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Attachment Part A

Middleton Flood Study

Flood Mitigation Assessment Report

Alexandrina Council

23 January 2024
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Contents

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1	Introduction	6
1.1	Purpose of this report	6
1.2	Background	7
1.3	Overall Approach	9
2	Model Inputs.....	10
2.1	Feature Survey	10
2.2	LiDAR	10
2.3	Site Visit	12
2.4	Hydrological Inputs	15
3	Hydrological Modelling	17
3.1	RORB Model	17
3.2	RORB Results	19
3.3	Comparison to Other Flow Estimates	19
3.4	Design Flow Hydrographs	20
4	November 2022 Event Assessment.....	22
5	Hydraulic Modelling	24
5.1	HEC-RAS Model.....	24
5.2	Flood Hazard Mapping.....	25
6	Existing Infrastructure Results.....	27
6.1	20-year ARI event.....	27
6.2	50-year ARI event.....	28
6.3	100-year ARI event.....	29
7	Potential Flood Mitigation Options	31
7.1	Identified Options.....	31
7.2	Scenario Summary	38
8	Modelling Results and Discussion	41
8.1	Scenario 1: Creek Clearing	41
8.2	Scenario 2: Channel Widening	41
8.3	Scenario 3: Flood Levees	42



8.4	Scenario 4: Flood Detention	43
8.5	Scenario 5: Goolwa Road Culvert Upgrade	43
8.6	Scenario 6: Creek Clearing and Levees	44
8.7	Scenario 7: Channel Widening and Levees.....	45
8.8	Scenario 8: Creek Clearing and Culvert Upgrade	46
8.9	Scenario 9: Channel Widening and Culvert Upgrade	46
8.10	Scenario 10: Creek Clearing, Levees and Culvert Upgrade.....	46
9	Mitigation Works Concept Cost Estimate.....	48
9.1	Cost Estimates Assumptions	48
9.2	Scenario Cost Summary	50
10	Discussion and Conclusions.....	51
10.1	By Scenario.....	51
10.2	Scenario Scores	55
10.3	Conclusion and Next Steps	58

Figures

Figure 1-1: Locality Plan, upstream catchment shown in blue	8
Figure 2-1: LiDAR and creek catchment.....	11
Figure 2-2: Typical Vegetation in the creek in Middleton (looking south from Goolwa Road)	12
Figure 2-3: Goolwa Road culverts (upstream side).....	13
Figure 2-4: Tree branch against the Goolwa Road culverts.....	14
Figure 3-1: Middleton RORB layout.....	18
Figure 3-2: Design flow hydrographs.....	21
Figure 4-1: BoM rainfall radar imagery from during the November 2022 event	22
Figure 5-1: Combined flood hazard curves (Smith et al, 2014).....	26
Figure 6-1: Flow hydrographs at key locations for the 20-year event (existing infrastructure).....	28
Figure 6-2: Flow hydrographs at key locations for the 50-year event (existing infrastructure).....	29
Figure 6-3: Flow hydrographs at key locations for the 100-year event (existing infrastructure).....	30
Figure 7-1: Example cross section of widened creek	33
Figure 7-2: Approximate locations of potential flood levees	34
Figure 7-3: Possible dam location.....	36

Tables

Table 1-1: Conversion between AEP and ARI.....	6
Table 2-1: ARR 2019 blockage factor assessment	14
Table 2-2: BoM IFDs (2016)	15
Table 3-1: K _c values.....	17
Table 3-2: RORB results.....	19
Table 3-3: Flow estimates comparison	20
Table 5-1: Adopted Manning's n values	25
Table 7-1: Flood mitigation scenario summary	39



Table 8-1: Required levee heights for Scenario 3	42
Table 8-2: Required levee heights for Scenario 6	44
Table 8-3: Required levee heights for Scenario 7	45
Table 8-4: Required levee heights for Scenario 10	47
Table 9-1: Summary of cost estimates (upfront costs in brackets).....	50
Table 10-1: Scenario scores (out of 5).....	56

Appendices

Appendix A – Flood Maps

Appendix B – Middleton Survey

Appendix C – Detailed Cost Breakdown



1 Introduction

1.1 Purpose of this report

This report is prepared for the Alexandrina Council, abbreviated hereafter in this document as “Council”. The purpose of this report is to summarise the flood modelling and assessments that have been undertaken along the main watercourse that passes through Middleton. It covers the following aspects:

- Details of the hydrological and hydraulic modelling process
- November 2022 flood investigations
- Hydraulic modelling of the performance of the existing infrastructure and analysis of the results
- Preparation of the 100-year, 50-year and 20-year average recurrence interval (ARI) flood maps for the existing infrastructure scenario
- Identification of several potential flood mitigation options
- Hydraulic modelling of these options and analysis of the results
- Preparation of the 100-year ARI flood maps for the flood mitigation scenarios
- Preliminary high-level cost estimates for each of the scenarios
- Discussion of the above and recommendations for potential next steps

In general, flood event magnitudes are referred to in this report in terms of their ARI e.g. a 100-year ARI event. It must be noted that a 100-year event does not mean that this event will occur once every hundred years. Rather, it refers to the average timespan between the occurrence of events of equal or greater magnitude. It is therefore possible for example (although unlikely) that two 100-year events may occur in consecutive years or even within the same year. Conversely, it is also possible that 150 years may pass without a 100-year event occurring in a given location.

Event magnitudes may also be referred to by their annual exceedance probability (AEP), which is the percentage chance of an event of a given magnitude or greater occurring within any one year. Table 1-1 below shows the conversion between AEP and ARI for a range of commonly referred-to event magnitudes.

Table 1-1: Conversion between AEP and ARI

Annual Exceedance Probability (AEP)	Average Recurrence Interval (ARI)
63.2%	1 year
50%	1.44 years
20%	4.48 years
10%	9.5 years
5%	20 years
2%	50 years
1%	100 years



0.5%

200 years

1.2 Background

Tonkin has been engaged by Council to undertake a flood study of the creek that runs through the centre of Middleton, including hydrological modelling and hydraulic modelling to assess existing flood conditions. Part of the outcome of this study has been to provide recommendations of possible flood mitigation works to alleviate flood risks within Middleton.

The need for a flood study was prompted by a large flood event that occurred in November 2022. Flooding was widespread across the greater Alexandrina Council area with significant flooding occurring within the Middleton township with a large number of private properties inundated, major road closures and damage to a variety of public and privately owned infrastructure within the town.

This flood was primarily due to flows coming from the creek which passes through the centre of town, and so the investigations undertaken have focused on the flood behaviour of the creek.

A locality map showing the creek catchment is shown below in Figure 1-1.

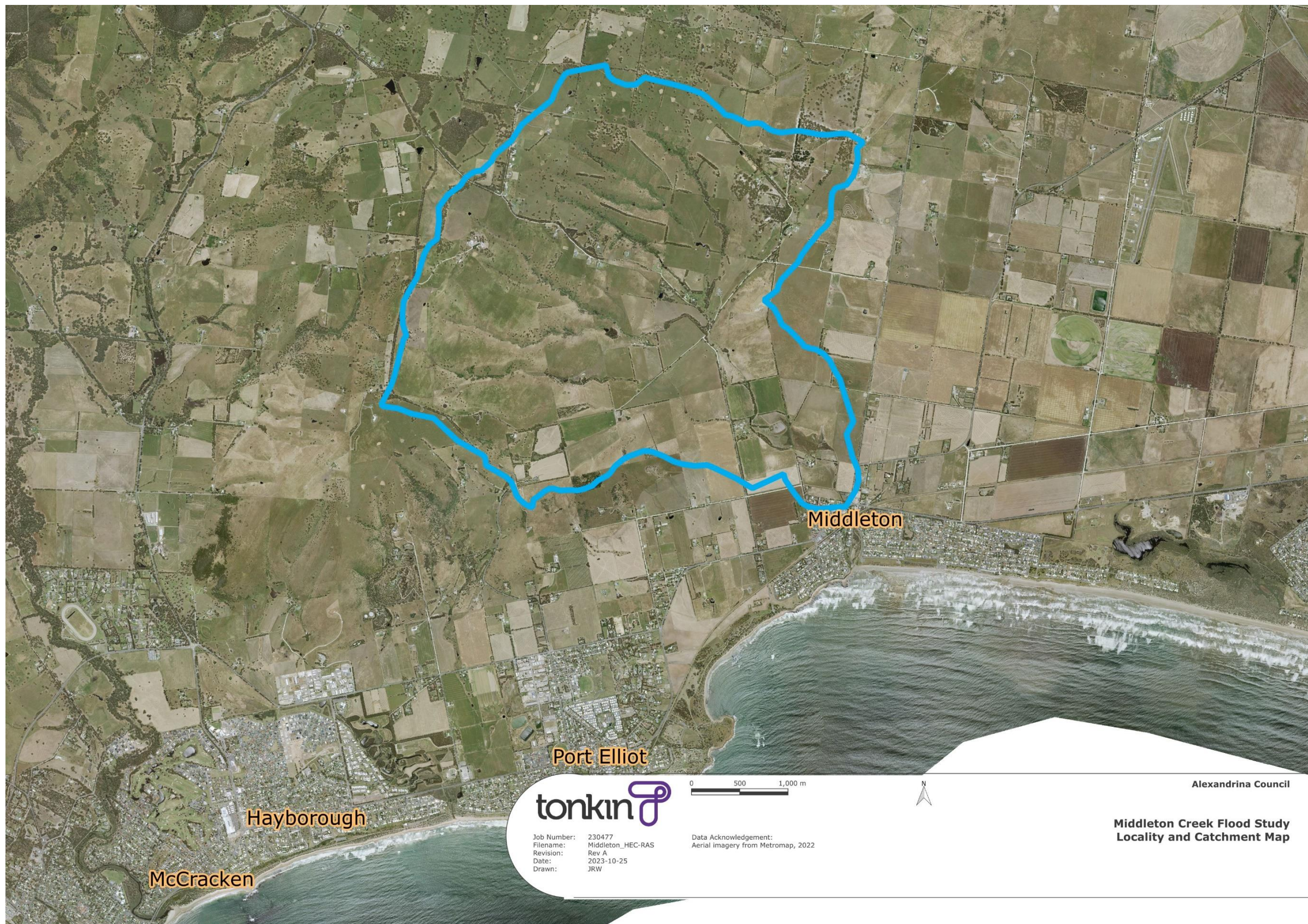


Figure 1-1: Locality Plan, upstream catchment shown in blue



1.3 Overall Approach

The basic approach to undertaking the flood study consisted of the following steps:

- Gather required input data and process into a format suitable for modelling.
- Undertake hydrological modelling in RORB (a software package) to determine the creek design flow hydrographs arriving just upstream of the Goolwa Road culverts for the 100-year, 50-year and 20-year ARI events.
- Develop a 2D hydraulic model in HEC-RAS of the creek and surrounding topography of Middleton, extending from 150m upstream of Goolwa Road to 300m downstream of the railway bridge.
- Assess the likely ARI and duration of the November 2022 flood event, and use this information in addition to recorded flood markers to verify the performance of the HEC-RAS model.
- Undertake design flood modelling in HEC-RAS for the existing infrastructure within Middleton, using the design flow hydrographs obtained from RORB as the input flow hydrograph boundary condition.
- Create flood maps for the 100-year, 50-year and 20-year ARI events showing the depth and extent of inundation within Middleton as well as the flood hazard (based on depth and flow velocities) for the existing infrastructure scenario.
- Based on the flood maps, develop mitigation option concepts to alleviate flood issues.
- Incorporate these scenarios into the HEC-RAS model and rerun the model to determine the effectiveness of these works in mitigating or reducing the negative effects of flooding.
- Undertake post-mitigation analysis for the 100-year, 50-year and 20-year ARI events.
- Undertake high-level construction cost estimates for each of the scenarios.
- Considering both the safety risks/benefits and the overall value for money of each scenario, provide a recommendation to Council of the potential next steps.



2 Model Inputs

The hydrological and hydraulic modelling utilised a number of inputs, either provided directly by Council or obtained by Tonkin from other sources.

2.1 Feature Survey

Survey of a number of features within Middleton were required. These included:

- The creek from 20m upstream of Goolwa Road to 10m downstream of the railway bridge;
- Goolwa Road between 32 Goolwa Road and 20m east of the creek;
- The culverts underneath Goolwa Road;
- The railway bridge;
- Survey of the pedestrian underpass underneath the railway (adjacent to the Middleton Caravan Park);
- Level survey of several known flood markers from the November 2022 flood event; and
- Floor levels of a number of buildings within Middleton.

Tonkin therefore engaged a surveyor to undertake the required survey, which was provided to Tonkin in the form of an AutoCAD .dwg file and PDF. The survey PDF is attached as Appendix C of this report.

