

ALBURY CITY COUNCIL

EIGHT MILE CREEK FLOOD STUDY REVIEW

MAY 2026

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FOREWORD

The NSW State Government’s Flood Prone Land Policy is directed at providing solutions to existing flooding problems in developed areas and to ensuring that new development is compatible with the flood hazard and does not create additional flooding problems in other areas.

Under the Policy, the management of flood liable land remains the responsibility of local government. The State subsidises flood mitigation works to alleviate existing problems and provides specialist technical advice to assist councils in the discharge of their floodplain management responsibilities.

The Policy provides for technical and financial support by the Government through the flood risk management process shown below.



The *Eight Mile Creek Flood Study Review* is funded by the Australian Government under the Housing Support Program and has been prepared to define flood behaviour under current catchment and near-term (Year 2030) climatic conditions, noting that Albury City Council has previously completed flood and flood risk management studies for the area. Technical assistance was also provided by the NSW Government, via the NSW Department of Climate Change, Energy, the Environment and Water.

TABLE OF CONTENTS

	Page No.
SUMMARY.....	S1
1 INTRODUCTION.....	1
1.1 Study Background	1
1.2 Community Consultation and Available Data.....	1
1.3 Layout of Report.....	2
2 BACKGROUND INFORMATION	4
2.1 Catchment Description	4
2.2 Flood History and Analysis of Historic Rainfall	6
2.2.1. General	6
2.2.2. 5-6 January 2022 Storm Event	7
2.2.3. 14-15 January 2022 Storm Event.....	7
3 HYDROLOGIC MODEL DEVELOPMENT AND CALIBRATION	19
3.1 Hydrologic Modelling Approach	19
3.2 Hydrologic Model Layout	19
3.3 Hydrologic Model Testing	20
3.3.1. General	20
3.3.2. Application of Historic Rainfall to the Hydrologic Model.....	20
3.3.3. Hydrologic Model Parameters	21
3.3.4. Results of Model Testing	21
4 HYDRAULIC MODEL DEVELOPMENT AND CALIBRATION	22
4.1 General	22
4.2 The TUFLOW Modelling Approach	22
4.3 TUFLOW Model Setup.....	22
4.3.1. Model Structure	22
4.3.2. Two-dimensional Model Domain	23
4.3.3. One-dimensional Model Elements.....	24
4.3.4. Model Parameters	24
4.4 Model Boundary Conditions	25
4.5 Results of Model Calibration Process	26
5 DERIVATION OF DESIGN FLOOD HYDROGRAPHS	34
5.1 Design Storms.....	34
5.1.1. Rainfall Intensity	34
5.1.2. Areal Reduction Factors	34
5.1.3. Temporal Patterns	34
5.1.4. Probable Maximum Precipitation.....	35
5.2 Design Rainfall Losses	35
5.3 Derivation of Design Discharges	36

Continued Over

TABLE OF CONTENTS (Cont'd)

	Page No.
6 HYDRAULIC MODELLING OF DESIGN FLOOD EVENTS	37
6.1 Hydraulic Model Structure.....	37
6.2 Critical Duration and Temporal Patterns	37
6.3 Presentation and Discussion of Results	38
6.3.1. Accuracy of Hydraulic Modelling	38
6.3.2. Design Flood Extents, Depths and Elevations.....	39
6.3.3. Description of Flood Behaviour	39
6.3.4. Comparison with Previous Studies.....	46
6.4 Economic Impacts of Flooding	49
6.5 Flood Hazard and Flood Function Zones	50
6.5.1. Flood Hazard Vulnerability Classification	50
6.5.2. Flood Function.....	51
6.6 Flood Emergency Response Classification of Communities	52
6.7 Sensitivity Analyses.....	54
6.7.1. General	54
6.7.2. Sensitivity of Flood Behaviour to an Increase in Hydraulic Roughness ...	55
6.7.3. Sensitivity of Flood Behaviour to a Partial Blockage of Hydraulic Structures	55
6.7.4. Sensitivity of Coincident Flooding in Murray River.....	55
6.8 Climate Change Sensitivity Analysis	56
6.8.1. General	56
6.8.2. Sensitivity to Increased Rainfall Intensities	56
6.9 Potential Impacts of Future Development.....	57
6.10 Selection of Interim Flood Planning Levels	58
7 REFERENCES	59
8 FLOOD-RELATED TERMINOLOGY	60

APPENDICES

- A. Community Newsletter and Questionnaire
- B. Details of Available Data and Community Consultation
- C. Photographs Showing Observed Flood Behaviour in Parts of the Study Area
- D. Design Input Data from ARR Data Hub
- E. ARR 2019 Design Blockage Assessment at Drainage Structures
- F. Flood Data for Individual Road and Rail Crossings
- G. Design Peak Flows
- H. Flood Damages

LIST OF MAIN REPORT FIGURES (BOUND IN VOLUME 2)

- 1.1 Location Plan

- 2.1 Catchment Plan
- 2.2 Existing Stormwater Drainage System at Eight Mile Creek (12 Sheets)
- 2.3 Intensity-Frequency-Duration Curves - Historic Storm Events (3 Sheets)
- 2.4 Cumulative Rainfall – Historic Storm Events
- 2.5 Isohyetal Map – 5-6 January 2022 Storm Event
- 2.6 Isohyetal Map – 14-15 January 2022 Storm Event

- 3.1 Eight Mile and Sandy Creek DRAINS Model Layout (3 Sheets)

- 4.1 Eight Mile and Sandy Creek TUFLOW Model Layout (3 Sheets)
- 4.2 Eight Mile and Sandy Creek TUFLOW Hydraulic Roughness Values (3 Sheets).
- 4.3 TUFLOW Schematisation of Floodplain
- 4.4 Eight Mile and Sandy Creek TUFLOW Model Results – 5-6 January 2022 Storm Event (12 Sheets)
- 4.5 Eight Mile and Sandy Creek TUFLOW Model Results – 14-15 January 2022 Storm Event (12 Sheets)
- 4.6 Water Surface Profiles – Historic Storm Events (3 Sheets)

- 5.1 Adjusted Probability Neutral Burst Initial Loss Values

- 6.1 Eight Mile and Sandy Creek TUFLOW Model Results – 20% AEP (12 Sheets)
- 6.2 Eight Mile and Sandy Creek TUFLOW Model Results – 10% AEP (12 Sheets)
- 6.3 Eight Mile and Sandy Creek TUFLOW Model Results – 5% AEP (12 Sheets)
- 6.4 Eight Mile and Sandy Creek TUFLOW Model Results – 2% AEP (12 Sheets)
- 6.5 Eight Mile and Sandy Creek TUFLOW Model Results – 1% AEP (12 Sheets)
- 6.6 Eight Mile and Sandy Creek TUFLOW Model Results – 0.5% AEP (12 Sheets)
- 6.7 Eight Mile and Sandy Creek TUFLOW Model Results – 0.2% AEP (12 Sheets)
- 6.8 Eight Mile and Sandy Creek TUFLOW Model Results – PMF (12 Sheets)
- 6.9 Capacity of Existing Stormwater Drainage System (12 Sheets)
- 6.10 Water Surface Profiles – Design Storm Events (3 Sheets)
- 6.11 Design Stage Hydrographs (8 Sheets)
- 6.12 Flood Hazard Vulnerability Classification – 5% AEP (12 Sheets)
- 6.13 Flood Hazard Vulnerability Classification – 1% AEP (12 Sheets)
- 6.14 Flood Hazard Vulnerability Classification – PMF (12 Sheets)
- 6.15 Flood Function – 5% AEP (12 Sheets)
- 6.16 Flood Function – 1% AEP (12 Sheets)
- 6.17 Flood Function – PMF (12 Sheets)

Cont'd Over

LIST OF MAIN REPORT FIGURES (Cont'd)
(BOUND IN VOLUME 2)

- 6.18 Flood Emergency Response Classification – 5% AEP (3 Sheets)
- 6.19 Flood Emergency Response Classification – 1% AEP (3 Sheets)
- 6.20 Flood Emergency Response Classification – PMF (3 Sheets)
- 6.21 Sensitivity of Flood Behaviour to 20% Increase in Hydraulic Roughness Values – 1% AEP (3 Sheets)
- 6.22 Sensitivity of Flood Behaviour to Partial Blockage of Hydraulic Structures – 1% AEP (3 Sheets)
- 6.23 Sensitivity of Flood Behaviour to Coincident 10% AEP Murray River Flood – 1% AEP (3 Sheets)
- 6.24 Sensitivity of Flood Behaviour to 10% Increase in Rainfall Intensity – 1% AEP (3 Sheets)
- 6.25 Sensitivity of Flood Behaviour to 30% Increase in Rainfall Intensity – 1% AEP (3 Sheets)
- 6.26 Impact of Increased Rainfall Intensities on Extent of Flooding – 1% AEP (3 Sheets)
- 6.27 Potential Impact of Future Development on Flood Behaviour – 20% AEP (3 Sheets)
- 6.28 Potential Impact of Future Development on Flood Behaviour – 5% AEP (3 Sheets)
- 6.29 Potential Impact of Future Development on Flood Behaviour – 1% AEP (3 Sheets)
- 6.30 Interim Flood Planning Area – Main Stream Flooding and Major Overland Flow Affected Areas (12 Sheets)

NOTE ON FLOOD FREQUENCY

The frequency of floods is generally referred to in terms of their Annual Exceedance Probability (**AEP**) or Average Recurrence Interval (**ARI**). For example, for a flood magnitude having 5% AEP, there is a 5% probability that there will be floods of greater magnitude each year. As another example, for a flood having a 5 year ARI, there will be floods of equal or greater magnitude once in 5 years on average. The approximate correspondence between these two systems is:

Annual Exceedance Probability (AEP) (%)	Average Recurrence Interval (ARI) (years)
0.2	500
0.5	200
1	100
2	50
5	20
10	10
20	5

The report also refers to the Probable Maximum Flood (**PMF**). This flood occurs as a result of the Probable Maximum Precipitation (**PMP**). The PMP is the result of the optimum combination of the available moisture in the atmosphere and the efficiency of the storm mechanism as regards rainfall production. The PMP is used to estimate PMF discharges using computer models which simulates the conversion of rainfall to runoff. The PMF is defined as the limiting value of floods that could reasonably be expected to occur. It is an extremely rare flood, generally considered to have a return period greater than 1 in 10^6 years.

NOTE ON QUOTED LEVEL OF ACCURACY

Peak flood levels have on occasion been quoted to more than one decimal place in the report to identify minor differences in values. For example, to demonstrate minor differences between peak heights reached by both historic and design floods, as well as minor differences in peak flood levels which will result from, for example, a partial blockage of hydraulic structures. It is not intended to infer a greater level of accuracy than is possible in hydrologic and hydraulic modelling.

ABBREVIATIONS

AEP	Annual Exceedance Probability (%)
AHD	Australian Height Datum
AMC	Antecedent Moisture Condition
ARF	Areal Reduction Factor
ARI	Average Recurrence Interval (years)
ARR	Australian Rainfall and Runoff
AWS	All Weather Station
BoM	Bureau of Meteorology
Council	Albury City Council
DCCEEW	Department of Climate Change, Energy, the Environment and Water
DEM	Digital Elevation Model
FRMM	Flood Risk Management Manual (NSW Government, 2023)
IFPL	Interim Flood Planning Level
IFPA	Interim Flood Planning Area
FRMS&P	Flood Risk Management Study and Plan
GSDM	Generalised Short Duration Method
GS	Gauging Station
IFD	Intensity-Frequency-Duration
LiDAR	Light Detecting and Ranging (type of aerial based survey)
NSW SES	New South Wales State Emergency Service
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
TUFLOW	A true two-dimensional hydrodynamic computer model which has been used to define flooding patterns as part of the present study.

Chapter 8 of the report contains definitions of flood-related terms used in the study.

SUMMARY

S.1 Study Objective

The objective of the study was to define the nature of the following two types of flooding that are experienced in the catchment of Eight Mile Creek and the adjacent Sandy Creek catchment (**study catchments**) for flood frequencies ranging between 20 and 0.2 per cent Annual Exceedance Probability (**AEP**), together with the Probable Maximum Flood (**PMF**):

- **Main Stream Flooding**, which occurs when floodwater surcharges the inbank area of Eight Mile Creek and Sandy Creek, and their tributaries. Main Stream Flooding is typically characterised by relatively deep and fast flowing floodwater but can include shallower and slower moving floodwater on overbank areas.
- **Major Overland Flow**, which is experienced during periods of heavy rain and is generally characterised by local runoff that is conveyed overland in an uncontrolled manner toward the abovementioned watercourses.

The findings of the study, along with the hydrologic and hydraulic models that have been developed as part of the study, will be used as the basis for preparing flood impact and risk assessments for future development that is proposed in the study catchments, as well as the future review and update of the *Albury Flood Risk Management Study and Plan (Albury FRMS&P)*.

S.2 Study Area

Figure 1.1 is a location plan, while **Figure 2.1** shows the extent of the Eight Mile Creek catchment and its three major tributaries, those being Nine Mile Creek, Seven Mile Creek and Woolshed Creek. **Figure 2.1** also shows the extent of the Sandy Creek catchment which lies immediately to the east of the Eight Mile Creek catchment, while **Figure 2.2** (12 sheets) shows the layout of the existing stormwater drainage system in the study catchments.

S.3 Study Method

The flood study involved the following activities:

- The forwarding of a *Community Newsletter and Questionnaire* to approximately 3,700 residents and business owners in the study area. The *Community Newsletter and Questionnaire*, a copy of which is contained in **Appendix A** of this report, introduced the study objectives and sought information on historic flood behaviour. Of the 532 respondents, about 20% noted that they had been affected by flooding. Respondents provided information on flooding that occurred on the following dates:

• March 2010	• September 2021	• October 2022
• August 2010	• November 2021	• November 2022
• August 2016	• January 2022	• June 2023
• March 2017	• August 2022	
- The collection of flood data, details of which are set out in **Appendix B** of this report, including, but not limited to:
 - Pluviographic rainfall data recorded by Bureau of Meteorology (**BoM**) and WaterNSW operated rain gauges, as well as several privately owned and operated rain gauges that are in the vicinity of the study catchments were obtained.

- Several photographs were provided by respondents to the *Community Questionnaire* showing historic flood behaviour in the study area, copies of which are contained in **Appendix C** of this report.
 - LiDAR survey data dated November 2020 and September 2024, the latter which was captured specifically for use in the present study.
 - Details of the existing stormwater drainage system that were taken from detailed survey data and previous flooding investigations, as well as Council and Transport for NSW's asset databases.
- Development of a hydrologic model of the study catchment. The RAFTS and IL-CL sub-models in the DRAINS software were used to simulate the hydrologic response of the rural and urbanised parts of the study catchment, respectively. The DRAINS-based hydrologic model was used to generate discharge hydrographs resulting from both historic and design storms.
 - Development of a hydraulic model of the study area using the TUFLOW two-dimensional modelling software. Discharge hydrographs that were generated by the DRAINS-based hydrologic model were input to the TUFLOW-based hydraulic model.
 - Tuning of the hydrologic and hydraulic models (collectively referred to herein as “the flood models”) using data that were available for the 5-6 January 2022 and 14-15 January 2022 storm events.
 - Definition of flood behaviour for design storms ranging between 20 and 0.2% AEP, as well as the PMF in accordance with the procedures set out in the 2019 edition of *Australian Rainfall and Runoff* (Geoscience Australia, 2019) (**ARR 2019**).
 - Presentation of study results as diagrams showing indicative extents and depths of inundation, flood hazard vulnerability and the categorisation of the floodplain into floodway, flood storage and flood fringe areas.
 - Assessment of the performance of existing detention basins that are located in the Eight Mile Creek catchment.
 - Assessment of the economic impacts of flooding, including the number of affected properties and an estimation of flood damages.
 - Sensitivity studies to assess the effects on model results resulting from variations in model parameters such as hydraulic roughness of the floodplain and a potential partial blockage of hydraulic structures. The effects of potential coincident flooding on the Murray River and increases in rainfall intensities associated with future climate change could have on flood behaviour were also assessed.
 - Assessment of the potential impacts of future development within the study catchments absent the provision of detention storage.
 - Spatial mapping of flood emergency response classifications based on the definitions set out in *Flood Risk Management Guideline EM01 Support for Emergency Management Planning*.
 - Derivation of appropriate Flood Planning Levels and the spatial mapping of the Flood Planning Area within the study catchments.

S.4 Analysis of Historic Rainfall

Figure 2.3, sheets 1 and 2 show a comparison between rainfall that was recorded at the following pluviographic rain gauges during several intense storms that have been experienced in the vicinity of Eight Mile Creek dating back to March 2010 and design intensity-frequency-duration curves:

- *Albury Airport AWS* (GS 72160);
- *Hume Reservoir* (GS 72023);
- *Murray River at Hume Dam* (GS 401027); and
- *Bowna Creek at Yambla* (GS 401015).

Figure 2.3, sheet 3 shows a comparison between rainfall that was recorded at four privately owned and operated rain gauges during an intense storm event that occurred on 14 January 2022 and design intensity-frequency-duration curves.

Based on a review of the available rainfall and flood related data, two separate storm events that occurred on 5-6 and 14-15 January 2022 were relied upon for model tuning purposes. **Figure 2.4** shows the cumulative rainfall that was recorded by the aforementioned rain gauges during the two storm events, while **Figures 2.5** and **2.6** show the spatial variation in the depth of rainfall which was associated with the two storm events.

An analysis of the available rainfall data found that:

- a) the most intense rain that fell prior to midnight on 5 January 2022 was confined to the south-western parts of the catchment in the vicinity of Albury Airport and approximated a 10% AEP storm event; and
- b) the most intense rain that fell prior to midnight on 14 January 2022 was confined to the southern parts of the study catchment and approximated between a 0.2% and 0.05% AEP storm event.

S.5 Flood Model Development and Tuning

Figure 3.1 (3 sheets) shows the layout of the sub-catchments which comprise the hydrologic (DRAINS) model (**Eight Mile and Sandy Creek DRAINS Model**), while **Figure 4.1** (3 sheets) shows the layout of the hydraulic (TUFLOW) model (**Eight Mile and Sandy Creek TUFLOW Model**) that were developed as part of the present study. **Figure 4.2** (3 sheets) shows the spatial distribution of hydraulic roughness values that were applied to the Eight Mile and Sandy Creek TUFLOW Model.

Figures 4.4 and **4.5** (12 sheets each) show the indicative extent and depth of inundation as defined by the Eight Mile and Sandy Creek TUFLOW Model for the 5-6 January 2022 and 14-15 January 2022 storm events, respectively, while **Tables 4.3** and **4.4** in **Chapter 4** of this report provide a comparison of observed and modelled flood behaviour for the two storm events.

S.6 Design Flood Estimation

Design storms for frequencies of 20%, 5%, 2%, 1%, 0.5% and 0.2% AEP were derived for storm durations ranging between 15 minutes and seven days. The Intensity-Frequency-Duration (IFD) dataset was downloaded from the BoM's *2016 Rainfall IFD Data System* (**BoM 2016 IFD Data**).

A recent update to ARR 2019 (**ARR 2019 Rev 4.2**) includes multiplication factors that are to be applied to the BoM 2016 IFD data to account for increases in design rainfall intensity and loss estimates associated with climate change. For the purpose of the present study, the multiplication factors for the Year 2030 and Shared Socioeconomic Pathway (**SSP**) 2-4.5 conditions have been applied to the BoM 2016 IFD data (**SSP2-4.5 Year 2030 IFD Data**).¹ The SSP2-4.5 Year 2030 IFD Data were then used as the basis for defining the nature of flooding in the study catchments for design storms with AEPs of between 20% and 0.2%, noting that the same approach was adopted for defining the nature of flooding for the PMF event.

Figures 6.1 to 6.8 (12 sheets each) show the hydraulic (TUFLOW) model results for the 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP storm events, together with the PMF. These diagrams show the indicative extent and depth of inundation in the study catchments for each design storm event.

Table F1 in **Appendix F** sets out the peak flood level and maximum depth of inundation at the locations where the existing road and rail network cross the various watercourses in the study catchments, while **Table G1** in **Appendix G** sets out design peak flows and corresponding critical storm durations at key locations throughout the study catchments. The key features of Mainstream Flooding and Major Overland Flow in different parts of the study catchments are described in **Section 6.3.3** of this report.

S.7 Comparison with Previous Studies

While two previous flood studies that have been undertaken for the Eight Mile Creek catchment (URS, 2012 and WMAwater, 2016) were based on procedures set out in the 1987 edition of *Australian Rainfall and Runoff* (Institute of Engineers Australia, 1987) (**ARR 1987**), the peak flows generated by the flood models that were developed as part of the present study are generally comparable. The reason for this is that the application of the climate change factors that are recommended in the latest addition of ARR 2019 result in the Intensity-Frequency-Duration data being comparable to the values used in the previous studies.

The exception to the above finding is in the upper reaches of the Eight Mile Creek and Woolshed Creek catchments where the peak flows derived as part of the present study are up to three and two times higher than the two earlier studies, respectively. The reasons for the differences are attributed to the lumped nature of the sub-catchments adopted in the earlier studies coupled with the adoption of a 6-hour storm which was considered to be critical for maximising peak flows in the middle and lower reaches of the catchment.

The peak flows derived as part of the present study were also found to be significantly higher than those derived in the upper reaches of Eight Mile Creek as part of the *Nexus Industrial Precinct Flooding Investigation* (WMS, 2023). The reasons for this are that the hydrologic model that was developed as part of the study was tuned to reproduce the peak flows that were derived as part of WMAwater, 2016 and the study also adopted the 6-hour duration storm for deriving peak flows in the drainage system.

The peak flows derived as part of the present study were also found to be significantly higher than those derived as part of the *Thurgoona Wirlinga Drainage Strategy* (Spiire, 2020), noting that the earlier study adopted the Rational Method set out in ARR 1987 for design peak flow estimation purposes and a tuned RORB model for sizing the various stormwater detention basins that were envisaged to be located within the Thurgoona Wirlinga Growth Area. Based on this finding, there is the potential for the earlier study to have over-estimated the size of the various stormwater detention basins.

¹ Climate change factors relating to SSP2-4.5 conditions were adopted as they represent the middle range of projected increases in design rainfall intensities.

S.8 Economic Impact of Flooding

The economic impacts of flooding in the study area were assessed in accordance with the procedures set out in *Flood Risk Management Guideline MM01 – Flood Risk Management Measures* (DPE, 2023) and the associated *NSW Flood Risk Management Tool DT01 (FRM Tool DT01)*. The intent of the assessment is to provide a basis for assessing the economic merits of potential flood mitigation schemes as part of the future review and update of the *Albury FRMS&P*.

While a total of 27 residential dwellings and one public building would be above-floor inundated in a 1% AEP flood event, resulting in total flood damages of about \$3.0 Million, the depth of above-floor inundation would not exceed 0.16 m in all cases.

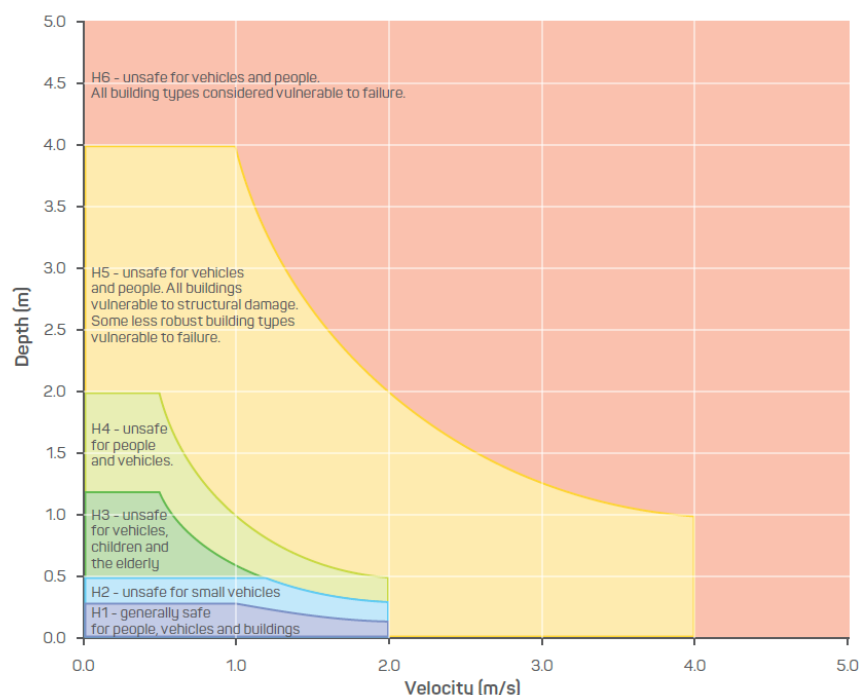
The number of buildings that would be flooded during a PMF event increases significantly, whereby there would be a total of 771 dwellings, eight commercial buildings and 11 public buildings that would be above-floor inundated, resulting in total flood damages of about \$192 Million.

The “*Net Present Value*” of damages resulting from all floods up to the 1% AEP for a discount rate of 5% and an economic life of 30 years is \$1.7 Million. This value represents the amount of capital spending that would be justified if one or more flood mitigation schemes costing up to this amount prevented flooding for all properties in the study catchments up to the 1% AEP event. While schemes costing more than this value would have a benefit/cost ratio less than 1, they may still be justified according to a multi-objective approach which considers other criteria in addition to economic feasibility.

Appendix H of this report contains further details on the economic assessment that was undertaken as part of the present study.

S.9 Flood Hazard Vulnerability Classification

Diagrams showing the flood hazard vulnerability classification of the floodplain for the 5% and 1% AEP flood events, as well as the PMF based on the relationship shown in the illustration below are shown on **Figures 6.12 to 6.14** (12 sheets each).



The study found that for floods up to 1% AEP in magnitude, areas that are classified as H5 and H6 are generally confined to the inbank area of the various watercourses which traverse the study catchments, while flooding in the urbanised parts of the study catchments is generally limited to H1 and H2 type hazard conditions.

The extent of land subject to H5 and H6 type hazard conditions in a PMF increases significantly, while the hazard category within the urbanised parts of the study catchments increases to H4 during a flood of this magnitude.

S.10 Flood Function

The classification of the flood affected areas into floodway, flood storage and flood fringe (collectively referred to as “flood function”) for the for the 5% and 1% AEP flood events, as well as the PMF are shown on **Figures 6.15 to 6.17** (12 sheets each).

While the study found that floodways are generally contained within the drainage and road reserve boundaries in the urbanised parts of the study catchments in a 1% AEP flood, they were found to be present in several existing residential estates. Flood storage areas were generally found to be confined to the major ponding areas that are located on the upstream side of the road and railway embankments, as well as in the local farm dams that have been constructed to capture surface runoff in different parts of the study catchments. The remainder of the flood affected areas were classified as flood fringe.

S.11 Flood Emergency Response Classifications

Figures 6.18, 6.19 and 6.20 show the flood emergency response classifications for the 5% and 1% AEP storm events, as well as the PMF, respectively, based on the definitions set out in *Flood Risk Management Guideline EM01 Support for Emergency Management Planning*.

Most of the study area is defined as ‘Rising Road Access’ in floods up to 1% AEP in magnitude, with isolated areas classified as ‘Low Flood Island’ and ‘High Flood Island’ in the following areas:

- Residential allotments that are located on Swindon Way in the Brooklyn Fields residential estate are located on a high flood island in a 5% AEP flood event.
- While the majority of the Brooklyn Fields residential estate is presently considered to be a high flood island in a 1% AEP flood event, there are isolated allotments that are considered to be low flood islands in an event of this magnitude. It is understood that consideration is currently being given to the provision of rising road access to the north.
- The Spring Park residential estate is a high flood island in a 1% AEP flood event.

S.12 Sensitivity Analyses

Analyses were undertaken to test the sensitivity of flood behaviour to:

- An increase in hydraulic roughness. **Figure 6.21** (3 Sheets) shows the effects a 20 per cent increase in the adopted ‘best estimate’ hydraulic roughness values would have on flood behaviour at the 1% AEP level of flooding.
- A partial blockage of major hydraulic structures by debris. **Figure 6.22** (3 Sheets) shows the effects a partial blockage of the major culvert structures would have on flood behaviour at the 1% AEP level of flooding.

- Coincident flooding on the Murray River. **Figure 6.23** (3 sheets) shows the impact that the adoption of a 10% AEP tailwater level in the Murray River coincident with a 1% AEP flood on the study catchments would have on flood behaviour.
- Increases in rainfall intensity associated with future climate change. **Figures 6.24, 6.25 and 6.26** show the effects a 10 and 30 per cent increase in design 1% AEP rainfall intensities would have on flood behaviour in the study catchments.

The sensitivity analyses identified that:

- a) A 20% increase in the adopted 'best estimate' hydraulic roughness values increases peak 1% AEP flood levels in the areas that are subject to Main Stream Flooding generally in the range 20 to 100 mm, with increases of up to 200 mm shown to occur in isolated areas, while increases in peak flood levels in areas subject to Major Overland Flow are generally in the range 10 to 50 mm.
- b) The removal of the probability neutral blockage factors on the major hydraulic structures that are located at the following locations results in an increase in the rate at which flow discharges to the downstream reach of watercourse:
 - the Main Southern Railway and Hume Highway crossings of Eight Mile Creek, Seven Mile Creek and a tributary of Eight Mile Creek; and
 - the Elizabeth Mitchell Road crossing of an unnamed watercourse to the south of Ettamogah Road.
- c) The removal of the probability neutral blockage factors increases peak flood levels along the main arm of Eight Mile Creek downstream of the Main Southern Railway and Hume Highway crossings by up to 200 mm and by up to 100 mm on the aforementioned tributaries, while it generally has a negligible effect on flood behaviour in areas subject to Major Overland Flow.
- d) The adoption of a 10% AEP flood on the Murray River coincident with a 1% AEP local catchment flood does not alter patterns of Main Stream Flooding and Major Overland Flow in the study catchments.
- e) A 10 per cent increase in 1% AEP rainfall intensities would increase peak flood levels in areas that are subject to Main Stream Flooding by between 20 and 100 mm, with increases of up to 500 mm shown to occur on the upstream side of the Main Southern Railway crossing of the various watercourses. Increases in peak flood levels of generally between 10 to 50 mm would also occur in areas subject to Major Overland Flow.
- f) A 30 per cent increase in 1% AEP rainfall intensities would increase peak flood levels in areas that are subject to Main Stream Flooding by between 50 and 200 mm, with increases of up to 700 mm shown to occur on the upstream side of the Main Southern Railway crossing of the various watercourses. Increases in peak flood levels of generally up to a maximum of 100 mm would also occur in areas subject to Major Overland Flow.
- g) The extent of land that would be inundated by floodwater should 1% AEP rainfall intensities increase by up to 30% is generally negligible, with the following exceptions:
 - in the Nexus Industrial Precinct where new flow paths arise due to increased overtopping of the existing levee which runs along its western boundary;
 - on the upstream side of the Main Southern Railway and Hume Highway, where the extent of ponding increases significantly;

- on the left (eastern) overbank of Seven Mile Creek and on the right (western) overbank Eight Mile Creek where flow from both watercourses combine; and
- on land that lies immediately to the north of Thurgoona Drive and east of Table Top Road where the extent of inundation increases significantly.

S.13 Potential Impacts of Future Development

Figures 6.27, 6.28 and 6.29 (3 sheets each) show the impact that uncontrolled development could have on flood behaviour in the study area in a 20%, 5% and 1% AEP storm event, respectively, noting that the assessment undertaken as part of the present study is of a broad-scale and strategic nature, and that more detailed site specific assessments would need to be undertaken as part of any future development.

The study found that peak flood levels could increase by up to 300 mm on the tributaries of Eight Mile Creek if uncontrolled development were allowed to occur in the catchment for the range of assessed storm events, while minor reductions in peak flood levels are shown to occur along the main arm of Eight Mile Creek due to changes in the timing of the flood wave in its tributaries. This finding reinforces the need for Council to require detention basins to be incorporated in the design of future development to ensure that post-development peak flows are no higher than pre-development conditions.

S.14 Derivation of Flood Planning Levels and Extent of the Flood Planning Area

Figure 6.30 (12 sheets) shows the extent of the Interim Flood Planning Area (**IFPA**) in the study catchments as it relates to both Main Stream Flooding and Major Overland Flow. The extent of the IFPA has been defined as follows:

- in areas subject to Main Stream Flooding, the extent of the IFPA was defined as land lying at or below the peak 1% AEP flood level plus a freeboard allowance of 0.5 m (**Main Stream Flooding IFPA**);² and
- in areas subject to Major Overland Flow and that also lie outside the extent of the Main Stream Flooding IFPA, the extent of the IFPA was defined as land inundated to a depth greater than 100 mm or within the extent of the floodway (**Major Overland Flow IFPA**).³

In advance of the adoption of any precinct specific development control plan and/or the future review and update of the *Albury FRMS&P*, it is recommended that the following freeboard allowances be adopted when setting minimum habitable floor levels of future development:

- for development that lies within the extent of the Main Stream Flooding IFPA, habitable floor levels should be set a minimum 500 mm above peak 1% AEP flood levels; and
- for development that lies within the extent of the Major Overland Flow IFPA, habitable floor levels should be set a minimum 300 mm above peak 1% AEP flood levels.

An assessment should also be undertaken by Council as part of any future Development Application to confirm that the proposed development will not form an obstruction to the passage of flow through the subject site.

² For the purpose of the present study, the Main Stream Flooding IFPA was defined along watercourses that are identified in the NSW Government's *SixMaps* "hydrolines" online database and where the peak 1% AEP flow exceeded 2 m³/s.

³ The extent of Major Overland Flow IFPA was filtered to remove pockets of flooding where the area was less than 100 m².

Figure 6.30 also shows the extent of the *Interim Outer Floodplain*, which is the area that lies between the IFPA and the extent of the PMF. Consideration will need to be given during the preparation of the future review and update of the *Albury FRMS&P* as to whether Council should adopt clause 5.22 in the *Albury Local Environmental Plan 2010*, noting that this clause relates to the need for Council to assess sensitive and hazardous type development that is proposed on land that lies between the IFPA and the PMF.

1 INTRODUCTION

1.1 Study Background

This report presents the findings of a review that was commissioned by Albury City Council (**Council**) into flooding in the Eight Mile Creek catchment, as well as in the adjacent Sandy Creek catchment. **Figure 1.1** is a location plan showing the extent of the Eight Mile Creek and Sandy Creek catchments where they drain to the Murray River near the eastern limits of Albury (**study area**).

The primary objective of the study was to review flood related studies that have been prepared for the study area such as the *Eight Mile Creek Flood Study* (URS, 2012) and the *Albury Floodplain Risk Management Study and Draft Plan* (WMAwater, 2016), and to develop a contemporary set of computer models that were then used to define flood behaviour in terms of flows, water levels and velocities for floods ranging between 20 and 0.2 per cent Annual Exceedance Probability (**AEP**), as well as for the Probable Maximum Flood (**PMF**). The study involved rainfall-runoff hydrologic modelling of the catchments to assess flows in the drainage systems of the study catchment and application of these flows to a hydraulic model to assess peak water levels and flow velocities (collectively referred to herein as 'flood modelling'). The model results were interpreted to present a detailed picture of flooding under present day conditions.

The present study focuses on the following two types of flooding which are present in different parts of the study area:

- **Main Stream Flooding**, which occurs when floodwater surcharges the inbank area of Eight Mile Creek and Sandy Creek and their tributaries. Main Stream Flooding is typically characterised by relatively deep and fast flowing floodwater but can include shallower and slower moving floodwater on overbank areas.
- **Major Overland Flow**, which is experienced during periods of heavy rain and is generally characterised by relatively shallow and slow-moving floodwater that is conveyed overland in an uncontrolled manner toward the abovementioned watercourses.

The present study forms the first and second steps in the flood risk management process for the Eight Mile Creek and Sandy Creek catchments (refer process diagram presented in the Foreword) and is a precursor to the review and update of the *Albury Flood Risk Management Study and Plan* (**Albury FRMS&P**) which will further consider measures which are aimed at reducing the existing, future and continuing flood risk in the study area.

1.2 Community Consultation and Available Data

To assist with data collection and promotion of the study to the community, a total of 3,700 *Community Newsletter* and *Questionnaires* were distributed by Council in September 2024 to residents and business owners in the study area. A copy of the *Community Newsletter* and *Questionnaire* is contained in **Appendix A** of this report. Two community information sessions were also held at the Thurgoona Country Club Resort at the commencement of the study.

A total of 532 responses were received by the closing date of submissions (a response rate of approximately 14%). Of the 532 respondents, 112 noted that they had been affected by flooding.

The following events were identified during the community consultation process:

- 17 March 2010
- August 2010 (exact day not known)
- October 2010 (exact day not known)
- August 2016 (exact day not known)
- 22 March 2017
- 22 March 2017
- 5 September 2021
- 25 November 2021
- 5-6 January 2022
- 11 January 2022
- 14 January 2022
- 15 January 2022
- 8 August 2022
- 14 October 2022
- 22 October 2022
- 1 November 2022
- 13-14 November 2022
- June 2023 (exact day not known)
- 2 August 2024

The following four events were also documented in the *Eight Mile Creek Flood Study* (URS, 2012):

- January 1974
- August 1983
- 16 March 1992
- 6 October 2005

Information on historic flooding patterns obtained from the responses assisted with “ground-truthing” the results of the flood modelling.

Appendix B contains details of the data that were available for the present study and a detailed summary of the responses to the *Community Questionnaire*, while **Appendix C** contains several photos that were provided by Council and respondents to the *Community Questionnaire* which show historic flood behaviour in parts of the study area during storms that occurred in 16 March 1992, 6 October 2005, October 2010 (day not known), 5 September 2021, 5-6 January 2022, and 14-15 January 2022.

The draft *Eight Mile Creek Flood Study Review* was publicly exhibited from 18 February to 2 April 2026. In accordance with Council’s *Community Participation Plan*, all landowners within the study area (approximately 4,300 properties) were directly notified by letter, and the exhibition period was advertised in the *Border Mail* and on Council’s website. The project webpage included access to the 1% AEP flood extent map online. Two community information sessions were held at the Thurgoona Country Club on 18 March 2026.

A total of 13 written submissions were received, none of which raised issues requiring changes to the Review methodology or findings. Formal responses to submissions were reported to Council on 18 May 2026.

1.3 Layout of Report

Chapter 2 contains background information including a brief description of the Eight Mile Creek and Sandy Creek catchments and their drainage systems, a brief history of flooding and an analysis of the available rain gauge record.

Chapter 3 deals with the hydrology of the study catchment and describes the development and calibration of the DRAINS-based hydrologic model that was used to generate discharge hydrographs for input to the hydraulic model.

Chapter 4 deals with the development and calibration of the TUFLOW hydraulic model that was used to analyse flood behaviour in the study area.

Chapter 5 deals with the derivation of design discharge hydrographs, which involved the determination of design storm rainfall depths over the catchment for a range of storm durations and conversion of the rainfalls to discharge hydrographs.

Chapter 6 details the results of the hydraulic modelling of the design floods in the study area. Results are presented as plans showing indicative extents and depths of inundation for a range of design flood events up to the PMF. This chapter also includes an assessment of flood hazard and hydraulic categorisation. It also presents the results of various sensitivity studies undertaken using the TUFLOW model, including the effects changes in hydraulic roughness, a partial blockage of the hydraulic structures, coincident flooding on the Murray River and potential increases in rainfall intensities due to future climate change would have on flood behaviour. This chapter also deals with the derivation of a preliminary set of *Flood Planning Levels* for the study area.

Chapter 7 contains a list of references, whilst **Chapter 8** contains a list of flood-related terminology that is relevant to the scope of the study.

The following appendices are included in the report:

- **Appendix A**, which contains a copy of the *Community Newsletter* and *Questionnaire* that were distributed at the commencement of the study to residents and business owners in the study area.
- **Appendix B**, which contains a list of data that were available for the present study and a summary of the responses to the *Community Questionnaire*.
- **Appendix C**, which contains photographs showing flood behaviour that was observed in parts of the study area during storms that occurred on 16 March 1992, 6 October 2005, October 2010 (day not known), 5 September 2021, 5-6 January 2022 and 14-15 January 2022.
- **Appendix D**, which contains a copy of the design input data that were extracted from the *Australian Rainfall and Runoff (ARR) Data Hub* for the study area.
- **Appendix E**, which summarises design blockage values that were assigned to each individual headwall structure in the study area.
- **Appendix F**, which contains a table containing flood data on individual road and rail crossings in the study area.
- **Appendix G**, which contains a table listing the peak flow at key locations in the study area for the full range of design storm events.
- **Appendix H**, which contains an assessment of the economic impacts of flooding to existing residential, commercial and industrial development, as well as public buildings in the study area.

Figures referred to in the main body of the report are bound separately in **Volume 2**.

2 BACKGROUND INFORMATION

2.1 Catchment Description

Figure 1.1 shows the extent of the study area which includes the Eight Mile Creek and Sandy Creek catchments (hereon collectively referred to as the “*study catchments*”) which discharge to the right (northern) bank of the Murray River a short distance downstream of Lake Hume. **Figure 1.1** shows that the study catchments cover the suburbs of Ettamogah, Table Top, Thurgoona and Wurlinga, as well as the Australian Defence Force’s (**ADF’s**) Wurlinga Defence Establishment.

Figure 2.1 shows the extent of the Eight Mile Creek catchment and its three major tributaries, those being Nine Mile Creek, Seven Mile Creek and Woolshed Creek. **Figure 2.1** also shows the extent of the Sandy Creek catchment which lies immediately to the east of the Eight Mile Creek catchment. **Figure 2.2** (12 sheets) shows the layout of the existing stormwater drainage system in the study catchments.

Eight Mile Creek has a total catchment area of about 63.5 km² where it crosses the Riverina Highway, downstream of where it discharges onto the Murray River floodplain. The headwaters of the Eight Mile Creek catchment are located approximately 11 km to the north of the Riverina Highway crossing in the vicinity of the intersection of Table Top Road and the Hume Highway. Eight Mile Creek generally flows in a southerly direction through the study area to a location approximately 2.3 km downstream (south) of the Riverina Highway where it continues in a westerly direction before discharging to the Murray River to the south of Albury Airport.

While the inbank area of Eight Mile Creek generally comprises an incised 20 m wide by 2 m deep channel which has a grade of about 0.2% where it runs through the study area, catchment slopes of up to 25% are present in its headwaters upstream of Gerogery Road. The grade of the creek reduces to about 0.12% along its 8.5 km reach downstream of the Riverina Highway where it runs in a westerly direction perpendicular to the general fall of the land. Floodwater has historically surcharged the banks of the creek at this location during relatively minor storm events where it flows in a southerly direction through rural land that is located between the highway and the Murray River.

Nine Mile Creek generally runs in a southerly direction and has a total catchment area of about 16 km² at the point at which it discharges to Eight Mile Creek immediately downstream of Table Top Road. The Ettamogah Forest Dam is located in the Nine Mile Creek catchment approximately 5 km to the north of its confluence with Eight Mile Creek. The dam was constructed in 1994 to provide storage capacity for excess process water for the adjacent paper mill during the winter months and to irrigate the adjacent pine plantation in the summer months.

While **Figure 2.1** shows that the Ettamogah Forest Dam is located on Nine Mile Creek based on the information contained on the NSW Government’s *SixMaps* online database, **Figure 2.2**, sheet 4 shows that diversion drains have been constructed to divert runoff from the headwaters of the catchment around the eastern and western sides of the dam (herein referred as the “*Eastern and Western Diversion Drains*”). While low flows in the creek are conveyed along the Western Diversion Drain where they do not appear to discharge directly into the dam, it was not possible to determine as part of the present study if there are operational structures along the two drains which can be used to divert surface runoff into the dam.

While the inbank area of Nine Mile Creek is generally indistinct upstream of the Ettamogah Forest Dam, it generally comprises an incised 10 m wide by 1.5 m deep channel which has a grade of about 0.4% downstream of the dam embankment.

Seven Mile Creek generally runs in an easterly direction and has a total catchment area of about 4.7 km² at the point at which it discharges to Eight Mile Creek in the vicinity of the intersection of Table Top Road and Brooklyn Drive. The inbank area of Seven Mile Creek generally comprises an incised 9 m wide by 1.5 m deep channel which has a grade of about 0.7% where it runs through the study area.

Woolshed Creek generally runs in a westerly direction and has a total catchment area of about 11.4 km² at the point at which it discharges to Eight Mile Creek approximately 0.4 km to the north of Thurgoona Drive. The inbank area of Woolshed Creek generally comprises an incised 20 m wide by 1.8 m deep channel which has a grade of about 0.5%.

Sandy Creek runs in a southerly direction through the western parts of Wirlinga and has a total catchment area of about 7 km² where it crosses the Riverina Highway, after which it discharges to a billabong which is located on the right (northern) overbank of the Murray River. While the watercourse is not officially known as Sandy Creek on the NSW Governments *SixMaps* online database, it is signposted as Sandy Creek where it crosses the Riverina Highway. The inbank area of Sandy Creek increases from a 5 m wide by 1 m deep channel in the upper reaches of the catchment to a 10 m wide by 2.5 m deep channel in the vicinity of the Riverina Highway. Sandy Creek generally has a grade of about 0.8%.

Figure 2.2 shows that the Main Southern Railway and Hume Highway run in a north-south direction through the study area. Both the railway and highway generally comprise embankments that are elevated several metres above the floodplain where they cross Eight Mile Creek and its tributaries. **Figure 2.2** also shows that the Riverina Highway runs in an east-west direction near the southern limits of the study area and comprises an embankment that is elevated several metres above the adjacent floodplain.

Figure 2.2 shows that the northern and eastern portions of the study area generally comprise rural type farm land and forests, with residential type development generally located in its south-western quadrant. There is a pocket of industrial type development that is located on the western side of the Hume Highway to the north-west of its intersection with Wagga Road which is known locally as the Nexus Industrial Precinct.

The study catchments have been subject to relatively rapid development over the past four decades as the city of Albury has expanded to its east. **Figure 2.2** shows the approximate extent of the following 26 existing housing type developments that are located in the study area:

- | | | |
|--------------------|----------------------|--------------------|
| ➤ Beaumont Park | ➤ Huntingdale | ➤ St Johns Green |
| ➤ Brooklyn Fields | ➤ Iluka Views | ➤ St Johns Hill |
| ➤ Chisholm Park | ➤ Kensington Gardens | ➤ The Elms |
| ➤ Corriedale Court | ➤ Kerrford Park | ➤ Thurgoona Court |
| ➤ Corrys Wood | ➤ Mitchell Park | ➤ Tweed Court |
| ➤ Ettamogah Rise | ➤ Somerset Rise | ➤ University Close |
| ➤ Fairway Gardens | ➤ Spring Park | ➤ Woolshed Estate |
| ➤ Hereford Court | ➤ Springview Estate | ➤ Yarrabee Park |
| ➤ Hillford Estate | ➤ St Hilaire | |

The eastern and northern portions of the study catchments are zoned for future development, with development expected to continue to extend into these areas in the near future.

