WOLLONDILLY SHIRE COUNCIL

STONEQUARRY CREEK (PICTON) FLOOD STUDY UPDATE

FINAL REPORT
VOLUME I

SEPTEMBER 2020

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Project Stonequarry Creek (Picton) Flood Study Update	**Project Number** 117094
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Date 16 September 2020September 2020	**Verified by** ***Erin Askew***

Revision	**Description**	**Distribution**	**Date**
1	Flood Study Update Draft Final Report	WSC, NSW OEH	June 2019
2	Flood Study Update Final Report	WSC, NSW DPIE	September 2020

Cover photo: Picton Railway Viaduct, WMAwater 2018

STONEQUARRY CREEK (PICTON) FLOOD STUDY UPDATE

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LIST OF ACRONYMS

AEP	Annual Exceedance Probability
ARI	Average Recurrence Interval
ALS	Airborne Laser Scanning
ARR	Australian Rainfall and Runoff
BOM	Bureau of Meteorology
DECC	Department of Environment and Climate Change (now OEH)
DNR	Department of Natural Resources (now OEH)
DRM	Direct Rainfall Method
DTM	Digital Terrain Model
GIS	Geographic Information System
GPS	Global Positioning System
IFD	Intensity, Frequency and Duration (Rainfall)
mAHD	meters above Australian Height Datum
OEH	Office of Environment and Heritage
PMF	Probable Maximum Flood
SRMT	Shuttle Radar Mission Topography
TUFLOW	one-dimensional (1D) and two-dimensional (2D) flood and tide simulation software (hydraulic model)
WBNM	Watershed Bounded Network Model (hydrologic model)

ADOPTED TERMINOLOGY

Australian Rainfall and Runoff (ARR, ed Ball et al, 2016) recommends terminology that is not misleading to the public and stakeholders. Therefore, the use of terms such as “recurrence interval” and “return period” are no longer recommended as they imply that a given event magnitude is only exceeded at regular intervals such as every 100 years. However, rare events may occur in clusters. For example, there are several instances of an event with a 1% chance of occurring within a short period, for example the 1949 and 1950 events at Kempsey. Historically the term Average Recurrence Interval (ARI) has been used.

ARR 2016 recommends the use of Annual Exceedance Probability (AEP). Annual Exceedance Probability (AEP) is the probability of an event being equalled or exceeded within a year. AEP may be expressed as either a percentage (%) or 1 in X. Floodplain management typically uses the percentage form of terminology. Therefore a 1% AEP event or 1 in 100 AEP has a 1% chance of being equalled or exceeded in any year.

ARI and AEP are often mistaken as being interchangeable for events equal to or more frequent than 10% AEP. The table below describes how they are subtly different.

For events more frequent than 50% AEP, expressing frequency in terms of Annual Exceedance Probability is not meaningful and misleading particularly in areas with strong seasonality. Statistically a 0.5 EY event is not the same as a 50% AEP event, and likewise an event with a 20% AEP is not the same as a 0.2 EY event. For example, an event of 0.5 EY is an event which would, on average, occur every two years. A 2 EY event is equivalent to a design event with a 6-month Average Recurrence Interval where there is no seasonality, or an event that is likely to occur twice in one year.

The Probable Maximum Flood is the largest flood that could possibly occur on a catchment. It is related to the Probable Maximum Precipitation (PMP). The PMP has an approximate probability. Due to the conservativeness applied to other factors influencing flooding a PMP does not translate to a PMF of the same AEP. Therefore, an AEP is not assigned to the PMF.

This report has adopted the approach recommended by ARR and uses % AEP for all events rarer than the 50 % AEP and EY for all events more frequent than this.

Frequency Descriptor	EY	AEP (%)	AEP (1 in x)	ARI
Very Frequent	12			
	6	99.75	1.002	0.17
	4	98.17	1.02	0.25
	3	95.02	1.05	0.33
	2	86.47	1.16	0.5
	1	63.21	1.58	1
Frequent	0.69	50	2	1.44
	0.5	39.35	2.54	2
	0.22	20	5	4.48
	0.2	18.13	5.52	5
	0.11	10	10	9.49
Rare	0.05	5	20	20
	0.02	2	50	50
	0.01	1	100	100
Very Rare	0.005	0.5	200	200
	0.002	0.2	500	500
	0.001	0.1	1000	1000
	0.0005	0.05	2000	2000
Extreme	0.0002	0.02	5000	5000
			PMP/ PMPDF	

FOREWORD

The NSW State Government's Flood Prone Land Policy provides a framework to ensure the sustainable use of floodplain environments. The primary objective of the NSW Government's Flood Prone Land Policy is to reduce the impact of flooding and flood liability on individual owners and occupiers of flood prone property, and to reduce private and public losses resulting from floods. At the same time, the policy recognises the benefits flowing from the use, occupation and development of flood prone land (Reference 4).

Under the Policy, the management of flood liable land remains the responsibility of local government. The State Government 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 five sequential stages:

1. ***Data Collection***
   - Compilation of existing data and collection of additional data.
2. ***Flood Study***
   - Determine the nature and extent of the flood problem.
3. ***Floodplain Risk Management***
   - Determines options in consideration of social, ecological and economic factors relating to flood risk.
4. ***Floodplain Risk Management Plan***
   - Preferred options are publicly exhibited and subject to revision in light of responses. Formally approved by Council after public exhibition and any necessary revisions due to public comments.
5. ***Implementation of the Plan***
   - Implementation of flood, response and property modification measures (including mitigation works, planning controls and flood warnings for example) by Council.

1. INTRODUCTION

The Stonequarry Creek (Picton) Flood Study Update has been prepared by WMAwater on behalf of Wollondilly Shire Council (Council) and will form the basis of the Stonequarry Creek (Picton) Floodplain Risk Management Study and Plan.

This Flood Study Update follows on from the Draft Stonequarry Creek (Picton) Flood Study (Reference 5) which determined the nature and extent of the flood problem in the township of Picton under existing conditions and in accordance with industry guidelines that were current at the time.

In 2016 (late in the previous Flood Study project), the Australian Rainfall and Runoff guidelines were updated due to the availability of numerous technological developments, a significantly larger rainfall dataset since the previous edition (in 1987) and development of updated methodologies. A key input to the update is information derived from rainfall gauges, and the dataset now includes a larger number of rainfall gauges which continuously recorded rainfall (pluviometers) and a longer record of storms, including additional rainfall data recorded between 1985 and 2012.

This report details the factors that led to the decision to update the flood models developed in the Draft Flood Study (Reference 5) using the methodologies described in ARR 2016, and provides updated design flood results for the for the 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% Annual Exceedance Probability (AEP) events. It is noted that the Probable Maximum Flood (PMF) flows were derived using the Bureau of Meteorology's Generalised Short Duration Method (Reference 25) to estimate the probable maximum precipitation (PMP), and the methodology was not revised as part of the updates to the Australian Rainfall and Runoff guidelines.

This report also describes updates made to the modelling to incorporate recent development in the catchment, in particular along Racecourse Creek, and changes to model packages to efficiently apply ARR 2016 methodologies.

All levels provided in this report are in metres to Australian Height Datum (AHD) or relate to the Stonequarry Creek gauge (m) at Picton (Gauge number: 212053) which will be referred to as the Picton Gauge in this report for ease of reference. Note that the local gauge datum (referred to as "Gauge Zero") equates to 147.803 mAHD (Australian Height Datum). A glossary of terms is provided in Appendix A.

1.1. Study Area

Picton is located approximately 90 km south west of Sydney in the Wollondilly Shire Council Local Government Area (LGA). The township is located on the banks of Stonequarry Creek, approximately 4.5 km upstream of its confluence with the Nepean River (see Figure 1). Stonequarry Creek is a tributary of the Nepean River, and has a catchment of 84 km^2. Stonequarry

Creek receives inflows from four main tributaries: Racecourse Creek from the east, Crawfords Creek from the north, and Cedar and Mathews Creek to the west of Picton.

Picton has a population of approximately 3,500 (2016 census) with land use in the township predominantly composed of low-density residential development with some commercial development along the main street (Argyle Street) and light industrial areas at the southern end of the town. In addition, there are large areas of open space (rural landscape) surrounding the town centre, characterised by hills sloping down towards Stonequarry Creek. The local topography is presented on Figure 2,

Flooding in Picton can occur as a result of flow breaking out of the main channel of Stonequarry Creek and inundating the surrounding floodplain. In larger events, water that overtops the banks of Stonequarry Creek can inundate parts of the Town Centre and surrounding ubran areas. Local rainfall over Picton can also cause flash flooding, as runoff from the surrounding slopes enters the Town Centre and can exceed the stormwater network capacity. The Study Area (displayed on Figure 1) covers areas of Picton that contribute to overland flow, as well as the Stonequarry Creek floodplain between Abbotsford Road (in the town's west) and about 1 km downstream (south) of the railway viaduct. In addition, approximately 1.5 km of Racecourse Creek between its confluence with Stonequarry Creek and the eastern boundary of Antill Country Golf Course. The Study Area covers an area of approximately 84.6 km^2.

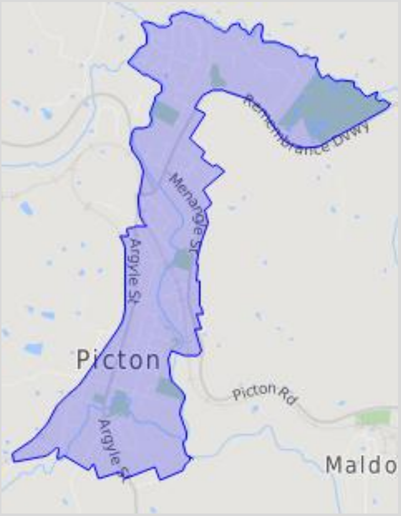
1.2. Land Use

Land use zoning is defined by the Wollondilly Local Environmental Plan (LEP 2011) and is shown on Figure 3. The majority of residential development within Picton is comprised of lots zoned *R2 Low Density Residential* with areas of *R3 Medium Density Residential* behind the town centre, and *R5 Large Lot Residential* west of Stonequarry Creek. A *B2 Local Centre* area which allows for commercial/industrial uses is situated along Argyle Street. Stonequarry Creek itself is classified as *E2 Environmental Conservation*, and it is bordered, generally on both sides, by *RE1 Public Recreation* and *RE2 Private Recreation* allowing for multiple uses including playing fields and golf a course. There is a relatively small amount of *IN2 Light Industrial* area at the southern end of the town. Land use outside of the township of Picton is generally zoned *RU2 Rural Landscape*.

1.3. Demographic Overview

Understanding the social characteristics of the Study Area can help in ensuring appropriate risk management practices are adopted, and shape the methods used for community engagement. Census data regarding house tenure and age distribution can also provide an indication of the community's lived experience with recent flood events, and hence an indication of their flood awareness. The following information has been extracted from the 2016 Census for the town of Picton and is considered relevant, while Table 1 below shows some of the characteristics of Picton compared to the NSW average.

Picton Demographic Overview



Population: 3,454
No. of Private Dwellings: 1,365
No. of lone person households: 257
Property Tenure:

- 73.7% owned (either outright or with a mortgage)
- 23.6% rented

Language

- 90.2% of people speak only English at home

No. persons over the age of 75: 199
Elderly people may be unable to respond as quickly to flood emergencies without requiring some assistance.

No. single parent families: 160
Single parent families can mean a low adult-to-child ratio within the household and therefore can make evacuation more difficult.

Statistics from quickstats.censusdata.abs.gov.au/census_services/getproduct/census/2016/quickstat/UCL115112?opendocument

Table 1: Characteristics of Picton (Australian Bureau of Statistics, 2016)

	Picton (Town)	NSW
Population Age:		
0 – 14 years	20.0%	18.5%
15 - 64 years	66.1%	65.1%
> 65 years	13.9%	16.2%
Average people per dwelling	2.7	2.6
Own/mortgage property	73.7%	64.5%
Rent property	23.6%	31.8%
Other tenure type/not stated	2.7%	3.7%
No cars at dwelling	4.3%	9.2%
Speak only English at home	90.2%	68.5%

The characteristics noted above are considered in the community engagement strategy and when considering response modification options, such as flood education, warning or evacuation systems. Given the high proportion of English-only households, the delivery of community consultation material and flood warnings/ information in English is deemed appropriate. The proportion of residents over the age of 65 is lower than the NSW state average, however aged residents are more likely to be frail and unable to respond as quickly to flood emergencies. These residents may also prefer to receive hardcopy newsletters than via online methods. Provision of assistance to such residents should be a key consideration when developing flood evacuation systems and the lead time with which warnings are provided.

The family composition within a residence can also affect flood awareness and capacity to respond. In Picton there are 257 lone person households, who are at greater risk of being unaware of flood warnings or evacuation orders. There are also 160 single parent families, which can mean a low adult-child ratio and result in difficulties preparing for and safely undertaking evacuations.

1.4. Local Environment

1.4.1. Riparian Vegetation

The Stonequarry Creek catchment is characterised by grassed hills and areas of moderate to dense tree cover, with urban areas within the Picton township and parts of Thirlmere to the south. The Stonequarry Creek channel itself is characterised by a degraded sandstone gully forest with high levels of weed infestation primarily of privet, moth vine and honeysuckle. The most prominent native species of trees along the creek include the Broad-leaved Apple, Forest Red Gum, and River Oak (Reference 12).

Privet is the dominant roost tree particularly along the middle and upper reaches of the creek banks. Other non-natives along the banks dominate the mid and upper storey stratum. Many of these weeds outcompeted native growth following extensive clearing on both sides of the creek. There is also evidence of land slippage as a result of removal of native vegetation (non-natives species can be less effective at stabilising creek banks), or due to the removal of riparian vegetation entirely. Garden plants from residential properties in close proximity to the creek have also established themselves on the creek banks including Pampas Grass and Giant Bamboo. Mature eucalyptus trees occupy the upper banks of the creek particularly on the eastern side. There has been work done in regard to actively removing weeds in the area and restoring some native vegetation. However, this can be challenging due to the steep banks of the creek (Reference 12).

1.4.2. Vegetation Management Practices

Riparian vegetation management can affect a range of factors including flood conveyance (by reducing hydraulic roughness), bank stability, reducing the occurrence of channel blockages, improving safety and amenity, and protecting ecological and geomorphic assets. Wollondilly Shire Council undertakes regular vegetation management activities as per guidelines in Council's *Stonequarry Creek Vegetation Management Plan, 1994)*, which strives to achieve a balance of the aforementioned factors.

Following the June 2016 flood event, Council engaged Soil Conservation Service in September 2016 to provide an assessment of river processes and erosion in Stonequarry Creek, review current vegetation management practices from a stream stability perspective, and provide a prioritised list of remedial works (Reference 13). The resulting report noted that *"Council's vegetation management practices, particularly crown lifting of in-channel trees, selective removal of regrowth, and weed control, appear to be producing no reach-scale instabilities (of the kind related to increased velocity of resultant floodwaters). It is recommended these be continued, due to their lessening backwater effects in flooding, and therefore favourable outcome in lessening flood peaks."*

In the past year, Council has specifically been working in Racecourse Creek and Stonequarry Creek north of Picton, to remove tree trunks (specifically Casuarinas) out of the creek bed and anchor them to the bank, such that the logs are aligned with the direction of flow. This significantly reduces the obstruction to flow, while achieving ecological outcomes in regards to the protection of native habitats. An example is shown in Photo 1 below. It is important to note the sensitive balance between maintaining a 'clear' channel (e.g. to increase flow conveyance), bank stability, and conservation of native species rather than weeds that may grow in their place if removed. It is also noted that appropriate vegetation may assist in attenuating flood flows and reducing downstream flood levels, and that 'creek clearing' in unsuitable locations may cause flood behaviour to be worsened elsewhere.

Photo 1: Fallen Casuarina logs anchored to Stonequarry Creek bank, just downstream of the Racecourse Creek confluence (WMAwater, 26/11/2018)

1.4.3. Local Fauna

There are a number of animal species that occupy the area, including frogs such as the Verreaux's Frog and Common Eastern Froglet. Additionally, there are species of skink including the Eastern Water-skink and Dark-flecked Garden Sunskink. With the exception of the Grey Headed Flying Fox there is little evidence of mammal activity (Reference 12).

The Stonequarry Creek flying fox camp is located between the railway Viaduct at the end of Webster Street and the Prince Street Bridge. The camp is home to some 2000 Grey Headed Flying Foxes (as of November 2016) which seasonally occupy the area. Council staff regularly undertake flying fox counts and are in the process of developing a camp plan of management. In addition, several bird species occupy the area which most commonly include the Superb Fairy Wren, Red-browed Finch and Australian Magpie (Reference 12).

2. PREVIOUS INVESTIGATIONS

2.1. Picton Flood Study Report, Department of Water Resources, 1989 (Reference 8)

In 1989, the NSW Department of Water Resources (DWR) completed the 'Picton Flood Study Report'. The project began in May 1986, when Wollondilly Shire Council requested the then Water Resources Commission initiate the flood study because of the increasing demand to develop areas that may be liable to flooding and the need to develop a floodplain management plan to reduce risk to life and property in Picton.

A RAFTS hydrological model was used to convert rainfall to runoff hydrographs. Once the hydrographs were determined, a one-dimensional HEC2 (Hydrologic Engineering Centre (1981)) model was developed for the hydraulic component of the flood study. The floodplain topography was defined by a series of surveyed cross sections across the channel or floodplain perpendicular to the direction of flow.

The study provided flood profiles and levels for the 20 year, 50 year, and 100 year ARI design events at 23 cross sections, determined hazard and hydraulic category mapping, and estimated flood damages. The results indicated that the floodway would pass through a large portion of the commercial centre of the town (i.e. Argyle Street). It was considered likely that any flows in this area would be extremely turbulent with localised variations in water level and velocity between buildings. The report identified that there was likely to be significant flood damages with 58 residential and 48 commercial properties subject to inundation in the 100 year ARI design flood. Many of these properties are along Argyle Street (between Menangle Street and Stonequarry Creek).

The Picton Flood Study Report also provided a preliminary consideration of a range of flood risk mitigation measures that may be suitable and effective in Picton. Below is a summary of the various types of structural mitigation options, and the findings of the report. The report also recommended improvements to flood warning systems, and the use of zoning and development controls as the most effective means of containing the growth of flood damages and complement any structural mitigation works. The report however noted that any decision to pursue such works would require detailed consideration by Council.

