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Spring Street Drain, Muddy Creek and Scarborough Ponds Catchments Flood Study Review

Final Report

February 2017

Bayside Council

Spring Street Drain, Muddy Creek and Scarborough Ponds Catchments Flood Study Review

Prepared for:	Rockdale City Council
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Executive Summary

Introduction

The Spring Street Drain, Muddy Creek and Scarborough Ponds Catchments Flood Study Review has been prepared for Rockdale City Council (Council) to define the existing flood behaviour in the catchment and establish the basis for subsequent floodplain management activities.

The primary objective of the Flood Study is to define the flood behaviour within the study area through the establishment of appropriate numerical models. The study has produced information on flood flows, velocities, levels and extents for a range of flood event magnitudes under existing catchment and floodplain conditions. Specifically, the study incorporates:

- Compilation and review of existing information pertinent to the study;
- Development and calibration of appropriate hydrologic and hydraulic models;
- Determination of design flood conditions for a range of design event including the 50% AEP, 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 0.5% AEP and PMF event; and
- Presentation of study methodology, results and findings in a comprehensive report incorporating appropriate flood mapping.

Catchment Description

The Spring Street Drain, Muddy Creek and Scarborough Ponds catchments are located within the Rockdale City Council LGA. The study area occupies an area of approximately 13.1 km^2 that is drained via the existing stormwater drainage system, with the Spring Street Drain and Muddy Creek catchments draining to Cooks River and the Scarborough Ponds catchment draining to Botany Bay.

The topography of the study area is relatively flat, particularly east of the Illawarra Railway. To the west of the Illawarra railway, the topography slopes gradually, with a peak elevation of 68.5 m AHD to the south west of the Muddy Creek catchment. The upper reaches of the Muddy Creek catchment generally slope in a south-easterly direction with the lower reaches draining north east towards Cooks River. The Spring Street Drain has a peak elevation of 55.5m AHD, with the catchment generally draining eastwards. The Scarborough Ponds catchment has a peak elevation of 31.5m AHD. The catchment generally drains towards the Scarborough Ponds with the ponds draining southwards to an artificial outlet to Botany Bay.

The catchment is a highly modified landscape, comprising medium to high-density residential and commercial developments. It also includes major infrastructure assets including the Princes Highway, Illawarra Railway and the NGRS sewer line. These infrastructure assets, where raised above the natural ground level, restrict surface flows from west to east.

Historical Flooding

There is limited surveyed data of historic flood levels available for this study area. Model calibration and validation primarily relied upon anecdotal reports of flooding from the community, Council records, Sydney Water records and photographs of flood behaviour. Photographs cannot be assumed to record the peak flood behaviour, however, they are important for identifying flooding hotspots.

Where sufficient anecdotal information was available on the historical depth of flooding, Council undertook field surveys to measure the water level in m AHD. This information has been used as part of the hydraulic model calibration.

Model Development

Development of hydrologic and hydraulic models has been undertaken to simulate flood conditions in the catchments. The hydrological model developed using XP-RAFTS software provides for simulation of the rainfall-runoff process using the catchment characteristics of the study catchments and historical and design rainfall data. The hydraulic model, simulating flood depths, extents and velocities utilises the TUFLOW two-dimensional (2D) software developed by BMT WBM. The 2D modelling approach is suited to model the complex interaction between channels and floodplains and converging and diverging of flows through structures and urban environments.

The floodplain topography is defined using a digital elevation model (DEM) derived from topographic, hydrographic and topographic survey data provided by Council.

Model Calibration and Validation

The selection of suitable historical events for calibration of computer models is largely dependent on available historical flood information. Ideally the calibration and validation process should cover a range of flood magnitudes to demonstrate the suitability of a model for the range of design event magnitudes to be considered.

Through consultation with Council a set of flood events were identified as being suitable for use in the model calibration and validation process. These are events of a reasonable flood magnitude, for which there are observed flood data available for comparison with the model performance. The principal event selected for model calibration is the April 1998 event, as this is the flood event with the most intense rainfall of recent years. There is also a wealth of observed flood data available.

The February 1993 and October 2014 flood events have been selected for model validation. The October 2014 event was almost as intense as the April 1998 storm, but the April 1998 event had a greater total rainfall. It is therefore the largest recent flood event in the lower-lying areas of the catchment such as the Scarborough Ponds. The February 1993 event was not as significant as the other two, but still has some useful flood data available for comparison.

Design Event Modelling and Output

The developed models have been applied to derive design flood conditions within the study catchments. A range of storm durations using standard AR&R (2001) temporal patterns, were modelled in order to identify the critical storm duration for design event flooding in the catchment.

A range of design flood conditions were modelled. The simulated design events included the 50% AEP, 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 0.5% AEP and PMF event. The model results for the design events considered have been presented in a detailed flood mapping series for the catchment (see Mapping Compendium). The flood data presented includes design flood inundation, peak flood water levels and depths and peak flood velocities.

Hydraulic categories (floodway, flood fringe and flood storage) and provisional flood hazard categories have been mapped for flood affected areas within the catchment.

Sensitivity Testing

A number of sensitivity tests have been undertaken to identify the impacts of the adopted model conditions on the design flood levels. Sensitivity tests included:

- Structure and stormwater pipe blockages;
- Changes in the adopted roughness parameters;
- Variation of the adopted rainfall losses;
- Variation of the adopted downstream boundary condition; and
- Various factors that influence the peak flood level in the Scarborough Ponds.

Climate Change

The NSW Sea Level Rise Policy Statement (DECCW, 2009) provided projected increases in mean sea level for NSW of 0.4m and 0.9m, by the years 2050 and 2100 respectively. These increases are no longer prescribed by the state government but have been adopted for the purpose of this study in the absence of other suitable recommendations. Therefore, design ocean boundaries have been raised by 0.4m and 0.9m to assess the potential impact of sea level rise on flood behaviour in the study catchment.

Current research predicts that a likely outcome of future climatic change will be an increase in flood producing rainfall intensities. Climate Change in New South Wales (CSIRO, 2004) provides projected regional changes in rainfall intensities for each season and annually for the years 2030 and 2070. The Muddy Creek catchment falls into the South-East region of NSW where compared to other regions in the state, projected increases are not as significant. It has been projected that 2.5% AEP 24 hour duration annual rainfall depths will increase by more than 5% by the year 2030 and 2070 in the study catchment. The 2.5% AEP 72 hour duration annual rainfall depth projections are increases of 10% for the year 2030 and 3% for the year 2070.

The NSW Government has also released a guideline (DECCW, 2007) for Practical Consideration of Climate Change in the floodplain management process that advocates consideration of increased design rainfall intensities of up to 30%.

In line with this guidance note, additional tests incorporating a 10% and a 30% increase to design rainfall have been undertaken. The design rainfall for the 0.5% AEP is around 10% higher than those of the 1% AEP, so comparison of these two events provides an appropriate assessment for potential impacts of increased design rainfall depths of 10%. Additional simulations have also been undertaken to assess the 30% increase.

Flood Risks

Flooding to the west of the railway is located along a number of gully lines that drain to Muddy Creek and Spring Street Drain. There are a number of locations along which the overland flow path alignment is not within the roadway, but instead traverses blocks of residential development. The floodway is usually situated along the yards to the rear of the properties and/or where flow is funnelled between buildings. The affected locations include:

- Properties located along two flow paths between Botany Street and High Street;
- The rear of properties located along High Street and Mill Street;
- Properties located along the flow path between Short Street and Edgehill Street;
- Properties located along the flow path between Guinea Street and Robinson Street;
- Properties located along the flow path between Percival Street and Queen Victoria Street;
- The rear of properties located along Robertson Street and Warialda Street;
- The rear of properties located along Campbell Street and Lymington Street;
- Properties located along two flow paths between Northbrook Street and Beaconsfield Street;
- Properties located along the flow path between Dunmore Street South and Warialda Street;
- Properties located along the flow path between Goyen Avenue and Watkin Street;
- The rear of properties located along Frederick Street;
- Properties located along the flow path between Heathcote Street and Arlington Street;
- The rear of properties located along Oswin Lane and Gloucester Street; and
- The rear of properties located along Godfrey Street and Bowmer Street.

The areas between the Princes Highway and Short Street, between Terry Street and Spring Street and between the Princes Highway and Cross Street (on the eastern side of the railway) also experience similar issues to the above. Further downstream the flooding problem areas are typically limited to locations where the capacity of the drainage channels is significantly exceeded. Such areas include:

- The properties along Spring Street Drain between Shaaron Court and West Botany Street;
- The properties along Muddy Creek between Harrow Road and Bay Street; and
- Properties along West Botany Street where local drainage to Muddy Creek is exceeded.

There are also a number of properties bordering the Scarborough Ponds that are affected.

There are also a number of areas that are particularly exposed to increased flood risk through potential blockage of structures, including:

- Properties situated between Prospect Street and Union Lane (~0.5m);
- Properties situated between Guinea Street and Cadia Street (~0.5m);
- Properties along Warialda Street (~1.0m);
- Properties situated between the railway and the Princes Highway (~1.1m);
- Properties around the Railway Street – Frederick Street intersection (~0.6m);

- Properties situated between Roach Street and the railway (~0.6m to 0.7m); and
- Properties situated between the Princes Highway and Short Street (~0.3m to 0.4m).

Conclusions

The primary objective of the study was to undertake a detailed flood study of the Muddy Creek, Spring Street Drain and Scarborough Ponds catchments and to establish models as necessary for design flood level prediction

In completing the flood study, the following activities were undertaken:

- Compilation and review of existing information pertinent to the study;
- Development and calibration of appropriate hydrologic and hydraulic models;
- Calibration of the developed models using the available flood data, including the recent events of 1993, 1998 and 2014; and
- Prediction of design flood conditions in the study area and production of design flood mapping series.

The principal outcome of the flood study is the understanding of flood behaviour in the study area and in particular design flood level information. The study provides updated and more detailed flooding information than the previous studies, to be used to inform floodplain risk management within the study area.

Given the significant increase in flood risk across certain areas under potential blockage scenarios the incorporation of blockage allowances within the design flood levels should be considered for flood planning purposes, particularly for the Warialda Street to Princes Highway section. It is expected that management of food risk within this area will be one of the key focuses of future floodplain risk management activities.

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Glossary

annual exceedance probability (AEP)	The chance of a flood of a given size (or larger) 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 (i.e. a 1 in 20 chance) of a peak discharge of 500 m^3/s (or larger) occurring in any one year. (see also average recurrence interval)
Australian Height Datum (AHD)	National survey datum corresponding approximately to mean sea level.
Astronomical Tide	Astronomical Tide is the cyclic rising and falling of the Earth's oceans water levels resulting from gravitational forces of the Moon and the Sun acting on the Earth.
attenuation	Weakening in force or intensity.
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 20yr ARI design flood will occur on average once every 20 years. ARI is another way of expressing the likelihood of occurrence of a flood event. (see also annual exceedance probability)
calibration	The adjustment of model configuration and key parameters to best fit an observed data set.
catchment	The catchment at a particular point is the area of land that drains to that point.
design flood event	A hypothetical flood representing a specific likelihood of occurrence (for example the 100yr ARI or 1% AEP flood).

development	Existing or proposed works that may or may not impact upon flooding. Typical works are filling of land, and the construction of roads, floodways and buildings.
discharge	The rate of flow of water measured in tems of vollume 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).
flood	Relatively high river or creek flows, which overtop the natural or artificial banks, and inundate floodplains and/or coastal inundation resulting from super elevated sea levels and/or waves overtopping coastline defences.
flood behaviour	The pattern / characteristics / nature of a flood.
flood fringe	Land that may be affected by flooding but is not designated as floodway or flood storage.
flood hazard	The potential risk to life and limb and potential damage to property resulting from flooding. The degree of flood hazard varies with circumstances across the full range of floods.
flood level	The height or elevation of floodwaters relative to a datum (typically the Australian Height Datum). Also referred to as “stage”.
flood liable land	see flood prone land
floodplain	Land adjacent to a river or creek that is periodically inundated due to floods. The floodplain includes all land that is susceptible to inundation by the probable maximum flood (PMF) event.

floodplain management	The co-ordinated management of activities that occur on the floodplain.
floodplain risk management plan	A document outlining a range of actions aimed at improving floodplain management. The plan is the principal means of managing the risks associated with the use of the floodplain. A floodplain risk management plan needs to be developed in accordance with the principles and guidelines contained in the NSW Floodplain Development Manual. The plan usually contains both written and diagrammatic information describing how particular areas of the floodplain are to be used and managed to achieve defined objectives.
Flood planning levels (FPL)	Flood planning levels selected for planning purposes are derived from a combination of the adopted flood level plus freeboard, as determined in floodplain management studies and incorporated in floodplain risk management plans. Selection should be based on an understanding of the full range of flood behaviour and the associated flood risk. It should also take into account the social, economic and ecological consequences associated with floods of different severities. Different FPLs may be appropriate for different categories of landuse and for different flood plans. The concept of FPLs supersedes the “standard flood event”. As FPLs do not necessarily extend to the limits of flood prone land, floodplain risk management plans may apply to flood prone land beyond that defined by the FPLs.
flood prone land	Land susceptible to inundation by the probable maximum flood (PMF) event. Under the merit policy, the flood prone definition should not be seen as necessarily precluding development. Floodplain Risk Management Plans should encompass all flood prone land (i.e. the entire floodplain).
flood source	The source of the floodwaters.

flood storage	Floodplain area that is important for the temporary storage of floodwaters during a flood.
floodway	A flow path (sometimes artificial) that carries significant volumes of floodwaters during a flood.
freeboard	A factor of safety usually expressed as a height above the adopted flood level thus determing the flood planning level. Freeboard tends to compensate for factors such as wave action, localised hydraulic effects and uncertainties in the design flood levels.
geomorphology	The study of the origin, characteristics and development of land forms.
gauging (tidal and flood)	Measurement of flows and water levels during tides or flood events.
historical flood	A flood that has actually occurred.
hydraulic	Relating to water flow in rivers, estuaries and coastal systems; in particular, the evaluation of flow parameters such as water level and velocity
hydrodynamic	Pertaining to the movement of water.
hydrograph	A graph showing how a river or creek's discharge changes with time.
hydrographic survey	Survey of the bed levels of a waterway.
hydrologic	Pertaining to rainfall-runoff processes in catchments

hydrology	The term given to the study of the rainfall-runoff process in catchments.
hyetograph	A graph showing the distribution of ranfall over time.
Intensity Frequency Duration (IFD) Curve	A statistical representation of rainfall showing the relationship between rainfall intensity, storm duration and frequency (probability) of occurrence.
isohyet	Equal rainfall contour.
morphological	Pertaining to geomorphology.
peak flood level, flow or velocity	The maximum flood level, flow or velocity that occurs during a flood event.
pluviographmeter	A rainfall gauge capable of continously measuring rainfall intensity
probable maximum flood (PMF)	An extreme flood deemed to be the maximum flood likely to occur.
probability	A statistical measure of the likely frequency or occurrence of flooding.
riparian	The interface between land and waterway. Literally means “along the river margins”
runoff	The amount of rainfall from a catchment that actually ends up as flowing water in the river or creek.
stage	See flood level.
stage hydrograph	A graph of water level over time.

sub-critical	Refers to flow in a channel that is relatively slow and deep
topography	The shape of the surface features of land
velocity	The speed at which the floodwaters are moving. A flood velocity predicted by a 2D computer flood model is quoted as the depth averaged velocity, i.e. the average velocity throughout the depth of the water column. A flood velocity predicted by a 1D or quasi-2D computer flood model is quoted as the depth and width averaged velocity, i.e. the average velocity across the whole river or creek section.
validation	A test of the appropriateness of the adopted model configuration and parameters (through the calibration process) for other observed events.
water level	See flood level.

1 Introduction

The Spring Street Drain, Muddy Creek and Scarborough Ponds Catchments Flood Study Review has been prepared for Rockdale City Council (Council) to define the existing and potential future flood behaviour in the study area and establish the basis for subsequent floodplain management activities.

This project has received technical and financial support from the NSW Government's Floodplain Management Program.

1.1 Study Location

This study area comprises three separate stormwater catchments, namely Spring Street Drain, Muddy Creek and Scarborough Ponds, which are located within Councils Local Government Area (LGA). The study area is bounded by Botany Bay to the east, Cooks River and the suburb of Arncliffe to the north and the suburbs of Ramsgate, Kogarah Bay and Bexley to the south and west. Figure 1-1 Study Locality the location of the catchments within Councils LGA.

The catchments drain an area of approximately 13.1 km^2, and is fully developed consisting primarily of medium to high-density housing and commercial developments. There are some large open spaces within the study area including the reserves and parks along Scarborough Ponds, Barton Park, McCarthy Reserve and Gardiners Park.

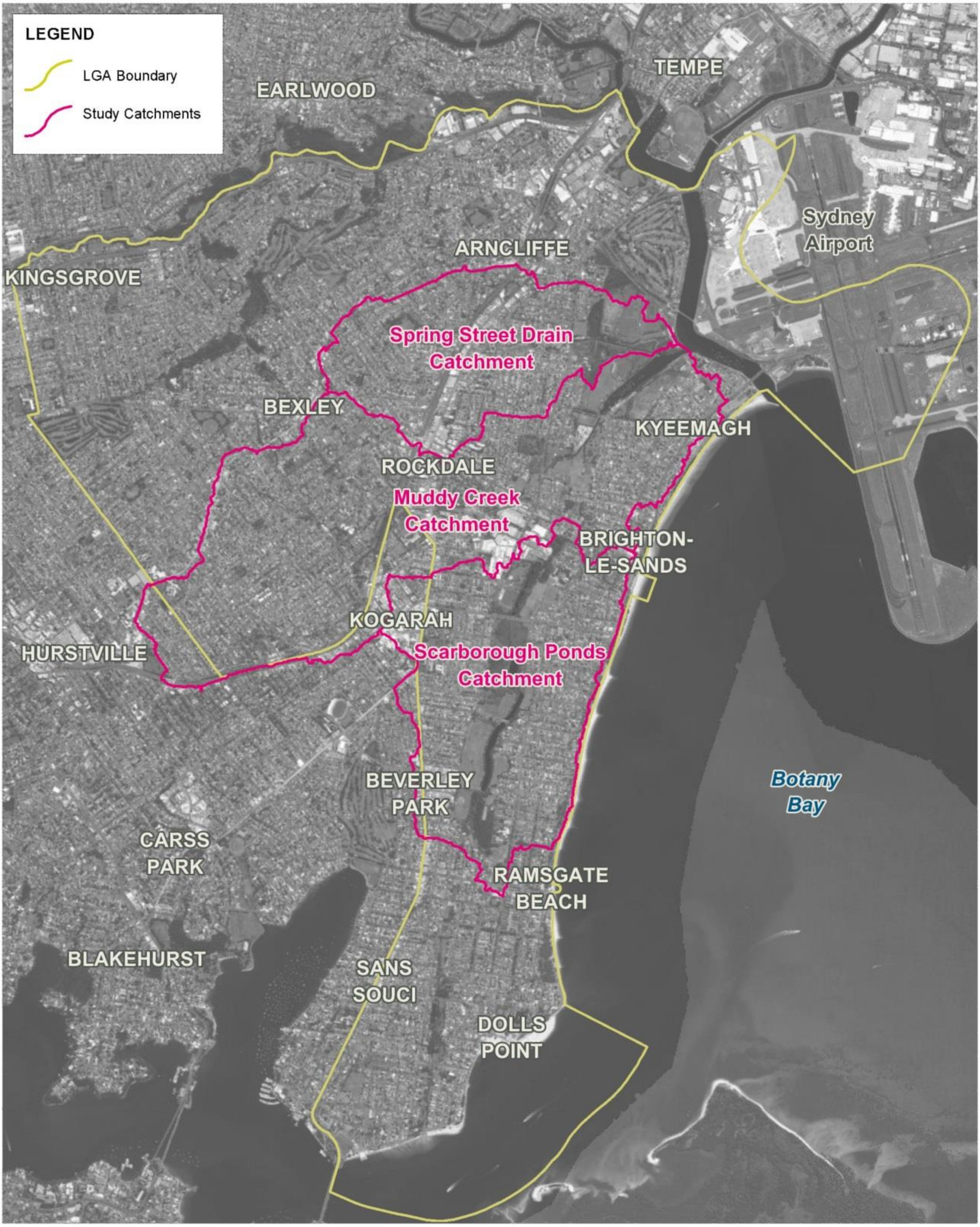
The Spring Street Drain and Muddy Creek catchments drain to Cooks River with the Scarborough Ponds catchment draining to Botany Bay. To the west of the Illawarra railway, the topography slopes gradually to the catchment boundaries. To the east of the Illawarra railway, the surface slopes are generally quite flat.

The study area is drained via the existing stormwater drainage system which consists mainly of sub-surface pipes, culverts and covered channels. There are also an extensive reaches of open channel in the Spring Street Drain and Scarborough Ponds catchments which generally have a constructed geometry and therefore have a regular profile. The Scarborough Ponds catchment consists of a number of linked dredged ponds and semi-natural wetlands which have formed behind the low beach ridge fronting Botany Bay.

There are a number of large infrastructure assets which traverse the study area. In order to provide flood-free transport, the Illawarra railway is often elevated above the natural floodplain levels restricting surface flows from west to east. Structures associated with the Princes Highway and the North Georges River Submain (NGRS) sewer also impact on the west to east surface flows.

1.2 The Floodplain Management Process

The NSW State Government's Flood Prone Land Policy is directed towards providing solutions to existing flooding problems in developed areas and ensuring that new development is compatible with the flood hazard and does not create additional flooding problems in other areas. Policy and practice are defined in the NSW State Government's Floodplain Development Manual (2005).



Title:
Study Locality

Figure: 1-1

Rev: A

BMT WBM endeavours to ensure that the information provided in this map is correct at the time of publication. BMT WBM does not warrant, guarantee or make representations regarding the currency and accuracy of information contained in this map.

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The implementation of the *Flood Prone Land Policy* culminates in the preparation and implementation of a Floodplain Management Plan in accordance with the Floodplain Management Process (see Figure 1-2) outlined in the *Floodplain Development Manual*. Periodic reviews of Floodplain Management Plans form part of the Floodplain Management Process. Under the Policy the management of flood liable land remains the responsibility of Local Government. The NSW 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 NSW State Government through the five sequential steps as shown in Figure 1-2.

As part of this study, steps 1 and 2 of this process will be undertaken which aims to provide an understanding of the existing and future flood behaviour due to climate change influences within the study area.

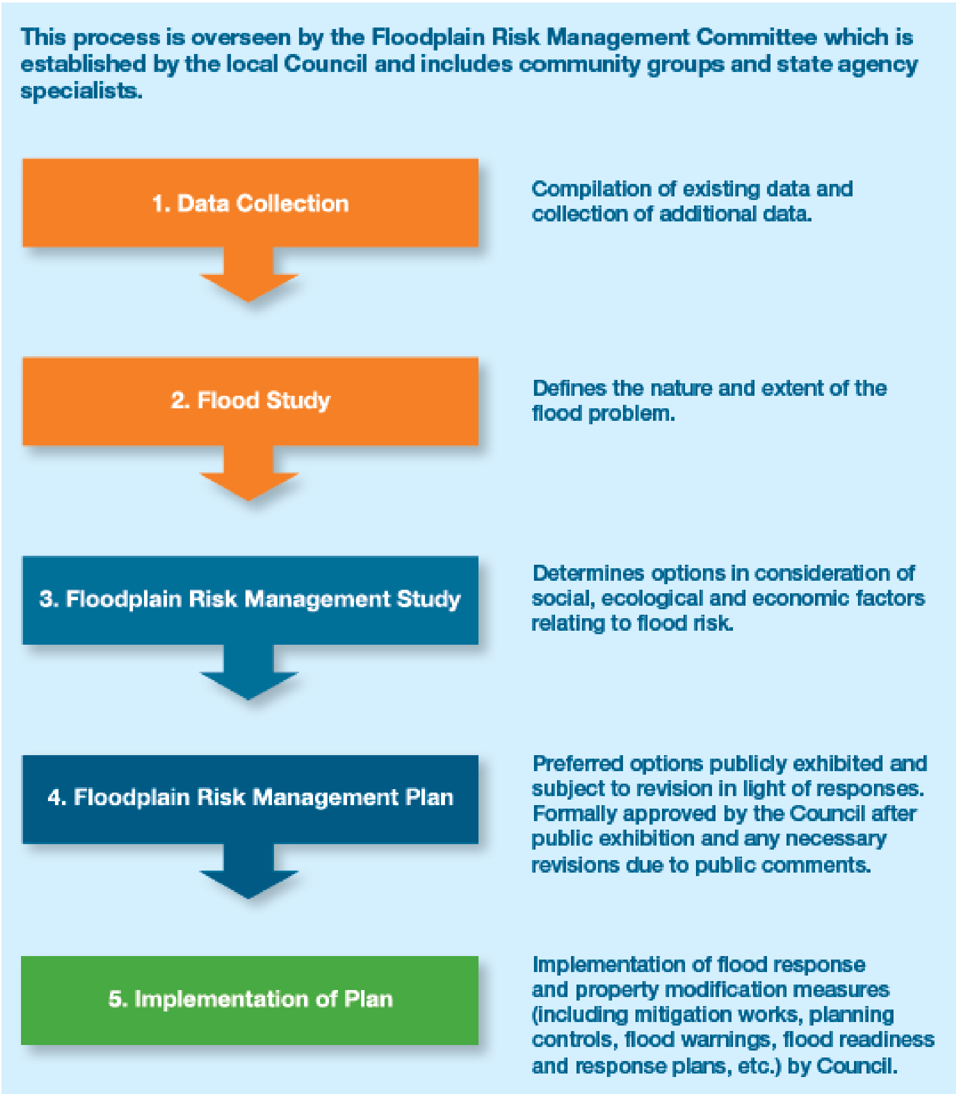


Figure 1-2 Steps of the Floodplain Management Process

1.2.1 The Floodplain Risk Management Committee

This study has been overseen by the Floodplain Risk Management Committee (Committee). The Committee has assisted and advised Council in the development of the flood study review. The Committee is responsible for recommending the outcomes of the study for formal consideration by Council. Members of the Floodplain Management Committee include representatives from the following:

- Rockdale City Council - Councillors;
- Staff from Rockdale City Council;
- Representatives from the NSW Office of Environment and Heritage (OEH); and
- Community representatives.

1.3 The Need for a Review of the Existing Flood Studies

There are a number of drivers which necessitate a review of the existing catchment flood studies including:

- Incorporation of latest sea level rise benchmarks;
- The availability of additional detailed ground survey data;
- Advances in modelling technology (use of two dimensional (2D) modelling);
- The implementation of some flood mitigation measures since the completion of the previous studies;
- Residential and commercial developments which have been completed since the completion of the previous flood studies; and
- The availability of additional numerical model calibration/validation data from recent flood events.

1.3.1 Climate Change Policy

Climate change is expected to have adverse impacts upon sea levels and rainfall intensities, both of which may have significant influence on flood behaviour at specific locations. The primary impacts of climate change in coastal areas are likely to result from sea level rise, which, coupled with a potential increase in the frequency and severity of storm events, may lead to increased coastal erosion, tidal inundation and flooding.

In 2009 the NSW State Government announced the NSW Sea Level Rise Policy Statement (DECCW, 2009) that adopted sea level rise planning benchmarks to ensure consistent consideration of sea level rise in coastal areas of NSW. These planning benchmarks adopt increases (above 1990 mean sea level) of 40 cm by 2050 and 90 cm by 2100. However, on 8 September 2012 the NSW Government announced its Stage One Coastal Management Reforms which no longer recommends state-wide sea level rise benchmarks for use by local councils. Instead councils have the flexibility to consider local conditions when determining future hazards of potential sea level rise.

Accordingly, it is recommended by the NSW Government that councils should consider information on historical and projected future sea level rise that is widely accepted by scientific opinion. This may include information in the NSW Chief Scientist and Engineer's Report entitled 'Assessment of the Science behind the NSW Government's Sea Level Rise Planning Benchmarks' (2012).

The NSW Chief Scientist and Engineer's Report (2012) acknowledges the evolving nature of climate science, which is expected to provide a clearer picture of the changing sea levels into the future. The report identified that:

- The science behind sea level rise benchmarks from the 2009 NSW Sea level Rise Policy Statement was adequate;
- Historically, sea levels have been rising since the early 1880's;
- There is considerable variability in the projections for future sea level rise; and
- The science behind the future sea level rise projections is continually evolving and improving.

Given that the Chief Scientist and Engineer's Report identifies the science behind these sea level rise projections is adequate, the potential impacts of sea level rise for the study area should be based on the best available information during preparation of this report.

For the study area, rising sea level is expected to increase the frequency, severity and duration of flooding along the foreshore of Botany Bay and the downstream reaches of Cooks River, Spring Street Drain and Muddy Creek. Council has previously engaged BMT WBM to undertake a study to assess the impacts of predicted sea level rises on the western foreshore of Botany Bay between the Cooks River and the Georges River and its effects on Lady Robinsons Beach / Cook Park and the surrounding environments.

In 2007 the NSW State Government released a guideline for practical consideration of climate change in the floodplain management process that advocates consideration of increased design rainfall intensities of up to 30%. Increased rainfall intensities will translate into increased fluvial flood inundation in the study area. Future planning and floodplain management will need to take due consideration of this increased flood risk.

In consultation with Council and the OEH, a range of climate change sensitivity tests incorporating combinations of sea level rise and increased design rainfall intensity have been formulated. The results of these sensitivity tests (refer to Chapter □) have been compared to the base case (i.e. numerical models with existing sea level and climate) model results in order to assess the potential increase in flood risk due to climate change.

1.3.2 LiDAR Data

Light Detention and Ranging (LiDAR) survey has been purchased from NSW Land and Property Information (LPI). The survey provides complete coverage of the study area and was captured in 2013. Horizontal and vertical accuracy is 0.8m and 0.3m respectively (95% confidence intervals).

The data has been supplied in a range of digital elevation model (DEM) grids ranging from 1m to 10m. The DEM is a natural surface dataset with features such as vegetation and buildings removed. The 1m DEM has been used for the development of the ground surface in the hydraulic model (refer to Section3.3). The 5m DEM has been used for the delineation of the catchments for the XP-RAFTS hydrologic model (refer to Section 3.2).

1.3.3 Modelling Techniques

Due to the complex nature of floodplain flow patterns in urban catchments, dynamically linked 2D/one dimensional (1D) hydrodynamic numerical models and are currently the most accurate, cost-effective and efficient tools to predict the flood behaviour.

The preferred method is to develop a catchment hydraulic model that consists of a high resolution 2D domain of the floodplain that is dynamically linked to a series of 1D domains that simulate the drainage characteristics of the stormwater network (i.e. pits and pipes system, open channels and culverts). For the simulation of the catchment rainfall-runoff processes, a lumped hydrological model has been developed with flows from this hydrological model routed through the hydraulic model domain.

1.4 Study Objectives

The primary objective of this Flood Study Review is to define the flood behaviour under historical, existing and future conditions (incorporating potential impacts of climate change) in the Spring Street Drain, Muddy Creek and Scarborough Ponds study area for a full range of design flood events. The study will provide information on flood levels and depths, velocities, flows, hydraulic categories and provisional hazard categories. Specifically, the study incorporates:

- Compilation and review of existing information pertinent to the study and acquisition of additional data including survey as required;
- Development and calibration of appropriate hydrological and hydraulic models;
- Determination of design flood conditions for a range of design events - including the 20% annual exceedance probability (AEP) which equates to a 5 year average recurrence interval (ARI), 10% AEP (10 year ARI), 5% AEP (20 year ARI), 2% AEP (50 year ARI), 1% AEP (100 year ARI), 0.5% AEP (200 year ARI) and the Probable Maximum Flood (PMF), noting that AEP refers to an Annual Exceedance Probability event and ARI refers to an Average Recurrence Interval flood; and
- Examination of potential impact of climate change using the latest guidelines.

The models and results produced in this study are intended to:

- Outline the current and potential future flood behaviour within the study area to aid in Council's management of flood risk; and
- Form the basis for a subsequent floodplain risk management study where detailed assessment of flood mitigation options and floodplain risk management measures will be undertaken.

1.5 About this Report

This report documents the Study's objectives, results and recommendations in the following chapters:

Chapter 1 introduces the study.

Chapter 2 provides an overview of the study and summary of background information.

Chapter 3 details the development of the computer models.

Chapter 4 details the numerical model calibration and validation process.

Chapter 5 details the design flood conditions.

Chapter 6 details the design flood results and associated flood mapping.

Chapter 7 details the sensitivity testing conducted.

Chapter 8 details the climate change analysis.

2 Study Approach

2.1 The Study Area

The Spring Street Drain, Muddy Creek and Scarborough Ponds catchments are located within the Rockdale City Council LGA. The study area occupies an area of approximately 13.1 km^2 that is drained via the existing stormwater drainage system, with the Spring Street Drain and Muddy Creek catchments draining to Cooks River and the Scarborough Ponds catchment draining to Botany Bay. Figure 2-1 shows the individual catchments and study area boundary.