Figure 2.2 shows the layout of the existing stormwater drainage system in the study catchments which generally comprises connected piped drainage elements that in many cases discharge to Eight Mile Creek and its tributaries via a network of detention basins. **Figure 2.2** shows the plan location of the detention basins that have been constructed to control runoff from the residential housing type developments, while **Table 2.1** at the end of this chapter sets out the key features of each.⁴

2.2 Flood History and Analysis of Historic Rainfall

2.2.1. General

Respondents to the *Community Questionnaire* identified several storm events that have been experienced in the study area, the dates of which are given in **Section 1.2** of this report. Several respondents also provided photographic evidence (refer **Appendix C**), as well as descriptions of the patterns of overland flow in the vicinity of their properties.

Figure 1.1 shows the location of the nearby two Bureau of Meteorology (**BoM**) and two WaterNSW operated rain gauges that are located in the vicinity of the study area, noting that there are no official rain gauges that are located within the study catchments. **Figure 1.1** also shows the plan location of four privately operated rain gauges that are located in the vicinity of Thurgoona Park.

Table 2.2 at the end of this chapter shows a comparison of the 24-hour rainfall totals at the rain gauges that are located in the proximity of the study catchments and were in operation during the storm events that were identified during the community consultation and data collection process.

Table 2.3 also at the end of this chapter gives the approximate AEP of the recorded rainfall for durations ranging between 0.5 and 9 hours for the pluviographic rain gauges that are located in the vicinity of study catchments and were operational during the storm events identified during the community consultation and data collection process, while **Figure 2.3** (2 sheets) shows design versus historic intensity-frequency-duration (**IFD**) curves for the pluviographic rain gauges.⁵

Table 2.2 and **Figure 2.3** show that the storms identified by the respondents to the *Community Questionnaire* varied in intensity. **Table 2.2** shows that most of the storms identified were equivalent to design storm events more frequent than 50% (1 in 2) AEP, with the following exceptions:

- The storm that occurred on the raindays⁶ of 4-5 January 1974 was equivalent to about a 2-5% AEP design storm event at Hume Reservoir.
- The storm that occurred on the raindays of 9-10 January 1974 was equivalent to about a 10-20% AEP design storm event at Hume Reservoir.

⁴ Note that several of the basins have multiple compartments which are designed to provide habitat for the endangered Sloane's Froglet which is found in parts of the study area.

⁵ Approximate AEP of each storm event is based on near term climatic conditions (i.e. BoM 2016 Intensity-Frequency-Duration curves that have been adjusted based on Year 2030 and Shared Socioeconomic Pathway 2-4.5 conditions). Refer **Chapter 5** of this report for further details.

⁶ A rainday is the 24 hour period prior to 09:00 hours on a given day/date.

- The storm that occurred on the raindays of 25-27 August 1983 was equivalent to about a 10-20% AEP design storm event at Hume Reservoir.
- The storm that occurred on the raindays of 6-8 March 2010 was equivalent to about a 10-20% AEP design storm event at Bowna Creek.
- The storms that occurred on the raindays of 21 March 2017 and 5 September 2021 were equivalent to about a 20-50% AEP design storm event at Bowna Creek.
- The storm that occurred on the rainday of 25 November 2021 was equivalent to about a 20-50% AEP design storm event at Bowna Creek.
- The storm that occurred on the rainday of 5-6 January 2022 was equivalent to about a 10% AEP design storm at Albury Airport.
- The storm that occurred on the rainday of 14-15 January 2022 was equivalent to about a 0.2% AEP design storm at Albury Airport and the Hume Dam, and an event that is rarer than 0.05% AEP in the vicinity of Thurgoona Park (based on the privately owned rainfall data obtained from the Weather Underground online database).
- The storm that occurred on the rainday of 1 November 2022 was equivalent to about a 20-50% AEP design storm at Albury Airport.
- The storm that occurred on the raindays of 13-14 November 2022 was equivalent to about a 10-20% and a 20-50% AEP design storm at Albury Airport and Hume Dam, respectively.

Based on the limited available historic flood data, the storm events that occurred on 5-6 January 2022 and 14-15 January 2022 were selected for use in the calibration/validation of the hydrologic and hydraulic models that were developed as part of the present study. **Figure 2.4** shows the cumulative rainfall that was recorded at the nearby rain gauges for these two storm events, while **Figures 2.5** and **2.6** show the assessed spatial distribution of the rain that fell across the study catchment for the rain days of 6 January 2022 and 15 January 2022, respectively.

2.2.2. 5-6 January 2022 Storm Event

Figure 2.4 shows that 59.4 mm of rain fell between 18:00 hours on 5 January 2022 and 01:00 hours on 6 January 2022 at the Albury Airport AWS rain gauge, which **Table 2.2** shows was equivalent to a design storm with an AEP of about 10%. **Figure 2.4** shows that 21 mm and 33.4 mm of rain fell at the Hume Dam and Yambla rain gauges over the same period, which was equivalent to a design storm event more frequent than 50% AEP. **Figure 2.5** shows that based on the above, the 5-6 January 2022 storm was confined to the south-western parts of the catchment that are located in the vicinity of Albury Airport.

Fifty-seven respondents identified that they had experienced flooding as a result of the 5-6 January 2022 storm event, while two photos that were provided by respondents to the *Community Questionnaire* showed that flooding on the overbank area of Eight Mile Creek occurred downstream of the Riverina Highway on the morning of 6 January 2022 (refer **Plates C5.1** to **C5.2** in **Appendix C**).

2.2.3. 14-15 January 2022 Storm Event

Preliminary runs of the hydrologic and hydraulic model that were developed as part of the present study found that it was not possible to reproduce the observed flood behaviour in the vicinity of Thurgoona Park using the official rainfall records associated with the BoM and WaterNSW operated rain gauges. A review of the privately owned rain gauge data on the *Weather Underground* website found that there were four rain gauges that were operational in the vicinity of Thurgoona Park during the 15 January 2022 storm event.

Figure 2.4 shows that 68.8 mm and 69.6 mm of rain fell between 21:30 hours and 23:30 hours on 14 January 2022 at the Albury Airport AWS and Hume Dam rain gauges, respectively, which **Table 2.2** shows was equivalent to a design storm with an AEP of about 0.2%. **Figure 2.4** shows that between 61.1 and 100.1 mm of rain fell during the same period at the four privately owned Thurgoona Park rain gauges,⁷ which were equivalent to a design storm with an AEPs of about between 0.2% and an event that is rarer than 0.05%. **Figure 2.4** shows that 11 mm was recorded at the Yambla rain gauges over the same period, which was equivalent to a design storm event more frequent than 50% AEP.

Figure 2.6 shows that based on the above, the 14-15 January 2022 storm was generally more intense in the southern parts of the study catchments, with the most intense rainfall recorded in the vicinity of Thurgoona Park. The spatial distribution of rainfall corresponds with the location of the 83 respondents to the *Community Questionnaire* who identified that they had experienced flooding as a result of the storm.

Of those that had been impacted, a total of 14 indicated that their dwelling was inundated above-floor level in the 14-15 January 2022 storm event, six of which were located in Thurgoona Park, three in Fairway Gardens and one each in St Johns Hill, St Johns Green, Tweed Court, Kerrford Park and The Elms developments.

It was also found that a significant number of respondents to the *Community Questionnaire* identified that they were inundated from localised flows from adjacent properties and/or failure of their internal/inter-allotment piped drainage system. It is noted that the assessment of these localised drainage issues lies outside the scope of the present study.

⁷ The owner of rain gauge IALBUR88 advised that a blackout occurred at around 23:45 hours on 14 January 2022 which caused it to cease recording rainfall depths. The owner also advised that the recorded values underestimated the total recorded rainfall as rain continued to fall until around 01:30 hours on 15 January 2022. It is not known if the three other rain gauges were affected by the blackout.

TABLE 2.1
DETAILS OF EXISTING DETENTION BASINS^(1,2,3)

Basin ID	Basin Name	Number of Compartments	Outlet Structure		Defined Spillway Level (m AHD)	Crest Level (m AHD)
			Dimensions (mm)	Invert (m AHD)		
B01	Visy Detention Basin No. 1	1	Details not available for use in present study			
B02	Visy Detention Basin No. 2	1				
B03	Strand Terrace Basin No. 1	1	1 off 900 RCP	217.05	219.48	219.72
B04	Trafalgar Street Basin	1	1 off 300 RCP	244.92	247.79	248.03
B05	Strand Terrace Basin No. 2	1	1 off 525 RCP	216.98	218.69	219.29
B06	Whitehall Avenue Basin No. 1	1	1 off 600 RCP	213.92	216.66	217.11
B07	Whitehall Avenue Basin No. 2	1	1 off 450 RCP	206.83	208.72	209.49
B08	Marlborough Street Basin	1	1 off 450 RCP	207.00	208.24	208.51
B09	Brooklyn Drive Basin	3	Detention basin under construction at time of present study			
B10	Huntington Grove Basin	2	1 off 450 RCP	183.53	185.14	185.43
B11	Swansea Court Basin	1	1 off 450 RCP 3 off 375 RCP	180.34 180.95	No defined spillway	181.81
B12	Cardiff Drive Basin	2	1 off 600 RCP	179.31	No defined spillway	181.42
B13	Newport Crescent Basin No. 1	1	Details not available for use in present study		No defined spillway	176.43
B14	Newport Crescent Basin No. 2	3	4 off 450 RCP	177.40	177.94	178.37
B15	Whitebox Circuit Basin	4	1 off 450 RCP	191.90	192.54	192.73
B16	Springview Estate Basin	4	Detention basin under construction at time of present study			
B17	Longacre Road Basin	4	1 off 300 RCP	192.00	193.47	193.71

Refer over for footnotes to table.

TABLE 2.1 (Cont'd)
DETAILS OF EXISTING DETENTION BASINS^(1,2,3)

Basin ID	Basin Name	Number of Compartments	Outlet Structure		Defined Spillway Level (m AHD)	Crest Level (m AHD)
			Dimensions (mm)	Invert (m AHD)		
B18	Yulumbang Road Basin	1	1 off 750 RCP	195.00	No defined spillway	196.74
B19	Hartigan Street Basin	1	1 off 500 RCP	187.12	188.9	189.07
B20	Fairway Gardens Road Basin	1	1 off 300 RCP	187.66	188.90	189.10
B21	Diamond Drive Basin	4	1 off 300 RCP	172.69	No defined spillway	175.22
B22	Forest Drive Basin	3	1 off 375 RCP	202.15	No defined spillway	204.49
B23	Corriedale Court Basin	1	2 off 900 RCBC	196.83	No defined spillway	183.55
B24	Beaumont Park Basin	1	1 off 600 RCP	199.86	201.13	201.53
B25	Guy Place Basin	1	1 off 300 RCP	194.70	195.63	196.03
B26	The Grove Basin	1	2 off 1350 RCP	172.05	172.94	173.12
B27	Stirling Way Basin	1	1 off 300 RCP	191.39	No defined spillway	192.80
B28	Alpine Street Basin	2	1 off 900 RCP	184.23	187.09	187.45
B29	Peninsular Court Basin	1	1 off 600 RCP	170.55	No defined spillway	171.17
B30	Pickworth Street Basin	1	1 off 300 RCP	183.10	184.94	185.33
B31	Kerrford Drive Basin	1	1 off 525 RCP	183.90	185.49	186.69
B32	Shearer Court Basin	1	2 off 300 RCP	182.00	No defined spillway	183.23
B33	Woolshed Drive Basin No 1	1	Details not available for use in present study		No defined spillway	177.63
B34	Woolshed Drive Basin No. 2	3	1 off 750 RCP	176.76	177.61	178.03

Refer over for footnotes to table.

TABLE 2.1 (Cont'd)
DETAILS OF EXISTING DETENTION BASINS^(1,2,3)

Basin ID	Basin Name	Number of Compartments	Outlet Structure		Defined Spillway Level (m AHD)	Crest Level (m AHD)
			Dimensions (mm)	Invert (m AHD)		
B35	Kosciuszko Road Basin	2	1 off 600 RCP	185.51	186.41	186.92
B36	Zeil Circuit Basin	3	1 off 450 RCP	189.11	189.83	190.86
B37	Prestwick Street Basin	5	2 off 1230 RCBC	168.06	168.49	169.03
B38	Hillford Circuit Basin	3	1 off 300 RCP 1 off 450 RCP	167.49 167.47	No defined spillway	167.79
B39	Rodeo Drive Basin	1	1 off 450 RCP	182.71	183.62	183.91
B40	Kowari Street Basin	1	1 off 450 RCP	181.49	182.71	182.89
B41	Richmond Drive Basin	4	Details not available for use in present study		181.45	181.71
B42	Mulgara Drive Basin	1	1 off 450 RCP	181.89	No defined spillway	182.55
B43	Birdwood Lane Basin	1	1 off 300 RCP	182.89	184.05	184.42
B43	Kerr Road Basin	1	1 off 375 RCP	182.98	184.62	185.15

1. Refer **Figure 2.2**, 12 sheets for location of detention basins.
2. Inverts and elevations estimated from the LiDAR survey data.
3. Details of the outlet structure and spillway/crest elevations are for the most downstream compartment of the detention basin.

TABLE 2.2
RECORDED DAILY RAINFALL TOTALS FOR HISTORIC STORM EVENTS⁽¹⁾

Storm Event	Rainday	BoM Daily Rain Gauges				BoM AWS	BoM Pluvi	WaterNSW	
		Wodonga (GS 82056)	Albury Airport (GS 72146)	Hume Reservoir (GS 72023)	Albury Airport AWS (GS 72160)	Albury Airport AWS (GS 72160)	Hume Reservoir (GS 72023)	Murray River At Hume Dam - Storage Gauge No.2 (GS 401027)	Bowna Creek At Yambla At Yambla (GS 401015)
January 1974	4	56	Gauge not in operation at time of storm	74.8	Gauge not in operation at time of storm	Gauge not in operation at time of storm	74.8	Gauge not in operation at time of storm	Gauge not in operation at time of storm
	5	31		50.2			50.2		
	10	23.6		11			11		
	11	57.4		67.2			67.2		
August 1983	26	37	53.6	83.2			83.2		
	27	38.6	35.6	29.2			29.2		
March 2010	6	Gauge not in operation at time of storm	18	14.4	20.6		12.4	13.4	12.5
	7		4.2	2	5.8		9.4	10	9.5
	8		0	53	66		44.4	36.4	61.5
August 2010	1		4.2	4.6	4.2		4.4	2	5.4
	2		13.4	13	18.6		13	12.8	9.8
	3		0	0	0		N/A	0	1.5
	10		6.8	5.4	7.2		5.8	4	8.5
	11		7.6	11.8	8.2		12	10.8	7.7
	12		10.4	13.4	10.8		12.2	13.2	15.7
August 2016	1	6.2	4.2	4	3.8	3.8	Gauge not in operation at time of storm	3.6	3
	2	26.4	22.4	30.2	21	21		23.6	18.4
	3	0	0	0	0	0		0	0

Refer over for footnotes to table.

TABLE 2.2
RECORDED DAILY RAINFALL TOTALS FOR HISTORIC STORM EVENTS⁽¹⁾

Storm Event	Rainday	BoM Daily Rain Gauges				BoM AWS	BoM Pluvi	WaterNSW	
		Wodonga	Albury Airport	Hume Reservoir	Albury Airport AWS	Albury Airport AWS	Hume Reservoir	Murray River At Hume Dam - Storage Gauge No.2	Bowna Creek At Yambla
		(GS 82056)	(GS 72146)	(GS 72023)	(GS 72160)	(GS 72160)	(GS 72023)	(GS 401027)	(GS 401015)
March 2017	21	27.4	32.4	34	Gauge not in operation at time of storm	Gauge not in operation at time of storm	Gauge not in operation at time of storm	36.4	17.8
	22	25.4	24.5	16.2				24.2	19.2
	23	3.6	25.6	14.8				7.4	22.4
September 2021	4	39.4	Gauge not in operation at time of storm	41.4	38	38		45	51.6
	5	9.8		9.4	6.8	6.8		10.4	12.6
	6	0		0.2	0.2	0.2		0.2	0.4
November 2021	24	22	17	16.4	15.4	16.2		15.8	17.8
	25	15.2	13	12	12.4	12.8		15.6	36.2
	26	12.2	10.8	13	10.8	11.2		10.6	21
	27	5	11.6	10.4	10.6	11.2		4.6	7.6
January 2022	6	46	75	67.2	69.6	67		27.6	35.4
	7	32	36.4	34.4	33	40		33	31.8
	8	26.4	23.4	32.2	21.8	14.8		20.8	12.4
January 2022	14	0	0	0	0	0		0.8	0
	15 ⁽²⁾	23.4	72	95.4	69.2	69.2		72.6	11.2
	16	0	0	0	0	0		0.4	0.2
August 2022	7	2.2	2.8	2.6	1.6	0		7.4	9.4
	8	0	0	0	0.2	0		0.2	0
	9	0	0	0.2	0	0		0	0.2

Refer over for footnotes to table.

TABLE 2.2
RECORDED DAILY RAINFALL TOTALS FOR HISTORIC STORM EVENTS⁽¹⁾

Storm Event	Rainday	BoM Daily Rain Gauges				BoM AWS	BoM Pluvi	WaterNSW	
		Wodonga (GS 82056)	Albury Airport (GS 72146)	Hume Reservoir (GS 72023)	Albury Airport AWS (GS 72160)	Albury Airport AWS (GS 72160)	Hume Reservoir (GS 72023)	Murray River At Hume Dam - Storage Gauge No.2 (GS 401027)	Bowna Creek At Yambla (GS 401015)
October 2022	13	12.6	12.4	10.2	11.8	14.6	Gauge not in operation at time of storm	11.2	11.2
	14	40.8	38.4	41	38.2	35.6		35.6	27.4
	15	5.6	5.6	3.8	5	4.8		4.4	1.2
October 2022	21	3.8	4.4	3.4	4	4		2.6	4.4
	22	27.4	20	1.6	23.8	23.8		0.6	13.6
	23	5.6	14	3.6	14	14		3	6
October/November 2022	31	31.2	17.6	15.2	17.4	17.4		17.4	13
	1	18	55.6	55.6	51.2	51.2		42	18.6
	2	11.2	17.8	11	12	12		10.4	11.2
November 2022	12	10	42.8	0	0.2	0.2		0	0
	13	38	44.1	43.6	41	41		39.8	17.8
	14	30.2	3.4	36	43.2	43.2		33.6	34.8
	15	0	0.1	4.8	2.6	2.6		2.8	4.8
June 2023	23	15.4	0	15.2	17	17		13.8	8
	24	9.6	17	7.4	8.8	8.8		6.6	6.6
	25	3.6	6.4	3.2	4.4	4.4		2.6	0.4
August 2024	1	0	0	0.2	0	0		0	0
	2	0	0.1	0	0	0		0	0
	3	0	0	0	0	0		0	0

1. Refer **Figure 1.1** for gauge location.

2. Total rainfall depths of 70 mm, 85.6 mm, 91.7 mm and 105.2 mm were recorded on the rain day of 15 January 2022 at the privately owned IALBUR88, IALBURY19, IALBUR40 and IALBURY11 rain gauges, respectively.

TABLE 2.3
APPROXIMATE AEPs OF RECORDED RAINFALL FOR HISTORIC STORM EVENTS^(1,2)
(% AEP)

Storm Event	Gauge	Storm Duration							
		30 minute	1 hour	1.5 hrs	2 hrs	3 hrs	4.5 hrs	6 hrs	9 hours
4-5 January 1974	Hume Reservoir [GS 72023]	<50%	20-50%	20-50%	20-50%	20-50%	20-50%	2-5%	2-5%
9-10 January 1974	Hume Reservoir [GS 72023]	<50%	<50%	<50%	<50%	<50%	20-50%	20-50%	10-20%
25-27 August 1983	Hume Reservoir [GS 72023]	<50%	<50%	<50%	<50%	<50%	20-50%	20-50%	10-20%
6-8 March 2010	Hume Reservoir [GS 72023]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	Murray River at Hume Dam [GS 401027]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	Bowna Creek at Yambla [GS 401015]	20-50%	10-20%	10-20%	10-20%	10-20%	10-20%	10-20%	10-20%
1-3 August 2010	Hume Reservoir [GS 72023]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	Murray River at Hume Dam [GS 401027]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	Bowna Creek at Yambla [GS 401015]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
10-12 August 2010	Hume Reservoir [GS 72023]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	Murray River at Hume Dam [GS 401027]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	Bowna Creek at Yambla [GS 401015]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
2 August 2016	Albury Airport AWS [GS 72160]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	Murray River at Hume Dam [GS 401027]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	Bowna Creek at Yambla [GS 401015]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%

Refer over for footnotes to table.

TABLE 2.3 (Cont'd)
APPROXIMATE AEPs OF RECORDED RAINFALL FOR HISTORIC STORM EVENTS^(1,2)
(% AEP)

Storm Event	Gauge	Storm Duration							
		30 minute	1 hour	1.5 hrs	2 hrs	3 hrs	4.5 hrs	6 hrs	9 hours
21 March 2017	Murray River at Hume Dam [GS 401027]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	Bowna Creek at Yambla [GS 401015]	20-50%	20-50%	<50%	<50%	<50%	<50%	<50%	<50%
5 September 2021	Albury Airport AWS [GS 72160]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	Murray River at Hume Dam [GS 401027]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	50%
	Bowna Creek at Yambla [GS 401015]	<50%	<50%	<50%	<50%	<50%	<50%	50%	50%
25 November 2021	Albury Airport AWS [GS 72160]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	Murray River at Hume Dam [GS 401027]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	Bowna Creek at Yambla [GS 401015]	<50%	<50%	<50%	<50%	20-50%	20-50%	20-50%	20-50%
5-6 January 2022	Albury Airport AWS [GS 72160]	50%	20%	20%	20%	20%	10%	10%	10%
	Murray River at Hume Dam [GS 401027]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	Bowna Creek at Yambla [GS 401015]	<50%	50%	50%	50%	<50%	<50%	<50%	<50%
14-15 January 2022	Albury Airport AWS [GS 72160]	2-5%	0.2%	0.5%	0.5%	1%	2%	2-5%	5%
	Murray River at Hume Dam [GS 401027]	2%	0.2%	0.5%	0.5%	1%	2%	2-5%	5%
	Bowna Creek at Yambla [GS 401015]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	IALBURY11	0.05%	0.05%	0.05%	0.05%	0.05%	0.10%	0.1-0.2%	0.2-0.5%
	IALBURY19	0.05%	0.05-0.1%	0.1-0.2%	0.2-0.5%	0.2-0.5%	0.5-1%	1-2%	2-5%

Refer over for footnotes to table.

TABLE 2.3 (Cont'd)
APPROXIMATE AEPs OF RECORDED RAINFALL FOR HISTORIC STORM EVENTS^(1,2)
(% AEP)

Storm Event	Gauge	Storm Duration							
		30 minute	1 hour	1.5 hrs	2 hrs	3 hrs	4.5 hrs	6 hrs	9 hours
14-15 January 2022	IALBUR40	0.05%	0.05%	0.05-0.1%	0.10%	0.1-0.2%	0.2-0.5%	0.5-1%	1-2%
	IALBUR88	0.2-0.5%	0.5-1%	1-2%	1-2%	2-5%	2-5%	5-10%	10-20%
7 August 2022	Murray River at Hume Dam [GS 401027]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	Bowna Creek at Yambla [GS 401015]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
14 October 2022	Albury Airport AWS [GS 72160]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	Murray River at Hume Dam [GS 401027]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	Bowna Creek at Yambla [GS 401015]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
22 October 2022	Albury Airport AWS [GS 72160]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	Murray River at Hume Dam [GS 401027]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	Bowna Creek at Yambla [GS 401015]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
1 November 2022	Albury Airport AWS [GS 72160]	50%	<50%	50%	20-50%	50%	50%	50%	<50%
	Murray River at Hume Dam [GS 401027]	<50%	50%	50%	50%	50%	<50%	<50%	<50%
	Bowna Creek at Yambla [GS 401015]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%

Refer over for footnotes to table.

TABLE 2.3 (Cont'd)
APPROXIMATE AEPs OF RECORDED RAINFALL FOR HISTORIC STORM EVENTS^(1,2)
(% AEP)

Storm Event	Gauge	Storm Duration							
		30 minute	1 hour	1.5 hrs	2 hrs	3 hrs	4.5 hrs	6 hrs	9 hours
13-14 November 2022	Albury Airport AWS [GS 72160]	<50%	20-50%	10-20%	10-20%	20%	20-50%	20-50%	20-50%
	Murray River at Hume Dam [GS 401027]	<50%	<50%	20-50%	20-50%	20-50%	<50%	<50%	<50%
	Bowna Creek at Yambla [GS 401015]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
23 June 2023	Albury Airport AWS [GS 72160]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	Murray River at Hume Dam [GS 401027]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%
	Bowna Creek at Yambla [GS 401015]	<50%	<50%	<50%	<50%	<50%	<50%	<50%	<50%

1. Refer **Figure 1.1** for location.
2. Approximate AEP of each storm event is based on near term climatic conditions (i.e. BoM 2016 Intensity-Frequency-Duration curves that have been adjusted based on Year 2030 and Shared Socioeconomic Pathway (**SSP**) 2-4.5 conditions). Refer **Chapter 6** of this report for further details.

3 HYDROLOGIC MODEL DEVELOPMENT AND CALIBRATION

3.1 Hydrologic Modelling Approach

The present study required the use of a hydrologic model which is capable of representing the rainfall-runoff processes that occur within both the rural and urbanised parts of the study catchment. For hydrologic modelling, the practical choice is between the models known as DRAINS, RAFTS, RORB and WBNM. Whilst there is little to choose technically between these models, Initial Loss-Continuing Loss (**IL-CL**) and Hortonian loss models within the DRAINS software have been developed primarily for use in modelling the passage of a flood wave through highly urbanised catchments, whilst RAFTS, RORB and WBNM have been widely used in the preparation of rural flood studies.

Both the IL-CL and RAFTS modelling approaches which are built into the DRAINS software were used to generate discharge hydrographs from urban and rural areas, respectively, as this combined approach was considered to provide a more accurate representation of the rainfall runoff process. The discharge hydrographs generated by applying the IL-CL and RAFTS modelling approach were applied to the TUFLOW hydraulic model as either point or distributed inflow sources (refer **Section 4.4** of this report for further details).

3.2 Hydrologic Model Layout

Figure 3.1 (3 sheets) shows the layout of the hydrologic model that was developed as part of the review (**Eight Mile and Sandy Creek DRAINS Model**). Careful consideration was given to the definition of the sub-catchments which comprise the Eight Mile and Sandy Creek DRAINS Model to ensure peak flows throughout the drainage system would be properly routed through the hydraulic model. In addition to using the Light Detecting and Ranging (**LiDAR**) based contour data, the location of stormwater pits and headwalls were also taken into consideration when deriving the boundaries of the various sub-catchments (refer **Chapter B1** of **Appendix B** for details of LiDAR survey data and existing stormwater network data). The study catchments were split into a total of 3,100 sub-catchments.

Figure 3.1 shows that the RAFTS modelling approach has been used for sub-catchments predominately comprising the rural portion of the study catchment, while the IL-CL modelling approach has been applied in the more urbanised parts of study catchments. The rainfall-on-grid type approach which is built into the TUFLOW software was adopted in locations where the catchment boundaries were indistinct.

Sub-catchment slopes used for input to the hydrologic model were derived using the vectored average slope approach for sub-catchments characterised as rural (which are modelled using the RAFTS approach) and the average sub-catchment slope approach for sub-catchments characterised as urbanised (which are modelled using the IL-CL approach). Digital Elevation Models (**DEMs**) derived from the available LiDAR survey data were used as the basis for computing the slope.

Percentages of impervious area were based on a visual inspection of the aerial photography and experience in determining appropriate values for different land-use types. The Total Impervious Area (**TIA**) was used as input to the hydrologic model as questions have been raised in the industry about the appropriateness of adopting the Effective Impervious Area (**EIA**) approach set out in the 2019 edition of *Australian Rainfall and Runoff* (Geoscience Australia, 2019) (**ARR 2019**). One of

the identified issues with the approach is that it is based on a volume check rather than a peak flow check, with the adjustment factor seen as taking account of additional losses that occur in the urban environment. However, Kus et al, 2018 found that the adoption of TIA in DRAINS more closely reproduced peak flows generated by an urban catchment, as well as those derived by other peak flow estimation methods.

The adoption of the EIA approach when using a hydrologic model to generate inflow hydrographs to a two-dimensional hydraulic model is also problematic, as it is accounting for a loss of volume from each sub-catchment possibly from additional depression storage, as well as surface runoff ponding behind solid structures such as buildings and fences, a feature which is also partially accounted for in the two-dimensional model domain. If the TIA is reduced by up to 40% as recommended in ARR 2019, then the total volume and also the peak flow being input to the two-dimensional hydraulic model would be significantly reduced. This fact, coupled with the additional flood storage that is present in the two-dimensional model domain has the potential to result in an under-estimation of peak flow and volume estimates, and hence peak flood levels throughout the catchment.

3.3 Hydrologic Model Testing

3.3.1. General

Historic flood data suitable for use in the model calibration process comprises photographic and anecdotal evidence of flooding patterns that were observed during the storms that occurred on 5-6 January 2022 and 14-15 January 2022, which as discussed in **Section 2.2**, were equivalent to about 10% and 0.2% AEP storm events, respectively.

As there are no historic data on flood flows anywhere in the study area, the procedure adopted for the calibration of the hydrologic model involved an iterative process sometimes referred to as “tuning” or “validating”. This process involved the generation of discharge hydrographs for the historic storm events using a starting set of hydrologic model parameters. The discharge hydrographs were then input to the hydraulic model, which was then run with an initial set of hydraulic roughness parameters and the resulting flooding patterns compared with the photographic and anecdotal evidence.

Minimal iterations of this process were required, whereby changes were made to the hydrologic model parameters, after which the resulting adjusted discharge hydrographs were input to the hydraulic model until a good fit with observed flood behaviour was achieved (refer **Chapter 4** for further details).

3.3.2. Application of Historic Rainfall to the Hydrologic Model

Figure 2.4 (2 sheets) shows the bursts of rainfall that were incorporated in the Eight Mile and Sandy Creek DRAINS Model for the two storms of interest. **Figures 2.5** and **2.6** show the extent over which the recorded rainfall was applied to the various sub-catchments comprising the Eight Mile and Sandy Creek DRAINS Model, as well as isohyetal contours showing the cumulative depth of rainfall that was recorded over the study catchments over the rain days of 6 January 2022 and 15 January 2022, noting that the rainfall that was applied to each individual sub-catchment was factored based on the isohyetal contours.

3.3.3. Hydrologic Model Parameters

For the sub-catchments modelled using the RAFTS hydrologic modelling approach, a Manning's n value of 0.04 was applied to sub-catchments primarily characterised as rural pastoral land, while a value of 0.06 was applied to sub-catchments comprising a mixture of cleared pastoral land and dense vegetation. A Manning's n value of 0.08 was applied to sub-catchments comprising mostly dense vegetation. A Bx routing parameter of 1.0 was adopted for sub-catchments that were modelled in RAFTS.

An initial loss value of 6.2 mm was adopted for pervious areas based on the Probability Neutral Burst Initial Loss value extracted from the *ARR Data Hub* (a copy of which is contained in **Appendix D** of this report) for a 1% AEP 6 hour duration design storm event. A continuing loss value of 1.8 mm/hr was also applied to pervious areas in the Eight Mile and Sandy Creek DRAINS Model which was derived by factoring the raw continuing loss value obtained from the *ARR Data Hub* of 4.5 mm/hr by a factor of 0.4 based on the recommendations for deriving design losses in *Floodplain Risk Management Guide - Incorporating 2016 Australian Rainfall and Runoff in studies* (OEH, 2019).

Initial and continuing loss values of 1 mm and 0 mm/hr, respectively were applied to impervious areas in the Eight Mile and Sandy Creek DRAINS Model.

3.3.4. Results of Model Testing

When applied to the hydraulic model, the discharge hydrographs that were generated by the hydrologic model gave reasonable correspondence with observed flood behaviour. The hydrologic model parameters set out in this chapter were therefore adopted for design flood estimation purposes, noting that due to the limited availability of historic flood related data for use in the model calibration process, the initial and continuing loss values contained in the *ARR Data Hub* were adopted for design flood estimation purposes (refer **Chapter 5** of this report for further details).

4 HYDRAULIC MODEL DEVELOPMENT AND CALIBRATION

4.1 General

The present study required the use of a hydraulic model that is capable of analysing the time varying effects of flow in the local stormwater drainage system and the two-dimensional nature of flow on the floodplain and in the steeper parts of the study area that are subject to overland flow.

The TUFLOW modelling software was adopted as it is one of only a few commercially available hydraulic models which contain all the required features.

This chapter deals with the development and calibration of the TUFLOW model that was then used to define the nature of flooding in the study area for a range of design storm events (refer **Chapter 6** for further details).

4.2 The TUFLOW Modelling Approach

TUFLOW is a true two-dimensional hydraulic model which does not rely on a prior knowledge of the pattern of flood flows in order to set up the various fluvial and weir type linkages which describe the passage of a flood wave through the system.

The basic equations of TUFLOW involve all of the terms of the St Venant equations of unsteady flow. Consequently, the model is "fully dynamic" and once tuned will provide an accurate representation of the passage of the floodwave through the drainage system (both surface and piped) in terms of extent, depth, velocity and distribution of flow.

TUFLOW solves the equations of flow at each point of a rectangular grid system which represent overland flow on the floodplain and along streets. The choice of grid point spacing depends on the need to accurately represent features on the floodplain which influence hydraulic behaviour and flow patterns (e.g. buildings, streets, changes in channel and floodplain dimensions, hydraulic structures which influence flow patterns, hydraulic roughness etc.).

Piped drainage and channel systems can be modelled as one-dimensional elements embedded in the larger two-dimensional domain, which typically represents the wider floodplain. Flows are able to move between the one and two-dimensional elements of the model, depending on the capacity characteristics of the drainage system being modelled.

The TUFLOW model developed as part of the present study will allow for the future assessment of potential flood management measures, such as detention storage, increased channel and floodway dimensions, augmentation of culverts and bridge crossing dimensions, diversion banks and levee systems.

4.3 TUFLOW Model Setup

4.3.1. Model Structure

Figure 4.1 (3 sheets) shows the layout of the TUFLOW model that was developed as part of the present study (**Eight Mile and Sandy Creek TUFLOW Model**). The Eight Mile and Sandy Creek TUFLOW Model comprises a combination of one- and two-dimensional elements, details of which are set out in the following sections of the report.

4.3.2. Two-dimensional Model Domain

An important consideration of two-dimensional modelling is how best to represent the roads, fences, buildings and other features which influence the passage of flow over the natural surface. Two-dimensional modelling is very computationally intensive, and it is not practicable to use a mesh of very fine elements without excessive times to complete the simulation, particularly for long duration flood events. The requirement for a reasonable simulation time influences the way in which these features are represented in the model.

A grid spacing of 4 m with a smaller 2 m grid spacing embedded internal to the model in the vicinity of the residential estates (refer **Figure 4.1** for extent) was found to provide an appropriate balance between the need to define features on the floodplain versus model run times and was adopted for the investigation. Ground surface elevations for model grid points were initially assigned using the 2024 LiDAR derived digital elevation model (**DEM**) and supplemented using the 2020 LiDAR derived DEM (refer **Appendix B** of this report for the source of these data).

Ridge and gully lines were added to the Eight Mile and Sandy Creek TUFLOW Model where the grid spacing was too coarse to accurately represent important topographic features which influence the passage of overland flow. The elevations for these ridge and gully lines were determined from inspection of the LiDAR survey data or site-based measurements.

Gully lines were also used to represent the incised watercourses and channels in the study area. The use of gully lines ensured that positive drainage was achieved along the full length of these watercourses and thus avoided creation of artificial ponding areas as artefacts of the 'bumpy' nature of the underlying LiDAR survey data.

The Ettamogah Forest Dam, along with local farm dams, were assumed full at the start of the model simulation (i.e. at the onset of flood producing rain).⁸

The footprints of individual buildings located in the two-dimensional model domain were digitised and assigned a high hydraulic roughness value relative to the more hydraulically efficient roads and flow paths through allotments. This accounted for their blocking effect on flow while maintaining a correct estimate of floodplain storage in the model.

It was not practicable to model the individual fences surrounding the many allotments in the study area. For the purpose of the present study, it was assumed that there would be sufficient openings in the fences to allow water to enter the properties, whether as flow under or through fences and via openings at driveways. Individual allotments where development is present were digitised and assigned a high hydraulic roughness value (although not as high as for individual buildings) to account for the reduction in conveyance capacity which will result from obstructive fences, such as Colorbond or brick, and other obstructions stored on these properties.

⁸ The Ettamogah Forest Dam was part-full at the time of the 2020 LiDAR survey data was captured, enabling this dataset to be used as the basis for partially defining the bathymetry behind the dam wall. While the adoption of the 2020 LiDAR survey data underestimates the actual depth of the water storage, as it was assumed full at the commencement of the model simulation, its inclusion in the two-dimensional domain does not result in a reduction in downstream flood volumes.

4.3.3. One-dimensional Model Elements

Details of the existing piped drainage system, a summary of which is set out in **Table 4.1** over the page, were incorporated into the Eight Mile and Sandy Creek TUFLOW Model (refer **Section B1.2** of **Appendix B** for sources of data). Invert levels were derived from survey data, else they were estimated by assuming 700 mm cover below the natural surface level as defined by the LiDAR survey data. The estimated inverts were then adjusted to ensure they were positively graded. The Eight Mile Creek crossing of the Riverina Highway which comprises an 8.2 m wide by 3 m high ellipsoidal culvert was modelled as a rectangular box culvert with an equivalent waterway area.

TABLE 4.1
SUMMARY OF MODELLED DRAINAGE STRUCTURES

Pipes		Box Culverts		Inlet Pits	Junction Pits	Headwalls
No.	Length (m)	No.	Length (m)	No.	No.	No.
3,523	86,430	148	2,930	2,874	296	782

Several types of pits are identified on **Figure 4.1**, including junction pits which have a closed lid and inlet pits which are capable of accepting overland flow. Inlet pit types and dimensions were incorporated in the TUFLOW model based on a visual inspection of the existing stormwater drainage system. Inlet pit capacity relationships were taken from those in-built to the TUFLOW software where appropriate, else they were calculated using an in-house spreadsheet model.

Pit losses throughout the various piped drainage networks were modelled using the Engelund approach in TUFLOW. This approach provides an automatic method for determining time-varying energy loss coefficients at pipe junctions that are recalculated each time step based on a range of variables including the inlet/outlet flow distribution, the depth of water within the pit, expansion and contraction of flow through the pit, and the horizontal deflection and vertical drop across the pit. The losses derived using the automated Engelund approach in TUFLOW are generally within the range of expected values derived using other methods.

4.3.4. Model Parameters

The main physical parameter for TUFLOW is the hydraulic roughness. Hydraulic roughness is required for each of the various types of surfaces comprising the overland flow paths, as well as inbank areas of the creeks. In addition to the energy lost by bed friction, obstructions to flow also dissipate energy by forcing water to change direction and velocity and by forming eddies. Hydraulic modelling traditionally represents all of these effects via the surface roughness parameter known as "Manning's n". Flow in the piped system also requires an estimate of hydraulic roughness.

Manning's n values along the channel and immediate overbank areas along the modelled length of creeks were varied, with the values in **Table 4.2** over the page providing reasonable correspondence between recorded and modelled flood levels.

The adoption of a value of 0.02 for the surfaces of roads, along with an adequate description of their widths and centreline/kerb elevations, allowed an accurate assessment of their conveyance capacity to be made. A relatively high roughness value of 0.1 has been applied to the grassed and paved inter-allotment area to account for the blocking effect that various features in private properties such as fences, landscaping, vegetation etc. will have on flood behaviour.

TABLE 4.2
BEST ESTIMATE HYDRAULIC ROUGHNESS VALUES

Surface Treatment	Manning's n Value
Concrete piped elements	0.015 ⁽¹⁾
Asphalt or concrete road surface	0.02
Overbank area, including grass and lawns	0.045
Lightly Vegetated Areas, Wetlands	0.06
Densley Vegetated areas	0.08
Very Densley Vegetated areas	0.10
Rural Allotments (between buildings)	0.05
Urban Allotments (between buildings)	0.10
Buildings	10

1. It has been assumed that the piped elements are old and have a slightly higher Manning's n value than a new concrete pipe.

Figure 4.3 is a typical example of flow patterns derived from the above roughness values. This example applies to the 1% AEP design storm event and shows flooding patterns along the overland flow path that runs in an easterly direction through residential development between Fenchurch Drive and Whitehall Avenue in Ettamogah Rise. The left hand side of the figure shows the roads and inter-allotment areas, as well as the outlines of buildings, which have all been assigned different hydraulic roughness values in the model. The right hand side shows the resulting flow paths in the form of scaled velocity vectors and the depths of inundation. Similar information to that shown on **Figure 4.3** may be presented at any location within the model domain and will be of assistance to Council in assessing individual flooding problems in the study area.

4.4 Model Boundary Conditions

The locations where sub-catchment inflow hydrographs were applied to the Eight Mile and Sandy Creek TUFLOW Model are shown on **Figure 4.1**. These comprise inflow boundaries at selected locations around the perimeter of the two-dimensional model domain, point-source inflows at selected locations internal to the model (for example, at the location of surface inlet pits) and as distributed inflows via "Inflow Regions".

The Inflow Regions act to "inject" flow into the Eight Mile and Sandy Creek TUFLOW Model, firstly at a point which has the lowest elevation, and then progressively over the extent of the Inflow Regions as the grid in the two-dimensional model domain becomes wet as a result of overland flow. The Inflow Regions have been digitised at the outlet of the catchment to reduce the "double-routing" of runoff from the sub-catchment, as well as internal to the model (for example, at the location of surface inlet pits).

As mentioned in **Section 3.2**, the rainfall-on-grid type approach which is built into the TUFLOW software was adopted in locations where the catchment boundaries were indistinct. The direct-rainfall-on-grid approach involves the application of rainfall excess to the two-dimensional model domain, with the routing of the rainfall excess (runoff) simulated across each grid cell within the area shown on **Figure 4.1**.

The downstream boundary of the model comprises a static tailwater level in the Murray River of RL 154 m AHD which is considered to be representative of the water level in the river resulting from typical regulated flow releases from Hume Dam, noting that an analysis was undertaken to assess the sensitivity of the flood behaviour to elevated tailwater levels in the Murray River (refer **Section 6.7.4** for further discussion).

TUFLOW-derived normal depth relationships were also incorporated in the Eight Mile and Sandy Creek TUFLOW Model at selected locations along its western and northern boundary where surface runoff is diverted into adjacent catchments due to road/rail/farm dam embankments.

4.5 Results of Model Calibration Process

As previously mentioned, the hydrologic and hydraulic models were calibrated using data that were available for the storms that occurred on 5-6 January 2022 and 14-15 January 2022.

Figures 4.4 and 4.5 (12 sheets each) show the Eight Mile and Sandy Creek TUFLOW Model results for the 5-6 January 2022 and 14-15 January 2022 storm events, respectively, while **Table 4.3** at the end of this chapter briefly describes the flood behaviour that was observed during each storm event and how it compares to the results of the Eight Mile and Sandy Creek TUFLOW Model.^{9,10} **Figure 4.6** shows modelled water surface profiles along the main arms of Eight Mile Creek, Nine Mile Creek, Seven Mile Creek and Woolshed Creek for the two historic storm events.

While a significant number of respondents to the *Community Questionnaire* identified that they were inundated from localised flows from adjacent properties and/or failure of their internal/inter-allotment piped drainage system, the assessment of these localised drainage issues lies outside the scope of the review.

While the Eight Mile and Sandy Creek TUFLOW Model was generally able to reproduce flood behaviour that was observed during the 5-6 January 2022 and 14-15 January 2022 storm events, it is believed that a localised burst of intense rain that caused flooding in the vicinity of University Close is not reflected in the available rainfall record.

Based on the findings of the model calibration/validation process, the flood models were considered to give satisfactory agreement with the available historic flood data, noting that the accuracy of the model calibration/validation is limited by the accuracy of the underlying flood data. In the absence of more detailed flood data, the hydraulic model parameters set out in **Sections 4.3 and 4.4**, and in particular the hydraulic roughness values set out in **Table 4.2**, were considered appropriate for use in defining flood behaviour in the study area over the full range of design flood events. Further discussion and presentation of hydrologic model parameters that were adopted for design flood estimation purposes is provided in **Section 5.3**.

⁹ In order to correlate the hydraulic (TUFLOW) model results with observed flood behaviour, it was necessary to show areas where depths of inundation of less than 100 mm are present. While this approach is considered appropriate for model calibration/validation purposes, it is not considered to be appropriate when presenting the results of the design flood modelling (refer **Chapter 6** of this report for further details). This is because very shallow flow is sensitive to inaccuracies in the underlying LiDAR survey data, as well as localised features internal to individual allotments, the latter which are not able to be accurately represented in the hydraulic (TUFLOW) model. Shallow inundation of this nature is also not classified as Major Overland Flow and is therefore beyond the scope of the present study to identify.

¹⁰ Where a respondent to the Community Questionnaire did not specify if the observed flood behaviour was for the 5-6 January 2022 or 14-15 January 2022 storm event, it has been assumed that it relates to the latter.

TABLE 4.3
COMPARISON OF OBSERVED AND MODELLED FLOOD BEHAVIOUR⁽¹⁾
5-6 JANUARY 2022 STORM EVENT

Residential Estate	Observed Flood Behaviour/ Other Comment	Model Verification Comments
Somerset Rise	The front yard of a residential allotment that is located at the southern end of Hay Court was inundated.	Maximum depth of inundation in the vicinity of the front yard computed to be less than 0.1 m.
Thurgoona Park	The front yard of a residential allotment that is located on the eastern side of Roseborough Court was inundated to a maximum depth of 0.3 m. The dwelling on the residential allotment was subject to above-floor inundation.	Maximum depth of overland flow through the front yard computed to be about 0.2 m. Maximum depth of overland flow adjacent to the dwelling computed to be less than 0.1 m.
	The back yard of a residential allotment that is located on the western side of Rosborough Court was inundated. The footbridge across the watercourse at the northern end of the allotment was inundated by approximately 0.1 m.	Maximum depth of inundation in the vicinity of backyard computed to be about 0.6 m.
	The front yard, back yard and garage of a residential allotment that is located on the western side of Nelson Court was inundated.	Maximum depth of inundation in the vicinity of garage computed to be about 0.2 m.
University Close	Front yard, back yard and garage of a residential allotment that is located at the northern end of Briesse Court was inundated.	Flood behaviour in the vicinity of University Close could not be reproduced. The nature of the observed flood behaviour indicates that intense localised rainfall was experienced in this area which resulted in inundation of existing development. There are no BoM, WaterNSW or privately owned rain gauges in the area that were in operation at the time of the storm.
	A residential allotment on the southern side of Briesse Court experienced above-floor inundation of depths less than 0.1 m as a result of runoff from neighbouring properties.	
	A residential allotment on the western side of Paton Close experienced inundation of the front yard, back yard and garage as a result of runoff from neighbouring properties.	

