LORD HOWE ISLAND BOARD

LORD HOWE ISLAND FLOOD STUDY REVIEW AND UPDATE

FINAL REPORT

NOVEMBER 2021

Level 2, 160 Clarence Street
Sydney, NSW, 2000

Tel: (02) 9299 2855
Fax: (02) 9262 6208
Email: wma@wmawater.com.au
Web: www.wmawater.com.au

LORD HOWE ISLAND FLOOD STUDY REVIEW AND UPDATE

FINAL REPORT

NOVEMBER 2021

Project Lord Howe Island Flood Study Review and Update	**Project Number** 119040
Client Lord Howe Island Board	**Client's Representative** David Waterhouse
Authors Monique Retallick Mark Babister	**Prepared by** M.Retallick
Date 19 November 2021	**Verified by** M. Babister

Revision	Description	Distribution	Date
5	Final Draft	LHI Board	Nov 2021
4	Draft for Public Exhibition	Public	March 2021
3	Draft	LHI Board	May 20
2	Calibration - Draft		DEC 19
1	Data Collection- Draft Report	LHI Board	JUL 19

LORD HOWE ISLAND FLOOD STUDY REVIEW AND UPDATE

TABLE OF CONTENTS

PAGE

LIST OF TABLES

LIST OF FIGURES

LIST OF DIAGRAMS

LIST OF PHOTOS

LIST OF ACRONYMS

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

ADOPTED TERMINOLOGY

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

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

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

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

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

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

Frequency Descriptor	EY	AEP (%)	AEP (1 in x)	ARI
Very Frequent	12			
	6	99.75	1.002	0.17
	4	98.17	1.02	0.25
	3	95.02	1.05	0.33
	2	86.47	1.16	0.5
	1	63.21	1.58	1
Frequent	0.69	50	2	1.44
	0.5	39.35	2.54	2
	0.22	20	5	4.48
	0.2	18.13	5.52	5
	0.11	10	10	9.49
Rare	0.05	5	20	19.5
	0.02	2	50	49.5
	0.01	1	100	99.5
Very Rare	0.005	0.5	200	199.5
	0.002	0.2	500	499.5
	0.001	0.1	1000	999.5
	0.0005	0.05	2000	1999.5
Extreme	0.0002	0.02	5000	4999.5
			↓	
			PMP/ PMP Flood	

FOREWORD

The NSW State Government's Floodprone land Policy provides a framework to ensure the sustainable use of floodplain environments. The Policy is specifically structured to provide solutions to existing flooding problems in rural and urban areas. In addition, the Policy provides a means of ensuring that any 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 Government subsidises flood mitigation works to alleviate existing problems and provides specialist technical advice to assist Councils in the discharge of their floodplain management responsibilities.

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

1. Flood Study
- Determine the nature and extent of the flood problem.

2. Floodplain Risk Management
- 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.

WMAwater were engaged by the Lord Howe Island Board to update the Lord Howe Island Flood study for Kings Beach, Airport, and Pinetrees to Steve's Reserve catchments. This study forms the first stage of the process.

EXECUTIVE SUMMARY

The study area (Figure 1) covers three distinctly different catchments, Kings Beach, Airport, and Pinetrees to Steven's Reserve.

- The Kings Beach catchment lies immediately south of South Capella on the West Coast of the island. The total catchment area to the ocean is approximately 165 hectares.

- The Airport catchment has an approximate catchment area of 80 hectares, with ill-defined heavily vegetated drainage lines. The catchment can be subdivided into three sub-catchments; the Golf Course sub-catchment, the Blinky Beach to Cobbys Creek sub-catchment, and the third sub-catchment which drains the remaining airport area to the north of the runway.

- The Pinetrees to Steven's Reserve catchment commences immediately to the north of the airport catchment. This infiltration driven catchment has an approximate catchment area of 145 hectares.

This study builds upon an earlier study (Webb, McKeown & Associates Pty Ltd, 1998) to produce updated flood modelling for the Lord Howe Island Board. The hydraulic model has been used to reproduce the historical flood behaviour from events in 1996, and 1998. Once calibrated, the TUFLOW model has been used to define flood behaviour for a range of design events (20%, 5%, and 1% AEP and Probable Maximum Flood).

The study contains further investigations relating to:

- hydraulic and hazard analysis,
- emergency response classification,
- flood planning levels

The study also includes community consultation.

The updated Lord Howe Island hydraulic and hydrologic models are considered suitable for future use by the Board in the floodplain risk management planning process.

FLOODPLAIN RISK MANAGEMENT MEASURES

A list of possible floodplain risk management measures for the study areas based on the previous floodplain management plan and issues identified during this study was compiled. A desktop assessment of what options had been actioned and what should be assessed in the floodplain risk management plan was undertaken.

1. INTRODUCTION

Lord Howe Island was not recognised as having a flooding problem until the occurrence of two large flood events in January and June 1996. The June 1996 event was a particularly rare event. The event caused the worst flooding on record and was associated with a number of large landslips. A large rainfall event also occurred in February 1998. This confirmed the suspicions from the 1998 Flood Study (Reference 1) that the design rainfall estimates for the island were an underestimate. This led to a review of design rainfall estimates that resulted in substantial increase in design rainfalls. These revised rainfalls were used in the subsequent Floodplain Risk Management Study and Plan (Reference 2).

This study represents an update to the previous flood study. The purpose of the flood study is to define existing flood behaviour and provide tools for the investigation of management of flooding. The study area includes three separate catchments:

- The basin draining to Kings Beach
- The basin which includes the airport and golf course
- The main inhabited area of the Island extending from Pinetrees to Stevens Reserve.

This report details the investigations, results and findings of the flood study for the three catchments. This includes:

- a summary of available data,
- hydrologic model development,
- hydraulic model development,
- calibration of the hydraulic model, and
- definition of the design flood behaviour through the analysis and interpretation of model results,
- provisional hydraulic hazard,
- emergency response classifications.

A glossary of flood related terms is provided in Appendix A.

2. BACKGROUND

2.1. General

Prior to the January and June 1996 floods, the last significant flood event that could be recalled by local residents of Lord Howe Island was in the late 1920's. A number of historical floods have been recorded on a wall at Pinetrees but none of these approached the magnitude of the June 1996 event.

The June 17-18 1996 storm caused significant damage to Lord Howe Island leading to the Island being declared a Natural Disaster Area. Apart from flooding, which caused particular problems around the airport and within the Pinetrees to Steven's Reserve catchment, massive land slips changed the face of the island in many areas, closing roads and washing away foot tracks. The airport runway was badly damaged by water rising up through the pavement. The Bureau of Meteorology's (BoM) pluviography at the airport recorded most of the event but went under water in the latter part of the storm. Fortunately BoM personnel were able to provide an estimate of the residual rain after the gauge was submerged.

Within the southernmost part of the study area (Kings Beach catchment) no properties were flooded, but extensive inundation of rural lands occurred with land slips and gravel deposition. Extensive flooding occurred in the airport catchment with many parts still inundated several days after the event. Two houses were badly flooded. The Pinetrees to Steven's Reserve catchment was affected by substantial ponding of floodwaters due to the lack of any exit point for the floodwaters. A number of properties were extensively flooded.

2.2. Study Area

The study area (Figure 1) covers three distinctly different catchments, Kings Beach, Airport, and Pinetrees to Steven's Reserve.

Kings Beach

Kings Beach catchment lies immediate South of South Capella on the West Coast of the Island. The total catchment area to the ocean is approximately 165 hectares. The catchment consists of well-defined drainage lines, and an unrestricted and natural exit to the ocean. The lower flatter slopes are cleared grazing land with steep naturally forested areas in the upper catchment. The Kings Beach catchment can be considered a "conventional" catchment in that it has well defined drainage paths and has a natural exit to the ocean which is unrestricted and flood levels in the lower part are influenced by ocean conditions. There are two natural creeks draining the catchment from the southern and northern boundaries of the floodplain.

A man made channel was constructed which intersected the southern branch and directed the flow straight to the ocean through the middle of the floodplain. This is now the main drainage path. It carries all the flow in very small events (less than 20%) and spreads out over the whole floodplain in major events. Photographs 1 and 2 in Appendix B show the catchment in normal conditions, and Photograph 3, Photograph 4 and Appendix B show the extent of flooding near the peak of the June 1996 event.

Flood levels are determined by the amount of flow entering the floodplain basin, the ocean level at the downstream boundary (this only affects the reach downstream of the road), the road crossing (with its limited culvert capacity) and the overall shape of the floodplain and the drainage channels.

Airport

The total catchment area is approximately 80 hectares, with ill-defined heavily vegetated drainage lines. The catchment can be subdivided into three sub-catchments. The Golf Course sub-catchment originates in a steep forested area adjacent to the headwaters of Kings Beach Catchment, follows wide grassed valleys somewhat restricted by heavy vegetations, finally joining Cobbys Creek. There are small drains within the golf course proper but these would only carry the flows in very minor events. In larger events the flows would spread naturally across the flat valley bottoms.

The Blinky Beach to Cobbys Creek sub-catchment, covers a large part of the airport and a hill slope forested area to the south. The hill slope area drains to a natural swamp and to a series of small flood storage basins upstream of the airport road. Some of these have no outlet paths but the larger swamp near Blinky Beach has a culvert draining across to the table drain to the north of the airport access road. This culvert would clearly carry flow in either direction depending on where the runoff was sourced. The table drain on the northern side of the airport road drains in a south westerly direction towards the Cobbys Creek outlet. Near its confluence with Cobbys Creek the road access to the airport terminal crosses the table drain. Only one small culvert (diameter 300 mm approximately) is available to convey the flow. Downstream of this point the drain joins the Golf Course sub-catchment and the now well-defined creek passes through a heavily vegetated and then clear channel to the ocean. There is a beach berm at the end of the creek which is regularly maintained by the LHIB.

The third sub-catchment drains the remaining airport area to the north of the runway. A steep forested hill slope area forms the northern part of the sub-catchment. Flows originating from this area collect to the north of the airport road in a series of flood storage basins which connect to table drains on the northern and southern sides of the road. There are a number of small culverts connecting across the road. The table drains lead to a substantial single cell culvert which is the only defined outlet from this area. It passes under the runway before exiting to the ocean west of the airport terminal. The outflow is restricted by a beach berm which limits the low flow capacity of the culvert. This catchment, at least in a June 1996 type event, breaks out direct to the ocean near the north western corner of the runway.

Poor drainage and high flood levels are problems for this catchment. Even after the flood peak is reached it takes many days for the floodwaters to drain away leading to loss of vehicular access, waterlogging, and killed pasture grasses (the worst problem area is front of Mr Stan Fenton’s house).

It would appear that the flood and drainage problems of this area can be sourced back to the construction of the airport and runway in the mid 1970's. Fill for the runway was taken from the lower end of the Golf Course, and construction of the runway has meant that a major barrier to flood and drainage flows has been placed across the floodplain. It is quite apparent that the drainage provisions around the airport constructed as part of the airport re-development are completely inadequate to convey even moderate flows. They are incapable of conveying major flows and the whole area becomes a large pond which can drain only very slowly. Photographs 8 and 9 in Appendix B illustrate this.

Pinetrees to Steven's Reserve

The Pinetrees to Steven's Reserve catchment commences immediately to the north of the airport catchment (see Figure 1). This infiltration driven catchment has an approximate catchment area of 145 hectares. It largely consists of cleared land partially covered with low density urban development on the lower flatter slopes, with a relatively small surrounding catchment of forested hills draining down on to the floodplain.

It is characterised by ill-defined drainage lines and the lack of a clear outlet. With no drainage outlet to the ocean and no overflow path the only escape for floodwaters is infiltration. The infiltration rate within the catchment is high and most buildings are relatively high off the ground.

2.3. Previous Studies

A number of flood studies and assessments have previously been undertaken within the catchments. A Flood Study and a Floodplain Management Study have been previously carried out by WMAwater within the three catchment areas. After the January and June 1996 storms affecting the Island, the Lord Howe Island Board (LHIB) decided to undertake a Flood Study. A second phase included a Floodplain Management Study, which also involved a revision of the rainfall design data due to the occurrence of a new event in 1998, which was not considered in the original Flood Study. A brief overview of these reports is provided below along with other relevant studies undertaken in the catchments.

Lord Howe Island Flood Study (Webb, McKeown & Associates Pty Ltd, 1998)

The Lord Howe Island Flood Study (referred to herein as the 1998 Flood Study) investigated the flooding characteristics and behaviour (flows, flood levels and velocities) of the three catchments that form part of Lord Howe Island. A runoff routing hydrologic model (Watershed Bounded Network Model, WBNM) covering all three catchments were set up to determine the inflows to the hydraulic model. A number of large storms, with relatively good data, including the June 1996 Storm, were used to calibrate and validate the hydrologic model. A hydraulic (RUBICON) model of each catchment was developed. This study has been used for data for the current study.

Lord Howe Island Floodplain Management Study (Webb, McKeown & Associates Pty Ltd, 1999)

Following the February 1998 storm, a revision of the design rainfall data, was deemed appropriate. The previous models were tested against this event as a verification exercise. The design rainfalls were increased significantly across the whole range of design events. New design flood levels were determined and options for improving the future management of the floodplain were tested. Options recommended by this study will form a starting point for the current assessment.

Lord Howe Island Coastal Study Extract pp. 2 - 42 (Haskoning Australia Pty Ltd, 2014)

This study is a coastal study of the island but does provide some useful information for the current study. The study considers the impacts of a range of coastline management issue including beach erosion/shoreline recession, coastal lagoon/watercourse entrance instability, coastal cliff and slope instability and threats from climate change. The report considered these issues and identified immediate management actions and approvals required. The study contains a detailed discussion of local datums and a tidal planes analysis and design ocean levels which will be adapted for the current study.

3. AVAILABLE DATA

3.1. Rainfall Information

3.1.1. Historical rainfall data

Historical rainfall data was obtained at a number of locations within the study area and surrounds. Daily rainfall and pluviograph data was obtained for a number of gauges within the region from the Bureau of Meteorology (BoM) (refer to Figure 2).

The daily read stations record total rainfall for the 24 hours to 9:00 am of the day being recorded. For example, the rainfall received for the period between 9:00 am on 3 February 2008 until 9:00 am on 4 February 2008 would be recorded on the 4 February 2008.

Table 1 presents a summary of the rainfall gauges available for use in this study.

Table 1: Rainfall stations

Station Name	Station ID	Agency	Opened	Closed	Gauge type
Lord Howe Island (Milky Way)	200389	BOM	01/2000	Current	Daily
Lord Howe Island Aero NSW	200839	BOM	11/1988	Current	Daily
Lord Howe Island Aero NSW	200839	BOM	07/1994	Current	Pluviograph
Lord Howe Island (Orlando)	200375	BOM	07/2000	Current	Daily
Lord Howe Island	200440	BOM	02/1886	11/1998	Daily
Lord Howe Island	200440	BOM	09/1946	12/1998	Pluviograph
Lord Howe Island South End	200441	BOM	04/1933	12/1959	Daily

Historical rainfall data was available for a number of historic flood events including 1996 and 1998. Significant events occurred in both January and June 1996. From the daily rainfall data available the five largest events were identified. Table 2 shows these events. Based on the IFD analysis previously undertaken the June 1996 event was considered to be above the 1% AEP event for most durations. The January 1996 event is considered to be a 2% AEP event and the 1998 event close to a 1% AEP.

Table 2: Largest events determined from daily rainfall data

Date	24 hour Rainfall amount (mm)	Station ID
June 1996	449.0	200389
February 1998	374.6	200839
April 1930	304.8	200375
April 2009	265.0	200440

3.1.2. Design rainfall data

Design rainfall data available for the three catchments within Lord Howe Island is documented in References 1 and 2. This will be updated as part of the current study.

3.2. Water Level Data

3.2.1. Timeseries Water Level Data

Manly Hydraulics Laboratory (MHL) operates a water level recorder at Lord Howe Island (Table 3, Figure 2). Tide levels have been observed at the jetty since 1994 and some levels are available from the MSB prior to that date. There is anecdotal evidence that the highest level at the jetty was the underside of the girder which is approximately 2 m AHD71.

Stage hydrograph data was obtained from the MHL operated water level station. The recorded time-series of water levels was used for model calibration purposes. It should be noted that these water level recorders are located within the tidal limit and therefore provides no indication of flows. Water level recordings are available for the historic events.

Table 3: Water Level Recorder

Station Name	Agency	Station ID	Opened	Closed
Lord Howe Island	MHL	240402	08/1994	Current

Table 4 shows the water levels in the Lagoon, obtained from MHL, based on a review of data collected every 15 minutes from 1994 to 2013 for various tidal planes. Table 5 shows the exceedance probability for these levels.

Table 4: Tidal Levels in the Lagoon

Tidal plane	Water Level (m AHD71)
High High Water Solstice Springs	2.31
Mean High Water Springs	2.01
Mean High Water	1.83
Mean High Water Neaps	1.66
Mean Sea Level	1.23
Mean Low Water Neaps	0.81
Mean Low Water	0.63
Mean Low Water Springs	0.46
Indian Springs Low Water	0.24

Source: Table 1 from Lord Howe Island Coastal Study Extract pp. 2 – 42

Table 5: Probability of Exceedance for Lagoon Tidal Levels

Probability of exceedance (%)	Water Level (m AHD71)
0.1	2.53
1	2.30
5	2.05
10	1.91
50	1.23
90	0.58

Source: Table 2 from Lord Howe Island Coastal Study Extract pp. 2 – 42

3.2.1. Observed peak flood levels

A number of flood levels within the three catchments were identified in the 1998 Flood Study. Publicity through a newsletter and The Signal newspaper assisted in bringing forward information. The previous study (Reference 1) gathered a large number of photographs, which were used to identify flood levels (reproduced in Appendix B). After the June 1996 flood, permanent brass markers were placed by LHIB to identify flood levels at key locations. These were surveyed in the 1998 Flood Study. Some additional data has been included in this study from the 1996 flood event. This data was used to refine the location of the flood mark at the offices of the Lord Howe Island Board (Photo 1).

Photo 1: 1996 Flood mark plate at the office of the Lord Howe Island Board

A number of historical floods have been recorded on a wall at Pinetrees dating back to June 1995 but none of these approached the magnitude of the June 1996 event.

Historic peak flood levels reported in the 1998 Flood Study have been digitised spatially (as accurately as possible) (refer to Figure 4 and Table 30 in Appendix E). Sufficient calibration data exists for the 1996 and 1998 events for use in the current study.