The topography of the study area is relatively flat, particularly east of the Illawarra Railway. To the west of the Illawarra railway, the topography slopes gradually, with a peak elevation of 68.5 m AHD to the south west of the Muddy Creek catchment. The upper reaches of the Muddy Creek catchment generally slope in a south-easterly direction with the lower reaches draining north east towards Cooks River. The Spring Street Drain has a peak elevation of 55.5m AHD, with the catchment generally draining eastwards. The Scarborough Ponds catchment has a peak elevation of 31.5m AHD. The catchment generally drains towards the Scarborough Ponds with the ponds draining southwards to an artificial outlet to Botany Bay.

The catchment is a highly modified landscape, comprising medium to high-density residential and commercial developments. It also includes major infrastructure assets including the Princes Highway, Illawarra Railway and the NGRS sewer line. These infrastructure assets, where raised above the natural ground level, restrict surface flows from west to east.

Further discussion on the hydraulic features relevant to each of the catchments is provided in the following sections.

2.1.1 Spring Street Drain Catchment

The Spring Street Drain catchment area is approximately 2.7 km^2. The Spring Street Drain is a brick and concrete lined stormwater channel which runs for approximately 2 km from near Short Street, Banksia, to join the tidal section of Muddy Creek just upstream from its confluence with the Cooks River. The open channel of Spring Street Drain is crossed by several bridges, including West Botany Street, Banksia .The NGRS sewer also crosses the channel and has the potential to obstruct flows as it has no bypass flow arrangement.

An extensive stormwater network of pits, pipes, open channels and covered box sections form a tributary drainage system which drains to the main Spring Street Drain channel.

2.1.2 Muddy Creek Catchment

The Muddy Creek catchment area is approximately 6.2 km^2 with portions of the catchment extending into Hurstville and Kogarah LGA's. The catchment is drained by an extensive stormwater network which collects flows and diverts it to the Muddy Creek Stormwater Channel (SWC). The Muddy Creek SWC is a brick and concrete lined stormwater channel which runs for approximately 4.3 km through the catchment. The channel forms the main drainage system in the catchment and is owned by Sydney Water. The Muddy Creek SWC drains to the Cooks River estuary.

Downstream of Bestic Street, Rockdale, the channel has been dredged and widened to form a tidal basin.

Significant hydraulic features in the Muddy Creek catchment include several road bridges, footbridges and a culvert under the Illawarra railway. As with the Spring Street Drain catchment, the NGRS sewer is a key infrastructure asset which impacts on flooding. The NGRS sewer crosses the channel upstream of Princes Highway, Rockdale, where it obstructs flows in large floods.

2.1.3 Scarborough Ponds Catchment

The Scarborough Ponds catchment has a total area of approximately 4.2 km^2. Scarborough Ponds is a series of dredged ponds and semi-natural wetlands which have formed behind the low beach ridge fronting Botany Bay. An artificial outlet to Botany Bay, comprising three 1350mm diameter pipes, was constructed in the 1970's at Florence Street, Ramsgate Beach, to improve drainage.

Two road crossings, President Avenue, Brighton-Le-Sands and Barton Street, Monterey, cross the ponds. These crossings and the surrounding land are all relatively low-lying. A number of pipes and drainage ditches convey stormwater into the ponds from the surrounding catchment.

2.2 Compilation and Review of Available Data

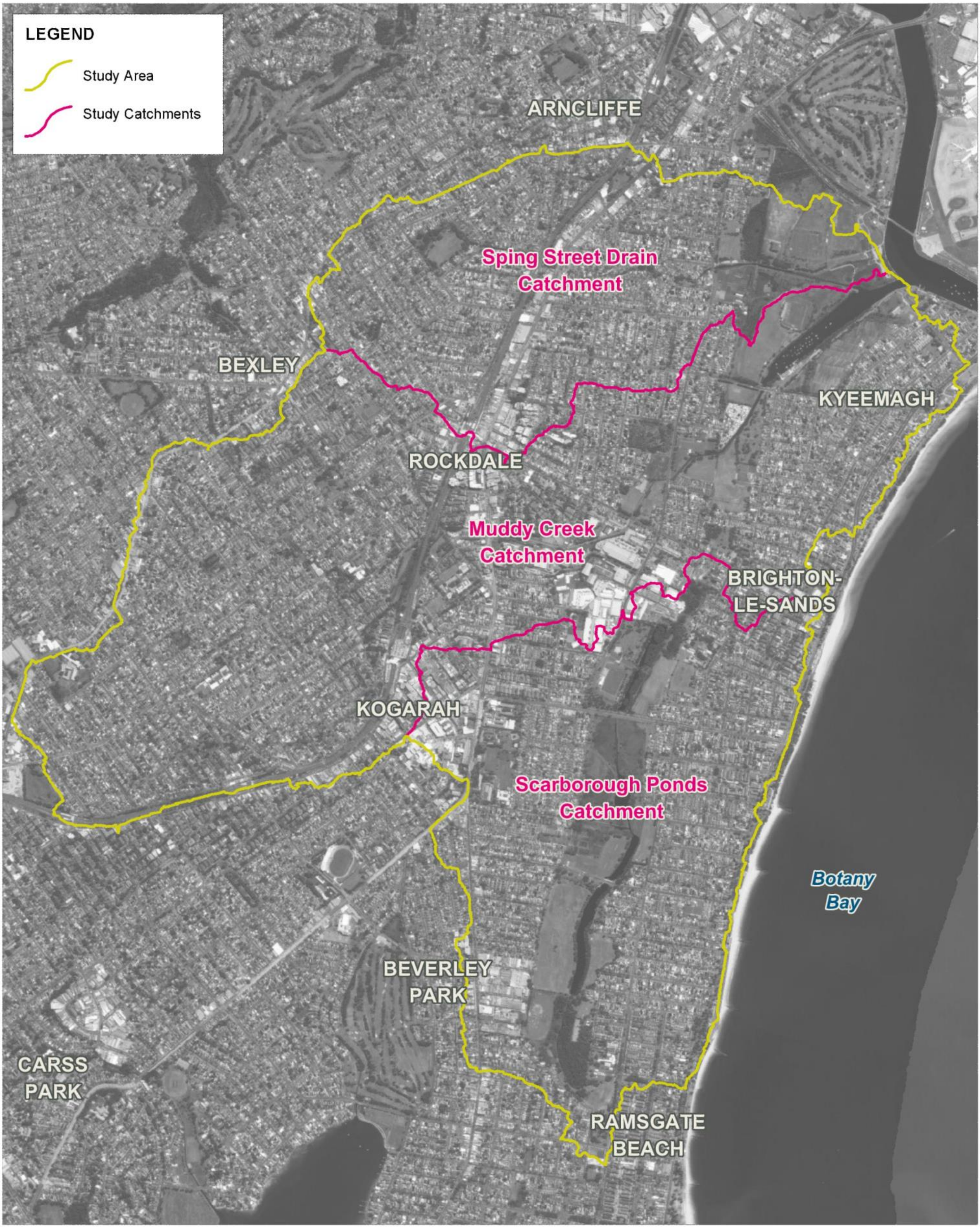
2.2.1 Introduction

The data compilation and review has been undertaken as the first stage in this flood study in order to consolidate and summarise all of the currently available data, and identify any significant data gaps that may affect the successful completion of the study. This allowed for the missing data to be collected during the initial phases of the study.

The review included:

- Previous studies undertaken within the catchment;
- Available water level, tide and rainfall data; and
- Register of data from historic flood events.

Council has provided digitally available information such as aerial photography, cadastral boundaries, watercourses, and drainage networks in the form of GIS datasets.



Title:
Study Area

Figure: 2-1

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Approx. Scale

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Summary of Previous Studies and Investigations

Details of previous flood studies, floodplain risk management plans and pipe drain and overland flow studies undertaken within the study area and the numerical models developed as part of these studies are listed in Table 2-1.

Table 2-1 Previous Studies, Reports and Numerical Models

Catchment Name Major Waterway Catchment(s)	Flood Studies	Floodplain Risk Management Plans	Pipe Drain and Overland Flow Studies
Spring Street Cooks River	Spring Street Drainage Catchment Flood Study, 1997, Lawson & Treloar (SS), (Mouse & Mike-11)	**Spring Street Drain, Muddy Creek and Scarborough Ponds, FRMP, 2000, Willing & Partners**, (RAFTS-XP & Mike-11, DRAINS or XP-SWMM ***)	**Spring Street Drain – Piped Drainage and Overland Flow Analysis, 2007, Brown Consulting**, (DRAINS & HEC-RAS)
Muddy Creek Cooks River	Scarborough Ponds, Muddy Creek and Sans Souci No 1 Drain Flood Study, 1997, AWACS (SS), (WBNM & Mike-11) **Muddy Creek Probable Maximum Flood Modelling, 2007, Cardno Willing**, (XP-SWMM)	**Spring Street Drain, Muddy Creek and Scarborough Ponds, FRMP, 2000, Willing & Partners**, (RAFTS-XP & Mike-11, DRAINS or XP-SWMM ***)	**Upper Muddy Creek Piped Drainage Analysis Stage 1, 1999, Webb McKeown & Associates**, (DRAINS) **Lower Muddy Creek and Scarborough Ponds catchments: Overland Flooding and Risk Assessment Study, 2004, Brown Consulting**, (DRAINS & HEC-RAS)
Scarborough Ponds Botany Bay	Scarborough Ponds, Muddy Creek and Sans Souci No 1 Drain Flood Study, 1997, AWACS (SS) (WBNM & Mike-11) **Muddy Creek Probable Maximum Flood Modelling, 2007, Cardno Willing**, (XP-SWMM)	**Spring Street Drain, Muddy Creek and Scarborough Ponds, FRMP, 2000, Willing & Partner**, (RAFTS-XP & Mike-11, DRAINS or XP-SWMM ***)	**Lower Muddy Creek and Scarborough Ponds catchments: Overland Flooding and Risk Assessment Study, 2004, Brown Consulting**, (DRAINS & HEC-RAS)

- Bold represents a current study/plan relied upon to advise developments.
- SS indicates superseded study.
- *** indicates only 'additional' modelling – using XP-RAFTS and MIKE-11 on the main open channel, and DRAINS or XP-SWMM for piped systems and overland flow paths.

The numerical models listed in Table 2-1 were developed as part of these previous studies to identify the flood behaviour in the study area and assess a range of options aimed at managing the flood risk. Typically these studies involved the development of a hydrological model to convert rainfall into runoff with a hydraulic model developed to route the runoff through the drainage network. A 1D hydraulic modelling approach was adopted to determine the flows and levels for model calibration and validation events and for a range of design flood events.

The topographic and structural features of the hydraulic models have been developed from a number of datasets including Councils pit and pipe database, topographic maps (i.e. ortho-photomaps) and topographic survey data (i.e. channel cross sections and structure dimensions). The hydraulic models developed as part of these previous studies have been used to inform the development of the hydraulic model as part of this study.

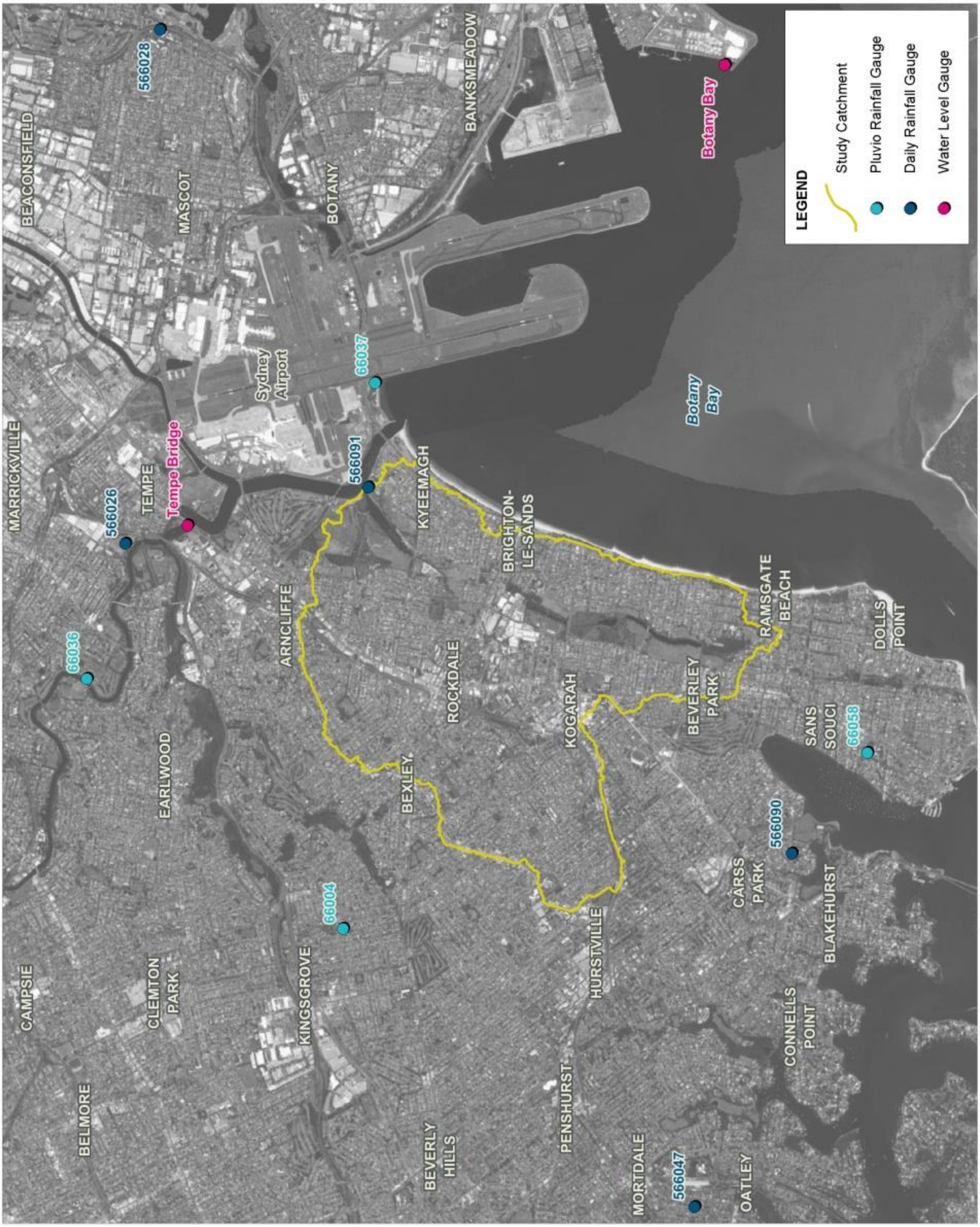
2.2.2 Council GIS Data

Digitally available GIS data such as aerial photography, cadastral boundaries, roads, drainage networks and park streetscapes have been provided by Council. This data provide a means to distinguish between land-use types across the study area to allow spatial variation of distinct hydrologic (e.g. rainfall losses) and hydraulic properties (e.g. Manning's roughness parameter 'n'). The data has also been used to identify any potential data gaps.

2.2.3 Rainfall Data

There is an extensive network of rainfall gauges across the Sydney area, the majority of which are operated by the Bureau of Meteorology (BoM) and Sydney Water Corporation (SW). There is one rainfall station located within the study area and a number of stations within close proximity to the study area which have data relevant to this study.

A list of rainfall stations relevant this study, the type of data available and their respective period of record are shown in Table 2-2, with the spatial distribution of the rainfall stations shown in Figure 2-2. The combination of daily rainfall stations and pluvio stations has been used to define the temporal pattern of historic rainfall events and provides a high quality rainfall data set for use in the model calibration and validation as part of this study.



Title:

Rainfall and Water Level Gauges

Figure: **2-2**

Rev: **A**

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Approx. Scale

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Table 2-2 Rainfall Gauges Within and in the Vicinity of the Study Area

Station #	Station Name	Record Period	Data Type	Authority
66037	Sydney Airport AMO	1929 - current	Daily/Pluvio	BoM
566090	Carss Park Bowling Club	1991 - 2006	Pluvio	SW
566091	Kyeemagh Bowling Club	Not available	Pluvio	SW
566026	Marrickville Sps	1904 – current	Pluvio	SW
66064	Bexley Bowling Club	1931 – 2008	Daily	BoM
66058	Sans Souci (Public School)	1899 – current	Daily	BoM
66036	Marrickville Golf Club	1904 – present	Daily	BoM
566028	Mascot Bowling Club	1974 – present	Pluvio	SW
566047	Oatley (Woronora Parade)	1981 – present	Pluvio	SW

2.2.4 Water Level and Tide Level Data

The study area catchments flow into Botany Bay and the lower reaches of Cooks River respectively. Consequently, the water level within both Botany Bay and Cooks River can act as a significant downstream control for both overland and piped flows under flooding conditions resulting from rainfall events. Given its proximity to Botany Bay, the water levels in the lower reaches of Cooks River are essentially equivalent to that in Botany Bay.

The tides in Botany Bay are typical of the NSW east coast, being semidiurnal, that is generally two high tides and two low tides each day with a diurnal inequality. These tidal conditionals also influence river levels along the downstream reaches of Cooks River which in turn affect Muddy Creek and Spring Street Drain.

For calibration and validation events, a tail water boundary for Botany Bay and Cooks River has been adopted based on water level records obtained from the BoM's National Tidal Centre and a river level gauge on Cooks River at Tempe Bridge, Tempe. Table 2-3 shows the tidal statistics obtained from the National Tidal Centre. Figure 2-2 shows the location of the tidal gauge in Botany Bay and river level gauge on Cooks River.

Table 2-3 Botany Bay Tide Levels

Tidal Level	Level (m)	
	Tide Gauge	AHD
Maximum Recorded Tide	2.320	1.395
Highest Astronomical Tide (HAT)	2.107	1.182
Mean High Water Springs (MHWS)	1.612	0.687
Mean High Water Neaps (MHWN)	1.369	0.444
Mean Sea Level (MSL)	0.992	0.067
Mean Low Water Neaps	0.615	-0.310
Mean Low Water Springs	0.372	-0.553
Lowest Astronomical Tide	0.073	-0.852

2.2.5 Historical Flood Level Data

There is limited surveyed data of historic flood levels available for this study area. Model calibration and validation primarily relied upon anecdotal reports of flooding from the community, Council records, Sydney Water records and photographs of flood behaviour. Photographs cannot be assumed to record the peak flood behaviour, however, they are important for identifying flooding hotspots.

Where sufficient anecdotal information was available on the historical depth of flooding, Council undertook field surveys to measure the water level in m AHD. This information has been used as part of the hydraulic model calibration.

2.2.1 Topographic Data

LiDAR survey data has been was purchased from NSW LPI as discussed in Section 1.3.2. Both the 1m DEM and 5m DEM supplied as part of this dataset have been used in the development of the numerical models.

2.2.2 Stormwater Drainage Network

An extensive network of stormwater drainage infrastructure exists in the study area to provide drainage of surface water runoff. The infrastructure primarily consists of a pit and pipe stormwater network and a number of open channels.

The drainage network has been previously been modelled using DRAINS software. A total of five separate DRAINS models have been provided for the study area. A comparison between the DRAINS models and Councils drainage records identified a number of locations with gaps in the drainage dataset. Additional surveys were undertaken to capture details of the drainage network in locations were gaps were identified. Further discussion on this survey is provided in Section 2.4.

A number of issues were also identified with the geographic projections of the DRAINS models. These issues were resolved as part of the development of the hydraulic model.

2.3 Site Inspections

Site inspections have been undertaken during the course of the study to gain an appreciation of local hydraulic features and their potential influence on the flood behaviour. Some of the key observations accounted for during the site inspections included:

- Presence of hydraulic controls;
- General nature of overland flow paths noting overland flow path obstructions including boundary walls and fences;
- General nature of the open channels noting channel shape, material and in-channel structures; and
- Location of development and infrastructure on the floodplain.

This visual assessment was useful for defining hydraulic properties within the hydraulic model and ground-truthing of topographic features identified in the DEM.

2.4 Additional Stormwater Drainage Survey

Following the review of available stormwater drainage network data, a number of locations were identified where additional pit and pipe survey data was required. A survey brief was prepared and a surveyor was engaged to capture the following pit and pipe details:

- Pit location coordinates;
- Reduced levels of the pit entry;
- Pit opening sizes;
- Number of pipes entering the pit;
- Number of pipes exiting the pit;
- Pipe invert levels;
- Pipe diameters; and
- Pipe material.

The survey was completed in late April 2015 after which the data was incorporated into the hydraulic model developed as part of the study.

2.5 Modelling Approach

2.5.1 Hydrological Model

A hydrologic model has been developed to simulate the rate of storm runoff from the catchment using XP-RAFTS software (refer to Chapter 3). The study area has been delineated into 190 sub-catchments with a flow hydrograph output at the outlet of each sub-catchment. These flow hydrographs form the inflow boundaries to the hydraulic model.

2.5.2 Hydraulic Model

A hydraulic model has been developed using TUFLOW software (refer to Chapter 3). The hydraulic model developed for this study includes:

- 2D representation of the floodplain of the combined catchments (i.e. complete coverage of the total study area);
- 2D representation of the open channel drainage network; and
- 1D representation of the stormwater pipe network,

The hydraulic model is applied to determine flood levels, velocities and depths across the study area for historical and design events.

2.6 Calibration/Validation and Sensitivity Testing of Models

The hydraulic model was calibrated and validated against available historical flood event data to establish the values of key model parameters and confirm that the models were capable of adequately simulating real flood events.

The following criteria are generally used to determine the suitability of historical events to use for calibration or validation:

- The availability, completeness and quality of rainfall and flood level event data;
- The amount of reliable data collected during the historical flood information survey; and
- The variability of events – preferably events would cover a range of flood sizes.

The available historical information highlighted three flood events with sufficient data to potentially support a calibration and validation process.

The calibration and validation of the hydraulic model is presented in Chapter 4. A series of sensitivity tests were also carried out to evaluate the model. These tests were conducted to examine the performance of the models and determine the relative importance of different hydrological and hydraulic parameters. The sensitivity testing of the model is detailed in Chapter 7.

2.7 Establishing Design Flood Conditions

Design floods are statistical-based events which have a particular probability of occurrence. For the study area, design floods were based on design rainfall estimates according to Australian Rainfall and Runoff (IEAust, 2001).

The design flood conditions form the basis for floodplain management in the catchment and in particular design planning levels for future development controls. The predicted design flood conditions are presented in Chapter 5.

2.8 Mapping of Flood Behaviour

Design flood mapping is undertaken using outputs from the hydraulic model. Maps are produced showing water level, water depth and velocity. The maps present the peak value of each parameter.

Provisional flood hazard categories and hydraulic categories are derived from the hydrodynamic model results and are also mapped. The mapping outputs are described in Chapter 6 and presented in a separate Flood Mapping Compendium.

3 Development of Computer Models

3.1 Modelling Methodology

The modelling approach adopted for this study has been developed through experience on a number of urban catchment overland flow studies across NSW. The key steps of the methodology include:

- Development of a detailed DEM for the catchment;
- Delineation of catchment flow paths and hydrological sub-catchments;
- Development of hydraulic roughness surfaces for the catchment;
- Development of a 1D stormwater drainage network;
- Representation of hydraulic structures; and
- Development of key hydraulic controls along main overland flow paths.

The modelling of overland flow paths in urban environments presents a number of challenges for flood modelling. It is limited by the resolution and accuracy of both the available data and the hydraulic model to represent intricate local hydraulic controls. The available data and hydraulic model representation generates much uncertainty within the model results, as many controls on flood mechanisms are not accurately captured. These mechanisms include:

- Stormwater pit capture for on-grade locations;
- Available flow capacity of kerb and gutter profiles;
- Impact of parked vehicles on the road and stormwater network hydraulic performance
- Crest level controls of driveway entrances;
- Complexity of urban lot vegetation;
- Flow under, over, around and through fences of various materials;
- Flood storage within underground basements;
- Flow under, around and between buildings and/or through gates; and
- Collection and re-distribution of debris by catchment runoff and the potential impact on the inlet capacity of the stormwater drainage network and/or hydraulic structures such as culverts.

The above list demonstrates the many difficulties in representing the real-world flood mechanics of small urban catchments within any modelling framework. This is particularly relevant higher up the catchment where flow paths are smaller, gradients steeper and flood depths lower. However, as the upstream contributing catchment size increases and the resultant overland flow path increases in significance, the effect of the many uncertainties reduces and a reasonable level of confidence can be drawn from the outputs of the flood modelling.

The purpose of modelling overland flow paths in urban catchments is to identify and quantify flood risk along the major overland flow path alignments. Measures with which these risks can be managed can then be assessed through use of the hydraulic model as an assessment tool. There may be many other issues throughout the catchment that are perceived by the community as being "flooding", which are in fact local drainage issues. These are typically located higher up the catchment in steeper areas, where either the gutter capacity is insufficient or the crest level of driveways too low to contain catchment runoff and inter-allotment drainage. This can initiate minor overland flow paths that direct flood waters into private properties. These issues are not readily represented in flood modelling due to scale limitations and data accuracy. However, solutions to the problems also do not require the assistance of flood modelling tools and local drainage improvements are typically sufficient.

The adopted modelling methodology is most suited for the intended purpose of the hydraulic model outputs. It utilises the advantages of both traditional hydrological models and the direct rainfall approach of 2D hydraulic models, whilst avoiding the associated disadvantages. The scale at which hydrological sub-catchments are defined results in the majority of catchment runoff routing occurring within the hydraulic model. This is advantageous compared to the simplified routing algorithms employed within hydrologic models. For areas upstream of the hydraulic model inflows the rainfall-runoff is processed within the hydrologic model. There are a number of advantages gained by excluding these areas from the hydraulic model, which cannot be achieved through a direct rainfall approach, including:

- Hydraulic roughness representation in hydraulic models (Manning's 'n') is not directly translatable to the representation of roughness for sheet flow conditions;
- Local depressions within the DEM do not drain in the hydraulic model and runoff volume is lost to these small distributed storages, that in reality would typically drain;
- The computational burden of a direct rainfall approach produces significantly larger model simulation times;
- Attempting to hydraulically model areas with slopes in excess of a 10% grade typically introduces numerous instabilities to the model solution; and
- Model results are not output for the entire catchment, prohibiting flood mapping within upper catchment areas, where the modelling uncertainty is significant and the adoption of model results for flood planning purposes is often inappropriate and/or erroneous.

Restricting hydraulic model computations to areas with a significant upstream contributing catchment area ensures that a reasonable level of confidence can be maintained across the full extent of the flood mapping output. It also prevents model outputs generating flood planning restrictions in areas that are dominated by shallow runoff, where flooding/drainage issues can be addressed through small-scale local measures and/or there is a low confidence level in the modelling to reproduce the actual flooding mechanisms and behaviour.

For this study the CatchmentSIM and XP-RAFTS software packages have been used for the purposes of hydrological modelling, with TUFLOW being used for hydraulic modelling.

3.2 Hydrologic Model

3.2.1 Flow Path Mapping and Catchment Delineation

The Spring Street Drain, Muddy Creek and Scarborough Ponds catchments drains an area of approximately 13.1 km^2. For the hydrological model this area has been delineated into 190 sub-catchments as shown in Figure 3-1.

Flow path mapping and sub-catchment delineation has been undertaken using the CatchmentSIM software. A 5m resolution DEM derived from the 2013 LPI LiDAR data set was imported into the software and following hydrologic conditioning (removal of flats and pits), flow paths and sub-catchment boundaries were generated.

In defining sub-catchment outlets, consideration has been given to the underlying pipe drainage network. Sub-catchment boundaries coincide with the location of major trunk drainage system infrastructure inlets, junctions and outlets where appropriate.

Sub-catchment properties calculated in CatchmentSIM, including catchment areas, impervious proportions and vectored slopes form the basis of the catchment data for the XP-RAFTS hydrological modelling undertaken. Flow hydrographs are output from the hydrological model at the outlet of each sub-catchment. These flow hydrographs form inflow boundaries to the hydraulic model.

The key catchment parameters adopted in the XP-RAFTS model include catchment area, vectored slope, impervious percentage and PERN (roughness) value, estimated from the available topographic and spatial information. The majority of the catchment consists of urban development and so the model has been configured using the second sub-catchment approach, where the pervious and impervious catchment areas are calculated separately. The impervious percentage of sub-catchments is typically in the order of 60%. A PERN value of 0.04 was adopted for the pervious catchment portions, with 0.015 adopted for the impervious portions.

3.2.2 Rainfall Data

Rainfall information is the primary input and driver of the hydrological model, which simulates the catchments response in generating surface run-off. Rainfall characteristics for both historical and design events are described by:

- Rainfall depth – the depth of rainfall occurring across a catchment surface over a defined period (e.g. 270mm in 36 hours or average intensity 7.5mm/h); and
- Temporal pattern – describes the distribution of rainfall depth at a certain time interval over the duration of the rainfall event.

Both of these properties may vary spatially across the catchment.

The procedure for defining these properties is different for historical and design events. For historical events, the recorded hyetographs at continuous rainfall gauges provide the observed rainfall depth and temporal pattern. Where only daily read gauges are available within a catchment, assumptions regarding the temporal pattern may need to be made.

Title:

RAFTS Model Hydrological Sub-catchments

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For design events, rainfall depths are most commonly determined by the estimation of intensity-frequency-duration (IFD) design rainfall curves for the catchment. Standard procedures for derivation of these curves are defined in AR&R (2001). Similarly AR&R (2001) defines standard temporal patterns for use in design flood estimation.

The rainfall inputs for the historical calibration/validation events are discussed in Chapter 4.

3.3 Hydraulic Model

3.3.1 Topography

The ability of the hydraulic model to provide an accurate representation of the flow distribution on the floodplain ultimately depends upon the quality of the underlying topographic model. A 1m by 1m gridded DEM was derived from the 2013 LPI LiDAR data set for the study area.

The channel topography has been incorporated into the 2D model representation and is discussed further in Section 3.3.4.

3.3.2 Extents and Layout

Consideration needs to be given to the following elements in constructing the hydraulic model:

- Topographical data coverage and resolution;
- Location of recorded data (e.g. levels/flows for calibration);
- Location of controlling features (e.g. detention basins, levees, bridges);
- Desired accuracy to meet the study's objectives; and
- Computational limitations.

With consideration to the available survey information and local topographical and hydraulic controls, a 2D model was developed incorporating the entire Spring Street Drain, Muddy Creek and Scarborough Ponds catchments. The model incorporates a number of open channels and drains totalling some 8km in length, as well as around 1700ha of water bodies within the Scarborough Ponds system. A total length of some 35km of trunk stormwater drainage is also included within the model.

A TUFLOW 2D domain model resolution of 2m was adopted for study area. It should be noted that TUFLOW samples elevation points at the cell centres, mid-sides and corners, so a 2m cell size results in DEM elevations being sampled every 1m. This resolution was selected to give necessary detail required for accurate representation of floodplain and channel topography and its influence on overland flows.

3.3.3 Hydraulic Roughness

The development of the TUFLOW model requires the assignment of different hydraulic roughness zones. These zones are delineated from aerial photography and cadastral data identifying different land-uses (e.g. forest, cleared land, roads, urban areas, etc.) for modelling the variation in flow resistance.

In the absence of suitable data from which to calibrate appropriate Manning's 'n' values, standard values have been adopted for this study as presented in Table 3-1. The spatial distribution of model materials inputs representing variations in hydraulic roughness is presented in Figure 3-2.