TABLE 4.4
COMPARISON OF OBSERVED AND MODELLED FLOOD BEHAVIOUR⁽¹⁾
14-15 JANUARY 2022 STORM EVENT

Residential Estate	Observed Flood Behaviour/ Other Comment	Model Verification Comments
Ettamogah Rise	The front yard of a residential allotment located to the east of the Fenchurch Drive and Strand Terrace intersection was inundated.	Depth of overland flow through front yard of property was computed to be less than 0.1 m.
	The back yard of the residential allotment located on the western side of Fenchurch Drive was inundated.	Maximum depth of overland flow through property was computed to be about 0.4 m.
	The intersection of Whitehall Ave and Euston St was completely inundated.	Maximum depth of inundation in the vicinity of the intersection computed to be about 0.5 m.
-	The front/back yard and garage of the residential allotment on the western side of Table Top Road was inundated.	Maximum depth of inundation in the vicinity of the property computed to be about 0.4 m.
Thurgoona Park	Floodwaters on the northern side of Kimball Court caused the backyard to be inundated for weeks with the shed flooding.	Maximum depth of inundation in the vicinity of the property computed to be about 0.6 m.
	The back yard and garage of the residential allotment on the northern side of Finlay Road was inundated.	Maximum depth of inundation in the rear of the property computed to be about 0.4 m.
	The watercourse running through the north-eastern corner of the allotment was approximately 2m deep.	Maximum depth of inundation in the watercourse computed to be about 1.8 m.
	The watercourse running through the rear of a residential allotment on northern side of Finlay Road was approximately 1 m deep and the width of flow was approximately 5 m.	The depth of flow in the watercourse was computed to be about 1.3 m. The width of flow was computed to be about 8 m.
	A residential allotment on the western side of Finlay Road experienced floodwater in the backyard due to floodwater surcharging the left bank Woolshed Creek.	Maximum depth of inundation in the vicinity of the back yard computed to be about 0.51 m.
Brooklyn Fields	Birchgrove Crescent was inundated to depths of approximately 0.3 m approximately 100 m to the south if its intersection with Cardiff Drive and the adjacent retarding basin (Basin B12) was full.	Maximum depth of inundation in the vicinity of the low point in Birchgrove Crescent computed to be about 0.3 m. Maximum depth of inundation in Basin B12 was computed to be about 2 m.

TABLE 4.4 (Cont'd)
COMPARISON OF OBSERVED AND MODELLED FLOOD BEHAVIOUR⁽¹⁾
14-15 JANUARY 2022 STORM EVENT

Residential Estate	Observed Flood Behaviour/ Other Comment	Model Verification Comments
Thurgoona Park	Overland flow entered property from direction of Bennett Road. Garage inundated to a depth of about 2 inches (i.e. 0.05 m). Back yard inundated to depths of about 4-8 inches (i.e. 0.1-0.2 m) from floodwater that originated from direction of Kerr Road.	Model results show shallow overland flow enter the western boundary of the property (i.e. from the direction of Bennett Road). Maximum depth of inundation in the vicinity of the garage computed to be about 0.2 m.
	The backyard and garage of a residential allotment on the western side of Rosborough Court was inundated.	Maximum depth of overland flow through property was computed to be about 0.1 m.
	Floodwater flowing down Lispett Road "like a raging river". The garage of a residential allotment on the eastern side of Lispett Road inundated to a depth of about 0.12m.	Maximum depth of flow in Lispett Road was computed to be about 0.3 m, with a maximum flow velocity of about 1 m/s. Maximum depth of overland flow in the vicinity of the garage computed to be about 0.2 m.
	Floodwater got to within 10 mm of inundating the floor level of a property that is located on the western side of Finlay Road.	Maximum depth of overland flow through adjacent to the dwelling was computed to be about 0.1 m.
	The front yard of a residential allotment that is located on the eastern side of Finlay Road intersection was inundated.	Maximum depth of overland flow through property was computed to be less than 0.1 m.
	A residential allotment at the northern end of Nelson Court experienced above-floor inundation.	Maximum depth of inundation in the vicinity of the dwelling computed to be about 0.5 m.
	Floodwater surcharged the table drains in Lispett Road and inundated the front and back yards of a property that is located on the western side of Lipsett Road.	Maximum depth of overland flow through property was computed to be about 0.2 m.

TABLE 4.4 (Cont'd)
COMPARISON OF OBSERVED AND MODELLED FLOOD BEHAVIOUR⁽¹⁾
14-15 JANUARY 2022 STORM EVENT

Residential Estate	Observed Flood Behaviour/ Other Comment	Model Verification Comments
Thurgoona Park	The front yard of a residential allotment that is located on the north-eastern corner of the intersection of Benzel Road and Finlay Road was inundated to a maximum depth of 0.05 m.	Maximum depth of overland flow through property was computed to be less than 0.1 m.
	The front yard, back yard and garage of a residential allotment that is located on the eastern side of Bennet Road was inundated.	Maximum depth of overland flow through property was computed to be about 0.3 m.
	A dwelling that is located on the western side of Nelson Court was inundated above-floor level by approximately 0.15 m by backflow from bathroom wastewater pipes.	The assessment of the backflow through internal sewer pipes is outside the scope of the review. Maximum depth of overland flow through adjacent to dwelling was computed to be about 0.1 m.
	The front yard of a residential allotment that is located on western side of intersection of Davenport Road and Lipsett Road was inundated.	Maximum depth of overland flow through the front yard was computed to be less than 0.1 m.
	The back yard of a residential allotment on the western corner of the intersection of Davenport Road and Bowles Court was inundated.	Maximum depth of overland flow through the back yard was computed to be about 0.2 m.
	The front yard of a residential allotment that is located on eastern side of Gale Court was inundated.	Maximum depth of overland flow through the front yard was computed to be about less than 0.1 m.
	Plate C6.1 shows parts of Gale Court was overtopped.	Maximum depth of flow across the crown of Gale Court was computed to be less than 0.1 m.
	Water backed up pipe from Kerr Road drainage line, inundating a dwelling that is located between Bennet Road and Kerr Road. The depth of above-floor inundation was about 0.3 m.	Maximum depth of inundation in the vicinity of the dwelling computed to be about 0.1 m.
	The dwelling and garage of a property that is located on the western side of Finlay Road was inundated above-floor level.	Maximum depth of inundation in the vicinity of the dwelling computed to be about 0.1 m.
	A dwelling that is located on the western side of Gale Court experienced above-floor inundation of depths up to 0.1 m as a result of Major Overland Flow surcharging the road and spoon drain to the east of the property.	Maximum depth of inundation in the vicinity of the dwelling computed to be about 0.2 m.

TABLE 4.4 (Cont'd)
COMPARISON OF OBSERVED AND MODELLED FLOOD BEHAVIOUR⁽¹⁾
14-15 JANUARY 2022 STORM EVENT

Residential Estate	Observed Flood Behaviour/ Other Comment	Model Verification Comments
Thurgoona Park	Garage of a residential allotment located on the southern side of Vaughan Road was inundated.	Maximum depth of inundation in the vicinity of the garage computed to be about 0.1 m.
	The front yard and garage of a residential allotment that is located on the southern side of Vaughan Road was inundated.	Maximum depth of inundation in the vicinity of the garage computed to be less than 0.1 m.
	The front/back yard, garage and dwelling of the residential allotment at the southern end of Bennet Road was inundated. The maximum depth of above-floor inundation in the garage and dwelling were 0.2 m and 0.15 m, respectively.	Maximum depth of inundation in the vicinity of the dwelling and garage computed to be about 0.2 m.
University Close	The garage of the residential allotment that is located on the northern side of Briese Court was inundated.	Maximum depth of overland flow through property was computed to be about 0.1 m.
Fairway Gardens	The front/back yard, garage and dwelling of a residential allotment that is located on the southern side of Fairway Gardens Road was inundated. Floodwater surcharged Fairway Gardens Road and flowed in a south-westerly direction through the Fairway Gardens playground.	Maximum depth of inundation in the vicinity of the dwelling computed to be about 0.2 m. Model shows that floodwater surcharged Fairway Gardens Road and flowed in a south-westerly direction through the Fairway Gardens playground
	A dwelling that is located to the south-west of the intersection of Newton Circuit and Norman Way experienced above-floor inundation to depths up to 0.15-0.2 m.	Maximum depth of overland flow adjacent to the dwelling was computed to be about 0.3 m.
	The front yard and garage of a residential allotment located at the eastern end of Scott Place was inundated as a result of flow from adjoining properties. The maximum depth of above-floor inundation in the garage was measured to be 0.16-0.18 m.	Maximum depth of inundation in the vicinity of the dwelling computed to be about 0.2 m.
	The back yard and garage of a residential allotment that is located on the north-eastern side of the intersection of Pickworth Street and Whitton Drive was inundated to depths of about 0.15 m and 0.075 m, respectively.	Maximum depth of inundation in the vicinity of the backyard computed to be about 0.2 m.

TABLE 4.4 (Cont'd)
COMPARISON OF OBSERVED AND MODELLED FLOOD BEHAVIOUR⁽¹⁾
14-15 JANUARY 2022 STORM EVENT

Residential Estate	Observed Flood Behaviour/ Other Comment	Model Verification Comments
Fairway Gardens	A dwelling that is located on the western side of Pickworth Street experienced above-floor inundation.	Maximum depth of inundation in the vicinity of the dwelling computed to be about 0.2 m.
St Johns Hill	A dwelling that is located in Cobbler Court experienced above-floor inundation.	Maximum depth of inundation in the vicinity of the dwelling computed to be about 0.2 m.
Huntingdale	The Grove was inundated to the east of its intersection with The Avenue	The crown of the road was overtopped by about 0.2 m.
Tweed Court	Approximately half of the front yard of a residential allotment that is located on the north-eastern corner of Nordic Street and Tweed Court was inundated.	Maximum depth of inundation in the vicinity of the front yard computed to be about 0.2 m.
	Residential allotments that are located at the western end of Alpine Street were "completely flooded", noting that this observation was made by a respondent to the <i>Community Questionnaire</i> that did not live at this location.	It is not likely that the allotments were inundated as they are set about 0.9 m higher than the low point in Alpine Street. The model results show that the allotments were isolated as the low point in the road was inundated to a maximum depth of about 0.5 m.
	The front/back yard and dwelling of the residential allotment that is located on the eastern side of Litchfield Drive was inundated with an approximate depth of 0.2m.	Maximum depth of overland flow through property was computed to be about 0.2 m.
	The front/back yard of a residential allotment that is located on the eastern side of Litchfield Drive was inundated to a depth of 0.1 m, with floodwater 0.02 m below the floor level of the dwelling.	Maximum depth of overland flow through property was computed to be about 0.2 m.
Kerrford Park	Plate C6.3 shows floodwater flowing in a westerly direction along Sittella Way, where it surcharged the western kerb approximately 50 m to the north of its intersection with Kerrford Drive and flowed through residential allotments. A dwelling on the western side of Sittella Way was inundated above-floor level.	The model shows floodwater surcharging the western kerb in Sittella Way and inundating the residential allotments to a maximum depth of about 0.3 m. Maximum depth of inundation in the vicinity of the dwelling computed to be about 0.2 m.

TABLE 4.4 (Cont'd)
COMPARISON OF OBSERVED AND MODELLED FLOOD BEHAVIOUR⁽¹⁾
14-15 JANUARY 2022 STORM EVENT

Residential Estate	Observed Flood Behaviour/ Other Comment	Model Verification Comments
Kerrford Park	The intersection of Kerrford Drive and Blue Wren Close was completely inundated.	Depth of overland flow in the vicinity of intersection computed to be about 0.5 m.
The Elms	The front yard, garage and dwelling of a residential allotment that is located on the eastern side of Loch Court were inundated above-floor level.	Maximum depth of overland flow through property was computed to be about 0.2 m.
St Johns Green	The front yard and garage of the residential allotment that is located on the southern side of Stockton Court was inundated.	Maximum depth of inundation in the vicinity of the front yard computed to be about 0.2 m.
-	A residential allotment that is located on the eastern side of Table Top Road had parts of its yard inundated to depths of up to 3 m (general comment about flooding in property).	Maximum depth of inundation in the vicinity of the rear of the computed to be about 2 m.
-	The paddocks of the rural residential allotment that is located on the northern side of the Riverina Highway west of its intersection with Table Top Road intersection were inundated.	Maximum depth of inundation in the vicinity of the paddocks computed to be about 2 m.
-	The acreage located to the north-west of the intersection of Kerr Road and Riverina Highway was inundated.	Maximum depth of inundation in the vicinity of the paddocks computed to be about 1.7 m.
-	Plate C6.2 shows that local access road to rural allotment that is located on the southern side of the Riverina Highway was inundated.	Property is located in an area that is subject to localised catchment runoff, the definition of which is outside the scope of the present study.
-	The paddocks of a rural residential allotment that is located on the southern side of the Riverina Highway were inundated.	Maximum depth of inundation in the vicinity of the backyard computed to be about 1 m.
-	The back yard of the rural residential allotment that is located on the southern side of the Riverina Highway was inundated.	Maximum depth of inundation in the vicinity of the backyard computed to be about 1.1 m.
-	The paddocks of the rural residential allotment that is located at the western end of Hawkscote Road were inundated.	Maximum depth of inundation in the vicinity of the paddocks computed to be about 0.6 m.

5 DERIVATION OF DESIGN FLOOD HYDROGRAPHS

5.1 Design Storms

5.1.1. Rainfall Intensity

The procedures used to obtain temporally and spatially accurate and consistent Intensity-Frequency-Duration (**IFD**) design rainfall curves for the assessment of flood behaviour in the study area are presented in ARR 2019. Design storms for frequencies of 20%, 5%, 2%, 1%, 0.5% and 0.2% AEP were derived for storm durations ranging between 15 minutes and seven days. The IFD dataset was downloaded from the BoM's *2016 Rainfall IFD Data System (BoM 2016 IFD Data)* using the co-ordinates set out in **Appendix D**.

A recent update to ARR 2019 (**ARR 2019 Rev 4.2**) includes multiplication factors which are to be applied to the BoM 2016 IFD data to account for increases in design rainfall intensity estimates out to the year 2100. For the purpose of the present assessment, the multiplication factors for the Year 2030 and Shared Socioeconomic Pathway (**SSP**) 2-4.5 condition have been applied to the BoM 2016 IFD data (**SSP2-4.5 Year 2030 IFD Data**) as this represents near-term conditions.¹¹

5.1.2. Areal Reduction Factors

The rainfalls derived using the processes outlined in ARR 2019 are applicable strictly to a point. In the case of a catchment of over tens of square kilometres area, it is not realistic to assume that the same rainfall intensity can be maintained. An Areal Reduction Factor (**ARF**) is typically applied to obtain an intensity that is applicable over the entire catchment.

It is noted that it is not appropriate to apply the above ARF to all sub-catchments in the Eight Mile and Sandy Creek DRAINS Model as the purpose of the present study was to define flood behaviour in areas subject to Major Overland Flow where the contributing catchments are relatively small. As such, an ARF value of 1.0 was applied to all sub-catchments in the Eight Mile and Sandy Creek DRAINS Model.

5.1.3. Temporal Patterns

ARR 2019 prescribes the analysis of an ensemble of 10 temporal patterns per storm duration for various zones in Australia. These patterns are used in the conversion of a design rainfall depth with a specific AEP into a design flood of the same frequency. The patterns may be used for AEPs down to 0.2 per cent where the design rainfall data is extrapolated for storm events with an AEP less than 1 per cent.

Point temporal pattern ensembles that are applicable to Frequent (more frequent than 14.4% AEP), Intermediate (between 14.4% and 3.2% AEP) and Rare (rarer than 3.2% AEP) storm events with durations of less than 12 hours were obtained from the *ARR Data Hub*¹². Temporal patterns for very rare events were taken from BoMs update of *Bulletin 53* (BoM, 2003) and Jordan et. al., 2005.

A copy of the data extracted from the *ARR Data Hub* is contained in **Appendix D**.

¹¹ Climate change factors relating to SSP2-4.5 conditions were adopted as they represent the middle range of projected increases in design rainfall intensities.

¹² It is noted that the temporal pattern data set for the *Murray-Darling Basin* region is suitable for use in the study area.

5.1.4. Probable Maximum Precipitation

Estimates of Probable Maximum Precipitation (**PMP**) were made using the Generalised Short Duration Method (**GSDM**) as described in the BoM, 2003. This method is appropriate for estimating extreme rainfall depths for catchments up to 1000 km² in area and storm durations up to 3 hours.

The steps involved in assessing PMP for the study catchment are briefly as follows:

- Calculate PMP for a given duration and catchment area using depth-duration-area envelope curves derived from the highest recorded US and Australian rainfalls.
- Adjust the PMP estimate according to the percentages of the catchment which are meteorologically rough and smooth, and also according to elevation adjustment and moisture adjustment factors.
- Apply the Year 2030, SSP2-4.5 climate change multiplication factors to the PMP estimates.
- Assess the design spatial distribution of rainfall using the distribution for convective storms based on US and world data but modified in the light of Australian experience.
- Derive storm hyetographs using the eleven temporal distributions contained in BoM, 2003 (one) and Jordan et. al., 2005 (ten), all of which are based on pluviographic traces recorded in major Australian storms.

Figure 3.1 shows the location and orientation of the PMP ellipses which were used to derive the rainfall estimates for the present study, while **Table 5.1** sets out the parameters that were adopted as part of the present study.

TABLE 5.1
ADOPTED GSDM PARAMETERS

Parameter	Value
Elevation Adjustment Factor [EAF]	1.0
Moisture Adjustment Factor [MAF]	0.61
Maximum Duration (hours)	3

5.2 Design Rainfall Losses

The initial and continuing loss values for pervious areas to be applied in flood hydrograph estimation were derived using the NSW jurisdictional specific procedures set out in the *ARR Data Hub*. The raw Probability Neutral Burst Initial Loss (**PNBIL**) values obtained from the *ARR Data Hub* were reviewed and adjusted to remove inconsistencies in values with varying storm probability and duration. **Figure 5.1** shows the original PNBIL curves derived from the tables obtained from the *ARR Data Hub*, together with the adopted PNBIL curves following the adjustments that were made as part of the present study.

The NSW jurisdictional advice recommends multiplying the raw (or unadjusted) continuing loss value that is contained on the *ARR Data Hub* of 4.5 mm/hr by a factor of 0.4 for design flood estimation. As such, a continuing loss value of 1.8 mm/hr has been applied to pervious areas for design flood estimation purposes.

Initial and continuing loss values of 1 mm and 0 mm/hr, respectively were applied to impervious areas in the Eight Mile and Sandy Creek DRAINS Model.

5.3 Derivation of Design Discharges

The Eight Mile and Sandy Creek DRAINS Model was run for a range of design storms using the design rainfall data set out in **Sections 5.1** and **5.2**, as well as the hydrologic model parameters set out in **Section 3.3.3** in order to obtain design discharge hydrographs for input to the Eight Mile and Sandy Creek TUFLOW Model.

6 HYDRAULIC MODELLING OF DESIGN FLOOD EVENTS

6.1 Hydraulic Model Structure

As per the requirements of ARR 2019, the potential for the existing drainage system to experience a partial blockage during a flood event was taken into account when deriving the design flood envelopes. **Table E1** in **Appendix E** provides a summary of the blockage factors that were derived for each individual inlet headwall and bridge structure in the study area based on the procedures set out in ARR 2019. As per the recommendations in ARR 2019, an L_{10}^{13} of 1.5 m was adopted for the blockage assessment, which is the recommended minimum value that should be adopted for urban areas in the absence of a record of past debris accumulated at the structure. Blockage factors of 20% and 50% were applied to on-grade and sag stormwater inlet pits, respectively.

The Eight Mile and Sandy Creek TUFLOW Model is representative of ground conditions that were current at the time that the most recent LiDAR survey data that were captured in September 2024, noting that there were several housing type developments that were under construction at the time which meant that their details have not been incorporated in the model. The figures presented in **Chapter 6** of this report show the extent over which development has occurred between September 2024 and December 2025 that has not been incorporated in the Eight Mile and Sandy Creek TUFLOW Model.

6.2 Critical Duration and Temporal Patterns

The critical storm durations and associated median temporal patterns for the design storm events were derived based on the results of running both the Eight Mile and Sandy Creek DRAINS Model and a coarse version of the Eight Mile and Sandy Creek TUFLOW Model in tandem.¹⁴ For example, design discharge hydrographs for the ensemble of temporal patterns for storm durations ranging between 15 minutes and 12 hours were exported from the Eight Mile and Sandy Creek DRAINS Model and input to the coarse Eight Mile and Sandy Creek TUFLOW Model. The assessment was undertaken for the 20%, 5% and 1% AEP storm events which represent the three temporal pattern bins (i.e. frequent, intermediate and rare, respectively) that were downloaded from the *ARR Data Hub*.

A similar process was adopted for determining the critical durations for the PMF using the procedures set out in BoM, 2003 and Jordan et al., 2005, whereby design discharge hydrographs for storm durations ranging between 15 minutes and 3 hours were exported from the Eight Mile and Sandy Creek DRAINS Model and input to the coarse Eight Mile and Sandy Creek DRAINS Model.

Table 6.1 sets out the storm durations and temporal patterns that were adopted as being critical for AEPs ranging from 20% and 0.2%, as well as the PMF.

¹³ L_{10} is defined as the average length of the longest 10% of the debris reaching the site.

¹⁴ To reduce model run times, the coarse version of the Eight Mile Creek TUFLOW Model incorporates an 8 m cell size. It also omits buildings to prevent the artificial blocking of overland flow paths.

TABLE 6.1
CRITICAL DURATIONS AND TEMPORAL PATTERNS

Design Storm Event	Temporal Pattern Bin	Critical Storm Duration and Temporal Pattern ⁽¹⁾
20%	Frequent	30 minute, temporal pattern 1 [3807] 1.5 hour, temporal pattern 7 [3926] 3 hour, temporal pattern 3 [3982] 4.5 hour, temporal pattern 6 [4016] 9 hour, temporal pattern 5 [4072]
10%	Intermediate	30 minute, temporal pattern 7 [3828] 1.5 hour, temporal pattern 8 [3915] 2 hour, temporal pattern 6 [3944] 3 hour, temporal pattern 1 [3974] 6 hour, temporal pattern 6 [4038] 9 hour, temporal pattern 6 [4063]
5%		
2%	Rare	30 minute, temporal pattern 6 [3815] 1 hour, temporal pattern 2 [3819] 1.5 hour, temporal pattern 8 [3907] 2 hour, temporal pattern 4 [3934] 3 hour, temporal pattern 6 [3963] 4.5 hour, temporal pattern 7 [3993]
1%		
0.5%		
0.2%		
PMF	Very Rare	15 minute, Melbourne 1972 temporal pattern 45 minute, Melbourne 1972 temporal pattern 1 hour, Melbourne 1972 temporal pattern 2 hour, Melbourne 1972 temporal pattern

1. Value in [] represent the Event ID for the critical storm duration and temporal pattern.

6.3 Presentation and Discussion of Results

6.3.1. Accuracy of Hydraulic Modelling

The accuracy of results depends on the precision of the numerical finite difference procedure used to solve the partial differential equations of flow, which is also influenced by the time step used for routing the flood wave through the system and the grid spacing adopted for describing the natural surface levels in the floodplain. Channels are described by cross-sections normal to the direction of flow, so their spacing also has a bearing on the accuracy of the results. The results are also heavily dependent on the size of the two-dimensional grid, as well as the accuracy of the LiDAR survey data which has a design accuracy based on 95% of points within +/- 150 mm. Given the uncertainties in the LiDAR survey data and the definition of features affecting the passage of flow, maintenance of a depth of flow of at least 200 mm is required for the definition of a “continuous” flow path in the areas subject to shallow overland flow. Lesser modelled depths of inundation may be influenced by the above factors and therefore may be spurious, especially where that inundation occurs at isolated locations and is not part of a continuous flow path. In areas where the depth of inundation is greater than the 200 mm threshold and the flow path is continuous, the likely accuracy of the hydraulic modelling in deriving peak flood levels is considered to be between 100 and 150 mm.

Use of the flood study results when applying flood related controls to development proposals should be undertaken with the above limitations in mind. Proposals should be assessed with the benefit of a site survey to be supplied by applicants in order to allow any inconsistencies in results to be identified and given consideration. This comment is especially appropriate in the areas subject to shallow overland flow, where the inaccuracies in the LiDAR survey data or obstructions to flow would have a proportionally greater influence on the computed water surface levels than in the deeper flooded areas.

6.3.2. Design Flood Extents, Depths and Elevations

Figures 6.1 to 6.8 (12 sheets each) show the TUFLOW model results for the 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP floods, together with the PMF. These diagrams show the indicative extent and depth of inundation for the full range of design storm events throughout the study area.

In order to create realistic results which remove most of the anomalies caused by inaccuracies in the LiDAR survey data, a filter was applied to remove depths of inundation over the natural surface less than 100 mm. This has the effect of removing the very shallow depths which are more prone to be artefacts of the model, but at the same time giving a reasonable representation of the various overland flow paths. The depth grids shown on the figures have also been trimmed to the building polygons, as experience has shown that property owners incorrectly associate depths of above-ground inundation at the location of buildings with depths of above-floor inundation.

Figure 6.9 (12 sheets) shows the hydrologic capacity of the existing stormwater drainage system in the study area in AEP terms, while water surface profiles for the modelled design flood events along the main arms of Eight Mile Creek, Nine Mile Creek, Seven Mile Creek, Woolshed Creek and Sandy Creek are shown on **Figure 6.10** (3 sheets). **Figure 6.11** (8 sheets) shows the time of rise of floodwater at key locations in the road and rail network in the study area. **Table F1** in **Appendix F** sets out the peak flood level and maximum depth of inundation at the low points that are present in road and rail crossings of Eight Mile Creek, Nine Mile Creek, Seven Mile Creek, Woolshed Creek and Sandy Creek. **Table G1** in **Appendix G** sets out design peak flows and corresponding critical storm durations at key locations throughout the study area.

The sensitivity studies and discussion presented in **Section 6.7** provide guidance on suitable freeboard provisions under present day catchment and climatic conditions.

In accordance with Department of Climate Change, Energy, the Environment and Water's (DCCEEWs) flood risk management guideline FB01 entitled "*Understanding and Managing Flood Risk*", sensitivity studies have also been carried out to assess the potential impacts of future climate change on flood behaviour (refer **Section 6.8**). While increases in flood levels due to future increases in rainfall intensities may influence the selection of Interim Flood Planning Levels (IFPLs), their final selection is a matter for more detailed consideration during the preparation of the future update of the *Albury FRMS&P*.

6.3.3. Description of Flood Behaviour

The key features of Main Stream Flooding along Eight Mile Creek are as follows:

- i. Floodwater overtops the levee that runs along the western boundary of the Nexus Industrial Precinct approximately 350 m to the south of Knowles Road in flood events as frequent as 20% AEP, where it flows in an easterly direction through an existing industrial allotment at depths less than 0.1 m before discharging onto McLaurin Road.

Floodwater also commences to overtop the levee at a location approximately 130 m to the south of Knowles Road in a 5% AEP flood.

Depths of overland flow across the industrial allotments in the Nexus Industrial Precinct are generally less than 0.1 m in a 1% AEP flood, with isolated pockets where the depth of overland flow is up to 0.2 m.

- ii. **Figure 6.11** and **Table F1** in **Appendix F** show that the road and rail crossings of Eight Mile Creek commence to become inundated by floodwater as follows:
 - a. Gerogery Road (refer Peak Flood Level Location (PFL) H001) will be inundated in a flood more frequent than 20% AEP.
 - b. R W Henry Drive (refer PFL H002) will be inundated in a 5% AEP flood.
 - c. The Hume Highway (refer PFL H004) will be inundated in a 0.5% AEP flood.
 - d. The Main Southern Railway (refer PFL H003) will be inundated in a 0.2% AEP flood.
- iii. A comparison of Peak Flow Locations (PFLs) Q006 and Q007 in **Table G1** of **Appendix G** shows that for flood events greater than 10% AEP, the culverts beneath the Main Southern Railway and Hume Highway embankments attenuate the flow in the watercourse by between about 20-30%.
- iv. **Figure 6.1** shows that the Williams Road low level causeway crossing of Eight Mile Creek will be inundated to depths of up to about 2.4 m in a 20% AEP flood, while **Figure 6.10** and **Table F1** in **Appendix F** show that the Table Top Road (refer PFL H005) will be inundated in a 2% AEP flood. **Figure 6.4**, sheet 6 shows that as a result, the Spring Park residential estate is isolated in a 2% AEP flood event as the Table Top Road crossing of Nine Mile Creek (refer PFL H024) is inundated in a 10% AEP flood.
- v. **Figure 6.2**, sheet 7 shows that floodwater surcharges the left (eastern) bank of the Eight Mile Creek upstream of Brooklyn Drive where it inundates Swindon Way in the Brooklyn Fields residential estate in a 10% AEP flood. In flood events greater than or equal to 2% AEP, floodwater that surcharges the inbank area of the watercourse at this location flows in a southerly direction along the road network in the estate before discharging to Woolshed Creek in the vicinity of Newport Crescent.
- vi. **Figures 6.2**, sheet 7 and **6.9**, as well as **Table F1** in **Appendix F** show that Brooklyn Drive (refer PFL H006) will be inundated to a relatively shallow depth in a 5% AEP flood.
- vii. **Figure 6.11** and **Table F1** in **Appendix F** show that the Vaughan Road crossing of Eight Mile Creek (refer PFL H007) will be inundated in floods more frequent than 20% AEP.
- viii. **Figure 6.3**, sheet 7 shows that floodwater surcharges the right (western) bank of Eight Mile Creek and inundates Diamond Drive immediately north of its intersection with Thurgoona Drive in a 5% AEP flood event.
- ix. **Figure 6.11** and **Table F1** in **Appendix F** show that Thurgoona Drive (refer PFL H008) will be inundated in a 2% AEP flood in the vicinity of its intersection with Evesham Place.
- x. The detailed ground survey of the 8.2 m wide by 3 m high ellipsoidal culvert beneath the Riverina Highway indicates that there is a blockage/raised bed elevation in the creek downstream of the highway which results in the backfilling of the culvert with standing water.
- xi. **Figure 6.11** and **Table F1** in **Appendix F** show that the Riverina Highway (refer PFL H011 and H013) will be inundated in a flood as frequent as 20% AEP.

- xii. **Table G1** of **Appendix G** shows that the inbank area of Eight Mile Creek where it runs in a westerly direction downstream of the Riverina Highway (refer PFL Q027b) has a capacity of about 5.6 m³/s. Once its capacity is exceeded, the majority of the total flow in the creek (refer PFL Q026) is conveyed on its left (eastern) overbank area (refer PFL Q027c) where it flows in a southerly direction through rural paddocks and discharges to the Murray River in the vicinity of the southern end of Hawkscote Road.
- xiii. **Table G1 4** in **Appendix G** shows that the peak flow in a PMF event is up to six times the corresponding peak 1% AEP flow in the upper reaches of the catchment, increasing to about 14 times in its lower reaches.
- xiv. **Figure 6.10**, sheet 1 shows that peak flood levels along Eight Mile Creek in a PMF event are generally 2-3 m higher than the corresponding peak 1% AEP flood levels, except downstream of the Riverina Highway where the PMF level is only about 0.3 m higher than the corresponding peak 1% AEP flood level.
- xv. **Figure 6.8** shows that the PMF will impact development that is located adjacent to Eight Mile Creek as follows:
 - a. The Nexus Industrial Precinct (refer sheet 3) will be inundated to depth of up to 1 m.
 - b. Existing residential allotments in the Spring Park residential estate (refer sheet 6) will be inundated to depths of up to 2.6 m.
 - c. Existing residential allotments in the Brooklyn Fields residential estate (refer sheet 5) will be inundated to depths of up to 2.4 m.
 - d. Existing commercial allotments that are located on Diamond Drive (refer sheet 5) will be inundated to depths of up to 2.8 m.
 - e. Existing residential development that is located on the eastern side of the Huntingdale, St Johns Creek, Kensington Gardens and Hillford Estate residential estates (sheet 9) will be inundated to depths of up to 2.3 m.

The key features of Main Stream Flooding along Nine Mile Creek are as follows:

- i. **Figure 6.11** and **Table F1** in **Appendix F** show that the road and rail crossings of Nine Mile Creek commence to become inundated by floodwater as follows:
 - a. Gerogery Road (refer PFL H020) and Lakeview Road (refer PFL H021) will be inundated in floods as frequent as 20% AEP.
 - b. Tabletop Road (refer PFL H024) will be inundated in a 10% AEP flood.
 - c. Maryvale Station Road (refer PFL H023) will be inundated in a 1% AEP flood.
 - d. The Main Southern Railway (refer PFL H022a) and Hume Highway (refer PFL H022b) will remain flood free up until the PMF.
- ii. **Figure 6.1**, sheet 4 shows that floodwater surcharges the Western Diversion Line and discharges to the Ettamogah Forest Dam in flood events as frequent as 20% AEP.
- iii. The Ettamogah Forest Dam acts to attenuate flood flows, limiting peak flows in the watercourse downstream of its location to a maximum of about 30 m³/s in a 0.2% AEP flood event (refer PFL Q051 in **Table G1** of **Appendix G**).
- iv. **Table G1** of **Appendix G** shows that the peak flow over the Ettamogah Forest Dam spillway (refer PFL Q062) in a PMF is about 114 m³/s, which is about 36 m³/s lower than the corresponding peak flow set out in the *Lake Ettamogah Winter Storage Dam – Design Review* (URS, 2014) (refer **Section B1.6.2** for further discussion).

The key features of Main Stream Flooding along Seven Mile Creek are as follows:

- i. **Figure 6.11** and **Table F1** in **Appendix F** show that the road and rail crossings of Seven Mile Creek commence to become inundated by floodwater as follows:
 - a. While the Wagga Road crossing (refer PFLH H032) will remain flood free up until the PMF, floodwater inundates the low point in the road that is located approximately 140 m to the north in a 20% AEP flood.
 - b. Wignell Road (refer PFLH H035) will be inundated in a 10% AEP flood.
 - c. Tabletop Road (refer PFLH H036) will be inundated in a 5% AEP flood.
 - d. Gerogery Road (refer PFLH H031) will be inundated in a 1% AEP flood.
 - e. The Main Southern Railway (refer PFLH H033) and Hume Highway will remain flood free in a PMF.
- ii. **Table G1 4** in **Appendix G** shows that the peak flow in a PMF event is about 10 times the corresponding peak 1% AEP flow.

The key features of Main Stream Flooding along Woolshed Creek are as follows:

- i. **Figure 6.11** and **Table F1** in **Appendix F** show that the Wedgetail Way (refer PFLH H048), Kywanna Road (refer PFLH H049) and Kerr Road (refer PFLH H050) crossings of Woolshed Creek will be inundated in a 20% AEP flood.
- ii. **Figure 6.3**, sheets 7 and 8 show that floodwater surcharges the left (southern) bank of Woolshed Creek and flows through existing residential allotments in the Thurgoona Park residential estate in a 5% AEP flood.
- iii. **Figure 6.5**, sheet 7 shows that floodwater surcharges the right (northern) bank and inundates Birchgrove Crescent in the Brooklyn Fields residential estate in a 1% AEP flood.
- iv. **Table G1 4** in **Appendix G** shows that the peak flow in a PMF event is about 10 times the corresponding peak 1% AEP flow.
- v. **Figure 6.8** shows that the PMF will impact development that is located adjacent to Woolshed Creek at the following location:
 - a. The Thurgoona Park residential estate (refer sheets 7 and 8) where low lying allotments will be inundated to depths of up to 2.5 m.
 - b. In the Brooklyn Fields residential estate, where floodwater originating from Woolshed Creek combines with floodwater that has surcharged Eight Mile Creek, inundates existing residential development to depths of up to 2.4 m.
- vi. **Figure 6.8**, sheet 8 shows that floodwater in a tributary of Woolshed Creek that runs in a northerly direction between Thurgoona Drive and Kerr Road surcharges a natural saddle that is located on the eastern side of the Yarrabee Park residential estate, where it then flows in a north-westerly direction through the Springview residential estate, inundating existing residential allotments to depths of up to 0.8 m.

The key features of Main Stream Flooding along Sandy Creek are as follows:

- i. Floodwater is generally contained within the banks of Sandy Creek for flood events up to 0.2% AEP in magnitude.
- ii. **Figure 6.11** and **Table F1** in **Appendix F** show that the road and rail crossings of Sandy Creek commence to become inundated by floodwater as follows:
 - a. Knoble Road (refer PFLH H057) will be inundated in a 5% AEP flood.

- b. The Riverina Highway Road (refer PFL H058) will be inundated in a 0.5% AEP flood.
- iii. **Table G1 4 in Appendix G** shows that the peak flow in a PMF event is about 10 times the corresponding peak 1% AEP flow.

The key features of Major Overland Flow in the urbanised parts of the study area are as follows:

- i. **Figure 6.9** (12 sheets) shows that the piped drainage system generally has a capacity of less than 20% AEP.
- ii. **Figure 6.5** shows that floodwater surcharges the piped drainage system and inundates existing development to a depth of greater than 0.3 m in a 1% AEP flood event at the following locations:
 - a. between Strand Terrace and Wagga Road to the south of Coventry Street in the Ettamogah Rise residential estate (refer sheet 5);
 - a. on the western side of Fenchurch Drive to the north of its intersection with Regent Street in the Ettamogah Rise residential estate (refer sheet 5);
 - b. on the western side of Fenchurch Drive to the north of its intersection with Whitehall Avenue in the Ettamogah Rise residential estate (refer sheet 5);
 - c. along the southern side of a drainage easement that runs between Fenchurch Drive and Whitehall Avenue on the northern side of Leicester Street in the Ettamogah Rise residential estate (refer sheet 5);
 - d. on the western side of Newton Circuit to the south of its intersection with Norman Way in the Fairway Gardens residential estate (refer sheet 7);
 - e. on the western side of Pickworth Street to the south of its intersection with Whitton Drive in the Fairway Gardens residential estate (refer sheet 7)
 - f. in the rear of the allotments that are located between Feathertop Circuit and Table Top Road in the St Johns Hill residential estate (refer sheet 9); and
 - g. on the northern side of Tweed Court at its intersection with Nordic Street in the Tweed Court residential estate (refer sheet 10).

Table 6.2 sets out the peak flood level in the 43 existing detention basins for the range of assessed design flood events, as well as the depth of flow over the lowest point of the basin embankment. In general, the basins surcharge in flood events of between 20-10% AEP and therefore it is unlikely that they reduce the post-development peak flow from the upstream development back to pre-development conditions.

It is noted that the details of the outlet structures are generally based on Council's stormwater asset database with inverts assumed based on the available LiDAR data. It is recommended that Council undertake detailed survey of each basin to determine the arrangement and dimensions of the drainage structures associated with each basin (internal drainage, as well as outlet structure), as well as the elevation of the spillways. These details should then be used to undertake a detailed assessment of the performance of the existing detention basins in the study catchments. It is recommended that Council review and updates its engineering standards to ensure that water cycle management type measures are being designed to meet contemporary standards.

TABLE 6.2
PERFORMANCE OF EXISTING DETENTION BASINS

Basin ID ⁽¹⁾	Basin Name	Spillway Elevation (m AHD)	Crest Elevation (m AHD)	Design Storm Event															
				20% AEP		10% AEP		5% AEP		2% AEP		1% AEP		0.5% AEP		0.2% AEP		PMF	
				Peak Flood Level (m AHD)	Depth of Flow Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Spillway (m)
B01	Visy Detention Basin No. 1			Not assessed as part of present study															
B02	Visy Detention Basin No. 2																		
B03	Strand Terrace Basin No. 1	219.48	219.72	219.39	-	219.82	0.34	219.92	0.44	219.95	0.47	219.97	0.49	219.99	0.51	220.01	0.53	220.30	0.82
B04	Trafalgar Street Basin	247.79	248.03	246.32	-	246.75	-	247.24	-	247.52	-	247.83	0.04	247.97	0.18	248.07	0.28	248.21	0.42
B05	Strand Terrace Basin No. 2	218.69	219.29	218.94	0.25	219.08	0.39	219.14	0.45	219.20	0.51	219.23	0.54	219.27	0.58	219.32	0.63	219.79	1.10
B06	Whitehall Avenue Basin No. 1	216.66	217.11	216.96	0.30	217.08	0.42	217.15	0.49	217.20	0.54	217.26	0.60	217.29	0.63	217.34	0.68	217.67	1.01
B07	Whitehall Avenue Basin No. 2	208.72	209.49	209.16	0.44	209.24	0.52	209.32	0.60	209.38	0.66	209.46	0.74	209.52	0.80	209.61	0.89	210.19	1.47
B08	Marlborough Street Basin	208.24	208.51	208.30	0.06	208.54	0.30	208.59	0.35	208.62	0.38	208.65	0.41	208.68	0.44	208.71	0.47	209.26	1.02
B09	Brooklyn Drive Basin	Not assessed as part of present study																	
B10	Huntington Grove Basin	185.14	185.43	185.06	-	185.17	0.03	185.22	0.08	185.25	0.11	185.26	0.12	185.28	0.14	185.30	0.16	186.59	1.45
B11	Swansea Court Basin	-(2)	181.81	181.65	-	181.68	-	181.76	-	181.85	0.04	181.91	0.10	181.94	0.13	181.99	0.18	184.38	2.57
B12	Cardiff Drive Basin	-(2)	181.42	180.91	-	181.02	-	181.06	-	181.12	-	181.15	-	181.20	-	181.27	-	182.91	1.49
B13	Newport Crescent Basin No. 1	-(2)	176.43	176.60	0.17	176.85	0.42	177.03	0.60	177.29	0.86	177.48	1.05	177.57	1.14	177.69	1.26	180.66	4.23
B14	Newport Crescent Basin No. 2	177.94	178.37	178.13	0.19	178.15	0.21	178.34	0.40	178.69	0.75	178.84	0.90	178.92	0.98	179.00	1.06	181.40	3.46
B15	Whitebox Circuit Basin	192.54	192.73	192.75	0.21	192.79	0.25	192.83	0.29	192.86	0.32	192.88	0.34	192.89	0.35	192.91	0.37	193.26	0.72
B16	Springview Estate Basin	Not assessed as part of present study																	
B17	Longacre Road Basin	193.47	193.71	193.66	0.19	193.73	0.26	193.76	0.29	193.78	0.31	193.79	0.32	193.80	0.33	193.81	0.34	194.16	0.69
B18	Yulumbang Road Basin	-(2)	196.74	195.93	-	196.10	-	196.13	-	196.14	-	196.14	-	196.15	-	196.17	-	196.33	-
B19	Hartigan Street Basin	188.9	189.07	189.14	0.24	189.17	0.27	189.20	0.30	189.22	0.32	189.24	0.34	189.25	0.35	189.27	0.37	189.60	0.70
B20	Fairway Gardens Road Basin	188.9	189.1	188.95	0.05	189.00	0.10	189.02	0.12	189.02	0.12	189.03	0.13	189.03	0.13	189.04	0.14	189.17	0.27
B21	Diamond Drive Basin	-(2)	175.22	174.12	-	174.51	-	176.41	1.19	176.73	1.51	176.86	1.64	176.91	1.69	176.98	1.76	179.06	3.84
B22	Forest Drive Basin	-(2)	204.49	202.42	-	202.47	-	202.50	-	202.56	-	202.60	-	202.63	-	203.21	-	203.26	-
B23	Corriedale Court Basin	-(2)	183.55	197.68	14.13	197.72	14.17	197.87	14.32	197.91	14.36	197.96	14.41	197.98	14.43	198.04	14.49	198.24	14.69
B24	Beaumont Park Basin	201.13	201.53	201.32	0.19	201.36	0.23	201.40	0.27	201.42	0.29	201.43	0.30	201.44	0.31	201.47	0.34	201.67	0.54
B25	Guy Place Basin	195.63	196.03	195.94	0.31	196.04	0.41	196.08	0.45	196.08	0.45	196.08	0.45	196.09	0.46	196.10	0.47	196.19	0.56
B26	The Grove Basin	172.94	173.12	173.34	0.40	173.48	0.54	173.56	0.62	173.72	0.78	173.82	0.88	173.89	0.95	173.96	1.02	176.07	3.13
B27	Stirling Way Basin	-(2)	192.8	192.81	0.01	192.90	0.10	192.97	0.17	193.03	0.23	193.06	0.26	193.08	0.28	193.11	0.31	193.38	0.58

Refer over for footnotes to table.

