PENRITH CITY COUNCIL

PENRITH CITY COUNCIL

CRANEBROOK OVERLAND FLOW FLOOD STUDY

OCTOBER 2022

DRAFT FOR PUBLIC EXHIBITION

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FOREWORD

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

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

The Policy provides for technical and financial support by the Government through the following four sequential stages:

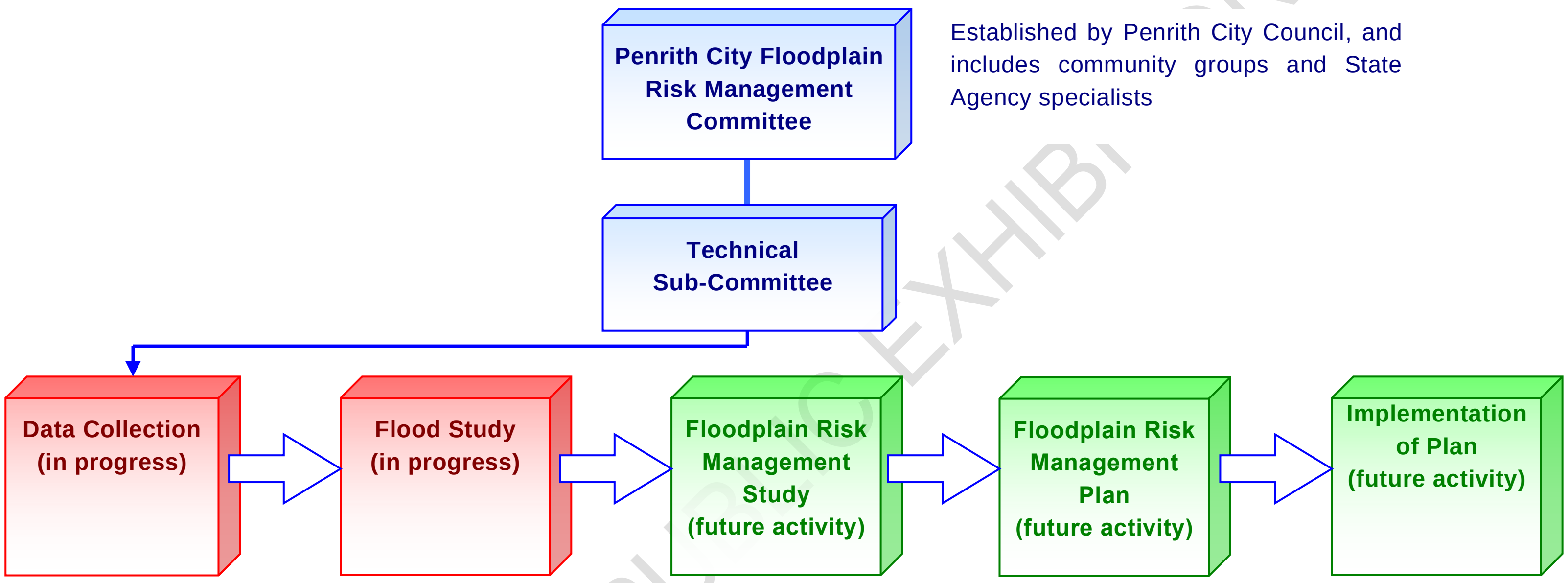
1.	Flood Study	Determines the nature and extent of flooding.
2.	Floodplain Risk Management Study	Evaluates management options for the floodplain in respect of both existing and proposed development.
3.	Floodplain Risk Management Plan	Involves formal adoption by Council of a plan of management for the floodplain.
4.	Implementation of the Plan	Construction of flood mitigation works to protect existing development. Use of Local Environmental Plans to ensure new development is compatible with the flood hazard. Improvements to flood emergency management measures.

The *Cranebrook Overland Flow Flood Study* is jointly funded by Penrith City Council and the NSW Government, via the Department of Planning and Environment. The Flood Study constitutes the first and second stage of the Floodplain Risk Management process (refer over) for this area and has been prepared for Penrith City Council to define flood behaviour under current conditions.

ACKNOWLEDGEMENT

Penrith City Council has prepared this document with financial assistance from the NSW Government through its Floodplain Management Program. This document does not necessarily represent the opinions of the NSW Government or the Department of Planning and Environment.

FLOODPLAIN RISK MANAGEMENT PROCESS



Involves the compilation of existing data and the collection of additional data.

Involves detailed hydrologic and hydraulic modelling of the existing stormwater drainage system at Cranebrook.

The Floodplain Risk Management Study will determine options which will seek to reduce the impact of flooding on the community in consideration of social, ecological and economic factors.

Preferred floodplain management options will be publicly exhibited and the responses from the community incorporated in the Plan. The Plan will then be formally approved by Council following the public exhibition period.

Implementation of the Plan will allow Council to reduce the impact of flooding on the community through flood, property, and response modification measures. The measures may include structural works, planning controls, flood warnings, flood readiness and response plans, ongoing data collection and monitoring.

TABLE OF CONTENTS

Page No.

SUMMARY S1

1 INTRODUCTION 1

1.1 Study Background 1
1.2 Community Consultation and Available Data 1
1.3 Previous Investigations 2
1.4 Layout of Report 2

2 BACKGROUND INFORMATION 4

2.1 Existing Environment 4
2.1.1. General 4
2.1.2. Study Catchments 4
2.1.3. Existing Drainage System 5
2.1.4. Penrith Lakes 5
2.2 Flood History and Analysis of Historic Rainfall 8
2.2.1. General 8
2.2.2. February 2012 Storm Event 9
2.2.3. January 2016 Storm Event 9
2.2.4. February 2020 Storm Event 10

3 HYDROLOGIC MODEL DEVELOPMENT AND CALIBRATION 13

3.1 Hydrologic Modelling Approach 13
3.2 Hydrologic Model Layout 13
3.3 Hydrologic Model Testing 14
3.3.1. General 14
3.3.1. Application of Historic Rainfall to the Hydrologic Model 14
3.3.2. Hydrologic Model Parameters 14
3.3.3. Results of Model Testing 16

4 HYDRAULIC MODEL DEVELOPMENT AND CALIBRATION 17

4.1 General 17
4.2 The TUFLOW Modelling Approach 17
4.3 TUFLOW Model Setup 17
4.3.1. Model Structure 17
4.3.2. Two-dimensional Model Domain 18
4.3.3. One-dimensional Model Elements 18
4.3.4. Model Parameters 19
4.4 Model Boundary Conditions 21
4.5 Hydraulic Model Calibration 22
4.5.1. General 22
4.5.2. Results of Model Testing 22
4.5.3. Summary 22

Cont'd Over

TABLE OF CONTENTS (Cont'd)

Page No.

5 DERIVATION OF DESIGN FLOOD HYDROGRAPHS 31

5.1 Design Storms 31
5.1.1. Rainfall Intensity 31
5.1.2. Areal Reduction Factors 31
5.1.3. Temporal Patterns 31
5.1.4. Probable Maximum Precipitation 31
5.2 Design Rainfall Losses 32
5.3 Derivation of Design Discharges 32

6 HYDRAULIC MODELLING OF DESIGN FLOOD EVENTS 33

6.1 Hydraulic Model Structure 33
6.1.1. Partial Blockage of Hydraulic Structures 33
6.1.2. Boundary Conditions 33
6.2 Presentation and Discussion of Results 34
6.2.1. Accuracy of Hydraulic Modelling 34
6.2.2. Critical Duration and Temporal Pattern Assessment 34
6.2.3. Design Flood Extents 35
6.2.4. Description of Local Catchment Flood Behaviour 37
6.2.5. Potential Flood Mitigation Measures at Identified Hot Spots 43
6.3 Flood Hazard Zones and Floodways 48
6.3.1. Flood Hazard Vulnerability Classification 48
6.3.2. Hydraulic Categorisation of the Floodplain 49
6.4 Flood Emergency Response Classification 51
6.5 Sensitivity Studies 51
6.5.1. General 51
6.5.2. Sensitivity to Rainfall Losses 52
6.5.3. Sensitivity to Hydraulic Roughness 52
6.5.4. Sensitivity to Partial Blockage 52
6.5.5. Sensitivity to Downstream Tailwater Level Conditions 53
6.5.6. Differences in Design Flood Estimation – ARR 1987 versus ARR 2019 53
6.5.7. Sensitivity to Cumulative Development 55
6.6 Climate Change Sensitivity Analysis 55
6.6.1. General 55
6.6.2. Sensitivity to Increased Rainfall Intensities 56
6.7 Flood Planning Information 57
6.7.1. Modelling of Coincident Flooding on the Nepean River 57
6.7.2. Selection of Flood Planning Levels 58

7 REFERENCES 59

8 FLOOD-RELATED TERMINOLOGY 60

APPENDICES

A. Information Sheet and Community Questionnaire

B. Details of Available Data and Community Consultation

C. Photographs Showing Observed Flood Behaviour in Study Area

D. Design Input Data from ARR Data Hub

E. ARR, 2019 Design Blockage Assessment at Cross Drainage Structures

F. Sensitivity Analyses Maps (Bound in Volume 2)

LIST OF MAIN REPORT FIGURES
(BOUND IN VOLUME 2)

1.1 Study Location Plan

2.1 Catchment Plan
2.2 Existing Stormwater Drainage System (3 Sheets)
2.3 Intensity-Frequency-Duration Curves and Historic Rainfall (2 Sheets)
2.4 Cumulative Rainfall – Historic Storms

3.1 Hydrologic Model Layout (3 sheets)
3.2 Application of Historic Rainfall to Hydrologic Model

4.1 TUFLOW Model Layout (3 sheets)
4.2 Hydraulic Roughness Values
4.3 TUFLOW Schematisation of Floodplain
4.4 TUFLOW Model Results – 10 February 2012 Storm Event (3 Sheets)
4.5 TUFLOW Model Results – 30 January 2016 Storm Event (3 Sheets)
4.6 TUFLOW Model Results – 9 February 2020 Storm Event (3 Sheets)

5.1 Extent of PMP Ellipses

6.1 TUFLOW Model Results – Local Catchment Flooding Only – 0.5 EY(3 Sheets)
6.2 TUFLOW Model Results – Local Catchment Flooding Only – 20% AEP (3 Sheets)
6.3 TUFLOW Model Results – Local Catchment Flooding Only – 10% AEP (3 Sheets)
6.4 TUFLOW Model Results – Local Catchment Flooding Only – 5% AEP (3 Sheets)
6.5 TUFLOW Model Results – Local Catchment Flooding Only – 2% AEP (3 Sheets)
6.6 TUFLOW Model Results – Local Catchment Flooding Only – 1% AEP (3 Sheets)
6.7 TUFLOW Model Results – Local Catchment Flooding Only – 0.5% AEP (3 Sheets)
6.8 TUFLOW Model Results – Local Catchment Flooding Only – 0.2% AEP (3 Sheets)
6.9 TUFLOW Model Results – Local Catchment Flooding Only – PMF (3 Sheets)
6.10 Capacity of Existing Stormwater Drainage System – Local Catchment Flooding Only (3 Sheets)
6.11 Design Stage Hydrographs
6.12 Flood Hazard Vulnerability Classification – Local Catchment Flooding Only – 5% AEP (3 Sheets)
6.13 Flood Hazard Vulnerability Classification – Local Catchment Flooding Only – 1% AEP (3 Sheets)
6.14 Flood Hazard Vulnerability Classification – Local Catchment Flooding Only – 0.5% AEP (3 Sheets)
6.15 Flood Hazard Vulnerability Classification – Local Catchment Flooding Only – PMF (3 Sheets)

Continued over

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

6.16 Hydraulic Categorisation of Floodplain – Local Catchment Flooding Only – 5% AEP (3 Sheets)

6.17 Hydraulic Categorisation of Floodplain – Local Catchment Flooding Only – 1% AEP (3 Sheets)

6.18 Hydraulic Categorisation of Floodplain – Local Catchment Flooding Only – 0.5% AEP (3 Sheets)

6.19 Hydraulic Categorisation of Floodplain – Local Catchment Flooding Only – PMF (3 Sheets)

6.20 Flood Emergency Response Classification – Local Catchment Flooding Only – 20% AEP (3 Sheets)

6.21 Flood Emergency Response Classification – Local Catchment Flooding Only – 5% AEP (3 Sheets)

6.22 Flood Emergency Response Classification – Local Catchment Flooding Only – 1% AEP (3 Sheets)

6.23 Flood Emergency Response Classification – Local Catchment Flooding Only – 0.5% AEP (3 Sheets)

6.24 Flood Emergency Response Classification – Local Catchment Flooding Only – PMF (3 Sheets)

6.25 Stage Hydrographs Extracted from Nepean River Flood Study

6.26 TUFLOW Model Results – Envelope of Local Catchment and Nepean River Flooding – 1% AEP (3 Sheets)

6.27 Peak Flow Velocities – Envelope of Local Catchment and Nepean River Flooding – 1% AEP (3 Sheets)

6.28 Flood Hazard Vulnerability Classification – Envelope of Local Catchment and Nepean River Flooding – 1% AEP (3 Sheets)

6.29 Hydraulic Categorisation of Floodplain – Envelope of Local Catchment and Nepean River Flooding – 1% AEP (3 Sheets)

6.30 Difference Between Envelope of Nepean River and Local Catchment Flooding and Local Catchment Flooding Only – 1% AEP (3 Sheets)

6.31 Flood Planning Area (3 Sheets)

NOTE ON FLOOD FREQUENCY

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

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

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

NOTE ON QUOTED LEVEL OF ACCURACY

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

ABBREVIATIONS

AEP	Annual Exceedance Probability (%)
AHD	Australian Height Datum
AMC	Antecedent Moisture Condition
ARF	Areal Reduction Factor
ARI	Average Recurrence Interval (years)
ARR	Australian Rainfall and Runoff (Geoscience Australia, 2019)
AWS	All Weather Station
BoM	Bureau of Meteorology
Council	Penrith City Council
DEM	Digital Elevation Model
DPE	Department of Planning and Environment
EY	Exceedances per Year
FDM	Floodplain Development Manual (NSW Government, 2005)
FPL	Flood Planning Level
FPA	Flood Planning Area
FRMS&P	Floodplain Risk Management Study and Plan
GDSM	Generalised Short Duration Method
GS	Gauging Station
IFD	Intensity-Frequency-Duration
LiDAR	Light Detecting and Ranging (type of aerial based survey)
NSW SES	New South Wales State Emergency Service
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
TUFLOW	A true two-dimensional hydrodynamic computer model which has been used to define flooding patterns as part of the present study.

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

SUMMARY

S.1 Study Objective

The primary objective of the *Flood Study* was to define the nature of local catchment flooding in the suburbs of Cranebrook, Penrith and Cambridge Gardens in the Penrith City Council (**Council**) Local Government Area (**LGA**) for flood frequencies ranging between 0.5 Exceedances per Year (**EY**) and 0.2 per cent Annual Exceedance Probability (**AEP**), as well as for the Probable Maximum Flood (**PMF**). The definition of Nepean River flooding in the study area for a flood with an AEP of 1% was also refined as part of the present study.

The findings of the *Flood Study* will be used as the basis for preparing the future *Floodplain Risk Management Study and Plan* (***FRMS&P***) which will assess options for flood mitigation and prepare a plan of works and measures for managing the existing, future and continuing flood risk in the study area.

S.2 Background Information

The study area generally comprises residential, commercial and industrial type land that drains in a westerly direction toward the Nepean River north of the Penrith Central Business District (**CBD**). **Figure 1.1** shows the extent of the 13.3 km^2 study area which encompasses parts of Cranebrook, Penrith and Cambridge Gardens and is generally bounded by the Nepean River and Penrith Lakes to the west, Cranebrook Road to the north, The Northern Road and Parker Street to the east and the Main Western Railway to the south.

S.3 Study Method

The flood study involved the following activities:

- The forwarding of a *Community Newsletter and Questionnaire* to approximately 7,800 residents and business owners in the study area. The *Community Newsletter and Questionnaire*, a copy of which is contained in **Appendix A** of this report, introduced the study objectives and sought information on historic flood behaviour. Of the 472 responses that were received, about 20% noted that they had observed flooding in or adjacent to their property. The respondents provided information on flooding that occurred in the following months:
    - March 1978;
    - January 1983;
    - November 1985;
    - August 1986;
    - July 1988;
    - August 1990;
    - 1995 (month not specified);
    - March 2005;
    - February 2012;
    - January 2016; and
    - February 2020.
- The collection of flood data, details of which are set out in **Appendix B** of this report. Pluviographic rainfall data recorded by a series of Bureau of Meteorology and Sydney Water operated rain gauges in the vicinity of the study catchment were obtained. A number of photographs were provided by respondents to the *Community Newsletter and Questionnaire* showing flood behaviour in the study area, copies of which are contained in **Appendix C** of this report.

- The hydrologic modelling of the study catchment. The RAFTS modelling approach in the DRAINS software was used to simulate the hydrologic response of the predominately rural parts of the study catchments, while the IL-CL modelling approach in DRAINS was used to stimulate the hydrologic response of the urbanised parts of the study catchments. The software generated discharge hydrographs resulting from both historic and design storms.
- Application of the discharge hydrographs to a hydraulic model comprising the main arm of Boundary Creek, its tributaries and the existing piped stormwater drainage system in the study area. The TUFLOW two-dimensional modelling system was adopted for the hydraulic analysis.
- Presentation of study results as diagrams showing indicative extents and depths of inundation, flood hazard vulnerability and the hydraulic categorisation of the floodplain into floodway, flood storage and flood fringe areas.
- Sensitivity studies to assess the effects on flood behaviour resulting from variations in model parameters such as rainfall losses, hydraulic roughness of the floodplain, the effects of a partial blockage of hydraulic structures and increases in rainfall intensity associated with future climate change.

After testing the models for the February 2012, January 2016 and February 2020 storm events, design storm rainfalls ranging between 0.5 EY and 0.2% AEP were derived using procedures set out in the 2019 edition of *Australian Rainfall and Runoff* (Editors, 2019) (**ARR 2019**) and applied to the hydrologic models to determine discharge hydrographs. The PMF was also modelled.

S.4 Design Flood Estimation

Figures 6.1 to **6.9** (3 sheets each) show the TUFLOW model results for the 0.5 EY, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP floods, together with the PMF. These diagrams show the indicative extent and depth of inundation for the full range of design storm events throughout the study area.

Figure 6.10 (3 sheets) shows the AEP of the local catchment storm event which results in individual pipes first flowing full, while **Figure 6.11** shows stage hydrographs at selected road crossings throughout the study area.

Flooding patterns derived by TUFLOW for the design storm events are described in **Chapter 6** of this report.

S.5 Flood Hazard Classification and Hydraulic Categorisation

Diagrams showing the flood hazard vulnerability classification for the 5%, 1%, and 0.5% AEP flood events, as well as the PMF are shown on **Figures 6.12**, **6.13**, **6.14** and **6.15**, respectively, while the hydraulic categorisation of the floodplain for the 5%, 1%, and 0.5% AEP flood events, as well as the PMF are shown on **Figures 6.16**, **6.17**, **6.18** and **6.19**, respectively.

The flood hazard vulnerability classification is dependent on the depth and velocity of flow in the channels and the floodplains. The floodplain has been divided into six hazard categories areas on the basis of these two variables based on the relationships set out in ARR 2019.

The study found that at the 1% AEP level of flooding areas classified as either H5 or H6 are generally limited to the inbank areas of the major watercourses and man-made farm dams and lakes that are scattered through the study catchments, while the major overland flow paths that are located within urbanised areas are generally classified as either H1 or H2. The exception to the latter is in areas where floodwater ponds on the upstream side of road formations or buildings that block overland flow paths, where the resultant flooding is generally classified as either H3.

The hydraulic categorisation requires the assessment of the main flow paths. Those areas of the floodplain where a significant discharge of water occurs during floods are denoted Floodways and are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flow or a significant increase in flood levels. The remainder of the floodplain is denoted *Flood Storage* or *Flood Fringe* areas.

As the hydraulic capacity of the creek channels and piped drainage system is not large enough to convey the 1% AEP flow, a significant portion of the total flow is conveyed overland. As a result, areas which lie on the overbank area also function as a floodway during the 1% AEP flood event. Floodways are also generally present along the major overland flow paths, while the majority of the remainder of the floodplain is designated as flood storage areas.

S.6 Flood Emergency Response Classification

Diagrams showing the flood emergency response for the 20%, 5%, 1% and 0.5% AEP flood events, as well as the PMF based on the procedures set out in AIDR, 2017 are presented on **Figures 6.20**, **6.21**, **6.22**, **6.23** and **6.24**, respectively. The flood emergency response classifications are based on whether or not an area is flooded, whether the flooded area has an exit to flood-free land and the consequence of flooding on the area for a given AEP storm event.

S.7 Sensitivity Analyses

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

a. An increase or decrease in rainfall losses (refer **Section 6.5.2** for details).

b. An increase or decrease in hydraulic roughness (refer **Section 6.5.3** for details).

c. A partial blockage of major hydraulic structures by debris (refer **Section 6.5.4** for details).

d. Application of lower tailwater levels to the hydraulic model (refer **Section 6.5.5** for details).

e. Adoption of the design rainfall data based on the procedures set out in the 1987 edition of *Australian Rainfall and Runoff* (**ARR 1987**) (The Institution of Engineers Australia, 1987) (refer **Section 6.5.6** for details).

f. Increased infill development in the study catchment (refer **Section 6.5.7** for details).

g. Increases in rainfall intensity associated with future climate change (refer **Section 6.6** for details).

The sensitivity analyses identified that:

- an increase in rainfall losses decreases peak flood levels generally by up to 20 mm while a decrease in rainfall losses would increase peak flood levels generally by up to 30 mm;

- a 20% increase in hydraulic roughness values would generally increase peak flood levels by up to 50 mm, while a 20% decrease in hydraulic roughness values would generally lower peak flood levels by up to 30 mm;
- a partial blockage of the stormwater drainage system results in increases in peak flood levels of up to 200 mm on the upstream side of road crossings and in the regional type flood detention basins;
- adopting lower tailwater levels in Penrith Lakes has a negligible impact on flood behaviour in the study area;
- peak flood levels derived using the procedures set out in ARR 1987 are generally higher than those derived using ARR 2019;
- infill development within the study area will generally increase peak flood levels by less than 20 mm; and
- an increase in the intensity of rainfall associated with future climate change has the potential to increase peak 1% AEP flood levels by a maximum of about 380 mm.

S.8 Flood Planning Information

The structure of the hydraulic model that was developed as part of the present study was updated so as to more accurately define the nature of flooding that would result in the study area from a 1% AEP flood on the Nepean River.

Figure 6.26 (3 sheets) shows the 1% AEP Nepean River and local catchment flood envelope in the study area, which comprises a combination of the following flooding scenarios:

- 5% AEP local catchment storm coincident with a 1% AEP Nepean River flood;
- 1% AEP local catchment storm coincident with a 5% AEP Nepean River flood.

Figure 6.30 (3 sheets) shows that peak flood levels in a 1% AEP Nepean River flood coincident with a 5% AEP local catchment flood are a maximum of about 2 m higher than peak flood levels in a 1% AEP local catchment flood event that is coincident with a 5% AEP Nepean River flood.

Figure 6.31 shows the extent of the Flood Planning Area (**FPA**) which was derived by adding 500 mm freeboard to the 1% AEP Nepean River and local catchment flood envelope. In areas that lie within the extent of the FPA it is recommended that a freeboard of 500 mm be applied to peak 1% AEP flood levels when setting the minimum floor level of future development. An assessment should also be undertaken by Council as part of any future Development Application to confirm that the proposed development will not form an obstruction to the passage of overland flow through the subject site.

1 INTRODUCTION

1.1 Study Background

This report presents the findings of an investigation of flooding in the suburbs of Cranebrook and Penrith in the Penrith City Council (**Council**) Local Government Area (**LGA**). The study has been commissioned by Council with financial support from the NSW Government, via the Department of Planning and Environment (**DPE**). **Figure 1.1** shows the extent of the study area.

The study objective was to define flood behaviour under existing catchment conditions in terms of flows, water levels and velocities for floods ranging between 0.5 Exceedances per Year (**EY**) and 0.2 per cent Annual Exceedance Probability (**AEP**), as well as for the Probable Maximum Flood (**PMF**). The investigation involved rainfall-runoff hydrologic modelling of the catchments to assess flows in the drainage systems of the study catchment, and application of these flows to a hydraulic model to assess peak water levels and flow velocities. The model results were interpreted to present a detailed picture of flooding under present day conditions.

The scope of the study included the investigation of main stream flood behaviour along Boundary Creek and its associated tributaries, as well as major overland flow which occurs during periods of heavy rain.[1]

The study forms the first and second step in the floodplain risk management process for the study area (refer process diagram presented in the Foreword) and is a precursor of the future *Floodplain Risk Management Study and Plan* (***FRMS&P***) which will consider measures which are aimed at reducing the existing, future and continuing flood risk in the study area.

1.2 Community Consultation and Available Data

To assist with data collection and promotion of the study to the community, an *Information Sheet* and *Community Questionnaire* were distributed by Council in September and December 2020 to residents and business owners in the study area. A copy of the *Information Sheet* and *Community Questionnaire* which were prepared by the Consultants is contained in **Appendix A** of this report, while the responses to the *Community Questionnaire* are summarised in **Appendix B**.

Council advised that approximately 7800 *Information Sheets* and *Community Questionnaires* were distributed to the residents and business owners in the study area. A total of 472 responses were received by the closing date of submissions (a response rate of about six per cent).

Of those that responded, about 20% noted that they had observed flooding in or adjacent to their property. The respondents provided information on flooding that occurred in the following months:

- March 1978;
- January 1983;
- November 1985;
- August 1986;
- July 1988;
- August 1990;
- 1995 (month not specified);
- March 2005;
- February 2012;
- January 2016; and
- February 2020.

[1] Note that the scope of the study does not include the definition of main stream flood behaviour along the Nepean River which has been defined as part of the *Nepean River Flood Study* (Advisian, 2018).

Information on historic flooding patterns obtained from the responses assisted with "ground-truthing" the results of the hydraulic modelling.

Appendix C contains several photos which show flood behaviour in the study area during storms that occurred on 9 February 2012, January 2016 (day not specified), 21 March 2017, 7 February 2020 and 9 February 2020.

1.3 Previous Investigations

The following flooding investigations have been undertaken in the immediate vicinity of the study area:

- *Cranebrook Local Hydraulics Specification Study (*Bewsher Consulting, 2002*)*
- *Boundary Creek Erosion Site Investigation* (Patterson Britton & Partners, 2006)
- *Penrith Overland Flow Flood "Overview Study"* (Cardno Lawson Treloar, 2006)
- *Penrith Lakes 2012 Water Management Plan: Stage 1* (Penrith Lakes Development Corporation (**PLDC**), 2012)
- *Penrith CBD Detailed Overland Flow Flood Study* (Cardno, 2015)
- *Nepean River Flood Study* (Advisian, 2018)
- *Peach Tree and Lower Surveyors Creek Flood Study* (Catchment Simulations Solutions (**CSS**), 2019)
- *Penrith Lakes Water Management Plan: Stage 2* (PLDC, 2020)
- *Penrith CBD Floodplain Risk Management Study and Plan* (Molino Stewart, 2020)
- *Emu Plain Overland Flow Flood Study* (BMT, 2020)

Section B1.1 of **Appendix B** contains a summary of the above studies.

1.4 Layout of Report

Chapter 2 contains background information including a brief description of the study catchment and its drainage systems, a brief history of flooding and an analysis of the available rain gauge record.

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

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

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

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

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

The following appendices are included in the report:

- **Appendix A**, which contains a copy of the *Information Sheet* and *Community Questionnaire* that were distributed at the commencement of the study to residents and business owners in the study area.
- **Appendix B**, which contains a list of data that were available for the present study, as well as a summary of the responses to the *Community Questionnaire*.
- **Appendix C** contains photographs showing flood behaviour in the study area during storms that occurred on 9 February 2012, January 2016 (day not specified), 21 March 2017, 7 February 2020 and 9 February 2020.
- **Appendix D** contains a copy of the design input data that were extracted from the *Australian Rainfall and Runoff* (**ARR**) *Data Hub* for the study area.
- **Appendix E** contains a copy of Generalised Short Duration Method (**GDSM**) calculation sheet which was used to drive the Probable Maximum Precipitation (**PMP**) for input to the hydrologic model.
- **Appendix F** contains a series of figures that show the results of the sensitivity analyses that were undertaken as part of the present study.

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

2 BACKGROUND INFORMATION

2.1 Existing Environment

2.1.1. General

The study area generally comprises residential, commercial and industrial type land that drains in a westerly direction toward the Nepean River north of the Penrith Central Business District (**CBD**). **Figure 1.1** shows the extent of the 13.3 km^2 study area which encompasses parts of Cranebrook, Penrith and Cambridge Gardens.

2.1.2. Study Catchments

The study area has been divided into seven study catchments, the extents of which are shown on **Figure 2.1**. Reference is made to several key features of Penrith Lakes in the following catchment description, further details of which are set out in **Section 2.1.4** of this report.

- The **Duralia Lake Catchment**, which predominantly comprises large lot residential type development and drains in a westerly direction, discharging to Cranebrook Lake where it is then conveyed to Duralia Lake via a submerged pipe beneath Castlereagh Road, the approximate alignment of which is shown on **Figure 2.1**. During significant rainfall events, runoff bypasses Cranebrook Lake and discharges directly to Duralia Lake via a series of piped crossings beneath Castlereagh Road in the vicinity of its intersection with Cranebrook Road. The total area draining to Duralia Lake is 1.53 km^2.

- **Boundary Road Catchment**, which comprises large lot residential type development that is located on the northern side of Boundary Road and low density residential type development to its south. The catchment drains in a westerly direction through a series of cascading detention basins and discharges to the Stilling Basin in the vicinity of Cranebrook Park. The total catchment area of the Boundary Road Catchment is 2.05 km^2.

- **Cranebrook Road South Catchment**, the eastern limit of which runs in a north-south direction to the west of The Northern Road, generally drains in a southerly direction through a network of cascading basins between Nereid Road and Laycock Street before discharging to the lakes system within the recently constructed Waterside development which is bounded by Castlereagh Road to the west, Nepean Street to the north, Laycock Street to the east and Lakeview Drive to the south (**Waterside Lakes**). Runoff then flows in a northerly direction to the Stilling Basin. The catchment generally comprises low and medium density residential type development to the east of Laycock Street and general residential type development to its west. The total catchment area of the Cranebrook Road South Catchment is 2.95 km^2.

- **Andrews Road Catchment**, which drains in a north-westerly direction between the intersection of Coreen Avenue, Parkes Street and Andrews Road where it discharges to the aforementioned Waterside Lakes. The eastern portion of the catchment comprises low density residential type development while the western portion comprises general industrial type development. The total catchment area of the Andrews Road Catchment is 3.12 km^2.

- **Boundary Creek Catchment**, which generally drains in a westerly direction between Parkes Street and the Nepean River. The catchment has a total catchment area of 2.45 km^2 and comprises a mix of residential and industrial type development.

- **Penrith Lakes Local Catchment** comprises general industrial type development and discharges to the Middle and Southern Basins within the Penrith Lakes. The total catchment area of the Penrith Lakes Local Catchment is 0.38 km^2.
- **Local Nepean River Catchment** comprises general industrial type development and discharges to the Nepean River immediately north of its confluence with Boundary Creek. The total catchment area of the Penrith Lakes Local Catchment is 0.34 km^2
- The **North Penrith Catchment**, the total catchment area of which is 0.4 km^2 generally comprises high density residential type development and drains in a southerly direction, discharging to the Penrith CBD catchment at three locations via pipes beneath the Main Western Railway.

2.1.3. Existing Drainage System

Figure 2.2 (3 sheets) shows the layout of the existing drainage system in the study area. The study catchments are highly urbanised and the natural drainage characteristics have been almost completely altered by development, with the natural drainage paths having been piped where they run through the urbanised areas.

Figure 2.2, sheet 3 shows the alignment of Boundary Creek which runs in a westerly direction from the intersection of Coombes Drive and Hickeys Lane and discharges to the Nepean River about 150 m downstream of Penrith Weir. The 180 m reach of Boundary Creek immediately downstream of Hickeys Lane comprises a 4 m wide concrete lined channel, after which it comprises an engineered vegetated channel with a base width of 2 m for a further 1000 m until it reaches Castlereagh Road. Downstream of Castlereagh Road the watercourse is generally in a more natural state ranging from about 3 m deep in the vicinity of Castlereagh Road to over 10 m deep at its confluence with the Nepean River.

Figure 2.2 (3 sheets) shows the plan location of 24 regional type detention basins that are located in the study area, the details of which are contained in **Table 2.1** over the page. *[Note also that numerous privately owned and maintained on-site stormwater detention systems have been constructed throughout the study area to control stormwater at the allotment level, the location and details of which are not known.]*

2.1.4. Penrith Lakes

Figure 1.1 shows the extent of Penrith Lakes where it borders the study area. Penrith Lakes comprises approximately 2,000 ha of rehabilitated quarries that have been converted to recreational parkland, lakes and nature reserves.