Table 3-1 Adopted Roughness Parameters

Land Use	Manning's 'n' Value
Concrete channel	0.015
Paved surfaces	0.02
Maintained grass	0.03
Grassed areas	0.04
Urban lots	0.07
Vegetated areas	0.09
Buildings	1.0

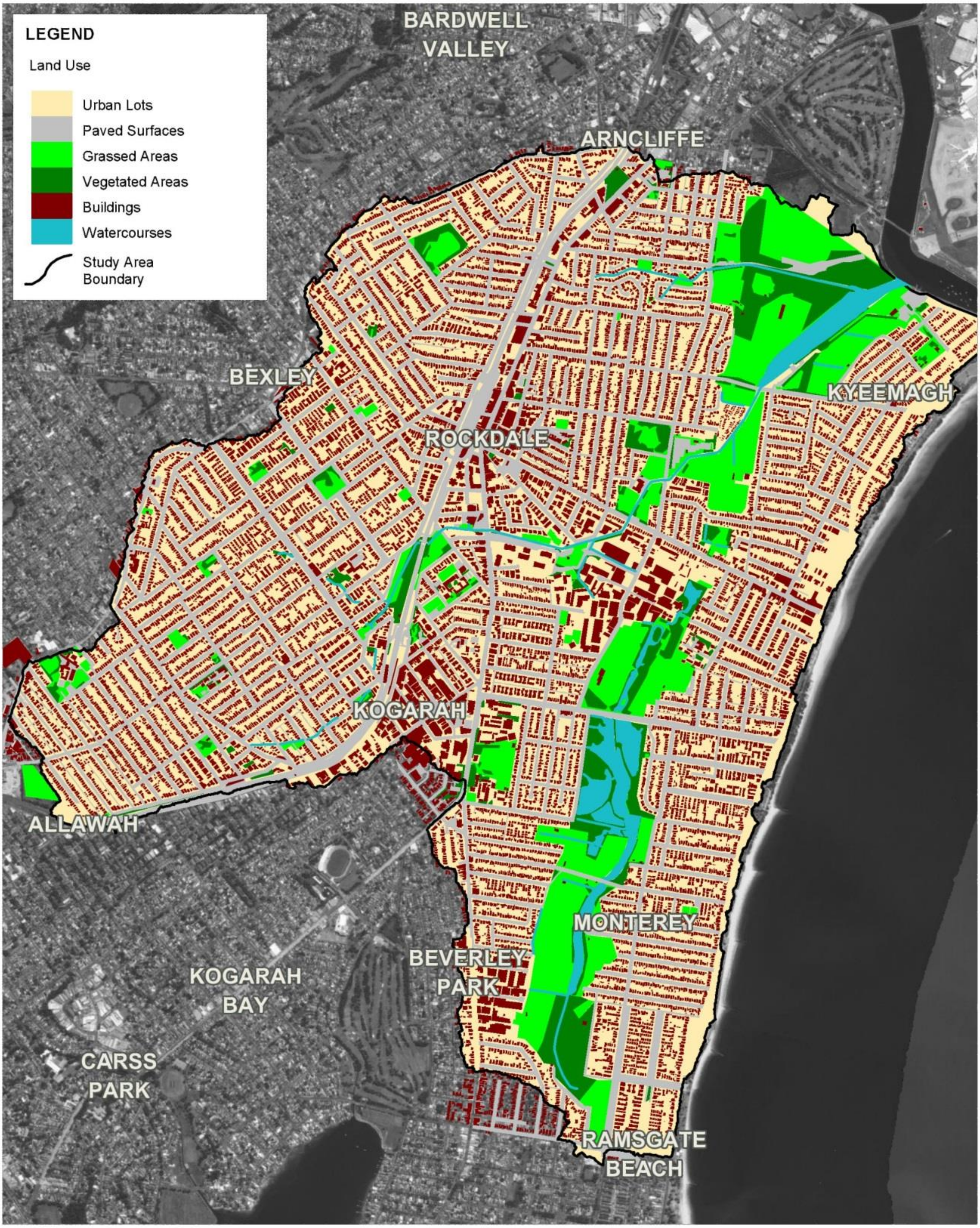
3.3.4 Channel Network

The LiDAR data provides an accurate representation of floodplain topography, but does not always maintain representative channel details, especially when submerged below a water surface. To accurately represent channel dimensions and flow capacity, the channel network representation must be further improved within the hydraulic model. The approach adopted in this study involved embedding the channel topography within the 2D model domain. This provides several advantages over a 1D channel representation, including:

- A smoother transition between channel and floodplain conveyance;
- A more spatially rich representation of the high-flow in-channel flood conveyance, taking account of local topographic controls both at and beneath bank-full level;
- An inherent representation of the channel sinuosity;
- Spatial variation of velocities across the width of the channel; and
- Improved flood mapping output for in-channel areas.

However, for steep-sided concrete channels, the 2D representation introduces additional losses that are not representative of the actual conveyance conditions. These lengths of channel were therefore modelled using a 1D representation of standard channel node cross-sections linked by channel reaches in a 1d_nwk layer. The channel is then dynamically linked to the 2D domain via 2d_bc HX connections, discharging water to the floodplain once the channel capacity is exceeded.

Upstream of the Cooks River three distinct watercourses were modelled: the main channel of Muddy Creek, the Spring Street Drain and the Scarborough Ponds. The extents of the modelled watercourses are presented in Figure 3-3. Reliable estimates of channel widths were measured from the aerial photography and reasonable assumptions were made as to the channel bed levels using the available data within the LiDAR survey and the stormwater drainage network pipe inverts.



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Modelled Land Use Map

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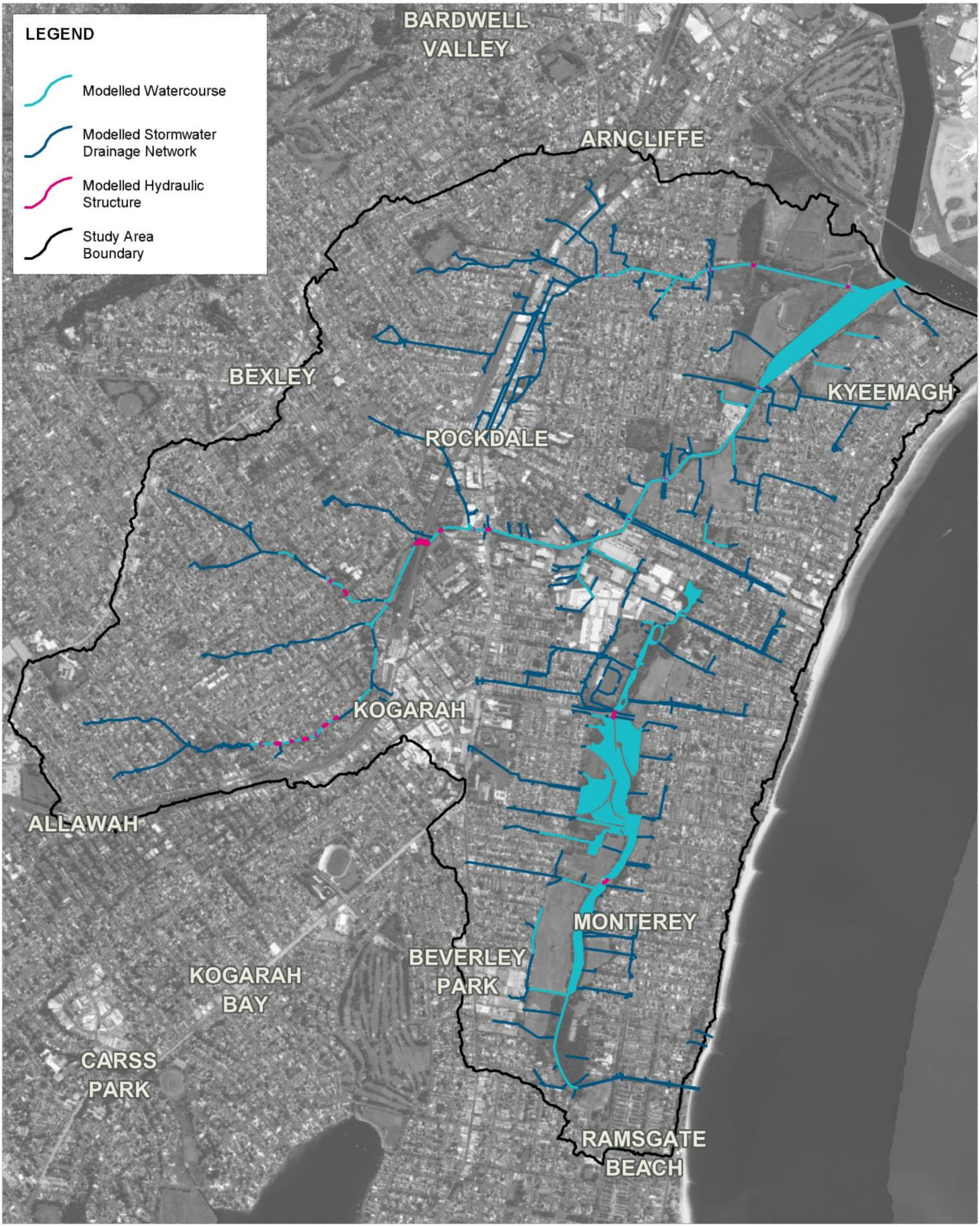

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Modelled Channel and Stormwater Network

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3.3.5 Structures

There are a number of bridges and culvert crossings over the open channel alignments within the model extents as presented in Figure 3-3. These structures vary in terms of construction type and configuration, with varying degrees of influence on local hydraulic behaviour. Incorporation of these hydraulic structures in the models provides for simulation of the hydraulic losses associated with these structures and their influence on peak water levels within the study area.

Bridge structures and larger culvert crossings have been modelled as flow constrictions within the 2D domain, unless within the 1D channel network, in which case they are also modelled in the 1D domain. Smaller culverts, where the flow width is typically less than one grid cell wide, have been modelled using 1D structures to provide flow through topographical features represented within the 2D domain.

3.3.6 Drainage Network

The study requires the modelling of the stormwater drainage system in the catchment areas upstream of the open channels. Council provided information on the existing drainage system where modelling was required. Data comprising pit/pipe locations, pit inlet type/dimensions and pipe sizes was received in a number of formats including GIS layers, survey details and DRAINS model files. These sources were used to build the necessary details of the stormwater pipe network into the TUFLOW model. Pipe sizes were generally available for most of the drainage network. Invert levels were taken from the provided data where available. Where invert levels were not available, they were estimated from the DEM, by assuming a minimum cover of 600mm from the known pipe size.

The pipe network, represented as a 1D layer in the TUFLOW model, is dynamically linked to the 2D domain at specified pit locations for inflow and surcharging, or directly to the 1D open channekl network where appropriate. An example of a typical representation is shown in Figure 3-4.The figure shows the pipes invert and obvert levels relative to the ground surface level.

3.3.7 Boundary Conditions

The catchment runoff is determined through the hydrological model and is applied to the TUFLOW model as flow vs. time inputs. These are applied at the upstream modelled drainage limits and also as distributed inflows along the modelled drainage alignments. For most sub-catchments with modelled stormwater drainage the hydrological model inflows are applied directly to the 1D pipe network and will surcharge to the 2D surface representation when pipe full capacity is exceeded. This assumes that there is sufficient pit capture within the drainage design to reach pipe full capacity, which is usually the case. For sub-catchment areas containing no stormwater drainage the catchment runoff is applied directly to the 2D domain, being distributed into the corresponding flow path or storage area.

The downstream model limit corresponds to the water level in the lower reaches of Cooks River and Botany Bay. Given its proximity to Botany Bay, the water levels in the lower reaches of Cooks River are essentially equivalent to that in Botany Bay. The adopted water levels for the downstream boundary condition for the calibration and design events are discussed in Chapter 4 and Chapter 5 respectively.

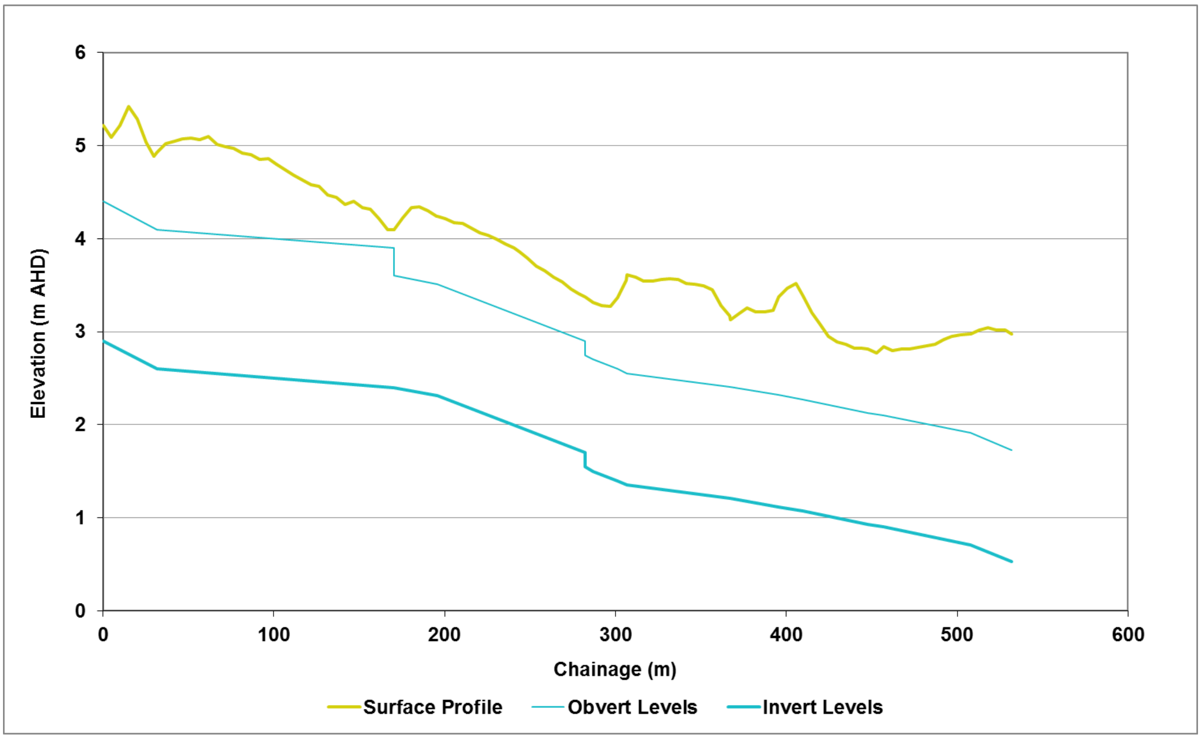


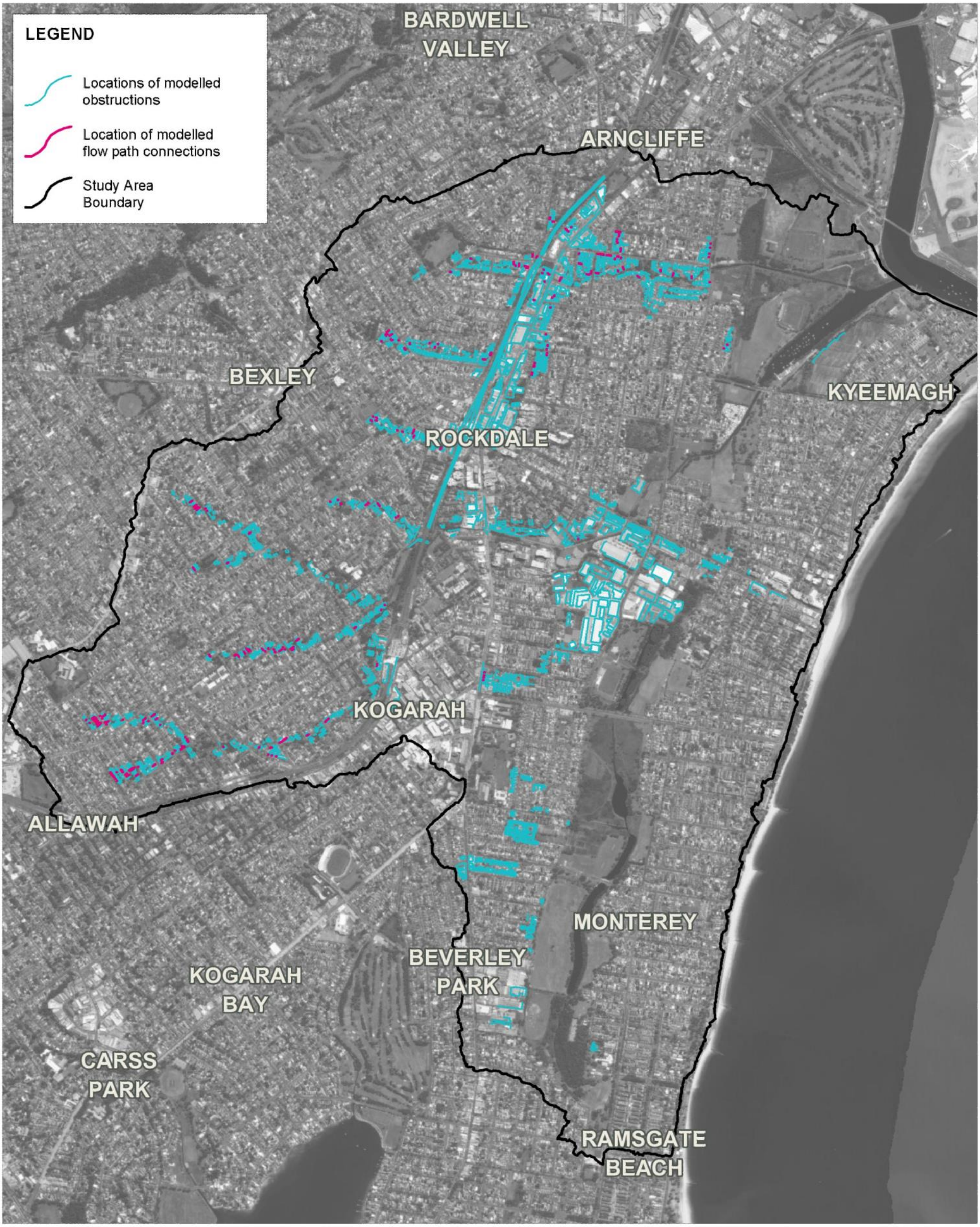
Figure 3-4 Example Drainage Line Long Section

3.3.8 Major Flowpath Representation

The adopted modelling approach serves to model the major overland flow paths of the Spring Street Drain, Muddy Creek and Scarborough Ponds catchments, utilising the best available data. Along these flow path alignments significant investment in model development has been undertaken to best represent the complex nature of hydraulic controls typical of the urban flood environment.

The process for model development along the overland flow paths has been to assess preliminary model outputs in the context of urban features that may influence or control the progression of flooding as it progresses downstream from the elevated areas of the upper catchment. The LiDAR elevation data typically provides a reasonable representation of the natural gully lines and their associated floodplains. However, local controls such as buildings, walls, gates and alleys can serve to alter the course of the natural catchment runoff. This can exacerbate flooding in some locations or even divert the preferred flood flow path to an alternative alignment.

Each modelled flood flow path has been verified in conjunction with the LiDAR elevation data, site visit notes, aerial photography and Google Street View imagery to incorporate local hydraulic controls into the TUFLOW model where appropriate. Much of this development involves the inclusion of brick and/or concrete walls as barriers to the progression of catchment runoff. Many of these obstructions are beyond the available resolution provided by the TUFLOW model grid. Therefore representative connections for flood waters have been provided through the obstructions using embedded 1D elements. This enables flood waters to progress downstream, rather than becoming “trapped” upstream of features such as walls. Such model inclusions are typically narrow alleys between buildings or gates through wall alignments. Other obstructions less sturdy in nature (such as wooden or Colorbond fences) have been omitted, as they typically fail when flood waters build on the upstream side. The distribution of the hydraulic controls developed for the TUFLOW model along the major flowpath alignments is presented in Figure 3-5. These developments have been guided by the available model calibration data where applicable.



Title:
Distribution of Modelled Hydraulic Controls

Figure: 3-5

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4 Model Calibration and Validation

4.1 Selection of Calibration and Validation Events

The selection of suitable historical events for calibration of computer models is largely dependent on available historical flood information. Ideally the calibration and validation process should cover a range of flood magnitudes to demonstrate the suitability of a model for the range of design event magnitudes to be considered.

Through consultation with Council a set of flood events were identified as being suitable for use in the model calibration and validation process. These are events of a reasonable flood magnitude, for which there are observed flood data available for comparison with the model performance. The principal event selected for model calibration is the April 1998 event, as this is the flood event with the most intense rainfall of recent years. There is also a wealth of observed flood data available.

The February 1993 and October 2014 flood events have been selected for model validation. The October 2014 event was almost as intense as the April 1998 storm, but the April 1998 event had a greater total rainfall. It is therefore the largest recent flood event in the lower-lying areas of the catchment such as the Scarborough Ponds. The February 1993 event was not as significant as the other two, but still has some useful flood data available for comparison.

4.2 April 1998 Model Calibration

4.2.1 Calibration Data

4.2.1.1 Rainfall Data

Given the lack of rainfall data within the study area (there is only one gauge at Kyeemagh Bowling Club) and the often high spatial variability of short duration, intense rainfall, it is difficult to determine a meaningful estimate of rainfall variability for the study area. However, there are a number of gauges situated around the study area that can be analysed to understand the likely range of rainfall intensities experienced within the catchment. Six pluvio gauges have been considered in this analysis and are summarised in Table 4-1.

Table 4-1 April 1998 Event Pluvio Gauges

Gauge Reference	Location	Approximate Locality from the Centre of Study Area
566091	Kyeemagh Bowling Club	1.8km to the NE
66037	Sydney Airport AMO	3.4km to the NE
566090	Carss Park Bowling Club	3.9km to the SW
566026	Marrickville SPS	4.3km to the N
566028	Mascot Bowling Club	5.7km to the NE
566047	Mortdale Bowling Club	6.3km to the SW

Analysis of the recorded rainfall at these sites shows that two bursts of heavy rainfall occurred over a two-to-three hour period on 10th April 1998, separated by a period of around ten hours. The first burst began at around 06:00 and was largely to the north of the study area. The second burst followed at around 16:00 and was largely to the south of the study area. Different locations within the study area may have received heavy rainfall coincident with the timing of the records to the north, to the south, or a combination of the two.

In order to gain an appreciation of the relative intensity and magnitude of the April 1998 event, the recorded rainfall depth for various durations within the storm is compared with the Intensity Frequency Duration (IFD) data for the catchment. The AR&R is in the process of revising the design flood estimate guidelines, and have released updated 2013 IFDs based on the extended history of rainfall records available since they were first developed in 1987. However, these are currently to be used for sensitivity purposes only and not adopted for design flood estimation, as their appropriate use is linked to the adopted design temporal rainfall patterns and design losses (the revision of which is still underway). Design IFD rainfall curves were obtained from AR&R (2001) based on the 1987 and 2013 datasets. Figure 4-1 presents the recorded April 1998 rainfall intensities against both the 1987 IFDs and 2013 IFDs, for comparison.

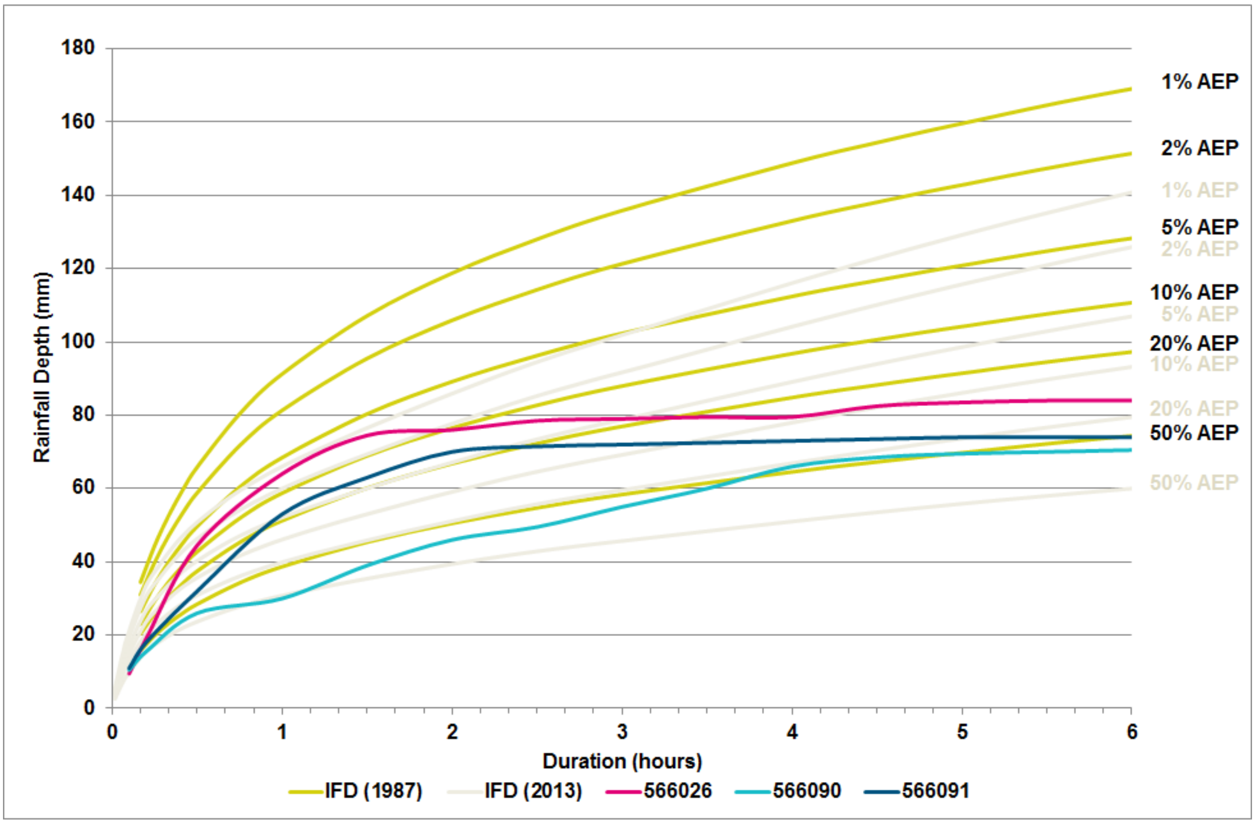


Figure 4-1 Comparison of Recorded April 1998 Rainfall with IFD Relationships

The IFD curves from the 1987 AR&R and 2013 revision become similar at longer durations such as the 12-hour, but for the shorter durations presented in Figure 4-1 the rainfall depths have been significantly reduced in the 2013 revision. The Kyeemagh Bowling Club gauge (566091) has been presented as it is the only gauge situated within the catchment and is likely to be most representative of the typical rainfall conditions experienced across the study area. The Carss Park Bowling Club (566090) and Marrickville SPS (566026) gauges provide the lowest and highest recorded rainfall intensities in the vicinity of the catchment and show the likely range of rainfall conditions experienced across the study area. The magnitude of the storm peaks at around the 1.5-hour duration, with an expected catchment rainfall depth of around 63mm and an expected range of between 39mm and 75mm.

The XP-RAFTS and TUFLOW models were simulated using the recorded data from the Kyeemagh Bowling Club gauge (566091).

4.2.1.2 Flood Data

As there are no stream gauges situated within the catchment, the verification of model performance against that which is expected is reliant upon the comparison with observed flood information obtained during or after the event. This can include observations of the main flow path alignments or specific peak flood levels read from flood marks.

For the April 1998 event there is an extensive set of observed data available. Most of this can only be used as an indication that significant flooding occurred at particular locations. This can be checked against the significant modelled flow paths to ensure a correct correlation. For some locations the available description of flooding combined with LiDAR elevation survey enables a threshold level that was exceeded. In some cases even a reasonable estimate of the actual peak flood level is able to be determined. Council also provided data for a few locations where peak flood levels have been surveyed from flood marks. The distribution of this data and level of detail obtained is presented in Figure 4-2.

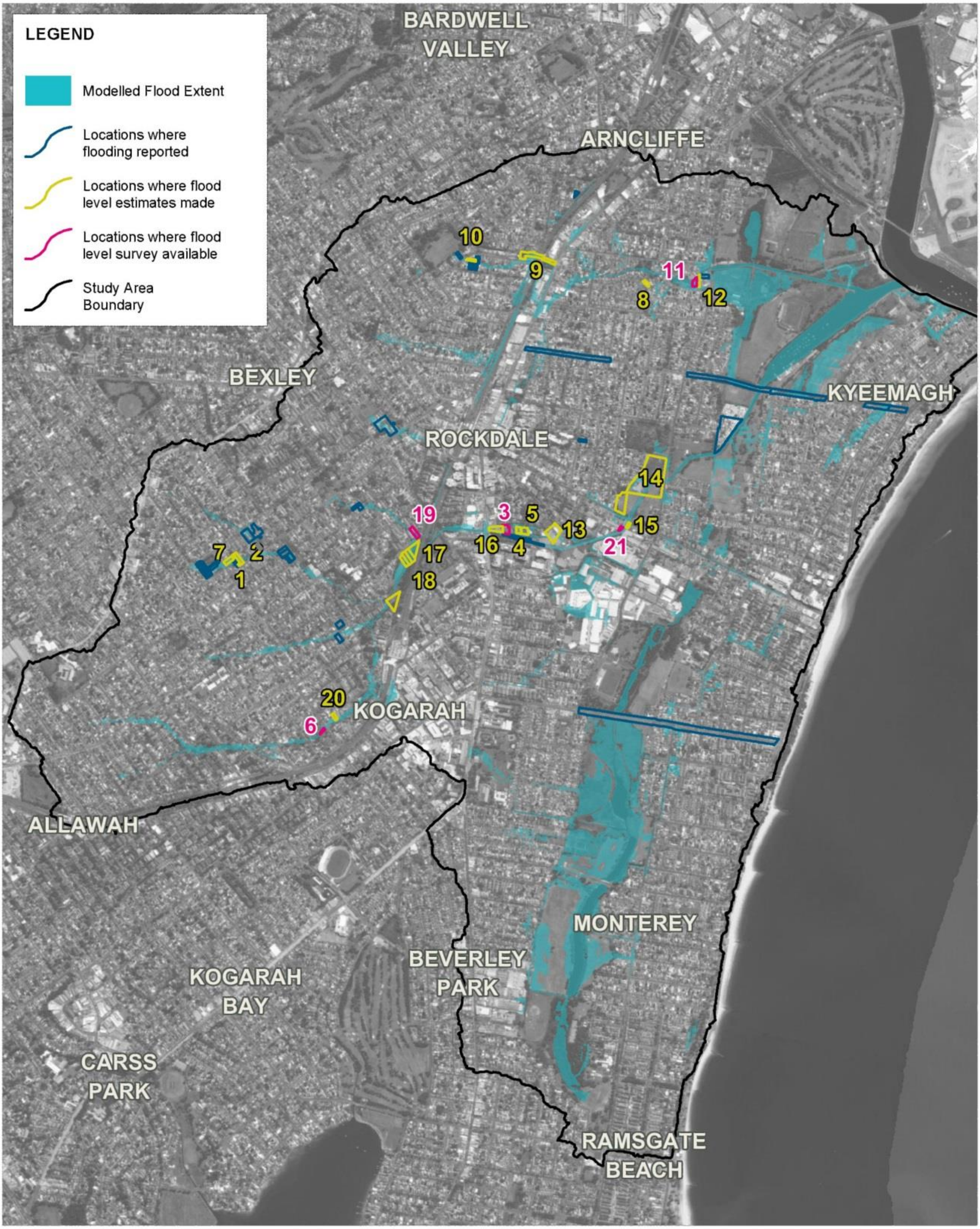
4.2.2 Downstream Boundary Condition

In most instances the downstream water level conditions will not be critical in determining upstream flood levels. However, for completeness the available recorded water level conditions at Tempe Bridge on the Cooks River have been obtained and used to represent the tailwater conditions within the model. This peaks at a level of just over 1.0m AHD.

4.2.3 Observed and Simulated Flood Behaviour

From Figure 4-2 it can be seen that there is a good correlation between the locations at which significant flooding was observed and the alignment of the major flood flow paths in the TUFLOW model results. For locations where some form of flood level estimation was possible these have been compared to the modelled flood conditions from the simulation of the Kyeemagh Bowling Club rainfall data (gauge 566091). Results from the 50% AEP and 5% AEP design rainfall (1987 IFDs) simulations are also presented, as these represent the likely bounds of expected catchment flood conditions during the event. This comparison of observed and modelled flood levels is presented in Table 4-2.

It can be seen from Table 4-2 that where reasonable estimates of the peak flood level can be made from the observed data, the modelled flood level is typically within 0.2m of this estimate. This indicates a good calibration considering the relative bounds of uncertainty.



Title:

Distribution of Observed Flood Data Available for the April 1998 Event

Figure: 4-2

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Approx. Scale

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Table 4-2 Comparison of Observed and Modelled April 1998 Flood Levels (m AHD)

Location (refer to Figure 4 2)	Observed	Modelled 50% AEP	Modelled Apr-98	Modelled 5% AEP
1	>27.6	28.0	28.0	28.4
2	>27.2	28.0	28.0	28.3
3*	3.35	N/A	N/A	3.1
4*	>3.0	2.9	3.0	3.1
5*	>3.0	2.9	3.0	3.1
6	19.67	N/A	19.59	19.93
7	>29.7	30.3	30.3	30.6
8	>3.3	N/A	N/A	3.4
9	>10.3	10.0	10.7	12.3
10	~19.0	18.9	18.9	19.1
11	2.44	2.15	2.22	2.39
12	~2.3	2.1	2.2	2.4
13*	~2.6	N/A	2.2	2.8
14	~2.3	N/A	N/A	2.3
15	~2.6	1.9	2.0	2.2
16	>3.7	4.2	4.5	4.6
17	~7.4	6.8	7.2	7.6
18	~7.5	6.9	7.3	7.8
19	7.86	6.92	7.49	7.81
20	~18.0	17.8	18.0	18.0
21	2.41	1.91	2.04	2.27

** The Strand Levee was constructed post-1998 but was included in the model simulation.*

4.3 February 1993 Model Validation

4.3.1 Validation Data

4.3.1.1 Rainfall Data

Given the lack of rainfall data within the study area (there is only one gauge at Kyeemagh Bowling Club) and the often high spatial variability of short duration, intense rainfall, it is difficult to determine a meaningful estimate of rainfall variability for the study area. However, there are a number of gauges situated around the study area that can be analysed to understand the likely range of rainfall intensities experienced within the catchment. Seven pluvio gauges have been considered in this analysis and are summarised in Table 4-3.

Analysis of the recorded rainfall at these sites shows that the intense rainfall occurred within a two hour period from around 08:00 on 17th February 1993.

