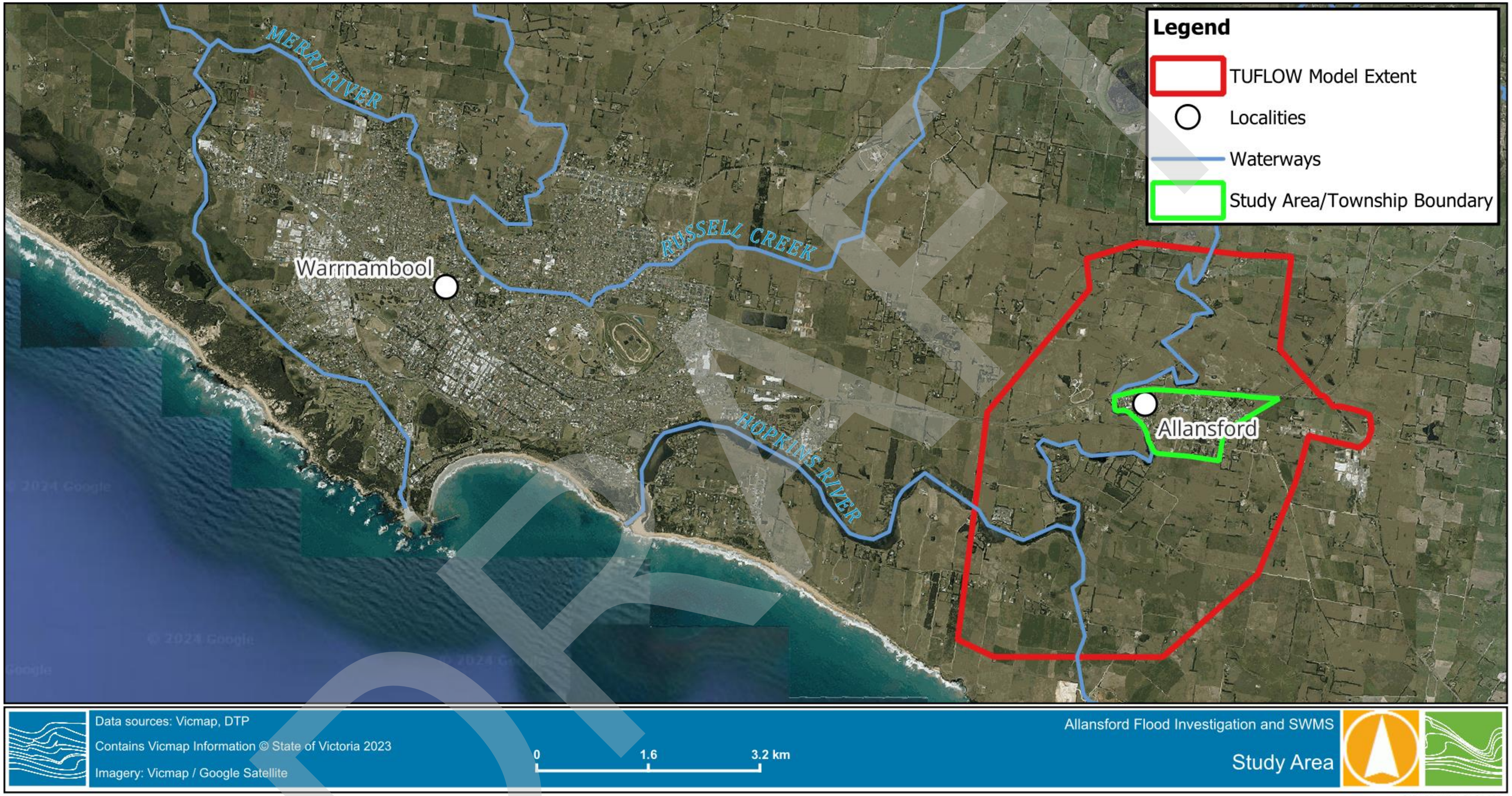

Figure 1-1 Study Area

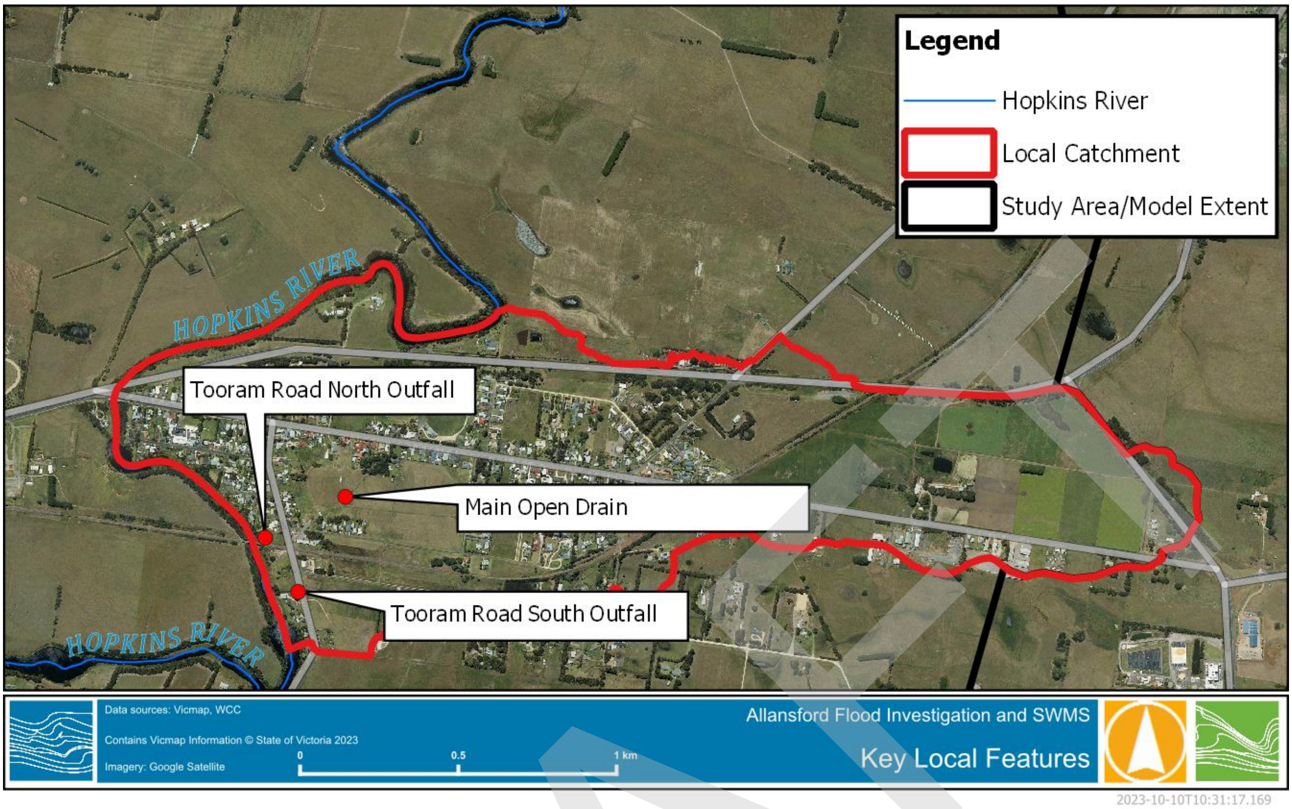

Figure 1-2 Local Catchment and Key Features

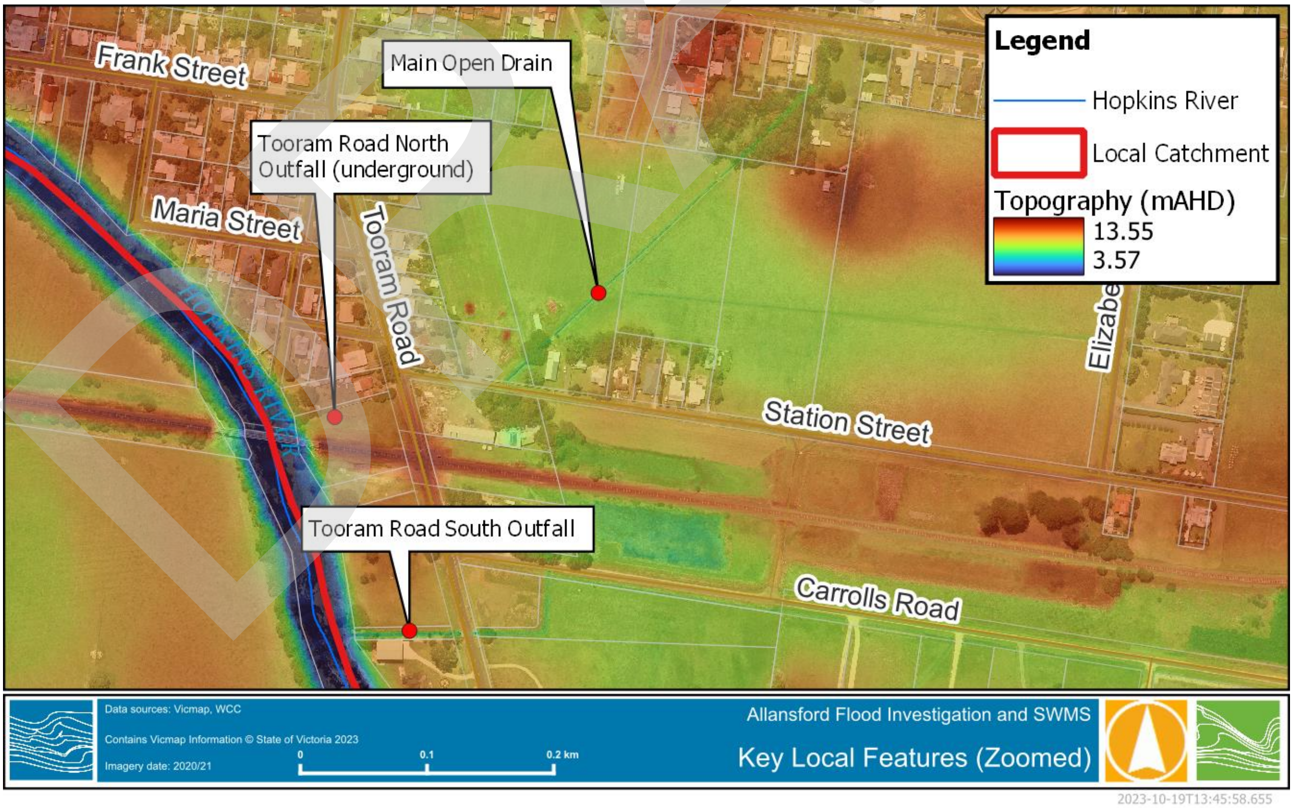

Figure 1-3 Key Features – close view with topography

2 DATA REVIEW

The initial stage of the project involved collation and reviewing available data related to flooding in Allansford. Data was collected, collated, and reviewed through both desktop analysis and on-site visits, supplemented by community consultation.

2.1 Desktop Data Collation and Review

Data that was collated and reviewed included the following:

- Previous flood studies and drainage investigations (Table 2-1), including,
  - Two previous studies related to stormwater pipe sizing requirements (Cardno and Hyder reports)
  - The GHCMA Hydrological Assessment, which included a flood frequency analysis on the Hopkins River at Hopkins Falls gauge

Table 2-1 Previous Studies

Study Name	Author	Year
Allansford Outfall Drain Options Evaluation Report	Hyder Consulting	2002
Allansford Township Drainage Requirements	Cardno Lawson Treloar	2008
Hydrological Assessment Report	GHCMA	2010

- Historic flood impact information
  - Written accounts of seven flood events from 1870 to present were uncovered
  - Only two previous events had recorded flood levels, although the single recorded level from 1946 was considered of low reliability
  - Several surveyed flood marks from the January 2011 event, along with aerial and ground photography were compiled
- Waterway and waterbody mapping, including significant catchment storages and lakes
- Historic daily and subdaily rainfall data, and streamflow data
- Riverine and stormwater hydraulic structure information such as bridges, culverts, pits and pipes
  - Council provided GIS pipe information which was supplemented with survey where required
- Topographic Data
  - The main topographic dataset utilised in the project was captured in 2023 as part of the Digital Twin Victoria project. This was compared against other previously available datasets and validated against feature survey captured for the project.
- Allansford Strategic Framework Plan 2021

The collated data was assessed for its suitability for use. All data that was utilised in the study was deemed suitable for use and verified against other datasets where possible.

2.2 Site Visits and Community Consultation

In addition, site visits were conducted in August and September 2023 to visit known stormwater hotspots, inspect drainage infrastructure and hydraulic structures, and consider potential flood mitigation asset placement. Numerous site visits were undertaken over the course of the project as and when required.

The first of two community consultation sessions occurred in Allansford on 29 August 2023. The session was attended by members of the community along with representatives from Warrnambool City Council, Glenelg Hopkins Catchment Management Authority and Water Technology. The purpose of the session was threefold:

- To inform the community of the study.
- To gather any flood intelligence or other information the community may have, such as historical impacts, extents, flood heights etc.
- To engage the community on possible mitigation options or other solutions to ongoing inundation issues in the town.

The session attracted excellent engagement from the community with 16 households/properties in attendance. Those in attendance were informed about the study and had the opportunity to put forward their ideas for flood mitigation options to alleviate flooding in the town. A number of options put forward by the community were assessed as part of the Stormwater Management Strategy development from a pre-feasibility perspective, and some options were tested in the hydraulic models as a strategy option.

KEY OPTIONS FROM COMMUNITY

WRITE ABOUT SECOND SESSION

3 HYDROLOGY AND HYDRAULIC MODELLING

In order to define flood behaviour in the Hopkins River floodplain and local Allansford stormwater catchment a detailed hydrological assessment was undertaken, accompanied by detailed hydraulic modelling of the floodplain and township. The key modelling software packages used were RORB (hydrology) and TUFLOW (hydraulics). The hydrologic assessment also included a flood frequency analysis of the Hopkins River at Hopkins Falls (236209) gauge, located some 11.5 km upstream of Allansford.

This section of the report is a technical summary of the modelling that informs the project outputs.

3.1 Hydrology

The following sections summarise the hydrologic analysis and modelling. More detailed information is available in R02 – Modelling Report.

3.1.1 General Methodology

The hydrologic component of the Allansford flood study followed the following steps:

- Completion of a flood frequency analysis (FFA) on the Hopkins River at Hopkins Falls gauge (gauge number 236209)
- Development of a catchment wide RORB rainfall runoff model
- Calibration of the RORB model to the January 2011 flood event and validation to the October 2022 event
- Adopting routing parameters based on calibration for design event modelling
- Design event modelling and reconciliation of design event peak flows at the Hopkins Falls gauge with expected quantiles as informed by the FFA

3.1.2 Hopkins River at Hopkins Falls Gauge

During the course of the project it became apparent that a detailed review of the Hopkins River at Hopkins Falls gauge rating table was required. A thorough investigation into the gauge was conducted, with the following steps undertaken:

1. Issues with gauge rating identified.
2. Consultation with relevant authorities (DEECA, Warrnambool City Council, ALS Hydrographic Services)
3. ALS completed a desktop assessment of the Hopkins River gauge network which resulted in a significant reduction in estimated flows for the 2011 event and the production of a new rating table (40.01) for the gauge.
   a. 2011 peak flow reduced from 813 to 675 m^3/s
4. The new flows were tested within the TUFLOW hydraulic model and found to remain too high to match observations and recordings from the event.
5. A TUFLOW hydraulic model of the gauge site was developed to create a modelled rating curve, with the model parameters (surface roughness) adjusted to provide agreement with observed gauging.
6. The modelled rating curve was adopted for water levels above 2.6 metres. Below this, the existing rating curve/published flows were adopted as the difference between the model results and existing rating was minimal.

The resultant rating curve further reduced peak flow in the 2011 event to 557 m^3/s. A comparison of the original rating curve (rating 40.00), the ALS revised rating curve (40.01) and the modelled rating curve which was adopted for the project hydrology is shown in Figure 3-1. While adoption of the modelled rating curve results in a significant reduction in peak flows for large events (e.g. a >30% reduction for 2011 peak flow), the decision to adopt it was based on the available evidence and is justified.

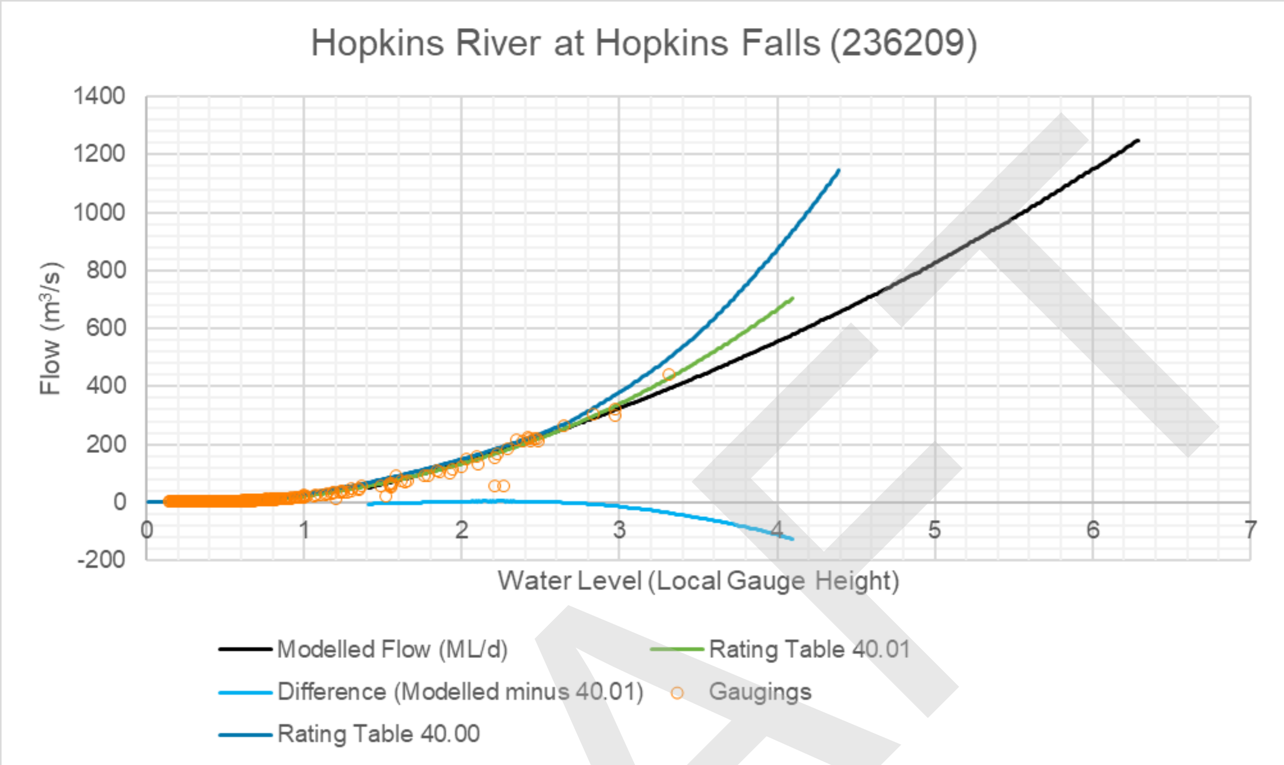

Figure 3-1 Hopkins Falls gauge rating comparisons: 40.00, 40.01 and modelled rating curves

3.1.3 Flood Frequency Analysis

A flood frequency analysis (FFA) was performed on the series of annual maximum flood flow from the Hopkins River at Hopkins Falls gauge. The FFA was conducted in accordance with the methods and guidelines in ARR Book 3 Chapter 2. The FLIKE software package was utilised.