Table 2: Summary of mitigation options considered in the 1989 Flood Study (Reference 8)

Option	Description	Conclusion
Retarding Basins	Temporarily store water during a storm runoff period, and lessen flow rates and water levels downstream. Typically most effective in upper reaches of the catchment.	Considered impractical in Picton due to the steep nature of the catchment and a lack of suitable sites near the township. No analysis of basins was carried out.
Levee	Typically an embankment structure used to protect properties from flooding, providing account is taken of potential flow redistribution and the possibility of overtopping of the levee in floods greater than the design flood.	A 1 in 100 year flood level plus 1m freeboard (the then Dept. standard) would necessitate a levee up to 4 m high, which would be considered unsightly and block internal drainage unless specific allowance was made. The levee would increase peak flood levels by up to 1 m (near

Option	Description	Conclusion
		Elizabeth Street), and increase in-stream velocities, leading to scour and erosion in the channel. Analysis of the levee option resulted in a cost-benefit ratio of 0.77, indicating it would not be economically viable.
Major stream clearing & linear park project	Stream clearing should concentrate on the removal of weed growth, exotic species and any willows within the channel, and creating a park along both banks. Mature trees would generally be maintained and only removed if they obstruct flow or threaten to fall across the channel.	The option was tested by adjusting hydraulic roughness coefficients and indicated that with 'clear conditions' flood levels may be reduced immediately downstream of Stonequarry Bridge, however the impacts on in-stream velocities, bank stability and downstream peak flood levels were not assessed.
Re-shaping the channel	This option involves changing the bed width and side slopes, testing widths of 8 m and side slopes of 2:1 (horizontal/vertical) to bed widths of 20 m and side slopes of 4:1, in addition to the clearing works described above.	At the lower range, flood levels were dropped by a further 0.25 m on those estimated with clearing alone (excavation of approx. 63,200 m^3). Excavation of ~1.4 million cubic metres would be required to achieve the higher range, a degree of work not considered warranted. Ongoing maintenance costs and environmental impacts were not considered in the initial option assessment.

2.2. Stonequarry Creek Floodplain Management Plan (Willing & Partners, 1996)

Wollondilly Shire Council engaged Willing & Partners to produce the Stonequarry Creek Floodplain Management Plan, which followed on from the Picton Flood Study (Reference 8) and previous work by Willing & Partners on the Stonequarry Creek Floodplain Management Study (completed in 1992). The hydraulic model developed in Reference 8 was used to simulate flood behaviour for several mitigation options. The structural measures assessed were:

- Vegetation management along Stonequarry Creek from the confluence with Racecourse Creek to the railway viaduct;
- Channel reconstruction and lining;
- Levee banks near the commercial centre and residences; and
- Retarding basins upstream of Picton, to hold back water during floods.

The Study found that the channel reconstruction and formalisation, levee bank and retarding basin measures were found to be very expensive in relation to the flood damage that could be prevented, and would also have a detrimental visual and environmental impact.
The Floodplain Management Study recommended:

- Vegetation management of riparian areas (then referred to as 'stream clearing';
- House raising;
- Building and development controls;
- Flood warning; and
- Flood response and evacuation planning.

The preparation of this Floodplain Management Plan occurred at the same time as a Vegetation Management Plan (VMP) for the Stonequarry Creek corridor (Reference 7).

2.3. Stonequarry Creek Vegetation Management Plan, Ian Perkins Consultancy Services, April 1996 (Reference 7)

The Vegetation Management Plan (VMP) was developed for Wollondilly Shire Council as part of the Stonequarry Creek Floodplain Management Plan (Reference 6) in response to the recommended option of "stream clearing". The objective of the VMP was to provide a strategy for vegetation planning that will create a valuable corridor of vegetation without increasing flooding. The plan stresses that the proposed VMP is a compromise between the need to manage the creek system for flood hydraulics and the desire to improve biodiversity and the environmental attributes of the site. Accordingly, the management of vegetation was not aimed at restoring the structure of the original plant communities to its original condition, due to Engineering/ Social/ Economic and Environmental constraints. A structurally modified representation of the original plant communities was therefore recommended for most sections of the creek, informed by detailed site assessment and computer modelling of flood hydraulics.

The VMP noted the need for ongoing reassessment of creek vegetation (including existing, replanted and regenerated vegetation), to monitor the vegetation density. If vegetation densities exceed the target ranges for each zone recommended by the VMP, flood level increases may occur. The VMP divided the creek line into three distinct management zones, and described individual strategies for weed control, clearing, site stabilisation, revegetation and regeneration developed for each of these zones:

- **Zone 1**
  Between Racecourse Creek and Elizabeth Street. Vegetation communities in this zone were identified in the flood model as having a low influence on flood levels.
- **Zone 2**
  Between Elizabeth Street and Coull Street. Vegetation in this zone was identified as having a significant influence on flood levels.
- **Zone 3**
  Between Coull Street and the Viaduct. Vegetation in this zone was identified as generally having a moderate influence on flood levels.

2.4. Stonequarry Creek – 2D Modelling and WaterRIDE Application (Patterson Britton & Partners, 2006) (Reference 9)

Wollondilly Shire Council engaged Advisian (*then Patterson Britton & Partners)* in 2006 to update the 1989 Flood Study using current two-dimensional hydraulic modelling techniques. This involved updating the 1989 RAFTS hydrologic model to the then current catchment conditions, including increases to impervious area where urbanisation had occurred since 1989. Instead of updating the 1989 HEC-2 model, a new two dimensional RMA-2 model was developed covering the same extent as the 1989 HEC-2 model. The RMA-2 model was developed based on a Digital Terrain Model (DTM) derived from the digitised HEC-2 cross-sections, with roughness parameters initially adopted from the original HEC-2 model, then revised based on aerial photography and water level comparisons. More detail about the modelling approach and results are available in Reference 5.

2.5. Stonequarry Creek – 2D Modelling and Climate Change Assessment (WorleyParsons, 2011) (Reference 10)

This study was commissioned by Council in order to extend the 2006 RMA-2 flood model further upstream along Stonequarry, Racecourse and Crawfords Creek. The topographic data was based on a combination of detailed survey data and 2 metre contours provided by Council. The updated hydrologic modelling (still in RAFTS) found that a critical duration of 9 hours applied to the study area, generating the greatest discharge at the most downstream model node, longer than the 6 hour duration previously identified in the 1989 Flood Study (Reference 8), and increasing peak discharges by 15%-20% at the node furthest downstream.

In addition, an assessment of Climate Change conditions was completed based on adoption of the methods outlined in the NSW Department of Environment and Climate Change (DECC, now OEH), document entitled 'Practical Consideration of Climate Change'. A sensitivity analysis was carried out by increasing the 1% AEP rainfall intensities by 10%, 20% and 30% in the RAFTS hydrologic model, then re-running the RMA-2 hydraulic model to determine the impact on peak flood levels. The maximum increase in peak 1% AEP flood levels for a 10%, 20% and 30% increase in rainfall intensity was 0.5 m, 0.9 m and 1.3 m respectively, occurring immediately upstream of the railway viaduct. Throughout the Picton CBD, the increases were substantially less; approximately 0.2 m, 0.4 and 0.6 m respectively.

2.6. Picton / Stonequarry Creek Flood Study, Advisian, September 2017 (Reference 5)

The models developed and improved in the aforementioned previous reports formed the basis of the modelling in the Picton / Stonequarry Creek Flood Study (Reference 5), with the following primary modifications:

Hydrologic Model:

- Updated to a recent version of RAFTS (XP-RAFTS, Version 7.0, 2008);
- Updated to reflect current catchment conditions, namely an increase in the proportion of impervious areas determined based on a review of newly urbanised areas identified in recent aerial photography;
- Application of a critical duration of 9 hours (not 6 hours as in the 1989 Flood Study);
- Revision of initial and continuing loss rates for urban areas; and
- IFD parameters were reviewed and updated.

Hydraulic Model:

- The previous RMA-2 two-dimensional flood model developed in Reference 9 and updated in Reference 10 formed the basis of this Flood Study, and was updated to the latest version of RMA-2 (Version 85S);
- The DTM was updated to incorporate the LiDAR survey available to Council in 2012;
- Refinement of the existing model mesh using the LiDAR that provided improved channel definition of Stonequarry Creek and its tributaries, followed by the refinement of floodplain areas, major roadways and building footprints.

- Validation of the flood model to historic floods and comparison with the 1989 Flood Study results.

In the updated study, flood behaviour was defined for the 20%, 5%, 1%, 0.5% and 0.2% AEP design flood events and the Probable Maximum Flood (PMF). In addition, the potential impact of climate change on the 1% AEP levels was assessed. These design events were completed in 2014, prior to the June 2016 event. After the flooding Council collected High Water Mark (HWM) information for 76 locations along the creek system and across the floodplain. This data as well as recorded rainfall data from nearby rainfall and streamflow gauges was used to validate the newly developed XP-RAFTS and RMA-2 models relied upon by the Flood Study. The data collection and validation methodology was reported in the Picton Post Flood Event Analysis (Reference 11), described in Section 2.7.

The results indicate that at the peak of the 1% AEP flood, the majority of overbank inundation occurs across undeveloped areas upstream of the Picton town centre and through the town centre itself. Further downstream, significant inundation occurs at Victoria Park, upstream of the railway viaduct. In the 1% AEP event it was found that peak velocities through the town centre (between Argyle Street and Elizabeth Street) typically ranged from 0.4 m/s to 0.8 m/s, while on Argyle Street itself flows are ‘channelled’ between buildings, reaching velocities of up to 1.5 m/s and becoming highly hazardous. This is consistent with the findings of the 1989 Flood Study.

Table 7 in Reference 5 provides a comparison of peak discharges from the 1989 flood study with the results of the updated XP-RAFTS model. At the downstream extent of the study area, the Flood Study resulted in a peak 1% AEP flow of 574 m^3/s, compared to 494 m^3/s previously estimated in the 1989 Flood Study (Reference 8).

It is noted also that the flood model developed in this Flood Study did not consider overland flow generated locally, that flows through the urban areas of Picton towards Stonequarry Creek. In the June 2016 event, local overland flow due to stormwater runoff was noted to significantly affect businesses and residences in the town centre *prior* to Stonequarry Creek breaking its banks (referred to as ‘mainstream flooding’). For this reason, this current report has incorporated overland flow into the flood modelling. This is discussed in detail in Section 6.2.

2.7. Picton Post Event Analysis, June 2016 Weather and Flood Event, Advisian, November 2016 (Reference 11)

Following the June 2016 flood event, Council collected High Water Mark (HWM) (as depth) information for 76 locations throughout the floodplain. These anecdotal or visual records of the peak flood depth are useful for calibrating and validating flood models. Council engaged Advisian to use the collected HWM information to validate the existing two dimensional RMA-2 model (most recently updated as per Reference 10, described in Section 2.5 above), and to comment on how the magnitude of the 2016 event compared to the 1% AEP event.

The model was validated by applying real rainfall data from the event to the XP-RAFTS hydrologic model, then running the model to produce the inflow hydrographs required for the RMA-2 hydraulic model. Initially, the XP-RAFTS model was used without adjusting any of the parameters, and was shown to predict flows within 20 m^3/s of the peak discharge determined from the gauged level and rating curve. However, the produced hydrograph did not align with the rising limb of the flood as per the then NSW Office of Water record. The initial and continuing loss rates were subsequently varied in the XP-RAFTS model to try to achieve a better 'fit' to the gauged data. The final values adopted were 35 mm and 2.2 mm/hr for initial and continuing loss respectively. The revised losses provided a much closer match to the peak flow rate recorded at the gauge (near the Railway Viaduct), with a modelled peak discharge of 578 m^3/s compared to the recorded peak flow of 575 m^3/s (as reported in 11), however still did not match the shape of the recorded rising limb. It was suggested that initial rainfall losses of 80 to 100 mm would need to be applied to achieve a good fit. The RMA-2 hydraulic model produced peak flood levels for the June 2016 event that were on average 0.18 m lower than all 76 High Water Marks. This exercise was considered to provide an acceptable agreement between flood levels simulated using RMA-2 to the recorded HWM levels, and the model was considered to be validated.

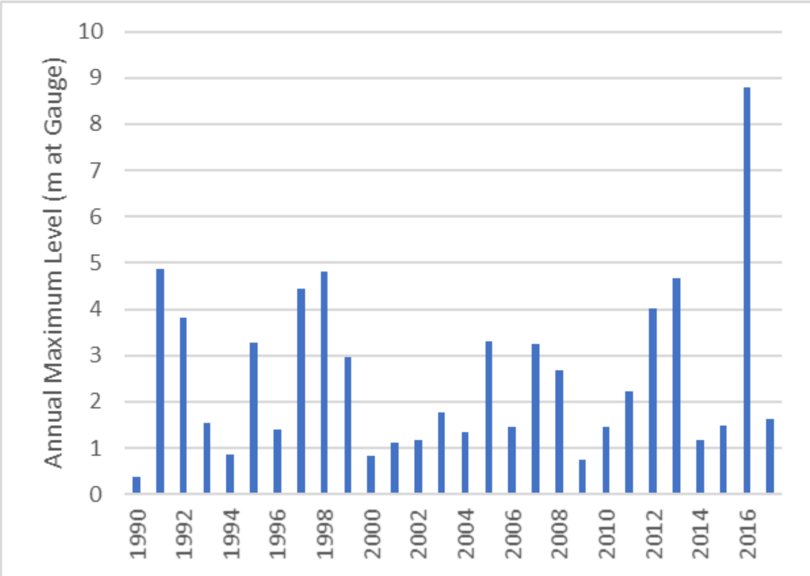
The analysis also noted that the modelled peak flood levels in the simulated June 2016 event are between 0.02 m to 0.22 m higher than those predicted for the 1% AEP design event, and that the recorded rainfall exceeded the amount predicted for a 1% AEP event.

3. FLOOD ENVIRONMENT

3.1. History of Flooding in Picton

Picton has a long history of flooding due to its location within the Stonequarry Creek floodplain, though formal gauging has only occurred since 1990 when the Stonequarry Creek at Picton gauge (no. 212053) was commissioned. In this period, the June 2016 event is by far the highest on record, as can be seen in Chart 1 below.

Chart 1: Annual Maximum Levels - Stonequarry Creek at Picton (Gauge No. 212053)



In order to gain a better understanding of floods that occurred prior to 1991, WMAwater has considered and researched the following:

- Historic floods described in previous reports (Draft Flood Study (Advisian, 2017), Picton Flood Study Report (NSW Department of Water Resources, 1989);
- Newspaper articles from the Picton Post (1855 to 1969), sourced through the National Library of Australia archives via *Trove* or previous reports;
- Long term rainfall record at the Picton Council Depot (dating back to 1880) and other nearby gauges;
- Anecdotal reports from members of the community and the Floodplain Management Committee referencing specific flood events in living memory.

The research revealed at least eight moderate to significant flood events that have occurred in Picton since 1911. There is also evidence of flooding prior to this date (e.g in 1860, in which a flood was reported to have washed the Stonequarry Bridge away), however it is more difficult to estimate their relative magnitude as they occurred prior to the commissioning of the rainfall gauge at the Picton Council Depot (1880). A brief summary of the flood events is provided in Table 3, noting that the research is limited by the availability of newspaper articles on *Trove* and level detail provided specifically on consequences of flooding in Picton, especially if other regions were more severely affected. Nevertheless, the investigation has provided insight into the flood history within the Study Area, which has been used in the estimation of design flood discharges, described in Section 4.

Table 3: Summary of some significant flood events in Picton

Year	24 Hour Rain (mm)	2 Day Total (mm)	Summary of consequences	Source
1911	160.3	160.3	*Creek has risen over the town bridge.* *Several residents evacuated their houses to seek higher shelter. 871 points (307.3 mm) of rainfall recorded at Picton.* *Flooded creeks and waterholes in a very short time.* *7 days rain, 1125 points* *Stonequarry Creek Bridge - halfway up handrailing of the bridge. Ald. Grahams Residence flooded over floor, two feet high. T. Moraghan's drapery shop (Argyle Street) 1 ft deep. Several houses in low lying areas flooded. Mrs. Murray in Menangle Street West, Mr. J. York in Argyle Street removed from houses.* *Mr. J. Corbett's Blacksmith shop - as high as his bellows.* *Messrs. G. Barr & Sons Store - Cellar filled with water.* *Portion of Menangle street under water, as far as the kitchen of Mrs. Reeve's residence.* *Mr. J. Jessup's house completely surrounded by water.* *Damage to fencing, gardens, roads and footpaths.* *Water 4ft over Windsor Bridge.*	The Bathurst Times, *'The Big Storm',* 14 January 1911. The Picton Post, *'The Rainfall',* 18 January 1911. The Sun, *'Floods at Picton',* 31 January 1911. Camden News, *'Sensational Accidents,'* 16 February 1911.
1933	211.6	211.6	833 points (293.8 mm) of rain recorded from 9am Sunday to 9am Monday. General comments about storms and damages verifying the event but no specific locations of high water marks.	The Picton Post, *'Rain Records Go',* 25 January 1933.
1943	84.1	95.5	*"Water flowed over Argyle Street for hundreds of yards. Inundated low-situated houses on Argyle Street. Water rose above the stone supports on the bridge over Stonequarry Creek, but did not cover the decking."*	The Picton Post *'Splendid Rain', 20 May 1943*
1950	204.7	204.7	General comments about flood warnings: *"Relieving Post Master at the Picton Post Office, Mr. A.Cooper, this morning was notified of expected floods and gales in Southern and South Eastern districts, with rises on all rivers."*	The Picton Post, *'Further Rain and Gales',* 19 January 1950.
1952	163.8	163.8	*6 inches of rain recorded at Picton (152 mm)* *Wide areas of rich grazing property between Camden and Picton are under water, ranging in depth from 3 feet to 25 feet.*	Camden News, *'Nepean River Again in Flood',* 31 July 1952.
1956	216.7	216.7	*"Flood is worst in history of the town"* *"Shops suffer thousands of pounds loss"* *"water two feet six inches in St Marks"*	Department of Water Resources New South Wales, *'Picton Flood Study Report',* February 1989, Section 10.2

Year	24 Hour Rain (mm)	2 Day Total (mm)	Summary of consequences	Source
1964	201.9	201.9	Widespread flooding across Sydney. 3 inches of water in St Marks Church, little damage. *"Water overflowed from Stonequarry Creek in the main street and entered several shops and adjoining homes…"*	The Canberra Times, *'Rivers Burst Banks, Dams Overflow: Widespread Floods Force Many to Flee',* 12 June 1964. Picton Post 18/6/1964
1966	245.9	245.9	High rainfall readings at Picton Council Depot and surrounding gauges, e.g. Oakdale,	BOM Daily rainfall data
1969	156	156	Reports that the flood peaked 1 m above Argyle St bridge. A range of observed flood levels are provided in the 1989 Flood Study.	Department of Water Resources New South Wales, *'Picton Flood Study Report',* February 1989, Table 4.1.
2016	266	331.5	Worst flood on record – See detailed description below.	Note: Rain from Stonequarry Ck Gauge (Pluviograph)

Some key notes and recorded or anecdotal high water marks from the above flood events are shown on Figure 4, and a selection excerpts from the Picton Post on Figure 5.