Table 4-3 February 1993 Event Pluvio Gauges

Gauge Reference	Location	Approximate Locality from the Centre of Study Area
566091	Kyeemagh Bowling Club	1.8km to the NE
66037	Sydney Airport AMO	3.4km to the NE
566062	Bexley Bowling Club	3.7km to the NW
566090	Carss Park Bowling Club	3.9km to the SW
566026	Marrickville SPS	4.3km to the N
566028	Mascot Bowling Club	5.7km to the NE
566047	Mortdale Bowling Club	6.3km to the SW

In order to gain an appreciation of the relative intensity and magnitude of the February 1993 event, the recorded rainfall depth for various durations within the storm is compared with design IFD rainfall curves obtained from AR&R (2001) based on the 1987 and 2013 datasets. Further discussion on both these IFD datasets is provided in Section 4.2.1.1. Figure 4-3 presents the recorded February 1993 rainfall intensities against both the 1987 and 2013 IFDs.

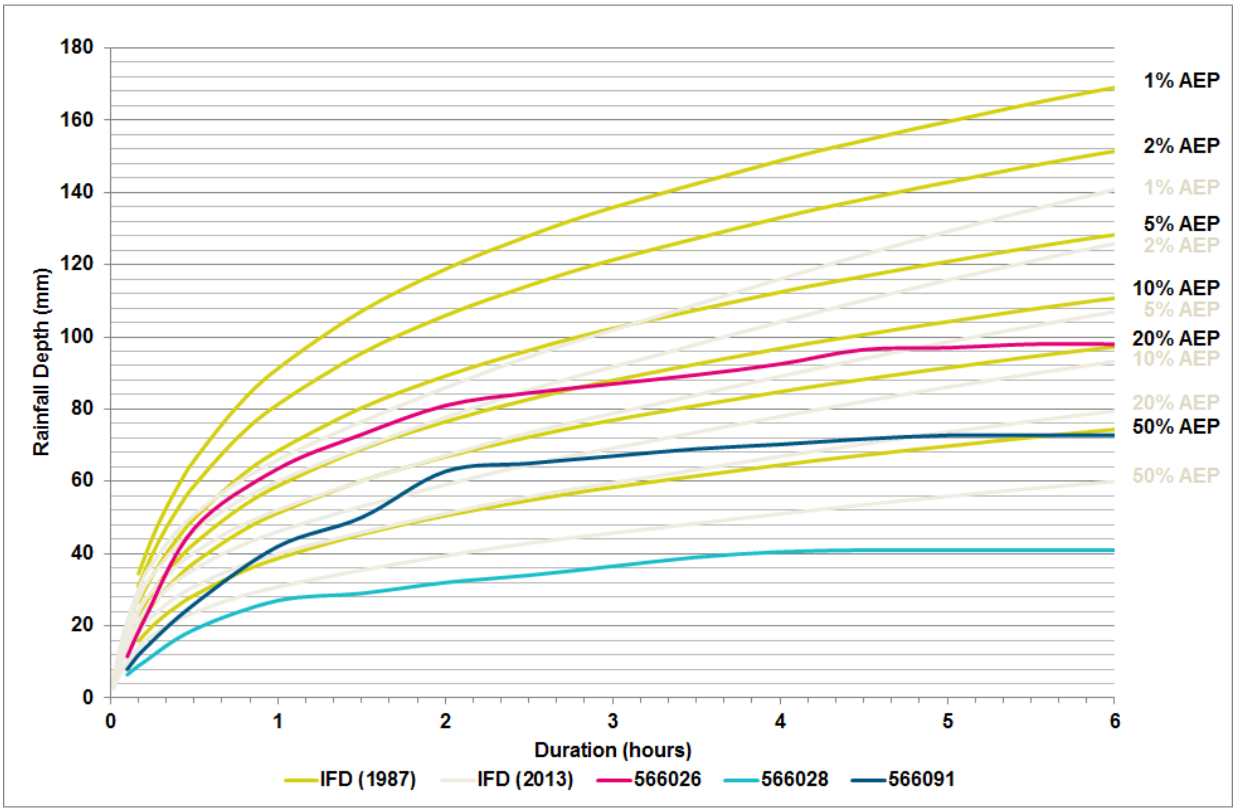


Figure 4-3 Comparison of Recorded February 1993 Rainfall with IFD Relationships

The IFD curves from the 1987 AR&R and 2013 revision become similar at longer durations such as the 12-hour, but for the shorter durations presented in Figure 4-3 the rainfall depths have been significantly reduced in the 2013 revision. The Kyeemagh Bowling Club gauge (566091) has been presented as it is the only gauge situated within the catchment and is likely to be most representative of the typical rainfall conditions experienced across the study area. The Mascot Bowling Club (566028) and Marrickville SPS (566026) gauges provide the lowest and highest recorded rainfall intensities in the vicinity of the catchment and show the likely range of rainfall conditions experienced across the study area. The magnitude of the storm peaks at around the 2-hour duration, with an expected catchment rainfall depth of around 63mm and an expected range of between 32mm and 81mm.

The XP-RAFTS and TUFLOW models were simulated using the recorded data from the Kyeemagh Bowling Club gauge (566091).

4.3.1.2 Flood Data

As there are no stream gauges situated within the catchment the verification of model performance against that which is expected is reliant upon the comparison with observed flood information obtained during or after the event. This can include observations of the main flow path alignments or specific peak flood levels read from flood marks.

For the February 1993 event there is a limited set of observed data available. However, these are flood marks that have been surveyed and so provide a reasonably accurate indication of the peak flood level during the event. The distribution of this data is presented in Figure 4-4.

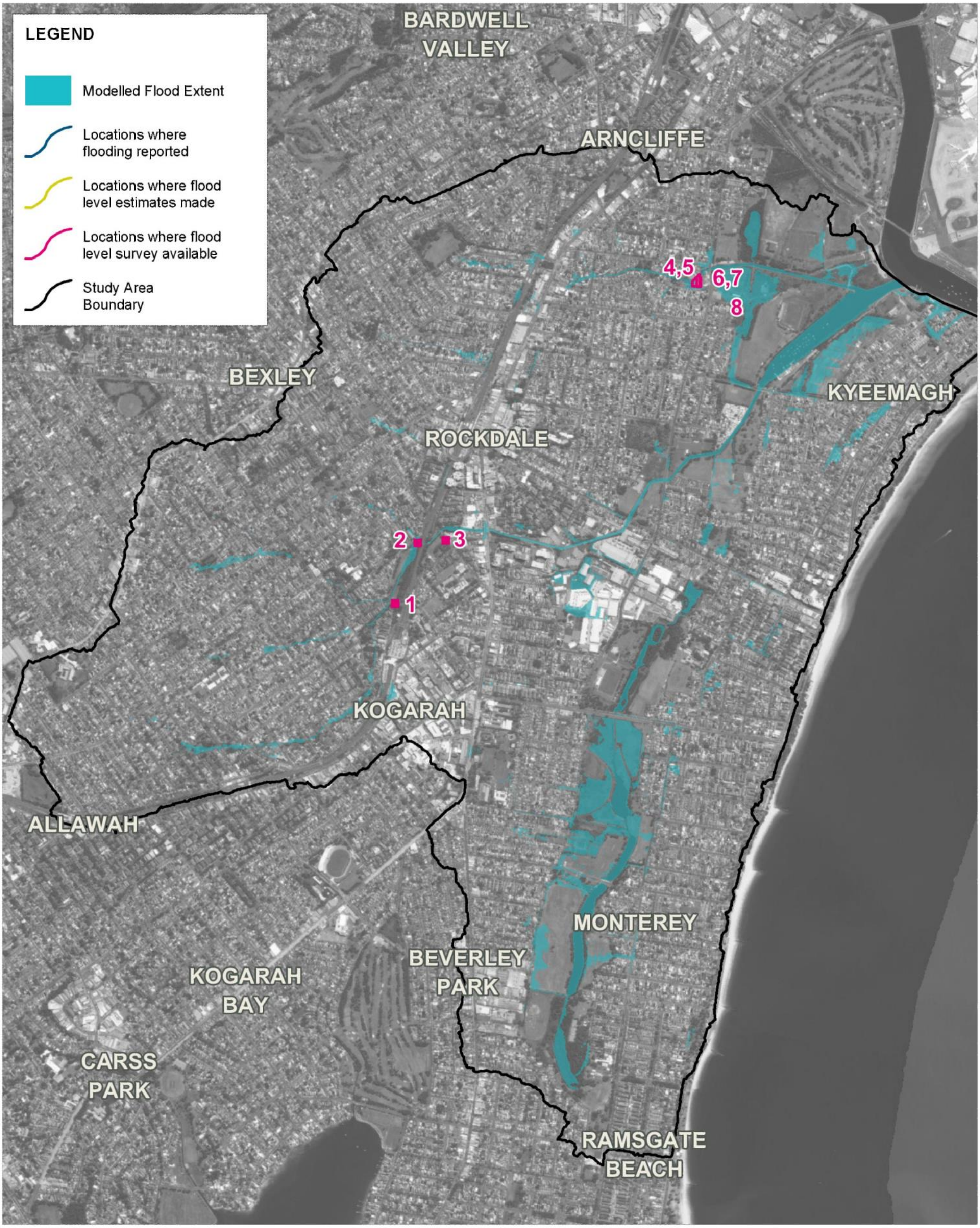
4.3.2 Downstream Boundary Condition

In most instances the downstream water level conditions will not be critical in determining upstream flood levels. However, for completeness the available recorded water level conditions in Botany Bay have been obtained and used to represent the tailwater conditions within the model. This peaks at a level of around 0.5m AHD.

4.3.3 Observed and Simulated Flood Behaviour

For locations where the flood mark survey is available the levels have been compared to the modelled flood conditions from the simulation of the Kyeemagh Bowling Club rainfall data (gauge 566091). This produces flood levels slightly lower than the 50% AEP design flood event (1987 IFDs). This comparison of observed and modelled flood levels is presented in Table 4-4.

It can be seen from Table 4-4 that there is typically a good match between modelled and surveyed flood levels along Muddy Creek (locations 1, 2 and 3). However, at the other locations on Spring Street Drain the modelled levels are typically around 0.6m lower than those that were surveyed after the flood. If the event was locally in the order of a 10% AEP to 5% AEP (as recorded at the Marrickville SPS rain gauge) then the flood levels would have been in the order of 0.4m higher than those modelled using the Kyeemagh data.



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Distribution of Observed Flood Data Available for the February 1993 Event

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Table 4-4 Comparison of Observed and Modelled February 1993 Flood Levels (m AHD)

Location (refer to Figure 4-4)	Observed	Modelled Feb-93
1	8.26	8.14
2	6.29	6.58
3	4.55	4.39
4	2.54	2.04
5	2.56	2.00
6	2.40	1.94
7	2.58	1.90
8	2.70	2.04

4.4 October 2014 Model Validation

4.4.1 Validation Data

4.4.1.1 Rainfall Data

Given the lack of rainfall data within the study area (there is only one gauge at Kyeemagh Bowling Club) and the often high spatial variability of short duration, intense rainfall, it is difficult to determine a meaningful estimate of rainfall variability for the study area. For the October 2014 event only the pluvio record from Marrickville Golf Club (66036) has been made available, which is around 4km north of the study area. However, there are a number of additional daily rainfall gauges situated around the catchment that show a reasonably consistent rainfall depth. Marrickville recorded 124mm, with the range of other nearby stations being between 107mm and 143mm, with an average of 123mm, as presented in Table 4-5.This suggests that the rainfall was reasonably evenly distributed and so the record from Marrickville should provide a reasonable estimate for the rainfall conditions across the study area.

Analysis of the recorded rainfall at Marrickville shows that the intense rainfall occurred predominantly over a three hour period from around 20:30 on 14th October 2014.

Table 4-5 October 2014 Event Rainfall Gauges

Gauge Reference	Location	Approximate Locality from the Centre of Study Area	Rainfall (mm)
66037	Sydney Airport AMO	3.4km to the NE	107
66036	Marrickville Golf Club	3.8km to the N	124
66058	Sans Souci Public School	4.5km to the S	143
66194	Canterbury Racecourse AWS	6.1km to the NW	121
66181	Oatley (Woronora Parade)	6.3km to the SW	127
66148	Peakhurst Golf Club	7.2km to the W	119

In order to gain an appreciation of the relative intensity and magnitude of the February 1993 event, the recorded rainfall depth for various durations within the storm is compared with design IFD rainfall curves obtained from AR&R (2001) based on the 1987 and 2013 datasets. Further discussion on both these IFD datasets is provided in Section 4.2.1.1. Figure 4-5 presents the recorded October 2014 rainfall intensities against both the 1987 and 2013 IFDs.

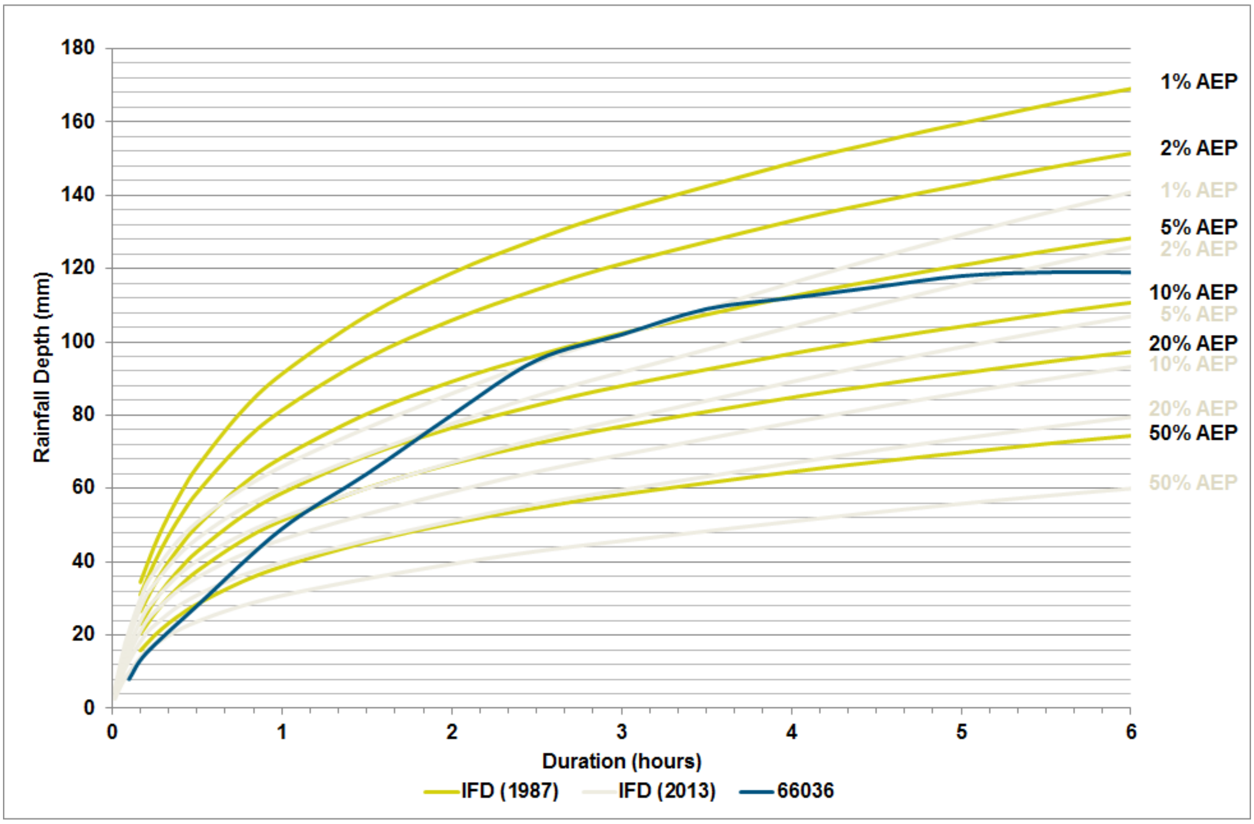


Figure 4-5 Comparison of Recorded October 2014 Rainfall with IFD Relationships

The IFD curves from the 1987 AR&R and 2013 revision become similar at longer durations such as the 12-hour, but for the shorter durations presented in Figure 4-5 the rainfall depths have been significantly reduced in the 2013 revision. The Marrickville Golf Club gauge (66036) has been presented as it is the only gauge for which pluvio rainfall data has been made available and should be typical of the rainfall conditions experienced across the study area. The magnitude of the storm peaks at around the 3-hour duration, with an expected catchment rainfall depth of around 102mm.

The XP-RAFTS and TUFLOW models were simulated using the recorded data from the Marrickville Golf Club gauge (66036).

4.4.1.2 Flood Data

As there are no stream gauges situated within the catchment the verification of model performance against that which is expected is reliant upon the comparison with observed flood information obtained during or after the event. This can include observations of the main flow path alignments or specific peak flood levels read from flood marks.

For the October 2014 event there is a limited set of observed data available from incidents reported to Council. However, some of these include sufficient detail to enable a reasonable estimate of the peak flood level to be derived, in conjunction with the LiDAR elevation data. The distribution of this observed data is presented in Figure 4-6.

4.4.2 Downstream Boundary Condition

In most instances the downstream water level conditions will not be critical in determining upstream flood levels. Therefore a fixed tailwater level of 1.1m AHD was adopted in the Cooks River.

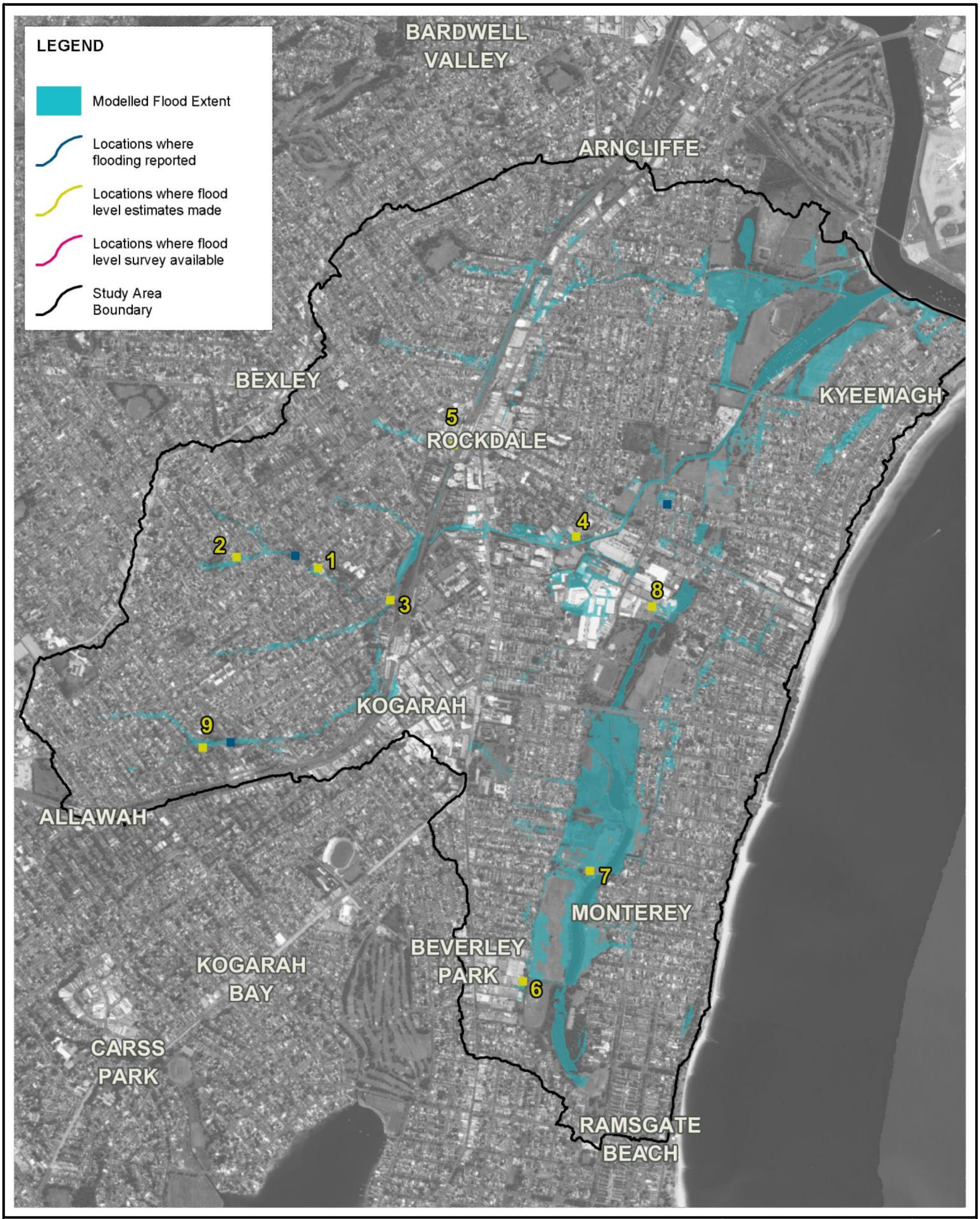
4.4.3 Observed and Simulated Flood Behaviour

From Figure 4-6 it can be seen that there is a good correlation between the locations at which significant flooding was observed and the alignment of the major flood flow paths in the TUFLOW model results. For locations where some form of flood level estimation was possible these have been compared to the modelled flood conditions from the simulation of the Marrickville Golf Club rainfall data (gauge 66036). This comparison of observed and modelled flood levels is presented in Table 4-6.

It can be seen from Table 4-6 that where reasonable estimates of the peak flood level can be made from the observed data, the modelled flood level is typically within 0.1m of this estimate. This indicates a good calibration.

Table 4-6 Comparison of Observed and Modelled October 2014 Flood Levels (m AHD)

Location (refer to Figure 4-6)	Observed	Modelled Oct-14
1	~16.4	16.3
2	~30.2	30.3
3	~8.7	7.9
4	>2.5	3.0
5	>19.5	N/A
6	>2.3	2.3
7	~1.7	1.6
8	>2.1	2.1
9	>29.5	29.8



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Distribution of Observed Flood Data Available for the October 2014 Event

Figure: 4-6

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4.5 Conclusion

The model calibration process has involved the development of appropriate hydrologic and hydraulic computer models in order to best represent the flooding conditions within the study area, within the limitations of the available data. The models have been constructed using actual survey data where available and informed assumptions elsewhere. Standard model parameters have been adopted where required, which are consistent with accepted theory and experiences learnt from other modelled catchments of a similar nature.

Appropriate rainfall inputs have been developed for the models for the three calibration/validation events; February 1993, April 1998 and October 2014. The resultant model simulations have shown the adopted model configuration to perform well across a range of events, producing reasonable matches to observed flood level data where available.

The main consideration for the simulation of design flood events to take forward form the model calibration process is the significant difference between the design IFD estimates of the AR&R 1987 release and the current revision (2013 IFDs). The short duration events (including the 2-hour duration, which is the critical storm duration for much of the catchment aside from the Scarborough Ponds) have had their design rainfall intensities reduced significantly in the 2013 revision of AR&R. This has implications for the adopted design flood conditions, as it will directly impact on the modelled peak flood levels. The official position is that the 1987 IFDs should be adopted for design until the revision of design rainfall temporal patterns and losses is complete, as this go hand-in-hand with derivation of design flood flows from the corresponding design rainfall depths. However, the potential reduction of design rainfall intensities for design flood conditions warrants consideration for this study given the significant change.

Figure 4-7 presents the design rainfall IFD curves for Sydney Airport from the 1987 release of AR&R and those proposed under the current revision from 2013. To provide some additional context, a site-specific rainfall frequency analysis (RFA) has been conducted using the available pluvio rainfall gauge record at Sydney Airport (66037). There is a complete record of 50 years' data available at this site. This was used to derive annual maxima rainfall depths for various storm durations in order to produce an annual maxima series for analysis with the TUFLOW FLIKE extreme value analysis package.

Developed by Professor George Kuczera from the School of Civil Engineering at the University of Newcastle Australia, TUFLOW FLIKE is compliant with the recent major revision of industry guidelines for flood estimation, documented in the draft update of AR&R. The FLIKE analyses used a Bayesian inference method with the generalised extreme value (GEV) probability model.

The significantly lower design rainfall intensities of the 2013 revision are evident when compared to the 1987 curves, although the shape of the curves is consistent. The site-specific RFA exhibits a different curve shape, more closely matching the 2013 IFD curves for the more frequent design flood events but becoming more intense than the 1987 IFD curves for the less frequent design flood events.

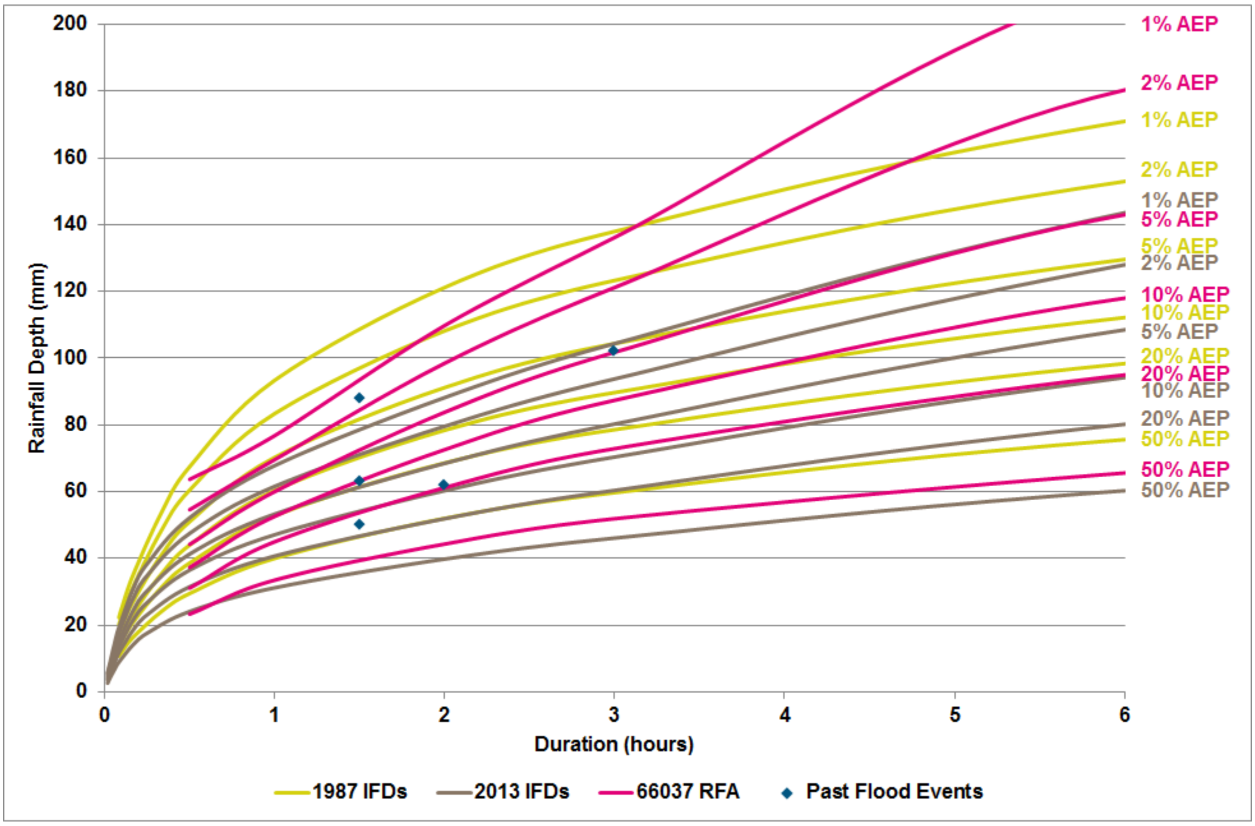


Figure 4-7 Comparison of Design Rainfall IFD Curves

It should be noted that with only 50 years of data available at the Sydney Airport gauge location, the rainfall frequency analysis will be much more reliable for the more frequent design flood events such as the 20% AEP than it will for the less frequent design flood events such as the 2% AEP. However, the analysis indicates that somewhere between the 1987 and 2013 design IFDs may be most appropriate for deriving design flood conditions for the Spring Street Drain, Muddy Creek and Scarborough Ponds catchments.

To demonstrate the impact of adopting different rainfall IFDs for the design flood events the relative intensities of some significant flood events of the past 50 years have been derived from the rainfall records and plotted on Figure 4-7 for comparison with the various design rainfall conditions. This information has also been summarised in Table 4-7.

Table 4-7 Summary of Design Rainfall IFDs with Past Events

Event	Equivalent Design Duration (h)	Rainfall Depth in Period (mm)	Design Equivalent (1987 IFDs)	Design Equivalent (2013 IFDs)	Design Equivalent (RFA)
Mar-75	~1.5	~88	~2-5% AEP	~0.5% AEP	~1-2% AEP
Mar-83	~1.5	~50	~20-50% AEP	~10-20% AEP	~20-50% AEP
Feb-93	~2.0	~62	~20-50% AEP	~10% AEP	~20% AEP
Apr-98	~1.5	~63	~20% AEP	~5% AEP	~10% AEP
Oct-14	~3.0	~102	~5% AEP	~1% AEP	~5% AEP

It can be seen from Table 4-7 that the relative design event magnitude of the five listed past flood events changes significantly depending on the adopted design rainfall conditions:

- The magnitude of the March 1975 flood event ranges from around a 2-5% AEP using the 1987 IFDs to around a 0.5% AEP using the 2013 IFDs;
- The magnitude of the March 1983 flood event ranges from around a 20-50% AEP using the 1987 IFDs or site-specific RFA to around a 10-20% AEP using the 2013 IFDs;
- The magnitude of the February 1993 flood event ranges from around a 20-50% AEP using the 1987 IFDs to around a 10-20% AEP using the 2013 IFDs;
- The magnitude of the April 1998 flood event ranges from around a 20% AEP using the 1987 IFDs to around a 5% AEP using the 2013 IFDs; and
- The magnitude of the October flood event ranges from around a 5% AEP using the 1987 IFDs or site-specific RFA to around a 1% AEP using the 2013 IFDs.

Alternatively, when considered from the perspective of each set of design rainfall conditions, over the past 50 years the following design flood magnitude thresholds have been met or exceeded:

- Using the 1987 IFDs there have been two floods greater than or equal to the 10% AEP, two floods greater than or equal to the 5% AEP and no floods greater than or equal to the 2% AEP;
- Using the 2013 IFDs there have been four floods greater than or equal to the 10% AEP, three floods greater than or equal to the 5% AEP and two floods greater than or equal to the 2% AEP; and
- Using the site-specific RFA there have been three floods greater than or equal to the 10% AEP, two floods greater than or equal to the 5% AEP and one flood greater than or equal to the 2% AEP.

5 Design Flood Conditions

5.1 Introduction

Design floods are hypothetical floods used for planning and floodplain management investigations. They are based on having a probability of occurrence specified as Annual Exceedance Probability (AEP) expressed as a percentage.

Refer to Table 5-1 for a definition of AEP.

Table 5-1 Design Flood Terminology

AEP	Comment
0.5%	A hypothetical flood or combination of floods which represent the worst case scenario with a 0.5% probability of occurring in any given year.
1%	As for the 0.5% AEP flood but with a 1% probability.
2%	As for the 0.5% AEP flood but with a 2% probability.
5%	As for the 0.5% AEP flood but with a 5% probability.
10%	As for the 0.5% AEP flood but with a 10% probability.
20%	As for the 0.5% AEP flood but with a 20% probability.
50%	As for the 0.5% AEP flood but with a 50% probability.
Extreme Flood / PMF[1]	A hypothetical flood or combination of floods which represent an extreme scenario.

1 A PMF (Probable Maximum Flood) is not necessarily the same as an Extreme Flood.

In determining the design floods it is necessary to take into account:

- Design rainfall parameters (rainfall depth, temporal pattern and spatial distribution). These inputs drive the hydrological model from which design flow hydrographs will be extracted as inputs to the hydraulic model;
- Design downstream ocean boundary levels; and
- The impact of future climate change on ocean levels and catchment inflows.

In accordance with Council's brief, the design events to be simulated include the 50% AEP, 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 0.5% AEP and PMF event. The 1% AEP flood is generally used as a reference flood for development planning and control for residential development.

The adopted storm durations are discussed in Section 5.2.5. The adopted ocean downstream boundary conditions are discussed in Section 5.3.

5.2 Design Rainfall

Design rainfall parameters are derived from standard procedures defined in AR&R (2001) which are based on statistical analysis of recorded rainfall data across Australia. The methods were first presented in 1987 and therefore only consider rainfall data available up to this time. The derivation of location specific design rainfall parameters (e.g. rainfall depth and temporal pattern) for the study catchment is presented below.

5.2.1 Rainfall Depths

Design rainfall depth is based on the generation of intensity-frequency-duration (IFD) design rainfall curves utilising the procedures outlined in AR&R (2001). These curves provide rainfall depths for various design magnitudes (up to the 1% AEP) and for durations from 5 minutes to 72 hours.

The Probable Maximum Precipitation (PMP) is used in deriving the Probable Maximum Flood (PMF) event. The theoretical definition of the PMP is “the greatest depth of precipitation for a given duration that is physically possible over a given storm area at a particular geographical location at a certain time of year” (AR&R, 2001). The ARI of a PMP/PMF event ranges between 10^4 and 10^7 years and is beyond the “credible limit of extrapolation”. That is, it is not possible to use rainfall depths determined for the more frequent events (1% AEP and less) to extrapolate the PMP. The PMP has been estimated using the Generalised Short Duration Method (GSDM) derived by the Bureau of Meteorology.