TABLE 6.2 (Cont'd)
PERFORMANCE OF EXISTING DETENTION BASINS

Basin ID ⁽¹⁾	Basin Name	Spillway Elevation (m AHD)	Crest Elevation (m AHD)	Design Storm Event															
				20% AEP		10% AEP		5% AEP		2% AEP		1% AEP		0.5% AEP		0.2% AEP		PMF	
				Peak Flood Level (m AHD)	Depth of Flow Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Spillway (m)
B28	Alpine Street Basin	187.09	187.45	187.54	0.45	187.57	0.48	187.60	0.51	187.63	0.54	187.66	0.57	187.68	0.59	187.71	0.62	188.14	1.05
B29	Peninsular Court Basin	-(2)	171.17	172.66	1.49	172.78	1.61	172.86	1.69	173.02	1.85	173.13	1.96	173.20	2.03	173.27	2.10	175.54	4.37
B30	Pickworth Street Basin	184.94	185.33	184.83	-	185.06	0.12	185.10	0.16	185.12	0.18	185.13	0.19	185.14	0.20	185.15	0.21	185.50	0.56
B31	Kerrford Drive Basin	185.49	186.69	186.57	1.08	186.62	1.13	186.66	1.17	186.68	1.19	186.74	1.25	186.77	1.28	186.80	1.31	187.18	1.69
B32	Shearer Court Basin	-(2)	183.23	183.37	0.14	183.46	0.23	183.50	0.27	183.53	0.30	183.55	0.32	183.55	0.32	183.56	0.33	183.74	0.51
B33	Woolshed Drive Basin No 1	-(2)	177.63	177.86	0.23	177.89	0.26	177.93	0.30	177.95	0.32	177.98	0.35	177.99	0.36	178.02	0.39	178.22	0.59
B34	Woolshed Drive Basin No. 2	177.61	178.03	177.61	0	177.62	0.01	177.67	0.06	177.68	0.07	177.70	0.09	177.71	0.10	177.75	0.14	178.21	0.60
B35	Kosciuszko Road Basin	186.41	186.92	186.91	0.50	186.92	0.51	186.93	0.52	186.93	0.52	186.94	0.53	186.95	0.54	186.96	0.55	187.13	0.72
B36	Zeil Circuit Basin	189.83	190.86	190.04	0.21	190.06	0.23	190.08	0.25	190.09	0.26	190.10	0.27	190.10	0.27	190.11	0.28	190.36	0.53
B37	Prestwick Street Basin	168.49	169.03	169.30	0.81	169.42	0.93	169.51	1.02	169.67	1.18	169.79	1.30	169.85	1.36	169.93	1.44	172.20	3.71
B38	Hillford Circuit Basin	-(2)	167.79	168.68	0.89	168.78	0.99	168.84	1.05	168.97	1.18	169.06	1.27	169.11	1.32	169.17	1.38	171.15	3.36
B39	Rodeo Drive Basin	183.62	183.91	183.47	-	183.63	0.01	183.85	0.23	183.89	0.27	183.92	0.30	183.93	0.31	183.95	0.33	184.03	0.41
B40	Kowari Street Basin	182.71	182.89	182.49	-	182.70	-	182.89	0.18	182.95	0.24	182.96	0.25	182.96	0.25	183.03	0.32	183.21	0.50
B41	Richmond Drive Basin	181.45	181.71	181.58	0.13	181.62	0.17	181.64	0.19	181.67	0.22	181.71	0.26	181.72	0.27	181.74	0.29	181.91	0.46
B42	Mulgara Drive Basin	-(2)	182.55	182.45	-	182.47	-	182.59	0.04	182.62	0.07	182.67	0.12	182.72	0.17	183.11	0.56	183.22	0.67
B43	Birdwood Lane Basin	184.05	184.42	184.05	0	184.21	0.16	184.28	0.23	184.34	0.29	184.40	0.35	184.43	0.38	184.48	0.43	184.74	0.69
B43	Birdwood Lane Basin	184.05	184.42	184.05	0	184.21	0.16	184.28	0.23	184.34	0.29	184.40	0.35	184.43	0.38	184.48	0.43	184.74	0.69

1. Refer **Figure 2.2** for plan location of existing detention basins.

2. Detention basin does not have a defined spillway.

6.3.4. Comparison with Previous Studies

Table 6.3 over the page shows a comparison of the design peak 1% AEP flows derived as part of the present study with those derived as part of URS, 2012 and WMAwater, 2016. It was found that the peak flows derived as part of the present study generally match those that were derived as part of URS, 2012 and WMAwater, 2016, for the reason that the SSP2-4.5 Year 2030 IFD Data is comparable to ARR 1987 IFD for the same storm duration.

The exception to the above finding is in the upper reaches of Eight Mile Creek (refer PFL Q005) and Woolshed Creek (refer PFL Q095) catchments where the peak flows derived as part of the present study are up to three and two times higher than the earlier studies, respectively. The reasons for the difference are:

- a) the early studies relied on larger lumped catchments with vectored average slopes to derive runoff from the very steep sub-catchments where the present study adopted smaller sub-catchments with generally steeper grades; and
- b) the earlier studies simply ran a 6 hour storm event which was considered the critical duration along the modelled watercourses, whereas the present study assessed storm durations ranging from 0.5 hours to 12 hours, noting that **Table G1** in **Appendix G** shows that storms durations of between 0.5-1 hour are critical for maximising peak flows in the headwaters of the two catchments.

The peak flows derived as part of the present study were also found to be significantly higher than those derived as part of the *Nexus Industrial Precinct Flooding Investigation* (WMS, 2023) in the upper reaches of Eight Mile Creek (refer PFL Q005), noting that:

- a) the WMS WBNM Model was tuned to reproduce the flows derived as part of WMAwater, 2016; and
- b) WMS, 2023 also found that the 6 hour storm was critical for maximising peak flows in the upslope catchment.

The reasons for the difference between WMS, 2023 and the present study are:

- a) The WBNM hydrologic modelling package does not require catchment slope as an input, instead using a catchment routing parameter to simulate catchment storage. As such, the WMS WBNM Model does not take into account the steep nature of the catchment upstream of the Nexus Industrial Precinct.
- b) The burst initial loss value incorporated in the WMS WBNM Model (i.e. 20.8 mm) was derived using the median pre-burst rainfall depths contained on the *ARR Data Hub* and is much higher than the value that was adopted for the present study which was derived from the Probability Neutral Burst Initial Loss Values (i.e. equivalent value of 6.2 mm) that are also contained on the *ARR Data Hub*. The lower loss values that were relied upon for the present study are considered to be more suitable as they are based on the values recommended in the *Floodplain Risk Management Guide - Incorporating 2016 Australian Rainfall and Runoff in studies* (OEH, 2019).

It was not possible to undertake a comparison of the peak flood levels derived as part of the present study with those derived as part of WMAwater, 2016 as the flood level data that is available from the earlier study is limited to water surface elevation contours at 3 m intervals.

TABLE 6.3
COMPARISON OF DESIGN PEAK 1% AEP FLOWS
DERIVED AS PART OF PREVIOUS INVESTIGATIONS

Peak Flow Location ID ⁽¹⁾	Watercourse	Location	Peak Flow			Difference ⁽³⁾	
			URS, 2012 ⁽²⁾	WMAwater, 2016	Present Study	URS, 2012	WMAwater, 2016
Q005	Eight Mile Creek	R W Henry Drive	55.3	55.6	115	59.7	59.4
Q007		Hume Highway	N/A	59.8	59.8	-(4)	0
Q012		Upstream Williams Road	74.1	74.4	72.5	-1.6	-1.9
Q014		Table Top Road	-(4)	77.3	73.6	-(4)	-3.7
Q020		Thurgoona Drive	-(4)	165.9	171	-(4)	5.1
Q025		Riverina Highway	-(4)	172.4	171	-(4)	-1.4
Q028a		Upstream Confluence with Murray River	170	169.3	170	0	0.7
-	Nine Mile Creek	Lakeview Drive	-(4)	3.1	4.9	-(4)	1.8
-		Hume Highway	-(4)	6.6	7.5	-(4)	0.9
Q051		Downstream Ettamogah Dam	29.3	31.1	25.1	-4.2	-6
Q052		Table Top Road	-(4)	39.3	34.6	-(4)	-4.7
Q053		Downstream Table Top Road	46.4	47.6	50.3	3.9	2.7
Q057	Tributary of Nine Mile Creek	Hume Highway	-(4)	6.2	6.7	-(4)	0.5
Q070			6.3	7.4	10.9	4.6	3.5
Q076	Seven Mile Creek	Wignell Road	13.2	13.5	14.3	1.1	0.8
Q079		Table Top Road	-(4)	19.1	16	-(4)	-3.1
Q087	Six Mile Creek		-(4)	24.1	14.2	-(4)	-9.9
Q095	Woolshed Creek		11.1	11.1	27.2	16.1	16.1
Q105	Tributary of Woolshed Creek		15.8	15.8	19.3	3.5	3.5
Q120	Tributary of Eight Mile Creek		6.5	6.7	11.5	5	4.8

1. Refer **Figures 6.1 to 6.8** (12 sheets each) for location of peak flow comparison.
2. Taken from WMAwater, 2016.
3. A positive value indicates that the design peak flows derived as part of the present study are higher, and conversely a negative value indicates they are lower than those derived as part of the previous studies.
4. Peak flow data not available.

Table 6.4 shows a comparison of the pre-development peak 1% AEP flows at the location of the 30 wetlands/retarding basins that were proposed as part of Spiire, 2020, with the flows derived as part of the present study. The Rational Method that is prescribed in the 1987 edition of *Australian Rainfall and Runoff* (Institute of Engineers Australia, 1987) was used to derive pre-development flows at the proposed sites as part of Spiire, 2020 based on the shape and size of the proposed post-development sub-catchments, noting that they are not always representative of the shape and size pre-development sub-catchments at the same location. It is noted that Spiire, 2020 states that “further catchment analysis be undertaken during the future development approval process to refine flow calculations”.

TABLE 6.4
COMPARISON OF PRE-DEVELOPMENT PEAK 1% AEP FLOWS
AT LOCATION OF BASINS RECOMMENDED AS PART OF SPIIRE, 2020

Basin ID ⁽¹⁾	Discharge (m ³ /s)		Difference ^(2,3) (m ³ /s)	Basin ID ⁽¹⁾	Discharge (m ³ /s)		Difference ⁽²⁾ (m ³ /s)
	Spiire, 2020	Present Study			Spiire, 2020	Present Study	
1	2.8	3.8	1.0 [36]	12	1.5	Model not able to derive discharge at a comparable location	
2.1	3.2	Model not able to derive discharge at a comparable location		13	4.2		
2.2	3.7			14	0.9		
3	3.4	5.6	2.2 [65]	15	5.0	18.6	13.6 [272]
4	5.2	9.3	4.1 [79]	16	5.2	6.8	1.6 [31]
5	4.3	5.6	1.3 [30]	17	4.5	10	5.5 [122]
6	5.3	13.7	8.4 [158]	18	5.0	7.6	2.6 [52]
7.1	7.8	19.3	11.5 [147]	19	4.1	7.6	3.5 [85]
7.2	6.0	15.8	9.8 [163]	20	4.8	Model not able to derive discharge at a comparable location	
7.3	2.9	10.2	7.3 [252]	21	4.1		
8	3.7	5.1	1.4 [38]	22	2.2	1.9	-0.3 [-14]
9.1	4.7	13.7	9 [191]	23	1.5	12.3	10.8 [720]
9.2	3.7	11.1	7.4 [200]	24	1.6		
10	4.3	Model not able to derive discharge at a comparable location		25	3.9	11.8	7.9 [203]
11	1.4			26	5.0	11.5	6.5 [130]

1. Refer **Annexure B1** of **Appendix B** for plan location of proposed basins.
2. A positive value indicates that the design peak flows derived as part of the present study are higher, and conversely a negative value indicates they are lower than those derived as part of Spiire, 2020.
3. Values in [] are percentage increases in peak flow when compared to those set out in Spiire, 2020.

In general, the peak 1% AEP flows that were derived using the Eight Mile and Sandy Creek TUFLOW Model are significantly higher than those that were derived as part of Spiire, 2020. This is principally due to the different modelling technique that was adopted as part of the present study (i.e. rainfall-runoff/hydrodynamic modelling versus simplistic Rational Method). Adoption of the lower pre-development flows as derived as part of Spiire, 2020 could mean that the size of the detention basins that are required to attenuate peak flows to pre-development conditions may be overstated in the report.

The results of the present study show that the patterns of overland flow in the area that is proposed for redevelopment are complex in nature due to the undulating topography which can generate shallow sheet flow over wide flow paths and the redistribution of flow between the indistinct drainage lines. Based on the above findings, it is recommended that the Eight Mile and Sandy Creek TUFLOW Model be used as the basis for assessing the flood mitigation requirements for future subdivision development within the study catchments.

6.4 Economic Impacts of Flooding

Table 6.5 sets out the number of properties that are subject to flooding in the Urban Centre and the estimated flood damages that would be occurred for storms of varying intensity.

TABLE 6.5
SUMMARY OF FLOOD DAMAGES

Design Flood Event (% AEP)	Number of Properties						Total Damage (\$ Million)
	Residential		Commercial/ Industrial		Public		
	Flood Affected	Flooded Above Floor Level	Flood Affected	Flooded Above Floor Level	Flood Affected	Flooded Above Floor Level	
20	10	0	0	0	0	0	0.0
10	21	1	0	0	1	0	0.1
5	31	5	0	0	1	0	0.7
2	52	15	0	0	1	1	1.3
1	81	27	0	0	1	1	3.0
0.5	95	37	1	1	1	1	4.9
0.2	126	52	2	1	3	2	7.5
PMF	1,059	769	11	8	11	11	191.3

A storm event more frequent than 10% AEP is the threshold at which tangible flood damages commence to occur in the study area, with a single dwelling subject to above-floor inundation in a 10% AEP storm event. **Table 6.5** shows that a total of 27 dwellings and one public building would experience above-floor inundation at the 1% AEP level of flooding, resulting in total damages of about \$3 Million.

During a PMF event, 769 individual dwellings, eight commercial buildings and 11 public buildings would experience above-floor inundation in the study area, resulting in total flood damages of about \$191 Million.

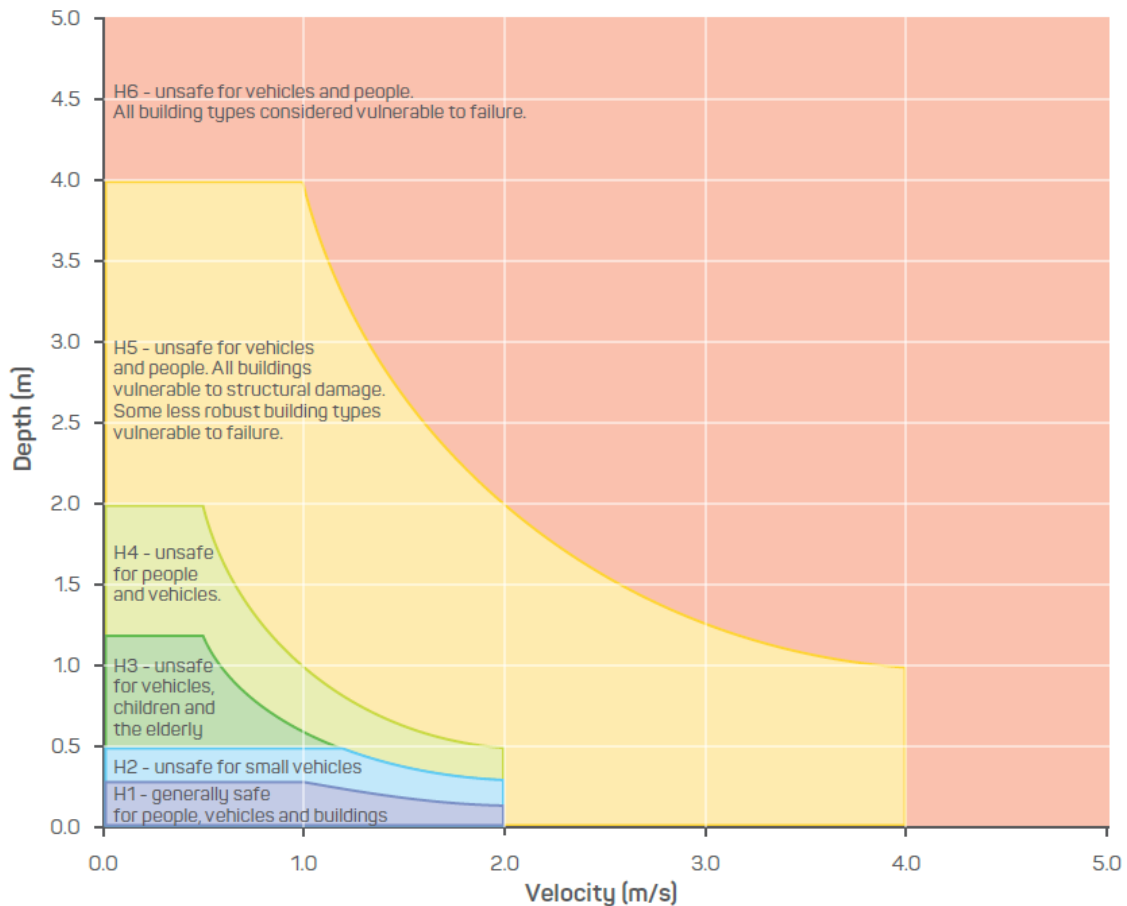
For a discount rate of 5% pa and an economic life of 30 years, the *Net Present Worth* of damages for all flood events up to the 1% AEP is \$1.7 Million. Therefore, one or more schemes costing up to this amount could be economically justified if they eliminated damages across the whole of the study area for all storm events up to 1% AEP. While schemes costing more than this value would have a benefit/cost ratio less than 1, they may still be justified according to a multi-objective approach which considers other criteria in addition to economic feasibility.

Appendix H of this report contains further details on the economic assessment that was undertaken as part of the present study.

6.5 Flood Hazard and Flood Function Zones

6.5.1. Flood Hazard Vulnerability Classification

Flood hazard categories may be assigned to flood affected areas in accordance with the definitions set out in ARR 2019. Flood prone areas may be classified into six hazard categories based on the depth of inundation and flow velocity that relate to the vulnerability of the community when interacting with floodwater as shown in the illustration below which has been taken from ARR 2019.



Flood Hazard Vulnerability Classification diagrams for the 5% and 1% AEP flood events, as well as the PMF based on the procedures set out in ARR 2019 are presented on **Figures 6.12 to 6.14**.

It was found that for floods up to 1% AEP in magnitude, areas that are classified as H5 and H6 are generally confined to the inbank area of both Eight Mile Creek and Sandy Creek, including their major tributaries, as well as in the vicinity of local farm dams, while the urbanised parts of the study catchments are generally classified as H1 and H2.

While the Ettamogah Forest Dam is shown to be subject to H4 type hazard conditions in a 5% and 1% AEP flood event, H6 type hazard conditions would be present due to the depth of ponding behind the dam wall, noting that only partial survey of the impoundment was available as part of the present study.

The extent of land subject to H5 and H6 type hazard conditions in a PMF increases significantly, while the hazard category in the urbanised parts of the study catchments increases to H4 during a flood of this magnitude.

6.5.2. Flood Function

According to the *FRMM*, the floodplain may be subdivided into the following three hydraulic categories:

- Floodways;
- Flood storage; and
- Flood fringe.

Floodways are those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with obvious naturally defined channels. Floodways are the areas that, even if only partially blocked, would cause a significant re-distribution of flow, or a significant increase in flood level which may in turn adversely affect other areas. They are often, but not necessarily, areas with deeper flow or areas where higher velocities occur.

Flood storage areas are those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. If the capacity of a flood storage area is substantially reduced by, for example, the construction of levees or by landfill, flood levels in nearby areas may rise and the peak discharge downstream may be increased. Substantial reduction of the capacity of a flood storage area can also cause a significant redistribution of flood flows.

Flood fringe is the remaining area of land affected by flooding, after floodway and flood storage areas have been defined. Development in flood fringe areas would not have any significant effect on the pattern of flood flows and/or flood levels.

Flood Risk Management Guideline FB02 Flood Function offers guidance in relation to two alternative procedures for identifying the portion of the floodplain that functions as floodways, flood storage and flood fringe areas.

The indicator technique set out in *Howells et al, 2003* was used to identify the preliminary extent of the floodway based on flow velocity and depth. Based on the findings of a trial and error process, the following criteria were adopted for identifying those areas which operate as a “floodway” in a 1% AEP event:

- Velocity x Depth greater than 0.25 m²/s **and** Velocity greater than 0.25 m/s; or
- Velocity greater than 1 m/s.

Manual assessment and cleaning of the raw model output data was then undertaken as recommended in *Flood Risk Management Guideline FB02 Flood Function*.

Flood storage areas are identified as those areas which do not operate as floodways in a 1% AEP flood event, but where the depth of inundation exceeds 300 mm. The remainder of the flood affected area was classified as flood fringe.

Figures 6.15 to 6.17 (12 sheets each) show the division of the floodplain into floodway, flood storage and flood fringe areas for the 5% and 1% AEP storm events, as well as the PMF.

Figure 6.16 shows that floodways are generally present along the alignment of both Eight Mile Creek and Sandy Creek, including their major tributaries, as well as the main flow paths in the rural parts of the study area in a 1% AEP flood. Floodways are also present in the rural properties that are located on the left (eastern) bank of Eight Mile Creek downstream of the Riverina Highway.

While the floodways are generally contained within the drainage and road reserve boundaries in the urbanised parts of the catchment in a 1% AEP flood, they were found to be present at the following locations within existing residential estates:

- on the western side of Fenchurch Drive to the north of its intersection with Whitehall Avenue in the Ettamogah Rise residential estate (refer sheet 5);
- along the southern side of a drainage easement that runs between Fenchurch Drive and Whitehall Avenue on the northern side of Leicester Street in the Ettamogah Rise residential estate (refer sheet 5)
- in the rear of several large lot residential allotments that are located in the Thurgoona Park residential estate and back onto Woolshed Creek (refer sheet 7);
- along the tributary of Woolshed Creek that runs between the intersection of Kerr Road and Hopewood Road in the Thurgoona Park residential estate (refer sheet 8);
- on the northern side of Tweed Court at its intersection with Nordic Street in the Tweed Court residential estate (refer sheet 10).

Flood storage areas are confined to the major ponding areas that are located on the upstream side of the roads and railways, as well as in the local farm dams that have been constructed to capture surface runoff in different parts of the study catchments.

6.6 Flood Emergency Response Classification of Communities

Figures 6.18, 6.19 and 6.20 show the flood emergency response classifications for the 5% and 1% AEP storm events, as well as the PMF, based on the definitions set out in *Flood Risk Management Guideline EM01 Support for Emergency Management Planning*. The flood emergency response classifications have only been defined in the immediate environs of the urbanised parts of the study area.

The steps involved in assessing flood emergency response classifications are briefly as follows:

- i. Define the potential vehicular evacuation routes based on roads that are identified as 'Motorway', 'Primary', 'Arterial', 'Sub-Arterial' and 'Local Roads' in the SixMaps 'road segment' database.
- ii. Identify where the potential vehicular evacuation routes that cross the main arms of Eight Mile Creek, Nine Mile Creek, Seven Mile Creek, Woolshed Creek and Sandy Creek become impassable as they are subject to H2 to H6 type flooding for the assessed AEP.¹⁵
- iii. Identify the allotments that are subject to depths of overland flow greater than 100 mm for the assessed AEP.
- iv. Classify the allotments identified as part of step iii. based on the definitions set out in *Flood Risk Management Guideline EM01 Support for Emergency Management Planning*

The majority of the study area is defined as 'Rising Road Access' in floods up to 1% AEP in magnitude, with isolated areas classified as 'Low Flood Island' and 'High Flood Island' in the following areas:

¹⁵ Vehicular evacuation routes that are subject to Major Overland Flow which is classified as H2 flood hazard vulnerability type conditions or greater were not included in the assessment as the short duration nature of the flooding is not considered to represent a major constraint from a flood evacuation perspective.

- Residential allotments that are located on Swindon Way in the Brooklyn Fields residential estate are located on a high flood island in a 5% AEP flood event;
- The Brooklyn Fields residential estate is presently considered to be a high flood island in a 1% AEP flood event, noting that it is understood that consideration is currently being given to provide rising road access to the north; and
- The Spring Park residential estate is a high flood island in a 1% AEP flood event.

The illustrations below are taken from *Flood Risk Management Guideline EM01 Support for Emergency Management Planning* demonstrating the above classifications that are applicable to the study area.

High flood island. The flood island has land higher than the limit of flooding for the event being considered (Figure 7 shows a high flood island in the PMF). During a flood these high islands are isolated from other areas of the community by floodwater, terrain, development, or infrastructure. However, there is an opportunity for people to retreat to higher ground within the island, and therefore, the direct risk to life is reduced. The area may require resupply by boat or air if not evacuated before the road is cut. If it is not possible to provide adequate support (such as community and medical facilities) during the period of isolation, evacuation will have to take place before isolation occurs. Isolation without these services is more likely to result in fatal decisions to cross floodwaters.

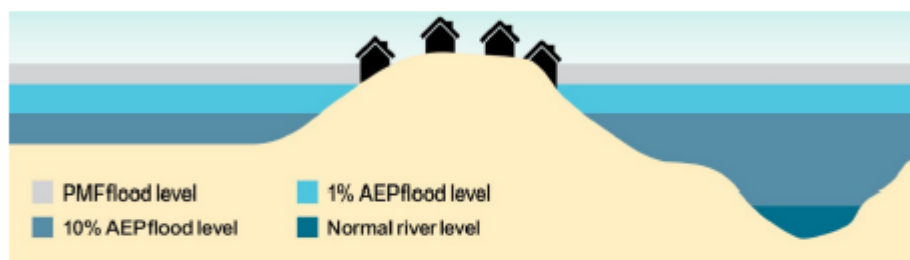


Figure 7 High flood island

Low flood island. The flood island is lower than the limit of flooding for the event being considered (Figure 8 and Figure 9 show a low flood island in the PMF). During a flood event the area initially becomes isolated by floodwater, terrain, development or infrastructure. If floodwater continues to rise after it is isolated, the land on the island will eventually be completely inundated by floodwaters. Evacuation of the community will be required prior to evacuation routes being closed as people left stranded on the island may drown.

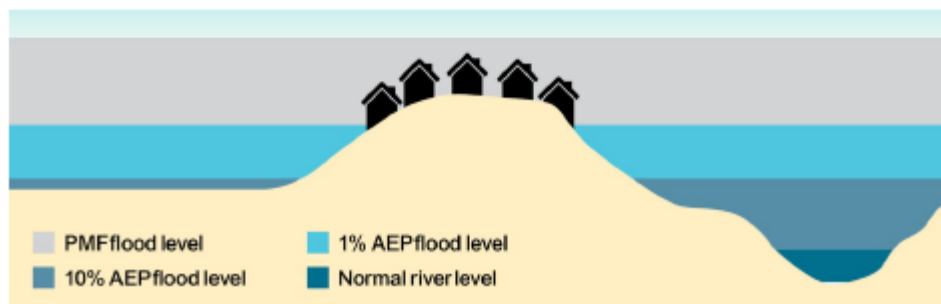


Figure 8 Low flood island

Areas with rising road access are those areas where access roads rise steadily uphill and away from the rising floodwaters (Figure 12 and Figure 13). The community will not be completely isolated before inundation reaches its maximum extent, even in the PMF. Evacuation can take place by vehicle or on foot along the road as floodwater advances. People should not be trapped unless they delay their evacuation from their homes, for example, people living in 2-storey homes may initially decide to stay but reconsider after water surrounds them.

These communities contain low-lying areas from which people will be progressively evacuated to higher ground as the level of inundation increases. This inundation could be caused either by direct flooding from the river system or by localised flooding from creeks.

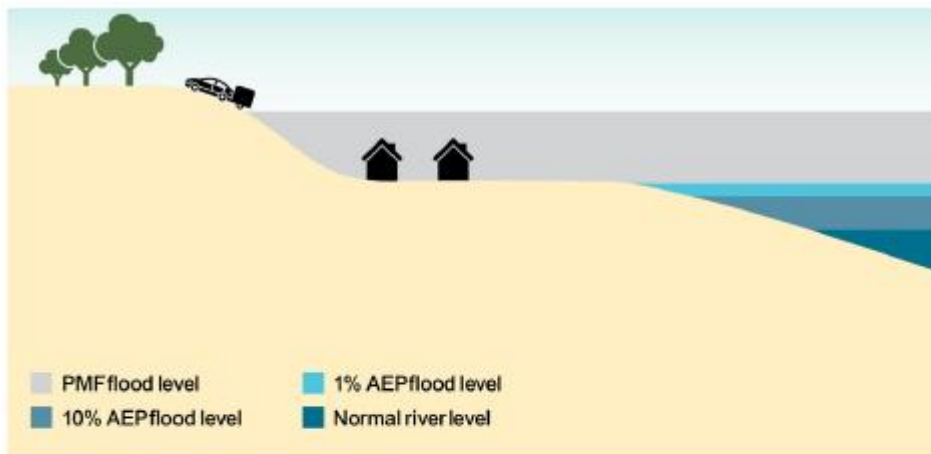


Figure 12 Area with rising road access

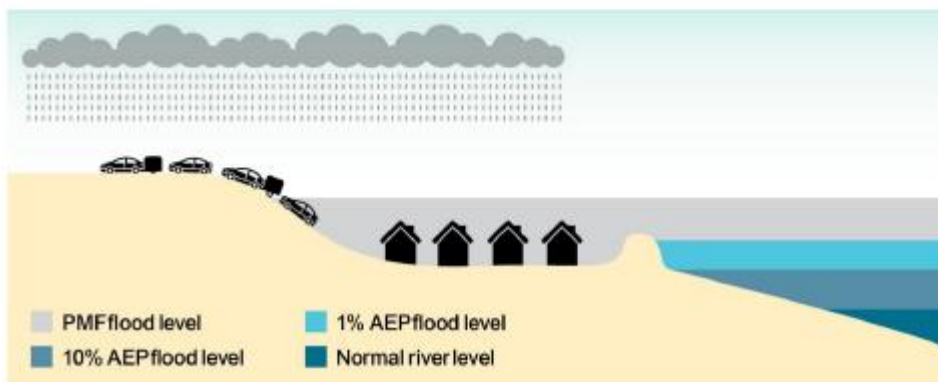


Figure 13 Area protected by a levee with rising road access

6.7 Sensitivity Analyses

6.7.1. General

The sensitivity of the hydraulic model to variations in model parameters such as hydraulic roughness, the partial blockage of major hydraulic structures by woody debris and coincident flooding in the Murray River was tested as part of the present study. The main purpose of these analyses was to give some guidance on:

- the freeboard to be adopted when setting minimum floor levels of development in flood prone areas, pending the completion of the review and update of the *Albury FRMS&P*; and
- areas where additional flood related planning controls should be implemented due to the development of new hazardous flow paths.

6.7.2. Sensitivity of Flood Behaviour to an Increase in Hydraulic Roughness

Figure 6.21 (3 sheets) shows the difference in peak flood levels (referred to as “afflux”) for the 1% AEP event resulting from an assumed 20% increase in hydraulic roughness (compared to the values given in **Table 4.2**).

The typical increases in peak flood level in the areas subject to Main Stream Flooding are generally in the range 20 to 100 mm, with increases of up to 200 mm shown to occur in isolated areas. Increases in peak flood levels in areas subject to Major Overland Flow are generally in the range 10 to 50 mm.

6.7.3. Sensitivity of Flood Behaviour to a Partial Blockage of Hydraulic Structures

As mentioned in **Section 6.1**, the design flood envelopes presented in this report incorporate the probability neutral blockage factors that are set out in **Table E1** in **Appendix E** of this report. As the degree to which each individual hydraulic structure experiences a blockage will vary during a real flood, the sensitivity of flood behaviour assuming no blockage of each structure was assessed as part of the present study.

The removal of the probability neutral blockage factors on the major hydraulic structures that are located at the following locations results in an increase in the rate at which flow discharges to the downstream reach of watercourse:

- the Main Southern Railway and Hume Highway crossings of Eight Mile Creek, Seven Mile Creek and a tributary of Eight Mile Creek; and
- the Elizabeth Mitchell Drive crossing of an unnamed watercourse to the south of Ettamogah Road.

Figure 6.22, 3 sheets shows that the removal of the blockage will increase peak flood levels along the main arm of Eight Mile Creek by up to 200 mm and by up to 100 mm on its tributaries.

The removal of the probability neutral blockage factors generally has a negligible effect on flood behaviour in areas subject to Major Overland Flow.

6.7.4. Sensitivity of Coincident Flooding in Murray River

As mentioned in **Section 4.4**, the downstream boundary of the model comprises a static tailwater level in the Murray River of RL 154 m AHD which is representative of the water level in the river resulting from typical regulated flow releases from Hume Dam. As it is possible that a local catchment storm event could occur coincident with a flood on the Murray River, the sensitivity of flood behaviour assuming a 1% AEP local catchment flood occurs coincident with a 10% AEP flood on the Murray River was assessed as part of the present study.¹⁶

Figure 6.23 (3 sheets) shows that elevated water levels in the Murray River would have a negligible effect on flood behaviour in the study catchments.

¹⁶ The downstream boundary of the model was updated to incorporate a static water level of RL 157 m AHD, which was based on the 10% AEP flood level at the confluence of Eight Mile Creek and the Murray River under present day conditions (i.e. in the absence of the proposed South Albury Levee upgrade) as defined in Water Technology, 2024.

6.8 Climate Change Sensitivity Analysis

6.8.1. General

At the present flood study stage, the principal issue regarding climate change is the potential increase in flood levels and extents of inundation throughout the study area. In addition, it is necessary to assess whether the patterns of flow will be altered by new floodways being developed for key design events, or whether the provisional flood hazard will be increased.

DCCEEW currently recommends that the advice set out in its flood risk management guideline FB01 entitled “*Understanding and managing flood risk*” be used as the basis for examining climate change in projects undertaken under the State Floodplain Management Program and the FRMM. The guideline recommends that flood behaviour resulting from 0.5% and 0.2% AEP events be used as a proxy for the scale of change that could occur to flood behaviour in a 1% AEP event as a result of future climate change.¹⁷

During the preparation of the review and update of the *Albury FRMS&P* it will be necessary to consider the impact of climate change on flood damages to existing development. Consideration will also be given both to setting floor levels for future development and in the formulation of works and measures aimed at mitigating adverse effects expected within the service life of development. Mitigating measures which could be considered in the review and update of the *Albury FRMS&P* include the implementation of structural works such as levees and channel improvements, improved flood warning and emergency management procedures and education of the population as to the nature of the flood risk.

6.8.2. Sensitivity to Increased Rainfall Intensities

As mentioned, the investigations undertaken at the flood study stage are mainly seen as sensitivity studies pending more detailed consideration in the review and update of the *Albury FRMS&P*. For the purposes of the present study, the design rainfalls for 0.5% and 0.2% AEP events were adopted for assessing the potential impacts of future climate change on flood behaviour in a 1% AEP event.

Figure 6.24 (3 sheets) shows the increase in peak flood levels resulting from a 10 per cent increase in 1% AEP rainfall intensities. The increase in peak flood levels in areas subject to Main Stream Flooding varies between 20 and 100 mm, with increases of up to 500 mm shown to occur on the upstream side of the Main Southern Railway crossing of the various watercourses. Increases in peak flood levels of generally between 10 to 50 mm are shown to occur in areas subject to Major Overland Flow.

Figure 6.25 (3 sheets) shows the increase in peak flood levels resulting from a 30 per cent increase in 1% AEP rainfall intensities. The increase in peak flood levels in areas subject to Main Stream Flooding varies between 50 and 200 mm, with increases of up to 700 mm shown to occur on the upstream side of the Main Southern Railway crossing of the various watercourses. Increases in peak flood levels of generally up to 100 mm are shown to occur in areas subject to Major Overland Flow.

¹⁷ While ARR 2019 updated the advice in relation to the impact that climate change will have on the BoM, 2016 design rainfall intensities, as well as initial and continuing losses for design flood estimation in late 2024, due to the timing of its release, the advice set out in OEH, 2019 has been adopted for undertaking the present study. Consideration will need to be given to the implications of the updated advice in ARR 2019 when preparing the review and update of the *Albury FRMS&P*.

Figure 6.26 (3 sheets) shows the increase in the extent of land that would be affected by floodwater should 1% AEP rainfall intensities increase by 10 or 30 per cent. The extent of land that would be inundated by floodwater should 1% AEP rainfall intensities increase by up to 30% is generally negligible, with the following exceptions:

- in the Nexus Industrial Precinct where new flows paths arise due to increased overtopping of the existing levee which runs along its western boundary;
- on the upstream side of the Main Southern Railway and Hume Highway, where the extent of ponding increases significantly;
- on the left (eastern) overbank of Seven Mile Creek and on the right (western) overbank Eight Mile Creek where flow from both watercourses combine; and
- on land that lies immediately to the north of Thurgoona Drive and east of Table Top Road where the extent of inundation increases significantly.

Consideration will need to be given to the identified changes that occur in flood behaviour during the preparation of the review and update of the *Albury FRMS&P*.

6.9 Potential Impacts of Future Development

Future infill development has the potential to increase the rate and volume of runoff conveyed by the various watercourses, as well as increase the frequency of surcharge of the local stormwater drainage system. It is also likely to result in changes to the existing drainage system. For example, while existing minor watercourses are likely to be retained and formalised in drainage reserves, piped drainage systems associated with urban subdivisions will result in significant amendments to existing overland flow paths leading to the watercourses.

The impact of future infill development was undertaken by adjusting the sub-catchment imperviousness in the Eight Mile and Sandy Creek DRAINS Model to reflect fully developed conditions within all land zoned for urban type development in the *Albury Local Environment Plan 2010*, with the resulting design discharge hydrographs used as input to the Eight Mile and Sandy Creek TUFLOW Model. As it was not possible to determine the layout of future development within the study area, no adjustments were made to the roughness values and topography of the Eight Mile and Sandy Creek TUFLOW Model.

Figures 6.27, 6.28 and 6.29 (3 sheets each) show the impact that uncontrolled development could have on flood behaviour in the study area in a 20%, 5% and 1% AEP storm event, respectively. Note that the assessment undertaken as part of the present study is of a broad-scale and strategic nature, and that more detailed site specific assessments would need to be undertaken as part of any future development.

Peak flood levels are shown to increase by up to 300 mm on the tributaries of Eight Mile Creek if uncontrolled development were allowed to occur in the study catchment for the range of assessed storm events, while minor reductions in peak flood levels are shown to occur along the main arm of Eight Mile Creek due to changes in the timing of the flood wave in its tributaries.

This finding reinforces the need for Council to require detention basins to be incorporated in the design of future development to ensure that post-development peak flows are no higher than pre-development conditions.

6.10 Selection of Interim Flood Planning Levels

After consideration of the findings of the present study and in advance of the future review and update of the *Albury FRMS&P*, the following criteria were adopted for defining the Interim Flood Planning Area (IFPA) for the study area:

- in areas subject to Main Stream Flooding, the extent of the IFPA was defined as land lying at or below the peak 1% AEP flood level plus a freeboard allowance of 0.5 m (**Main Stream Flooding IFPA**);¹⁸ and
- in areas subject to Major Overland Flow and that also lie outside the extent of the Main Stream Flooding IFPA, the extent of the IFPA was defined as land inundated to a depth greater than 100 mm or within the extent of the floodway (**Major Overland Flow IFPA**).¹⁹

Figure 6.30 (3 sheets) shows the extent of the Main Stream Flooding and Major Overland Flow IFPAs in the study area. In advance of the adoption of any precinct specific development control plan and/or the future review and update of the *Albury FRMS&P*, it is recommended that the following freeboard allowances be adopted when setting minimum habitable floor levels of future development:

- for development that lies within the extent of the Main Stream Flooding IFPA, habitable floor levels should be set a minimum 500 mm above peak 1% AEP flood levels; and
- for development that lies within the extent of the Major Overland Flow IFPA, habitable floor levels should be set a minimum 300 mm above peak 1% AEP flood levels.

Figure 6.30 also shows the area that lies between the IFPA and the extent of the PMF, which for the purpose of the present study has been denoted the “*Interim Outer Floodplain*”. Consideration will need to be given during the preparation of the future review and update of the *Albury FRMS&P* as to whether Council should adopt clause 5.22 in the *Albury Local Environmental Plan 2010*, noting that this clause relates to the need for Council to assess sensitive and hazardous type development that is proposed on land that lies between the IFPA and the PMF.

¹⁸ For the purpose of the present study, the Main Stream Flooding IFPA was defined along watercourses that are identified in the NSW Government’s *SixMaps* “hydrolines” online database and where the peak 1% AEP flow exceeded 2 m³/s.

¹⁹ The extent of Major Overland Flow IFPA was filtered to remove pockets of flooding where the area was less than 100 m².

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8 FLOOD-RELATED TERMINOLOGY

Note: For an expanded list of flood-related terminology, refer to glossary contained within the Floodplain Development Manual, NSW Government, 2005).

TERM	DEFINITION
Afflux	Increase in water level resulting from a change in conditions. The change may relate to the watercourse, floodplain, flow rate, tailwater level etc.
Annual Exceedance Probability (AEP)	The chance of a flood of a given or larger size occurring in any one year, usually expressed as a percentage. For example, if a peak flood discharge of 50 m ³ /s has an AEP of 5%, it means that there is a 5% chance (that is one-in-20 chance) of a 50 m ³ /s or larger events occurring in any one year (see average recurrence interval).
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
Average Recurrence Interval (ARI)	The average period in years between the occurrence of a flood of a particular magnitude or greater. In a long period of say 1,000 years, a flood equivalent to or greater than a 100 year ARI event would occur 10 times. The 100 year ARI flood has a 1% chance (i.e. a one-in-100 chance) of occurrence in any one year (see annual exceedance probability).
Catchment	The land area draining through the main stream, as well as tributary streams, to a particular site. It always relates to an area above a specific location.
Critical Duration	The storm duration which produces the highest peak flood level for a given design flood event.
Discharge	The rate of flow of water measured in terms of volume per unit time, for example, cubic metres per second (m ³ /s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving (e.g. metres per second [m/s]).
Flood fringe area	The remaining area of flood prone land after floodway and flood storage areas have been defined.
Interim Flood Planning Area (IFPA)	The area of land inundated at the Interim Flood Planning Level.
Interim Flood Planning Level (IFPL)	A combination of flood level and freeboard selected for planning purposes, as determined in floodplain risk management studies and incorporated in floodplain risk management plans.
Flood prone land	Land susceptible to flooding by the Probable Maximum Flood. Note that the flood prone land is synonymous with flood liable land.
Flood storage area	Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. The extent and behaviour of flood storage areas may change with flood severity, and loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation. Hence, it is necessary to investigate a range of flood sizes before defining flood storage areas.
Floodplain	Area of land which is subject to inundation by floods up to and including the probable maximum flood event (i.e. flood prone land).

TERM	DEFINITION
Flood Risk Management Plan	A management plan developed in accordance with the principles and guidelines in the <i>Flood Risk Management Manual, 2023</i> . Usually includes both written and diagrammatic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.
Floodway area	Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flow, or a significant increase in flood levels.
Freeboard	A factor of safety typically used in relation to the setting of floor levels, levee crest levels, etc. It is usually expressed as the difference in height between the adopted Flood Planning Level and the peak height of the flood used to determine the flood planning level. Freeboard provides a factor of safety to compensate for uncertainties in the estimation of flood levels across the floodplain, such as wave action, localised hydraulic behaviour and impacts that are specific event related, such as levee and embankment settlement, and other effects such as “greenhouse” and climate change. Freeboard is included in the flood planning level.
High hazard	Where land in the event of a 1% AEP flood is subject to a combination of flood water velocities and depths greater than the following combinations: 2 metres per second with shallow depth of flood water depths greater than 0.8 metres in depth with low velocity. Damage to structures is possible and wading would be unsafe for able bodied adults.
Low hazard	Where land may be affected by floodway or flood storage subject to a combination of floodwater velocities less than 2 metres per second with shallow depth or flood water depths less than 0.8 metres with low velocity. Nuisance damage to structures is possible and able bodied adults would have little difficulty wading.
Main Stream Flooding	Inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam.
Mathematical/computer models	The mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.
Merit approach	The merit approach weighs social, economic, ecological and cultural impacts of land use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and well-being of the State’s rivers and floodplains.
Major Overland Flow	Inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.
Interim Outer Floodplain	Area of land that lies between the IFPA and the PMF

TERM	DEFINITION
Peak discharge	The maximum discharge occurring during a flood event.
Peak flood level	The maximum water level occurring during a flood event.
Probable Maximum Flood (PMF)	The largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation coupled with the worst flood producing catchment conditions. Generally, it is not physically or economically possible to provide complete protection against this event. The PMF defines the extent of flood prone land (i.e. the floodplain). The extent, nature and potential consequences of flooding associated with events up to and including the PMF should be addressed in a floodplain risk management study.
Probability	A statistical measure of the expected chance of flooding (see annual exceedance probability).
Risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of the manual it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
Runoff	The amount of rainfall which ends up as stream flow, also known as rainfall excess.
Stage	Equivalent to water level (both measured with reference to a specified datum).

APPENDIX A

COMMUNITY NEWSLETTER AND QUESTIONNAIRE



Contact: CityPlanning
Reference: DOC24/161724

Dear

Eight Mile Creek Flood Study Review for

We're working with consultants Lyall & Associates Consulting Water Engineers to conduct a comprehensive review of previous flood studies for the Eight Mile Creek Study Area. This review will make any necessary updates to flood information based on current catchment conditions.

We're seeking input from landowners, residents, and businesses on historic flooding within the Eight Mile Creek Study Area. (Refer Figure 1, within the red outline).

Please complete the attached survey by **Monday 7 October 2024**. You can return the completed survey using the reply-paid envelope provided. Alternatively, you can fill out the survey online at **haveyoursay.alburycity.nsw.gov.au/eight-mile-creek-flood-study-review**

All information provided in the survey will remain confidential and used solely for this study.



Join us for a Community Information Session on **Wednesday, 25 September 2024**. To register visit **haveyoursay.alburycity.nsw.gov.au/eight-mile-creek-flood-study-review** or contact City Planning on 02 6023 8111.

T 02 6023 8111
F 02 6023 8190
info@alburycity.nsw.gov.au
alburycity.nsw.gov.au

PO Box 323
553 Kiewa Street
Albury NSW 2640
ABN 92 965 474 349

Why complete a flood study review?

The Eight Mile Creek Flood Study Review will develop an updated set of flood models which will be used to more accurately define the nature of both mainstream flooding and major overland flow under current catchment conditions.

The Eight Mile Creek Flood Study Review aims to enhance community resilience to flooding by improving development planning, emergency management and community awareness.

The Eight Mile Creek Flood Study Review will also help inform the Thurgoona Wirlinga Precinct Structure Plan Review. To find out more, visit **haveyoursay.alburycity.nsw.gov.au/growing-thurgoona-wirlinga**

How will the Eight Mile Creek Flood Study Review be conducted?

The Eight Mile Creek Flood Study Review will be undertaken in accordance with legislative requirements of the NSW Floodplain Risk Management Manual, in summary this includes:

- Data and desktop studies;
- Community engagement;
- Hydrologic and hydraulic modelling;
- Flood damages model and assessment;
- Processed model results, precinct area;
- Model specific results locations;
- Visualisations, and
- Reporting

Your responses to the survey questions will assist us in identifying the key flood issues within the Eight Mile Creek Study Area. For more information on this project, contact City Planning on 02 6023 8111.

Regards,



Steven Millett
Service Leader Assets, Environment and Sustainability

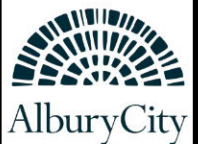
Eight Mile Creek Study Area

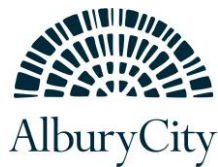


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SCALE:	1:75,000
DRAWN:	Ben VanKesteren
DATE:	16/07/2024 1:39 PM

MAP GRID AUSTRALIA, ZONE 55
TRANSVERSE MERCATOR PROJECTION
HORIZONTAL DATUM: GROUND DATUM AUSTRALIA 1994
VERTICAL DATUM: AUSTRALIAN HEIGHT DATUM 1972
PRODUCED BY THE MAPPING TEAM, ALBURYCITY COUNCIL





Eight Mile Creek Flood Study Review Community Survey

We're working with Lyall & Associates Consulting Water Engineers on the Eight Mile Creek Flood Study Review. Your responses to the survey questions will assist us in identifying the key flood issues within the Eight Mile Creek Catchment Area.

