West Gippsland Catchment Management Authority

WGCMA Floodplain Mapping Program

Floodplain mapping for Shady Creek, Welshpool

May 2025

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1 Introduction

1.1 Purpose

The project seeks to improve flood information for the study area where there is currently a knowledge gap as identified by the West Gippsland CMA Regional Floodplain Management Strategy (2018-2027).

1.2 Objective

The primary objective of the Flood Study is to conduct a desktop study of the catchment to identify the model requirements and key features which can then be used to develop a suitably robust hydrologic and hydraulic modelling system. The completed model can then be used to assist in Statutory Planning by defining flood behaviour, peak flood levels and inundation extents within the study area.

1.3 Catchment description

The Shady Creek catchment is located 1 km to the north of Welshpool town centre and 162 km east of Melbourne (Figure 1-1). Shady Creek forms part of the Corner Inlet area of the West Gippsland Catchment Authority region and is part of the South Gippsland Shire Council area. The catchment of Shady Creek above the township has a catchment area of approximately 13 km^2. Above the town of Welshpool, Shady Creek has a typically well-defined channel which develops into an expansive floodplain. The upper and mid catchment is underlain by predominantly sandstone becoming alluvial throughout the lower reaches of the study area.

Most of the catchment has been cleared for a range of urban and agricultural land uses. With a current population of 361 Welshpool has well-developed infrastructure including roads, housing, commercial buildings and recreational spaces. However, the major land use of the Shady Creek catchment is grazing.

There is limited underground stormwater drainage in and around the town. As such, stormwater is primarily conveyed through curb gutters adjacent to the roadways and discharging into the creeks.

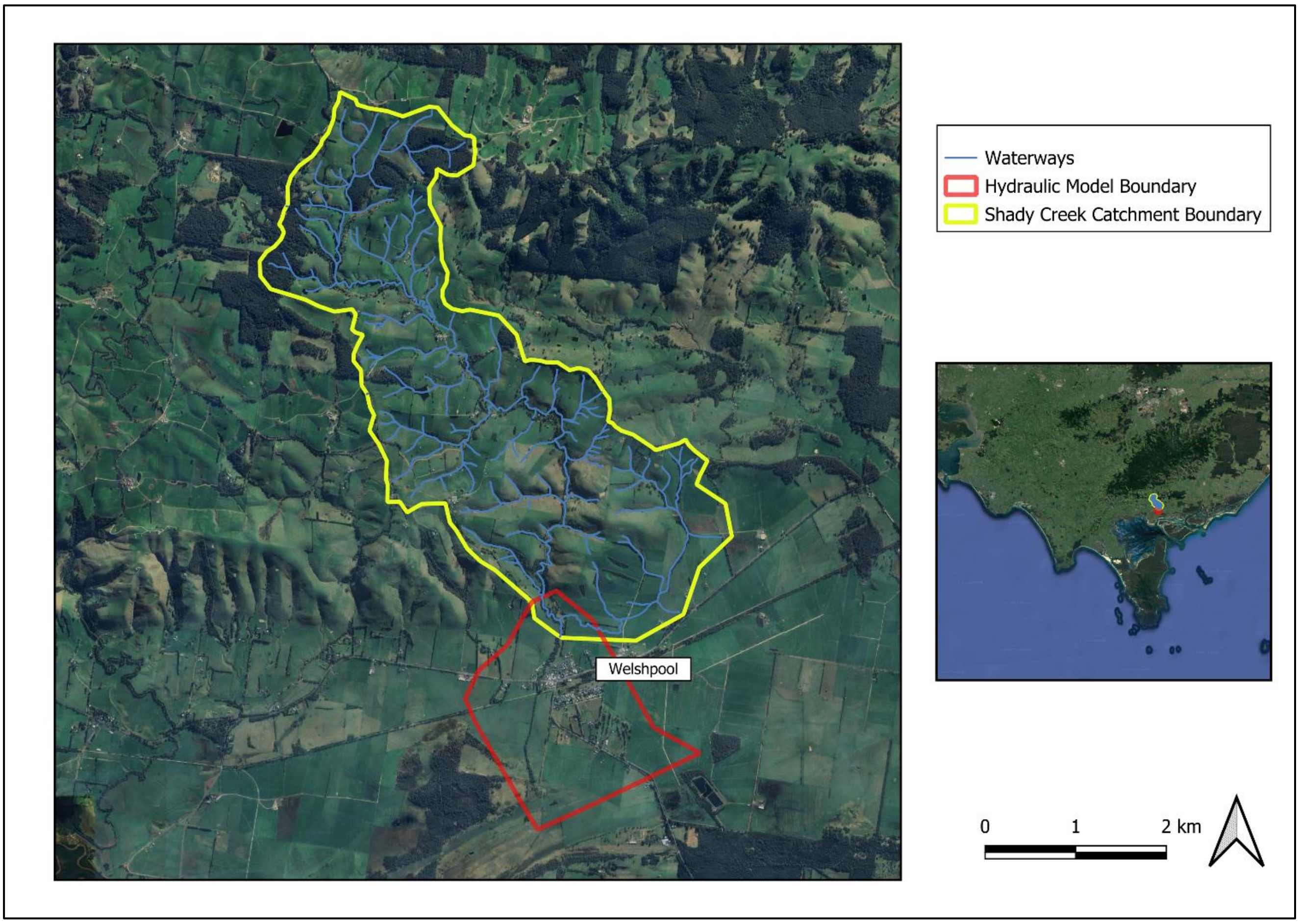

Figure 1.1- Shady Creek Catchment

1.4 Flood history

There have been several contemporary and historical storm events that are known to have caused flooding in the Welshpool township, though detailed information within the catchment is scarce. The most recent significant event occurring on the 26th of December 2023 (Figure 1-3, 1-4 and Appendix 1). No rainfall gauges are currently active in the catchment however, 145 mm of rainfall was recorded at Mount Best-Upper Toora station gauge 12.5km away (BOM, 2024). As a result of the storm there were multiple flooding events throughout the region and data collected by the South Gippsland Shire Council indicates that over floor flooding* occurred at 25 properties (Figure 1-5).

*" Over floor flooding" refers to a situation where floodwaters rise above the ground level and inundate the interior spaces of buildings or structures

Figure 1.-1 South Gippsland Highway, Welshpool looking east toward the Welshpool Hotel, Tuesday 26th December 2023 (source: Vic Emergency)

Figure 1.-2 South Gippsland Highway, Welshpool heading west, Tuesday 26th December 2023 (source: Vic Emergency)

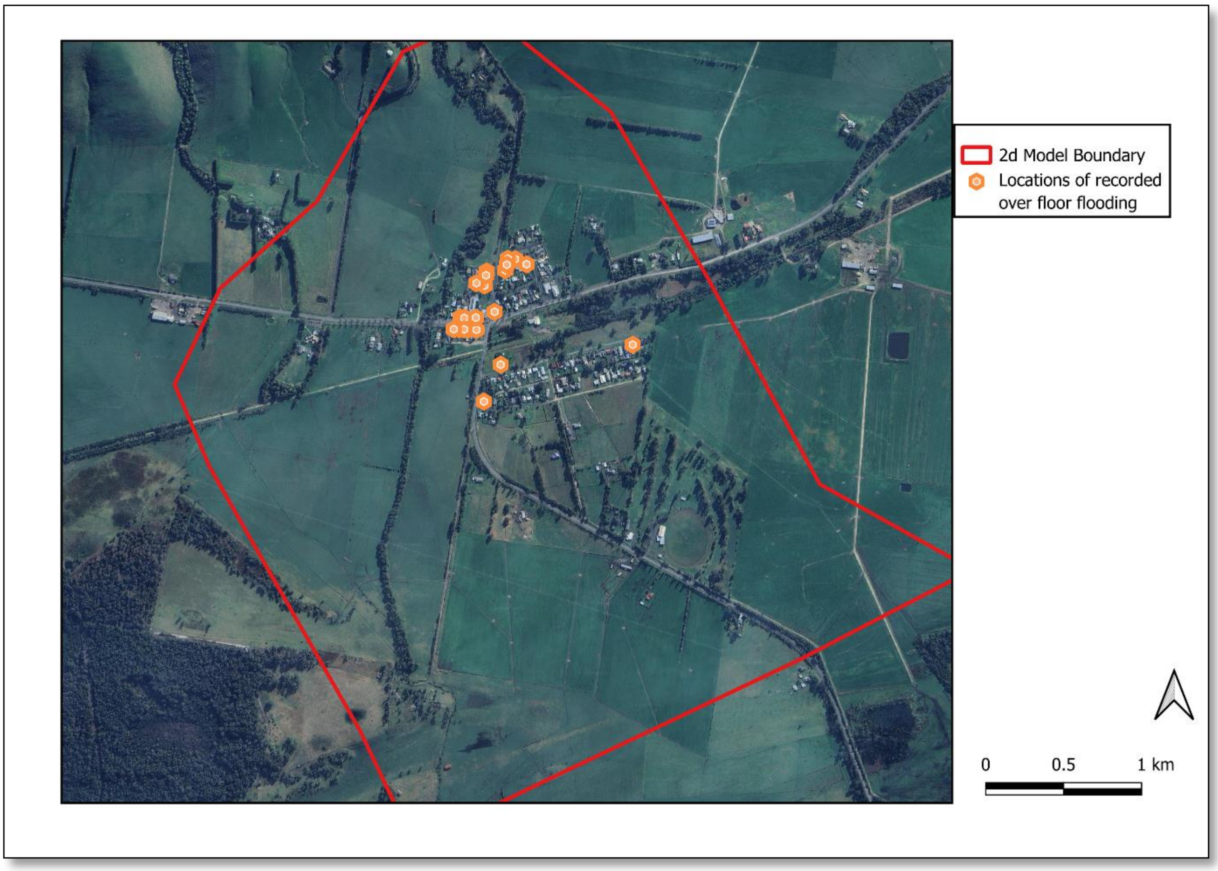

Figure 1.-3 Recorded incidences of over floor flooding in Welshpool township (December 2023).

Historically, floods have occurred in the town and evidence sourced from newspapers in the Trove database suggest that flooding events occurred in March 1911, December 1934, and November 1954.

November 1954 – "In the small townships of Gelliondale, Alberton, Hedley and Welshpool were flooded in some places, with a foot of water running through them." (The Herald (Melbourne, Vic), 1954) "The train was stopped at Welshpool, where floodwaters have almost surrounded the town and are flowing over the rail track". (The Argus 1954)

December 1934 – "At Welshpool the floods were the most serious in history. After 8in. of rain had fallen, streets and roads were submerged, and everywhere washaways and landslides made transport impossible. *(The Argus (Melbourne, Vic) - Wed 5 December 1934)*

March 1911 – "A heavy thunderstorm broke over the district on Wednesday evening. Shady Creek overflowed its banks, and the flood rushed through the township, entering several business places. At the Welshpool Hotel there was 4in. of water in all of the rooms excepting in the back cottage, which is much higher than the main building. This is the second occasion within a few weeks on which there has been a flood through the hotel, and much damage has been done". (The Argus (Melbourne, Vic), 1911)

2 Hydrology

2.1 Description of hydrologic modelling approaches adopted.

The aim of the hydrological modelling is to calculate runoff at locations throughout the study area for input into the TUFLOW hydraulic model. When determining the hydrological response of the study area, there are several factors that need to be considered. These include catchment characteristics, design rainfalls and model parameters determined through model calibration.