3.2. Picton Flood Event – June 2016

Early on Sunday 5th June 2016, an East Coast Low developed causing heavy rain, strong winds and large waves along the NSW coast. The low pressure system brought widespread heavy rainfall to the northern coast and ranges, before the main rainfall focus shifted southwards to impact the south coast and ranges of NSW. Rain persisted through both Saturday and Sunday and many locations reported their wettest June on record in the first week of the month. Severe coastal erosion was reported in areas including Coogee and Collaroy. In the western areas of the Sydney Basin, major flooding occurred at Picton and Camden, with over 330 mm of rainfall observed during the event.

The gauge at Stonequarry Creek recorded a peak water level of 8.799 m (156.6 mAHD). The flooding caused damage to commercial and residential properties. Properties throughout the study area, including many along Argyle Street in the centre of town, experienced significant inundation with depths in excess of 1.5 metres recorded. A large number of trees and other in-bank vegetation were up-rooted during the flood event and conveyed downstream; a reflection of the significant volume and velocity of floodwaters along Stonequarry Creek and its tributaries. Following the event Council collected High Water Marks at 76 locations throughout the floodplain, which have since been used to calibrate (and validate) hydraulic models. A selection of photos from the 2016 flood are shown on Figure 6.

3.3. Picton Flood Event – April 1969

Until the recent 2016 flood, the 1956 and 1969 floods were the largest floods on record at Picton. The Flood Study (Reference 5) notes the April 1969 flood is reported to have been the largest. Peaking at approximately 1 m above the deck of the Argyle Street Bridge (no equivalent gauge level recorded), the 1989 DWR Flood Study (Reference 8) determined that the flood was in the order of the 2% AEP flood event. For context, the 1969 flood reached 157.56 mAHD at the Westpac Bank, while the 2016 event was over a metre higher, reaching 158.70 mAHD at the same location (Reference 5).

4. AVAILABLE DATA

4.1. Topographic Data

Light Detection and Ranging (LiDAR) survey of the study area and its immediate surroundings was provided for the study by LPI. LiDAR is aerial survey data that provides a detailed topographic representation of the ground with a survey mark approximately every square metre. The data for the Picton area was collected in 2011. The accuracy of the ground information obtained from LiDAR survey can be adversely affected by the nature and density of vegetation, the presence of steeply varying terrain, the vicinity of buildings and/or the presence of water. The accuracy is typically ± 0.15 m for clear terrain. Topography in the immediate vicinity of the main creeks was retained from the RMA-2 model which used localised survey, and LiDAR was used in the remaining areas.

Where needed, the DEM was modified manually to represent recent development in the floodplain. In particular, parts of the Vault Hill Development had been constructed after the LiDAR was collected. Works as Executed Drawings of the North OSD basin were provided by Council (dated 31/1/28 and 12/4/18), and used to ensure details of the basin were appropriately represented. In addition, details of the roads, retaining walls and other features were taken from design drawings dated 26/8/16 (12122E4-SET F – Amended plans for Vault Hill, John M. Daly & Associates).

Towards the end of the project a revised LiDAR survey became available (captured 29/6/2019). Following public exhibition the DEM was subsequently revised in recent development areas where comprehensive details were not previously available, including the development at Jarvisfield. Mapping presented in this report utilises this updated DEM.

The data extent is shown on Figure 2. The model adopts a 2 m x 2 m grid resolution which is locally refined to show sub-grid elements such as kerbs and gutters (described in Section 7.5.5.4). A 4 m x 4 m grid was adopted for the PMF event to prevent model instability due to high velocities in some areas.

4.2. Hydraulic Structures

A site inspection was undertaken in April 2018 to identify and measure key hydraulic structures, including culverts, bridges, and elements of the pit and pipe network. For larger bridges, measurements were estimated from photographs, LiDAR data and Works As Executed (WAE) drawings provided by Council where available. Information on culvert inverts and dimensions were taken from WAE and stormwater plans where available. The locations of bridges are shown on Figure 13.

4.3. Pit and Pipe Network

A database of stormwater pits and pipes within the catchment was provided by Council. Where needed, additional details were gathered via visual inspection or assuming pipe diameters based on location and estimating pipe invert levels based on LiDAR data and reasonable pipe cover depths. Pit inverts were assumed to be 1-1.5 m below the ground level (from LiDAR), and were manually adjusted where needed to ensure no negative grades were assigned to pipes.

4.4. Design Rainfall

Design rainfall information for use with ARR 1987 methodologies was adopted directly from Reference 5. New Intensity Frequency Duration (IFD) for the Study Area was obtained from the Bureau of Meteorology (BoM) website for the purpose of the ARR 2016 Sensitivity Assessment.

4.5. Floor Level Database

A floor level survey was commissioned by Council for properties estimated to be inundated in the 1% AEP event, and was undertaken by LandTeam Australia Pty Ltd in 2012. The survey included 251 properties in Picton, collecting (where available) details such as the Lot and Section number, street address, building description (construction type, number of stories), lowest property level and if applicable, lowest habitable floor level. The following were identified:

- 214 ground floor levels were surveyed
- 168 of these were identified as 'habitable' floor levels;
- 32 spot heights were collected;
- 46 vacant lots were identified

This data set was supplemented by estimating floor levels of 902 (885 residential properties and 17 commercial) additional properties based on visual inspection to ensure all properties within the PMF extent were included in the database. For each property, the following details were recorded:

- Estimated floor height (m);
- Ground Level (m AHD);
- Street Address;
- Indication of house size (number of storeys);
- Location of the front entrance to the property; and
- Land Use (residential or commercial) based on information from the Wollondilly Local Environmental Plan (LEP) 2011.

The data was gathered in two stages. Stage 1 estimated properties within the preliminary PMF extent, excluding dwellings in the development zone north of Jarvisfield Road. The extension of the TUFLOW hydraulic model (discussed in Section 7.5.2) introduced Stage 2, which is a continuation of the estimation including the developing residential properties north of Jarvisfield Road and Stargard Crescent, additional dwellings at Margaret St next to the central business district, and additional properties in the southern parts of the hydraulic model.

5. FLOOD FREQUENCY ANALYSIS

A Flood Frequency Analysis (FFA) has been undertaken to improve and provide confidence in the estimates of design flood behaviour of Stonequarry Creek at Picton. FFA enables the magnitude of floods (5%, 1% AEP etc.) to be estimated based on statistical analysis of recorded floods. It can be undertaken graphically or using a probability distribution, and is advantageous as it does not require assumptions regarding the relationship between rainfall and runoff – all factors affecting flood magnitude are already integrated into the data. However, the reliability of the flood frequency approach depends largely upon the length and quality of the observed record and accuracy of the rating curve.

The FFA is made up of two stages: The first stage involves establishing a flood record using gauged data and information about events that occurred prior to the gauged record. The second stage involves fitting different probability distributions to the data, and using the resulting curve to determine the peak design flows. This section of the report describes the data used for this investigation, outlines the methodology undertaken and sets out the results that will be used in the hydrologic and hydraulic model development in the subsequent chapter. The analysis produces revised estimates for design peak flows (e.g. 1% AEP and 5% AEP events), and provides confirmation of the magnitude (and rarity) of the June 2016 event.

5.1. Rating Curve

The Stonequarry Creek at Picton Gauge continuously records water level data only, which is then converted into a flow rate using a rating curve. Rating curves define a relationship of height to flow at the gauge location. The relationship is defined by a series of instantaneous flow measurements at known heights (called ‘gaugings’). The Stonequarry Creek at Picton Gauge (212053) is located approximately 30 m upstream of the Railway Viaduct, and is managed by WaterNSW, formerly NSW Office of Water. The most recent rating curve produced by this agency was published on the 15/12/2015.

An investigation of the gauging record found that the highest gauging is 2.260 m above gauge datum, recorded on the 10th February 1992. This is approximately the level of a 25% AEP event (or a 4 year ARI). Above this level the rating curve has been extended using an extrapolation technique. The further the flow estimates are above this level the more unreliable they can become. This is particularly problematic when the rating curve is extended from in-bank to overbank flow, as the hydraulic behaviour and resistance to flow tends to change dramatically.

The TUFLOW hydraulic model developed for this project (see Section 7.5) is able to replicate the change in behaviour between in-bank and overbank flow and can be used to derive a new rating curve. While the WaterNSW rating curve is reliable in the lower flow estimates (where it is defined by the recorded gaugings), the model derived rating curve provides a more reliable estimate at higher flows. The model-derived curve was obtained by modelling floods of varying magnitude and obtaining the flow and peak level at the location of the gauge.

For the work subsequently documented in this report, a combination rating curve has been developed using the WaterNSW rating curve for river levels of up to 5.5 m, and the TUFLOW derived rating curve for all height – flow conversions above this stage height, i.e. the June 2016 event. A plot of the resulting combination rating curve is shown on Figure 7.

5.2. Annual Maximum Series

ARR (References 1 and 14) recommends that FFA should be applied to peak flows or discharges. In frequency analysis of flows, the fitting of a particular distribution may be carried out analytically, by fitting a probability distribution. The data may consist of an annual series, where the largest peak in each year is used, or a partial series, where all flows above a selected base value are used. The relative merits of each method are discussed in detail in AR&R. In general, an annual series approach is preferable as there are more methods and experience available, in addition using a series of annual maximums lowers the risk of two successive peaks being dependent. Observing the time series of monthly maximums showed that no year contained more than one major flood event, ensuring the annual series was not filtering out significant events. Validation against rainfall data confirmed no years were missing from the record. An annual data set was used for this study.

Water level data at the Stonequarry Creek at Picton Gauge (212053) was obtained from WaterNSW from 1990 to 2017. The peak annual heights were extracted from the data and converted to flow rates using the combined WaterNSW/TUFLOW rating curve described in Section 5.1. Table 4 contains the Annual Maximum Series used in the FFA.

Table 4: Annual Maximum Series

Date	Recorded Level (m)	Flow (m^3/s) Derived from combined rating curve
11/06/91	4.882	119.2
09/02/92	3.807	85.0
24/11/93	1.542	13.8
12/02/94	0.867	2.3
25/09/95	3.270	68.0
06/05/96	1.406	10.4
02/03/97	4.433	104.9
08/08/98	4.815	117.0
24/10/99	2.953	57.9
18/11/00	0.835	2.0
11/03/01	1.110	4.6
29/03/02	1.184	5.8
16/05/03	1.767	20.7
22/10/04	1.340	9.0
29/11/05	3.307	69.2
17/01/06	1.450	11.3
16/06/07	3.249	67.3
05/02/08	2.690	49.6
28/12/09	0.752	1.3
01/12/10	1.445	11.2
26/11/11	2.214	34.5
18/04/12	4.008	91.4
24/02/13	4.662	112.2
17/08/14	1.182	5.8
22/04/15	1.496	12.4
05/06/16	8.799	580.1
22/03/17	1.631	17.3

5.3. Extending the Flood Record

The Stonequarry Creek Gauge at Picton has been recording rainfall and water level data since December 1990, providing a record of less than 30 years. This is considered to be a short record. Short flood records, while giving a good indication of the behaviour of frequent flood events (say, up to the 5% AEP), typically provide less certainty in the estimation of larger events, as they do not capture the climatic variations that can occur across longer periods. Furthermore, the extreme flood event of June 2016 significantly skews the range of typical peak flows that appear to occur within a 30 year period.

For this reason, WMAwater has investigated the history of flooding in Picton prior to 1990 with a view to effectively supplement or “extend” the flood record, and in doing so, gain a better understanding of the magnitude of the 1% AEP event to be used moving forward in the Floodplain Risk Management Study. The investigation and its findings are detailed in Section 3.1.

As set out in Table 3, 8 notable flood events were identified either via newspaper reports or rainfall records (or both). The earliest significant event we can identify with confidence occurred in 1911, where there is a good correlation of recorded high rainfall as well as reported consequences for Picton. Selecting this year as a starting point provides an additional 79 years of data up to 1990, and extends the total record length to 107 years, which is considered appropriate for the purposes of estimating the 1% AEP design flow.

Significant flooding also occurred prior to 1911 (for example in 1860, in which 3 flood events were reported, one of which washing the Stonequarry Creek Bridge away (later rebuilt in the 1890s)), as well as 1863, and 1880. However, the magnitude of these earlier events cannot be estimated with the same degree of confidence as there is less anecdotal evidence of the consequences of the floods, and they occurred prior to the commissioning of the rain gauge at the Picton Council Depot in 1880.

5.4. Fitting the Probability Distribution

A Flood Frequency Analysis has been undertaken to fit a probability distribution to the extended flood record described above. Recent research has suggested that the fitting method is as important as the adopted distribution. The traditional fitting method has generally been based on moments and this makes the fit very sensitive to the highest and lowest values. Recent research has shown that L-moment and Bayesian likelihood approaches are much more robust than traditional moment fitting and are now the recommended methods.

For this analysis a Bayesian maximum likelihood approach has been adopted. The FLIKE FFA software developed by Kuczera (Reference 15) uses the Baeysian approach and was utilised in this study. Two probability distributions were tested, Log Pearson III (LP3), which is commonly used in FFA; and the Generalised Extreme Value (GEV) distribution, which is a more recently developed family of distributions that combine Gumbel, Frechet and Weibull families of distributions. The LP3 distribution produced the best fit and has been adopted for this analysis.

The Bayesian method allows for the inclusion of flood events outside of the gauged records, referred to as 'censored events'. WMAwater tested a number of scenarios to understand the effect of the additional historic data and to identify the scenario that produced the best fit to the available Annual Maximum Series data.

The accuracy of the FFA can be improved, substantially in some cases, by augmenting at-site information with regional information. The use of an informative prior based on regional analysis is strongly recommended in all Flood Frequency Analyses involving at-site data. Even with long at-site records, the shape parameter in the LP III and GEV distribution is subject to considerable uncertainty. Regional priors can substantially reduce the uncertainty in the shape (and even scale) parameter. To improve the accuracy of the Picton FFA, prior information was obtained from a similar sized adjacent catchment on the Nepean River and incorporated into the FFA using Flike (described above) and in accordance with guidance provided in ARR 2016 Book 3, Chapter 3 (Reference 2).

Following testing of a range of scenarios, it was found that adding 79 "events" (i.e. one flood event per additional year of record), each with a flow of below 450 m^3/s, and inclusion of the prior regional information was best approximated by the probability distribution and improved confidence limits. The 450 m^3/s threshold was selected following testing of a range of thresholds (between 300 m^3/s and 500 m^3/s), and the available research gives confidence that floods that have occurred prior to the gauge record would have been below (if not well under) this magnitude.

5.5. Results

5.5.1. Design Flow Estimates

The results of the FFA are provided in Table 5, and the resulting LP3 fit is presented on Figure 8.

Table 5: Flood Frequency Analysis Results – Stonequarry Creek at Picton Gauge

AEP	Peak Flow (m^3/s)		
	FFA	2017 Flood Study (Ref 5)	1989 Flood Study (Ref 8)
50%	23.4	Not Documented	Not Documented
20%	68	Not Documented	Not Documented
10%	121	Not Documented	Not Documented
5%	193	431	345
2%	331	509	424
1%	474	578	494

5.5.2. Magnitude of June 2016 Flood Event

In addition to design flow estimates, the FFA can also be used to estimate the magnitude of actual flood events. The analysis shows that the June 2016 flood event, which peaked at 8.799 m (with a flow rate of 580.1 m^3/s determined using the combined rating curve), has an annual recurrence interval of 157 years, approximately equivalent to a 0.6% AEP event. However, as the June 2016 event is the largest event on record, there is a high degree of uncertainty when estimating its magnitude. In this case, additional insight can be gained from other metrics including rainfall records and historic flood reports. Considering these sources, it is possible that the peak flow observed in the June 2016 event is even rarer than the FFA suggests, and could have a recurrence interval anywhere between 200 and 500 years.

5.6. Discussion

The results of the FFA differ significantly from design flow rates previously presented in the Flood Study (Reference 5) and 1989 Flood Study (Reference 8). There are a number of reasons the results would be expected to differ, especially the fact that Reference 5 did not use FFA, and based its peak flow estimates on the selection of parameters informed by limited calibration data (a limited number of recorded high water marks from the 1969 event). The Flood Study subsequently undertook a validation exercise using high water marks surveyed by Council. The RMA-2 hydraulic model produced peak flood levels for the June 2016 event that were on average 0.18 m lower than all 76 High Water Marks. At the time, this was considered to demonstrate an acceptable level of agreement between the model results and surveyed depths, however the average difference of 0.18 m and results being consistently below flood marks across the floodplain, and the results of the current FFA indicates that the original selection of parameters may not have been appropriate. A review of the selected parameters at the commencement of this FRMS also showed that each parameter would incrementally increase runoff, resulting in an overestimation of the design flows.

It is noted also that if the 79 censored events were excluded from the FFA, and only the 30 years of gauged data are used, the design flow estimates would increase significantly, producing a 5% AEP estimate of 326 m^3/s and 1% AEP of 945 m^3/s. These results are considered unreasonable, and are a function of the extreme June 2016 event skewing the results, as the analysis software effectively assumes an event of that size could statistically occur once every 30 years. However, based on the available historic data dating back to the 1860s, it was assuredly a much rarer flood event.

Given the gauge location immediately upstream of the Railway Viaduct, there has been discussion and speculation about the effect of blockage on the gauged water level during the June 2016 event. It is possible that during the flood, the water level in the creek was raised temporarily behind an obstruction, causing the gauge to record a higher flood level, although the recorded hydrograph does not explicitly indicate the effects of blockage. While we cannot verify if this occurred, or the extent to which the water level may have been influenced by blockage, we can assess how the gauge reading affects the design flow estimates.