Penrith Lakes was designed to achieve an equivalent or improved flood impact on the Nepean River and surrounding areas in a 1% AEP Nepean flood event. The *Penrith Lakes 2012 Water Management Plan: Stage 1* (PLDC, 2012) (a summary of which is contained in **Section B1.1.4** of **Appendix B**) and the *Penrith Lakes Water Management Plan: Stage 2* (PLDC, 2020) (a summary of which is contained in **Section B1.1.8** of **Appendix B**) set out the operational and water management requirements of the completed Penrith Lakes.

TABLE 2.1
DETAILS OF EXISTING REGIONAL FLOOD DETENTION BASINS AT CRANEBROOK

Basin ID[1]	Basin Name	Approximate Year of Construction[3]	Outlet Structure[2]		Spillway Elevation (m AHD)
			Dimensions (mm)	Invert Level (m AHD)	
B01	Tornado Crescent Basin No. 1	1984	1 off 750 RCP	34.3	37.15
B02	Tornado Crescent Basin No. 2	1984	1 off 600 RCP	32.26	34.48
B03	Hanlan Street Basin	1984	1 off 1,800 RCP	28.65	33.27
B04	Soling Crescent Basin No. 1	1984	1 off 600 RCP	27.38	30.75
B05	Soling Crescent Basin No. 2	1984	1 off 600 RCP	25.91	28.01
B06	Borrowdale Way Basin No. 1	1978-1986	1 off 450 RCP	39.89	42.63
B07	Borrowdale Way Basin No. 2	1978-1986	1 off 450 RCP	39.19	41.58
B08	Sherringham Road Basin No. 1	1978-1986	1 off 450 RCP	36.86	39.06
B09	Sherringham Road Basin No. 2	1978-1986	1 off 600 RCP	34.89	38.23
B10	McHenry Road Basin No. 1	1978-1986	1 off 600 RCP	32.63	35.64
B11	McHenry Road Basin No. 2	1978-1986	1 off 1,050 RCP	29.55	32.92
B12	McHenry Road Basin No. 3	1978-1986	1 off 1,350 RCP	28.12	31.19
B13	McHenry Road Basin 6 No.	1978-1986	1 off 900 RCP	26.64	30.13
B14	McHenry Road Basin No. 4	1978-1986	1 off 450 RCP	29.76	32.96
B15	McHenry Road Basin No. 5	1978-1986	1 off 450 RCP	28.64	31.17
B16	Laycock Street Basin No. 1	1978-1986	1 off 900 RCP	24.55	27.21
B17	Laycock Street Basin No. 2	1978-1986	1 off 900 RCP	23.56	26.2
B18	Cooper Street Basin	1978-1986	1 off 750 RCP 1 off 3,050 x 2,100 RCBC	29.35 29.96	33.26
B19	Andrews Road Basin No. 1	1978-1986	1 off 750 RCP	25.81	28.94
B20	Andrews Road Basin No. 2	1978-1986	1 off 750 RCP 1 off 1,050 RCP	24.11 24.88	27.13
B21	King Street Basin No. 1	1975-1978	1 off 750 RCP	45.99	49.1
B22	King Street Basin No. 2	1975-1978	1 off 600 RCP	40.86	44.09
B23	King Street Basin No. 3	1975-1978	1 off 600 RCP	37.39	40.84
B24	Coreen Avenue Basin	1975-1978	1 off 525 RCP	34.89	37.65

1. Refer **Figure 2.2**, 3 sheets for location
2. RCP = reinforced concrete pipe, RCBC = reinforced concrete box culvert.
3. Dates based on available aerial imagery on NSW Government *Historical Imagery* database.

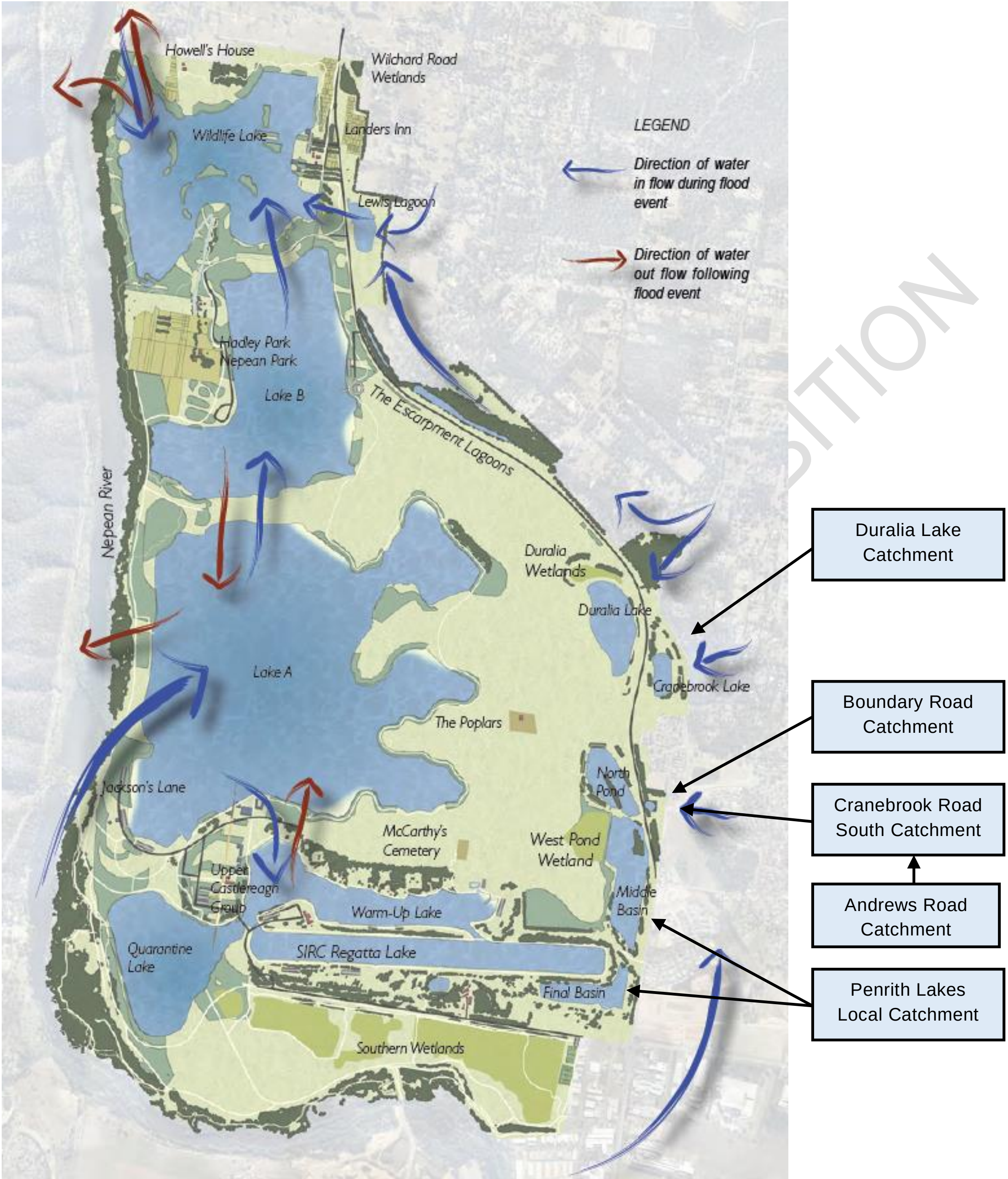
Figure 1.1 shows the location and extent of the thirteen permanent water bodies that comprise Penrith Lakes, while **Table 2.2** below sets out the operating water level and outlet structure of each. **Figure 2.1** shows the approximate location and alignment of the outlet structures, while the illustration over shows the direction floodwater takes through Penrith Lakes during a flood event.

Runoff from the Duralia Lake Catchment initially discharges to Cranebrook Lake before flowing in a westerly direction to Duralia Lake. The runoff then flows in a southerly direction to the North Pond where it is joined by runoff from the Andrews Road, Cranebrook Road South and Boundary Road catchments before again discharging in a southerly direction to the Middle Basin and then the Final Basin. Runoff from the Penrith Lakes Local Catchments discharges directly to both the Middle and Final Basins. From the Final Basin, flow then discharges in a northerly direction through the Regatta Lake, Lake A, Lake B and the Wildlife Lake before discharging to the Nepean River.

TABLE 2.2
DETAILS OF PERMANENT WATER BODIES COMPRISING PENRITH LAKES

Lake Name	Operating Level[1] (m AHD)	Maximum Tolerable Water Level[2] (m AHD)	Outlet Structure[3,4]
Cranebrook Lake	18.0	18.90	900 mm diameter pipe
Duralia Lake	18.0	18.90	900 mm diameter pipe[5]
Stilling Basin	17.7	18.05	3 off 9000 mm wide by 3000 mm high arch culverts
North Pond	16.5	18.05	1200 mm diameter pipe and concrete weir
Middle Basin	16.0	18.05	3000 mm wide by 1800 mm high box culvert
Final Basin	15.5	18.05	3000 mm wide by 1800 mm high box culvert[5]
Regatta Lake	15.0	15.4	1200 mm diameter pipe
Quarantine Lake	15.0	15.4	-[6]
Lake A	14.0	14.5	Open channel
Lake B	13.5	14.5	900 mm diameter pipe[5]
Wildlife Lake	10.0	11.0	Concrete weir

1. Taken from Table 1 of PLDC, 2020 (a copy of which is contained in **Annexure B3**).
2. Based on the maximum water level tolerance set out in Table 1 of PLDC, 2020 (a copy of which is contained in **Annexure B3**).
3. Taken from Figure 11 and Table 8 of PLDC, 2012 (a copy of which is contained in **Annexure B2**).
4. Refer **Figure 2.1** for approximate location and orientation of outlet structure.
5. A sluice gate is installed on the upstream end of the outlet structure to control the water level in the upstream lake.
6. Quarantine Lake is directly connected to Regatta Lake.



Source: PLDC, 2012

2.2 Flood History and Analysis of Historic Rainfall

2.2.1. General

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

Figure 1.1 shows the location of the Bureau of Meteorology (**BoM**) operated All Weather Station (**AWS**) and nearby Sydney Water operated pluviographic rain gauges that are located in the vicinity of the study area. It is noted that the BoM operated Penrith Lakes AWS and Sydney Water operated Cranebrook Reservoir and Penrith WRP pluviographic rain gauges have been relied upon to assess historic storm events that have occurred since 1991, while the decommissioned BoM operated Richmond RAAF pluviographic rain gauge that is located approximately 12 km north of the study area has been used to assess historic storm events prior to 1991.

Figure 2.3 (2 sheets) shows design versus historic intensity-frequency-duration (**IFD**) curves for the two BoM and two Sydney Water operated rain gauges for the storm events identified by the respondents to the *Community Questionnaire*, while **Table 2.3** at the end of this chapter gives the approximate AEP of the recorded rainfall for durations ranging between 0.25 and 6 hours.

Table 2.3 shows that the storm events identified by the respondents to the *Community Questionnaire* that occurred prior to 2005 were less intense than a storm that occurs once every year on average (i.e. less than 1 EY), with the exception of the 6-7 August 1986, 31 July – 2 August 1990 and September 1995 storm events, which where equivalent to design storm events with AEPs of about 5%, 20% and 50%, respectively.

Based on the availability of historic flood data that were obtained from the respondents to the *Community Questionnaire* (refer **Section B2.2.3** of **Appendix B** for discussion), the storm events that occurred in February 2012, January 2016 and February 2020 were selected for use in validating the hydrologic and hydraulic models that were developed as part of the present study. **Figure 2.4** shows the cumulative rainfall that was recorded at the nearby rain gauges for these storm events.

2.2.2. February 2012 Storm Event

A total of 26 respondents to the *Community Questionnaire* indicated that they had experienced flooding during a storm event that occurred in February 2012. **Plates C1.1** and **C1.2** in **Appendix C** show floodwater ponding in the rear of a property that is located in Soling Crescent, Cranebrook at about 19:00 hours on 9 February 2012.

Figure 2.4 shows that flooding occurred after a maximum of 85 mm of rain fell between 17:00 hours and 20:30 hours on 9 February 2012, which was preceded by a maximum of 20 mm which fell between 09:00 hours and 17:00 hours on the same day. **Table 2.3** and **Figure 2.3** show that this event was equivalent to a design storm with an AEP of between about 2% to 5%.

2.2.3. January 2016 Storm Event

While 33 respondents to the *Community Questionnaire* indicated that they were affected by flooding during January 2016, none of the respondent's provided information on the exact date that the event occurred.

A review of the rainfall recorded at the nearby gauges during January 2016 identified that the following storms occurred:

- Between 105 mm and 166 mm of rain fell on the raindays of **4-7 January 2016**. The most intense period of rain was between 05:00 hours and 18:00 hours on 5 January 2016 when between 55 mm and 80 mm fell. **Table 2.3** and **Figure 2.3** shows that the storm burst was equivalent to a design storm with an AEP of about 10% for storm durations longer than 3 hours in the northern parts of the study area in the vicinity of the Cranebrook Reservoir rain gauge, but equivalent to a 20%-50% AEP design storm in the southern portion of the study area.
- The rainday of **15 January 2016**, when between 25 mm and 42 mm of rainfall was recorded at the nearby rain gauges. **Table 2.3** and **Figure 2.3** show that this event was less intense than a storm that occurs once every year on average (i.e. less than 1 EY).
- Between 47 mm and 54 mm of rainfall was recorded at the nearby gauges on **22-23 January 2016**. **Figure 2.4** shows that the rainfall occurred over two separate storm bursts, the first burst occurred between 17:00 hours and 20:00 hours on 21 January 2016 when between 32 mm and 36 mm of rainfall was recorded and the second burst occurred between 14:00 hours and 18:00 hours on 22 January 2016 when about 15 mm of rainfall were recorded. **Table 2.3** and **Figure 2.3** show that these bursts were equivalent to a storm that occurs once every two years on average (i.e. 50% AEP).
- The raindays of **30-31 January 2016** when a total rainfall depth of between 40 mm and 70 mm was recorded at the nearby rain gauges. **Figure 2.4** shows that the recorded rainfall depth was higher in the southern parts of the catchment in the vicinity of the Penrith Lakes AWS and Penrith WRP rain gauges when compared with the rainfall that was recorded at the Cranebrook Reservoir gauge further to the north. **Figure 2.4** also shows that the rain fell over two distinct storm bursts, the largest of which occurred when between 24 mm and 41 mm fell between 14:30 hours and 16:00 hours on 30 January 2016. **Table 2.3** and **Figure 2.3** show that this event was equivalent to a 20-10% AEP design storm event in the southern part of the study area, but only equivalent to a 50-20% AEP storm event in the north.

Based on the above, it is likely that the respondents who indicated that they had experienced flooding during January 2016 were referring to the storm burst that occurred on the afternoon of 30 January 2016, as this was generally more intense for storm durations less than 1 hour in duration which are typically responsible for causing the surcharge of the piped drainage systems in the upper reaches of the study catchments where the majority of the anecdotal information on flood behaviour is available.

2.2.4. February 2020 Storm Event

Based on photographic evidence provided by respondents to the *Community Questionnaire*, flooding occurred at Cranebrook between about 12:00 hours and 15:00 hours on 9 February 2020. **Plates C5.1** and **C5.2** in **Appendix C** show floodwater ponding in Borrowdale Way Basin Nos. 1 and 2, the latter of which is shown to be overtopping at 15:00 hours, while **Plates C5.3** to **C5.6** show floodwater ponding in the streets and in the rear of properties in Woodside Glen and Linden Crescent.

Figure 2.4 shows that flooding occurred after continuous rain that fell on the rain days of 9 and 10 February 2020 where a total rainfall depth of between 169 mm and 206 mm was recorded at the nearby rain gauges. **Table 2.3** and **Figure 2.3** show that this event was generally equivalent to a design storm with an AEP of between 10% and 20%.

TABLE 2.3
APPROXIMATE AEPs OF RECORDED RAINFALL FOR HISTORIC STORM EVENTS[1]
(% AEP)

Storm Event	Rain Gauge[2]	Storm Duration (hours)							
		0.25	0.5	1	1.5	2	3	4.5	6
21-22 March 1978	Richmond RAAF (GS 67033)	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY
26 January 1983	Richmond RAAF (GS 67033)	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY
6-7 August 1986	Richmond RAAF (GS 67033)	50%	20%	20%	5%	5%	5%	5%	5%
6-7 July 1988	Richmond RAAF (GS 67033)	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY
31 July - 2 August 1990	Richmond RAAF (GS 67033)	< 1 EY	< 1 EY	1 EY	50%	20%	20%	50%	< 1 EY
25 September 1995	Cranebrook Reservoir (GS 567159)	< 1 EY	< 1 EY	< 1 EY	< 1 EY	1 EY-50%	50%	50%	50%
	Penrith WRP (GS 567107)	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY
17 March 2005	Cranebrook Reservoir (GS 567159)	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY
	Penrith WRP (GS 567107)	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY
	Penrith Lakes (GS 67113)	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY
9 February 2012	Cranebrook Reservoir (GS 567159)	10%	5%	5%	2%	2%	2%	5%	2%
	Penrith WRP (GS 567107)	20%	20%	5%	5%	5%	5%	5%	5%
	Penrith Lakes (GS 67113)	20%	20%	5%	5%	2%	2%	5-2%	2%
4-7 January 2016	Cranebrook Reservoir (GS 567159)	< 1 EY	< 1 EY	50%	50-20%	20%	20-10%	10%	20-10%
	Penrith WRP (GS 567107)	< 1 EY	< 1 EY	< 1 EY	1 EY-50%	50%	50-20%	20%	50-20%
	Penrith Lakes (GS 67113)	< 1 EY	< 1 EY	< 1 EY	50%	50%	50%	50%	50%

Refer over for footnotes to table.

Cont'd over

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

Storm Event	Rain Gauge(2)	Storm Duration (hours)							
		0.25	0.5	1	1.5	2	3	4.5	6
15 January 2016	Cranebrook Reservoir (GS 567159)	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY
	Penrith WRP (GS 567107)	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY
	Penrith Lakes (GS 67113)	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY
22-23 January 2016	Cranebrook Reservoir (GS 567159)	1 EY	50%	50%	50%	50%	50%	< 1 EY	< 1 EY
	Penrith WRP (GS 567107)	1 EY	1 EY-50%	50%	50%	50%	1 EY	< 1 EY	< 1 EY
	Penrith Lakes (GS 67113)	1 EY	1 EY-50%	50%	50%	50%	1 EY	< 1 EY	< 1 EY
30-31 January 2016	Cranebrook Reservoir (GS 567159)	50-20%	50%	1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY
	Penrith WRP (GS 567107)	20-10%	20%	20%	50-20%	50%	50%	< 1 EY	< 1 EY
	Penrith Lakes (GS 67113)	20%	10%	10%	20%	20%	50-20%	50%	1 EY
17 March 2017	Cranebrook Reservoir (GS 567159)	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY
	Penrith WRP (GS 567107)	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY
	Penrith Lakes (GS 67113)	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY	< 1 EY
9 February 2020	Cranebrook Reservoir (GS 567159)	< 1 EY	20%	20%	20%	20%	20%	10%	10%
	Penrith WRP (GS 567107)	< 1 EY	20%	20%	20%	20%	20%	10%	10%
	Penrith Lakes (GS 67113)	< 1 EY	20%	20%	20%	20%	10%	10%	10%

1. Unless otherwise noted, storm frequency is given as % AEP.
2. Refer **Figure 1.1** for location.

3 HYDROLOGIC MODEL DEVELOPMENT AND CALIBRATION

3.1 Hydrologic Modelling Approach

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

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

3.2 Hydrologic Model Layout

Figure 3.1 (3 sheets) shows the layout of the hydrologic model that was developed for the study area. Careful consideration was given to the definition of the sub-catchments which comprise the hydrologic model to ensure peak flows throughout the drainage system would be properly routed through the TUFLOW model. In addition to using the Light Detecting and Ranging (**LiDAR**) based contour data, the location of inlet pits and headwalls were also taken into consideration when deriving the boundaries of the various sub-catchments. The study area was split into a total of 1,623 sub-catchments.

As the primary function of the hydrologic model was to generate discharge hydrographs for input to the TUFLOW hydraulic model, individual reaches linking the various sub-catchments were not incorporated in the model.

Percentages of impervious area were based on a visual inspection of the aerial photography and experience in determining appropriate values for different land-use types. The Total Impervious Area (**TIA**) was used as input to the hydrologic model as questions have been raised in the industry about the appropriateness of adopting the Effective Impervious Area (**EIA**) approach set out in ARR 2019 (Kus et al, 2018). One of the identified issues with the approach is that it is based on a volume check rather than a peak flow check, with the adjustment factor seen as taking account of additional losses that occur in the urban environment. However, Kus et al, 2018 found that the adoption of TIA in DRAINS more closely reproduced peak flows generated by an urban catchment, as well as those derived by other peak flow estimation methods.

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

Sub-catchment slopes used for input to the hydrologic model were derived using the average sub-catchment slope computed via a region inspection in the QGIS software. Digital Elevation Models (**DEMs**) derived from the available LiDAR survey data were used as the basis for computing the slope.

Figure 3.1 shows that the RAFTS modelling approach has been used for sub-catchments comprising predominately large lot residential type development in the northern portion of the study area, while the IL-CL modelling approach has been applied in the more urbanised areas south of Boundary Road.

3.3 Hydrologic Model Testing

3.3.1. General

Historic flood data suitable for use in the model calibration process is limited to photographic and anecdotal evidence of flooding patterns for the storms that occurred on 9 February 2012, 30 January 2016 and 9 February 2020. As discussed in **Section 2.2**, the storm events for which flood data were available are equivalent to between 50% and 2% AEP design storm events.

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

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

3.3.1. Application of Historic Rainfall to the Hydrologic Model

Figure 2.4 shows the bursts of rainfall that were incorporated in the hydrologic model for the three historic storm events, noting that in order to reduce model run time only the most intense burst of rain was incorporated in the model.

As shown on **Figure 1.1**, there are no daily-read rainfall gauges internal to the study area. As a result, the continuous rainfall recorded at the two Sydney Water operated and one BoM operated pluviographic rain gauges was applied to the hydrologic model using the Thiessen polygon approach. **Figure 3.2** shows the extent over which the recorded rainfall was applied to the various sub-catchments comprising the hydrologic model.

3.3.2. Hydrologic Model Parameters

Sub-catchments in the north of the study area that were modelled using the RAFTS modelling approach in the DRAINS software had a Manning's n value of 0.04 and a Bx routing parameter of 1.0 applied to them.

The IL-CL hydrologic modelling approach in the DRAINS software requires information on the losses to be applied to determine the depth of rainfall excess. These loss rates differ for sub-catchment areas categorised as either impervious or pervious. Infiltration losses are of two types: an initial loss arising from water which is held in depressions which must be filled before runoff commences, and a continuing loss rate which depends on the type of soil and the duration of the storm event. The IL-CL approach also requires information on flow path characteristics in order to compute the time of travel of the flood wave through the sub-catchments.

The IL-CL model parameters set out in **Table 3.1** were found to give a good fit to the historic flood data:

<u>Travel Time Parameters</u>

- Paved flow path roughness = 0.02
- Grassed flow path roughness = 0.07

Table 3.1 over provides a comparison of the initial and continuing loss rates that were adopted for model calibration as part the *Peach Tree Creek and Lower Surveyors Creek Flood Study* (CSS, 2019) and *Emu Plains Overland Flow Flood Study* (BMT, 2020) with those relied upon as part of the present investigation to achieve a good match with the available historic flood data.

TABLE 3.1
ADOPTED INITIAL AND CONTINUING LOSS VALUES
HISTORIC STORM EVENTS

Study	Historic Storm Event	Initial Loss (mm)		Continuing Loss (mm/hr)	
		Impervious Area	Pervious Area	Impervious Area	Pervious Area
Peach Tree and Lower Surveyors Creek Flood Study (CSS, 2019)	9 February 2012	0	0	0	2.5[(1)]
	4 January 2016	1	10	0	2.5[(1)]
Emu Plains Overland Flow Flood Study (BMT, 2020)	31 January 2016	2	10	0	1.5[(2)]
Present Study	9 February 2012	0	0	0	1.4[(2)]
	30 January 2016	0	0	0	1.4[(2)]
	9 February 2020	0	0	0	1.4[(2)]

1. CSS, 2019 states that a continuing loss value of 2.5 mm/hr was adopted based on the procedures set out in ARR 1987.
2. Derived by applying a multiple of 0.4 to the raw continuing loss value obtained from the *ARR Data Hub* as per OEH, 2019.

Initial loss values of zero were deemed applicable for the present study as the three historic storm bursts occurred within larger storm events where there was prior rainfall that would likely have saturated the catchment (as shown on **Figure 2.4**).

The continuing loss value of 1.4 mm/hr was derived by multiplying the raw continuing loss values derived from the ARR Data Hub by a multiple of 0.4 as per the guidance in DPE's *Floodplain Risk Management Guide - Incorporating 2016 Australian Rainfall and Runoff in studies* (OEH, 2019).

Table 3.1 shows that the continuing loss value derived as part of the present study is similar to the value that was relied upon as part of BMT, 2020, while it is almost half of the value that was adopted as part of CSS, 2019. It is noted that the higher continuing loss value adopted in CSS, 2019 was derived using the procedures set out in ARR 1987 which has since been superseded by the approach set out in OEH, 2019.

3.3.3. Results of Model Testing

Through the aforementioned iterative process of running both the hydrologic and hydraulic models, it was found that the discharge hydrographs generated by the hydrologic model, when applied to the TUFLOW hydraulic model, gave reasonable correspondence with observed flood behaviour (refer **Section 4.5** for more detail). The IL-CL and RAFTS hydrologic model parameters set out in this chapter were therefore adopted for design flood estimation purposes.

4 HYDRAULIC MODEL DEVELOPMENT AND CALIBRATION

4.1 General

The present study required the use of a hydraulic model that is capable of analysing the time varying effects of flow in the local stormwater drainage system and the two-dimensional nature of flow on the floodplain and in the steeper parts of the study area that are subject to overland flow. The TUFLOW modelling software was adopted as it is one of only a few commercially available hydraulic models which contain all the required features.

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

4.2 The TUFLOW Modelling Approach

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

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

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

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

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

4.3 TUFLOW Model Setup

4.3.1. Model Structure

Figure 4.1 (3 sheets) shows the layout of the TUFLOW model that was developed as part of the present study. The model comprises the pit and pipe drainage system, while the inbank area of Boundary Creek is represented by a series of cross sections aligned normal to the direction of flow. Both out-of-bank and shallow “overland” flow are modelled by the rectangular grid.

The following sections provide further details of the model development process.

4.3.2. Two-dimensional Model Domain

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

A grid spacing of 2 m was found to provide an appropriate balance between the need to define features on the floodplain versus model run times and was adopted for the investigation. Ground surface elevations for model grid points were initially assigned using the LiDAR derived DEMs for the study area.

Figure 4.1 shows the alignment of ridge and gully lines were added to the TUFLOW model where the grid spacing was considered too coarse to accurately represent important topographic features which influence the passage of overland flow. The elevations for these ridge and gully lines were determined from inspection of LiDAR survey or site-based measurements.

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

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

It was not practicable to model the individual fences surrounding the many allotments in the study area.[2] For the purpose of the present study, it was assumed that there would be sufficient openings in the fences to allow water to enter the properties, whether as flow under or through fences and via openings at driveways. Individual allotments where development is present were digitised and assigned a high hydraulic roughness value (although not as high as for individual buildings) to account for the reduction in conveyance capacity which will result from obstructive fences, such as Colorbond or brick, and other obstructions stored on these properties.

4.3.3. One-dimensional Model Elements

Survey data provided by Cardno and Richard Hogan & Co were used as the primary source of details of the piped drainage system which were incorporated into the TUFLOW model. These data were supplemented with detailed design drawings (refer **Appendix B** for more detail). **Table 4.1** over the page summarises the pit and pipe data that were incorporated into the TUFLOW model.

[2] It is noted that no significant fences or structures were observed across the major overland flow paths during ground truthing that was undertaken in December 2021.

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

TABLE 4.1
SUMMARY OF MODELLED DRAINAGE STRUCTURES

Study Catchment	Pipes		Box Culverts		Inlet Pits	Junction Pits	Headwalls
	No.	Length (m)	No.	Length (m)	No.	No.	No.
Duralia Lake	50	1,137	7	123	31	1	70
Boundary Road	438	11,674	12	214	385	22	95
Cranebrook Road South	1,082	26,680	25	232	1,007	79	80
Andrews Road	754	20,739	46	916	695	81	66
Penrith Lakes Local	77	2,167	1	6	72	2	19
Nepean River Local	196	4,539	17	342	181	18	0
Boundary Creek	1,002	22,611	30	700	934	80	30
North Penrith	177	4,050	17	342	149	45	5
TOTAL	**3,776**	**93,597**	**155**	**2,875**	**3,454**	**328**	**365**

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

4.3.4. Model Parameters

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

Manning's n values along the channel and immediate overbank areas in the study area were varied, with the values in **Table 4.2** over the page providing reasonable correspondence between recorded and modelled flood levels. **Figure 4.2** shows the extent over which the hydraulic roughness values set out in **Table 4.2** were applied to the TUFLOW model.

TABLE 4.2
BEST ESTIMATE HYDRAULIC ROUGHNESS VALUES

Surface Treatment	Manning's n Value
Concrete piped elements	0.015(1)
Asphalt or concrete road surface; invert of concrete lined reach of Boundary Creek	0.02
Standing water bodies	0.03
Overbank area, including grass and lawns	0.045
Grassed banks of Boundary Creek	0.05
Invert of Boundary Creek	0.06
Lightly vegetated areas / Macrophytes	0.06
Moderately vegetated areas; vegetated banks of Boundary Creek	0.08
Densely vegetated banks of Boundary Creek	0.12
Allotments (between buildings)	0.1
Buildings	10

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

The adoption of a value of 0.02 for the surfaces of roads, along with an adequate description of their widths and centreline/kerb elevations, allowed an accurate assessment of their conveyance capacity to be made. Similarly, the high value of roughness adopted for buildings recognised that these structures will completely block the flow but are capable of storing water when flooded.

A relatively high roughness value of 0.1 has been applied to the grassed and paved inter-allotment area to account for the blocking effect that various features in private properties such as fences, landscaping, vegetation etc. will have on flood behaviour. While a higher roughness value may be justified in the newly developed residential areas in North Penrith and Waterside, it would have a negligible impact on overland flow behaviour as a preliminary 1% AEP result showed that there are no overland flow paths through existing development in these areas.

Figure 4.3 is a typical example of flow patterns derived from the above roughness values. This example applies to the 1% AEP design storm event and shows flooding patterns in the vicinity of the intersection of Bel-Air Road and Sunshine Avenue. The top half of the figure shows the roads and inter-allotment areas, as well as the outlines of buildings, which have all been assigned different hydraulic roughness values in the model. The bottom half shows the resulting flow paths in the form of scaled velocity vectors and the depths of inundation. The buildings with their high values of hydraulic roughness block the passage of flow, although the model recognises that they store floodwater when inundated and therefore correctly accounts for flood storage.[3] Similar information to that shown on **Figure 4.3** may be presented at any location within the model domain and will be of assistance to Council in assessing individual flooding problems in the study area.

[3] Note that the depth grid has been trimmed to the building polygons as based on previous experience, residents tend to interpret the figure as showing the depth of above-floor inundation, when in fact it is showing the depth of above-ground inundation over the footprint of the building. The same approach has been adopted for presenting the results for the various design flood events, details of which are contained in **Chapter 6**.

4.4 Model Boundary Conditions

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

The Rain Boundaries act to "inject" flow into the TUFLOW model, firstly at a point which has the lowest elevation, and then progressively over the extent of the Rain Boundary as the grid in the two-dimensional model domain becomes wet as a result of overland flow. The Rain Boundaries have been digitised at the outlet of the catchment in order to reduce the "double-routing" of runoff from the sub-catchment.

Figure 4.1 shows the location of the downstream boundaries of the TUFLOW model which have been located a sufficient distance downstream of the study area so as to not impact flood behaviour in the area of interest. **Table 4.3** sets out the source of the downstream boundary conditions that have been adopted for modelling purposes. The starting water levels in Penrith Lakes for the historic storm events were based on the "Operating Level" set out in **Table 2.2**.