2.2 LiDAR

A primary input was the Victor Harbor 50cm LiDAR (Light Detection And Ranging) which was flown in 2021 (publicly available from elevation.fsdf.org.au) and was used as the base DEM (Digital Elevation Model) for the project. The LiDAR covered the entirety of Middleton and the creek catchment. This LiDAR was used to derive the boundary of the catchment, which has a total area of 16 km². The DEM and the creek catchment are shown below in Figure 2-1.

It should be noted that the final DEM utilised in the hydraulic modelling (refer Section 5) used the more accurate survey information where it was available.

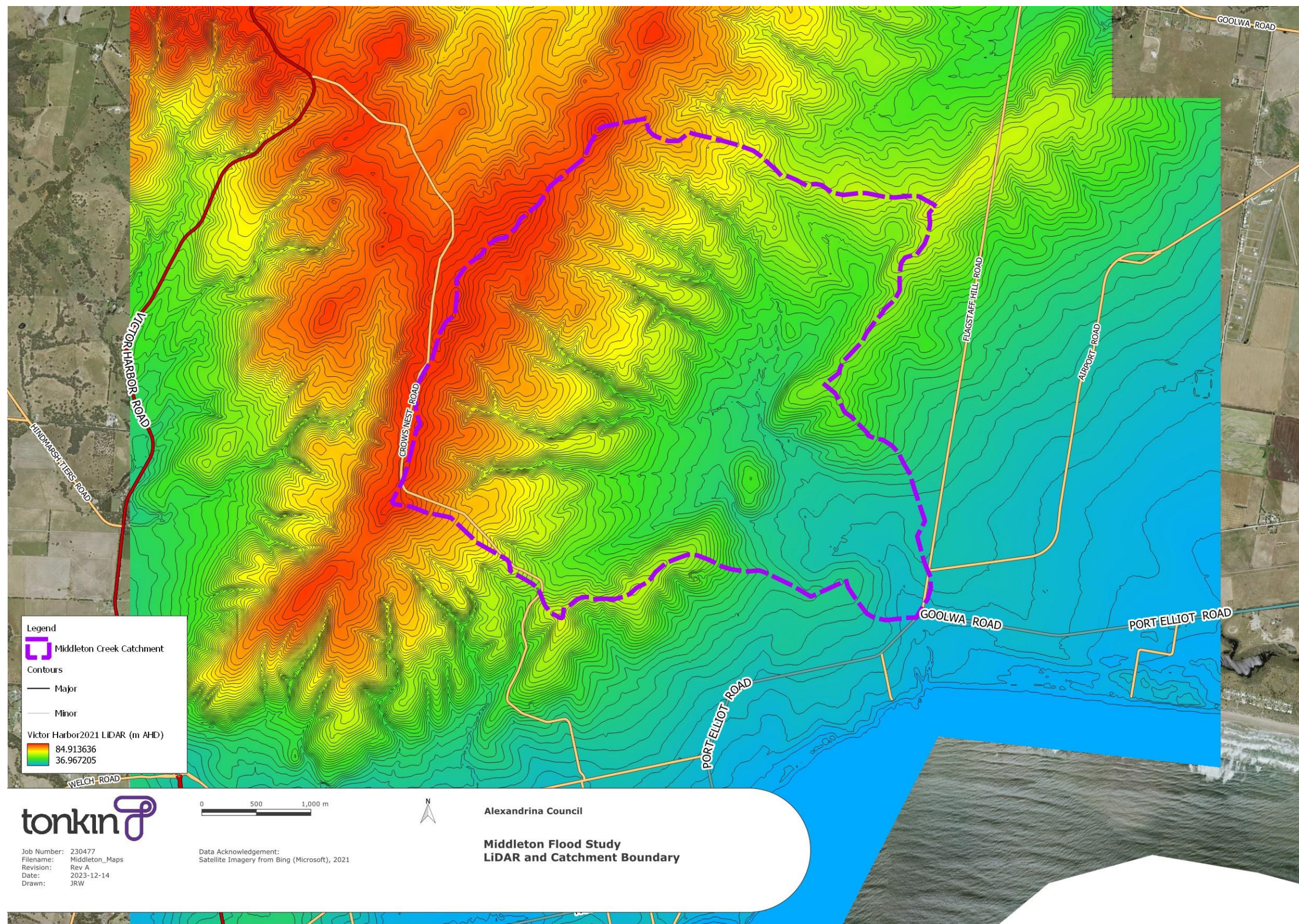


Figure 2-1: LiDAR and creek catchment

2.3 Site Visit

A site visit to the area of interest was undertaken during the start-up phase by two engineers from Tonkin. This provided much valuable information regarding site conditions, including:

- Vegetation type and density
- Amount and types of debris that may contribute to culvert blockages
- Ground surface roughness and erosion potential
- Overall visual inspection of the site and local topography.

2.3.1 Catchment land use

The land use in the upstream catchment is predominantly agriculture and farmland, with few buildings within the catchment. The township of Middleton is mostly residential, with a number of businesses mainly along Goolwa Road and Mill Terrace.

The reaches of the creek within Middleton are fairly densely vegetated, with reeds and grasses along the base of the creek and numerous trees along the banks of the creek, especially between Goolwa Road and the Mill Terrace/Padman Crescent footbridge. See Figure 2-2 below for a photograph taken during the site visit.



Figure 2-2: Typical Vegetation in the creek in Middleton (looking south from Goolwa Road)

2.3.2 Culvert blockage factor

A culvert blockage assessment was completed to estimate the design blockage factor for modelling the Goolwa Road culverts. The basis of this assessment was the procedure outlined in Australian Rainfall and Runoff (ARR) 2019, Book 6, Chapter 6 using information gathered during the site visit, anecdotal evidence from the November 2022 floods, and satellite imagery.

The debris relevant to this assessment is that debris which is sourced within the creek catchment upstream of the Goolwa Culverts, and located sufficiently closely to the creek such that it may be washed into the creek and carried to Goolwa Road.

The photo shown below in Figure 2-3 was fairly typical in terms of the level of transportable debris that was observed within the creek and through the catchment nearby to the creek. Overall, the amount of debris that may contribute in a “typical” event was fairly moderate. No sources of large rubbish (e.g. mattresses, shopping trolleys) upstream of the bridge were observed, and the majority of debris arriving at the culverts are relatively small branches and sticks.



Figure 2-3: Goolwa Road culverts (upstream side)

It is understood that during the November 2022 floods, the Goolwa Road culverts did become partially blocked due to tree branches becoming pinned against the central dividing wall between the culverts. Figure 2-4 is a photograph of a tree branch that remained against the culverts following the November 2022 floods.



Figure 2-4: Tree branch against the Goolwa Road culverts

For a summary of the blockage assessment, refer to Table 2-1 below.

Table 2-1: ARR 2019 blockage factor assessment

Assessment Criterion	Description	Classification
Debris Type	Medium sized floating debris, mostly small to medium sized sticks and branches.	$L_{10} = 1.0$ m (90 th percentile of debris size)
Debris Availability	Streams with steep bed slopes and banks showing signs of substantial past bed/bank movements. Upstream catchment primarily consists of well-maintained rural lands and paddocks with minimal outbuildings.	Medium
Debris Mobility	Steep source area with fast response time. Main source areas well away from streams.	Medium
Debris Transportability	Moderate stream velocities ($v < 2.5$ m/s). Deep stream relative to vertical debris dimension ($D > L_{10}$).	Medium-High



Wide stream relative to horizontal debris dimension ($W > L_{10}$).		
Debris Potential	MMM or MMH	Medium overall
AEP-adjusted Debris Potential	$5\% \leq AEP \leq 0.5\%$	Medium adopted
Design Inlet Blockage (%)	$L_{10} < \text{Culvert width (2.4m)} < 3 * L_{10}$	10% Blockage

Based on the above assessment, a blockage factor of 10% was adopted for the hydraulic modelling.

2.4 Hydrological Inputs

2.4.1 Rainfall depths

Bureau of Meteorology (BoM) Intensity-Frequency-Duration (IFD) 2016 rainfall data was obtained for the catchment centroid at a lat/lon of 35.5087° south, 138.7094° west. These are shown in Table 2-2 below. The table shows rainfall duration against rainfall depths for various AEPs as provided by BoM. Refer to Table 1-1 for conversions between AEP and ARI.

Table 2-2: BoM IFDs (2016)

Duration	Rainfall Depth by AEP (mm)						
	63.2%	50%	20%	10%	5%	2%	1%
30 min	9.09	10.3	14.5	17.7	21.1	26.1	30.2
45 min	10.6	12.1	16.9	20.6	24.6	30.3	35.1
1 hour	11.8	13.4	18.8	22.9	27.3	33.6	38.9
1.5 hours	13.7	15.5	21.8	26.5	31.6	38.8	45
2 hours	15.1	17.2	24.1	29.4	35	43	49.9
3 hours	17.3	19.7	27.8	33.9	40.3	49.8	57.8
4.5 hours	19.8	22.5	31.8	38.9	46.5	57.6	67
6 hours	21.7	24.7	35	42.8	51.2	63.8	74.4
9 hours	24.5	27.9	39.7	48.8	58.6	73.4	85.8
12 hours	26.7	30.4	43.3	53.3	64.2	80.6	94.5



Duration	Rainfall Depth by AEP (mm)						
	63.2%	50%	20%	10%	5%	2%	1%
18 hours	29.9	34	48.5	59.8	72.3	90.9	107
24 hours	32.3	36.6	52.1	64.5	78.1	98.1	115

2.4.2 Temporal patterns

Australian Rainfall and Runoff 2019 (ARR 2019) point temporal patterns for the S-SW Flatlands (East) region were sourced from the DataHub website (data.arr-software.org). This includes a set of ten temporal patterns for each duration.

2.4.3 Rainfall losses

One common method of rainfall loss estimation for a rural catchment would be calibration and verification of a hydrological model (such as RORB) to accurate gauged rainfall and streamflow data. However, no streamflow gauge data is available for the creek.

Loss values of 23.0 mm initial loss (IL) and 4.7 mm/hr continuing loss (CL) were sourced from the ARR 2019 DataHub website and adopted within the modelling.



3 Hydrological Modelling

3.1 RORB Model

In order to determine the design flow hydrographs to be used within the hydraulic modelling/flood mapping process, a RORB model of the catchment was developed. RORB is a runoff-routing software program that takes rainfall and rainfall losses as inputs and outputs flow hydrographs at a desired location (in this case, just upstream of the Goolwa Road bridge).

3.1.1 RORB catchment file

Delineation of the subareas and reaches was undertaken using the available LiDAR as well as available satellite imagery. A .catg file was then created to represent the catchment. The RORB model layout is shown below in Figure 3-1.

3.1.2 RORB modelling parameters

Apart from rainfall (refer Section 2.4) and catchment data, RORB has four primary modelling parameters. These are:

- IL – Rainfall loss subtracted from the start of the rainfall.
- CL – Rainfall loss subtracted as a constant rate throughout the rainfall.
- K_c and m – Routing parameters.

IL and CL were adopted as per Section 2.4.3. An m value of 0.8 (which is the industry standard) was adopted.

There are multiple ways to determine an appropriate value for K_c . For this project, as no streamflow gauge data was available to calibrate to, two relationships (obtained from between catchment area A and K_c (obtained from ARR 2019) were applied. These are shown in Table 3-1 below.