To test the sensitivity of the design flood flow results to the level of the June 2016 event, the FFA was re-assessed and the 2016 event was assigned a flow rate based on peak flood levels 0.5 m and 1.0 m lower (corresponding to flow rates of ~525 m^3/s and 450 m^3/s respectively). The results indicated that the 5% AEP design flow estimate was not sensitive to the magnitude of the 2016 event, as in each test, the 5% AEP flow estimate fell by less than 0.8%. Similarly, the 1% AEP event estimate fell by only 2% when the 2016 level was lowered by 1.0 m. These results improve confidence in the 5% AEP and 1% AEP design flow estimates.

To conclude, the FFA allows for the estimation of design flood flows with a reduced number of assumptions regarding the relationship between rainfall and runoff. This relationship is typically modelled using a combination of parameters, which provides ample opportunity for the accumulation of inaccuracies and errors, especially when there is limited calibration data with which to inform the selection of parameters. While the short gauge record is not ideal for FFA, the addition of censored historic events to effectively extend the record, and use of regional prior information significantly improve the confidence limits of the resulting fit. The results of the FFA as presented in the second column of Table 5 are therefore considered appropriate for the purpose of the development of the hydrologic and hydraulic models described in the subsequent section.

6. FLOOD MODEL REVIEW

6.1. Overview of Model Review

The Draft Picton/ Stonequarry Creek Flood Study (Reference 5), completed in September 2017, was carried out by Advisian for the Wollondilly Shire Council (Council) in accordance with the NSW Government's Flood Prone Land Policy. The Flood Study aimed to determine design flood behaviour in the area and used an XP-RAFTS hydrologic model and an RMA-2 hydraulic model. The models were reviewed by WMAwater to determine the suitability for use in the Floodplain Risk Management Study.

The Flood Study was developed using Australian Rainfall and Runoff (ARR) 1987 Guidelines. A new version of the ARR Guidelines were released in 2016 following numerous technological developments, availability of a significantly larger dataset, and development of updated methodologies. As part of the current Picton Flood Risk Management Study and Plan (FRMS&P), sensitivity of the design event modelling to the use of the ARR 2016 methodologies has been assessed, and subsequently, the ARR 2016 methodologies have been applied to the design flood estimation process. This process is described in detail in Section 9.2.

6.2. Hydrologic Model Review

In order to simulate flooding in a catchment, the amount of runoff that is conveyed in the creeks needs to be determined based on the rainfall over the catchment, and the catchment's response to that rainfall. A rainfall-runoff hydrologic model (which converts rainfall to runoff) is generally used to determine how a rainfall event translates into flow in creeks and streams. A review of the hydrologic model developed for the Flood Study (Reference 5) is described below.

6.2.1. Hydrologic Model Package: XP-RAFTS

The hydrologic model utilised in the Flood Study (Reference 5) was based on the previous RAFTS hydrologic modelling developed for the Picton Flood Study (1989, Reference 8). This model covered the entire Stonequarry Creek catchment to the Main Southern Railway Viaduct crossing (downstream of Picton), delineated into 30 sub-catchments and covering a total catchment area of 84 km^2. It is understood the sub-catchments adopted for this study were originally defined for the 1989 Flood Study (Reference 8). The model was not calibrated to recorded data due to the absence of any stream flow data.

The Flood Study updated the RAFTS model to Version 7.00 (2008) XP-RAFTS and made a range of revisions to reflect urbanisation and the changes to impervious areas since 1989. The critical duration was determined to be 9 hours (previously determined to be 6 hours in the 1989 Flood Study) following review of various storm durations.

6.2.2. Modelling Methodology: Australian Rainfall and Runoff

The estimation of design flood behaviour in the Flood Study (Reference 5) was completed using methodologies and inputs from ARR1987. A new edition of 'Australian Rainfall and Runoff' was released in December 2016, and revised Intensity, Frequency, Duration (IFD) data was made available at this time by the Bureau of Meteorology. Given the Flood Study was near completion, no consideration of the new ARR 2016 methodologies were made. The Flood Study noted however that ARR2016 data and methodologies would need to be considered in future studies.

Due to the differences in ARR 1987 (the method used for the Flood Study (Reference 5)) and ARR 2016 methodologies, there may be significant variation in the results produced by each, and these variations in turn may indicate that current design flood levels in Picton are under/overestimated. As part of the current Picton Flood Risk Management Study and Plan (FRMS&P), sensitivity of the 1% AEP and 5% AEP design event modelling to the use of the ARR 2016 methodologies was undertaken and a comparison made to the results produced using the updated models with the ARR 1987 process.

The sensitivity assessment (which also involved a review of the hydrologic model inputs, detailed in Section 6.2.3 to 6.2.5) was presented to Council via a memo in April 2018. The assessment found that the ARR 2016 design rainfall depths are lower, losses are higher, and areal reduction factors were applied. In addition, the ARR 1987 temporal pattern (1% AEP, 9 hour) has a more prominent internal burst where the most intense rainfall increments are clustered together, while the selected ARR 2016 temporal pattern (1% AEP, 12 hour) has a less intense burst, with a consistent rainfall distribution across the event, which is more representative of recorded rainfall events in Picton. The ARR 1987 temporal pattern tends to produce a peakier hydrography (and higher peak flows) as a result.

Consequently, the peak flows and volumes (of the whole storm) across several key locations produced using ARR 2016 methodologies were significantly lower than those produced using the ARR 1987 methodology. It was concluded that the models would be updated to use ARR 2016 methodologies across the full suite of design events (see Section 9).

6.2.3. Design Rainfall

Intensity-Frequency-Duration (IFD) parameters were obtained from the Bureau of Meteorology (BoM) based on ARR 1987. The Flood Study determined the 9-hour duration was critical for the 1% AEP event. From a review of the available ARR 1987 IFD data, the depth for the 1% AEP, 9 hour duration varied between a minimum of 156.3 mm to 165.5 mm, with a catchment average of 160.7 mm. The Flood Study however applied a uniform depth of 163.3 mm across the catchment in the XP-RAFTS model. The spatial variation that would naturally occur across the catchment has therefore not been represented, and furthermore, the depth applied is higher than the catchment average, which is likely to contribute to producing a greater amount of runoff, resulting in elevated peak flood levels.

6.2.4. Losses

Rainfall losses are generally categorised as initial and continuing. The initial loss represents the wetting of the catchment prior to runoff starting to occur and the filling of localised depressions, and the continuing loss represents the ongoing infiltration of water into the saturated soils while rainfall continues.

The Flood Study (Reference 5) applied an initial loss of 15 mm and continuing loss of 1.5 mm/hr for design flood estimation. The model adopted separate infiltration losses for urban areas, with initial loss of 2.5 mm and continuing losses of 0.5 mm/hr. When validating the model to the June 2016 event, the original Flood Study losses were applied, and revised to 35 mm (initial loss) and 2.2 mm/hr (continuing loss) to achieve a better fit to the available recorded data. The estimation of design flood events however was not revisited using these calibrated losses.

ARR 1987 Book Two – Design Rainfall Considerations (Reference 1) recommends initial loss values of 10 mm to 35 mm (varying with catchment size and mean annual rainfall) and a continuing loss of 2.5 mm/hr. The values selected in the Flood Study are at the lower end of the recommended range. Whilst the selected values are not unreasonable, they may be considered conservative when defining design flood levels and extents. This is due to the amount of rainfall lost to the ground via infiltration being underestimated, resulting in a greater amount of runoff being generated, which leads to higher modelled peak flood levels.

6.2.5. Areal Reduction Factors

Design rainfall information for flood estimation, in the form of IFD data, relates to specific points in a catchment rather than to the catchment area as a whole. An Areal Reduction Factor (ARF) provides a correction factor between the catchment rainfall depth (for a given combination of AEP and duration) and the average of the point rainfall depths across a catchment (for the same AEP/duration combination).

The Flood Study (Reference 5) did not apply an ARF to the IFD data. However, using ARR 1987 Guidelines, an ARF of 0.95 was determined for the 1% AEP event with a 9 hour critical duration is required. If this factor had been applied, the 1% AEP design rainfall would be reduced from 163.3 mm to 155.14 mm, which would lower the peak flow by ~40 m^3/s. By not applying an ARF the Flood Study has not considered the spatial variability of rainfall across the catchment, and applied the design rainfall depth uniformly to the total catchment area. This would have the effect of overestimating the amount of rainfall occurring within the catchment, resulting in conservative design flood flows.

6.3. Flood Study: Hydrologic Model Review Findings

The hydrologic model utilised in the Flood Study (Reference 5) is considered to have been appropriate. However, while individual input parameters (including design rainfall and losses) were within industry accepted ranges individually, the selection of each parameter has cumulatively acted to overestimate mainstream flood behaviour in Picton. Further to the descriptions of each parameter above, the Flood Study (Reference 5) estimated the 1% AEP flow using the rational method and found it to be 403 m^3/s, which is about 40% lower than the corresponding discharge derived from the XP-RAFTS model (572 m^3/s).

For the purposes of the Floodplain Risk Management Study, it is proposed to convert the XP-RAFTS model into a Watershed Bounded Network Model (WBNM) hydrologic model. This will allow for the following:

- Assessment of ARR 2016 methodologies and updated IFD data, as WBNM better facilitates the use of spatially varying rainfall compared to XP-RAFTS;
- Assessment of overland flow behaviour, which, following the June 2016 event, was identified as significantly contributing to flood risk in Picton; and
- Provides a cross check of the flows defined in the XP-RAFTS model.

The establishment of the WBNM model is described in detail in Section 7.1. In addition to the conversion from XP-RAFTS to WBNM, the following updates are required:

- Ensuring the proportion of each sub-catchment that is assumed impervious is reflective of the current conditions, including recent development in Picton; and
- Use results of the FFA (Section 4) and data from the June 2016 event to calibrate the hydrologic model.

6.4. Hydraulic Model Review

6.4.1. Modelling Package: RMA-2

Hydraulic modelling involves the simulation of the way in which floodwaters move through a particular terrain. The Flood Study (Reference 5) utilised RMA-2, a 2D hydraulic model, to estimate the flood levels, depths, velocities and extents across the model domain and over the duration of the flood event. The RMA-2 model was initially developed as part of the 2006 Stonequarry Creek – 2D modelling and WaterRide Application project, (Reference 9, described in Section 2.2), and covered the same extent as the 1989 HEC-2 model (originally defined by 23 cross sections).

The RMA-2 model uses an irregular mesh based on triangular and quadrilateral elements. The model solves the shallow water equations using a finite element scheme and has found wide scale adoption in coastal estuaries modelling but was rarely used in flood modelling. Some of the issues with RMA-2 include:

- the solution scheme tends to not conserve volume on a local scale during flood simulations;

- the solution scheme is not very stable under flood conditions, particularly at boundary inflow locations resulting in the overestimation of peak flood velocities, which may act to underestimate peak flood levels;
- the model stability is very dependent upon a well set out mesh which is time consuming to develop;
- the mesh often has to be modified for mitigation options, which can lead to potential stability issues;
- flow can leak under non-wet elements; and
- consultants find it to be an expensive and difficult modelling platform to use on projects.

These aspects mean that it is likely to be a costly and time-consuming exercise to model a range mitigation measures, which is a primary objective of this FRMS&P. With these issues in mind, WMAwater has converted the RMA-2 model into a 1D/2D TUFLOW model. A full description of the TUFLOW model set up is provided in Section 7.5. TUFLOW uses a finite difference numerical model for the solution of the depth averaged shallow water equations in two dimensions. The TUFLOW software has been widely used for a range of similar floodplain projects both internationally and within Australia and is capable of dynamically simulating complex overland flow regimes. With its ability to represent hydraulic structures, and the fact that the DEM can be readily modified to represent a range of flood risk mitigation options, TUFLOW is the preferred modelling package moving forward with this FRMS&P.

6.4.2. RMA-2 Hydraulic Model Extent

The RMA-2 hydraulic model boundary extended approximately 1.2 km along Racecourse Creek upstream of its confluence with Stonequarry Creek, and along Stonequarry Creek approximately 200 m upstream (west) of Fairleys Road. While this extent was appropriate for the Flood Study, a number of potential flood mitigation options have been suggested at locations beyond the current hydraulic model boundary. It is therefore necessary to extend the hydraulic model boundary for use in the FRMS to allow for various options to be assessed. This is described further in Section 7.5.2. Review of the RMA-2 upstream and downstream model boundaries is provided in Section 6.4.6.

6.4.3. RMA-2 Hydraulic Roughness

The channel and floodplain roughness parameter values were assigned to the RMA-2 model based on analysis of available aerial photography and oblique photography of Stonequarry Creek and its channel and overbank vegetation. The MHL Peer Review (Reference 23) identified areas where the roughness values could be refined to better delineate significant differences in floodplain roughness. The adopted roughness values (Manning's 'n') were typically reflective of industry accepted values, with the exception of "heavily vegetated creek channel", which at 0.060 is considered low, and "industrial development" (paved areas), which at 0.065 is considered high. These values will be reviewed and confirmed during development of the TUFLOW hydraulic model, described in Section 7.5.4.

6.4.4. RMA-2 Hydraulic Structures

Hydraulic structures were modelled by increasing the roughness parameters in the vicinity of the bridges and culverts to represent the energy and friction losses that would be caused by various structures. This approach was adopted for all bridge crossings except the railway viaduct and the Argyle Street Bridge, in which piers were individually blocked out of the model domain in combination with increase in roughness. For small structures, where the performance of the structure and associated losses do not vary significantly under a range of different flows, this approach may be appropriate if details of the structure are not known.

For larger structures (such as the Argyle Street Bridge), however, this methodology is not considered appropriate. The structure will perform quite differently under different flow conditions (including depths and velocities) that will not be represented by a single Mannings ‘n’ roughness value. The losses at a bridge structure will be quite different when the water level is below the deck, at the deck and above the deck level. A culvert will have similar issues in trying to replicate flows through the culvert and overtopping the road or embankment with just a single Mannings ‘n’ roughness value. Hydraulic effects associated with the contraction and expansion of flow may also not be accounted for. Furthermore, there is no standardised approach for representing structures in this way, so it is difficult to verify or check that the selected parameters are reasonable.

The shortcomings of the above approach further support moving to a 1D/2D TUFLOW model, which would allow smaller culverts to be modelled as 1D elements, while larger bridges could be modelled within the 2D domain as a layered flow constriction. The 1D culverts will more accurately represent the flow and velocity through the culvert based on upstream and downstream water levels, as well as flow overtopping the road or embankment in the 2D domain. 2D layered flow constriction elements will allow the different layers of the bridge (below deck, deck, railings and above railings) to be modelled so that the losses associated with different flow conditions can be more accurately simulated. This is described in detail in Section 7.5.5, along with other updates to the representation of hydraulic structures, including the addition of the local pit and pipe network which is integral to the modelling of overland flow, especially in frequent events.

6.4.5. RMA-2 Representation of Buildings

Buildings were nulled out of the model domain using polygons defined by AAM Pty Ltd in 2012 in conjunction with a review of aerial photography. This approach is deemed suitable, as it allows building to be *‘blocked out’* of the model to simulate the significant obstructions they impose on floodwaters. This approach would be applied in the TUFLOW model set-up, as described in Section 7.5.5.5. It is noted that the number of buildings has increased since the establishment of the RMA-2 model, and will require revision to include recent development in Picton, for example, Jarvisfield Estate to the north of the town centre. A review of the existing building polygons also found that some building extents had been defined by roof extents (including overhanging eaves) rather than the actual building footprint. Where necessary, the building extents will be adjusted to ensure flow paths between buildings are represented appropriately.

6.4.6. RMA-2 Boundary Conditions

The upstream boundary conditions for the RMA-2 model were derived from the discharge hydrographs generated from the XP-RAFTS modelling of the upstream catchment. This is considered appropriate and is the accepted practice when linking a hydrologic model to a hydraulic model. This approach will be applied using the outputs from the WBNM model described in Section 7.1, and the location of inflows will be adjusted to account for the extended hydraulic model boundary described in Section 7.5.2. It is noted also that inflows from the XP-RAFTS subcatchments 1.07, 1.08, 1.09 and 1.10 (located within Picton along Stonequarry Creek) were not input into the RMA-2 model. These subcatchments account for approximately 4.2% of the catchment area, and represent the runoff within Picton itself. Exclusion of these subcatchments reduces the inflow into the model, thereby underestimating peak flood levels and extent.

The downstream boundary condition was defined by a time-varying water level, also known as a stage-discharge relationship. This was preferred over a static tailwater (at 154.85 mAHD, as applied in the 1989 HEC-2 model, Reference 8) which was considered overly conservative as it produced higher flood levels as far upstream as the Stonequarry Bridge crossing (at Argyle Street). The stage-discharge relationship is based on the WaterNSW rating curve. However, the rating curve is specific to the gauge location, whereas it was applied 700 m downstream of the gauge location. This is not considered appropriate, as the geometry of the Stonequarry Creek channel varies considerably, meaning that for the same water level a different corresponding flow rate would be expected. Furthermore, there is little faith in the rating curve at higher levels, as the highest gauging was only 2.2 m above gauge datum. An alternative downstream boundary location and tailwater condition is proposed for the new TUFLOW model, and is described in Section 7.5.6.

6.4.7. RMA-2 Calibration of Hydraulic Model

Due to the limited availability of historic flood level, stream flow and/or rainfall data at the time, the RMA-2 model was not calibrated to any historic floods. The Flood Study notes that the modelled 1% AEP flood levels were compared with those determined in the 1989 HEC-2 results (Reference 8). The comparison between the two sets of results showed that flood levels predicted by the RMA-2 model are on average higher than those predicted in 1989, however notes that this is due to use of a different DEM, and points to the fact that peak flood discharges produced in the XP-RAFTS model are 20-30% higher than flows in the original RAFTS model.

Late in the study, a significant event occurred and the opportunity was taken to validate the models using data collected during the June 2016 Flood Event (described in Section 3.2). The results indicate that modelled flood levels were on average 180 mm lower in most locations across the study area, when compared to the High Water Marks collected by Council. This suggests that the flood model may be underestimating flood levels across the floodplain. Had a calibration been undertaken using the 2016 flood event, model parameters may have been able to be adjusted to obtain a closer match than in the validation. Given the nature of the validation results (systematically lower by approximately 0.2 m), it would have been desirable to adjust the model parameters to have confidence in the model's ability to replicate actual flood behaviour.

A calibration to the 2016 event as well as the FFA results (Section 4) will be undertaken as part of the development of the new hydrologic (WBNM) and hydraulic (TUFLOW) models for this FRMS&P to ensure that the modelled flood behaviour is consistent with actual flood behaviour in Picton. The calibration process is described further in Section 8.