TABLE 4.3
ADOPTED DOWNSTREAM BOUNDARY CONDITIONS

<table>
<tr><th rowspan="2">Boundary ID[1]</th><th rowspan="2">Boundary Description</th><th colspan="2">Source of Boundary Condition</th></tr>
<tr><th>Historic Storm Events</th><th>Design Storm Events</th></tr>
<tr><td>Henry_Out</td><td>Overland flow across Henry Street, Penrith</td><td colspan="2" rowspan="2">TUFLOW derived rating curve</td></tr>
<tr><td>Railway_Out</td><td>Overland flow in Main Western Railway corridor</td></tr>
<tr><td>NP_Out_1</td><td>Piped outflow to Penrith CBD in vicinity of Henry Street</td><td rowspan="3">Assumed tailwater level[2]</td><td rowspan="3">Molino Stewart, 2020[3]</td></tr>
<tr><td>NP_Out_2</td><td>Piped outflow to Penrith CBD in vicinity of Doonmore Street</td></tr>
<tr><td>NP_Out_3</td><td>Piped outflow to Penrith CBD in vicinity of Doonmore Street</td></tr>
<tr><td>CR_Out</td><td>Overland flow in Castlereagh Road corridor</td><td colspan="2">TUFLOW derived rating curve</td></tr>
<tr><td>PTC_Out</td><td>Piped outflow to Peachtree Road</td><td colspan="2">Assumed tailwater level[2]</td></tr>
<tr><td>NR_Out</td><td>Peak flood level in Nepean River</td><td>Nepean River at Penrith stream gauge</td><td>Advisian, 2018</td></tr>
<tr><td>RL_OUT_1</td><td>Piped outflow from Final Basin</td><td colspan="2">Assumed tailwater level[4]</td></tr>
<tr><td>RL_OUT_2</td><td>Surcharge across Final Basin spillway</td><td colspan="2" rowspan="2">TUFLOW derived rating curve</td></tr>
<tr><td>LAKE_A_OUT</td><td>Surcharge across Duralia Lake spillway</td></tr>
</table>

1. Refer **Figure 4.1** for location.
2. Tailwater assumed to be equivalent to the downstream obvert of the pipe at the location that the boundary is applied.
3. Tailwater derived from results of TUFLOW model that was developed as part of the *Penrith CBD Floodplain Risk Management Study and Plan* (Molino Stewart, 2020).
4. Tailwater assumed to be equal to the Maximum Tolerable Water Level in the Regatta Lake.

4.5 Hydraulic Model Calibration

4.5.1. General

As previously mentioned, the hydrologic and hydraulic models were tested for storms that occurred on 9 February 2012, 30 January 2016 and 9 February 2020 using the available rain gauge data. The TUFLOW model was run using discharge hydrographs that were generated by the hydrologic model, parameters for which are set out in **Section 3.3**.

4.5.2. Results of Model Testing

Figures 4.4, **4.5** and **4.6** show the TUFLOW model results for the 9 February 2012, 30 January 2016 and 9 February 2020 storms. Also shown on the figures are the plan locations of the respondents to the *Community Questionnaire* who observed flooding in or adjacent to their property during each storm event. **Tables 4.5**, **4.6** and **4.7** at the end of this chapter provide a comparison of the observed and modelled flood behaviour for the 9 February 2012, 30 January 2016 and 9 February 2020 storms, respectively.
The majority of the anecdotal descriptions of flood behaviour related to localised drainage issues in individual properties which the TUFLOW model was not capable of reproducing. As such, **Tables 4.5**, **4.6** and **4.7** also identify whether the information provided by the respondents to the *Community Questionnaire* were relied upon for model calibration purposes.

In general, the model was able to reproduce the observed flood behaviour which was approximated from the photographs and descriptions provided by respondents to the *Community Questionnaire*. The model was also able to reproduce the timing of the flooding that was experienced during the 9 February 2020 storm event.

4.5.3. Summary

Based on the findings of the model testing process, the hydrologic and hydraulic models were considered to give satisfactory correspondence with the available historic flood data. As such, the hydraulic model parameters set out in **Sections 4.3** and **4.4**, and in particular the hydraulic roughness values set out in **Table 4.2,** were considered appropriate for use in defining flood behaviour in the study area over the full range of design flood events. Further discussion and presentation of hydrologic and hydraulic model parameters that were adopted for design flood estimation purposes is provided in **Chapter 5**.

TABLE 4.5
SUMMARY OF QUESTIONNAIRE RESPONSES RELATED TO OBSERVED FLOOD BEHAVIOUR
9 FEBRUARY 2012

Response Identifier	Figure Reference	Observed Flood Behaviour/ Other Comment	Model Verification Comments	Relied Upon for Model Calibration
C044	Figure 4.4, sheet 1	• Flowpath approximately 5 m wide and up to 500 mm deep through front of property.	• The TUFLOW model shows depths of overland flow up to 160 mm in the front of the property, covering a width of approximately 10 m. • Anecdotal observation considered unreliable as, based on the LiDAR survey data, if the depth of overland flow was 500 mm, the width of flow would be 20-30 m wide.	No
C432		• Floodwater surcharged the local dam that is located upstream of the property and ponded to a depth of about 500 mm on the western side of the dwelling.	• TUFLOW model shows local dam overtopping and floodwater ponging to a depth of 430 mm.	Yes
C046a		• Overland flow that originated from neighbour's yard flows through property.	• Property is located in an area that is subject to localised catchment runoff, the definition of which is outside the scope of the present study.	No
C268		• Floodwater originating from the "top of hill" ponded in property to a depth of 300 mm.	• Property is located in an area that is subject to localised catchment runoff, the definition of which is outside the scope of the present study.	No
C046b	Figure 4.4, sheet 2	• Cranebrook Road was inundated to a depth of about 500 mm (exact location no specified).	• TUFLOW model shows Cranebrook Road inundated to a depth of about 300 mm in the vicinity of its intersection with Olive Lane.	Yes
C269		• Floodwater ponding in gutter on southern side of Bluebird Road at its intersection with Soling Crescent inundated the interior of a car that was parked at the location.	• TUFLOW model shows floodwater ponded to a depth of about 400 mm on the southern side of Bluebird Road.	Yes
C225		• Dwelling inundated above-floor level causing damage to carpets.	• Property is located in an area that is subject to localised catchment runoff, the definition of which is outside the scope of the present study.	No
C027		• Garage (carport) inundated. Items in garage were ruined and needed to be thrown out.	• TUFLOW model does not reproduce observed flood behaviour. • Observed flood behaviour may have been a result of a partial blockage of the piped drainage system and/or gutter in Moxham Street.	No
C277		• Floodwater ponded to a depth of about 100 mm in the rear of the property.	• TUFLOW model does not reproduce observed flood behaviour. • Observed flood behaviour may have been a result of localised heavy rainfall that was not recorded by rain gauges or alternatively a partial blockage of the piped drainage system. Observed flood behaviour may also be a result of localised catchment runoff, the definition of which is outside the scope of the present study	No

Cont'd Over

TABLE 4.5 (Cont'd)
SUMMARY OF QUESTIONNAIRE RESPONSES RELATED TO OBSERVED FLOOD BEHAVIOUR
9 FEBRUARY 2012

Response Identifier	Figure Reference	Observed Flood Behaviour/ Other Comment	Model Verification Comments	Relied Upon for Model Calibration
C238	Figure 4.4, sheet 2	• Dwelling inundated to a depth of about 100 mm. Stormwater entered property as overland flow from a southerly direction. Respondent attributes flow to recent development in nearby property.	• Property is located in an area that is subject to localised catchment runoff, the definition of which is outside the scope of the present study.	No
C381a		• Park opposite church on Callisto Drive (Borrowdale Way Basin No. 1) inundated to depths of at least 1 m.	• The TUFLOW model show the Borrowdale Way Basin No. 1 inundated to a maximum depth of about 1.2 m.	Yes
C381b		• Park opposite skatepark (Sherringham Road Basin No. 1) inundated to depths of at least 1 m.	• The TUFLOW model show the Sherringham Road Basin No. 1 inundated to a maximum depth of about 1.3 m.	Yes
C381c		• Grey Gums Oval inundated to depths of at least 1 m.	• The TUFLOW model show the Grey Gums Oval inundated to a maximum depth of about 1.1 m.	Yes
C120		• Entry foyer in house inundated to a depth of 500-600 mm. Backyard inundated to depth of about 1 m.	• Entry foyer is set at an elevation about 1.5 m higher than the street level. • Local catchment potentially contributing to runoff in back yard of property is about 0.4 ha. • Anecdotal description not considered reliable for model calibration.	No
C319		• Overland flow originating from neighbouring properties through rear of property to a depth of 100-150 mm.	• TUFLOW model does not reproduce observed flood behaviour. • Observed flood behaviour have been a result of localised heavy rainfall that was not recorded by rain gauges or a partial blockage of the piped drainage system.	No
		• Road in vicinity of property (location not specified) inundated to shallow depths (depth not given).	• TUFLOW model shows Greygums Road inundated to a depth of about 250 mm in the vicinity of its intersection with Andrews Road.	Yes
C148		• Backyard inundated to a depth of about 250 mm.	• Property is located in an area that is subject to localised catchment runoff, the definition of which is outside the scope of the present study.	No
C135	Figure 4.4, sheet 3	• Floodwater filled reserve on the eastern side of Landy Avenue (i.e. Cooper Street Basin).	• The TUFLOW model shows the Cooper Street Basin inundated to a maximum depth of about 2.2 m.	Yes

Cont'd Over

TABLE 4.5 (Cont'd)
SUMMARY OF QUESTIONNAIRE RESPONSES RELATED TO OBSERVED FLOOD BEHAVIOUR
9 FEBRUARY 2012

Response Identifier	Figure Reference	Observed Flood Behaviour/ Other Comment	Model Verification Comments	Relied Upon for Model Calibration
C210	Figure 4.4, sheet 3	• Floodwater inundated the front gate of property. • Coombes Drive inundated to a depth of about 500 mm.	• TUFLOW model shows the depth of inundation in the vicinity of the front gate is about 200 mm. • TUFLOW model shows Coombes Drive inundated to a maximum depth of about 460 mm.	Yes
C087a		• Stormwater backed up and inundated new pool pump and overtopped into garage.	• Property is located in an area that is subject to localised catchment runoff, the definition of which is outside the scope of the present study.	No
C087b		• The depth of inundation at the intersection of Bel-Air Road and Sunshine Avenue was "thigh deep" (approximately 700 mm).	• TUFLOW model shows Bel-Air Road inundated to a depth of about 800 mm at a location approximately 75 m to the east of its intersection with Sunshine Avenue. • TUFLOW model shows Sunshine Avenue inundated to a depth of about 1050 mm at the low point that is located approximately 40 m north of its intersection with Bel-Air Road.	Yes
C467		• Runoff from neighbour's house inundated back yard to a depth of about 300 mm.	• Property is located in an area that is subject to localised catchment runoff, the definition of which is outside the scope of the present study.	No
C012		• Floodwater ponded up to front veranda and entered the house via the front door. • Floodwater inundated garage and damaged two cars.	• A review of the aerial photography shows that the property (located in North Penrith subdivision) did not exist at time of storm event.	No

TABLE 4.6
SUMMARY OF QUESTIONNAIRE RESPONSES RELATED TO OBSERVED FLOOD BEHAVIOUR
30 JANUARY 2016

Response Identifier	Figure Reference	Observed Flood Behaviour/ Other Comment	Model Verification Comments	Relied Upon for Model Calibration
C279	Figure 4.5, sheet 1	• Concentrated flow from school discharges to a narrow channel	• Property is located in an area that is subject to localised catchment runoff, the definition of which is outside the scope of the present study.	No
C176		• Kenilworth Crescent and Boundary Road inundated to a depth of about 300-400 mm.	• TUFLOW model shows Kenilworth Crescent inundated to a depth of about 150 mm in the vicinity of its intersection with Woodside Glen. • TUFLOW model shows Boundary inundated to a depth of about 250 mm.	Yes
C214	Figure 4.5, sheet 2	• Front yard and garage inundated to depths of about 20-30 mm.	• TUFLOW model shows floodwater surcharges road reserve and ponds in front yard of property to depths less than 100 mm.	Yes
C269		• Floodwater ponding in gutter on southern side of Bluebird Road at its intersection with Soling Crescent inundated the interior of a car that was parked at the location.	• TUFLOW model shows floodwater ponded to a depth of about 350 mm on the southern side of Bluebird Road.	Yes
C225		• Dwelling inundated to shallow depths causing damage to carpets and furniture.	• Property is located in an area that is subject to localised catchment runoff, the definition of which is outside the scope of the present study.	No
C030		• Shallow inundation of front third of property. Floodwater ponded to shallow depths in garage.	• Property is located in an area that is subject to localised catchment runoff, the definition of which is outside the scope of the present study.	No
C082	Figure 4.5, sheet 3	• Backyard inundated to a maximum depth of about 2 inches (i.e. 50 mm)	• Property is located in an area that is subject to localised catchment runoff, the definition of which is outside the scope of the present study.	No
C106		• Cooper Street inundated to depths of between 50-100 mm (exact location not specified).	• TUFLOW model does not reproduce observed flood behaviour. • Observed flood behaviour may have been a result of a partial blockage of the piped drainage system in Cooper Street.	No
C135		• Floodwater filled reserve on the eastern side of Landy Avenue (i.e. Cooper Street Basin).	• The TUFLOW model show the Cooper Street Basin inundated to a maximum depth of about 1.8 m.	Yes

Cont'd Over

TABLE 4.6 (Cont'd)
SUMMARY OF QUESTIONNAIRE RESPONSES RELATED TO OBSERVED FLOOD BEHAVIOUR
30 JANUARY 2016

Response Identifier	Figure Reference	Observed Flood Behaviour/ Other Comment	Model Verification Comments	Relied Upon for Model Calibration
C263	Figure 4.5, sheet 3	• Garage inundated to a depth of about 100 mm.	• TUFLOW model does not reproduce observed flood behaviour. • Observed flood behaviour may have been a result of localised heavy rainfall that was not recorded by rain gauges or alternatively a blockage of the piped drainage system in Illawong Avenue.	No
C467		• Runoff from neighbour's house inundated back yard to a depth of about 300 mm.	• Property is located in an area that is subject to localised catchment runoff, the definition of which is outside the scope of the present study.	No

TABLE 4.7
SUMMARY OF QUESTIONNAIRE RESPONSES RELATED TO OBSERVED FLOOD BEHAVIOUR
9 FEBRUARY 2020

Response Identifier	Figure Reference	Observed Flood Behaviour/ Other Comment	Model Verification Comments	Relied Upon for Model Calibration
C044	Figure 4.6, sheet 1	• Flowpath approximately 3 m wide and up to 300 mm deep through front of property.	• The TUFLOW model shows depths of overland flow less than 100 mm in the front of the property. • Anecdotal observation considered unreliable as, based on the LiDAR survey data, if the depth of overland flow was 300 mm, the width of flow would be 10-20 m wide.	No
C279		• Concentrated flow from school discharges to a narrow channel	• Property is located in an area that is subject to localised catchment runoff, the definition of which is outside the scope of the present study.	No
C432		• Floodwater surcharged the local dam that is located upstream of the property and ponded to a depth of about 300 mm on the western side of the dwelling.	• TUFLOW model shows local dam overtopping and floodwater ponding to a depth of 250 mm.	Yes
C176a		• Kenilworth Crescent and Boundary Road (exact locations not specified) inundated to a depth of about 300-400 mm.	• TUFLOW model shows Kenilworth Crescent inundated to a depth of less than 100 mm. • TUFLOW model shows Boundary Road inundated to a depth of about 170 mm immediately west of its intersection with Hindmarsh Street.	Yes
C417		• Depth of inundation in yard exceeded 1.5 m. • **Plates C5.7** and **C5.8** of **Appendix C** show floodwater surcharging the banks of the channel and inundating the overbank area.	• TUFLOW model shows the maximum depth of inundation in the channel is about 500 mm. It is noted that flooding conditions would have been much more severe than shown in the photographs if the depth on inundation was greater than 1.5 m. • TUFLOW model shows floodwater surcharging the banks of the channel and inundating the overbank areas to depths of up to 300 mm.	Yes
C046a		• Overland flow that originated from neighbour's yard flows through property.	• Property is located in an area that is subject to localised catchment runoff, the definition of which is outside the scope of the present study.	No
C429		• Runoff originating from three neighbouring properties inundates back yard.	• Property is located in an area that is subject to localised catchment runoff, the definition of which is outside the scope of the present study.	No
C359		• Floodwater ponded to a depth of about 20 mm beneath back patio.	• TUFLOW model does not reproduce observed flood behaviour. • Observed flood behaviour may have been a result of localised heavy rainfall that was not recorded by rain gauges or alternatively a blockage of the piped drainage system in Illawong Avenue.	No

Cont'd Over

TABLE 4.7 (Cont'd)
SUMMARY OF QUESTIONNAIRE RESPONSES RELATED TO OBSERVED FLOOD BEHAVIOUR
9 FEBRUARY 2020

Response Identifier	Figure Reference	Observed Flood Behaviour/ Other Comment	Model Verification Comments	Relied Upon for Model Calibration
C028		• Cranebrook Road inundated to depths of between 200 and 250 mm immediately to the north of its intersection with Olive Lane.	• TUFLOW model shows Cranebrook Road inundated to a depth of about 280 mm	Yes
C306		• Cranebrook Road inundated to depths of about 4 feet (i.e. 1.2 m) immediately to the north of its intersection with Olive Lane.	• TUFLOW model shows Cranebrook Road inundated to a depth of about 280 mm	No
C235		• Cranebrook Road inundated to depths of about 400 mm immediately to the north of its intersection with Olive Lane.	• TUFLOW model shows Cranebrook Road inundated to a depth of about 280 mm	Yes
C381	Figure 4.6, sheet 2	• **Plate C5.2** of **Appendix C** shows that floodwater was surcharging the spillway of the Borrowdale Road Basin No. 2 at 15:00 hours.	• TUFLOW model shows basin overtopping between 12:00 hours and 17:00 hours.	Yes
C327		• Floodwater in channel was flowing like "rapids" and moving very fast.	• TUFLOW model shows the flow velocity in the channel is about 0.7-0.9 m/s	Yes
C421		• Floodwater inundated yard to a depth of about 30 to 60 mm which resulted in very shallow inundation of garage.	• Property is located in an area that is subject to localised catchment runoff, the definition of which is outside the scope of the present study.	No
C028		• Floodwater inundated the intersection of Andrews Road and Castlereagh Road to a depth of about 150-200 mm.	• TUFLOW model does not reproduce observed flood behaviour. • Observed flood behaviour may have been a result of a partial blockage of the piped drainage system.	No
C411	Figure 4.6, sheet 3	• Low point in Cudgee Road inundated to a depth of about 4 feet (i.e. 1.2 m).	• TUFLOW model does not reproduce observed flood behaviour. • Observed flood behaviour have been a result of localised heavy rainfall that was not recorded by rain gauges or a partial blockage of the piped drainage system which would have resulted in runoff ponding in the trapped low point in Cudgee Road.	No

Cont'd Over

TABLE 4.7 (Cont'd)
SUMMARY OF QUESTIONNAIRE RESPONSES RELATED TO OBSERVED FLOOD BEHAVIOUR
9 FEBRUARY 2020

Response Identifier	Figure Reference	Observed Flood Behaviour/ Other Comment	Model Verification Comments	Relied Upon for Model Calibration
C263	Figure 4.6, sheet 3	• Floodwater inundated garage to a depth of about 100 mm.	• TUFLOW model does not reproduce observed flood behaviour. • Observed flood behaviour may have been a result of localised heavy rainfall that was not recorded by rain gauges or alternatively a blockage of the piped drainage system in Illawong Avenue.	No
C467		• Runoff from neighbour's house inundated back yard to a depth of about 300 mm.	• Property is located in an area that is subject to localised catchment runoff, the definition of which is outside the scope of the present study.	No
C159		• Floodwater inundated property to "knee-deep or lower" (i.e. less than 500 mm).	• Property is located in an area that is subject to localised catchment runoff, the definition of which is outside the scope of the present study.	No
C343		• Floodwater in "canal" inundated pavement for a long period of time (exact location not specified).	• TUFLOW model shows footpath on the southern side of Thornton Drive inundated to a depth of about 400 mm.	Yes

5 DERIVATION OF DESIGN FLOOD HYDROGRAPHS

5.1 Design Storms

5.1.1. Rainfall Intensity

The procedures used to obtain temporally and spatially accurate and consistent Intensity-Frequency-Duration (**IFD**) design rainfall curves for the assessment of flood behaviour in the study area are presented in the 2019 edition of *Australian Rainfall and Runoff* (Geoscience Australia, 2019) (**ARR 2019**). Design storms for frequencies of 0.5 EY, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP were derived for storm durations ranging between 15 minutes and seven days. The IFD dataset was downloaded from the BoM's *2016 Rainfall IFD Data System*.

5.1.2. Areal Reduction Factors

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

However, as the study catchments are relatively small (i.e. between 0.34 and 3.12 km^2 in area), the reduction in rainfall intensity would be quite small. Furthermore, the purpose of this present study is to define flood behaviour not only in the lower reaches of each catchment, but also in the middle and upper reaches where the contributing catchment areas are much smaller in size.

Accordingly, no reduction in design point rainfalls was made for this present study (i.e. an ARF of 1.0 was adopted).

5.1.3. Temporal Patterns

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

The temporal pattern ensembles that are applicable to Frequent (more frequent than 14.4% AEP), Intermediate (between 14.4% and 3.2% AEP) and Rare (rarer than 3.2% AEP) storm events were obtained from the ARR Data Hub[4], while those for the very rare events were taken from the BoMs update of *Bulletin 53* (BoM, 2003). A copy of the data extracted from the ARR Data Hub is contained in **Appendix D**.

5.1.4. Probable Maximum Precipitation

Estimates of Probable Maximum Precipitation (**PMP**) were made using the Generalised Short Duration Method (**GSDM**) as described in the BoM, 2003. This method is appropriate for estimating extreme rainfall depths for catchments up to 1000 km^2 in area and storm durations up to 3 hours.

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

- Calculate PMP for a given duration and catchment area using depth-duration-area envelope curves derived from the highest recorded US and Australian rainfalls.

[4] It is noted that the temporal pattern data set for the *East Coast South* region is suitable for use in the study area.

- Adjust the PMP estimate according to the percentages of the catchment which are meteorologically rough and smooth, and also according to elevation adjustment and moisture adjustment factors.
- Assess the design spatial distribution of rainfall using the distribution for convective storms based on US and world data but modified in the light of Australian experience.
- Derive storm hyetographs using the temporal distribution contained in BoM, 2003, which is based on pluviographic traces recorded in major Australian storms.

Figure 5.1 shows the location and orientation of the PMP ellipses which were used to derive the rainfall estimates for the present study. Note that three orientations of the PMP ellipses were adopted for the northern, central and southern sections of the study catchment in order to accurately derive the upper limit of flooding in the upper, middle and lower reaches of the drainage system.

5.2 Design Rainfall Losses

The initial and continuing loss values to be applied in flood hydrograph estimation were derived using the NSW jurisdictional specific procedures set out in the *ARR Data Hub*. A continuing loss value of 1.4 mm/hr[5] was adopted for design flood estimation purposes, while a copy of the raw *ARR Data Hub* data, which includes the Probability Neutral Burst Initial Loss values that were adopted for design flood estimation purposes, is contained in **Appendix D**.

5.3 Derivation of Design Discharges

The hydrologic model was run with the design rainfall data set out in **Sections 5.1** and **5.2**, as well as the hydrologic parameters set out in **Section 3.3.2** in order to obtain design discharge hydrographs for input to the TUFLOW model.

[5] The continuing loss value of 1.4 mm/hr was derived by multiplying the raw (or unadjusted) continuing loss value contained on the *ARR Data Hub* (i.e. 3.9 mm/hr) by a factor of 0.4.

6 HYDRAULIC MODELLING OF DESIGN FLOOD EVENTS

6.1 Hydraulic Model Structure

6.1.1. Partial Blockage of Hydraulic Structures

As per the requirements of ARR 2019, the potential for the existing drainage system to experience a partial blockage during a flood event was taken into account when deriving the design flood envelopes. **Table E1** in **Appendix E** provides a summary of the blockage factors that were derived for each individual headwall and bridge structure in the study area based on the procedures set out in ARR 2019. As per the recommendations in ARR 2019, an L_{10}[6] of 1.5 m was adopted for the blockage assessment, which is the recommended minimum value that should be adopted for urban areas in the absence of a record of past debris accumulated at the structure. **Table 6.1** sets out the blockage factors that were applied to the stormwater inlet pits based on Council's *Pit Blockage Policy*.

TABLE 6.1
ADOPTED PIT INLET BLOCKAGE FACTORS

Pit Conditions	Inlet Type	Adopted Blockage Factor
Sag	Side Entry	20%
	Grated	50%
	Combination	Side inlet capacity only (i.e. grate 100% blocked)
	Letterbox	50%
On-Grade	Side Entry	20%
	Grated	50%
	Combination	10%

6.1.2. Boundary Conditions

Table 4.3 in **Chapter 4** sets out the source of the downstream boundary conditions that were adopted for design flood modelling purposes, while **Table 6.2** over the page sets out the combination of local catchment storms and Nepean River flood events that were adopted for defining the nature of short duration local catchment flooding in the study area.

The starting water levels in Penrith Lakes that were adopted for design flood modelling purposes were based on the "Maximum Tolerable Water Levels" which are set out in **Table 2.2** in **Chapter 2**.[7] The starting water levels in the privately owned dams that are located in the rural part of the study area in the Duralia Lake and Boundary Road catchments have been set at their respective spillway elevations (i.e. the dams are assumed to be full at the commencement of the design storm event).

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

[7] Note that a sensitivity analysis was undertaken to assess the impact that adopting the "Operating Levels" in Penrith Lakes that are set out in **Table 2.2** would have on a 1% AEP local catchment flood event (refer **Section 6.5.5** for details).

TABLE 6.2
NEPEAN RIVER TAILWATER LEVELS INCORPORATED IN DESIGN FLOOD ENVELOPES

Design Flood Envelope	Local Catchment Storm Event	Nepean River Flood Event
0.5 EY	0.5 EY	No flooding on Nepean River
20%	20%	
10%	10%	
5%	5%	5% AEP
2%	2%	
1%	1%	
0.5%	0.5%	
0.2%	0.2%	
PMF	PMF	

6.2 Presentation and Discussion of Results

6.2.1. Accuracy of Hydraulic Modelling

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

6.2.2. Critical Duration and Temporal Pattern Assessment

The critical storm durations and associated median temporal patterns for the design storm events were derived based on the results of running both the DRAINS and TUFLOW models in tandem. For example, design discharge hydrographs for the ensemble of temporal patterns for storm durations ranging between 15 minutes and 18 hours were exported from the DRAINS model and input to the TUFLOW model. The assessment was undertaken for the 20%, 5% and 1% AEP storm events which represent the three temporal pattern bins (i.e. frequent, infrequent and rare, respectively) that were downloaded from the *ARR Data Hub.*

A similar process was adopted for determining the critical durations for the PMF using the procedures set out in BoM, 2003, whereby design discharge hydrographs for storm durations ranging between 15 minutes and 6 hours were exported from the DRAINS model and input to the TUFLOW model.

Table 6.3 sets out the storm durations and temporal patterns that were adopted as being critical for AEPs ranging from 50% and 0.2%, as well as the PMF.

TABLE 6.3
CRITICAL DURATIONS AND TEMPORAL PATTERNS

Design Storm Event	Temporal Pattern Bin	Critical Storm Duration and Temporal Pattern[1]
0.5 EY	Frequent	15 minute, Storm Burst 7[4423] 30 minute, Storm Burst 5 [4519] 1 hour, Storm Burst 8 [4581] 1.5 hour, Storm Burst 8 [4608] 6 hour, Storm Burst 8 [4740]
20%		
10%	Infrequent	15 minute, Storm Burst 5[4413] 30 minute, Storm Burst 4 [4509] 1 hour, Storm Burst 3 [4565] 1.5 hour, Storm Burst 6 [4593] 6 hour, Storm Burst 7 [4726]
5%		
2%	Rare	15 minute, Storm Burst 8[4400] 30 minute, Storm Burst 5 [4498] 1 hour, Storm Burst 7 [4558] 6 hour, Storm Burst 4 [4596]
1%		
0.5%		
0.2%		
PMF	Very Rare	30 minute 2 hour 6 hour

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

6.2.3. Design Flood Extents

Figures 6.1 to **6.9** (3 sheets each) show the TUFLOW model results for the 0.5 EY, 20%, 10%, 5%, 2%, 1%, 0.5% and 0.2% AEP floods, together with the PMF. These diagrams show the indicative extent and depth of inundation for the full range of design storm events throughout the study area.

In order to create realistic results which remove most of the anomalies caused by inaccuracies in the LiDAR survey data (refer below for details), a filter was applied to remove depths of inundation over the natural surface less than 150 mm and then remove isolated pockets of flooding that are less than 100 m^2 in area. This has the effect of removing the very shallow depths which are more prone to be artefacts of the model, but at the same time giving a reasonable representation of the various overland flow paths. The depth grids shown on the figures have also been trimmed to the building polygons, as experience has shown that property owners incorrectly associate depths of above-ground inundation at the location of buildings with depths of above-floor inundation.

Figure 6.10 (3 sheets) shows the AEP of the local catchment storm event which results in individual pipes first flowing full, while **Figure 6.11** shows stage hydrographs at selected road crossings throughout the study area (refer **Figure 6.10** for plan location of stage hydrographs). **Table 6.4** over the page summarises the design peak flood levels and depth of flow over the spillway in the 24 regional detention basins that have been constructed to control local catchment runoff in the upper and middle reaches of the drainage system.