Please complete the attached survey by **Monday 7 October 2024**. You can return the completed survey using the reply-paid envelope provided. Alternatively, you can fill out the survey online at haveyoursay.alburycity.nsw.gov.au/eight-mile-creek-flood-study-review



If you have misplaced the reply-paid envelope or wish to submit additional information, please use the following address:

Lyall & Associates Consulting Water Engineers
Reply Paid 85163
NORTH SYDNEY NSW 2060

All information provided in the survey will remain confidential and used solely for this study.

Q1. What is the property address?

Q2. What is the primary use of the land at the above property?

- | | |
|--|--|
| ☐ Residential | ☐ Special purpose (Education/Defence) |
| ☐ Commercial | ☐ Agriculture/farming |
| ☐ Industrial | ☐ Vacant land |
| ☐ Other, please specify | ☐ Under development |
-

Q3. What is the occupancy status of the property?

- | | |
|--|---|
| ☐ Owner occupier | ☐ Industrial property |
| ☐ Tenant occupier | ☐ Commercial property |
| ☐ Investment property | ☐ Other (please specify) |
-

Q4. How long have you lived, worked, or owned property at this address in the Eight Mile Creek Catchment area?

- ☐ 0-5 years ☐ 5-10 years ☐ 10-20 years ☐ More than 20 years

Q5. How long have you lived, worked, or owned property in the general area (Albury-Wodonga or surrounding region)?

- ☐ 0-5 years ☐ 5-10 years ☐ 10-20 years ☐ More than 20 years

Q6. Have you ever been affected by flooding in the Eight Mile Creek Catchment area?

- ☐ Yes (Go to Q.7) ☐ No (Go to Q.10)

Q7. When and how were you affected by flooding?

	Flood 1	Flood 2
Date of flood (tick all that apply)	☐ 5 September 2021 ☐ 6 January 2022 ☐ 15 January 2022 ☐ 1 November 2022 ☐ Other (please specify)	☐ 5 September 2021 ☐ 6 January 2022 ☐ 15 January 2022 ☐ 1 November 2022 ☐ Other (please specify)
How were you affected by flooding? (tick all that apply)	☐ Road closed ☐ Front/backyard ☐ Garage ☐ Building/ home ☐ Floodwater-borne vehicle ☐ Other (please specify)	☐ Road closed ☐ Front/backyard ☐ Garage ☐ Building/ home ☐ Floodwater-borne vehicle ☐ Other (please specify)
Description of flood behaviour (e.g. depths, levels, duration of inundation etc) <i>Please be as descriptive as possible</i>		



Q8. In your opinion, what was the main cause of the flooding (tick all that apply)?

- ☐ Insufficient stormwater capacity
- ☐ Blockage of channels, stormwater inlets, pipes or drains by debris
- ☐ Overland flow impediments (e.g. fences or buildings)
- ☐ Changes caused by new development
- ☐ Other (please specify) _____

Q9. Do you have any photos or videos of these floods?

- ☐ Yes ☐ No

If yes, please email a copy of photos/videos up to 20MB, to eightmile@lyallandassociates.com.au for files over 20MB, please use a files sharing service.

Q10. Is there any other information you would like to share that may assist the Eight Mile Creek Flood Study Review?

[illegible]

Q11. Contact details (optional)

Email: _____

APPENDIX B
DETAILS OF AVAILABLE DATA
AND COMMUNITY CONSULTATION

TABLE OF CONTENTS

	Page No.
B1 COLLECTION OF MISCELLANEOUS DATA	B-1
B1.1 Airborne Laser Scanning Survey.....	B-1
B1.2 Existing Stormwater Network	B-1
B1.3 Historic Rainfall Data	B-1
B1.4 Detailed Ground Survey	B-3
B1.5 Photographic Record	B-3
B1.6 Previous Investigations.....	B-3
B1.6.1. Eight Mile Creek Flood Study (URS, 2012)	B-3
B1.6.2. Lake Ettamogah Winter Storage Dam – Design Review (URS, 2014)...	B-4
B1.6.3. Albury Floodplain Risk Management Study and Draft Plan (WMAwater, 2016)	B-4
B1.6.4. Sloane’s Froglet Stormwater Wetland Design Guidelines (Spiire, 2017)	B-5
B1.6.5. Thurgoona Wirlinga Drainage Strategy (Spiire, 2020)	B-5
B1.6.6. Nexus Flood Mitigation Study (WMS, 2023)	B-6
B1.6.7. South Albury Levee Upgrade - Murray River Flood Study (Water Technology, 2024).....	B-9
B2 COMMUNITY CONSULTATION	B-10
B2.1 Background	B-10
B2.2 Summary of Findings	B-10
B2.2.1. General	B-10
B2.2.2. Resident Profile	B-10
B2.2.3. Experiences of Flooding	B-11
B3 REFERENCES	B-14

ANNEXURES

B1	Extract from Spiire, 2020
B2	Extract from Water Technology, 2024
B3	Responses to Community Questionnaire

LIST OF FIGURES (BOUND IN VOLUME 2)

Figure B1.1	Location of Rain Gauges and LiDAR Survey Data
Figure B1.2	Location and Source of Stormwater Drainage System Data
Figure B2.1	Location of Respondents to Community Questionnaire and Observed Flood Data

B1 COLLECTION OF MISCELLANEOUS DATA

B1.1 Airborne Laser Scanning Survey

Figure B1.1 shows the extent of LiDAR survey data that are available in the vicinity of the study area, while **Table B1.1** sets out the details of the available LiDAR survey data.

TABLE B1.1
LiDAR SURVEY DATA SPECIFICATIONS

Data Set	Date of Capture	Data Provider	Horizontal Accuracy	Vertical Accuracy	DEM Resolution
Albury202011	27 November 2020	Geoscience Australia	±800 mm	±300 mm	1 m
Albury202409	11 September 2024	Aerometrex	±200 mm	±100 mm	1 m

B1.2 Existing Stormwater Network

Figure B1.2 shows the alignment of the existing stormwater drainage network in the study area. Details of the existing stormwater drainage network were taken from the following sources (listed in order of priority):

- Detailed structure survey that was undertaken by Ovens & Murray Land Survey in December 2024.
- Detailed structure survey along Kerr Road and Thurgoona Drive that was undertaken by Council in October 2024.
- Detailed structure survey along the Main Southern Railway that were provided by ARTC.
- URS, 2012 contained details of key drainage structures along Eight Mile Creek, Nine Mile Creek and Seven Mile Creek, as well as a tributary of Eight Mile Creek.
- Council's GIS database of the stormwater drainage system.
- Transport for NSW (**TfNSW**), provided details of the culvert and bridge crossings along the state-operated roads in the study area.
- Assumed based on a desktop analysis of aerial photography, LiDAR survey data, Google Streetview and adjacent stormwater drainage infrastructure.

B1.3 Historic Rainfall Data

Figure B1.1 shows the plan location of two WaterNSW operated pluviographic rain gauges, one BoM operated pluviographic rain gauge and one BoM operated All Weather Station, as well as a large number of daily rain gauges that are located in the vicinity of the study area, while **Table B1.2** over the page sets out the details of each. **Figure B1.1** and **Table B1.2** also show details of four privately owned rain gauges that are located in the vicinity of Thurgoona Park and were in operation at the time of the 15 January 2022 storm, details of which were obtained from the *Weather Underground* website.

TABLE B1.2
SUMMARY OF AVAILABLE RAIN GAUGE DATA⁽¹⁾

Gauge Number	Gauge Name	Gauge Type	Site Commence	Site Cease	Distance from Study Area (km)
72160	Albury Airport AWS	BoM AWS ⁽²⁾	September 2008	Ongoing	0.8
401015	Bowna Creek At Yambla	WaterNSW	December 1973	Ongoing	7.3
401027	Murray River at Hume Dam - Storage Gauge No.2		August 2002	Ongoing	2.2
72023	Hume Reservoir	BoM Pluvi ⁽³⁾	March 1955	June 2017	3.2
74236	Bungowannah (Roseleigh)	BoM Daily	June 1893	Ongoing	21.3
72146	Albury Airport		June 1983	Ongoing	0.6
72023	Hume Reservoir		January 2022	Ongoing	3.2
82056	Wodonga		November 1898	Ongoing	7.7
82117	Bethanga		January 1971	12 June 2013	9.7
72168	Albury (Table Top (Nyrimbla))		January 1904	December 2012	1.5
74056	Jindera Post Office		October 1884	August 2013	7.4
72001	Albury		February 1858	July 1962	6.9
72059	Albury Grammar School		November 1960	November 1969	2.7
72057	Albury Pmg Repeater Station		September 1954	June 1962	6.9
72097	Albury Pumping Station		May 1970	August 1986	0.4
82005	Bethanga Lower		July 1907	December 1970	7.3
82100	Bonegilla		June 1968	June 1986	6.4
74015	Brae Springs		June 1892	May 1943	12.5
74024	Bungowannah Park		February 1883	December 1998	17.6
72093	Cumberoona		Jan 1887	August 1915	11.8
72021	Hawksview		October 1929	July 1947	6.4
74134	Jindera (Murrumbateman)		November 1913	December 1918	5.0
74057	Jindera Public School		March 1919	December 1955	5.0
72041	Table Top (Bellevue)		August 1950	May 1969	0.4
74177	The Olives		May 1886	January 1925	29.7
72095	Thurgoona		Jan 1887	October 1913	0.2
82088	Wodonga South		September 1956	August 1994	10.9
74225	Woodlands (Murrumbateman)		May 1907	December 1918	5.0
IALBURY11	Albury - IALBURY11	Privately owned	Not known	Ongoing	-
IALBUR40	Drage's Thurgoona - IALBUR40				-
IALBURY19	woodenco - IALBURY19				-
IALBUR88	Ecowitt HP2551 - IALBUR88				-

1. Refer **Figure B1.1** for location
2. AWS = All Weather Station
3. Pluvi = Pluviographic rain gauge

B1.4 Detailed Ground Survey

Figure B1.2 shows the extent of detailed ground survey that was captured on behalf of Council along Kerr Road where it runs between Thurgoona Drive and Woolshed Creek.

B1.5 Photographic Record

Appendix C contains a number of photographs that were provided by Council and respondents to the *Community Questionnaire* showing flood behaviour that was observed in parts of the study area during storms that occurred on 16 March 1992, 6 October 2005, October 2010 (day not known), 5 September 2021, 5-6 January 2022, and 14-15 January 2022.

B1.6 Previous Investigations

B1.6.1. Eight Mile Creek Flood Study (URS, 2012)

This report describes the results of a detailed flood study that was undertaken for the Eight Mile Creek catchment in 2012 by URS. The study included the development of a hydrologic (RAFTS) model of the Eight Mile Creek catchment (**URS RAFTS Model**) and a hydraulic (MIKE FLOOD) model (**URS MIKE FLOOD Model**), both of which were then used to define flood behaviour along the main arms of Eight Mile Creek, Seven Mile Creek, Nine Mile Creek and Woolshed Creek, as well as their associated tributaries.

The report refers to two historic flood events that occurred in the Eight Mile Creek catchment in January 1974 and August 1983 which were equivalent to design flood events with AEPs of about 1.4-2% and 50%, respectively, noting that there are no flood marks or photographs showing flood behaviour from these two events. URS, 2012 also contains photographs showing flooding that occurred in the lower reaches of Eight Mile Creek on 16 March 1992 and 6 October 2005, copies of which are reproduced in **Appendix C** of this report.

The URS RAFTS Model was not calibrated as part of URS, 2012 due to the limited data that was available. **Table B1.3** sets out the loss values that were adopted for design flood estimation purposes, noting that URS, 2012 makes no mention of the catchment storage coefficient (Bx) that was adopted as part of the study.

TABLE B1.3
ADOPTED LOSS VALUES - URS, 2012

Design Storm Event	Pervious Areas		Impervious Area	
	Initial Loss (mm)	Continuing Loss (mm/hr)	Initial Loss (mm)	Continuing Loss (mm/hr)
1 EY – 0.2% AEP	15	2.5	1	0
PMP	0	1	1	0

The URS MIKE FLOOD Model incorporated the main arm of Eight Mile Creek and its tributaries, including 19 hydraulic structures beneath the road and railway crossings of the watercourses which were surveyed as part of the study (refer **Figure B1.2** for location of structures). The URS MIKE FLOOD Model was schematised so that 1d elements were relied upon to model the inbank area of the watercourses, while 2d grid elements were relied upon to represent the overbank flows in the catchment. A static tailwater level of RL 158 m AHD was adopted as the downstream boundary condition of the URS MIKE FLOOD Model based on the level of the 1917 Murray River flood at the confluence with Eight Mile Creek.

The results of the hydraulic models were used to define flood behaviour in a 1EY, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP flood event, as well as the PMF.

B1.6.2. Lake Ettamogah Winter Storage Dam – Design Review (URS, 2014)

URS undertook the *Lake Ettamogah Winter Storage Dam – Design Review* on behalf of Norske Skog to address a request of NSW Dam Safety Committee to undertake an updated flood study in order to re-assess the consequence category for the dam. The assessment which was undertaken in accordance with *ANCOLD Guidelines on the Consequence Categories for Dams* from 2012 determined that the consequence category for both flood and sunny day failure of the dam is High C.

URS, 2014 states that the dam has a capacity of approximate 2,100,00 m³ at the spillway which is set at about RL 213.3 m AHD, which increases to about 3,867,000 m³ at the crest of the dam embankment which is set at RL 215.35 m AHD. The dam was constructed in 1994 and its function is to provide storage capacity for excess process water for the Norske Skog paper mill during the winter months and to irrigate the adjacent pine plantation in the summer months.

URS, 2012 found that the dam embankment is not overtopped in a PMF and the total flow over the spillway in an event of this magnitude is about 150 m³/s.

B1.6.3. Albury Floodplain Risk Management Study and Draft Plan (WMAwater, 2016)

Council commissioned the *Albury Floodplain Risk Management Study and Draft Plan* (WMAwater, 2016) to assess floodplain management issues in the vicinity of the city of Albury and review policies and options for the management of flood affected land along the Murray River and its tributaries.

The sub-catchments that were incorporated in the URS RAFTS Model was refined as part of WMAwater, 2016 (**WMA RAFTS Model**), while URS MIKE FLOOD Model was updated as follows:

- the size of the grid was reduced from 10 m to 5 m;
- the schematisation of the model was changed so that both the inbank and overbank flows were represented in 2d
- additional hydraulic structures were incorporated in the model; and
- the downstream boundary of model was updated to incorporate 1% AEP flow in the Murray River.

The updated MIKE Flood model (**WMAwater MIKE FLOOD Model**) was used to define flood behaviour in a 5%, 2%, 1%, 0.5% and 0.2% AEP flood event, as well as the PMF. WMAwater, 2016 found that the results of the WMAwater MIKE FLOOD Model were about 0.2 to 0.3 m higher than the URS MIKE FLOOD Model along the alignment of the watercourses, and up to 1 m higher on the overbank areas. WMAwater, 2016 states that the results of the updated modelling was considered to show a “*far more realistic representation of the floodplain flows when compared to the original schematisation*” and are a “*more robust and reliable representation of flood mechanisms in Eight Mile Creek*”.

WMAwater, 2016 assessed four potential detention basins that are located in the Woolshed Creek catchment to the east of Kerr Road that were identified in the *Thurgoona/Wirlinga Structure Plan – Technical Report* which was prepared by Council in 2013. The assessment found that the four basins had the capacity to eliminate almost all adverse flooding impacts from future development downstream of the basin locations. The *Floodplain Risk Management Plan* which was developed

as part of WMAwater, 2016 includes the recommendation to develop a retarding basin strategy for the Eight Mile Creek catchment to offset the impact of flooding on future development in the catchment.

B1.6.4. Sloane's Froglet Stormwater Wetland Design Guidelines (Spiire, 2017)

The *Sloane's Froglet Stormwater Wetland Design Guidelines* detail key design considerations and hold points associated with the creation of Sloane's froglet breeding habitat and stormwater treatment wetlands in urban development projects in the Local Area Management Plan (**LAMP**) zone, the extent of which is shown in **Plate B1** over the page.

The guidelines are supplemented by existing planning, design, maintenance and construction documents. These guidelines provide a common understanding of and consistency in requirements for development approvals for proponents. The guidelines are dynamic in that they will be refined over time to incorporate new knowledge and considerations, as required.

The primary aims of the guideline are to ensure for all future development within the LAMP (where stormwater management is required) to achieve:

- **Habitat outcomes** - Sloane's froglet breeding habitat requirements are incorporated into all new stormwater wetlands irrespective of the presence of existing habitat.
- **Stormwater quality outcomes** - Stormwater design delivers water quality outcomes in accordance with current state regulations.

The guideline does not set out requirements for mitigating the impact that future subdivision development would otherwise have on the rate of flow discharging to the receiving drainage lines, such as the provision of stormwater detention basins, whether they be integrated with the Sloane's Froglet habitat requirements or not.

B1.6.5. Thurgoona Wirlinga Drainage Strategy (Spiire, 2020)

Council engaged Spiire to undertake the *Thurgoona Wirlinga Drainage Strategy* in 2020 to develop an overarching strategy that defines concept locations and sizes of wetlands/retarding basins to offset future development in the Eight Mile Creek catchment as per the recommendations of WMAwater, 2016. Spiire, 2020 proposed a total of 30 new wetlands/retarding basins in the Eight Mile Creek catchment. The hydrologic and hydraulic assessment that was undertaken as part of Spiire, 2020 was based on the procedures set out in the 1987 edition of *Australian Rainfall and Runoff* (Institution of Engineers Australia, 1987) (**ARR 1987**).

Annexure B1 contains a plan location of the 30 basins that were proposed as part of Spiire, 2020, while **Table B1.4** over the page sets out the details of each. The rational method was used to derived pre-development flows at the proposed sites based on the proposed post-development sub-catchments which are not always representative of the pre-developed sub-catchments. A hydrologic (RORB) model was developed as part of Spiire, 2020 (Spiire RORB Model) (**Spiire RORB Model**) to assess the detention storage requirements for each wetlands/retarding basin.

While Spiire, 2020 provides an overarching approach to stormwater management across the growth related areas within the study area, it recommends that further catchment analysis be undertaken during the future development approval process to refine flow calculations.



Plate B1: Sloane's Froglet Local Area Management Plan (**LAMP**)

B1.6.6. Nexus Flood Mitigation Study (WMS, 2023)

Council commissioned Water Modelling Solutions (**WMS**) to undertake a flooding investigation to better understand the current flood risk at the site of the Nexus Industrial Precinct in Ettamogah which was identified as being inundated by floodwater that surcharges the left (eastern) bank of Eight Mile Creek as part of WMAwater, 2016.

A hydrologic (WBNM) model of the 10.3 km² catchment contributing to flow in Eight Mile Creek at a location approximately 600 m to the east of Gerogory Lane was developed as part of WMS, 2023 (**WMS WBNM Model**) based on the procedures set out in ARR 2019. **Table B1.5** is principally a reproduction of *Table 4-1* in WMS, 2023 which shows a comparison of the hydrologic model parameters that were adopted as part of URS, 2012 (refer column B), WMAwater, 2016 (refer column C) and WMA, 2023 (refer column D).

TABLE B1.4
DETAILS OF WETLANDS/DETENTION BASINS RECOMMENDED AS PART OF SPIIRE, 2020

Basin ID ⁽¹⁾	Catchment Area (ha)	Pre-Developed Conditions ^(2,3)	Post-Developed Conditions ^(2,4)			
		Peak 1% AEP Flow	Peak 1% AEP Inflow	Peak 1% AEP Outflow	Outlet Pipe Dimensions	Required Storage Volume (m ³)
1	46.3	2.80	11.34	1.99	2 off 900 RCP's	15,000
2.1	54.8	3.17	12.87	1.89	2 off 825 RCP's	14,000
2.2	65.2	3.66	9.37	2.53	6 off 600 RCP's	9,000
3	58.5	3.38	13.82	2.12	2 off 900 RCP's	13,000
4	89.0	5.16	13.81	3.48	4 off 750 RCP's	18,000
5	80.7	4.28	4.98	2.18	2 off 750 RCP's	18,000
6	104.0	5.30	5.82	3.49	9 off 450 RCP's	14,400
7.1	178.8	7.78	36.59	6.83	3 off 825 RCP's	46,500
7.2	128.8	6.04	20.27	5.54	4 off 825 RCP's	22,300
7.3	49.9	2.94	25.27	2.97	2 off 700 RCP's	25,800
8	64.9	3.70	16.41	2.34	3 off 750 RCP's	21,000
9.1	89.4	4.65	21.45	4.66	5 off 750 RCP's	22,600
9.2	69.2	3.70	17.21	3.49	2 off 700 RCP's	18,800
10	77.6	4.27	17.15	2.72	1 off 1050 RCP	19,000
11	19.3	1.36	4.53	1.25	3 off 600 RCP's	4,420
12	20.9	1.49	5.13	1.26	4 off 525 RCP's	6,000
13	77.0	4.18	18.76	3.37	8 off 525 RCP's	20,000
14	10.7	0.87	2.79	0.83	3 off 525 RCP's	3,000
15	99.4	4.95	20.82	3.90	3 off 900 RCP's	19,000
16	106.5	5.21	20.22	4.39	4 off 900 RCP's	22,000
17	86.9	4.51	20.07	3.54	3 off 900 RCP's	22,000
18	98.1	5.00	7.10	4.01	7 off 750 RCP's	13,500
19	74.2	4.06	9.92	3.89	4 off 750 RCP's	29,800
20	93.1	4.76	32.69	4.55	3 off 825 RCP's	53,400
21	77.6	4.12	5.22	2.30	2 off 825 RCP's	29,000
22	46.5	2.15	10.72	1.59	2 off 825 RCP's	12,000
23	20.3	1.47	5.37	1.34	3 off 600 RCP's	4,920
24	23.7	1.64	4.42	1.50	5 off 525 RCP's	4,560
25	70.8	3.87	14.30	2.99	5 off 675 RCP's	15,000
26	101.2	5.01	10.87	4.57	5 off 750 RCP's	17,000

1. Refer **Annexure B1** of this report for extract from Spiire, 2020 showing the plan location of proposed wetland/retarding basin.
2. Hydrologic and hydraulic analysis based on procedures set out in 1987 Edition of Australian Rainfall and Runoff.
3. Pre-developed flows were derived using Rational Method calculations.
4. Post-development flows and basin dimensions were derived using Spiire RORB Model.

TABLE B1.5
COMPARISON OF HYDROLOGIC MODEL PARAMETERS AND MODEL PEAK FLOWS⁽¹⁾

Parameter [A]	Eight Mile Creek Flood Study (URS, 2012) [B]	Albury FRMS (WMAwater, 2016) [C]	NEXUS Flood Mitigation Study (WMS, 2022) [D]
ARR Version	ARR 1987	ARR 1987	ARR 2019
Initial Loss (mm)	15	15	26
Continuing Loss (mm/hr)	2.5	2.5	1.84
Median Pre-burst Depth for the 1% AEP (mm)	N/A	N/A	5.2
Burst Loss (mm) [Initial Loss - Pre-burst Depth]	N/A	N/A	20.8
1% AEP Critical Duration (hrs)	6	6	6
Rainfall Intensity for the 1% AEP Critical Duration (mm/hr)	14	14	11.8
Total Rainfall Depth for the 1% AEP Critical Duration (mm/hr)	84	84	70.9
Rainfall Depth after Burst Losses for the 1% AEP Critical Duration	69	69	50.1
Catchment Area within Study Area (ha)	1028.6	1028.6	1028.6
No. of Sub-catchments within Study Area	10	10	17
Peak Flow Upstream of Site (m ³ /s/)	9.3	7.7	7.6

1. Taken from Tables 4-1 and 4-2 of WMS, 2023.

While WMS, 2023 followed the NSW Jurisdictional Advice on the *ARR Data Hub* to derive an appropriate continuing loss value, it relied on the Median Pre-Burst Rainfall Depth in order to derive a Burst Loss instead of the Probability Neutral Burst Initial Loss values that are contained on the *ARR Data Hub* and recommended for use in NSW. As a result, the Burst Loss that was applied to the WMS WBNM Model for the 1% AEP 6 hour storm which was found to be critical at the site (i.e. 20.4 mm) is more than three times the Probability Neutral Burst Initial Loss value for the same storm (i.e. 6.2 mm).

The WMS WBNM Model was found to achieve a good match with the design peak flows in Eight Mile Creek upstream of the Nexus Industrial Precinct that were derived from the WMA RAFTS Model, but were lower than those derived as part of URS, 2012.

The WMS WBNM Model was used to derived design discharge hydrographs for input to a hydraulic (TUFLOW) model of a 3.2 km reach of Eight Mile Creek in the vicinity of the Nexus Industrial Precinct (**WMS TUFLOW Model**). The WMS WBNM and TUFLOW Models were used to define flood behaviour in a 5% and 1% AEP storm event.

The WMS TUFLOW Model was then used to assess four potential levee alignments that would prevent floodwater from inundating the Nexus Industrial Precinct. WMS, 2023 identified that the preferred option was to construct a new 560 m long earthen levee along the western boundary of the precinct with the crest elevation set 0.3 m above the 1% AEP design peak flood level.

B1.6.7. South Albury Levee Upgrade - Murray River Flood Study (Water Technology, 2024)

Council engaged Water Technology to update the Murray River Flood Study as part of the South Albury Levee Upgrade project. A hydraulic (TUFLOW) model of the Murray River where it runs between the Hume Dam and a location approximately 20 km to the west of Albury was developed as part Water Technology, 2024 and used to define flood behaviour on the Murray River in the vicinity of Albury for floods ranging from 20% to 0.2% AEP, as well as the PMF under the following two scenarios:

- Present day conditions; and
- Post-South Albury levee upgrade conditions.

Annexure B2 contains a plan that has been extracted from Water Technology, 2024 that shows the design flood extents for the full range of assessed floods under present day conditions. The results of Water Technology, 2024 show that:

- a) the Murray River inundates the billabong that Sandy Creek discharges into, but does not inundate land on the northern side of the Riverina Highway; and
- b) the Murray River backs up Eight Mile Creek to a location in the vicinity of the intersection of the Riverina Highway and Elizabeth Mitchell Drive.

B2 COMMUNITY CONSULTATION

B2.1 Background

At the commencement of the study, the Consultants prepared a *Community Newsletter and Questionnaire* which were distributed by Council to residents and business owners in the study area (refer **Appendix A**).

The purpose of the *Community Newsletter* was to introduce the objectives of the study so that the community would be better able to respond to the *Community Questionnaire* and contribute to the study process. The *Community Newsletter* contained a plan showing the extent of the study area and a summary of the proposed methodology and outcomes.

The *Community Questionnaire* was structured with the objectives of collecting information on historic flood behaviour in the study area.

The *Community Newsletter and Questionnaire* were advertised in the local newspaper and posted to 3,700 residents and business owners in the study area in September 2024. The *Community Newsletter and Questionnaire* were also advertised on Council's website and social media platforms.

B2.2 Summary of Findings

B2.2.1. General

Residents and business owners were requested to complete the *Community Questionnaire* by 7 October 2024. The Consultants received 532 responses in total, which amounted to about 14 per cent of the total number of questionnaires that were distributed to the community.

Figure B2.1 shows the plan location of the 504 respondents that provided their address, while the collated responses to the *Community Questionnaire* are shown in graphical format in **Annexure B3** of this Appendix.

B2.2.2. Resident Profile

The first five questions of the *Community Questionnaire* canvassed resident information such as whether the respondent was a resident or business owner, length of time at the property and the type of property (e.g. residential, commercial, farmland, etc.).

Of the 532 responses, 489 respondents occupied residential type property (**Question 2**), one occupied commercial type property, 21 occupied agricultural farming type land and five were owners of vacant land in the study area.

In response to **Question 3**, 452 respondents were property owners and 55 rented the property, 22 were investment properties and one respondent owned commercial property in the study area.

The length of time respondents had been at their current address was found to be varied, with approximately 38% (203 respondents) having lived at the residence for between '0-5 years', 23% (119 respondents) for '5 to 10 years', 19% (101 respondents) for '10 to 20 years', and 20% (105 respondents) for 'more than 20 years' (**Question 4**).¹

¹ Three respondents did not provide a response to **Question 4**.

Of those that answered **Question 5**, approximately 49% (260 respondents) have lived in the general area for more than 20 years, 17% (89 respondents) for 10-20 years, 14% (74 respondents) for 5-10 years and 19% (102 respondents) for less than five years.²

B2.2.3. Experiences of Flooding

In **Question 6**, 112 of the respondents advised that they had been affected by flooding, while 398 advised that they had not been affected by flooding.

In response to **Question 7a**, respondents advised that they had been affected by flooding during a number of storm events. The most commonly reported storm events occurred on 14-15 January 2022 (83 respondents), 5-6 January 2022 (57 respondents), 1 November 2022 (50 respondents) and 5 September 2021 (44 respondents). Respondents had also been affected by storm events that occurred on the following dates:

- 17 March 2010 (one respondent)
- August 2010 (one respondent)
- August 2016 (one respondent)
- 22 March 2017 (one respondent)
- 25 November 2021 (one respondent)
- 11 January 2022 (one respondent)
- 14 January 2022 (one respondent)
- 8 August 2022 (one respondent)
- 14 October 2022 (two respondents)
- 22 October 2022 (one respondent)
- 13-14 November 2022 (one respondent)
- June 2023 (one respondent)
- 2 August 2024 (one respondent)

Of those that have been affected by flooding, 97 indicated that their front or back yard was inundated, 42 indicated that roadways were cut by floodwater, 61 indicated that their garage was flooded and 35 indicated that their house or business was flooded (**Question 7b**). In particular, a total of 14 respondents indicated that their dwelling was inundated above-floor level in the 15 January 2022 storm event, six of which were located in Thurgoona Park, three in Fairway Gardens and one each in St Johns Hill, St Johns Green, Tweed Court, Kerrford Park and The Elms developments.

Respondents to the *Community Questionnaire* also identified that they had been affected by flooding in the following way:

- Farmland being damaged, leading to ruined pasture and damaged fencing (six respondents); and
- Fencing on residential allotments being damaged (four respondents).

In response to **Question 8**, respondents identified the following features as the main cause of flooding in the study area:

- insufficient stormwater capacity (92 respondents);
- blockage of channels, stormwater inlets, pipes and drains by debris (56 respondents);
- overland flow impediments (i.e. fences) (36 respondents); and
- changes caused by new development (67 respondents).

² Twenty-one (21) respondents did not provide an answer to **Question 5**.

Of the 47 respondents that answered 'other' to **Question 8**, the following were identified as the main cause of flooding:

- floodwater backing up through wastewater drains causing internal flooding within dwellings (13 respondents);
- Eight Mile Creek and its tributaries being obstructed by silt and debris (four respondents), with respondents specifically mentioning the Kerrs Road flood works behind Finlay Road and the creek running through the golf course;
- overflow from neighbouring properties (three respondents);
- overflow from nearby retarding basins (one respondent);
- poor upkeep of open swale systems (one respondent); and
- waterway modification in the Hillford Rise Estate (one respondent).

In **Question 9**, respondents were asked if they have photographs that show the flooding and to provide them directly to the Consultant. **Appendix C** of this report contains a number of photographs that were provided by respondents to the *Community Questionnaire* showing flood behaviour in the study area during storms that occurred on 16 March 1992, 6 October 2005, October 2010 (day not known), 5 September 2021, 5-6 January 2022 and 14-15 January 2022.

In **Question 10** respondents were asked if they had any other information that they would like to share that may assist the present study. A total of 185 respondents answered this question. Respondents made the following observations about the drainage system in the study area:

- the silt retention dams in Thurgoona Golf Club are full;
- a number of respondents mentioned the persistence of water for days following storm events at a number of locations including the Williams Road crossing of Eight Mile Creek the vacant land to the north of Redbox Drive, Pickworth Street and Kerr Road;
- the banks and surrounding pastures of Eight Mile Creek becoming heavily eroded due to the increased flow in recent years, leading to trees on its banks collapsing into the river creating more debris (five respondents); and
- in the November 2022 storm event, the dam located on the eastern side of Kerr Road burst.

In response to **Question 10**, the following suggestions were made for measures to reduce the impact of flooding on the community:

- a large number of respondents suggested ensuring new developments have proper stormwater drainage systems in place (such as adequate piping and ensuring that downhill courts have water escape routes that aren't running through residential dwellings) and that these new developments have no downstream effects on existing properties;
- the regular maintenance and clearing of stormwater drainage systems;
- the clearing of obstructions such as trees and raised footpaths from overland flow paths leading to Eight Mile Creek and its tributaries;
- clearing of debris from Eight Mile Creek and its tributaries; and
- implementation of grassed floodways, drains, table drains and other systems that can be easily maintained.

Five respondents to the *Community Questionnaire* also raised significant concerns over the data being gathered in the present study being used for the purpose of altering insurance costs and/or property values.

While many respondents described significant damage to their properties, three respondents to the *Community Questionnaire* provided the total costs of repairing the damages caused by floodwater, with the costs ranging from \$40,000 - \$100,000.

B3 REFERENCES

The Institution of Engineers Australia, 1987. ***“Australian Rainfall and Runoff – A Guide to Flood Estimation”***, Volumes 1 and 2.

Spiire, 2020. ***“Thurgoona Wirlinga Drainage Strategy”***

URS, 2012. ***“Eight Mile Creek Flood Study”***

URS, 2014. ***“Lake Ettamogah Winter Storage Dam – Design Review”***

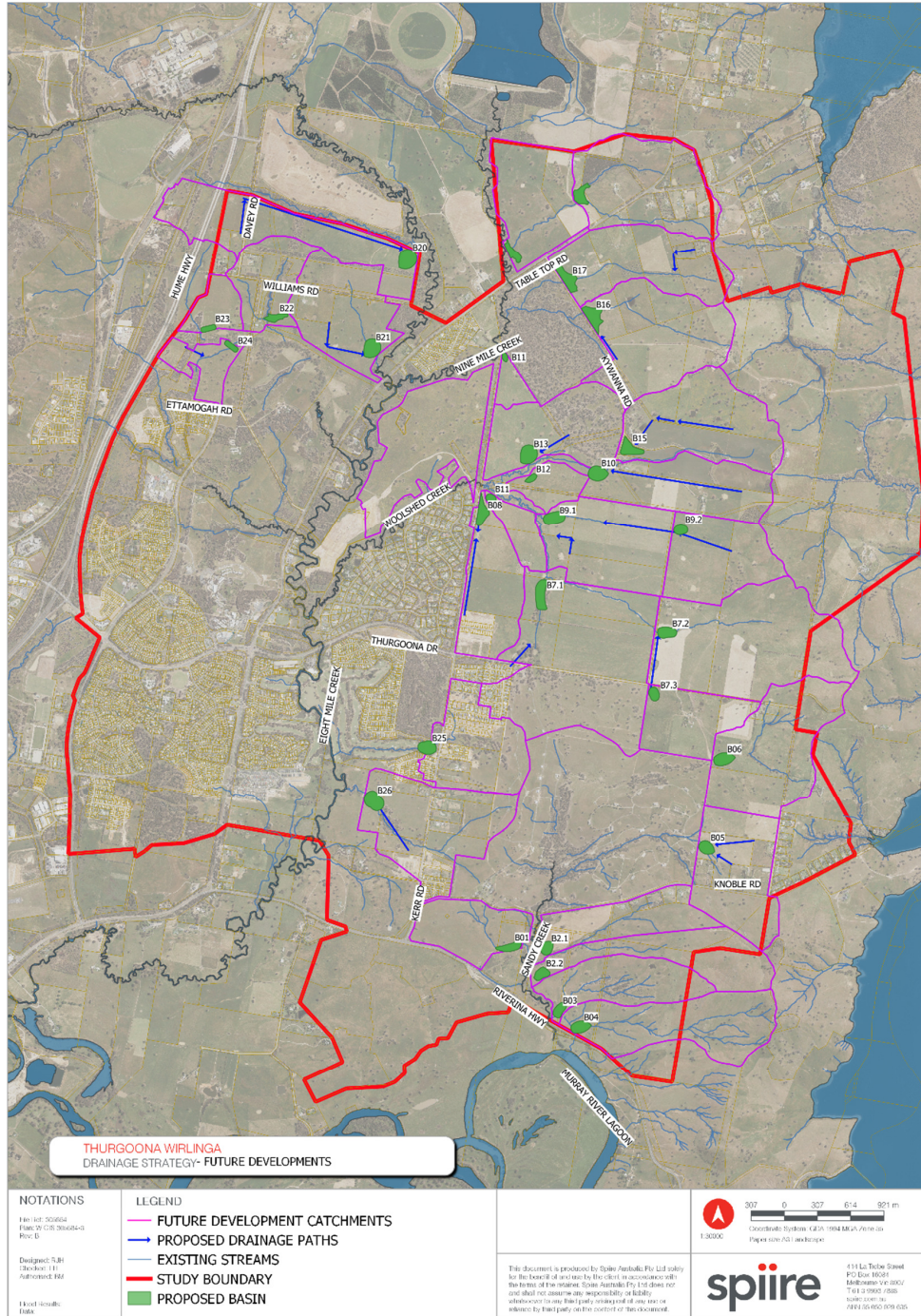
Water Technology, 2024. ***“South Albury Levee Upgrade - Murray River Flood Study”***

WMAwater, 2016. ***“Albury Floodplain Risk Management Study and Draft Plan”***

WMS (Water Modelling Solutions), 2023. ***“Nexus Flood Mitigation Study”***

ANNEXURE B1

EXTRACT FROM SPIIRE, 2020



ANNEXURE B2

EXTRACT FROM WATER TECHNOLOGY, 2024

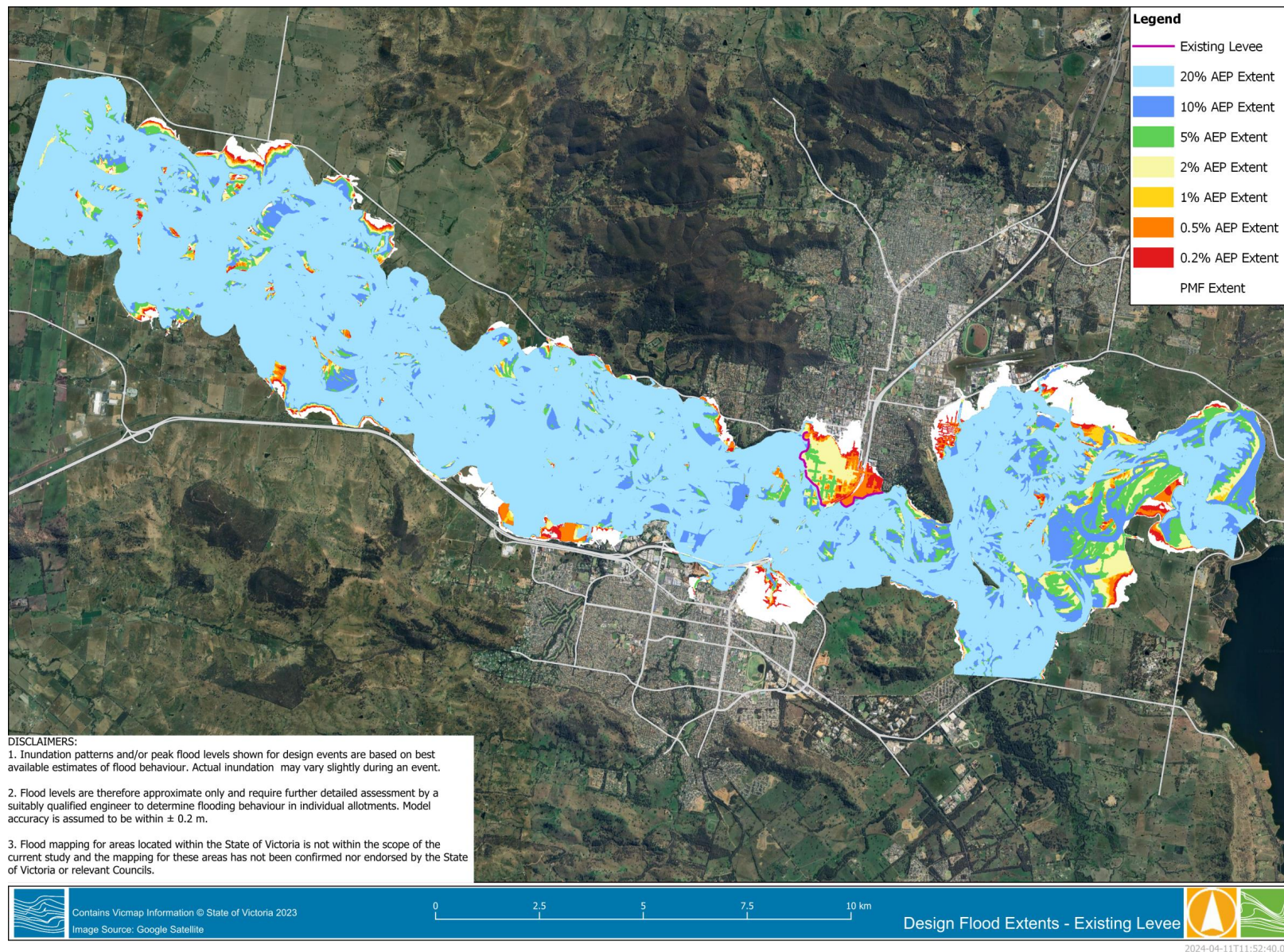
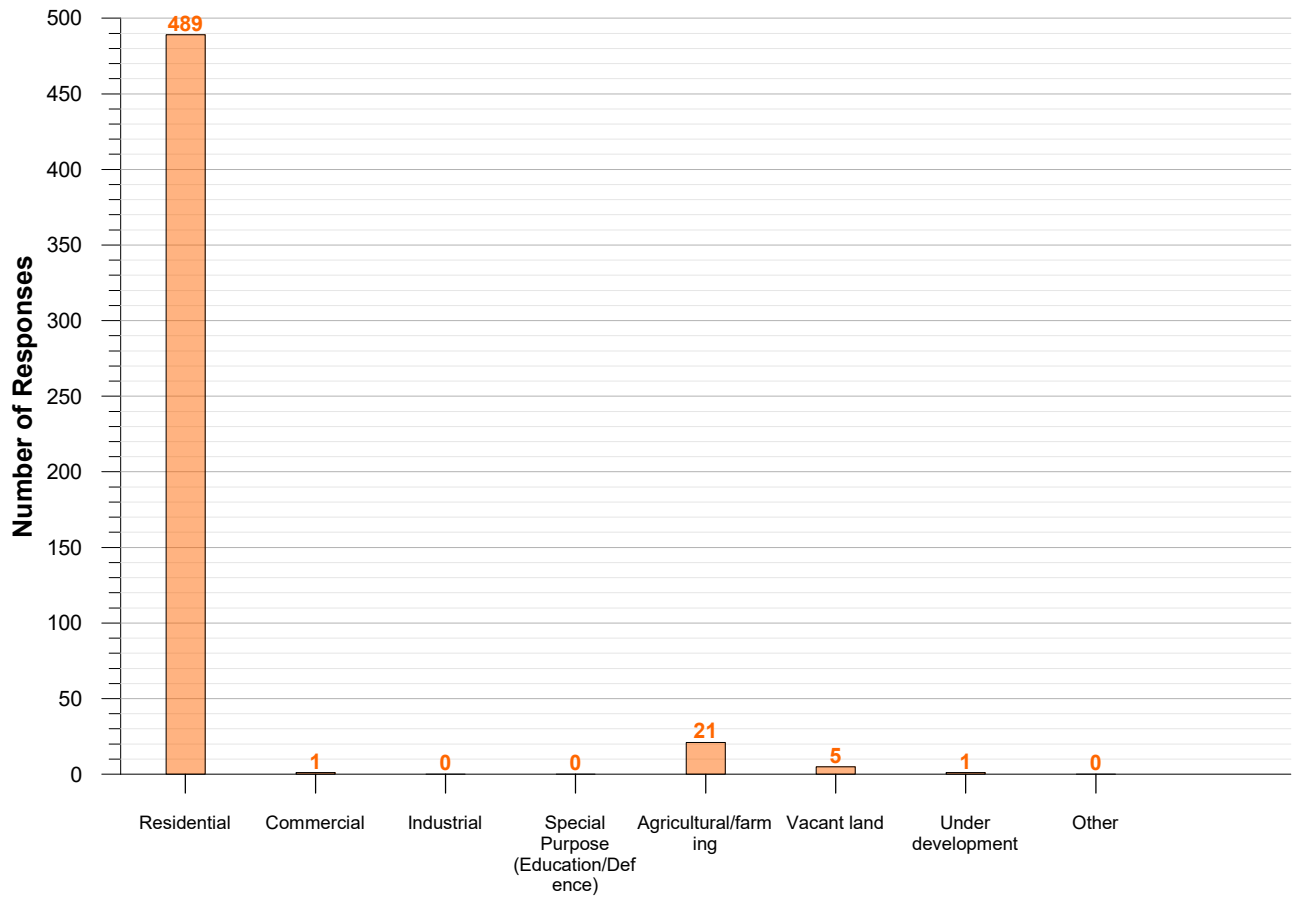


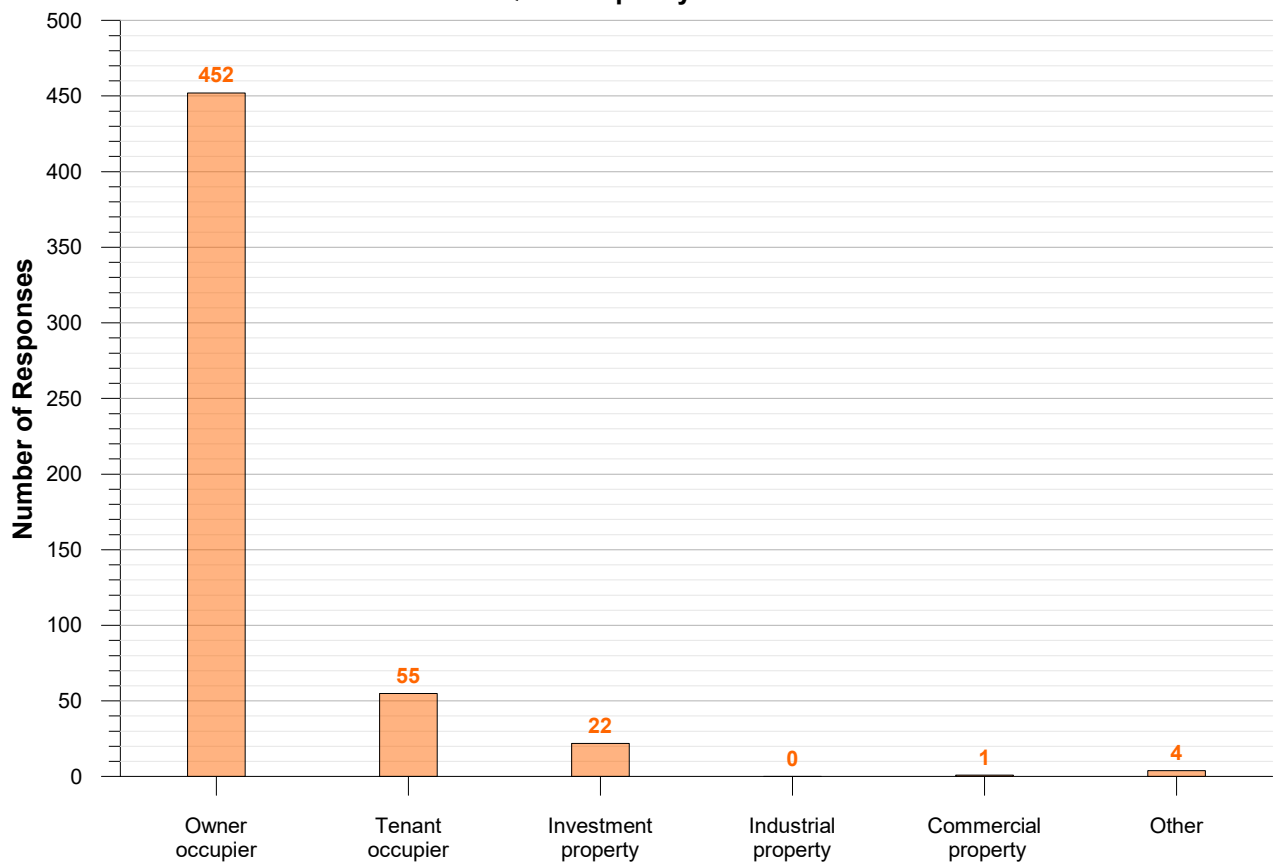
Figure 6-2 Design modelling flood extents for all modelled AEP events – Existing levee conditions