7. FLOOD MODEL UPDATE

7.1. Selection of Hydrologic Model: WBNM

While the XP-RAFTS hydrologic model developed in the Flood Study (Reference 5) was suitable for mainstream flood estimation, the sub-catchments were considered too coarse to appropriately represent overland flow. Furthermore, with the need to assess the sensitivity of modelled flood behaviour to the methodology prescribed in Australian Rainfall and Runoff 2016 (ARR 2016) (Reference 2), it is preferable to utilise a hydrologic model that supports spatially varying rainfall inputs.

A range of runoff routing hydrologic models is described in ARR 2016 (Reference 2). These models allow the rainfall to vary in both space and time over the catchment and will calculate the runoff generated by each sub-catchment. The generated flow hydrographs then serve as inputs at the boundaries of the hydraulic model.

The Watershed Bounded Network Model (WBNM) hydrologic runoff-routing model was selected to determine flows from each sub-catchment. The WBNM model has a relatively simple but well supported method, where the routing behaviour of the catchment is primarily assumed to be correlated with the catchment area. The biggest advantage of using WBNM is that rainfall and losses can be applied spatially, and testing of temporal patterns, losses, and design rainfall is easily interchangeable for more efficient review. WBNM Version 2017_000A was used for this assessment.

The results of the flood frequency analysis (Section 4) were used to validate results from the hydrologic models. WBNM parameters (such as losses and stream routing factors) were adjusted where appropriate to reconcile the WBNM flows against the results of the flood frequency analysis and observed flood levels in the June 2016 event.

7.2. WBNM Sub-catchment Delineation

In total, the catchment represented by WBNM covers 84.61 km^2, consisting of 166 sub-catchments (compared to 31 in the XP-RAFTS model for the same area). The sub-catchments were derived from LiDAR topographic data and delineated with consideration of hydraulic controls such as bridge crossings and road/rail embankments. The sub-catchment delineation is shown on Figure 11.

7.3. WBNM Impervious Surface Area

Runoff from connected impervious surfaces such as roads, gutters, roofs or concrete surfaces occurs significantly faster than from vegetated surfaces. This results in a faster concentration of flow within the downstream area of the catchment, and increased peak flow in some situations. This is less important in rural studies as they consist of relatively few impervious areas, and those areas are typically not hydraulically connected to the waterway (i.e. the water flows across pervious areas on the route between the impervious surface and the receiving waterway). Impervious percentages for each sub-catchment were derived from aerial imagery. Overall, approximately 7% of the hydrologic model extent was considered impervious, with majority occurring in the urban areas.

7.4. WBNM Adopted Hydrologic Model Parameters

The WBNM model input parameters for each subcatchment are listed below, with adopted values provided in Table 6:

- A lag factor (termed ‘C’), which can be used to accelerate or delay the runoff response to rainfall;
- A stream flow routing factor, which can accelerate or decelerate in-channel flows occurring through each subcatchment;
- An impervious area lag factor;
- An areal reduction factor;
- The percentage of catchment area with a pervious/impervious surface; and
- Rainfall losses calculated by initial and continuing losses to represent infiltration.

Table 6: WBNM model parameters

Parameter	Value
C (Catchment Routing)	1.6
Impervious Catchment Area	3%
Stream Routing Factor	1
Impervious Area Lag Factor	0.1
Initial loss	Varies (see Section 7.5)
Continuing loss	

7.5. Flood Model Update - Hydraulic Model

7.5.1. Selection of Hydraulic Model: TUFLOW

As part of this FRMS, the RMA-2 model established in Reference 5 has been converted to a 1D/2D TUFLOW hydraulic model. The main benefits of moving to a 1D/2D TUFLOW model is the ability to appropriately represent hydraulic structures, including culverts and bridges. These types of structures are particularly important in frequent flood events, especially with the addition of overland flow estimation which is influenced greatly by the local stormwater drainage network. TUFLOW is also the preferred platform for the assessment of flood modification options later in the study, as the digital elevation model (DEM) can be easily altered to model basins, levees, etc.

In 2017, TUFLOW offered Heavily Parallelised Computing (HPC) an alternate 2D Shallow Water Equation (SWE) solver to TUFLOW Classic. Whereas TUFLOW Classic is limited to running a simulation on a single CPU core, HPC provides parallelisation of the TUFLOW model allowing modellers to run a single TUFLOW model across multiple CPU cores or GPU graphics cards. Simulations using GPU hardware has been shown to provide significantly quicker model run times than those modelled using CPU cores. As such, the GPU model was used for the initial stages of this study, and there is potential to continue using the GPU model moving forward to flood mitigation option assessment later in the study. Alternatively, the models can be run over a longer timeframe using CPU. TUFLOW Version 2018-03-AC was used for this Study.

7.5.2. TUFLOW Hydraulic Model Extent

The hydraulic model extent has been extended upstream (approximately 1.5 km) along Racecourse Creek, with the upstream model boundary approximately at the eastern boundary of the Antill Country Golf Course. Throughout Picton itself, the hydraulic model has been broadened to include all development east of Argyle Street. The extension of the hydraulic model allows for potential mitigation options to be tested in these areas. The downstream boundary has been relocated to a point 1.25 km downstream of the gauge (an additional 500 m from the original downstream boundary location) to ensure the modelled tailwater conditions do not artificially influence flood behaviour upstream.

7.5.3. TUFLOW Topographic Data

With the extension of the hydraulic model area, it was necessary to obtain additional topographic data to cover the extended Study Area. Light Detection and Ranging (LiDAR) survey of the study area and its immediate surroundings was provided for the study by LPI. LiDAR is aerial survey data that provides a detailed topographic representation of the ground with a survey mark approximately every square metre. The data for the Picton area was collected in 2011 and supplemented with 2019 data. The accuracy of the ground information obtained from LiDAR survey can be adversely affected by the nature and density of vegetation, the presence of steeply varying terrain, the vicinity of buildings and/or the presence of water. The accuracy is typically ± 0.15 m for clear terrain. The data extent is shown on Figure 2. The model adopts a 2 m x 2 m grid resolution which is locally refined to show sub-grid elements such as kerbs and gutters (described in Section 7.5.5.4). A 4 m x 4 m grid is used for the PMF event to ensure the model does not become unstable at locations with particularly high velocities.

The LiDAR was utilised over the whole extended Study Area. However, the RMA topography had a well-defined creek invert (due to use of localised survey), whereas due to vegetation and sharp changes in elevation, the LiDAR did not represent the creek channel appropriately. For this reason, the RMA topography in the immediate vicinity of the main creeks was retained for this FRMS&P, with LiDAR used in the remaining areas.

7.5.4. TUFLOW Hydraulic Roughness

Roughness, represented by the Mannings ‘n’ coefficient, is an influential parameter in hydraulic modelling. Values for Manning’s ‘n’ are initially set based on industry experience of appropriate values for each surface type (e.g. concrete, grass, heavy vegetation). As part of the calibration process, these roughness values are adjusted within ranges defined in the literature so that the model better matches observed peak flood levels at a variety of locations. Chow (Reference 16) provides the definitive reference work in regards to the setting of the of the roughness values for hydraulic calculations. The adopted value of Manning’s ‘n’ for each surface type were based on those used in the Flood Study (Reference 5) are listed in Table 7. It is noted that the Manning’s ‘n’ value for roadways has been lowered from n = 0.03 to 0.01, and that the GIS layers used to assign Manning’s ‘n’ values were extended to cover the areas now included in the hydraulic model extent (refer to Section 7.5.2). The resulting Manning’s ‘n’ GIS layers are shown on Figure 12.

Table 7: Manning's 'n' hydraulic roughness parameters

Surface Type	Manning’s n
Creek Channel Clear of Vegetation	0.03
Creek Channel with moderate Vegetation	0.04
Heavily Vegetated Creek	0.06
Grassed Floodplain	0.04
Floodplain with sparse trees	0.06
Floodplain with moderate coverage of trees	0.075
Floodplain with dense trees	0.09
Bridge Crossings*	0.1
Roadway	0.01
Industrial Development	0.065
Urban/ Residential	0.04
Other	0.075
Paved surfaces	0.01
Buildings*	0.1
Dams	0.01

Note: The Flood Study used high Manning’s ‘n’ values to represent large bridges – refer to Section 7.5.5 for updated approach to modelling hydraulic structures, no longer requiring a Manning’s ‘n’ coefficient. The Manning’s ‘n’ value listed for bridge crossings (and buildings) in the above table are not applied in the updated TUFLOW model.

7.5.5. TUFLOW Hydraulic Structures

7.5.5.1. Bridges

A number of bridges were modelled in the 2D model domain using a 2D layered flow constriction. 2D layered flow constriction elements will allow the different layers of the bridge (below deck, deck, railings and above railings) to be modelled so that the losses associated with different flow conditions can be more accurately simulated. A site inspection was undertaken in April 2018 to identify and measure key hydraulic structures. For larger bridges, measurements were estimated from photographs, LiDAR data and WAE drawings provided by Council where available. The locations of bridges are shown on Figure 13.

7.5.5.2. Culverts

Road culverts were modelled in the 1D domain allowing a more accurate representation of the flow and velocity through the culvert based on upstream and downstream water levels, as well as flow overtopping the road or embankment in the 2D domain, based on WAE drawings and stormwater plans. Entry and exit losses and minor losses through structures are also incorporated using industry standard parameters. Key culvert structures are shown on Figure 13 and include Fairleys Road at Evelyn Bridge, Jarvisfield Park and Picton Road (South of Baxter Lane).

7.5.5.3. Pit and Pipe Network

Pit and pipe networks play an important role in managing runoff in frequent events. With the addition of modelling overland flow as a flood mechanism in Picton it is especially important to understand the capacity of the local stormwater network in the urban areas. A database of stormwater pits and pipes within the catchment was provided by Council. Where needed, additional details were gathered via visual inspection or assuming pipe diameters based on location and estimating pipe invert levels based on LiDAR data and reasonable pipe cover depths. Pit inverts were assumed to be 1-1.5 m below the ground level (from LiDAR), and were manually adjusted where needed to ensure no negative grades were assigned to pipes. This approach is considered to provide a reasonable level of detail and modelling accuracy in light of the overall study objectives. However, it is noted that there may be localised inaccuracies that should be taken into account when considering detailed flood behaviour on an individual property scale.

7.5.5.4. Kerbs and Guttering

The 2D domain consists of a 2 m grid that defines the topography throughout the study area. This is an appropriate cell size to represent flood behaviour in an urban area with a high level of detail whilst retaining manageable model run times. Some features of the urban environment, however, may not be well represented with a 2 m grid, such as the kerb and gutter systems (known as 'sub-grid features'). With the addition of the pit and pipe network and the estimation of overland flow, the street drainage becomes an important topographic feature and needs to be adequately represented in the model.

The locations and dimensions of roads, kerbs and gutters throughout the study were obtained from the 1 m LiDAR DEM and confirmed via visual inspection during site visits and though a desktop assessment. These features were explicitly represented in the terrain as breaklines to ensure their effect on flood behaviour was modelled appropriately.

7.5.5.5. Buildings

The representation of buildings within the study area is based on the same approach used in the Flood Study (Reference 5). In this method, buildings are 'nulled out', or removed from the computational grid to effectively exclude any flow from entering buildings. While this is not necessarily realistic (as flow can enter buildings), it is an appropriate method that simulates the obstruction that buildings can impose on floodwaters.

The original buildings GIS layer was adopted from the RMA-2 model. The following modifications were made before implementing the layer in TUFLOW:

- Minor distortions to building boundaries formed as a result of the RMA-2 irregular mesh were manually corrected;
- Building polygons had originally been digitised to include roof overhangs. This had the effect of overestimating the footprint size (sometimes impinging on road corridors), and not adequately allowing for flow between buildings. The polygons were slightly reduced to correct this; and
- Additional buildings in recently developed areas were incorporated (e.g. Jarvisfield) based on aerial imagery available at the time of model development. It is noted however that as development continues in this area, the building GIS layer may require further revision in the future.

7.5.6. TUFLOW Boundary Conditions

7.5.6.1. Inflows

For sub-catchments within the TUFLOW model domain, local runoff hydrographs were extracted from the WBNM model (see Section 6.2). These were applied to the downstream end of the sub-catchments within the 2D domain of the hydraulic model. Flows entering the model extent from upstream of the model boundary (i.e. east of Antill Golf Course on Racecourse Creek and west of Abbotsford Road on Stonequarry Creek) were applied to the boundary of the model. The inflow locations are shown on Figure 14.

7.5.6.2. Downstream Boundary

A water level vs flow curve was applied to the downstream hydraulic model boundary. This curve is generated by TUFLOW using the gradient and cross-section of the flow path. The flood gradient was assumed based on the topographic gradient of the DEM, and was found to be 0.5% at the boundary location.

8. CALIBRATION

8.1. Objectives

The objective of the calibration process is to build a robust hydrologic and hydraulic modelling system that can replicate historical flood behaviour in the catchment being investigated. If the modelling system can replicate historical flood behaviour then it can more confidently be used to estimate design flood behaviour. The resulting outputs from design flood modelling are used for planning purposes and for infrastructure design, and in this FRMS specifically, assessment of flood mitigation options. For this study, recorded streamflow data and surveyed high water marks from the June 2016 event were available to use for calibration purposes.

8.2. Methodology

The Stonequarry Creek at Picton Gauge (Site No. 212053) recorded water level data (converted to flow data using the rating curve described in Section 5.1) was suitable for model calibration. The gauge location is indicated on Figure 2. In addition, 76 surveyed high water marks (HWM) were collected by Council following the event. These points measured the depth of water above ground at various locations through the town.

A rainfall grid was produced to incorporate recorded depths from 12 gauges in the vicinity of the catchment, listed in Table 8. The grid was manually corrected to appropriately account for the influence of recorded depths occurring outside of the Stonequarry Creek catchment boundary. The gridded rainfall depths are shown on Figure 15.

Table 8: Rainfall Gauges in and around Picton

Station ID	Name	Type
68052	Picton Council Depot	Daily
68122	Cawdor (Woodburn)	Daily
68125	Oakdale (Cooyong Park)	Daily
68159	Wedderburn (Booalbyn)	Daily
68166	Buxton (Amaroo)	Daily
68200	Douglas Park (St. Marys Towers)	Daily
68216	Menangle Br (Nepean River)	Daily
68254	Mount Annan Botanic Garden	Daily
68192	Camden Airport AWS	Daily
212053	Stonequarry Creek at Picton	Pluviometer
212063	Lake Nerrigorang	Pluviometer
568296	Thurns Rd	Pluviometer

The approach to model calibration involved using the gridded rainfall data to input rainfall depths into the WBNM hydrologic model, and adjusting the rainfall loss parameters in the WBNM model. Multiple combinations of initial and continuing losses were investigated until the best fit to the flow hydrograph at the gauge and recorded HWM in the study area could be achieved.

8.3. Results

A comparison between the recorded and modelled flow hydrographs at the Stonequarry Creek at Picton gauge is shown on Figure 16. An initial loss of 120 mm and continuing loss of 0.5 mm/hr was found to produce the best fit to the recorded data. Note that the ‘recorded’ flow data was produced by converting recorded water level data to flow using the rating curve described in Section 5.1.

The modelled peak flood depths were compared to the surveyed HWM across the study area, with the results shown on Figure 17. Out of the 76 provided HWM locations, only 62 points contained measured depths useful for calibration purposes. Out of the 62 points, the model produced peak flood depths at 42 locations within +/- 0.3 m of the recorded depth, with an even spread of points above and below this threshold. The comparison found that, on average, the model produced peak flood depths 0.02 m lower than the recorded depths. The low average variance, and even distribution of points above and below the recorded depths indicates that the model is generally reproducing historic flood behaviour to a suitable degree, and further, that there is no systematic bias that would overestimate or underestimate flood levels. This indicates that the selection of parameters is appropriate for use moving forward with design flood estimation.

9. DESIGN FLOOD EVENT MODELLING

9.1. Overview

The hydrologic and hydraulic models developed as part of the calibration process (Section 8) have been adopted for use in design flood modelling. Key parameters such as topography, catchment routing lag and Manning's "n" remain unchanged from the June 2016 calibration event modelled. All other input parameters, data and assumptions that form the basis of the design flood modelling are based on inputs from ARR 2016 and are detailed below.

9.2. ARR 2016 Methodology

The Australian Rainfall and Runoff (AR&R) guidelines were updated in 2016 due to the availability of numerous technological developments, a significantly larger rainfall dataset since the previous edition (in 1987) and development of updated methodologies. A key input to the process is information derived from rainfall gauges, and the dataset now includes a larger number of rainfall gauges which continuously recorded rainfall (pluviometers) and a longer record of storms, including additional rainfall data recorded between 1985 and 2012.

Following a detailed comparison of ARR 1987 and ARR 2016 guidelines (described in Section 6.2.2), it was resolved that the ARR 2016 guidelines would be adopted for design flood modelling for this study. This includes the use of ARR 2016 IFD information and temporal patterns for the 20%, 10%, 5%, 2%, 1%, 0.5%, 0.2% AEP events. The PMF flows were derived using the Bureau of Meteorology's Generalised Short Duration Method (Reference 25) to estimate the probable maximum precipitation (PMP).

The ARR2016 temporal patterns, the procedure for the selection of the critical duration, temporal pattern and adopted hydrologic model parameters are discussed in the following sections. The Flood Frequency Analysis was used to validate the results produced by the WBNM model). The flows generated by the WBNM model for the critical duration for each design flood event were then used as inflows in the calibrated TUFLOW model to define the flood behaviour across the catchment. The resulting flood behaviour simulated in the TUFLOW model is subsequently presented, including an analysis of the results.

9.3. ARR 2016 IFD

Design rainfalls (ARR 2016 IFDs) were obtained from the BoM website (Bureau of Meteorology, 2017) for specific AEP and duration combinations across the catchment at a 2.3 km by 2.8 km grid. The IFD values for the catchment centroid (Easting 277462.2, Northing 6216538.7) are presented in Table 2.