Table 5-2 shows the design rainfall intensities calculated for the Muddy Creek catchment from the methods first presented by AR&R in 1987. Discussion regarding current use of the 2013 IFDs and their differences against the 1987 IFDs was included in Section 4.5

Table 5-2 Average Design Rainfall Intensities (mm/hr)

Duration	Design Rainfall Intensities (mm/hr)						
	50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP
0.5	56.9	74.9	85.6	99.4	118	132	145
1	38.7	51.2	58.7	68.4	81.3	91.2	100
2	25.3	33.4	38.3	44.6	53.0	59.4	65.3
3	19.5	25.7	29.3	34.1	40.4	45.3	49.8
6	12.4	16.2	18.5	21.4	25.2	28.2	31.1
9	9.56	12.4	14.1	16.3	19.2	21.5	23.7
12	7.97	10.4	11.7	13.6	16.0	17.8	19.6
18	6.19	8.03	9.10	10.5	12.4	13.8	15.2
24	5.18	6.71	7.62	8.80	10.3	11.5	12.7

5.2.2 Areal Reduction Factor

The design rainfall intensities derived according to AR&R are applicable strictly to a point location. For larger catchments, it is not realistic to assume that the same rainfall intensity can be maintained over the entire area and an areal reduction factor (ARF) is typically applied. The adopted methodology for determining ARFs is that proposed in the Review of ARFs Final Report (AR&R Revision Project 2, 2013).

Under the revised AR&R guidelines appropriate ARFs are calculated separately for both long duration events (18 hours or greater) and short duration events (18 hours or less). These calculations incorporate the catchment area, storm duration, event AEP and a set of published parameters which vary according to the geographical location of the study area. The Muddy Creek catchment is situated within the NSW GSAM zone. An ARF of 0.95 was adopted for all design events as:

- The calculated ARF for an area of 3km^2 (similar to that of Muddy Creek upstream of the railway) is 0.95 for the 2-hour duration storms; and
- The calculated ARF for an area of 13km^2 (similar to that of the entire study area) is 0.95 for the 9-hour duration storms.

5.2.3 Temporal Patterns

The IFD data presented in Table 5-2 provides the average intensity (or total depth) that occurs over a given storm duration. Temporal patterns are required to define what percentage of the total rainfall depth occurs over a given time interval throughout the storm duration. The temporal patterns adopted in the current study are based on the standard patterns presented in AR&R (2001).

5.2.4 Rainfall Losses

The hydrologic model parameters adopted for the design floods were based on the initial and continuing loss model, with a continuing loss of 2.5mm/h as recommended in AR&R (2001). For the initial loss AR&R recommends values between 10mm and 35mm for eastern NSW.

An initial loss of 10mm and continuing loss of 2.5mm/h was adopted for the pervious portions of the Muddy Creek catchment, with an initial loss of 2mm and continuing loss of 0mm/h being adopted for the impervious areas.

5.2.5 Critical Storm Duration

The critical duration is the storm duration for a given event magnitude that provides for the peak flood conditions at the location of interest. For example, small catchments are more prone to flooding during short duration storms, while for large catchments longer durations will be more critical.

A range of storm durations were modelled in order to identify the critical storm duration for design event flooding in the catchment. The duration producing the highest flow rate out of the hydrological model may not necessarily result in the peak flood level in the hydraulic model as catchment characteristic come into play. Storage effects of floodplain topography may attenuate the flood wave as it moves down the catchment. Durations producing a greater volume of floodwater may result in higher flood levels, as opposed to the duration that produces the peak flow rate.

The 1% AEP flood event was run for all durations to determine the critical duration for each location in the study area. The critical duration for the upper reaches was found to be the 2 hour storm, whereas for lower catchment areas the 9 hour storm was critical. Adopting both the 2 hour and 9 hour storm durations provided the critical condition across most of the modelled area. In locations where the 2 hour or 9 hour storm is not the critical duration it is expected that the difference between the actual critical duration and that modelled would be minimal.

The flood conditions of the Scarborough Ponds system are driven by total catchment runoff volume rather than peak flows and as such display a significantly different critical storm duration to the rest of the study area. The critical storm duration for this peak storage volume is typically between 30-hours and 48-hours, depending on the adopted design conditions. The peak flood levels attained within Scarborough Ponds can be readily estimated using XP-RAFTS, through the representation of the stage-storage relationship and piped drainage discharge within the retarding basin module.

The PMP has been estimated using the Generalised Short Duration Method (GSDM) derived by the Bureau of Meteorology. The critical storms using this method were found to be the 15-min, 45-min and 90-min durations for the upper reaches, middle reaches and lower catchment areas, respectively.

5.3 Design Ocean Boundary

Design ocean boundaries for use in flood risk assessments are recommended by the Flood Risk Management Guide (OEH, 2015), where the recommended design ocean water levels have been determined based on long term records from Fort Denison in Sydney Harbour. The design levels include the following considerations:

- Barometric pressure set up of the ocean surface due to the low atmospheric pressure of the storm;
- Wind set up due to strong winds during the storm “piling” water upon the coastline;
- Astronomical tide, particularly the HHWS(SS); and
- Wave set up.

OEH (2015) recommends different design ocean peak water levels are to be adopted based on the type of ocean entrance. Type A entrances are subject to little ocean tide attenuation and are not influenced by wind and wave set up, e.g. Botany Bay. Type B estuaries are typically open but may be affected by shoaling and may have some potential for wave set up. Type C estuaries are prone to heavy shoaling and often close completely (also known as Intermittently Closed and Open Lakes and Lagoons (ICOLLS)). Peak design ocean water levels for each of the different entrance types for locations south of Crowdy Head are presented in Table 5-3. The different peak levels reflect the degree of influence of wave set up applicable to the various types of entrances.

Given the close proximity of the Muddy Creek outlet at Cooks River to Botany Bay, it is appropriate to adopt the design flood levels for Entrance Type A as the downstream boundary for this study.

Table 5-3 Design Peak Ocean Water Levels (OEH, 2015) for Various Entrance Types, located South of Crowdy Head

Ocean Event	Peak Ocean Water Level (m AHD)		
	Entrance Type A	Entrance Type B	Entrance Type C
5% AEP	1.4	1.9	2.35
1% AEP	1.45	2.0	2.55

For simplicity a fixed level downstream boundary has been adopted, to represent the catchment runoff being coincident with the peak ocean level.

The ocean boundary level recommended by OEH (2015) for each design catchment flood scenario is presented in Table 5-4 and has been adopted for design simulations in this study.

Table 5-4 Design Peak Ocean Water Levels

Catchment Event	Ocean Event	Peak Ocean WL (m AHD)
50% AEP	HHWS	1.1
20% AEP	HHWS	1.1
10% AEP	HHWS	1.1
5% AEP	HHWS	1.1
2% AEP	5% AEP	1.4
1% AEP	5% AEP	1.4
0.5% AEP	1% AEP	1.45
PMF	1% AEP	1.45

5.4 Blockage Scenarios

The modelled design event conditions consider no blockages to the stormwater drainage network or bridge and culvert structures. However, during flood events, blockages can significantly increase local flood levels. The adopted methodology for determining appropriate consideration of blockages is that proposed in the Blockage of Hydraulic Structures Stage 2 Final Report (AR&R Revision Project 11, 2013).

Under the revised AR&R guidelines appropriate blockages to consider for design flood conditions are based on a number of criteria relating to the nature of the source catchment, in order to determine at-site debris potential. For the Muddy Creek catchment, that is steep and heavily urbanised, the potential is high. Therefore, when considering blockages for design events a high blockage potential is applied for events of a 5% AEP magnitude or greater, and a medium blockage potential for events smaller than a 5% AEP magnitude. The recommended method for the application of structure blockage is then determined using the information in Table 3.6 of the AR&R Project 11 report, reproduced below in Table 5-5.

Table 5-5 Most Likely Blockage Levels for Design Events

Control Dimension	At-site Debris Potential		
	High	Medium	Low
$W < L_{10}$ (5m)	100%	50%	25%
$W >= L_{10}$ (5m) $<= 3*L_{10}$ (15m)	20%	10%	0%
$W > 3*L_{10}$ (15m)	10%	0%	0%

L_{10} is the length for which the longest 10% of potential blockage items exceed. This has been taken as around the length of a car, being approximately 5m.

For the stormwater drainage network, sensitivity to blockages has been assessed by modelling both a 50% and a 100% blockage.

5.5 Modelled Design Events

5.5.1 Catchment Derived Flood Events

The catchment derived flood events that have been simulated for the design flood scenarios are summarised in Table 5-6.

Table 5-6 Modelled Design Flood Events

ID	Name	Event Magnitude	Event Duration	Tailwater Level (m AHD)
1	2y2h	50% AEP	2h	1.1
2	2y9h	50% AEP	9h	1.1
3	5y2h	20% AEP	2h	1.1
4	5y9h	20% AEP	9h	1.1
5	10y2h	10% AEP	2h	1.1
6	10y9h	10% AEP	9h	1.1
7	20y2h	5% AEP	2h	1.1
8	20y9h	5% AEP	9h	1.1
9	50y2h	2% AEP	2h	1.4
10	50y9h	2% AEP	9h	1.4
11	100y2h	1% AEP	2h	1.4
12	100y9h	1% AEP	9h	1.4
13	200y2h	0.5% AEP	2h	1.45
14	200y9h	0.5% AEP	9h	1.45
15	PMF15m	PMF	15m	1.45
16	PMF45m	PMF	45m	1.45
17	PMF90m	PMF	90m	1.45

5.5.2 Tidal Inundation

In terms of predicting flood extents from tidal inundation an approximate HAT level has been mapped using an elevation of 1.1m AHD. The future sea-level rise scenarios considering potential climate change for the 2050 and 2100 planning horizons have also been mapped using levels of 1.5m AHD and 2.0m AHD respectively, based on the 0.4m and 0.9m sea-level rise predictions discussed late in Section 8.1.1.

6 Design Flood Results

A range of design flood conditions were modelled, the results of which are presented and discussed below. The simulated design events included the 50% AEP, 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP and 0.5% AEP. The PMF event has also been modelled. The impact of future climate change on flooding in the study catchment was also considered for the 1% AEP design flood event.

The design flood results are presented in a separate mapping compendium. For the simulated design events including the 50% AEP, 20% AEP, 10% AEP, 5% AEP, 2% AEP, 1% AEP, 0.5% AEP and PMF events, a map of peak flood level, depth and velocity is presented covering the modelled area.

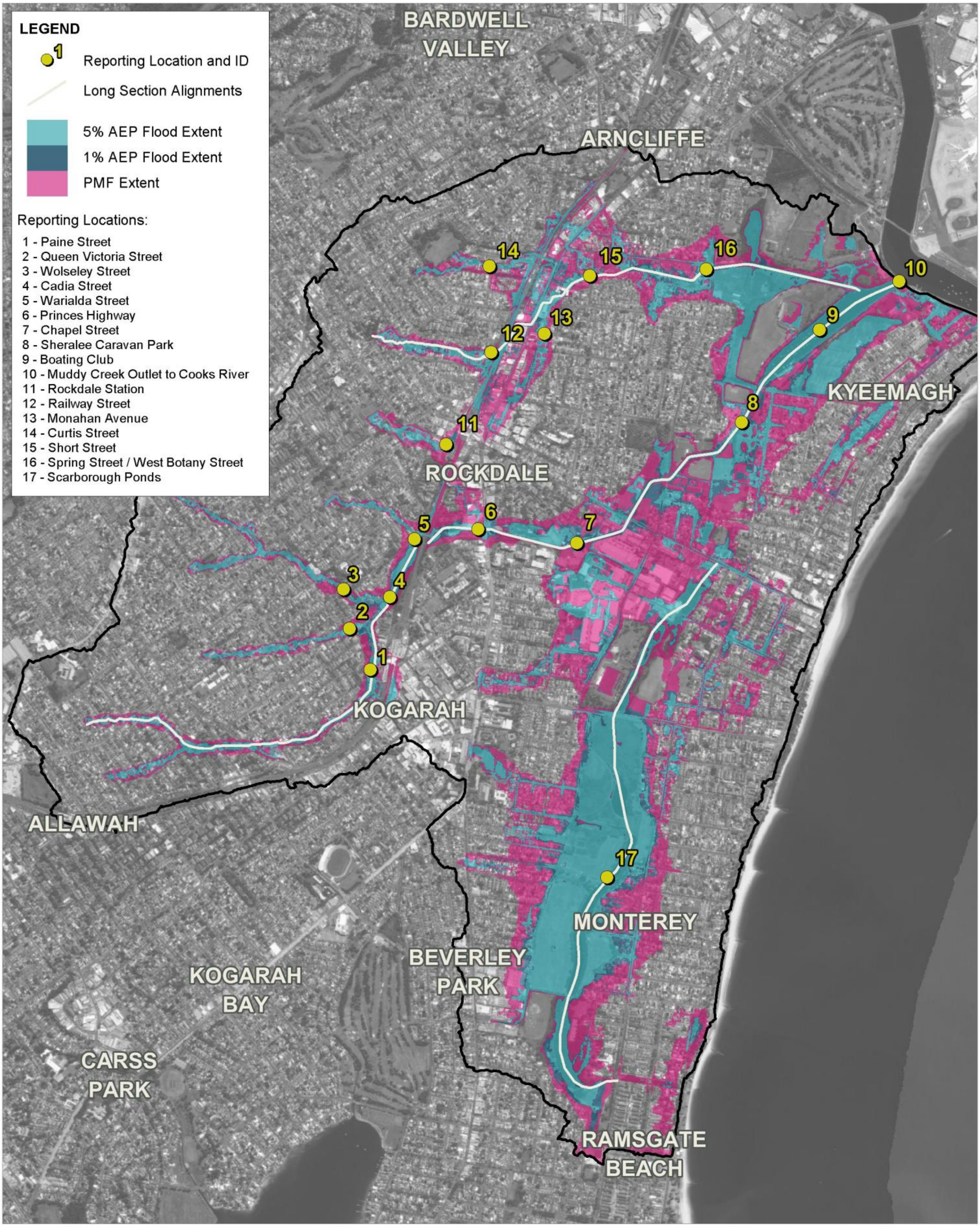
6.1 Peak Flood Conditions

6.1.1 Catchment Derived Flood Events

Predicted flood levels at selected locations (as presented in Figure 6-1) are shown in Table 6-1 for the full range of design flood events considered.

Table 6-1 Modelled Peak Flood Levels (m AHD) for Design Flood Events

ID	Location	Flood Event Frequency							PMF
		50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	
1	Paine St	13.7	13.7	13.8	13.8	13.9	13.9	13.9	15.0
2	Queen Vic St	14.1	14.3	14.4	14.4	14.4	14.5	14.5	14.8
3	Wolseley St	10.7	11.0	11.9	12.1	12.3	12.4	12.4	13.2
4	Cadia St	7.6	7.8	7.8	8.2	8.5	8.6	8.6	11.9
5	Warialda St	6.9	7.6	7.7	7.8	7.9	8.2	8.4	12.0
6	Princes Hwy	4.0	4.3	4.5	5.1	5.4	5.7	5.8	7.6
7	Chapel St	2.2	2.3	2.4	2.5	2.7	2.8	2.9	4.2
8	Caravan Park	1.3	1.4	1.4	1.5	1.8	1.9	2.0	3.2
9	Boating Club	1.2	1.2	1.3	1.3	1.6	1.6	1.7	3.0
10	Cooks River	1.1	1.1	1.1	1.1	1.4	1.4	1.5	1.5
11	Rockdale Stn.	19.7	19.9	19.9	19.9	19.9	20.3	20.4	21.1
12	Railway St	17.9	18.0	18.0	18.0	18.0	18.0	18.1	18.2
13	Monahan Ave	11.1	11.1	11.2	11.2	11.2	11.4	11.4	12.5
14	Curtis St	17.9	18.0	18.0	18.1	18.1	18.2	18.2	18.5
15	Short St	7.1	7.2	7.3	7.4	7.5	7.5	7.6	8.5
16	W Botany St	1.9	2.0	2.1	2.2	2.3	2.4	2.4	3.5
17	S'boro. Ponds	1.8	1.9	2.0	2.1	2.3	2.4	2.5	3.4



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Design Flood Inundation Extents and Reporting Locations

Figure: 6-1

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Longitudinal profiles (see Figure 6-1 for alignments) showing predicted flood levels along the upper and lower reaches of Muddy Creek are shown in Figure 6-2 and Figure 6-3. Similar longitudinal profiles along Spring Street Drain and Scarborough Ponds are shown in Figure 6-4 and Figure 6-5.

Predicted peak flood flows at selected locations (as presented in Figure 6-1) are shown in Table 6-2 for the full range of design flood events considered.

Table 6-2 Modelled Peak Flood Flows (m^3/s)

ID	Location	Flood Event Frequency							PMF
		50% AEP	20% AEP	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	
1	Paine St	12.7	15.4	18.5	22.3	26.6	30.6	34.7	143
2	Queen Vic St	4.0	4.8	6.1	7.8	9.6	11.3	12.8	43.4
3	Wolseley St	9.0	11.2	14.7	18.5	22.9	25.9	29.6	102
4	Cadia St	26.2	32.1	38.4	46.8	64.3	76.5	77.2	275
5	Warialda St	28.2	40.4	41.0	48.7	59.5	74.8	80.9	277
6	Princes Hwy	35.5	42.7	48.3	56.3	65.0	78.2	87.4	326
7	Chapel St	36.4	44.1	48.9	57.6	66.1	77.5	86.6	265
8	Caravan Park	46.1	54.9	58.7	65.7	69.2	75.0	78.9	278
9	Boating Club	49.7	59.2	63.1	70.1	89.6	97.6	103	297
10	Cooks River	61.4	72.2	80.1	95.4	118	130	142	412
11	Rockdale Stn.	4.3	4.7	4.9	5.3	5.5	5.8	6.0	29.8
12	Railway St	4.8	6.2	7.0	8.8	10.5	12.1	13.5	45.5
13	Monahan Ave	2.0	2.8	3.3	4.1	4.8	5.5	6.2	38.9
14	Curtis St	4.2	6.8	7.7	9.3	11.4	13.3	15.2	44.4
15	Short St	13.8	15.5	17.0	20.8	25.3	28.7	32.8	147
16	W Botany St	23.8	28.0	30.3	33.1	38.6	43.8	48.7	190
17	S'boro. Ponds	1.2	1.8	2.2	2.7	3.5	3.7	4.0	5.8

Figure 6-1 shows the design flood inundation extents for the 5% AEP, 1% AEP and PMF events. Only isolated areas of residential or commercial development appear to be inundated at the 5% AEP and 1% AEP events, with extensive flooding of development at the PMF event. The flood extents for the 5% AEP event and 1% AEP event are broadly similar.

Peak in-channel flood velocities are typically around 3.0m/s, ranging from 2m/s to 4m/s for the 5% AEP event. At the 1% AEP event they increase to around 3.5m/s and are closer to 4m/s at the PMF. Flood velocities on the developed floodplain areas are typically less than 0.5m/s, but may be locally high around control structures and buildings. On roadways the velocities are typically higher, often being between 1m/2 and 2m/s.

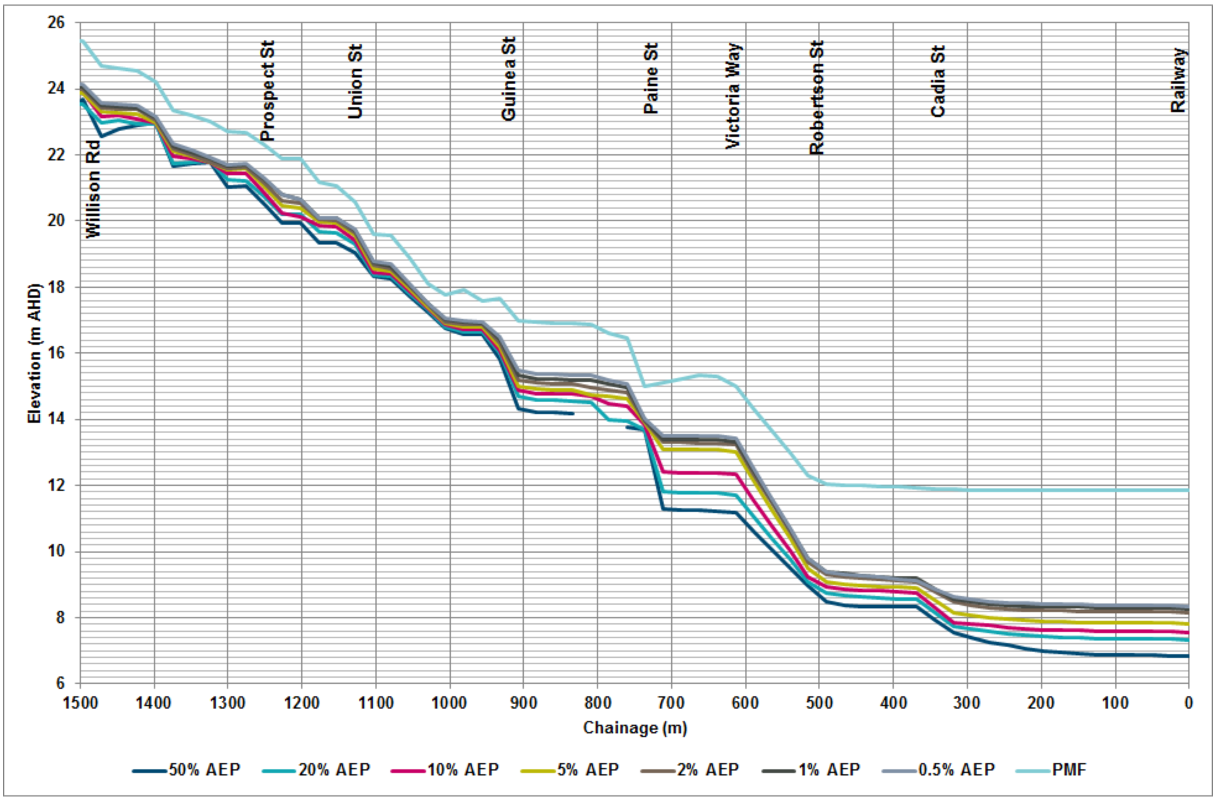


Figure 6-2 Long Section along the upper Muddy Creek for Design Flood Events

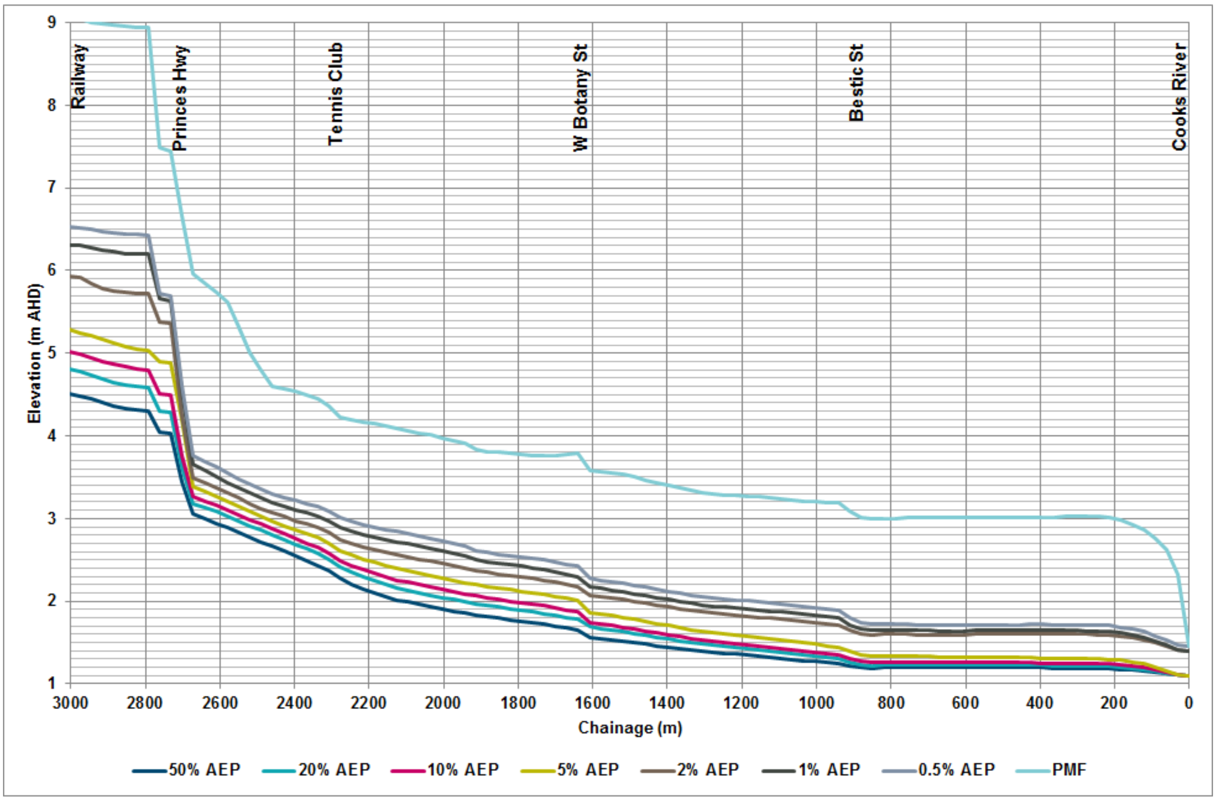


Figure 6-3 Long Section along the lower Muddy Creek for Design Flood Events

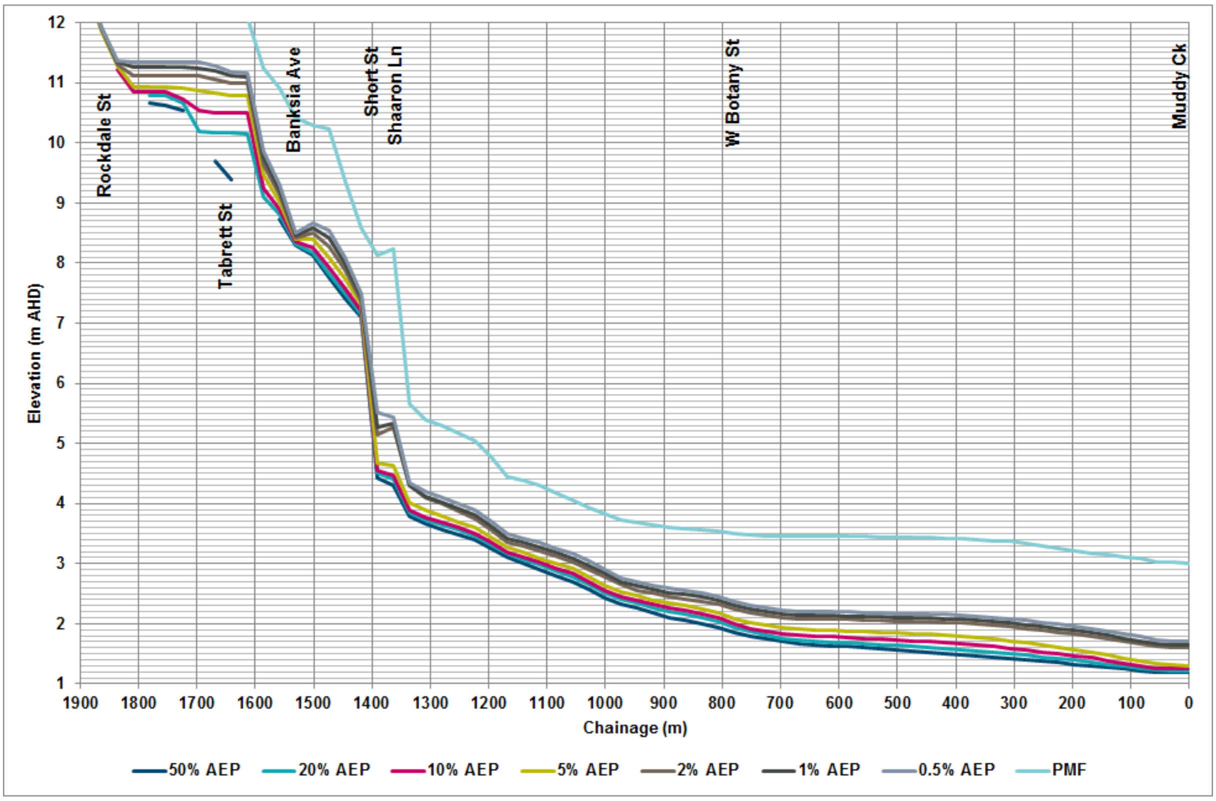


Figure 6-4 Long Section along the Spring Street Drain for Design Flood Events

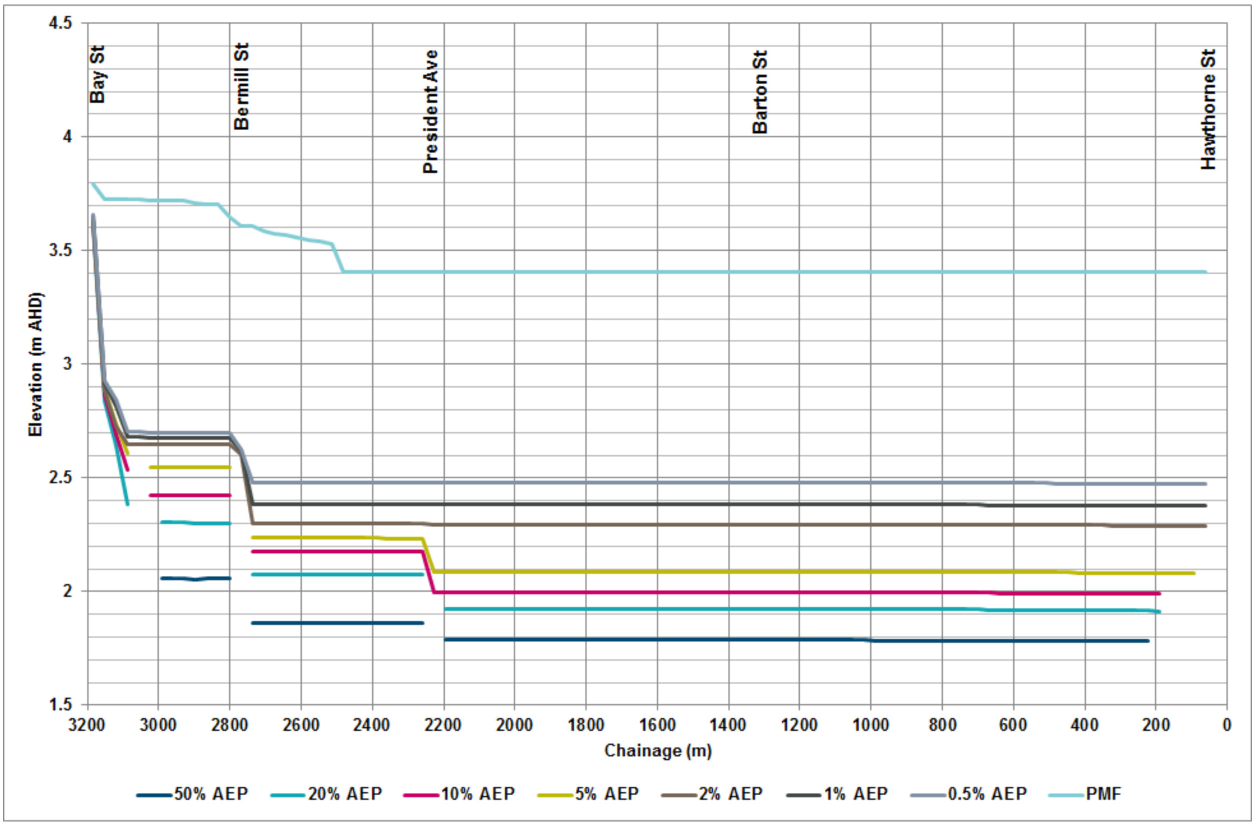


Figure 6-5 Long Section along the Scarborough Ponds for Design Flood Events

6.1.2 Tidal Inundation

The results of the tidal inundation mapping for the current, 2050 and 2100 planning horizons are presented in Figure 6-6. It shows that under current conditions the extent of tidal inundation is essentially restricted to the defined waterways, including the lower reaches of the drainage channels and the Scarborough Ponds system. At the 2050 planning horizon the increased inundation extents begin to impact on areas of adjacent open space, including agriculture and at the 2100 planning horizon some low-lying roads and properties are also impacted.