ANNEXURE B3
RESPONSES TO COMMUNITY QUESTIONNAIRE

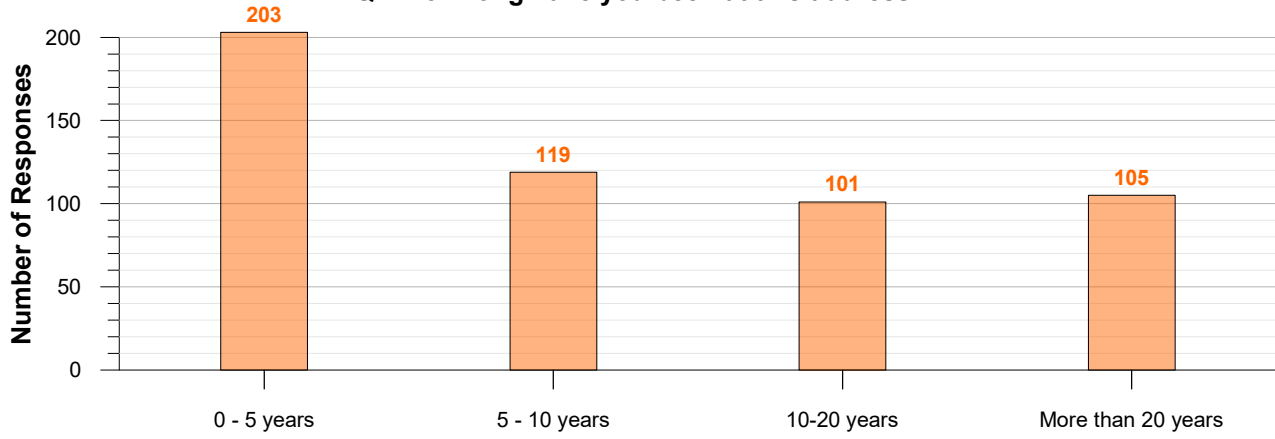
Q2. Type of Property



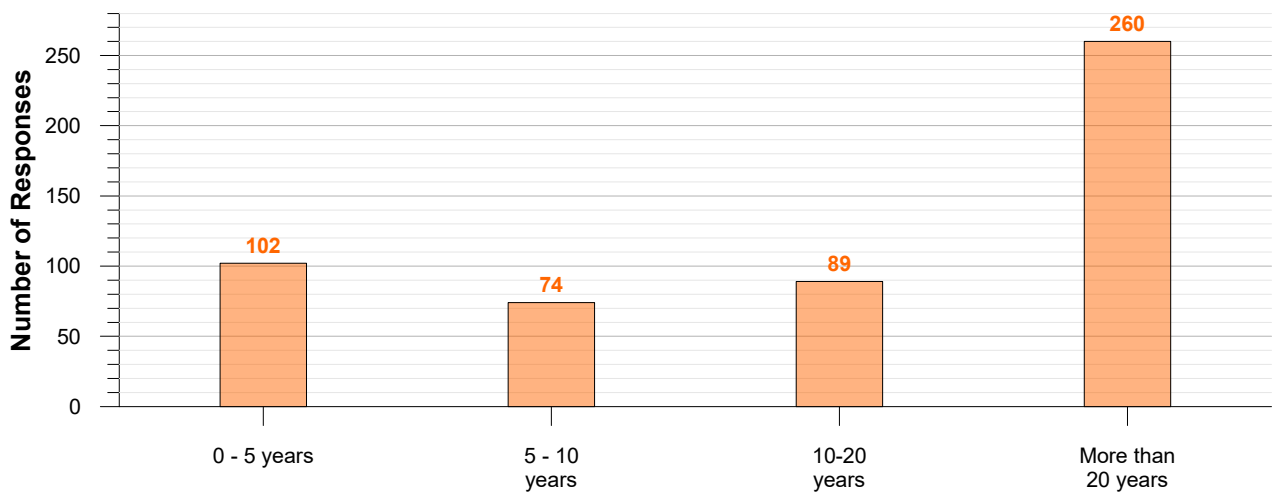
Q3. Occupancy Status



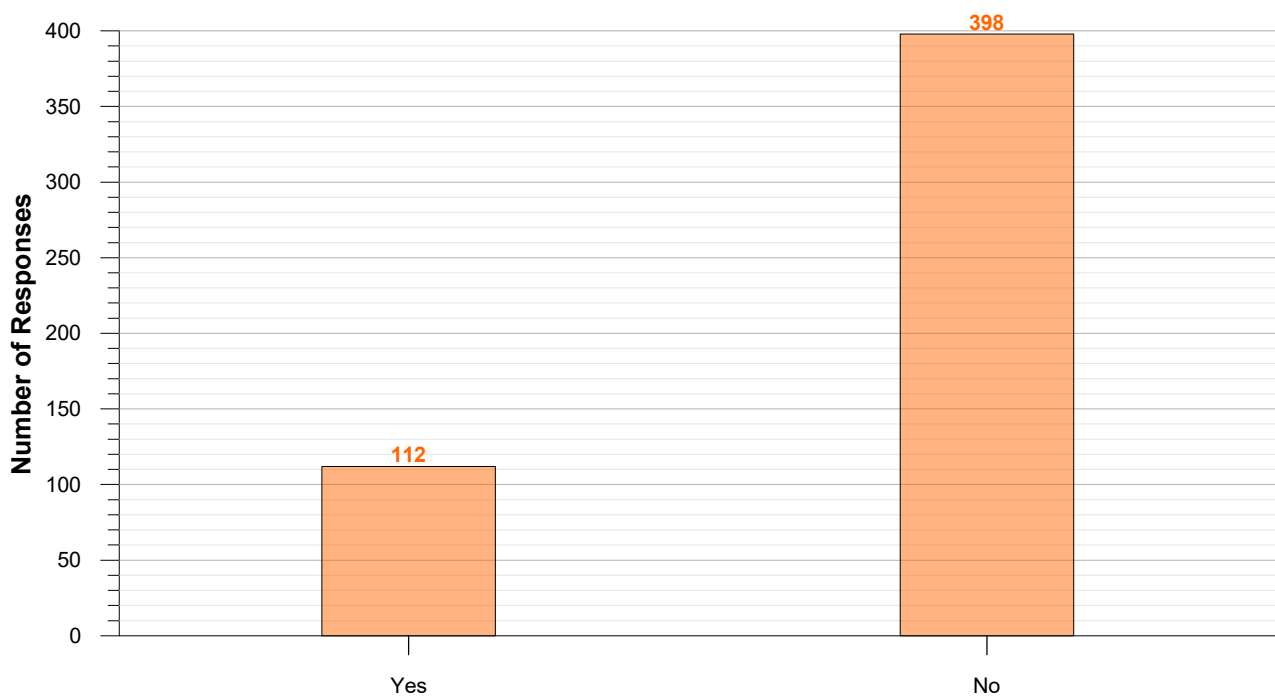
Q4. How long have you been at this address?



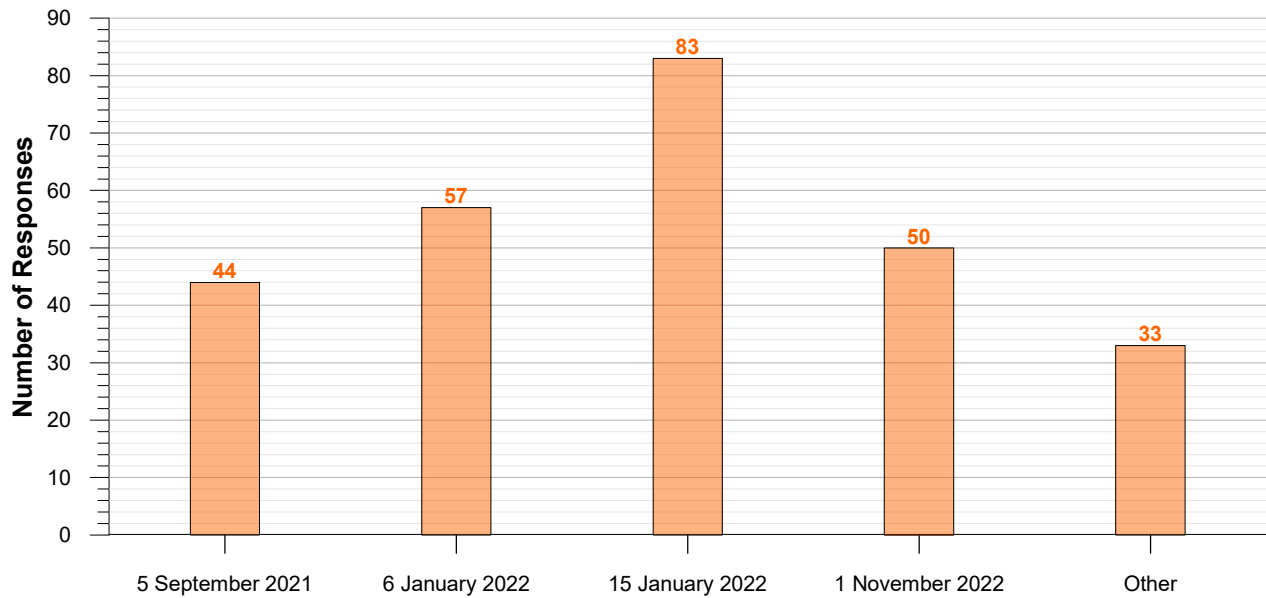
Q5. How long have you lived in the general area?



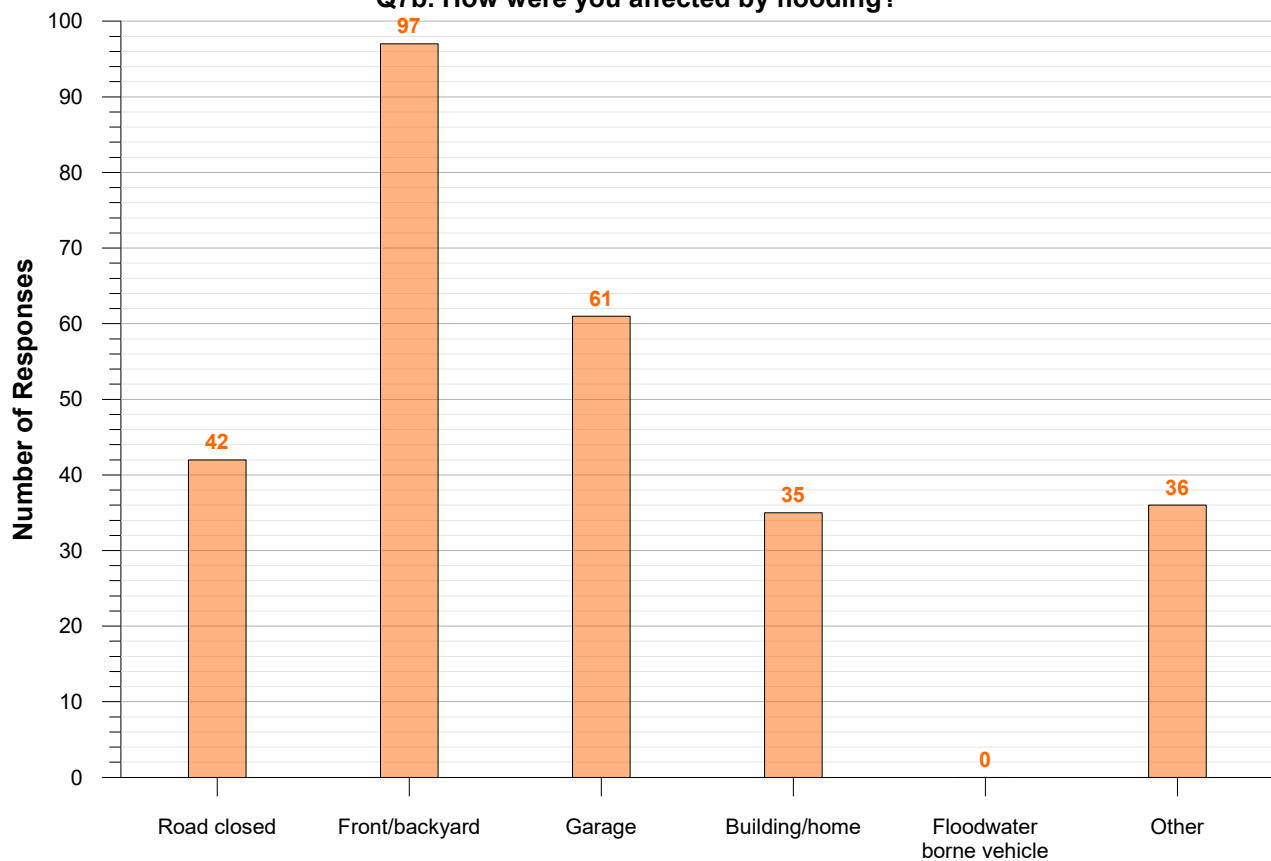
Q6. Have you ever been affected by flooding?



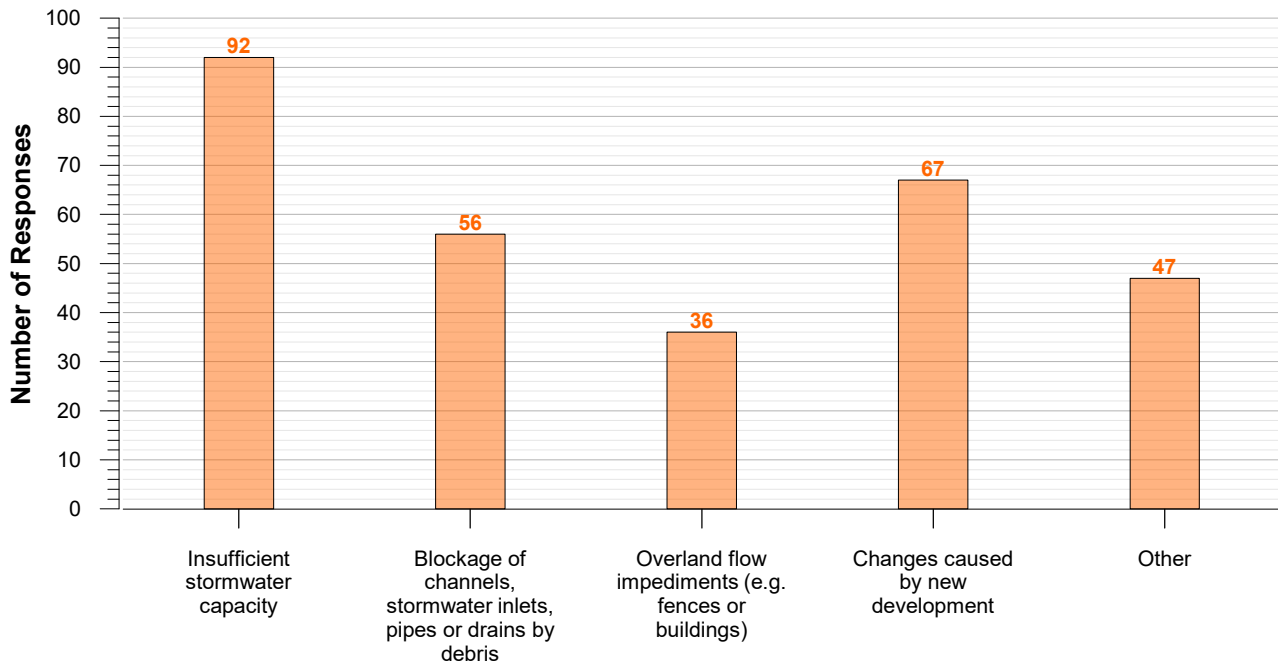
Q7a. On which dates were you affected by flooding?



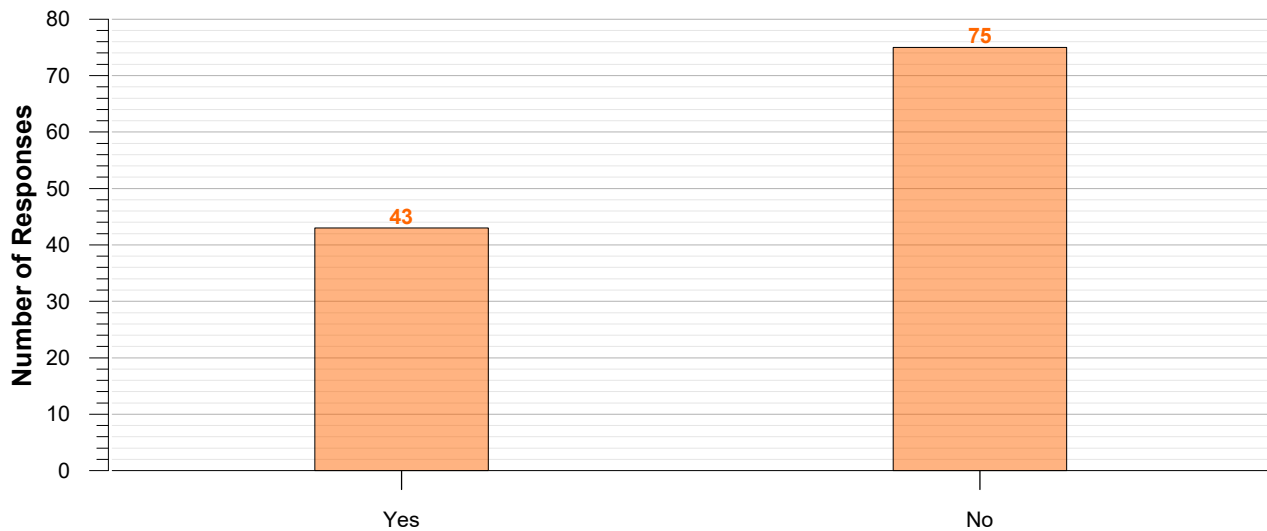
Q7b. How were you affected by flooding?



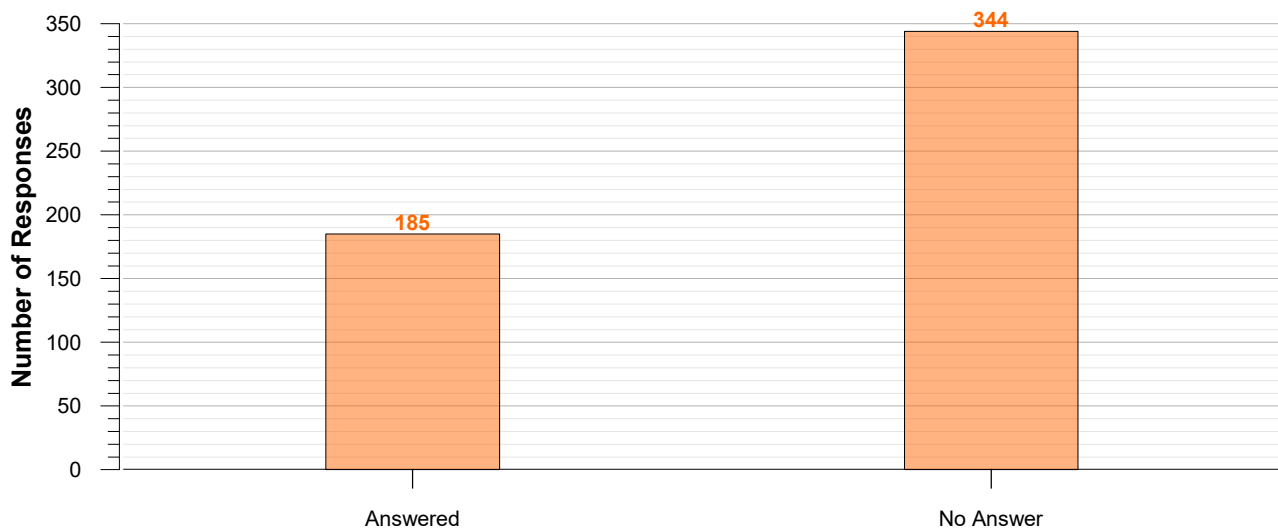
Q8. In your opinion, what was the main cause of the flooding?



Q9. Do you have any photos or videos of these floods?



Q10. Are there any other information to share that may assist the Eight Mile Creek Flood Study Review?



APPENDIX C

PHOTOGRAPHS SHOWING OBSERVED FLOOD BEHAVIOUR IN PARTS OF THE STUDY AREA

16 MARCH 1992
(Source; URS, 2012)



Plate C1.1 – Hawksote Road looking west.



Plate C1.2 – Hawksote Road looking west.



Plate C1.3 – Hawksote Road looking west.



Plate C1.4 – Hawksote Road looking west.



Plate C1.5 – Hawksote Road looking west.



Plate C1.6 –Riverina Highway looking north east.

16 MARCH 1992
(Source; URS, 2012)



Plate C1.7 – End of Hawkscote Road.



Plate C1.8 – End of Hawkscote Road.



Plate C1.9 –Hawkscote Road looking west.



Plate C1.10 – Hawkscote Road looking west.



Plate C1.11 –Hawkscote Road looking west.



Plate C1.12 –Hawkscote Road looking west.

6 OCTOBER 2005



Plate C2.1 – Mungabareena Lagoon looking west from bottom of farm at Hawkscote Road.



Plate C2.2 – Mungabareena Lagoon looking west from bottom of farm at Hawkscote Road.

OCTOBER 2010



Plate C3.1 – Looking west across Eight Mile Creek from Table Top Road.

5 SEPTEMBER 2021



Plate C4.1 – Looking north across Woolshed Creek in the vicinity of Kimball Crescent.



Plate C4.2 – Looking north west across Eight Mile Creek at Thurgoona Golf Club.



Plate C4.3 – Looking north west across Woolshed Creek in the vicinity of Kimball Crescent.



Plate C4.4 – Looking south east towards Thurgoona Drive crossing of Eight Mile Creek.



Plate C4.5 – Looking north east across Woolshed Creek in the vicinity of Kimball Crescent

6 JANUARY 2022



Plate C5.1 – Looking north at floodwater that originated from Eight Mile Creek inundating the rear of a property that is located on the southern side of the Riverina Highway



Plate C5.2 – Looking north at floodwater that originated from Eight Mile Creek inundating the rear of a property that is located on the southern side of the Riverina Highway (Photo taken on the morning of 7 January 2022).

15 JANUARY 2022



Plate C6.1 – Looking south along left side of Gale Court.



Plate C6.2 – Looking north at floodwater that originated from Eight Mile Creek inundating the rear of a property that is located on the southern side of the Riverina Highway (Photo taken on 17 Jan 2022)



Plate C6.3 – Looking west along Sittella Way, approximately 100m to the west of its intersection with Litchfield Drive.



Plate C6.4 – Looking north at the western end of Scott Place.

UNKNOWN DATES



Plate C7.1 – Looking west along the northern side of Stirling Way.



Plate C7.2 – Looking north-west from the southern side of the Riverina Highway, approximately 140m to the east of its intersection with Elizabeth Mitchell Drive.

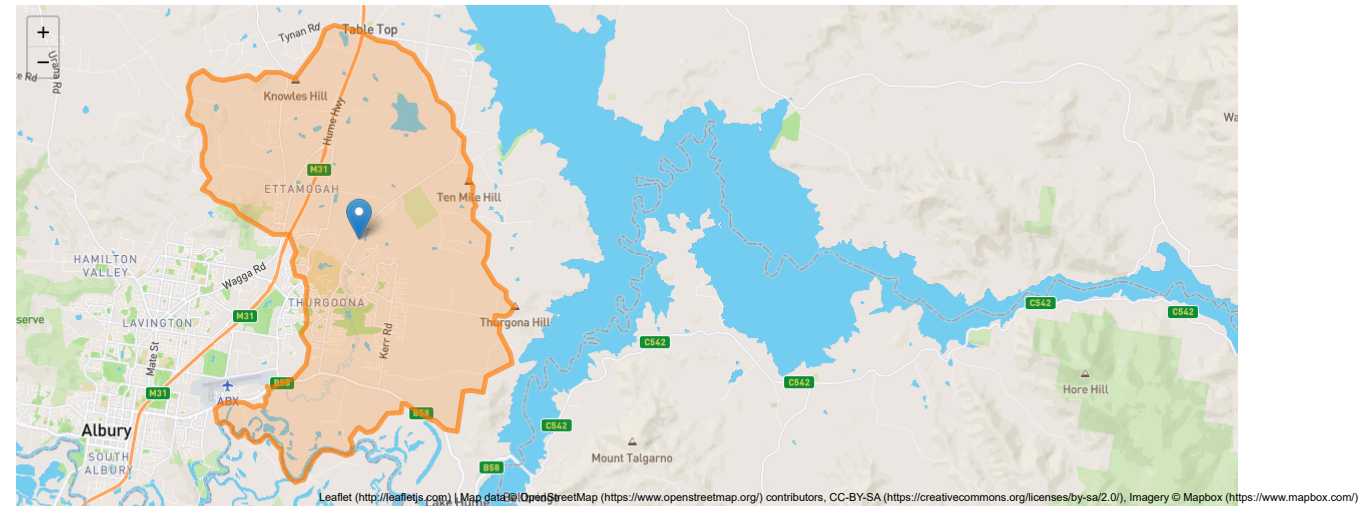
APPENDIX D

DESIGN INPUT DATA FROM ARR DATA HUB

Australian Rainfall & Runoff Data Hub - Results

Input Data

Longitude	147.003
Latitude	-36.028
Selected Regions (clear)	
River Region	show
ARF Parameters	show
Storm Losses	show
Temporal Patterns	show
Areal Temporal Patterns	show
BOM IFDs	show
Median Preburst Depths and Ratios	show
10% Preburst Depths	show
25% Preburst Depths	show
75% Preburst Depths	show
90% Preburst Depths	show
Climate Change Factors	show
Probability Neutral Burst Initial Loss (/nsw_specific)	show



Data

River Region

Division	Murray-Darling Basin
River Number	10
River Name	Murray Riverina
Shape Intersection (%)	97.1

Layer Info

Time Accessed	12 March 2025 09:30AM
Version	2016_v1

ARF Parameters

$$ARF = Min \left\{ 1, \left[1 - a \left(Area^b - c \log_{10} Duration \right) Duration^{-d} \right. \right. \\ \left. \left. + e Area^f Duration^g \left(0.3 + \log_{10} AEP \right) \right. \right. \\ \left. \left. + h 10^{\left(\frac{Area \times Duration}{1440} \right)} \left(0.3 + \log_{10} AEP \right) \right] \right\}$$

Zone	a	b	c	d	e	f	g	h	i	Shape Intersection (%)
Southern Temperate	0.158	0.276	0.372	0.315	0.000141	0.41	0.15	0.01	-0.0027	100.0

Short Duration ARF

$$ARF = Min \left[1, 1 - 0.287 \left(Area^{0.265} - 0.439 \log_{10} (Duration) \right) \cdot Duration^{-0.36} \right. \\ \left. + 2.26 \times 10^{-3} \times Area^{0.226} \cdot Duration^{0.125} \left(0.3 + \log_{10} (AEP) \right) \right. \\ \left. + 0.0141 \times Area^{0.213} \times 10^{-0.021 \frac{(Duration - 180)^2}{1440}} \left(0.3 + \log_{10} (AEP) \right) \right]$$

Layer Info

Time Accessed	12 March 2025 09:30AM
Version	2016_v1

Storm Losses

Note: Burst Loss = Storm Loss - Preburst
Note: These losses are only for rural use and are **NOT FOR DIRECT USE** in urban areas

Note: As this point is in NSW the advice provided on losses and pre-burst on the NSW Specific Tab of the ARR Data Hub (/nsw_specific) is to be considered. In NSW losses are derived considering a hierarchy of approaches depending on the available loss information. The continuing storm loss information from the ARR Datahub provided below should only be used where relevant under the loss hierarchy (level 5) and where used is to be multiplied by the factor of 0.4.

Storm Initial Losses (mm)	26.0
Storm Continuing Losses (mm/h)	4.5

Layer Info

Time Accessed	12 March 2025 09:30AM
Version	2016_v1

Temporal Patterns | Download (.zip) (static/temporal_patterns/TP/MB.zip)

code	MB
Label	Murray Basin
Shape Intersection (%)	100.0

Layer Info

Time Accessed	12 March 2025 09:30AM
Version	2016_v2

Areal Temporal Patterns | Download (.zip) (/static/temporal_patterns/Areal/Areal_MB.zip)

code	MB
arealabel	Murray Basin
Shape Intersection (%)	100.0

Layer Info

Time Accessed	12 March 2025 09:30AM
Version	2016_v2

BOM IFDs

Click here (http://www.bom.gov.au/water/designRainfalls/revised-ifd/?year=2016&coordinate_type=dd&latitude=-36.0275069812&longitude=147.002995854&sdrmin=true&sdrh=true&sdday=true&user_label=) to obtain the IFD depths for catchment centroid from the BoM website

Layer Info

Time Accessed	12 March 2025 09:30AM
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Median Preburst Depths and Ratios

Values are of the format depth (ratio) with depth in mm

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	2.5 (0.128)	1.7 (0.065)	1.2 (0.038)	0.7 (0.019)	0.9 (0.020)	1.0 (0.021)
90 (1.5)	1.9 (0.084)	1.6 (0.053)	1.4 (0.039)	1.2 (0.030)	0.7 (0.015)	0.4 (0.007)
120 (2.0)	5.1 (0.211)	3.1 (0.098)	1.9 (0.050)	0.6 (0.015)	0.4 (0.008)	0.2 (0.004)
180 (3.0)	2.0 (0.073)	2.6 (0.072)	2.9 (0.071)	3.3 (0.070)	1.5 (0.028)	0.2 (0.003)
360 (6.0)	3.2 (0.097)	2.7 (0.062)	2.3 (0.046)	1.9 (0.034)	3.8 (0.058)	5.2 (0.072)
720 (12.0)	0.1 (0.004)	0.5 (0.009)	0.7 (0.012)	0.9 (0.014)	2.1 (0.026)	2.9 (0.033)
1080 (18.0)	0.0 (0.000)	0.3 (0.005)	0.5 (0.008)	0.7 (0.009)	1.6 (0.017)	2.2 (0.021)
1440 (24.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.8 (0.008)	1.4 (0.013)
2160 (36.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
2880 (48.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
4320 (72.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)

Layer Info

Time Accessed	12 March 2025 09:30AM
Version	2018_v1

Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.
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10% Preburst Depths

Values are of the format depth (ratio) with depth in mm

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
90 (1.5)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
120 (2.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
180 (3.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
360 (6.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
720 (12.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
1080 (18.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
1440 (24.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
2160 (36.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
2880 (48.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
4320 (72.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)

Layer Info

Time Accessed	12 March 2025 09:30AM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

25% Preburst Depths

Values are of the format depth (ratio) with depth in mm

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	0.1 (0.004)	0.0 (0.002)	0.0 (0.001)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
90 (1.5)	0.1 (0.003)	0.0 (0.001)	0.0 (0.001)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
120 (2.0)	0.1 (0.004)	0.1 (0.002)	0.0 (0.001)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
180 (3.0)	0.0 (0.001)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
360 (6.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
720 (12.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
1080 (18.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
1440 (24.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
2160 (36.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
2880 (48.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)
4320 (72.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)

Layer Info

Time Accessed	12 March 2025 09:30AM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

75% Preburst Depths

Values are of the format depth (ratio) with depth in mm

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	17.8 (0.906)	16.9 (0.640)	16.4 (0.525)	15.8 (0.442)	14.1 (0.336)	12.8 (0.274)
90 (1.5)	16.9 (0.764)	17.1 (0.579)	17.3 (0.497)	17.4 (0.437)	13.3 (0.285)	10.2 (0.196)
120 (2.0)	18.3 (0.763)	17.9 (0.560)	17.6 (0.471)	17.4 (0.406)	14.9 (0.300)	13.1 (0.237)
180 (3.0)	14.1 (0.522)	15.6 (0.437)	16.5 (0.399)	17.4 (0.369)	14.5 (0.265)	12.3 (0.203)
360 (6.0)	14.0 (0.424)	14.6 (0.339)	15.0 (0.301)	15.3 (0.272)	17.9 (0.275)	19.9 (0.276)
720 (12.0)	5.7 (0.138)	7.9 (0.150)	9.4 (0.155)	10.9 (0.158)	15.1 (0.189)	18.2 (0.206)
1080 (18.0)	2.3 (0.049)	5.0 (0.083)	6.8 (0.099)	8.6 (0.109)	10.7 (0.118)	12.3 (0.122)
1440 (24.0)	0.9 (0.017)	4.2 (0.063)	6.4 (0.084)	8.4 (0.098)	10.1 (0.100)	11.3 (0.101)
2160 (36.0)	0.0 (0.000)	0.6 (0.008)	1.0 (0.012)	1.4 (0.014)	3.1 (0.027)	4.4 (0.034)
2880 (48.0)	0.0 (0.000)	0.2 (0.002)	0.3 (0.003)	0.4 (0.004)	0.5 (0.004)	0.6 (0.004)
4320 (72.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.1 (0.001)

Layer Info

Time Accessed	12 March 2025 09:30AM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

90% Preburst Depths

Values are of the format depth (ratio) with depth in mm

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	42.6 (2.169)	35.9 (1.356)	31.4 (1.008)	27.2 (0.758)	26.8 (0.638)	26.6 (0.567)
90 (1.5)	37.2 (1.682)	37.5 (1.266)	37.6 (1.082)	37.8 (0.948)	30.4 (0.653)	24.9 (0.481)
120 (2.0)	34.2 (1.423)	35.8 (1.119)	36.8 (0.983)	37.8 (0.883)	41.5 (0.832)	44.3 (0.801)
180 (3.0)	26.2 (0.971)	33.7 (0.946)	38.6 (0.931)	43.3 (0.917)	36.5 (0.666)	31.4 (0.517)
360 (6.0)	32.6 (0.984)	30.3 (0.704)	28.7 (0.578)	27.3 (0.484)	35.9 (0.550)	42.3 (0.587)
720 (12.0)	14.1 (0.345)	20.1 (0.381)	24.1 (0.395)	27.9 (0.405)	29.6 (0.370)	30.9 (0.349)
1080 (18.0)	14.6 (0.313)	20.2 (0.336)	23.9 (0.345)	27.5 (0.351)	27.3 (0.299)	27.2 (0.268)
1440 (24.0)	12.6 (0.248)	20.4 (0.309)	25.5 (0.335)	30.4 (0.354)	26.0 (0.259)	22.7 (0.203)
2160 (36.0)	3.0 (0.052)	7.3 (0.097)	10.1 (0.117)	12.8 (0.131)	12.8 (0.111)	12.9 (0.100)
2880 (48.0)	1.7 (0.027)	5.0 (0.061)	7.2 (0.076)	9.3 (0.086)	19.9 (0.157)	27.9 (0.197)
4320 (72.0)	0.0 (0.000)	1.5 (0.017)	2.5 (0.024)	3.5 (0.029)	10.8 (0.076)	16.2 (0.102)

Layer Info

Time Accessed	12 March 2025 09:30AM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

Climate Change Factors

Rainfall Factors

SSP1-2.6

Year	<1 hour	1.5 Hours	2 Hours	3 Hours	4.5 Hours	6 Hours	9 Hours	12 Hours	18 Hours	>24 Hours
2030	1.18	1.17	1.16	1.14	1.13	1.12	1.12	1.11	1.1	1.1
2040	1.21	1.19	1.17	1.16	1.15	1.14	1.13	1.12	1.11	1.11
2050	1.22	1.2	1.18	1.17	1.15	1.15	1.14	1.13	1.12	1.11
2060	1.23	1.21	1.2	1.18	1.17	1.16	1.15	1.14	1.13	1.12
2070	1.24	1.22	1.2	1.18	1.17	1.16	1.15	1.14	1.13	1.12
2080	1.23	1.21	1.2	1.18	1.17	1.16	1.15	1.14	1.13	1.12
2090	1.23	1.21	1.2	1.18	1.17	1.16	1.15	1.14	1.13	1.12
2100	1.22	1.2	1.19	1.17	1.16	1.15	1.14	1.13	1.12	1.12

SSP2-4.5

Year	<1 hour	1.5 Hours	2 Hours	3 Hours	4.5 Hours	6 Hours	9 Hours	12 Hours	18 Hours	>24 Hours
2030	1.18	1.17	1.16	1.14	1.13	1.12	1.12	1.11	1.1	1.1
2040	1.22	1.2	1.19	1.17	1.16	1.15	1.14	1.13	1.12	1.12
2050	1.27	1.24	1.23	1.21	1.19	1.18	1.17	1.16	1.15	1.14
2060	1.3	1.27	1.25	1.23	1.21	1.2	1.19	1.18	1.16	1.16
2070	1.33	1.3	1.28	1.26	1.24	1.22	1.21	1.19	1.18	1.17
2080	1.37	1.33	1.31	1.28	1.26	1.24	1.22	1.21	1.2	1.19
2090	1.4	1.36	1.34	1.31	1.28	1.26	1.24	1.23	1.21	1.2
2100	1.41	1.37	1.35	1.32	1.29	1.27	1.25	1.24	1.22	1.21

SSP3-7.0

Year	<1 hour	1.5 Hours	2 Hours	3 Hours	4.5 Hours	6 Hours	9 Hours	12 Hours	18 Hours	>24 Hours
2030	1.18	1.17	1.16	1.14	1.13	1.12	1.12	1.11	1.1	1.1
2040	1.23	1.21	1.2	1.18	1.17	1.16	1.15	1.14	1.13	1.12
2050	1.29	1.26	1.24	1.22	1.2	1.19	1.18	1.17	1.16	1.15
2060	1.35	1.32	1.3	1.27	1.25	1.23	1.22	1.2	1.19	1.18
2070	1.42	1.38	1.35	1.32	1.29	1.28	1.26	1.24	1.22	1.21
2080	1.5	1.45	1.42	1.38	1.35	1.33	1.3	1.28	1.26	1.25
2090	1.59	1.53	1.49	1.44	1.4	1.38	1.35	1.33	1.3	1.29
2100	1.66	1.59	1.55	1.5	1.45	1.42	1.39	1.37	1.34	1.32

SSP5-8.5

Year	<1 hour	1.5 Hours	2 Hours	3 Hours	4.5 Hours	6 Hours	9 Hours	12 Hours	18 Hours	>24 Hours
2030	1.2	1.18	1.17	1.16	1.14	1.13	1.13	1.12	1.11	1.11
2040	1.26	1.24	1.22	1.2	1.18	1.17	1.16	1.15	1.14	1.14
2050	1.34	1.31	1.29	1.26	1.24	1.23	1.21	1.2	1.18	1.18
2060	1.42	1.38	1.35	1.32	1.29	1.28	1.26	1.24	1.22	1.21
2070	1.52	1.47	1.43	1.4	1.36	1.34	1.31	1.29	1.27	1.26
2080	1.63	1.57	1.52	1.48	1.43	1.4	1.37	1.35	1.33	1.31
2090	1.77	1.69	1.64	1.58	1.52	1.49	1.45	1.42	1.39	1.37
2100	1.86	1.77	1.71	1.64	1.58	1.54	1.5	1.47	1.43	1.41

Loss Factors

Initial Loss (Adjustment Factors)

	Losses SSP1-2.6	Losses SSP2-4.5	Losses SSP3-7.0	Losses SSP5-8.5
2030	1.04	1.04	1.04	1.04
2040	1.04	1.04	1.05	1.05
2050	1.04	1.05	1.06	1.07
2060	1.05	1.06	1.07	1.08
2070	1.05	1.07	1.08	1.1
2080	1.05	1.07	1.09	1.11
2090	1.05	1.07	1.11	1.13
2100	1.04	1.08	1.12	1.15

Continuing Loss (Adjustment Factors)

	Losses SSP1-2.6	Losses SSP2-4.5	Losses SSP3-7.0	Losses SSP5-8.5
2030	1.08	1.08	1.08	1.09

	Losses SSP1-2.6	Losses SSP2-4.5	Losses SSP3-7.0	Losses SSP5-8.5
2040	1.09	1.1	1.1	1.11
2050	1.1	1.11	1.13	1.14
2060	1.1	1.13	1.15	1.18
2070	1.1	1.14	1.18	1.21
2080	1.1	1.16	1.21	1.25
2090	1.1	1.17	1.24	1.3
2100	1.1	1.17	1.27	1.33

Temperature Changes (Degrees, Relative to 1961-1990 Baseline)

Year	SSP1-2.6	SSP2-4.5	SSP3-7.0	SSP5-8.5
2030	1.2	1.2	1.2	1.3
2040	1.3	1.4	1.5	1.6
2050	1.4	1.7	1.8	2.1
2060	1.5	1.9	2.2	2.5
2070	1.5	2.1	2.5	3
2080	1.5	2.2	2.9	3.5
2090	1.5	2.4	3.3	4.1
2100	1.4	2.5	3.6	4.5

Layer Info

Time Accessed	12 March 2025 09:30AM
Version	2024_v1
Note	Updated climate change factors for IFD Initial loss and continuing loss based on IPCC AR6 temperature increases from the updated Climate Change Considerations (Book 1: Chapter 6) in ARR (Version 4.2). ARR recommends the use of Current and near-term (2030 midpoint), Medium-term (2050 midpoint) and Long-term (2090 midpoint)

Probability Neutral Burst Initial Loss

min (h)\AEP(%)	50.0	20.0	10.0	5.0	2.0	1.0
60 (1.0)	17.0	9.0	8.4	9.2	9.5	8.5
90 (1.5)	17.4	9.7	8.8	9.2	9.2	9.0
120 (2.0)	16.4	9.7	9.0	9.4	9.0	7.7
180 (3.0)	18.1	11.1	9.6	9.6	9.0	8.1
360 (6.0)	17.5	11.6	10.6	11.4	9.7	6.2
720 (12.0)	21.5	15.6	14.1	14.2	12.3	7.6
1080 (18.0)	22.3	16.7	15.5	15.6	14.0	10.1
1440 (24.0)	23.1	17.4	16.2	16.3	15.6	11.6
2160 (36.0)	25.4	20.7	20.7	21.2	19.6	15.1
2880 (48.0)	25.9	21.6	21.9	22.0	21.0	12.9
4320 (72.0)	26.6	22.5	23.4	23.9	23.1	17.5

Layer Info

Time Accessed	12 March 2025 09:30AM
Version	2018_v1
Note	As this point is in NSW the advice provided on losses and pre-burst on the NSW Specific Tab of the ARR Data Hub (/nsw_specific) is to be considered. In NSW losses are derived considering a hierarchy of approaches depending on the available loss information. Probability neutral burst initial loss values for NSW are to be used in place of the standard initial loss and pre-burst as per the losses hierarchy.