The annual series included data from 1955 to 2023, the entire period of record for the gauge at the time of the analysis. There was no suitable available record available on which to extend the gauge record, thus the record was used as is. The record was adjusted in line with the rating curve review, i.e. the annual series was downloaded as both water level and stream flow, with heights above 2.6 metres at the gauge converted to new stream flow based on the adopted rating curve.

The analysis adopted the Log Pearson III distribution after also considering the Generalised Extreme Value, Log Natural and Generalised Pareto distributions. The resultant flood quantile estimates are shown in Figure 3-2 and Table 3-1 below, with the estimated magnitude (in annual exceedance probability, AEP) of the 5 largest recorded flood events shown in Table 3-2.

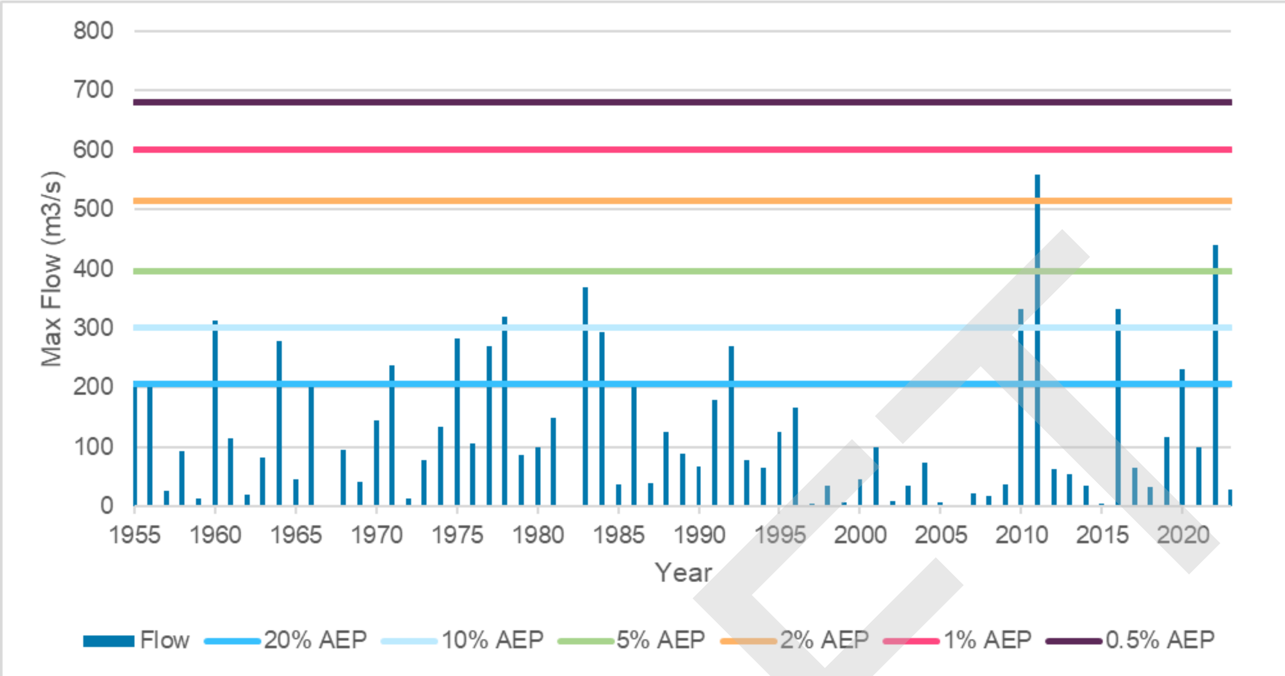

Figure 3-2 Annual Flow Series and Expected Quantiles from FFA

Table 3-1 Adopted FFA Quantiles for Hopkins River at Hopkins Falls (236209)

AEP	Peak Design Flow (m³/s)			
	Log Pearson III	LPIII 5% Limit	LPIII 95% Limit	GHCMA FFA (2010)
20%	205.6	166.4	257.5	191.1
10%	301.1	244.5	375.1	278.5
5%	395.4	318.0	507.3	373.4
2%	515.0	403.7	735.3	510.8
1%	600.1	457.0	931.2	623.9

The 5 largest flows since records began have been compared against the design FFA distribution to assign an approximate AEP (as defined by the FFA) to each event as shown in Table 3-2 below.

Table 3-2 Approximate AEP of 5 largest recorded flows

Year	Flow (m³/s)	Approximate AEP
2011	559.5	Between 1% and 2%
2022	441.0	Between 2% and 5%
1983	368.1	Between 5% and 10%
2016	331.9	Between 5% and 10%
2010	331.2	Between 5% and 10%

3.1.4 Hydrologic (RORB) Model Build

A hydrologic model of the Hopkins River catchment was developed. Modelling utilised the RORB rainfall runoff modelling software package to determine flow hydrographs at gauged locations within the catchment and at the upstream extent of the hydraulic model. The model utilised the Vicmap 10m resolution Digital Elevation Model (DEM) to inform topography used to delineate sub catchments and flow paths within the catchment.

The RORB model adopted type 1 (natural) reaches throughout the catchment. Three storages were included in the model as a special storage: Lake Burrumbeet, Lake Goldsmith and Lake Bolac. The final model adopted a single set of flood routing parameters (Kc and m) across the entire catchment, however calibration runs tested the adoption of varying routing parameters across each gauge. Ultimately the increased complexity of the varying parameter model was deemed to not increase confidence in the final model results in the context of this project based on the available data, thus the single parameter model was adopted. Due to this, care should be taken with results if the model is appropriated to produce flows for areas other than Allansford.

Fraction imperviousness (FI) of the model subareas was adopted in line with industry standard values (including Melbourne Water Technical Specification for Flood Mapping Projects 2024), based on land use zoning and inspection of aerial imagery. The modelling did not adopt an Effective Impervious Area/Indirectly Connected Area (EIA/ICA) approach as the vast majority of the catchment is not urban. With the exception of Ballarat and Beaufort, the catchment can generally be split into 3 land use types as follows:

- Farming zone makes the clear majority of the catchment, assigned FI = 0.025 (2.5%)
- Some forested areas in the upper catchment were assigned FI = 0.01 (1%)
- Waterbodies were assigned FI = 1 (100%)

A map showing the applied FI values to the catchment is provided in Figure 3-4 below.

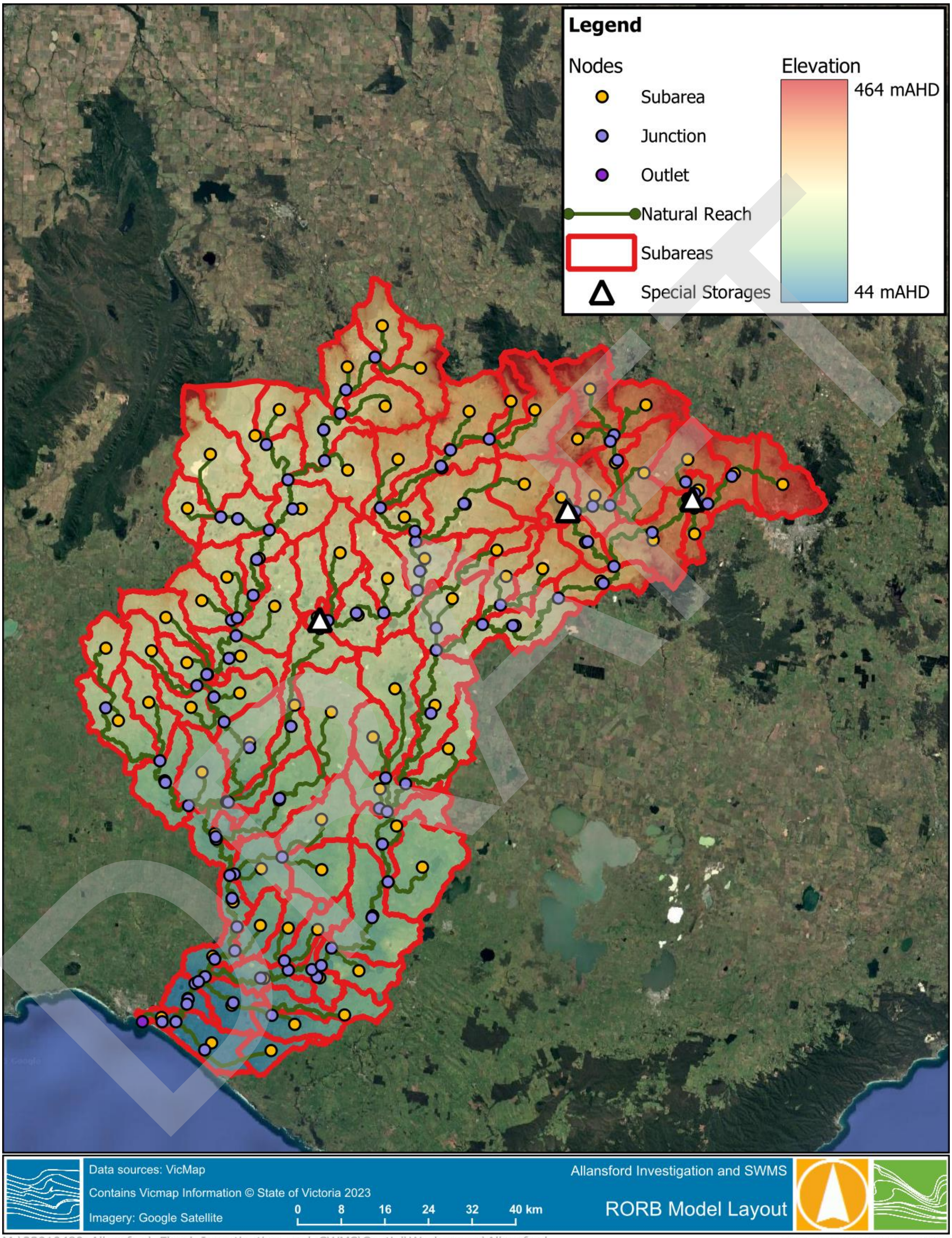

Figure 3-3 RORB model layout

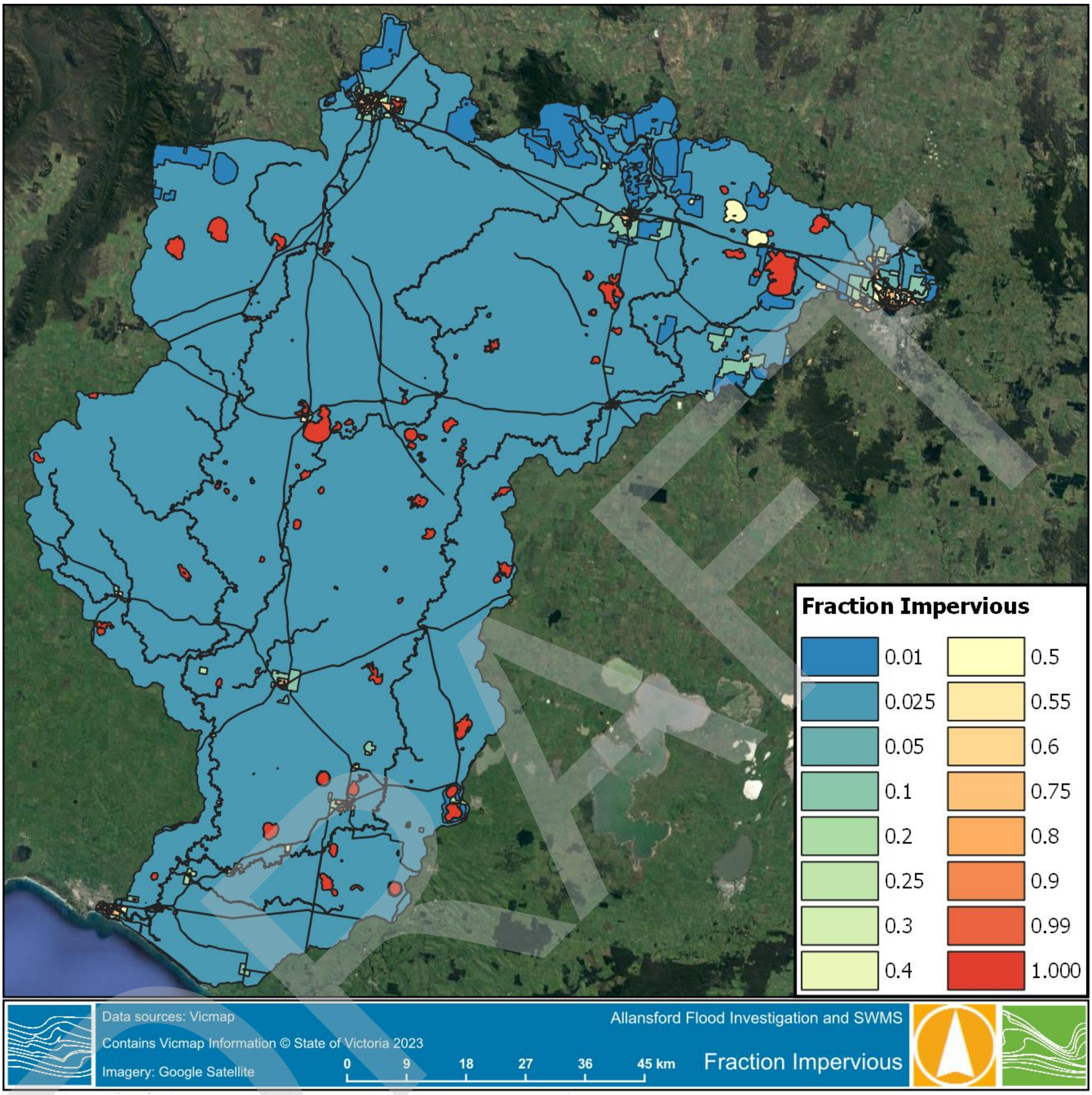

Figure 3-4 Adopted Fraction Impervious

3.1.5 RORB Calibration

The RORB model was calibrated to the January 2011 flood event and validated to the October 2022 flood event. Initial project scoping had included a validation scenario of the 1946 flood event however a lack of rainfall data meant that rainfall patterns would have to be largely assumed, reducing the confidence in any validation results. The October 2022 event was used as a validation event in place of 1946.

The calibration/validation results are shown in Figure 3-5 and Figure 3-6 below, with the parameters adopted for each event shown in Table 3-3 and Table 3-4. The modelling adopted the regional Kc/Dav ratio of 1.25, taken from Pearse et. al. (2002)[1] which has been found to represent many Victorian catchments well. While a reasonable agreement between the modelled and recorded flows was reached for the 2011 event, the 2022 event was poorly reproduced with the available data. Some experimentation to achieve a better fit for the 2022 event was largely unsuccessful and it was concluded that the recorded rainfall data may have missed key parts of the storm responsible for the peak. In the absence of alternative rainfall data, the agreement between modelled and recorded flows could not be significantly improved.

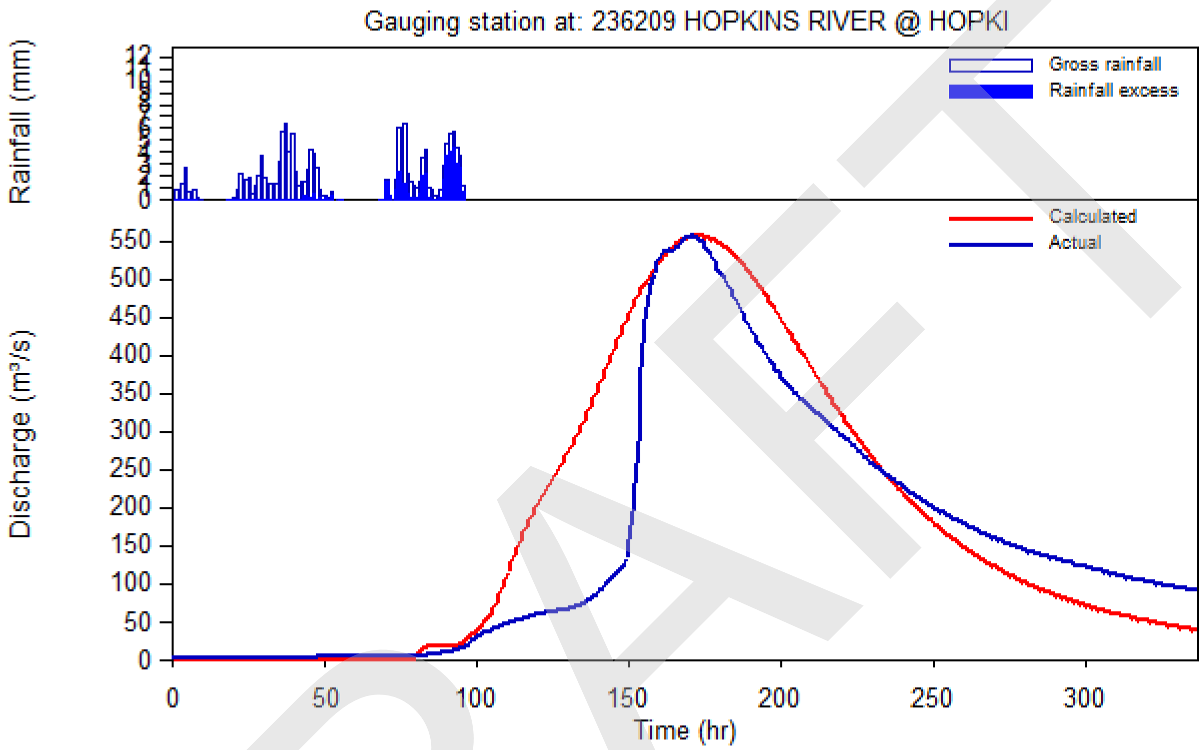

Figure 3-5 January 2011 RORB Calibration at Hopkins Falls

Table 3-3 January 2011 RORB Calibration A parameters

Interstation Area/Gauge	Kc	m	IL (mm)	CL (mm/hr)
236202 Hopkins River at Wickliffe	252.88	0.8	100.0	2.50
236204 Fiery Creek at Streatham			85.0	1.30
236210 Hopkins River at Framlingham			95.0	1.00
236203 Mt Emu Creek at Skipton			85.0	0.90
236216 Mt Emu Creek at Taroon			100.0	1.60
236209 Hopkins River at Hopkins Falls			80.0	2.00
Catchment Outlet			80.0	2.00

[1] Pearse, M., P. Jordan and Y. Collins (2002). A simple method for estimating RORB model parameters for ungauged rural catchments. Water Challenge: Balancing the Risks: Hydrology and Water Resources Symposium 2002., Institution of Engineers, Australia: Barton, A.C.T.: 128-134.