Catchment and sub-catchment areas together with other physical catchment characteristics were determined from topographic information. Once the physical characteristics of a catchment have been determined and design rainfall calculated it is necessary to determine the hydrological model parameters. These parameters are commonly determined through calibration. The approach to calibration is dependent on the available data. If there is sufficient data available, the hydrological model should be calibrated to this data. As a minimum this would require streamflow data at one location.

2.2 Rainfall and Streamflow Data

Streamflow, pluviographic and daily rainfall records are required for the hydrological model calibration. pluviographic rainfall data is used to understand the temporal distribution of rainfall during calibration events while daily rainfall data provides the spatial variation and rainfall depths for the specific calibration event. There are currently no operating rainfall or stream gauges in the Shady Creek catchment.

Rainfall Gauge Data

There are currently no rain gauges in the Shady Creek catchment therefore the absence of historical rainfall data, specifically continuous rainfall observations represent a significant data gap. A network of continuous and daily read gauges exists in the broader region, which may be used to provide insight into rainfall behaviour during historical flood events. The rain gauges in the vicinity of Shady Creek are listed in Table 1.

Table 1 Available weather station data record including operation period, types and distance from Welshpool.

Distance (km)	Station Number	Station Name	Operational Dates	Type
1.22	085094	Welshpool	1889-2012	Daily
4.15	085037	Hazel Park	1932-1966	Daily
5.51	085001	Agnes River	1900-2000	Daily
8.48	085109	Binginwarri	1906-1937	Daily
9.32	085120	Headley (Vivaleigh)	1899-1917	Daily
9.88	085084	Toora	1905-2021	Daily
12.55	085063	Mount Best (Upper Toora)	1903 - current	Daily
18.8	085053	Madalya	1900- current	Daily
25.98	085301	Corner Inlet (Yanakie)	2013 - current	Continuous
29.04	085151	Yarram Airport	2010 - current	Continuous

Rainfall data was obtained from the BOM for the two closest continuous reading stations (Corner inlet, and Yarram airport) and daily reading stations (Mount Best and Madalya) for the Boxing Day flood event. Analysis of the rainfall indicated that the continuous stations (Corner Inlet and Yarram Airport) likely received significantly less rainfall than the Shady Creek/ Welshpool catchment and was therefore unsuitable for use as a proxy temporal pattern in the model. A comparison of closest operating gauge readings for the Boxing Day flood event are shown in Table 2.

Table 2 Total Daily Rainfall readings for the Boxing Day flood event (26/12/2024)

Station	Daily Rainfall Total (mm)
Mount Best (Upper Toora)	145.0
Madalya	94.0
Corner Inlet (Yanakie)	61.0
Yarram Airport	36.2

Stream Flow Gauge Data

There are currently no stream gauges in the Shady Creek catchment, the absence of historical stream flow data represents a significant data gap. A list of the closest gauges to the Shady Creek catchment are shown in Table 3. Due to lack of available data the RORB model could not be calibrated.

Table 3 Nearest available stream flow gauge data

Distance	Gauge Number	Name	Operational Dates	Type
6km	227211	Agnes River @ Toora	1952-current	Instantaneous
27km	227200	Tarra River @ Yarram	1965 – current	Instantaneous

2.3 Regional Flood frequency estimation

Flood frequency estimation was provided by the ARR Regional Flood Frequency Estimation Model tool. The RFFE Model 2015 is based on the concept of regionalisation where data from gauged catchments are utilised to make flood quantile estimates at ungauged locations. Flood quantiles are estimated using a regional log Pearson Type 3 (LP3) distribution where the location, scale and shape parameters are estimated based on prediction equations. In the ARR RFFE model, the model coefficients have been embedded in an application software (known as RFFE Model 2015), which enables the user to obtain design flood estimates relatively easily using simple input data such as latitude, longitude and catchment area of the ungauged catchment of interest. Data retrieved from the RFFE tool are displayed in Table 5, it is worth noting that estimates can be inaccurate in small catchments as well as those located further away than 300km from the nearest gauged catchment location used to develop the RFFE technique.

Table 4 Design flows based ARR Regional Flood Frequency Estimation Model tool.

Average Exceedance Probability (AEP)	Flow based on flood frequency analysis
%	m^3/s
50	9.56
20	18.3
10	25.9
5	34.6
2	48.1
1	60.0

2.4 RORB hydrologic model

RORB is the standard hydrology model used by the West Gippsland Catchment Management Authority (WGCMA). RORB is a general runoff and streamflow routing program used to calculate flood hydrographs from rainfall and other channel inputs. In RORB the catchment is represented by network of sub-areas and reaches, rainfall is applied at the centroid of each sub-area and runoff is calculated by subtracting losses and multiplying the result by the area of the sub-area. The runoff from each sub-catchment is then routed from the centroid of that sub-catchment, along the main reach, to the next downstream node where the runoff hydrograph is combined with (a) runoff hydrographs from other tributaries and/or (b) rainfall excess hyetograph from the sub-catchment of the downstream node reach. The combined runoff hydrograph is then routed downstream to the next node until the outlet is reached.

The delineation of the catchment and sub-catchment for the model was done using ArcGIS Pro and LiDAR available to WGCMA. The model was then constructed from within RORB using the Graphical Editor available in the software and all information was input directly into the program (Figure 2-1).

Design flood events for the 20, 10, 5, 2, 1, 0.5, 0.2 and 0.1 AEP events were run for multiple durations for use in the Hydraulic model.

2.5 Model Schematisation

Figure 2.1 shows the RORB model layout within RORB's graphical editor.

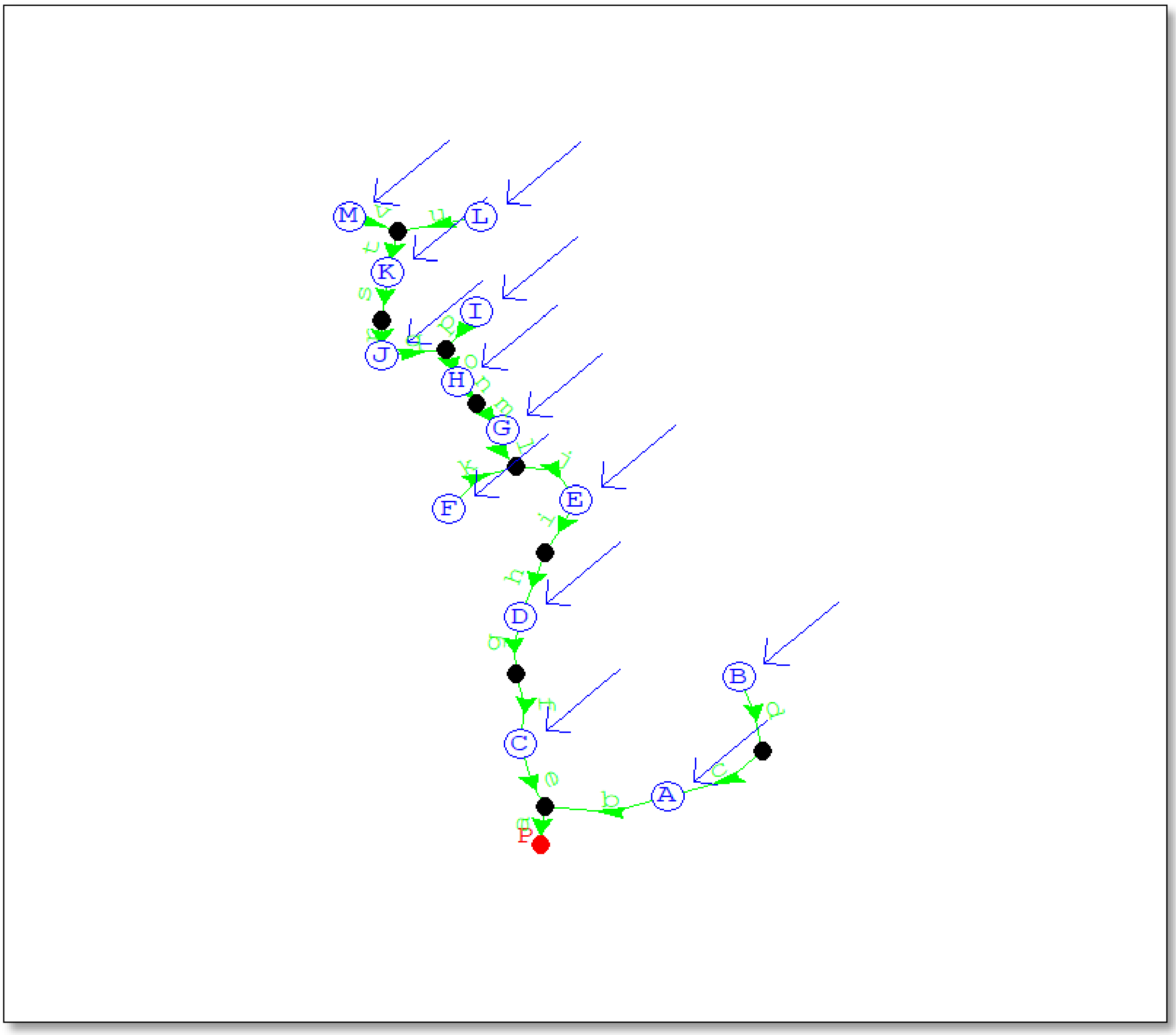

Figure 2-.1 Rainfall runoff and routing model schematisation.

Sub-catchment Delineation and Reach Types

The hydrologic catchment area covered a region of 13.12 km^2. This area was defined by the topographical ridges that form the upper bounds of the watershed area. The most downstream point in a catchment, the catchment outlet was established approximately 700 meters beyond the final tributary convergence. The development of the sub-catchments was based on the stream network and the drainage characteristics of the catchment. Where possible similarity in sub-catchment area and shape was sought after and a total of 13 sub-catchments were delineated across the total hydrologic catchment which fits within the 5-20 sub-areas as recommended in the RORB manual. The catchment and sub-catchment extents are shown in Figure 2-2.

Reach types were determined from site visits and high definition basemaps/imagery. Much of the flow within the catchment is via natural unlined channels reflecting the predominantly rural nature of the catchment. Therefore, Reach Type 1 was adopted for all reaches.

