Woy Woy Floodplain Risk Management Study – Technical Volume

Final Report

Central Coast Council

Final Report

April 2023

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Woy Woy Floodplain Risk Management Study – Technical Volume

Final Report

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1 Glossary of terms

Annual Exceedance Probability (AEP)	The probability of an event occurring or being exceeded in any given year. Usually expressed as a percentage.
Aquifer	An underground layer of water-bearing permeable rock or unconsolidated material from which water can be extracted.
Australian height datum (AHD)	A common national plane of level corresponding approximately to mean sea level.
Australian Rainfall and Runoff (ARR)	A national guideline document, data and software suite that can be used for the estimation of design flood characteristics in Australia.
Average recurrence interval (ARI)	The average time period between occurrences equalling or exceeding a given value. ARI is another way of expressing the likelihood of occurrence of a flood event. ARR discourages the use of the terminology ARI which are often seen in previous flood studies, as it leads to confusion with the public for rare events.
Cadastre, cadastral base	Information in map or digital form showing the extent and usage of land, including streets, lot boundaries, water courses etc.
Catchment	An area where water is collected to a location. This could be by to the natural landscape or by storm drainage network.
Design Rainfall	Design rainfalls are a probabilistic or statistically-based estimate of the likelihood of a specific rainfall depth being recorded at a particular location within a defined duration. It is generally classified by Annual Exceedance Probability (AEP) or Exceedance per Year (EY)

Discharge	The rate of flow of water measured in terms of volume per unit time, for example, cubic metres per second (m^3/s). Discharge is different from the speed or velocity of flow, which is a measure of how fast the water is moving for example, metres per second (m/s).
Exceedance Per Year (EY)	Events more frequent than 50% AEP is expressed as X Exceedances per Year (EY) as expressing frequency in AEP is misleading. ARR provides an example "2 EY is equivalent to a design event with a 6 month recurrence interval when there is no seasonality in flood occurrence."
Flood	Relatively high stream flow which overtops the natural or artificial banks in any part of a stream, river, lake or dam and/or overland flooding associated with major drainage before entering a watercourse and/or coastal inundation resulting from super-elevated sea levels and/or waves overtopping coastline defences excluding tsunami.
Flood fringe areas	The remaining area of flood prone land after floodway and flood storage areas have been defined.
Flood planning area (FPA)	A flood planning area is the area where flood related development controls may be applied for development. The flood planning area is the area where the topography is below the Flood Planning Level.
Flood planning level (FPL)	Typically, the height used to set floor levels for development of properties in flood prone areas.
Floodplain	Area of land, which is subject to inundation by floods up to, and including the probable maximum flood event, that is, flood prone land.
Floodplain risk management options	The measures that might be feasible for the management of a particular area of the floodplain. Preparation of a floodplain risk management plan requires a detailed evaluation of floodplain risk management options.

Floodplain risk management plan	A management plan developed in accordance with the principles and guidelines of the NSW Government Floodplain Management Manual 2005. Usually includes both written and diagrammatic information describing how particular areas of flood prone land are to be used and managed to achieve defined objectives.
Flood Hazard	The potential loss of life, injury and economic loss caused by future flood events. The degree of hazard varies with the severity of flooding and is affected by flood behaviour (extent, depth, velocity, isolation, rate of rise of floodwaters, duration), topography and emergency management.
Flood storage areas	Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. The extent and behaviour of flood storage areas may change with flood severity and loss of flood storage can increase the severity of flood impacts by reducing the natural flood attenuation. Hence it is necessary to investigate a range of flood sizes before defining flood storage areas.
Floodway areas	Those areas of the floodplain where a significant discharge of water occurs during floods. They are often aligned with naturally defined channels. Floodways are areas that, even if only partially blocked, would cause a significant redistribution of flood flow, or significant increase in flood levels.
Freeboard	A factor of safety usually expressed as a height above the adopted flood level thus determining the flood planning level. Freeboard tends to compensate for factors such as wave action, localised hydraulic effects and uncertainties in the design flood levels.
Geographical information systems (GIS)	A system of software and procedures designed to support the management, manipulation, analysis, and display of spatially referenced data.

Groundwater	Water that is located beneath the ground surface in soil pore spaces and fractures of lithologic formations.
LiDAR	A surveying method which is widely used to surface topography. It is measuring the reflection with a sensor by targeting with laser light.
Numerical/computer models	The mathematical representation of the physical processes involved in runoff and stream flow. These models are often run on computers due to the complexity of the mathematical relationships between runoff, stream flow and the distribution of flows across the floodplain.
Probable Maximum Flood (PMF)	The largest flood that could conceivably occur at a particular location, usually estimated from probable maximum precipitation. Generally, it is not physically or economically possible to provide complete protection against this event. The PMF defines the extent of flood prone land, that is, the floodplain.
Probable Maximum Precipitation (PMP)	The theoretical maximum precipitation for a given duration under modern meteorological conditions.
Runoff	The amount of rainfall that ends up as stream flow, also known as rainfall excess.
Topography	A surface which defines the ground level of a chosen area. Ground levels are typically presented in relation to the Australian Height Datum.
Unconfined aquifer	Aquifer with an upper boundary being the water table or phreatic surface.

2 Purpose

Theis report is intended as an additional technical volume of the *Woy Woy Floodplain Risk Management Study* (DHI & Rhelm, 2022). It contains a majority of the highly technical aspect of the project including details on modelling methodologies and results.

This document should be read in conjunction with:

- The Woy Woy Floodplain Risk Management Study (DHI & Rhelm, 2022)
- The Woy Woy Floodplain Risk Management Plan (Rhelm, 2022)

3 Australian Rainfall and Runoff

3.1 ARR 2019

The Australian Rainfall and Runoff 1987 (ARR 1987) has undergone some fundamental changes since its last major release in 1987. The main factor driving the changes of the new ARR are based around the uncertainty associated with the design rainfalls. These uncertainties can arise from various sources, including errors in the data due to short record length, gaps in the data, limitations in the adopted methods, etc.

A major change has been the appreciation of the fact that temporal patterns exhibit significant variability between rainfall events of similar magnitude, and that the adopted pattern can have significant effect on the estimated peak flow. The new Australian Rainfall and Runoff 2019 (ARR 2019) therefore recommends using Monte Carlo or ensemble modelling techniques, which try to overcome the problems associated with this simplification by using an ensemble of temporal patterns.

The new ARR2019 approach reflects on the significant uncertainties that exist in predicting extreme rainfall and runoff events. The core advantage of the analysis is a better understanding of the uncertainties associated with highly variable physical processes and the challenges associated with predicting extreme events based upon a limited historical record. Computational effort and increased complexity of analyses aside, there is also an additional burden of responsibility for the decision maker to consider risk and make decisions based upon a range of likely events rather than a specific single event.

In summary, the revised ARR2019 has introduced the following main changes to design rainfall estimation and flood hydrograph estimation:

- a largely updated dataset upon which rainfall IFD (intensity-frequency-duration) estimates are based,
- fundamental changes to the design rainfall temporal patterns,
- updated initial and continuing losses,
- incorporation of preburst rainfalls, and
- updated areal reduction factors.

3.2 Adopted Modelling Approach to Derive Design Events

It is not feasible and practical to run all design scenarios (7AEPs x 8 durations x 10 temporal patterns + 8 durations of PMF) with the 2D model. Instead, the variability in rainfall and associated uncertainties in resulting runoff and inundation can be accounted for by undertaking parts of the hydrologic analysis on a sub-catchment and use findings to conduct further analyses on the entire peninsula. A lumped hydrological model for the Kahibah Creek catchment was set up to run all design scenarios to narrow down which temporal pattern to be applied for the 2D model of the entire study area. The selected durations and selected temporal pattern were run with the 2D model to determine which durations are critical for inundation in the study area.

More detail is given in **Section 7.6**.

3.3 OEH guide for incorporation of ARR2019 in NSW

In January 2019, the NSW Office of Environment and Heritage (OEH) released a guide "Floodplain Risk Management Guide - Incorporating 2016 Australian Rainfall and Runoff in studies." This guide is intended to assist councils to transition to ARR2019 with their floodplain risk management plans. This guide explains the major differences between ARR 1987 and ARR2019 and provides specific advice and techniques how to incorporate ARR 2019 in NSW. It also describes rainfall loss estimation approaches specific to NSW, in contrast to the national approaches outlined in ARR 2019. It is however noted that this recommendation is not particularly relevant in this study as losses are also modelled dynamically by groundwater components in the model.

4 Data Compilation and Literature Review

Most data were provided by Council as part of the *Woy Woy Integrated Water Management and Case Study Everglades Catchment* (DHI, 2021), including:

- Reports of previous studies
- Groundwater records
- LiDAR topographic data
- Photos of nuisance flooding on the peninsula
- GIS layers
- Old design drawings of cross-sections at Main Drain
- Drainage design plans

The list of previous study reports and GIS layers previously provided by Council are summarised in**Table A 1** and **Table A 2** respectively in **Appendix A**.