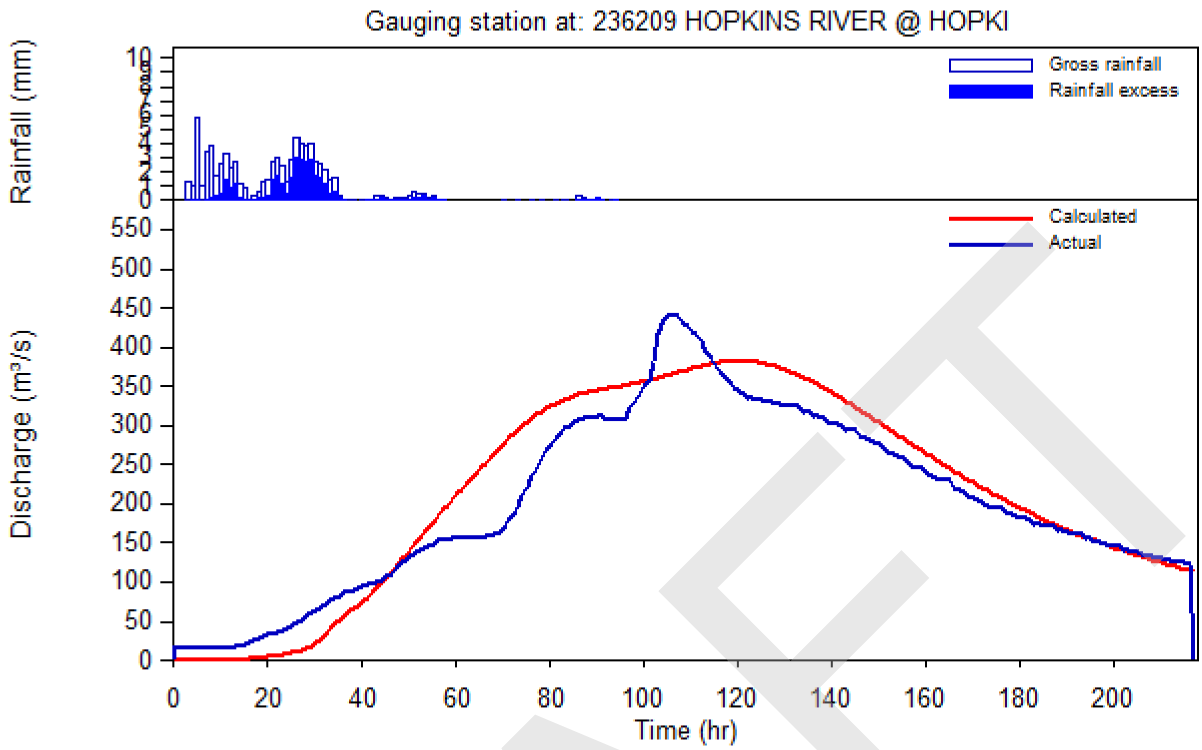

Figure 3-6 October 2022 RORB Validation at Hopkins Falls

Table 3-4 October 2022 RORB validation parameters

Interstation Area/Gauge	Kc	m	IL (mm)	CL (mm/hr)
236202 Hopkins River at Wickliffe	252.88	0.8	40.0	1.50
236204 Fiery Creek at Streatham			40.0	0.50
236210 Hopkins River at Framlingham			20.0	1.50
236203 Mt Emu Creek at Skipton			15.0	0.00
236216 Mt Emu Creek at Taroon			20.0	1.50
236209 Hopkins River at Hopkins Falls			20.0	2.00
Catchment Outlet			20.0	2.00

While the resultant hydrographs are an imperfect match against the recorded hydrographs, it is important to consider the purpose of the RORB model in the context of the study. The purpose of the RORB model is to determine the shape of the design hydrographs only; peak flows for design events (other than the PMF) are taken from the flood frequency analysis. Design modelling adopted a regionally derived routing parameter set which represented the calibration event well. With that in mind, the adopted parameters are considered fit for purpose.

3.1.6 Design Hydrology

Design rainfall depths were obtained from the Bureau of Meteorology Design Rainfall Data System[2]. Rainfall depths were obtained in ascii grid format to enable spatial variation of rainfall to be considered in line with the recommendations of ARR2019 for catchments exceeding 20km^2. Areal reduction factor (ARF) parameters and

[2] http://www.bom.gov.au/water/designRainfalls/revised-ifd/

temporal patterns were obtained from the ARR Datahub[3]. The ARF was calculated based on the full catchment area using RORB's internal ARF calculator which adopts the method specified in Book 2, Chapter 4 of ARR v4.2.

Temporal patterns for the catchment were adopted from the Southern Slopes (Vic) region. Due to the size of the catchment, areal temporal patterns are recommended for use by ARR2019. Areal temporal patterns are available for storms 12 hours in duration and longer. Given the critical duration was shown to be longer than 12 hours, point temporal patterns were not considered for design rainfall distribution. Spatial variation of rainfall was considered in line with the method outlined in Book 2, Chapter 6 of ARR v4.2. Pre-burst rainfall was considered by subtracting the pre-burst depth from initial loss. Median values of pre-burst depth were used. Embedded burst filtering was utilised within RORB, which ensures that sub-burst rainfalls of a certain AEP do not exceed rarer AEPs (e.g. a 1% AEP 1 hour storm embedded within a 2% AEP 6 hour storm).

In line with ARR v4.2, Book 1 Chapter 6, the study has considered the impact of climate change on rainfall intensity and losses. A range of timeframes and projected climate scenarios are available within the literature as shown in Figure 3-7 below.

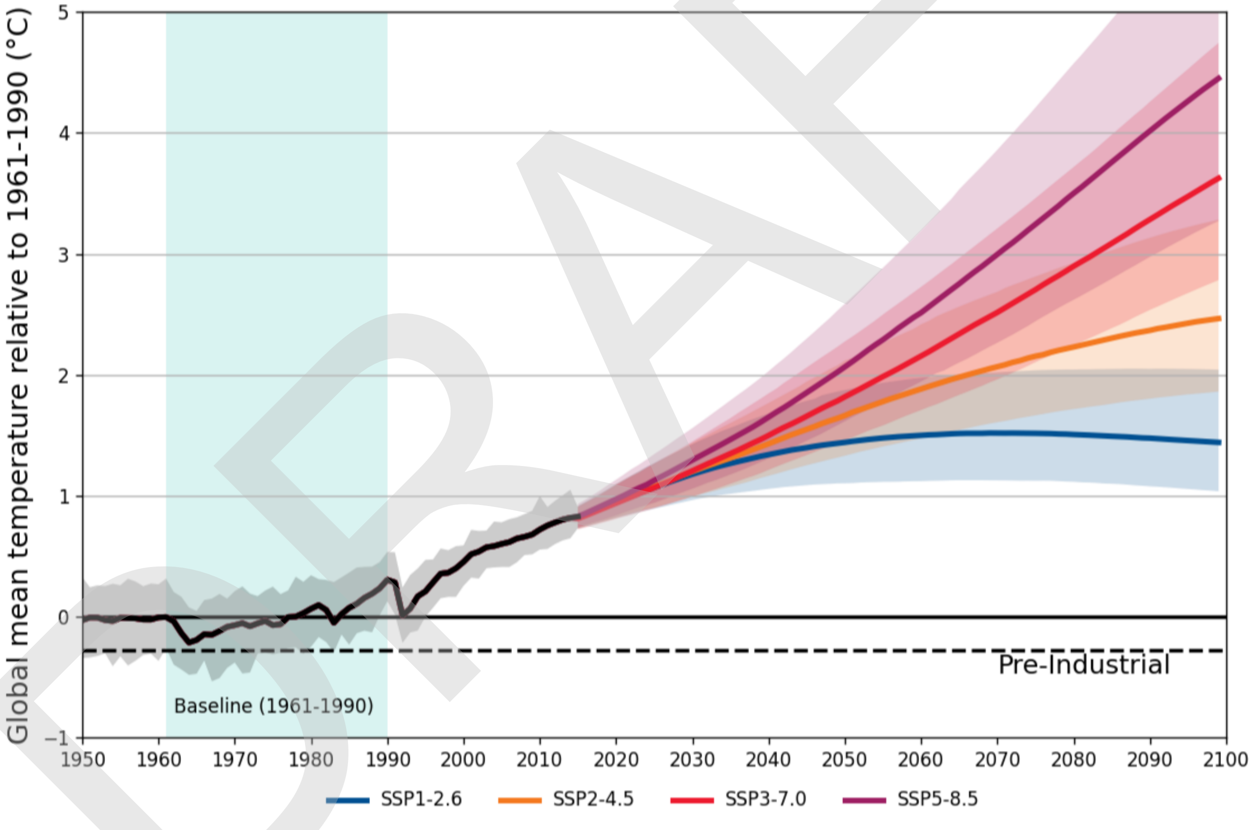

Figure 3-7 Projected temperature increases associated with varying socioeconomic pathways (SSPs), relative to a baseline at 1961-1990

This study has considered and modelled the following climate change scenarios:

- 20% and 1% AEP baseline scenarios (i.e. unscaled IFDs as provided by the Bureau of Meteorology)
- 20% and 1% AEP SSP5 to the year 2100

[3] https://data.arr-software.org/

- 20% and 1% AEP SSP5 to the year 2030 (as a proxy for "present day")
- 1% AEP SSP3 to the year 2100

The relevant rainfall scaling factors and other design rainfall data are available in R02 – Modelling Report and appendices. Note that "baseline" refers to the average temporal period 1961-1990, which agrees with the period of the annual series utilised for flood frequency analysis.

Design losses were determined by reconciliation of the modelled peak flows at the Hopkins Falls gauge with the results of the Flood Frequency Analysis described in Section 3.1.1. Regional losses were sourced from the ARR datahub as a starting point and the model ran iteratively with adjustments made. After some iteration it was clear that a constant continuing loss across the range of flood magnitudes would not reconcile with the FFA, thus the continuing loss was varied across the range of flood magnitudes to achieve a good fit – similar to a proportional loss model.

For the climate change scenarios, design losses were scaled by the relevant factor (see R02).

Design RORB Parameters (Baseline Scenario)

The calibration routing parameters were adopted for design flow derivation. While the calibration events adopted varying losses across the catchment/interstation areas, design modelling adopted a single IL and CL for the whole model based on the reconciliation of modelled flows with FFA. The design parameters are shown in Table 3-5 below.

Table 3-5 Design RORB Parameters

AEP	Kc	Dav (km)	m	IL (storm)	CL (mm/hr)
20%	252.88	202.31	0.8	21 mm	1.19
10%					1.35
5%					1.58
2%					1.99
1%					2.33
0.5%					2.90
0.2%					3.42

Design Flows

Design peak flows at the Hopkins Falls gauge are shown in Table 3-6 below. The flows at Allansford are similar with a slight increase as a result of the larger contributing catchment area and Brucknell Creek contributing inflows. The critical duration and temporal patterns are equal for the two sites across the modelled events.

The design flows at Hopkins Falls and Allansford, along with the FFA results and confidence limits, are shown in Table 3-6 below.

Table 3-6 Design Flows (Baseline Climate), Critical Durations and Representative Temporal Patterns

AEP		Hopkins Falls			Allansford		
	Hopkins Falls Peak Flow (m³/s)	FFA 5% Confidence Limit (m³/s)	FFA Expected Quantile (m³/s)	FFA 95% Confidence Limit (m³/s)	Peak Flow (m³/s)	Critical Duration	Representative Temporal Pattern

		Hopkins Falls			Allansford		
20%	206.6	166.4	205.6	257.5	207.6	24 hour	8
10%	301.5	244.5	301.1	375.1	303.0	24 hour	10
5%	394.4	318.0	395.5	507.3	400.0	24 hour	10
2%	513.6	403.7	515.0	735.3	522.6	24 hour	7
1%	600.5	457.0	600.1	931.2	612.5	24 hour	7
1 in 200	673.4	500.8	679.8	1161.1	690.1	24 hour	1
1 in 500	778.7	545.8	776.3	1493.4	802.3	24 hour	7

The peak flow at Allansford for the modelled climate change events, along with a percentage increase over the equivalent baseline event, is shown in Table 3-7 below. The critical 1% AEP hydrograph for the baseline, SSP5-8.5 2030 and SSP5-8.5 2100 events at Allansford is shown in Figure 3-8 below.

Table 3-7 RORB peak flow at Allansford and % increase over baseline

	1960-1990 Baseline	2030 SSP5-8.5	2100 SSP5-8.5	2100 SSP3-7.0
20%	207.6 m³/s	243.1 m³/s (17%)	349.7 m³/s (68%)	-
1%	612.5 m³/s	711.2 m³/s (16%)	974.1 m³/s (59%)	902.3 m³/s (47%)

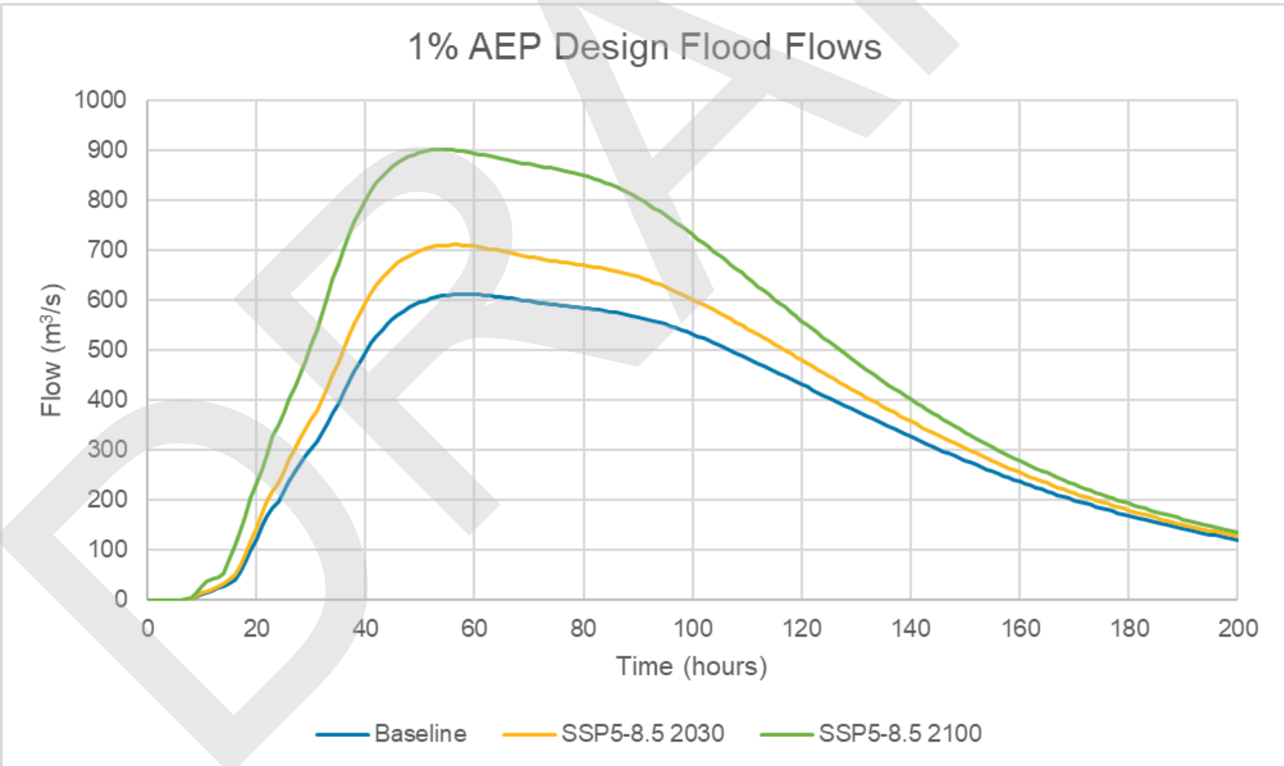

Figure 3-8 1% AEP Hydrograph at Allansford under three climate scenarios

The probable maximum flood (PMF) was also determined in accordance with the method outlined in Book 8 of ARR. The impact of climate change on the PMF was considered as recommended by Book 1. The resultant PMF peak flow rates at Allansford under the climate change scenarios considered are shown in Table 3-8 below.

Table 3-8 PMP rainfall depths and PMF flow rates, Allansford

	Baseline	SSP5-8.5 2030	SSP5-8.5 2100
PMP Depth (mm)	600	666	846
PMF Flow (m^3/s)	18,516	20,879	27,347

3.2 Hydraulic Modelling

The following sections summarise the key hydraulic model parameters and set up. More detailed information is available in R02 – Modelling Report.