TABLE 6.4
SUMMARY OF HYDRAULIC PERFORMACE OF REGIONAL DETENTION BASINS[(1)]

Basin ID[(2)]	Basin Name	Study Catchment	Spillway Elevation (m AHD)	Historic Storm Events						Design Storm Events																	
				February 2012		January 2016		February 2020		0.5 EY		20% AEP		10% AEP		5% AEP		2% AEP		1% AEP		0.5% AEP		0.2% AEP		PMF	
				Peak Flood Level (m AHD)	Depth of Flow Over Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Over Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Over Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Over Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Over Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Over Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Over Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Over Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Over Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Over Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Over Spillway (m)	Peak Flood Level (m AHD)	Depth of Flow Over Spillway (m)
B01	Tornado Crescent Basin 1	Boundary Road	37.15	37.61	0.46	37.48	0.33	37.28	0.13	37.19	0.04	37.48	0.33	37.54	0.39	37.59	0.44	37.63	0.48	37.67	0.52	37.70	0.55	37.74	0.59	38.28	1.13
B02	Tornado Crescent Basin 2		34.48	35.01	0.53	34.86	0.38	34.68	0.20	34.64	0.16	34.86	0.38	34.93	0.45	34.98	0.50	35.03	0.55	35.07	0.59	35.10	0.62	35.15	0.67	35.69	1.21
B03	Hanlan Street Basin		33.27	33.53	0.26	32.07	-	31.58	-	31.49	-	32.36	-	32.86	-	33.27	-	33.52	0.25	33.66	0.39	33.72	0.45	33.79	0.52	34.43	1.16
B04	Soiling Crescent Basin 1		30.75	31.14	0.39	31.01	0.26	30.96	0.21	30.89	0.14	31.01	0.26	31.04	0.29	31.06	0.31	31.10	0.35	31.18	0.43	31.22	0.47	31.27	0.52	31.81	1.06
B05	Soiling Crescent Basin 2		28.01	28.31	0.30	28.09	0.08	28.17	0.16	27.79	-	28.16	0.15	28.22	0.21	28.24	0.23	28.27	0.26	28.31	0.30	28.34	0.33	28.38	0.37	28.82	0.81
B06	Borrowdale Way Basin 1	Cranebrook Road South	42.63	43.02	0.39	42.81	0.18	42.91	0.28	42.72	0.09	42.91	0.28	42.94	0.31	42.97	0.34	43.00	0.37	43.02	0.39	43.04	0.41	43.06	0.43	43.34	0.71
B07	Borrowdale Way Basin 2		41.58	41.98	0.40	41.11	-	41.81	0.23	41.12	-	41.75	0.17	41.78	0.20	41.84	0.26	41.90	0.32	41.93	0.35	41.95	0.37	42.00	0.42	42.81	1.23
B08	Sherringham Road Basin 1		39.06	39.44	0.38	39.13	0.07	39.26	0.20	39.13	0.07	39.19	0.13	39.23	0.17	39.29	0.23	39.34	0.28	39.39	0.33	39.41	0.35	39.44	0.38	39.97	0.91
B09	Sherringham Road Basin 2		38.23	38.72	0.49	37.69	-	38.48	0.25	37.45	-	38.36	0.13	38.44	0.21	38.51	0.28	38.58	0.35	38.65	0.42	38.68	0.45	38.72	0.49	39.37	1.14
B10	McHenry Road Basin 1		35.64	36.13	0.49	35.28	-	35.88	0.24	34.97	-	35.54	-	35.85	0.21	35.90	0.26	35.97	0.33	36.06	0.42	36.10	0.46	36.15	0.51	36.79	1.15
B11	McHenry Road Basin 2		32.92	33.28	0.36	32.05	-	32.97	0.05	-	-	32.16	-	32.52	-	32.97	0.05	33.16	0.24	33.19	0.27	33.26	0.34	33.36	0.44	34.15	1.23
B12	McHenry Road Basin 3		31.19	31.68	0.49	31.43	0.24	31.40	0.21	31.11	-	31.44	0.25	31.46	0.27	31.47	0.28	31.55	0.36	31.59	0.40	31.65	0.46	31.79	0.60	32.58	1.39
B13	McHenry Road Basin 6		30.13	30.55	0.42	30.10	-	30.37	0.24	29.84	-	30.24	0.11	30.35	0.22	30.39	0.26	30.47	0.34	30.51	0.38	30.53	0.40	30.68	0.55	32.01	1.88
B14	McHenry Road Basin 4		32.96	33.10	0.14	32.72	-	32.68	-	32.41	-	32.76	-	33.01	0.05	33.07	0.11	33.08	0.12	33.09	0.13	33.09	0.13	33.11	0.15	33.52	0.56
B15	McHenry Road Basin 5		31.17	31.13	-	30.54	-	31.00	-	30.46	-	30.68	-	30.85	-	30.99	-	31.23	0.06	31.31	0.14	31.31	0.14	31.32	0.15	32.04	0.87
B16	Laycock Street Basin 1		27.21	27.55	0.34	27.28	0.07	27.46	0.25	27.13	-	27.37	0.16	27.44	0.23	27.48	0.27	27.51	0.30	27.53	0.32	27.54	0.33	27.57	0.36	27.98	0.77
B17	Laycock Street Basin 2		26.20	26.52	0.32	25.54	-	26.43	0.23	25.42	-	25.79	-	26.36	0.16	26.43	0.23	26.48	0.28	26.50	0.30	26.51	0.31	26.54	0.34	27.02	0.82
B18	Cooper Street Basin	Andrews Road	33.26	33.10	-	32.82	-	32.60	-	32.38	-	32.76	-	32.95	-	33.13	-	33.27	0.01	33.39	0.13	33.43	0.17	33.49	0.23	34.21	0.95
B19	Andrews Road Basin 1		28.94	29.51	0.57	29.39	0.45	29.32	0.38	29.11	0.17	29.36	0.42	29.41	0.47	29.45	0.51	29.49	0.55	29.52	0.58	29.55	0.61	29.58	0.64	30.18	1.24
B20	Andrews Road Basin 2		27.13	27.59	0.46	27.28	0.15	27.34	0.21	26.77	-	27.32	0.19	27.41	0.28	27.47	0.34	27.54	0.41	27.59	0.46	27.63	0.50	27.68	0.55	28.39	1.26
B21	King Street Basin 1	Boundary Creek	49.10	49.12	0.02	48.80	-	47.97	-	-	-	48.61	-	48.81	-	48.96	-	49.13	0.03	49.27	0.17	49.30	0.20	49.32	0.22	49.58	0.48
B22	King Street Basin 2		44.09	43.93	-	43.32	-	42.77	-	-	-	43.11	-	43.40	-	43.60	-	43.74	-	44.17	0.08	44.28	0.19	44.34	0.25	44.80	0.71
B23	King Street Basin 3		40.84	41.01	0.17	40.55	-	40.57	-	39.45	-	40.37	-	40.63	-	40.85	0.01	41.02	0.18	41.04	0.20	41.06	0.22	41.15	0.31	41.87	1.03
B24	Coreen Avenue Basin		37.65	37.82	0.17	37.74	0.09	37.78	0.13	37.52	-	37.75	0.10	37.77	0.12	37.78	0.13	37.81	0.16	37.82	0.17	37.83	0.18	37.87	0.22	38.52	0.87

1. Cells highlighted green indicate that the road is not overtopped while cells highlighted yellow indicate that the road is inundated.
2. Refer **Figure 6.10** (3 sheets) for location.

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

Minimum floor levels for residential and commercial developments should be based on the 1% AEP flood level plus appropriate freeboard (this planning level is defined as the *"Flood Planning Level"* (**FPL**)), to cater for uncertainties such as wave action, effects of flood debris conveyed in the overland flow stream and precision of modelling. Note that a freeboard of 500 mm has been adopted for defining an set of FPLs (**FPLs**) along the main drainage paths in the study area pending the completion of the future *FRMS&P*. Derivation of an Flood Planning Area (**FPA**) based on the FPLs is presented in **Section 6.7**.

The sensitivity studies and discussion presented in **Section 6.5** provide guidance on the suitability of the recommended allowance for freeboard under present day climatic conditions.

In accordance with DPE recommendations (DECC, 2007), sensitivity studies have also been carried out to assess the potential impacts of future climate change on flood behaviour (refer **Section 6.6**). While increases in flood levels due to future increases in rainfall intensities may influence the selection of FPLs, final selection of FPLs is a matter for more detailed consideration during the preparation of the future *FRMS&P*.

6.2.4. Description of Local Catchment Flood Behaviour

The key features of local catchment flood behaviour in the **Duralia Lake Catchment** (refer sheet 1 in figure set) are as follows:

- **Figure 6.10** shows that while the piped drainage system generally has a capacity of 20% AEP or higher, the local piped drainage system in Karen Court is at capacity in a 0.5 EY storm event.
- **Figure 6.2** shows that the depth of flow along the overland flow paths is generally less than 300 mm deep in a 20% AEP storm event with the following exceptions:
  - In the vicinity of the privately owned dams located in rural residential properties where depths exceed 1 m.
  - Along the incised channel that runs in a southerly direction on the western side of Cranebrook Road.
- **Figure 6.6** shows that the depth of flow along the abovementioned overland flow paths is generally greater than 500 mm in a 1% AEP storm event.
- Floodwater surcharges the piped drainage system and ponds in residential properties at the western end of Karen Court in a 5% AEP storm event (refer **Figure 6.4**).
- **Figure 6.11** shows that floodwater commences to inundate the southbound lane of Cranebrook Road at a location approximately 150 m north of its intersection with Vincent Road in a 20% AEP storm event (refer Stage Hydrograph Location (**SHL**) H01 on **Figure 6.10** for location). Cranebrook Road is inundated to a maximum depth of about 550 mm at this location in a 1% AEP storm event.

- Cranebrook Road remains flood free where it runs between Castlereagh Road and Karen Court in storm events up to 0.2% AEP in intensity. As a result, this section of road could be used to evacuate the residential properties that are located in the vicinity of the intersection of Boundary Road and Cranebrook Road during storms up to this intensity.
- **Figure 6.9** shows that Castlereagh Road will be overtopped at a location approximately 200 m to the north of its intersection with Cranebrook Road to a maximum depth of about 150 mm during a PMF event.

The key features of local catchment flood behaviour in the **Boundary Road Catchment** (refer sheets 1 and 2 in figure set) are as follows:

- **Figure 6.10** shows that the trunk drainage system that runs in a westerly direction between the intersection of Grays Lane and Hindmarsh Street, and Olive Lane is at capacity in a 0.5 EY design storm event, as are the lower reaches of the lateral piped drainage lines that connect to the trunk drainage line along this reach. The majority of the remaining piped drainage system in the catchment generally has a capacity of 20% AEP.
- **Table 6.4** shows that the spillways of five regional detention basins that are located in the catchment are surcharged as follows (refer **Figure 6.10** for plan location of each):
    - The Tornado Crescent Basin Nos. 1 and 2 (refer B01 and B02, respectively) and Soling Crescent Basin No. 1 (refer B04) surcharge in storm events as frequent as 0.5 EY.
    - The Soling Crescent Basin No. 2 (refer B05) commences to surcharge in a 20% AEP storm event.
    - The Hanlan Crescent Basin (refer B03) commences to surcharge in a 2% AEP storm event.
- **Figure 6.2** shows that the floodwater is generally contained within the banks of the flowpaths through rural residential properties that are located between Kenilworth Crescent and Boundary Road in a 20% AEP storm event. **Figure 6.6** shows that floodwater surcharges the banks at this location in a 1% AEP storm event where it inundates the overbank area to a maximum depth of about 500 mm.
- Floodwater surcharges the piped drainage system and discharges through residential properties at the following locations:
    - between the northern end of Wiggan Place and Laycock Street at its intersection with Fireball Avenue in a 10% AEP storm event (refer **Figure 6.3**).
    - between Fireball Avenue and Tornado Crescent on the western side of Laycock Street in a 5% AEP storm event (refer **Figure 6.4**).
    - between Fireball Avenue and Tornado Crescent to the south of the latter's intersection with Cobra Street in a 5% AEP storm event (refer **Figure 6.4**,).
    - between Marrett Way and Hindmarsh Drive west of their intersections with Middleton Avenue in a 5% AEP storm event (refer **Figure 6.4**).
    - between the western end of Ellim Place and Hindmarsh Drive at its intersection with Grays Lane in a 2% AEP storm event (refer **Figure 6.6**).
- **Figure 6.6** shows that the depth of flow along the abovementioned overland flow paths generally does not exceed 300 mm in a 1% AEP storm event.

- **Figure 6.11** shows that floodwater commences to inundate Boundary Road at a location approximately 220 m to the east of its intersection with Cranebrook Road in a 10% AEP storm event (refer SHL H02 on **Figure 6.10** for location). Cranebrook Road is inundated to a maximum depth of about 500 mm at this location in a 1% AEP storm event.
- **Figure 6.11** shows that floodwater inundates Cranebrook Road at a location approximately 180 m to the east of its intersection with Castlereagh Road by 130 mm in a 50% AEP storm event (refer SHL H03 on **Figure 6.10** for location). Cranebrook Road is inundated to a maximum depth of about 430 mm at this location in a 1% AEP storm event.
- **Figure 6.9** shows that the depth of overland flow through existing residential properties during a PMF event is generally less than 400 mm, with depths of up to 800 mm shown to occur in isolated areas where water ponds on the upstream side of existing buildings.

The key features of local catchment flood behaviour in the **Cranebrook Road South Catchment** (refer sheet 2 in figure set) are as follows:

- **Figure 6.10** shows that the trunk drainage system that runs in a southerly direction between Marrett Way and Laycock Street is at capacity in a 0.5 EY design storm event, as are the lower reaches of the lateral piped drainage lines that connect to the trunk drainage line. The majority of the remaining piped drainage system in the catchment generally has a capacity of 20% AEP.
- **Table 6.4** shows that the spillways of twelve regional detention basins that are located in the catchment are surcharged as follows (refer **Figure 6.10**, sheet 2 for plan location of each):
  - The Borrowdale Way Basin No. 1 (refer B06) and Sherringham Road Basin No. 1 (refer B08) surcharge in storm events as frequent as 0.5 EY.
  - The Borrowdale Way Basin No. 2 (refer B07), Sherringham Road Basin No. 2 (refer B09), McHenry Road Basin Nos. 3 (refer B12) and 6 (refer B13), and the Laycock Street Basin No. 1 (B16) commence to surcharge in a 20% AEP storm event.
  - The McHenry Road Basin Nos. 1 (refer B10) and 4 (refer B14), as well as the Laycock Street Basin No. 2 (refer B17) commence to surcharge in a 10% AEP storm event.
  - The McHenry Road Basin No. 2 (refer B11) commences to surcharge in a 5% AEP storm event.
  - The McHenry Road Basin No. 5 (refer B15) commences to surcharge in a 2% AEP storm event.
- Floodwater surcharges the piped drainage system and flows through residential properties at the following locations:
  - In a westerly direction between the northern end of Whitbeck Place and Mellfell Road in a 20% AEP storm event (refer **Figure 6.2**).
  - In a south-westerly direction between the intersection of Sherringham Road and Pendock Road and McHenry Road in a 10% AEP storm event (refer **Figure 6.3**).
  - In a southerly direction to the east of Callisto Drive between its intersections with Marrett Way and Mellfell Road in a 2% AEP storm event (refer **Figure 6.4**).

- In a westerly direction between the Cranebrook Reservoir and the northern end of Whitbeck Place in a 2% AEP storm event (refer **Figure 6.5**).

➢ **Figure 6.6** shows that the depth of flow along the abovementioned overland flow paths is generally between 200 mm to 400 mm in a 1% AEP storm event, with depths of up to 600 mm shown to occur in isolated areas where water ponds on the upstream side of existing buildings.

➢ **Figure 6.9** shows that the depth of overland flow through existing residential properties during a PMF event is generally less than 400 mm, with depths of up to 1.1 m shown to occur in isolated areas where water ponds on the upstream side of existing buildings.

The key features of local catchment flood behaviour in the **Andrews Road Catchment** (refer sheet 2 in figure set) are as follows:

➢ **Figure 6.10** shows that the majority of the piped drainage system is at capacity in a 0.5 EY design storm event.

➢ **Table 6.4** shows that the spillways of three regional detention basins that are located in the catchment are surcharged as follows (refer **Figure 6.10**, sheet 2 and 3 for plan location of each):

- The Andrews Road Basin No. 1 (refer B19) surcharges in storm events as frequent as 0.5 EY.
- The Andrews Road Basin No. 2 (refer B20) commences to surcharge in a 20% AEP storm event.
- The Cooper Street Basin (refer B18) commences to surcharge in a 2% AEP storm event.

➢ Floodwater surcharges the piped drainage system and flows through residential properties at the following locations:

- In a westerly direction between Moonbi Road and Fox Place in a 0.5 EY storm event (refer **Figure 6.1**).
- In a southerly direction from the intersection of Andrews Road and Greygums Road to Allard Street in a 20% AEP storm event (refer **Figure 6.2**).
- Between Kareela Avenue and Moonbi Road in a 20% AEP storm event (refer **Figure 6.2**).
- Along the alignment of the piped drainage system that runs through the rear of residential properties that are location between Illawong Avenue and Caloola Avenue in a 20% AEP storm event (refer **Figure 6.2**).
- In a westerly direction between Cudgee Road and Illawong Avenue in a 20% AEP storm event (refer **Figure 6.2**).
- In a northerly direction between Hilltop Road and Illawong Avenue in a 20% AEP storm event (refer **Figure 6.2**).
- In a westerly direction from Grange Crescent and Allard Street in a 10% AEP storm event (refer **Figure 6.3**).
- In a westerly direction between Orana Avenue and Arakoon Avenue in a 10% AEP storm event (refer **Figure 6.3**).

- In a westerly direction between Newham Drive and The Northern Road in a 1% AEP storm event (refer **Figure 6.6**).

- **Figure 6.6** shows that the depth of flow along the abovementioned overland flow paths is generally between 300 mm to 500 mm in a 1% AEP storm event, with depths of up to 1 m shown to occur in isolated areas where water ponds on the upstream side of existing buildings.
- **Figure 6.11** shows that floodwater commences to inundate Andrews Road at a location approximately 280 m to the east of its intersection with Lambridge Place in a 0.2% AEP storm event (refer SHL H04 on **Figure 6.10** for location).
- **Figure 6.9** shows that the depth of overland flow along the abovementioned flow paths exceed 1 m during a PMF event, with depths of up to 2.5 m shown to occur in isolated areas where water ponds on the upstream side of existing buildings.

The key features of local catchment flood behaviour in the **Boundary Creek Catchment** (refer sheet 3 in figure set) are as follows:

- While floodwater is generally contained within the inbank area of Boundary Creek for storm events up to 1% AEP in intensity, it surcharges the banks at the following locations:
  - On the western side of Hickeys Land Reserve where floodwater surcharges the right (northern) bank in a 10% AEP storm event and flows in a northerly direction to the Cranebrook Road South Catchment.
  - Approximately 150 m west of Hickeys Lane Reserve where floodwater commences to surcharge the banks of the creek in a 20% AEP event and inundate low lying land in the vicinity of the Penrith Water Recycling Plant.
  - Adjacent to Dean Place where floodwater surcharges the left (southern) bank of the creek and inundates the northern end of the street in a 10% AEP storm event.
- **Figure 6.10** shows that while the majority of the trunk drainage line that runs between the Parker Street Reserve and the upstream end of Boundary Creek is at capacity in a 0.5 EY design storm event, the reach of the drainage line downstream of Robert Street reaches capacity during storm events ranging between 5% and 10% AEP in intensity.
- **Figure 6.10** also shows that the majority of the piped drainage system to the east of Coombes Drive is at capacity in a 0.5 EY storm event, while the parts of the catchment that have been more recently developed to the east generally have a higher capacity (i.e. they commence to flow full in storm events less frequent that 20% AEP).
- **Table 6.4** shows that the spillways of four regional detention basins that are located in the catchment are surcharged as follows (refer **Figure 6.10**, sheet 2 and 3 for plan location of each):
  - The Coreen Avenue Basin (refer B24) commences to surcharge in a 20% AEP storm event.
  - The King Street Basin No. 3 (refer B23) commences to surcharge in a 5% AEP storm event.
  - The King Street Basin No. 1 (refer B21) commences to surcharge in a 2% AEP storm event.
  - The King Street Basin No. 2 (refer B22) commences to surcharge in a 1% AEP storm event.

- Floodwater surcharges the piped drainage system and flows through residential properties at the following locations:
  - In a westerly direction between Bel-Air Road and Sunshine Avenue in a 20% AEP storm event (refer **Figure 6.2**).
  - In a westerly direction between Orton Close and Bel-Air Road and between Sunshine Avenue and Coombes Drive in a 10% AEP storm event (refer **Figure 6.3**).
  - Along the alignment of the existing piped drainage line that runs in a westerly direction between Robert Street and Coreen Avenue in a 5% AEP storm event (refer **Figure 6.4**).
  - In a westerly direction between King Street and Blaxland Avenue in a 5% AEP storm event (refer **Figure 6.4**).
- **Figure 6.6** shows that the depth of flow along the abovementioned overland flow paths generally does not exceed 500 mm in a 1% AEP storm event, except in the vicinity of Sunshine Avenue where the depth of overland flow exceeds 1 m.
- **Figure 6.3** shows that floodwater commences to surcharge the existing piped drainage system in Coreen Avenue and Coombes Drive in a 10% AEP storm event. Floodwater that surcharges the existing piped drainage system at this location in a 1% AEP storm event inundates existing industrial development to depths of up to 500 mm.
- **Figure 6.11** shows that the Castlereagh Road crossing of Boundary Creek remains flood free in a 0.2% AEP local catchment storm event (refer SHL H05 on **Figure 6.10** for plan location).
- **Figure 6.11** shows that floodwater commences to inundate Coreen Avenue at a location approximately 140 m to the east of its intersection with Castlereagh in a 10% AEP storm event Road (refer SHL H06 on **Figure 6.10** for plan location). Coreen Avenue is inundated to a maximum depth of about 900 mm at this location in a 1% AEP storm event.
- **Figure 6.9** shows that the depth of overland flow along the abovementioned flow paths through residential development exceeds 1 m during a PMF event, with depths of up to 2.8 m occurring in some areas where water ponds on the upstream side of existing buildings.
- **Figure 6.9** also shows that the existing industrial development in the vicinity of Coreen Avenue and Coombes Drive will be inundated to depths of up to 1.6 m in a PMF event.

The key features of local catchment flood behaviour in the **North Penrith Catchment** (refer sheet 3 in figure set) are as follows:

- **Figure 6.10** shows that the majority of the piped drainage system is at capacity in a 0.5 EY or 20% AEP design storm event.
- Floodwater surcharges the piped drainage system and flows through residential properties at the following locations:
  - On the eastern side of King Street to the south of its intersection of Copeland Street in a 20% AEP storm event (refer **Figure 6.2**).
  - To the north of the Main Western Railway where floodwater that is unable to flow through the existing pipe beneath the railway ponds and commences to back

flood into existing development that is located to the north-east of the intersection of The Crescent and Haynes Street in a 20% AEP storm event (refer **Figure 6.2**).

  - Between Gascoigne Street and Copeland Street in a 10% AEP storm event (refer **Figure 6.3**).
  - West of King Street approximately 40 m to the south of its intersection with Gascoigne Street in a 10% AEP storm event (refer **Figure 6.3**).
  - Between Parker Street and Gascoigne Street in a 2% AEP storm event (refer **Figure 6.5**).

- **Figure 6.6** shows that the depth of flow along the abovementioned overland flow paths generally does not exceed 400 mm in a 1% AEP storm event, with depths of up to 800 mm occurring in isolated areas.
- **Figure 6.9** shows that the depth of overland flow along the abovementioned flow paths through residential development exceeds 1 m in a PMF event.

The key features of local catchment flood behaviour in the **Penrith Lakes Local Catchment** (refer sheet 2 in figure set) are as follows:

- **Figure 6.10** shows that the majority of the piped drainage system is at capacity in a 0.5 EY or 10% AEP design storm event.
- Floodwater surcharges the piped drainage system and flows through industrial development in the area bounded by Camden Street to the east, Gordon Street to the south, Leland Street to the west and Old Castlereagh Road to the north in a 10% AEP storm event (refer **Figure 6.3**). **Figure 6.6** shows that the depth of overland flow in his area generally does not exceed 450 mm in a 1% AEP storm event.
- **Figure 6.9** shows that the depth of overland flow through industrial development exceeds 1 m in a PMF event.
- **Figure 6.9** shows that a 700 m section of Castlereagh Road immediately north of its intersection with Andrews Road is inundated in a PMF event.

The key features of local catchment flood behaviour in the **Nepean River Local Catchment** (refer sheet 3 in figure set) are as follows:

- **Figure 6.10** shows that the piped drainage system in the northern portion of the catchment is at capacity in a 20% AEP design storm event, while the piped drainage system in the southern portion commences to reach its capacity in a 5% AEP storm event.
- **Figure 6.9** shows that the depth of overland flow through industrial development generally does not exceed 500 mm in a PMF event.

Table 6.5 over the page sets out the number of properties that are inundated either partially or entirely by floodwater (to depths greater than 150 mm for the assessed storm events.

6.2.5. Potential Flood Mitigation Measures at Identified Hot Spots

Table 6.6 on page 45 contains a qualitative assessment of potential flood mitigation measures for the identified hot spots described in **Section 6.2.4**.

TABLE 6.5
NUMBER OF FLOOD AFFECTED PROPERTIES IN STUDY AREA

Design Storm Event	Study Catchment								Total
	Duralia Lake	Boundary Road	Cranebrook Road South	Andrews Road	Penrith Lakes Local	Nepean River Local	Boundary Creek	North Penrith	
50% AEP	33	45	179	52	5	4	25	4	347
20% AEP	36	58	189	123	5	4	46	18	479
10% AEP	45	75	220	177	11	7	73	31	639
5% AEP	47	85	213	202	14	10	92	43	706
2% AEP	50	117	277	236	15	15	105	62	877
1% AEP	51	119	276	247	18	17	127	71	926
0.5% AEP	51	124	296	272	19	23	132	72	989
0.2% AEP	55	138	346	288	22	25	163	77	1,114
PMF	77	338	921	537	49	47	464	110	2,543

TABLE 6.6
QUALITATIVE ASSESSMENT OF POTENTIAL FLOOD MITIGATION MEASURES AT IDENTIFIED HOT SPOTS

Hot Spot Location	Study Catchment	Potential Flood Mitigation Measure	Qualitative Assessment
Karen Court	Duralia Lake Catchment	• Upgrade existing stormwater inlet capacity and existing 1 off 375 mm diameter pipe draining cul-de-sac in Karen Court	• Council maintains easement along alignment of existing pipe. • Remove shallow inundation in the front of up to eight residential properties in a 1% AEP storm event. • Reduce ponding in road and improve evacuation. • Unlikely to have significant economic benefits.
Kenilworth Crescent	Cranebrook Road South	• Upgrade existing stormwater inlet capacity and existing 1 off 750 mm diameter pipe that runs on the southern side of Kenilworth Crescent between Woodside Glen and Grays Lane.	• Remove shallow inundation in the front of up to four residential properties. • Unlikely to have significant economic benefits.
Whitbeck Place	Cranebrook Road South	• Upgrade existing 1 off 375 mm diameter pipe that runs between the northern end of Whitbeck Place and Mellfell Road.	• Council maintains easement along alignment of existing drainage line. • Remove inundation in up to three residential properties in a 1% AEP storm event. • Reduce deep ponding in rear of one residential property in a 1% AEP storm event. • Unlikely to have significant economic benefits. • Inspection required to confirm surveyed dimensions of existing pipe.
Intersection of Pendock Road and Sherringham Road	Cranebrook Road South	• Upgrade existing stormwater inlet capacity in Sherringham Road and existing piped draining system between intersection with Pendock Street and downstream detention basins	• Council maintains easement along alignment of existing pipe. • Potentially requires the upgrade of about 400 m of piped drainage system to a depth of up to 5 m. • Remove hazardous flooding in three residential properties in a 1% AEP storm event. • Unlikely to have significant economic benefits.
		• Potential to construct detention basin in reserve that is located to the north-west of intersection.	• Extent of land may not be large enough for an effective detention basin. • Construction of basin will require the removal of a significant number of trees in the reserve.

Cont'd Over

TABLE 6.6 (Cont'd)
QUALITATIVE ASSESSMENT OF POTENTIAL FLOOD MITIGATION MEASURES AT IDENTIFIED HOT SPOTS

Hot Spot Location	Study Catchment	Potential Flood Mitigation Measure	Qualitative Assessment
Intersection of Trinity Drive and The Northern Road	Andrews Road	• Investigate potential to construct new drainage line along western side of The Northern Road and the southern side of Andrews Road and discharge to Andrews Road Basin No. 1.	• New drainage line is about 800 m long. • Removes overland flow through five residential properties in a 1% AEP storm event. • Unlikely to have significant economic benefits.
Fox Place		• Upgrade existing drainage line that runs between Kareela Avenue and the Cooper Street Basin.	• Council maintains easement along alignment of existing drainage line. • May remove overland flow through up to 11 residential properties in a 1% AEP storm event.
		• Investigate if surcharge pit on eastern side of Cooper Street Basin is causing a backwater up existing piped drainage system, thereby reducing the capacity of the existing piped drainage system upstream.	
Cudgee Place		• Limited potential to upgrade existing piped drainage system due to downstream constraints. • Investigate potential for voluntary purchase due to depth of inundation and hazard.	• Obtain survey of existing floor levels for incorporation in flood damages assessment to be undertaken as part of future *FRMS&P*.
Hilltops Road to Illawong Avenue		• Limited potential to upgrade existing piped drainage system due to downstream constraints. • Investigate potential for detention basin in vacant land immediately to the north of Hilltops Road.	• Confirm if vacant land is slated for future transport / utilities. • Difficult to construct detention basins large enough to remove flooding problem based on natural topography.
		• Investigate potential for voluntary purchase due to depth of inundation and hazard.	• Obtain survey of existing floor levels for incorporation in flood damages assessment to be undertaken as part of future *FRMS&P*.

Cont'd Over

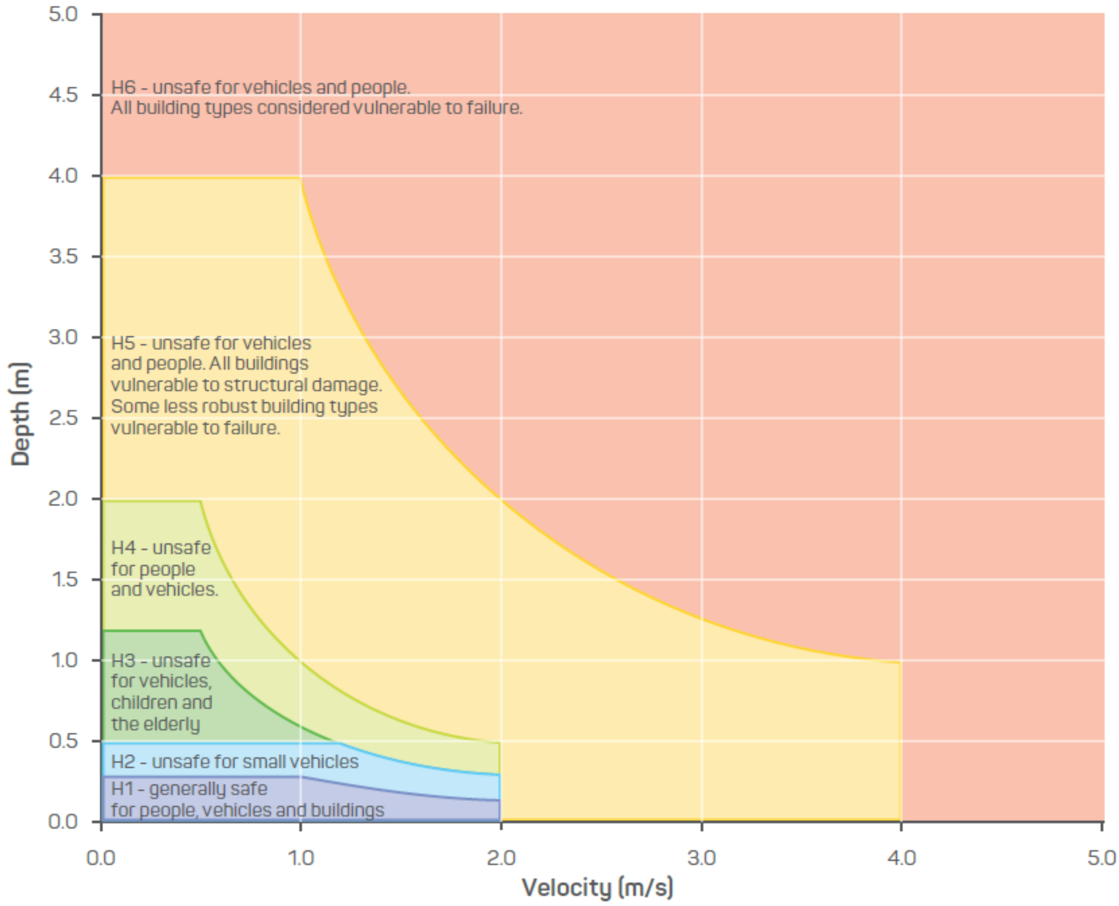
TABLE 6.6 (Cont'd)
QUALITATIVE ASSESSMENT OF POTENTIAL FLOOD MITIGATION MEASURES AT IDENTIFIED HOT SPOTS

Hot Spot Location	Study Catchment	Potential Flood Mitigation Measure	Qualitative Assessment
Bel-Air Road and Sunshine Avenue	Boundary Creek	• Upgrade existing piped drainage system between Bel-Air Road and Coombes Drive, noting that the existing piped drainage system in Coombes Drive has capacity to convey more water in 0.5 EY and 20% AEP storm events.	• Council maintains easement along alignment of existing pipe. • Remove overland flow and ponding in up to 12 residential properties.
		• Investigate potential for voluntary purchase due to depth of inundation and hazard.	• Obtain survey of existing floor levels for incorporation in flood damages assessment to be undertaken as part of future *FRMS&P*.
Between Coreen Avenue and Boundary Creek		• Install one way value on outlet of existing piped drainage system drainage that drains low point in Coreen Avenue that is located about 140 m east of Castlereagh Road and discharges to Boundary Creek.	• Dimensions of existing drainage system could not be confirmed as land owner would not grant surveyor permission to enter. • Confirm dimension of drainage system and whether flood gate is presently installed on existing outlet. • Flood gate has the potential to prevent back flooding of existing substation on the southern side of Thornton Drive in Nepean River flood events.
King Street and The Crescent	North Penrith	• Investigate potential to upgrade drainage system beneath railway line to reduce backwater on existing piped drainage system.	• Ensure North Penrith catchment is incorporated in future *Flood Study* or *FRMS&P* investigation for the Penrith CBD catchment.

6.3 Flood Hazard Zones and Floodways

6.3.1. Flood Hazard Vulnerability Classification

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



Flood Hazard Vulnerability Classification diagrams for the 5, 1 and 0.5% AEP flood events, as well as the PMF event based on the procedures set out in ARR, 2019 are presented on **Figures 6.12**, **6.13**, **6.14** and **6.15**, respectively.

It was found that areas classified as H5 and H6 are generally limited to the inbank areas of Boundary Creek and incised channelled areas, local farm dams and the man-made lakes in the lower reaches of the study catchments in storm events up to 0.5% AEP in intensity.

The overland flow paths in the densely urbanised residential parts of the study area are generally classified as either H1 or H2 in storms up to a 0.5% AEP event in intensity, with the following exceptions which are classified as H3:

<u>**Cranebrook Road South Catchment**</u>

- between the northern end of Whitbeck Place and Mellfell Road in a 5% AEP storm event (refer **Figure 6.12**, sheet 2); and

- between the intersection of Sherringham and Pendock Roads and McHenry Road in a 1% AEP storm event (refer **Figure 6.13**, sheet 2).