Table 3-1: K_c values

Equation	Reference	Applicability	K_c value
$K_c = 0.60A^{0.67}$	ARR 2019 Eqn. 7.6.17	Catchment area < 100 km ²	3.8
$K_c = 0.89A^{0.55}$	ARR 2019 Eqn. 7.6.21	Mean annual rainfall for the catchment > 500mm	4.0

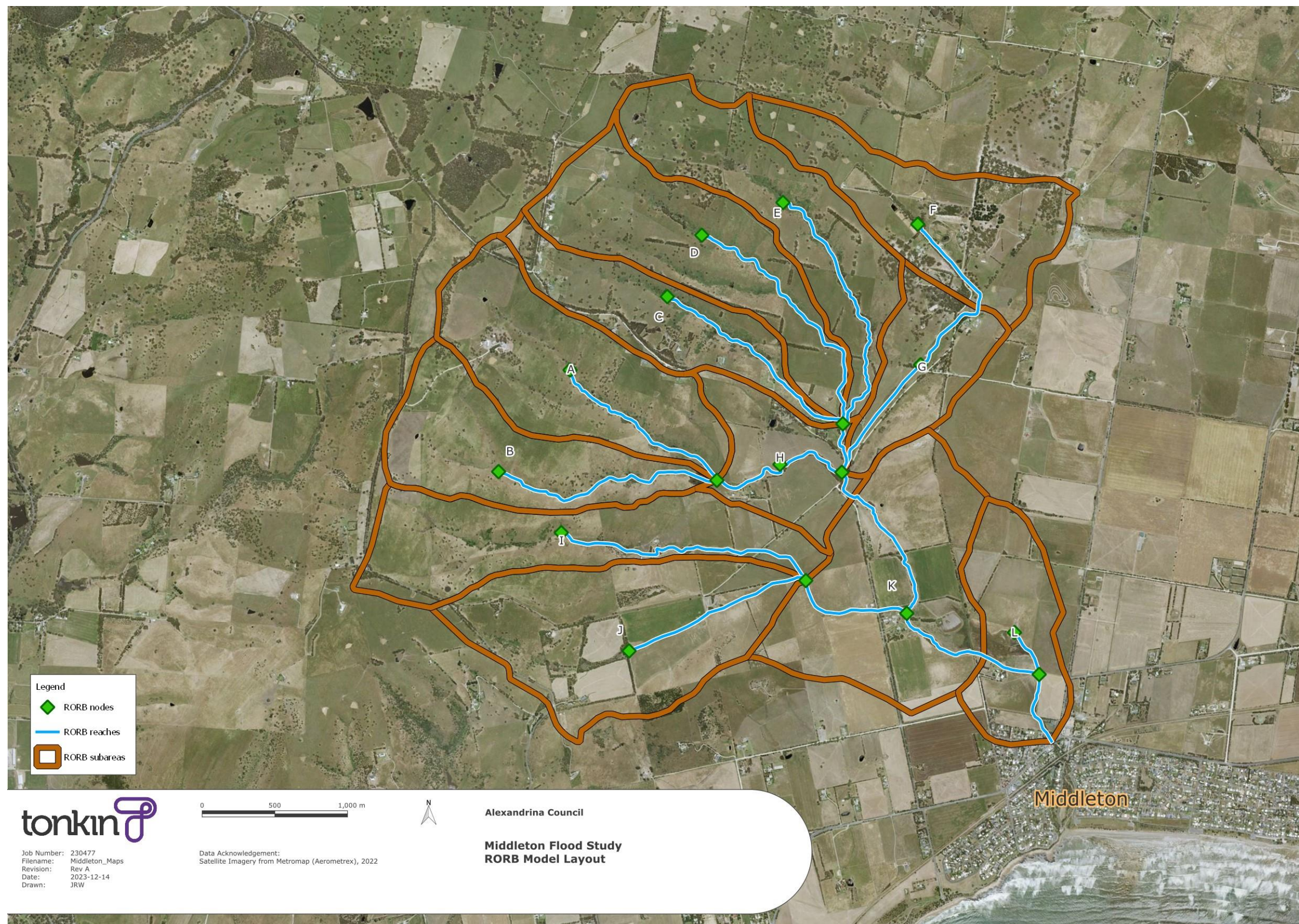


Figure 3-1: Middleton RORB layout



3.2 RORB Results

The RORB model was run for a range of AEPs and durations (for each of the ten temporal patterns per duration) for both K_c values. The results of the modelling are shown below in Table 3-2. The peak flow rates shown are the median peak flow rate out of the ten temporal patterns for the critical duration at each AEP. Refer to Table 1-1 for conversions between AEP and ARI.

Table 3-2: RORB results

Parameter	10% AEP		5% AEP		2% AEP		1% AEP	
	$K_c = 3.8$	$K_c = 4.0$	$K_c = 3.8$	$K_c = 4.0$	$K_c = 3.8$	$K_c = 4.0$	$K_c = 3.8$	$K_c = 4.0$
Peak Flow (m ³ /s)	8.2	7.8	14.4	14.0	21.1	20.2	29.8	28.8
Critical Duration (hours)	12	12	12	12	9	9	9	9

3.3 Comparison to Other Flow Estimates

In order to validate the RORB modelling results, and determine the K_c value to be adopted, a number of other estimates of peak flow were obtained.

The first category of flow estimates were regression estimates relating flow (for a particular event) to catchment area. Each of the regression formulae are of the following form:

$$Q_{ARI} = CA^B$$

where Q_{ARI} is the design flow rate (in m³/s) for a given event ARI, A is catchment area (in km²) and B and C are regression parameters unique to each regression relationship.

The regression formulae used were:

- Eusuff (High)
- Akter and Daniell
- Transport SA (Kemp)

The other flow estimate was the Regional Flood Frequency Estimate (RFFE), using the publicly available online tool at rffe.arr-software.org. A comparison of the flow estimates is shown below in Table 3-3. Refer to Table 1-1 for conversions between AEP and ARI.



Table 3-3: Flow estimates comparison

Method	Peak Flow Rate (m ³ /s)			
	10% AEP	5% AEP	2% AEP	1% AEP
RORB ($K_c = 4.0$)	7.8	14.0	20.2	28.8
RORB ($K_c = 3.8$)	8.2	14.4	21.1	29.8
Eusuff (High)	15.3	19.2	23.8	28.4
Akter and Daniell	14.4	-	25.1	30.6
Transport SA (Kemp)	18.9	24.0	31.3	37.7
RFFE lower confidence limit (5%)	0.5	0.6	0.8	0.9
RFFE best estimate	1.6	2.1	2.8	3.4
RFFE upper confidence limit (5%)	5.0	7.0	9.6	12.3

It can be seen that the regression estimates are typically higher than the RORB estimates for the more frequent events but have a reasonably high level of agreement for the 2% and 1% AEPs. In particular, the 1% AEP RORB estimate (with $K_c = 3.8$) is the middle value in the range of these estimates. It should be noted that the 10% AEP is not one of the design events being considered but is included here for comparison only.

On the other hand, the RFFE estimates (even the upper confidence limit) are much lower across all AEPs compared to both the RORB and regression estimates. It should be noted that the RFFE tool can give unpredictable results depending on how similar the catchments are which are being used for the estimate. In this case, the nearest gauged catchment was Station 4260504, which is 21.2 km away from the catchment, near Mount Compass. Based on the other flow estimates as well as anecdotal evidence regarding the frequency of flooding at Middleton, the RFFE estimates appear to be unreliable and have thus been excluded from the analysis.

3.4 Design Flow Hydrographs

Based on the comparison to the other flow estimates, a design K_c value of 3.8 was adopted. The temporal pattern for each of the design ARIs (100-year which produced the median peak flow rate for the critical duration was identified, and the entire event hydrograph was adopted as the inflow boundary condition to the HEC-RAS model of Middleton. The design hydrographs are shown below in Figure 3-2. Note that the initial periods of no flow (i.e. prior to commencement of runoff due to initial rainfall losses) have been removed from the hydrograph.

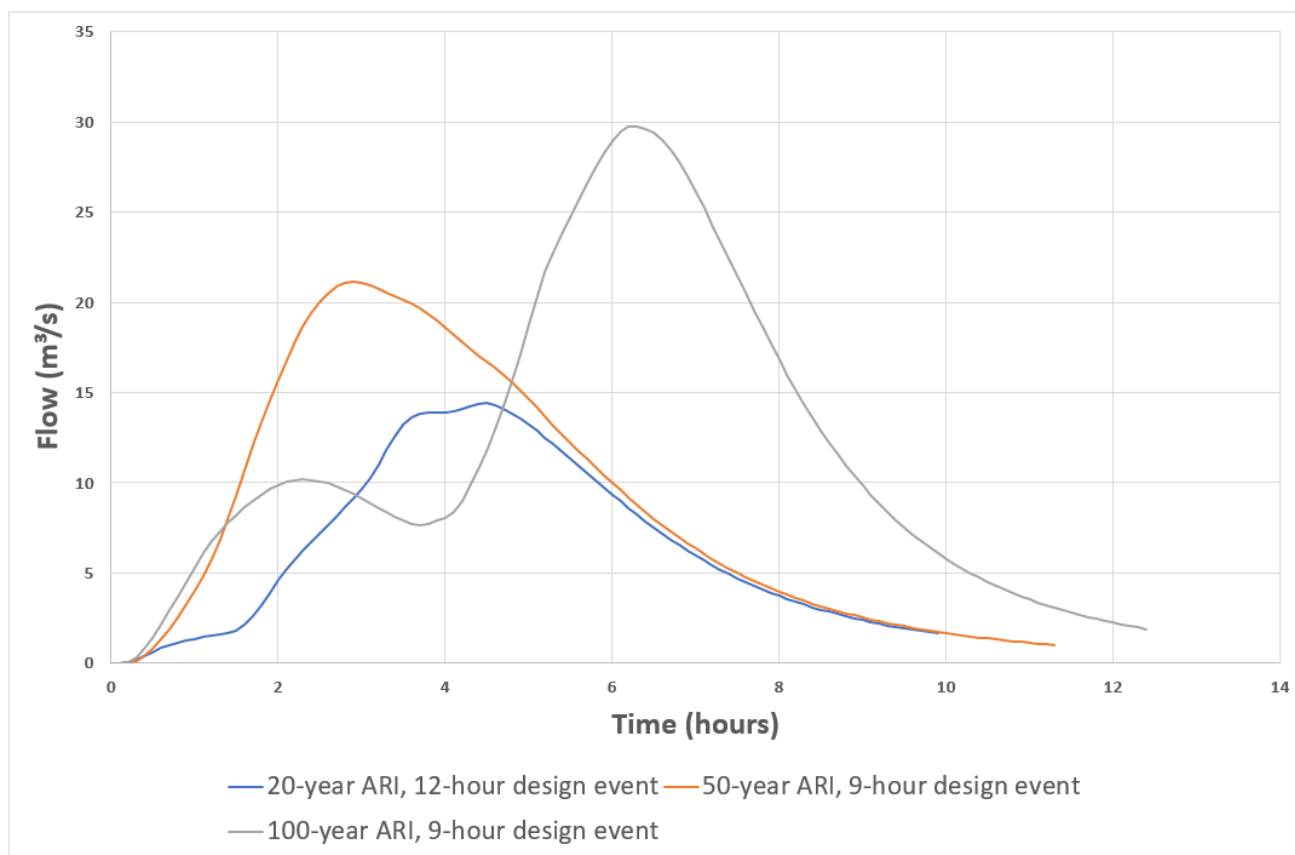


Figure 3-2: Design flow hydrographs

4 November 2022 Event Assessment

In November 2022, a significant rainfall event occurred across a large portion of South Australia, including most of the Fleurieu Peninsula. The time of greatest rainfall occurred on the 12th of November. This rainfall caused widespread flooding and damage, including within the township of Middleton. A number of roads and pedestrian bridges suffered significant damage during the event, including Glenford Gully Road, Flagstaff Hill Road, and three separate footbridges crossing the creek from Mills Terrace. A number of private residences, businesses and the Middleton Caravan Park were inundated by floodwaters during the event.

Shown below in Figure 4-1 is a screenshot of the BoM rainfall radar loop taken during the event.

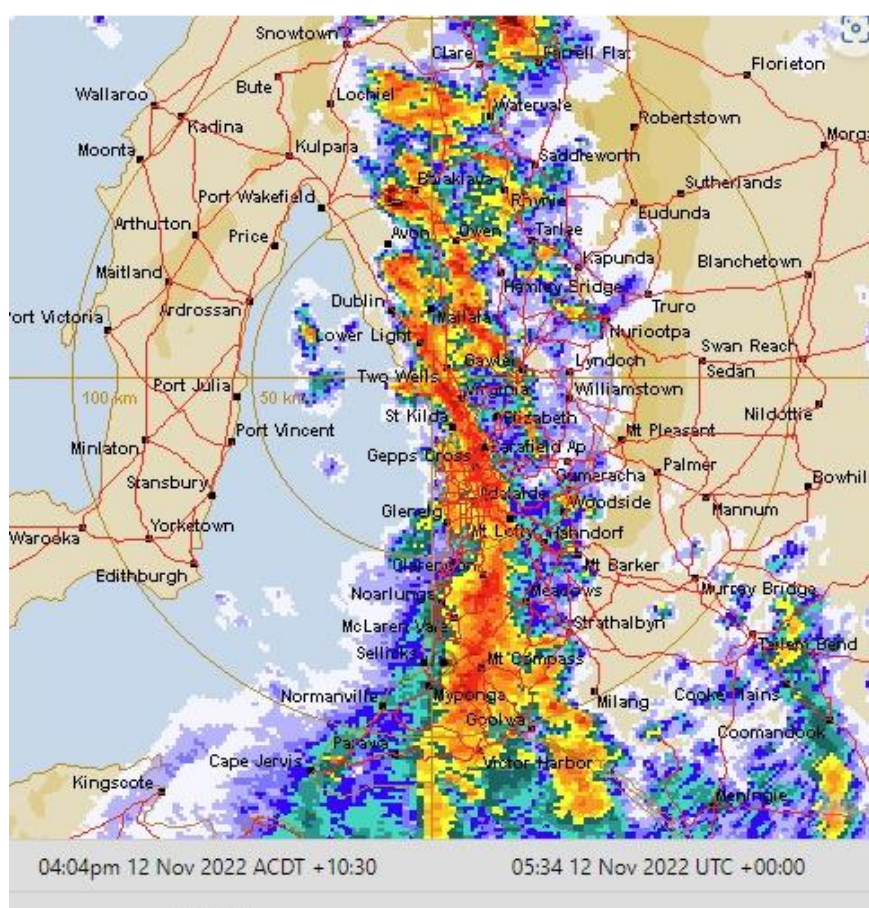


Figure 4-1: BoM rainfall radar imagery from during the November 2022 event

The information available regarding the November 2022 event is as follows:

- Photographs and videos from residents and news agencies from during the event;
- Flood markers (e.g. deposited rubbish on a chain-link fence);
- Other anecdotal evidence and reports from the event;
- Sub-daily rainfall data at the Strathalbyn gauging station (023747); and
- Daily rainfall totals at a number of other sites.