3.2.1 TUFLOW Model Summary

A hydraulic model of Allansford was built using the TUFLOW modelling package. The model has two uses: simulation of riverine flooding as a result of rainfall within the Hopkins River catchment, and localised stormwater flooding as a result of rainfall over Allansford and its upstream catchment. Key TUFLOW model parameters are shown in Table 3-9 below, with the table differentiating between the riverine and stormwater models where necessary.

Table 3-9 Key TUFLOW Parameters

Parameter	Value
Model Build	2023-03-AC-iSP-w64
Model Precision	Single Precision
Grid Cell Size	4 metres (riverine), 2 metres (stormwater)
Model Orientation	North/South
Sub Grid Sampling	1 metre
Solution Scheme	HPC
Inflows	Single inflow from RORB output (riverine) Direct rainfall (stormwater)
Outflow	Riverine: Height-Flow Slope of 0.1% Stormwater: Height-Flow Slope of 5% at catchment (upstream) boundaries, water levels in river set based on riverine modelling
Hydraulic Roughness	Manning's 'n', varies with land use
1-Dimensional elements	Culverts and pipes linked to 2-D domain (note bridges modelled as 2-D flow constrictions)

3.2.2 Topography

The base model topography adopted the most recent Digital Twin Victoria LiDAR dataset, captured in 2023. Calibration events adopted an older 2017 LiDAR dataset, which better represented the floodplain for the historic events modelled. Topographic alterations were made as necessary, with the river centreline lowered by 1m to account for the riverine bathymetry below the water surface (as captured by LiDAR). Roads were reinforced as breaklines in the model to ensure no "leaking" of flow through roads. Figure 3-9 below shows the adopted design event model topography including modifications.

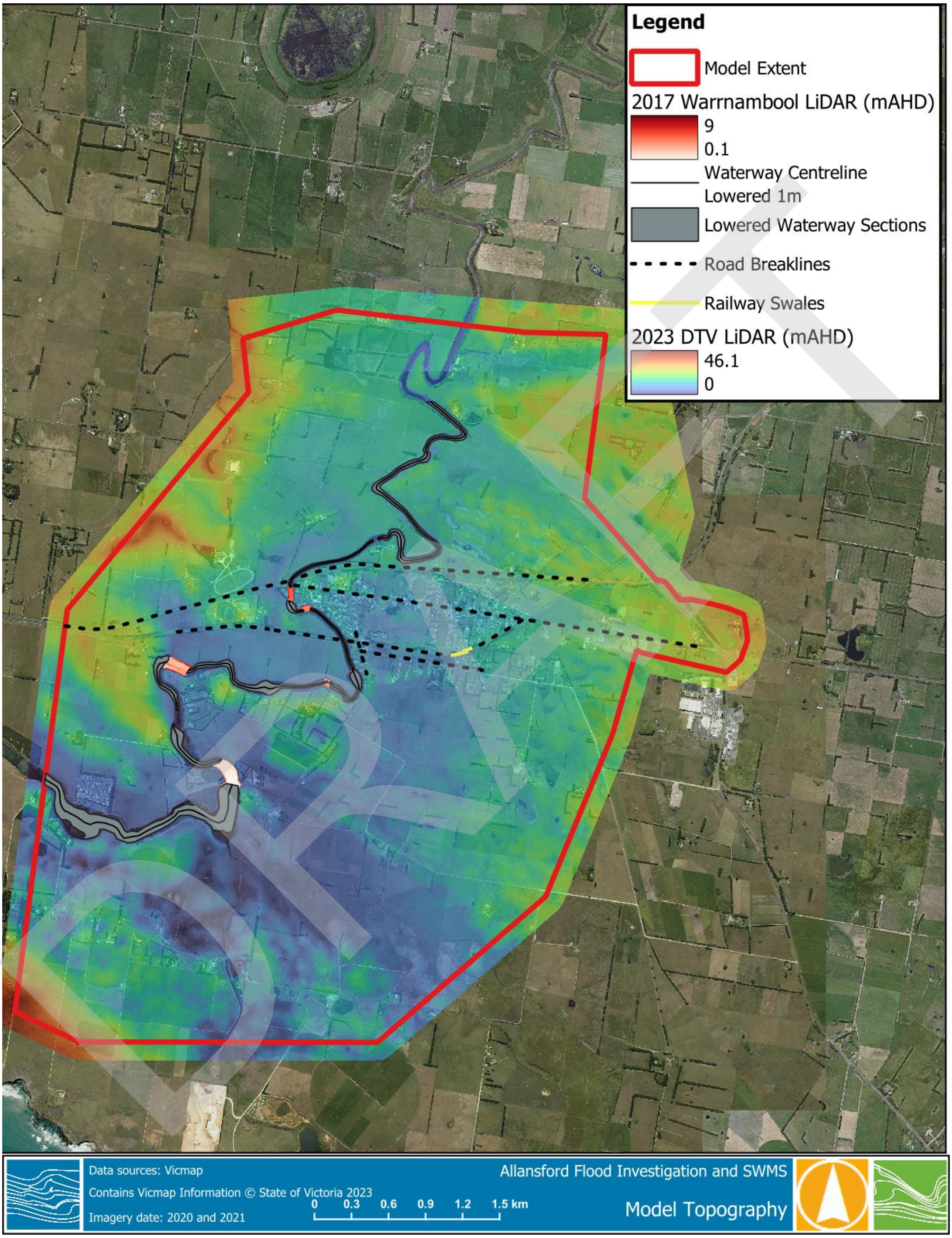

Figure 3-9 Model Topography

3.2.3 Model Extent, Boundaries and Mapping Extent

The model extends approximately 4.7km upstream of the Highway bridge, and approximately 5km downstream of the railway bridge. This ensures sufficient distance from the model boundaries to the area of interest to avoid any influence on model results from boundary conditions. While the extent ensures that no boundary conditions impact flooding in Allansford, it is noted that rural drainage and culverts in the floodplains to the south of Allansford are not represented in the model.

Because of this, model results in the area influenced by such drainage cannot be relied upon. A "mapping area" was defined as the area of high confidence in model results, and most result grids clipped to the mapping area so as to avoid the publication of lower confidence results. The model extent, boundary conditions and mapping area are shown in Figure 3-10 below.

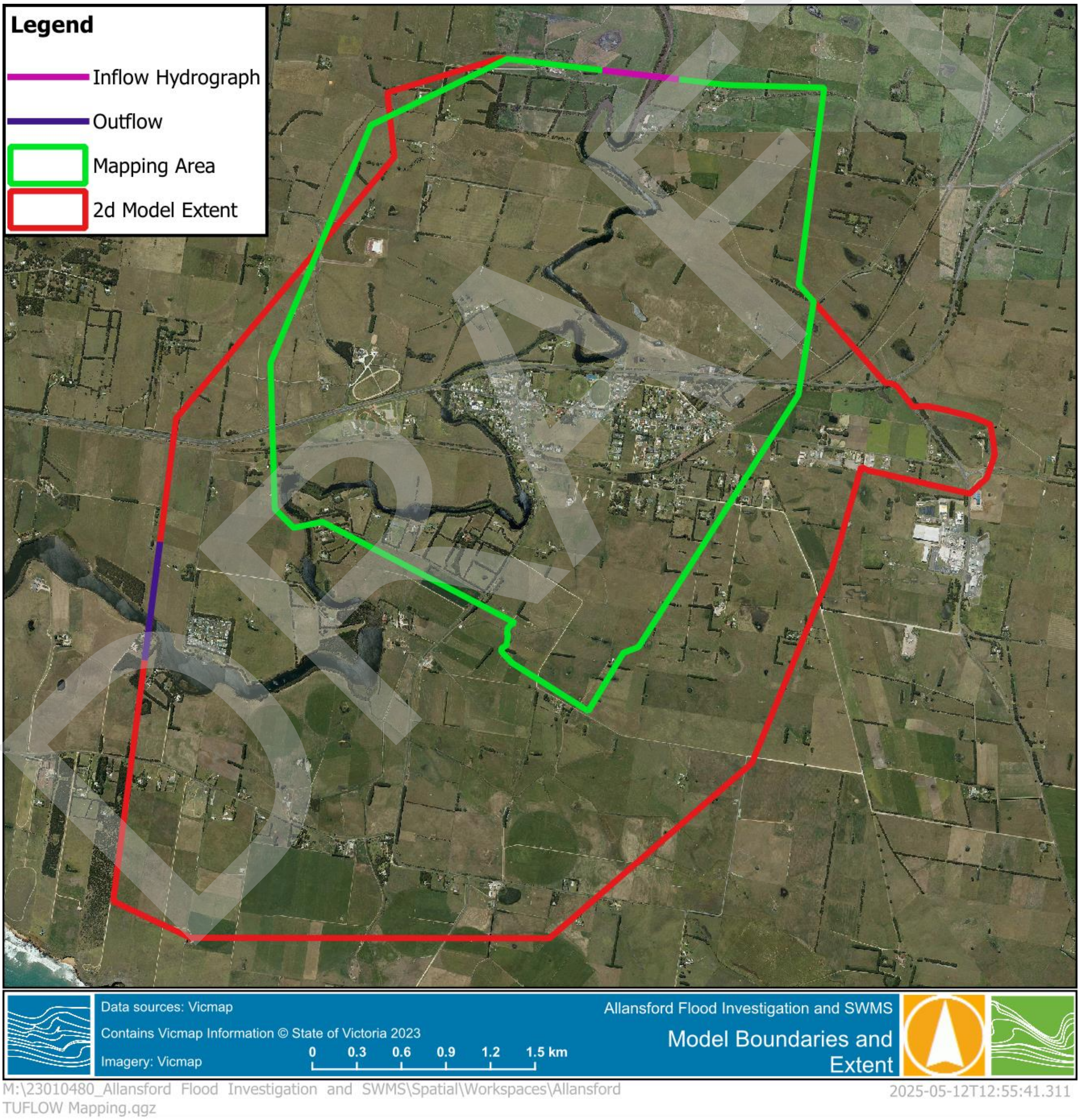

Figure 3-10 TUFLOW Extent and Model Boundaries

3.2.4 Hydraulic Roughness

Hydraulic roughness within the 2-dimensional model domain was represented by the Manning's 'n' roughness coefficient. Manning's 'n' was determined using land use classifications as determined from the Warrnambool and Moyne Planning Schemes and adjusted based on inspection of aerial imagery. Roughness coefficients were determined using industry standard/expected values such as those found in (Chow, 1959) and (Melbourne Water, 2023) and adjusted during the calibration model runs.

During the calibration process, roughness values were adjusted after further inspection of aerial photography, aerial photographs of the January 2011 flood event, and analysis of results against available surveyed flood marks. This resulted in the waterway roughness being decreased.

The adopted roughness coefficients are summarised in Table 3-10. Figure 3-11 shows a map of the adopted roughness values.

Table 3-10 Hydraulic Roughness

Land use / Topographic description	Roughness coefficient (Manning's n)
Pasture and Grasses/Open Space	0.04
Moderate Vegetation	0.08
Low Density Residential	0.1 (dwellings and lot single roughness)
Residential/Township	0.35 (dwellings and lot single roughness)
School	0.5
Industrial	0.5
Sealed Roads (entire reserve)	0.02
Unsealed Roads (entire reserve)	0.03
Waterbody	0.02
Railway	0.03
Channel (Hopkins River)	0.025

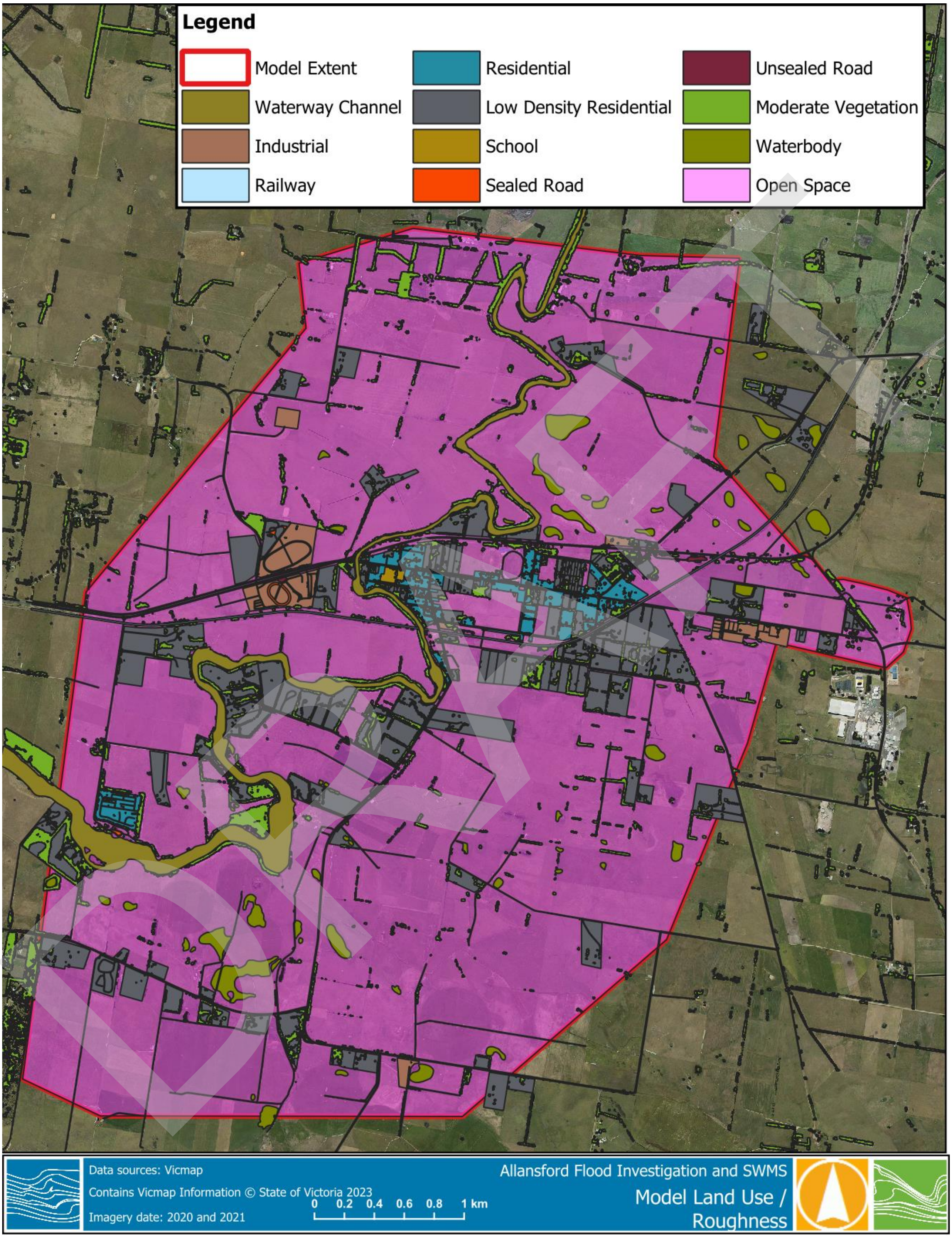

Figure 3-11 Hydraulic model surface roughness

3.2.5 Hydraulic Structures

Information on hydraulic structures was sourced from council GIS databases, structure design drawings, and survey. Four major riverine hydraulic structures were included in the model 2D domain: Princes Highway bridge, Ziegler Parade bridge, railway bridge and railway culverts/bridge in the floodplain west of the Hopkins River crossing. Minor structures such as culverts, pits and pipes were modelled as 1 dimensional elements. Figure 3-12 below shows the location of hydraulic structures included in the model.

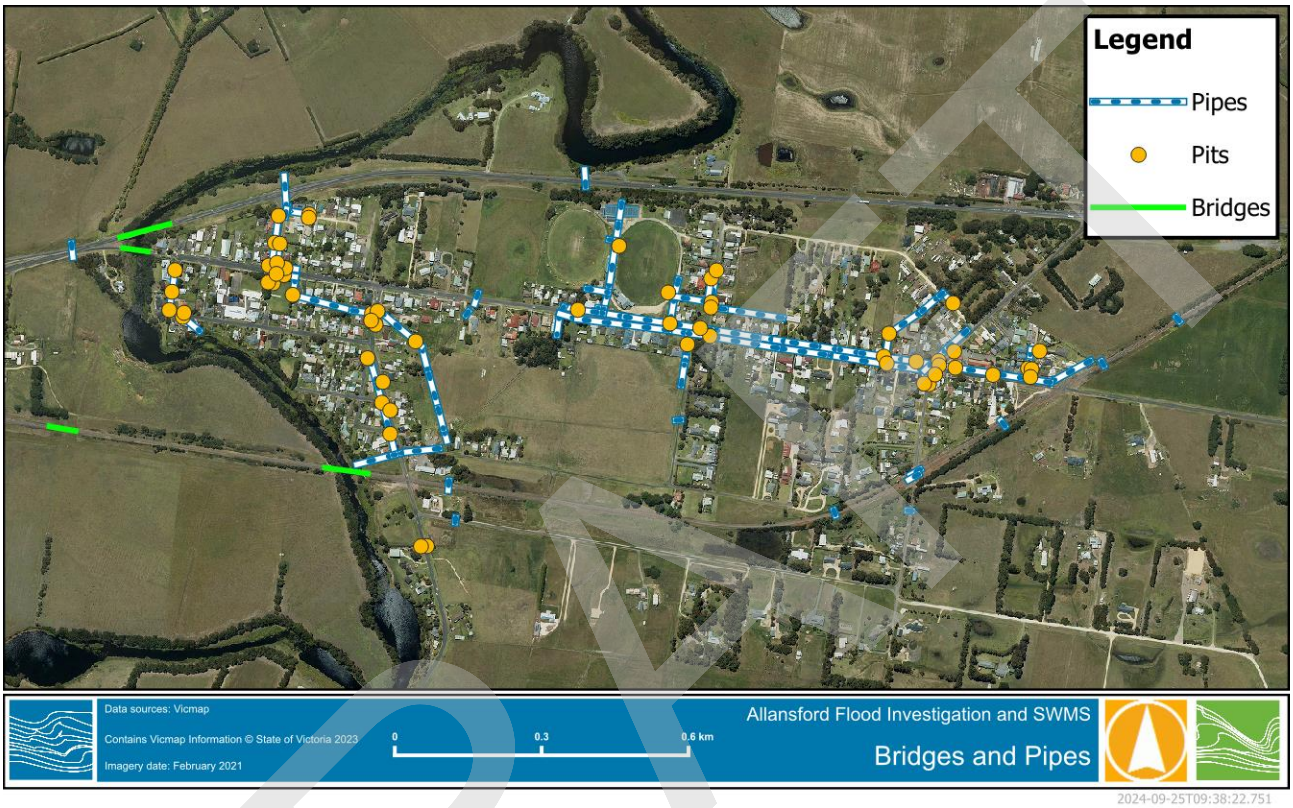

Figure 3-12 Hydraulic Structures

The Tooram Road pipe, the main pipe outlet for the town from Tooram Road, which outflows to the Hopkins River just upstream of the railway bridge; and the highway culvert immediately north of the football ovals, has been replaced with a new pipe and installed with a flap-gate on the outlet. As a result this pipe has been modelled as unidirectional. Similarly, the outlet to the north of the football oval has been modelled as unidirectional since its capability for backflow is well known and it is expected to be manually blocked in the event of a flood.