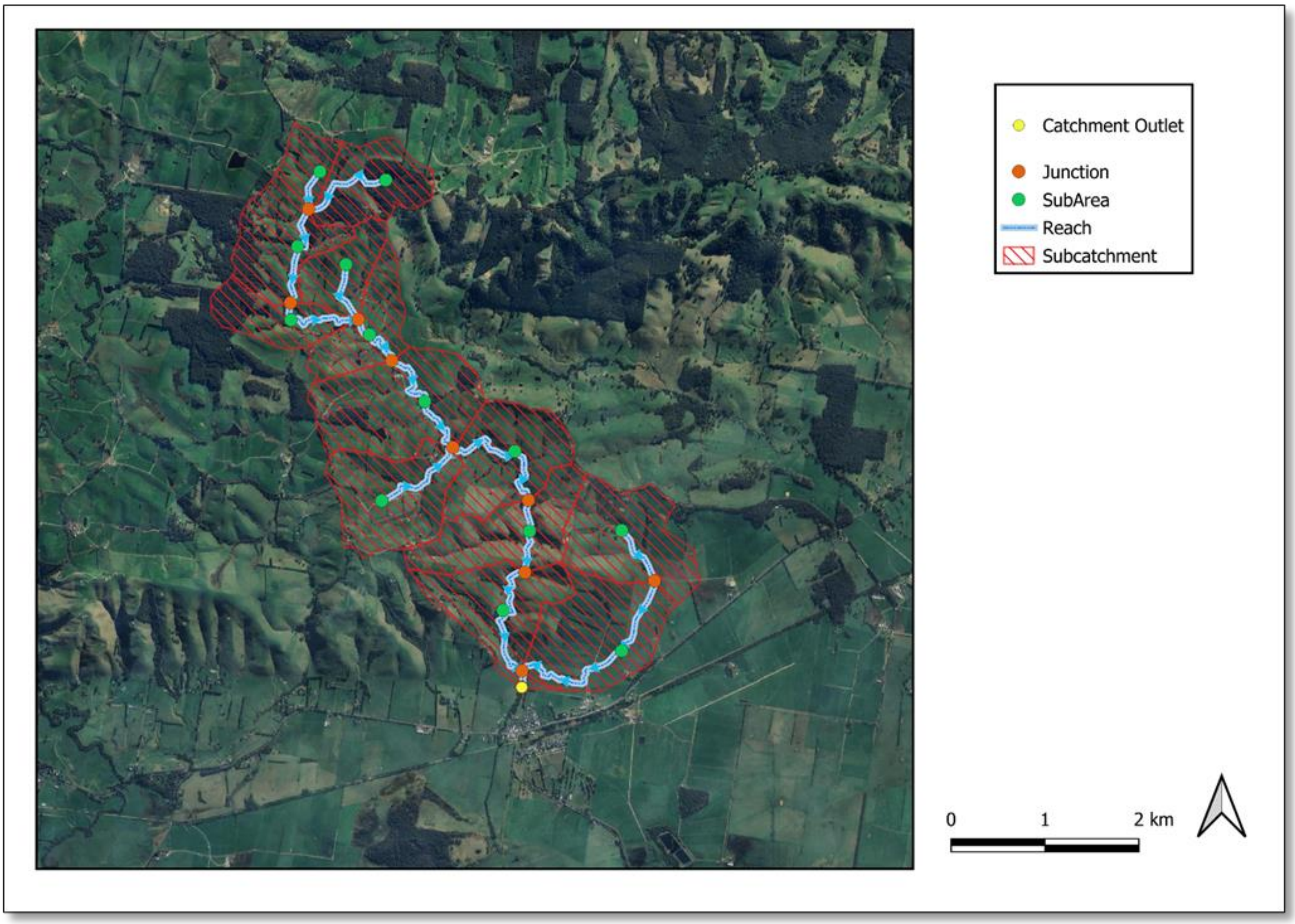

Figure 2-.2 RORB Hydraulic model sub-catchment delineation with Outlet, Sub-catchment and Junction nodes.

Fraction Impervious Data

The RORB model requires an input of fraction impervious values for the subareas. The total imperviousness of the catchment was set as 0.1 as derived from Melbourne Water's fraction effective impervious for different land uses. This figure reflects the predominantly rural nature of the catchment.

Intensity Frequency Duration (IFD) Parameters

Storm data was generated using IFD parameters sourced from the Bureau of Meteorology 2016 Design Rainfall program (Bureau of Meteorology, 2024). The design rainfall intensities for centroid of the catchment upstream of Welshpool obtained from the BOM website are shown in Table 6.

Table 5 IFD Table for Welshpool (Centroid: -38.6375S, 146.4375E)

Storm Duration	1 EY	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.50% AEP
1 min	1.42	1.6	2.2	2.63	3.08	3.7	4.21	4.75
2 min	2.41	2.74	3.84	4.66	5.52	6.78	7.76	8.86
3 min	3.24	3.68	5.14	6.22	7.35	8.98	10.2	11.7
4 min	3.95	4.48	6.22	7.5	8.82	10.7	12.2	13.8
5 min	4.57	5.17	7.14	8.58	10.1	12.2	13.9	15.7
10 min	6.82	7.68	10.5	12.5	14.6	17.5	19.9	22.4
15 min	8.34	9.39	12.8	15.3	17.8	21.4	24.3	27.3
20 min	9.51	10.7	14.6	17.5	20.4	24.5	27.9	31.4
25 min	10.5	11.8	16.2	19.3	22.6	27.2	30.9	34.9
30 min	11.3	12.7	17.5	21	24.6	29.6	33.7	38.1
45 min	13.3	15	20.8	25	29.4	35.6	40.7	46
1 hour	14.8	16.8	23.4	28.2	33.3	40.5	46.4	52.5
1.5 hour	17.3	19.7	27.6	33.5	39.6	48.5	55.7	63
2 hour	19.3	22	31.1	37.8	44.8	55	63.3	71.5
3 hour	22.7	25.9	36.7	44.7	53.2	65.4	75.5	85.1
4.5 hour	26.6	30.4	43.3	52.9	63	77.6	89.7	101
6 hour	29.9	34.2	48.7	59.5	71	87.3	101	113
9 hour	35.4	40.4	57.4	70.1	83.6	103	119	133
12 hour	39.8	45.3	64.3	78.5	93.6	115	133	149
18 hour	46.8	53.2	75	91.5	109	134	155	174
24 hour	52.3	59.3	83.3	101	121	148	172	194
30 hour	56.8	64.2	89.9	109	130	160	185	214
36 hour	60.6	68.4	95.4	116	138	170	197	229
48 hour	66.7	75	104	126	150	185	214	250
72 hour	75.3	84.3	116	140	166	205	237	275
96 hour	81.1	90.6	123	149	176	216	250	287
120 hour	85.5	95.2	129	155	182	222	257	293
144 hour	88.9	98.8	133	158	186	224	260	296
168 hour	91.7	102	136	161	188	224	260	296

Loss Model

RORB generates rainfall excess (runoff) by subtracting losses at each time-step from the rainfall occurring in that period. The "initial loss followed by a continuing loss" loss model was adopted. The adopted initial loss and continuing loss were 29.0 mm and 3.8 mm/hr respectively as recommended by ARR datahub when using the catchment centroid (Figure 2-3). A Kc value of 8.11 was reviewed using the regional equation (Equation 1) for eastern

Victoria regions with mean annual rainfall greater than 800 mm. This value was compared to the “Victorian” equation (Equation 2) from Pearse et al. (2002) and found to be similar (6.02). The lower value of 6.02 was adopted (refer to section 2.8 Assumptions for further discussion). An m value of 0.80 was used in accordance with the RORB manual for use in ungauged catchments.

Equation 1:

$K_c = 2.57A^{0.45}$ (ARR Book 7, Eqn 7.6.15, Hansen et al. (1986a, b))

Equation 2:

$K_c = 1.25d_{av}$ (“Victorian” equation from Pearse et al. (2002))

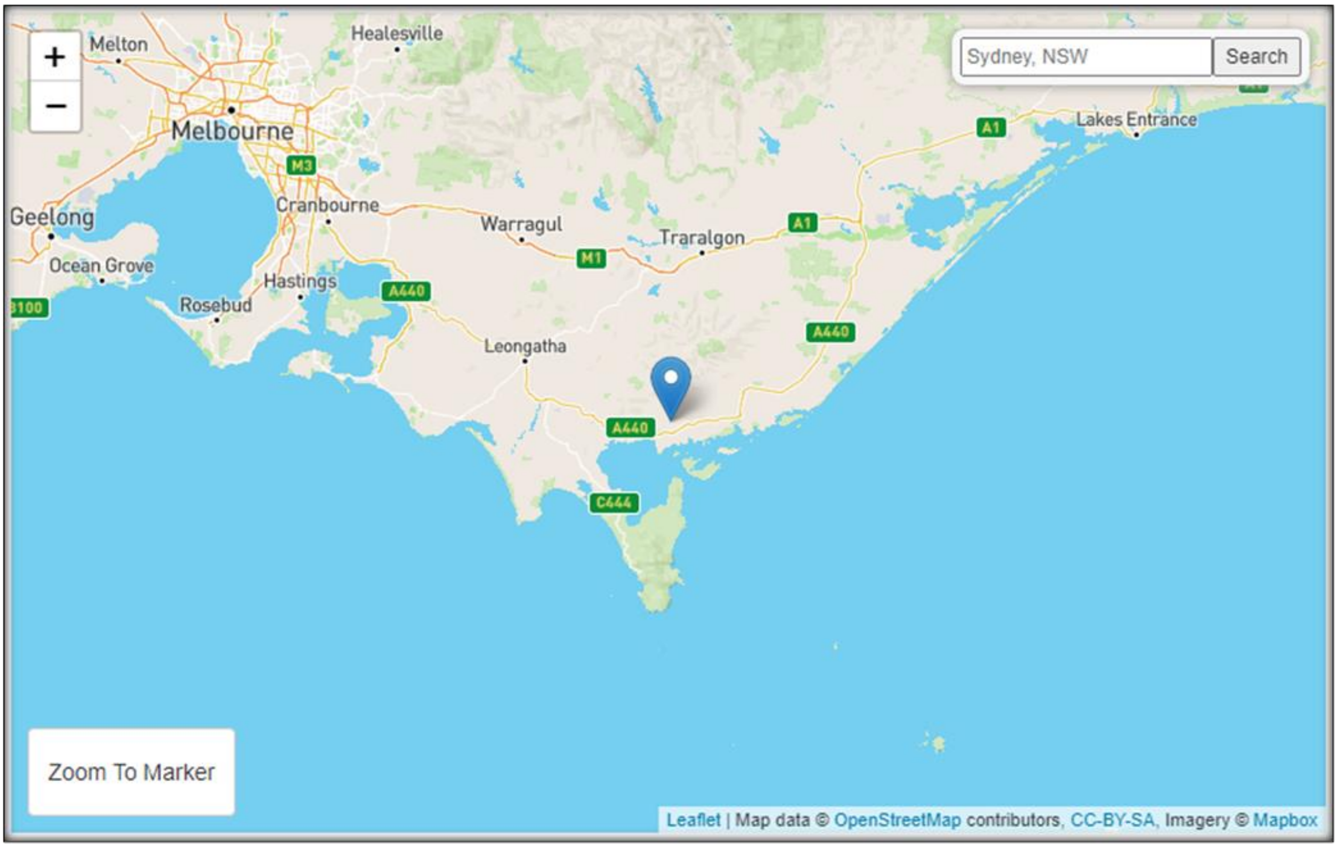

Figure 2-.3 Position of Catchment centroid used in the ARR Datahub

2.6 Monte Carlo Simulation

Design flood estimation within RORB was undertaken using the Monte Carlo simulation method for the catchment. The Monte Carlo approach involves undertaking thousands of simulations where the stochastic factors (such as rainfall, temporal patterns, and initial loss) are sampled to represent the joint probability of such factors to provide a more realistic representation of the flood peak.