4.1 Site Visits

Several site inspections were conducted, mainly focusing on the Kahibah Creek and Everglades catchments, the escarpment above the Everglades catchment and the Woy Woy town centre. These site inspections were attended by senior DHI, Rhelm and/or Council staff and were carried out on the following dates:

- 13/02/2019: Inception Walkover
- 25/09/2019: Structure survey and introduction to the maintenance program by a field officer
- 10/02/2020: Post-rainfall event
- 20/07/2020: Floor level survey scoping

In addition, DHI had undertaken inspections as a part of *Woy Woy Integrated Water Management and Case Study Everglades Catchment Study* (DHI, 2021) prior to this study.

4.2 Previous studies

Detailed literature review focusing on groundwater and nuisance flooding in the Everglades catchment was carried out as part of the *Woy Woy Integrated Water Management and Case Study Everglades Catchment Study* (DHI, 2021).

4.2.1 Ettymalong Creek Flood Study and Environmental Effects Assessment (Willing & Partners, 1989)

Ettymalong Swamp, which used to be located at the south-west of Umina, was reclaimed and rezoned for subdivision for subdivision into residential blocks in the 1980s. Flooding issues became serious since development of the residential area commenced. The development was approved with the condition of undertaking the stormwater drainage works in Ettymalong Creek to reduce flood levels. Prior to filling, the available storage in the swamp was estimated to be of the order of 150,000m^3.

The study carried out an independent analysis of the environmental effects of proposed stormwater drainage works which were prepared by an engineering firm commissioned by the developer. Since flood levels produced by the 1% AEP design flood were lower than those produced by the 6th January 1989 flood due to the rainfall distribution, the study used a combination of the January 1989 flood and the May 1974 tide as the design flood for investigations.

The report concluded the proposed drainage works would not reduce the flood level below the lowest existing floor levels in Neera Road and recommended further enlargement of the channel of the southern branch of Cowper Creek.

The report also contains the information about Palmtree Grove Detention Basin, and these are summarised in the Woy Woy FRMS (DHI, 2022).

4.2.2 Kahibah Creek Flood Study (Willing & Partners, 1991)

The study assessed the nature and extent of flooding in the catchments of five branches of Kahibah Creek.

Flood behaviour in the system is influenced by:

- The storage effects of remnants of the swamps at the foot of the escarpment;
- Timing of storms over the catchment and by tidal influences.

The study found that the 2hr duration design event produced higher flood levels than the 1.5 hr storm which had the highest peak discharge.

4.2.3 Kahibah Creek Floodplain Management Study (Willing & Partners, 1991)

Severe flooding was experienced in the Kahibah Creek catchment in 1988, 1989 and 1990. The floodplain management study was undertaken as the second stage of the management process for the Kahibah Creek catchment after the flood study, to investigate the feasibility of various floodplain management strategies. The study considered the existing catchment conditions at the time and urbanised catchment scenario conditions. The urbanised scenario assumed a 40% increase of the impervious areas on the flatter sub-catchments in the northern and eastern parts of the study area.

Using the 1% AEP event as the design event, it was estimated that 18 houses are flood liable under the existing conditions at the time and the average annual damage cost was estimated to be $16,000.

Both the structural and non-structural measures were investigated as mitigation options and combinations of these options were modelled.

The study considered the following for flood mitigation under the existing catchment conditions such as:

- Construction of the bridge at Mt Ettalong Road
- Channel works at Neera Road
- Enlargement of the Ettymalong Creek channel between Cowper Road bridge and McLaurin Road
- Enlargement of the Greenhaven Drive arm of the creek
- Removal of silt from Iluka Lagoon and installation of a sediment trap upstream
- building controls for new develop to ensure the adequate freeboard of 500mm at least
- Zoning restrictions to preserve floodways. to prevent filling in flood storage areas and to prevent filling within property boundaries until other compensatory works are carried out
- A flood warning system

The following additional measures to reduce flood levels to below house floor levels in the catchment were considered for fully urbanised conditions.

- Doubling the flow capacity of the culverts at Brisbane Avenue and Calypta Road
- Lowering the invert along the Australia Avenue arm of the creek between Australia Avenue and Osborne Avenue and replacing of the culvert at McEvoy Avenue
- Tripling the capacity of the culvert at Etta Road east

Additional three measures were provided to increase freeboard:

- Construction of a retarding basin with a capacity of 50,000 m3 at the Council depot site west of the Ettymalong Swamp arm.
- Excavation of an extra cutting through Mt Ettalong Road
- Acquisition or raising of low-lying houses with marginal freeboard

The report also contains surveyed floor levels.

4.2.4 Kahibah Creek Floodplain Management Plan (Willing & Partners, 1996)

Following *Kahibah Creek Floodplain Management Study* (Willing & Partners, 1991), the Kahibah Creek Floodplain Management Plan was prepared. The Kahibah system floodplain was divided into eleven management areas and the plan was prepared to establish development controls which are required to complement the proposed structural work to manage the flood risk.

The high priority works recommended by Kahibah Creek Floodplain Management Study were already implemented by the time when the plan was prepared. They are:

- Construction of bridge at the downstream of Mt Ettalong Road and the related channel works
- Widening of Greenhave Drive Channel
- Enlargement of Neera Road Channel
- Enlargement of Ettalong Creek Channel between Cowper Road and McLaurin Road

The report contains the recommended works to excavate the channels.

4.2.5 Kahibah Creek Review of Floodplain Management Measures (Willing & Partners, 2001)

This report was prepared for Council to provide advice on the effectiveness of the structural floodplain management measures, such as culvert augmentation, channel excavation, rock lining of banks, implemented after the recommendations provided in the Kahibah Creek Floodplain Management Plan (Willing & Partners, 1996) and the effect of vegetation at the channel and reserve on the flood risk.

The report reviewed the implemented structural work and vegetation management works in the Kahibah Creek and recommended high priority works for the Ettalong Swamp arm and Australia Avenue Arm of the system to increase the flow capacities.

It also warned that excessive clearing of the channel may lead to lower local flood levels but increase the level of flooding further downstream.

The report contains the cross-sections along Ettymalong Swamp arm, surveyed in October 2000.

4.2.6 Woy Woy Flood Risk Management Study (DHI, 2010)

This flood study was undertaken to determine the existing flood behaviour of flood prone areas for a range of flood risk levels from the 50% Annual Exceedance Probability (AEP) event through to the Probable Maximum Flood (PMF). Flood behaviour was determined for flood prone areas using mathematical modelling tools developed specifically for the study. Catchment groundwater behaviour, runoff generation, overland flow, channel flow and pipe flow were calculated using MIKE SHE. The model allows a distributed, physically based approach to rainfall runoff, with rainfall time series applied directly to a two-dimensional grid representation of the catchment surface.

The model was calibrated to the 1988 storm event using flood depths obtained from community consultation and council maps indicating areas historically prone to flooding.

The flood model predictions indicate that in many areas of the catchment the groundwater table rises to the ground surface, preventing infiltration of rainfall and creating significant areas of ponded water. The existing flow channels and stormwater drainage conduits can be effective in removing this water if the ponded areas are connected to the drainage system and the drainage system is operating effectively.

The Kahibah Creek catchment was not included in the study.

This study was uses as a basis for further developing the current Floodplain Risk Management Study and Plan.

4.2.7 Brisbane Water Foreshore Flood Study (Cardno, 2013)

This study aimed at establishing water levels in Brisbane Water for the full range of flood and ocean events due to various natural conditions such as catchment flooding, heavy rainfall directly onto Brisbane Water, elevated ocean levels and local winds. The study also considered joint probability of combination of these processes to some extent.

The study concluded that severe ocean storms cause the highest water levels except Fagans Bay where catchment floods of the same probability cause higher levels.

The study established downstream boundary water levels to be used for individual creek flood studies. This corresponds to the 1% exceedance level, which will not be exceeded during any creek flood event with 99% confidence.

The report summarised the residence responses to the distributed questionnaires. Some of them are located at the foreshore within the Woy Woy peninsula.

4.2.8 Drainage Studies

Report Stormwater Investigations to Catchment Blackwall Mountain, Springwood Street, Waitangi Street, Warrigal Street, Wyalong Street, Memorial Avenue, Umina (Giammarco Engineering, 1989) investigated the extent of stormwater flooding in severe flood events, particularly those experienced in April 1988 and January 1989. The report provides a general description of the catchment and flooding behaviour and recommends strategies and drainage options to relieve flooding problems.

Woy Woy, Umina, Ettalong Peninsula, Drainage Strategy Study, (Webb McKeown & Associates, 1992) was commissioned by Gosford City Council to assist in planning possible future trunk drainage works and in the preparation of a Development Control Plan. The study deteimned that the existing pipe drainage system then had capacities ranging from zero to the 1 in 100 AEP flood. The study also determined the catchments of the existing drainage system for the development at that time and also future drainage catchments with drainage reserves and easements. The required pipes and culverts were estimated for different scenarios to meet the contemporary design standards and costs were calculated.