Table 9: 2016 IFD Data (mm)

Duration (hours)	AEP							
	50%	20%	10%	5%	2%	1%	0.5%	0.2%
1	23.9	33.1	39.8	46.8	56.6	64.7	69.4	77.8
1.5	27.3	37.7	45.2	52.9	63.8	72.6	77.9	87.4
2	30.1	41.5	49.6	57.9	69.7	79.2	84.8	95.1
3	34.6	47.8	57.2	66.7	79.9	90.5	96.6	108
4.5	40.1	55.7	66.8	78.0	93.3	105.0	112	125
6	44.8	62.7	75.3	88.1	105.0	119.0	126	140
9	52.7	74.6	90.1	106.0	127.0	143.0	151	168
12	59.3	84.7	103.0	121.0	145.0	164.0	173	192
18	70.0	101.0	124.0	147.0	176.0	199.0	211	235
24	78.4	114.0	141.0	168.0	201.0	227.0	243	272
30	85.2	125.0	154.0	185.0	222.0	251.0	274	309
36	91.0	134.0	166.0	199.0	239.0	271.0	299	339
48	99.9	148.0	183.0	221.0	266.0	301.0	336	384
72	112.0	165.0	205.0	248.0	299.0	338.0	379	436

It is noted that across all durations and AEPs, that the eastern portion of the catchment has higher design rainfall depths compared to the rest of the catchment. That is, the long-term rainfall records indicate that the eastern portion of the catchment has higher rainfall depths compared to the west catchment. For the 1% and the 5% AEP event, the spatial variability (i.e. change of rainfall depths across the catchment) was as much as 19.5% for the 1% AEP 12 hour duration. The spatial distributions were derived by generating a rainfall depths grid across the catchment using 0.025 decimal spacing. Each point was queried using BoM's online 2016 IFD tools. Rainfall depths were retrieved for each point for the 2016 IFD dataset. A surface grid was generated for each AEP / duration combination by interpolating point values. The spatial distribution for the 1% AEP, 12 hour is shown on Figure 18.

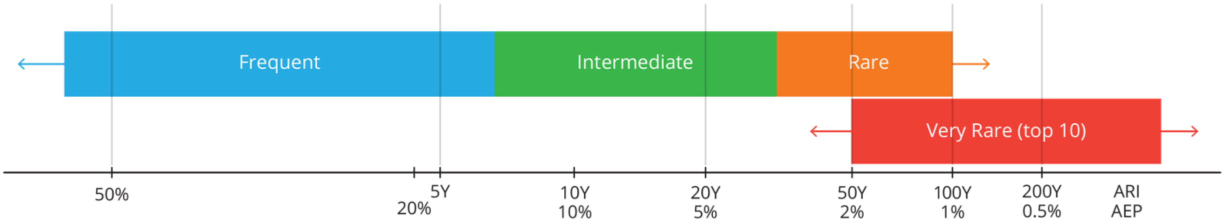
9.4. ARR 2016 Temporal Patterns

Temporal patterns describe how rain falls over time and form a component of storm hydrograph estimation. Previously, with ARR 1987 guidelines (Reference 1), a single temporal pattern was adopted for each rainfall event duration. However, ARR 2016 (Reference 2) discusses the potential deficiencies of adopting a single temporal pattern. It is widely accepted that there are a large variety of temporal patterns possible for rainfall events of similar magnitude. This variation in temporal pattern can result in significant effects on the estimated peak flow. As such, the revised temporal patterns have adopted an ensemble of ten different temporal patterns for a particular design rainfall event. Given the rainfall-runoff response can be quite catchment specific, using an ensemble of temporal patterns attempts to produce the median catchment response.

As hydrologic modelling has advanced, it is becoming increasingly important to use realistic temporal patterns. The ARR 1987 temporal patterns only provided a pattern of the most intense burst within a storm, whereas the 2016 temporal patterns look at the entirety of the storm including pre-burst rainfall, the burst and post-burst rainfall. There can be significant variability in the burst loading distribution (i.e. depending on where 50% of the burst rainfall occurs an event can be defined as front, middle or back loaded). The ARR 2016 method provides patterns for 12 climatic regions across Australia, with the Stonequarry Creek catchment falling within the East Coast South region. In particular, the ARR2016 temporal patterns in this region (especially the 1% AEP 12 hr duration temporal patterns) are characterised by having a slightly lower rainfall distribution across the start of the event, with the burst occurring later in the storm, which is more representative of recorded rainfall events in Picton than the ARR1987 temporal pattern.

ARR 2016 provides patterns for each duration which are sub-divided into three temporal pattern bins based on the frequency of the events. Diagram 1 shows the three categories of bins (frequent, intermediate and rare) and corresponding AEP groups. The "very rare" bin is currently unavailable and was not used in this flood study. There are ten temporal patterns for each AEP/duration in ARR 2016 that have been utilised in this study for the 20% AEP to 0.2% AEP events.

Diagram 1: Temporal Pattern Bins



Temporal patterns for this study were obtained from the ARR 2016 data hub (Reference 2, http://data.arr-software.org/). A summary of the data hub information at the catchment centroid is presented in Appendix B. The method employed to estimate the PMP utilises a single temporal pattern (Reference 25).

9.4.1. ARR 2016 Areal Temporal Patterns

ARR 2016 recommends that areal temporal patterns should be considered in catchments larger than 75 km^2 to account for the spatial smoothing of rainfall that occurs over larger catchments. At Picton, Stonequarry Creek has a catchment of approximately 84 km^2, putting it just above the threshold at which areal temporal patterns should be considered. It was found that areal temporal patterns tended to underestimate flows compared to the Flood Frequency Analysis results, and the distribution of rain across the areal temporal patterns were less representative of recorded events at Picton than the standard temporal patterns described in Section 9.4. In addition, ARR 2016 suggests that the shape of a catchment can influence the variability of results. The Stonequarry Creek catchment at Picton is made up of two key areas: the Racecourse Creek catchment (around 35 km^2) in the north east, and the remainder of the Stonequarry Creek catchment through the centre of Picton and to the west (approximately 50 km^2). Given the two catchments are each well below the 75 km^2 threshold, the total catchment may be better characterised as two smaller catchments. With these factors in mind, it was considered appropriate to not apply areal temporal patterns in this Study.

9.5. Critical Duration Assessment

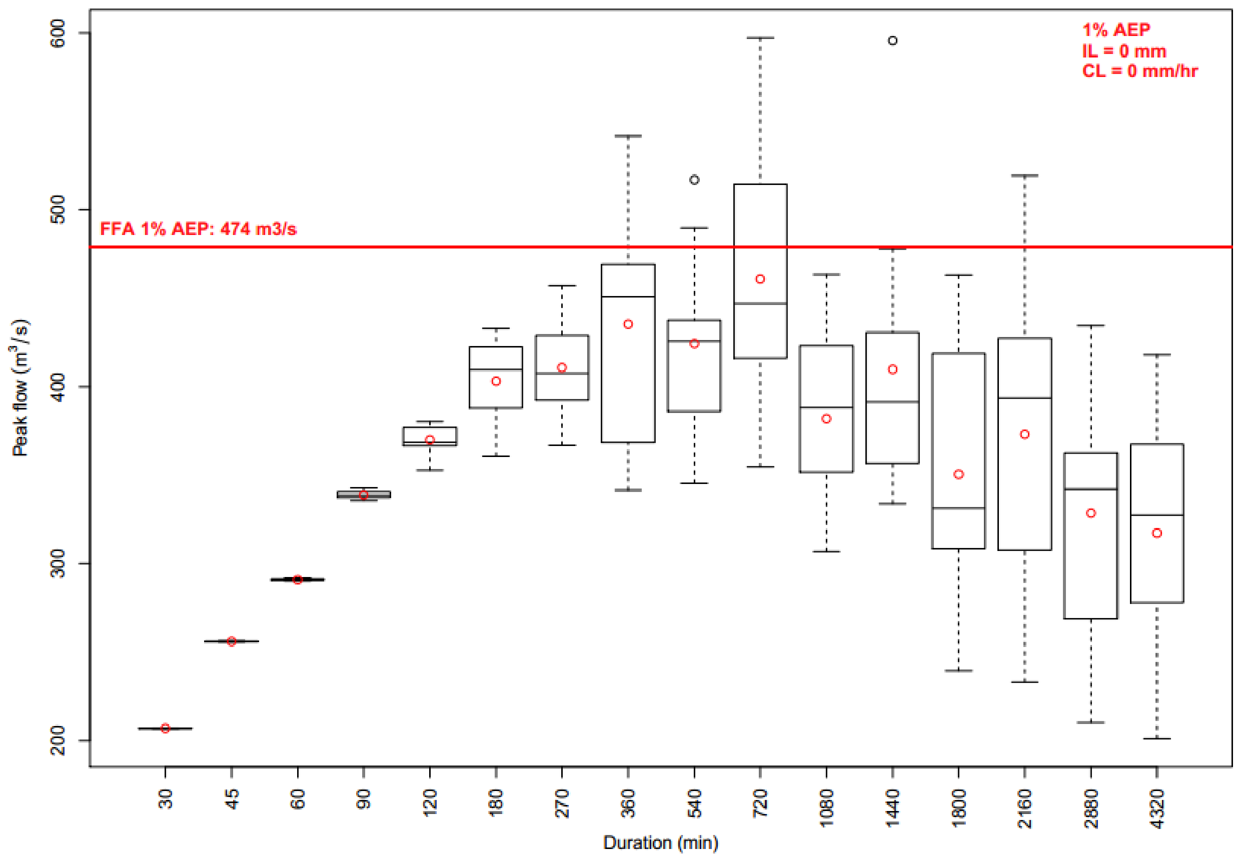
9.5.1. Assessment Approach

In Picton, flooding is caused by both mainstream flooding from Stonequarry Creek, and local overland flow from the relatively small, hilly catchment immediately east of the town centre. Typically, mainstream flooding in catchments of this size is generated by longer storm durations, whereas local overland catchments are generally more responsive to shorter, more intense storms. For this reason, the critical duration for mainstream and overland flooding in Picton were assessed using different approaches. For mainstream flooding, the critical duration was based on peak flow, while for overland flow, the storm duration that produced the highest peak flood levels was selected. Subsequently, the temporal pattern that best represented the median catchment response and flood behaviour for a specific design event. This process is described below.

9.5.1.1. Mainstream Flooding – Critical Duration Assessment

The Stonequarry Creek at Picton gauge location was selected to assess the peak flows produced by the WBNM hydrologic model. The WBNM model was used to determine the flows produced by each of the ten temporal patterns, across all storm durations, as provided by the ARR Data Hub (See metadata provided in Appendix B). The peak flow results were analysed at this location using box plots, shown in Diagram 2.

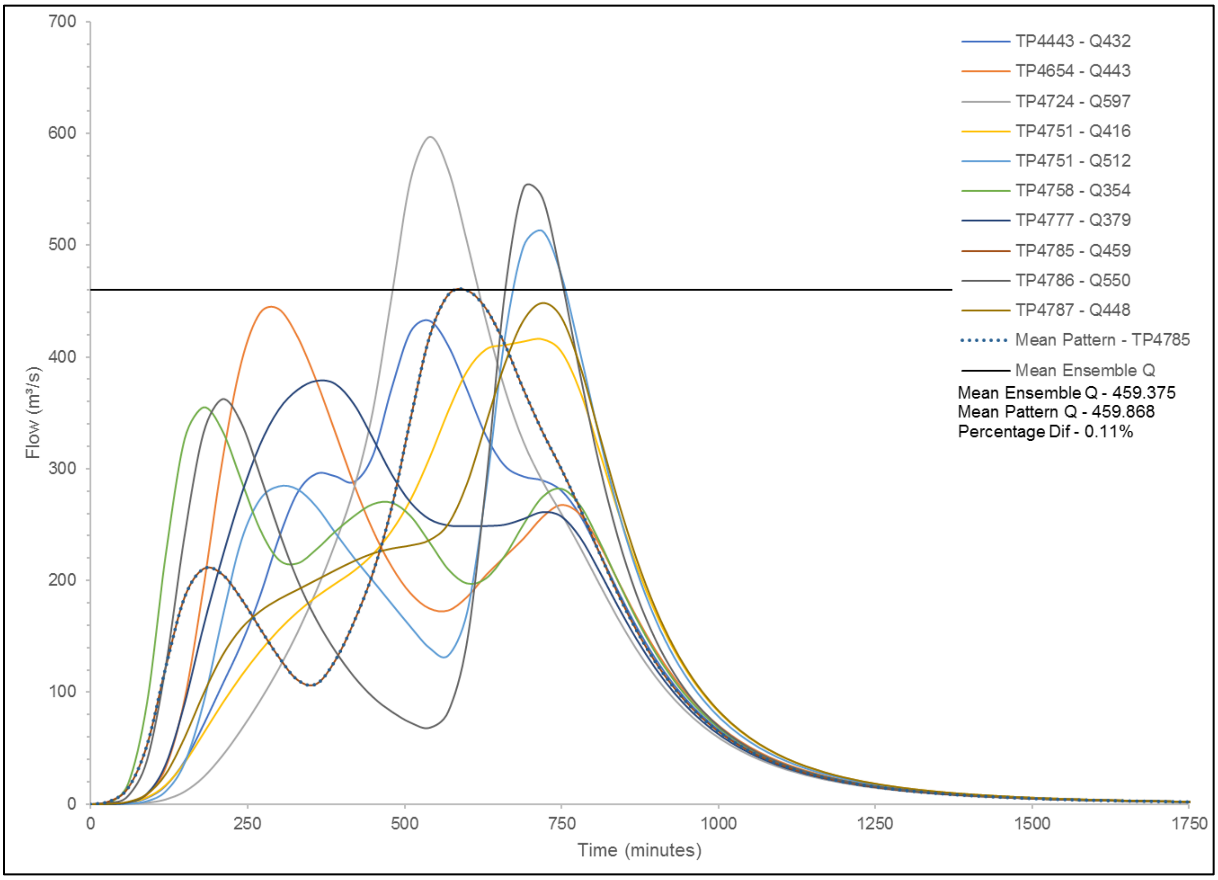
Diagram 2: Box Plot of Peak Flows at Stonequarry Creek Gauge – 1% AEP Event



The box and whiskers for each duration indicate the spread of results obtained from the ensemble of temporal patterns. The box defines the first quartile to the third quartile of the results and the bottom and top line (also called 'whiskers') represent the maximum and minimum values. The black circles beyond these lines are statistical outliers. The horizontal line within the box represents the median value. The red circle is the mean value.

For the 1% AEP event, the critical duration was determined to be the 720 minute storm (12 hour) duration. Within this duration, the temporal pattern that produced the peak flow just above the mean peak flow was selected in accordance with ARR 2016 guidelines, with the selected temporal pattern shown in Diagram 3. Note that this is *not* the pattern that produces the *largest* peak flow for that storm duration. The resulting adopted temporal patterns derived from this process are listed in Table 14.

Diagram 3: 720 minute 1% AEP flow hydrographs for subcatchment PIC_165



9.5.1.2. Overland Flow – Critical Duration Assessment

As overland flow is characterised by multiple shallow flow paths and sheet flow, it was not appropriate to select any one point for the comparison of peak flows. Instead, the full 10 temporal patterns for the 1, 2, 3, 6, 9 and 12 hour storms were run through the TUFLOW model to produce peak flood level result grids. An analysis of enveloped grids revealed that the 1 hour duration was critical in the overland-flow affected areas of Picton, particularly to the north and east of the town centre, while the 12 hour and 6 hour storms produced the highest peak flood levels within the Stonequarry Creek channel itself. Further analysis indicated that the 6 hour storm produced only marginally higher peak flood levels within the Stonequarry Creek channel upstream of Elizabeth

Street and in Racecourse Creek (less than 100 mm higher than the 12 hour storm). Further, given that the 12 hour storm produced the highest peak flow at the gauge and highest peak flood levels in the Picton Town Centre, the 12 hour duration was deemed critical for mainstream flooding in Picton. The critical duration map is presented on Figure 19.

Once the critical duration was identified, the representative temporal pattern was selected. This was done by producing a 'mean grid' by averaging the 10 peak flood level grids (each produced by a different temporal pattern). The peak flood level results of each temporal pattern were then compared to the mean grid to assess the differences. The temporal pattern that produced results as close to and just above the mean grid was selected as the 'adopted temporal pattern' for overland flow flooding.

The adopted temporal pattern and critical duration for the largest event in each bin (See Diagram 1) was applied to the more frequent event within the same bin, for example, the adopted temporal pattern for the 1% AEP event was applied to the 2% AEP event, and that which was selected for the 5% AEP event was applied to the 10% AEP event. To ensure this approach was appropriate in this catchment, the same analysis described above was undertaken for the 2% AEP overland flow event independently, whereby the peak flood level results produced by the 10 patterns were each compared to the mean grid produced by averaging results of the 10 patterns. While the analysis revealed that temporal pattern No. 4559 would be technically the preferred selection for the 2% AEP event, the peak flood level results produced by the adopted 1% AEP temporal pattern (TP4463) were less than 0.002 m higher, indicating that applying the same temporal pattern as the 1% AEP event would not materially affect results of the 2% AEP event.

9.5.2. Critical Duration Assessment Results

Table 10 below presents a summary of the critical duration and adopted temporal pattern for each design flood event, both for mainstream and overland flow.

Table 10: Summary of adopted temporal patterns and critical durations

Event	Overland Flow		Mainstream Flooding	
	Duration	TP#	Duration	TP#
20% AEP	18 hr	4846*	18 hr	4846*
10% AEP	6 hr	4678	9 hr	4763
5% AEP	6 hr	4678	9 hr	4763
2% AEP	1 hr	4463	12 hr*	4785
1% AEP	1 hr	4463	12 hr*	4785
0.5% AEP	1 hr	4463	12 hr*	4785
0.2% AEP	1 hr	4463	12 hr*	4785
PMF	3 hr	GSDM	3 hr	GSDM

*Note: For the 20% AEP event, temporal pattern 4846 yields flows that are slightly below the mean, however this temporal pattern was chosen as it produced flows that were closest to the mean flow.

9.5.3. Probable Maximum Precipitation

The Probable Maximum Precipitation (PMP) is 'the greatest depth of precipitation for a given duration meteorologically possible...' (Reference 25. It is used together with spatial and temporal distributions to estimate the Probable Maximum Flood (PMF). Reference 25 indicates that for Picton, the GSDM can be used to determine durations of up to 6 hours. The GSDM was therefore applied for the 1, 2, 3 and 6 hour durations and the Generalised Southeast Australia Method (GSAM) was then used to assess the 12, 24 and 36 hour duration events. Analysis of enveloped peak flood level results showed that the three hour duration was critical. The factors used in the estimation of the PMP is outlined in Table 11 below. These factors were used in conjunction with the Initial Mean Rainfall Depths (taken from Table 2 of Reference 25) to produce Adjusted Mean Rainfall Depth, presented in Table 12.