The Monte Carlo method was selected as it recognises that design floods (e.g. peak flows) can result from a variety of combinations/factors, rather than from a single combination as is assumed with the typical ‘design event’ approach. For example, the same peak flood could result from a large, front-loaded storm on a dry catchment, or a moderate, more uniformly distributed storm on a saturated catchment. In the absence of rainfall and stream gauges within a given catchment the Monte Carlo simulation is particularly useful in deriving estimates of design flood characteristics without introducing additional biased from storm parameter and temporal pattern selection.

The simulation used a range of rainfall depths, durations, temporal patterns, and initial losses to produce a flood frequency distribution. Initial loss values are taken by sampling within an expected variability range of the original value (i.e. 29 mm), while rainfall depths/durations, temporal patterns and areal reduction factors are drawn from the information sourced by Data Hub. The flood frequency distribution is used to estimate the peak flow for each ARI event and identify critical storm durations and temporal patterns within the Monte Carlo result output.

Design Runs

RORB Parameters used in the Design runs were adopted from the Monte Carlo simulation run. The critical duration which produced the maximum flows was different for specific design floods such that 20%, 10%, 5%, 2%, 1%, 0.5%, 0.2% and 0.1% AEPs had a critical duration of 12 hours (Table 8) and 5%, 2%, 1%, and 0.5% AEPs had a critical duration of 9 hours (Table 9). The storm events that produced the maximum flows for each critical duration and AEP are listed in Table 8 and Table 9. Each of the events were modelled separately in RORB using the parameters listed in conjunction with the established kc value of 6.02, continuing loss of 3.8, and m value of 0.8.

Table 6 12hr RORB parameters used in design runs.

AEP (%)	ARI	Depth	TPat	Initial Loss	Peak 01
20	5	62.8	4	14.21	5.82
10	9.7	76	13	17.4	10.56
5	19.8	90.8	17	39.73	14.75
2	50.8	111.8	30	35.38	25.32
1	99	128.1	30	11.31	33.95
0.5	197.2	143.4	25	76.27	39.37
0.2	525.9	171.3	22	27.55	51.68
0.1	997.8	190.6	29	14.21	59.79

Table 7 9 hr RORB parameters used in design runs

AEP (%)	ARI	Depth	TPat	IL	Peak 01
5	20.9	81.9	13	8.41	15.19
2	49.7	99.3	29	48.72	25.29
1	102.3	114.8	27	27.26	34.38
0.5	199.5	127.8	27	25.81	40.91

2.7 RORB results

Design flood hydrographs were extracted for the two storm durations and 12 AEP to produce maximum flows for the study area. Results from the 9 and 12-hour duration design rainfall events are shown in Table 10 and Figures 2-4 and 2-5 respectively.

Table 8 Design flows at model outlet from RORB model

AEP (%)	Flow at outlet based on RORB design run model (m^3/s)	
	9 Hour	**12 Hour**
20	N/A	5.8
10	N/A	10.7
5	14.9	14.8
2	25.2	25.1
1	34.2	34.1
0.5	40.8	39.3
0.2	N/A	51.0
0.1	N/A	59.6

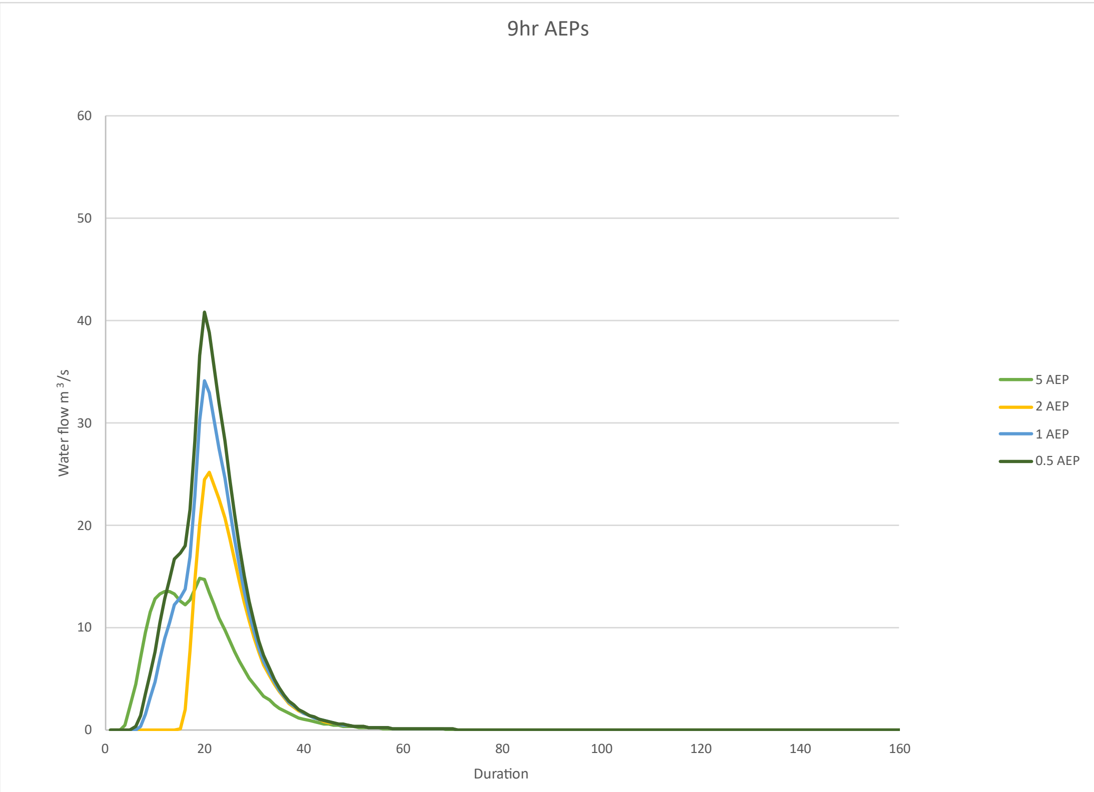

Figure 2-.4 Rainfall hydrograph for 9-hour duration design events 5 ,2, 1 and 0.5 AEPs.

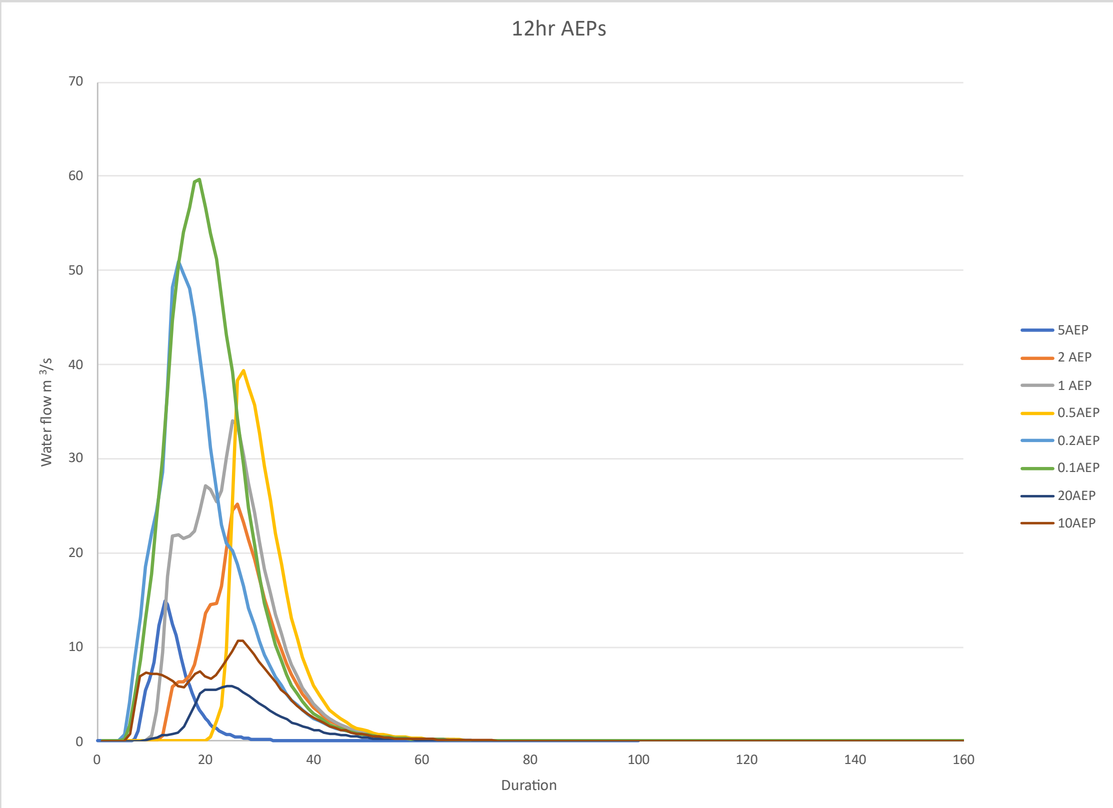

Figure 2-.5 Rainfall hydrograph for 12-hour duration design events 20, 10, 5, 2, 1, 0.5, 0.2 and 0.1 AEPs.

In ungauged catchments where calibration is not possible a suitable proxy model validation can be performed by measuring the similarity between a modeled result and a real-life scenario. Therefore, a general comparison was made by inputting the above 9- and 12-hour hydrographs into a 2D 5 x 5m grid TUFLOW hydraulic model. The model extents were compared against observed flooding by residents and data collected by the South Gippsland Shire Council for the 2023 Boxing Day flood event. The 12 hour 0.1 AEP modelled extents were consistent with observed flooding. The results of the review demonstrated that the modelled flows resulted in flood extents that were consistent with observed flooding from 2023 Boxing Day flood event. This suggests that the hydrographs produced by RORB are likely to be appropriate and therefore fit for purpose.

The results suggest that the 2023 Boxing Day flood event may have been a 1 in 1000-year flood event.

2.8 Assumptions

Kc value: The use of regional data in an ungauged catchment is standard practice, however, the results are expected to have a higher level of uncertainty attached. Additionally, the smaller size of the catchment is below the regional threshold of 38km^2 which is why the Pearse equation may be more suitable.