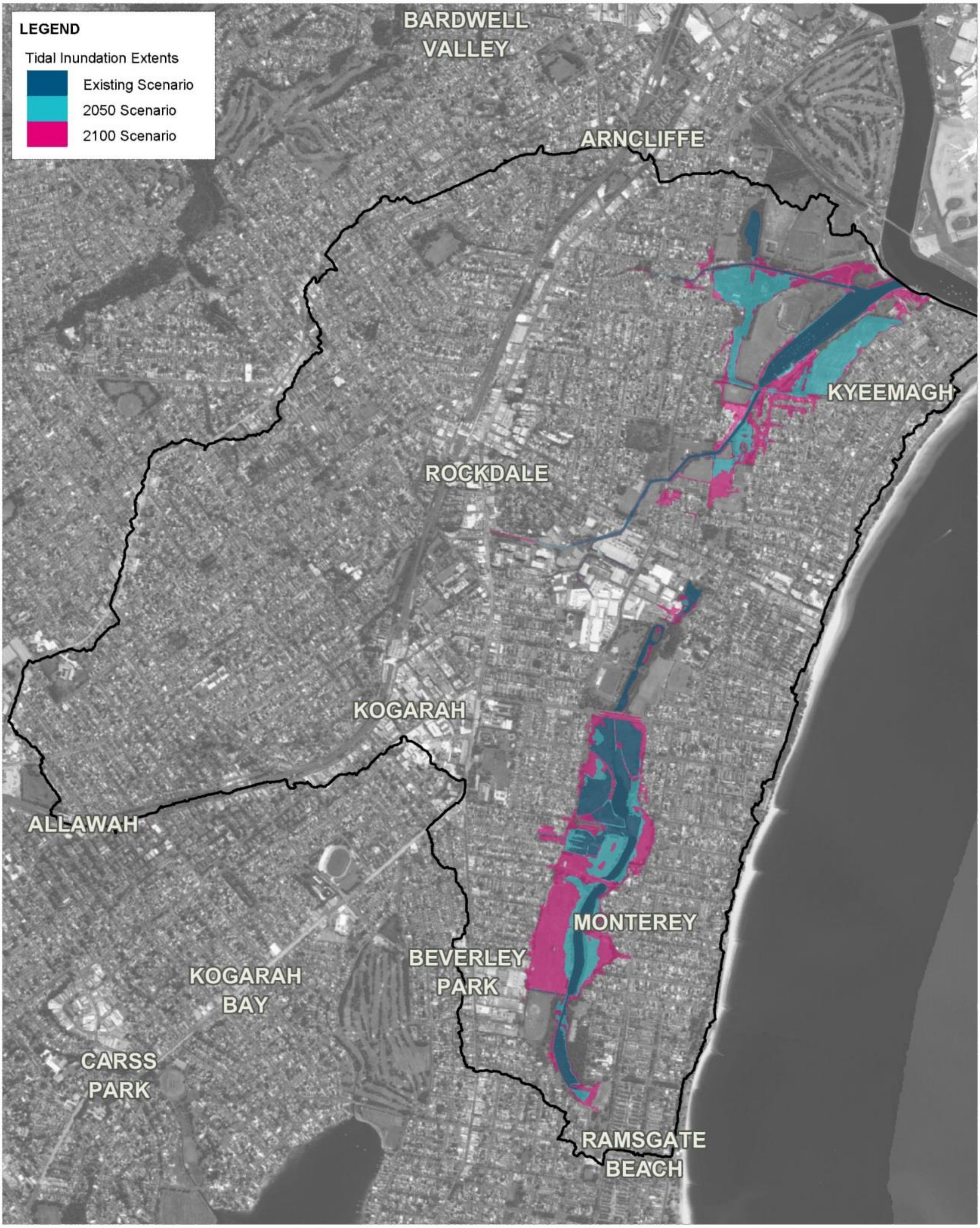
6.1.3 Potential Flooding Problem Areas

Figure 6-7 shows the properties that have modelled flood inundation within their cadastral boundary at the 1% AEP event. It helps to provide an overview of where flooding problems are located within the catchment.

Flooding to the west of the railway is located along a number of gully lines that drain to Muddy Creek and Spring Street Drain. There are a number of locations along which the overland flow path alignment is not within the roadway, but instead traverses blocks of residential development. The floodway is usually situated along the yards to the rear of the properties and/or where flow is funnelled between buildings. The affected locations include:

- Properties located along two flow paths between Botany Street and High Street;
- The rear of properties located along High Street and Mill Street;
- Properties located along the flow path between Short Street and Edgehill Street;
- Properties located along the flow path between Guinea Street and Robinson Street;
- Properties located along the flow path between Percival Street and Queen Victoria Street;
- The rear of properties located along Robertson Street and Warialda Street;
- The rear of properties located along Campbell Street and Lymington Street;
- Properties located along two flow paths between Northbrook Street and Beaconsfield Street;
- Properties located along the flow path between Dunmore Street South and Warialda Street;
- Properties located along the flow path between Goyen Avenue and Watkin Street;
- The rear of properties located along Frederick Street;
- Properties located along the flow path between Heathcote Street and Arlington Street;
- The rear of properties located along Oswin Lane and Gloucester Street; and
- The rear of properties located along Godfrey Street and Bowmer Street.

The areas between the Princes Highway and Short Street, between Terry Street and Spring Street and between the Princes Highway and Cross Street (on the eastern side of the railway) also experience similar issues to the above. Further downstream the flooding problem areas are typically limited to locations where the capacity of the drainage channels is significantly exceeded. Such areas include:



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Tidal Inundation Extents

Figure: 6-6

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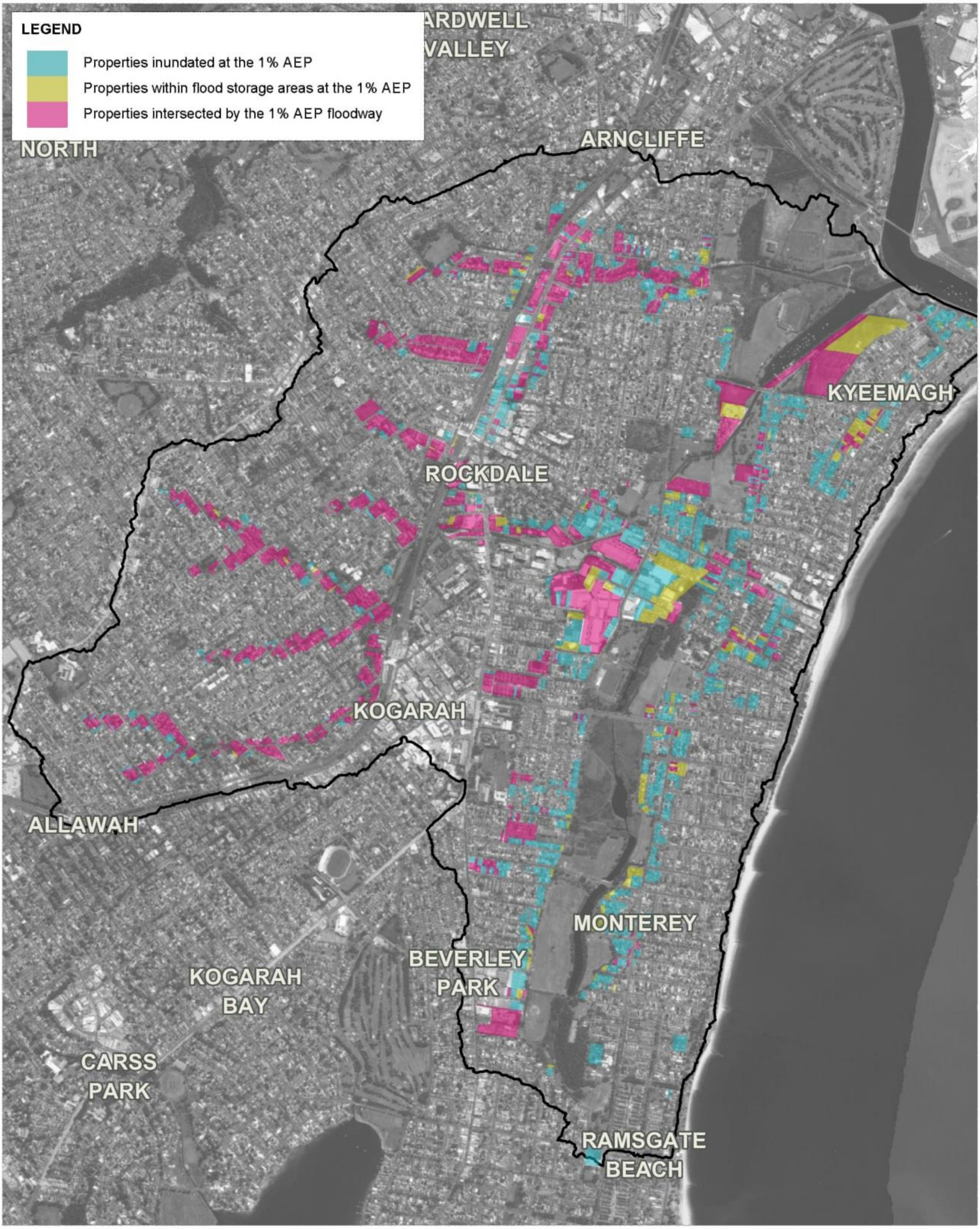

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Flood Affected Properties at the 1% AEP Event

Figure: 6-7

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- The properties along Spring Street Drain between Shaaron Court and West Botany Street;
- The properties along Muddy Creek between Harrow Road and Bay Street; and
- Properties along West Botany Street where local drainage to Muddy Creek is exceeded.

There are also a number of properties bordering the Scarborough Ponds that are affected.

6.2 Design Flood Hydrographs

The flood flow hydrographs for the modelled events at the Warialda Street, West Botany Street, Cooks River and Scarborough Ponds locations are presented in Figure 6-8 to Figure 6-11 respectively. The Scarborough Ponds hydrographs are taken from the 9 hour duration storm, as this is the critical event at that location. The hydrographs at the other three locations are taken from the 2 hour duration storm. At Warialda Street and West Botany Street the hydrographs peak at around 1 hour after the onset of the storm, with the peak at the outlet to Cooks River occurring at around 1.5 hours. The flood storage in the Scarborough Ponds system reaches its peak around 9 hours after the onset of the storm, before beginning to drain.

The flood flow hydrographs for the 1% AEP event at each of the four locations are also presented in Figure 6-12, to gain an appreciation of the relative timings at the various locations, and the different nature of the shape and magnitude of the catchment response. The peaky response due to the steep nature of the upper Muddy Creek catchment is evident in the Warialda Street hydrographs, with the Spring Street Drain exhibiting a similar shape and timing, albeit with a less peaky response. The highly different nature of the Scarborough Ponds system is also evident.

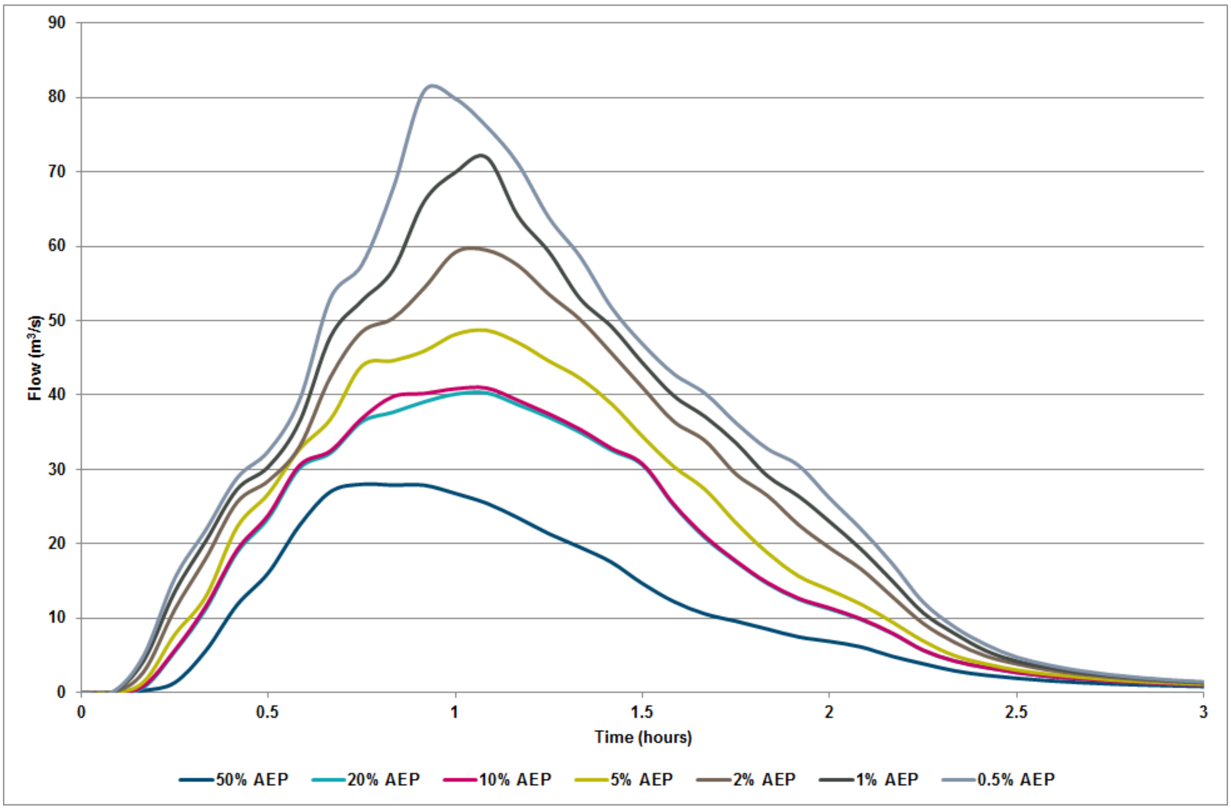


Figure 6-8 Modelled Design Event Hydrographs at Warialda Street

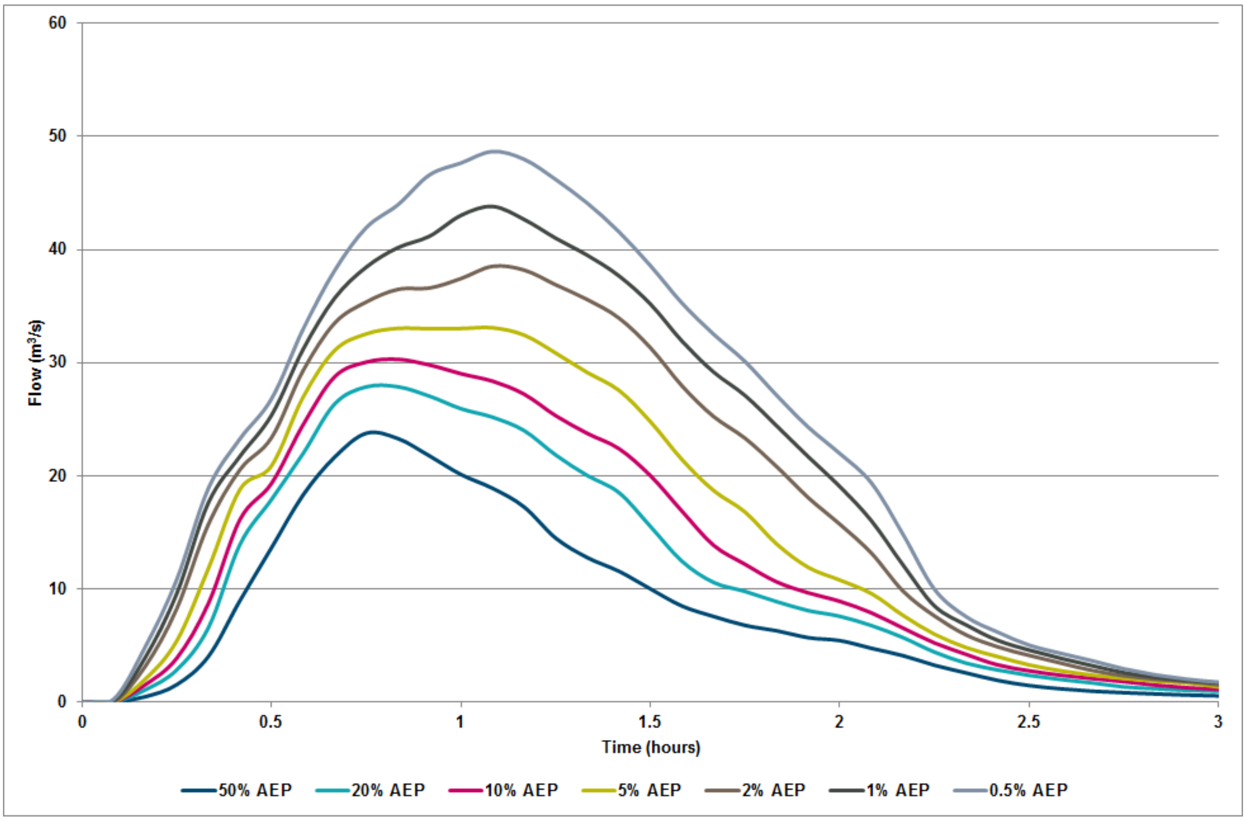


Figure 6-9 Modelled Design Event Hydrographs at West Botany Street

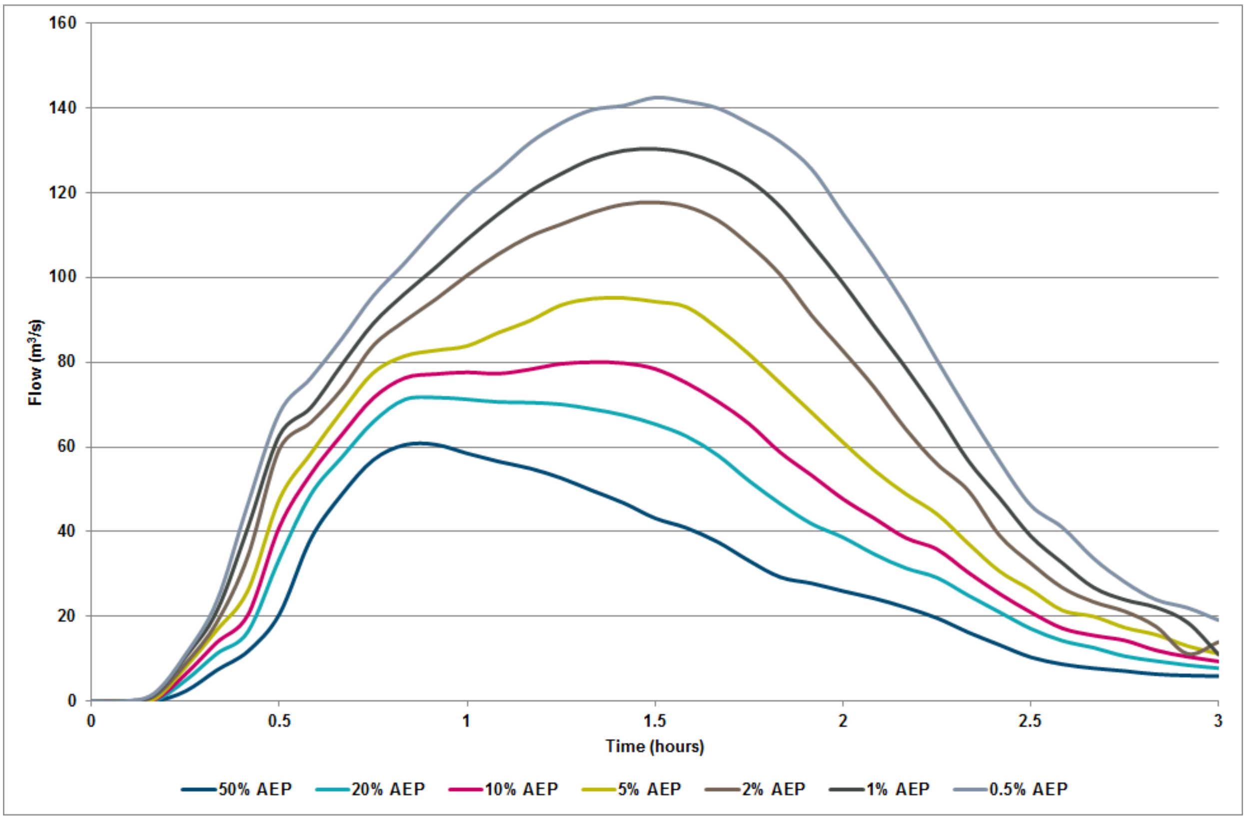


Figure 6-10 Modelled Design Event Hydrographs at the Muddy Creek Outlet to Cooks River

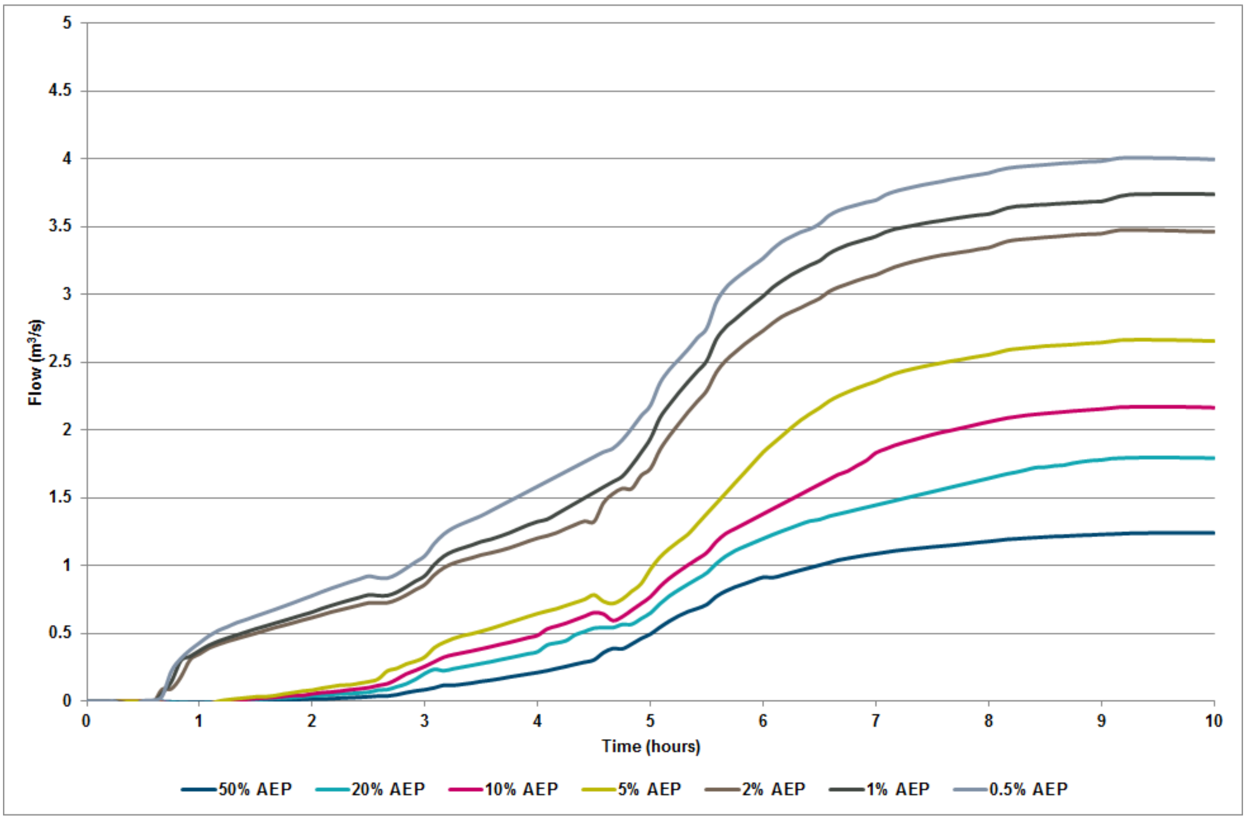


Figure 6-11 Modelled Design Event Hydrographs in the Scarborough Ponds System

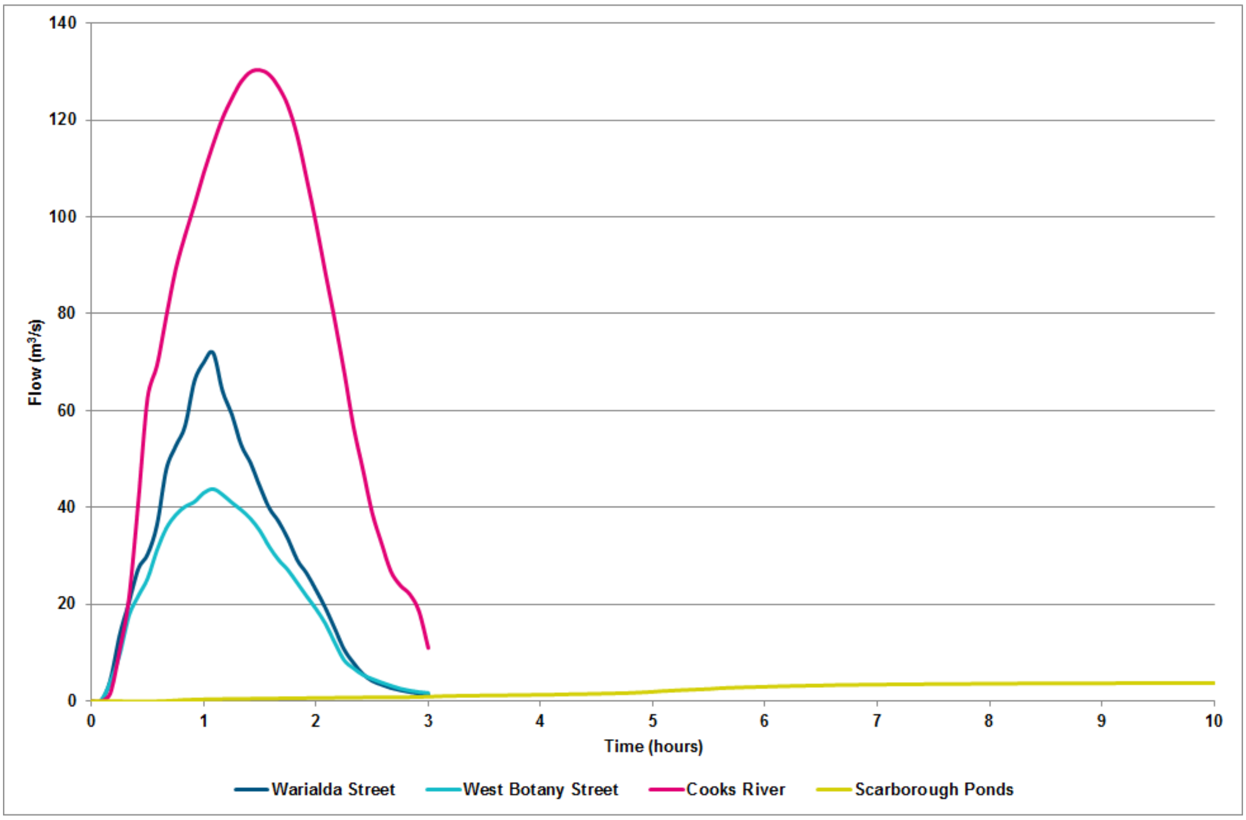


Figure 6-12 Modelled 1% AEP Event Hydrographs at Various Locations

6.3 Hydraulic Classification

There are no prescriptive methods for determining what parts of the floodplain constitute flood ways, flood storages and flood fringes. Descriptions of these terms within the NSW Floodplain Development Manual (DIPNR, 2005) are essentially qualitative in nature. Of particular difficulty is the fact that a definition of flood behaviour and associated impacts is likely to vary from one floodplain to another depending on the circumstances and nature of flooding within the catchment.

The hydraulic categories as defined in the Floodplain Development Manual are:

- **Floodway** – Areas that convey a significant portion of the flow. These are areas that, even if partially blocked, would cause a significant increase in flood levels or a significant redistribution of flood flows, which may adversely affect other areas.
- **Flood Storage** – Areas that are important in the temporary storage of the floodwater during the passage of the flood. If the area is substantially removed by levees or fill it will result in elevated water levels and/or elevated discharges. Flood Storage areas, if completely blocked would cause peak flood levels to increase by 0.1m and/or would cause the peak discharge to increase by more than 10%.
- **Flood Fringe** – Remaining area of flood prone land, after Floodway and Flood Storage areas have been defined. Blockage or filling of this area will not have any significant effect on the flood pattern or flood levels.

A number of approaches were considered when attempting to define flood impact categories across the study catchment. The approach that was adopted derived a preliminary floodway extent from the velocity * depth product (sometimes referred to as unit discharge). The peak flood depth was used to define flood storage areas. The adopted hydraulic categorisation is defined in Table 6-3.

Table 6-3 Hydraulic Categories

Floodway	Velocity * Depth > 0.3m^2/s at the 1% AEP event	Areas and flow paths where a significant proportion of floodwaters are conveyed (including all bank-to-bank creek sections).
Flood Storage	Velocity * Depth < 0.3m^2/s and Depth > 0.5m at the 1% AEP event	Areas where floodwaters accumulate before being conveyed downstream. These areas are important for detention and attenuation of flood peaks.
Flood Fringe	Flood extent of the PMF event	Areas that are low-velocity backwaters within the floodplain. Filling of these areas generally has little consequence to overall flood behaviour.

Preliminary hydraulic category mapping is included in the Mapping Compendium, and is presented for each of the design events.

6.4 Provisional Hazard Categories

The Updating National Guidance on Best Practice Flood Risk Management (NFRAG, 2014) considers a holistic approach to consider flood hazards to people, vehicles and structures. It recommends a composite six-tiered hazard classification, reproduced in Figure 6-13. The six hazard classifications are summarised in Table 6-4.

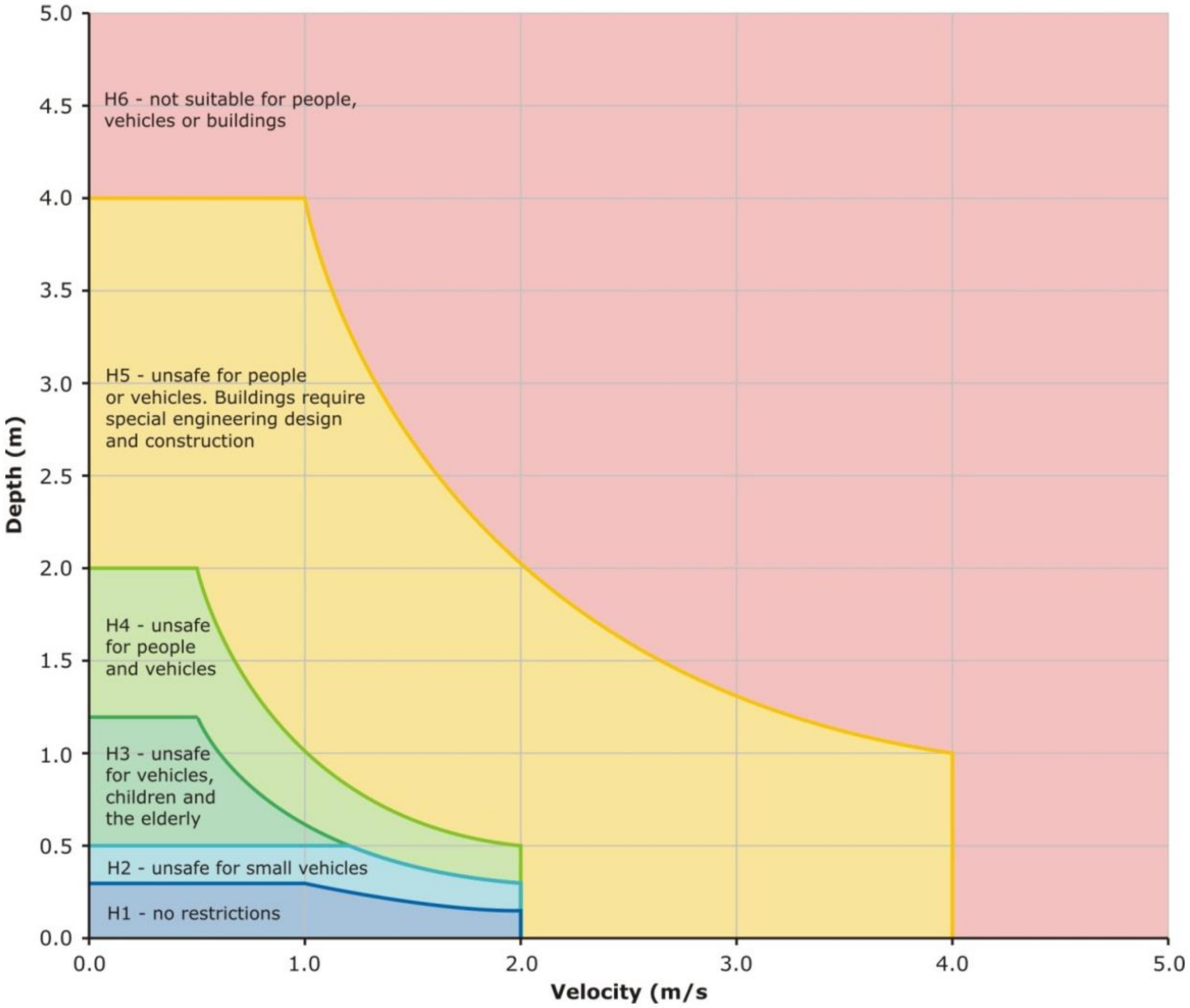


Figure 6-13 Combined Flood Hazard Curves

The key factors influencing flood hazard or risk are:

- Size of the Flood
- Rate of Rise - Effective Warning Time
- Community Awareness
- Flood Depth and Velocity
- Duration of Inundation
- Obstructions to Flow

- Access and Evacuation

Table 6-4 Combined Flood Hazard Curves – Vulnerability Thresholds

Hazard Classification	Description
H1	Relatively benign flow conditions. No vulnerability constraints.
H2	Unsafe for small vehicles.
H3	Unsafe for all vehicles, children and the elderly.
H4	Unsafe for all people and vehicles.
H5	Unsafe for all people and all vehicles. Buildings require special engineering design and construction.
H6	Unconditionally dangerous. Not suitable for any type of development or evacuation access. All building types considered vulnerable to failure.