3.3. Topographic Information

There is a considerable amount of topographic data available for the study area (Figure 3). However, the accuracy and suitability of these existing datasets for use in the present study varies. This includes contours, hydrosurvey, cross sections and Light Detection and Ranging (LiDAR) survey.

LiDAR survey of Lord Howe Island was obtained for the study from ELVIS. This LiDAR data has a 1 m grid resolution. The accuracy of the ground information obtained from LiDAR survey can be adversely affected by the nature and density of vegetation, the presence of steep varying terrain, the vicinity of buildings and/or the presence of water. Spatial accuracy of the LiDAR in the horizontal and vertical directions was reported as 0.8 m and 0.3 m respectively (95% confidence).

A DEM (Digital Elevation Model) at a 1 m grid resolution was used in order to:

- confirm sub-catchment and catchment watershed boundaries; and
- inform the two dimensional hydraulic model used in the study.

Contour layers (10 m and 20 m) generated from different elevation surfaces were provided by LHIB. Data was available for the Airport, Kings Beach and Pinetrees to Steven's Reserve catchments. Metadata has not been provided to indicate accuracy or the elevation dataset from which they were derived.

NSW Maritime conducted a hydrographic survey of the Lagoon (eastern side of LHI) in October 2008. This was recorded using ODOM CVX3 Echo Sounder and is based on LHI AHD 71 Datum. Mapping of the Survey has been obtained in PDF format.

The Port Authority of New South Wales conducted a survey of the Lagoon in March 2015. This data set is reduced to zero and is based on the Lord Howe Island Hydro Datum being approximately the level of Lowest Astronomical Tide. This zero is 0.144 metres above Local AHD.

Cross sections were also available from the 1998 RUBICON model. These were used to confirm that the LiDAR had recorded the channel inverts.

3.1. Aerial Imagery

High resolution aerial imagery of Lord Howe Island has been obtained from NSW Six Maps for this study. This was utilised in the assigning of Manning's n values and identifying catchment changes.

3.2. Culvert and Structure Data

Some culvert and structure data was available from the 1998 Flood Study for inclusion in the hydraulic model.

In addition, site photos, measurements of opening widths and pipe network layout figures were collected by Lord Howe Island Board Staff (refer to Figure 3 and Appendix D).

3.3. Cadastre and LPI Data

A cadastre of Lord Howe Island has been obtained from LHIB for this project. Additionally, Land and Property Information (LPI) layers for drainage paths, drainage areas, areas of interest and the mean high water tidal extent were provided by LHIB, however the date at which the data was recorded was not provided.

3.4. Previous models

The previous Flood study of Lord Howe Island (Reference 1) developed a WBNM hydrologic model. This model will be further refined using the most up to date data and techniques for the present study.

A 1D RUBICON hydraulic model was developed in the previous Flood Study (Reference 1). This model will be used as reference data only. A new two dimensional TUFLOW model will be set up for the current study.

3.5. Community Consultation

One of the central objectives of the Flood Study process is to provide the local community with a community accepted resource that can be utilised for all flood related issues including development, flood warning, response and management/remediation.

Newsletters were posted to the community by LHIB. A total of 12 responses were received through both email responses and online survey. Of these 2 reporting flooding since 1998. The respondents did not indicate they had photos or flood marks for the flood events since 1998 that could be used in the calibration process.

4. MODELLING APPROACH

The primary objective of this study is to define the flood behaviour under historical and existing floodplain conditions in the Study Area while addressing possible future variations in flood behaviour due to climate change and provide information for its management.

The approach adopted for this study has been influenced by the study objectives, accepted practice and the quality and quantity of available data. There are two basic approaches to determining design flood levels namely:

- a *flood frequency approach* based upon a statistical analysis of the flood record, and
- using a *rainfall/runoff routing* approach (hydrologic modelling) to obtain flows, and then inputting these flows into a hydraulic model of the study area

A rainfall/runoff routing approach was adopted for the current study due to the lack of a long-term water level gauge for use in flood frequency analysis.

A hydrologic (WBNM, Watershed Bounded Network Model) model was established for each catchment to determine inflows into the hydrodynamic model. A two-dimensional hydrodynamic (TUFLOW) model was used to define the flood behaviour using LiDAR and hydrosurvey.

The TUFLOW models were calibrated and verified to the June 1996 and February 1998 events.

The calibrated hydraulic models were then used to assess the flood levels and hydraulic flood hazard for the 20, 5, 1% AEP and PMF events.

5. INTENSITY FREQUENCY DURATION INFORMATION

To determine the design flood behaviour within the catchment, it is necessary to obtain design rainfall data. Design rainfall is based on statistical analysis of historical rainfall events to determine rainfall that has a certain probability of occurring; often identified as an ARI or AEP.

Design rainfalls derived by Bureau of Meteorology specifically for Lord Howe Island (Reference 1) were used for the 1998 Flood Study. These were revised by Webb McKeown and Associates (now WMAwater) in the 1999 Floodplain Risk Management Study after the occurrence of a significant rainfall event in February 1998.

Since this time, the distributions and methods used to fit design rainfalls have changed and additional rainfall data (20 years) is available. Design rainfalls (Intensity Frequency Duration, IFDs) over Australia were developed by Bureau of Meteorology for the 2019 version of Australian Rainfall and Runoff (ARR2019). However, these IFDs did not cover Lord Howe Island. Therefore, IFDs were derived for use in this flood study, based on recorded rainfall data (refer to section 3.1) on Lord Howe Island using updated techniques. The derivation of IFDs is described in this section.

5.1. Method

The following method was used to develop IFDs for Lord Howe Island.

1. At each site, rainfall was totalled for a range of durations from 30 minutes to 4320 minutes, based on a moving window. For daily rainfall sites, only durations of one day or greater were calculated.
2. Annual maximum series were extracted for each site (refer to Appendix C) for each duration. In some records there were flags to indicate if data recordings at the fixed time step were missed but that the total equals the total in the missed timesteps. These values were included if the burst duration being calculated was larger than the gap. Sensitivity testing showed that inclusion of these values made very little difference to the results.
3. Factors were applied to the annual maximum series for the daily rainfall sites (ARR2019 Book 2, Table 2.3.4) to convert rainfall from the restricted period of 9 am to 9 am, to an unrestricted period.
4. The annual maximum series for the two pluviograph sites were pooled, as they did not include concurrent data.
5. For durations of 24 hours or greater, a combined annual maximum series was derived based on the following procedure:
   - Where pluviograph and daily data was available at the same gauge for a given year, the pluviograph data was used,
   - In years where data was only available from one site, the annual maximum from that site was used,
   - In years where data was available at multiple sites, the maximum of the annual maxima from all sites was used. Using the mean of the annual maxima was also investigated and the difference in results was found to be insignificant.
6. Generalised Extreme Value (GEV) distributions were fitted to the individual and combined annual maximum series using the method of L-moments.

5.2. Results

The design rainfalls derived for each duration are shown in Figure 6 and Table 6. Overall the IFDs have reduced from the 1998 estimates. This is not unexpected due to the lack of major storms in the last 20 years.

Table 6: Design rainfalls Depths (mm)

Duration (minutes)	Annual Exceedance Probability (%)						
	50	**20**	**10**	**5**	**2**	**1**	**0.5**
30	26.4	33.8	39.0	44.0	50.8	56.1	61.5
60	35.7	47.4	56.5	66.4	81.0	93.6	107.7
120	48.5	65.1	77.6	90.9	110.4	126.8	144.8
180	56.3	75.9	92.0	110.2	138.6	164.2	194.0
360	68.8	93.5	116.1	144.0	192.3	240.2	301.1
720	82.1	114.4	143.3	178.7	238.8	297.6	371.1
1440	114.5	166.5	207.4	252.3	319.8	378.6	445.1
2880	137.3	197.4	242.9	291.5	362.5	422.4	488.4
4320	145.6	205.9	250.9	298.3	366.6	423.3	485.1
5760	151.9	212.6	258.1	306.0	375.2	432.8	495.6
7200	157.7	218.7	265.1	314.5	386.8	447.8	515.1
8640	163.5	226.1	273.7	324.7	399.1	462.0	531.5
10080	170.4	233.8	281.3	331.5	404.1	464.7	530.8

6. HYDROLOGIC MODELLING

6.1. Overview

Hydrologic models of the Kings Beach, Airport, and Pinetrees to Steven's Reserve catchments were established as part of the study. All models were developed using the Watershed Bounded Network Model (WBNM).

The three main hydrologic models used in Australia are RORB, RAFTS and WBNM. WBNM (Watershed Bounded Network Model) is the simplest to set up as it only uses area whereas RORB and RAFTS require stream length and/or stream slope to be quantified. These two parameters can be map source and scale dependent. By using WBNM this source of uncertainty will be eliminated.

WBNM (Reference 4) is widely used throughout Australia and particularly NSW. WBNM simulates a catchment and its tributaries as a series of sub-catchment areas linked together to replicate the rainfall and runoff process through the natural stream network. Input data includes the definition of physical catchment characteristics including surface area of sub-catchments, proportion of impervious surfaces, stream length adjustments, initial and continuing losses, temporal and spatial patterns over the catchment.

Key parameters for WBNM represent the physical characteristics of the catchment. Typical model parameters include;

- Rainfall Losses: two values, initial and continuing loss, modify the amount of rainfall excess to be routed through the model sub-catchments;
- Lag Parameter: this affects the timing of the runoff response to the rainfall and is subject to catchment size, shape and slope; and
- Non Linearity Exponent: adjustment of the non-linearity of catchment response.

The parameters adopted for this study were based on the previous experience and calibration. Details of the parameters used for each of the catchments can be found in Sections 6.2 and 6.3.4.

6.2. Old model

For the 1998 flood study, a WBNM hydrologic model was set up to cover all three catchments. As no flow data was available for calibration, the model parameters were based on recommendations in the WBNM modelling guide, for ungauged catchments. The adopted C value was 1.29. Initial loss was 0.0 mm and continuing loss was 2.5 mm/h for the Kings Beach and Airport catchments, and 30 mm and 10 mm/hr for the Pinetrees to Steven's Reserve catchment.

6.3. Hydrologic Model Update

Some changes were made to the layout of sub-catchments compared with 1998 hydrologic model. The sub-catchments were redefined based on the fine resolution DEM. An additional sub-catchment was included in the north of the Pinetree catchment and an additional area on the west coast was included in the Kings Beach catchment. The model sub-catchments were altered to align with required input locations for the hydraulic model, and two of the larger sub-catchments were split into smaller sub-catchments to give more consistent sub-catchment areas over the model. The hydrologic model layout is shown in Figure 7, and the difference in catchment area is shown in Table 7.

Table 7: Change in catchment area – 2019 to 1998

Catchment	Change in catchment area
Airport	4%
Kings Beach	11%
Pinetrees to Steven's Reserve	24%

6.3.1. Calibration to Historical Events

The WBNM models were calibrated to historical events for which sufficient rainfall and observed hydrograph data existed. Adopted calibration events were those used in previous studies. No events have occurred since 1998 of a large magnitude or with sufficient calibration data available for inclusion in the study.

6.3.2. June 1996 Event

A large rainfall event occurred on the 18th June 1996, with almost 450 mm of rain recorded at the Lord Howe Island Aero pluviometer. The maximum rainfall occurred between 2 am and 4 am, with a maximum intensity estimated at 125 mm/h. The rainfall data from the Lord Howe Island Aero pluviograph was used as input to the WBNM model for both the rainfall depths and temporal pattern. This rainfall was adopted for all catchments and sub-catchments.

The hydrographs produced from the hydrologic model were input to the TUFLOW model for calibration to historic water levels.

6.3.3. February 1998 Event

A rainfall event was recorded on 12 February 1998. A total of approximately 375 mm recorded at Lord Howe Island Aero pluviometer with the maximum rainfall occurring between 4 pm and 10 pm. The rainfall data from the Lord Howe Island Aero pluviograph was used as input to the WBNM model for both the rainfall depths and temporal pattern. This rainfall was adopted for all catchments and sub-catchments.

The hydrographs produced from the hydrologic model were input to the TUFLOW model for calibration to historic water levels.

6.3.4. Parameters

The adopted loss parameters are similar to those used in the 1998 Flood study. Losses were set at 0 mm initial loss and 2.5 mm/hr continuing loss for Airport and Kings Beach catchments, and 30 mm initial loss and 10 mm/hr continuing loss for Pinetrees to Steven's Reserve catchment. The exception to this is the February 1998 event for the Pinetrees to Steven's Reserve catchment only which has an applied initial loss of 30 mm and continuing loss of 20 mm/hr. Higher than normal losses were applied for the Pinetrees to Steven’s Reserve catchment however this is compatible with the soils in the catchment and dense vegetation. Further details are provided in Section 7.5 and 7.6. The adopted Lag parameter (C) was 1.6, which is the current default parameter recommended in WBNM manual (Reference 4) for ungauged catchments.

6.3.5. Comparison with Previous Hydrologic Models – Historic Events

A comparison of the results of the hydrologic modelling with the results presented in the 1998 Flood Study shows that the hydrographs produced from the hydrologic modelling are similar in both studies (Diagram 1).

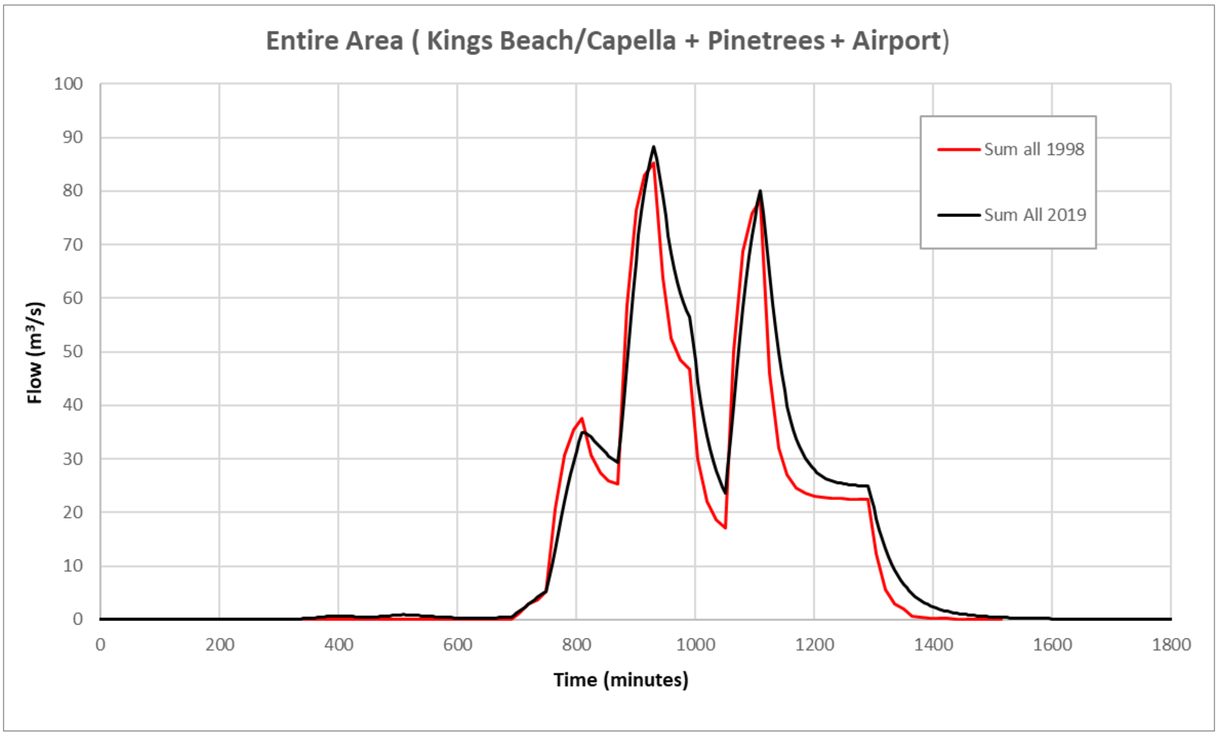

Diagram 1: Total flow from all Lord Howe Island catchments for June 1996 event.

6.3.6. Design Event Modelling

Design flow estimates were obtained for AEPs of 20%, 5%, and 1% design events. Design rainfalls used are described in Section 5.

Other inputs to the hydrologic model are the spatial and temporal patterns of rainfall. In real rainfall events and particularly in large catchments, depths of rainfall can vary significantly from one location in the catchment to another. When this occurs, spatially non-uniform rainfall (spatial pattern) can be applied to the hydrologic model. A uniform spatial pattern was used. Areal reduction factors are used to convert point IFDs to areal rainfalls. These were calculated based on formula for South East Coast region, using the equation in ARR 2019 (Reference 6). These are shown in Table 8.

Table 8: Areal reduction factors

Duration (minutes)	Annual Exceedance Probability					
	50%	20%	10%	5%	2%	1%
30	0.95	0.95	0.95	0.94	0.94	0.94
60	0.97	0.96	0.96	0.96	0.95	0.95
120	0.98	0.97	0.97	0.96	0.96	0.96
180	0.98	0.98	0.97	0.97	0.96	0.96
360	0.99	0.99	0.98	0.98	0.98	0.98
720	0.99	0.99	0.99	0.99	0.99	0.99
1440	0.99	0.99	0.99	0.99	0.99	0.99
2880	0.99	0.99	0.99	0.99	0.99	0.99
4320	1.00	1.00	1.00	1.00	1.00	1.00

Temporal patterns are a representation of how the rainfall fell over time. The temporal patterns of real storms can vary significantly, and catchments can respond very differently to the shape of the temporal pattern. For example, some rainfall events can have a significant portion of the rainfall occurring at the start of the storm burst (front loaded), and the catchment response will vary from that to a storm where a large portion of the rainfall occurs towards the end of the rainfall burst (back loaded).

ARR 2019 (Reference 6) provides ensembles of temporal patterns, i.e. 10 temporal patterns for each rainfall duration and across three AEP groups. The application of the AEP groups is shown in Table 9.

Table 9: Temporal Pattern Bins

AEP Group Name	Design AEP's Applied to
Rare	2% and 1%
Intermediate	10% and 5%
Frequent	50% and 20%

East Coast temporal patterns, obtained from ARR datahub (Reference 7), were applied to the design rainfalls as described in Section 5.2. This is considered valid as the topographic features within the zone are similar to Lord Howe Island. Ten temporal patterns were run for each duration and AEP.