Ross-Rowan Catchment, Woy Woy Channel to Ocean Beach Road, Trunk Drainage Management Study and Management Plan (Webb McKeown & Associates, 1993) is a trunk drainage management study and plan for the Ross-Rowan Catchment. It includes a review of the existing drainage system and presents trunk drainage options with cost estimates, impacts and benefits of proposed works.

Woy Woy Peninsula Catchments 'B' and 'C' Drainage Study, (Webb McKeown & Associates, 1996) investigated further drainage strategies and concept design for catchments B and C. Various allowable surface water ponding options and two alignments of the pipe system were considered. The study includes detailed survey to identify low areas and locate existing utilities and prepared a revised drainage strategy.

Woy Woy Peninsula - Catchments 'P' and 'O' Drainage Investigation - Draft Report (Issue 1), (Patterson Britton & Partners, 1997) identifies conceptual drainage options for catchments P and O at Woy Woy and recommends the preferred drainage concepts. The study covers catchment characteristics and a history of development as well as the existing drainage problems in the region. An investigation of current stormwater management practices and detailed modelling was undertaken for which a number of alternative drainage concepts were examined. These included open channel systems, piped drainage systems and a retarding basin combined with piped drainage systems.

Drainage Investigation Veron Road / Dulkara Road Catchment Umina / South Woy Woy, (Kinhill Engineers, 1999), investigates the extent of stormwater flooding and develops a drainage management plan to solve or relieve identified flood problems in the catchment. The existing system capabilities were investigated using the ILSAX model and a number of feasible options were developed to achieve council's stated design standard. The majority of flow problems investigated were found to be caused by development in natural flow paths and often where piped drainage system was capable of conveying only the 1 or 2 year ARI event. Solutions investigated mainly involved structural measures such as piped system upgrading and construction of detention basins.

Woy Woy Peninsula Catchments 'D' and 'E' Drainage Study (Ivan Tye and Associates, 2000) details further drainage investigations and prepare a concept design for catchments D and E. The 100 year ARI capacity trunk drainage options for catchment are investigated.

Everglades Lagoon System Precinct, Plan of Management (Kellogg Brown & Root, 2005) provides the framework for the short, medium and long term management of the Everglades Lagoon System Precinct.

4.2.9 Woy Woy Integrated Water Management and Case Study Everglades Catchment (DHI, 2021)

The study updated the groundwater model for the Woy Woy peninsula developed as a part of the previous *Woy Woy Peninsula Flood Study* (2010). The update includes

- Extension of the model domain to include the Kahibah Creek catchment
- Update of the topography using the 2013 LiDAR
- Recalibration of the model against the long-term groundwater level records including newly compiled data since the 2010 study.

The groundwater model was run with a long-term rainfall timeseries and the average sea level, for more than 100 years, to estimate the groundwater trend in the catchment.

Then, a flood model for the case study Everglades Catchment was derived by refining the Peninsula groundwater model and coupling to a stormwater drainage network model. The model was calibrated against a series of nuisance flooding events in 2017. The Everglades catchment lies in the north-western section of the peninsula and is prone to nuisance flooding.

Simulation of a series of nuisance flooding events in 2017 as well as a larger event in February 1990 which is the equivalent of the 1% to 0.5% AEP rainfall event revealed the following flooding characteristics at the Everglades catchment. The study assessed a selection of integrated management options for alleviating flooding in the Everglades catchment. This is further summarised in the Woy Woy FRMS (DHI, 2022).

It recommends that revision to Council's Black Spot Policy which currently restricts developments in the vicinity of the historically reported drainage issues would need to carefully consider any site specific black spot in the context of the flooding mechanics (e.g. groundwater driven flooding) and should utilise the groundwater information from this study.

4.3 GIS layers and topographic data

4.3.1 Topographic data

Topographic grid data was developed from the provided LiDAR 2013 data.

4.3.2 Heritage

Heritage layer is available at the portal for Sharing and Enabling Environmental Data (SEED) published by NSW Government.
https://geo.seed.nsw.gov.au/Public_Viewer/index.html?viewer=Public_Viewer&locale=en-AU

4.3.3 Buildings

Council confirmed that a shapefile of buildings was not available. DHI tried to find other sources without success. Therefore, a building shapefile was generated from Building Class (6) in LiDAR data.

Although this is unlikely to cover paved area of allotments, e.g. parking spots, this is the best available and affordable approach to generate paved areas in allotments.

4.3.4 Landuse map

Council confirmed a land use GIS layer was not available. Therefore, it was created manually using available GIS layers.

Council provided an additional spreadsheet which contained road widths and road pavement types to help generating road polygons using the road polyline shapefile. However, joining the spreadsheet table to the existing road shapefile was not feasible due to lack of identical IDs which can be used as a primary key for joining two tables.

Therefore, an approximate land use map was generated using shapefiles of Cadastre, Council Reserves, Bells Vegetation and National Parks.

4.4 Drainage Evaluation

The provided drainage data were missing information about pipe sizes and invert levels at numerous locations. **Figure C.1** in **Appendix C** shows the pipes missing a diameter and **Figure C.2** shows the locations (upstream and downstream ends of pipes) where invert levels are missing.

Given the magnitude of missing data, it was more appropriate to make assumptions on this information rather than undertaking a detailed survey of all pipes. Invert levels were interpolated from locations where estimated depths were available. If no depth information was available, the invert levels were estimated from the ground surface levels by maintaining a reasonable slope towards the outlet. Ground surface levels were estimated from LiDAR.

Piping under the railway near Brief Street

Several pipes and culverts were found in the shapefile along Brief Street towards the sea, as shown in **Figure 4.1**. DHI consulted Council on 11 July 2019, whether all these pipes are actively functioning.

In the late 1990s and early 2000s, Council's Flooding and Drainage section at the time completed many drainage studies as summarised in **Section 4.2.8**. One of the design plans which came out of the studies was to upgrade the drainage system to cater for the 1-in-100 year event for catchment B & C which is bounded by Ocean Beach Road to east and Dunban Road to the south, as detailed in *Woy Woy Peninsula Catchments 'B' and 'C' Drainage Study*, (Webb McKeown & Associates, 1996). The design plan included upgrading of the trunk drainage system to the 1-in-100 year standard and required upgrading of the drainage outlet of the catchment at Brief Street. This involved updating of the pipes under the Great Northern Railway Line near the outlet.

Although the construction started and two 1650mm pipes (PI24120 and PI24121) out of the designed four were installed under the railway line, the project was put on hold due to its high costs. Currently these two pipes are abandoned and the existing 750mm pipe is connected to the downstream culvert flowing to the bay.

In summary, the drainage model was adjusted as follows:

- PI24120 and PI24121 were abandoned.
- PI10450 was also abandoned.
- PI24119 is connected to BC895.
- Drainage Channel was left in the model, as per in the shapefile.

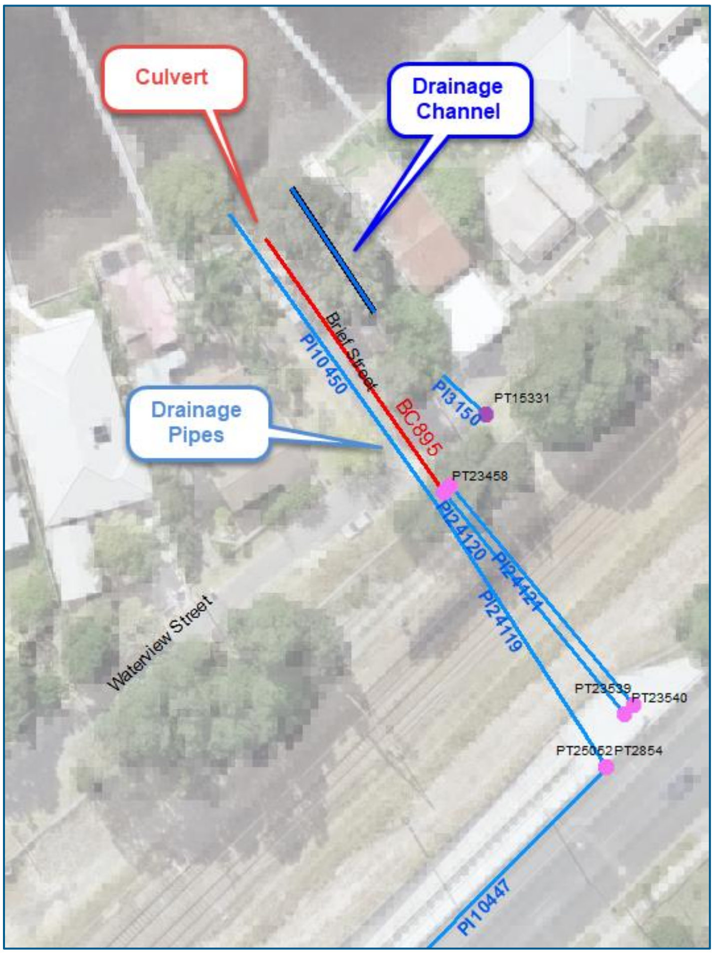

Figure 4.1 Storm drainages around Railway Street and Brief Street

4.5 Cross-sections for Kahibah Creek

Surveyed cross-sections are available for parts of the Kahibah Creek system in the following reports:

- **Ettalong Swamp Arm:** Kahibah Creek Review of Floodplain Management Measures (Willing & Partners Consulting Engineers, 2001). Includes surveyed cross-sections from October 2000
- **Kahibah Creek**: Kahibah Creek Flood Study (Willing & Partners Consulting Engineers, 1991). Used surveyed creek transects from 1990, but the exact locations and surveyed levels are not provided in the report.