The provisional flood hazard level is often determined on the basis of the predicted flood depth and velocity. This is conveniently done through the analysis of flood model results. A high flood depth will cause a hazardous situation while a low depth may only cause an inconvenience. High flood velocities are dangerous and may cause structural damage while low velocities generally have no

Provisional hazard category mapping is included in the Mapping Compendium, and is presented for the 1% AEP and PMF design events.

6.5 Flood Emergency Response Classification

The SES classifies communities according to the impact that flooding has on them. The primary purpose for doing this is to assist SES in the planning and implementation of response strategies. Flood impacts relate to where the normal functioning of services is altered due to a flood, either directly or indirectly, and relates specifically to the operational issues of evacuation, resupply and rescue.

Flood Islands

Flood Islands are inhabited areas of high ground within a floodplain which are linked to the flood free valley sides by only one access / egress route. If the road is cut by floodwaters, the community becomes an island, and access to the area may only be gained by boat or aircraft. Flood islands are classified according to what can happen after the evacuation route is cut as and are typically separated into:

- High Flood Islands;
- Low Flood Islands

A *High Flood Island* include sufficient land located at a level higher than the limit of flooding (i.e., above the PMF) to provide refuge to occupants. During flood events properties may be inundated and the community isolated, however, as there is an opportunity for occupants to retreat to high ground, the direct risk to life is limited. If it will not be possible to provide adequate support during the period of isolation, evacuation will have to take place before isolation occurs.

The highest point of a *Low Flood Island* is lower than the limit of flooding (i.e., below the PMF) or does not provide sufficient land above the limit of flooding to provide refuge to the occupants of the area. During flood events properties may be inundated and the community isolated. If floodwater continues to rise after it is isolated, the island will eventually be completely covered. People left stranded on the island may drown.

Trapped Perimeter Areas

Trapped Perimeter Areas are inhabited areas located at the fringe of the floodplain where the only practical road or overland access is through flood prone land and unavailable during a flood event. The ability to retreat to higher ground does not exist due to topography or impassable structures. Trapped perimeter areas are classified according to what can happen after the evacuation route is cut as follows.

High Trapped Perimeter Areas include sufficient land located at a level higher than the limit of flooding (i.e., above the PMF) to provide refuge to occupants. During flood events properties may be inundated and the community isolated, however, as there is an opportunity for occupants to retreat to high ground, the direct risk to life is limited. If it will not be possible to provide adequate support during the period of isolation, evacuation will have to take place before isolation occurs.

Low Trapped Perimeter Areas is lower than the limit of flooding (i.e., below the PMF) or does not provide sufficient land above the limit of flooding to provide refuge to the occupants people of the area. During a flood event the area is isolated by floodwater and property may be inundated. If floodwater continues to rise after it is isolated, the area will eventually be completely covered. People trapped in the area may drown.

Areas Able to be Evacuated

These are inhabited areas on flood prone fringe areas that are able to be evacuated. However, their categorisation depends upon the type of evacuation access available, as follows.

Areas with Overland Escape Route are those areas where access roads to flood free land cross lower lying flood prone land. Evacuation can take place by road only until access roads are closed by floodwater. Escape from rising floodwater is possible but by walking overland to higher ground. Anyone not able to walk out must be reached by using boats and aircraft. If people cannot get out before inundation, rescue will most likely be from rooftops.

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

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

Indirectly Affected Areas

These are areas which are outside the limit of flooding and therefore will not be inundated nor will they lose road access. However, they may be indirectly affected as a result of flood damaged infrastructure or due to the loss of transport links, electricity supply, water supply, sewage or telecommunications services and they may therefore require resupply or in the worst case, evacuation.

Overland Refuge Areas

These are areas that other areas of the floodplain may be evacuated to, at least temporarily, but which are isolated from the edge of the floodplain by floodwaters and are therefore effectively flood islands or trapped perimeter areas. They should be categorised accordingly and these categories used to determine their vulnerability.

Note that Flood Management Communities identified as Overland Refuge Areas on Low Flood Island have been classified according to the SES Flow Chart for Flood Emergency Response Classification. These are areas where vehicular evacuation routes are inundated before residential areas of the Community.

6.5.1.1 Local Classification

Being a heavily urbanised area it is difficult to provide an appropriate classification, as the local flood conditions to each individual property will vary for varying event magnitudes and may be highly spatially variable. However, the flooding within the catchment is principally overland flow, with limited out-of-bank mainstream flooding. Given the relatively steep nature of the catchment and the extensive network of roads, the most appropriate classification would be ***Areas with Rising Road Access****.* However, the roadways would often have hazardous conditions during a major flood and so it may be safer for people to remain in their homes. These properties would then be ***High or Low Flood Islands*** if surrounded by flood waters, or ***High or Low Trapped Perimeter Areas*** if located on the edge of the floodplain.

With limited potential response time available during flash flood events it would usually be safer for residents to take refuge in their homes, rather than evacuate along potentially hazardous and grid-locked roads. The exception to this is buildings that would be at risk of collapse due to structural damage during the flood.

6.6 Preliminary Residential Flood Planning Level

Flood Planning Levels (FPLs) are used for planning purposes, and directly determine the extent of the Flood Planning Area (FPA), which is the area of land subject to flood-related development controls. The FPL is the level below which a Council places restrictions on development due to the hazard of flooding. Traditional floodplain planning has relied almost entirely on the definition of a singular FPL, which has usually been based on the 1% AEP flood level, for the purposes of applying floor level controls.

The FPA for the study area has been derived through the addition of a 0.5m freeboard to the modelled 1% AEP flood level. Through spatial analysis within a GIS platform this level has then been projected horizontally until it intersects with the LiDAR DEM to provide the associated area of extent over which the FPL and associated planning controls should apply. A few minor modifications to this approach were adopted to overcome some shortcomings with application of the method, including:

- Within flat areas such as those between Scarborough Ponds and Botany Bay, the addition of a 0.5m freeboard exceeds the height of the surrounding topography, unreasonably extending the FPA beyond credible limits. The extent of the 1% AEP +30% rainfall (~0.2% AEP) event has been used as a surrogate definition of the FPA in such instances; and
- Where the addition of a 0.5m freeboard exceeded the level of the PMF the resultant FPA has been trimmed back to the extent of the PMF inundation.

6.7 Conclusion

Following model calibration a standard approach has been adopted to derive appropriate design flood conditions, as presented. These results will form the basis for future flood planning and floodplain risk management activities within the study area.

7 Sensitivity Testing

7.1 Hydraulic Roughness

The sensitivity of modelled peak flood levels to the adopted Manning's 'n' roughness values were tested for the 1% AEP design event. Roughness values for all materials types within the channel and floodplain were increased and decreased by 20%. Longitudinal profiles showing the result of this assessment for the upper and lower reaches of Muddy Creek are shown in Figure 7-1 and Figure 7-2. Peak modelled flood levels are presented in Table 7-2 at the end of this Section.

7.2 Blockages

As discussed previously in Section 5.4, the consideration of potential structure blockage is an important consideration of the design flood modelling. Blockages were assessed using a total of four separate model simulations:

- Application of a 50% blockage to the stormwater drainage network;
- Application of a 100% blockage to the stormwater drainage network; and
- Application of appropriate structure blockages on Spring Street Drain and Muddy Creek.

Longitudinal profiles showing the result of the blockage assessment for the upper and lower reaches of Muddy Creek are shown in Figure 7-3 and Figure 7-4. Peak modelled flood levels are presented in Table 7-2 at the end of this Section.

Figure 7-5 presents the spatial distribution of peak blockage impacts of the combined three modelled blockage scenarios for the 1% AEP event. It highlights areas that are particularly exposed to increased flood risk through potential blockage of structures, including:

- Properties situated between Prospect Street and Union Lane (~0.5m);
- Properties situated between Guinea Street and Cadia Street (~0.5m);
- Properties along Warialda Street (~1.0m);
- Properties situated between the railway and the Princes Highway (~1.1m);
- Properties around the Railway Street – Frederick Street intersection (~0.6m);
- Properties situated between Roach Street and the railway (~0.6m to 0.7m); and
- Properties situated between the Princes Highway and Short Street (~0.3m to 0.4m).

Given the significant increase in flood risk across these areas under potential blockage scenarios the incorporation of blockage allowances within the design flood levels should be considered for flood planning purposes, particularly for the Warialda Street to Princes Highway section. It is expected that management of food risk within this area will be one of the key focuses of future floodplain risk management activities.

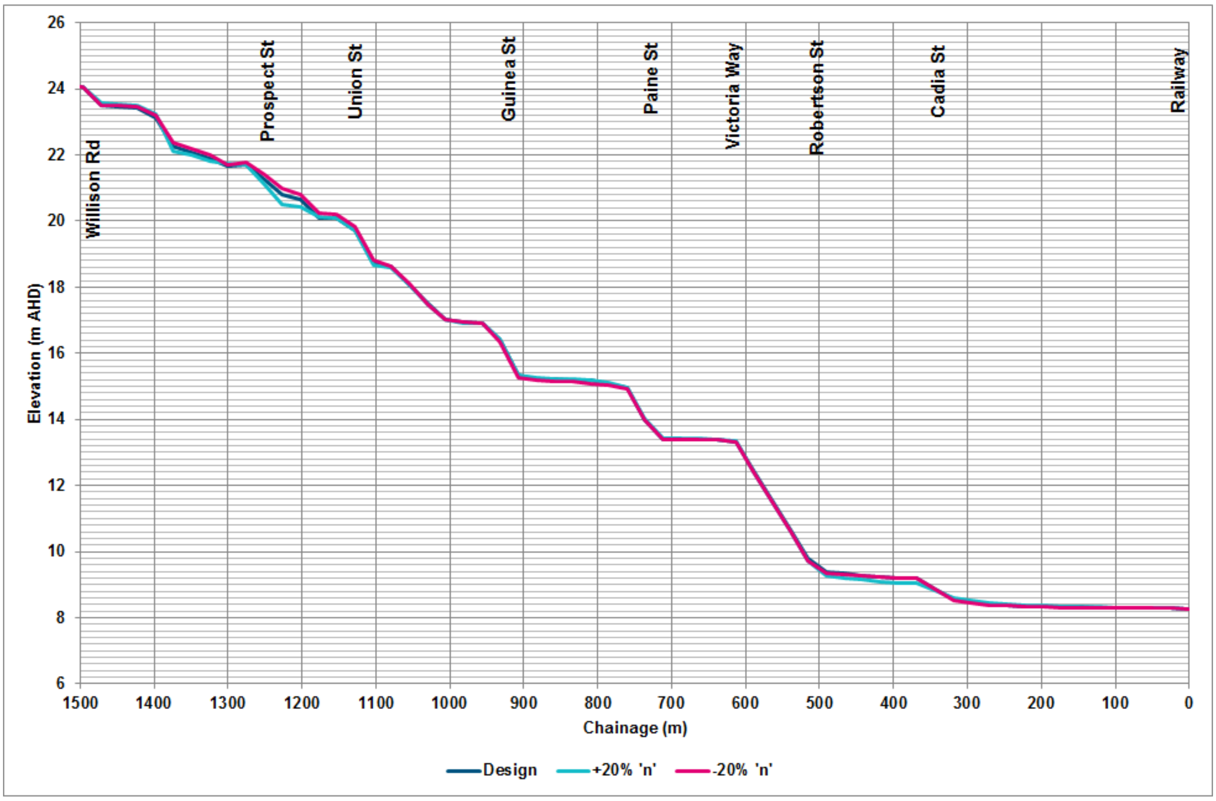


Figure 7-1 Impact of Adopted Hydraulic Roughness along the Upper Muddy Creek

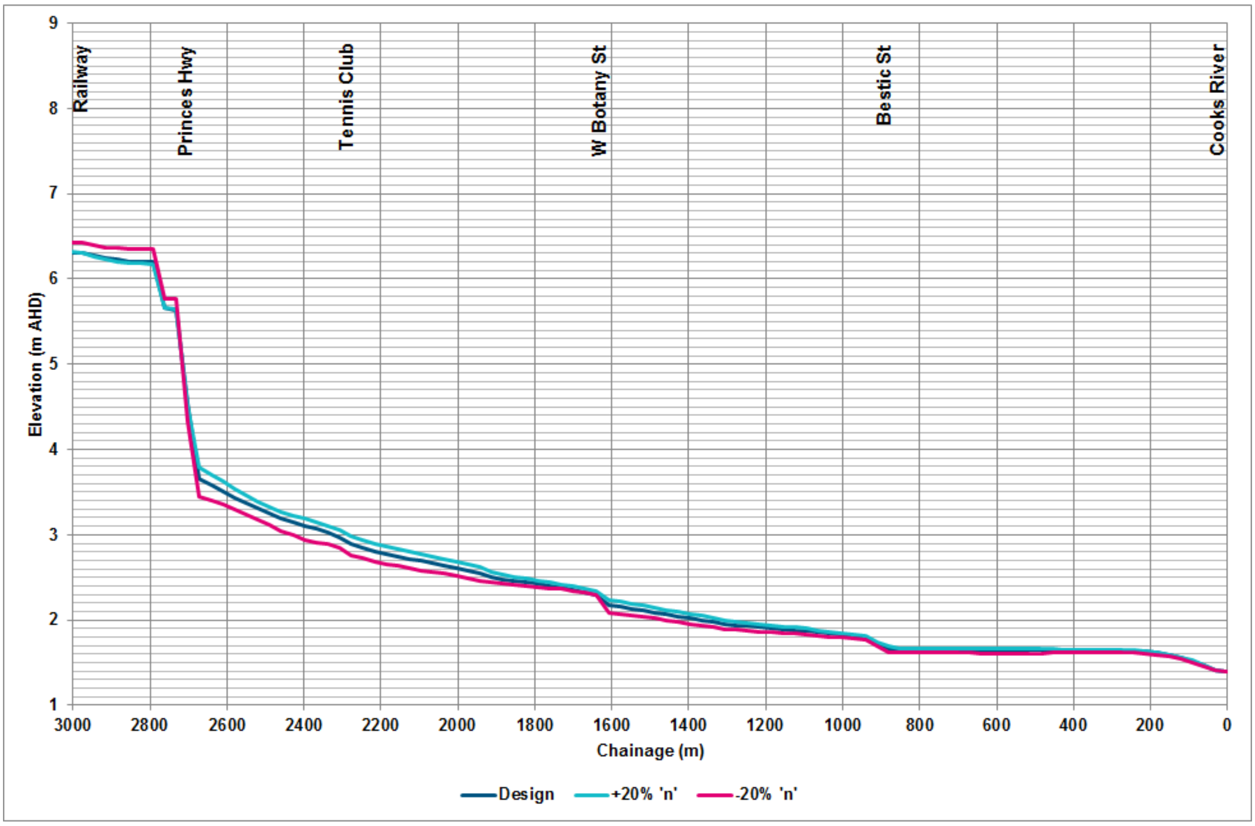


Figure 7-2 Impact of Adopted Hydraulic Roughness along the Lower Muddy Creek

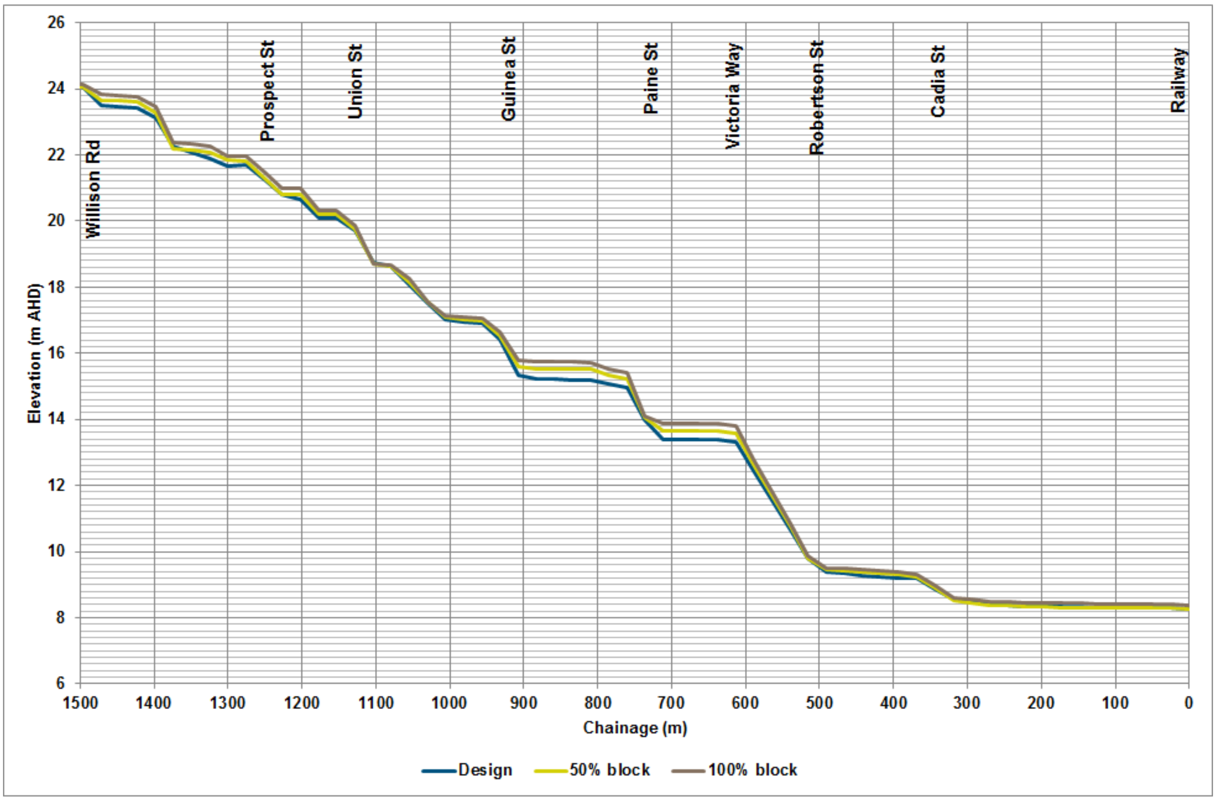


Figure 7-3 Impact of Hydraulic Structure Blockage along the Upper Muddy Creek

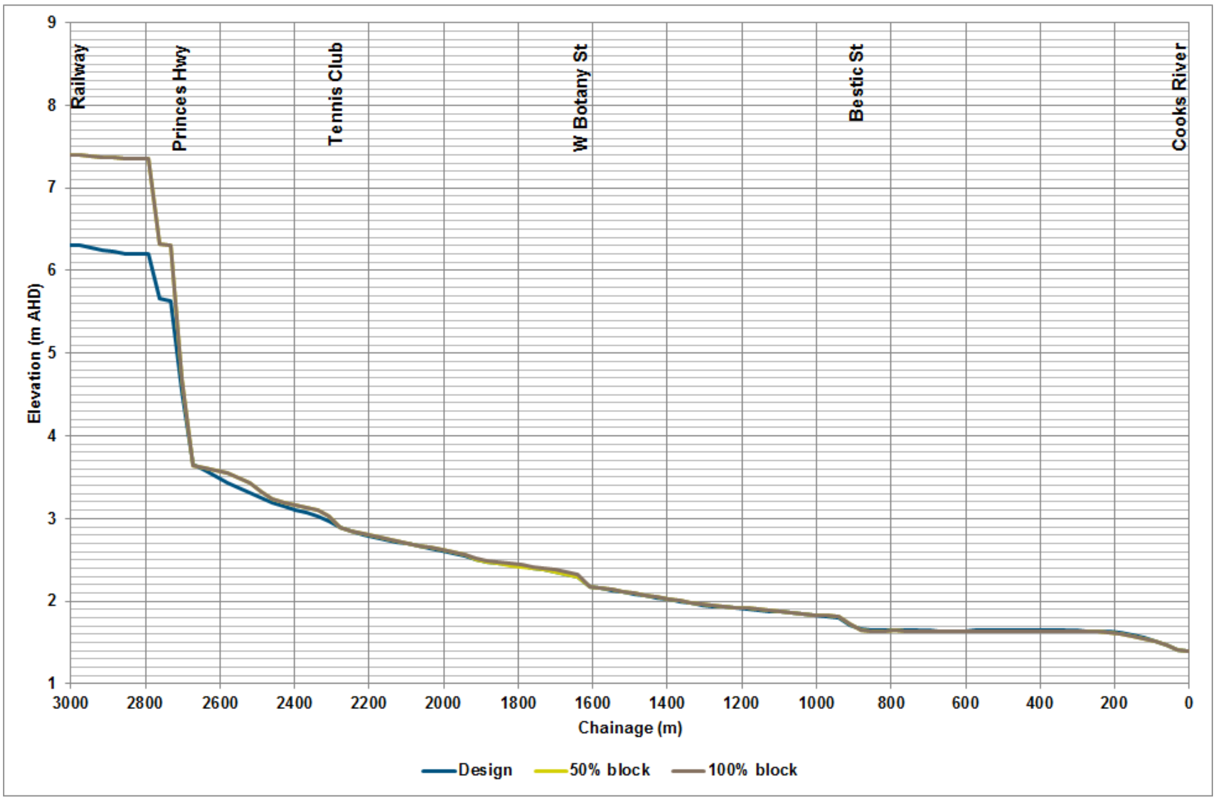
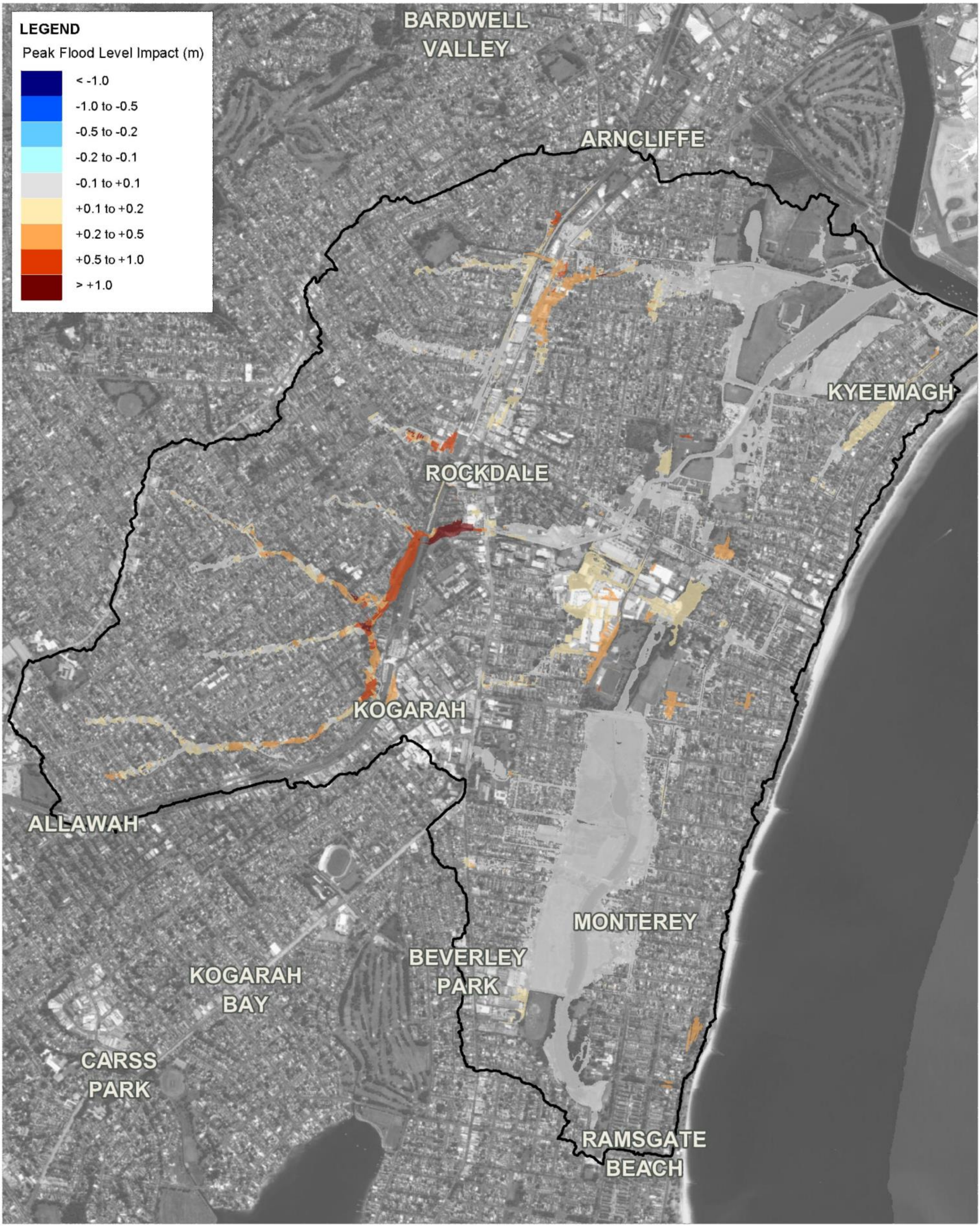


Figure 7-4 Impact of Hydraulic Structure Blockage along the Lower Muddy Creek



Title:

Impact of Combined Blockage Scenarios on the Modelled 1% AEP Peak Flood Level

Figure: 7-5

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Approx. Scale

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7.3 Rainfall Losses

The assessment of model sensitivity to changes in the adopted rainfall losses can be readily determined through the XP-RAFTS hydrological model. When the adopted initial loss of pervious areas is increased from 10mm to 20mm the effective rainfall (that which results in runoff from the catchment) is reduced by 10mm. For the 1% AEP event this results in a typical reduction of modelled peak flows in the order of 6%, which is comparable to a condition halfway between the 2% AEP and 1% AEP design events. When the adopted initial and continuing losses are reduced to zero, the resultant flows for the 1% AEP event increase in the order of 4%, which is comparable to a condition around 30% of the way between the 1% AEP and 0.5% AEP design events.

Longitudinal profiles showing the expected hydraulic result of this assessment for the upper and lower reaches of Muddy Creek are shown in Figure 7-6 and Figure 7-7. Peak modelled flood levels are presented in Table 7-2 at the end of this Section.

7.4 Downstream Boundary

The adopted downstream boundary conditions were discussed in Section 5.3. They consider a coincident flood condition in the Cooks River (albeit to a lesser magnitude) and the study catchment runoff. For the 1% AEP design event this was a 5% AEP Botany Bay level of 1.4m AHD. The impact of adopting a typical downstream boundary, with no consideration of coincident flooding, was simulated for the 1% AEP event. A coincident 1% AEP fluvial flood condition on the Cooks River was also tested. This level is around 1.8m AHD.

Longitudinal profiles showing the result of the adopted downstream boundary condition for the lower Muddy Creek and Spring Street Drain are shown in Figure 7-8 and Figure 7-9. Peak modelled flood levels are presented in Table 7-2 at the end of this Section.

7.5 Scarborough Ponds

The flood conditions of the Scarborough Ponds system are driven by total catchment runoff volume rather than peak flows and as such display a significantly different critical storm duration to the rest of the study area. The critical storm duration for this peak storage volume is typically between 30-hours and 48-hours, depending on the adopted design conditions.

The peak flood conditions for Scarborough Ponds in the design flood mapping and results presentation are derived from the 9-hour storm duration, which is the longest duration that is the critical condition elsewhere in the study area. The length of time required to simulate longer durations within the hydraulic model is overly restrictive. However, the peak flood levels attained within Scarborough Ponds can be readily estimated using XP-RAFTS, through the representation of the stage-storage relationship and piped drainage discharge within the retarding basin module.

A range of design flood conditions were simulated for the 1% AEP event within the XP-RAFTS model to assess the sensitivity of peak flood levels in Scarborough Ponds. The resultant flood condition within Scarborough Ponds for any given event will vary depending on:

- The initial water level in the Ponds at the onset of the event;
- The total volume of catchment runoff during the event;

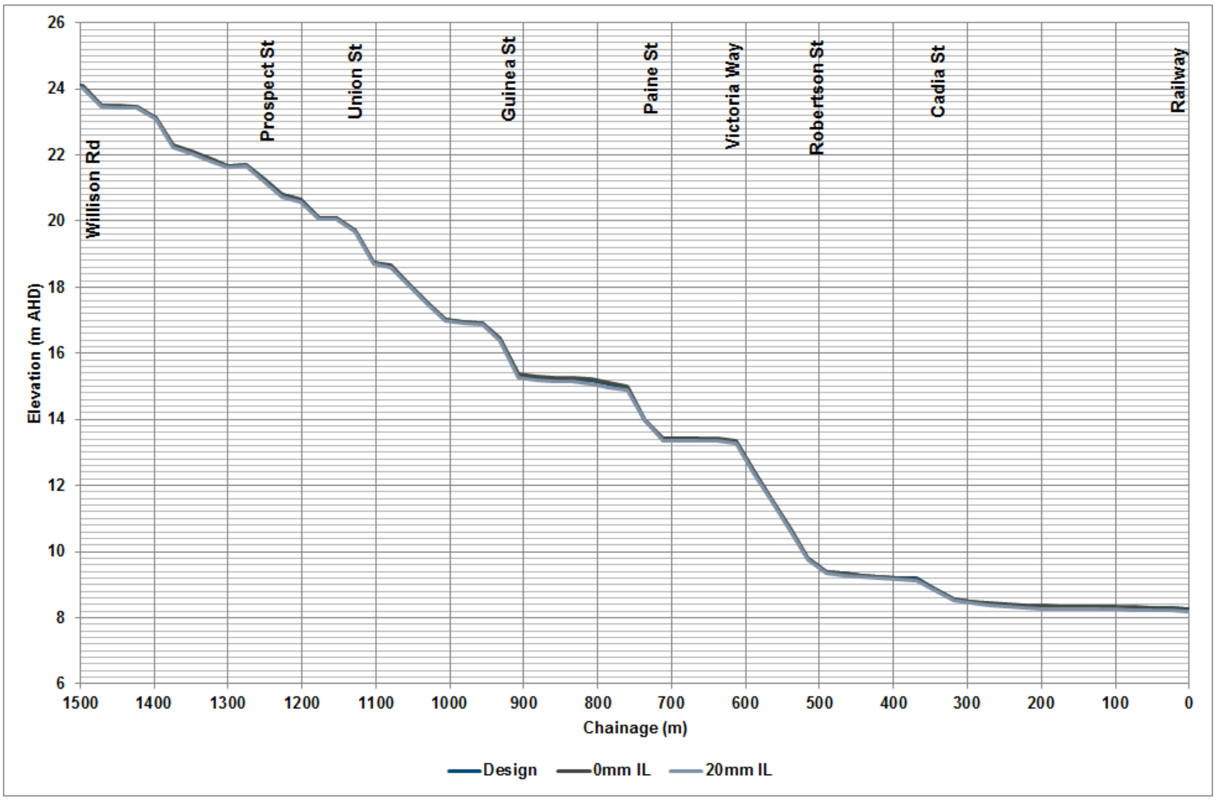

Figure 7-6 Expected Impact of Adopted Rainfall Losses along the Upper Muddy Creek

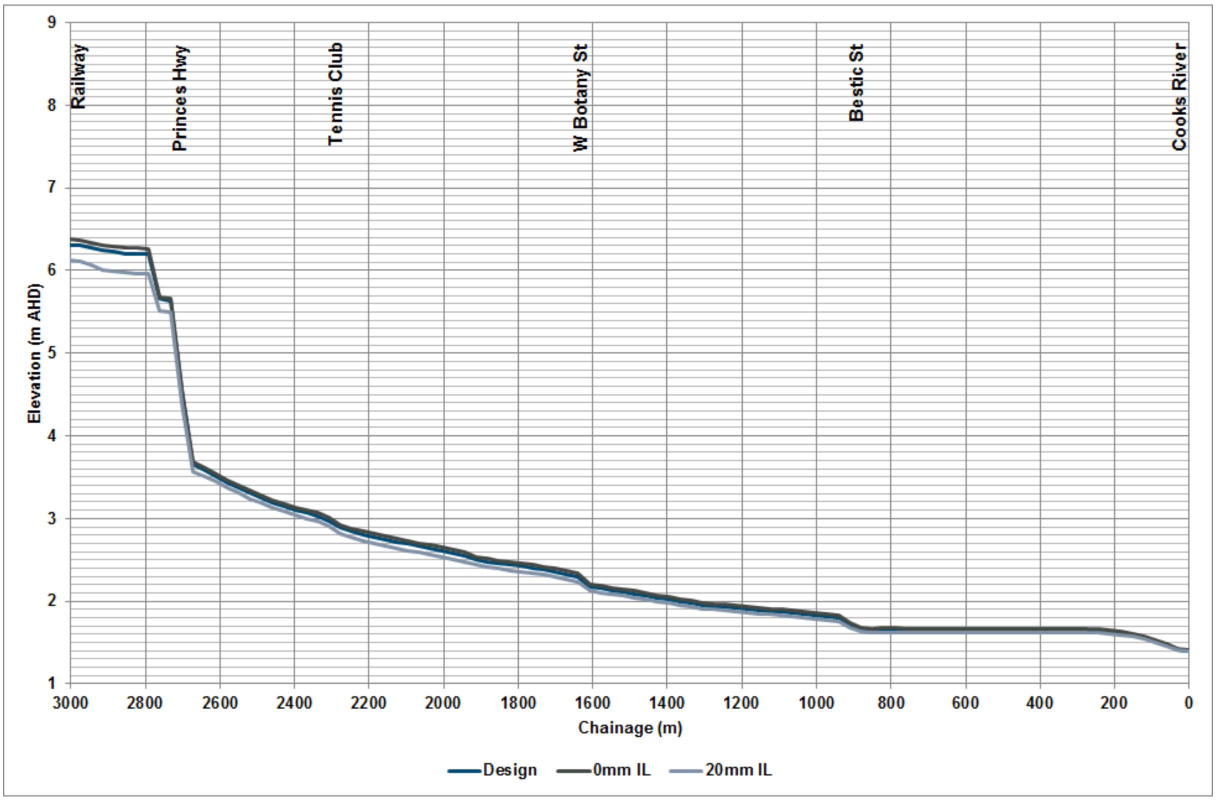

Figure 7-7 Expected Impact of Adopted Rainfall Losses along the Lower Muddy Creek