Initial and continuing losses were as per the design runs in the previous study (refer to Table 10). A lower continuing loss than the calibration events was used for Pinetrees to Steven's Reserve to be consistent with the previous study. A lag parameter value of C = 1.6 was adopted for all catchments.

Table 10: Design Losses

Catchment	Initial Loss (mm)	Continuing Loss (mm/h)
Airport and Kings Beach	0	2.5
Pinetrees to Steven's Reserve	30	10

The results of the design event hydrologic modelling are shown in Figure 8 to Figure 13 for selected sub-catchments and AEPs. The critical duration of flows from the hydrologic model for Airport and Kings Beach catchments for all AEPs was 60 minutes. For Pinetree catchment, critical duration was 360 minutes. However, all patterns were run through the hydraulic model to determine the critical durations for design flood levels.

6.3.7. Comparison with previous Hydrologic Models – Design Events

The critical duration, 1% AEP event was compared between the 1999 Floodplain management study (Reference 2) and the current study, for each catchment outlet (Table 11 and Diagram 2). The differences in the peak flows are largely due to the different temporal patterns used for design, and different lag parameters in the two studies. The differences in hydrograph volumes between the two studies reflect the change in catchment area and differences in design rainfall inputs. Note that the Pinetrees to Steven's Reserve area includes an additional catchment from the previous study that drains separately.

Table 11: Comparison of 1% AEP design event – 1999 and 2019 studies

Catchment	Crit. Dur'n (mins)	Peak			Volume			1% AEP Design rainfall difference	Catchment area difference
		1998 (m^3/s)	2019 (m^3/s)	Percent difference	1998 (m^3)	2019 (m^3)	Percent difference		
Airport	60	48.5	29.0	-40%	2.62	2.35	-10%	-5%	4%
Kings Beach	60	28.9	29.7	3%	2.32	2.70	17%	16%	11%
Pinetrees to Steven's Reserve	360	34.0	22.3	-34%	14.76	13.99	-5%	-8%	24%

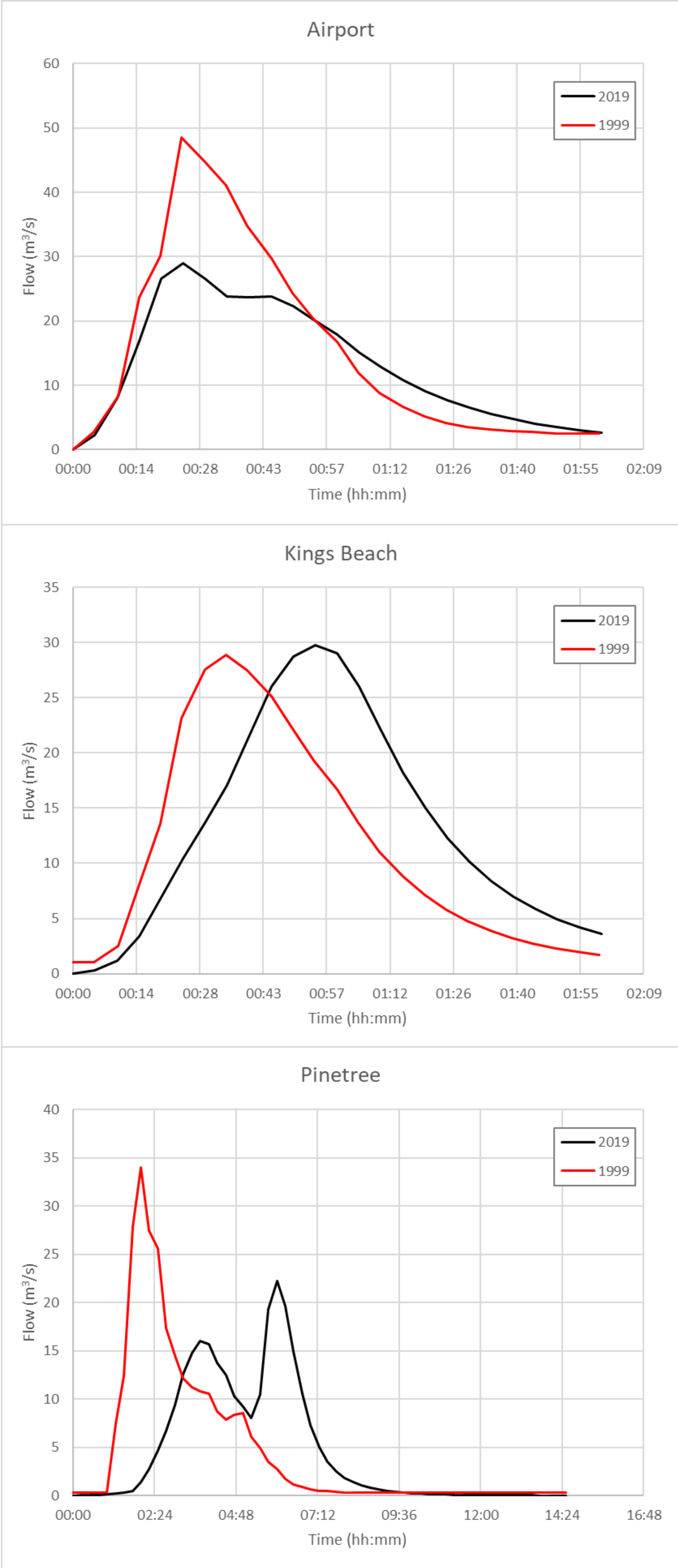

Diagram 2: 1% AEP critical duration event, comparison between 2019 and 1999 studies

6.3.8. Probable Maximum Flood

6.4. Probable Maximum Precipitation rainfall depths

Probable Maximum Precipitation (PMP) rainfall depths were calculated using the Generalised Tropical Storm Method as revised (GTSMR) (Reference 11) for durations of 24 hours and above and the Generalised Short Duration Method (GSDM) (Reference 12) for durations up to 6 hours and then interpolated between these durations. PMP estimates were calculated using the entire catchment area of all sub catchments on Lord Howe Island. As the gridded inputs for the PMP estimates (topographical adjustment factor (TAF), decay amplitude facto (DAF), extreme precipitable water (EPW)) do not extend to Lord Howe Island these were derived by moving the shapefile of Lord Howe Island catchments to the NSW coast. For DAF and EPW this was moved to the coast at the same latitude. For TAF the shapefile was further shifted to the area near Coffs Harbour so that the TAF was calculated on an area with steep elevation changes, as this occurs on Lord Howe Island. Rainfall depths were calculated for the 0.5, 1, 2, 3, 4.5, 6, 9, 12, 18, 24, 48, 72 hour durations and are presented in Table 12 and Diagram 3.

Table 12: Probable Maximum Precipitation Depths (mm)

Duration (hours)	PMP rainfall (mm)
0.5	220
1	330
2	490
3	590
4	680
4.5	720
6	790
9	890
12	970
18	1120
24	1280
48	1840
72	2310

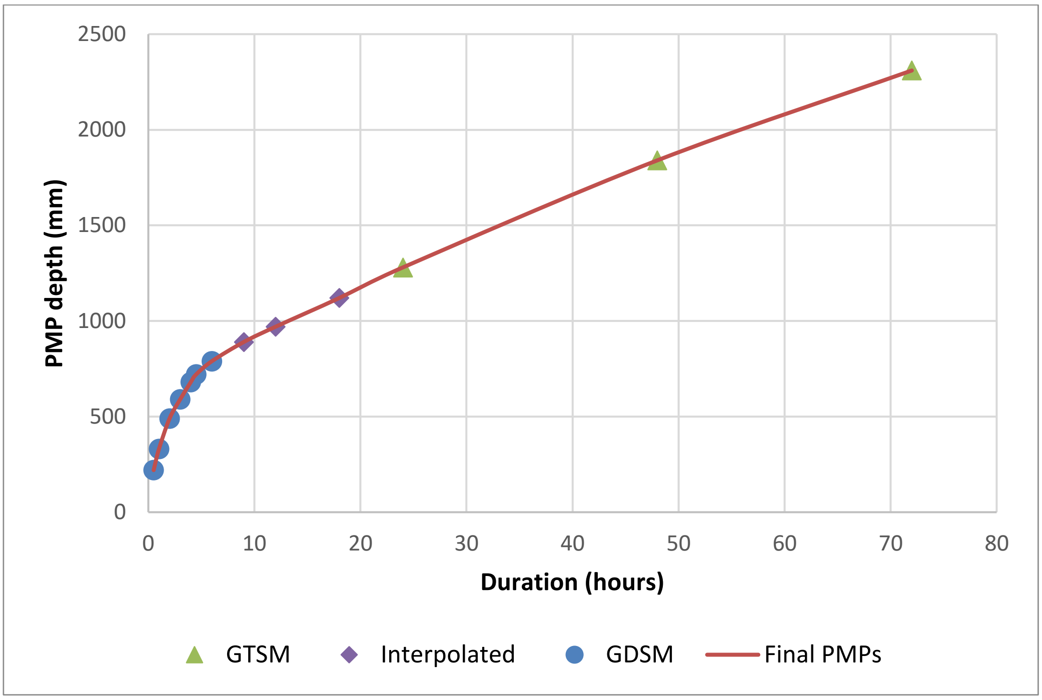

Diagram 3: PMP depths showing values derived using GSDM and GTSM and the interpolation between the two methods

6.5. Probable Maximum Flood

Probable Maximum Flood (PMF) estimates were calculated using the method in the NSW Floodplain Risk Management Guide (Reference 13). The WBNM model was run using the PMP rainfall depths. GSDM temporal patterns were used for durations up to and including 12 hours and GSTM temporal patterns were used for durations greater than 12 hours. As the catchment is small a uniform spatial pattern was used. As recommended in NSW Floodplain Risk Management Guide (Reference 13), initial loss and continuing loss values of 0 mm and 1 mm/hr respectively were used.

The WBNM model was run for all durations that the PMP rainfall depths were calculated at (see Section 6.4) and these flows were then adopted as inflows to the hydraulic model.

7. HYDRAULIC MODELLING

A model of the study area was developed in the hydrodynamic modelling package (TUFLOW). TUFLOW (Reference 5) is widely used in Australia and internationally for assessing flood behaviour and hydraulic hazard. TUFLOW is a finite difference numerical model which is capable of solving the depth averaged shallow water equations in both the one and two-dimensional domains.

The model extent for each catchment was determined based on the previous model extents and the catchments delineated for the hydrologic WBNM model.

A separate two-dimensional hydrodynamic model (TUFLOW) model of the Kings Beach, Airport, and Pinetrees to Steve's Reserve catchments was established.

7.1. Model Configuration

The model consists of a 2D 2 m grid defining the overbank and the channel for the Kings Beach, Airport, and Pinetrees to Stevens Reserve catchments. The extent of the TUFLOW models are shown on Figure 14 to Figure 16.

The model extends a sufficient distance upstream and downstream of the study area such that the imposed boundary conditions do not influence the model results in the region of interest. The TUFLOW model limits were:

Pinetree to Stevens Reserve (PTSR)

- Upstream extent to Skyline Drive,
- Upstream extent follows the ridgeline through Middle Beach Common to Transit Hill summit, and
- Downstream limit of approx. 500 m into the Lagoon

Airport

- Upstream extent to the summit of Transit Hill,
- Extent continues to Blinky Beach and approx. 600 m upstream of the Lord Howe Island Golf Club, and
- Downstream limit of approx. 600 m into the Lagoon

Kings Beach

- Upstream limit of Summit Creek extending to Mount Lidgbird,
- Upstream of Lagoon Road extending to Intermediate Hill, and
- Downstream limit of approx. 300 m into the Lagoon

A 2 metre digital terrain model (DTM) was created using the topographic data outlined in Section 3.3.

7.2. Topographic Data

The TUFLOW hydraulic models make use of the available topographic data as outlined in Section 3.3. The extents of the available data are provided on Figure 3 and includes the following:

- LiDAR survey at 1 m grid resolution
- Hydrographic survey of the Lagoon from October 2008 and March 2015

This data has been applied where available in the following order:

- LiDAR survey applied as the base elevation data
- October 2008 hydrographic survey applied where available
- March 2015 hydrographic survey applied where available

In addition to the use of the available LiDAR data, there were also some minor topographic changes made within the TUFLOW models in order to improve hydraulic representation. These changes include:

- Lowering of flow paths to improve hydraulic continuity and ensure channel inverts are correctly represented in the models,
- Smoothing of boundary between hydrodynamic survey and LiDAR survey data at Signal Point (PTSR model) and Kings Beach, and
- Smoothing of LiDAR data to improve continuity between Middle Beach Road and TC Douglass Drive (PTSR model) and downstream of Lagoon Road (Kings Beach Model).

7.3. Hydraulic Structures

Pit structures and culverts under a number of roads and the airport were incorporated in the model based on data from site inspections undertaken by LHIB. Where pit and culvert sizes and culvert lengths were not provided, they have been estimated based on provided photos and aerial imagery. Similarly, where pit and culvert invert levels have not been provided, they have been estimated based on nominal pit and culvert depths to ensure minimum grades and culvert structure are not exposed. Typically, when culvert invert levels are not provided, minimum cover of 400 mm is also used to determine culvert invert levels, but site photos indicate that minimum cover is not achieved at most locations. Locations and culvert details are provided in Table D 1 and Table D 2 (APPENDIX D).

7.4. Boundary Conditions

Inflows and boundary conditions for the TUFLOW model consist of a number of time varying flow hydrographs developed using the WBNM model. At the downstream boundary of the model, a tailwater level defining the tide level in the lagoon was used. The tailwater conditions were based on recorded tide levels at the Lord Howe Island Gauge for historic flood modelling. Figure 14 to Figure 16 show the inflow and boundary locations.

7.5. Model Calibration

Model calibration was undertaken using historical data for the 1996 and 1998 flood events. These events were adopted as a reasonable amount of observed data exists within the catchment. Time varying water level data is also available in the lagoon for these events. Previous studies on the Lord Howe Island catchments have used these events for calibration and been able to reproduce observed flood behaviour.

Inflows to the hydraulic model for these events were developed as part of the study (refer to Section 6.3).

7.5.1. Manning's n Value

The hydraulic efficiency of the creeks is represented (in part) within the TUFLOW model by the roughness or friction factor, Manning's "n" value. Manning's "n" is used to describe the influence of the following factors on flow behaviour:

- channel roughness,
- channel sinuosity,
- vegetation and other debris/obstructions in the channel, and
- bed forms and shapes

As part of the calibration process the Manning's "n" roughness value was adjusted within reasonable limits to best match the recorded flood heights along the creek system. Adopted values were selected based on an assessment of the ground cover types and vegetation density within the floodplain. The adopted values (refer to Table 13 and Figure 17) were then used for the hydraulic modelling of the design events.

Table 13: Adopted Manning's "n" Values

Description	Manning's "n" Value
General	0.040
Roads	0.020
Maintained grass	0.035
Vegetated area	0.050
Sports-field/grass	0.035
Beach/sand	0.025
Water	0.020
Buildings	0.020 – 3.000

Buildings have been represented using a depth varying Manning's "n" whereby the Manning's "n" value is dependent on the depth of flow. Manning's "n" roughness of 0.020 is applied between depths of 0 – 0.3 m. Between depths of 0.03 – 0.1 m the roughness will vary linearly between 0.02 and 3. For depths equal to and greater than 0.1 m a Manning's of 3 is applied.

7.5.2. Infiltration - Pinetrees to Steven's Reserve Catchment

The Pinetrees to Steven's Reserve catchment has no drainage outlet to the ocean and no overflow path. The only way flood waters are drained is via infiltration. The catchment sandy soils mean that a high amount of infiltration occurs. The area around Stevens Reserve is known for its high infiltration rates with water lost to "caves" or "sink holes".

As part of the calibration process the infiltration parameter was varied in order to match observed levels for the Pinetrees to Steven's Reserve catchment. A similar approach was adopted in the 1998 Flood Study.

The infiltration is modelled via two mechanism infiltration areas (as per) and a sinkhole. The 1998 Flood Study noted possible reasons for the spatial variation in infiltration rates. The fixed infiltration rate areas are defined as either 150 mm/hr or 300 mm/hr and total infiltration in the model in these areas is dependent on the length and extent of inundation. The infiltration rate has limited effect on the peak level but is key to the rate of recession of the flood.

The sinkhole location infiltration is dependent on the depth of water. In the hydraulic model this is modelled as flow vs water depth as shown on and varies up to 3.6 m^3/s at 2.5 m depth.

This value was initially adjusted from those adopted in the 1998 model in order to better match the 1996 and 1998 event recorded flood levels. These infiltration rates were also adopted in the design event modelling.

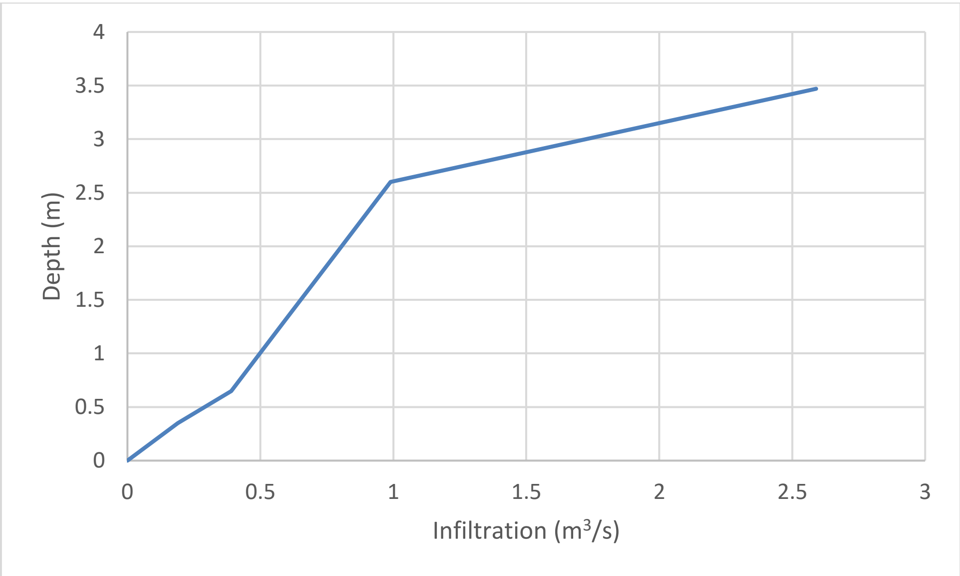

Diagram 4: Sinkhole infiltration rate

7.6. Calibration Results and Discussion

The 1996 and 1998 events were used for calibration of the hydraulic model. The TUFLOW hydraulic models, similar to the 1998 Flood Study, have been calibrated against observed peak flood levels. The location of the observed levels have been estimated spatially as shown in Figure 4 and Figure 5. Peak flood depths and levels at calibration points are shown on Figure 20 to Figure 25. Some text within this section has been copied from the 1998 Flood Study which provides a more contemporary report of the flood behaviour.