Unfortunately, no daily or sub-daily rainfall data at Middleton was available, nor any flood level gauge data. The Strathalbyn gauge (023747) is located approximately 32km north of Middleton. Full sub-daily rainfall data was available for this event at that gauge.

Based on reported values from other locations nearer to Middleton, the rainfall that fell at Strathalbyn has been increased by a factor of 1.2 in order to estimate the AEP of the event. According to the BoM 2016 IFD values for Middleton, the November 2022 rainfall event had an estimated rainfall AEP of approximately 2.4% or about a 40-year ARI event.

Weather conditions in the week leading up to the event were not particularly wet, with only scattered rainfall in the two days prior. However, rainfall totals in August, September and October were well above average, with data at the Goolwa Barrage gauge (23825) showing that the rainfall in August 2022 was well above the 90th percentile monthly total for August. Overall, the flood event in November occurred in a very wet year, and it is likely that farm dams in the upstream catchment were at or near full capacity, potentially leading to increased runoff from the catchment compared to average conditions.

In addition, anecdotal evidence suggests that partial blockage of the culverts under Goolwa Road occurred during the event due to a buildup of debris (including tree branches and rubbish). This is likely to have resulted in worsened flooding upstream of the culverts due to the reduction in culvert capacity and an increase in spill over Goolwa Road to properties between Goolwa Road and the rail line.

Based on the preliminary results from the HEC-RAS flood modelling (refer Section 5) that has been undertaken and based on the wet catchment conditions as outlined above, the runoff event that occurred within the creek catchment upstream of Middleton is likely to have been more extreme than suggested by recorded rainfall, and was potentially closer to a 100-year event. This is based on the model showing flood depths lower than the measured flood depths when the estimated 40-year flow event was run through the model. The extent of flooding and pattern of flood behaviour observed in modelling the design 2% and 1% AEP events suggests this as well.

Overall, the model was shown to be able to recreate the flood extents and patterns of flood behaviour as observed in the November 2022 event.



5 Hydraulic Modelling

5.1 HEC-RAS Model

A HEC-RAS 2D hydraulic model was developed in order to simulate the flood behaviour of the creek within Middleton and determine the depth and extent of flooding for each of the design AEPs. HEC-RAS is a program developed by the US Army Corps of Engineers that can undertake one- and two-dimensional unsteady flow calculations. Unlike some other hydraulic modelling software packages (e.g. TUFLOW), HEC-RAS has flexible mesh functionality which means that breaklines and the direction of cell faces can be easily aligned to important features, such as bridges or the crown of a road.

The base cell size for the model was 4m x 4m, with refinements for the following features:

- Goolwa Road
- The creek channel and banks
- Railway embankment
- Pedestrian underpass under the railway
- Railway bridge structure

The HEC-RAS model extended from 150m upstream of Goolwa Road to 300m downstream of the railway bridge. It should be noted that the HEC-RAS model represented riverine flooding only (due to flows from the creek) and does not include runoff from the immediate urban catchment of Middleton.

5.1.1 Digital Elevation Model

The DEM for the model was created using the survey contours to accurately model the creek bed and banks. Survey of the pedestrian underpass under the railway, and the creek railway bridge was also incorporated. LiDAR was then used to supplement the survey to create the base model DEM.

Some smoothing of the DEM was required at the interface between the edge of the survey and the LiDAR where there was some non-systematic discrepancy between survey and LiDAR levels.

Known floor levels of a number of buildings were enforced within the DEM to create the final model DEM.

5.1.2 Boundary conditions

As noted previously, the upstream inflow boundary condition for each design event were the design flow hydrographs (refer Section 3.4).

The downstream boundary condition of the model is a normal depth boundary. The boundary condition and end of the model was set sufficiently far away from the area of interest such that the modelling results within the area of interest would not be affected by the choice of boundary condition.

5.1.3 Manning's n values

Manning's n values were determined for the catchment according to information obtained from the site visit, satellite imagery and knowledge gained from previous modelling experience.

It should be noted that a Manning's n value of 0.5 was adopted within building footprints to represent the significant obstruction to flow caused by the external and internal walls and obstacles within buildings.

The Manning's n values adopted for each of the surface types is shown below in Table 5-1.



Table 5-1: Adopted Manning's n values

Surface Type	Manning's n value
Grassed areas	0.035
Road reserves	0.025
Building footprints	0.5
Creek	0.08
Concrete structures	0.02
Railway embankment	0.05
All other areas	0.08

5.1.4 1D elements

As this flood study is focused only on riverine flooding from the creek and does not include local runoff from the immediate urban catchment, the underground stormwater network within Middleton has not been included within the model.

The culverts underneath Goolwa Road (which the creek flows through) were modelled as a 1D element within HEC-RAS. The culverts consist of two culvert cells, each 2.4m wide by 2.2m high. A design blockage factor of 10% (refer Section 2.3.2) was applied to these culverts by blocking the bottom 10% of the height of these culverts.

5.2 Flood Hazard Mapping

In terms of floodplain management, flooding is a hazard which has the potential to cause damage to the community. Personal danger and physical property damage caused by floods varies both in time and space across the floodplain. In some areas floodwaters can flow deeply and slowly, while in other areas they may be more shallow and swift; both situations may unbalance people and vehicles and sweep them away. Structures can be undermined or have their structural integrity damaged by floodwater and debris, and contents are vulnerable to contact with floodwater.

Flood hazard mapping assists with identifying the relative degree of hazard in a floodplain. This allows for effective floodplain management and emergency response planning.

Flood hazard maps based on the modelling undertaken in HEC-RAS have been developed using the combined flood hazard threshold curves developed by Smith et al (2014) as shown in Figure 5-1 below. The combined flood hazard curves are divided into a number of hazard classifications that are based on thresholds for the stability of people, vehicles and buildings in floods.

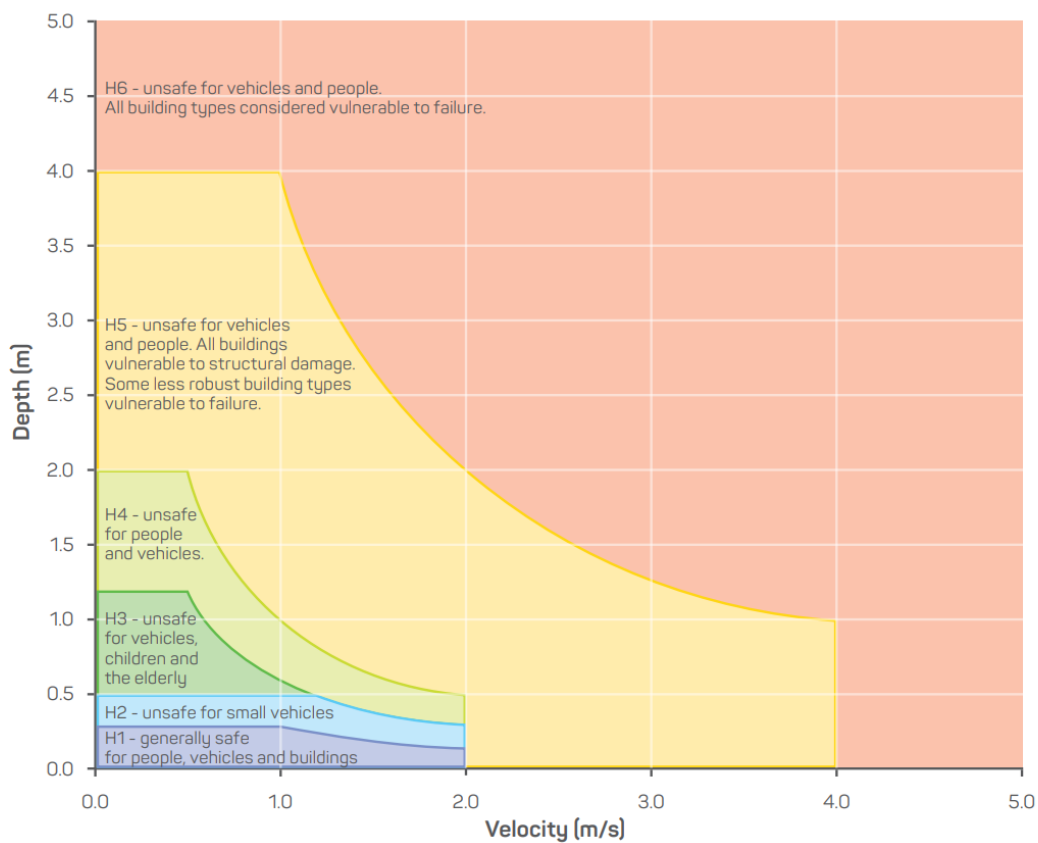


Figure 5-1: Combined flood hazard curves (Smith et al, 2014)

Flood maps showing the flooding depth and hazard (separately) are shown in Appendix A for the 100-year, 50-year and 20-year events for existing catchment conditions.



6 Existing Infrastructure Results

The results of the HEC-RAS modelling demonstrate that the current capacity of the creek itself downstream of Goolwa Road begins to be exceeded as the flow rate through the creek exceeds approximately 10 m³/s, with minor breakouts on the western side of the creek entering the Middleton Caravan Park.

The results also show that the capacity of the existing culverts under Goolwa Road begins to be exceeded when the arriving flow rate exceeds 18.5 m³/s, with flows backing up behind Goolwa Road and breaking out of the creek to the east and west. Properties on the western side are inundated as the water level increases and begins to spill over William Street. As flow rates increase and the natural flood storage behind Goolwa Road is filled, flows begin to spill over the western side of Goolwa Road, inundating the road, the pharmacy and supermarket, and the Middleton Caravan Park.

Analysis of the hydrology results suggests that people within Middleton may have between three and six hours of warning time following the commencement of rainfall before flood flows arrive at Middleton. This may be sufficient time so as to allow evacuation of vulnerable people in flood prone areas to safety.

Discussed below are the results from each of the specific design AEPs that were modelled. Hydrographs at a number of key locations have been included in the model, and colour-coded so that upstream inflows (blue), flows under/over Goolwa Road (green) and under the railway (red) may be compared.

Overall, the modelling results indicate that the current combined system (including the creek and the Goolwa Road culverts) have a combined flood standard of a 20-year ARI. This is defined as the event magnitude at which flooding of private property begins to occur. Flooding of private property in this context does not necessarily indicate a risk of damage to buildings or a safety risk, but instead means that a flood depth of greater than zero is occurring within private property outside of the main creek corridor.

6.1 20-year ARI event

In the 20-year event, some localised flooding occurs in the Middleton Caravan Park adjacent to the creek as the capacity of the creek is exceeded during the event. However, this water does not present a significant life safety or damage risk, with all of the breakout flows having a Hazard value of H1.

Within the creek itself, the Hazard category is up to a value of H5. This presents a major life safety risk for any person that may enter the creek during the flood.

See Figure 6-1 below for key hydrographs. It can be seen that the capacity of the Goolwa Road culverts is not exceeded, with only a short lag time between flows reaching the culverts and passing under the railway bridge. The inflow line (blue) is not shown as it is essentially identical to the flow rate passing through the culverts.