Table 11: GSDM Input Parameters

Parameter	Value	Reference
Terrain Category	Rough	Section 4.2 (Reference 25)
Elevation Adjustment Factor (EAF)	1	Section 4.3 (Reference 25)
Moisture Adjustment Factor (MAF)	0.675	Section 4.4 (Reference 25)

Table 12: Adjusted Mean Rainfall Depth Between Successive Ellipses (A-E)

Duration (hrs)	A	B	C	D	E
1	333	297	268	237	200
2	502	444	400	349	328
3	608	535	485	428	413
6	810	717	645	562	536

The rainfall identified in Table 12 was applied to the WBNM model to produce inflow hydrographs, which were then applied to the TUFLOW model for the 1, 2, 3 and 6 hour durations. The resulting peak flood levels were enveloped to determine the critical duration across the study area. The results showed that the 3 hour storm produced the highest flood levels across the Stonequarry Creek floodplain, while Racecourse Creek and the overland areas in the north of the study areas were dominated by the 2 hour and 1 hour durations, respectively. However, analysis of results showed that the 3 hour duration produced peak flood levels within 0.02 m of each of the other event results. The 3 hour duration was therefore selected as the representative storm for the estimation of the PMF across the catchment. The peak flow and flood level results are presented in Section 10.

9.6. Areal Reduction Factors

An Areal Reduction Factor (ARF) is an estimate of how the intensity of a design rainfall event varies over a catchment, based on the assumption that large catchments will not have a uniform depth of rainfall over the entire catchment. The ARF is extracted via the ARR Data Hub (Reference 3), and applied to each sub-catchment. An ARF of 0.92 is applied for the 1% AEP event (12 hour duration). The full suite of ARFs across all design events and durations are taken directly from the Data Hub, and are presented in Appendix B.

9.7. Losses

The Australian Rainfall and Runoff 2016 Data Hub (Reference 3) recommends for the Stonequarry Creek catchment centroid a storm initial loss rate of 42 mm, and a continuing loss rate of 4.9 mm/hr. During finalisation of the design flood modelling, in January 2019, the NSW Office of Environment and Heritage released new guidance regarding the implementation of ARR2016 methodologies in NSW: *"Incorporating 2016 Australian Rainfall and Runoff in Studies Section 3.7.1 Initial and continuing losses, pre-burst and burst losses in NSW".* The new guidance was developed in response to work that found that the losses originally prescribed in the ARR 2016 Data Hub were overly high, which causes lower flood levels as it assumes more rainfall would infiltrate into the soil rather than contribute to flooding as runoff. The new guidance requires continuing losses (derived from the ARR 2016 Data Hub) to be multiplied by a factor of 0.4, and that the probability-neutral burst initial losses are to be used.

The continuing loss prescribed by the ARR 2016 Data Hub (4.9 mm/hr) was multiplied by 0.4, resulting in a continuing loss rate of 1.96 mm/hr. On this basis, a continuing loss rate of 2 mm/hr was applied as a starting point for the hydrologic assessment of design flows, and subsequently adjusted as part of the model refinement process. The adopted initial and continuing loss values for each design flood event are listed in Table 13.

Table 13: Adopted Initial and Continuing Losses

Event	Initial Loss (mm)	Continuing Loss (mm/hr)
20% AEP	60	3.5
10% AEP	50	2.5
5% AEP	50	2.5
2% AEP	20	2
1% AEP	0	0
0.2% AEP	0	0
PMF	0	0

The design flood modelling process, including critical duration assessment and selection of loss values, was validated to the results produced by the Flood Frequency Analysis. A comparison of peak flows for each design event are provided in the subsequent section.

10. DESIGN FLOOD EVENT MODELLING RESULTS

The results for the design flood events are presented in the following maps:

- Peak flood depth, extents and level contours on Figure 20 to Figure 24;
- Peak flood velocities on Figure 25 to Figure 27;
- Peak flood level profiles (long sections) on Figure 28 to Figure 30 (chainages shown on Figure 31);
- Key Reporting Locations located on Figure 31;
- Hydraulic categories on Figure 34 to Figure 36;
- Hydraulic hazard based on the Australian Disaster Resilience Handbook (Reference 26) on Figure 37 to Figure 39;

A discussion of these results is provided in the following sections.

10.1. Description of Results

10.1.1. Mainstream Flooding

In events up to and including the 20% AEP event, mainstream flows are generally contained within the main channels of Stonequarry Creek, Racecourse Creek and other tributaries. However, in events greater than the 20% AEP, flow breaks out of the main Stonequarry Creek channel and inundates the benched area along the left bank at the rear of properties along Davies Place (across the creek from Hume Oval). The capacity of this benched area is generally sufficient to contain the 1% AEP flow, however in events as frequent as the 5% AEP, flow breaks out from the main channel a little further downstream and flows northwards through an open drainage channel parallel to Barkers Lodge Road. In the 1% AEP event, Davies Place is overtopped to a depth of approximately 0.5 m.

Moving downstream to the town centre, the Stonequarry Creek channel contains flows in events up to the 5% AEP. However, in the 2% AEP event and above, the right bank is breached and flows break out into Argyle Street and Davidson Lane, inundating the St Mark’s Anglican Church grounds and open areas around Elizabeth Street to depths of approximately 1.5 m in the 1% AEP event. The Argyle Street bridge has its deck level at 156.62 mAHD, and is overtopped to a depth of approximately 0.6 m in the 2% AEP event. The 5% AEP event reaches the underside of the bridge but is not shown to overtop the bridge deck.

Flood levels at the downstream end of Racecourse Creek are influenced by tailwater levels in Stonequarry Creek at the confluence, with elevated water levels in Stonequarry Creek causing flows to ‘back up’ along Racecourse Creek. The extent of this backwatering is evident in the design peak flood level profiles on Figure 29, and is most pronounced in the PMF, in which the backwatering extends for approximately one kilometre upstream of the confluence. Crawfords Creek, a tributary of Racecourse Creek, is similarly controlled by the water levels in Racecourse Creek, as indicated by the relatively flat water level on Figure 30.

10.1.2. Overland Flow

There are two main areas of Picton affected by overland flow: the eastern part of the town centre which receives runoff from Vault Hill, and the recently developed areas just south of Racecourse Creek. In each of these areas, overland flow is generally shallow (less than 0.1 m) in the 5% AEP event, deepening only in flatter, low lying areas closer to the main creeks. Flow from the north east of town approaches Margaret Street and continues down Argyle Street towards Stonequarry Creek. In the 1% AEP event and greater, depths in the major drainage lines in Jarvisfield such as the open channel between Coldenham Road and the Golf Course reach up to 0.8 m. High velocities in this channel leads to its classification as floodway, described in Section 10.4. Overland flow approaching Stonequarry Creek from the western side is generally limited, though there is a minor flow path along the Old Hume Highway (Argyle Street). In events including and greater than the 20% AEP event, Menangle Street is overtopped just south of Baxter Lane, to a depth of approximately 0.3 m in the 20% AEP event.

10.2. Design Peak Flows and Levels

The design peak design flows and levels at the Stonequarry Creek Gauge are presented in Table 14.

Table 14: Design Peak Discharges at Stonequarry Creek Gauge (Gauge Zero: 147.803 mAHD)

Event	Peak Flow (WBNM) (m^3/s)	Peak Flow (TUFLOW) (m^3/s)	FFA Estimate (m^3/s)	Peak Flood Level (mAHD)	Peak Flood Level (m Gauge Height)
20% AEP	67	66	68	151.0	3.2
10% AEP	135	138	121	152.2	4.4
5% AEP	191	193	193	152.9	5.1
2% AEP	356	349	330	154.5	6.7
1% AEP	461	452	473	155.3	7.5
0.5% AEP	492	484	-	155.6	7.8
0.2% AEP	553	542	-	156.0	8.2
PMF	3465	2774*	-	165.2	17.4

*Note: Due to a change in tributary inflow timing (especially from Racecourse Creek) due to backwatering, attenuation in the upper western areas and significant storage of floodwaters on the Victoria Park playing fields, the TUFLOW hydraulic model produces a lower peak flow rate than the WBNM hydrologic model in the PMF event. The total volume passing the gauge however is consistent between the two models.

The peak design levels for the 5%, 1%, and PMF events at key locations throughout the town centre are presented in Table 15. A map with the key locations has been provided in Figure 31.

Table 15: Peak Flood Heights at Key Locations

Location	Design Peak Flood Level (mAHD)		
	5% AEP	1% AEP	PMF
Picton Hotel (Corner of Menangle St & Argyle ST)	157.0	158.0	166.6
Argyle Street Bridge (Over Stonequarry Creek)	155.6	157.8	166.4
Khan's SUPA IGA (Magnolis Ln)	157.8	158.2	166.8
George IV Inn (Corner of Argyle St & Crakanthorp Ln)	157.0	158.0	166.5
Liquorland Picton (Argyle St)	163.4	163.4	166.8

10.3. Climate Change Sensitivity Assessment

The sensitivity of the simulated 1% AEP peak flood levels to climate change was investigated. Climate change is expected to have adverse impacts upon sea levels and rainfall intensities. Sensitivity analysis of an increase in rainfall intensity was undertaken by comparing the 0.5% and 0.2% AEP events with the 1% AEP event. These events are commonly used as proxies to assess an increase in rainfall intensity. Within the Stonequarry Creek catchment, these events correspond to an increase in rainfall intensity of approximately 7% for the 0.5% AEP event and 20% for the 0.2% AEP event (see Table 9). The peak flood depth and level results of the 1%, 0.5% and 0.2% AEP events are shown on Figure 21, Figure 22 and Figure 23 respectively. A comparison of flood levels has been provided on Figure 32 and Figure 33.

The 0.5% AEP event is approximately 0.06 to 0.28 m higher along Stonequarry Creek than the 1% AEP event. The increase in flood level along the tributary flow paths is typically below 0.03 m. The largest increase in flood level occurs at Victoria Park, directly north of Webster Street and adjacent to Stonequarry Creek. Flood levels increases of up to 0.28 m are observed in this area. In the 0.2% AEP event, the increase in flood levels on Stonequarry Creek through the town is approximately 0.20 to 0.75 m. The increase in flood level on the overland flow paths is typically less than 0.03 m. Again, the largest increase in flood level occurs at Victoria Park, where the increase in flood level is up to 0.75 m. Racecourse Creek experiences increases in flood level by 0.1-0.2 m and 0.1-0.6 m respectively for the 0.2% and 0.5% AEP events. In both cases, there is little to no change in the tributary flow paths of Racecourse Creek.

10.3.1. Comparison to Previous Studies

Stonequarry Creek at Picton has been subject to a number of investigations over the years, described in Section 2. The below table presents a brief summary of how design flow estimates have changed since the 1989 and 2017 (draft) Flood Studies (Reference 8 and 5 respectively). Differences in modelled flow estimates are a product of a range of factors, including the application of different editions of ARR (1987 and 2016), inclusion of overland flow, different model types and calibration processes. A detailed description of these factors (and other contributing elements) is provided in Section 6, which provides a thorough review of the hydrologic and hydraulic models developed in the 2017 Draft Flood Study (Reference 5).

Table 16: Comparison of Hydrologic Design Peak Discharges

Tributary	XP-RAFTS Model Node	WBNM Model Node	Peak Discharge (m³/s)					
			1% AEP			5% AEP		
			1989 RAFTS model (Ref 8)	2017 Draft XP Rafts (Ref 5)	WBNM	1989 RAFTS model (Ref 8)	2017 Draft XP Rafts (Ref 5)	WBNM
Stonequarry Creek	1.06	PIC_025	273	305	230	194	230	91
Racecourse Creek	6.04	PIC_090	99	117	118	67	85	55
Crawfords Creek	5.01	PIC_043	58	68	53	40	51	22
Unnamed Creek	4.02	PIC_035	48	60	46	33	44	20
Downstream Extent of Study Area	1.10	PIC_165	494	574	461	345	431	191

*Refer to Figure 11 for XP-RAFTS node locations and WBNM subcatchments.

Table 17: Peak Design Discharges at the Stonequarry Creek at Picton Gauge

Event	1989 XP-RAFTS (m³/s)	XP-RAFTS (m³/s)	WBNM (m³/s)	TUFLOW (m³/s)	FFA Target Flows (m³/s)
5% AEP	345	431	191	193	193
1% AEP	494	574	461	452	473
PMF	-	-	3465	2774	-

Table 18: Peak Flood Level Comparison at Key Locations

Location	1% AEP Peak Flood Level (mAHD)		
	HEC-2 1989 Picton Flood Study (“Natural”) (Reference 8)	RMA-2 2017 Draft Flood Study (Reference 5)	TUFLOW
Picton Hotel (Corner of Menangle St, Argyle St)	158.1	158.2	158.0
Argyle Street Bridge (Over Stonequarry Creek)	158.0	157.9	157.8
Khan’s SUPA IGA (Magnolia Ln)	157.9	158.6	158.2
George IV Inn (Corner of Argyle St, Crakanthorp Ln)	158.0	158.0	158.0
Liquorland Picton (Argyle St)	158.0	157.9	157.8

10.4. Hydraulic Categorisation

Hydraulic categorisation of the floodplain is used in the Floodplain Risk Management process to assist in the assessment of the suitability of future types of land use and development, and the formulation of floodplain risk management plans. The Floodplain Development Manual (Reference 4) defines land inundated in a particular event as falling into one of the three hydraulic categories listed in Table 19.

Table 19: Hydraulic Categorisation Definitions (*Floodplain Development Manual* (Reference 4))

Category	Definition
Floodway	• Those areas where a significant volume of water flows during floods; • Often aligned with obvious natural channels; • Areas that, even if only partially blocked, would cause a significant increase in flood levels and/or a significant redistribution of flood flow, which my adversely affect other areas; and • Often, but not necessarily, areas with deeper flow or areas where higher velocities occur.
Flood Storage	• 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, for example by the construction of levees or by landfill, flood levels in nearby areas may rise and the peak discharge downstream may be increased; and • Substantial reduction of the capacity of a flood storage area can also cause a significant redistribution of flood flows.
Flood Fringe	• 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.

To define the floodway, the Howells et al. (Reference 19) methodology was applied, which differentiates the floodway from other hydraulic categories by selecting a velocity-depth product criteria that exceeds a specific threshold. These parameters were confirmed iteratively through encroachment analysis, in which all areas not defined as 'floodway' were totally excluded from the modelling domain, and the subsequent impact on flood levels examined. If the reduction in conveyance area resulted in an increase in greater than 0.1 m to existing flood levels, the floodway area was increased. This approach is informed by Section L4 of the Floodplain Development Manual (Reference 4), which defines Flood Storage areas as *"those areas outside floodways which, if completely filled with solid material, would cause peak flood levels to increase anywhere by more than 0.1 m and/or would cause the peak discharge anywhere downstream to increase by more than 10%."* The resulting parameters are provided in Table 20. Following application of these criteria, the resulting floodway areas were examined to ensure continuity of flowpaths, and to remove any isolated grid cells inappropriately classified as floodway (for example as an artefact of the modelling).

Table 20: Hydraulic Category Definition Parameters

Category	Floodway Definition Parameters
Floodway	VD > 0.3 m^2/s and V > 0.3 m/s;
Flood Storage	Areas outside floodway where D > 0.4 m
Flood Fringe	Areas outside floodway where D < 0.4 m

Hydraulic Categorisation for the 5% AEP, 1% AEP and PMF events are shown on Figure 34 to Figure 36 respectively. The analysis indicates that in the 5% AEP event, only the main creek channel and its tributaries are classified as floodways. Similarly, in the 1% AEP event, most of the floodway remains within the main Stonequarry Creek channel and its tributaries, with a few exceptions (described below), and out of bank flooding generally classified as flood storage or flood fringe. In particular. a major flood storage area is formed in and around Elizabeth Street. In the PMF event, most of the study area becomes a floodway with some flood storage and fringe areas closer to the edge of the floodplain.

In Stonequarry Creek in the 1% AEP event, the floodway extends along Argyle Street (between Coull Street and Walton Lane) and Davidson Lane at the rear of several commercial properties. The playing fields in Victoria Park become critical to the conveyance of flow in this size event, and are also classified as floodway due to the high velocities occurring in the open space. This is consistent with the flood behaviour observed in the June 2016 event.

In the northern section of the study area south of Racecourse Creek, three of the major local drainage lines area classified as floodway, including the Yallambi Street drain, the open channel behind properties on the western side of Old Racecourse Close, and the flow path between Coldenhan Road and the golf course.

10.5. Hydraulic Hazard Classification

Hazard classification plays an important role in informing floodplain risk management in an area as it reflects the likely impact of flooding on development and people. In the Floodplain Development Manual (Reference 4) hazard classifications are essentially binary – either Low or High Hazard as described on Figure L2 of that document. However, in recent years there has been a number of developments in the classification of hazard especially in *Managing the floodplain: a guide to best practice in flood risk management in Australia (Third Edition)* (Reference 17). The Flood Study (Reference 5) presents hazard categorisation mapping based on the Floodplain Development Manual, while this study presents revised mapping based on the methodology outlined in Reference 17. The classification is divided into 6 categories (H1-H6), listed in Table 21, which indicate constraints of hazard on people, buildings and vehicles appropriate to apply in each zone. The criteria and threshold values for each of the hazard categories are presented in Diagram 4.