7.6.1. 1996 Event

7.6.1.1. Overview

A total of nearly 450 mm of rain was recorded at the Bureau of Meteorology - Lord Howe Island - Airport pluviograph on 18 June 1996. The maximum falls were between 0200 hours and 0400 hours, with a maximum intensity estimated to be about 125 mm/h.

This event was generated from a conglomeration of three thunderstorm cells, which formed over a large area within the Tasman Sea to the south west of Lord Howe Island. The weather system was slow moving, and this coupled with the high mountains on the Island, had the effect of anchoring the system and causing it to lift. These factors contributed to intense rainfall on the Island over a significant length of time. Whilst sea temperatures in June are generally lower than during the summer months, it is not known whether the formation of this storm system in winter would have resulted in significantly lower rainfalls than would be expected during the warmer months for a similar type event. Although a localised system of this nature is observed on average about once every year somewhere within the eastern part of Australia, its frequency of occurrence over ocean areas is not known because of the lack of observed rainfall data. Nevertheless, this type of high intensity rainfall event at any individual location is considered to be fairly rare. Statistical analysis of available pluviography data for the Island over a 50 year period (1946 to 1996) suggests that the June 1996 event is very rare.

7.6.1.2. Pinetrees to Steven's Reserve

In the June 1996 flood event water ponded throughout all the low lying areas. However, it was also noted that there was a general northerly flow of floodwaters from the Pinetrees Resort area towards Stevens Reserve at the far northern end of the catchment. There are two possible explanations for this flow (or maybe a combination of the two). The first is that the larger catchment area from the surrounding hills in the south, together with direct rainfall over the floodplain, produced a larger inflow per unit area of floodplain than the areas to the north. The higher flood levels arising by this means created a flood gradient to the north and the flow moved in this direction.

A second possible explanation lies in the relative infiltration rates. The area in and around Stevens Reserve is known for its high infiltration rates (water is lost into the "caves" or "sink holes"). The relatively higher infiltration rates, according to this explanation, thus meant that water levels dropped faster in this area creating a flood gradient for floodwaters from areas to the south to flow to the north. The modelling therefore had to be cognisant of these two possibilities to ensure that the correct mechanism was reflected in the ultimate results.

Flood levels in this catchment are therefore determined by the amount of surface flow entering the floodplain basin, the rate of infiltration, and the various hydraulic controls within the floodplain, these mainly being roads crossing generally from east to west. Even if the rate of infiltration proves to be the main mechanism driving flows from south to north, it is unlikely to control the peak levels. The June 1996 storm fell over a relatively short period of time, and the infiltration rates even at their highest, would be only a small proportion of the rainfall rate. However, the differing infiltration rates could have caused localised flow patterns to develop (as discussed above) and the overall infiltration rate is vital for the rate of recession of the floodwaters as there are no overland escape paths for floodwaters.

The width of the floodplain varies significantly. Flows originating on the slopes of Transit Hill either flow directly or are diverted by a cutoff drain to a wide floodplain lying to the east of Pinetrees. Some flows from the hill slopes also flow directly into the Pinetrees Resort. A diversion bank has been constructed on a north/south alignment between Pinetrees and the Bowling Club. This effectively keeps flows originating from the Transit Hill area out of the Resort in small, more frequent, storm events. Flows ponding within the Resort can flow northwards towards The Oval, which is a large ponding area adjoining the Bowling Club. From this area flows move northwards through the school and the LHIB office complex where they join with flows passing to the east of Pinetrees Resort/the Bowling Club. At this point the floodplain is still wide but has narrowed when compared with the width at Pinetrees.

After leaving the LHIB area flows move northward through open paddock areas to the Anglican/Catholic Churches on Middle Beach Road. The floodplain has narrowed considerably at this point, and flows to the north are constricted by the slightly elevated road. Just north of the road there is a further constriction to the floodplain caused by a rocky ridge intruding from the east. After this point the floodplain opens slightly again and flows proceed down T C Douglas Drive before discharging into an open paddock area with considerable flood storage. From this point the flows proceed northwards into another paddock area before entering Stevens Reserve. This forms the end of the floodplain as ground levels start rising again to the catchment divide.

Photographs 11 to 16 in Appendix B show the extent of flooding in this catchment during the June 1996 flood. Photograph 11 in Appendix B shows the tennis court under water at Pinetrees near the peak of the flood. Quite a large number of motel units were flooded as well as the laundry building and staff quarters. The main building was not affected. Photograph 12 in Appendix B shows a view of the Bowling Club from the “Bowling Green” near the peak of the flood. Photographs 13 and 14 in Appendix B were taken from the balcony of the Bowling Club. Photograph 13 is taken looking towards the south-east and shows the flooding of the bowling greens in the foreground with the easterly flow path referred to above in the background towards Transit Hill. Photograph 14 shows the view towards the south-west with the bowling green in the foreground and The Oval in the background with Pinetrees behind the Pinetrees to the left. Photograph 15 in Appendix B shows the ponding around the churches taken from Middle Beach Road looking south. Photograph 16 in Appendix B is taken looking to the north from T C Douglas Drive towards Stevens Reserve in the far distance.

Good flood level information was available within the catchment. Starting from the south, a number of levels were available at Pinetrees, both on a wall near the laundry building and also on flood photographs provided by the owners:

- wall near laundry (16 June 1995, 4.03 m AHD71; 7 January 1996, 4.13 m AHD71; 27 January 1996, 4.00 m AHD71),
- flood photograph June 1996, unit 47 (4.55 m AHD71).

At Pinetrees a number of the staff units at the rear were flooded in the January 1996 flood.

A very clear debris mark (4.57 m AHD71) was available at the Bowling Club for June 1996 together with a plaque placed by the LHIB (4.57 m AHD71). The Bowling Club was flooded to a depth of about 50 mm.

The LHIB were very badly flooded. Flood marks were permanently identified by plaques and these were recorded as part of the survey during the 1998 Flood Study. A plaque was found on the rear of the Anglican Church Hall and a good flood mark was available inside the church proper. Further to the north a good flood mark was available at Mr J Lonergan Senior's house. The next area affected was along the north/south section of T C Douglas Drive which was the main flood path in this area. Water almost entered the house of Ms Marj Rayward, and a good flood mark was located here. Further to the north on the edge of Stevens Reserve, water almost entered the house of Ms Patricia Dignam, and a good flood mark was located and surveyed at this location during the 1998 Flood Study. Within Stevens Reserve, a flood mark was provided by Mr Ian Hutton at the Wood Hen Breeding building.

It was reported by residents, and documented in the 1998 Flood Study, that water remained in low lying areas for some time, but that the worst of flooding was over within 2 days of the flood peak. Therefore, the infiltration rate is very high in this area. This is assisted by the fact that the normal depth to water table is almost 3 metres (personal communication - Anglican Church Minister documented in the 1998 Flood Study) allowing a substantial depth of soil to be saturated before the infiltration rate would be affected by the need for lateral flow of the groundwater towards the ocean. Given the catchment area involved, it would appear that up to 500 mm of rain could be absorbed in a reasonable period of time before saturation of the soil would occur and the groundwater table elevated above the ground surface.

7.6.1.3. Airport

In the June 1996 flood the water built up to such a level that the floodwaters broke out to the ocean at the north-western end of the runway. Photographs 8 and 9 in Appendix B show the area looking along the airport road towards Blinky Beach during the June 1996 flood (Ms May Shick's house is on the left of Photograph 8 behind the trees). Photograph 10 in Appendix B shows the ponded, poorly drained area in front of Mr Stan Fenton's house referred to above.

A large amount of ponding occurred around the airport. Two houses were flooded during the June 1996 flood. Ms Judy Wilson was flooded in her house within the Golf Course sub-catchment. She was isolated at her house and had to wade out during the peak of the storm at great personal risk. Floodwaters entered her house, and the adjoining flat to the rear, to a depth of approximately 300 mm. Good flood marks were photographed at the time and these were surveyed in as part of the 1998 Flood Study. They consisted of a level in the rear flat, a level on the rear fence, and a flood debris mark in the Golf Course at the rear of the house. The photographs showed that the floodwaters were relatively clean and very little silt was deposited within the house, which made the clean-up much easier than it is in some floods (the November 1996 flood in Coffs Harbour left up to 50 mm of silt in some houses). Ms Wilson's house was almost flooded in the January 1996 event with water getting to the top of the top step at the front of the house. Flood levels surveyed in the vicinity of Ms Wilson's house were:

- rear fence (4.22 m AHD71, June 1996),
- flood mark on rear of flat (4.38 m AHD71, June 1996),
- debris mark from Photograph 6 in Appendix B on Golf Course (4.4 m AHD71, June 1996),
- front step of house (4.13 m AHD71, January 1996).

At the peak of the June 1996 flood, water escaped to the ocean across the road adjoining Ms Judy Wilsons house causing significant scour to the road and beach dune (see Photograph 17 in Appendix B). Photographs 5 and 6 in Appendix B show the lower reaches of the Golf Course sub-catchment after the June 1996 flood had receded. Debris marks and some erosion/deposition can be seen in the photographs (see also Photograph 18 in Appendix B).

The other house that was flooded in June 1996 lies within the third sub-catchment to the north of the airport and belongs to Ms May Shick. Floodwaters peaked at approximately 100 mm within the house. Damages amounted to approximately $100,000. A flood debris mark was identified on the front wall of the house and levelled (4.35 m AHD71). Ms Shick described the runoff coming off the hill slope beside her house as like a waterfall. She was almost flooded in the January 1996 flood with the floodwaters being within 250 mm of entering the house (approx. level 4 m AHD71). Floodwaters from the sub-catchment broke out naturally to the ocean near the house likely preventing even higher flood damages.

A property owned by Mr Stan Fenton within the northern sub-catchment and at the eastern end of the runway suffered from access problems and loss of pasture grasses due to the long drainage times. Part of the airport runway drains into the property via a culvert across the road and any drainage from this area is dependent on the long flow path along the table drain, through the culvert under the runway and to the ocean through the beach berm. Photograph 7 in Appendix B shows the catchment in June 1996 conditions with the head loss through the small culvert on the airport access road being apparent. The property did not drain properly after the June 1996 event for several weeks by which time all the pasture had died and took 12 months to recover.

The flood profiles produced by the model were then compared with the flood levels at Ms Judy Wilson's house and at Ms May Shick's house. A good match was obtained which gave confidence in the model representation of the flood. Comparison with the general extent of flooding shown in Photographs 7, 9 and 10 in Appendix B also showed that the model was correctly representing the conditions experienced in June 1996.

7.6.1.4. Kings Beach

There were no flood marks in the Kings Beach catchment for the June 1996 event because no properties were inundated and there are no flood level gauges within the catchment. The flood photographs collected during the 1998 Flood Study, together with other similar photographs supplied by Mr R Shick, provided a good record at the flood peak and which enable intuitive calibration of the hydraulic model. Mr Shick was also able to describe the flood in January 1996 which almost entered his house due to diversion of flows from upstream caused by a debris blockage in the man-made drain. However, the flooding was worse in the June flood because of the volume of flow. Significant land slips occurred within the catchment and a substantial slip also intruded into the floodplain on the property of Mr Esven Fenton, partially blocking the northern branch of the creek.

The residents interviewed in the Kings Beach catchment during the 1998 Flood Study expressed the view that the rainfall in their area tended to be higher than the rest of the Island given the proximity of the high mountain peaks in the south of the Island. Given the absence of any tangible information, and the relatively close proximity of Kings Beach to the BOM airport station, no attempt was made to take this into account in the modelling.

The flood profile produced by the model was then compared with the visual information available from the previously described photographs. The model results showed that the whole floodplain was inundated to depths of up to 0.5 m, with depths in the vicinity of where the photographs were taken (near Mr R Shick's house) corresponding well with those shown in the photographs.

7.6.1.5. Results and discussion

A comparison of the June 1996 observed flood levels to the current study results and the previous study are presented in Table 14.

Table 14: June 1996 Event Calibration Results

Catchment	Location	Observed Level (m AHD71)	Modelled Level Current Study (m AHD71)	Difference Current Study (m)	Modelled Level 1998 Study (m AHD71)	Difference 1998 Study (m)
Airport	May Shick's House	4.35	4.26	-0.09	-	-
	Judy Wilson's House (flat)	4.38	4.28	-0.10	-	-
	Judy Wilson's House (back fence)	4.22	4.28	0.06	-	-
Pinetrees to Steven's Reserve	Wood Hen Pen	3.60	3.73	0.13	3.61	0.01
	Patricia Dignam	3.60	3.83	0.23	3.61	0.01
	Marj Rayward	4.00	4.13	0.13	4.10	0.10
	Jim Lon. Jnr.	4.35	4.31	-0.04	4.37	0.02
	Jim Lon. Snr.	4.45	4.48	0.03	4.47	0.02
	Anglican Church	4.52	4.49	-0.03	4.52	0.00
	LHIB	4.57	4.53	-0.04	4.57	0.00
	Bowling Club	4.57	4.53	-0.04	4.57	0.00
	Pinetrees	4.55	4.53	-0.02	4.57	0.02

The TUFLOW modelled levels for the 1996 event are generally within the range of ±0.1 m of observed values except for the levels recorded at Wood Hen Pen, Patricia Dignam's and Marj

Rayward's properties with differences of +0.13 m, +0.23 m and +0.13 m. This is still considered a reasonable calibration to the observed values. The 1998 study model had a closer alignment with the observed levels (refer to Diagram 5 to Diagram 7). This is likely due to the simplistic nature of the 1D model and how it easy it is to match limited data with simple parameter adjustments without matching true flow behaviour.