When adopting a Kc value of 6.02 the suitability of fitness was evaluated by calibrating the resultant AEP 1% flow of 34.1 m^3/s to the RFFE estimate of 60 m^3/s. It was found that to reach 60 m^3/s the Kc would need to be adjusted to 2.1 (Figure 2-6) if the initial loss and continuing loss were maintained at 11.31 and 3.8 respectively. Alternatively, by reducing the initial loss and continuing loss to 0 a Kc value of 3.25 would be required to meet the 60 m^3/s. Using reverse substitution into Equation 1 the area represented by these Kc values would be approximately 0.64km^2 and 1.68km^2 compared to the actual area of 13.12km^2.

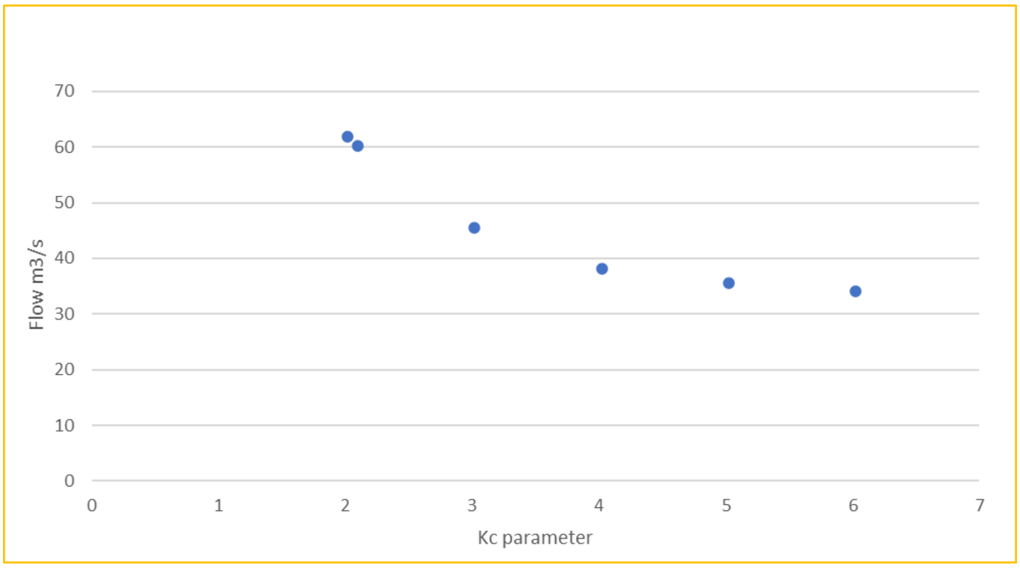

Figure 2-.6 Welshpool catchment flow rate versus RORB Kc parameter when initial loss and continuing loss are maintained at 11.31 and 3.8 respectively.

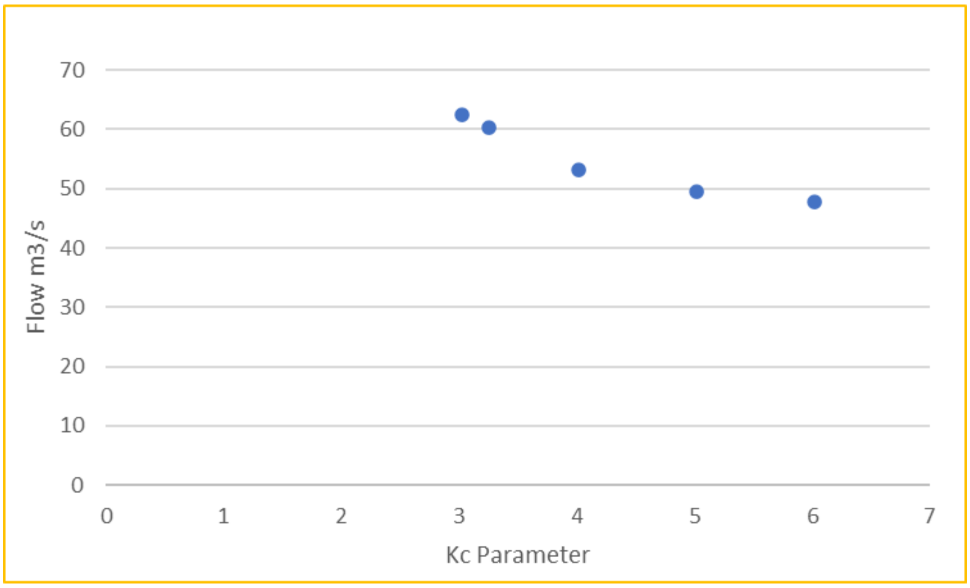

Figure 2-.7 Welshpool catchment flow rate versus RORB Kc parameter when initial loss and continuing loss are reduced to 0.

2.9 Summary of hydrology results

A RORB hydrological model was used to generate design flows for the study. The model was then used to generate design flows for the 20%, 10%, 5%, 2%, 1%, 0.5%, 0.2% and 0.1% AEP events. The choice of hydrological model parameters used to generate design flows was checked against observed flood data using a 5 x 5 grid 2D TUFLOW model and has been found to adequately represent observed flood behaviour. The design flows indicate that the Boxing Day 2023 flood event was approximately a 1000-year ARI event in Welshpool.

Table 2-9 Summary of design flows based on estimates and model

Average Exceedance Probability (AEP)	Flow based on Australian Regional Flood Frequency Estimation Model (RFFE-ARR)	Flow at outlet based on 9-hour RORB design run model	Flow at outlet based on 12-hour RORB design run model
%	m^3/s	m^3/s	m^3/s
20		N/A	5.8
10		N/A	10.7
5	34.6	14.9	14.8
2	48.1	25.2	25.1
1	60.0	34.2	34.1
0.5	N/A	40.8	39.3
0.2	N/A	N/A	51.0
0.1	N/A	N/A	59.6

2.10 Climate Change scenarios

Climate change scenario hydrographs were calculated based on Interim Climate Change Factors given by the ARR datahub in line with CSIRO and BOM (2015) recommendations. The WGCMA uses an RCP 8.5 projected to the year 2100. Values obtained for the Welshpool catchment were plotted in an Excel spreadsheet and extrapolated according to the linear equation below (Equation2).

Equation 2:

$$y = 0.0021x - 4.2501$$

The RCP 8.5 for the year 2100 was determined to be equivalent to an 18.3% increase in rainfall.

3 Hydraulic Modelling

This section provides a description of the TUFLOW modelling process undertaken for the catchment. A 2-dimensional TUFLOW hydraulic model was developed as part of this study with the aim of flood mapping the catchment for the calibration and design flood events. TUFLOW is a computer program that models depth-averaged, one and two-dimensional free-surface flows and is used to simulate the hydrodynamic behaviour of rivers, floodplains and urban drainage environments. The software is well-suited to small scale catchment studies such as the Welshpool Flood Study, as it is equally capable of modelling stream network and floodplain environments such as those found in the Shady Creek environments.

3.1 Model description

To produce flood extents, depths, velocities and other hydraulic properties for the study area a 2D hydraulic model was developed using TUFLOW. The area modelled within the 2D domain comprises a total area of 3.503 km^2 which represents the entire town of Welshpool and surrounding infrastructure. Shady Creek, including its floodplains and the town of Welshpool, were represented in the 2D domain.

Model Schematisation

The floodplain topography and other significant hydraulic features, such as roads and bridges, were represented within the 2D domains. A 2D domain with a 1m grid resolution was used to represent the floodplain. The major watercourse, Shady Creek was represented in the 2D domain of the hydraulic model. External inflows boundaries were applied to the model to represent flow in Shady Creek. No internal inflow boundaries were modelled.

3.2 Hydraulic Modelling Overview

The following sections provide an overview of methodology and assumptions used to establish the key elements of the hydraulic model.

TUFLOW Model Version

Model runs were performed with the TUFLOW HPC 2020-01-AB-iSP-w64.

Design Event Modelling

The hydraulic model that was run for each of the design events as well as the critical durations of 9 and 12 hours are discussed below. The following events were run in the hydraulic model:

- 5% AEP (20-year ARI) event;
- 2% AEP (50-year ARI) event;
- 1% AEP (100-year ARI) event;
- 0.5% AEP (200-year ARI) event;
- 0.2% AEP (500-year ARI) event; and
- 0.1% AEP (1000-year ARI) event.

TUFLOW model runs were controlled through a TUFLOW Event File (.tef) and a series of batch files constructed for use in this project. The use of the .tef file and batch files ensures that the base .tcf (TUFLOW Control File) does not change between runs, with all event specific parameters specified in the .tef file. This reduces the potential for error and assists in reducing model run and processing times.

Model Extent

Consideration was given to the following in constructing the model:

- Desired accuracy to meet the study's objectives.
- Topographic data coverage and resolution.
- Location of controlling features (e.g. Catchment Stream outlet and out of channel flow).

The upper bounds of the 2D domain were established based upon the final convergence of Shady Creek as it exits the upper catchment in addition to LiDAR data extent, while the lower boundaries were based on the extent of available single origin LiDAR data. The area modelled within the 2D domain comprises a total area of 3.503 km^2 which represents the entire town of Welshpool and surrounding infrastructure (Figure 3-1).

Parameters and settings

TUFLOWs HPC mode used an adaptive timestep and the grid size that was adopted for this model was 1x1 m. This grid size meant that the 0.1 % AEP (1 in 1000 ARI) model finished its simulation in approximately 26 hours. This level of detail enabled us to provide fine enough resolution to maintain the DEM definition as well as maintain a channel width around 6 cells across in most areas of the study.