Download TXT (downloads/62ff0e07-8677-4478-a5b6-5711bbb76220.txt)

Download JSON (downloads/7814b9a4-7fbb-439b-ac42-c51e548a199b.json)

Generating PDF... (downloads/3dc2b4c8-ea97-4faf-9b8d-a748d3f22482.pdf)

APPENDIX E

ARR 2019 DESIGN BLOCKAGE ASSESSMENT AT DRAINAGE STRUCTURES

TABLE E2
ARR, 2019 DESIGN BLOCKAGE ASSESSMENT AT HYDRAULIC DRAINAGE STRUCTURES

ID ⁽¹⁾	Structure Details				Floating Debris																						Adopted Design Blockage B _{DES} %		
	Structure Type ⁽²⁾	Bridge Span / Width / Diameter (m)	Height (m)	No. of Barrels / Spans	L ₁₀ ⁽³⁾	Debris Availability	Debris Mobility	Debris Transportability	Debris Potential	Debris Potential at Structure	Adjusted Debris Potential			Most Likely Design Inlet Blockage (B _{DES} %)			Approximate Average Flow Velocity (m/s)	Likelihood of Deposition	Debris Potential at Structure	Adjusted Debris Potential			Most Likely Design Barrel Blockage (B _{DES} %)						
											> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP				> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP				
EMCPP-2759	R Culvert	2.1	0.5	0	1.5	H	H	H	HHH	High	Medium	High	High	10%	20%	20%	1.1	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%	
EMCPP-2763	R Culvert	2.1	0.5	0	1.5	H	H	H	HHH	High	Medium	High	High	10%	20%	20%	2.8	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%	
EMCPP-2769	C Culvert	0.45	0	0	1.5	H	H	H	HHH	High	Medium	High	High	50%	100%	100%	0.2	Medium	High	Medium	High	High	40%	60%	60%	50%	100%	100%	
EMCPP-2945	R Culvert	2.1	1.8	2	1.5	H	H	H	HHH	High	Medium	High	High	10%	20%	20%	5.7	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%	
EMCPP-2871	R Culvert	2.7	2.56	3	1.5	H	H	H	HHH	High	Medium	High	High	10%	20%	20%	3.3	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%	
EMCPP-2921	C Culvert	0.825	0	0	1.5	H	H	H	HHH	High	Medium	High	High	50%	100%	100%	2.6	Low	High	Medium	High	High	15%	25%	25%	50%	100%	100%	
EMCPP-2935	R Culvert	3.03	2.4	3	1.5	H	H	H	HHH	High	Medium	High	High	10%	20%	20%	2.9	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%	
EMCPP-2943	R Culvert	2.7	1.8	3	1.5	H	H	H	HHH	High	Medium	High	High	10%	20%	20%	3.4	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%	
EMC_6	C Culvert	0.75	0	1	1.5	H	H	H	HHH	High	Medium	High	High	50%	100%	100%	2.3	Low	High	Medium	High	High	15%	25%	25%	50%	100%	100%	
EMCPP-0296	C Culvert	1.5	0	0	1.5	H	H	H	HHH	High	Medium	High	High	10%	20%	20%	2.2	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%	
EMCPP-2707	C Culvert	0.3	0	0	1.5	H	H	H	HHH	High	Medium	High	High	50%	100%	100%	2.3	Low	High	Medium	High	High	15%	25%	25%	50%	100%	100%	
EMCPP-2960	R Culvert	3.2	2.7	2	1.5	H	H	H	HHH	High	Medium	High	High	10%	20%	20%	2.7	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%	
EMCPP-2959	R Culvert	2.4	1.6	2	1.5	H	H	H	HHH	High	Medium	High	High	10%	20%	20%	2.2	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%	
EMCPP-0584	C Culvert	1.65	0	0	1.5	H	H	H	HHH	High	Medium	High	High	10%	20%	20%	4.4	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%	
EMCPP-2712	R Culvert	1.8	1.2	3	1.5	H	H	H	HHH	High	Medium	High	High	10%	20%	20%	1.5	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%	
EMCPP-2741	R Culvert	3.6	3.2	2	1.5	H	H	H	HHH	High	Medium	High	High	10%	20%	20%	1.3	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%	
EMCPP-2826	R Culvert	3.65	2.95	2	1.5	H	H	H	HHH	High	Medium	High	High	10%	20%	20%	1.3	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%	
EMCPP-2827	R Culvert	3.56	2.95	0	1.5	H	H	H	HHH	High	Medium	High	High	10%	20%	20%	1.3	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%	
EMCPP-2828	R Culvert	2.4	2.55	2	1.5	H	H	H	HHH	High	Medium	High	High	10%	20%	20%	1.3	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%	
EMCPP-2829	R Culvert	2.36	2.36	2	1.5	H	H	H	HHH	High	Medium	High	High	10%	20%	20%	1.2	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%	
EMCPP-2660	R Culvert	2.1	1.8	2	1.5	H	H	H	HHH	High	Medium	High	High	10%	20%	20%	2	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%	
WCPP-699	C Culvert	0.3	0	0	1.5	H	H	H	HHH	High	Medium	High	High	50%	100%	100%	2.2	Low	High	Medium	High	High	15%	25%	25%	50%	100%	100%	
WCPP-710	R Culvert	3	2.8	4	1.5	H	H	H	HHH	High	Medium	High	High	10%	20%	20%	1.7	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%	
EMCPP-2951	C Culvert	0.45	0	0	1.5	H	H	H	HHH	High	Medium	High	High	50%	100%	100%	3.3	Low	High	Medium	High	High	15%	25%	25%	50%	100%	100%	
EMCPP-2952	C Culvert	0.45	0	0	1.5	H	H	H	HHH	High	Medium	High	High	50%	100%	100%	2	Low	High	Medium	High	High	15%	25%	25%	50%	100%	100%	
EMCPP-2738	C Culvert	0.5	0	2	1.5	H	H	H	HHH	High	Medium	High	High	50%	100%	100%	3.2	Low	High	Medium	High	High	15%	25%	25%	50%	100%	100%	
EMCPP-2819	C Culvert	0.75	0	0	1.5	H	H	H	HHH	High	Medium	High	High	50%	100%	100%	3.2	Low	High	Medium	High	High	15%	25%	25%	50%	100%	100%	
WCPP-696	C Culvert	1.82	0	2	1.5	H	H	H	HHH	High	Medium	High	High	10%	20%	20%	2.4	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%	
EMCPP-2832	R Culvert	1.9	1.2	3	1.5	H	H	H	HHH	High	Medium	High	High	10%	20%	20%	1.7	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%	
EMCPP-2790	C Culvert	0.525	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.6	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2794	C Culvert	0.375	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.9	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2801	C Culvert	0.375	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.2	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2804	C Culvert	0.375	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.3	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2805	C Culvert	0.675	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.5	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2807	C Culvert	0.375	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.8	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2808	C Culvert	0.675	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.3	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-0783	C Culvert	0.6	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.1	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-0789	C Culvert	0.3	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.5	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-0792	C Culvert	0.6	0	3	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2793	C Culvert	0.3	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	0.8	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMC_3	C Culvert	0.45	0	1	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.6	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2746	C Culvert	0.525	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	0	Medium	Medium	Low	Medium	High	15%	40%	60%	25%	50%	100%	
EMCPP-2747	C Culvert	0.375	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.5	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2757	C Culvert	0.375	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2728	C Culvert	0.6	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.8	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2729	C Culvert	0.6	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	3	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2937	R Culvert	1.53	1.53	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	0%	10%	20%	5.4	Low	Medium	Low	Medium	High	0%	15%	25%	0%	15%	25%	
EMCPP-2938	R Culvert	1.53	1.53	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	0%	10%	20%	5.4	Low	Medium	Low	Medium	High	0%	15%	25%	0%	15%	25%	
EMCPP-2762	R Culvert	1.2	0.45	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	10													

TABLE E2
ARR, 2019 DESIGN BLOCKAGE ASSESSMENT AT HYDRAULIC DRAINAGE STRUCTURES

ID ⁽¹⁾	Structure Details				Floating Debris																				Adopted Design Blockage B _{DES} %			
	Structure Type ⁽²⁾	Bridge Span / Width / Diameter (m)	Height (m)	No. of Barrels / Spans	L ₁₀ ⁽³⁾	Debris Availability	Debris Mobility	Debris Transportability	Debris Potential	Debris Potential at Structure	Adjusted Debris Potential			Most Likely Design Inlet Blockage (B _{DES} %)			Approximate Average Flow Velocity (m/s)	Likelihood of Deposition	Debris Potential at Structure	Adjusted Debris Potential			Most Likely Design Barrel Blockage (B _{DES} %)					
											> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP				> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP
EMCPP-2949	C Culvert	0.6	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.1	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2950	C Culvert	0.9	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.2	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-0500	C Culvert	0.45	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.1	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-1356	C Culvert	0.825	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.2	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-1670	C Culvert	0.6	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.1	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-0774	C Culvert	0.3	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	0	Medium	Medium	Low	Medium	High	15%	40%	60%	25%	50%	100%
EMCPP-2719	C Culvert	1.8	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	0%	10%	20%	2.3	Low	Medium	Low	Medium	High	0%	15%	25%	0%	15%	25%
EMCPP-2723	R Culvert	2.45	2.45	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	0%	10%	20%	1.1	Low	Medium	Low	Medium	High	0%	15%	25%	0%	15%	25%
EMCPP-2726	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	0	Medium	Medium	Low	Medium	High	15%	40%	60%	25%	50%	100%
EMCPP-2868	C Culvert	0.75	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.1	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2870	R Culvert	2.45	2.45	1	1.5	L	H	L	LHL	Medium	Low	Medium	High	0%	10%	20%	2.1	Low	Medium	Low	Medium	High	0%	15%	25%	0%	15%	25%
EMCPP-2936	R Culvert	1.51	1.51	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	0%	10%	20%	2.8	Low	Medium	Low	Medium	High	0%	15%	25%	0%	15%	25%
EMCPP-2731	R Culvert	1.5	1.5	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	0%	10%	20%	2.7	Low	Medium	Low	Medium	High	0%	15%	25%	0%	15%	25%
EMCPP-2670	C Culvert	0.525	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.7	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2672	C Culvert	0.525	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.8	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2674	C Culvert	0.3	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.8	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2675	C Culvert	0.45	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.1	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2676	C Culvert	0.6	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	0.9	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-1673	C Culvert	0.3	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.7	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2686	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2705	C Culvert	1.2	0	3	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.7	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2677	R Culvert	1.8	0.5	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	0%	10%	20%	0.7	Low	Medium	Low	Medium	High	0%	15%	25%	0%	15%	25%
EMCPP-2684	C Culvert	0.3	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.4	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2665	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.9	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2666	C Culvert	0.9	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	3.1	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2667	C Culvert	0.3	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.9	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2668	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.8	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2669	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	3.9	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2671	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	0.9	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2673	C Culvert	0.675	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	3	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2727	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	0.8	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2709	R Culvert	0.9	0.45	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	3.6	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2713	R Culvert	1.5	1.5	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	0%	10%	20%	2.3	Low	Medium	Low	Medium	High	0%	15%	25%	0%	15%	25%
EMCPP-2717	R Culvert	1.2	1.2	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.1	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2836	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	0	Medium	Medium	Low	Medium	High	15%	40%	60%	25%	50%	100%
EMCPP-2724	C Culvert	0.3	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.8	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2859	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.9	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2739	R Culvert	2.1	0.5	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	0%	10%	20%	4.8	Low	Medium	Low	Medium	High	0%	15%	25%	0%	15%	25%
EMCPP-2742	R Culvert	2.1	0.5	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	0%	10%	20%	4.1	Low	Medium	Low	Medium	High	0%	15%	25%	0%	15%	25%
EMCPP-2690	C Culvert	0.75	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2701	R Culvert	0.9	0.45	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	4	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2703	C Culvert	0.6	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.5	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2836	R Culvert	0.62	0.24	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	0.7	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2837	R Culvert	0.57	0.6	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.4	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-0233	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	0.9	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-0887	C Culvert	0.3	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	3.6	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2834	R Culvert	2.74	2.2	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	0%	10%	20%	3.2	Low	Medium	Low	Medium	High	0%	15%	25%	0%	15%	25%
EMCPP-1791	C Culvert	0.3	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.8	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-0847	C Culvert	0.375	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	0	Medium	Medium	Low	Medium	High	15%	40%				

TABLE E2
ARR, 2019 DESIGN BLOCKAGE ASSESSMENT AT HYDRAULIC DRAINAGE STRUCTURES

ID ⁽¹⁾	Structure Details				Floating Debris																				Adopted Design Blockage B _{DES} %			
	Structure Type ⁽²⁾	Bridge Span / Width / Diameter (m)	Height (m)	No. of Barrels / Spans	L ₁₀ ⁽³⁾	Debris Availability	Debris Mobility	Debris Transportability	Debris Potential	Debris Potential at Structure	Adjusted Debris Potential			Most Likely Design Inlet Blockage (B _{DES} %)			Approximate Average Flow Velocity (m/s)	Likelihood of Deposition	Debris Potential at Structure	Adjusted Debris Potential			Most Likely Design Barrel Blockage (B _{DES} %)					
											> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP				> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP			
											> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP				> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP			
EMCPP-2740	C Culvert	0.375	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.5	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2744	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.2	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2745	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.4	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2748	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.1	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2831	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.5	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2736	C Culvert	0.3	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	0.7	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2737	C Culvert	0.3	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	0.1	Medium	Medium	Low	Medium	High	15%	40%	60%	25%	50%	100%
EMCPP-2825	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	0.9	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2730	C Culvert	0.75	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	3	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2733	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.5	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2954	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.1	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2955	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-0129	R Culvert	3.5	2.45	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	0%	10%	20%	1.7	Low	Medium	Low	Medium	High	0%	15%	25%	0%	15%	25%
WCPP-093	C Culvert	0.3	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.5	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
WCPP-117	R Culvert	0.9	0.3	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.7	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
WCPP-203	C Culvert	0.3	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.5	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
WCPP-290	C Culvert	0.375	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.1	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
WCPP-700	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.6	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2811	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2812	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.3	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2813	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.1	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2775	C Culvert	0.6	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.4	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2776	C Culvert	0.6	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.8	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2778	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.2	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2779	C Culvert	0.3	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.6	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2780	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	0	Medium	Medium	Low	Medium	High	15%	40%	60%	25%	50%	100%
EMCPP-2782	C Culvert	0.4	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.3	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2783	C Culvert	0.6	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.1	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2785	C Culvert	0.375	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.2	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2788	C Culvert	0.45	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.5	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2791	C Culvert	0.45	0	3	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.4	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2784	R Culvert	1.2	0.46	3	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	4	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2787	C Culvert	0.45	0	3	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2796	C Culvert	0.75	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.5	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2799	C Culvert	0.75	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.5	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2806	C Culvert	0.45	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.5	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2809	C Culvert	0.75	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	3.3	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2821	C Culvert	0.75	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	0	Medium	Medium	Low	Medium	High	15%	40%	60%	25%	50%	100%
EMCPP-2822	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.4	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2774	C Culvert	0.6	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.5	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
WCPP-008	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.9	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
WCPP-170	C Culvert	0.825	0	3	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.4	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
WCPP-259	C Culvert	0.9	0	3	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.5	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-0637	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.9	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2050	C Culvert	0.6	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.5	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2455	C Culvert	0.6	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.5	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2768	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.3	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2770	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	3.2	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%
EMCPP-2753	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.1	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	10

TABLE E2
ARR, 2019 DESIGN BLOCKAGE ASSESSMENT AT HYDRAULIC DRAINAGE STRUCTURES

ID ⁽¹⁾	Structure Details				Floating Debris																						Adopted Design Blockage B _{DES} %		
	Structure Type ⁽²⁾	Bridge Span / Width / Diameter (m)	Height (m)	No. of Barrels / Spans	L ₁₀ ⁽³⁾	Debris Availability	Debris Mobility	Debris Transportability	Debris Potential	Debris Potential at Structure	Adjusted Debris Potential			Most Likely Design Inlet Blockage (B _{DES} %)			Approximate Average Flow Velocity (m/s)	Likelihood of Deposition	Debris Potential at Structure	Adjusted Debris Potential			Most Likely Design Barrel Blockage (B _{DES} %)						
											> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP				> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP				
EMCPP-2798	C Culvert	0.9	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.8	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2800	C Culvert	0.6	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.5	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2802	C Culvert	0.9	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	0	Medium	Medium	Low	Medium	High	15%	40%	60%	25%	50%	100%	
EMCPP-2816	C Culvert	0.6	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	0	Medium	Medium	Low	Medium	High	15%	40%	60%	25%	50%	100%	
EMCPP-2946	C Culvert	2.55	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	0%	10%	20%	2.8	Low	Medium	Low	Medium	High	0%	15%	25%	0%	15%	25%	
EMCPP-2947	C Culvert	2.25	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	0%	10%	20%	2.8	Low	Medium	Low	Medium	High	0%	15%	25%	0%	15%	25%	
EMCPP-2714	R Culvert	1.2	0.6	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.1	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2715	C Culvert	0.6	0	3	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.6	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2820	R Culvert	0.6	0.3	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.5	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2971	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.8	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-0636	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.8	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-1280	C Culvert	1.05	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	3.1	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-1482	C Culvert	0.45	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2689	R Culvert	1.8	1.2	3	1.5	L	H	L	LHL	Medium	Low	Medium	High	0%	10%	20%	1.6	Low	Medium	Low	Medium	High	0%	15%	25%	0%	15%	25%	
EMCPP-2696	C Culvert	0.75	0	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.4	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2708	C Culvert	0.6	0	0	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	2.5	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2777	C Culvert	1.2	0	4	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	4.7	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2700	R Culvert	2	1.5	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	0%	10%	20%	3.5	Low	Medium	Low	Medium	High	0%	15%	25%	0%	15%	25%	
EMCPP-0762	R Culvert	1.8	1.6	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	0%	10%	20%	1.5	Low	Medium	Low	Medium	High	0%	15%	25%	0%	15%	25%	
EMCPP-2810	C Culvert	0.75	0	3	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	1.7	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-2792	C Culvert	0.6	0	3	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	3	Low	Medium	Low	Medium	High	0%	15%	25%	25%	50%	100%	
EMCPP-0162	C Culvert	0.9	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2.2	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%	
EMCPP-0401	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.1	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%	
EMCPP-0669	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	0	Medium	Low	Low	Low	Medium	15%	15%	40%	25%	25%	50%	
EMC_14	C Culvert	0.45	0	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.2	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%	
EMC_1	C Culvert	0.45	0	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%	
EMCPP-0671	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2.4	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%	
EMCPP-1083	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	0	Medium	Low	Low	Low	Medium	15%	15%	40%	25%	25%	50%	
EMCPP-0215	C Culvert	0.375	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.2	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%	
EMCPP-2735	C Culvert	0.45	0	4	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.6	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%	
EMCPP-2941	C Culvert	0.45	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	0.6	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%	
EMCPP-2942	C Culvert	0.45	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	0.9	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%	
EMCPP-2752	R Culvert	1	0.9	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	6.1	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%	
EMCPP-2754	C Culvert	0.45	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	3.5	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%	
EMCPP-2944	C Culvert	0.45	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	3.2	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%	
EMCPP-0163	C Culvert	0.9	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.1	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%	
EMCPP-0503	C Culvert	0.9	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2.1	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%	
EMCPP-2750	R Culvert	1.5	0.6	2	1.5	L	M	L	LML	Low	Low	Low	Medium	0%	0%	10%	4.3	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%	
EMCPP-2751	C Culvert	0.45	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2.5	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%	
EMCPP-2939	R Culvert	1.8	1.8	0	1.5	L	M	L	LML	Low	Low	Low	Medium	0%	0%	10%	2.3	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%	
EMCPP-2940	R Culvert	1.8	1.8	0	1.5	L	M	L	LML	Low	Low	Low	Medium	0%	0%	10%	3.2	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%	
EMCPP-2694	R Culvert	1.8	0.6	0	1.5	L	M	L	LML	Low	Low	Low	Medium	0%	0%	10%	3.8	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%	
EMCPP-0206	C Culvert	0.9	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	4.5	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%	
EMCPP-0868	C Culvert	0.375	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%	
EMCPP-2688	C Culvert	0.525	0	2	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.3	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%	
EMCPP-2691	R Culvert	1.5	0.6	0	1.5	L	M	L	LML	Low	Low	Low	Medium	0%	0%	10%	2.2	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%	
EMCPP-2697	C Culvert	0.45	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	0	Medium	Low	Low	Low	Medium	15%	15%	40%	25%	25%	50%	
EMCPP-1304	C Culvert	0.525	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2.6	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%	
EMCPP-2695	C Culvert	0.75	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	3.4	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%	
EMCPP-2956	C Culvert	0.45	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%													

TABLE E2
ARR, 2019 DESIGN BLOCKAGE ASSESSMENT AT HYDRAULIC DRAINAGE STRUCTURES

ID ⁽¹⁾	Structure Details				Floating Debris																						Adopted Design Blockage B _{DES} %		
	Structure Type ⁽²⁾	Bridge Span / Width / Diameter (m)	Height (m)	No. of Barrels / Spans	L ₁₀ ⁽³⁾	Debris Availability	Debris Mobility	Debris Transportability	Debris Potential	Debris Potential at Structure	Adjusted Debris Potential			Most Likely Design Inlet Blockage (B _{DES} %)			Approximate Average Flow Velocity (m/s)	Likelihood of Deposition	Debris Potential at Structure	Adjusted Debris Potential			Most Likely Design Barrel Blockage (B _{DES} %)						
											> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP				> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP				
											> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP				> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP				
EMCPP-2698	R Culvert	3	0.9	2	1.5	L	M	L	LML	Low	Low	Low	Medium	0%	0%	10%	2.4	Low	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%
EMCPP-2353	C Culvert	0.6	0	2	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.4	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-0648	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.9	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-2704	R Culvert	1.8	0.9	2	1.5	L	M	L	LML	Low	Low	Low	Medium	0%	0%	10%	1.9	Low	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%
EMCPP-2706	C Culvert	0.675	0	2	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.8	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-2716	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.7	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-0933	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	0	Medium	Low	Low	Low	Low	Medium	15%	15%	40%	25%	25%	50%
EMCPP-0938	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	0	Medium	Low	Low	Low	Low	Medium	15%	15%	40%	25%	25%	50%
EMCPP-0213	C Culvert	0.375	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.4	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-2957	C Culvert	0.45	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2.1	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMC_4	C Culvert	0.375	0	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	0.9	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-2958	C Culvert	0.45	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2.2	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-0160	C Culvert	0.525	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2.4	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-0399	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2.2	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-0585	C Culvert	1.65	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	0%	0%	10%	1.8	Low	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%
EMCPP-0590	C Culvert	0.45	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.6	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-2711	C Culvert	0.45	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2.1	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-2830	C Culvert	0.45	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	0	Medium	Low	Low	Low	Low	Medium	15%	15%	40%	25%	25%	50%
EMCPP-2710	C Culvert	0.3	0	2	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.6	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-0591	C Culvert	0.45	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2.1	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMC_8	C Culvert	0.3	0	2	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.5	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMC_9	C Culvert	0.45	0	4	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.2	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-0116	C Culvert	0.525	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	3.1	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-0775	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.5	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-1883	C Culvert	0.225	0	3	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-2414	C Culvert	1.35	0	2	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2.5	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-2953	C Culvert	0.6	0	2	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2.2	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-2749	C Culvert	0.45	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.7	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-2993	C Culvert	0.3	0	2	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-1242	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	3.1	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
WCPP-003	R Culvert	0.9	0.3	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.6	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
WCPP-116	R Culvert	0.9	0.3	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.3	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
WCPP-346	R Culvert	0.9	0.3	2	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.4	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
WCPP-437	R Culvert	0.9	0.3	2	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.4	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
WCPP-021	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.1	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
WCPP-059	C Culvert	0.1	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.2	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
WCPP-074	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2.9	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
WCPP-161	C Culvert	0.525	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.7	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
WCPP-392	C Culvert	0.45	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.5	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
WCPP-421	C Culvert	0.75	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.3	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMC_10	C Culvert	0.45	0	2	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.5	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
WCPP-110	C Culvert	0.6	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.6	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
WCPP-187	C Culvert	0.6	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2.9	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-0094	R Culvert	0.45	0.45	2	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	0.2	Medium	Low	Low	Low	Low	Medium	15%	15%	40%	25%	25%	50%
EMCPP-0265	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2.2	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-0285	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2.7	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-1494	C Culvert	0.65	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	3	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-0292	C Culvert	0.525	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.4	Low	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-0824	C Culvert	0.375	0	0																									

TABLE E2
ARR, 2019 DESIGN BLOCKAGE ASSESSMENT AT HYDRAULIC DRAINAGE STRUCTURES

ID ⁽¹⁾	Structure Details				Floating Debris																					Adopted Design Blockage B _{DES} %		
	Structure Type ⁽²⁾	Bridge Span / Width / Diameter (m)	Height (m)	No. of Barrels / Spans	L ₁₀ ⁽³⁾	Debris Availability	Debris Mobility	Debris Transportability	Debris Potential	Debris Potential at Structure	Adjusted Debris Potential			Most Likely Design Inlet Blockage (B _{DES} %)			Approximate Average Flow Velocity (m/s)	Likelihood of Deposition	Debris Potential at Structure	Adjusted Debris Potential			Most Likely Design Barrel Blockage (B _{DES} %)					
											> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP				> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP			
EMCPP-0856	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2.5	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-0369	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.7	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-1175	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.4	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
KENPP-035	R Culvert	1.7	0.8	3	1.5	L	M	L	LML	Low	Low	Low	Medium	0%	0%	10%	2.4	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%
KENPP-036	R Culvert	1.23	0.9	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.4	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
KENPP-037	R Culvert	1.23	0.83	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	0.4	Medium	Low	Low	Low	Medium	15%	15%	40%	25%	25%	50%
KENPP-038	R Culvert	1.23	0.9	2	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2.3	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
KENPP-039	R Culvert	1.23	0.6	2	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.5	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-0640	C Culvert	0.45	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.5	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-0773	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.8	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-1785	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.3	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-0763	C Culvert	0.3	0	0	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	1.3	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
EMCPP-2124	C Culvert	0.375	0	3	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	2.2	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
Thurgoona_Drive_Road_Bridge	B	14	0	0	1.5	H	H	H	HHH	High	Medium	High	High	0%	10%	10%	1.7	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%
M31 - Hune HWY Dual Bridg	B	27	0	0	1.5	H	H	H	HHH	High	Medium	High	High	0%	10%	10%	3.2	Low	High	Medium	High	High	15%	25%	25%	15%	25%	25%

1. Note that the plan location of each structure can be identified in the GIS layers contained in the data handover for the present study.

2. C Culvert = Circular Pipe Culvert

3. L₁₀ is the average length of the longest 10% of the debris that could arrive at the culvert.

APPENDIX F

FLOOD DATA FOR INDIVIDUAL ROAD AND RAIL CROSSINGS

TABLE F1
PEAK FLOOD LEVEL AND MAXIMUM DEPTH OF INUNDATION AT INDIVIDUAL ROAD AND RAIL CROSSINGS AT EIGHT MILE CREEK ^(1,2)

ID ⁽³⁾	Tributary	Location	Road/ Rail/ Spillway Level (m AHD)	6 January 2022		15 January 2022		20% AEP		10% AEP		5% AEP		2% AEP		1% AEP		0.5% AEP		0.2% AEP		PMF	
				Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)
[A]	[B]	[C]	[D]	[E]	[F]	[G]	[H]	[I]	[J]	[K]	[L]	[M]	[N]	[O]	[P]	[Q]	[R]	[S]	[T]	[U]	[V]	[W]	[X]
H001	Eight Mile Creek	Gerogery Road	229.20	229.27	0.07	229.62	0.42	229.33	0.13	229.45	0.25	229.54	0.34	229.60	0.40	229.67	0.47	229.71	0.51	229.77	0.57	230.89	1.69
H002		R W Henry Drive	220.00	218.72	NF	220.42	0.42	219.16	NF	219.88	NF	220.26	0.26	220.36	0.36	220.48	0.48	220.54	0.54	220.61	0.61	221.85	1.85
H003		Main Southern Railway	219.11	216.38	NF	218.44	NF	216.57	NF	217.06	NF	217.61	NF	218.06	NF	218.51	NF	218.84	NF	219.23	0.12	220.95	1.84
H004		Hume Highway	217.15	215.00	NF	216.84	NF	215.39	NF	215.86	NF	216.50	NF	216.78	NF	217.06	NF	217.27	0.12	217.69	0.54	219.24	2.09
H005a		Williams Road	190.20	192.21	2.01	193.46	3.26	192.44	2.24	192.70	2.50	192.86	2.66	193.05	2.85	193.23	3.03	193.33	3.13	193.43	3.23	195.37	5.17
H005b		Table Top Road	188.50	188.34	NF	188.84	0.34	188.00	NF	188.36	NF	188.47	NF	188.54	0.04	188.59	0.09	188.62	0.12	188.66	0.16	190.77	2.27
H006		Brooklyn Drive	180.80	179.94	NF	181.19	0.39	180.33	NF	180.57	NF	180.87	0.07	181.08	0.28	181.20	0.40	181.26	0.46	181.33	0.53	183.70	2.90
H007		Vaughan Road	175.70	176.02	0.32	177.14	1.44	176.26	0.56	176.50	0.80	176.67	0.97	176.94	1.24	177.09	1.39	177.17	1.47	177.26	1.56	179.73	4.03
H008		Thurgoona Drive	176.50	174.63	NF	176.88	0.38	174.72	NF	174.86	NF	176.41	NF	176.74	0.24	176.85	0.35	176.91	0.41	176.97	0.47	178.94	2.44
H009			177.80	175.20	NF	176.42	NF	175.56	NF	175.84	NF	176.03	NF	176.34	NF	176.47	NF	176.54	NF	176.62	NF	178.79	0.99
H010		Riverina Highway	169.55	169.03	NF	169.20	NF	169.02	NF	169.09	NF	169.16	NF	169.17	NF	169.19	NF	169.19	NF	169.21	NF	169.32	NF
H011			167.30	167.22	NF	167.64	0.34	167.47	0.17	167.51	0.21	167.55	0.25	167.62	0.32	167.67	0.37	167.71	0.41	167.75	0.45	169.10	1.80
H012			167.40	9999.00	NF	167.58	0.18	167.35	NF	167.40	NF	167.43	0.03	167.49	0.09	167.54	0.14	167.57	0.17	167.60	0.20	169.01	1.61
H013			167.35	167.19	NF	167.62	0.27	167.42	0.07	167.47	0.12	167.50	0.15	167.57	0.22	167.63	0.28	167.66	0.31	167.70	0.35	169.16	1.81
H014	Eight Mile Creek Tributary	Gerogery Road and Hub Road Intersection	235.85	236.93	1.08	237.10	1.25	236.85	1.00	236.92	1.07	236.99	1.14	237.03	1.18	237.07	1.22	237.09	1.24	237.14	1.29	237.82	1.97
H015		R W Henry Drive	217.00	216.07	NF	216.55	NF	216.04	NF	216.14	NF	216.27	NF	216.38	NF	216.55	NF	216.71	NF	217.72	0.72	218.31	1.31
H016		Wagga Road	217.55	215.46	NF	216.43	NF	215.52	NF	215.76	NF	216.08	NF	216.26	NF	216.43	NF	216.54	NF	216.99	NF	218.24	0.69
H017		Main Southern Railway	217.00	213.52	NF	213.92	NF	213.40	NF	213.48	NF	213.48	NF	213.54	NF	213.61	NF	213.66	NF	213.47	NF	216.03	NF
H018		Hume Highway	212.30	210.47	NF	211.00	NF	210.59	NF	210.70	NF	210.86	NF	211.00	NF	211.34	NF	211.46	NF	212.34	0.04	212.67	0.37
H019		Davey Road	208.50	208.58	0.08	208.71	0.21	208.59	0.09	208.60	0.10	208.62	0.12	208.63	0.13	208.65	0.15	208.66	0.16	208.63	0.13	209.05	0.55
H020	Nine Mile Creek	Lakeview Drive	239.95	240.01	0.06	240.14	0.19	239.96	0.01	240.01	0.06	240.04	0.09	240.07	0.12	240.11	0.16	240.13	0.18	240.16	0.21	240.56	0.61
H021		Lara Lake Road	235.10	235.26	0.16	235.36	0.26	235.23	0.13	235.27	0.17	235.29	0.19	235.31	0.21	235.34	0.24	235.35	0.25	235.38	0.28	238.68	3.58
H022a		Main Southern Railway	238.65	233.90	NF	234.89	NF	233.60	NF	233.94	NF	234.17	NF	234.42	NF	234.68	NF	234.92	NF	235.45	NF	238.68	0.03
H022b		Hume Highway	236.20	233.68	NF	234.49	NF	233.44	NF	233.73	NF	233.93	NF	234.10	NF	234.27	NF	234.49	NF	234.79	NF	236.42	0.22
H023		Maryvale Station Road	232.25	232.13	NF	232.30	0.05	231.97	NF	232.13	NF	232.19	NF	232.23	NF	232.26	0.01	232.27	0.02	232.29	0.04	232.54	0.29
H024		Table Top Road	195.30	194.46	NF	195.56	0.26	195.05	NF	195.31	0.01	195.44	0.14	195.50	0.20	195.56	0.26	195.60	0.30	195.64	0.34	196.58	1.28

Refer over for footnotes to table.

TABLE F1
PEAK FLOOD LEVEL AND MAXIMUM DEPTH OF INUNDATION AT INDIVIDUAL ROAD AND RAIL CROSSINGS AT EIGHT MILE CREEK ^(1,2)

ID ⁽³⁾	Tributary	Location	Road/ Rail/ Spillway Level (m AHD)	6 January 2022		15 January 2022		20% AEP		10% AEP		5% AEP		2% AEP		1% AEP		0.5% AEP		0.2% AEP		PMF	
				Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)
[A]	[B]	[C]	[D]	[E]	[F]	[G]	[H]	[I]	[J]	[K]	[L]	[M]	[N]	[O]	[P]	[Q]	[R]	[S]	[T]	[U]	[V]	[W]	[X]
H025	Nine Mile Creek Tributary	Hume Hwy	238.90	237.96	NF	-	NF	237.49	NF	237.89	NF	238.34	NF	238.42	NF	238.47	NF	238.50	NF	238.59	NF	239.14	0.24
H026		Maryvale Station Rd	217.70	217.01	NF	217.67	NF	216.78	NF	217.11	NF	217.59	NF	217.66	NF	217.70	NF	217.73	0.03	217.80	0.10	218.90	1.20
H027			217.55	217.06	NF	217.66	0.11	216.81	NF	217.23	NF	217.58	0.03	217.65	0.10	217.69	0.14	217.72	0.17	217.80	0.25	218.84	1.29
H028			216.88	216.56	NF	216.72	NF	216.57	NF	216.62	NF	216.65	NF	216.68	NF	216.70	NF	216.72	NF	216.76	NF	217.32	0.44
H029		Kywanna Rd	201.10	200.89	NF	201.37	0.27	201.23	0.13	201.26	0.16	201.30	0.20	201.33	0.23	201.36	0.26	201.39	0.29	201.44	0.34	201.81	0.71
H030			201.70	201.21	NF	201.81	0.11	201.70	NF	201.73	0.03	201.77	0.07	201.79	0.09	201.81	0.11	201.83	0.13	201.87	0.17	202.26	0.56
H031	Seven Mile Creek	Gerogery Rd	220.57	220.08	NF	220.54	NF	219.86	NF	220.11	NF	220.54	NF	220.57	NF	220.59	0.02	220.60	0.03	220.68	0.11	220.98	0.41
H032		Wagga Rd	214.50	213.67	NF	214.22	NF	213.66	NF	213.80	NF	213.94	NF	214.04	NF	214.13	NF	214.18	NF	214.26	NF	215.06	0.56
H033		Main Southern Railway	211.30	207.74	NF	209.37	NF	207.51	NF	207.91	NF	208.33	NF	208.59	NF	208.88	NF	209.08	NF	209.56	NF	211.08	NF
H034		Hume Hwy	209.33	204.88	NF	205.26	NF	204.98	NF	205.05	NF	205.10	NF	205.15	NF	205.21	NF	205.26	NF	205.29	NF	207.20	NF
H035		Wignell Rd	197.38	197.42	0.04	197.67	0.29	197.37	NF	197.46	0.08	197.52	0.14	197.55	0.17	197.60	0.22	197.62	0.24	197.65	0.27	198.53	1.15
H036		Table Top Rd	185.40	185.04	NF	185.61	0.21	185.00	NF	185.19	NF	185.42	0.02	185.51	0.11	185.54	0.14	185.56	0.16	185.59	0.19	187.21	1.81
H037	Eight Mile Creek Tributary	Wagga Rd	216.25	214.97	NF	215.71	NF	214.89	NF	215.29	NF	215.43	NF	215.52	NF	215.61	NF	215.65	NF	215.73	NF	216.29	0.04
H038		Main Southern Railway	210.15	209.20	NF	209.67	NF	209.12	NF	209.23	NF	209.29	NF	209.34	NF	209.39	NF	209.43	NF	209.47	NF	210.36	0.21
H039		Hume Hwy	207.45	205.67	NF	205.97	NF	205.56	NF	205.64	NF	205.68	NF	205.71	NF	205.75	NF	205.78	NF	205.78	NF	207.77	0.32
H040		Ettamogah Rd	201.50	201.55	0.05	201.74	0.24	201.55	0.05	201.64	0.14	201.67	0.17	201.69	0.19	201.70	0.20	201.71	0.21	201.73	0.23	202.09	0.59
H041		Stanton Dr	197.70	196.85	NF	197.99	0.29	196.71	NF	196.94	NF	197.27	NF	197.53	NF	197.79	0.09	197.86	0.16	197.90	0.20	198.81	1.11
H042		Elizabeth Mitchell Dr	195.00	193.15	NF	195.08	0.08	192.92	NF	193.37	NF	193.94	NF	194.32	NF	194.64	NF	194.86	NF	195.05	0.05	195.75	0.75
H043		Table Top Rd	182.65	179.76	NF	181.45	NF	179.70	NF	179.90	NF	180.15	NF	180.42	NF	180.64	NF	180.79	NF	181.01	NF	183.79	1.14
H044		Wagga Rd	207.90	206.87	NF	207.41	NF	206.61	NF	206.88	NF	207.00	NF	207.09	NF	207.15	NF	207.20	NF	207.31	NF	208.01	0.11
H045		Main Southern Railway	205.80	204.81	NF	205.24	NF	204.72	NF	204.88	NF	204.94	NF	205.00	NF	205.08	NF	205.12	NF	205.19	NF	205.79	NF
H046		Hume Hwy	204.05	202.14	NF	203.26	NF	201.97	NF	202.10	NF	202.19	NF	202.28	NF	202.48	NF	202.62	NF	203.25	NF	204.88	0.83
H047		Ettamogah Rd	190.20	190.15	NF	190.23	0.03	190.16	NF	190.18	NF	190.20	NF	190.20	NF	190.21	0.01	190.21	0.01	190.22	0.02	190.34	0.14
H048	Woolshed Creek	Wedgetail Way	203.75	203.85	0.10	204.08	0.33	203.87	0.12	203.91	0.16	203.94	0.19	203.97	0.22	204.00	0.25	204.01	0.26	204.04	0.29	204.54	0.79
H049		Kywanna Rd	199.90	199.98	0.08	200.39	0.49	200.02	0.12	200.08	0.18	200.14	0.24	200.20	0.30	200.26	0.36	200.31	0.41	200.36	0.46	201.31	1.41
H050		Kerr Rd	187.30	186.52	NF	188.12	0.82	187.51	0.21	187.71	0.41	187.85	0.55	187.94	0.64	188.05	0.75	188.12	0.82	188.21	0.91	189.74	2.44

Refer over for footnotes to table.

TABLE F1
PEAK FLOOD LEVEL AND MAXIMUM DEPTH OF INUNDATION AT INDIVIDUAL ROAD AND RAIL CROSSINGS AT EIGHT MILE CREEK ^(1,2)

ID ⁽³⁾	Tributary	Location	Road/ Rail/ Spillway Level (m AHD)	6 January 2022		15 January 2022		20% AEP		10% AEP		5% AEP		2% AEP		1% AEP		0.5% AEP		0.2% AEP		PMF	
				Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)	Peak Flood Level (m AHD)	Depth of Overtopping (m)
[A]	[B]	[C]	[D]	[E]	[F]	[G]	[H]	[I]	[J]	[K]	[L]	[M]	[N]	[O]	[P]	[Q]	[R]	[S]	[T]	[U]	[V]	[W]	[X]
H051	Woolshed Creek	Wedgetail Way	212.30	212.20	NF	212.41	0.11	212.25	NF	212.28	NF	212.31	0.01	212.34	0.04	212.36	0.06	212.38	0.08	212.39	0.09	212.64	0.34
H052	Woolshed Creek Tributary	Thurgoona Dr	198.60	198.59	NF	198.90	0.30	198.64	0.04	198.66	0.06	198.69	0.09	198.70	0.10	198.73	0.13	198.75	0.15	198.79	0.19	199.70	1.10
H053			200.15	200.04	NF	200.20	0.05	200.14	NF	200.16	0.01	200.18	0.03	200.20	0.05	200.21	0.06	200.21	0.06	200.23	0.08	200.36	0.21
H054		Kerr Rd	189.00	188.76	NF	189.75	0.75	189.58	0.58	189.61	0.61	189.63	0.63	189.66	0.66	189.69	0.69	189.72	0.72	189.75	0.75	190.24	1.24
H055		Rosborough Ct	187.75	186.95	NF	188.14	0.39	187.80	0.05	187.89	0.14	187.97	0.22	188.00	0.25	188.04	0.29	188.09	0.34	188.18	0.43	189.09	1.34
H056	Eight Mile Creek Tributary	Kerr Rd	184.05	183.60	NF	184.65	0.60	184.31	0.26	184.39	0.34	184.47	0.42	184.51	0.46	184.54	0.49	184.57	0.52	184.66	0.61	185.16	1.11
H057	Sandy Creek	Knoble Rd	176.50	174.44	NF	177.38	0.88	175.58	NF	176.50	NF	176.80	0.30	176.90	0.40	177.01	0.51	177.08	0.58	177.31	0.81	178.80	2.30
H058		Riverina Hwy	164.20	161.22	NF	164.88	0.68	161.92	NF	162.29	NF	162.78	NF	163.25	NF	163.87	NF	164.27	0.07	164.62	0.42	166.63	2.43
H059			164.20	163.26	NF	165.32	1.12	164.34	0.14	164.82	0.62	165.07	0.87	165.14	0.94	165.21	1.01	165.25	1.05	165.36	1.16	166.01	1.81

1. While elevations have generally been rounded to nearest 0.1 m, the depth of overtopping shows where the depth of overtopping is less than 0.1 m.
2. NF = Not flooded.
3. Refer relevant figures in Volume 2 for location of Peak Flood Level Location.

APPENDIX G

DESIGN PEAK FLOWS

TABLE G1
DESIGN PEAK FLOWS DERIVED BY TUFLOW MODEL⁽¹⁾

Peak Flow Location Identifier ⁽²⁾	Watercourse	Location	20% AEP			10% AEP			5% AEP			2% AEP			1% AEP			0.5% AEP			0.2% AEP			PMF ⁽⁵⁾	
			Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)
[A]	[B]	[C]	[D]	[E]	[F]	[G]	[H]	[I]	[J]	[K]	[L]	[M]	[N]	[O]	[P]	[Q]	[R]	[S]	[T]	[U]	[V]	[W]	[X]	[Y]	[Z]
Q001	Eight Mile Creek	Upstream Knowles Road	8.4	120	s7	12.1	90	s6	14.9	30	s8	17.7	30	s6	22.9	30	s6	26.5	30	s6	31.8	30	s6	161	45
Q002		Knowles Road	11.9	120	s7	17.4	90	s6	21.4	60	s8	26.2	30	s2	32.3	30	s6	37	30	s6	44.5	30	s6	237	45
Q003		0.8 km upstream Gerogery Road	21.7	120	s7	32.1	90	s6	39.8	60	s8	48	60	s2	57.5	60	s2	64.4	60	s2	74.3	60	s2	377	45
Q004		0.1 km upstream Gerogery Road	29.4	120	s7	45.4	90	s6	58.6	60	s8	71.9	60	s2	88.3	60	s2	100	60	s2	117	60	s2		
Q005		Upstream R W Henry Drive	34.9	120	s7	56.1	120	s6	74.7	60	s6	91.6	60	s2	115	60	s2	131	60	s2	153	60	s2	692	45
Q006		Downstream R W Henry Drive	33.1	120	s7	46.2	120	s6	62.7	90	s6	73	90	s8	86.1	90	s8	95.3	90	s8	108	90	s8		
Q007		0.2 km downstream Hume Highway	31.3	120	s6	42.7	120	s6	47.1	90	s6	53.7	90	s8	59.8	90	s8	64	90	s8	78.2	90	s8		
Q008		0.7 km downstream Hume Highway	33.4	120	s6	44.9	120	s6	50.1	90	s6	57.3	90	s8	64.2	90	s8	69.1	90	s8	82.4	90	s8		
Q009		1.5 km downstream Hume Highway	33.8	120	s6	45.3	120	s6	50.5	120	s6	57.9	90	s4	65	90	s8	69.9	90	s8	83.1	90	s8		
Q010		2.2 km downstream Hume Highway	34.1	120	s6	45.7	120	s6	51.2	120	s6	58.8	120	s4	66.2	90	s4	71.2	90	s8	84	90	s8		
Q011		1.9 km upstream Williams Road	34.6	120	s6	46.2	120	s6	51.9	120	s6	59.8	120	s4	67.6	120	s4	72.8	90	s4	85.1	90	s4		
Q012		1 km upstream Williams Road	36	120	s6	47.5	120	s6	53.7	120	s6	63.6	180	s4	72.5	120	s6	78.8	90	s4	87.7	90	s4		
Q013		0.1 km upstream Williams Road	36.2	120	s6	47.7	120	s6	54.7	120	s6	65	180	s4	75.2	180	s6	81.7	90	s6	90.3	90	s6		
Q014		Upstream confluence with Nine Mile Creek	35.5	120	s6	46.5	120	s6	54.5	180	s6	64.4	180	s6	73.6	180	s6	78.9	90	s6	85.5	90	s6		
Q015		Downstream confluence with Nine Mile Creek	50.3	180	s6	65.7	180	s1	79.8	180	s1	99.2	180	s6	116	180	s6	127	180	s6	139	180	s6		
Q016		Upstream confluence with Seven Mile Creek	49.5	180	s6	64.6	180	s1	79.2	180	s1	98.9	180	s6	117	180	s6	128	180	s6	141	180	s6		
Q017		Downstream confluence with Seven Mile Creek	53.2	180	s6	69.8	180	s1	87.3	180	s1	111	180	s6	133	180	s6	148	180	s6	167	180	s6		
Q018		Downstream Brooklyn Drive	53.7	180	s6	70.2	180	s1	89.7	270	s1	116	180	s7	137	180	s6	151	180	s6	169	180	s6		
Q019		Upstream confluence with Woolshed Creek	53.4	180	s6	69.8	180	s1	90.2	270	s1	117	180	s7	137	180	s6	152	180	s6	170	180	s6		
Q020		Downstream confluence with Woolshed Creek	59.3	360	s6	80.5	360	s6	103	270	s6	138	270	s7	171	270	s7	192	180	s7	219	180	s7		
Q021b		Downstream Thurgoona Drive	58.7	360	s6	79	360	s6	95.4	270	s6	130	270	s7	152	270	s7	164	270	s7	181	270	s7		
Q021a		Downstream Thurgoona Drive	5.7	30	s1	6.1	30	s7	6.6	270	s7	7.2	270	s6	18.1	270	s7	26.5	270	s7	37.9	270	s7		
Q022		Thurgoona Country Club	57.9	360	s6	78.9	360	s6	95.5	270	s6	135	270	s7	169	270	s7	191	270	s7	219	270	s7		
Q023		2.4 km upstream Riverina Highway	58	540	s6	77.4	540	s6	94.4	270	s6	137	270	s7	172	270	s7	194	270	s7	223	270	s7		
Q024		2.0 km upstream Riverina Highway	57.9	540	s6	77.6	540	s6	95.1	270	s6	136	270	s7	172	270	s7	194	270	s7	223	270	s7		

Refer over for footnotes to table.