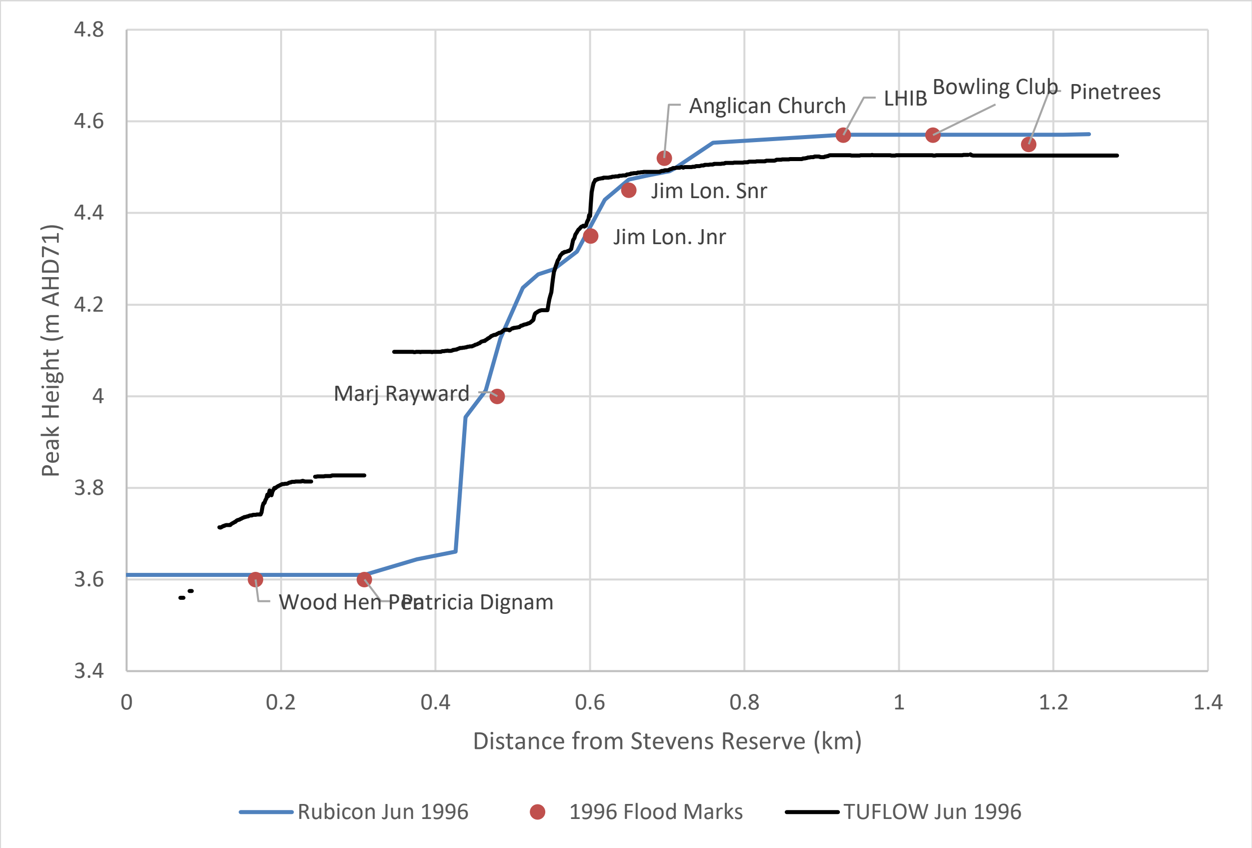

Diagram 5: June 1996 Pinetrees to Steven's Reserve Flood Level Comparison

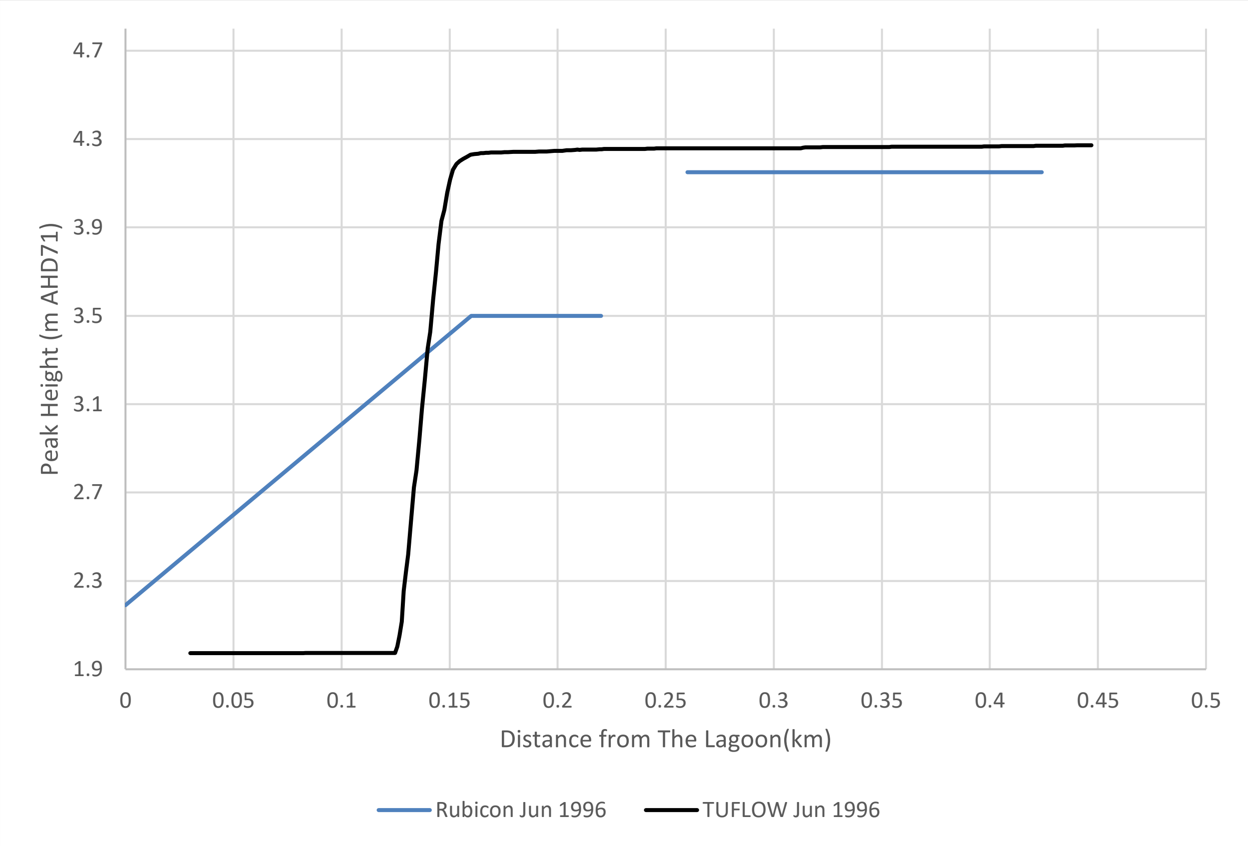

Diagram 6: June 1996 Airport Flood Level Comparison

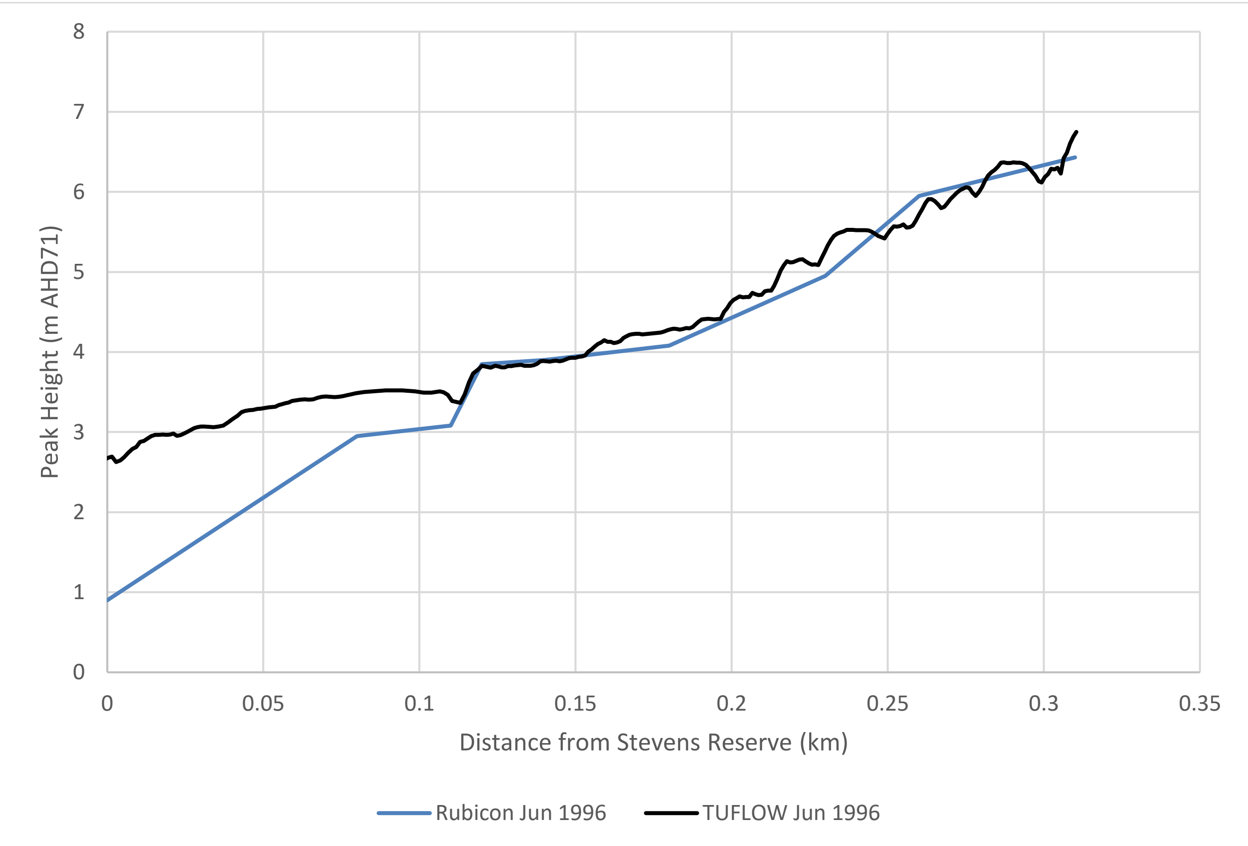

Diagram 7: June 1996 Kings Beach Flood Level Comparison

7.6.2. 1998 Event

7.6.2.1. Overview

The February 1998 storm was a very severe one for Lord Howe Island but was nowhere near as severe as the June 1996 storm. The tide gauge at the wharf recorded the ocean conditions during the storm and these were not unusually elevated. Given the poor hydraulic connection between the three catchments and the ocean, the ocean level was not a factor in determining peak flood levels.

7.6.2.2. Pinetrees to Steven's Reserve

Within the Pinetrees to Steven's Reserve catchment significant flooding occurred with a number of units flooded within Pinetrees Resort and water entering the workshop at the Lord Howe Island Board. Water was within 50 mm approximately of entering the Anglican Church. The observed levels at the LHIB and church were very approximate.

7.6.2.3. Airport

In the Airport catchment significant flooding occurred. The level at Ms Judy Wilson's house was approximately 0.3 m lower than in June 1996 and a slightly higher than in January 1996. Water did not enter Ms May Shick's house but it did enter the Eastern Airlines office to a depth of about 25 mm. Since this is in a low point it is not known whether this was due to local runoff or overall ponding.

7.6.2.4. Kings Beach

There were no reports of flooding in the Kings Beach catchment although deposited sediment was noted in the lower reaches of the man-made channel. A slip occurred near South Capella, but this was well outside of the floodplain.

7.6.2.5. Results and discussion

A comparison of the February 1998 observed flood levels to the current study results and the previous study are presented in Table 15.

Table 15: February 1998 Event Calibration Results

Catchment	Location	Observed Level (m AHD71)	Modelled Level Current Study (m AHD71)	Difference Current Study (m)	Modelled Level 1998 Study (m AHD71)	Difference 1998 Study (m)
Airport	Golf Course	4.16	4.24	0.09	-	-
Pinetrees to Steven's Reserve	Anglican Church	4.14	4.30	0.16	4.25	0.11
	LHIB	4.17	4.32	0.16	4.25	0.08
	Pinetrees	4.24	4.32	0.08	4.25	0.01

The 1998 calibration model results are generally within the range of ±0.1 m of the observed values except for the Anglican Church and LHIB where the difference is +0.16 m. These values were noted in the 1999 Floodplain Risk Management Study to be of low accuracy. As stated previously, this is considered a reasonable calibration to the observed values. There is a slight positive bias on the 1998 event.

Calibration to the observed events are dependent on the hydrologic model inputs as well as hydraulic infiltration and roughness. Significant sensitivity analysis to adjust model parameters were made in an effort to better calibrate the hydraulic model to the both events. Although this was undertaken, it was observed that, similar to the 1998 Flood Study, the modelled values were slightly higher than the observed values in the Pinetree's to Stevens Reserve catchment (refer to Diagram 8). It was therefore required that the continuing loss in the hydrologic model be increase from 10 mm/hr to 20 mm/hr for the 1998 event. It is likely that the either small errors in the recorded rainfall, either caused by the instrument or due to the location of the rainfall gauge during the 1998 event occurred. Additionally, the result could be due to high infiltration rates which have results in higher observed flood levels.

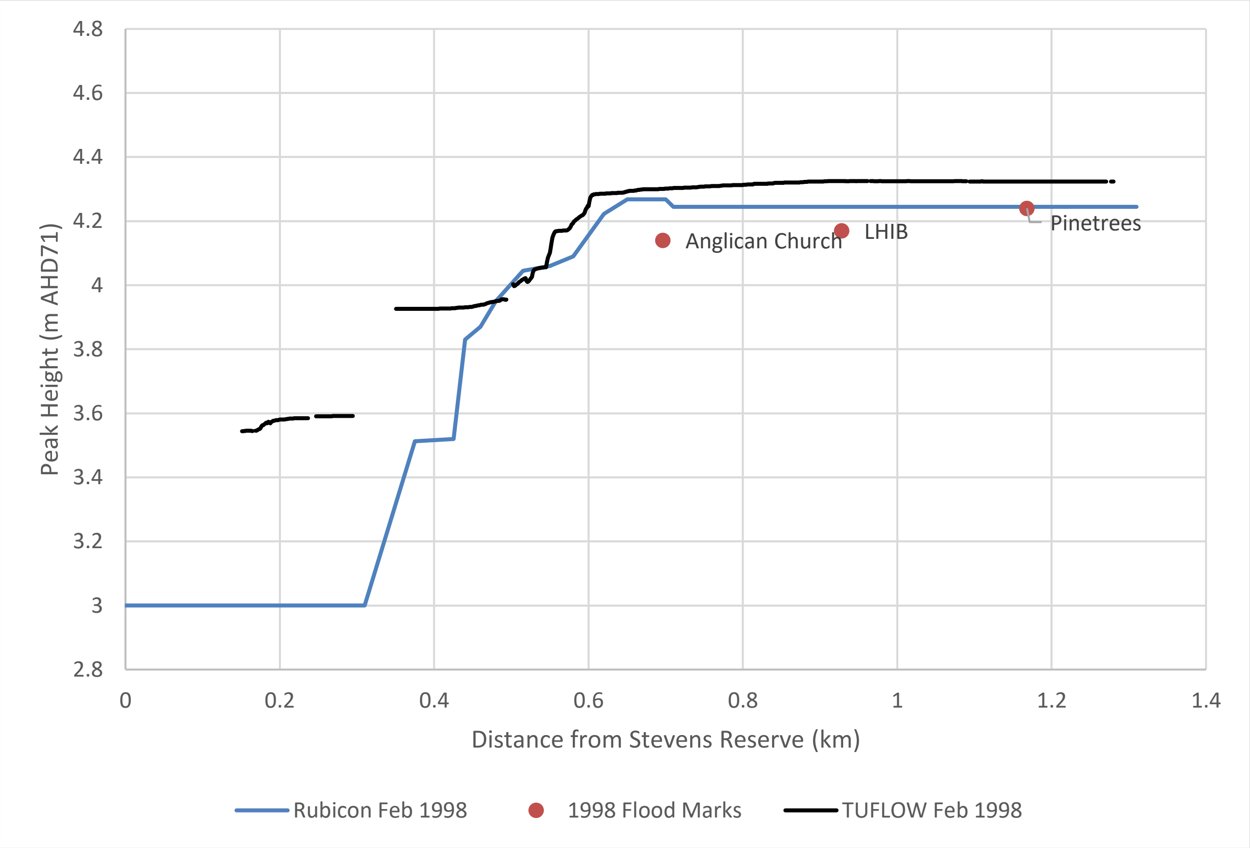

Diagram 8: June 1998 Pinetrees to Steven's Reserve Flood Level Comparison

8. DESIGN FLOOD BEHAVIOUR

8.1. Boundary Conditions

8.1.1. Design Inflows

As with the historical events the TUFLOW inflows for the 20%, 5% and 1% AEP and Probable Maximum Flood (PMF) design events were obtained from a number of time varying flow hydrographs taken from the WBNM model (refer to Section 6). These inflow hydrographs were then applied to the calibrated TUFLOW hydraulic model to produce design flood levels.

8.2. Tailwater Conditions

In addition to runoff from the catchment, the lower reaches of the catchment for Airport and Kings Beach can also be influenced by backwater effects resulting from elevated ocean levels. Hence, the height of the tide at the time of the arrival of the peak runoff from the catchment can also have an influence on flood levels in the lower reaches. However, these two distinct flooding mechanisms may or may not result from the same storm. Consideration must therefore be given to accounting for the joint probability of coincident flooding from both catchment runoff and backwater effects due to elevated ocean levels.

A full joint probability analysis is beyond the scope of the present study. Traditionally, it is common practice to estimate design flood levels in these situations using a 'peak envelope' approach that adopts the highest of the predicted levels from the two mechanisms.

Design tidal hydrographs in this study were based on a statistical analysis of ocean levels was undertaken by the Lord Howe Island Coastal Study (Reference 3). The design ocean levels in Reference 3 and reproduced in Table 5 are lower than the berm on the Pinetrees to Steven's catchment. Therefore, only rainfall dominated events were run for this catchment.

The adopted levels are significantly higher than those adopted for the 1998 Flood Study. Design ocean levels used in the 1998 Flood Study were based anecdotal evidence indicated that the highest level at the Jetty came up to the underside of the girder, this being approximately 2 m AHD71. As such, 2 m AHD71 was adopted for the 1% AEP event and slightly reduced levels of 1.6 and 1.4 m AHD71 were adopted for the 5% and 20% AEP events respectively. The current 1% AEP design ocean level is 2.3 m AHD71. Table 16 summarises the adopted ocean levels.

In addition to the above it is not unreasonable to expect that the effects of a severe storm in terms of ocean levels and runoff could be coincident for a catchment of this size. Hence to establish the design flood levels in the present study, the relative phasing of the ocean levels was adjusted such that the peak of the tidal hydrograph would approximately coincide with the peak of the catchment runoff. For example a 1% AEP catchment event was run with a mean high water springs variable tide. A 1% AEP ocean event was run with a 20% AEP catchment event. These 2 scenarios were enveloped to form the 1% AEP event.

Table 16: Adopted Tailwater and concurren flows for Design Events

Rainfall Dominated Cases		Ocean Dominated Cases		Enveloped Design Grid
Rainfall	Ocean	Ocean	Rainfall	
20% AEP Rainfall run with 20% Ocean Level (1.74m AHD71 interpolated between 10% and 50% AEP levels)				20% AEP
5% AEP	Mean High Water Springs 2.01 m AHD71	5% AEP 2.05 m AHD71	20% AEP	5% AEP
1% AEP	Mean High Water Springs 2.01 m AHD71	1% AEP 2.30 m AHD71	20% AEP	1% AEP
PMF	1% AEP 2.30 m AHD71	0.1% AEP 2.53 m AHD71	20% AEP	PMF

8.3. Design Event Results

Peak flood depths for the 20%, 5% and 1 % AEP and Probable Maximum Flood (PMF) design events are presented in Figure 26 to Figure 37. Peak flood levels a represented in Figure 38 to Figure 49. Peak velocities within the study area for the design events are presented in Figure 50 to Figure 61. Table 17 documents the design flood levels at key locations.

Table 17: Design Event Levels (Existing Conditions)

Catchment	ID	Location	Flood Level (m AHD71)			
			20% AEP	5% AEP	1% AEP	PMF
Airport	A01	Golf Club	4.01	4.11	4.22	4.56
	A02	Airport	-	-	4.22	4.58
	A03	Lagoon Road	3.95	4.08	4.22	4.57
	A04	Judys House	4.01	4.11	4.22	4.55
	A05	Airstrip	3.99	4.04	4.20	4.52
Kings Beach	KB01	Near Cappella Lodge	10.53	10.54	10.56	10.69
	KB02	Soldiers Creek US Lagoon Road	4.60	4.66	4.69	4.75
	KB03	Lagoon Rd	3.59	3.73	3.81	4.76
Pinetrees to Steven's Reserve	PT01	Pine trees	-	4.17	4.44	5.58
	PT02	Lord Howe Island Bowling Club	-	4.17	4.44	5.58
	PT03	Anglican Church		4.16	4.41	5.58
	PT04	Police Station	3.85	3.90	4.05	5.57

8.3.1. Pinetrees to Steven's Reserve

Flooding in this catchment is characterised by the ponding that occurs in the upper catchment, near Pinetrees and in Steven's Reserve. In the 5% AEP event, the radio station is inundated to depths of 0.43 m, increasing to 0.8 m in the 1% AEP event. Further downstream, in frequent events such as the 20% AEP event ponding occurs at a number of properties and businesses, with depths reaching 0.41 m at properties west of the Police station. In the 5% and 1% AEP events, this inundation increases to 0.50 m and 0.65 m respectively.

Peak flood velocities are typically less than 0.5 m/s where the flood extent intersects with properties in events up to the 1% AEP.

There is no tidal interaction in the Pinetrees to Steven's Reserve catchment in all design events.

8.3.2. Airport

The airstrip is subject to shallow depths of flooding. Significant flood depths occur south of the airport on Lagoon road and the Golf Course. In the 20% AEP event, flood depths are 1.29 m on the Golf Course. This increases to 1.38 m in the 5% AEP event, and 1.49 m in the 1% AEP event.

Peak flood velocities are typically less than 0.5 m/s where the flood extent intersects with properties in events up to the 1% AEP.

The catchment is negligibly impacted by tidal conditions, with the ocean dominated events generating peak flood levels, only in the creek between the downstream of the intersection with Lagoon Road at Cobbys Corner.