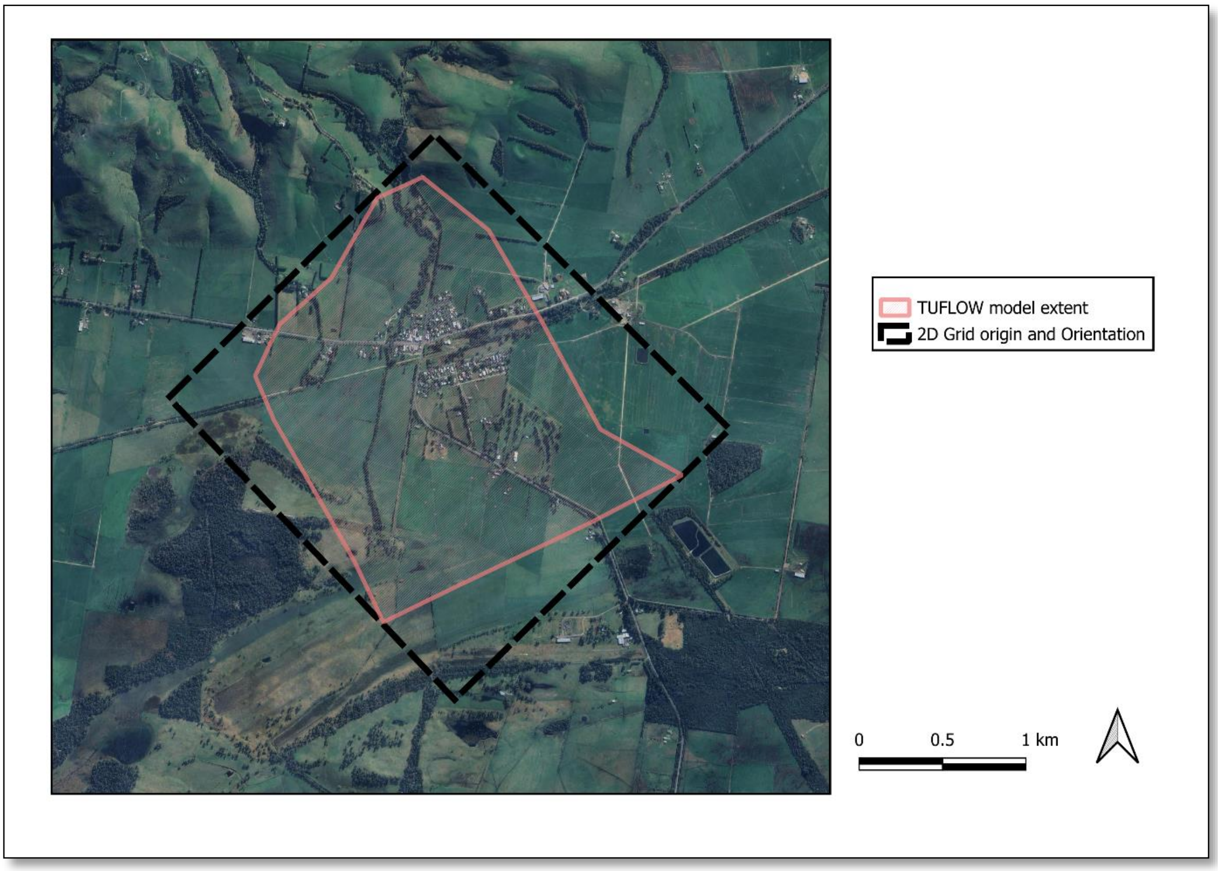

Figure 3-.1 TUFLOW Hydraulic Model Extent

3.3 2D Domain

Topography

The geometry of the 2D floodplain and watercourses were established by constructing a uniform grid of square elements from the DEM. This TUFLOW grid (or zpt layer) provides the topography for the hydraulic model. The DEM used in the hydraulic model was based the LiDAR available to the WGCMA (Table 11). The DEM was converted to an ASCII file for use in TUFLOW. The extent of the utilised DEM is illustrated in Figure 3-2.

Table 10 Digital Elevation Dataset Summary

Dataset	Resolution	Vertical Accuracy (1 sigma)
2010-11 Floodplains LiDAR Stage 2 – West Gippsland	1 m x 1 m	± 0.1 m

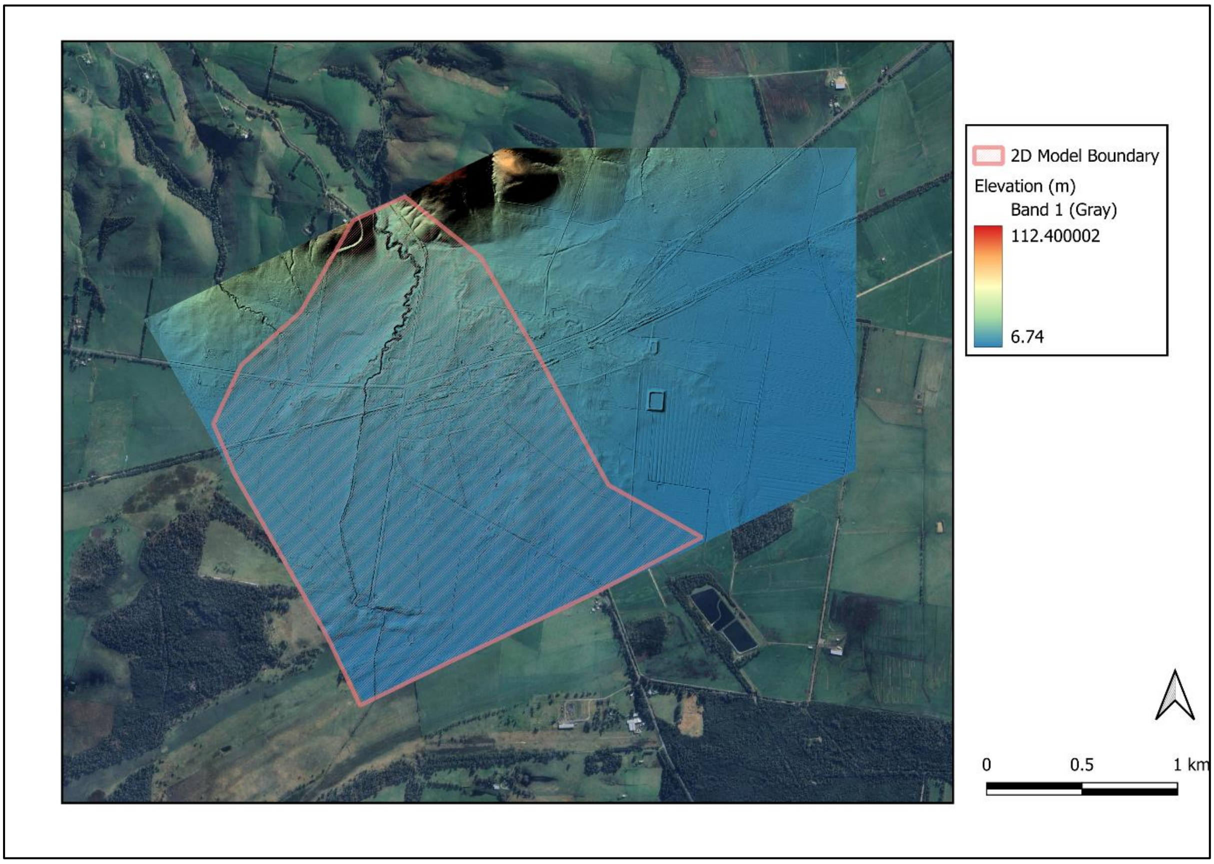

Figure 3-.2 Available DEM data for Hydraulic Model

Grid Resolution

One of the key considerations in establishing a 2D hydraulic model relates to the selection of an appropriate grid element size. Element size affects the resolution, or degree of accuracy, of the representation of the physical properties of the study area as well as the size of the computer model and its resulting run times. TUFLOW samples elevation points at the cell centres, mid-sides, and corners therefore, a 4 m cell size results in DEM elevations being sampled every 2 m. Selecting a very small grid element size will result in both higher resolution and longer model run times.

A 1 m grid cell size has been adopted for the Welshpool model. This grid size was selected due to the shorter processing time taken to model a catchment of this size and simultaneously give necessary detail required for accurate representation of floodplain and channel topography and its influence on overland flows.

3.4 2D Hydraulic Features

It is important to ensure that large (2D grid size or larger) impediments and constrictions to flow are properly incorporated in the TUFLOW model. A site inspection was undertaken in the initial stages of the study to gain an appreciation of local features influencing flooding behaviour. Some of the key observations from the site inspection included the location and dimensions of existing infrastructure including bridges and culverts.

Bridges

Bridge structures were modelled as 2D flow constrictions The layered flow constriction also allows for typical bridge characteristics such as bridge deck height and thickness as well as any blockages associated with guard or handrails to be incorporated directly in the 2D domain. Photographs and locations of the bridge structures modelled are shown in Figure 3-3 and 3-4 respectively.

a) South Gippsland Highway

c) Rail Trail Creek Crossing

b) Pedestrian Bridge adjacent to Highway

d) Rail Trail bridge

Figure 3-.3 Photographs of Bridges in Welshpool

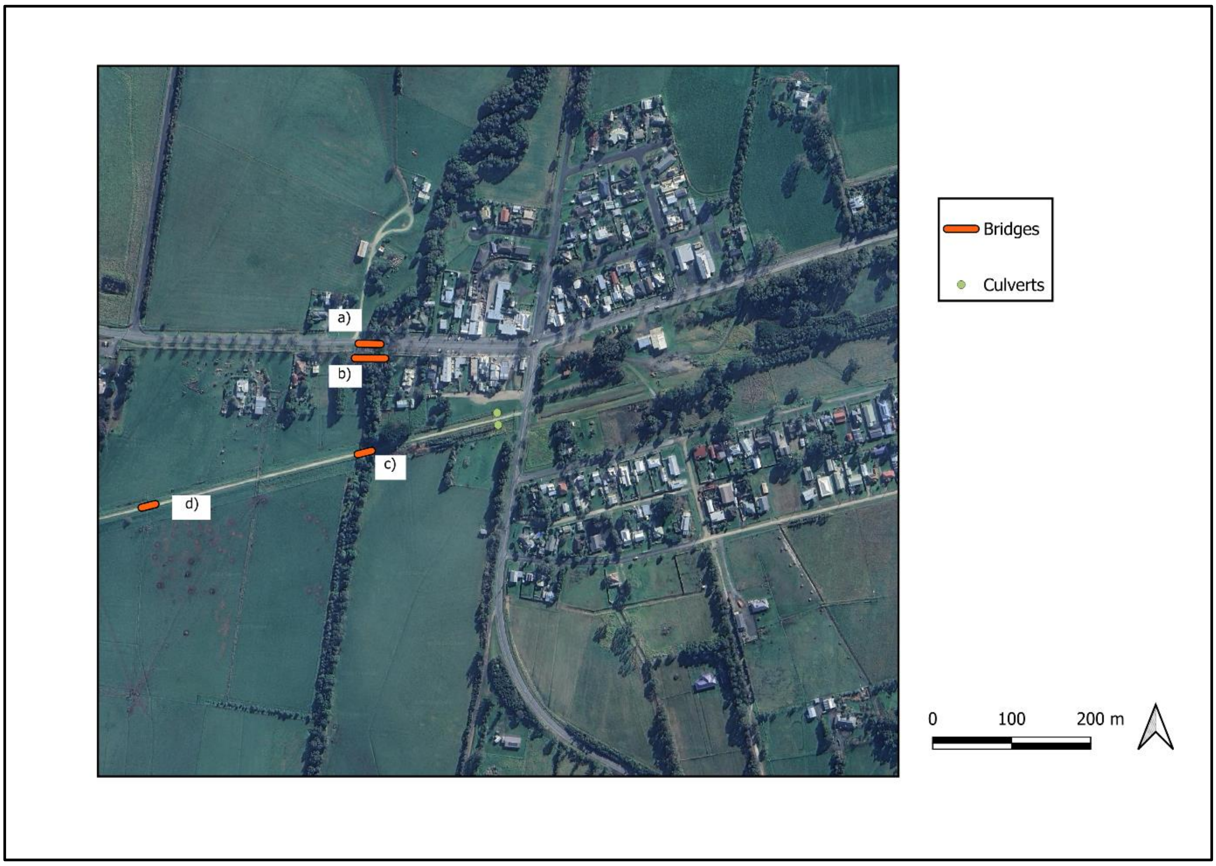

Figure 3-.4 Location of Bridges within the model domain

As previously mentioned, bridge crossings have been modelled as a TUFLOW 'layered flow constrictions' embedded in the 2D model. A zsh modifying polygon has been used to merge and modify the road crest thereby maintaining the channel topography and allowing water to pass underneath. The form loss coefficient for the various parts of the bridge (Layer 1 to Layer 3) have been determined using typical values in accordance with Australian Rainfall and Runoff Guidelines (2019) and are shown below in Table 12.