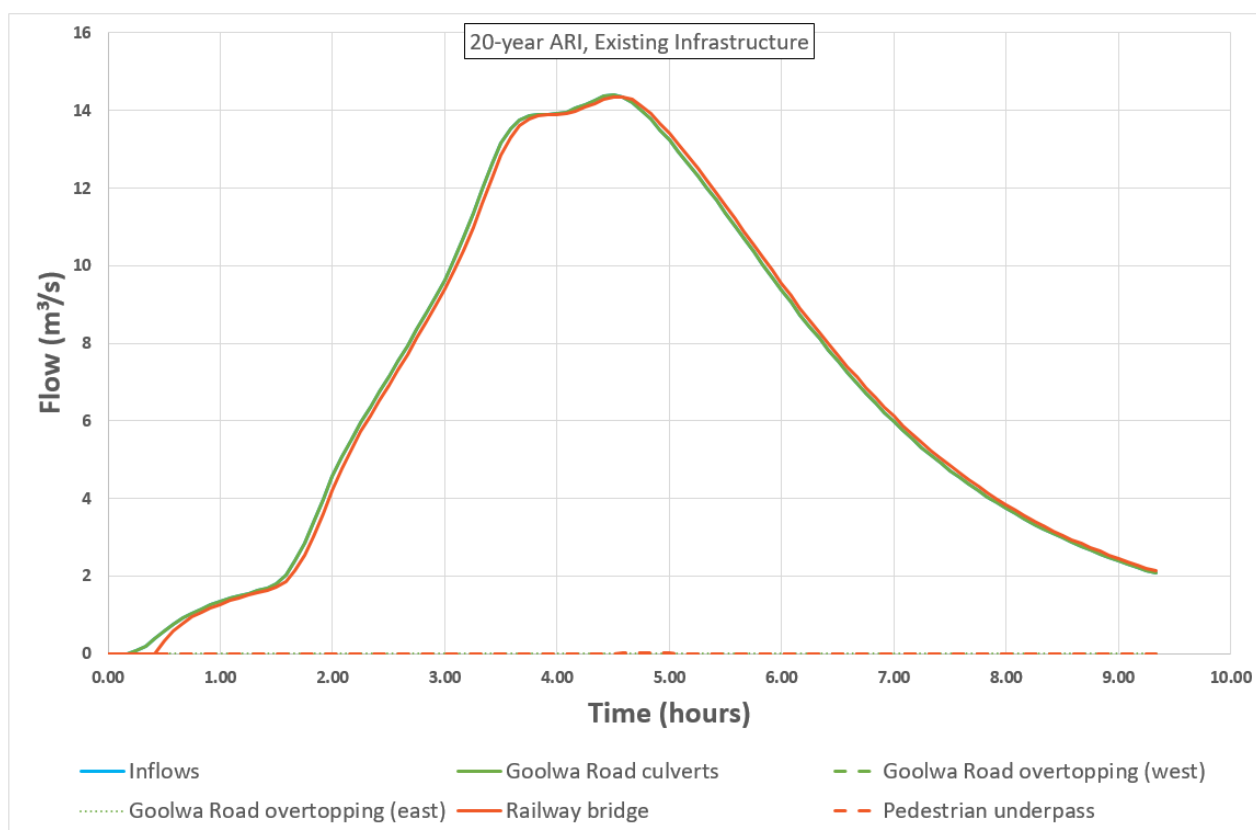


Figure 6-1: Flow hydrographs at key locations for the 20-year event (existing infrastructure)

6.2 50-year ARI event

In the 50-year event, flooding occurs in a significant portion of the Middleton Caravan Park. This is primarily due to flows overtopping Goolwa Road on the western side (1.7 m³/s at peak) and flowing south through the caravan park. The flood hazard within the park varies between H1 at the northern end of the park and H3 where the water is much deeper. A number of small units are located in the region with a H3 Hazard category which presents a life risk to children and the elderly. It is noted that these demographics may make up a large percentage of visitors at the caravan park.

It is noted that there are small regions of Hazard category H2 on Goolwa Road, which may present a risk to the vehicle stability of small cars, but overall the safety risk on the road is fairly low.

A number of private properties north of Goolwa Road and along William Street are inundated as flows break out of the creek. The Hazard category through this area is H1-H2, so it is unlikely that life safety will be an issue in this location. However, these flood flows are likely to cause non-structural damage to these houses e.g. water damage to carpets, walls or goods stored near floor level.

A Hazard category of up to H5 is observed within the pedestrian underpass under the railway line, with velocities of greater than 1.7 m/s and depths of 0.8 m (which may be at or above waist level for most people). This presents a significant safety risk for anyone in this location.

Within the creek itself, the Hazard category is up to a value of H5. This presents a major life safety risk for any person that may enter the creek during the flood.

Flows passing through this underpass flow over Ellensford Terrace and through Abbots Reserve. It appears that no properties are inundated by these waters.



See Figure 6-2 below for the key hydrographs from the model. It can be seen that most of the flows that spill over Goolwa Road do not enter back into the creek and through the railway bridge but instead flow through the caravan park and through the pedestrian underpass.

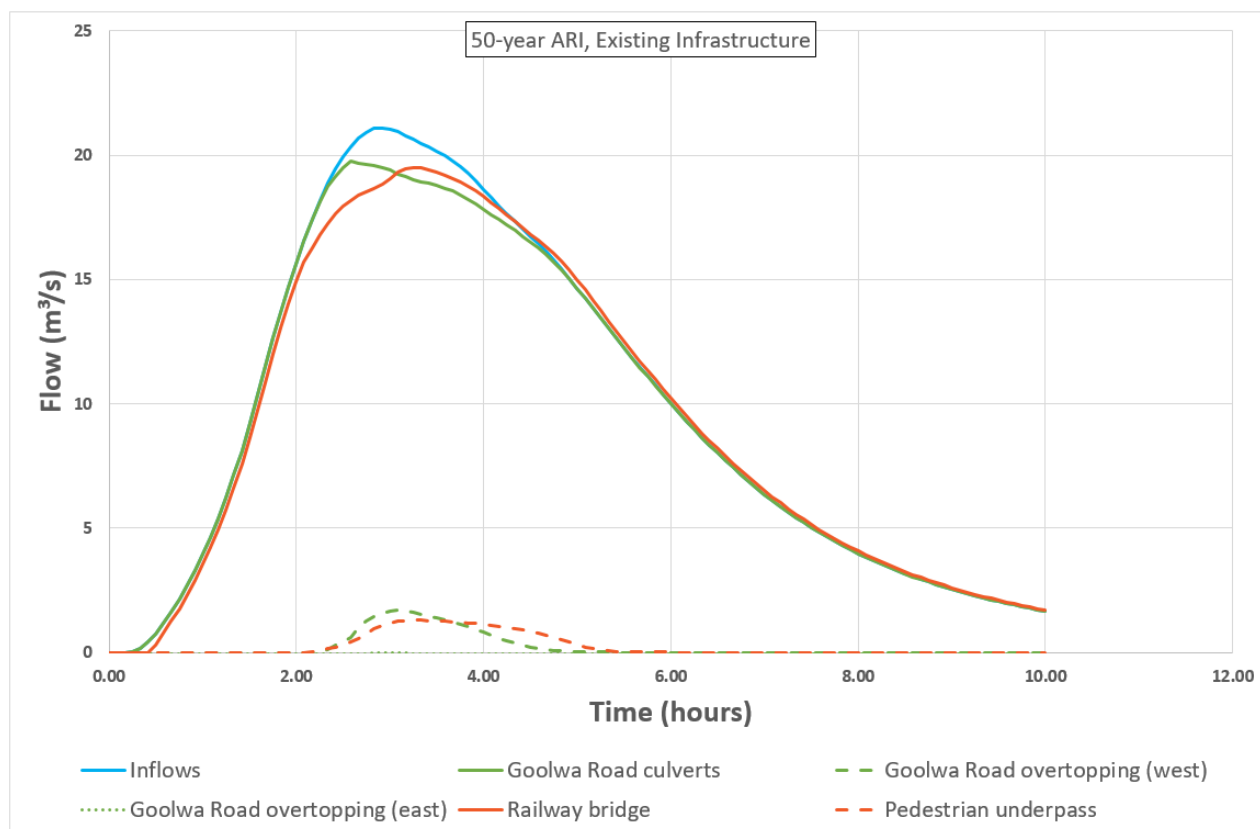


Figure 6-2: Flow hydrographs at key locations for the 50-year event (existing infrastructure)

6.3 100-year ARI event

In the 100-year event, flooding across Middleton is fairly widespread. Almost the entire Middleton Caravan Park is inundated, with Hazard categories of H3 widespread across the park presenting a significant life safety risk.

In the 1% event flows are observed to overtop Goolwa Road on both the western and eastern side of the creek. The flows spilling over Goolwa Road on the west inundate the pharmacy and supermarket and private property adjacent to the creek.

Flows spilling over the eastern side of Goolwa Road appears to occur when the inflow flow rate exceeds roughly 22 m³/s. These flows cause inundation to a number of private properties that were not flooded in the 50-year event.

It is noted that there are small regions of Hazard category H2 on Goolwa Road, which may present a risk to the vehicle stability of small cars, but overall the safety risk on the road is fairly low.

A Hazard category of up to H5 is observed within the pedestrian underpass under the railway line, with velocities of greater than 2.5 m/s and depths of 1.0 m (which is above waist level for most people). This presents a significant safety risk for anyone in this location.



Flows passing through this underpass flow over Ellensford Terrace and through Abbots Reserve. It appears that the backyards of several properties are inundated by these waters but no actual buildings are affected.

A number of private properties north of Goolwa Road and along William Street are inundated as flows break out of the creek. The Hazard category through this area is increased compared to the 50-year event, with Hazard values of up to H3. This flooding presents a significant life safety risk and is likely to cause significant non-structural damage to these properties.

Within the creek itself, the Hazard category is up to a value of H6. This presents a major life safety risk for any person that may enter the creek during the flood.

It is noted that the railway bridge and pedestrian underpass remain the controlling flow outlets, with no flows spilling over the top of the railway embankment.

It is also worth noting that the flood extents shown for the 100-year event modelling are closer to the estimated flood extents from November 2022. The pattern of flood behaviour, particularly the inundation of several properties on the eastern side of the creek downstream of Goolwa Road, broadly represents the flood behaviour observed in November 2022.

See Figure 6-3 below for key hydrographs.

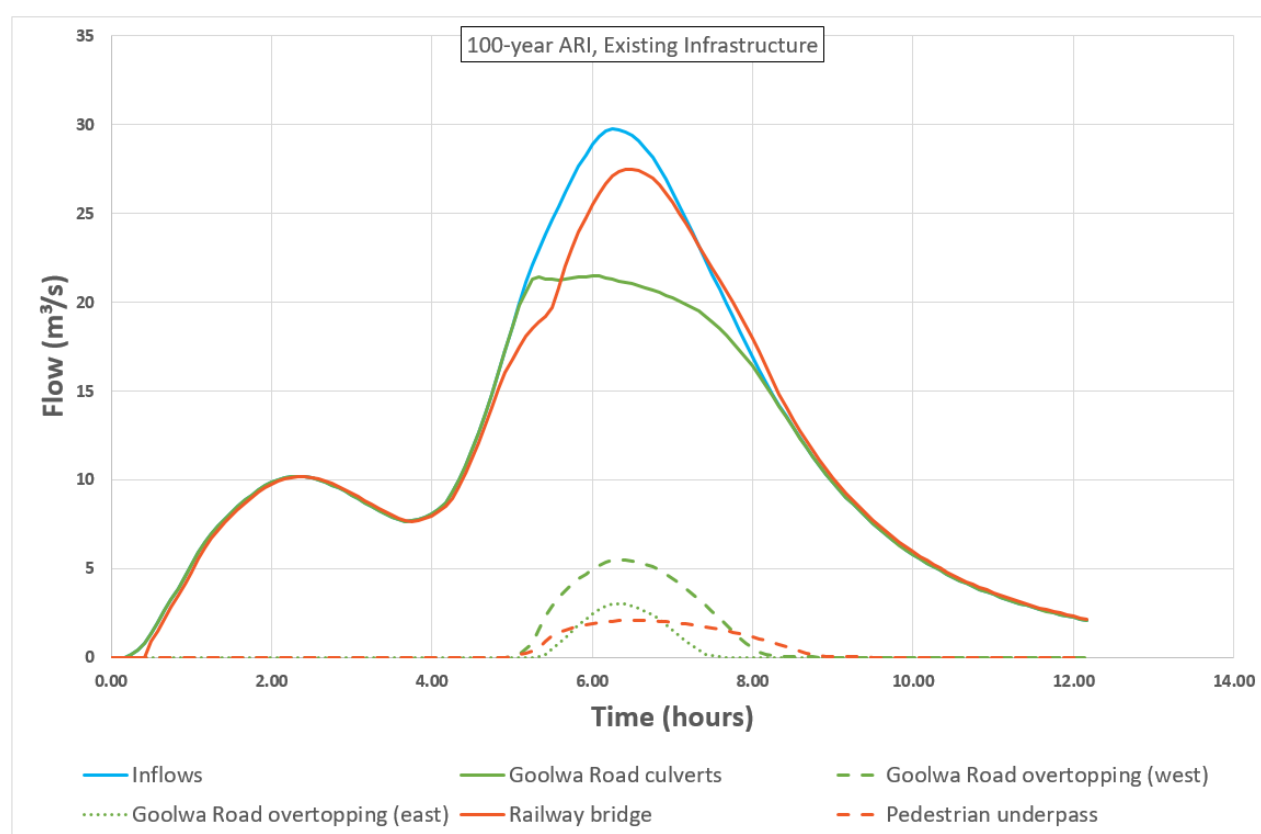


Figure 6-3: Flow hydrographs at key locations for the 100-year event (existing infrastructure)



7 Potential Flood Mitigation Options

7.1 Identified Options

Based on the results of the modelling for the existing infrastructure scenario, a shortlist of potential flood mitigation options to be investigated further was developed. These are as follows:

- Creating levee banks along the creek to prevent floodwaters breaking out of the creek;
- Widening the creek channel to increase the creek flow capacity;
- Increasing the culvert size under Goolwa Road to increase the capacity under the road and reduce the likelihood of culvert blockage;
- Reducing the amount of understory vegetation along the creek lines to increase the creek's capacity;
- Providing flood detention storage upstream of Middleton to reduce the peak flow rates that arrive at Middleton; and
- Providing an early warning system to allow for early evacuations and flood response plans to be activated prior to the arrival of floodwaters.