The surveyed cross-sections used in these previous studies are more than 20 years old and actual data are not available in digital format. Several channel works have been carried out since then. Therefore, cross-sections derived from LiDAR were used in this study, complemented by the available longitudinal profiles in these previous studies.

4.6 Existing Floor level survey

Council collected available surveyed floor levels from the Kahibah flood studies (Willing & Partners, 1991) and the Brisbane Water Foreshore Flood Study (Cardno, 2013). The provided tables were converted to GIS layers by DHI, by comparing the address with the Cadastre data set.

Properties where these survey data are available are shown in **Figure 4.2**. Floor levels surveyed in the Brisbane Water Foreshore Flood Study are concentrated along the coastline due to the nature of the study. The central part of the peninsula has no floor level surveys.

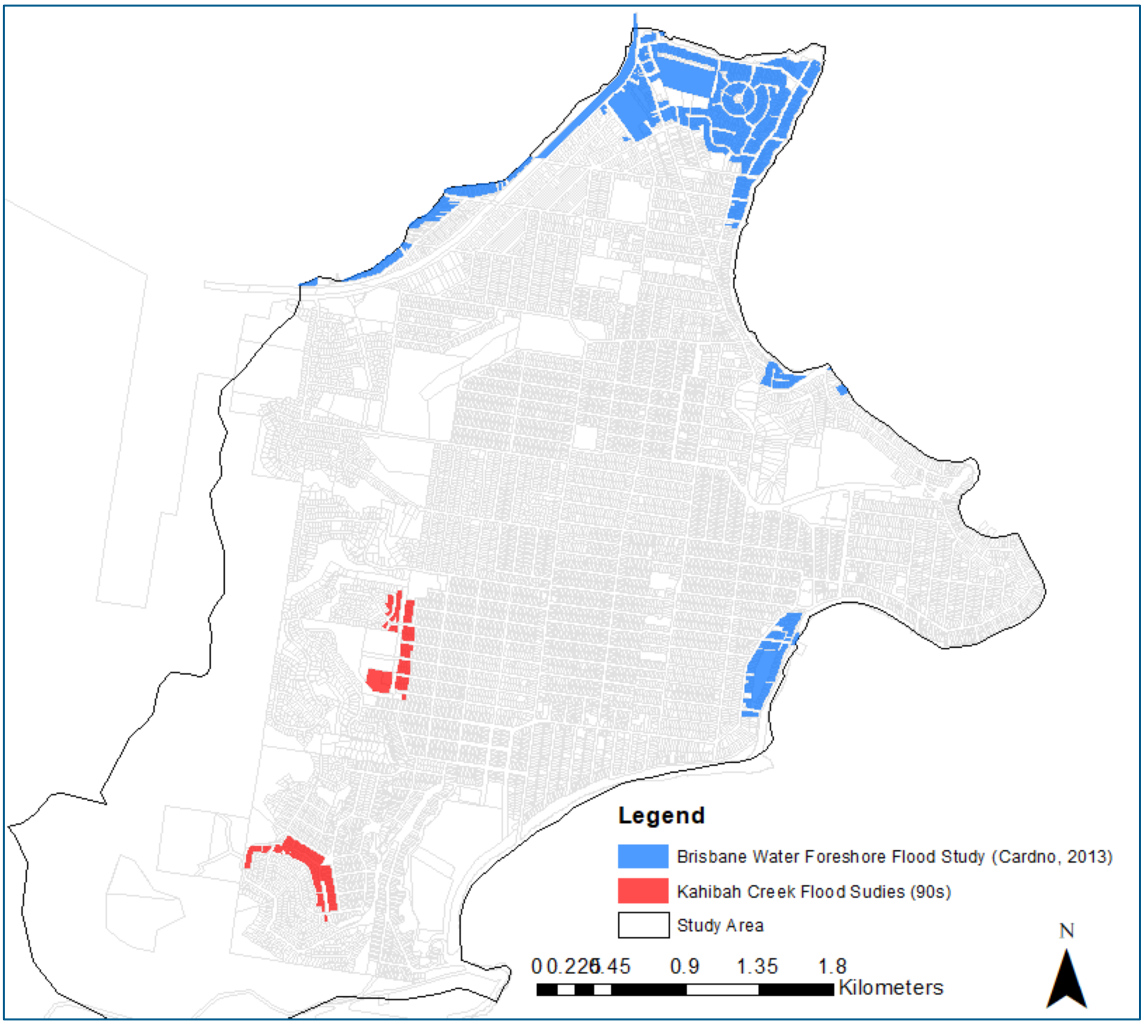

Figure 4.2 Floor Level Surveys in the previous studies

4.7 Structure Survey

Dimensions of some of the major structures were missing in the provided data. Survey of the major structures was therefore carried out by DHI following the site visit on 25 September 2019. This includes:

- Mt Ettalong Rd Bridge
- Cowper Rd Bridge
- Cowper Rd Culverts

- Hillview St pedestrian crossing
- Iluka Creek pedestrian bridge
- Kahibah Creek pedestrian bridge
- Main Drain Railway Bridge

Dimensions of structures that could not get accessed were estimated from photos.

5 Hydrological/Hydraulic Modelling

Given that the flood behaviour on the peninsula is impacted by both surface water and groundwater processes, a traditional modelling approach of decoupling the groundwater from the surface water component is not suitable. The dynamics between the two components have some unusual or unique mechanism in the hydrological cycle during a rainfall event and can only be captured by an appropriate model. Therefore, the integrated groundwater-surface water modelling tool MIKE SHE was used in this study and coupled to the pipe network modelling tool MIKE URBAN and the river modelling tool MIKE HYDRO:

- The MIKE SHE component calculates local runoff, infiltration and evapotranspiration (ET), as well as groundwater discharge to the surface water and storm water systems. It applies rainfall directly onto the grid.
- The MIKE URBAN model calculates the storm water drainage flow, including potential surcharging to the surface.
- The MIKE HYDRO model calculated flow through the open channels.
- Surface runoff in MIKE SHE discharges to the MIKE URBAN storm water drains and the MIKE HYDRO open channels, while storm water surcharge in MIKE URBAN and open channel flows MIKE HYDRO discharges onto the MIKE SHE topography.
- The combined model framework closes the internal water balance so that all inflows, discharges and internal storage changes are accounted for.

The details of the model setups are provided in this section.

5.1 MIKE SHE model

As part of the Woy Woy Integrated Water Management and Case Study Everglades Catchment (DHI, 2021), a MIKE SHE model was developed for the entire peninsula. The model was adopted for this study and modified to better address the nuisance flooding phenomenon. The model modification focused on the refinement of the model topography (finer model grid size for more accurate ground level representation) and inclusion of open drains, waterways and hydraulic structures.

The different MIKE SHE model components are described in detail in the following sections.

5.1.1 Overland Flow

Overland flow describes the physical process of water movement over the land surface outside of main river channels. Overland flow occurs when the rainfall rate exceeds the soil's infiltration capacity. In the MIKE SHE model, overland flow is represented by a diffusive wave approximation of the Saint Venant equations. Flow occurs in two dimensions, with water able to move in the x and y directions, but not diagonally.

5.1.2 Manning's roughness

Resistance to overland flow is controlled by surface roughness. The model uses a Manning's roughness coefficient to define the overland roughness (see **Figure.5.1)**. The roughness values vary to represent roads, buildings and undeveloped areas.