It is extremely important that the above assumptions are noted, and the mapping produced by this study interpreted appropriately. If the highway culvert is not blocked during a flood event it will cause nuisance flooding through the town. If the flap gate on the Tooram Road culvert fails, or is intentionally wedged open, significant inundation of the town may occur via backflow.

4 RIVERINE MODEL RESULTS/FLOOD MAPPING

4.1 January 2011 Calibration

The results of the January 2011 calibration modelling are presented herein. For a full discussion of the calibration process, please refer to R02 – Modelling Report.

The 2011 event was calibrated to two datasets: a set of flood marks which were surveyed at or near the peak of the event, and aerial photographs captured at or near the peak. The calibrated model achieved excellent agreement with many of the surveyed points. Some points had obvious errors (e.g. spot heights taken on the road, not the flood height) and were discounted from the calibration process. The difference between modelled and surveyed flood heights are shown in Table 4-1 and Figure 4-1 below.

The results indicate a strong calibration of the hydraulic model downstream of the highway bridge. Surveyed flood marks upstream of the highway appear to lack sufficient flood height to allow for head loss as water travels through the bridge and along the waterway, thus were deemed to contradict the points further downstream. That is, the water level does not sufficiently drop as it travels downstream (based on the surveyed peak flood marks). Given the model's close agreement with the surveyed points downstream of the bridge and its close agreement with aerial photography, more weight was given to the points downstream of the bridge as being accurate.

Mapping of peak flood depths across the broader floodplain is shown in Figure 4-2, with an aerial photograph taken near the peak shown in Figure 4-3. The two images show an excellent agreement between the modelled and observed flood behaviour.

Table 4-1 Surveyed and Modelled Flood Levels, January 2011

Name/Location	Flood Level (mAHD)	Survey Method	Modelled Flood Level (mAHD)	Difference (m)	Absolute Difference (m)
Upstream of Highway					
Princes Highway Road reserve 1	11.01	GPS	11.45	0.44	0.44
Princes Highway Road reserve 2	11.05	GPS	11.37	0.32	0.32
Princes Highway Road reserve 3	10.97	GPS	11.29	0.32	0.32
Princes Highway Road reserve 4	10.79	GPS	11.26	0.47	0.47
Princes Highway Road reserve 5	10.8	GPS	11.20	0.40	0.40
Princes Highway Road reserve 6	10.82	GPS	11.15	0.33	0.33
Princes Highway Road reserve 7	10.74	GPS	10.97	0.23	0.23
Downstream of Highway					
Maria Street	9.903	GPS	9.86	-0.05	0.05
11 Frank Street	10.269	GPS	10.21	-0.06	0.06
10215 Princes Highway	10.607	GPS	10.58	-0.03	0.03
1 Ziegler Parade	10.631	GPS	10.75	0.12	0.12
Princes Highway Road reserve 10	10.76	GPS	10.88	0.12	0.12
Princes Highway Road reserve 11	11	GPS	10.99	0.04	0.04
Princes Highway Road reserve 12	10.95	GPS	11.00	0.00	0.00
1 Ziegler Parade	10.75	GPS	10.74	-0.01	0.01
1 Ziegler Parade	10.69	GPS	10.73	0.04	0.04
1 Ziegler Parade	10.68	GPS	10.73	0.05	0.05
	10.7	GPS	10.63	-0.07	0.07

Name/Location	Flood Level (mAHD)	Survey Method	Modelled Flood Level (mAHD)	Difference (m)	Absolute Difference (m)
1 Ziegler Parade	10.69	GPS	10.63	-0.06	0.06
1 Frank Street	10.59	GPS	10.59	0.01	0.01
3 Alice Street	10.61	GPS	10.51	-0.10	0.10
3 Alice Street	10.45	GPS	10.57	0.12	0.12
	10.53	GPS	10.19	-0.34	0.34
3 Frank Street	10.38	GPS	10.26	-0.12	0.12
3 Frank Street	10.34	GPS	10.28	-0.06	0.06
5 Frank Street	10.39	GPS	10.31	-0.08	0.08
Suspected erroneous points					
Princes Highway	11.1	GPS	10.54	-0.56	0.56
Princes Highway Road reserve 8	12.63	GPS	10.95	-1.68	1.68
Princes Highway Road reserve 9	12.39	GPS	11.05	-1.34	1.34

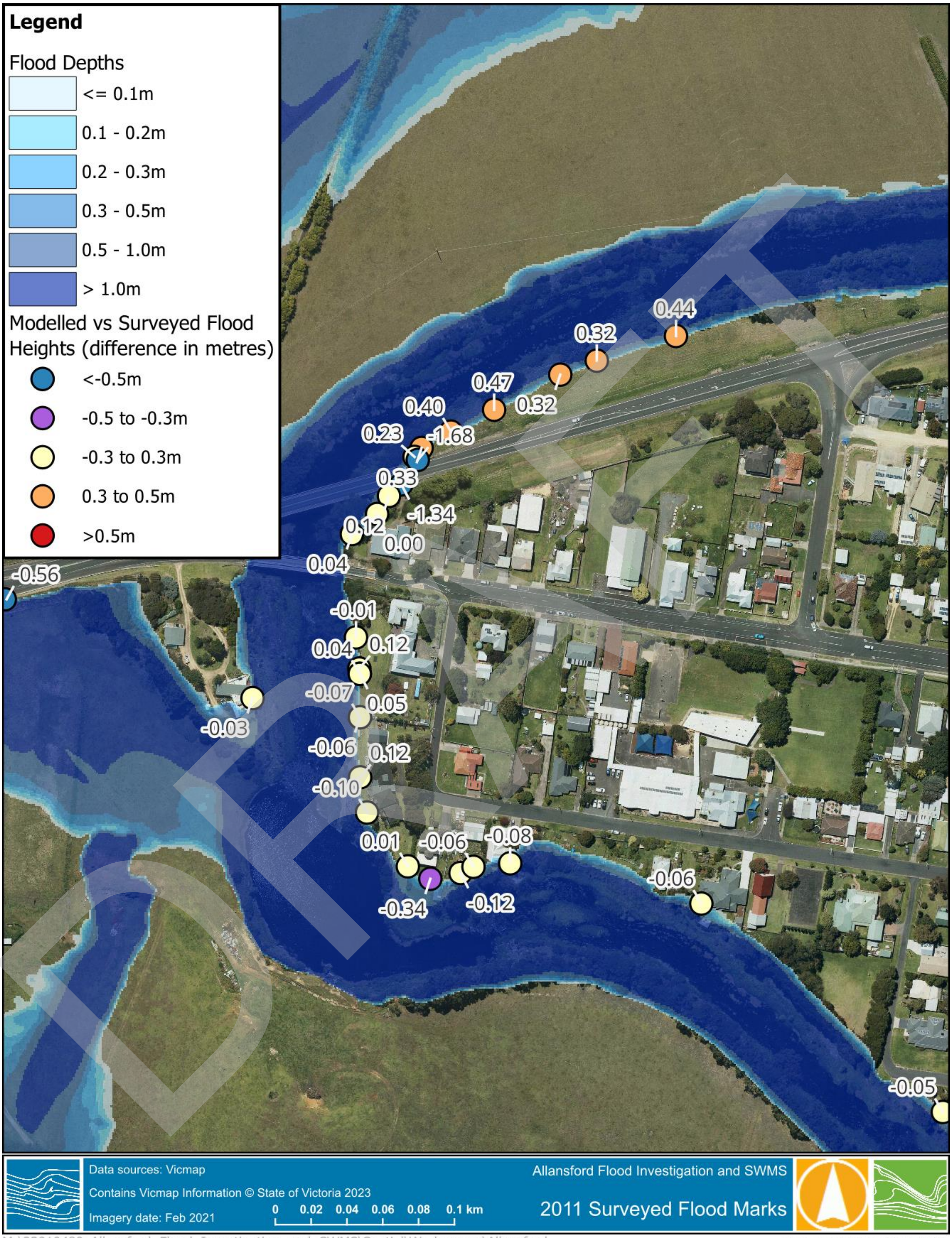

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Figure 4-1 Modelled and Surveyed Flood Levels

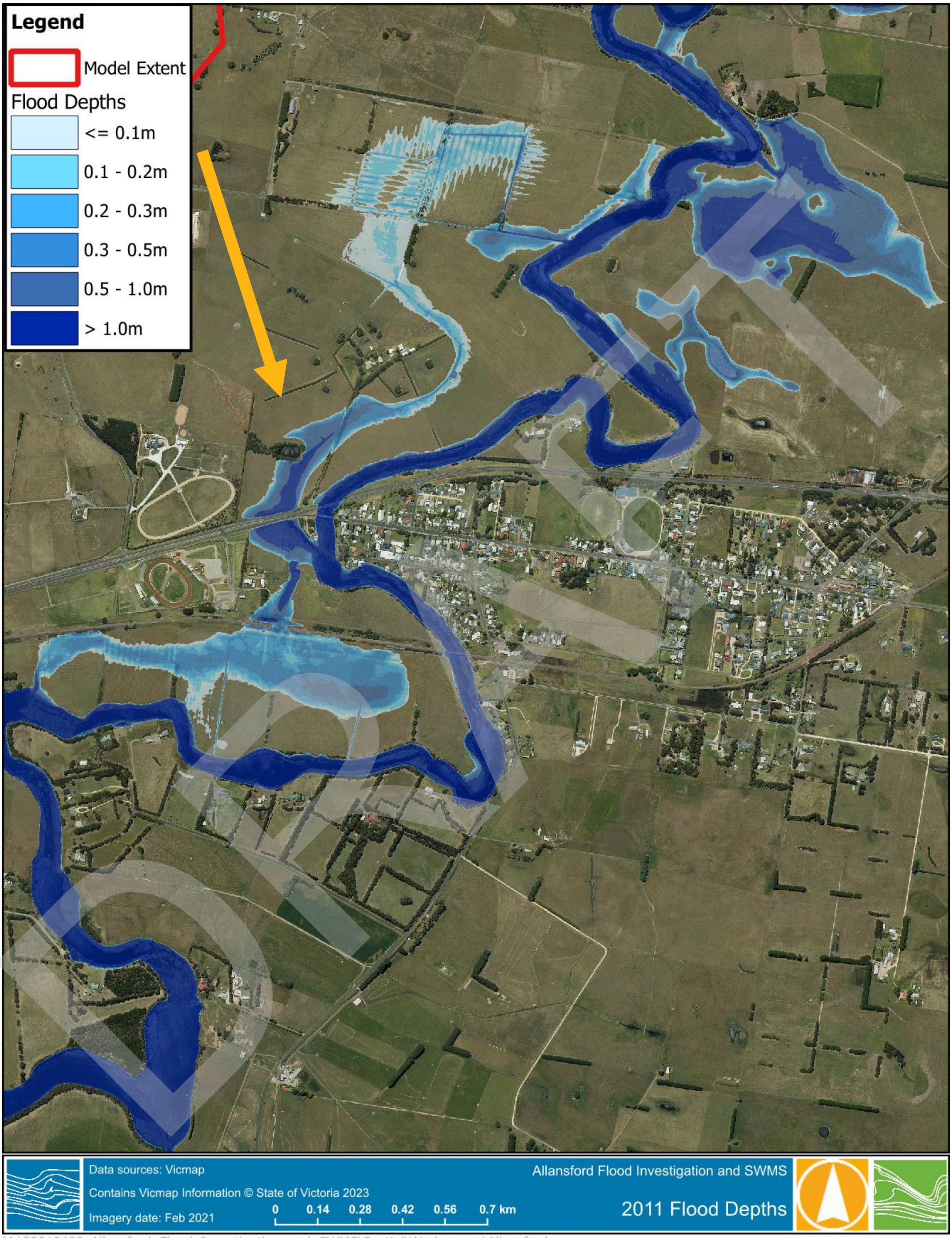

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Figure 4-2 January 2011 modelled flood depths. The arrow indicates the direction of view for Figure 4-3 below.

Figure 4-3 Aerial photo of the January 2011 event.

4.2 Design Mapping

The calibrated flood models were simulated for the design flood events shown in Table 4-2 below (section 3).

Table 4-2 RORB peak flow at Allansford and % increase over baseline

	1960-1990 Baseline	2030 SSP5-8.5	2100 SSP5-8.5	2100 SSP3-7.0
20%	207.6 m³/s	243.1 m³/s (17%)	349.7 m³/s (68%)	-
1%	612.5 m³/s	711.2 m³/s (16%)	974.1 m³/s (59%)	902.3 m³/s (47%)

In general, the 20% AEP events (across all modelled climate scenarios) remain within the bed and banks of the Hopkins River. The 1% AEP results break out of the waterway confines, spilling into the floodplain to varying degrees.

The baseline 1% AEP event, which does not consider climate change that has already occurred since the period 1961-1990, is the only 1% AEP event modelled that does not overtop the Princes Highway. Events which overtop the highway cause significant inundation in Allansford, particularly in the low area north of Station Street. Events which overtop the highway have varying impacts within Allansford. As water overtops the highway, low areas are inundated. Drainage which usually drains these areas is overwhelmed by high water levels in the Hopkins River, thus no drainage can occur until the event is passing and water levels in the river begin to lower.

Figure 4-4 below shows the 1% AEP flood extents under baseline, SSP5-8.5 2030 and SSP5-8.5 2100 climate scenarios. Detailed flood depth, velocity and hazard mapping are available in R02 – Modelling Report Appendix A for all modelled events.

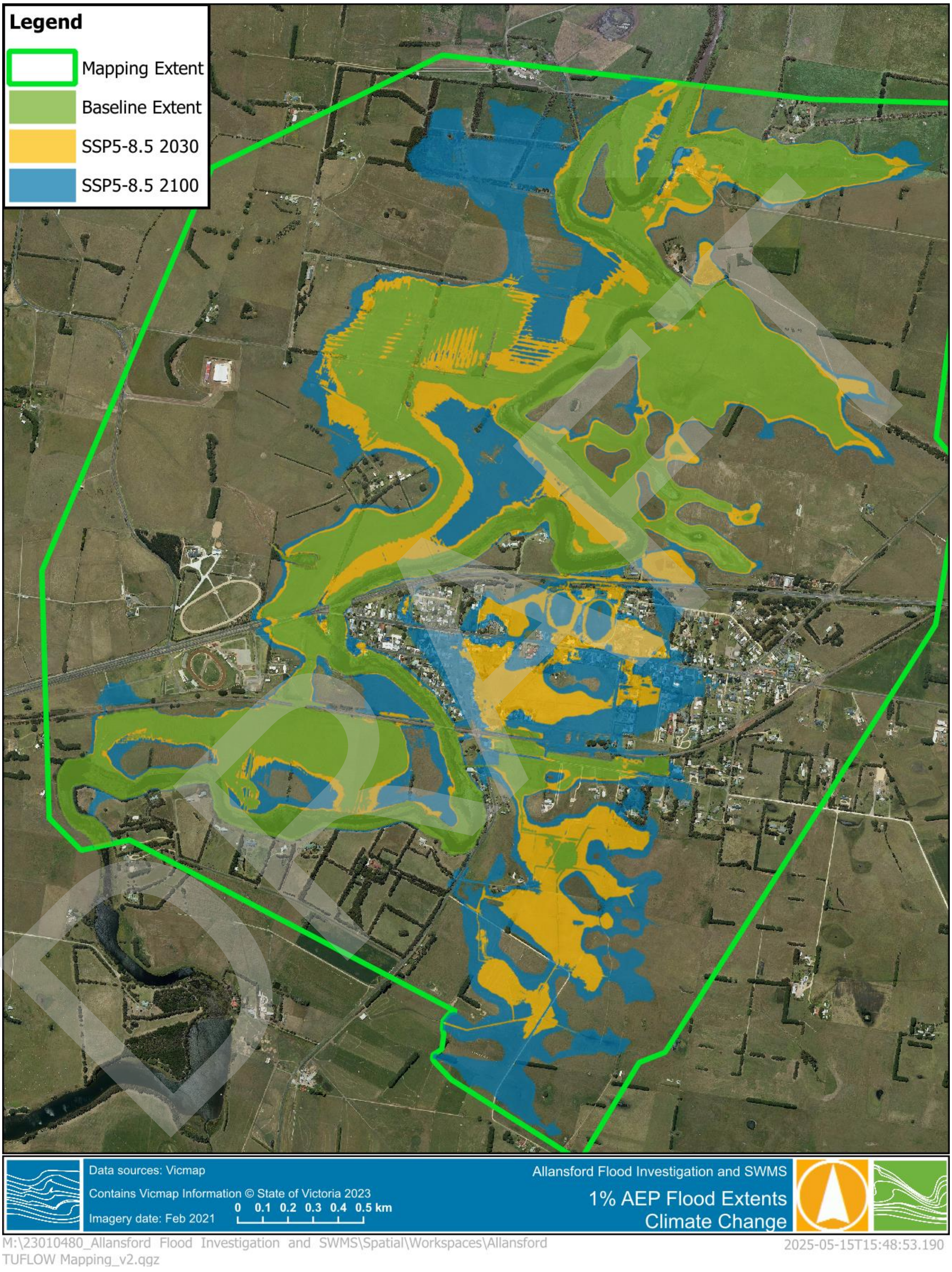

Figure 4-4 1% AEP flood extent under three climate scenarios

4.3 Draft Planning Scheme Mapping

Draft planning scheme overlay mapping has been prepared to the specifications of Warrnambool City Council and Glenelg Hopkins CMA.