Andrew Road Catchment

- between Moonbi Road and Fox Place in a 5% AEP storm event (refer **Figure 6.12**, sheet 3);
- along the alignment of the piped drainage system that runs through the rear of residential properties that are location between Illawong Avenue and Caloola Avenue in a 5% AEP storm event (refer **Figure 6.12**, sheet 3);
- between Hilltop Road and Illawong Avenue in a 5% AEP storm event (refer **Figure 6.12**, sheet 3);
- between Cudgee Road and Illawong Avenue in a 1% AEP storm event (refer **Figure 6.13**, sheet 3); and
- between Newham Drive and The Northern Road in a 1% AEP storm event (refer **Figure 6.13**, sheet 3).

Boundary Creek Catchment

- between Bel-Air Road and Coombes Drive in a 5% AEP storm event (refer **Figure 6.12**, sheet 3); and
- along the alignment of the existing piped drainage line that runs in a westerly direction between Robert Street and Coreen Avenue in a 1% AEP storm event (refer **Figure 6.13**, sheet 3).

North Penrith Catchment

- on the eastern side of King Street to the south of its intersection of Copeland Street in a 20% AEP storm event (refer **Figure 6.12**, sheet 3);
- between King Street and The Crescent in a 1% AEP storm event (refer **Figure 6.13**, sheet 3); and
- between Jenkins Avenue and King Street in a 0.5% AEP storm event (refer Figure 6.14, sheet 3).

The overland flow paths in the vicinity of industrial development that is located in the study area are generally classified as either H1 or H2 in storms up to a 0.5% AEP event, except in the areas where floodwater ponds on the upstream side of roads where it is generally classified as either H3 or H4.

For the PMF event, the hazard in the detention basins and man-made lakes generally increase to H6, while the hazard along the abovementioned flowpaths increases to between H4 and H5. The hazard category along the majority of the remaining drainage lines increases to between H2 and H3 during a storm event of this intensity.

6.3.2. Hydraulic Categorisation of the Floodplain

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

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

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

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

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

Floodplain Risk Management Guideline No. 2 Floodway Definition, offers guidance in relation to two alternative procedures for identifying floodways. They are:

- **Approach A**. Using a *qualitative approach* which is based on the judgement of an experienced hydraulic engineer. In assessing whether or not the area under consideration was a floodway, the qualitative approach would need to consider; whether obstruction would divert water to other existing flow paths; or would have a significant impact on upstream flood levels during major flood events; or would adversely re-direct flows towards existing development.
- **Approach B**. Using the hydraulic model, in this case TUFLOW, to define the floodway based on *quantitative experiments* where flows are restricted or the conveyance capacity of the flow path reduced, until there was a significant effect on upstream flood levels and/or a diversion of flows to existing or new flow paths.

One quantitative experimental procedure commonly used is to progressively encroach across either floodplain towards the channel until the designated flood level has increased by a significant amount (for example 0.1 m) above the existing (un-encroached) flood levels. This indicates the limits of the hydraulic floodway since any further encroachment will intrude into that part of the floodplain necessary for the free flow of flood waters – that is, into the floodway.

The *quantitative assessment* associated with **Approach B** is technically difficult to implement. Restricting the flow to achieve the 0.1 m increase in flood levels can result in contradictory results, especially in unsteady flow modelling, with the restriction actually causing reductions in computed levels in some areas due to changes in the distribution of flows along the main drainage line.

Accordingly the *qualitative approach* associated with **Approach A** was adopted, together with consideration of the portion of the floodplain which conveys approximately 80% of the total flow and also the findings of *Howells et al, 2004* who defined the floodway based on velocity of flow and depth. Based on the findings of a trial and error process, the following criteria were adopted for identifying those areas which operate as a "floodway" in a 1% AEP event:

- Velocity x Depth greater than 0.1 m^2/s[8] **and** Velocity greater than 0.25 m/s; or
- Velocity greater than 1 m/s.

[8] Note that a Velocity x Depth product of 0.1 m^2/s was used to define floodways as part of a recent flooding investigation that was undertaken in the nearby Emu Plains catchment (i.e. BMT, 2020).

Flood storage areas are identified as those areas which do not operate as floodways in a 1% AEP event but where the depth of inundation exceeds 200 mm.[9] The remainder of the flood affected area was classified as flood fringe.

Figures 6.16, **6.17** and **6.18** show the division of the floodplain into floodway, flood storage and flood fringe areas for the 5, 1 and 0.5% AEP storm events, respectively, while **Figure 6.19** shows the hydraulic categorisation of the floodplain for the PMF. The figures also show the parts of the floodplain that primarily function as flood storage areas, but also act to convey a large portion of the total flow and hence also function as floodways (refer areas denoted "Floodway / Flood Storage").

As the hydraulic capacity of the piped drainage system is not large enough to convey the 1% AEP flow, a significant portion of the total flow is conveyed on the floodplain. As a result, areas which lie on the major overland flow paths described in **Sections 6.2.4** are generally defined as floodways.

As the flood mapping is only shown where depths are greater than 150 mm, and flood storage areas have been defined as areas where depths are greater than 200 mm, the majority of the remaining land inundated outside the extent of the floodway is defined as flood storage area.

6.4 Flood Emergency Response Classification

Flood emergency response categories may be assigned to flood affected areas in accordance with the definitions contained in AIDR, 2017. The flood emergency response classifications are based on whether or not an area is flooded, whether the flooded area has an exit to flood-free land and the consequence of flooding on the area for a given AEP storm event. This information will assist NSW SES in emergency management planning during flood events.

Flood Emergency Response Classification diagrams for the 20%, 5%, 1% and 0.5% AEP flood events, as well as the PMF based on the procedures set out in AIDR, 2017 are presented on **Figures 6.20**, **6.21**, **6.22**, **6.23** and **6.24**, respectively. Sensitivity Studies

6.5 Sensitivity Studies

6.5.1. General

The sensitivity of the hydraulic model was tested to variations in model parameters such as rainfall losses, hydraulic roughness, downstream tailwater conditions and the partial blockage of the major hydraulic structures by debris. The main purpose of these studies was to give some guidance on:

a) the freeboard to be adopted when setting minimum floor levels of development in flood prone areas, pending the completion of the future *FRMS&P*; and

b) areas where additional flood related planning controls should be implemented due to the development of new hazardous flow paths.

The results of the sensitivity analyses are presented in a series of figures that are bound in **Appendix F** of this report.

[9] Note that depths of 200 mm or greater were used to define flood storage areas as part of recent flooding investigations that have been undertaken in the adjacent Peach Tree Creek and nearby Emu Plains catchments.

6.5.2. Sensitivity to Rainfall Losses

Figures F1.1, **F1.2** and **F1.3** (2 sheets each) show the difference in peak flood levels (i.e. the "afflux") that increasing the initial loss values by 20% and the continuing loss value by 0.5 mm/hr would have for the 5%, 1% and 0.5% AEP storm events, respectively. Increasing the initial and continuing loss values reduces peak levels by less than 20 mm for the range of assessed storm events.

Figures F1.4, **F1.5** and **F1.6** (2 sheets each) show the impact that decreasing the initial loss values by 20% and the continuing loss value by 0.5 mm/hr would have for the 5%, 1% and 0.5% AEP storm events, respectively. Decreasing the initial and continuing loss values increases peak levels by a maximum of about 30 mm for the range of assessed storm events.

6.5.3. Sensitivity to Hydraulic Roughness

Figures F2.1, F2.2 and **F2.3** (2 sheets each) show the afflux for the 5%, 1% and 0.5% AEP storm events, respectively resulting from an assumed 20% increase in hydraulic roughness (compared to the values given in **Table 4.2**).

The typical increases in peak flood level in the major watercourses and standing water bodies are generally in the range 20 to 50 mm for the assessed storm events, with increases of up to 100 mm shown to occur in isolated areas. Increases in peak flood levels in areas subject to major overland flow are generally in the range 10 to 20 mm. The increases in peak flood level have a negligible impact on the extent of inundation. It is noted that the increase in assumed hydraulic roughness in the upper reaches of the study catchments has an attenuating effect on peak flow, resulting in minor reductions in peak flood levels in the lower reaches of the study catchments.

Figures F2.4, F2.5 and **F2.6** (2 sheets each) show the impact that an assumed 20% decrease in hydraulic roughness (compared to the values given in **Table 4.2**) has on peak flood levels for the 5%, 1% and 0.5% AEP storm events, respectively.

While the reduction in hydraulic roughness values results in a general reduction in peak flood levels by up to 30 mm for the assessed storm events, it has had the effect of accelerating the flow of stormwater, resulting in minor increases in peak flood levels in the lower reaches of the study catchments.

6.5.4. Sensitivity to Partial Blockage

Figures F3.1, **F3.2** and **F3.3** (2 sheets each) show the impact that a partial blockage of the drainage system (which has been incorporated in the design flood modelling present in **Section 6.2** of this report) has on flood behaviour for the 5%, 1% and 0.5% AEP storm events, respectively. Note that a positive afflux indicates that the modelled peak flood levels resulting from a partial blockage of the drainage system are higher than those derived assuming free flowing (i.e. unblocked) conditions.

Partial blockage of the drainage system has a negligible impact on flood behaviour in the upper reaches of the study catchments that are subject to major overland flow, but increases peak flood levels on the upstream side of road crossings and in the regional type flood detentions basins by up to 200 mm. The exception is in Coreen Avenue at a location approximately 140 m to the east of its intersection with Castlereagh Road (refer sheet 2 of figure set), where peak flood levels are increased by between 200 mm and 300 mm.

6.5.5. Sensitivity to Downstream Tailwater Level Conditions

Figure F4.1 (2 sheets) shows the impact that adopting lower starting tailwater levels in the Penrith Lakes based on the "Operating Levels" set out in **Table 2.2** has on flood behaviour in a 1% AEP storm event. **Figure F4.1** shows that adopting lower tailwater levels reduces peak flood levels in Cranebrook Lake and the Still Basin but has no impact on flood behaviour elsewhere in the study catchments.

6.5.6. Differences in Design Flood Estimation – ARR 1987 versus ARR 2019

For comparison purposes, design flood modelling was undertaken for the 5% and 1% AEP design storm events based on the procedures set out in the 1987 edition of *Australian Rainfall and Runoff* (**ARR 1987**) (The Institution of Engineers Australia, 1987).

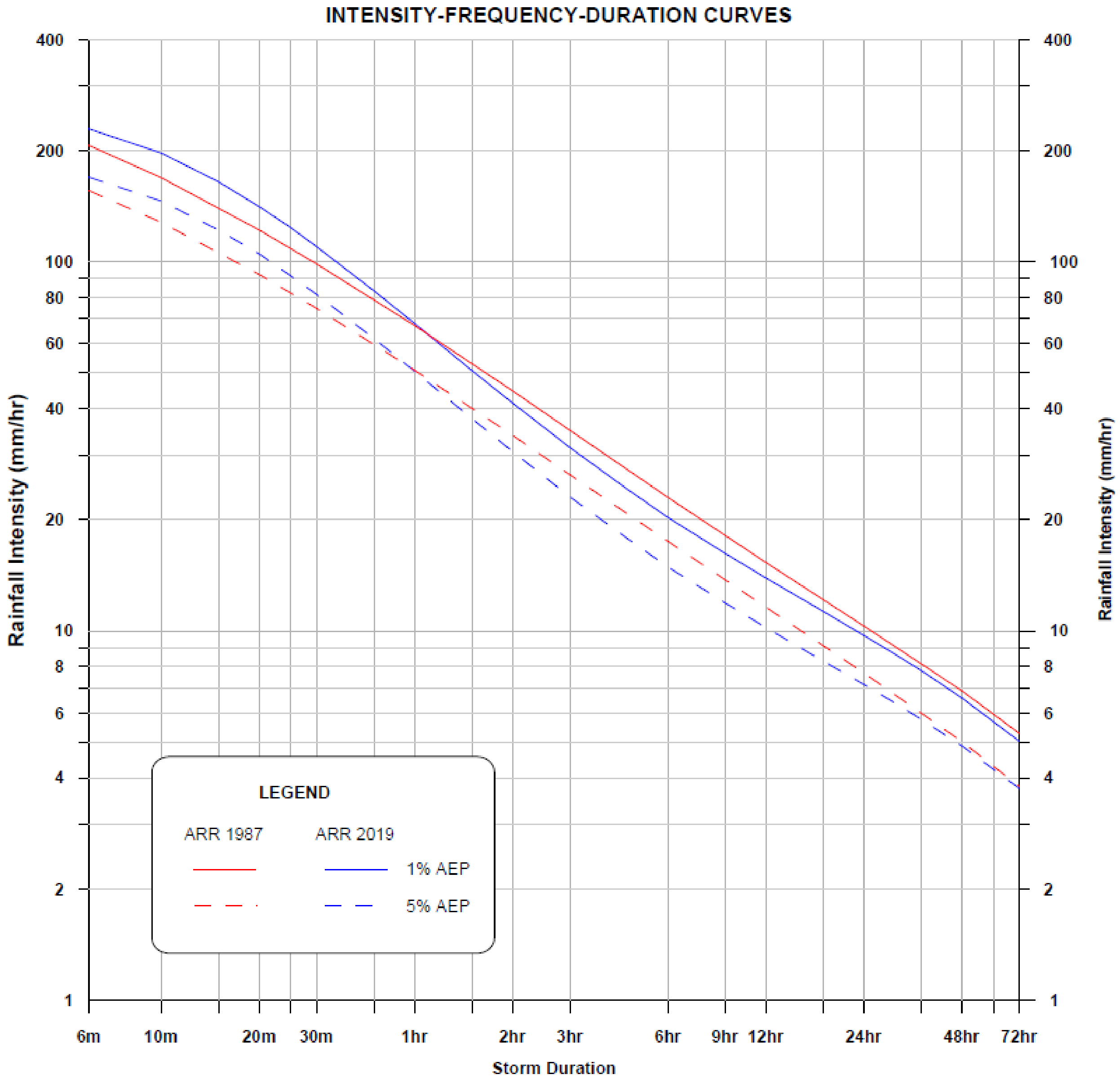


TABLE 6.7
COMPARISON OF ADOPTED MODEL PARAMETERS
ARR 2019 VERSUS ARR 1987

Model Parameter	ARR 2019	ARR 1987
Design Rainfall	BoM 2016 IFD's	BoM 1987 IFD's
Initial Loss	14.6 mm – 16.8 mm (5% AEP) 10.4 mm – 12.6 mm (1% AEP)[1]	15 mm[2]
Continuing Loss	1.4 mm/hr[3]	2.5 mm/hr[2]
Temporal Patterns	Suite of ten 2016 temporal patterns per event / duration	Single 1987 temporal patterns for Zone 1 per event / duration

1. Based on the Probability Neutral Burst Initial Loss values taken from the *ARR Data Hub*.
2. Based on *Initial Losses for Design Flood Estimation in New South Wales* (Walsh et. al., 1991).
3. Derived by multiplying the raw continuing loss value obtained from the ARR Data Hub by a factor of 0.4.

The illustration above shows a comparison of the intensity-frequency-duration curves for the 5% and 1% AEP design events derived using the procedures set out in ARR 1987 and ARR 2019, while **Table 6.7** shows a comparison of the adopted model parameters.
The key differences in the adopted model parameters are as follows:

- The ARR 2019 derived IFD's are lower than the ARR 1987 derived values for storm durations less than 1 hour and higher for longer durations storm.
- While the initial loss values are comparable between the two procedures for the 5% AEP storm event, the ARR 2019 initial loss value for 1% AEP storms are about 2-3 mm lower than the single ARR 1987 initial loss value of 15 mm.
- The ARR 2019 derived continuing loss value of 1.4 mm/hr is about 40% lower than the ARR 1987 value of 2.5 mm/hr.
- The shape of the suite of ten ARR 2019 temporal patterns vary significantly to the single ARR 1987 temporal patterns.

Figures F5.1 and **F5.2** (2 sheets each) show the difference in the extent and depth of inundation resulting on the application of the procedures set out in ARR 1987 and ARR 2019 for the 5% and 1% AEP events, respectively. Note that a positive afflux indicates that the modelled peak flood levels derived using the procedures set out in ARR 1987 are higher than those derived using ARR 2019.

Figure F5.1 shows that peak flood levels derived using the procedures set out in ARR 1987 are about 10-50 mm higher than those derived using the ARR 2019 approach in the upper reaches of the study catchments subject to major overland flow and up to 250 mm higher in the flood storage areas in the lower reaches of the study catchments.

Figure F5.2 shows that peak flood levels derived using the procedures set out in ARR 1987 are about 10-50 mm higher than those derived using the ARR 2019 approach in the detention basins that are located in the study catchment and lower by up to 30 mm in other areas. The increase in peak flood levels in the storages is a result of higher rainfall excess for longer duration storms, while the reduction in peak flood levels in other parts of the study catchments is likely caused by differences in the temporal variability of the design rainfall.

6.5.7. Sensitivity to Cumulative Development

While the study catchments are already highly developed, there is potential for further redevelopment or intensification of development. Future development has the potential to increase the rate and volume of runoff conveyed by the various watercourses, as well as increase the frequency of surcharge of the local stormwater drainage system. It is also likely to result in changes to the existing drainage system.

While there is evidence that Council is requiring developers to incorporate flow control measures such as detention basins in residential subdivisions, infill development at an individual allotment scale has the potential to increase flow in the receiving drainage lines.

As the future scope of urbanisation at the time of the present study is not quantifiable, it has been assumed that the current impervious percentage of the catchment would increase by 10% to a maximum of 90% in areas zoned for residential type development and 100% in areas zoned for commercial and industrial type development.

Figures F6.1, **F6.2** and **F6.3** (2 sheets each) show the potential impact that future development within the study catchments could have on flood behaviour for the 5%, 1% and 0.5% AEP storm events, respectively. Peak flood levels in the portion of the catchment north of boundary that is currently zoned for rural type residential would generally increase by up to 50 mm, while peak flood levels in the more densely urbanised areas to the south would generally increase by up to 20 mm.

6.6 Climate Change Sensitivity Analysis

6.6.1. General

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

DPE recommends that its guideline *Practical Considerations of Climate Change, 2007* be used as the basis for examining climate change induced increases in rainfall intensities in projects undertaken under the State Floodplain Management Program and NSWG, 2005. The guideline recommends that until more work is completed in relation to the climate change impacts on rainfall intensities, sensitivity analyses should be undertaken based on increases in rainfall intensities ranging between 10 and 30 per cent. On current projections the increase in rainfalls within the service life of developments or flood management measures is likely to be around 10 per cent, with the higher value of 30 per cent representing an upper limit. Under present day climatic conditions, increasing the 1% AEP design rainfall intensities by 10 per cent is analogous to a 0.5% AEP flood; and increasing those rainfalls by 30 per cent is analogous to a 0.2% AEP event.

The impacts of climate change and associated effects on the viability of floodplain risk management options and development decisions may be significant and will need to be taken into account in the future *FRMS&P* for the study area using site specific data.

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

In the *FRMS&P* it will be necessary to consider the impact of climate change on flood damages to existing development. Consideration will also be given both to setting floor levels for future development and in the formulation of works and measures aimed at mitigating adverse effects expected within the service life of development.

Mitigating measures which could be considered in the *FRMS&P* include the implementation of structural works such as levees and channel improvements, improved flood warning and emergency management procedures and education of the population as to the nature of the flood risk.

6.6.2. Sensitivity to Increased Rainfall Intensities

As mentioned, the investigations undertaken at the flood study stage are mainly seen as sensitivity studies pending more detailed consideration in the *FRMS&P*. For the purposes of the present study, the design rainfalls for 0.5 and 0.2 per cent AEP events were adopted as being analogous to flooding which could be expected should present day 1% AEP rainfall intensities increase by 10 and 30 per cent, respectively.

Figure F7.1 (2 sheets) shows the afflux resulting from a 10 per cent increase in 1% AEP rainfall intensities. The increase in peak flood levels generally varies between 10 to 50 mm, with increases of up to 100 mm shown to occur in isolated areas in the lower reaches of the study catchments.

Figure F7.2 (2 sheets) shows the afflux for a 30 per cent increase in 1% AEP rainfall intensities.[10] Peak flood levels generally increase by between 50 and 200 mm, with increases of up to 380 mm shown to occur in isolated areas in the lower reaches of the study catchments. Decreases in peak flood level are shown to occur in Cranebrook Lake due to the higher blockage value that has been applied to the culvert beneath Cranebrook Road immediately to the east of its intersection with Castlereagh Road in a 0.2% AEP storm event when compared to a 1% AEP event.

Figure F7.3 (2 sheets) shows the increase in the extent of land affected by floodwater should 1% AEP rainfall intensities increase by 10 or 30 per cent. The extent of land that is affected by floodwater increases significantly at the following locations:

- in the area bound by Boundary Creek to the north Combewood Avenue and Coombes Drive to the east, the Main Western Railway to the south and Castlereagh Road to the west (refer sheet 2);
- between the Sydney Water Recycling Plant and Andrews Road (refer sheet 2); and
- between Old Castlereagh Road and the Final Basin to the west of Castlereagh Road (refer sheet 2).

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

[10] Probability neutral blockage values derived using the procedures set out in ARR 2019 were also incorporated in the assessment, noting that the blockage values applied to the 0.2% AEP storm event are higher than those applied to the 1% and 0.5% AEP storm events.

6.7 Flood Planning Information

6.7.1. Modelling of Coincident Flooding on the Nepean River

As discussed in **Section 6.1.2**, the local catchment flood modelling undertaken for the present study assumes that a 5% AEP Nepean River flood occurs coincident with a 1% AEP local catchment storm event. In order to define the 1% AEP design flood envelope in the study area, the TUFLOW model that was developed as part of the present study was utilised to assess the impact that a 1% AEP Nepean River would have on a 5% AEP local catchment flood behaviour within the study area.

Figure 6.25 shows the stage hydrographs that were extracted from the *Nepean River Flood Study* (Advisian, 2018) at the confluence of the Nepean River and Boundary Creek, as well as in the Regatta Lake and Lake A in Penrith Lakes and then applied to the TUFLOW model. Lag times were applied to the local catchment discharge hydrographs to ensure that the flood peak of the local catchment runoff coincided with the flood peak in the Nepean River.

The results of the TUFLOW model that was developed as part of the present study show that a total flow of about 28 m^3/s surcharges the right (northern) bank of Boundary Creek in a 1% AEP Nepean River flood event, where it flows in a northerly direction across Andrews Road to Lake Cranebrook. It is noted that the findings of the present study differ to those presented in Advisian, 2018 due to the incorporation of hydraulic structures such as transverse drainage culverts and water level control weirs, as well as updated topographic data in the hydraulic model.

Figure 6.26 (3 sheets) shows depth and extent of inundation of the 1% AEP Nepean River and local catchment flood envelope in the study area, which comprises a combination of the following flooding scenarios:

- 5% AEP local catchment storm coincident with a 1% AEP Nepean River flood;
- 1% AEP local catchment storm coincident with a 5% AEP Nepean River flood.

Figures 6.26, **6.27** and **6.29** (3 sheets each) show the peak flow velocities, flood hazard vulnerability classification and hydraulic categorisation of the floodplain, respectively for the 1% AEP Nepean River and local catchment flood envelope.

Figure 6.30 (3 sheets) shows the difference in flood behaviour between the 1% AEP Nepean River and local catchment flood envelope (as shown on **Figure 6.26**) and 1% AEP local catchment flooding coincident with a 5% AEP Nepean River flood (as shown on **Figure 6.6**). **Figure 6.30** shows that peak flood levels in the Nepean River and local catchment flood envelope are higher by:

- between 1.2 m and 1.6 m to the north of the Stilling Basin (refer sheet 1);
- about 2.1 m in the Stilling Basin (refer sheet 2);
- about 200 mm in the Waterside Lakes (refer sheet 2);
- between 400 mm and 800 mm between Andrews Road and Boundary Creek (refer sheets 2 and 3);
- up to 1.8 m along the reach of Boundary Creek that runs between Hickeys Lane and Castlereagh Road (refer sheet 3); and

- between 400 mm and 500 mm is areas that are located to the south of Coreen Avenue (refer sheet 3).

6.7.2. Selection of Flood Planning Levels

The 1% AEP Nepean River and local catchment flood envelope was used to define FPLs and FPA for the study area, noting that they should be reviewed and updated (if necessary) as part of the future *FRMS&P*.

After consideration of the TUFLOW model results and the findings of sensitivity studies outlined in **Sections 6.5** and **6.6** which found that the best estimate of the peak 1% AEP flood levels in the study area may increase by a maximum of 380 mm as a result of future climate change, the FPA was defined as land lying below the peak 1% AEP flood level plus a freeboard allowance of 500 mm. **Figure 6.31** (3 sheets) shows the extent of the FPA in the study area.

In areas that lie within the extent of the FPA it is recommended that a freeboard of 500 mm be applied to peak 1% AEP flood levels when setting the minimum floor level of future development. An assessment should also be undertaken by Council as part of any future Development Application to confirm that the proposed development will not form an obstruction to the passage of overland flow through the subject site.

Consideration will need to be given during the preparation of the future *FRMS&P* to the appropriateness of the adopted freeboard allowance of 500 mm given the impact changes in hydraulic roughness and future increases in rainfall intensity could have on peak flood levels. Consideration will also need to be given to the setting of an appropriate freeboard for areas subject to major overland flow given that the adopted value of 500 mm may be found to be too conservative.

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8 FLOOD-RELATED TERMINOLOGY

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

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

TERM	DEFINITION
Floodplain Risk Management Plan	A management plan developed in accordance with the principles and guidelines in the *Floodplain Development Manual, 2005*. Usually includes both written and diagrammatic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.
Floodway area	Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flow, or a significant increase in flood levels.
Freeboard	A factor of safety typically used in relation to the setting of floor levels, levee crest levels, etc. It is usually expressed as the difference in height between the adopted Flood Planning Level and the peak height of the flood used to determine the flood planning level. Freeboard provides a factor of safety to compensate for uncertainties in the estimation of flood levels across the floodplain, such and wave action, localised hydraulic behaviour and impacts that are specific event related, such as levee and embankment settlement, and other effects such as "greenhouse" and climate change. Freeboard is included in the flood planning level.
High hazard	Where land in the event of a 1% AEP flood is subject to a combination of flood water velocities and depths greater than the following combinations: 2 metres per second with shallow depth of flood water depths greater than 0.8 metres in depth with low velocity. Damage to structures is possible and wading would be unsafe for able bodied adults.
Low hazard	Where land may be affected by floodway or flood storage subject to a combination of floodwater velocities less than 2 metres per second with shallow depth or flood water depths less than 0.8 metres with low velocity. Nuisance damage to structures is possible and able bodied adults would have little difficulty wading.
Main stream flooding	Inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam.
Mathematical/computer models	The mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.
Merit approach	The merit approach weighs social, economic, ecological and cultural impacts of land use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and well-being of the State's rivers and floodplains.
Major overland flow	Inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.
Peak discharge	The maximum discharge occurring during a flood event.

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

APPENDIX A

INFORMATION SHEET AND COMMUNITY QUESTIONNAIRE

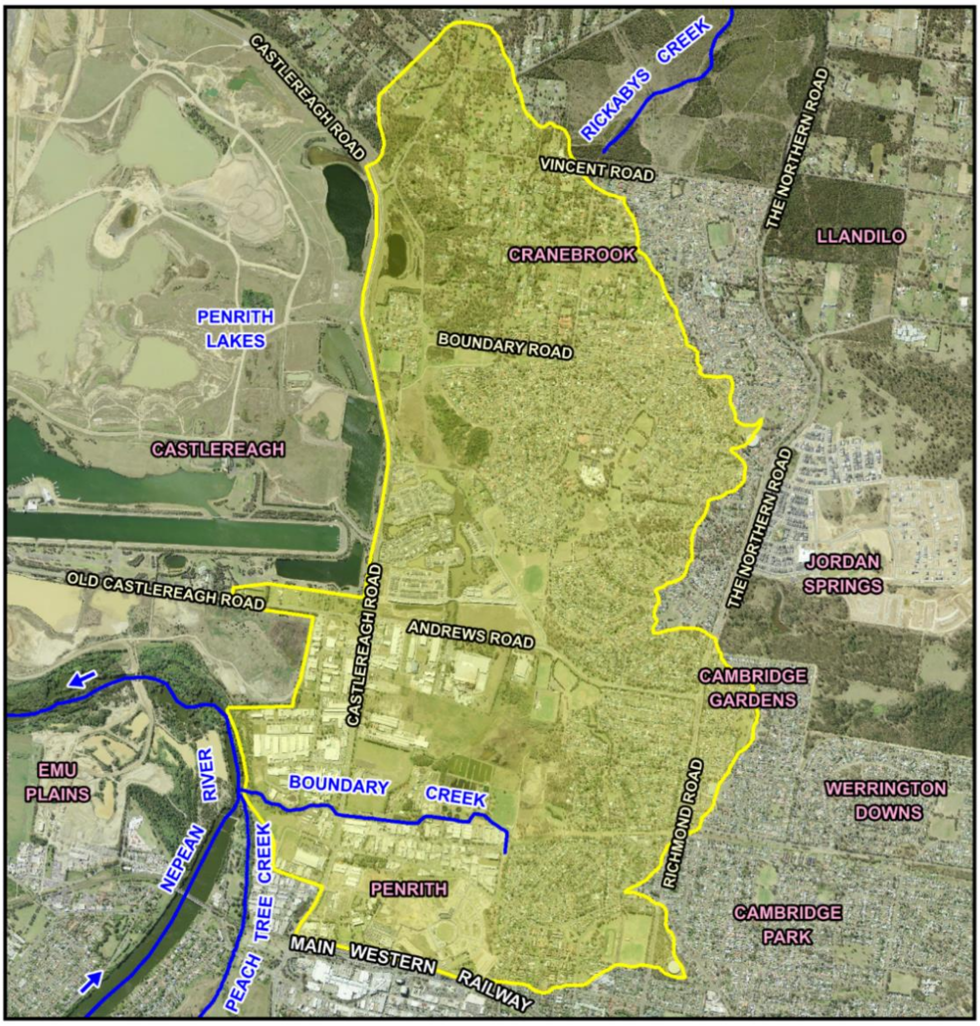
CRANEBROOK OVERLAND FLOW FLOOD STUDY

INFORMATION SHEET

INTRODUCTION

Council is in the initial stages of preparing the Cranebrook Overland Flow Flood Study, and we would like your help. The study will help Council identify and understand the existing flooding problem within the catchment. The extent of the study area is shown on the map below and includes the suburbs of Cranebrook and North Penrith.

During most rainfall events across the catchment, runoff is carried by the stormwater drainage pipes and channels in a westerly direction across Castlereagh Road where it discharges in either the Nepean River or Penrith Lakes. However, during heavy rainfall there is the potential for the capacity of the stormwater drainage pipes and channels to be exceeded, leading to overland flooding. Floodwater can also overtop the banks of the local creeks, as well as the Nepean River during large floods, resulting in the inundation of the adjoining floodplain.



Extent of the Cranebrook study area

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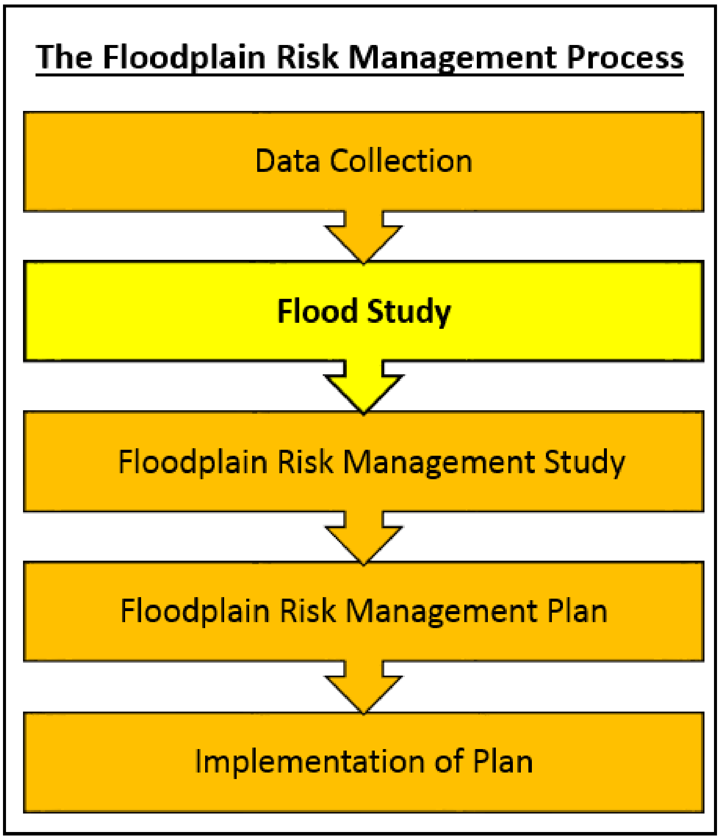
Council has appointed engineering consultants Lyall & Associates to prepare the study on our behalf. The study will be overseen by the Penrith Floodplain Risk Management Committee and is receiving financial and technical support from the NSW Government under its Floodplain Management Program.

WHY DO WE NEED TO PREPARE A FLOOD STUDY?