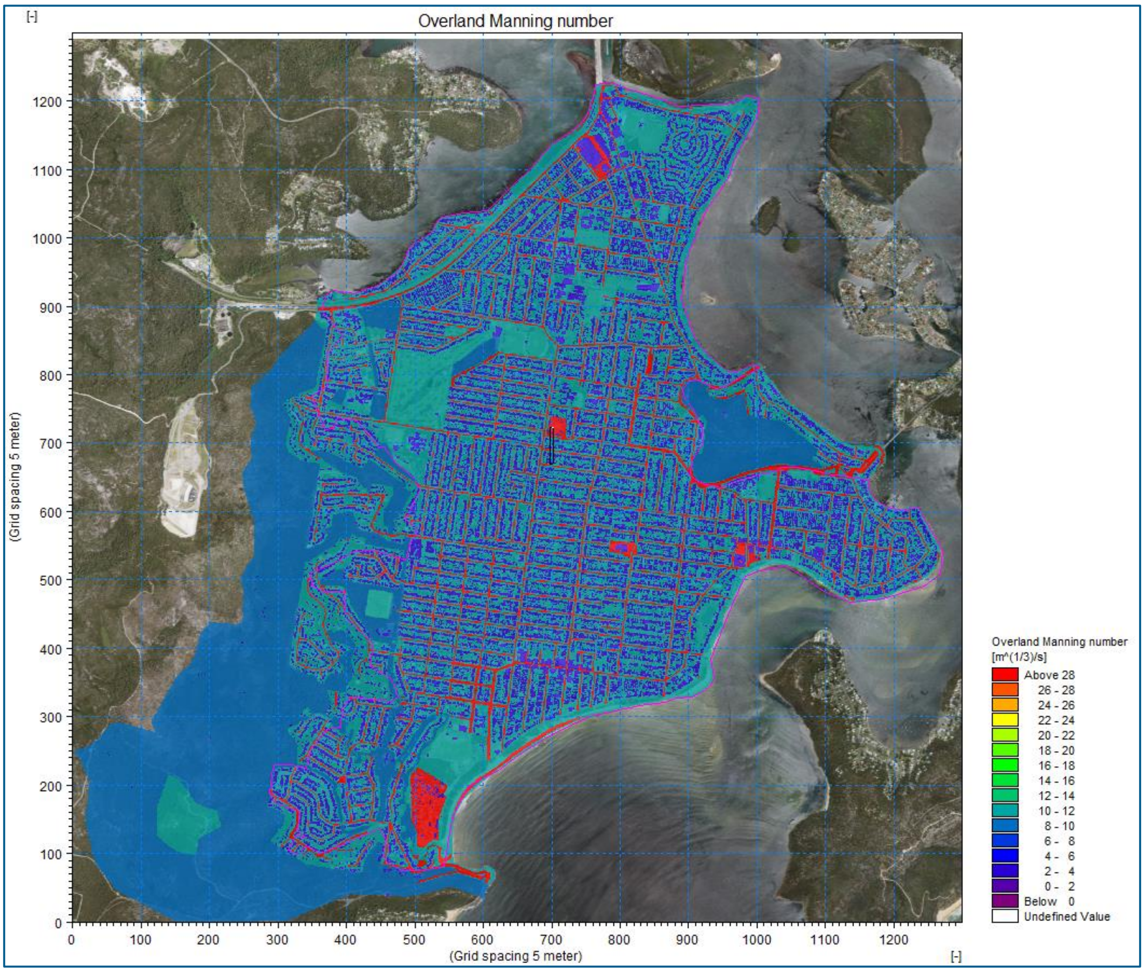

Figure.5.1 Overland Manning's M roughness

5.1.3 Areas of restricted overland/subsurface exchange

In urban and built up areas the potential for rainfall and overland flow to infiltrate into groundwater is significantly reduced by impervious surfaces (e.g. roads). To represent this in the model, areas of reduced exchange have been applied via use of an additional leakage coefficient (see **Figure 5.2**) which reduces the infiltration rate at the ground surface to account for soil compaction or pavement.

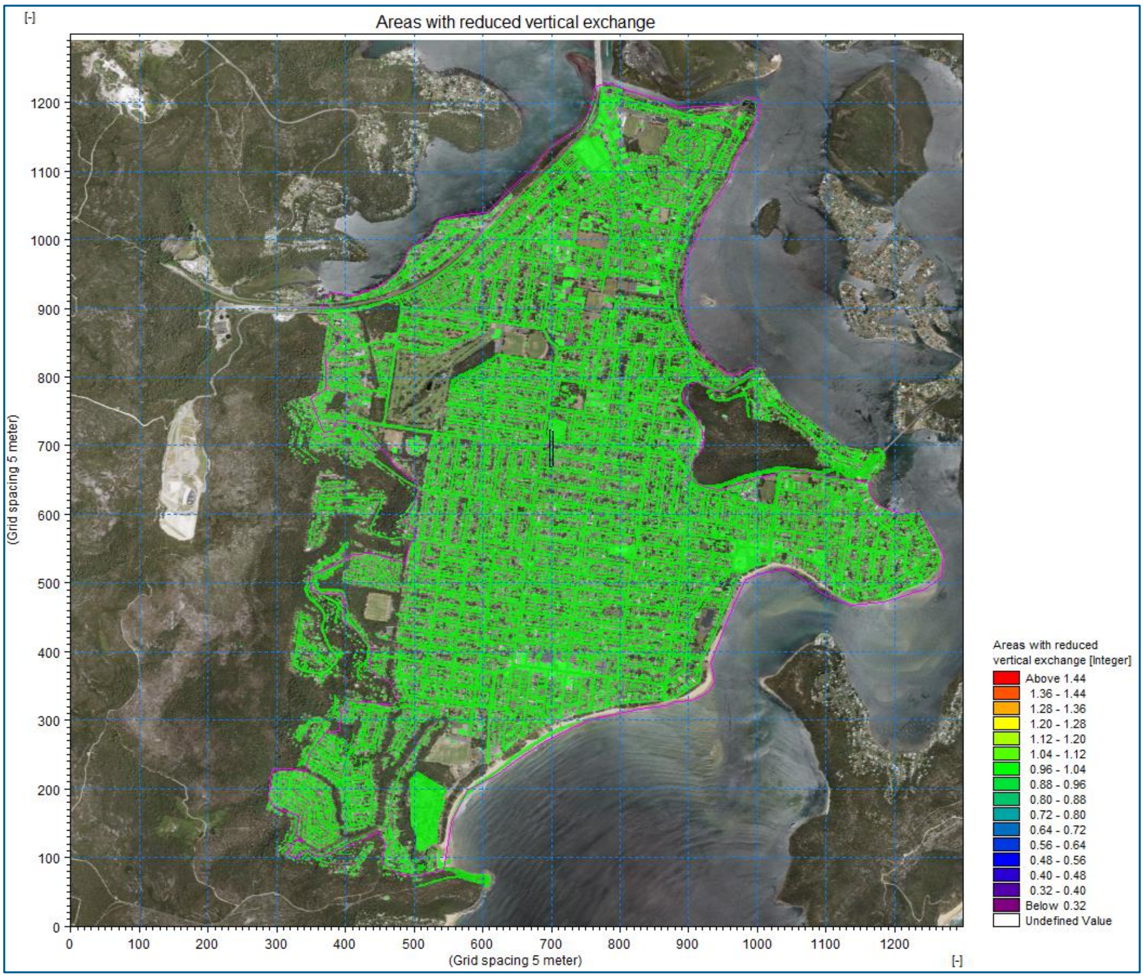

Figure 5.2 Leakage coefficient used in areas of reduced overland/subsurface exchange

5.1.4 Unsaturated flow

The unsaturated zone (or vadose zone) is the area of variable saturation that extends from the land surface to the groundwater table. The physical properties of the unsaturated zone affect the timing and volume of recharge to groundwater, and conversely the volume of water that runs off rather than infiltrating during high-intensity rainfall events.

Mathematically, MIKE SHE can represent the unsaturated zone using three methods: 1-D Richards equation (1931), 1-D gravity flow or a 2-layer soil moisture balance. While the Richards equation is physically the most realistic approach, it is also the most computationally and data-intensive. Part of what makes the Richards equation so computationally intensive is the inclusion of capillary fringe movement. As the dynamics of the soil capillary fringe are not critical to this study, the gravity flow method has been used. The gravity flow method employed in this study is a simplification of the Richards equation without capillary fringe and is used when timing and volume of recharge or discharge of groundwater are important. The 2-layer soil moisture balance model is generally only used in situations where the water table is very close to the surface, and the timing of recharge to groundwater does not need to be considered. As such, the 2-layer soil moisture balance model has not been employed for this study.

The gravity flow method uses a soil profile prescribed in tabular form, which provides the model's vertical discretisation. In order to maintain numerical stability in MIKE SHE, the soil profile must extend from land surface to slightly below the first saturated zone layer. The table below details the vertical discretisation of the soil profiles. Horizontal discretisation follows the model grid resolution.

Table 5.1 Soil profile discretisation

From depth (m)	To depth (m)	Cell height (m)	Number of cells
0.0	0.5	0.1	5
0.5	1.5	0.2	5
1.5	3.0	0.5	3

Movement of water through the soil profile and the timing and volumes of recharge are dependent on specified soil properties. The model in this study utilises the soil properties developed for the Woy Woy Integrated Water Management and Case Study Everglades Catchment (DHI, 2021). Specifically, 11 soil types are represented in the model. These soil types include ten sand classes and one sandstone class. Ten sand classes are made to match the different hydraulic conductivities used in Saturated Zone. Full details of each soil class are provided in Woy Woy Integrated Water Management and Case Study Everglades Catchment (DHI, 2021).