8.3.3. Kings Beach

Flooding in the Kings Beach catchment is characterised by the steep topography of Soldiers Creek. Extensive flooding occurs in the flatter region north of Lagoon Road. Although properties are inundated in events as frequent as the 20% AEP, flood depths do not exceed 0.1 m. Similarly, in the 5% AEP event, despite widespread property affectation, depths remain shallow outside the Soldiers Creek channel, and do not exceed 0.2 m at properties. In the 1% AEP the maximum depth reached at properties is 0.31 m.

The steep topography in the Kings Beach catchment generates higher velocities compared to the other catchments. Peak flood velocities are typically 2.0 m/s where the flood extent intersects with properties in events up to the 1% AEP.

In the Kings Beach catchment, peak flood levels in low lying land downstream of Lagoon Road is generated by ocean dominated flood events.

8.4. Comparison to Previous Studies

The TUFLOW hydraulic model has been compared to the 1998 Flood Study and 1999 Floodplain Risk Management Study (Reference 1 and 2) RUBICON model results. Table 18 compares the 1% AEP flood levels at key locations. Differences between the 1998/1999 Studies and the current study are a result:

- The use of a two dimensional model (current study) compared to a one dimensional model (1998/9 study), and
- Improved IFD estimates in the current study.

The results are largely similar to the flood study. The Airport catchment results are flat pond which shows the benefit on the use of a two dimensional model over a one dimensional model (as a one dimensional model will show gradient where there is none). Notable differences occur at KB 01, which would be a function of the limited ground level data outside of the main channel and is representative of the increased reliability of the two dimensional model results.

Table 18: Comparison of results to the 1998/1999 Flood Studies

Catchment	ID	Description	1% AEP Flood level (m AHD71)	
			Original Study	Current study
Airport	A01	Golf Club	4.70	4.22
	A02	Airport	4.00	4.22
	A03	Lagoon Road	4.10	4.22
	A04	Judy's House	4.40	4.22
	A05	Airstrip	3.20	4.20
Kings Beach	KB01	Near Cappella Lodge	3.80	10.56
	KB02	Soldiers Creek US Lagoon Road	4.00	4.69
	KB03	Lagoon Rd	3.80	3.81
Pinetrees to Steven's Reserve	PT01	Pine trees	4.15	4.44
	PT02	Lord Howe Island Bowling Club	4.15	4.44
	PT03	Anglican Church	4.45	4.41
	PT04	Police Station	4.10	4.05

8.5. Sensitivity Analysis

The following scenarios were considered to represent the envelope of likely parameter values:

- ± change in loss rates in the WBNM hydrologic model,
- ± 20% change in the C storage routing parameter in the WBNM hydrologic model,
- ± 20% change in Manning's "n" value, and
- Blockage of culverts

For the hydrologic model scenarios listed above the hydrologic model were run for the 1% AEP design storm. The Hydraulic model sensitivity was assessed with both the 1% AEP and 5% AEP events. The results for the hydrologic model sensitivity are provided in Table 19, Table 20 and Table 21 for the Pinetrees to Steven's Reserve, Airport and Kings Beach catchments respectively. Table 22 presents the results of the hydraulic model sensitivity assessment.

Changes in the continuing losses resulted in a change in peak flow of <1 m^3/s. A ±20% change in the storage routing parameter resulted in up to a 2.3 m^3/s change in peak flow.

A 20% increase and decrease in Manning's n value resulted in a maximum changing in flood levels of 0.03 m at Soldiers Creek US Lagoon Road in the Airport Catchment for the 5% AEP event.

All culverts were blocked by 100% to determine sensitivity to blockage. This is a likely situation based on site photos provided by the LHIB. The impacts of blockage are localised to the structures and minimal. There are no impacts due to blockage of culverts in the Pinetrees to Steven's Reserve catchment due to the placement of the culverts with respect to the primary flow paths. The model is relatively insensitive to changes in parameter values. Lagoon Road is particularly sensitive to blockage with a change in flood level of 0.12 m in the 1% AEP event and 0.15 m in the 5% AEP event.

Table 19: Pinetrees to Steven's Reserve catchment hydrologic model sensitivity

Pinetrees to Steven's Reserve Catchment Flow (m^3/s)			
C value	Losses		
	Low (24 mm IL, 8 mm/h CL)	Original (30 mm IL, 10 mm/h CL)	High (36 mm IL, 12 mm/h CL)
1.3	27.0	26.2	25.3
1.6	24.8	23.9	23.0
1.9	22.8	21.9	20.9

Table 20: Airport catchment hydrologic model sensitivity

Airport Catchment Flow (m^3/s)			
C value	Losses		
	Low (0 mm IL, 2 mm/h CL)	Original (0 mm IL, 2.5 mm/h CL)	High (10 mm IL, 3 mm/h CL)
1.3	17.9	17.8	17.6
1.6	16.7	16.5	16.4
1.9	15.8	15.6	15.5

Table 21: Kings Beach catchment hydrologic model sensitivity

Kings Beach Catchment Flow (m^3/s)			
C Value	Losses		
	Low (0 mm IL, 2 mm/h CL)	Original (0 mm IL, 2.5 mm/h CL)	High (10 mm IL, 3 mm/h CL)
1.3	28.6	28.4	28.2
1.6	26.7	26.5	26.3
1.9	25.4	25.2	25.0

Table 22: Sensitivity Assessment – Hydraulic model

Catchment	ID	Description	Flood Level (m AHD71)		Impact – Mannings decrease (m)		Impact Mannings Increase (m)		Blockage (m)	
			1% AEP	5% AEP	1% AEP	5% AEP	1% AEP	5% AEP	1% AEP	5% AEP
Airport	A01	Golf Club	4.22	4.11	-0.008	-0.014	0.009	0.015	0.020	0.028
	A02	Airport	4.22	-	-0.010	0.000	0.008	0.000	0.028	Newly Flooded
	A03	Lagoon Road	4.22	4.08	-0.010	-0.006	0.008	0.006	0.025	0.032
	A04	Judy's House	4.22	4.11	-0.008	-0.013	0.008	0.013	0.020	0.028
	A05	Airstrip	4.20	4.04	-0.009	-0.008	0.007	0.009	0.050	0.050
Kings Beach	KB01	Near Cappella Lodge	10.56	10.54	-0.003	-0.005	0.007	0.005	0.000	0.000
	KB02	Soldiers Creek US Lagoon Road	4.69	4.66	-0.025	-0.022	0.018	0.029	0.000	0.000
	KB03	Lagoon Rd	3.81	3.73	-0.006	-0.006	0.006	0.005	0.122	0.154
Pinetrees to Steven's Reserve	PT01	Pinetrees	4.44	4.17	-0.010	-0.003	0.008	0.002	0.000	0.000
	PT02	Lord Howe Island Bowling Club	4.44	4.17	-0.011	-0.003	0.007	0.003	0.000	0.000
	PT03	Anglican Church	4.41	4.16	-0.007	0.001	0.005	-0.003	0.000	0.000
	PT04	Police Station	4.05	3.90	-0.006	-0.011	0.004	0.010	0.000	0.000

8.6. Climate Change

The 2005 Floodplain Development Manual (Reference 9) requires that Flood Studies and Floodplain Risk Management Studies consider the impacts of climate change (sea level rise and rainfall increase) on flood behaviour. The following climate change scenarios (rainfall by the year 2070) are considered in this climate change assessment:

- **Increase in peak rainfall and storm volume:**
  - low level rainfall increase = 10%,
- **Sea level rise:**
  - a 0.4 m increase in level by year 2050
  - a 0.9 m increase in level by year 2100

A 10% increase is in line with the numbers determined by Engineers Australia, CSIRO and the Bureau of Meteorology as part of the revision of Australian Rainfall and Runoff.

Sea level rise was not assessed for the Pinetrees to Steven's Reserve catchment as it has no outlet to the ocean and the berm is above 4 m AHD71.

A 10% increase in rainfall results in up to a 0.04 m increase in flood levels.

A 0.4 m and 0.9 m sea level rise result in an increase in flood levels in the lower reaches of the Kings beach catchment.

Table 23 and Table 24 summarises the impact of climate change on the 1% AEP and 5% AEP flood levels respectively.

Table 23: Impact of Sea level Rise and Rainfall Increase with Climate Change for the 1% AEP event

Catchment	ID	Description	1% AEP Flood Level (m AHD71)	1% AEP Impact (m)		
				Rainfall Increase	Rainfall Increase, SLR +0.4 m	Rainfall Increase, SLR +0.9 m
Airport	A01	Golf Club	4.22	0.04	0.04	0.04
	A02	Airport	4.22	0.04	0.04	0.04
	A03	Lagoon Road	4.22	0.04	0.04	0.04
	A04	Judy's House	4.22	0.04	0.04	0.04
	A05	Airstrip	4.20	0.04	0.04	0.04
Kings Beach	KB01	Near Cappella Lodge	10.56	0.01	0.01	0.01
	KB02	Soldiers Creek US Lagoon Road	4.69	0.01	0.01	0.01
	KB03	Lagoon Rd	3.81	0.04	0.04	0.04
Pinetrees to Steven’s Reserve	PT01	Pine trees	4.44	-0.07	-	-
	PT02	Lord Howe Island Bowling Club	4.44	-0.07	-	-
	PT03	Anglican Church	4.41	-0.07	-	-
	PT04	Police Station	4.05	-0.06	-	-

Table 24: Impact of Sea level Rise and Rainfall Increase with Climate Change for the 5% AEP event

Catchment	ID	Description		1% AEP Impact (m)		
			5% AEP (m AHD71)	Rainfall Increase	Rainfall Increase, SLR +0.4 m	Rainfall Increase, SLR +0.9 m
Airport	A01	Golf Club	4.11	0.02	0.02	0.02
	A02	Airport	-	Newly Flooded	Newly Flooded	Newly Flooded
	A03	Lagoon Road	4.08	0.03	0.03	0.03
	A04	Judy's House	4.11	0.02	0.02	0.02
	A05	Airstrip	4.04	0.02	0.02	0.02
Kings Beach	KB01	Near Cappella Lodge	10.54	0.02	0.02	0.02
	KB02	Soldiers Creek US Lagoon Road	4.66	0.04	0.04	0.04
	KB03	Lagoon Rd	3.73	0.12	0.12	0.12
Pinetrees to Steven's Reserve	PT01	Pine trees	4.17	-0.11	-	-
	PT02	Lord Howe Island Bowling Club	4.17	-0.10	-	-
	PT03	Anglican Church	4.16	-	-	-
	PT04	Police Station	3.90	-0.01	-	-

8.7. Hydraulic and Hazard Categories

Managing the Floodplain: a guide to best practice in flood risk management in Australia (AIDR 2017) provides a revised flood hazard classification, relating combinations of flood depths and velocities to risks to vehicles, people and buildings. The classification is divided into six categories (Diagram 9):

- H1 - Generally safe for people, vehicles and buildings
- H2 - Unsafe for small vehicles
- H3 - Unsafe for vehicles, children and the elderly
- H4 - Unsafe for people and vehicles
- H5 - Unsafe for people and vehicles. All buildings vulnerable to structural damage. Some less robust building types vulnerable to failure
- H6 - Unsafe for people and vehicles. All buildings types considered vulnerable to failure.

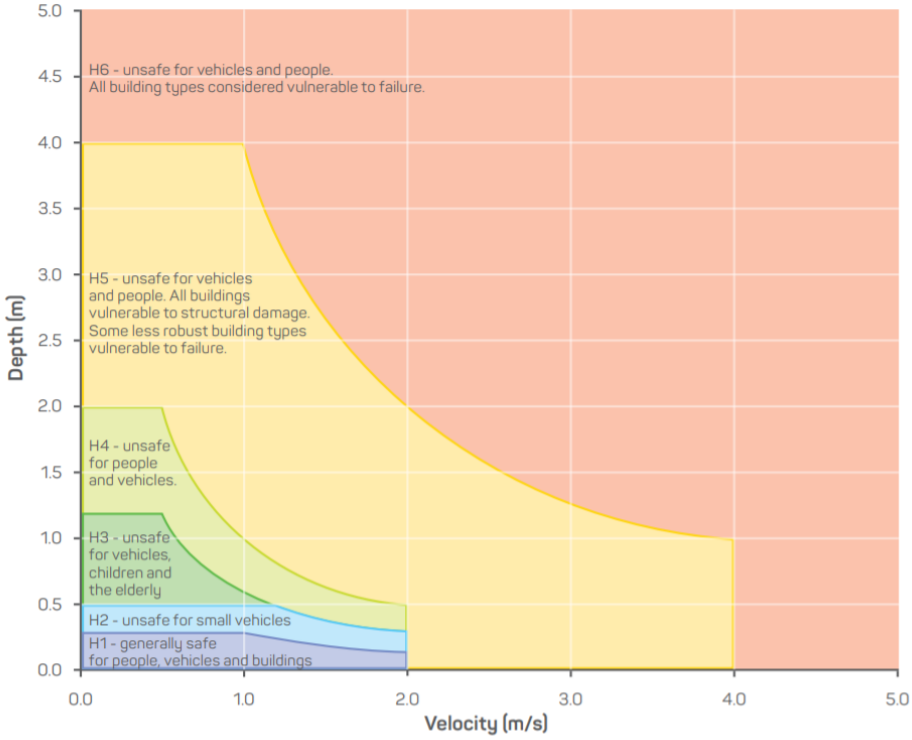

Diagram 9: Flood hazard vulnerability curves (AIDR 2017)

The *Floodplain Development Manual* (NSW Government, 2005) requires that other factors be considered in determining the 'true' hazard including: size of flood, effective warning time, flood readiness, rate of rise of floodwaters, depth and velocity of floodwaters, duration of flooding, evacuation problems, effective flood access, type of development within the floodplain, complexity of the stream network and the inter-relationship between flows. However, to assess the full flood hazard all adverse effects of flooding have to be considered. As well as considering the provisional (hydraulic) hazard it also incorporates threat to life, danger and difficulty in evacuating people and possessions and the potential for damage, social disruption and loss of production.

The conversion from 'provisional' hazard to 'true' hazard requires subjective decisions on how these aspects interact with the population at risk. To overcome this problem the practice has evolved to map provisional hazard and to separately identify evacuation risk over the full range of flood events. For this reason, a true hazard conversion has not been carried out.

Hazard classification was carried out on the 5% AEP, 1% AEP and PMF events adopting gridded depth and velocity results output from the TUFLOW 2D hydraulic model.

Figure 62 to Figure 70 present the provisional flood hazard classifications for the design events. Under this classification for a 1% AEP event, the majority of the floodplain is considered relative safe for vehicles and people. For all catchments, there is very little area subject to Hazard categories H5 or H6, meaning while areas of the floodplain for dangers to people and vehicles, the hazard it not sufficient to mean that well-constructed buildings are vulnerable.

Provisional flood hazard classifications for other events are also provided. In a probable maximum flood (PMF), a greater portion of the floodplain is classified as H5, however it does not intercept properties or businesses.

8.8. Hydraulic categories

Hydraulic categories describe the flood behaviour by categorising areas depending on their function during the flood event, specifically, whether they convey large quantities of water (floodway), store a significant volume of water (flood storage), or do not play a significant role in either storing or conveying water (flood fringe). As with categories of flood hazard, hydraulic categories play an important role in informing floodplain risk management in an area. Although the three categories of hydraulic function are described in the *Floodplain Development Manual* (NSW Government, 2005), their definitions are largely qualitative, and the manual does not prescribe a method to determine each area.

The manual gives an indication of criteria for the quantification of flood storage areas. The manual defines flood storage areas as areas outside of the floodway which if completely filled with solid material, would increase peak flood levels by ‘more than 0.1 metres and/or would cause the peak discharge anywhere downstream to increase by more than 10 per cent’.

A range of methods have been developed that aim to define these areas such as Howells et al. (Reference 14), encroachment and conveyance methods. The Hydraulic Categories have been defined for the catchments in Lord Howe based on an iterative application of Howells method.

The use of velocity and depth to delineate areas of different hydraulic category follows the approach proposed by Howells et al. in their 2004 paper. At each grid cell, the peak velocity (v), peak depth (d) and their product (v*d) is considered, and the cell is categorised based on the following criteria.

1. *If both v*d > 0.08 and v > 0.045, then ‘floodway’*
2. *If both v > 0.14 and d > 0.05, then ‘floodway’*
3. *If neither of the above apply and d > 0.08, then ‘flood storage’*
4. *Otherwise, ‘flood fringe’.*

8.9. Flood Planning Area

The flood planning level (FPL) is used to define land subject to flood related development controls and is generally adopted as the minimum level to which floor levels in the flood affected areas must be built. The FPL includes a freeboard above the design flood level. It is common practice to set minimum floor levels for residential buildings, garages, driveways and even commercial floors as this reduces the frequency and extent of flood damages. Freeboards provide reasonable certainty that the reduced level of risk exposure selected (by deciding upon a particular event to provide flood protection for) is actually provided.

The Flood Planning Area is defined as the 1% AEP event plus a freeboard. For Lord Howe Island the use of a 0.3 m freeboard is considered appropriate. Figure 83 to Figure 85 show the proposed Flood Planning Area.

8.10. Flood Planning Constraint Categories

AIDR National Manual provides guidance on the how to classify land within the floodplain based on its Flood Risk. The guidance takes into account the Hazard Categorisation and Hydraulic Categorisation of the Design Flood Event and a flood event larger than the Design Flood Event, the Flood Planning Area, the PMF extent. The Flood planning Constraint Categories are presented in Figure 100 to Figure 102.

9. EMERGENCY REPONSE

9.1. Flood Emergency Response Planning Classification of Communities

9.2. Communities

The Floodplain Development Manual (NSW State Government, 2005) requires flood studies to address the management of continuing flood risk to both existing and future development areas. As continuing flood risk varies across the floodplain so does the type and scale of emergency response problem and therefore the information necessary for effective Emergency Response Planning (ERP). Classification provides an indication of the vulnerability of the community in flood emergency response and identifies the type and scale of information needed by the State Emergency Services (SES) to assist in emergency response planning (ERP).

Criteria for determining flood ERP classifications and an indication of the emergency response required for these classifications are provided in the Floodplain Risk Management Guideline, 2007 (Flood Emergency Response Planning: Classification of Communities). Table 25 summarises the response required for areas of different classification. However, these may vary depending on local flood characteristics and resultant flood behaviour, i.e. in flash flooding or overland flood areas.