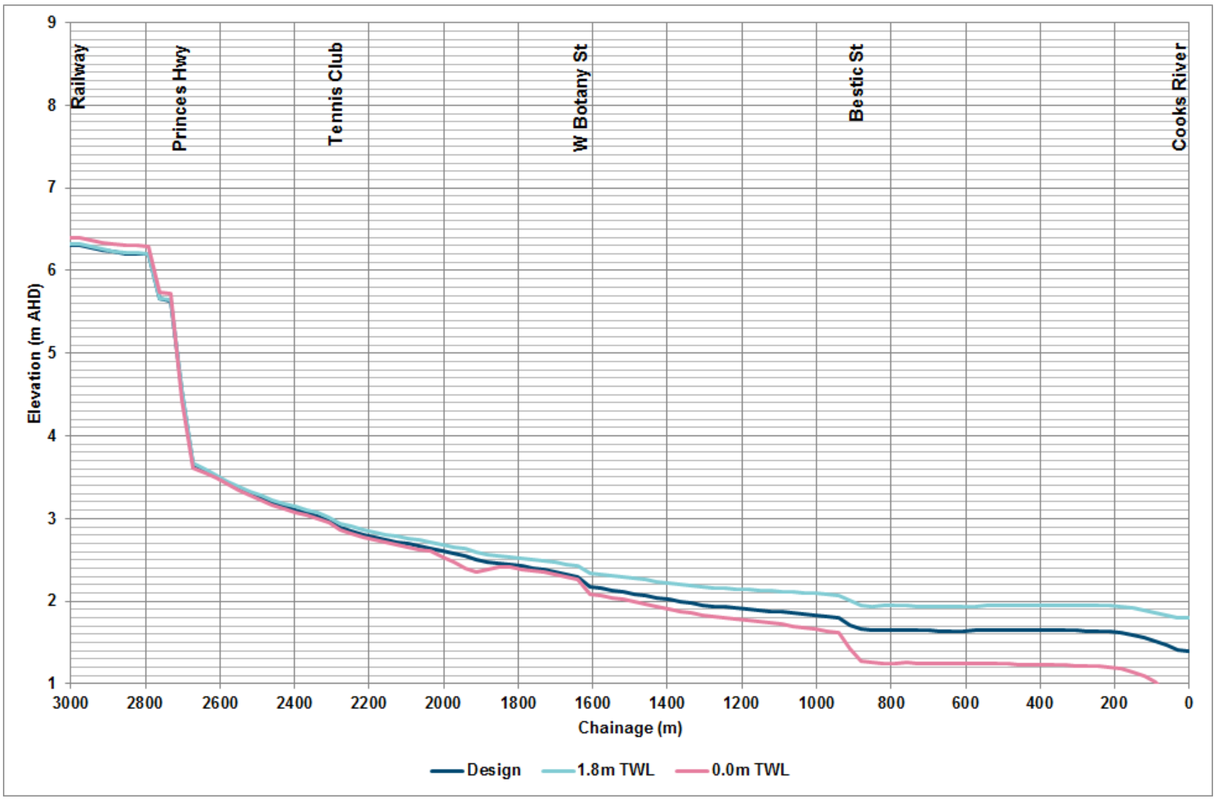

Figure 7-8 Impact of Adopted Downstream Boundary along the Lower Muddy Creek

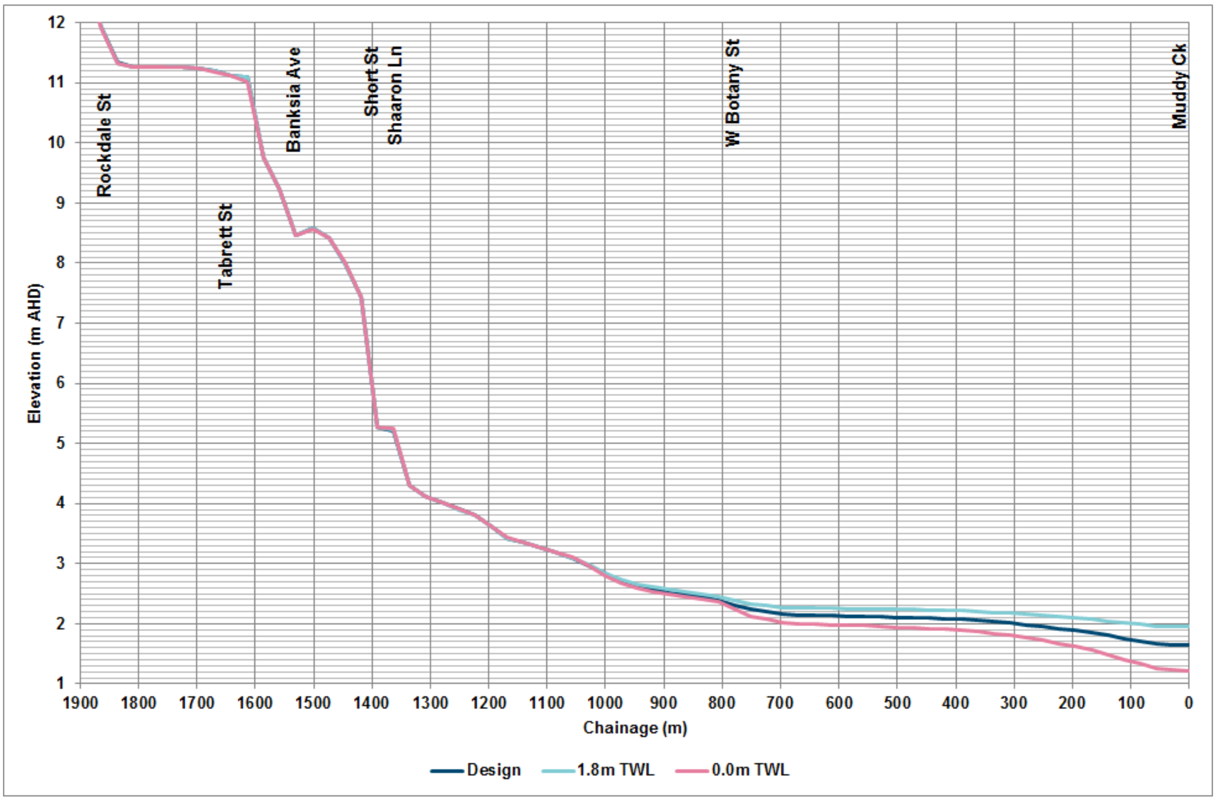

Figure 7-9 Impact of Adopted Downstream Boundary along Spring Street Drain

- The duration of the event;
- The potential blockage of the outlet structures; and
- The coincident tide and/or surge conditions within Botany Bay.

The full range of event durations was simulated for the 1% AEP event, with three scenarios considered, the results of which are presented in Table 7-1:

- The adopted design conditions of a fixed high tailwater level in Botany Bay (1.4m AHD) with an initial water level of 1.4m AHD;
- A fixed low tailwater level in Botany Bay (0.0m AHD) with an initial water level of 0.0m AHD; and
- The adopted design conditions with a 100% blockage of the Scarborough Ponds outlet structures.

It can be seen from Table 7-1 that the critical duration for flood levels in Scarborough Ponds is the 36-hour to 48-hour storm. Given the range of potential flood conditions in Scarborough Ponds, it is considered that the design flood results presented within this study (the 9-hour duration with a fixed high tailwater level) provide a reasonable estimate of a representative design flood condition for adoption. The peak level of 2.4m AHD sits between the 2.2m AHD and 2.6m AHD (low and high tailwater) peak levels for the critical duration storms. The addition of a 0.5m freeboard to the 2.4m AHD level is similar to the fully blocked outlet condition, as above around 2.9m AHD an overflow relief from the Ponds becomes active to the south.

Table 7-1 Scarborough Ponds 1% AEP Flood Levels

Duration	Design	Low TWL	Blockages
2h	2.1	1.7	2.2
3h	2.2	1.8	2.3
6h	2.3	1.9	2.4
9h	2.4	2.0	2.5
12h	2.4	2.1	2.5
18h	2.5	2.1	2.7
24h	2.5	2.1	2.7
30h	2.6	2.2	2.8
36h	**2.6**	**2.2**	**2.9**
48h	**2.6**	**2.2**	**2.9**
72h	2.5	1.9	2.9

7.6 Conclusion

The impact of the model sensitivity tests considered for the 1% AEP event is summarised in Table 7-2, in terms of modelled peak flood levels at the reporting locations identified in Figure 6-1. The results presented for the 50% and 100% blockage to the stormwater drainage network also include the blockage tests undertaken for the mainstream alignments.

Table 7-2 Modelled Peak Flood Levels (m AHD) for Sensitivity Tests

ID	Location	Modelled Condition for the 1% AEP Event								
		Design	+20% 'n'	-20% 'n'	50% block	100% block	0mm IL/CL	20mm IL	1.8m TWL	0.0m TWL
1	Paine St	13.9	13.9	13.9	14.0	14.0	13.9	13.9	13.9	13.9
2	Queen Vic St	14.5	14.5	14.5	14.5	14.6	14.5	14.5	14.5	14.5
3	Wolseley St	12.4	12.4	12.4	12.6	12.7	12.4	12.3	12.4	12.4
4	Cadia St	8.6	8.6	8.5	9.2	9.2	8.6	8.5	8.6	8.6
5	Warialda St	8.2	8.2	8.2	9.2	9.2	8.3	8.1	8.2	8.2
6	Princes Hwy	5.7	5.8	5.6	6.3	6.3	5.8	5.6	5.7	5.7
7	Chapel St	2.8	2.9	2.7	2.8	2.8	2.9	2.7	2.9	2.8
8	Caravan Park	1.9	1.9	1.8	1.9	1.9	1.9	1.8	2.1	1.7
9	Boating Club	1.6	1.7	1.6	1.6	1.6	1.7	1.6	2.0	1.2
10	Cooks River	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.8	0.2
11	Rockdale Stn.	20.3	20.3	20.2	20.7	20.9	20.3	20.1	20.3	20.3
12	Railway St	18.0	18.1	18.0	18.1	18.1	18.0	18.0	18.0	18.0
13	Monahan Ave	11.4	11.4	11.3	11.6	11.7	11.4	11.3	11.4	11.4
14	Curtis St	18.2	18.2	18.2	18.3	18.3	18.2	18.2	18.2	18.2
15	Short St	7.5	7.5	7.5	7.6	7.7	7.5	7.5	7.5	7.5
16	W Botany St	2.4	2.4	2.4	2.4	2.4	2.4	2.3	2.4	2.3
17	S'boro. Ponds	2.4	2.4	2.4	2.4	2.4	2.4	2.3	2.4	2.0

8 Climate Change Analysis

8.1 Potential Climate Change Impacts

8.1.1 Ocean Water Level

The NSW Sea Level Rise Policy Statement (DECCW, 2009) provided projected increases in mean sea level for NSW of 0.4m and 0.9m, by the years 2050 and 2100 respectively. These increases are no longer prescribed by the state government but have been adopted for the purpose of this study in the absence of other suitable recommendations. Therefore, design ocean boundaries have been raised by 0.4m and 0.9m to assess the potential impact of sea level rise on flood behaviour in the Muddy Creek catchment.

8.1.2 Design Rainfall Intensity

Current research predicts that a likely outcome of future climatic change will be an increase in flood producing rainfall intensities. Climate Change in New South Wales (CSIRO, 2004) provides projected regional changes in rainfall intensities for each season and annually for the years 2030 and 2070. The Muddy Creek catchment falls into the South-East region of NSW where compared to other regions in the state, projected increases are not as significant. It has been projected that 2.5% AEP 24 hour duration annual rainfall depths will increase by more than 5% by the year 2030 and 2070 in the study catchment. The 2.5% AEP 72 hour duration annual rainfall depth projections are increases of 10% for the year 2030 and 3% for the year 2070.

The NSW Government has also released a guideline (DECCW, 2007) for Practical Consideration of Climate Change in the floodplain management process that advocates consideration of increased design rainfall intensities of up to 30%.

In line with this guidance note, additional tests incorporating a 10% and a 30% increase to design rainfall have been undertaken. The design rainfall for the 0.5% AEP is around 10% higher than those of the 1% AEP, so comparison of these two events provides an appropriate assessment for potential impacts of increased design rainfall depths of 10%. Additional simulations have also been undertaken to assess the 30% increase.

8.2 Climate Change Model Conditions

The range of model simulations that were undertaken in order to assess the potential impact of future climate change, both in terms of increased sea levels in Botany Bay and increased rainfall intensities in the study catchment, are summarised in Table 8-1

8.3 Climate Change Results

Longitudinal profiles (see Figure 6-1 for alignments) showing the various climate change scenario flood levels along the upper and lower reaches of Muddy Creek are shown in Figure 8-1 and Figure 8-2. Similar longitudinal profiles along Spring Street Drain and Scarborough Ponds are shown in Figure 8-3 and Figure 8-4.

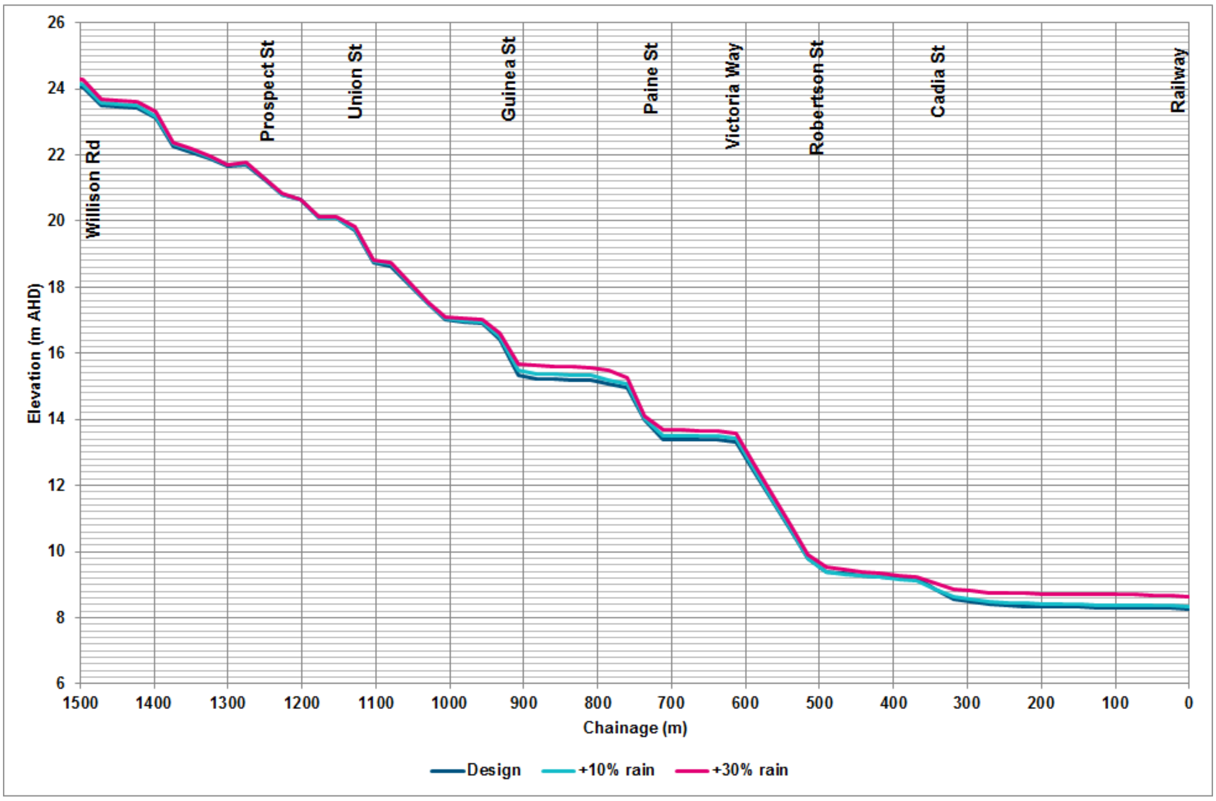

Figure 8-1 Long Section along the upper Muddy Creek for Climate Change Events

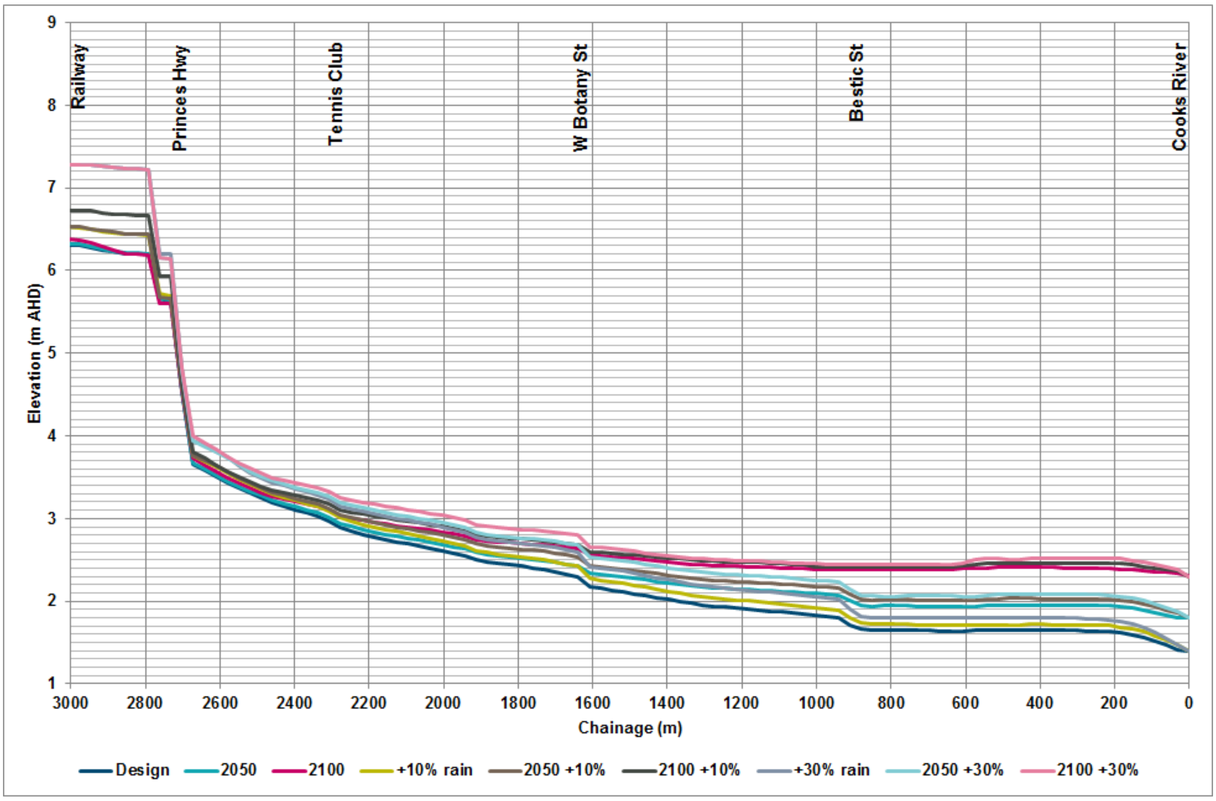

Figure 8-2 Long Section along the lower Muddy Creek for Climate Change Events

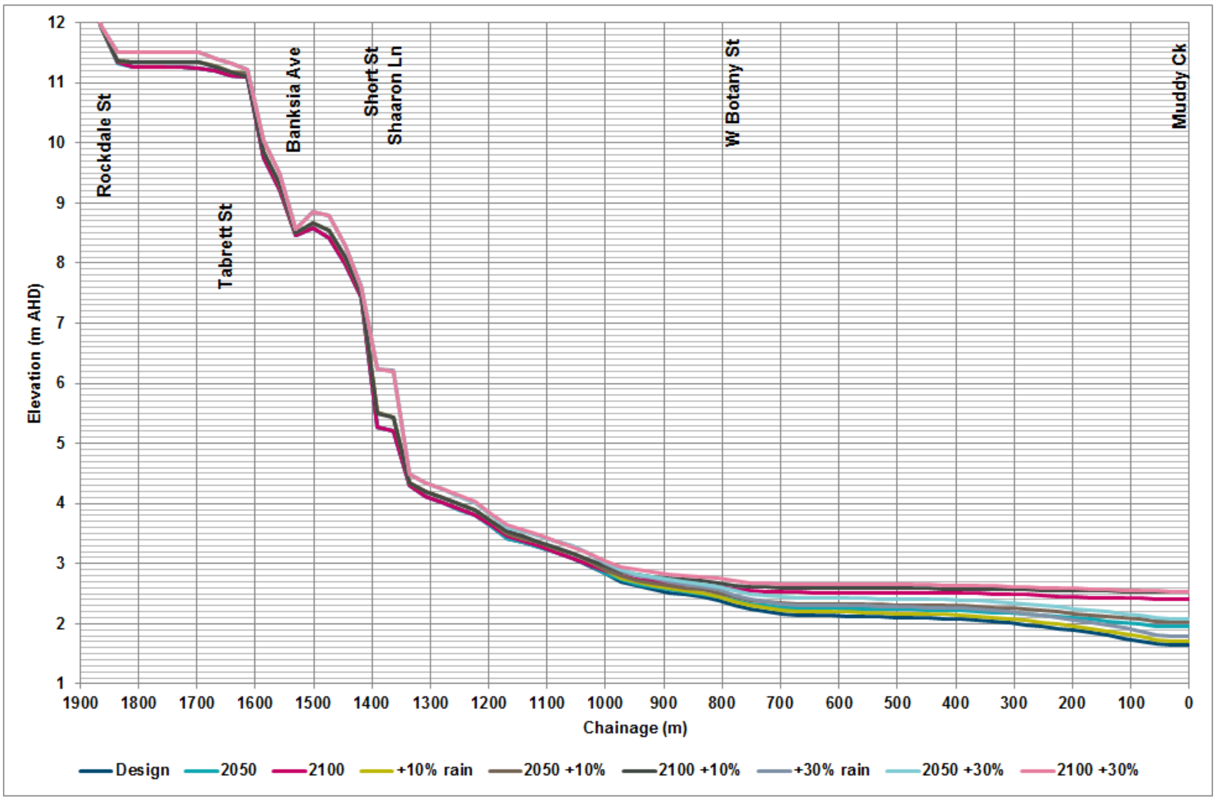

Figure 8-3 Long Section along the Spring Street Drain for Climate Change Events

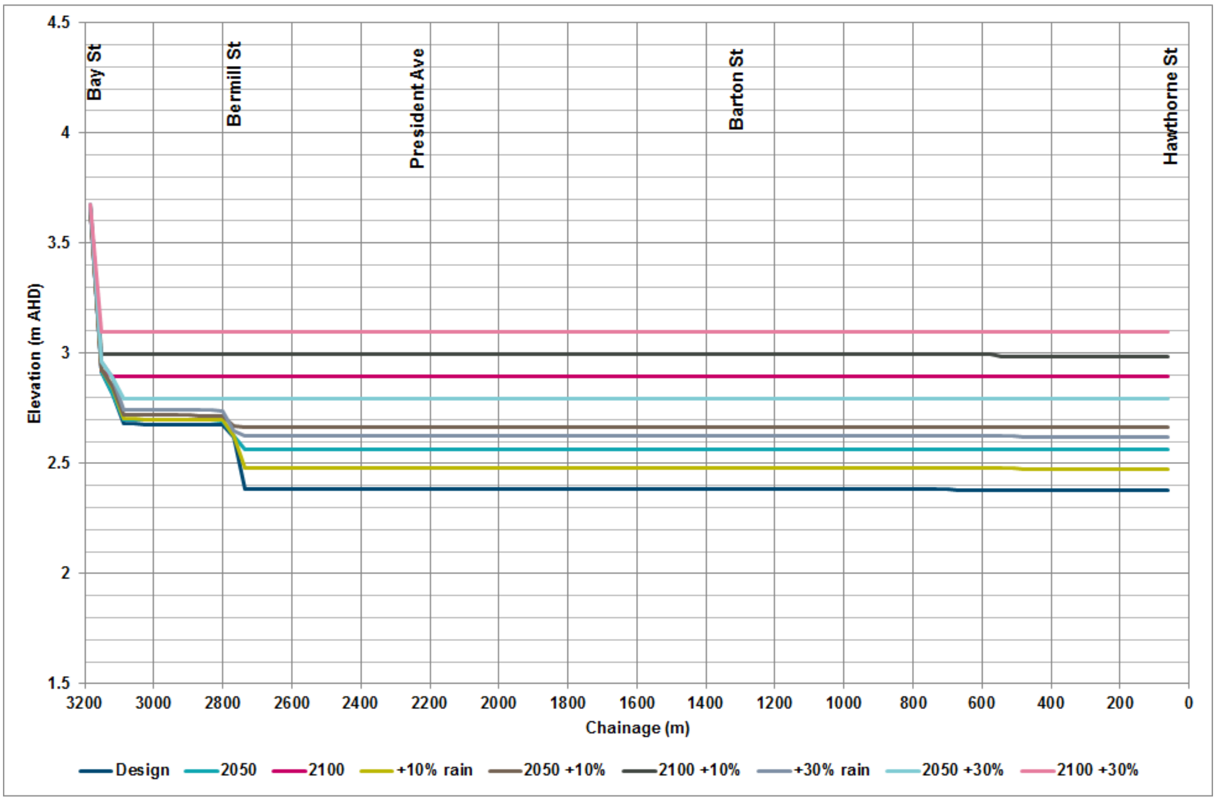

Figure 8-4 Long Section along the Scarborough Ponds for Climate Change Events

Table 8-1 Modelled Climate Change Scenarios

ID	Name	Event Magnitude	Event Duration	Tailwater Level (m AHD)
1	100y2h 2050	1% AEP	2h	1.8
2	100y9h 2050	1% AEP	9h	1.8
3	100y2h 2100	1% AEP	2h	2.3
4	100y9h 2100	1% AEP	9h	2.3
5	200y2h	1% AEP +10%	2h	1.4
6	200y9h	1% AEP +10%	9h	1.4
7	200y2h 2050	1% AEP +10%	2h	1.8
8	200y9h 2050	1% AEP +10%	9h	1.8
9	200y2h 2100	1% AEP +10%	2h	2.3
10	200y9h 2100	1% AEP +10%	9h	2.3
11	100y2h_plus30	1% AEP +30%	2h	1.4
12	100y9h_plus30	1% AEP +30%	9h	1.4
13	100y2h_plus30 2050	1% AEP +30%	2h	1.8
14	100y9h_plus30 2050	1% AEP +30%	9h	1.8
15	100y2h_plus30 2100	1% AEP +30%	2h	2.3
16	100y9h_plus30 2100	1% AEP +30%	9h	2.3

The impact of the climate change modelling considered for the 1% AEP event is summarised in Table 8-2, in terms of modelled peak flood levels at the reporting locations identified in Figure 6-1.

Table 8-2 Modelled Peak Flood Levels (m AHD) for Climate Change Conditions

ID	Location	Modelled Condition for the 1% AEP Event								
		Design	2050 TWL	2100 TWL	+10% rain	2050 +10%	2100 +10%	+30% rain	2050 +30%	2100 +30%
1	Paine St	13.9	13.9	13.9	13.9	13.9	13.9	14.0	14.0	14.0
2	Queen Vic St	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5
3	Wolseley St	12.4	12.4	12.4	12.4	12.4	12.4	12.5	12.5	12.5
4	Cadia St	8.6	8.6	8.6	8.6	8.6	8.6	8.9	8.9	8.9
5	Warialda St	8.2	8.2	8.2	8.4	8.4	8.4	8.7	8.7	8.7
6	Princes Hwy	5.7	5.7	5.7	5.8	5.8	5.9	6.2	6.2	6.2
7	Chapel St	2.8	2.9	3.0	2.9	3.0	3.1	3.1	3.1	3.2
8	Caravan Park	1.9	2.1	2.4	2.0	2.2	2.5	2.1	2.3	2.5
9	Boating Club	1.6	2.0	2.4	1.7	2.0	2.5	1.8	2.1	2.5
10	Cooks River	1.4	1.8	2.3	1.4	1.8	2.3	1.4	1.8	2.3
11	Rockdale Stn.	20.3	20.3	20.3	20.4	20.4	20.4	20.6	20.6	20.6
12	Railway St	18.0	18.0	18.0	18.1	18.1	18.1	18.1	18.1	18.1
13	Monahan Ave	11.4	11.4	11.4	11.4	11.4	11.4	11.6	11.6	11.6
14	Curtis St	18.2	18.2	18.2	18.2	18.2	18.2	18.3	18.3	18.3
15	Short St	7.5	7.5	7.5	7.6	7.6	7.6	7.7	7.7	7.7
16	W Botany St	2.4	2.4	2.6	2.4	2.5	2.7	2.5	2.6	2.7
17	S'boro. Ponds	2.4	2.6	2.9	2.5	2.7	3.0	2.6	2.8	3.1

9 Conclusions

The primary objective of the study was to undertake a detailed flood study of the Muddy Creek, Spring Street Drain and Scarborough Ponds catchments and to establish models as necessary for design flood level prediction

In completing the flood study, the following activities were undertaken:

- Compilation and review of existing information pertinent to the study;
- Development and calibration of appropriate hydrologic and hydraulic models;
- Calibration of the developed models using the available flood data, including the recent events of 1993, 1998 and 2014; and
- Prediction of design flood conditions in the study area and production of design flood mapping series.

The principal outcome of the flood study is the understanding of flood behaviour in the study area and in particular design flood level information. The study provides updated and more detailed flooding information than the previous studies, to be used to inform floodplain risk management within the study area.

Flooding to the west of the railway is located along a number of gully lines that drain to Muddy Creek and Spring Street Drain. There are a number of locations along which the overland flow path alignment is not within the roadway, but instead traverses blocks of residential development. The floodway is usually situated along the yards to the rear of the properties and/or where flow is funnelled between buildings. The affected locations include:

- Properties located along two flow paths between Botany Street and High Street;
- The rear of properties located along High Street and Mill Street;
- Properties located along the flow path between Short Street and Edgehill Street;
- Properties located along the flow path between Guinea Street and Robinson Street;
- Properties located along the flow path between Percival Street and Queen Victoria Street;
- The rear of properties located along Robertson Street and Warialda Street;
- The rear of properties located along Campbell Street and Lymington Street;
- Properties located along two flow paths between Northbrook Street and Beaconsfield Street;
- Properties located along the flow path between Dunmore Street South and Warialda Street;
- Properties located along the flow path between Goyen Avenue and Watkin Street;
- The rear of properties located along Frederick Street;
- Properties located along the flow path between Heathcote Street and Arlington Street;
- The rear of properties located along Oswin Lane and Gloucester Street; and

- The rear of properties located along Godfrey Street and Bowmer Street.

The areas between the Princes Highway and Short Street, between Terry Street and Spring Street and between the Princes Highway and Cross Street (on the eastern side of the railway) also experience similar issues to the above. Further downstream the flooding problem areas are typically limited to locations where the capacity of the drainage channels is significantly exceeded. Such areas include:

- The properties along Spring Street Drain between Shaaron Court and West Botany Street;
- The properties along Muddy Creek between Harrow Road and Bay Street; and
- Properties along West Botany Street where local drainage to Muddy Creek is exceeded.

There are also a number of properties bordering the Scarborough Ponds that are affected.

There are also a number of areas that are particularly exposed to increased flood risk through potential blockage of structures, including:

- Properties situated between Prospect Street and Union Lane (~0.5m);
- Properties situated between Guinea Street and Cadia Street (~0.5m);
- Properties along Warialda Street (~1.0m);
- Properties situated between the railway and the Princes Highway (~1.1m);
- Properties around the Railway Street – Frederick Street intersection (~0.6m);
- Properties situated between Roach Street and the railway (~0.6m to 0.7m); and
- Properties situated between the Princes Highway and Short Street (~0.3m to 0.4m).

Given the significant increase in flood risk across these areas under potential blockage scenarios the incorporation of blockage allowances within the design flood levels should be considered for flood planning purposes, particularly for the Warialda Street to Princes Highway section. It is expected that management of food risk within this area will be one of the key focuses of future floodplain risk management activities.

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11 Acknowledgements

The advice and information provided by Rockdale City Council, the NSW Office of Environment and Heritage and members of the Floodplain Management Committee is gratefully acknowledged.

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