5.1.5 Saturated zone

The saturated zone describes groundwater flow and transport below the water table. The behaviour of water in the saturated zone (groundwater levels, flow and transport) is controlled by the hydraulic properties of the aquifer and boundary conditions. The key hydraulic properties related to flow are the aquifers hydraulic conductivity, specific yield and specific storage. The physical properties that control the transport of nutrients and contaminants in the groundwater system are porosity and dispersivity.

It should be noted that the saturated zone was already calibrated as a part of the Woy Woy Integrated Water Management and Case Study Everglades Catchment (DHI, 2021) and this project will focus only on the calibration of parameters relating to flooding.

5.1.6 Grid structure and layering

The saturated zone module grid structure follows that of the overall model, e.g. 5 m by 5 m. Vertically the model is discretised with a single saturated zone layer which extends variably from the land-surface to between 0.5 m and 65 m below ground level. The saturated zone thickness is presented in **Figure 5.3**.

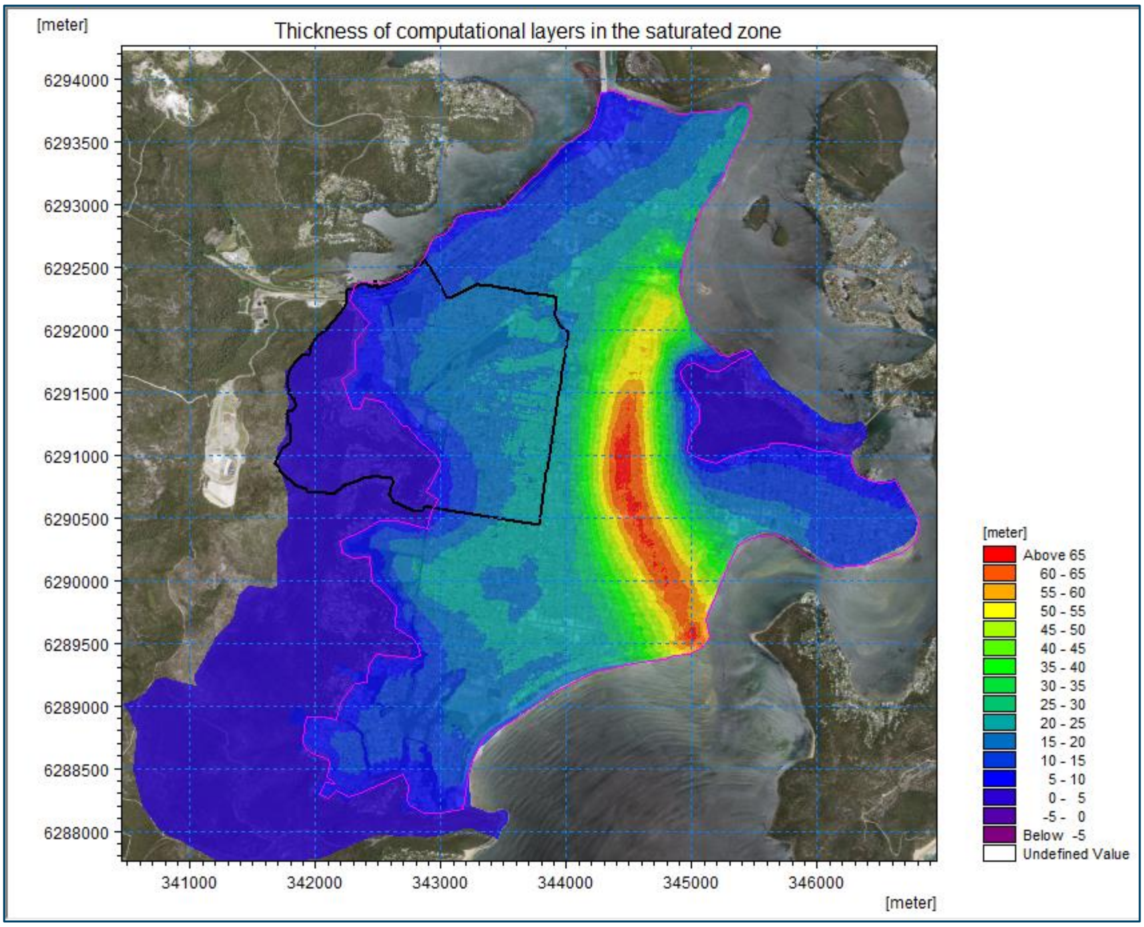

Figure 5.3 Saturated zone thickness

5.1.7 Boundary conditions

For the saturated zone module, boundary conditions are the sources and sinks for inputs and outputs from the model. Boundary conditions in the model include no-flow boundaries, head-dependent boundaries, river gains and losses, drainage boundaries, pumping wells and recharge from and discharge to the unsaturated zone/surface.

5.1.8 External head-dependent boundaries

In MIKE SHE, head-dependent boundary conditions can take one of three forms:

1. Specified head, where the hydraulic head is prescribed on the boundary and can be either fixed or time-varying.
2. Gradient/flux dependent, where the gradient of the hydraulic head across the boundary is prescribed. Where the unsaturated zone module is not used, recharge is specified as a flux boundary.
1. Head-dependent flux, where the head-dependent flux is prescribed on the boundary (synonymous with river linkages).

The Woy Woy model uses both the no-flow boundary and the specified head options for the model. The inland boundary is specified as no-flow, meaning no water may enter or exit the model via the saturated zone in the inland extent. The time-varying tidal levels measured at Ettalong 212423 Station and Koolewong 212422 Station was used as the coastal boundary for calibration purpose. The coastal boundary between two stations were linearly interpolated.

5.1.9 Internal boundary conditions

Pumping wells

Pumping wells in the model are a flux dependent boundary condition. They are specified as either a fixed or variable rate of flux to or from the saturated zone grid cell they intercept. The Woy Woy model includes assumed 17 pumping well boundaries to represent municipal and private water supply wells (highlighted in **Figure 5.4**). The assumed pumping rates were taking from the previous studies (DHI, 2010).