Table 25: Response Required for Different Flood ERP Classifications

Classification	Response Required		
	Resupply	**Rescue/Medivac**	**Evacuation**
High Flood Island	Yes	Possibly	Possibly
Low Flood Island	No	Yes	Yes
Area with Rising Road Access	No	Possibly	Yes
Area with Overland Escape Routes	No	Possibly	Yes
Low Trapped Perimeter	No	Yes	Yes
High Trapped Perimeter	Yes	Possibly	Possibly
Indirectly Affected Areas	Possibly	Possibly	Possibly

In undertaking this assessment for Lord Howe Island, all roads have been considered trafficable in a flood event, both paved and unsealed. The suitability for use of particularly unsealed roads should be reviewed with the SES, and Lord Howe Island. Figure 80 and Figure 82 presents the ERP classifications.

Most of the main population centres of the Pinetrees and Steven's Reserve are classified as Rising Road Access as the properties are inundated but flood free access roads provide a retreat to flood free land. Small parts of this catchment are classified as Low Flood Island as roads are cut prior to the inundation of the properties. The evacuation route along Lagoon road is also cut prior to the inundation of properties in Kings Beach, classifying properties up Smoking Tree Ridge Track as within a Low Flood Island.

9.3. Length of inundation

Time of inundation in a 1% AEP event shown on Figure 103 to Figure 105 for all grid cells within the model. The majority of areas are inundated for between 4 and 10 hrs. Some extremely low lying areas may experience inundation for longer. The time of inundation is measured as the duration the cell is inundated with a depth greater than 0.1 m.

Table 26 provides the levels of low points in key roads within the catchment and the flood levels at these locations for a range of events. These low points were derived from the ALS. The length of time in a 1% AEP event till a low point in the road is cut and how long it can be expected to be cut is presented in Table 27. The time the road is cut is determined as the time when the depth reaches 0.1 m.

Table 26: Peak Flood Levels at Road Low Points

Catchment	Road Name	Low point in road level (m AHD71)	Flood Level (m AHD71)			
			20% AEP	5% AEP	1% AEP	PMF
Airport	Lagoon Road at Airstrip	3.42	4.01	4.23	4.50	4.50
	Lagoon Rd Airstrip South	3.26	3.95	4.22	4.58	4.58
Kings Beach	Lagoon Rd	3.21	3.36	3.62	4.74	4.74
	Smoking Tree Ridge track	7.31	7.48	7.56	7.82	7.82
Pinetrees to Steven's Reserve	Rear St Police Station	3.69	3.76	3.84	4.03	5.57
	Middle Beach Rd	4.20	-	-	4.40	5.58
	Bowker Ave	4.12	-	-	4.43	5.58

Table 27: Time to cut and Time of inundation of Road Low Points

Catchment	Road Name	Low point in road level (m AHD71)	Hours before inundated in 1% AEP Event	Total time of inundation (hrs)*
Airport	Lagoon Road at Airstrip	3.42	0.59	9.42
	Lagoon Rd Airstrip South	3.26	1.06	8.95
Kings Beach	Lagoon Rd	7.31	1.24	5.47
	Smoking Tree Ridge track	3.69	1.59	5.70
Pinetrees to Steven's Reserve	Rear St Police Station	4.20	1.40	4.76
	Middle Beach Rd	4.12	3.38	2.84
	Bowker Ave	3.42	3.53	3.36

10. PRELIMINARY OPTIONS INDENTIFICATION

10.1. Overview

A desktop preliminary options identification was undertaken based on the recommended options in the *Lord Howe Island Floodplain Management Study, 1999* and flood modelling results from the current study. In undertaking the assessment consideration was given to the 2005 NSW Government Floodplain Development Manual (NSW State Gov, 2005) which separates risk management measures into three broad categories:

Flood modification measures modify the physical behaviour of a flood (depth, velocity and redirection of flow paths) and include flood mitigation dams, retarding basins and levees.

Property modification measures modify land use and development controls. This is generally accomplished through means such as flood proofing (house raising or sealing entrances), strategic planning (such as land use zoning), building regulations (such as flood-related development controls), or voluntary purchase.

Response modification measures modify the community's response to flood hazard by educating flood affected property owners about the nature of flooding so that they can make informed decisions. Examples of such measures include provision of flood warning and emergency services, improved information, awareness and education of the community and provision of flood insurance.

Table 28 provides a summary of the floodplain risk management measures that could be considered for the Lord Howe Island catchments.

Table 28: Floodplain Risk Management Measures

Flood Modification	Property Modification	Response Modification
Flood mitigation dams	Land zoning	Community awareness/preparedness
Retarding basins	Voluntary purchase	Flood warning
Bypass floodways	Building & development controls	Evacuation planning
Channel modifications	House raising	Evacuation access
Levees	Flood proofing	Flood plan / recovery plan
Temporary Flood Barriers	Flood access	Flood insurance

10.1.1. Relative Merits of Management Measures

Indicative costs associated with each option are included in Table 29 as per the 1999 Management Study. A detailed cost benefit assessment should be undertaken in the next plan.

The potential environmental or social impacts of any proposed flood mitigation measure must be considered in the assessment of any management measure and these cannot be evaluated using the classical benefit/cost approach.

Table 29 presents a summary of options presented in the 1999 Risk Management Study and their status. Additional options identified during the study have been added. Recommendations are also made for options that should be modelled in the next stage.

Table 29: Options Summary

ID	Measure	Description	Environmental/Social Implications	Indicative Cost ($) from 1999 Study	Priority	Current Status	Recommendation
THE KINGS BEACH CATCHMENT							
1	Development Controls.	Ensure by appropriate planning measures that flood problems do not increase in the future. Include flood information in the Building Code.	Long term benefits both environmentally and socially.	No direct costs.	High.	Controls are in place. The LHIB Development Application Statement of Environmental Effects captures information from the applicant about flooding and what measures will be undertaken to mitigate effects. During planning assessment project the LHIB Team Leader Projects and Compliance assesses application against Webb McKeown & Associates LHI Floodplain Management Study 1999 and makes recommendations to planners. Planners then include appropriate conditions in recommendation to LHIB.	Amend controls to refer to the current study and continue to assess developments in line with controls
2	Construct control works along man made drain.	Construction of such works over time would reduce erosion and stabilise the drain.	Reduction in sediment discharge to Lagoon.	$50,000 (could be staged). Ongoing $5000 p.a. maintenance.	Medium.	Planting and stabilisation work has been completed and receives ongoing maintenance.	Consider in the next phase as part of ongoing maintenance
3	Install depth indicators.	Depth indicators to be placed across floodplain to show depth over pavement.	Minor social benefit.	$1,000.00	Medium.	Completed - depth indicators have been installed.	Nil
4	Increase culvert capacity under Lagoon Road.	More culverts under the road would reduce flood peaks and allow faster drainage.	No adverse environ-mental effects. Slight improvement in access.	$20,000.00	Low.	Completed. Culverts have been included in the current study and culverts are maintained.	Consider in the next phase as part of ongoing maintenance
THE AIRPORT CATCHMENT							
5	Evacuation Plan for Ms Judy Wilson.	Provide specific provision in flood emergency plan for Ms Wilson to be evacuated before major flooding.	Major social benefit.	$1,000.00	High.	A depth marker has been installed near the mouth of Cobbys Creek to indicate when the flood level as at the bottom step of Mrs Wilson's house. This was to be used to trigger an evacuation however Mrs Wilson now lives on the mainland. The current tenants at the house are staff from a nearby lodge and they are able to self evacuate. The catchment has been planted with Sallywood swamp. The creek outlet is maintained and periodically opened to ocean.	Nil
6	Clear golf course tributary creek.	Creek adjoining Ms J Wilson's house and downstream to the road should be cleared of excess vegetation/debris (including removal of tin fence) and regularly maintained.	Will improve visual amenity and reduce flood damages.	Annual cost $2000.	High.	Completed. Golf course tributary creek is Cobbys Creek (as referenced above). Regular maintenance is carried out.	Consider in the next phase as part of ongoing maintenance
7	Maintain creek ocean outlet sand berms.	The two ocean outlets, at Cobbys corner and downstream of airport culvert, should be monitored and regularly lowered.	No adverse environmental or social effects.	$3000 p.a.	High.	Cobbys corner is maintained and regularly opened to ocean. The other outlet (downstream of airport culvert) no longer receives the inflow volume as it did at the time of previous reporting, due to changes to airport drainage. Sand berm is monitored but has not required maintenance.	Consider in the next phase as part of ongoing maintenance
8	Development Controls.	Ensure by appropriate planning measures that flood problems do not increase in the future. Include flood information in the Building Code.	Long term benefits both environmentally and socially.	No direct costs.	High.	As per PM01	Amend controls to refer to the current study and continue to assess developments in line with controls
9	Improve culvert capacity under airport access road.	Present culverts too small and blocked. Larger culverts will reduce flood levels and improve drainage.	Reduce period of inundation and dieback of grass.	$50,000.00	Medium.	Completed. Culverts have been included in the current study and culverts are maintained.	Nil – maintain culverts and remove blockages
10	Install depth indicators.	Depth indicators to be placed around airport road to show depth over pavement.	Minor social benefit.	$2,000.00	Medium.	Completed – Depth Indicators have been installed (although one indicator has been damaged then removed and not replaced).	Replace depth indicator at Blinky Corner.

11	Resolve drainage problems on Mr S Fenton's land.	Can be rectified by land purchase or land swap. Some improvement may result from drainage works associated with seawall if works extended.	Social benefit.	Nil to ?	Low.	Resolved by drainage works undertaken in conjunction with 2015 airport runway reseal.	Nil
12	Provide mobile pumpout system.	A mobile pump out system could be used to drain low lying areas around the airport after floods.	Minor social benefit.	$20,000.00	Very Low.	In place. Lord Howe Island fire truck (Rural Fire Service) is available as a mobile pump unit. LHIB also has a portable diesel pump.	Consider in the next phase as part of ongoing use.
13	House relocation of Ms Wilson's house.	Even with creek clearing will still be regularly flooded. Preferable to relocate house in longer term.	Will improve environmental amenity. Gives clear social benefit.	$40,000 to $100,000.	Long term objective	No action. Not under consideration.	Nil
14	Duplicating or otherwise amplifying the capacity of the culvert under the runway	This would be an extremely expensive and disruptive measure in terms of its likely effects on airport operation.				The side drains parallel to runway were enlarged as part of 2015 airport runway reseal works, given extra holding capacity. No culvert installed under runway. Extra culverts installed under Lagoon Road (details have been supplied for this study).	Recommend modelling in the next phase to confirm if effective.
PINETREES TO STEVENS RESERVE CATCHMENT							
15	Maintenance of high infiltration areas.	Since there are no drainage outlets it is vital that areas of high infiltration and any sinkholes be retained and maintained.	Social benefit.	Nil.	High.	High infiltration areas have been and continue to be retained. The sports oval has been lowered as a detention basin.	Consider in the next phase as part of ongoing maintenance
16	Maintenance of existing road levels.	It is vital that the existing levels of the east-west roads crossing the floodplain be maintained or flood flows could redistribute benefiting some and adversely affecting others.	No adverse environmental or social effects.	Nil.	High.	None have been raised except for possible small section adjacent to churches on Middle Beach Road where road has been resealed. New height is no more than 50 mm above previous.	Consider in the next phase as part of ongoing maintenance
17	Provision of flood proofing advice (applies to all catchments).	Advice to be given regularly to the floodplain dwellers to ensure that they take appropriate actions if flooding threatens.	Social benefit.	Minor.	High.	No direct communication from LHIB currently. Lord Howe Island SES maintains community contact.	Although the community generally has a high level of awareness of weather events (from collective experience), recommended that clear advice is provided and that this is considered in the next phase.
18	Development Controls.	Ensure by appropriate planning measures that flood problems do not increase in the future. Include flood information in the Building Code.	Long term benefits both environmentally and socially.	No direct costs.	High.	As per Option 1	Amend controls to refer to the current study and continue to assess developments in line with controls
19	Pinetrees to Stevens Reserve detention basin.	A detention basin located in the paddock adjoining Pinetrees would reduce flood inundation within Pinetrees and give some minor reductions in levels and damages elsewhere.	Social benefit. No adverse environmental effects.	$60,000 approx. (Cost will depend on design and construction. method).	Medium.	Comparison of the 1999 Flood Study report figures with aerial imagery suggests the existing levee drawing from 1999 was not completely accurate. The levee visible in the SixMaps imagery was in place in 1999 and remains unchanged to the present. The level of oval has been lowered to provide detention however there is no documentation on these works.	Recommend modelling impact of oval in the next phase.
20	Provision of flood warnings and flood emergency plan (applies to all catchments) .	Board to investigate whether it is possible to collect and disseminate warnings based on observed rainfalls and use these to trigger a flood emergency plan.	Social benefit.	$15,000.00	Medium.	No action on warnings based on observed rainfalls used for trigger Lord Howe Island Local emergency Management Plan 2017 has a consequence management guide for storm/flash flood.	Maintain consideration
21	Restriction on flow downstream of Middle Beach Rd. Seems to be caused by fill	Option to remove fill to be modelled in the next study	Potential reduction in damages and flood levels	Low	High	Mentioned in 1998 flood study. The ALS contains some fill that is restricting flow which can be seen in results. The area is to the SW of a Board residence known as the Doll's House.	Recommend modelling the removal of fill in the next phase.

11. CONCLUSIONS

A detailed hydraulic model (TUFLOW) has been developed to quantify the flood behaviour of the

This model has been used to reproduce the historical flood behaviour from events in 1996 and 1998. The TUFLOW model has been used to define flood behaviour for a range of design events (20%, 5%, 1% and Probable Maximum Flood).

Community consultation and hazard classification were undertaken. The model developed for the current study is suitable for further floodplain planning and use in setting planning levels within the study area.

12. ACKNOWLEDGEMENTS

This study was carried out by WMAwater and funded by the Lord Howe Island Board and the Office of Environment and Heritage.

The assistance of the following in providing data and guidance to the study is gratefully acknowledged:

- Lord Howe Island Board,
- Residents of Lord Howe Island within the study area,
- Bureau of Meteorology,
- Office of Environment and Heritage.

13. REFERENCES

1. Lord Howe Island Board
Lord Howe Island Flood Study
Webb, McKeown & Associates Pty Ltd, Australia, 1998.

2. Lord Howe Island Board
Lord Howe Island Floodplain Management Study
Webb, McKeown & Associates Pty Ltd, Australia, 1999.

3. Lord Howe Island Board
Lord Howe Island Coastal Study
Royal Haskoning Australia Pty Ltd, 2014

4. Boyd M, Rigby T, VanDrie R, and Schymitzek I
WBNM User Guide
2017

5. WBM BMT
TUFLOW Classic/HPC User Manual
2018

6. Ball J, Babister M, Nathan R, Weeks W, Weinmann E, Retallick M, Testoni I, (Editors)
Australian Rainfall and Runoff: A Guide to Flood Estimation
Commonwealth of Australia, Australia, 2019

7. Babister, M., Trim, A., Testoni, I. & Retallick, M
The Australian Rainfall & Runoff Datahub
37th Hydrology and Water Resources Symposium Queenstown NZ, 2016

8. Babister, M & Barton C (Editors)
Australian Rainfall and Runoff Revision Project 15: Two Dimensional Modelling in Urban and Rural Floodplains- Stage 1&2 Report P15/S1/009
Engineers Australia, 2012

9. NSW Government
Floodplain Development Manual: The management of flood liable land
April 2005

10. Commonwealth of Australia
Australian Disaster Resilience Handbook 7 Managing the Floodplain: A Guide to Best Practice in Flood Risk Management in Australia
AIDR 2017

11. BoM 2003a
Guidebook to the Estimation of Probable Maximum Precipitation: Generalised Tropical Storm Method, Bureau of Meteorology November 2003

12. BoM 2003b
The Estimation of Probable Maximum Precipitation in Australia: Generalised Short-Duration Method, Bureau of Meteorology June 2003

13. State of NSW and Office of Environment and Heritage (NSW OEH)
Floodplain Risk Management Guide Incorporating 2016 Australian Rainfall and Runoff in studies, Office of Environment and Heritage, Sydney, January 2019

14. Howells L, McLuckie D., Collings G., Lawson N.
Defining the Floodway – Can One Size Fit All?
2004

Figures

Appendix A

APPENDIX A. GLOSSARY

Taken from the Floodplain Development Manual (April 2005 edition)

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

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

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

discharge The rate of flow of water measured in terms of volume per unit time, for example, cubic metres per second (m^3/s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving for example, metres per second (m/s).