Table 11 Modelled Bridge attributes

Parameters	South Gippsland Highway	Pedestrian	Rail Trail Creek Crossing	Rail Trail (other)
Layer 1 (Under the Bridge)				
Pier blockage (%)	5%	5%	10%	10%
Form Loss (k)	0.02	0.04	0.02	0.04
Bridge soffit level (mAHD)	2.4	3.2	2.0	0.5
Layer 2 (Bridge Deck)				
Pier blockage (%)	98%	85%	90%	100%
Form Loss (k)				
Top of Layer 2 (mAHD)	2.9	3.7	2.4	0.9
Layer 3 (Above Top of Deck)				
Pier blockage (%)	5%	45%	65%	15%
Form Loss (k)	0.02	0.03	0.02	0.02
Top of Layer 3 (mAHD)	3.7	4.5	3.6	1.7

Surface Roughness

The Manning's roughness coefficient represents friction losses associated with the bed material of a channel/floodplain, and drag losses associated with vegetation or other obstructions. The Mannings values that have been used throughout the model were derived from the Australian Rainfall and Runoff Guidelines (2019) as per Table 6.2.1 – Values of Roughness Coefficient n for different channel conditions and Table 6.2.2 - Valid Manning 'n' Ranges for Different Land Use Types (Ball, et al., 2019). Table 13 displays the Manning's coefficient and land use categories described within the model. The roughness coefficients in the study area were derived from satellite images, planning zone maps and field observations and digitised into land-use polygons representing zones of similar loss characteristics within the study area (Figure 3-5). A global material ID for all cells that fall outside the defined material polygons was set to 0.03 reflecting the predominant agricultural usage in the catchment.

Table 12 Mannings Roughness (n) Values applied to the model for different land use types (Source: (Ball, et al., 2019).

Land Use Type	Mannings 'n' Value
Residential areas	0.2
Dams	0.01
Moderate Vegetation	0.04
Vegetated Waterways	0.08
Paved /Unpaved Roads	0.02
Pasture/grass	0.03

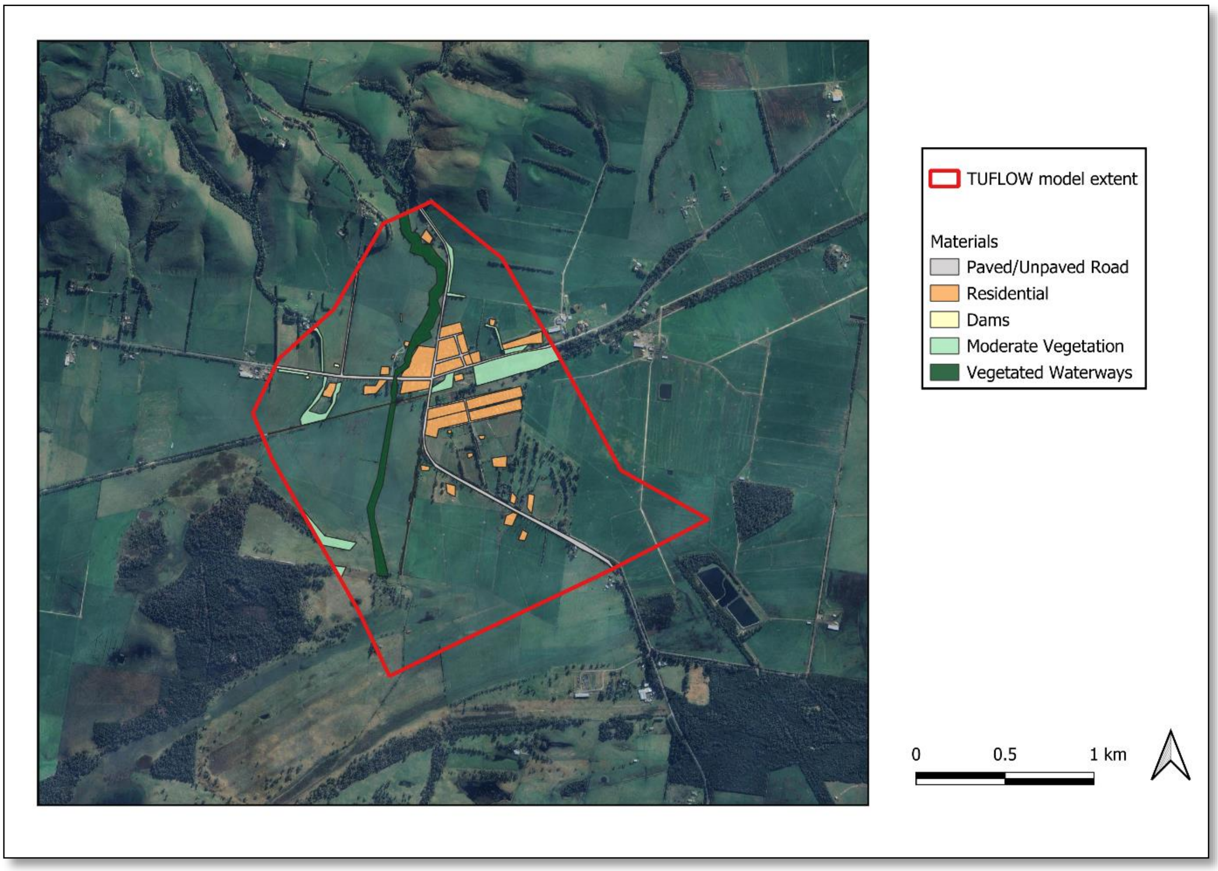

Figure 3-.5 Materials layer with allocated Manning's n surface roughness

Buildings

Buildings were simulated in the hydraulic model for the town as a materials layer within the 2D domain.

Breaklines

Z points derived from LiDAR alone may not adequately capture changes in topographic breaks such as road crests and creek beds in flood models. These topographic breaks can be reinforced in TUFLOW as breaklines. Breaklines were manually digitized using a hill shade LiDAR and google aerial imagery to suitably represent raised roadways and reinforce the bed of Shady Creek (Figure 3-6). Roads and the rail trail were represented as thick lines with 'Max' shape in this way the Zpt elevation only changes when the Z shape elevation at the Zpt is higher which enforces the highest elevations along the road crest. Shady creek was represented as a gully shape. The TUFLOW asc_to_asc utility was used to create a breakline points layer which provides a more consistent interpolation of elevations along the digitised line. The digitised points were manually inspected to remove any positive gradients created due to vegetation interference.

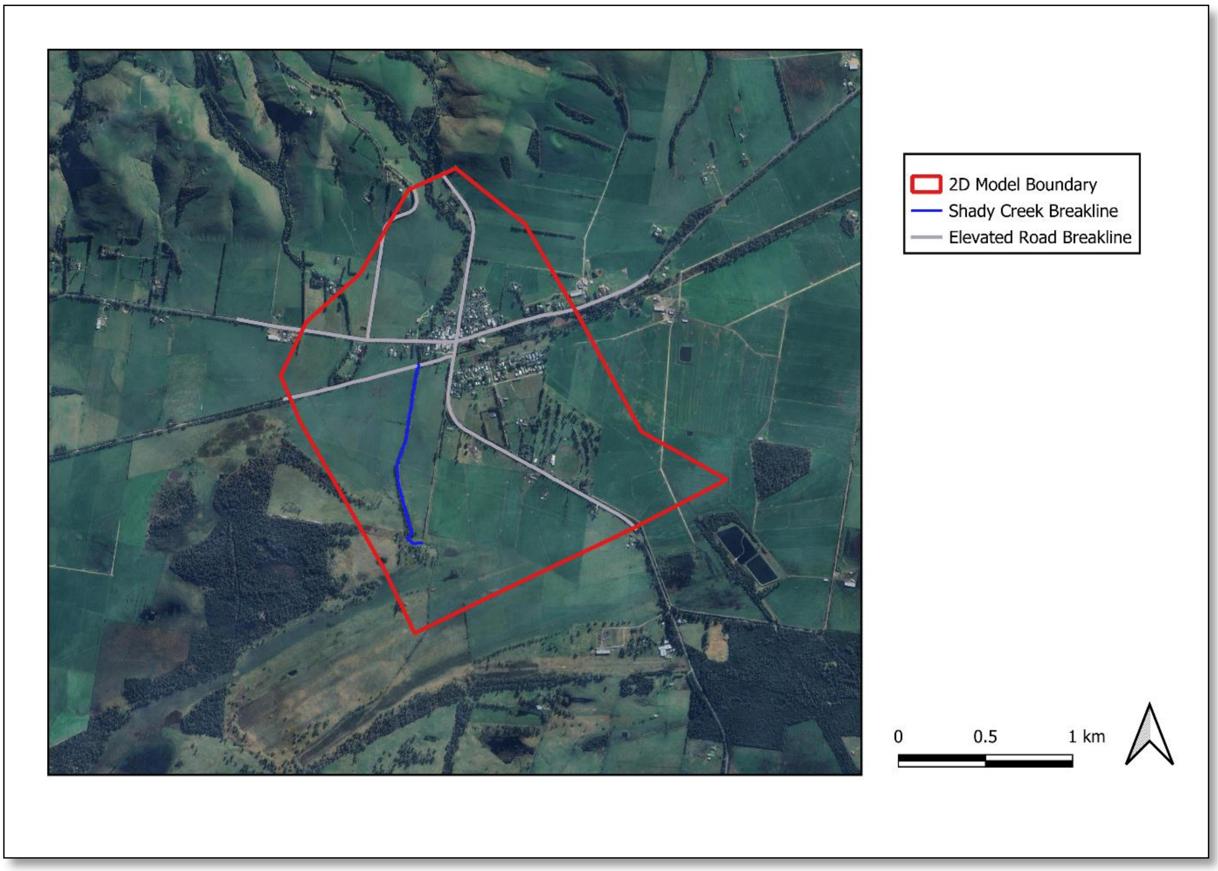

Figure 3-.6 Modified Breaklines used to reinforce road crests and Shady creek bed within the model boundary.