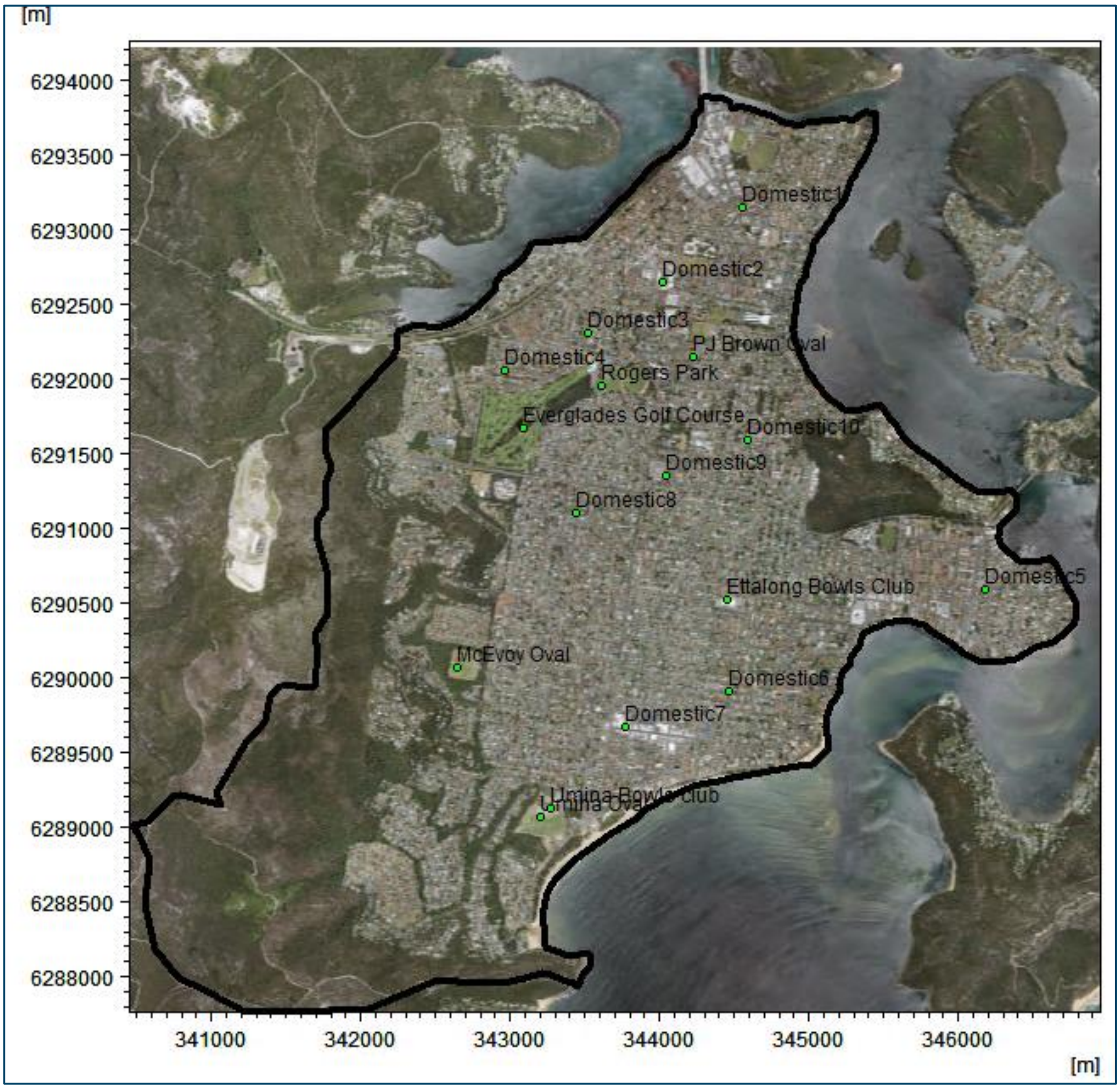

Figure 5.4 Pumping well locations and boundary locations

Stream linkages/MIKE URBAN

Another internal boundary condition used in the Woy Woy saturated zone model is that coupling the model to the MIKE URBAN package. This represents leakage from streams or slotted pipes to and from the groundwater.

Stream linkages/MIKE HYDRO
The final internal boundary condition of the saturated zone is that coupling to the MIKE HYDRO river network. This includes leakage from/to the open channel systems.

5.1.10 Aquifer properties

As mentioned above, aquifer properties control the water levels and flow through the aquifer. Hydraulic conductivity along with recharge controls the long-term average groundwater levels. Hydraulic conductivity is the volume of fluid at the existing kinematic viscosity, that will move in a unit time under unit hydraulic gradient at right angles to the direction of flow. Specific yield and specific storage relate to the volume of water that may be released by the aquifer, and along with recharge volumes control the magnitude of seasonal water level variations. Specifically, specific yield (also known as effective porosity) is the volume of water that an unconfined aquifer releases from storage per unit surface area of aquifer per unit decline in water table and is hence unitless (Freeze and Cherry, 1979). Specific storage is the unit volume of water elastically release by the aquifer under a unit decline in hydraulic head, in units of 1/m.

The Woy Woy model features 16 hydraulic conductivity zones, each with isotropic properties to represent the various soil types beneath the land surface. The hydraulic conductivities adopted range from 1x10-7 m/s to 4.9x10-4 m/s. A visual representation of the spatial variation in the horizontal conductivity adopted in the MIKE SHE model is given in **Figure 5.5.**

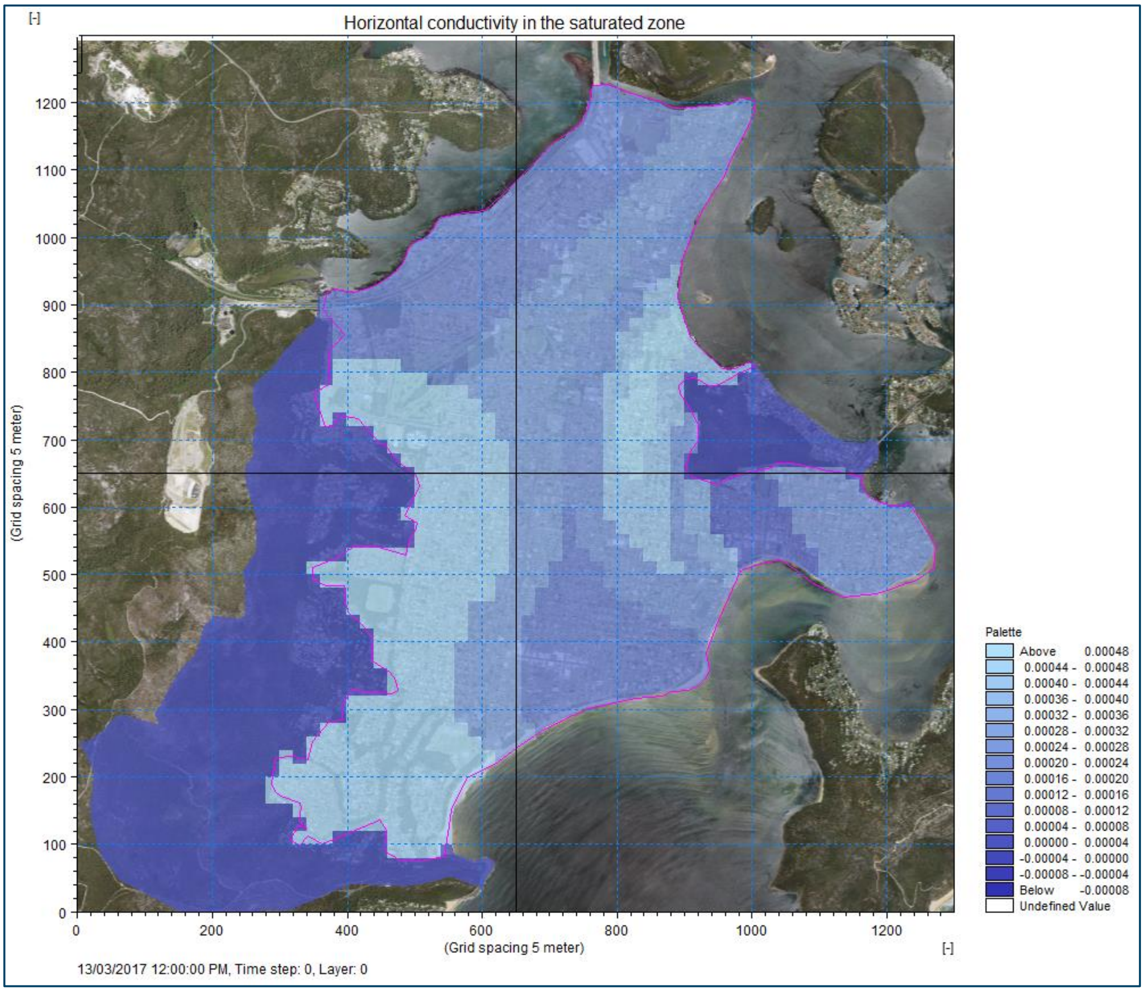

Figure 5.5 Modelled horizontal conductivity

The adopted specific yield properties are taken from the effective porosity of each overlying vadose zone soil types in order to maintain numerical stability and prevent mass balance errors. The specific yield values range from 0.08 to 0.34 (**Figure 5.6**).

Because only one aquifer is modelled and has unconfined conditions specific storage is not used by the model.