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

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

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

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

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

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

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

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

flood liable land Is synonymous with flood prone land (i.e. land susceptible to flooding by the probable maximum flood (PMF) event). Note that the term flood liable land covers the whole of the floodplain, not just that part below the flood planning level (see flood planning area).

flood mitigation standard	The average recurrence interval of the flood, selected as part of the floodplain risk management process that forms the basis for physical works to modify the impacts of flooding.
floodplain	Area of land which is subject to inundation by floods up to and including the probable maximum flood event, that is, flood prone land.
floodplain risk management options	The measures that might be feasible for the management of a particular area of the floodplain. Preparation of a floodplain risk management plan requires a detailed evaluation of floodplain risk management options.
floodplain risk management plan	A management plan developed in accordance with the principles and guidelines in this manual. Usually includes both written and diagrammetic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.
flood plan (local)	A sub-plan of a disaster plan that deals specifically with flooding. They can exist at State, Division and local levels. Local flood plans are prepared under the leadership of the State Emergency Service.
flood planning area	The area of land below the flood planning level and thus subject to flood related development controls. The concept of flood planning area generally supersedes the Aflood liable land@ concept in the 1986 Manual.
Flood Planning Levels (FPLs)	FPL=s are the combinations of flood levels (derived from significant historical flood events or floods of specific AEPs) and freeboards selected for floodplain risk management purposes, as determined in management studies and incorporated in management plans. FPLs supersede the Astandard flood event@ in the 1986 manual.
flood proofing	A combination of measures incorporated in the design, construction and alteration of individual buildings or structures subject to flooding, to reduce or eliminate flood damages.
flood prone land	Is land susceptible to flooding by the Probable Maximum Flood (PMF) event. Flood prone land is synonymous with flood liable land.
flood readiness	Flood readiness is an ability to react within the effective warning time.
flood risk	Potential danger to personal safety and potential damage to property resulting from flooding. The degree of risk varies with circumstances across the full range of floods. Flood risk in this manual is divided into 3 types, existing, future and continuing risks. They are described below. **existing flood risk:** the risk a community is exposed to as a result of its location on the floodplain. **future flood risk:** the risk a community may be exposed to as a result of new development on the floodplain. **continuing flood risk:** the risk a community is exposed to after floodplain risk management measures have been implemented. For a town protected by levees, the continuing flood risk is the consequences of the levees being overtopped. For an area without any floodplain risk management measures, the continuing flood risk is simply the existence of its flood exposure.
flood storage areas	

	Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. The extent and behaviour of flood storage areas may change with flood severity, and loss of flood storage can increase the severity of flood impacts by reducing natural flood attenuation. Hence, it is necessary to investigate a range of flood sizes before defining flood storage areas.
floodway areas	Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flows, or a significant increase in flood levels.
freeboard	Freeboard provides reasonable certainty that the risk exposure selected in deciding on a particular flood chosen as the basis for the FPL is actually provided. It is a factor of safety typically used in relation to the setting of floor levels, levee crest levels, etc. Freeboard is included in the flood planning level.
habitable room	**in a residential situation:** a living or working area, such as a lounge room, dining room, rumpus room, kitchen, bedroom or workroom. **in an industrial or commercial situation:** an area used for offices or to store valuable possessions susceptible to flood damage in the event of a flood.
hazard	A source of potential harm or a situation with a potential to cause loss. In relation to this manual the hazard is flooding which has the potential to cause damage to the community. Definitions of high and low hazard categories are provided in the Manual.
hydraulics	Term given to the study of water flow in waterways; in particular, the evaluation of flow parameters such as water level and velocity.
hydrograph	A graph which shows how the discharge or stage/flood level at any particular location varies with time during a flood.
hydrology	Term given to the study of the rainfall and runoff process; in particular, the evaluation of peak flows, flow volumes and the derivation of hydrographs for a range of floods.
local overland flooding	Inundation by local runoff rather than overbank discharge from a stream, river, estuary, lake or dam.
local drainage	Are smaller scale problems in urban areas. They are outside the definition of major drainage in this glossary.
mainstream flooding	Inundation of normally dry land occurring when water overflows the natural or artificial banks of a stream, river, estuary, lake or dam.
major drainage	Councils have discretion in determining whether urban drainage problems are associated with major or local drainage. For the purpose of this manual major drainage involves: • the floodplains of original watercourses (which may now be piped, channelised or diverted), or sloping areas where overland flows develop along alternative paths once system capacity is exceeded; and/or

- water depths generally in excess of 0.3 m (in the major system design storm as defined in the current version of Australian Rainfall and Runoff). These conditions may result in danger to personal safety and property damage to both premises and vehicles; and/or

- major overland flow paths through developed areas outside of defined drainage reserves; and/or

- the potential to affect a number of buildings along the major flow path.

mathematical/computer models — The mathematical representation of the physical processes involved in runoff generation and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.

merit approach — The merit approach weighs social, economic, ecological and cultural impacts of land use options for different flood prone areas together with flood damage, hazard and behaviour implications, and environmental protection and well being of the State=s rivers and floodplains.

The merit approach operates at two levels. At the strategic level it allows for the consideration of social, economic, ecological, cultural and flooding issues to determine strategies for the management of future flood risk which are formulated into Council plans, policy and EPIs. At a site specific level, it involves consideration of the best way of conditioning development allowable under the floodplain risk management plan, local floodplain risk management policy and EPIs.

minor, moderate and major flooding — Both the State Emergency Service and the Bureau of Meteorology use the following definitions in flood warnings to give a general indication of the types of problems expected with a flood:

minor flooding: causes inconvenience such as closing of minor roads and the submergence of low level bridges. The lower limit of this class of flooding on the reference gauge is the initial flood level at which landholders and townspeople begin to be flooded.

moderate flooding: low-lying areas are inundated requiring removal of stock and/or evacuation of some houses. Main traffic routes may be covered.

major flooding: appreciable urban areas are flooded and/or extensive rural areas are flooded. Properties, villages and towns can be isolated.

modification measures — Measures that modify either the flood, the property or the response to flooding. Examples are indicated in Table 2.1 with further discussion in the Manual.

peak discharge — The maximum discharge occurring during a flood event.

Probable Maximum Flood (PMF) — The PMF is the largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation, and where applicable, snow melt, coupled with the worst flood producing catchment conditions. Generally, it is not physically or economically possible to provide complete protection against this event. The PMF defines the extent of flood prone land, that is, the floodplain. The extent, nature and potential consequences of flooding associated with a range of events rarer than the flood used for designing mitigation works and controlling development, up to and including the PMF event should be addressed in a floodplain risk management study.

Probable Maximum Precipitation (PMP)	The PMP is the greatest depth of precipitation for a given duration meteorologically possible over a given size storm area at a particular location at a particular time of the year, with no allowance made for long-term climatic trends (World Meteorological Organisation, 1986). It is the primary input to PMF estimation.
probability	A statistical measure of the expected chance of flooding (see AEP).
risk	Chance of something happening that will have an impact. It is measured in terms of consequences and likelihood. In the context of the manual it is the likelihood of consequences arising from the interaction of floods, communities and the environment.
runoff	The amount of rainfall which actually ends up as streamflow, also known as rainfall excess.
stage	Equivalent to Awater level@. Both are measured with reference to a specified datum.
stage hydrograph	A graph that shows how the water level at a particular location changes with time during a flood. It must be referenced to a particular datum.
survey plan	A plan prepared by a registered surveyor.
water surface profile	A graph showing the flood stage at any given location along a watercourse at a particular time.
wind fetch	The horizontal distance in the direction of wind over which wind waves are generated.

Appendix B

APPENDIX B. PHOTOGRAPHS OF FLOODING

KINGS BEACH CATCHMENT

Photograph 1: Looking down the man made drain towards the ocean

Photograph 2: Looking upstream along the man made drain

Photograph 3: Looking downstream across the floodplain (June 1996)

Photograph 4: Looking down the man made drain adjoining Photograph 3

AIRPORT CATCHMENT

Photograph 5: Golf course upstream of Ms Wilson's house after June 1996 storm

Photograph 6: Golf course debris marks after June 1996 storm

Photograph 7: Southern sub-catchment looking towards Blinky Beach June 1996 storm

Photograph 8: Northern sub-catchment looking towards Blinky Beach June 1996 storm

Photograph 9: Northern sub-catchment looking along runway to Blinky Beach June 1996

Photograph 10: Ponded area in front of Mr S Fenton June 1996

PINETREES TO STEVEN'S RESERVE CATCHMENT

Photograph 11: Tennis Court under water at Pinetrees - June 1996 -

Photograph 12: View of the Bowling Club from the "Bowling Green" - June 1996

Photograph 13: From Bowling Club looking towards Transit Hill - June 1996

Photograph 14: From Bowling Club looking towards the Oval - June 1996

Photograph 15: Flooding of the churches on Middle Beach Road - June 1996

Photograph 16: Paddock north of TC Douglas Drive looking to Stevens Reserve - June 1996

STORM DAMAGE

Photograph 17: Road Damage at Cobbys Corner caused by flood overflow

Photograph 18: Silt deposition on Golf Course upstream of Ms Wilson's house

Photograph 19: Major land slip closing road south of Kings Beach

Photograph 20 : Typical land slip initiation near Clear Place

Appendix C

APPENDIX C. UNRESTRICTED ANNUAL SERIES 1947 - 1998

Year	Duration										
	6 m	12 m	18 m	30 m	1 hr	2 hr	3 hr	6 hr	12 hr	24 hr	48 hr
1947	11.5	15.1	21.4	32.0	54.9	58.3	64.2	79.4	96.5	134.9	158.4
1948	12.5	18.4	27.6	46.0	48.2	52.7	57.1	75.1	103.3	162.5	186.1
1949	12.6	15.8	22.3	37.1	74.3	93.5	111.3	176.0	198.9	201.2	201.2
1950	12.9	14.6	15.9	20.7	31.6	54.1	67.1	83.3	85.1	85.3	89.7
1951	5.1	9.7	10.7	17.5	28.1	46.1	58.1	58.7	58.7	62.3	73.1
1952	12.6	18.5	21.4	25.4	40.5	66.3	73.8	81.4	81.5	81.7	96.0
1953	8.1	13.2	15.0	22.9	36.5	42.9	48.6	56.5	79.7	105.4	106.4
1954	11.3	18.4	23.4	26.8	50.7	69.0	83.4	88.0	88.1	113.5	122.0
1955	7.9	11.2	14.5	21.2	35.9	67.5	96.6	163.9	177.2	186.6	190.2
1956	9.7	15.1	21.5	24.0	33.9	41.6	52.5	52.7	83.8	89.2	95.5
1957	12.4	15.2	17.9	25.8	30.5	43.4	55.6	58.3	73.2	146.3	153.2
1958	7.1	10.5	13.0	15.6	22.3	29.3	35.8	52.9	63.7	68.6	82.8
1959	8.9	14.6	19.0	31.7	43.7	45.2	45.3	46.6	51.1	59.6	82.8
1960	16.3	25.5	27.5	35.5	37.8	48.0	57.4	79.5	93.1	100.6	103.6
1961	7.5	12.8	14.4	16.4	27.5	39.6	44.9	73.9	108.9	120.0	122.7
1962	9.1	18.2	27.2	35.0	36.5	50.6	51.0	65.3	79.7	92.1	115.6
1963	12.3	14.9	17.4	20.9	29.4	37.0	49.0	64.6	106.1	111.5	120.0
1964	11.5	15.9	22.4	25.2	31.2	40.9	49.9	70.0	70.1	89.1	107.5
1965	6.7	9.7	12.6	17.8	21.5	28.4	31.0	32.9	33.1	37.3	42.4
1966	7.9	10.1	12.8	18.6	24.3	37.2	43.9	56.5	70.6	92.2	92.2
1967	6.8	13.3	19.2	27.5	42.8	53.9	55.9	59.3	67.3	93.7	101.0
1968	13.2	21.1	24.4	31.6	37.4	38.5	39.8	58.9	61.2	75.6	93.9
1969	13.7	18.9	20.4	26.9	45.0	63.0	70.3	89.1	91.3	100.6	103.4
1970	9.2	16.4	21.7	30.4	46.4	55.1	74.6	77.7	85.3	93.0	93.0
1971	7.9	13.7	16.2	24.4	34.0	41.4	41.4	46.1	59.7	73.8	86.6
1972	6.7	11.6	14.6	21.0	25.3	46.7	51.5	77.9	89.8	92.4	156.8
1973	29.2	33.3	36.7	46.2	59.0	70.8	72.5	91.5	99.5	112.0	115.8
1974	12.0	18.9	25.7	36.3	54.8	65.6	66.7	69.6	71.7	76.1	85.4
1975	14.3	18.6	22.5	35.2	46.9	66.9	71.3	95.8	99.4	102.5	146.9
1976	12.1	16.7	22.2	31.7	55.6	60.9	74.5	93.1	94.0	123.9	141.0
1977	10.5	14.5	18.2	21.3	27.8	31.6	33.1	51.1	54.1	55.0	67.3
1978	10.7	18.1	20.1	27.0	35.5	44.5	50.2	67.6	77.6	88.7	104.7
1979	19.7	28.6	37.2	39.6	50.3	61.8	67.3	79.5	97.0	152.9	167.0
1980	14.6	19.4	22.0	38.3	59.6	75.4	88.0	97.2	114.5	121.3	125.6
1981	17.6	21.9	24.2	30.4	36.0	55.4	64.6	85.2	134.9	163.3	174.9
1982	16.3	25.2	32.9	40.6	42.7	52.7	59.8	60.0	67.9	114.1	124.5
1983	14.9	24.1	25.0	26.7	32.5	41.0	43.2	47.0	59.4	68.7	82.0
1984	17.0	21.9	27.5	31.5	43.6	52.0	53.1	64.3	64.7	64.7	85.4
1985	10.4	13.0	16.6	20.7	31.9	49.6	54.3	59.0	59.0	59.4	63.7
1986	15.1	19.3	22.1	24.2	31.7	46.3	57.4	71.6	79.0	80.3	112.1

1987	13.8	16.3	20.6	24.7	29.9	37.0	44.6	56.1	83.8	91.5	94.8
1988	16.3	25.6	28.8	32.8	44.7	50.3	60.9	92.3	136.7	152.7	152.8
1989	12.5	19.1	23.7	24.8	29.5	34.0	43.5	73.9	90.3	93.8	100.3
1990	7.8	11.8	13.8	19.1	33.2	42.1	50.6	65.4	73.2	85.2	92.4
1991	17.3	23.0	32.5	36.8	48.2	55.7	61.4	61.8	62.0	107.9	177.4
1992	10.0	18.2	19.8	21.4	31.1	34.2	52.8	70.2	100.7	118.4	121.2
1993	12.2	17.1	20.2	21.8	26.0	31.4	37.3	57.2	78.9	81.4	84.0
1994	11.6	19.7	27.5	41.3	63.6	77.1	78.9	80.8	91.6	98.0	112.3
1995	18.3	25.3	26.1	28.3	37.9	64.4	69.0	84.9	123.1	178.3	185.0
1996	24.9	37.2	50.1	65.1	104.3	156.9	226.0	72.5	434.9	449.4	450.7
1997	n/a	n/a	n/a	35.0	45.5	71.5	72.5	72.5	72.5	72.5	n/a
1998	n/a	n/a	n/a	47.6	79.3	121.4	193.9	323.9	373.6	391.2	n/a

Appendix D

APPENDIX D. HYDRAULIC STRUCTURES

Table D 1: Culvert Structures included in models

ID	Culvert Details	Length (m)	Latitude	Longitude
L17_Culv01	600 RCP	6.00	-31.554	159.078
L17_Culv02	650 x 380 RCBC	5.80	-31.554	159.078
NLN_Culv01	450 RCP	7.00	-31.553	159.077
L14_Culv01	1510 x 930 RCBC	5.27	-31.552	159.077
L14_Culv02	1510 x 860 RCBC	4.50	-31.552	159.077
L15_Culv01	600 RCP	6.00	-31.552	159.077
L9_Culv01	1840 x 940 RCBC	11.70*	-31.544	159.078
L8_Culv1	1220 x 920 RCBC	16.72*	-31.543	159.078
L8_Culv02	910 x 285 RCBC	13.15*	-31.543	159.079
L7_Culv01	900 x 320 RCBC	10.61*	-31.541	159.081
L6_Culv01	900 x 280 RCBC	11.73*	-31.539	159.08
L16_Culv02	900 x 280 RCBC	12.20*	-31.539	159.08
L5_Culv01	900 RCP	101.19*	-31.538	159.075
L4_Culv01	900 x 330 RCBC	11.12*	-31.537	159.075
CL15_C01	455 RCP	46.48*	-31.536	159.071
CL15_Culv02	455 RCP	9.45*	-31.536	159.072
CL16_C01	380 RCP	9.70*	-31.536	159.072
CL16_C02	380 RCP	29.85*	-31.536	159.072
CL13_C01	455 RCP	9.21*	-31.536	159.071
CL13_C02	2 x 900 RCP	5.63*	-31.536	159.071
CL13_C03	2 x 900 RCP	50.18*	-31.536	159.071
CL13_C04	2 x 900 RCP	9.59*	-31.536	159.071
L1_Culv02	450 RCP	13.46*	-31.525	159.061
L1_Culv03	400 RCP	19.06*	-31.525	159.061
L1_Culv01	450 RCP	9.72*	-31.524	159.061

*please note culvert lengths are estimated based on aerial imagery

Table D 2: Pit Structures included in models

ID	Details*	Latitude	Longitude
CL15_Pit03	2400 Side Entry Pit	-31.536	159.071
CL15_Pit02	600 x 600 Grated Inlet Pit	-31.536	159.072
CL15_Pit01	600 x 600 Grated Inlet Pit	-31.536	159.072
CL16_Pit01	600 x 600 Grated Inlet Pit	-31.536	159.072
CL16_Pit02	600 x 600 Grated Inlet Pit	-31.536	159.072
CL13_Pit01	3600 x 3400 Grated Inlet Pit	-31.536	159.071
CL13_Pit02	2000 x 1800 Grated Inlet Pit	-31.536	159.071
L1_Pit03	2400 Side Entry Pit	-31.525	159.061
L1_Pit01	2400 Side Entry Pit	-31.524	159.061
L1_Pit02	2400 Side Entry Pit	-31.524	159.061

*please note all pit structures sizing and types are based on provided site photographs and aerial imagery

Appendix E

APPENDIX E. HISTORIC REPORTING LOCATIONS

Table 30: Locations Identified in the Lord Howe Island Flood Study (1998)

ID	Property name referenced in report	Deposited Plan Number	Lot Number and Road Name	Latitude, Longitude
1	Stan Fenton's house	DP757515	Lot 114, Lagoon Road	-31.53785 159.079282
2	Esven Fenton's property	DP48320	Lot 300, Mulley Drive	-31.549763 159.078301
3	May Shick's house	DP757515	Lot 313, Lagoon Road	-31.535614 159.0713
4	Mr Ray Shick's house	Unknown	Unknown – Kings Beach Catchment	Unknown
5	Judy Wilson	DP757515	Lot 191, Lagoon Road	-31.544634 159.077418
6	J. Lonergan senior's house	DP1127467	Lot 141, Middle Beach Road	-31.527539 159.066864
7	Marj Rayward	DP757515	Lot 18, TC Douglass Drive	-31.526722 159.066727
8	Patricia Dignam	DP757515	Lot 31 Lagoon Road	-31.525979 159.064448
9	Seventh Day Adventist Church	DP822355	Lot 322 Middle Beach Road	-31.527212 159.068086
10	Catholic Church	DP822355	Lot 323 Middle Beach Road	-31.527847 159.068224
11	Anglican Church	DP822355	Lot 324 Middle Beach Road	-31.52777 159.067735
12	Pinetrees	DP48213	Lot 236, Lagoon Road	-31.532356 159.069879
13	Woodhen breeding building Stevens Reserve (Ian Hutton supplied) **Now removed.**	DP757515	Lot 29	-31.524818 159.064001
14	LHIB	DP757515	Lot 37, Bowker Avenue	-31.529672 159.068766
15	Bowling club	DP757515	Lot 39, Lagoon Road	-31.53016, 159.069491
16	Golf course	DP757515	Lot 120, Lagoon Road	-31.544991 159.07919
17	Police station	DP757515	Lot 10, TC Douglass Drive	-31.526823 159.066263
18	Capella	DP1216287	Lot 41, Lagoon Road	-31.550071 159.075976
19	Airport/airstrip	DP757515	Lot 180, Lagoon Road	-31.540685 159.078136
20	J. Lonergan Junior's house	Unknown	Lot 141, Middle Beach Road	-31.527539 159.066864