Flooding is the costliest natural disaster in Australia, causing an estimated $314 million worth of damage each year. Over 2,000 people have lost their lives due to floods in Australia. However, flooding is also one of the most manageable natural disasters as we can reasonably predict what areas will be subject to inundation.

Under the NSW Government's Flood Prone Land Policy, the management of flood prone land is, primarily, the responsibility of local councils. The policy specifies a staged process to manage the flood risk. This includes data collection, a flood study, a floodplain risk management study and plan, and implementation of the plan (see flowchart below).



Council will follow this staged process to manage the floodplain in your area. The "Data Collection" stage of work is largely complete, and we are now in the initial phases of the "Flood Study" stage of the process.

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PENRITH

WHAT IS INVOLVED IN PREPARING A FLOOD STUDY?

The primary objective of the flood study is to identify the nature and extent of the existing flooding problem. The preparation of a flood study will typically involve the following tasks:

- collection and review of all available flood-related information for the area.
- development of computer models to simulate the transformation of rainfall into runoff and to determine how that runoff would be distributed across the catchment.
- calibration of the computer models to reproduce historic floods.
- use of the computer models to simulate a range of hypothetical floods from relatively frequent storms right up to the largest flood that could possibly occur.
- preparation of a flood study report and maps summarising the outcomes of all stages of the investigation.

HOW CAN YOU BE INVOLVED?

Council recognises that the community holds important information about past floods that will help identify flooding 'trouble spots' and assist in calibrating the computer flood models. The study team will consult with the community in two stages:

- **Questionnaire** – Please complete the questionnaire included with this information sheet and share with us your experience and knowledge of local flooding within the study area. The questionnaire is also available online.
- **Public Exhibition** - once the draft flood study report is prepared, the report will be placed on public exhibition and a **community session** will be held to give you an opportunity to find out more about the study and to ask questions about any aspect of the study. Any comments from the public exhibition will be reviewed and addressed as part of the final report.

STAY UP TO DATE

Our website will be updated throughout the study and plan process to provide the latest available information including details of the above community consultations. Go to the Floodplain Management page of www.penrith.city/fps.

MORE INFORMATION

If you have any question or would like to submit any information you think may be helpful or relevant to the study, please contact:

Penrith City Council
Janahan Jivajirajah
Phone: 4732 7777
Email: Janahan.Jivajirajah@penrith.city

Lyall & Associates
Tom Rooney
Phone: 9929 4466
Email:
cranebrook@lyallandassociates.com.au

COMMUNITY QUESTIONNAIRE

CRANEBROOK OVERLAND FLOW FLOOD STUDY

Council has appointed Lyall & Associates to undertake a detailed flood study for the Cranebrook catchment, which includes the suburbs of Cranebrook and North Penrith. The enclosed Information Sheet and the "Floodplain Management" page of the Penrith City Council web page www.penrith.city/fps provide information on the steps involved in the preparation of the flood study.

We encourage you to complete and return this questionnaire to share your local knowledge and experience of flooding which will help us prepare a flood study that is shaped by local information that may otherwise go unrecorded.

Please complete the questionnaire and return it by Friday 9 October 2020.

You can do this by:

- filling out an online questionnaire at yoursaypenrith.com.au/CranebrookFS
- filling out the enclosed questionnaire, scanning and emailing it to cranebrook@lyallandassociates.com.au, or
- filling out the enclosed questionnaire and post it to Council using the enclosed pre-paid envelope.

Please answer as many questions as you can and provide as much detail as possible (attach additional pages if necessary).

If you have any questions or require further information, please contact:

1. Council's Senior Engineer – Stormwater, Janahan Jivajirajah on 4732 7777, or
2. Lyall & Associates – Senior Engineer, Mr Tom Rooney on 9929 4466.

CONTACT DETAILS

Please provide your street and suburb details.

Street Address: ____

____

Suburb: ______________________________ **Postcode:** ___________

Providing full contact details is optional, but useful so we can contact you for more information if required. If you choose to provide full contact details, this information will remain confidential at all times and will not be published.

Name: ____

Phone number: ____

Email: ____

Please indicate if and how you would like us to contact you for more information or to provide you with study updates:

☐ Yes – telephone/ email/ mail (circle your preferred method of contact)

☐ No

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PENRITH CITY COUNCIL | NSW GOVERNMENT Planning, Industry & Environment | SES STATE EMERGENCY SERVICE | Lyall & Associates

PENRITH

1) WHAT TYPE OF PROPERTY DO YOU LIVE IN/OWN?

☐ Residential

☐ Commercial

☐ Industrial

☐ Vacant land

☐ Other (please specify): ________________________________

2) WHAT IS THE OCCUPIER STATUS OF THIS PROPERTY?

☐ Owner occupied

☐ Rental property

☐ Business

☐ Other (please specify): ________________________________

3) HOW LONG HAVE YOU LIVED, WORKED OR OWNED PROPERTY IN THE AREA?

(a) At this address?

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

(b) In the general area?

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

4) HAVE YOU EVER BEEN AFFECTED BY FLOODING?

☐ Yes

☐ No (please go to Question 9)

5) HOW WERE YOU AFFECTED BY FLOODING? (YOU CAN SELECT MORE THAN ONE)

☐ Roadway was cut by water

☐ My front/back yard was flooded

☐ My garage was flooded

☐ My house/business was flooded

☐ Other (please specify) ________________________________

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6) CAN YOU PROVIDE ADDITIONAL INFORMATION ON THESE PAST FLOODS? (PLEASE ATTACH ADDITIONAL PAGES IF YOU HAVE INFORMATION FOR MORE THAN TWO FLOODS)

	Flood #1	Flood #2
Date of flood(s)	☐ February 2020 ☐ January 2016 ☐ February 2012 ☐ August 1990 ☐ July 1988 ☐ August 1986 ☐ March 1978 Other: __________________	☐ February 2020 ☐ January 2016 ☐ February 2012 ☐ August 1990 ☐ July 1988 ☐ August 1986 ☐ March 1978 Other: __________________
What was the flood water depth/height & location		
How confident are you of the height/depth of the flood?	☐ High (exact) ☐ Medium (within 10cm) ☐ Low (within 50cm)	☐ High (exact) ☐ Medium (within 10cm) ☐ Low (within 50cm)

7) DO YOU HAVE ANY PHOTOS OR VIDEOS OF THESE FLOODS?

☐ Yes

☐ No

If 'Yes', a copy of these photos/videos would assist our study. Please email a copy of the photos/videos to cranebrook@lyallandassociates.com.au.

8) WAS YOUR PROPERTY DAMAGED BY FLOODWATERS?

☐ Yes

☐ No

If ‘yes’, please provide details____

____

____

9) IN YOUR OPINION, WHAT WAS THE MAIN CAUSE OF THE FLOODING? (YOU CAN SELECT MORE THAN ONE)

☐ Insufficient creek capacity

☐ Insufficient stormwater capacity

☐ Blockage of creeks, stormwater inlets, bridges or drains

☐ Overland flow impediments (e.g. fences, buildings)

☐ Other (please specify) ____

10) CAN WE CONTACT YOU TO OBTAIN ADDITIONAL INFORMATION AND/OR CLARIFY ANY OF YOUR RESPONSES?

☐ Yes (please ensure that you have completed your contact details on page 1).

☐ No

11) DO YOU HAVE ANY OTHER COMMENTS, INFORMATION OR SUGGESTIONS YOU THINK MAY ASSIST THE STUDY?

____

____

____

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____

____

____

____

____

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APPENDIX B

DETAILS OF AVAILABLE DATA AND COMMUNITY CONSULTATION

TABLE OF CONTENTS

Page No.

B1 COLLECTION OF MISCELLANEOUS DATA B-1

B1.1 Previous Reports B-1
B1.1.1. Cranebrook Local Hydraulics Specification Study (Bewsher Consulting, 2002) B-1
B1.1.2. Boundary Creek Erosion Site Investigation (Patterson Britton & Partners (PBP), 2006) B-1
B1.1.3. Penrith Overland Flow Flood “Overview Study” (Cardno Lawson Treloar, 2006) B-1
B1.1.4. Penrith Lakes 2012 Water Management Plan: Stage 1 (Penrith Lakes Development Corporation, 2012) B-2
B1.1.5. Penrith CBD Detailed Overland Flow Flood Study (Cardno, 2015) B-2
B1.1.6. Nepean River Flood Study (Advisian, 2018) B-2
B1.1.7. Peach Tree and Lower Surveyors Creek Flood Study (Catchment Simulations Solutions (CSS), 2019) B-3
B1.1.8. Penrith Lakes Water Management Plan: Stage 2 (Penrith Lakes Development Corporation, 2020) B-4
B1.1.9. Penrith CBD Floodplain Risk Management Study and Plan (Molino Stewart, 2020) B-5
B1.1.10. Emu Plains Overland Flow Flood Study (BMT, 2020) B-5
B1.2 Airborne Laser Scanning Survey B-5
B1.3 Existing Stormwater Network B-6
B1.4 Cross Sectional Survey Data B-7
B1.5 Historic Rainfall Data B-7
B1.6 Stream Gauge Data B-7
B1.7 Photographic Record B-7

B2 COMMUNITY CONSULTATION B-9

B2.1 Background B-9
B2.2 Summary of Findings B-9
B2.2.1. General B-9
B2.2.2. Resident Profile B-9
B2.2.3. Experiences of Flooding B-10

B3 REFERENCES B-11

ANNEXURES

B1 Responses to Community Questionnaire
B2 Extracts from *Penrith Lakes 2012 Water Management Plan: Stage 1* (PLDC, 2012)
B3 Extracts from *Penrith Lakes Water Management Plan: Stage 2* (PLDC, 2020)

LIST OF FIGURES (BOUND IN VOLUME 2)

B1.1 Location and Source of Data (3 sheets)
B2.1 Location of Respondents to Community Questionnaire and Observed Flood Data

B1 COLLECTION OF MISCELLANEOUS DATA

B1.1 Previous Reports

B1.1.1. Cranebrook Local Hydraulics Specification Study (Bewsher Consulting, 2002)

The *Cranebrook Local Hydraulics Specification Study* was undertaken by Bewsher Consulting in 2002 to determine the hydraulic specification of the flood evacuation routes along Cranebrook Road, Vincent Road and Grays Lane in Cranebrook.

A hydrologic (RAFTS) model was developed as part of Bewsher Consulting, 2002 in order to define the runoff from the catchments contributing to flow at the low points of the aforementioned flood evacuation routes. Bewsher Consulting, 2002 found that a RAFTS storage routing parameter (Bx) of 1.7 was required to achieve a match between the peak flow derived from the RAFTS model and those derived using the Probabilistic Rational Method (**PRM**), procedures for which are set out in the 1987 edition of *Australian Rainfall and Runoff* (The Institution of Engineers Australia, 1987).

Given the relatively small size of the sub-catchments, the hydraulic capacity of the low points and their associated culverts were analysed using culvert and weir flow calculations, while hydraulic models of the four larger crossings were assessed using the HEC-RAS software. While survey data of a number of culverts were obtained as part of Bewsher Consulting, 2002, they have not been relied upon as they were superseded by survey data that were obtained as part of the present study (refer **Section B1.3** for more detail).

B1.1.2. Boundary Creek Erosion Site Investigation (Patterson Britton & Partners (PBP), 2006)

Patterson Britton & Partners (**PBP**) were engaged by Sydney Water to investigate and identify the likely cause of erosion of a 30 m section of the right bank of Boundary Creek immediately upstream of its confluence with the Nepean River. The study was undertaken as private land owners on the northern bank of Boundary Creek were concerned that discharge from Sydney Water's Penrith Sewage Treatment Plant (**STP**) was causing the bank instability and erosion.

A hydrologic (RAFTS) and hydraulic (HEC-RAS) model were developed as part of PBP, 2006 to assist with the investigation. PBP, 2006 found that the erosion was likely triggered by the removal of supporting talus along the toe of the creek bank by both Boundary Creek and Nepean River dominant flood events. PBP, 2006 also found that the discharge from the STP (which was about 70 ML/day) does not have sufficient energy or depth to initiate or complete the erosion process, but may speed up the process of the channel attaining a new state of dynamic equilibrium after a natural flood event.

While cross-sectional survey data were obtained as part of PBP, 2006, they have not been relied upon as they were superseded by survey data that were obtained as part of the present study (refer **Section B1.3** for more detail).

B1.1.3. Penrith Overland Flow Flood "Overview Study" (Cardno Lawson Treloar, 2006)

The *Penrith Overland Flow Flood "Overview Study"* was undertaken by Cardno Lawson Treloar in 2006 to define the broad nature of overland flow flood behaviour in the Local Government Area (**LGA**) and to assist Council in establishing priorities for undertaking detail flood studies. A hydrologic and hydraulic model of the LGA were developed using the direct-rainfall-on-grid approach in the TUFLOW software.

While some survey data were available from Cardno Lawson Treloar, 2006, they have not been relied upon as they were superseded by survey data that were obtained as part of the present study (refer **Section B1.3** for more detail).

B1.1.4. Penrith Lakes 2012 Water Management Plan: Stage 1 (Penrith Lakes Development Corporation, 2012)

The aim of the *Penrith Lakes 2012 Water Management Plan: Stage 1* (Penrith Lakes Development Corporation (**PLDC**), 2012) is to provide a contemporary and holistic response to water management across Penrith Lakes, which includes flood management, water supply, water reticulation, water quality and lake operating levels. PLDC, 2012 was adopted before construction of Penrith Lakes was complete.

PLDC, 2012 includes a description of the stormwater infrastructure that is used to convey stormwater through Penrith Lakes which is designed to maintain the optimum operating levels in the lakes while preventing levels reaching a height which may result in damage to infrastructure. **Annexure B2** contains the following extracts from PLDC, 2012:

- *Table 6* which sets out the dimensions and operating levels of the thirteen lakes that comprise Penrith Lakes;
- *Figure 11* which shows the indicative layout of the stormwater infrastructure that is used to convey stormwater through Penrith Lakes; and
- *Table 8* which sets out the dimensions of abovementioned stormwater infrastructure.

The information contained in PLDC, 2012, particularly that contained in **Annexure B2** has been used to understand how the lakes system functions during a local catchment storm event.

B1.1.5. Penrith CBD Detailed Overland Flow Flood Study (Cardno, 2015)

Cardno was commissioned by Council to undertake the *Penrith CBD Overland Flow Flood Study* in 2015 which defined patterns of overland flow through the Penrith CBD catchment which lies immediately to the south of the study area. Hydrologic (TUFLOW and RAFTS) and hydraulic (TUFLOW) models were developed as part of Cardno, 2015 and validated to observed flood behaviour from a storm event that occurred in February 2006.

The hydrologic and hydraulic models were used to define patterns of overland flow through the Penrith CBD for design flood events with AEPs of 1 EY, 50%, 20%, 10%, 2%, 1% and 0.5%, as well as the PMF. **Table B1.1** over shows that the downstream boundary condition applied to the TUFLOW model for the range of assessed design flood events comprised a free flowing discharge into Peach Tree Creek.

B1.1.6. Nepean River Flood Study (Advisian, 2018)

The *Nepean River Flood Study* was undertaken by Advisian in 2018 to define the nature of flooding along a 25 km reach of the Nepean River between a location approximately 2.5 km upstream (south) of the M4 Motorway and the Springdale Road crossing of the river in the vicinity of Agnes Banks.

TABLE B1.1
ADOPTED TAILWATER CONDITIONS FOR FLOODING INVESTIGATIONS UNDERTAKEN IN THE VICINITY OF THE STUDY AREA

<table>
<tr><th>Local Catchment AEP</th><th>Penrith CBD Detailed Overland Flow Flood Study
(Cardno, 2015)</th><th>Peach Tree and Lower Surveyors Creek Flood Study (CSS, 2019)</th><th>Emu Plains Overland Flow Flood Study
(BMT, 2020)</th></tr>
<tr><td>1EY</td><td rowspan="8">Free flowing discharge</td><td rowspan="10">5% AEP</td><td>Not Assessed</td></tr>
<tr><td>50%</td><td rowspan="3">Free flowing discharge</td></tr>
<tr><td>20%</td></tr>
<tr><td>10%</td></tr>
<tr><td>5%</td><td rowspan="6">5% AEP</td></tr>
<tr><td>2%</td></tr>
<tr><td>1%</td></tr>
<tr><td>0.5%</td></tr>
<tr><td>0.2%</td><td>Not Assessed</td></tr>
<tr><td>PMF</td><td>Free flowing discharge</td></tr>
</table>

The RUBICON hydrologic model that was originally developed as part of the *Warragamba Flood Mitigation Dam Environmental Impact Statement Flood Study* by Webb, McKeon & Associates in 1994 was used to derive inflow hydrographs to a hydraulic model of the Nepean River that was developed using the RMA2 software. The RMA2 model was used to define flood behaviour along the Nepean River for the 5%, 2%, 1%, 0.5%, 0.2%, 0.01% and 0.05% AEP design flood events, as well as the PMF.

The results of Advisian, 2018 were used to define tailwater conditions along the Nepean River in the TUFLOW model that was developed as part of the present study.

B1.1.7. Peach Tree and Lower Surveyors Creek Flood Study (Catchment Simulations Solutions (CSS), 2019)

Catchment Simulation Solutions undertook the *Peach Tree and Lower Surveyors Creek Flood Study* in 2019 to define flood behaviour along the main arms of Peach Tree Creek and Lower Surveyors Creek. CSS, 2019 also defined the nature of overland flow in the area bounded by the Nepean River to the west, M4 Motorway to the south, The Northern Road to the east and the Penrith CBD catchment to the north.

Hydrologic (RAFTS) and hydraulic (TUFLOW) models that were developed as part of CSS, 2019 were calibrated to observed flood behaviour during storm events that occurred on 9-10 February 2012 and 4 January 2016 (both of which were used to validate the flood models developed as part of the present study), and also validated against a storm event that occurred on 26 February 2006. **Table B1.2** over sets out the hydrologic model parameters that were found to achieve a good match with observed flood behaviour for the February 2012 and January 2016 storm events, as well as those adopted for design flood modelling.

Table B1.1 shows that CSS, 2019 adopted a 5% AEP Nepean River flood as the downstream tailwater condition that was applied to the TUFLOW model for the full range of assessed design storm events.

TABLE B1.2
ADOPTED HYDROLGOIC MODEL PARAMETERS
FROM PREVIOUS INVESTIGATIONS

Storm Event	Hydrologic Model Parameter	Surface Type	*Peach Tree and Lower Surveyors Creek Flood Study* (CSS, 2019)	*Emu Plains Overland Flow Flood Study* (BMT, 2020)	Present Study
9 February 2012	Initial Loss (mm)	Pervious Areas	0	Not Assessed	0
		Impervious Areas	0		2
	Continuing Loss (mm/hr)	Pervious Areas	2.5		1.4
		Impervious Areas	0		0
4 January 2016	Initial Loss (mm)	Pervious Areas	10	Not Assessed	
		Impervious Areas	1		
	Continuing Loss (mm/hr)	Pervious Areas	2.5		
		Impervious Areas	0		
5 January 2016	Initial Loss (mm)	Pervious Areas	Not Assessed		0
		Impervious Areas			2
	Continuing Loss (mm/hr)	Pervious Areas			1.4
		Impervious Areas			0
31 January 2016	Initial Loss (mm)	Pervious Areas	Not Assessed	10	Not Assessed
		Impervious Areas		2	
	Continuing Loss (mm/hr)	Pervious Areas		1.5	
		Impervious Areas		0	
9 February 2020	Initial Loss (mm)	Pervious Areas	Not Assessed		0
		Impervious Areas			2
	Continuing Loss (mm/hr)	Pervious Areas			1.4
		Impervious Areas			0
Design Events	Initial Loss (mm)	Pervious Areas	Varies	10	PNBIL[(1)]
		Impervious Areas	1	2	2
	Continuing Loss (mm/hr)	Pervious Areas	2.5	1.5	1.4
		Impervious Areas	0	0	0

1. PNBIL – Probability Neutral Burst Initial Losses.

B1.1.8. Penrith Lakes Water Management Plan: Stage 2 (Penrith Lakes Development Corporation, 2020)

The *Penrith Lakes Water Management Plan: Stage 2* (PLDC, 2020) completements the *Stage 1 Water Management Plan* (PLDC, 2012) and makes recommendations regarding the future operational and water management requirements of the completed Penrith Lakes.

Table 1 of PLDC, 2020 (a copy of which is contained in **Annexure B3**) sets out the final dimensions and operating levels of the thirteen lakes that comprise Penrith Lakes, as well as the recommended water level tolerances in each which were based on operational requirements and ecological considerations. PLDC, 2020 indicates that the monitoring and management of the lakes water levels is assisted by numerous sluice gates and level monitoring stations.

The information contained PLDC, 2020, particularly that contained in **Annexure B3** has been used to define the tailwater conditions that has been applied to the TUFLOW model that has been developed as part of the present study (refer **Section 4.4** of Main Report for further discussion).

B1.1.9. Penrith CBD Floodplain Risk Management Study and Plan (Molino Stewart, 2020)

The *Penrith CBD Floodplain Risk Management Study and Plan* undertaken by Molino Stewart in 2020 identified practical, affordable and acceptable means of managing the existing flood risk in the Penrith CBD catchment which is located immediately to the south of the study area.

The hydrologic (TUFLOW and RAFTS) and hydraulic (TUFLOW) models that were originally developed as part of Cardno, 2015 were updated as part of Molino Stewart, 2020 to incorporate recent data and changes to the study catchment. The updated flood models were then used define flood behaviour in the Penrith CBD and assess measures aimed at mitigating the flood risk.

The results of Molino Stewart, 2020 were used to define the downstream tailwater conditions that were applied to the TUFLOW model that was developed as part of the present study at three locations where runoff from the study area discharges to the Penrith CBD catchment (refer **Section 4.4** of the Main Report for details).

B1.1.10. Emu Plains Overland Flow Flood Study (BMT, 2020)

BMT undertook the *Emu Plains Overland Flow Flood Study* in 2020 which defined flood behaviour in the suburbs of Emu Plains, Emu Heights and Leonay. Hydrologic (RAFTS) and hydraulic (TUFLOW) models were developed as part of BMT, 2020 and calibrated to observed flood behaviour that was observed during storms that occurred in January 2016 and April 2015, the former of which has also been used to validate the flood models that have been developed as part of the present study.

Table B1.1 sets out the tailwater conditions that were adopted for design flood estimation as part of BMT, 2020, while **Table B1.2** sets out the hydrologic model parameters that were found to give a good match with the observed flood data from the January 2016 storm event, as well as those adopted for design flood estimation.

B1.2 Airborne Laser Scanning Survey

Table B1.3 sets out the details of the three sets of LiDAR survey data that cover the study area. The data comprising each set were captured in accordance with the International Committee on Surveying and Mapping guidelines for digital elevation data with a 95% confidence interval on horizontal accuracy of ±800 mm and a vertical accuracy of ±150 mm.

TABLE B1.3
LiDAR SURVEY DATA SPECIFICATIONS

Data Set	Date of Capture	Data Provider
Penrith201907	July 2019	Geoscience Australia
Penrith_Lakes_1607	July 2016	Penrith City Council
NepeanRiverWEST0211	February 2011	Lands & Property Management Authority

B1.3 Existing Stormwater Network

Figure B1.1 shows the plan location of the existing stormwater in the study area. Details of the stormwater drainage network were taken from the following sources:

➢ **<u>Structure Survey</u>**

Survey of the stormwater network in the Cranebrook study area was initially undertaken by Cardno in July 2020 (**Cardno Survey Data**) and supplemented by additional data that were captured by Richard Hogan & Co in November 2020 (**RH&C Survey Data**). **Figure B1.1** shows the plan location of the structure survey data which were provided in MapInfo format.

Data provided for the surveyed pipes and culverts included (but was not limited to):

- Diameter for piped structures;
- Width and height for box culvert type structures;
- Upstream and downstream invert levels of the structure in m AHD;
- Number of barrels;
- Length of structure;
- Structure material; and
- Location of structure in the MGA (GDA 94) co-ordinate system.

Data provided for the surveyed stormwater pits included (but was not limited to):

- Pit width, length and depth;
- Pit type;
- Dimensions of lintel and/or grate;
- Elevation of pit invert, kerb inlet and top of kerb in m AHD;
- Structural condition;
- Observed blockage of structure; and
- Location of structure in the MGA (GDA 94) co-ordinate system.

- **<u>Detailed Design Drawings</u>**

  At the commencement of the study, Council provided Work-As-Executed (**WAE**) plans of the stormwater drainage network associated within the following subdivisions (refer **Figure B1.1** for location):

  - Lambridge Estate Subdivision;
  - North Penrith Subdivision;
  - Waterside Subdivision; and
  - Hickeys Lane Subdivision.

  Council also provided WAE drawings for the *Castlereagh Road Relocation* project between Smith Road and Andrews Road and the *Mulgoa Road and Castlereagh Road Upgrade* project in the vicinity of Jane Street.

B1.4 Cross Sectional Survey Data

RH&C was also engaged to undertake inbank cross sectional survey at regular intervals along Boundary Creek between Hickeys Lane and the Nepean River (refer **Figure B1.1** for location). Cross section data were provided as tabulations of offset versus elevation in an Excel spreadsheet. An AutoCAD file was also provided in the MGA (GDA 94) co-ordinate system showing the extent of each cross section. A photographic record of each cross section was also compiled by the surveyor.

B1.5 Historic Rainfall Data

Rainfall data were available at two All Weather Station (**AWS**) rain gauges which are operated by the Bureau of Meteorology (**BoM**) and eight pluviographic rainfall gauges which are operated by Sydney Water. **Figure 1.1** of the Main Report shows the plan location of the abovementioned gauges, while **Table B1.4** over sets out the details of the rain gauge network.

B1.6 Stream Gauge Data

Figure B1.1 shows the location of the WaterNSW operated *Nepean River at Penrith* stream gauge (Gauge No. 212201) which was used to define water levels in the Nepean River for the three historic storm events that were used to validate the TUFLOW model developed as part of the present study.

B1.7 Photographic Record

Appendix C contains a number of photographs that were provided by respondents to the *Information Sheet* and *Community Questionnaire* showing flood behaviour in the study area during storms that occurred on 9 February 2012, January 2016 (day not specified), 21 March 2017, 7 February 2020 and 9 February 2020.

TABLE B1.4
SUMMARY OF AVAILABLE PLUVIOGRAPHIC RAIN GAUGE DATA[1]

Gauge Owner	Gauge Number	Gauge Name	Site Commence	Site Cease
Bureau of Meteorology	67033[2]	Richmond RAAF	February 1953	November 1994
	67113	Penrith Lakes AWS	August 1995	Ongoing
Sydney Water	567107	Penrith WRP	August 1991	Ongoing
	567159	Cranebrook Reservoir	August 1991	Ongoing
	567155	INTL Transmitter Station, Shanes Park	August 1991	Ongoing
	563146	Winmalee WWTP	April 1990	Ongoing
	563064	Glenbrook RAAF Base	June1997	Ongoing
	567163	Regentville Rural Fire Service	September 1992	Ongoing
	567158	Kingswood Rd Reservoir Orchard Hills	August 1991	Ongoing
	567087	St Marys WRP	August 1983	Ongoing

1. Refer **Figure 1.1** of the Main Report for location.
2. BoM's Richmond RAAF rain gauge is located approximately 13 km north of the study area and not shown on **Figure 1.1**.

B2 COMMUNITY CONSULTATION

B2.1 Background

At the commencement of the study, the Consultants prepared an *Information Sheet* and *Community Questionnaire*, both of which were distributed by Council to residents and business owners in the study area (a copy of which is contained in **Appendix A**).

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

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

The *Information Sheet* and *Community Questionnaire* were advertised in the local newspaper on 17 September 2020 and posted to 7135 residents and business owners in the study area on 21 September 2020. The *Information Sheet* and *Community Questionnaire* were also advertised on Council's website and social media platforms.

An additional 665 *Information Sheet* and *Community Questionnaires* were also sent to residents and business owners in the vicinity of North Penrith on the 7 December 2020.

B2.2 Summary of Findings

B2.2.1. General

Residents and business owners were requested to complete the *Community Questionnaire* and return it to the Consultants by 9 October 2020. The deadline was extended to include any submissions that were received after this date. The Consultants received 434 responses in total, which amounted to about six per cent of the total number of questionnaires that were distributed to the community.

Of the 665 additional *Information Sheet* and *Community Questionnaires* that were distributed in December 2020, a total of 38 responses were received by the return date of 10 January 2021, which amounts to a return rate of about six per cent.

Figure B2.1 shows the plan location of the 472 respondents, while the collated responses are shown in graphical format in **Annexure B1**.

B2.2.2. Resident Profile

The first three questions of the *Community Questionnaire* canvassed resident information such as whether the respondent was a resident or business owner, length of time at the property, the type of property (e.g. house, unit/flat).

Of the 472 responses, 450 respondents occupied residential type property (**Question 1**), nine occupied commercial type property and seven occupied industrial type property. One response received was concerned with property which is vacant land, while four respondents were occupants of rural-residential type property.

In response to **Question 2**, approximately 84% of respondents were property owners, about 14% rented the property and 1% occupied commercial premises, while 1% of respondents did not provide a response to the question.

The length of time respondents had been at their current address was found to be varied, with approximately 17% of respondents having lived at the residence for between '0-5 years', 14% for '5 to 10 years', 19% for '10 to 20 years', and 49% for 'more than 20 years' (**Question 3**).

B2.2.3. Experiences of Flooding

In **Question 4**, 27% of respondents indicated that they had been affected by flooding while 71% had not been affected.[1] Of those that have been affected by flooding, 11 indicated that their house or business was flooded, 33 indicated that their garage was flooded and 59 indicated that their front or back yard was inundated (**Question 5**). A total of 49 respondents had experienced roadways being cut off by floodwater, specifically the section of Cranebrook Road between Ashley Avenue and Olive Lane (ten respondents) and at the Andrews Road and Castlereagh Road intersection (three respondents). Fifteen respondents referenced the flood detention basins and/or floodways that convey and store overland flow during storm events.

In response to **Question 6**, the majority of respondents to the *Community Questionnaire* had been affected by flooding as a result of storm events in February 2012 (26 respondents), January 2016 (33) and more recently in February 2020 (51). Respondents also identified storm events that occurred on the following months:

- March 1978 (two);
- January 1983 (one);
- November 1985 (one);
- August 1986 (six);
- July 1988 (five);
- August 1990 (seven);
- 1995 (month not specified) (one); and
- March 2005 (one).

Of the 29 respondents who had indicated their property was damaged by floodwaters (**Question 8**), the majority had been damaged during the February 2012, January 2016 and February 2020 storm events. Based on the responses to the *Community Questionnaire*, above-floor inundation was reported in five properties in February 2012, two in January 2016 and four in February 2020.

One respondent indicated that their dwelling had been inundated above-floor level four times in 38 years (i.e. November 1985, August 1986, February 2012 and January 2016). Another respondent had incurred costs of $45,000 to repair damage to walls, cupboards, fittings and furnishings as a result of the February 2020 storm event.

In **Question 9**, respondents were asked what the main cause of flooding was in the study area. The majority of respondents indicated it was insufficient stormwater capacity (74), blockage of creeks, stormwater inlets, bridges and drains (43) and overland flow impediments (33), while fourteen respondents identified that the creeks had insufficient capacity. Other causes of flooding identified by respondents were:

- Increased density of housing;
- Change in landform due to upstream sub-division;
- Lack of kerb and gutter / drains in rural areas
- Neighbours pools/dams overflowing

Figure B2.1 shows the plan location of observed flood data identified by respondents to the Community Questionnaire.

[1] 2% of respondents did not provide an answer to Question 4.