TABLE G1 (Cont'd)
DESIGN PEAK FLOWS DERIVED BY TUFLOW MODEL⁽¹⁾

Peak Flow Location Identifier ⁽²⁾	Watercourse	Location	20% AEP			10% AEP			5% AEP			2% AEP			1% AEP			0.5% AEP			0.2% AEP			PMF ⁽⁵⁾	
			Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)
[A]	[B]	[C]	[D]	[E]	[F]	[G]	[H]	[I]	[J]	[K]	[L]	[M]	[N]	[O]	[P]	[Q]	[R]	[S]	[T]	[U]	[V]	[W]	[X]	[Y]	[Z]
Q025	Eight Mile Creek	0.1 km upstream Riverina Highway	57.7	360	s6	79.6	360	s6	96.5	270	s6	136	270	s7	171	270	s7	194	270	s7	224	270	s7	2,392	120
Q026		1.3 km downstream Riverina Highway	57.4	360	s6	79.5	360	s6	96.5	270	s6	136	270	s7	171	270	s7	194	270	s7	223	270	s7		
Q027a		2.3 km downstream Riverina Highway	1.7	360	s6	2.2	540	s6	2.8	270	s6	4.4	270	s7	6.4	270	s7	8	270	s7	10.2	270	s7	534	120
Q027b			5.5	360	s6	5.6	540	s6	5.6	90	s6	5.6	270	s8	5.6	270	s7	5.6	270	s7	5.6	270	s7	77.9	120
Q027c			50.2	360	s6	72	540	s6	89.1	270	s6	126	270	s7	160	270	s7	182	270	s7	209	270	s7	1,797	120
Q028a		Upstream Koowong Lane	4.8	540	s5	6.1	540	s6	7.1	270	s6	7.5	270	s7	10.1	270	s7	11.5	270	s7	13.6	270	s7	247	120
Q028b			0	540	s5	0	540	s6	0	270	s6	0	270	s7	0.1	270	s7	0	270	s7	0.1	270	s7	420	120
Q028c		Downstream Hawkscote Road	51.2	360	s6	73.3	540	s6	90.5	270	s6	127	270	s7	160	270	s7	182	270	s7	210	270	s7	1,752	120
Q029	Eight Mile Creek Tributary	N/A	8.1	120	s7	12.2	120	s6	15.1	30	s6	18.4	30	s6	24.1	30	s6	28.2	30	s6	34.7	30	s6	181	45
Q030a		Upstream Rockwood Lane	0.1	120	s7	0.4	120	s6	0.9	60	s6	1.5	30	s2	2.3	30	s6	2.9	30	s6	3.8	30	s6	39.1	45
Q030b			9.1	120	s7	13.5	90	s6	16.6	60	s8	20	60	s2	24.2	60	s2	27.2	60	s2	31.5	60	s2	174	45
Q031		Upstream Central Reserve Road	6.7	120	s7	11	120	s6	14.2	30	s6	17.6	30	s6	22.8	30	s6	26	30	s6	30.5	30	s6	167	45
Q032a		Upstream Rockwood Lane	0.4	90	s7	0.7	90	s8	1	60	s8	1.4	60	s2	1.8	60	s2	2.1	60	s2	2.6	60	s2	23.7	45
Q032b			2.5	120	s7	3.9	120	s6	5	60	s6	6.2	30	s2	7.6	30	s6	8.9	30	s6	10.8	30	s6	82.2	45
Q032c			5.6	120	s7	8.4	120	s6	11.6	60	s6	13.9	30	s2	17.6	30	s6	20.6	30	s6	24.7	30	s6	134	45
Q033		Upstream Gerobery Road	5.3	90	s7	7.8	90	s8	9.9	60	s8	12.3	60	s2	15	60	s2	17.1	60	s2	19.5	60	s2	128	45
Q034		Downstream Gerobery Road	6.3	90	s7	9.6	90	s8	12.5	60	s8	15.7	60	s2	19.6	60	s2	22.3	60	s2	25.8	60	s2	163	45
Q035		Upstream confluence with Eight Mile Creek	9.3	90	s7	14.4	90	s8	18.7	60	s8	23.6	60	s2	29.1	60	s2	32.8	60	s2	37.8	60	s2		
Q036		Downstream Hub Road	3.7	90	s7	5.5	120	s8	6.9	60	s6	8.3	30	s2	10.1	30	s6	11.5	30	s6	13.4	30	s6		
Q037		Upstream confluence with Eight Mile Creek	3.3	120	s7	5.3	90	s6	7.1	90	s8	8.6	60	s8	10.5	60	s2	11.7	60	s2	13.1	60	s2	82.3	60
Q038		Upstream R W Henry Drive	0.6	120	s7	0.9	120	s6	1	90	s6	1.2	90	s8	1.4	90	s8	1.6	60	s8	1.7	60	s8	58.2	60
Q039		Upstream Wagga Road	1.3	120	s7	1.4	120	s6	1	180	s6	1	180	s6	1.1	180	s6	1.1	270	s6	0.1	270	s6		
Q040		Downstream Wagga Road	1.4	120	s7	1.5	120	s6	1.1	120	s6	1.1	90	s4	1.2	90	s8	1.2	270	s8	0	270	s8		
Q041		Upstream Hume Highway	2	120	s7	2.3	90	s6	2.3	90	s8	2.8	90	s8	3	90	s8	3.1	270	s8	1.1	270	s8	30.6	60
Q042		Downstream Davey Road	2.9	120	s3	3.5	90	s6	4.2	90	s8	5.2	60	s8	6.3	60	s2	7	60	s2	4.9	60	s2	52.8	60

Refer over for footnotes to table.

TABLE G1 (Cont'd)
DESIGN PEAK FLOWS DERIVED BY TUFLOW MODEL⁽¹⁾

Peak Flow Location Identifier ⁽²⁾	Watercourse	Location	20% AEP			10% AEP			5% AEP			2% AEP			1% AEP			0.5% AEP			0.2% AEP			PMF ⁽⁵⁾	
			Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)
[A]	[B]	[C]	[D]	[E]	[F]	[G]	[H]	[I]	[J]	[K]	[L]	[M]	[N]	[O]	[P]	[Q]	[R]	[S]	[T]	[U]	[V]	[W]	[X]	[Y]	[Z]
Q043	Eight Mile Creek Tributary	Upstream confluence with Eight Mile Creek	4	120	s3	5.2	90	s6	6.6	90	s8	8.1	60	s8	10	60	s2	11.3	60	s2	10.3	60	s2	92.9	60
Q044	Nine Mile Creek	Upstream Lakeview Drive	0.5	120	s7	0.7	90	s6	0.8	30	s8	1.1	30	s6	1.4	30	s6	1.6	30	s6	1.9	30	s6	10.1	45
Q045		Downstream Lara Lake Road	2.6	90	s7	4	90	s8	5	60	s8	6.1	60	s2	7.1	60	s2	7.9	60	s2	8.6	60	s2		
Q046		0.2 km downstream Maryvale Station Road	4.3	90	s7	6	90	s8	7.4	60	s8	8.7	60	s2	10.1	60	s2	11.1	60	s2	10.9	60	s2	40.8	60
Q047		1.2 km downstream Maryvale Station Road	6	90	s7	8.7	90	s8	11.1	60	s8	13.1	60	s2	15.8	60	s2	17.6	60	s2	18.7	60	s2	92.4	60
Q048		0.8 km upstream Plantation Lane	10.3	120	s3	15.8	120	s6	19	90	s6	23	90	s8	27.4	90	s8	30.5	90	s8	33.1	90	s8	135	60
Q049		0.1 km upstream Plantation Lane	5.9	120	s3	6.5	120	s6	6.7	120	s6	7	120	s4	7.3	90	s4	7.4	90	s8	7.6	90	s8	9.8	60
Q050		0.9 km downstream Plantation Lane	5.4	180	s3	5.7	120	s1	6	180	s6	6.2	120	s6	6.5	120	s4	6.5	90	s4	6.7	90	s4	9.1	60
Q051		1.6 km downstream Plantation Lane	11.4	120	s3	14.2	120	s6	17.7	90	s6	21.2	90	s8	25.1	90	s8	27.8	90	s8	31.3	90	s8	166	60
Q052		Upstream Table Top Road	14.3	120	s3	18.4	120	s6	23.5	120	s6	28.6	90	s4	34.6	90	s8	39	90	s8	44.3	90	s8	273	60
Q053		Downstream Table Top Road	17.6	180	s6	23.4	180	s1	30.7	120	s1	40	120	s4	50.3	120	s4	57.3	90	s4	67.2	90	s4	472	60
Q054		Upstream confluence with Eight Mile Creek	18	180	s6	22.7	180	s1	30	180	s1	39.9	180	s6	48.1	180	s6	54.1	180	s6	62.6	180	s6		
Q055a	Nine Mile Creek Tributary	Main Southern Railway	0.1	120	s1	0.2	120	s6	0.2	30	s6	0.2	30	s6	0.2	30	s6	0.2	30	s6	0.2	30	s6	4.1	60
Q055b		Hume Highway	0	30	s1	0	30	s7	0	30	s7	0	30	s6	0	30	s6	0	90	s6	0.3	90	s6	1.7	60
Q055c		Maryvale Station Road	0.4	90	s7	0.7	90	s8	0.9	60	s8	1	60	s2	1.3	60	s2	1.5	60	s2	1.7	60	s2	10.2	60
Q056			3.8	90	s7	5.6	90	s8	7.2	60	s8	8.9	30	s2	11	30	s6	12.8	30	s6	15.3	30	s6	88.5	45
Q057			4.5	120	s7	6.2	120	s6	6.3	90	s6	6.5	60	s8	6.7	60	s2	7	60	s2	7	60	s2	39.5	45
Q058		Upstream confluence with Nine Mile Creek	4.7	120	s3	7.2	120	s6	8.4	120	s6	9.5	120	s4	10.7	90	s4	11.6	90	s8	12.4	90	s8	115	60
Q059		Upstream Lake Ettamogah	3.8	120	s7	5.9	120	s6	7.8	90	s6	9.6	90	s8	11.9	60	s8	13.7	60	s2	16.5	60	s2	107	60
Q060			0.2	120	s3	0.3	120	s6	0.5	90	s6	0.7	90	s8	1	90	s8	1.2	60	s8	1.5	60	s8	10.8	60
Q061		Downstream Maryvale Station Road	5	120	s3	10.1	120	s6	13.2	90	s6	17.1	90	s8	21.8	90	s8	24.9	90	s8	28	90	s8		
Q062		Lake Ettamogah Spillway	0	540	s1	0	540	s6	0.2	270	s6	1	270	s7	2.3	270	s7	3.5	270	s7	5.4	270	s7		
Q063		Upstream confluence with Nine Mile Creek	4.7	120	s3	7.1	90	s6	9.2	60	s8	11.4	60	s2	14.2	60	s2	16	60	s2	18.3	60	s2	133	120
Q064			2.2	120	s3	3.2	90	s6	4.1	60	s8	5.2	60	s2	6.8	60	s2	7.8	60	s2	9.2	60	s2	65.2	60
Q065a		Upstream confluence with Nine Mile Creek	0	30	s1	0	120	s7	0	90	s6	0.1	90	s8	0.1	60	s8	0.2	60	s2	0.3	60	s2	2.7	60

Refer over for footnotes to table.

TABLE G1 (Cont'd)
DESIGN PEAK FLOWS DERIVED BY TUFLOW MODEL⁽¹⁾

Peak Flow Location Identifier ⁽²⁾	Watercourse	Location	20% AEP			10% AEP			5% AEP			2% AEP			1% AEP			0.5% AEP			0.2% AEP			PMF ⁽⁵⁾	
			Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)
[A]	[B]	[C]	[D]	[E]	[F]	[G]	[H]	[I]	[J]	[K]	[L]	[M]	[N]	[O]	[P]	[Q]	[R]	[S]	[T]	[U]	[V]	[W]	[X]	[Y]	[Z]
Q065b	Nine Mile Creek Tributary	Upstream confluence with Nine Mile Creek	1	180	s6	1.4	120	s1	1.7	120	s6	2.3	120	s4	2.8	90	s4	3.3	90	s8	3.9	90	s8	27.1	60
Q066			2.9	120	s7	4.3	90	s6	5.1	60	s8	6.1	60	s2	7.6	60	s2	8.6	60	s2	9.6	60	s2	66.9	60
Q067		Upstream Kywanna Road	2.8	90	s7	4.1	90	s8	5.4	60	s8	7	60	s2	9.3	60	s2	10.8	60	s2	13.6	60	s2	88.7	60
Q068		Downstream Kywanna Road	3.1	90	s7	4.5	120	s8	5.8	60	s6	7.5	60	s2	9.9	60	s2	11.6	60	s2	14.7	60	s2		
Q069		Upstream Kywanna Road	1.9	90	s7	2.7	90	s8	3.5	60	s8	4.4	60	s2	5.5	60	s2	6.2	60	s2	7.3	60	s2	45.6	60
Q070		Downstream Kywanna Road	3.7	120	s7	5.4	90	s6	7.1	90	s8	8.6	60	s8	10.9	60	s2	12.5	60	s2	14.9	60	s2		
Q071			1	120	s3	1.4	90	s6	1.8	90	s8	2.3	90	s8	2.8	90	s8	3.2	60	s8	3.8	60	s8	25.9	60
Q072	Seven Mile Creek	Upstream Gerogery Road	0.5	120	s7	0.8	120	s6	0.9	30	s6	1.1	30	s6	1.5	30	s6	1.7	30	s6	2	30	s6	11.3	45
Q073		Downstream Gerogery Road	1.3	90	s7	1.8	90	s8	2.4	60	s8	3	60	s2	3.8	60	s2	4.4	60	s2	5.4	60	s2	37.8	45
Q074		Downstream Twynam Court	4.2	90	s7	6.6	90	s8	8.8	60	s8	11.2	60	s2	14.8	60	s2	17.4	60	s2	21.4	60	s2	146	60
Q075		Upstream Hume Highway	4.2	120	s7	6.4	90	s6	8	90	s8	9.8	60	s8	11.9	60	s2	13.5	60	s2	14.7	60	s2		
Q076		Upstream Wignell Road	5.3	120	s6	7.6	120	s6	9.5	90	s6	11.7	90	s8	14.4	90	s8	16.4	90	s8	18.1	90	s8	194	60
Q077		Downstream Wignell Road	5.7	120	s6	8.2	120	s6	10.3	120	s6	12.7	120	s4	16.8	120	s4	19.6	120	s4	22.6	120	s4		
Q078a		Upstream Table Top Road	6.3	120	s3	8.2	120	s6	9.9	120	s6	11.8	120	s4	13.9	120	s4	15.3	120	s4	16.7	120	s4	51.4	60
Q078b			0.8	120	s3	1.3	120	s6	2.3	120	s6	4	120	s4	6	120	s4	8.1	120	s4	11	120	s4	236	60
Q079		Upstream confluence with Eight Mile Creek	7.9	180	s6	9.3	180	s1	11.2	180	s1	13.7	180	s6	16.1	180	s6	18	180	s6	21	180	s6		
Q080		Upstream confluence with Seven Mile Creek	0.5	120	s3	0.7	90	s6	1	90	s8	1.3	60	s8	1.7	60	s2	2.1	60	s2	2.5	60	s2	20.3	45
Q081	Seven Mile Creek Tributary	Downstream Wagga Road	0.8	90	s7	1	90	s8	1.1	60	s8	1.1	60	s2	1.2	60	s2	1.2	60	s2	0.8	60	s2		
Q082		Downstream Hume Highway	1.1	120	s7	1.4	90	s6	1.7	90	s8	1.9	60	s8	2.1	60	s2	2.3	60	s2	2	60	s2	18.3	60
Q083		Upstream Ettamogah Road	1	120	s6	1.4	120	s6	1.6	90	s6	1.9	90	s8	2.1	60	s8	2.3	90	s2	2	90	s2	19.6	60
Q084		Downstream Stanton Drive	1.5	120	s3	2.2	120	s6	2.8	120	s6	3.4	90	s4	4.2	90	s8	4.9	90	s8	5.1	90	s8	44.5	60
Q085		Downstream Elizabeth Mitchell Drive	5.3	120	s6	6.4	120	s6	6.8	180	s6	7.5	180	s6	8	180	s6	8.3	90	s6	11.2	90	s6	117	60
Q086		Hume and Hovell Walking Track	5.8	180	s3	6.9	180	s1	7.5	180	s1	8.5	90	s6	9.6	90	s8	10.3	90	s8	11.6	90	s8	144	60
Q087		Upstream Table Top Road	6.9	180	s3	8.7	180	s1	10.2	180	s1	12.3	180	s6	14.2	90	s6	15.8	90	s8	18.5	90	s8		
Q088		Downstream Hume Highway	3.1	120	s7	4.3	90	s6	5.4	90	s8	6.5	60	s8	9	60	s2	11	60	s2	13.6	60	s2	21.4	60

Refer over for footnotes to table.

TABLE G1 (Cont'd)
DESIGN PEAK FLOWS DERIVED BY TUFLOW MODEL⁽¹⁾

Peak Flow Location Identifier ⁽²⁾	Watercourse	Location	20% AEP			10% AEP			5% AEP			2% AEP			1% AEP			0.5% AEP			0.2% AEP			PMF ⁽⁵⁾	
			Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)
[A]	[B]	[C]	[D]	[E]	[F]	[G]	[H]	[I]	[J]	[K]	[L]	[M]	[N]	[O]	[P]	[Q]	[R]	[S]	[T]	[U]	[V]	[W]	[X]	[Y]	[Z]
Q089	Eight Mile Creek Tributary	Upstream Elizabeth Mitchell Drive	3.6	120	s7	4.9	120	s6	6	180	s6	6.2	120	s6	8.4	120	s4	9.7	90	s4	13.6	90	s4		
Q090		Downstream Birramal Road	1.7	90	s7	2.7	120	s8	3.5	60	s6	4.3	30	s2	5.6	30	s6	6.4	30	s6	7.4	30	s6	41.9	60
Q091		Downstream Ettamogah Road	0.7	120	s3	1.1	120	s6	1.4	90	s6	2	90	s8	2.6	90	s8	3.1	60	s8	3.8	60	s8	41.1	60
Q092	Woolshed Creek	Upstream Wedgetail Way	1.4	120	s7	2.1	120	s6	2.7	30	s6	3.4	30	s6	4.4	30	s6	5.4	30	s6	6.9	30	s6	45.5	45
Q093		Upstream Kywanna Road	4.9	90	s7	7.6	90	s8	10.3	60	s8	13.4	60	s2	17.2	60	s2	19.8	30	s2	23.8	30	s2		
Q094		0.4 km downstream Kywanna Road	6.6	120	s7	10.5	90	s6	14	60	s8	18	60	s2	23	60	s2	26.7	60	s2	32.1	60	s2		
Q095		0.8 km downstream Kywanna Road	7.6	120	s7	12.3	90	s6	16.5	60	s8	21	60	s2	27.2	60	s2	31.6	60	s2	38.1	60	s2		
Q096		Upstream Kerr Road	8	120	s7	12.9	120	s6	17.2	60	s6	21.8	60	s2	28.2	60	s2	32.8	60	s2	39.5	60	s2		
Q097		Downstream Kerr Road	18.1	120	s7	28.6	120	s6	38	90	s6	45.9	60	s8	59.3	60	s2	68.8	60	s2	82.9	60	s2		
Q098			21.1	120	s6	32.2	120	s6	42.2	90	s6	52.4	90	s8	66.9	60	s8	77.6	60	s2	94.4	60	s2		
Q099			20.8	120	s6	31.5	120	s6	41.7	90	s6	51.8	90	s8	66.3	90	s8	76.8	60	s8	92.6	60	s8		
Q100		Upstream confluence with Eight Mile Creek	20.8	360	s6	23.6	360	s6	29.7	270	s6	38.3	270	s7	44.9	270	s7	50.1	270	s7	57.4	270	s7		
Q101		Upstream confluence with Woolshed Creek	0.9	120	s7	1.3	90	s6	1.6	60	s8	2	60	s2	2.5	60	s2	3	60	s2	3.7	60	s2	27.0	45
Q102			0.5	90	s7	0.7	90	s8	0.9	60	s8	1.1	60	s2	1.4	60	s2	1.5	30	s2	1.8	30	s2	13.6	45
Q103		Upstream Thurgoona Drive	0.3	120	s7	0.5	120	s6	0.6	30	s6	0.7	30	s6	1	30	s6	1.1	30	s6	1.3	30	s6	7.1	45
Q104		Downstream Thurgoona Drive	3.6	120	s3	5.1	120	s6	7.1	90	s6	9	90	s8	11.4	90	s8	13.2	90	s8	15.8	90	s8		
Q105			5.8	120	s7	9	120	s6	11.8	180	s6	14.8	120	s6	19.3	120	s4	22.5	120	s4	27.3	120	s4	230	60
Q106		Upstream confluence with Woolshed Creek	9	120	s6	14.1	120	s6	18.5	90	s6	22.2	90	s8	27.8	90	s8	31.7	60	s8	38	60	s8	300	60
Q107		Thurgoona Drive	1.3	120	s7	2.1	120	s6	2.8	60	s6	3.5	30	s2	4.5	30	s6	5.3	30	s6	6.5	30	s6	32.7	45
Q108		Upstream Dairy Lane	4.6	120	s7	7.7	120	s6	9.7	30	s6	12.4	30	s6	16.5	30	s6	19	30	s6	22.5	30	s6	107	45
Q109			0.6	120	s7	1	30	s6	1.2	30	s7	1.6	30	s6	2.1	30	s6	2.3	30	s6	2.7	30	s6	13.9	45
Q110			1.8	30	s1	2.7	30	s7	3.6	30	s7	4.4	30	s6	5.4	30	s6	6.1	30	s6	7.1	30	s6	32.4	15
Q111a			2.8	120	s7	4.3	90	s6	5.2	60	s8	6.3	30	s2	7.2	30	s6	8.2	30	s6	9.8	30	s6	63.8	45
Q111b			0.6	120	s7	0.9	120	s6	1.2	30	s6	1.5	30	s6	1.9	30	s6	2.2	30	s6	2.7	30	s6	18.7	45
Q111c			1.8	90	s7	2.6	90	s8	3.3	60	s8	4.1	60	s2	4.8	30	s2	5.5	30	s6	6.6	30	s6	37.6	60

Refer over for footnotes to table.

TABLE G1 (Cont'd)
DESIGN PEAK FLOWS DERIVED BY TUFLOW MODEL⁽¹⁾

Peak Flow Location Identifier ⁽²⁾	Watercourse	Location	20% AEP			10% AEP			5% AEP			2% AEP			1% AEP			0.5% AEP			0.2% AEP			PMF ⁽⁵⁾	
			Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m³/s)	Critical Storm Duration ⁽³⁾ (minutes)
[A]	[B]	[C]	[D]	[E]	[F]	[G]	[H]	[I]	[J]	[K]	[L]	[M]	[N]	[O]	[P]	[Q]	[R]	[S]	[T]	[U]	[V]	[W]	[X]	[Y]	[Z]
Q111d	Woolshed Creek Tributary		4.3	120	s7	6.8	120	s6	8.4	60	s6	9.6	60	s2	11.3	30	s2	12.3	30	s6	13.9	30	s6	58.8	45
Q111e			0.9	120	s7	2	120	s6	2.7	60	s6	3	30	s2	3.7	60	s6	4.3	30	s2	5.2	30	s2	41.9	60
Q112		Upstream Grasslands Road	0.5	90	s7	0.7	90	s8	1	60	s8	1.3	60	s2	1.6	60	s2	1.8	60	s2	2.1	60	s2	15.0	45
Q113		Upstream Kerr Road	0.8	540	s7	1.2	180	s6	1.5	90	s1	2.4	90	s8	3.4	60	s8	4.1	60	s2	5.3	60	s2	90.4	60
Q114			4.4	90	s7	6.3	90	s8	7.4	60	s8	8.8	60	s2	10.5	60	s2	12.9	60	s2	16.8	60	s2		
Q115	Eight Mile Creek Tributary	Upstream confluence with Eight Mile Creek	1.6	30	s1	2.2	120	s7	2.9	30	s6	3.6	30	s6	4.8	30	s6	5.7	30	s6	6.9	30	s6	39.4	15
Q116		Downstream Kerr Road	4.4	120	s7	5.5	90	s6	7.2	30	s8	9.5	30	s6	12.3	30	s6	14.3	30	s6	17	30	s6	89.8	45
Q117		Upstream confluence with Eight Mile Creek	4.9	120	s7	6.3	90	s6	8.6	60	s8	11.2	30	s2	15	30	s6	17.4	30	s6	20.9	30	s6		
Q118		Upstream Kerr Road	2.1	90	s7	3.2	90	s8	3.8	60	s8	4.8	60	s2	5.9	60	s2	6.7	60	s2	7.9	60	s2	49.5	60
Q119		Downstream Kerr Road	3.1	120	s7	4.7	90	s6	6.3	60	s8	8	60	s2	9.9	60	s2	11.1	60	s2	13.4	60	s2	80.2	60
Q120		Upstream confluence with Eight Mile Creek	3	120	s3	4.6	120	s6	6.6	90	s6	9.1	60	s8	11.5	60	s2	13.6	60	s2	16.7	60	s2	108	60
Q121			0.4	90	s7	0.6	30	s8	0.7	30	s7	0.9	30	s6	1.2	30	s6	1.3	30	s6	1.4	30	s6	8.5	45
Q122			0.5	30	s1	0.7	30	s7	1	30	s7	1.5	30	s6	2.1	30	s6	2.4	60	s6	4.2	60	s6	31.7	60
Q123	Sandy Creek	Downstream Hawksview Road	3.5	120	s7	5.4	120	s6	6.8	30	s6	8.4	30	s6	11.3	30	s6	13.1	30	s6	15.7	30	s6	72.7	45
Q124			5.4	90	s7	8.2	90	s8	10.4	60	s8	13	60	s2	16	60	s2	18.5	30	s2	22.3	30	s2	132	45
Q125			6.6	120	s7	10.2	120	s6	12.7	60	s6	15.5	60	s2	19.3	60	s2	22.5	60	s2	27.2	60	s2	172	60
Q126			6.7	120	s7	10.5	120	s6	13.5	60	s6	16.4	60	s2	20.8	60	s2	24.2	60	s2	28.3	60	s2	183	60
Q127			7.9	120	s6	12.3	120	s6	16.1	90	s6	18.7	90	s8	23.6	60	s8	28.3	60	s2	32.8	60	s2	228	60
Q128		Upstream Knoble Road	11.6	120	s6	18.1	120	s6	23.9	90	s6	29	90	s8	36.5	90	s8	41.6	60	s8	48.5	60	s8		
Q129		Downstream Knoble Road	11.8	120	s3	18	120	s6	24.1	90	s6	29.5	90	s8	37.4	90	s8	42.8	60	s8	49.4	60	s8	352	60
Q130		Upstream Riverina Highway	17	120	s3	23.9	120	s6	30.9	120	s6	38.8	90	s4	48.2	90	s8	55	90	s8	66.7	90	s8		

Refer over for footnotes to table.

TABLE G1 (Cont'd)
DESIGN PEAK FLOWS DERIVED BY TUFLOW MODEL⁽¹⁾

Peak Flow Location Identifier ⁽²⁾	Watercourse	Location	20% AEP			10% AEP			5% AEP			2% AEP			1% AEP			0.5% AEP			0.2% AEP			PMF ⁽⁵⁾	
			Peak Flow (m ³ /s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m ³ /s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m ³ /s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m ³ /s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m ³ /s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m ³ /s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m ³ /s)	Critical Storm Duration ⁽³⁾ (minutes)	Critical Temporal Pattern ⁽⁴⁾	Peak Flow (m ³ /s)	Critical Storm Duration ⁽³⁾ (minutes)
[A]	[B]	[C]	[D]	[E]	[F]	[G]	[H]	[I]	[J]	[K]	[L]	[M]	[N]	[O]	[P]	[Q]	[R]	[S]	[T]	[U]	[V]	[W]	[X]	[Y]	[Z]
Q131	Sandy Creek Tributary		0.9	90	s7	1.2	90	s8	1.6	60	s8	2	60	s2	2.4	60	s2	2.7	60	s2	3.1	60	s2	17.7	60
Q132			4.6	90	s7	6.4	90	s8	8.1	60	s8	10	60	s2	12	60	s2	13.4	30	s2	16	30	s2	89.8	60
Q133			5.4	90	s7	7.5	90	s8	9.5	60	s8	11.7	60	s2	14.2	60	s2	15.9	60	s2	18.6	60	s2	106	60
Q134			6	90	s7	8.6	90	s8	10.8	60	s8	13.3	60	s2	16.1	60	s2	18.1	60	s2	21.3	60	s2	120	60

1. Peak flows less than 100 m³/s have been quoted to one decimal place in order to show minor differences.
2. Refer **Figures 6.2 to 6.12** for location of Flow Location Identifiers.
3. Relates to storm duration that is critical for maximising the peak flood level at each location, not necessarily the peak flow.
4. Relates to temporal pattern that is critical for maximising the peak flood level at each location, not necessarily the peak flow.
5. PMF values not stated at all locations as it was not possible to determine comparable flow rates at individual locations due to the merging of adjacent flow paths and/or lateral inflows.

APPENDIX H

FLOOD DAMAGES

TABLE OF CONTENTS

	Page No.
H1. INTRODUCTION AND SCOPE.....	H-1
H1.1 Introduction	H-1
H1.2 Scope of Investigation	H-1
H1.3 Terminology	H-1
H2. DESCRIPTION OF APPROACH.....	H-2
H3. SOURCES OF DATA	H-4
H3.1 General	H-4
H3.2 Property Data	H-4
H3.3 Flood Levels Used in the Analysis	H-5
H4. RESIDENTIAL DAMAGES	H-6
H4.1 Damage Functions	H-6
H4.2 Total Residential Damages	H-6
H5. COMMERCIAL AND INDUSTRIAL DAMAGES	H-8
H5.1 Damage Functions	H-8
H5.2 Total Commercial and Industrial Damages	H-8
H6. DAMAGES TO PUBLIC BUILDINGS.....	H-10
H6.1 Damage Functions	H-10
H6.2 Total Damages – Public Buildings.....	H-10
H7. SUMMARY OF TANGIBLE DAMAGES	H-11
H7.1 Tangible Damages	H-11
H7.2 Definition of Terms	H-11
H7.3 Average Annual Damages	H-11
H7.4 Net Present Value of Damages.....	H-11
H8. REFERENCES	H-14

FIGURES (BOUND IN VOLUME 2)

- H7.1 Location and AEP at which Buildings first become Above-Floor Inundated (3 sheets)

H1. INTRODUCTION AND SCOPE

H1.1 Introduction

Damages from flooding belong to two categories:

- **Tangible Damages**
- **Intangible Damages**

Tangible damages are defined as those to which monetary values may be assigned and may be subdivided into direct and indirect damages. Direct damages are those caused by physical contact of floodwater with damageable property. They include damages to commercial and residential building structures and contents as well as damages to infrastructure services such as electricity and water supply.

Intangible damages resulting from flooding includes a number of various factors that can have a significant effect on the community. Such factors may include:

- a) risk of injury or loss of life;
- b) mental health impacts such as depression, anxiety and post-traumatic stress disorder; and
- c) social and wellbeing impacts such as isolation, inconvenience, or disruption of family and social activities.

H1.2 Scope of Investigation

In the following sections, both tangible and intangible damages to residential, commercial and industrial properties, and public buildings have been estimated resulting from flooding in the study area.

For the present investigation, the procedures set out in *Flood Risk Management Guideline MM01 – Flood Risk Management Measures* (DPE, 2023) and the associated *NSW Flood Risk Management Tool DT01 (FRM Tool DT01)* were used to undertake an assessment of both the tangible and intangible damages resulting from flooding in the Eight Mile Creek catchment.

The threshold floods at which damages may commence to infrastructure and community assets have also been estimated, mainly from site inspection and interpretation of flood level data. However, there are no data available to allow a quantitative assessment of damages to be made to this category.

H1.3 Terminology

Definitions of the terms used in this Appendix are presented in **Section H8** which also summarises the value of Tangible Flood Damages.

H2. DESCRIPTION OF APPROACH

The damage caused by a flood to a particular property is a function of the depth of flooding above floor level and the value of the property and its contents. The warning time available for residents to take action to lift property above floor level also influences damages actually experienced. The *FRM Tool DT01* was used to estimate damages on a property by property basis according to the type of development, the location of the property and the depth of inundation.

Using the results of the updated flood study, a peak flood elevation was derived for each event at each property. The property flood levels were input to the *FRM Tool DT01* which also contained property characteristics and depth-damage relationships. The depth of flooding was computed as the difference between the interpolated flood level and the floor elevation at each property.

The floor levels of 750 individual dwellings/buildings were assessed by adding the height of floor above a representative natural surface within the allotment (as estimated by visual inspection using Google Street View and archival photos from real estate websites) to the natural surface elevation determined from LiDAR survey. The type of structure and potential for property damage were also assessed during the visual inspection. If a property was not accessible to undertake a visual inspection, the height of the floor was generally assumed to be 300 mm above the adjacent natural surface level, with the exception of dwellings that are located in a number of recently constructed residential estates where a desktop search of real estate websites was used to estimate the floor heights.

A series of depth-damages curves in the *FRM Tool DT01* were used to estimate the cost of tangible damages to residential, commercial, industrial and public properties. The spreadsheet model also includes procedures that were used to estimate intangible damages associated with:

- a) risk of injury or loss of life correlated to the hazard vulnerability classification of flooding;
- b) mental health costs correlated to the depth of above-floor inundation; and
- c) social and wellbeing costs correlated to the frequency of above-floor inundation.

It should be understood that this approach is not intended to identify individual properties liable to flood damages and the values of damages in individual properties, even though it appears to be capable of doing so. The reason for this caveat lies in the various assumptions used in the procedure, the main ones being:

- the assumption that computed water levels and topographic data used to define flood extents are exact and without any error;
- the assumption that the water levels as computed by the hydraulic model are not subject to localised influences;
- the estimation of property floor levels by visual inspection rather than by formal field survey;
- the use of "average" stage-damage relationships, rather than a unique relationship for each property;
- the uncertainties associated with assessing appropriate factors to convert *potential damages* to *actual flood damages* experienced for each property after residents have taken action to mitigate damages to contents.

The consequence of these assumptions is that some individual properties may be inappropriately classified as flood liable, while others may be excluded. Nevertheless, when applied over a broad area these effects would tend to cancel, and the resulting estimates of overall damages, would be expected to be reasonably accurate.

For the above reasons, the information contained in the spreadsheets used to prepare the estimates of flood damages for the study area should not be used to provide information on the depths of above-floor inundation of individual properties.

H3. SOURCES OF DATA

H3.1 General

To estimate *Average Annual Flood Damages* for a specific area it is necessary to estimate the damages for several floods of different magnitudes, i.e., of different frequencies, and then to integrate the area beneath the damage – frequency curve over the whole range of frequencies. To do this it is necessary to have data on the damages sustained by all types of property over the likely range of inundation. There are several ways of doing this:

- The ideal way would be to conduct specific damage surveys in the aftermath of a range of floods, preferably immediately after each. An example approaching this ideal is the case of Nyngan where surveys were conducted in May 1990 following the disastrous flood of a month earlier (DWR, 1990). This approach is not possible in the study area as specific damage surveys have not been conducted following the historic flood events.
- The second best way is for experienced loss adjusters to conduct a survey to estimate likely losses that would arise due to various depths of inundation. This approach is used from time to time, but it can add significantly to the cost of a floodplain management study. It was not used for the present investigation.
- The third way is to use generalised data that are considered to be suitable for broad regional studies. They are not considered to be suitable for use in specific areas unless none of the other approaches can be satisfactorily applied.
- The fourth way is to adapt or transpose data from other flood liable areas. The approach set out in DPE, 2023 and the *FRM Tool DT01* is based on data collected following major flooding in various urban centres across NSW and has been adopted for the present study.

H3.2 Property Data

The properties were divided into three categories: residential, commercial/industrial and public buildings.

For residential properties, the data used in the damages estimation included:

- the location/address of each property
- an assessment of the type of structure
- representative natural surface level of the allotment
- floor level of the residence

For commercial/industrial properties, the data used in the damages estimation included:

- the location of each property
- the nature of each enterprise
- an estimation of the floor area
- natural surface level
- floor level

The property descriptions were used to classify the commercial/industrial developments into categories (i.e., high, medium or low value properties) which relate to the magnitude of likely flood damages.

The total number of residential properties, commercial / industrial and public buildings in the study area is shown in **Table H3.1**.

TABLE H3.1
NUMBER OF PROPERTIES INCLUDED IN DAMAGES DATABASE

Development Type	Number of Properties
Residential	3,610
Commercial / Industrial	38
Public	116
Total	3,764

H3.3 Flood Levels Used in the Analysis

Damages were computed based on the design flood envelope levels that were determined from the hydraulic models that were developed as part of the present investigation. The design levels assume that the drainage system is operating at optimum capacity, with only the AEP neutral blockage factors applied to major hydraulic structures. They do not allow for any increase in levels resulting from wave action and significant debris build-ups at the location of major hydraulic structures, as well as other local hydraulic effects. These factors are usually taken into account by adding a factor of safety (freeboard) to the “nominal” flood level when assessing the “level of protection” against flooding of a particular property. Freeboard could also include an allowance for the future effects of climate change.

H4. RESIDENTIAL DAMAGES

H4.1 Damage Functions

The procedures identified in DPE, 2023 allow for the preparation of a depth versus damage relationship which incorporates structural damage, damage to contents, external damage, relocation costs and clean-up costs. In limited cases, the additional damage costs related to structural integrity due to building failure may also warrant consideration. Depth versus damage curves are computed for single and double storey residences.

The level of flood awareness and available warning time are taken into account by factors which are used to reduce “potential” damages to contents to “actual” damages. “Potential” damages represent losses likely to be experienced if no action were taken by residents to mitigate impacts. A reduction in the potential damages to “actual” damages is usually made to allow for property evacuation and raising valuables above floor level, which would reduce the damages actually experienced. The ability of residents to take action to reduce flood losses is mainly limited to reductions in damages to contents, as damages to the structure and clean-up costs are not usually capable of significant mitigation.

The reduction in damages to contents is site specific, being dependent on a number of factors related to the time of rise of floodwaters, the recent flood history and flood awareness of residents and emergency planning by the various Government Agencies (BoM and NSW SES).

The actual damage to contents in an event can be reduced by actions taken during the warning time available in response to a flood threat. The actual to potential damage ratio is dependent on the effective warning time, likely duration of inundation of contents, flood awareness of the community, the likelihood of at least one resident being present at the time of the flood, the ability of the individual to lift goods and the height to which goods would need to be raised. As there is minimal warning time available in the study area, the default actual to potential damage ratio of 0.9 was adopted for the present study.

H4.2 Total Residential Damages

Table H4.1 over the page summarises the residential damages in the study area for floods between 20% AEP and the PMF which were modelled hydraulically as part of the present study.

A flood slightly smaller than a 10% AEP event is the threshold at which dwellings commence to be subject to above-floor inundation. At the 1% AEP level of flooding, 27 dwellings would experience above-floor inundation resulting in a total flood damages of about \$2.7 Million, comprised of about \$1.8 Million of structural damage, \$0.4 Million of contents damage and \$0.5 Million of external damage.

During a PMF event, a total of 769 individual dwellings would experience above-floor inundation resulting in total flood damages of about \$164 Million, with the maximum depth of above-floor inundation in the worst affected dwelling reaching about 2.4 m.

TABLE H4.1
TOTAL RESIDENTIAL FLOOD DAMAGES

Design Flood Event	No. of Properties		Flood Damages (\$ Million)				
	Flood Affected	Flooded Above Floor Level	Structural	Contents	External	Intangibles	TOTAL
20% AEP	10	0	0.0	0.0	0.0	0.0	0.0
10% AEP	21	1	0.1	0.0	0.0	0.0	0.1
5% AEP	31	5	0.4	0.1	0.1	0.0	0.6
2% AEP	52	15	0.8	0.1	0.3	0.0	1.2
1% AEP	81	27	1.8	0.4	0.5	0.0	2.7
0.5% AEP	95	37	2.6	0.6	0.7	0.0	3.9
0.2% AEP	126	52	3.9	1.0	1.0	0.1	6.0
PMF	1,059	769	91.9	52.6	14.4	5.5	164.4

H5. COMMERCIAL AND INDUSTRIAL DAMAGES

H5.1 Damage Functions

The procedures identified in DPE, 2023 allow for the preparation of a depth versus damage relationship for commercial and industrial buildings. The damage costs include the indirect costs associated with loss of trading and post-flood clean-up for commercial and industrial buildings.

Commercial and industrial property damages are highly variable, with the particular use and associated contents (rather than the structure) generally dominating the overall damage. The damage category assigned to each enterprise may vary between "low", "medium" or "high", depending on the nature of the enterprise set out in **Table H5.1** below. Damages also depend on the floor area.

TABLE H5.1
ASSESSED COMMERCIAL AND INDUSTRIAL DAMAGE CATEGORIES

Proposed classification	Adjustment to average value curve	Representative uses
Low to medium	60% of average	Restaurants, cafes, offices, doctor's surgeries, retail/food outlets, butchers, bakeries, newsagencies, service stations, hardware
Medium/default	100%	Proposed as a representative average, where the particular use is not known
Medium to high	150% of average	Chemists, electrical goods, clothing stores, bottle shops, electronics

H5.2 Total Commercial and Industrial Damages

Table H5.2 over the page summarises the estimated commercial and industrial damages in the study area. Similar to residential development, a flood with an AEP of between 1% and 0.5% is the threshold at which individual commercial/industrial buildings commence to be subject to above-floor inundation. A total of eight commercial buildings would experience above-floor inundation in a PMF event, with the flood damages estimated to be about \$9.4 Million.

TABLE H5.2
COMMERCIAL / INDUSTRIAL FLOOD DAMAGES

Design Flood Event	No. of Properties		Total Damages ⁽¹⁾ (\$ Million)
	Flood Affected	Flooded Above Floor Level	
20% AEP	0	0	0.0
10% AEP	0	0	0.0
5% AEP	0	0	0.0
2% AEP	0	0	0.0
1% AEP	0	0	0.0
0.5% AEP	1	1	0.6
0.2% AEP	2	1	0.9
PMF	11	8	9.4

1. Total damages are a combination of structural and internal flood damages

H6. DAMAGES TO PUBLIC BUILDINGS

H6.1 Damage Functions

The procedures identified in DPE, 2023 allow for the preparation of a depth versus damage relationship for public buildings. The damage costs include the indirect costs associated with post-flood clean-up for public buildings.

As part of the FRM tool DT01, depth versus damage relationship for public buildings have been classified for three categories which are schools, hospitals and other buildings, the latter of which comprises the following uses:

- Health (e.g. aged care, nursing home);
- Emergency Services (e.g. police station, fire station, ambulance station, NSE SES facilities etc.); and
- Government Buildings (e.g. courthouse, government administration buildings, diplomatic facilities, consulate facilities, major defence facilities, correctional facilities etc).

H6.2 Total Damages – Public Buildings

Table H6.1 summarises the estimated public damages in the study area. A flood with an AEP of between 5% and 2% is the threshold at which individual public buildings commence to be subject to above-floor inundation. The same building would be flooded up to 0.5% AEP level of flooding, while a total of 11 public buildings would experience above-floor inundated in a PMF event, resulting in total flood damages of about \$1.1 Million.

TABLE H6.1
PUBLIC FLOOD DAMAGES

Design Flood Event	No. of Properties		Total Damages ^(1,2) (\$ Million)
	Flood Affected	Flooded Above Floor Level	
20% AEP	0	0	0
10% AEP	1	0	0
5% AEP	1	0	0
2% AEP	1	1	0
1% AEP	1	1	<0.1
0.5% AEP	1	1	<0.1
0.2% AEP	3	2	<0.1
PMF	11	11	1.1

1. Total damages are a combination of structural and internal flood damages
2. Public damages quoted to two decimal places to highlight small amounts

H7. SUMMARY OF TANGIBLE DAMAGES

H7.1 Tangible Damages

Flood damages have been computed for a range of flood frequencies from 20% AEP up to the PMF. As set out in **Table H7.1** over, about \$3 Million of damages would be incurred at the 1% AEP level of flooding in the Eight Mile Creek catchment, increasing to a total of about \$192 Million for the PMF.¹ **Figure H7.1** (3 Sheets) shows the plan location of the properties that were included in the flood damages assessment and the AEP at which each is first inundated above-floor level.

H7.2 Definition of Terms

Average Annual Damages (also termed “expected damages”) are determined by integrating the area under the damage-frequency curve. They represent the time stream of annual damages, which would be expected to occur on a year by year basis over a long duration.

Using an appropriate discount rate, average annual damages may be expressed as an equivalent “*Net Present Value*” (**NPV**) of damages and used in the economic analysis of potential flood management measures.

A flood management scheme which has a design 1% AEP level of protection, by definition, will eliminate damages up to this level of flooding. If the scheme has no mitigating effect on larger floods then these damages represent the benefits of the scheme expressed on an average annual basis and converted to the NPV via the discount rate.

Using the procedures outlined in DPE, 2023 and NSW Treasury Guidelines, economic analyses were carried out assuming a 30 year economic life for projects and discount rates of 5% pa. (best estimate) and 7% and 3% pa (sensitivity analyses).

H7.3 Average Annual Damages

The average annual damages for all flood events up to the PMF are shown below in **Table H7.2**. Note that values have been quoted to two decimal places to highlight the relatively small recurring damages.

H7.4 Net Present Value of Damages

The NPV of damages likely to be experienced for all flood events up to the 5% AEP and 1% AEP, as well as the PMF, for a 30 year economic life and discount rates of 3, 5 and 7 per cent are shown in **Table H7.3**.

For a discount rate of 5% pa, the NPV of total damages for flood events up to the 1% AEP flood in the Eight Mile Creek catchment is about \$1.7 Million. Based on this finding, one or more schemes costing up to this amount could be economically justified if they eliminated damages across the whole of the Eight Mile Creek catchment for all floods up to 1% AEP in magnitude. While schemes costing more than this value would have a benefit/cost ratio less than 1, they may still be justified according to a multi-objective approach which considers other criteria in addition to economic feasibility.

¹ Note that the total flood damages include a provision for damage to general public infrastructure which is assumed to equate to 10% of the total residential flood damages.

**TABLE H7.1
TOTAL FLOOD DAMAGES
\$ MILLION**

Design Flood Event	Residential	Commercial/Industrial	Public	General Public Infrastructure	Total
20% AEP	0.0	0.0	0.0	0.0	0.0
10% AEP	0.1	0.0	0.0	0.0	0.1
5% AEP	0.6	0.0	0.0	0.1	0.7
2% AEP	1.2	0.0	0.0	0.1	1.3
1% AEP	2.7	0.0	0.0	0.3	3.0
0.5% AEP	3.9	0.6	0.0	0.4	4.9
0.2% AEP	6.0	0.9	0.0	0.6	7.5
PMF	164.4	9.4	1.1	16.4	191.3

**TABLE H7.2
AVERAGE ANNUAL DAMAGES
\$ MILLION**

Design Flood Event	Residential	Commercial/Industrial	Public	General Public Infrastructure	Total	
					Contribution to AAD ⁽¹⁾	Cumulative AAD ⁽²⁾
20% AEP	0	0	0	0	0	0
10% AEP	0.01	0	0	0	0.01	0.01
5% AEP	0.02	0	0	0	0.02	0.03
2% AEP	0.03	0	0	0	0.03	0.06
1% AEP	0.02	0	0	0	0.02	0.08
0.5% AEP	0.02	0	0	0	0.02	0.10
0.2% AEP	0.01	0	0	0	0.01	0.11
PMF	0.17	0.01	0	0.02	0.20	0.31

1. Represents the contribution to the total average annual damages for the specified design flood event
2. Represents the cumulative annual average damages for all floods up to the specified design flood event in magnitude.

TABLE H7.3
NET PRESENT VALUE OF DAMAGES
\$ MILLION

Discount Rate (%)	All Floods up to 5% AEP	All Floods up to 1% AEP	All Floods up to the Extreme Flood
3	1.0	2.1	6.7
5	0.8	1.7	5.3
7	0.6	1.3	4.3

H8. REFERENCES

DPE (Department of Planning), 2023. ***“Flood Risk Management Guideline MM01 – Flood Risk Management Measures”***