The latest guidance from GHCMA, supplied to Water Technology during the project, requires the use of the 1% AEP event projected to 2100 under SSP5-8.5 as the basis for planning scheme flood mapping. This mapping was then delineated into "flood fringe" and "floodway" based on the criteria in Table 4-3 below, which also aligns with the vulnerability thresholds defined in the Australian Disaster Resilience Handbook Collection Guideline 7-3, *Food Hazard* (AIDR, 2017). Land Subject to Inundation Overlay has been applied to the *flood fringe* and Floodway Overlay applied to the *floodway*.

Table 4-3 Floodway Delineation

Overlay	Vulnerability Threshold (AIDR)	Limiting Depth (D_{max}, m)	Limiting Velocity (V_{max}, m/s)	Limiting Classification Limit $(V*D)_{max}$, (m^2/s)
Land Subject to Inundation Overlay	H1 and H2	0.5	3	0.6
Floodway Overlay	H3, H4, H6 and H6	>0.5	>3	>0.6

The initially delineated map is then further manipulated to improve its clarity and reinforce the purpose of the mapping, i.e. to inform planners of a flood risk and trigger the necessary permit process when development is proposed. The resultant draft planning scheme maps for the town of Allansford are shown in Figure 4-5 below.

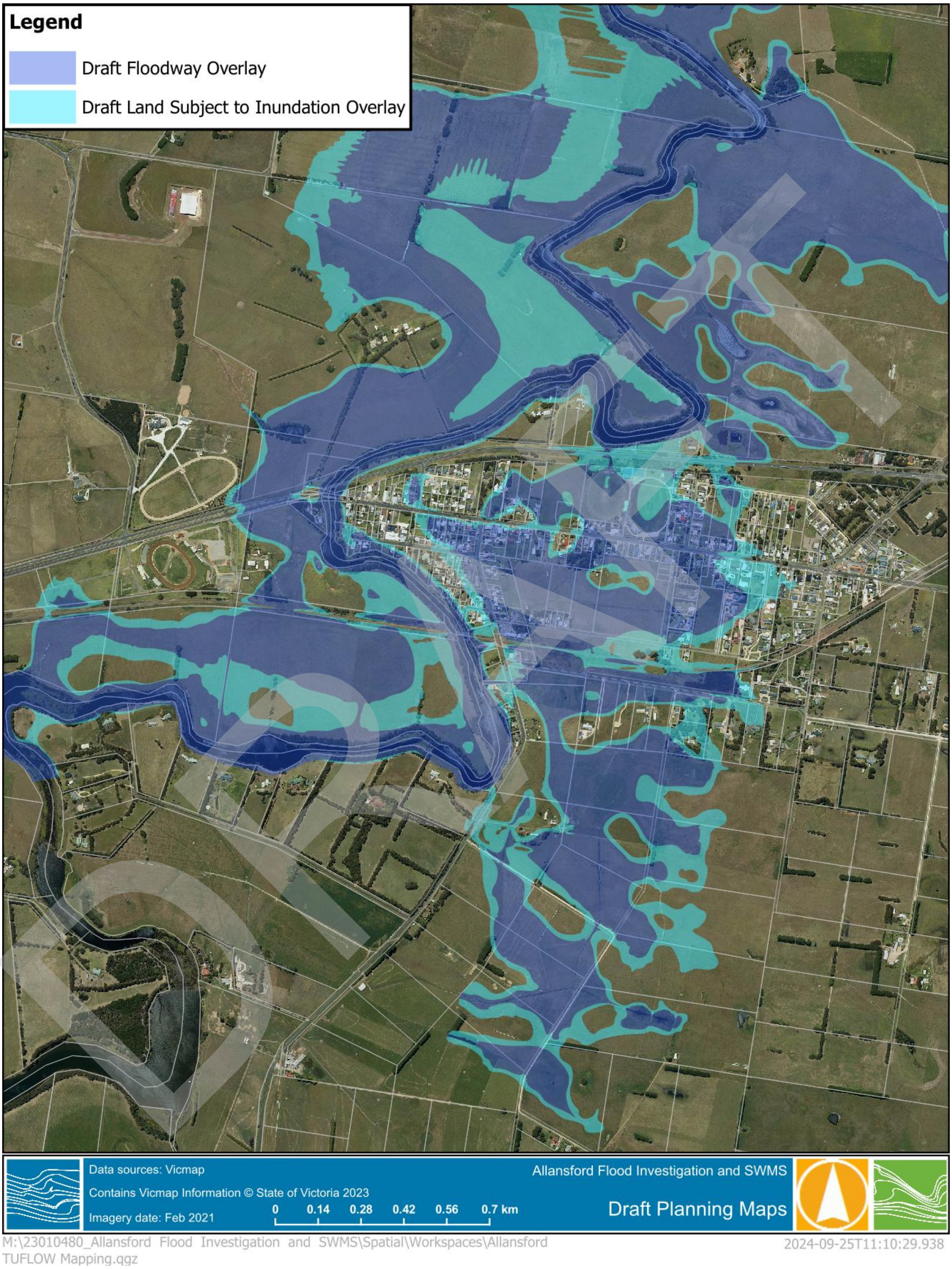

Figure 4-5 Draft maps for inclusion in the Warrnambool Planning Scheme

4.4 Mitigation Modelling

Riverine flooding within the township of Allansford occurs via one of two key mechanisms: backflow from the river into the stormwater system flooding low lying areas, and overtopping of the highway north of the football ovals. Backflow into the stormwater system is prevented via the installation of flap valves, one of which has already been installed on the main outflow pipe immediately north of the railway. Ensuring the prevention of backflow in other pipes is a recommendation of the study. **Modelling has assumed that the flap gate installed on the Tooram Road outlet operates perfectly (i.e. no backflow occurs), and that the culvert north of the highway cannot backflow (i.e. sandbagged/decommissioned/flap valve installed).**

Overtopping of the highway occurs in the 2030/present day 1% AEP flood event and the 2100 climate change scenario 1% AEP event. Overtopping can theoretically be prevented via the installation of a flood wall, levee or other arrangement (e.g. raising the highway). A conceptual arrangement was simulated in the hydraulic model to test the potential impact it has on flooding in the Hopkins and floodplains. The aim of the modelling was to determine the following:

- If breakout inundation of the township from the Hopkins River could be stopped using flood walls/levees,
- If the proposed levee/walls would impact water levels in the remaining floodplain,
- If any impacts could be easily dealt with and further mitigated to reduce or alleviate raised flood levels on private property and dwellings.

Modelling did not consider the cost or economic benefit of the mitigation options tested.

Mitigation modelling was conducted for the SSP5-8.5 2100 1% AEP event. Modelling showed that a wall along the north of the highway alone did not prevent inundation of the township – other areas (for example near the Tooram Road outlet) required treatment in addition to the highway area. The modelling found that a flood wall/levee arrangement on its own would raise flood levels in the Hopkins River and floodplains, with some private properties on the east side of the river between the highway and railway bridges experiencing raised flood levels in the order of 12cm higher than the current floodplain conditions.

In order to address the increased flood levels in the Hopkins River, particularly those on residential lots, further modelling (referred to as option 2 and shown below in Figure 4-6) aimed to encourage flow to pass through the flood runner at 10235 Princes Highway. This was modelled by lowering/excavating the western riverbank downstream of Ziegler Parade and the flow path between the river and railway bridge. The modelling attempted to assume a reasonable/achievable area of excavation in the first instance, however this had negligible impact on flood behaviour and did not alleviate the impact of the flood walls/levees.

Finally, option 3 aimed to reduce the impact within the Hopkins River and floodplain by implementing large-scale earthworks. In the model, a channel has been assumed from the Hopkins River to the railway bridge within 10235 Princes Highway. The channel has been assumed to be 50m wide at the river end, widening to 100m through 10235 Princes Highway up to the railway culverts. No alteration of the railway culverts or earthworks downstream of the culverts was modelled as part of this option. An estimated ~50,000 m^3 of material would be required to be excavated to achieve ground levels as modelled, with even more required to achieve batters to natural ground surface.

Regardless of the viability (or lack thereof) of construction, option 3 mitigates the raised flood levels between the highway and railway bridges. Raised flood levels remain on the farmland floodplains north of Allansford, through 10235 Princes Highway and south of the railway line, however no residential land or dwellings are within the impacted area.

The modelled mitigation options are shown in Figure 4-6 and Figure 4-7, with the flood level afflux (change in flood level compared to non-mitigated) results shown in Figure 4-8 and Figure 4-9.

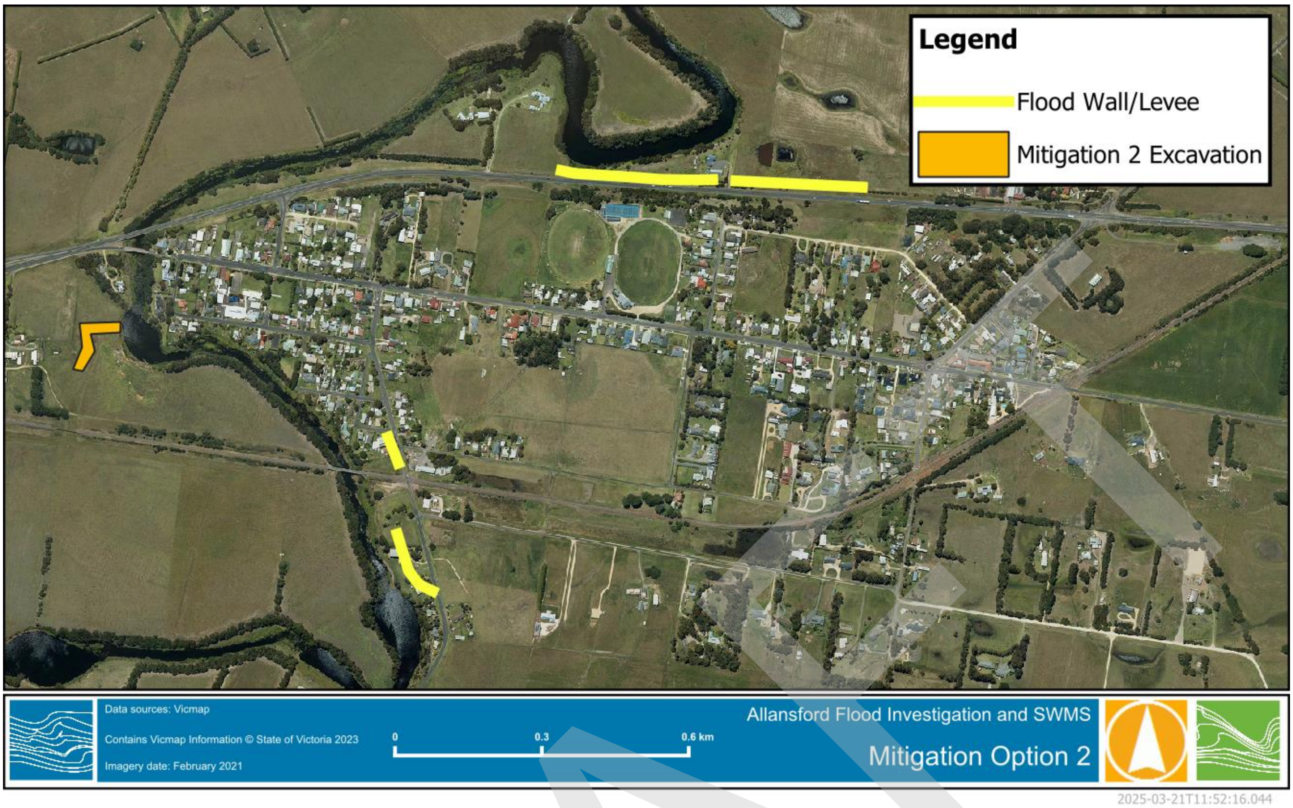

Figure 4-6 Mitigation Option 2 – Modelled Works

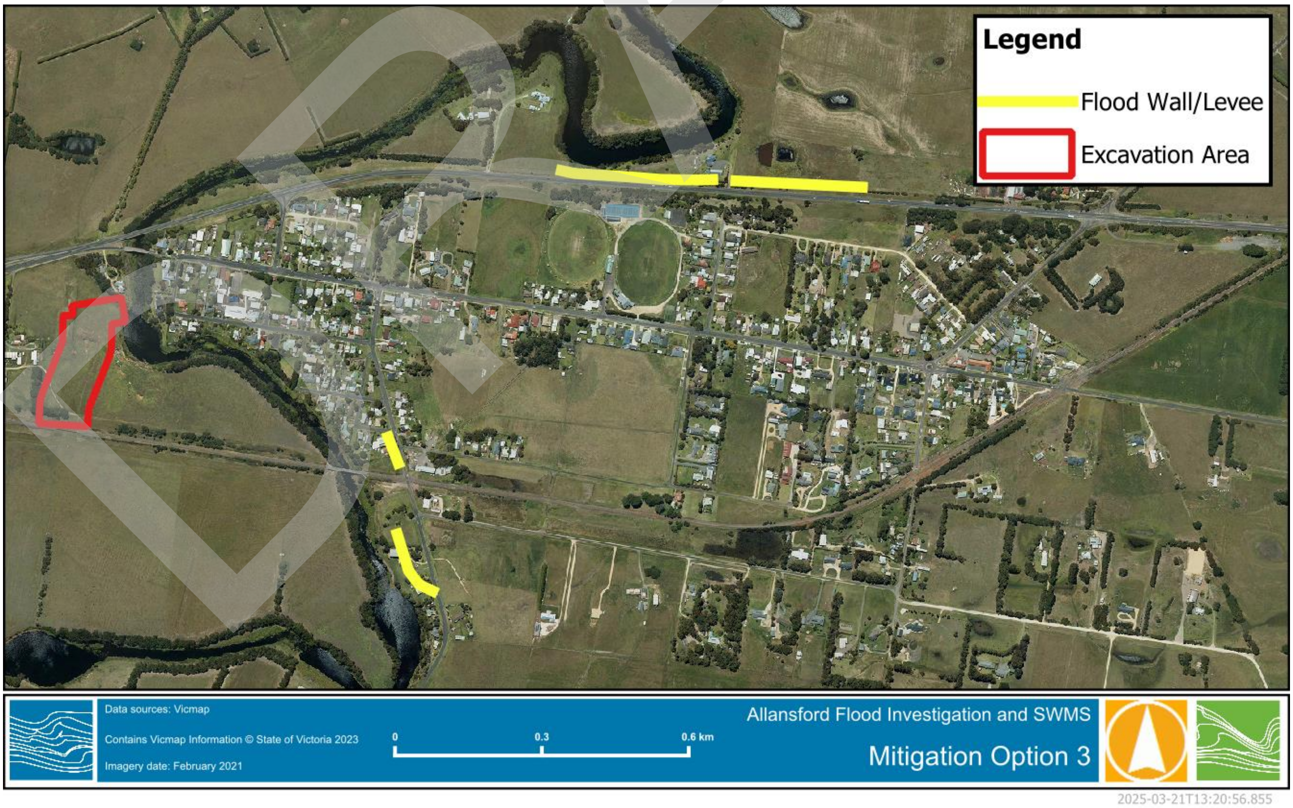

Figure 4-7 Mitigation Option 3 – Modelled Works

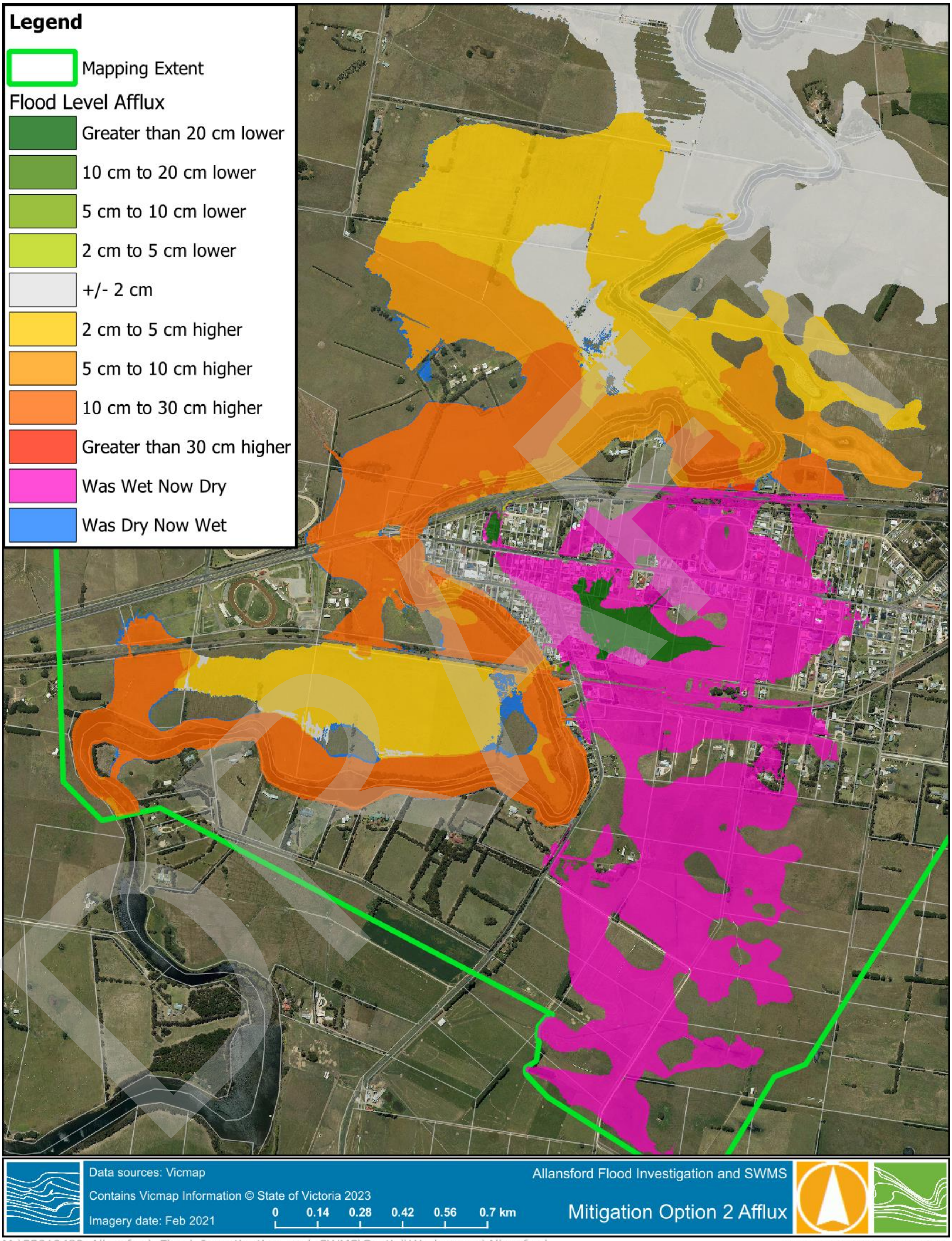

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Figure 4-8 Mitigation Option 2 – 1% AEP SSP5-8.5 2100 Flood Level Afflux