B3 REFERENCES

Advisian, 2018*.* ***"Nepean River Flood Study"***

Bewsher Consulting, 2002. ***"Cranebrook Local Hydraulics Specification Study"***

BMT, 2020. ***"Emu Plain Overland Flow Flood Study"***

Cardno Lawson Treloar, 2006. **"*Penrith Overland Flow Flood "Overview Study""***

Cardno, 2015 *.* ***"Penrith CBD Detailed Overland Flow Flood Study"***

CSS (Catchment Simulations Solutions), 2019). ***"Peach Tree and Lower Surveyors Creek Flood Study*"**

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

Molino Stewart, 2020. ***"Penrith CBD Floodplain Risk Management Study and Plan"***

PLDC (Penrith Lakes Development Corporation), 2012. ***"Penrith Lakes 2012 Water Management Plan: Stage 1"***

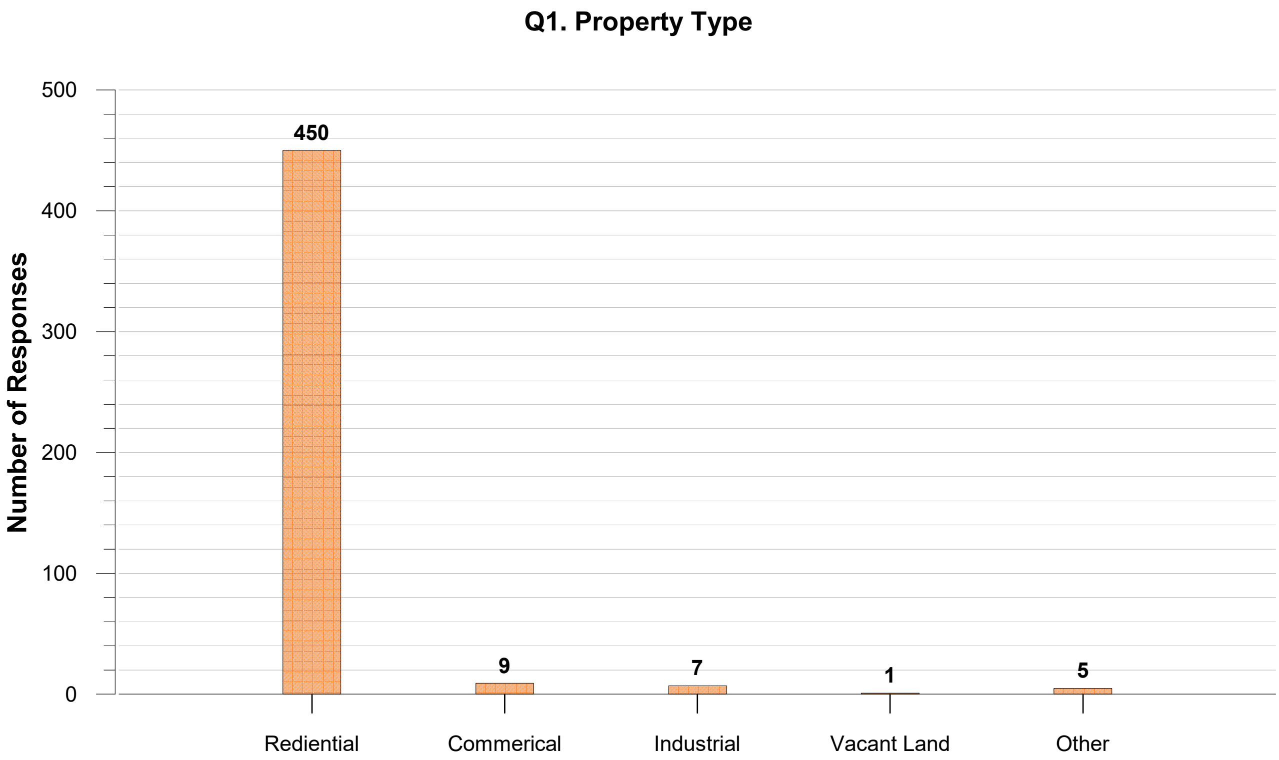
PLDC (Penrith Lakes Development Corporation), 2020. ***"Penrith Lakes Water Management Plan: Stage 2"***

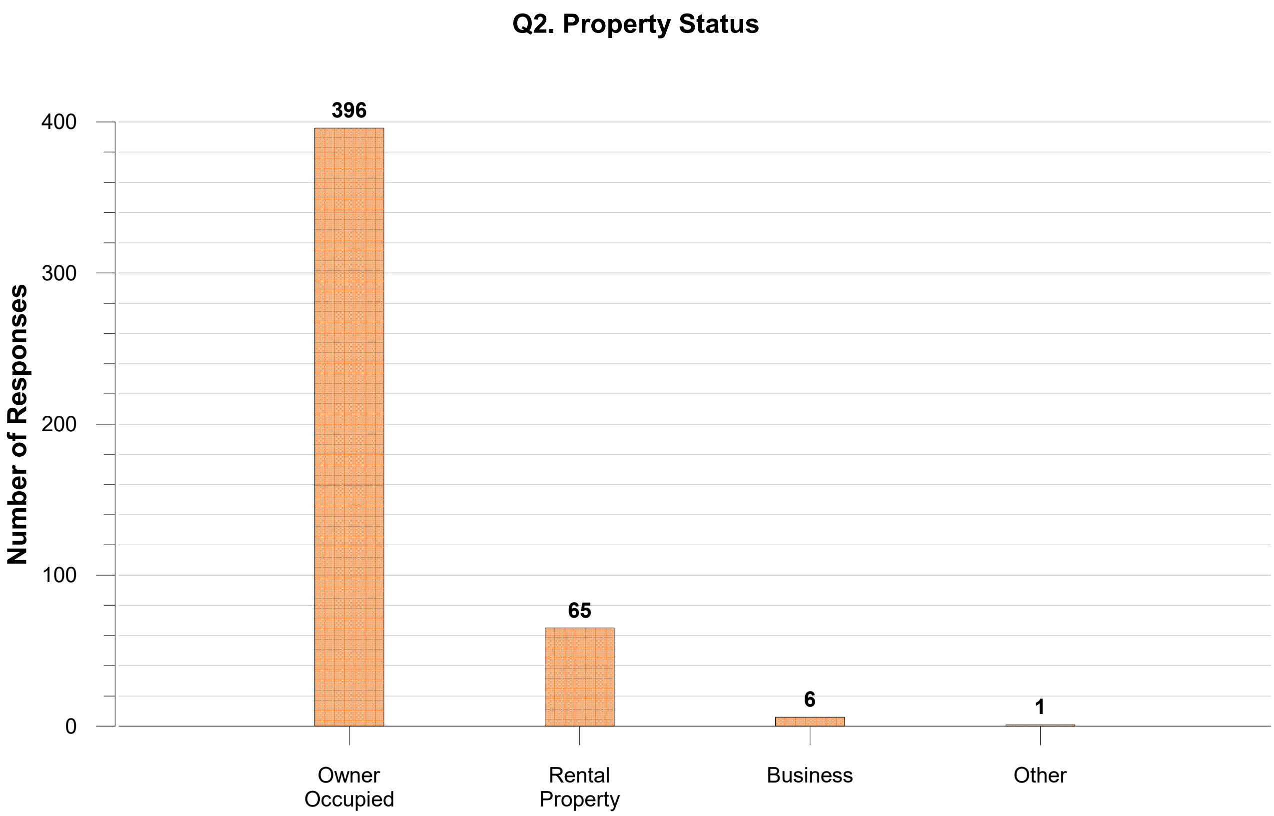
PBP (Patterson Britton & Partners), 2006. ***"Boundary Creek Erosion Site Investigation"***

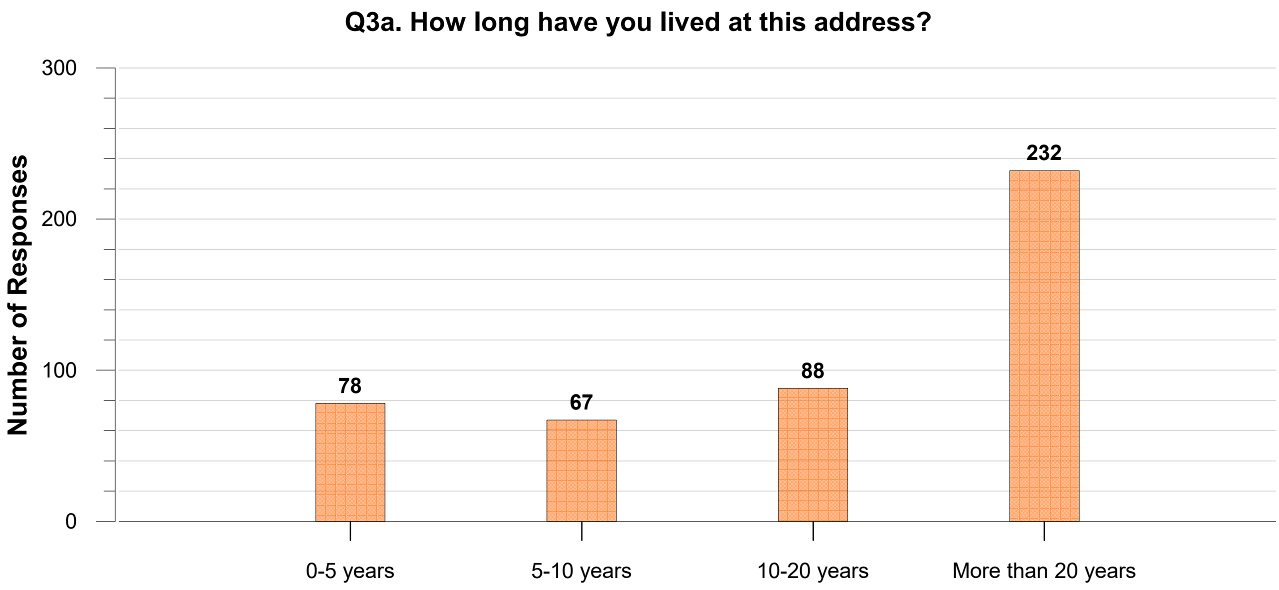
Webb, McKeon & Associates, 1994. ***"Warragamba Flood Mitigation Dam Environmental Impact Statement Flood Study"***

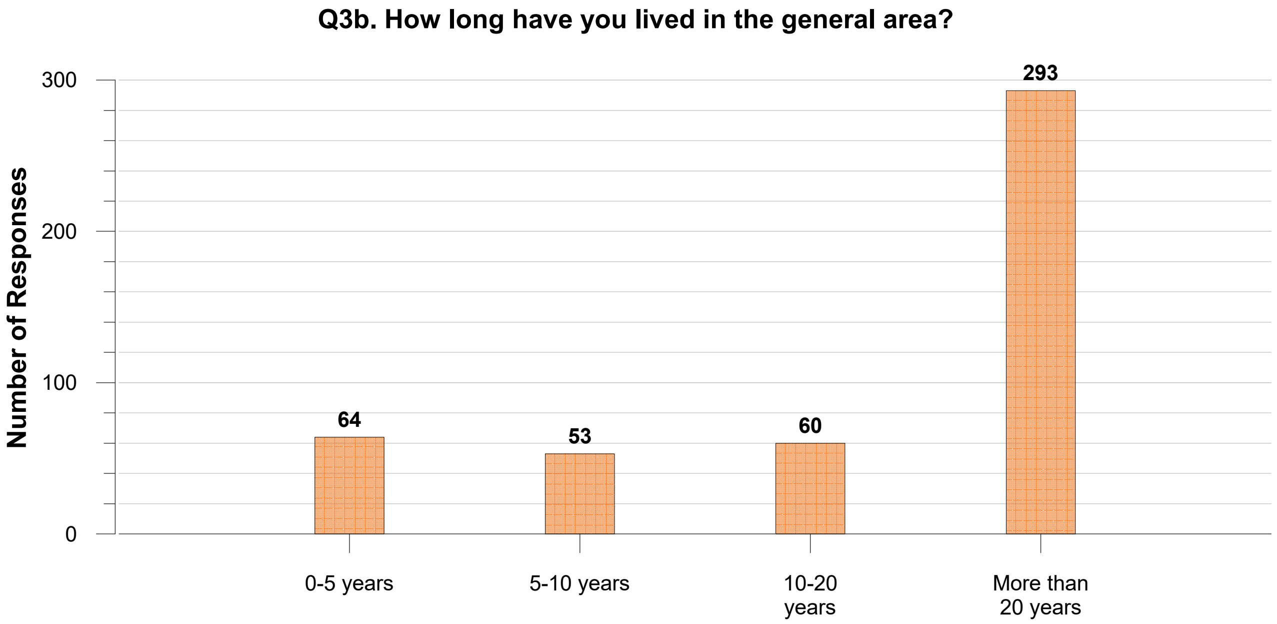
ANNEXURE B1

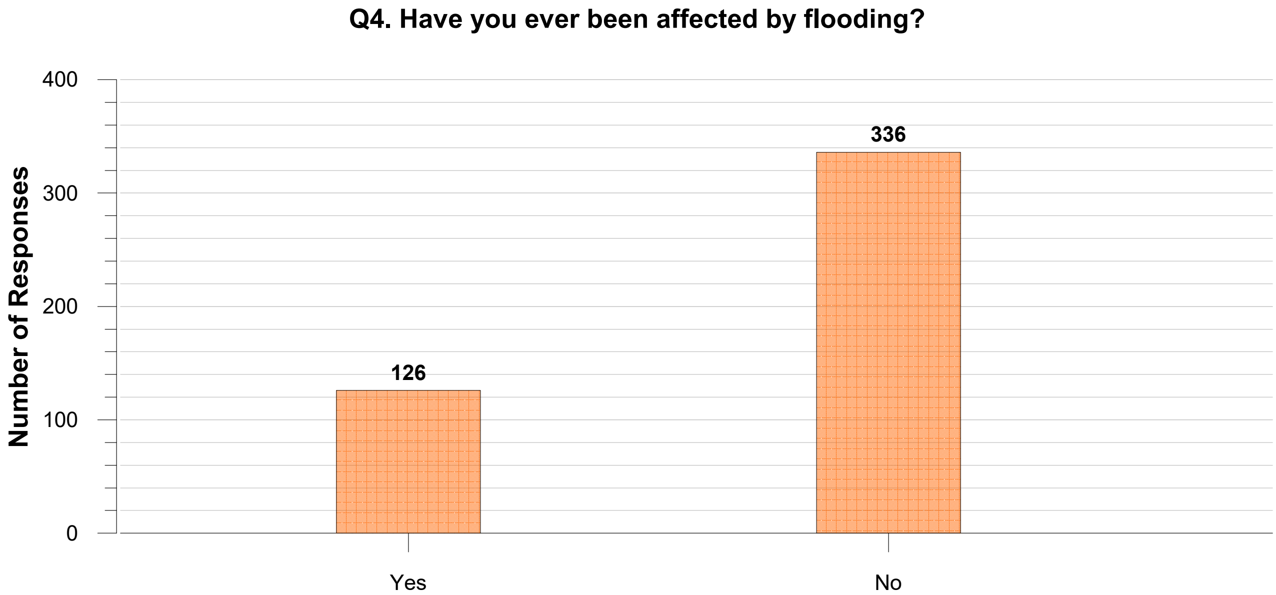
RESPONSES TO COMMUNITY QUESTIONNAIRE


Q1. Property Type
Number of Responses
500
400
300
200
100
0
450
9
7
1
5
Rediential
Commerical
Industrial
Vacant Land
Other

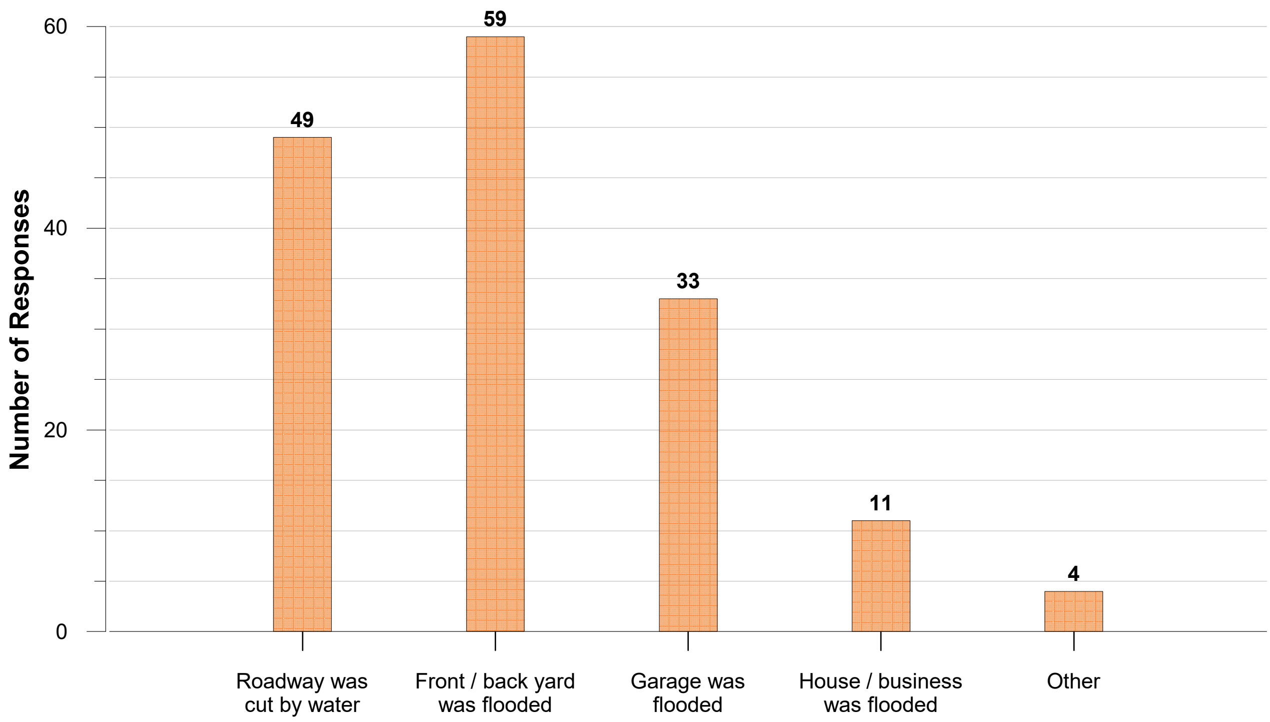

Q2. Property Status
Number of Responses
400
300
200
100
0
396
65
6
1
Owner Occupied
Rental Property
Business
Other


Q3a. How long have you lived at this address?
Number of Responses
300
200
100
0
78
67
88
232
0-5 years
5-10 years
10-20 years
More than 20 years

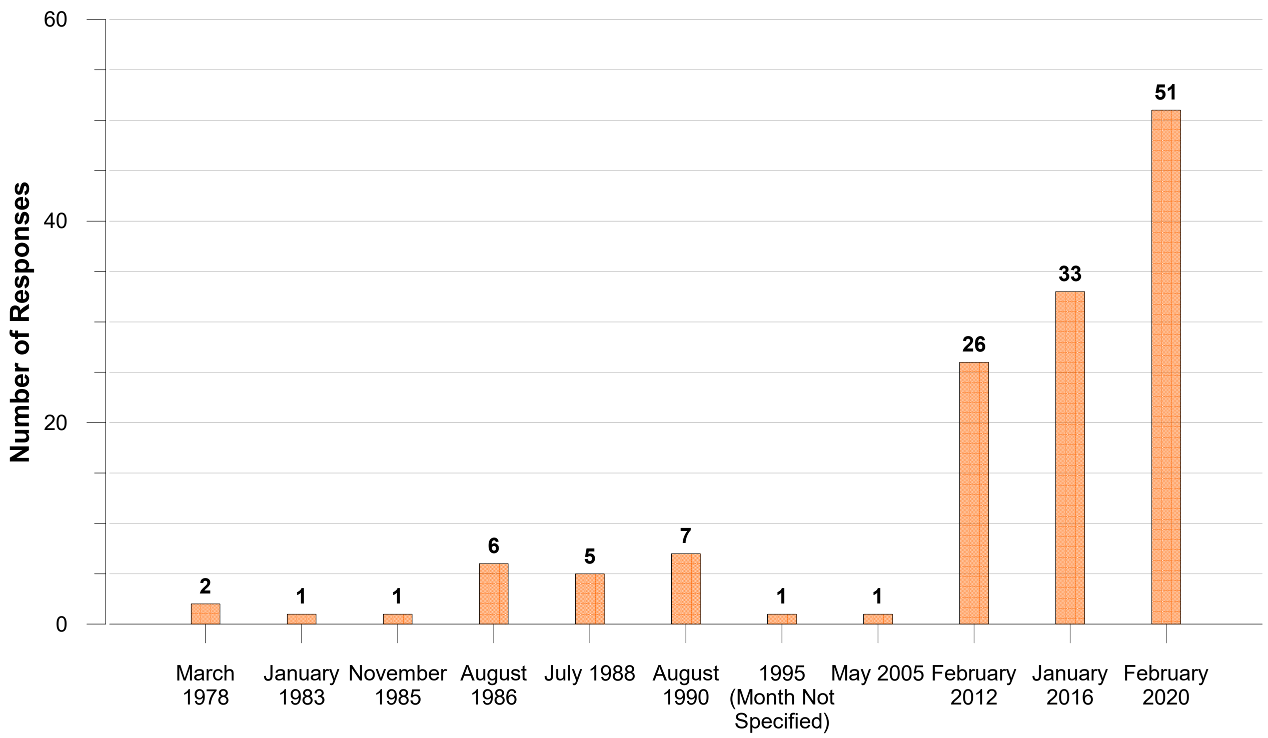

Q3b. How long have you lived in the general area?
Number of Responses
300
200
100
0
64
53
60
293
0-5 years
5-10 years
10-20 years
More than 20 years

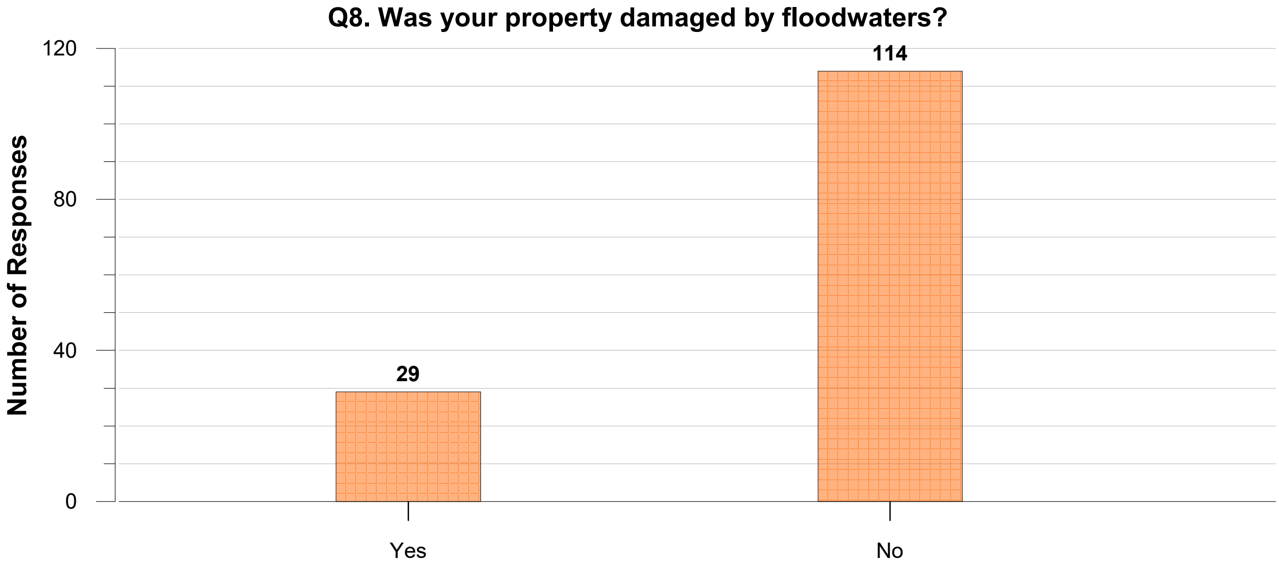

Q4. Have you ever been affected by flooding?
Number of Responses
400
300
200
100
0
126
336
Yes
No

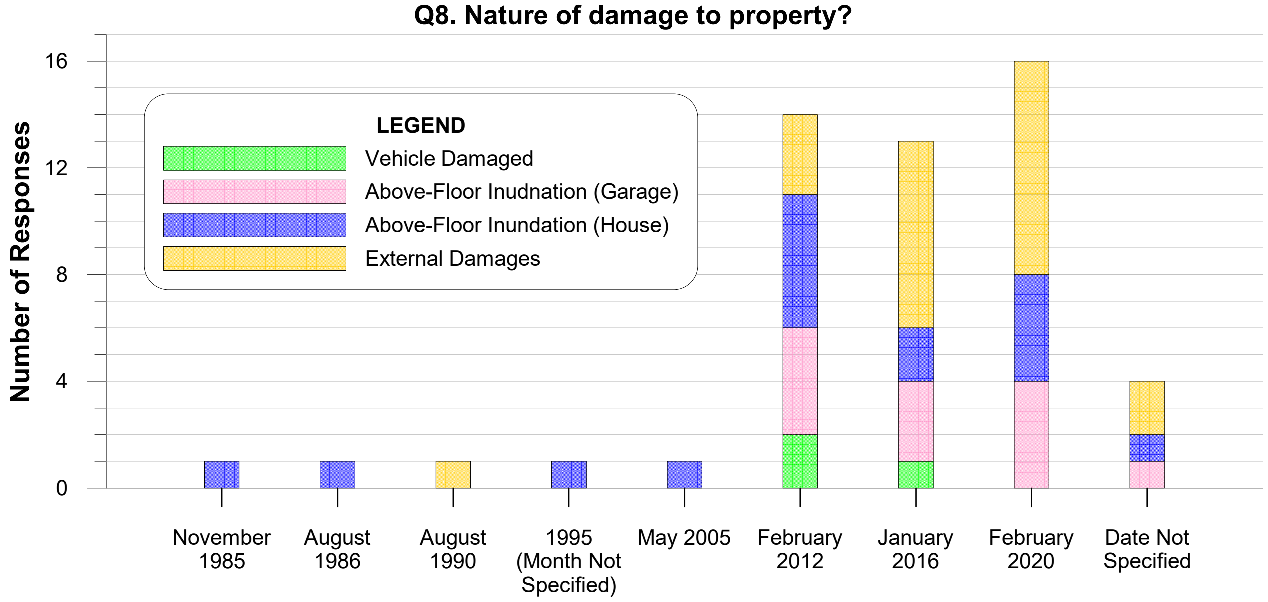
Q5. How were you affected by flooding?

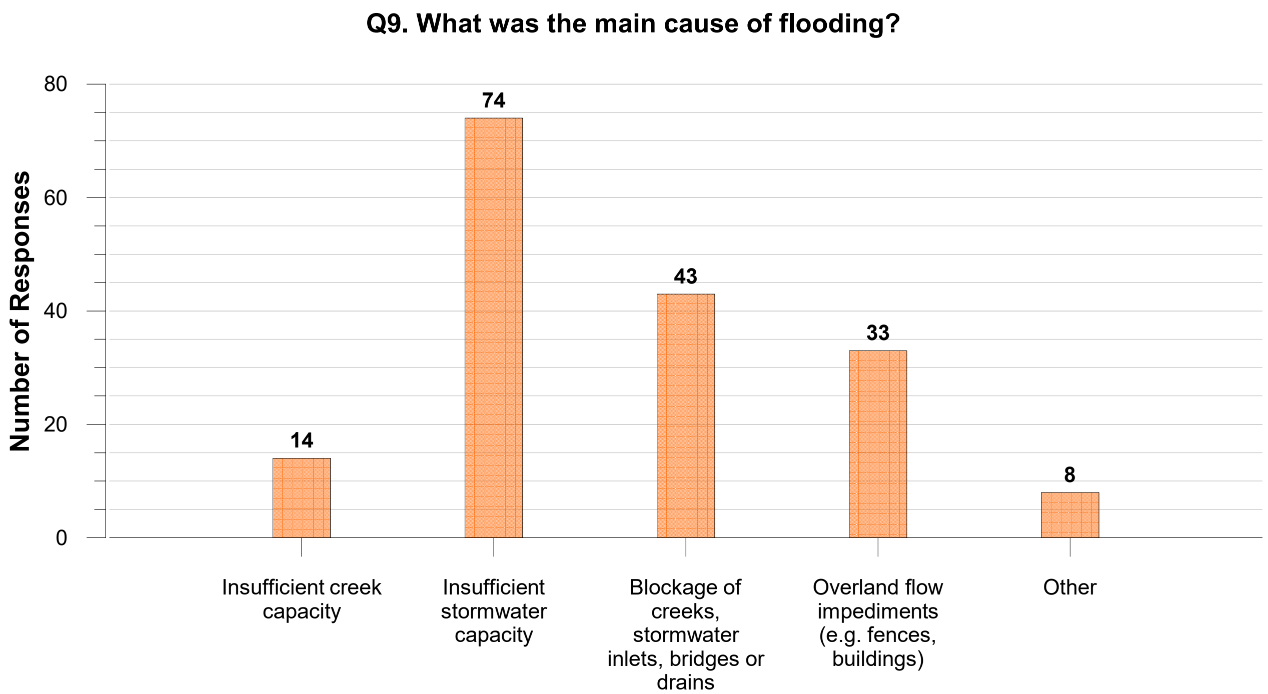


Q6. On what dates did you experience flooding?









RESPONSE TO COMMUNITY QUESTIONNAIRE

ANNEXURE B2

EXTRACTS FROM *PENRITH LAKES 2012 WATER MANAGEMENT PLAN: STAGE 1* (PLDC, 2012)

Table 6 - Summary of Lake Details

LAKE	SIZE (HA)	AVERAGE DEPTH (M)	OPERATING LEVEL (M AHD)	VOLUME AT OPERATING LEVEL (GL)	TIME TO RETURN TO OPERATIONAL LEVEL (POST 100YR ARI) (DAYS)
Primary Lakes					
Wildlife Lake	110	4-5	10	3.9	15
Lake B	121	6-7	13.5	7.3	29+
Lake A	318	5-6	14	17.8	29
Quarantine Lake	42	6-7	15	2.4	29
Regatta Lake	80	5-6	15	4.2	29
Treatment Lakes					
Duralia Lake	13	10-11	18	0.9	35++
Cranebrook Lake	3	10-11	18	0.13	35++
Detention Basins					
Lewis Lagoon	3	5	14	0.08	15
North Pond	7	4	16.5	0.17	35++
Stilling Basin	0.6	2.5	17.7	0.01	35++
West Pond (Pioneer Pond)	5	1-2	17	0.04	35++
Middle Basin	13	4-5	16.0	0.5	35++
Final Basin	7	3-4	15.5	0.07	35++
Lake Totals	**723**			**38.0**	

WETLANDS	SIZE (HA)	OPERATING RANGE (M AHD)
Southern Wetlands*	40	26 - 20
Duralia Wetlands	3.7	19 - 18
Cranebrook Wetlands	2.2	24.5 - 18
Eastern Chain of Ponds	3.6	24 - 17

** The Southern Wetlands have not been designed at this stage. The details shown in this table are estimates only and will change following the conceptual design that is being undertaken as part of the SSD application for the Nepean River Pump and Pipeline.*

+ Drawdown time is to 0.5m above operational level except for Lake B where it is to 1m above operational level. Increased tolerances are to assist future managers in maintaining water volumes.

++ These drawdown times have been assumed. In reality they are controlled by sluice gates and the draw down will be governed by lake operators.

Figure 11: Completed Water Supply and Reticulation Works

9.0 WATER MANAGEMENT SCHEDULES

Figure 11: Completed Water Supply and Reticulation Works shows the water supply and reticulation works already completed by PLDC.

Table 8: Completed Water Supply and Reticulation Works

REFERENCE	ITEM DESCRIPTION
2	1200 mm diameter pipeline to SIRC
3	1500 mm diameter pipeline SIRC to Lake A
5	900 mm diameter pipeline Lake B to Wildlife Lake
7	5 x 3m x 0.9m culverts Lewis Lagoon to Wildlife Lake
8	900 mm diameter pipeline Cranebrook Lake to Duralia Lake
9	900 mm diameter pipeline Duralia Lake to North Pond
10	2 x 900 mm pipes connecting West Pond and North Pond
11	Overflow weir 20m crest width on Farrell's Creek
12	1200 mm pipe plus a weir connecting North Pond and Middle Basin
13	3m x 1.8m culvert connecting Middle and Final Basins
14	3m x 1.8m culvert connecting Final Basin and SIRC
16	Sluice gate to control flows between Lake B and Wildlife Lake
17	Sluice gate to control flows between Duralia Lake and North Pond
18	Sluice gate to control flows between Final Basin and SIRC

ANNEXURE B3

EXTRACTS FROM *PENRITH LAKES WATER MANAGEMENT PLAN: STAGE 2* (PLDC, 2020)

2.2 LAKE STRUCTURE

The lakes have been designed to provide optimal operational flexibility in achieving the desired end water uses and maximise public amenity. The ultimate design included wetlands to assist with improving the overall ecosystem performance and water quality and the capacity for significant water storage buffer above the prescribed operating levels approved in the Stage 1 Water Management Plan. Operational flexibility is also optimised through adopting the hierarchy of lakes as set out above. Physical attributes including the size, capacity and recommended water level tolerances of the lakes are provided below in Table 1 and details on the wetlands in Table 2.

.