3.5 Boundary Conditions

A hydraulic model requires inflow boundaries and outlet boundaries to allow water into and out of the model in a realistic manner. The external inflow boundaries accounts for flow generated from outside of the model extents (external boundaries). Flow is removed from the model through downstream boundaries, which are generally a fixed water level or a stage discharge relationship.

The TUFLOW model for Welshpool has been modelled with one external flow vs time (QT) boundary from Shady Creek. The model outflow boundaries were applied as stage vs discharge boundaries and covered most of the extent of the southern and western boundary. Terrain slope was used as the outflow boundary condition and was calculated based on the gradient/slope between points on either side of the model extent. The model extents and inflow and outflow boundaries are illustrated in Figure 3-7

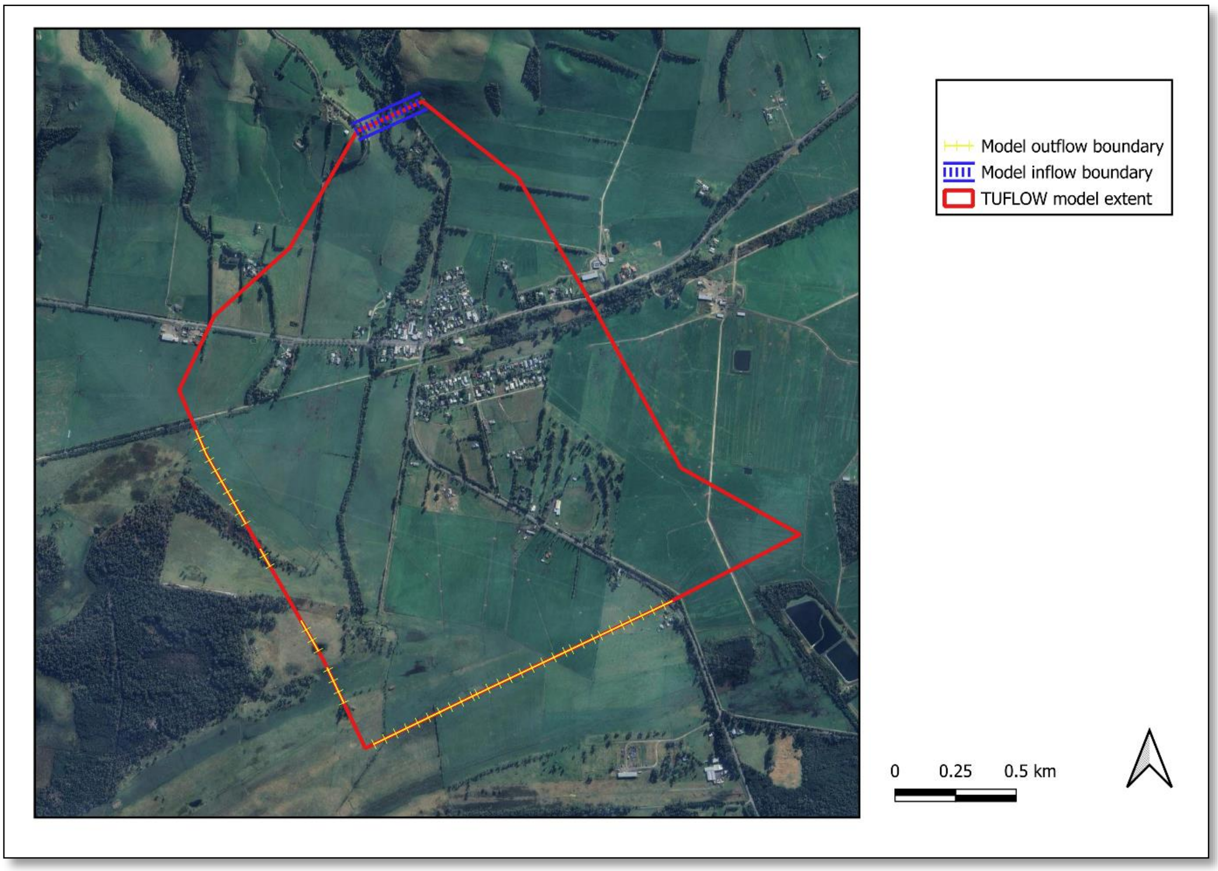

Figure 3-.7 Model extents, showing inflow and outflow boundaries

3.6 Assumptions

The accuracy of all model results provided in this report is dependent on the accuracy of the input data sets and the ability of the modelling approach to accurately replicate recorded flood events. As additional data becomes available from future flood events the accuracy of the results may be improved upon. In general, consideration of topography information, rainfall intensity duration data and hydraulic structure integrity are worthy of consideration in this study.

The topographic information used in this report had a high level of accuracy in most areas. In locations with more complex terrain, particularly the vegetated creek areas, the accuracy is likely to be much lower due to vegetation masking underlying creek topography. Topographical errors resulted in the amount of in-channel flow being affected below the town of Welshpool. While the use of a gully breakline helped to alleviate some of the in-channel flow issues they were not fully rectified. In the future a field survey of channels may improve accuracy in this regard.

The final flood frequency levels are based solely on regionalised intensity–frequency–duration (IFD) design rainfall data provided by the Bureau of Meteorology. Therefore, specific at site factors are not considered.

Blockage of hydraulic structures can occur with the transportation of different materials by flood waters. Blockage from vegetation is of particular importance to the Shady Creek catchment with debris still being present - following the 2023 Boxing Day event - during the site visit. The potential effects of blockage include:

- decreased conveyance of flood waters through the blocked hydraulic structure or drainage system;
- variation in peak flood levels;
- variation in flood extent due to flows diverting into adjoining flow paths; and
- overtopping of hydraulic structures.

3.7 Sensitivity analysis

A sensitivity analysis was undertaken to evaluate how sensitive the hydraulic model was to changes in model cell size and surface roughness. The sensitivity analysis was completed for the 1% AEP, 12-hour event using a cell size of 2.5 where applicable. The range of values tested is summarised in Table 13.

Table 13 Summary of parameters adopted for sensitivity analysis

Parameter Assessed			
Cell size (m)	10	5	2.5
Roughness (Mannings 'n')	25% increase	25% decrease	
Boundary conditions	Double gradient	Halve gradient	

The following conclusions were reached from the sensitivity analysis:

Cell Size

Overall depths and areas of inundation increased slightly with higher model resolution (Table 14). This is to be expected due to cell convergence as cell size decreases and the model is better able to match the scale of the topography

Table 14 Sensitivity analysis of cell size on hydraulic model

Cell Size	Depth (m)	Mean	SD	Area (km^2)
10	4.384	0.1502	0.2619	1.5957
5	3.783	0.1579	0.2885	1.4627
2.5	3.747	0.1676	0.3062	1.4350

Mannings 'n' value

Generally, there was a marked decrease in both the depth and area of inundation with a decrease in Mannings 'n' values (Table 15). Velocity also increased from 2.017 at a higher roughness values to 2.829 at lower values. This is to be expected when water can move with less resistance across a landscape.

Table 15 Sensitivity analysis of alterations to surface roughness (Mannings 'n') on hydraulic model

Mannings 'n'	Depth (m)	Mean	SD	Area (km2)
25 % increase	3.959	0.1789	0.3184	1.4832
Normal	3.747	0.1676	0.3062	1.4350
25% decrease	3.514	0.1581	0.2956	1.3484

Boundary Conditions

Alterations in the gradient of the outflow boundaries had very little impact on the depth and area of inundation (Table 16).

Table 16 Sensitivity analysis of alterations to HQ boundary conditions on hydraulic model

Boundary conditions	Depth (m)	Mean	SD	Area (km2)
Double gradient	3.747	0.1672	0.3062	1.4343
Normal	3.747	0.1676	0.3062	1.4350
Halve gradient				

3.8 Results

The overall health of the model indicated by the Final Cumulative Mass Error was given as 0.01% which falls well within the recommended ±1%. This means that the timestep was well within acceptable range, boundary conditions were stable, and there were no occurrences of negative depths in the model.

Peak flood depths, extents and velocities for the 12 hour 1% AEP are shown in Figure 3-8 and Figure 5-9. The 1% AEP event had a maximum depth of 3.2 m and maximum velocity of 1.873 m^2/s. At its peak the 12-hour duration storm event ultimately covered 1.441km^2 of the 3.514km^2 study area (approximately 41%) with a significant portion of the town west of Woorarra Road being inundated.

Figure 3-.8 1% AEP flood extent maximum depth for Welshpool and surrounds

Figure 3-.9 1% AEP flood extent maximum velocity for the town of Welshpool

Figure 3-.10 1% AEP flood extent maximum depth for the town of Welshpool

3.9 Climate Change Scenario

To address uncertainty in future concentrations of greenhouse gases and emissions of aerosols, WGCMA uses Representative Concentration Pathways (RCPs). ARR recommends reporting on modeling changes under RCP 4.5 and 8.5 which represent medium and high-emission scenarios respectively. RCP 8.5 is the highest baseline emissions scenario in which emissions continue to rise throughout the twenty-first century. Climate change projected under RCP 8.5 will be more severe than under RCP 4.5. A conservative approach has been adopted utilizing the RCP 8.5 scenario. For details regarding the adjusted parameters refer to Section 2.9. Table 14 shows the statistical analysis of climate change peak flood levels from existing conditions for the 1% AEP event. There is a change in the maximum depth from 3.929 to 4.148 which represents a difference of around 0.2 m. The greatest changes were confined to the upper reaches of Shady Creek above the Welshpool township. Overall, the average depth increased by only 0.006m over the entire inundated area.

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Table 17 Statistical comparison of 1% AEP flood extent for the current year and the RCP 8.5-year 2100 scenario

	Maximum Depth (m)	Mean depth (m)	Standard Deviation
Current	3.929	0.170	±0.303
RCP 8.5	4.148	0.176	±0.313

4 References

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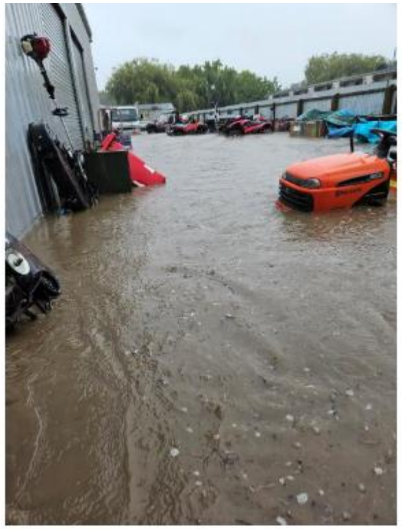
5 Appendix A: Preliminary data collection

5.1 Photographs Welshpool floods

5.2 Photographs Welshpool floods


21/23 Main Street, Welshpool


19 Main St Welshpool


Rear 19 Main St Welshpool


55 Main St Welshpool


Corner Woorarra Road/Main Street (facing south)


24 Main Street


Cnr Saunders Street / Gippsland Highway


24 Main Street

Figure 5-1 A: 21/23 Main Street, Welshpool