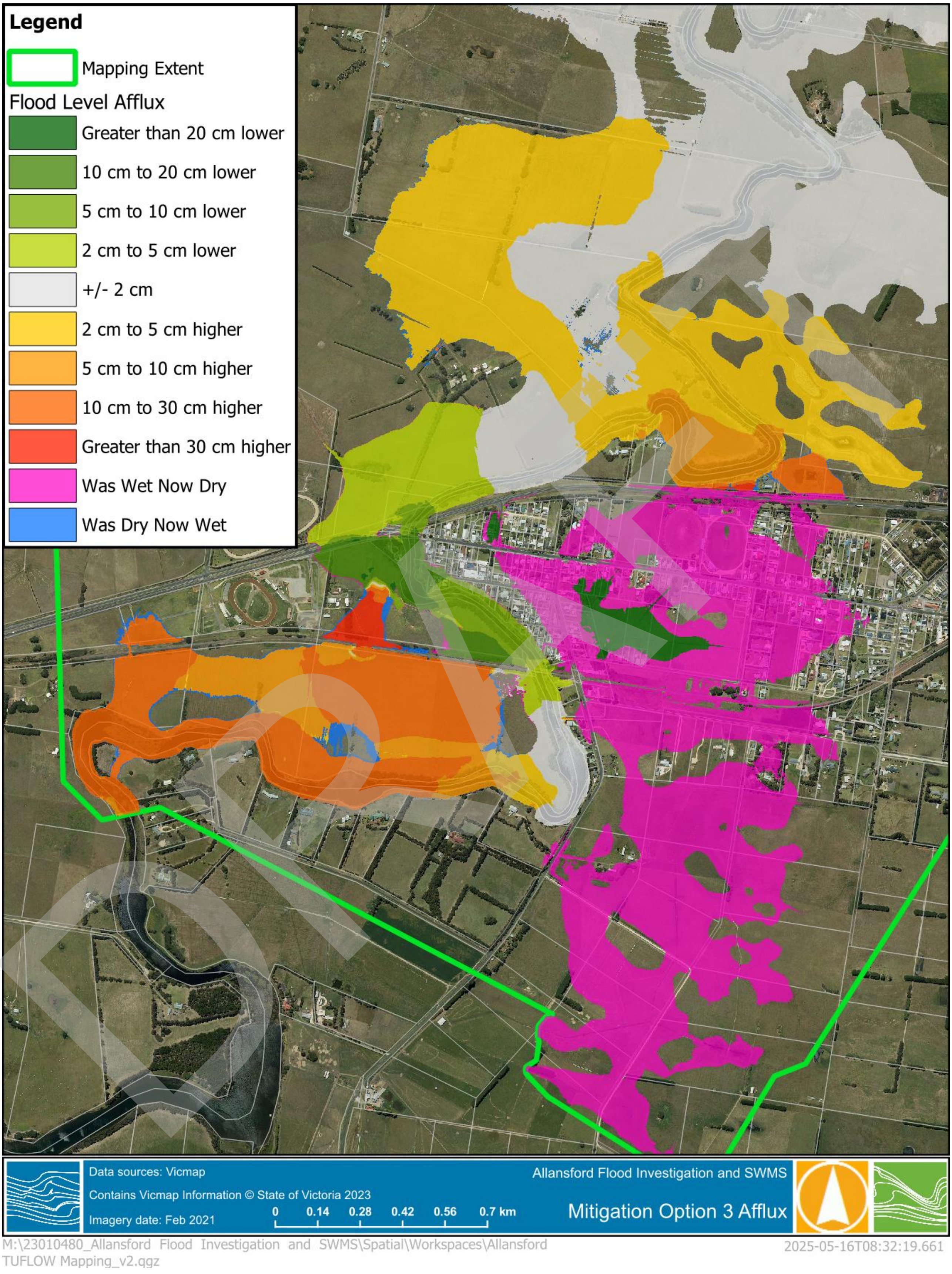

Figure 4-9 Mitigation Option 3 – 1% AEP SSP5-8.5 2100 Flood Level Afflux

4.5 Model Sensitivity Tests

Sensitivity tests were conducted to determine the models' responsiveness to particular parameters. The draft 1% AEP event was utilised to test hydraulic model parameters. Model parameters tested included:

- Initial and Continuing Loss (tested in RORB)
- Storage Drawdown (RORB)
- Hydraulic Roughness (TUFLOW)
- Bridge Losses (TUFLOW)
- Boundary Conditions (TUFLOW)
- Routing Parameter Kc (RORB)

The full details of model sensitivity tests can be found in R02 – Modelling Report.

5 STORMWATER MANAGEMENT STRATEGY

5.1 Overview

The need for improved management of stormwater in Allansford has been well known for some time. Two previous studies have investigated underground drainage in the town: *Allansford Outfall Drain Options Evaluation Report* (Hyder Consulting, 2002) and *Allansford Township Drainage Requirements* (Cardno, 2008). These assessments both recommended upgrades to the underground pipe network within the town, which were intended to improve drainage of the area.

The strategy options developed as part of this project considers existing conditions, overland stormwater conveyance, and water quality treatment by developing a local catchment rain on grid TUFLOW model and MUSIC model of the town. Three distinct drainage options have been developed and assessed.

Hydrologic/hydraulic modelling will consider climate change in accordance with the recommendations of ARR version 4.2. The SSP5-8.5 climate scenario has been adopted in consultation with Council and the Glenelg Hopkins CMA. The performance of the existing drainage network has been tested for the year 2030, which has been adopted as being the best representation of present-day conditions, alongside 2100, which has been adopted as the design case against which mitigation options have been tested.

5.2 Methodology

A trimmed down TUFLOW model of Allansford which modelled the whole town and its contributing catchment, but did not explicitly model the Hopkins River riverine flooding, was used for the hydraulic assessment of stormwater inundation. The stormwater modelling extent is shown in Figure 5-1 below. In order to account for the influence of riverine flooding on the stormwater network, the model adopted the 20% AEP riverine flood level at each outlet to the river. Full modelling details can be found in R03 – Stormwater Management Strategy.

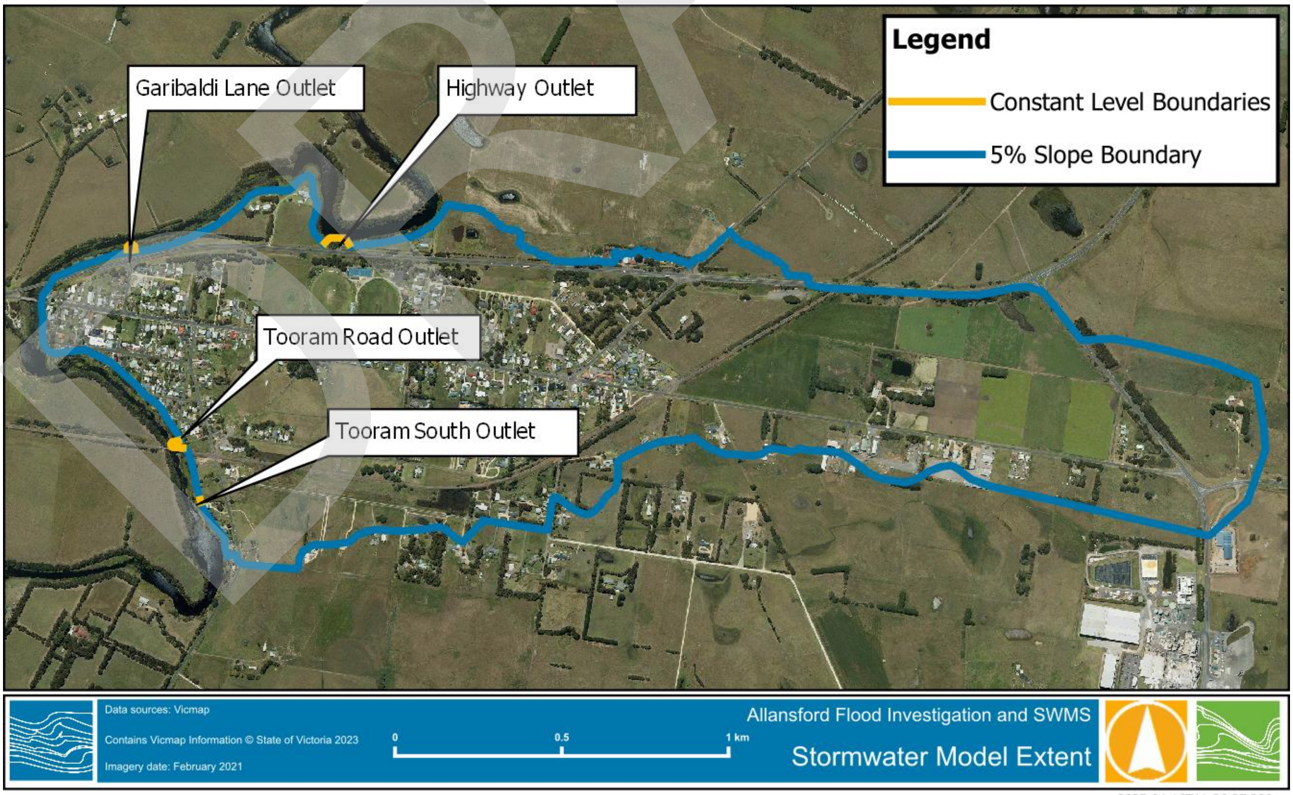

Figure 5-1 Stormwater Model Extent

5.3 Existing Inundation Conditions and Consultation

The model was simulated for a range of design events to determine the critical flooding conditions for the 1% and 20% AEP events under SSP5-8.5 2030 and 2100 climate scenarios. The 2100 climate scenario was adopted for design conditions and thus mitigation options were tested against that scenario.

Peak inundation depths for the 1% AEP SSP5-8.5 2100 scenario with existing drainage infrastructure are shown in Figure 5-2 below.

The consideration of potential stormwater conveyance improvements began at the community consultation stage of the project as detailed in R01 – Data Review Report. A number of measures were proposed by members of the community and were assessed via a pre-feasibility assessment. A number of community member suggestions were adopted and incorporated into the three mitigation options tested.

Figure 5-2 1% AEP Critical Flood Depths – Current Infrastructure, SSP5-8.5 to 2100

5.4 SWMS Options

Three distinct options were tested within the hydraulic model, referred to as options A, B and C. All three options included a basin located to the north east of the Tooram Road/Station Street intersection, with additional pipes and swales added to convey water to and from the basin. The key features of each option are summarised below.

- Option A
  - Tooram Road/Station Street basin, with duplicated and upsized outlet pipes (1.2m pipe).
  - Upgraded culvert on Elizabeth Street to assist drainage from east to west.
  - New 1.2m pipe from White Street to the west side of Elizabeth Street.
- Option B
  - Tooram Road/Station Street basin with current outlet configuration
  - Upgraded drainage from White Street to south of Ziegler Parade
  - Large 1.2m diversion pipe from upstream of the railway culverts, travelling along the south side of the railway. Pipe daylights into existing basin/borrow pit, with open drainage swales linking to the Tooram Road south outlet.
- Option C
  - Tooram Road/Station Street basin, with duplicated and upsized outlet pipes (1.2m pipe).
  - Upgraded drainage from White Street to south of Ziegler Parade
  - 1.2m pipe from downstream of the railway culverts, travelling under Ziegler Parade and Elizabeth Street to the west side of Elizabeth Street

The basin functions as a wetland for treatment of stormwater (see section 5.5 below) while allowing excavation and filling of the surrounding area to confine inundation to a smaller area. The basin is not intended or designed to retard flow rates entering the Hopkins River, in fact duplication of the outlet increases flow rates form Allansford to the Hopkins. The impact of increased flows from Allansford to the Hopkins River is negligible given the following flood behaviour characteristics downstream:

- There are no known riverine flood impacts between Allansford and Warrnambool, and the unclipped riverine model results have not revealed any impacts.
- Riverine and local flooding are caused by different types of storm, with local flooding caused by high intensity localised rainfall over a shorter period.
- Flooding downstream of Tooram Stones in the Hopkins estuary is a primarily volume driven process, with water levels raising behind the sand berm until it overtops and scours away or is artificially opened.

The total volume of flood flows will not be reduced by implementing a retarding basin, thus the decision not to retard flows is not expected to have any influence on volume based flooding of the estuary. Regarding flow rates, it is noted that Option A increases peak flow in the Tooram Road outfall pipe from 1.2 m^3/s (existing infrastructure) to ~3.1 m^3/s (Option A). This increase of 1.9 m^3/s represents a 0.205% increase in flow if it were to coincide perfectly with a 1% AEP riverine event under design conditions (SSP5-8.5 to 2100). Such a coincidence is considered extremely unlikely.

The three options and their cost are presented in detail in R03 – Stormwater Management Strategy. Summarised plans of each option are shown in the below figures, along with the 1% AEP peak inundation depths for the SSP5-8.5 2100 scenario, and the change in flood level (afflux) from the non-mitigated scenario (i.e. Figure 5-2 above).

Each modelled option has its benefits, impacts and associated costs. The key considerations associated with each option are summarised, along with the estimated cost of each option, as follows:

- All options:
  - Significant increase in flood free area around the proposed basin.
  - Minor increases in flood level and extent to the north of the proposed basin can be offset by increasing conveyance into the basin at this location.
- Option A ($2.84 million):
  - Least costly option.
  - Greatest benefit for White Street low point.
- Option B ($4.85 million):
  - Most expensive option.
  - Large areas of increased flood levels and extents will require landholder consent.
  - Use of the railway corridor for pipe asset may not be accepted by VicTrack.
  - Large area of benefit along main flow path on south side of Ziegler Parade.
- Option C ($3.84 million):
  - Middle cost option.
  - Pipe alignment utilises road reserve, eliminating need to place assets in VicTrack alignment.
  - Further option to add connections to allow additional drainage into Ziegler Parade pipe alignment – e.g. properties drain directly to the network (note this was not included in modelling). This could improve benefits along the main flow path, and potentially address flooding on the north of Ziegler Parade also.

Given the significant cost associated with Option B combined with the raised flood levels and additional flooding on private land, and the need to place the asset within the rail corridor, the diversion pipe along the railway is not recommended for further consideration. While the option does create a significant benefit to flooding in Allansford, it is unlikely to be the most cost-effective solution.

Option C provides a significantly larger area of beneficial impact compared to Option A. If further pipe connections are made to the proposed Ziegler Parade trunk drainage, the benefits would likely increase particularly for smaller events where the pipe capacity is not reached.

Given the above, Option C is recommended as the preferred option for implementation. Detailed design should investigate additional drainage links to directly connect dwelling and lot drainage to the main trunk where possible.