Table 1: Surface areas, operating water levels, volumes and recommended water level tolerances in the various Scheme water bodies

LAKE	SIZE (HA)	AVERAGE DEPTH (M)	OPERATING LEVEL (M AHD)	VOLUME AT OPERATING LEVEL (GL)	RECOMENDED WATER LEVEL TOLERANCES
Primary Lakes					
Wildlife Lake	110	4-5	10	3.9	-1.00m / +1.00m
Main Lake B	121	6-7	13.5	7.3	-1.00m / +1.00m
Main Lake A	318	5-6	14	17.8	-1.00m / +0.50m
Quarantine Lake	42	6-7	15	2.4	-0.25m / +0.40m
Regatta Lake	80	5-6	15	4.2	-0.25m / +0.40m
Treatment Lakes					
Duralia Lake	13	10-11	18	0.9	-1.50m / +0.90m
Cranebrook Lake	3	10-11	18	0.13	-1.50m / +0.90m
Detention Basins					
Lewis Lagoon	3	5	14	0.08	-1.50m / +1.00m
North Pond	7	4	16.5	0.17	-1.50m / +1.55m
Stilling Basin	0.6	2.5	17.7	0.01	-1.50m / +0.35m
Middle Basin Wetland	5	1-2	17	0.04	-1.50m / +1.05m
Middle Basin	13	4-5	16.0	0.5	-1.50m / +2.05m
Final Basin	7	3-4	15.5	0.07	-1.50m / +2.55m
Lake Totals	723			37.5	

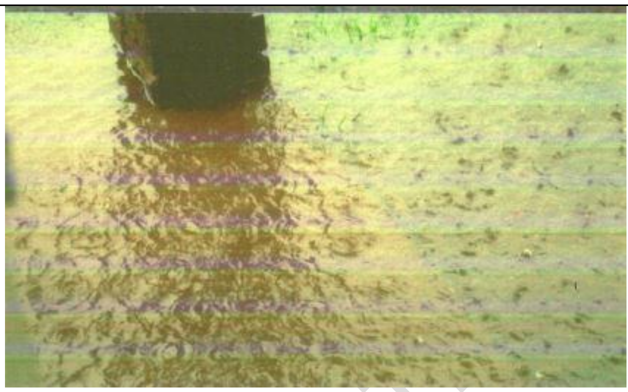
Table 2: Surface areas and recommended water level tolerances of the Scheme wetland systems

WETLANDS	SIZE (HA)	RECOMENDED WATER LEVEL TOLERANCES (M AHD)
Southern Wetlands	23	24.4 – 18
Duralia Wetlands	3.7	19 – 18
Cranebrook Wetlands	2.2	24.5 – 18
Eastern Chain of Ponds	3.6	24 – 17

APPENDIX C

PHOTOGRAPHS SHOWING OBSERVED FLOOD BEHAVIOUR IN STUDY AREA

9 FEBRUARY 2012	
	
Plate C1.1 – (Photo taken at 19:00 hours) Floodwater ponding in rear of property that is located on Soling Crescent, Cranebrook.	**Plate C1.2** – (Photo taken at 19:00 hours) Floodwater ponding at side of property that is located on Soling Crescent, Cranebrook.

JANUARY 2016	
	
Plate C2.1 – Floodwater ponding at front of property that is located on Barry Coe Place, Cranebrook.	**Plate C2.2** – Floodwater ponding at front of property that is located on Barry Coe Place, Cranebrook.

21 MARCH 2017	
	
Plate C3.1 – (Photo taken at 17:30 hours) Overland flow along driveway of property that is located on Linden Crescent, Cranebrook.	**Plate C3.2** – (Photo taken at 17:30 hours) Floodwater ponding at rear of property that is located on Linden Crescent, Cranebrook.

7 FEBRUARY 2020

(Source: George Guirguis)

Plate C4.1 – Floodwater ponding between road and front of properties that are located on the western side of Linden Crescent, Cranebrook.

9 FEBRUARY 2020

Plate C5.1 – (Photo taken at 12:00 hours) Floodwater ponding in the Borrowdale Way Basin No. 1.

Plate C5.2 – (Photo taken at 15:00 hours) Floodwater surcharging the spillway of the Borrowdale Road Basin No. 2.

Plate C5.3 – Flooding on Woodside Glen, Cranebrook.

Plate C5.4 – (Photo taken at 11:40 hours) Flooding in rear of property that is located on Woodside Glen, Cranebrook.

Plate C5.5 – (Photo taken at 10:00 hours) Concentrated flow discharging from gutter in Linden Crescent, Cranebrook.

Plate C5.6 – (Photo taken at 13:30 hours) Floodwater ponding at rear of property that is located on Linden Crescent, Cranebrook.

<table>
<tr><th colspan="2">9 FEBRUARY 2020</th></tr>
<tr><td></td><td></td></tr>
<tr><td>Plate C5.7 – Flooding on property that is located on Kenilworth Crescent, Cranebrook.</td><td>Plate C5.8 – Flooding on property that is located on Kenilworth Crescent, Cranebrook.</td></tr>
</table>

DATE NOT SPECIFIED	
	
Plate C6.1 – Location not specified.	**Plate C6.2** – Location not specified.
	
Plate C6.3 – Flooding at rear of property that is located on Waterfall Crescent, Cranebrook.	

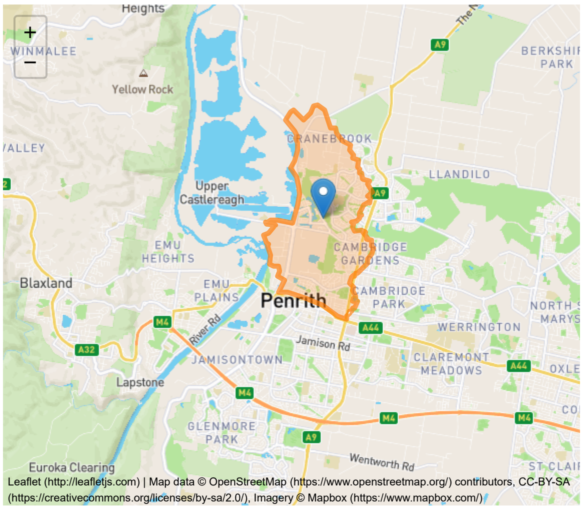
APPENDIX D

DESIGN INPUT DATA FROM ARR DATA HUB

Australian Rainfall & Runoff Data Hub - Results

Input Data

Longitude	150.704
Latitude	-33.728
Selected Regions (clear)	
River Region	show
ARF Parameters	show
Storm Losses	show
Temporal Patterns	show
Areal Temporal Patterns	show
BOM IFDs	show
Median Preburst Depths and Ratios	show
10% Preburst Depths	show
25% Preburst Depths	show
75% Preburst Depths	show
90% Preburst Depths	show
Interim Climate Change Factors	show
Probability Neutral Burst Initial Loss (./nsw_specific)	show



Data

River Region

Division	South East Coast (NSW)
River Number	12
River Name	Hawkesbury River
Shape Intersection (%)	100.0

Layer Info

Time Accessed	23 July 2021 05:37PM
Version	2016_v1

ARF Parameters

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

Zone	a	b	c	d	e	f	g	h	i	Shape Intersection (%)
SE Coast	0.06	0.361	0.0	0.317	8.11e-05	0.651	0.0	0.0	0.0	100.0

Layer Info

Time Accessed	23 July 2021 05:37PM
Version	2016_v1

Short Duration ARF

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

Storm Losses

Note: Burst Loss = Storm Loss - Preburst

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

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

Storm Initial Losses (mm)	47.0
Storm Continuing Losses (mm/h)	3.4

Layer Info

Time Accessed	23 July 2021 05:37PM
Version	2016_v1

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

code	ECsouth
Label	East Coast South
Shape Intersection (%)	100.0

Layer Info

Time Accessed	23 July 2021 05:37PM
Version	2016_v2

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

code	ECsouth
arealabel	East Coast South
Shape Intersection (%)	100.0

Layer Info

Time Accessed	23 July 2021 05:37PM
Version	2016_v2

BOM IFDs

Click here (http://www.bom.gov.au/water/designRainfalls/revised-ifd/?year=2016&coordinate_type=dd&latitude=-33.7276626104&longitude=150.704234787&s to obtain the IFD depths for catchment centroid from the BoM website

Layer Info

Time Accessed	23 July 2021 05:37PM

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)	1.2 (0.046)	1.1 (0.032)	1.1 (0.026)	1.1 (0.022)	2.1 (0.035)	2.9 (0.043)
90 (1.5)	1.7 (0.059)	1.7 (0.043)	1.7 (0.036)	1.8 (0.031)	1.3 (0.020)	1.0 (0.014)
120 (2.0)	0.0 (0.000)	0.6 (0.014)	1.0 (0.019)	1.4 (0.023)	1.6 (0.022)	1.8 (0.022)
180 (3.0)	1.0 (0.026)	1.6 (0.033)	2.1 (0.035)	2.5 (0.037)	2.4 (0.029)	2.2 (0.024)
360 (6.0)	1.8 (0.039)	7.0 (0.110)	10.5 (0.138)	13.8 (0.155)	16.8 (0.157)	19.0 (0.157)
720 (12.0)	1.0 (0.017)	4.9 (0.056)	7.4 (0.071)	9.8 (0.081)	16.0 (0.110)	20.7 (0.125)
1080 (18.0)	0.6 (0.008)	5.0 (0.048)	8.0 (0.064)	10.8 (0.073)	15.0 (0.084)	18.1 (0.090)
1440 (24.0)	0.0 (0.000)	2.9 (0.025)	4.9 (0.034)	6.7 (0.040)	10.0 (0.049)	12.4 (0.054)
2160 (36.0)	0.0 (0.000)	2.2 (0.015)	3.7 (0.021)	5.1 (0.025)	5.7 (0.023)	6.1 (0.022)
2880 (48.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.5 (0.002)	0.8 (0.003)
4320 (72.0)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)	0.0 (0.000)

Layer Info

Time Accessed	23 July 2021 05:37PM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

10% Preburst Depths

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

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

Layer Info

Time Accessed	23 July 2021 05:37PM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

25% Preburst Depths

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

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

Layer Info

Time Accessed	23 July 2021 05:37PM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

75% Preburst Depths

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

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	16.6 (0.641)	15.7 (0.436)	15.1 (0.350)	14.5 (0.288)	17.9 (0.298)	20.4 (0.302)
90 (1.5)	16.6 (0.566)	19.2 (0.476)	20.9 (0.435)	22.6 (0.403)	19.9 (0.297)	18.0 (0.237)
120 (2.0)	8.9 (0.279)	19.4 (0.442)	26.3 (0.504)	33.0 (0.542)	34.0 (0.467)	34.8 (0.422)
180 (3.0)	18.4 (0.501)	31.5 (0.634)	40.3 (0.680)	48.6 (0.706)	43.5 (0.527)	39.6 (0.423)
360 (6.0)	19.8 (0.422)	36.3 (0.569)	47.3 (0.621)	57.7 (0.650)	72.1 (0.676)	82.8 (0.684)
720 (12.0)	22.3 (0.356)	30.1 (0.350)	35.3 (0.342)	40.3 (0.332)	52.4 (0.360)	61.5 (0.372)
1080 (18.0)	20.8 (0.279)	31.9 (0.307)	39.3 (0.313)	46.4 (0.314)	53.4 (0.301)	58.7 (0.291)
1440 (24.0)	14.3 (0.170)	24.5 (0.206)	31.3 (0.217)	37.8 (0.222)	40.2 (0.197)	42.0 (0.182)
2160 (36.0)	8.8 (0.089)	16.4 (0.115)	21.4 (0.124)	26.3 (0.128)	34.1 (0.138)	39.9 (0.143)
2880 (48.0)	2.9 (0.026)	4.3 (0.027)	5.2 (0.027)	6.1 (0.026)	11.5 (0.041)	15.6 (0.050)
4320 (72.0)	0.0 (0.000)	0.2 (0.001)	0.4 (0.002)	0.5 (0.002)	7.2 (0.023)	12.3 (0.034)

Layer Info

Time Accessed	23 July 2021 05:37PM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

90% Preburst Depths

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

min (h)\AEP(%)	50	20	10	5	2	1
60 (1.0)	47.0 (1.810)	49.3 (1.369)	50.8 (1.180)	52.3 (1.042)	78.0 (1.300)	97.3 (1.435)
90 (1.5)	50.1 (1.704)	79.3 (1.962)	98.6 (2.045)	117.1 (2.087)	91.6 (1.365)	72.4 (0.955)
120 (2.0)	30.1 (0.936)	76.6 (1.745)	107.4 (2.053)	136.9 (2.250)	122.6 (1.684)	111.8 (1.358)
180 (3.0)	38.2 (1.044)	65.6 (1.320)	83.8 (1.416)	101.2 (1.469)	115.6 (1.402)	126.4 (1.352)
360 (6.0)	46.2 (0.982)	76.0 (1.190)	95.7 (1.257)	114.7 (1.290)	126.8 (1.189)	135.9 (1.122)
720 (12.0)	45.0 (0.720)	69.9 (0.812)	86.3 (0.836)	102.1 (0.843)	112.8 (0.774)	120.8 (0.730)
1080 (18.0)	44.7 (0.601)	60.5 (0.582)	70.9 (0.566)	80.9 (0.549)	101.0 (0.569)	116.0 (0.576)
1440 (24.0)	44.0 (0.523)	52.5 (0.441)	58.0 (0.403)	63.4 (0.373)	84.1 (0.412)	99.6 (0.431)
2160 (36.0)	33.9 (0.342)	43.1 (0.302)	49.1 (0.283)	54.9 (0.268)	72.5 (0.294)	85.7 (0.308)
2880 (48.0)	16.9 (0.153)	17.8 (0.111)	18.4 (0.094)	18.9 (0.082)	48.9 (0.176)	71.3 (0.228)
4320 (72.0)	12.6 (0.101)	22.6 (0.123)	29.2 (0.130)	35.6 (0.134)	38.2 (0.120)	40.1 (0.112)

Layer Info

Time Accessed	23 July 2021 05:37PM
Version	2018_v1
Note	Preburst interpolation methods for catchment wide preburst has been slightly altered. Point values remain unchanged.

Interim Climate Change Factors

	RCP 4.5	RCP6	**RCP 8.5**
2030	**0.869 (4.3%)**	0.783 (3.9%)	**0.983 (4.9%)**
2040	**1.057 (5.3%)**	1.014 (5.1%)	**1.349 (6.8%)**
2050	**1.272 (6.4%)**	1.236 (6.2%)	**1.773 (9.0%)**
2060	**1.488 (7.5%)**	1.458 (7.4%)	**2.237 (11.5%)**
2070	**1.676 (8.5%)**	1.691 (8.6%)	**2.722 (14.2%)**
2080	**1.810 (9.2%)**	1.944 (9.9%)	**3.209 (16.9%)**
2090	**1.862 (9.5%)**	2.227 (11.5%)	**3.679 (19.7%)**

Layer Info

Time Accessed	23 July 2021 05:37PM
Version	2019_v1
Note	ARR recommends the use of RCP4.5 and RCP 8.5 values. These have been updated to the values that can be found on the climate change in Australia website.

Probability Neutral Burst Initial Loss

min (h)\AEP(%)	50.0	20.0	10.0	5.0	2.0	1.0
60 (1.0)	26.0	18.6	16.5	16.8	15.6	12.6
90 (1.5)	29.4	17.5	15.1	15.0	14.8	12.9
120 (2.0)	32.1	20.9	16.1	14.6	12.6	10.5
180 (3.0)	35.4	19.6	15.7	15.2	14.0	10.4
360 (6.0)	33.5	19.5	15.9	14.8	12.4	8.4
720 (12.0)	34.4	23.4	21.5	21.0	18.5	9.7
1080 (18.0)	35.3	25.5	24.5	23.1	20.7	11.6
1440 (24.0)	37.5	29.2	28.8	28.6	25.9	17.4
2160 (36.0)	40.9	34.0	32.9	33.4	29.7	15.3
2880 (48.0)	46.2	40.6	41.5	45.8	35.0	19.0
4320 (72.0)	48.3	41.7	42.3	47.1	38.9	27.4

Layer Info

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

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Download JSON (downloads/ffb21886-a048-4e07-9dc0-78eb2ee037ee.json)

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APPENDIX E

ARR, 2019 DESIGN BLOCKAGE ASSESSMENT AT HYDRAULIC DRAINAGE STRUCTURES

TABLE E1
ARR, 2019 DESIGN BLOCKAGE ASSESSMENT AT HYDRAULIC DRAINAGE STRUCTURES

ID[1]	Study Catchment	Structure Details: Structure Type[2]	Structure Details: Width	Structure Details: Height (m)	Structure Details: No. of Barrels	Floating Debris: L_{10}[3]	Floating Debris: Debris Availability	Floating Debris: Debris Mobility	Floating Debris: Debris Transportability	Floating Debris: Debris Potential	Floating Debris: Debris Potential at Structure	Floating Debris: Adjusted Debris Potential > 5% AEP	Floating Debris: Adjusted Debris Potential 5% - 0.5% AEP	Floating Debris: Adjusted Debris Potential < 0.5% AEP	Floating Debris: Most Likely Design **Inlet** Blockage (B_{DES}%) > 5% AEP	Floating Debris: Most Likely Design **Inlet** Blockage (B_{DES}%) 5% - 0.5% AEP	Floating Debris: Most Likely Design **Inlet** Blockage (B_{DES}%) < 0.5% AEP	Non-Floating Debris: Likelihood of Deposition	Non-Floating Debris: Debris Potential at Structure	Non-Floating Debris: Adjusted Debris Potential > 5% AEP	Non-Floating Debris: Adjusted Debris Potential 5% - 0.5% AEP	Non-Floating Debris: Adjusted Debris Potential < 0.5% AEP	Non-Floating Debris: Most Likely Design **Barrel** Blockage (B_{DES}%) > 5% AEP	Non-Floating Debris: Most Likely Design **Barrel** Blockage (B_{DES}%) 5% - 0.5% AEP	Non-Floating Debris: Most Likely Design **Barrel** Blockage (B_{DES}%) < 0.5% AEP	Adopted Design Blockage B_{DES}% > 5% AEP	Adopted Design Blockage B_{DES}% 5% - 0.5% AEP	Adopted Design Blockage B_{DES}% < 0.5% AEP
NorthC1	North Penrith	C Culvert	1.2	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
NorthC2	North Penrith	R Culvert	1.2	1.2	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
NP_HW_102	North Penrith	C Culvert	0.75	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
LA_58	Boundary Creek	C Culvert	0.375	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
LA_59	Boundary Creek	C Culvert	0.375	-	1	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3611	Boundary Creek	C Culvert	0.375	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
CB_m1	Boundary Creek	R Culvert	1	0.45	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
CB_m2	Boundary Creek	R Culvert	1.6	0.3	2	1.5	L	M	L	LML	Low	Low	Low	Medium	0%	0%	10%	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%
CB_m31	Boundary Creek	R Culvert	1.6	0.3	2	1.5	L	M	L	LML	Low	Low	Low	Medium	0%	0%	10%	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%
NP_Cul	Boundary Creek	R Culvert	3.3	2.4	2	1.5	L	M	M	LMM	Low	Low	Low	Medium	0%	0%	10%	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%
PI2242	Boundary Creek	C Culvert	0.9	-	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI2243	Boundary Creek	C Culvert	0.375	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3695	Boundary Creek	C Culvert	0.225	-	1	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3746	North Penrith	C Culvert	0.375	-	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3049a	Boundary Creek	C Culvert	0.45	-	2	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
CB_m5	Boundary Creek	R Culvert	4.3	0.9	1	1.5	L	M	L	LML	Low	Low	Low	Medium	0%	0%	10%	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%
PI3820a	Boundary Creek	C Culvert	0.9	-	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
LA	Boundary Creek	C Culvert	0.525	-	3	1.5	L	H	M	LHM	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3620	Boundary Creek	R Culvert	2.7	2.4	2	1.5	M	H	M	MHM	Medium	Low	Medium	High	0%	10%	20%	Low	Low	Low	Low	Medium	0%	15%	25%	0%	15%	25%
PI2744	Boundary Creek	R Culvert	1.2	0.42	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3678	Boundary Creek	C Culvert	1.5	-	2	1.5	L	M	M	LMM	Low	Low	Low	Medium	0%	0%	10%	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%
LA_8	Andrews Road	C Culvert	0.6	-	2	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI1481	Boundary Creek	C Culvert	0.75	-	4	1.5	M	H	M	MHM	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI1481a	Boundary Creek	C Culvert	0.45	-	2	1.5	M	H	M	MHM	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
RHCO_2	Penrith Lakes Local	C Culvert	0.6	-	2	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
RHCO_5	Penrith Lakes Local	C Culvert	0.3	-	1	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
RHCO_4	Penrith Lakes Local	C Culvert	0.375	-	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
RHCO_3	Penrith Lakes Local	C Culvert	0.6	-	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
LA_51	Penrith Lakes Local	C Culvert	0.375	-	1	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
LA_52	Penrith Lakes Local	C Culvert	0.375	-	1	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3547	Andrews Road	R Culvert	3.6	0.9	1	1.5	L	M	L	LML	Low	Low	Low	Medium	0%	0%	10%	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%
PI3634	Andrews Road	R Culvert	3.6	1.18	5	1.5	L	M	M	LMM	Low	Low	Low	Medium	0%	0%	10%	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%
PI2131	Andrews Road	R Culvert	1.5	0.9	1	1.5	L	M	M	LMM	Low	Low	Low	Medium	0%	0%	10%	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%
PI3117	Andrews Road	C Culvert	0.15	-	2	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
RHCO_7a	Penrith Lakes Local	C Culvert	0.25	-	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
RHCO_7b	Penrith Lakes Local	C Culvert	0.25	-	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3672	Andrews Road	C Culvert	0.9	-	1	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3670	Andrews Road	C Culvert	0.6	-	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3671	Andrews Road	R Culvert	4.2	0.88	1	1.5	M	H	M	MHM	Medium	Low	Medium	High	0%	10%	20%	Low	Low	Low	Low	Medium	0%	15%	25%	0%	15%	25%
PI3671a	Andrews Road	C Culvert	0.9	-	1	1.5	M	H	M	MHM	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3673	Andrews Road	C Culvert	0.9	-	7	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3006	Andrews Road	C Culvert	0.75	-	1	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3674	Andrews Road	R Culvert	3.6	1.8	3	1.5	L	M	M	LMM	Low	Low	Low	Medium	0%	0%	10%	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%
PI3008	Andrews Road	C Culvert	0.75	-	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3122	Andrews Road	C Culvert	0.75	-	1	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3537	Andrews Road	C Culvert	0.75	-	1	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3570	Andrews Road	C Culvert	0.6	-	1	1.5	M	H	M	MHM	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3013	Andrews Road	C Culvert	0.45	-	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3012	Andrews Road	C Culvert	0.75	-	1	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3610	Andrews Road	C Culvert	0.6	-	1	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI1640	Andrews Road	R Culvert	1.2	0.38	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
LA_26	Andrews Road	C Culvert	0.375	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3369	Cranebrook Road South	R Culvert	0.9	0.6	1	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI0922	Cranebrook Road South	R Culvert	2.4	0.9	1	1.5	L	H	M	LHM	Medium	Low	Medium	High	0%	10%	20%	Low	Low	Low	Low	Medium	0%	15%	25%	0%	15%	25%
PI0446	Cranebrook Road South	R Culvert	2.4	0.9	1	1.5	L	H	M	LHM	Medium	Low	Medium	High	0%	10%	20%	Low	Low	Low	Low	Medium	0%	15%	25%	0%	15%	25%
PI0461	Cranebrook Road South	R Culvert	2.7	1.2	2	1.5	L	H	M	LHM	Medium	Low	Medium	High	0%	10%	20%	Low	Low	Low	Low	Medium	0%	15%	25%	0%	15%	25%
LA_55	Penrith Lakes Local	C Culvert	0.375	-	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
RHCO_9	Penrith Lakes Local	C Culvert	0.45	-	1	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
RHCO_8a	Penrith Lakes Local	C Culvert	0.3	-	2	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3659	Penrith Lakes Local	C Culvert	0.375	-	2	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
WS_2	Cranebrook Road South	R Culvert	2.4	0.75	2	1.5	L	M	M	LMM	Low	Low	Low	Medium	0%	0%	10%	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%

TABLE E1

ARR, 2019 DESIGN BLOCKAGE ASSESSMENT AT HYDRAULIC DRAINAGE STRUCTURES

ID[1]	Study Catchment	Structure Details				Floating Debris												Non-Floating Debris								Adopted Design Blockage B_{DES}%		
		Structure Type[2]	Width	Height (m)	No. of Barrels	L_{10}[3]	Debris Availability	Debris Mobility	Debris Transportability	Debris Potential	Debris Potential at Structure	Adjusted Debris Potential			Most Likely Design Inlet Blockage (B_{DES}%)			Likelihood of Deposition	Debris Potential at Structure	Adjusted Debris Potential			Most Likely Design Barrel Blockage (B_{DES}%)					
												> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP			> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP
WS_1	Cranebrook Road South	R Culvert	2.1	0.6	5	1.5	M	M	M	MMM	Medium	Low	Medium	High	0%	10%	20%	Low	Low	Low	Low	Medium	0%	15%	25%	0%	15%	25%
PI2354	Cranebrook Road South	R Culvert	1.8	0.52	1	1.5	M	M	L	MML	Low	Low	Low	Medium	0%	0%	10%	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%
Lam_b3	Andrews Road	C Culvert	0.45	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI1721	Andrews Road	C Culvert	0.75	-	2	1.5	M	H	M	MHM	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3647	Cranebrook Road South	R Culvert	3	2.4	1	1.5	L	H	L	LHL	Medium	Low	Medium	High	0%	10%	20%	Low	Low	Low	Low	Medium	0%	15%	25%	0%	15%	25%
CRR_Arch	Olive Lane	Archway	9.5	3	3	1.5	M	M	M	MMM	Medium	Low	Medium	High	0%	0%	10%	Low	Low	Low	Low	Medium	0%	15%	25%	0%	15%	25%
CRR_2a	Olive Lane	R Culvert	3	0.9	3	1.5	M	H	M	MHM	Medium	Low	Medium	High	0%	10%	20%	Low	Low	Low	Low	Medium	0%	15%	25%	0%	15%	25%
CRR_1b	Cranebrook Road South	C Culvert	0.45	-	3	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
LA_2	Cranebrook Road South	C Culvert	0.375	-	2	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
LA_50	Cranebrook Road South	C Culvert	0.375	-	1	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI1143	Olive Lane	C Culvert	0.45	-	1	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3613	Olive Lane	C Culvert	0.6	-	3	1.5	M	M	M	MMM	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3526	Olive Lane	C Culvert	0.45	-	2	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3525	Olive Lane	C Culvert	0.75	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3523	Olive Lane	C Culvert	0.525	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3522	Olive Lane	C Culvert	0.375	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
CRR_6	Duralia Lake	C Culvert	0.375	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
CRR_7	Olive Lane	C Culvert	0.9	-	4	1.5	M	M	M	MMM	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
CRRa	Olive Lane	R Culvert	6	2	1	1.5	L	M	L	LML	Low	Low	Low	Medium	0%	0%	0%	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%
PI3615	Olive Lane	C Culvert	1.05	-	3	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
LA_42	Cranebrook Road South	R Culvert	1.8	0.3	1	1.5	M	M	L	MML	Low	Low	Low	Medium	0%	0%	10%	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%
RHCO_10	Cranebrook Road South	C Culvert	0.525	-	6	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
RHCO_10a	Cranebrook Road South	R Culvert	3	0.75	1	1.5	M	M	L	MML	Low	Low	Low	Medium	0%	0%	10%	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%
RHCO_10b	Cranebrook Road South	C Culvert	0.9	-	2	1.5	M	M	M	MMM	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
CRR_8	Duralia Lake	C Culvert	0.9	-	3	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
CRR_9	Duralia Lake	R Culvert	2.4	0.9	3	1.5	M	H	L	MHL	Medium	Low	Medium	High	0%	10%	20%	Low	Low	Low	Low	Medium	0%	15%	25%	0%	15%	25%
CRR_1	Duralia Lake	C Culvert	0.9	-	2	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3588	Duralia Lake	C Culvert	0.9	-	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3534	Duralia Lake	R Culvert	1.2	0.35	2	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3533	Duralia Lake	R Culvert	0.9	0.3	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3532	Duralia Lake	C Culvert	0.375	-	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
LA_34	Duralia Lake	C Culvert	0.9	-	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3658	Duralia Lake	C Culvert	0.9	-	2	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
LA_5	Duralia Lake	C Culvert	0.375	-	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
LA_6	Duralia Lake	C Culvert	0.9	-	2	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3515	Duralia Lake	C Culvert	0.375	-	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3662	Duralia Lake	C Culvert	0.6	-	1	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
LA_3	Duralia Lake	C Culvert	0.375	-	1	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3665	Duralia Lake	C Culvert	0.375	-	1	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3667	Duralia Lake	C Culvert	0.375	-	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3488	Duralia Lake	C Culvert	0.75	-	2	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3587	Duralia Lake	C Culvert	0.6	-	2	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3487	Duralia Lake	C Culvert	0.375	-	1	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3586	Duralia Lake	C Culvert	1.2	-	2	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3591	Duralia Lake	C Culvert	0.6	-	2	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3589	Duralia Lake	R Culvert	1.2	0.45	2	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3590	Duralia Lake	C Culvert	0.375	-	2	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
LA_4	Duralia Lake	C Culvert	0.375	-	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
LA_1	Duralia Lake	C Culvert	0.525	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3608	Duralia Lake	C Culvert	0.6	-	2	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3535	Duralia Lake	C Culvert	0.3	-	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3655	Olive Lane	C Culvert	0.3	-	2	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3654	Olive Lane	C Culvert	0.3	-	2	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3653	Olive Lane	C Culvert	0.525	-	2	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3652	Olive Lane	C Culvert	0.75	-	1	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3649	Olive Lane	C Culvert	0.375	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3648	Olive Lane	C Culvert	0.375	-	1	1.5	M	M	L	MML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3494	Olive Lane	C Culvert	0.75	-	3	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3493	Olive Lane	R Culvert	0.6	0.3	1	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3606	Olive Lane	C Culvert	0.525	-	1	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3605	Olive Lane	C Culvert	0.375	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%

TABLE E1
ARR, 2019 DESIGN BLOCKAGE ASSESSMENT AT HYDRAULIC DRAINAGE STRUCTURES

ID[1]	Study Catchment	Structure Details				Floating Debris												Non-Floating Debris								Adopted Design Blockage B_{DES}%		
		Structure Type[2]	Width	Height (m)	No. of Barrels	L_{10}[3]	Debris Availability	Debris Mobility	Debris Transportability	Debris Potential	Debris Potential at Structure	Adjusted Debris Potential			Most Likely Design Inlet Blockage (B_{DES}%)			Likelihood of Deposition	Debris Potential at Structure	Adjusted Debris Potential			Most Likely Design Barrel Blockage (B_{DES}%)					
												> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP			> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP	> 5% AEP	5% - 0.5% AEP	< 0.5% AEP
PI3598	Olive Lane	C Culvert	0.375	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3597	Olive Lane	R Culvert	0.45	0.3	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3596	Olive Lane	C Culvert	0.375	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3595	Olive Lane	C Culvert	0.375	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3492	Olive Lane	C Culvert	0.375	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3491	Olive Lane	C Culvert	0.375	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3601	Olive Lane	C Culvert	0.375	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3599	Olive Lane	C Culvert	0.375	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI0283	Olive Lane	R Culvert	0.6	0.3	1	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3490	Olive Lane	C Culvert	0.375	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3600	Olive Lane	C Culvert	0.375	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
LA_7	Olive Lane	C Culvert	0.375	-	1	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI0306	Olive Lane	C Culvert	0.375	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3609	Olive Lane	R Culvert	0.3	0.2	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3627	Olive Lane	C Culvert	0.3	-	1	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI1524	Olive Lane	C Culvert	0.375	-	1	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI1520	Olive Lane	C Culvert	0.3	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3607	Olive Lane	C Culvert	0.375	-	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI3602	Olive Lane	R Culvert	0.75	0.3	1	1.5	L	M	L	LML	Low	Low	Low	Medium	25%	25%	50%	Low	Low	Low	Low	Medium	0%	0%	15%	25%	25%	50%
PI0709	Cranebrook Road South	C Culvert	0.375	-	1	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
RHCO_11	Cranebrook Road South	R Culvert	3	2.4	1	1.5	L	M	L	LML	Low	Low	Low	Medium	0%	0%	10%	Low	Low	Low	Low	Medium	0%	0%	15%	0%	0%	15%
PI0258	Cranebrook Road South	C Culvert	0.45	-	1	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI3582	Cranebrook Road South	R Culvert	0.61	0.47	1	1.5	L	H	L	LHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI1528	Olive Lane	C Culvert	1.05	-	3	1.5	M	H	M	MHM	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%
PI1132	Olive Lane	C Culvert	0.375	-	1	1.5	M	H	L	MHL	Medium	Low	Medium	High	25%	50%	100%	Low	Low	Low	Low	Medium	0%	15%	25%	25%	50%	100%

1. Note that the plan location of each structure can be identified in the GIS layers contained in the data handover for the present study.
2. C Culvert = Circular Pipe Culvert, R Culvert = Rectangular Box Culvert
3. L_{10} is the average length of the longest 10% of the debris that could arrive at the culvert.