These options are described in further detail below.

In addition to the above options, it is recommended that community engagement and education be undertaken regardless of which mitigation options are implemented, as it is likely to provide significant benefit in terms of the reduction of flood damages and safety risks at a low cost and with minimal approvals required. As such, this has not been included as a separate mitigation option/modelling scenario.

A number of the mitigation options require access to the creek. However, at present the majority of this land is privately owned with no easement, with only a small block of land east of the creek publicly owned apart from the road corridor. This means therefore that many of the mitigation options will require land access approvals, creation of a drainage easement or purchase of private property. Figure 7-1 below shows the demarcation between public and private land.



Figure 7-1: Land ownership in Middleton



7.1.1 Clearing of creek vegetation

The first mitigation option is to undertake clearing/reduction of vegetation within and adjacent to the creek to improve flow conveyance along the creek and reduce the risk for blockage. Grassy weeds, small bushes and fallen organic debris (e.g. branches) would be removed from the channel bed and banks, with trees and other larger native vegetation to be retained.

In order to remain effective, the creek would be required to be maintained regularly to retain the benefit of creek clearing. No Council-owned easement exists along the creek, so this option would require access approval to the creek to be granted from multiple private landowners. It is likely that this maintenance work may be required every five years (subject to a full assessment of the required works). Refer to Section 9.1.2 for details of how the cost estimate for this option was derived.

This was implemented into the HEC-RAS model by reducing the Manning's n value for the creek bed from 0.08 to 0.04 and the creek banks from 0.08 to 0.05.

7.1.2 Channel widening

This mitigation option involves widening the creek cross-section to increase the flow capacity of the creek. A base width of 3m was adopted with a typical top width of 14m. A typical cross section (showing the existing and widened sections) of the creek is shown below in Figure 7-2.

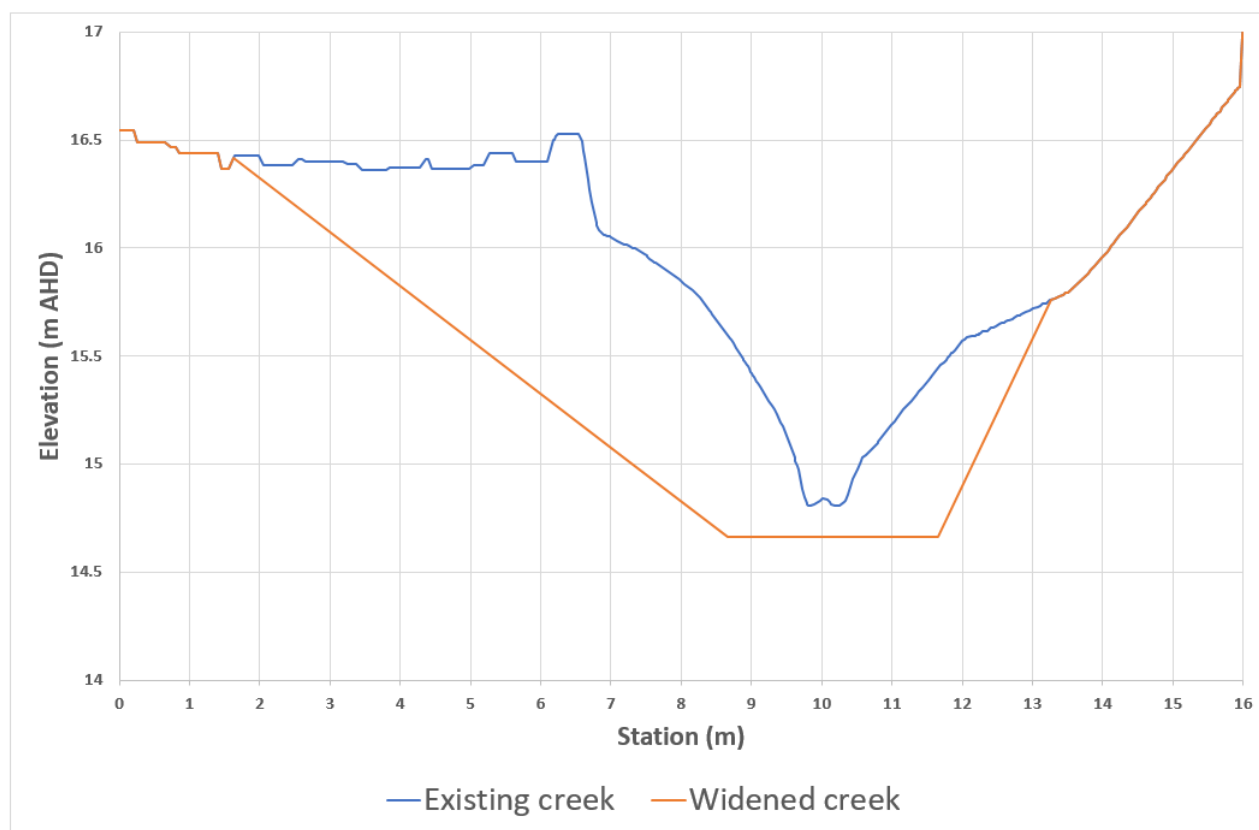


Figure 7-2: Example cross section of widened creek

A number of private residences lie immediately adjacent to the creek, and so undertaking the widening will not be possible along all the entire length of the creek within Middleton.

It should be noted that similarly to the creek clearing option, the majority of the land in which widening would be undertaken is private property and so this will require landholder acceptance or even acquisition of private property.



As the associated earthworks are being undertaken, the majority of vegetation will be removed. It has been assumed that the creek area will then be replanted to achieve a condition the same as for the creek clearing option. For this reason, the same reduced Manning's n values were adopted during the HEC-RAS modelling.

7.1.3 Creek levee banks

This mitigation option involves creating levees at the top of the channel banks or within the floodplain to contain water within the levees, preventing floodwaters from inundating private property. These levees would be created at key locations identified during the modelling of the existing infrastructure scenario where floodwaters break out of the creek.

In some locations, there would be adequate room (pending approval from landholders) to construct an earthen levee. In other locations, owing to a lack of space between the creek banks and buildings, a vertical flood wall would need to be constructed. The locations of the levees are shown below in Figure 7-3.



Figure 7-3: Approximate locations of potential flood levees



It should be noted that as with the previous two options, much of the space required lies within private property and landholder approval would be required.

The required height of the levees to prevent flood inundation varies by location and also due to the combined effects of other mitigation options (e.g. creek clearing), and so the required levee height at each location is explained in the post-mitigation modelling results (see Section 8). The required levee heights typically range between 0.4 and 1.2 metres above existing ground levels.

7.1.4 Flood detention

This mitigation option involves creating a detention basin upstream of Middleton by constructing an earthen dam on the creek with an outlet. The low flow outlet would allow for regular environmental flows along the creek whilst detaining larger flows in the storage and greatly reducing the peak flow rate downstream of the dam.

This option is different from the other options considered in that it involves no direct work within the township of Middleton and does not improve the conditions of the creek itself.

The maximum dam height would be approximately seven to eight metres above existing levels. This would create a basin with a storage volume of approximately 250 ML below dam crest level.

Depending on the configuration and storage volume of the basin, additional benefits may be provided such as the inclusion of retention storage, to provide a water quality improvement or water harvesting opportunity.

It should be noted that like any dam this option would come with the risk associated with the potential for dam failure. Should the dam fail during a large rainfall event, dambreak flows may cause significant flooding downstream that may even be in excess of the flooding that would have occurred naturally prior to the construction of the dam. The dam would need to have a large spillway structure to help offset this risk.

The modelling undertaken for the existing infrastructure scenario indicated that the flow capacity of the creek (without flows breaking out of the creek within Middleton) is approximately 10 m³/s. Preliminary modelling in DRAINS (1-dimensional hydrologic/hydraulic modelling software) indicated that a basin with a volume of 200ML would be required to reduce the peak flows during a 100-year flood to less than 10 m³/s.

A site immediately southeast of the intersection between Lakeview Road and Glenford Gully Road has been identified as a possible suitable location for such a dam. It should be noted that this lies on private property and would require purchase of a portion of the land or the creation of an easement. This location is shown below in Figure 7-4.

The outflow hydrograph from this basin has been modelled in DRAINS and inserted into HEC-RAS as the upstream boundary condition to represent this option.

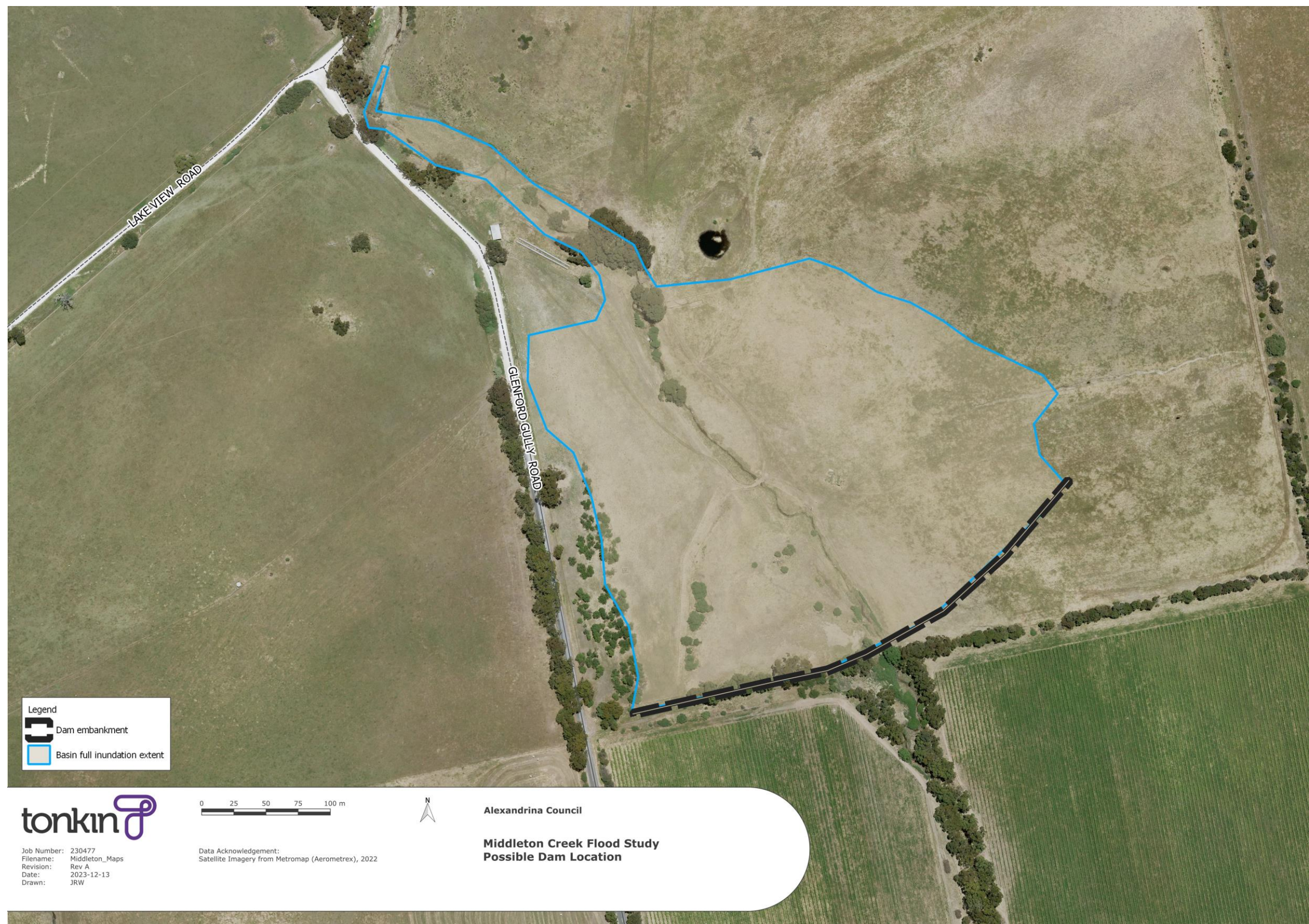


Figure 7-4: Possible dam location



7.1.5 Early warning system

This option involves the installation of creek gauge and rainfall sensors that would provide a warning to the residents of Middleton and local emergency services prior to the arrival of flood flows from upstream. The hydrological modelling (refer Section 3) demonstrated that the time delay between the commencement of runoff and the arrival of the peak flood wave is between two and four hours. This may be sufficient time for both residents and local emergency services to take appropriate action, which may involve simply moving valuable items off the floor within houses to actual evacuation of residents at risk.