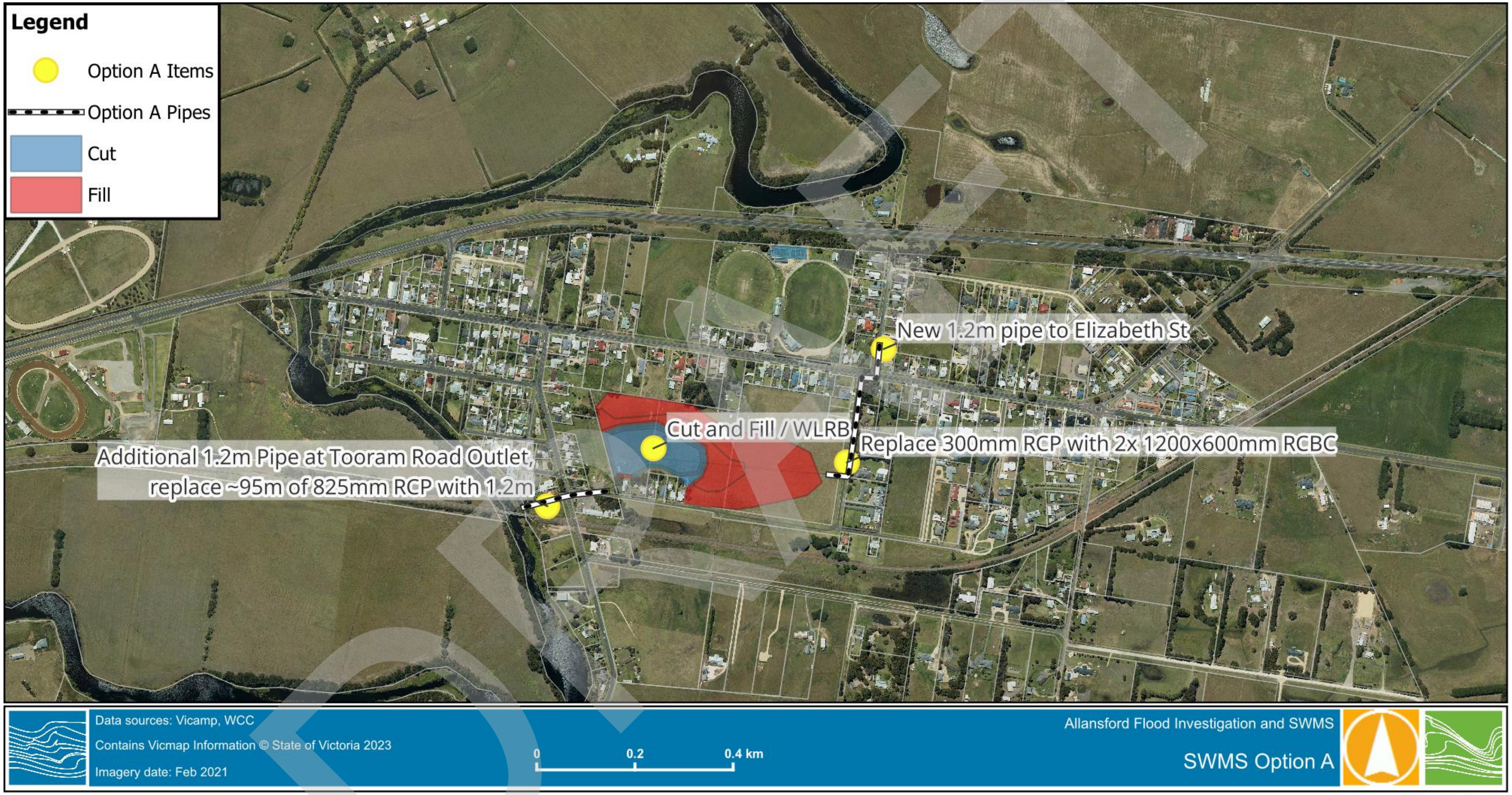

Figure 5-3 Proposed Infrastructure – Option A

Figure 5-4 SWMS Option A – 1% AEP SSP5-8.5 2100 Flood Depths

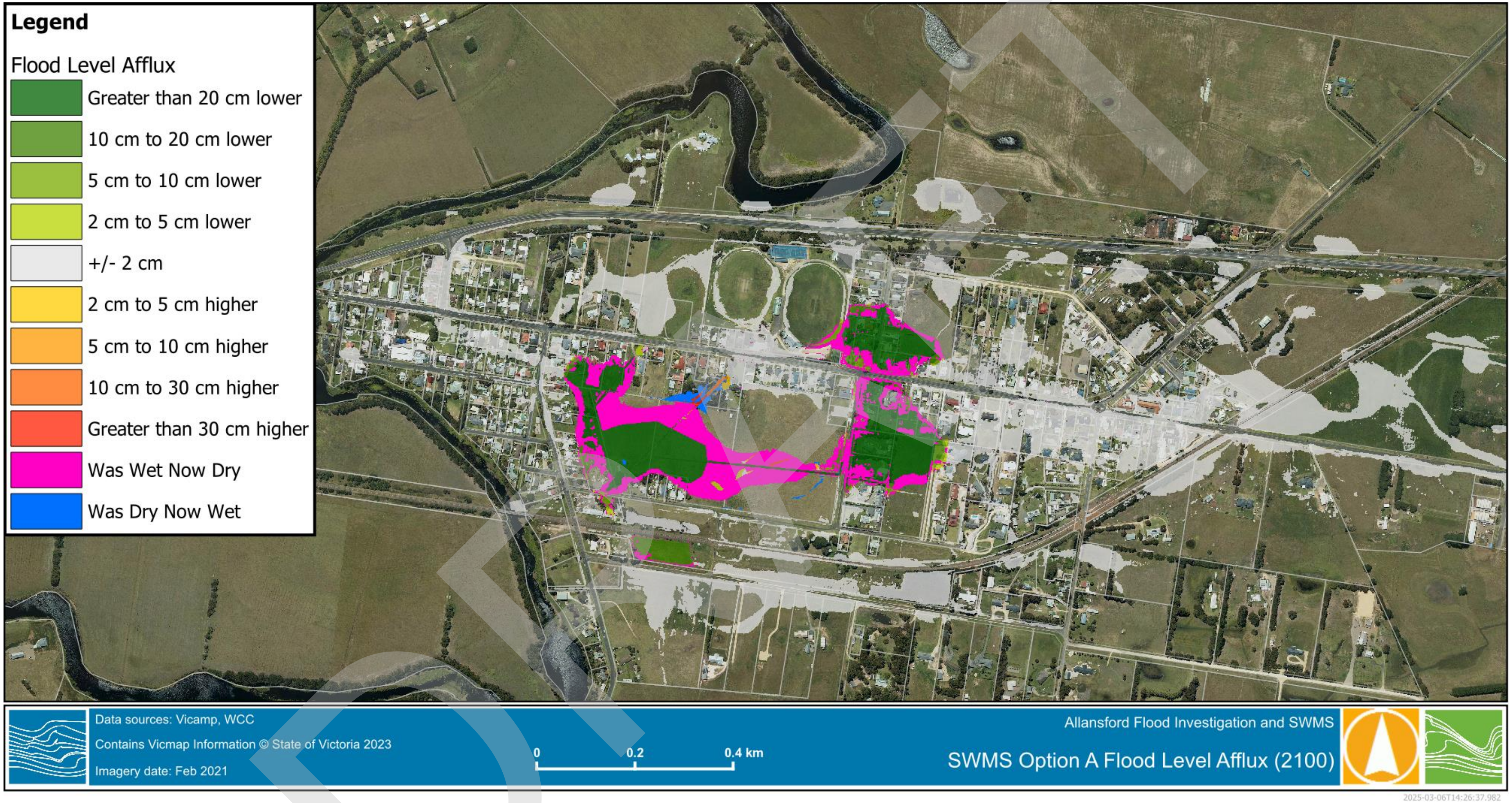

Figure 5-5 SWMS Option A – 1% AEP SSP5-8.5 2100 Flood Level Afflux

Figure 5-6 Proposed Infrastructure – Option B

Figure 5-7 SWMS Option B – 1% AEP SSP5-8.5 2100 Flood Depths

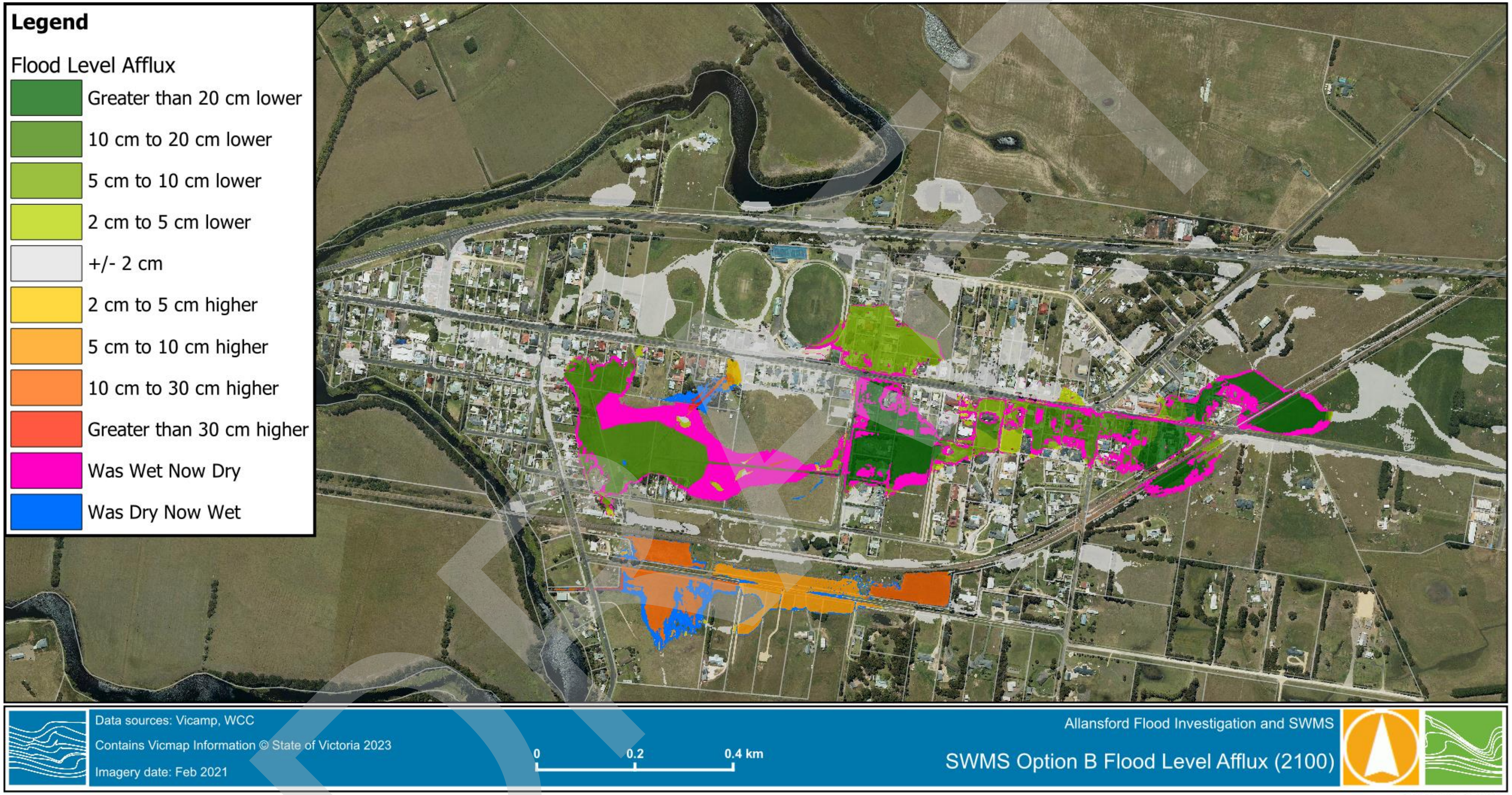

Figure 5-8 SWMS Option B – 1% AEP SSP5-8.5 2100 Flood Level Afflux

Figure 5-9 Proposed Infrastructure – Option C

Figure 5-10 SWMS Option C – 1% AEP SSP5-8.5 2100 Flood Depths

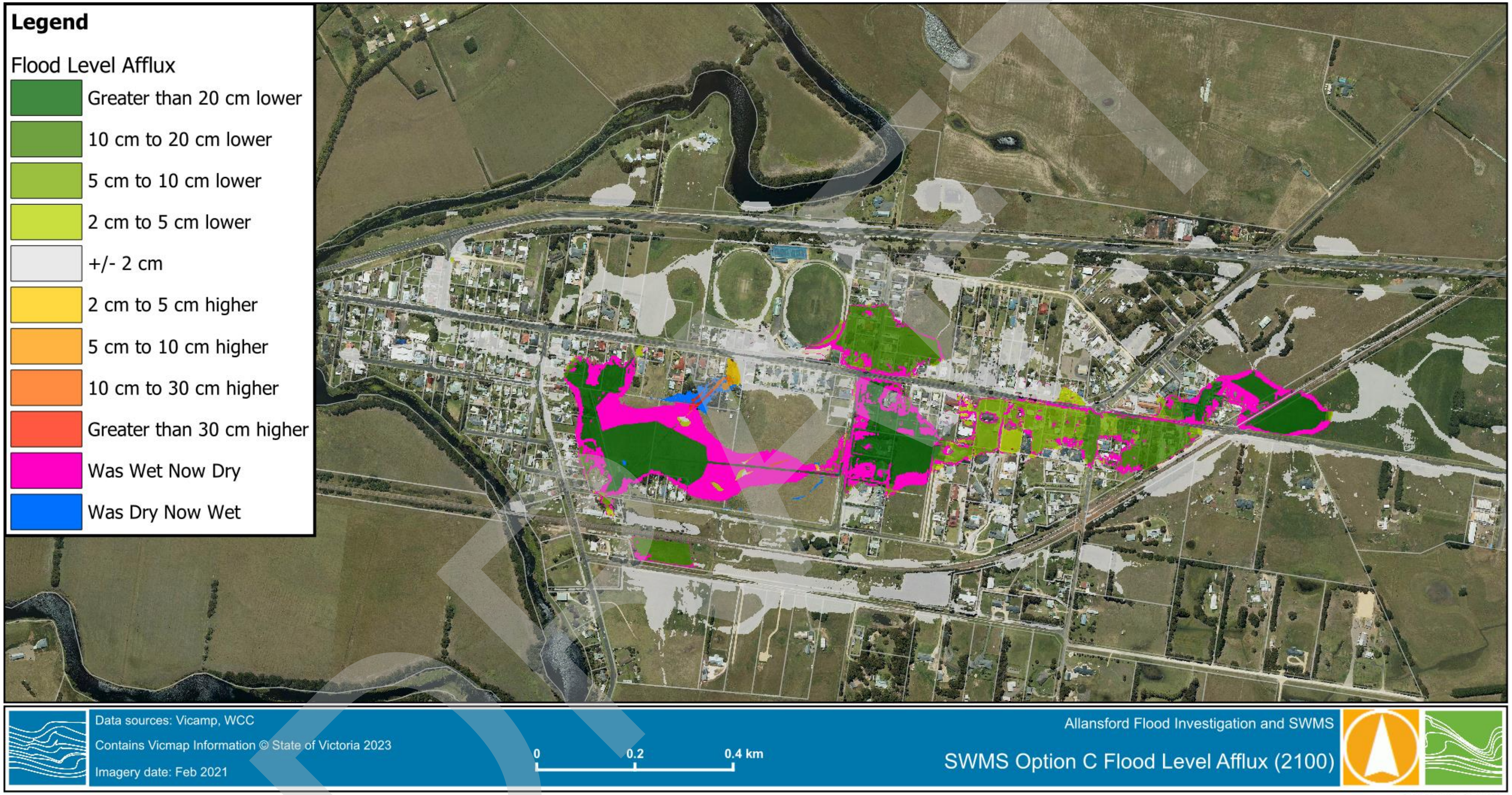

Figure 5-11 SWMS Option C – 1% AEP SSP5-8.5 2100 Flood Level Afflux

5.5 Water Quality Assessment

Water quality of stormwater runoff from Allansford was simulated within MUSIC (Model for Urban Stormwater Improvement Conceptualisation). The modelling aimed to conceptually size a basin for the treatment of stormwater from the town such that runoff meets the current Best Practice Environmental Management Guidelines (BPEMG) for stormwater treatment, which are:

- 70% removal of the Total Gross Pollutant Load (Litter);
- 80% removal of Total Suspended Solids (TSS);
- 45% removal of Total Phosphorus (TP); and
- 45% removal of Total Nitrogen (TN).

The township catchments as modelled in MUSIC are shown in Figure 5-12 below.

Figure 5-12 MUSIC Catchments - Allansford

Given the existing topography and flow paths present in the township, external catchments (EXT_S1, EXT_W2 and EXT_N1) do not flow into the proposed basin and thus are not treated. Two separate modelling set ups were completed: one which aimed to treat only catchments E1 and E2, and another which aimed to treat E1, E2 and W1. In each setup, the wetland was sized to achieve the BPEMG for only the catchments flowing into it, not the entirety of Allansford. Because of this, the option which treats a smaller area results in a smaller wetland, however this results in less pollutant removal from water entering the Hopkins River as a whole.

Ultimately any development of a wetland should utilise as much land is available, given that Allansford currently has no formal stormwater treatment systems or assets in place. This will likely be guided by the flooding and the need to provide a basin to store floodwater as included in each of the mitigation options modelled for the SWMS. It is noted that wetland option 1 is likely to fit within the currently designed basin (as modelled) while wetland option 2 will require a greater land take than currently proposed.

The two tested options are summarised in Table 5-1 below.

Table 5-1 Wetland Treatment Summary

	Option 1	Option 2
Area Treated	141 hectares	123 hectares
Allansford Area Untreated	87 hectares	105 hectares
Size of Wetland Treatment Area*	13,500 m^2	11,500 m^2
% Load Reduction – Suspended Solids (Allansford total)	50.7%	42.4%
% Load Reduction – Phosphorus (Allansford total)	43.6%	36.3%
% Load Reduction – Nitrogen (Allansford total)	30%	25.4%

* Note this is the treatment area, additional area would be required for construction of batters, access tracks etc.

6 RECOMMENDATIONS

The key recommendations made as a result of the investigations and modelling completed during this project are as follows:

1. Council engage with the Department of Transport and Planning (DTP) regarding the culvert north of the football ovals. An investigation of the intended purpose of the culvert should be carried out and, if possible, the culvert decommissioned or otherwise blocked if deemed safe to do so.
    a. Failing this, a note should be placed in the Warrnambool MFEP to manually block this culvert if significant flooding in the Hopkins River is expected.
2. While decommissioning is not an option for the Garibaldi Lane culvert, backflow prevention should be considered for this culvert. Until permanent backflow prevention (e.g. a flap-gate) is installed, a note should be added to the Warrnambool MFEP to manually block the culvert if significant flooding in the Hopkins River is expected.
3. Further community consultation and education should be carried out regarding the flap-gate installed on the Tooram Road outlet. Future upgrade works should consider the viability of installing a penstock gate in the upstream pit to allow isolating the outlet from the river manually without the need for sandbags etc.

In addition to the above drainage recommendations, the following recommendations are made regarding the findings and outputs of the study:

4. Council and Glenelg Hopkins CMA consider the draft planning scheme amendment mapping with a view to updating the planning scheme.
5. Council give further consideration to flood mitigation works, with further investigation into the feasibility and constructability of works to alleviate flooding in Allansford.
6. DEECA consider the findings of the Hopkins River at Hopkins Falls gauge review.
7. Council investigate funding opportunities and cost-share arrangements to implement option C of the SWMS.

7 SUMMARY

The Allansford Flood Investigation and Stormwater Management Strategy has investigated inundation at Allansford from both a riverine and stormwater perspective. Riverine flooding at Allansford is typically driven by prolonged rainfall on a saturated Hopkins River catchment (~8700km^2), while stormwater inundation is typically caused by much shorter, more intense storms on the 2.4 km^2 catchment.

Inundation of Allansford from the Hopkins River occurs via two mechanisms: backflow into the stormwater network and overtopping of the highway. Flooding from the stormwater network can be managed via the implementation of backflow prevention such that the former mechanism no longer presents a threat. Overtopping of the highway can be prevented through the use of a series of flood walls/levees however these are likely to cause worsened flooding on residential properties within the town. Such impacts can potentially be managed with significant earthworks however the viability of such a plan has not been tested.

Non-structural mitigation of future riverine flooding impacts has been proposed in the form of flood related planning scheme mapping. Mapping has been based on the 1% AEP SSP5-8.5 2100 flood event as per Glenelg Hopkins CMA's guidance. Land Subject to Inundation and Floodway have been delineated according to categorised hazard as per the same guidance.

Stormwater management infrastructure has been proposed with consideration of the available land and its form. A stormwater management strategy has developed three distinct options for infrastructure development aimed at reducing the extent and depth of inundation in Allansford, with a single option recommended for implementation once funding is sourced. Water quality has been considered and a conceptual wetland proposed to afford some treatment to runoff originating within Allansford, noting that the entire catchment does not meet the Best Practice Environmental Management guidelines for stormwater under the proposed arrangement.

Flood mapping outputs have been provided in GIS and PDF format, with a set of riverine mapping outputs provided to the FloodZoom platform in the required format.

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