Table 21: Hazard Categories

Category	Constraint to people/vehicles	Building Constraints
H1	Generally safe for people, vehicles and buildings	*No constraints*
H2	Unsafe for small vehicles	*No constraints*
H3	Unsafe for vehicles, children and the elderly	*No constraints*
H4	Unsafe for vehicles and people	*No constraints*
H5	Unsafe for vehicles and people	All buildings vulnerable to structural damage. Some less robust building types vulnerable to failure.
H6	Unsafe for vehicles and people	All building types considered vulnerable to failure

Diagram 4: Hazard Classifications

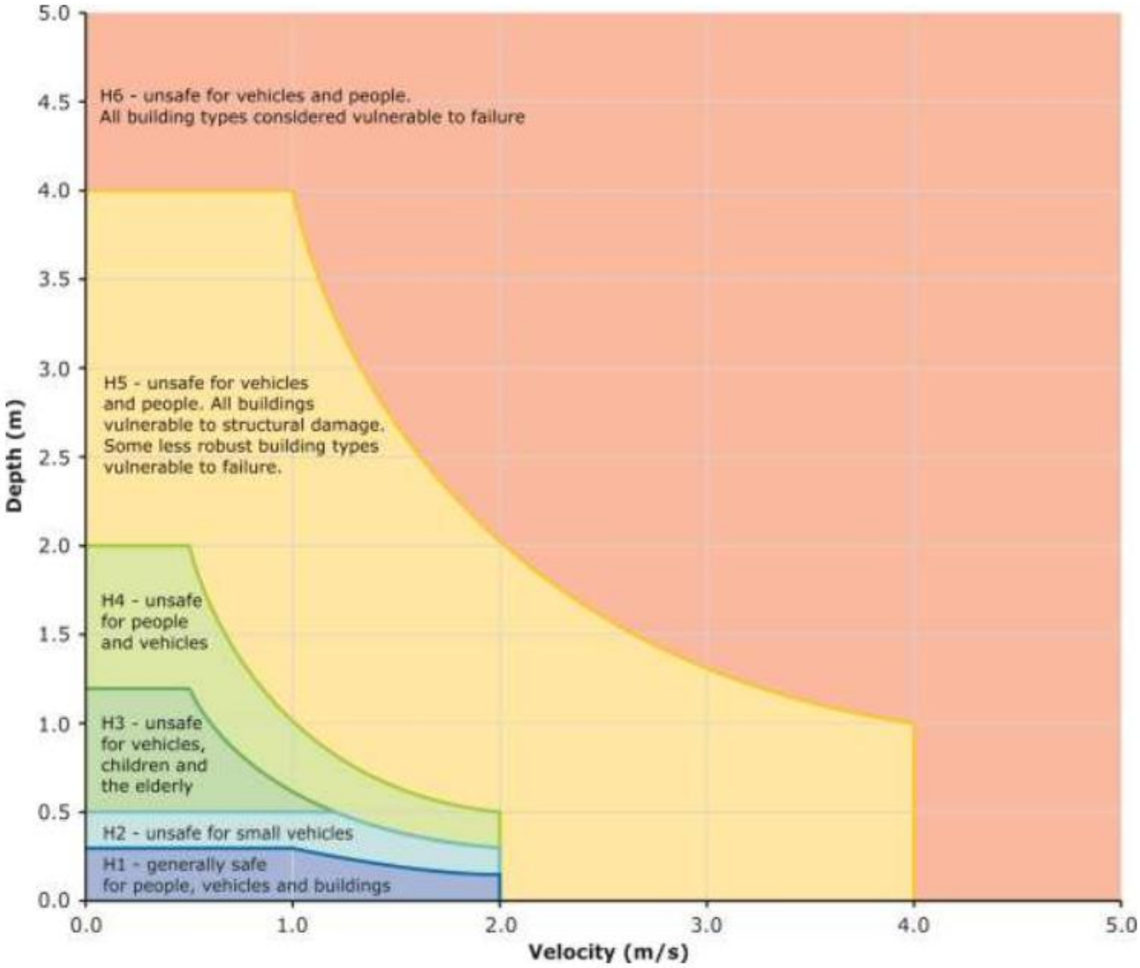


Figure 37 to Figure 39 present the hazard classifications based on the H1-H6 delineations for the 5% AEP, 1% AEP and PMF events respectively. In the 5% AEP event, all areas outside of the main channels of Stonequarry Creek and its tributaries are generally classified as H1 "generally safe for people, vehicles and buildings". However, in the 1% AEP event, parts of the town centre become much more hazardous, with Argyle Street classified as H5 between Menangle Street and Stonequarry Creek.

Further south, the Victoria Park playing fields are also classified as H3-H5, indicating that they would be dangerous for people and vehicles, and in parts, even buildings. Recently developed parts of Jarvisfield in the town's north are generally classified as H1, indicating a relatively low level of hazard constraint. In sections where flow becomes faster, for example along defined drainage channels, or deeper (in small dams within the golf course) the hazard classification is elevated. The Yallambi Street drain in particular is classified as H5 in the 1% AEP, and given its proximity to residential development, public safety and the suitability of on-street parking may warrant further investigation as part of the flood risk mitigation option assessment.

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Appendix A

Appendix A: Glossary

Taken from the Floodplain Development Manual (April 2005 edition)

acid sulfate soils	Are sediments which contain sulfidic mineral pyrite which may become extremely acid following disturbance or drainage as sulfur compounds react when exposed to oxygen to form sulfuric acid. More detailed explanation and definition can be found in the NSW Government Acid Sulfate Soil Manual published by Acid Sulfate Soil Management Advisory Committee.
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 500 m^3/s has an AEP of 5%, it means that there is a 5% chance (that is one-in-20 chance) of a 500 m^3/s or larger event occurring in any one year (see ARI).
Australian Height Datum (AHD)	A common national surface level datum approximately corresponding to mean sea level.
Average Annual Damage (AAD)	Depending on its size (or severity), each flood will cause a different amount of flood damage to a flood prone area. AAD is the average damage per year that would occur in a nominated development situation from flooding over a very long period of time.
Average Recurrence Interval (ARI)	The long term average number of years between the occurrence of a flood as big as, or larger than, the selected event. For example, floods with a discharge as great as, or greater than, the 20 year ARI flood event will occur on average once every 20 years. ARI is another way of expressing the likelihood of occurrence of a flood event.
caravan and moveable home parks	Caravans and moveable dwellings are being increasingly used for long-term and permanent accommodation purposes. Standards relating to their siting, design, construction and management can be found in the Regulations under the LG Act.
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.
consent authority	The Council, government agency or person having the function to determine a development application for land use under the EP&A Act. The consent authority is most often the Council, however legislation or an EPI may specify a Minister or public authority (other than a Council), or the Director General of DIPNR, as having the function to determine an application.
development	Is defined in Part 4 of the Environmental Planning and Assessment Act (EP&A Act). **infill development:** refers to the development of vacant blocks of land that are generally surrounded by developed properties and is permissible under the current zoning of the land. Conditions such as minimum floor levels may be imposed on infill development. **new development:** refers to development of a completely different nature to that associated with the former land use. For example, the urban subdivision of an area previously used for rural purposes. New developments involve rezoning and

typically require major extensions of existing urban services, such as roads, water supply, sewerage and electric power.

redevelopment: refers to rebuilding in an area. For example, as urban areas age, it may become necessary to demolish and reconstruct buildings on a relatively large scale. Redevelopment generally does not require either rezoning or major extensions to urban services.

disaster plan (DISPLAN) A step by step sequence of previously agreed roles, responsibilities, functions, actions and management arrangements for the conduct of a single or series of connected emergency operations, with the object of ensuring the coordinated response by all agencies having responsibilities and functions in emergencies.

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 for example, metres per second (m/s).

ecologically sustainable development (ESD) Using, conserving and enhancing natural resources so that ecological processes, on which life depends, are maintained, and the total quality of life, now and in the future, can be maintained or increased. A more detailed definition is included in the Local Government Act 1993. The use of sustainability and sustainable in this manual relate to ESD.

effective warning time The time available after receiving advice of an impending flood and before the floodwaters prevent appropriate flood response actions being undertaken. The effective warning time is typically used to move farm equipment, move stock, raise furniture, evacuate people and transport their possessions.

emergency management A range of measures to manage risks to communities and the environment. In the flood context it may include measures to prevent, prepare for, respond to and recover from flooding.

flash flooding Flooding which is sudden and unexpected. It is often caused by sudden local or nearby heavy rainfall. Often defined as flooding which peaks within six hours of the causative rain.

flood Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, estuary, lake or dam, and/or local overland flooding associated with major drainage before entering a watercourse, and/or coastal inundation resulting from super-elevated sea levels and/or waves overtopping coastline defences excluding tsunami.

flood awareness Flood awareness is an appreciation of the likely effects of flooding and a knowledge of the relevant flood warning, response and evacuation procedures.

flood education Flood education seeks to provide information to raise awareness of the flood problem so as to enable individuals to understand how to manage themselves an their property in response to flood warnings and in a flood event. It invokes a state of flood readiness.

flood fringe areas The remaining area of flood prone land after floodway and flood storage areas have been defined.

flood liable land	Is synonymous with flood prone land (i.e. land susceptible to flooding by the probable maximum flood (PMF) event). Note that the term flood liable land covers the whole of the floodplain, not just that part below the flood planning level (see flood planning area).
flood mitigation standard	The average recurrence interval of the flood, selected as part of the floodplain risk management process that forms the basis for physical works to modify the impacts of flooding.
floodplain	Area of land which is subject to inundation by floods up to and including the probable maximum flood event, that is, flood prone land.
floodplain risk management options	The measures that might be feasible for the management of a particular area of the floodplain. Preparation of a floodplain risk management plan requires a detailed evaluation of floodplain risk management options.
floodplain risk management plan	A management plan developed in accordance with the principles and guidelines in this manual. Usually includes both written and diagrammetic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.
flood plan (local)	A sub-plan of a disaster plan that deals specifically with flooding. They can exist at State, Division and local levels. Local flood plans are prepared under the leadership of the State Emergency Service.
flood planning area	The area of land below the flood planning level and thus subject to flood related development controls. The concept of flood planning area generally supersedes the Aflood liable land concept in the 1986 Manual.
Flood Planning Levels (FPLs)	FPLs are the combinations of flood levels (derived from significant historical flood events or floods of specific AEPs) and freeboards selected for floodplain risk management purposes, as determined in management studies and incorporated in management plans. FPLs supersede the Astandard flood event in the 1986 manual.
flood proofing	A combination of measures incorporated in the design, construction and alteration of individual buildings or structures subject to flooding, to reduce or eliminate flood damages.
flood prone land	Is land susceptible to flooding by the Probable Maximum Flood (PMF) event. Flood prone land is synonymous with flood liable land.
flood readiness	Flood readiness is an ability to react within the effective warning time.
flood risk	Potential danger to personal safety and potential damage to property resulting from flooding. The degree of risk varies with circumstances across the full range of floods. Flood risk in this manual is divided into 3 types, existing, future and continuing risks. They are described below. **existing flood risk:** the risk a community is exposed to as a result of its location on the floodplain. **future flood risk:** the risk a community may be exposed to as a result of new development on the floodplain.

	continuing flood risk: the risk a community is exposed to after floodplain risk management measures have been implemented. For a town protected by levees, the continuing flood risk is the consequences of the levees being overtopped. For an area without any floodplain risk management measures, the continuing flood risk is simply the existence of its flood exposure.
flood storage areas	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.
floodway areas	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 flows, or a significant increase in flood levels.
freeboard	Freeboard provides reasonable certainty that the risk exposure selected in deciding on a particular flood chosen as the basis for the FPL is actually provided. It is a factor of safety typically used in relation to the setting of floor levels, levee crest levels, etc. Freeboard is included in the flood planning level.
habitable room	**in a residential situation:** a living or working area, such as a lounge room, dining room, rumpus room, kitchen, bedroom or workroom. **in an industrial or commercial situation:** an area used for offices or to store valuable possessions susceptible to flood damage in the event of a flood.
hazard	A source of potential harm or a situation with a potential to cause loss. In relation to this manual the hazard is flooding which has the potential to cause damage to the community. Definitions of high and low hazard categories are provided in the Manual.
hydraulics	Term given to the study of water flow in waterways; in particular, the evaluation of flow parameters such as water level and velocity.
hydrograph	A graph which shows how the discharge or stage/flood level at any particular location varies with time during a flood.
hydrology	Term given to the study of the rainfall and runoff process; in particular, the evaluation of peak flows, flow volumes and the derivation of hydrographs for a range of floods.
local overland flooding	Inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.
local drainage	Are smaller scale problems in urban areas. They are outside the definition of major drainage in this glossary.
mainstream flooding	Inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam.

major drainage	Councils have discretion in determining whether urban drainage problems are associated with major or local drainage. For the purpose of this manual major drainage involves: • the floodplains of original watercourses (which may now be piped, channelised or diverted), or sloping areas where overland flows develop along alternative paths once system capacity is exceeded; and/or • water depths generally in excess of 0.3 m (in the major system design storm as defined in the current version of Australian Rainfall and Runoff). These conditions may result in danger to personal safety and property damage to both premises and vehicles; and/or • major overland flow paths through developed areas outside of defined drainage reserves; and/or • the potential to affect a number of buildings along the major flow path.
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 States rivers and floodplains. The merit approach operates at two levels. At the strategic level it allows for the consideration of social, economic, ecological, cultural and flooding issues to determine strategies for the management of future flood risk which are formulated into Council plans, policy and EPIs. At a site specific level, it involves consideration of the best way of conditioning development allowable under the floodplain risk management plan, local floodplain risk management policy and EPIs.
minor, moderate and major flooding	Both the State Emergency Service and the Bureau of Meteorology use the following definitions in flood warnings to give a general indication of the types of problems expected with a flood: **minor flooding:** causes inconvenience such as closing of minor roads and the submergence of low level bridges. The lower limit of this class of flooding on the reference gauge is the initial flood level at which landholders and townspeople begin to be flooded. **moderate flooding:** low-lying areas are inundated requiring removal of stock and/or evacuation of some houses. Main traffic routes may be covered. **major flooding:** appreciable urban areas are flooded and/or extensive rural areas are flooded. Properties, villages and towns can be isolated.
modification measures	Measures that modify either the flood, the property or the response to flooding. Examples are indicated in Table 2.1 with further discussion in the Manual.
peak discharge	The maximum discharge occurring during a flood event.

Probable Maximum Flood (PMF)	The PMF is the largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation, and where applicable, snow melt, 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, that is, the floodplain. The extent, nature and potential consequences of flooding associated with a range of events rarer than the flood used for designing mitigation works and controlling development, up to and including the PMF event should be addressed in a floodplain risk management study.
Probable Maximum Precipitation (PMP)	The PMP is the greatest depth of precipitation for a given duration meteorologically possible over a given size storm area at a particular location at a particular time of the year, with no allowance made for long-term climatic trends (World Meteorological Organisation, 1986). It is the primary input to PMF estimation.
probability	A statistical measure of the expected chance of flooding (see AEP).
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 actually ends up as streamflow, also known as rainfall excess.
stage	Equivalent to A water level. Both are measured with reference to a specified datum.
stage hydrograph	A graph that shows how the water level at a particular location changes with time during a flood. It must be referenced to a particular datum.
survey plan	A plan prepared by a registered surveyor.
water surface profile	A graph showing the flood stage at any given location along a watercourse at a particular time.
wind fetch	The horizontal distance in the direction of wind over which wind waves are generated.

Appendix B

Australian Rainfall & Runoff Data Hub - Results

Input Data

Longitude	150.586
Latitude	-34.168
Selected Regions	
River Region	
ARF Parameters	
Storm Losses	
Temporal Patterns	
Areal Temporal Patterns	
Interim Climate Change Factors	

Region Information

Data Category	Region
River Region	Hawkesbury River
ARF Parameters	SE Coast
Temporal Patterns	East Coast South

Data

River Region

division	South East Coast (NSW)
rivregnum	12
River Region	Hawkesbury River

Layer Info

Time Accessed	30 April 2019 01:43PM
Version	2016_v1

ARF Parameters

Long Duration ARF

$$Areal\ reduction\ factor = Min\left\{1, \left[1 - a\left(Area^{b} - c\log_{10}Duration\right)Duration^{-d} + eArea^{f}Duration^{g}\left(0.3 + \log_{10}AEP\right) + h10^{iArea\frac{Duration}{1440}}\left(0.3 + \log_{10}AEP\right)\right]\right\}$$

Zone	SE Coast
a	0.06
b	0.361
c	0.0
d	0.317
e	8.11e-05
f	0.651
g	0.0
h	0.0
i	0.0

Short Duration ARF

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

Layer Info

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Storm Losses

Note: Burst Loss = Storm Loss - Preburst

Note: These losses are only for rural use and are NOT FOR USE in urban areas

id	2254.0
Storm Initial Losses (mm)	42.0
Storm Continuing Losses (mm/h)	4.9

Layer Info

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Version	2016_v1

Temporal Patterns

code	ECsouth
Label	East Coast South

Layer Info

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Version	2016_v2

Areal Temporal Patterns

code	ECsouth
arealabel	East Coast South

Layer Info

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Version	2016_v2

BOM IFD Depths

Click here to obtain the IFD depths for catchment centroid from the BoM website

No data	No data found at this location!

Layer Info

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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)	0.4 (0.018)	0.9 (0.027)	1.2 (0.030)	1.5 (0.032)	1.4 (0.025)	1.3 (0.020)
90 (1.5)	1.0 (0.036)	1.2 (0.033)	1.4 (0.031)	1.6 (0.030)	1.0 (0.016)	0.6 (0.008)
120 (2.0)	0.0 (0.000)	0.8 (0.018)	1.3 (0.025)	1.7 (0.030)	1.6 (0.023)	1.5 (0.019)
180 (3.0)	1.3 (0.039)	1.8 (0.038)	2.1 (0.038)	2.5 (0.037)	2.6 (0.032)	2.6 (0.029)
360 (6.0)	2.0 (0.045)	9.3 (0.148)	14.1 (0.188)	18.8 (0.213)	21.8 (0.207)	24.1 (0.203)
720 (12.0)	1.2 (0.020)	6.1 (0.072)	9.3 (0.091)	12.4 (0.102)	17.2 (0.119)	20.8 (0.127)
1080 (18.0)	0.2 (0.003)	6.3 (0.062)	10.3 (0.083)	14.2 (0.096)	15.6 (0.088)	16.6 (0.083)
1440 (24.0)	0.0 (0.000)	2.2 (0.019)	3.6 (0.026)	5.0 (0.030)	8.0 (0.040)	10.2 (0.045)
2160 (36.0)	0.0 (0.000)	2.0 (0.015)	3.4 (0.020)	4.6 (0.023)	5.4 (0.022)	5.9 (0.022)
2880 (48.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.2 (0.001)	0.4 (0.001)
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

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Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

Interim Climate Change Factors

Values are of the format temperature increase in degrees Celcius (% increase in rainfall)

	RCP 4.5	RCP6	**RCP 8.5**
2030	**0.892 (4.5%)**	0.775 (3.9%)	**0.979 (4.9%)**
2040	**1.121 (5.6%)**	1.002 (5.0%)	**1.351 (6.8%)**
2050	**1.334 (6.7%)**	1.28 (6.4%)	**1.765 (8.8%)**
2060	**1.522 (7.6%)**	1.527 (7.6%)	**2.23 (11.2%)**
2070	**1.659 (8.3%)**	1.745 (8.7%)	**2.741 (13.7%)**
2080	**1.78 (8.9%)**	1.999 (10.0%)	**3.249 (16.2%)**
2090	**1.825 (9.1%)**	2.271 (11.4%)	**3.727 (18.6%)**

Layer Info

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Version	2016_v1
Note	ARR recommends the use of RCP4.5 and RCP 8.5 values