This may also involve education programs within the community, especially to encourage the preparation of flood response plans prior to any actual flood occurring. This education and planning would need to be refreshed on a regular basis to ensure that the community retains their preparedness and experience from previous flooding (including November 2022).

Table 3-5 in the Rapid Appraisal Method (RAM) developed for the Department of Natural Resources and Environment (Read Sturgess and Associates, May 2000) includes proposed ratios of damage reductions for various warning times and community preparedness/flooding experience.

Based on this table, an estimated reduction of damages of 30% to 40% may be possible given the warning times for the creek and the community being prepared.

While construction activities would be required for the installation and maintenance of these gauges/systems, no construction work would be required within Middleton.

The key stakeholders involved in managing and maintaining this system would be the Department for Environment and Water (DEW) and the Bureau of Meteorology (BoM).

7.1.6 Upgrade of Goolwa Road culverts

This option involves upgrading the culverts underneath Goolwa Road. Goolwa Road is a Department for Infrastructure and Transport (DIT) asset and Council may be able to advocate for funding to upgrade the culverts.

The modelling results for the existing infrastructure scenario show that the creek is the current “weak point” in the system in that flows begin to break out of the creek well before the capacity of the culverts is exceeded. However as stated previously, no creek easement currently exists and approval from private landowners, or even land purchase, would be required for most of the other mitigation options.

If this is either not possible or not desirable for Council to undertake, upgrade of the culverts may still provide some benefit in increasing the flow able to pass through the culverts, thereby reducing the flows spilling over Goolwa Road and inundating properties south of the road.

Previous modelling by DIT suggests that the current capacity of the culverts is approximately 32.6 m³/s (assuming no blockage of the culverts) or 19.4 m³/s (with 25% blockage). However, this was based upon a road overtopping level of 18.08 m AHD. In reality, flows begin to break out of the creek and inundate properties at a level of 17.70 m AHD. It is expected that the primary issue is the lack of flow capacity within the creek, but it does suggest that in order to pass the 100-year peak flow (29.8 m³/s) with an upstream level of less than 17.70 m AHD, an upgrade of the culvert may be required.

In order to have a capacity equal to the 100-year design flow rate, the culvert has been modelled within HEC-RAS as a single-span 7.2m wide by 2.2m high culvert (effectively a small bridge). As the central wall dividing the culvert cells is removed, the risk for blockage due to branches/trees being pushed against the central wall is eliminated, and therefore the culvert has been modelled as having 0% blockage.



7.2 Scenario Summary

Each of the mitigation options detailed above may be considered in isolation; however, combining multiple options (provided the necessary approvals were granted) may provide an even greater benefit. For this reason, eleven scenarios (ten mitigation scenarios plus the existing infrastructure scenario) have been modelled in HEC-RAS, with various possible combinations of mitigation options.

It should be noted that as the early warning system option involves no actual change to flooding behaviour, this has not been modelled as a scenario within HEC-RAS. This has been included in this report as Scenario 11.

It is recommended that the community education programme be undertaken regardless of which other mitigation options are implemented, as it is likely to provide significant benefit in terms of the reduction of flood damages and safety risks at a low cost and with minimal approvals required. As such, this has not been included as a separate mitigation option/modelling scenario.

As stated previously, some of the options focus on improving the flow conveyance of the creek. This also serves to lower the peak water levels within the creek and therefore the height of levee required to contain flows within the creek. The required levee height for each of these scenarios will be shown in the results section (Section 8).

Table 7-1 below shows a summary of the twelve scenarios.



Table 7-1: Flood mitigation scenario summary

Scenario	Mitigation Options	Land Status	Possible Action
Scenario 0	Existing infrastructure (no mitigation)	None affected	
Scenario 1	Creek clearing	Private land	Council to pursue drainage easement or land purchase for potential future works
Scenario 2	Channel widening	Private and public land	Council to pursue drainage easement or land purchase for potential future works
Scenario 3	Levees	Private and public land	Council to pursue drainage easement or land purchase for potential future works
Scenario 4	Flood detention	Private land	Council to pursue drainage easement or land purchase for potential future works
Scenario 5	Goolwa Road culvert upgrade	Public land	Council to seek approval from DIT
Scenario 6	Creek clearing, levees	Private and public land	Council to pursue drainage easement or land purchase for potential future works
Scenario 7	Channel widening, levees	Private and public land	Council to pursue drainage easement or land purchase for potential future works



Scenario 8	Creek clearing, Goolwa Road culvert upgrade	Private and public land	Council to pursue drainage easement or land purchase for potential future works, and approval from DIT
Scenario 9	Channel widening, Goolwa Road culvert upgrade	Private and public land	Council to pursue drainage easement or land purchase for potential future works, and approval from DIT
Scenario 10	Creek clearing, levees, Goolwa Road culvert upgrade	Private and public land	Council to pursue drainage easement or land purchase for potential future works, and approval from DIT
Scenario 11	Early warning system (not modelled in HEC-RAS)	Private land	Council to pursue drainage easement or land purchase for potential future works



8 Modelling Results and Discussion

Modelling results for each of the mitigation scenarios (not including Scenario 0 or Scenario 11) are discussed in this section. Flood maps for the 100-year ARI event for these mitigation scenarios, showing the maximum flood depths, hazard category and change in flood depths compared to existing, are shown in Appendix A.

The flood depth change maps (referred to as depth difference maps) are calculated by subtracting the maximum depths for the existing case (Scenario 0) from the scenario being considered. A side-effect of this method is that where the terrain is lowered at a certain point (e.g. in the channel widening scenarios), the depth at that point may increase even if the overall peak water surface level is reduced. This can show in the depth difference maps as an increase in flood depths. However, this does not necessarily indicate a worsening of flooding in that location.

The results discussion for each of the scenarios includes the new standard of the creek and culvert system, compared with the existing standard of a 20-year ARI.

This standard is defined as the event magnitude at which flooding of private property begins to occur. Flooding of private property in this context does not necessarily indicate a risk of damage to buildings or a safety risk, but instead means that a flood depth of greater than zero is occurring within private property, outside of the main watercourse corridor.

All discussion in this section related to the changes in flood risk due to the 100 year ARI event, which is typically the standard adopted for the protection of dwellings and private property.

8.1 Scenario 1: Creek Clearing

The results show that clearing weeds and small bushes from the creek results in some improvement in flow depths within the creek itself (150mm to 250mm generally). This is due to an improvement to the flow conveyance of the creek.

However, this improvement does not translate to a similar improvement in terms of flooding within the properties upstream of Goolwa Road. Flood depths in and around private properties upstream of Goolwa Road are generally reduced only by up to 30mm, and the flood hazard category generally remains the same.

Depths of flow over Goolwa Road are also only reduced by roughly 40mm which is not likely to have a material impact on the driveability of the road.

Downstream of Goolwa Road, the Middleton Caravan Park and properties to the east of the creek see a reduction of depths of approximately 120mm and 150mm respectively. Again, generally speaking the hazard category and hence safety/property damage risk is minimally impacted.

This option results in a new flood standard of the overall system of a 40-year ARI.

8.2 Scenario 2: Channel Widening

The results show that channel widening of the creek (including reducing the amount of vegetation within the creek) results in significant improvement in flow depths within the creek, with reductions of 300mm and 750mm upstream and downstream of Goolwa Road respectively.

This improvement translates to a minor improvement in terms of flooding within the town. Flow depths in and around the private properties upstream of Goolwa Road as well as over Goolwa Road itself are reduced by up to 60mm. The hazard category in these locations is generally unchanged.



Flood depths within the caravan park are reduced by approximately 250mm, and the flood hazard category within the caravan park is reduced in some locations from a category 2 to a category 1. This represents a small benefit in terms of safety/property damage risks.

Flood depths within the properties to the east of the creek downstream of Goolwa Road are reduced by up to 220mm, and the flood hazard category is reduced by one category (e.g. from a 3 to a 2, or a 2 to a 1) in some locations.

This option results in a new flood standard of the overall system of a 40-year ARI.

8.3 Scenario 3: Flood Levees

The results show that should the levees be constructed, and they do not fail during the event, that all of the properties may be protected at least up to a 100-year flood event.

However, within the creek flood depths are significantly increased, with increases of 750mm and 300mm upstream and downstream of Goolwa Road respectively. This is due to the fact that floodwaters that previously spilled out through private property and over Goolwa Road now must be contained within the creek and passed through the Goolwa Road culverts.

The difference in flood level increase between upstream and downstream of Goolwa Road shows that the capacity of the Goolwa Road culverts does become a limiting factor in passing flows through the creek.

Table 8-1 shows the required heights for each levee section for Scenario 3. The required height varies along the length of each levee depending on the elevation of the existing terrain where the levee is to be situated. It should be noted that levees are typically constructed with an additional freeboard allowance (e.g. 300mm) to account for local variations in the water surface profile e.g. as a result of waves.

Table 8-1: Required levee heights for Scenario 3

Levee location	Approximate height required ¹ (m)	
	Minimum	Maximum
A	0.6	1.3
B	0.5	1.4
C	0.5	0.5
D	0.4	0.8
E	0.7	0.7

1 The required height for each levee varies along its length according to the variation in water surface elevation down the creek. This value does not include any freeboard allowance.

Within the creek area bounded by the levees, the flood hazard categories are generally increased to a 3 or 4, which represents a significant safety risk should someone enter this area.

This option results in a new flood standard of the overall system of a 100-year ARI.



8.4 Scenario 4: Flood Detention

The results show that constructing a dam to create flood storage and hence hydrograph attenuation has the potential to entirely prevent flood inundation (up to a 100-year event) within the town due to flows from upstream. Flow depths within the creek in this scenario are reduced by up to a metre.

This statement does not mean that all flooding would be prevented within Middleton however. It should be noted that this option, like all the other options, would provide little benefit to flooding situations arising from the immediate local catchment. Any local stormwater issues would need to be resolved potentially via upgrades to the stormwater infrastructure within the town. However, the local stormwater issues would be at a much smaller scale than the regional flooding that occurs due to the creek's capacity being exceeded.

Ideally, the dam would be sized such that Middleton is provided with a 100-year ARI level of flood protection, as a result of riverine flooding.

8.5 Scenario 5: Goolwa Road Culvert Upgrade

The results show that upgrading the culverts under Goolwa Road as previously described would increase the peak flow through the culvert from 20.5 m³/s (existing) to 25.0 m³/s (upgraded).

However, this increase would have only a small benefit to flooding. A decrease in flood depths is observed of up to 60mm in and around the houses upstream of Goolwa Road, and flow depths over Goolwa Road are reduced by up to 45mm and 80mm west and east of the creek respectively. The upgrade appears to have little effect on the flood hazard category.

Flood depths within the creek immediately downstream of the culverts are increased by up to 120mm as a result of the additional flow through the culverts.

Flood depths within the caravan park are generally unchanged, whilst depths east of the creek are reduced by up to 140mm. Flooding behaviour within the caravan park is generally dominated by the volume of water that becomes banked up behind the railway embankment over time, and so an increase in the maximum flow capacity through the culverts (which only takes effect during the peak of the event) makes little difference to this.

However, flooding behaviour to the east of the creek (downstream of Goolwa Road) occurs as a result of flows overtopping Goolwa Road on the eastern side and this occurs only during the peak of the event. The ~20% increase to flow capacity in the culverts results in the flows overtopping Goolwa Road on the eastern side to be reduced from 3.1 m³/s down to 0.7 m³/s, or an 80% reduction. This greatly reduces the flood flows inundating properties to the east.

The upgrade to the culvert, particularly by making the culvert single-span, would significantly reduce the probability of major culvert blockage by a large tree or branches as there would be no central wall for debris to be caught against.

This option results in a flood standard of the overall system of a 20-year ARI which is unchanged compared to the base case (Scenario 0). Although flood depths upstream of Goolwa Road are reduced, breakout flows still occur on the downstream side of the road at the same flow rate.



8.6 Scenario 6: Creek Clearing and Levees

The results show that as with Scenario 3, should the levees be constructed, and they do not fail during the event, that all of the properties may be protected at least up to a 100-year flood event.

However, within the creek flood depths are significantly increased, with increases of 530mm and 110mm upstream and downstream of Goolwa Road respectively. The water levels resulting from this scenario are an improvement to the peak water levels compared to Scenario 3 of approximately 200mm and is reflected in a similar reduction in the required levee heights.

This improvement is due to the improvement in the flow conveyance of the creek as a result of the clearing of vegetation. However, as with Scenario 3, the Goolwa Road culverts remain a limiting factor in passing flows through the creek.

Table 8-2 shows the required heights for each levee section for Scenario 6. The required height varies along the length of each levee depending on the elevation of the existing terrain where the levee is to be situated. It should be noted that levees are typically constructed with an additional freeboard allowance (e.g. 300mm) to account for local variations in the water surface profile e.g. as a result of waves.

Table 8-2: Required levee heights for Scenario 6

Levee location	Approximate height required ¹ (m)	
	Minimum	Maximum
A	0.4	1.1
B	0.3	1.1
C	0.2	0.2
D	0.1	0.5
E	0.4	0.4

¹ The required height for each levee varies along its length according to the variation in water surface elevation down the creek. This value does not include any freeboard allowance.

Within the creek area bounded by the levees, the flood hazard categories are generally increased to a 3 or 4, which represents a significant safety risk should someone enter this area.

This option results in a new flood standard of the overall system of a 100-year ARI.



8.7 Scenario 7: Channel Widening and Levees

The results show that as with the other scenarios including a levee, should the levees be constructed, and they do not fail during the event, that all of the properties may be protected at least up to a 100-year flood event.

Upstream of Goolwa Road, flood depths are increased within the creek by up to 400mm. This is an improvement to peak water levels compared to Scenario 3 and Scenario 6 of 350mm and 150mm respectively.

Downstream of Goolwa Road, flood depths are decreased by up to 180mm. This is an improvement of approximately 500mm and 300mm compared to Scenario 3 and Scenario 6 respectively. Given that the levees are in place, the channel widening appears to have a much greater impact on peak water levels compared to the clearing of creek vegetation. This results in a reduction of the required levee heights by 400mm compared with Scenario 3.

The Goolwa Road culverts appear to still remain a limiting factor in the peak water levels upstream of Goolwa Road.

Table 8-3 shows the required heights for each levee section for Scenario 7. The required height varies along the length of each levee depending on the elevation of the existing terrain where the levee is to be situated. It should be noted that levees are typically constructed with an additional freeboard allowance (e.g. 300mm) to account for local variations in the water surface profile e.g. as a result of waves.

Table 8-3: Required levee heights for Scenario 7

Levee location	Approximate height required ¹ (m)	
	Minimum	Maximum
A	0.2	0.9
B	0.1	1.0
C	Not required	
D	Not required	
E	0.4	0.4

¹ The required height for each levee varies along its length according to the variation in water surface elevation down the creek. This value does not include any freeboard allowance.

Due to the reduction in water surface elevation due to the channel widening works, levee C and levee D (refer Figure 7-3) are not required as flows are largely contained within the existing creek banks. Some water does break out of the creek on the western side close to the railway embankment, however this is not considered to be an issue as it is not likely to impact on any actual buildings/caravan sites within the caravan park. Consequently some minor flooding is still shown in this location in the flood maps.

Within the creek area bounded by the levees, the flood hazard categories are generally increased to a 3 or 4, which represents a significant safety risk should someone enter this area.

This option results in a new flood standard of the overall system of a 100-year ARI.



8.8 Scenario 8: Creek Clearing and Culvert Upgrade

The results show that reducing the vegetation within the creek as well as upgrading the culverts under Goolwa Road results in significant flooding improvements. Flood depths are decreased by up to 220mm within the creek, and by 170mm in and around the private properties upstream of Goolwa Road. The hazard category in this location is also generally reduced by one.

Flow depths over Goolwa Road on the western side are reduced by up to 140mm, and overtopping of Goolwa Road is entirely eliminated on the eastern side.

As flows over Goolwa Road on the eastern side are eliminated, flood depths at the private properties on the eastern side downstream of Goolwa Road are reduced by up to 450mm, and the flood hazard category is reduced to 1 which represents a major improvement to safety/property damage risks.

Depths within the Middleton Caravan Park are reduced by approximately 220mm, and the hazard category is generally reduced by one.

This option results in a new flood standard of the overall system of a 50-year ARI.

8.9 Scenario 9: Channel Widening and Culvert Upgrade

The results show that widening the creek (including reducing the amount of vegetation within the creek) and upgrading the culverts under Goolwa Road results in significant flooding improvements. Flood depths are decreased within the creek by up to 300mm, and by up to 210mm in and around the private properties upstream of Goolwa Road. The hazard category in this location is also generally reduced by one.

Flow depths over Goolwa Road on the western side are reduced by up to 190mm, and overtopping of Goolwa Road is entirely eliminated on the eastern side.

As flows over Goolwa Road on the eastern side are eliminated, flood depths at the private properties on the eastern side of the creek downstream of Goolwa Road are reduced by up to 570mm. Flooding is almost totally prevented in this area.

Depths within the Middleton Caravan Park are reduced by 450mm, with the hazard category within the park reduced to 1.

It should also be noted that the upgrade to the culvert, particularly by making the culvert single-span, would significantly reduce the probability of major culvert blockage by a large tree or branches as there would be no central wall for debris to be caught against.

This option results in a new flood standard of the overall system of a 60-year ARI.

8.10 Scenario 10: Creek Clearing, Levees and Culvert Upgrade

The results show that should the levees be constructed, and they do not fail during the event, that all of the properties may be protected at least up to a 100-year flood event.

Flood depths are reduced by up to 80mm and 120mm upstream and downstream of Goolwa Road respectively. This reduction is a result of the improved flow conveyance of the creek due to the channel widening (and the associated reduction of vegetation) and the upgrade to the culverts under Goolwa Road. It should be noted that the maximum water surface level is less than that for Scenario 3,



Scenario 6 and Scenario 7 due to the improved flow conveyance of the creek due to the creek clearing and the culvert upgrade.

Table 8-4 shows the required heights for each levee section for Scenario 10. The required height varies along the length of each levee depending on the elevation of the existing terrain where the levee is to be situated. It should be noted that levees are typically constructed with an additional freeboard allowance (e.g. 300mm) to account for local variations in the water surface profile e.g. as a result of waves.

Table 8-4: Required levee heights for Scenario 10

Levee location	Approximate height required ¹ (m)	
	Minimum	Maximum
A	0.1	0.5
B	0.0	0.5
C	0.2	0.2
D	0.1	0.5
E	0.4	0.4

¹ The required height for each levee varies along its length according to the variation in water surface elevation down the creek. This value does not include any freeboard allowance.

This option results in a new flood standard of the overall system of a 100-year ARI.



9 Mitigation Works Concept Cost Estimate

Initial costs estimates were prepared for each of the scenarios. These were based largely from information provided with Rawlinsons (2023), supplemented by estimates from similar previous Tonkin work.

It should be noted that these are high-level cost estimates for the purposes of comparison between options and should not be treated as final estimates for construction. They are only based on conceptual designs. Each of the options were developed primarily to assess their hydraulic feasibility and have not been optimised for cost.

The actual true costs incurred during detailed design and construction may vary considerably from these estimates due to possible optimisations (e.g. refinement of dam size and outlets), changes to material and labour costs and other unforeseeable complications.

Due to the challenging site conditions, particularly the difficulty in gaining access to undertake construction within or adjacent to private land, the costs estimated may not have included adequate allowance for these construction challenges.

A detailed breakdown of the costs associated with each scenario is included as Appendix C.

Future work is likely to involve the formal assessment of annual average damages (AAD), which would be used in conjunction with these cost estimates to calculate the cost/benefit ratio of the flood mitigation options. This would be used to further inform Council in making their decision and in seeking funding.

9.1 Cost Estimates Assumptions

9.1.1 General assumptions

When estimating the cost of infrastructure, an allowance is made for design fees and preliminaries in addition to the cost of materials and labour. In the concept stage it is assumed that the cost of these other items will scale with the overall cost of the infrastructure. In addition, a contingency allowance (as a percentage of all other costs) has been included to account for the high degree of uncertainty in estimating costs at the concept stage.

No allowance has been made for land acquisition due to the large degree of uncertainty in estimating land costs. However, this is likely to be required as part of a number of the proposed options.

No allowance has been made for service relocation costs.

The adopted factors applied to each of the costs are adopted as follows:

- Preliminaries – 10% of the cost of labour and materials
- Design fees – 5% of the cost of labour and materials
- Contingency – 30% of the cost of all other items

9.1.2 Clearing of creek vegetation

As stated previously, in order to maintain the benefit of creek clearing it would need to be maintained at regular intervals. To capture this recurring cost in the estimate, a net present value (NPV) analysis of costs was undertaken assuming the following:

- The initial clearing was estimated to cost \$50,000.
- Ongoing periodic creek maintenance/clearing was estimated to have a base cost of \$15,000.
- Ongoing periodic creek clearing would be required every 5 years.



- An inflation rate of 2.5% was adopted based on the current (as of December 2023) Reserve Bank of Australia target inflation rate.
- A discount rate of 5% was adopted based on guidelines developed by the Department of Treasury and Finance.
- The NPV analysis of costs was undertaken based on a 30-year evaluation period.

9.1.3 Channel widening

As the channel widening scenarios include a reduction in creek vegetation, the ongoing cost for creek maintenance is included in these estimates as per the creek clearing scenarios. No other specific assumptions, other than the general assumptions previously mentioned, were made to estimate the costs for channel widening.

9.1.4 Creek levee banks

Each scenario that included the construction of levee banks required different heights and extents of the levees. This variation has been included in the cost estimates for these scenarios.

9.1.5 Flood detention

The cost estimate for the dam includes only the labour and materials cost (including earth fill, outlet pipes and spillway) and preliminaries. The price of land acquisition has not been incorporated into the overall cost calculation.

However, based on an internet search for prices per hectare of rural land in the Fleurieu Peninsula, it is estimated that land acquisition costs may be in the order of \$2M. The actual price of the land required to construct the dam would need to be negotiated with relevant landowners and may vary considerably from this value.

This price is based on a maximum surface area of the dam of 9 ha when full. It should be noted that the roughly 9 ha may not necessarily need to be purchased outright. Most of the time (outside of large and rare events) the land should still be usable for some purposes e.g. cattle, so it may be possible to acquire an easement within the property for the storage area of the dam at significantly reduced cost.

9.1.6 Early warning system

A high-level estimate of \$100k was made for the early warning system.

9.1.7 Upgrade of Goolwa Road culverts

No specific assumptions other than those previously mentioned were made to estimate the costs for upgrading the Goolwa Road culverts. It is noted however that Goolwa Road is a Department of Infrastructure and Transport (DIT) asset. DIT may be willing to provide funding towards these works.



9.2 Scenario Cost Summary

Table 9-1 below shows the concept cost estimates for each of the scenarios. Costs incurred as a result of ongoing maintenance are included in the total cost, so the upfront cost only is shown in brackets.

It is noted that potential land acquisition costs have not been included in these figures.

Table 9-1: Summary of cost estimates (upfront costs in brackets)

Scenario	Mitigation Options	Total Cost Estimate
Scenario 0	Existing infrastructure (no mitigation)	-
Scenario 1	Creek clearing	\$158,000 (\$72,000)
Scenario 2	Channel widening	\$449,000 (\$359,000)
Scenario 3	Levees	\$239,000 (\$239,000)
Scenario 4	Flood detention	\$5,600,000 (\$5,600,000)
Scenario 5	Goolwa Road culvert upgrade	\$1,370,000 (\$1,370,000)
Scenario 6	Creek clearing, levees	\$291,000 (\$201,000)
Scenario 7	Channel widening, levees	\$542,000 (\$452,000)
Scenario 8	Creek clearing, Goolwa Road culvert upgrade	\$1,530,000 (\$1,440,000)
Scenario 9	Channel widening, Goolwa Road culvert upgrade	\$1,700,000 (\$1,610,000)
Scenario 10	Creek clearing, levees, Goolwa Road culvert upgrade	\$1,610,000 (\$1,520,00)
Scenario 11	Early warning system	\$100,000 (\$100,000)