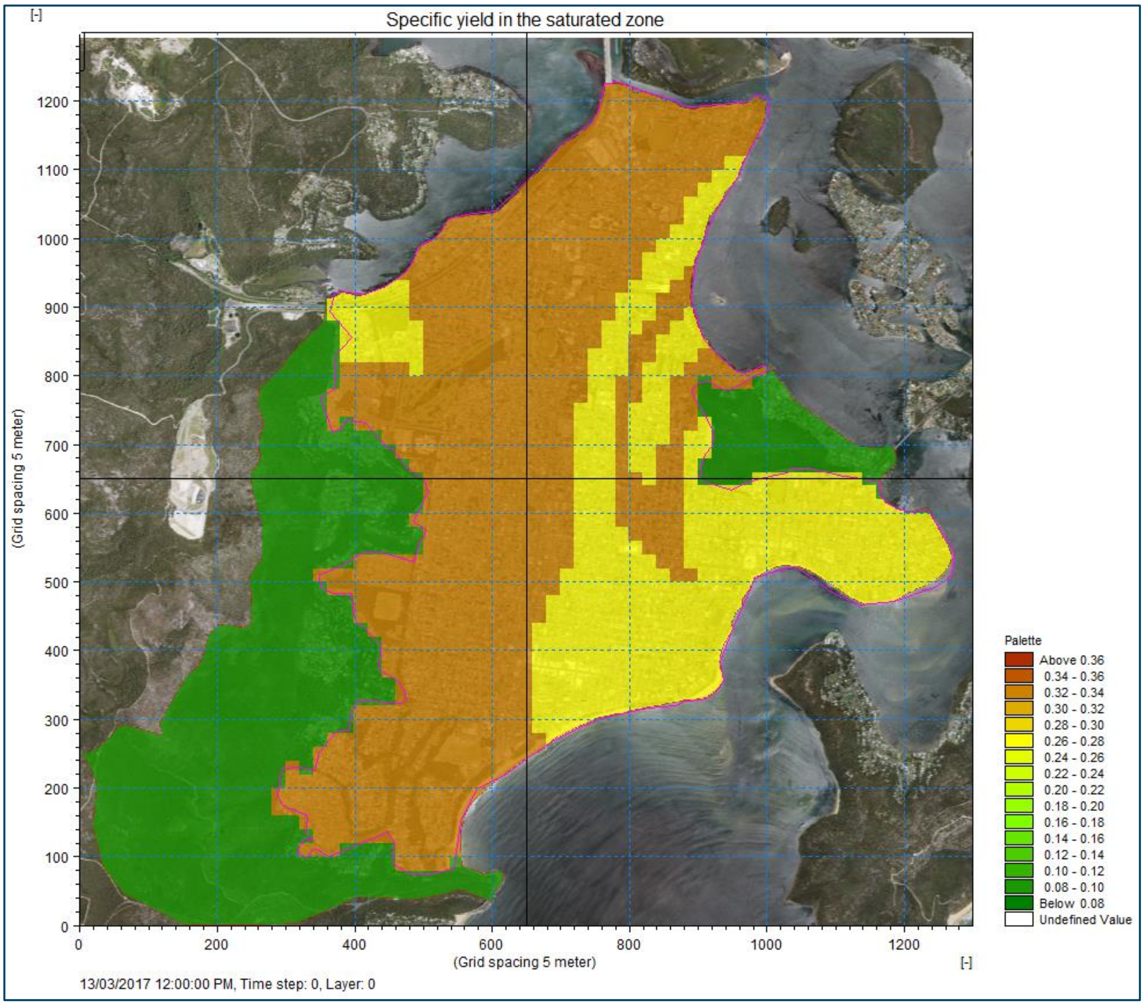

Figure 5.6 Model specific yield

5.2 MIKE URBAN model

MIKE URBAN is DHI's commercial package which hosts the MOUSE model, a computational tool for simulations of 1-D unsteady flows in pipe networks with alternating free surface and pressurised flow conditions. The computation is based on an implicit, finite difference numerical solution of the Saint-Venant equation.

MIKE URBAN was used to represent the storm drainage network in the catchment. Stormwater asset data was provided by Council in GIS format (shapefiles) and imported into MIKE URBAN. As discussed in **Section 4.4**, the majority of the drainage network was manually adjusted in MIKE URBAN to allow for a smooth slope of the stormwater pipes. The adjustments included manual modifications of pipe invert levels and assumptions of pipe sizes and pit volumes.

The open drainage Main Drain was also represented in MIKE URBAN, as many stormwater drains are conveying runoff to the Main Drain.

Council provided Everglades main channel.pdf which contains data of cross-sections and structures of the Main Drain. This was incorporated in the model and supplemented with cross-sections extracted from the DEM.

An overview of the stormwater drainage network included in the MIKE URBAN model is given in **Figure 5.7**.

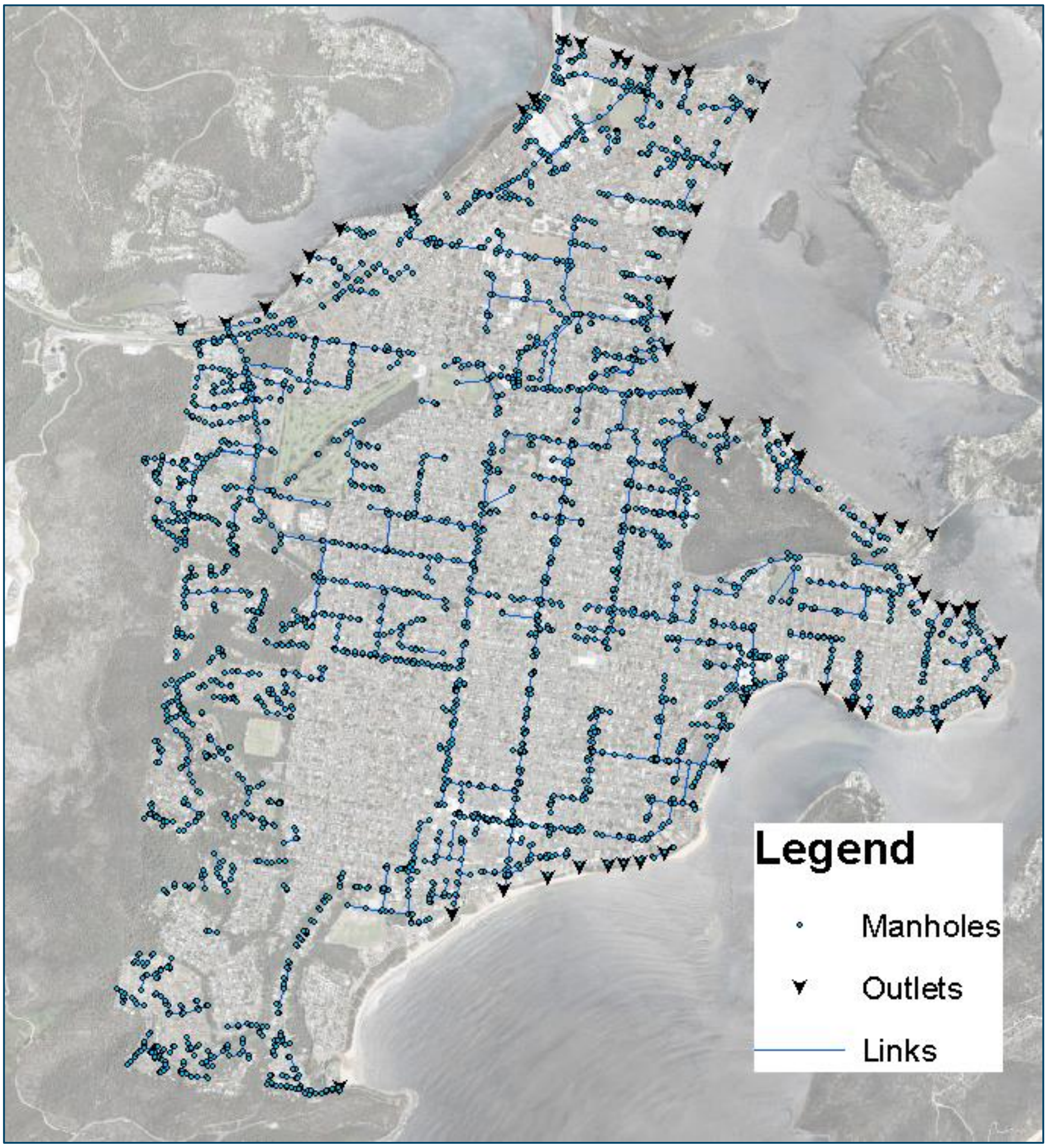

Figure 5.7 Overview of stormwater drainage network in MIKE URBAN

As discussed in 4.4, there is significant information missing in the provided drainage dataset and the following assumptions were made:

- Most pit volumes were set equal to the volume of a round 1 m manhole, if no other information is given.
- “Dummy pipes” were not included in the model.

- Drainage pipe sizes were estimated from the neighbouring pipe sizes, if the information is not provided.
- Drainage invert levels were interpolated from the provided estimated invert levels at the neighbouring pipes.

5.3 MIKE HYDRO model

MIKE HYDRO is a one-dimensional hydraulic modelling package for open channel and river modelling. It was used in this study to better represent the open channels and creeks in the Kahibah Creek System (in contrast to using MIKE SHE only).

A few site visits revealed that small culverts at Australia Avenue Arm and the culverts on Iluka Creek at Kahibah Road are usually blocked by sediment deposit and vegetations. The blockage level was set to 20% at culverts on Australia Avenue Arm and to 50% at the culverts at Kahibah Road for calibration purpose and design runs.

5.4 Coupling of MIKE SHE and MIKE URBAN

The MIKE SHE model was coupled to the MIKE URBAN model to allow for transfer of flow between the overland and groundwater components (MIKE SHE) and the stormwater component (MIKE URBAN). The coupling between the two models was achieved by an external coupling definition file (.adp).

The coupling can occur at nodes or links, which means that water transfer between the two models is possible at manholes (MIKE URBAN model nodes) and open channels (MIKE URBAN model open links). Groundwater exchange through the bottom of the open channels was included for natural streams such as the Main Drain and Kahibah Creek.

5.5 Coupling of MIKE SHE and MIKE HYDRO

The MIKE SHE model was coupled to the MIKE HYDRO model to allow transfer of water between the river branch (MIKE HYDRO) and the surface and groundwater components (MIKE SHE).

6 Calibration of the model

The MIKE SHE model developed in this study was based on the groundwater model with a coarser resolution established in the *Woy Woy Integrated Water Management and Case Study Everglades Catchment* (DHI, 2021). As the parameters related to soil/groundwater were already calibrated against the long-term groundwater level observations, calibration of the model in this study focused on parameters affecting surface water.

6.1 Calibration Event

In the absence of any gauged water level or discharge data, calibration of the model was carried out against the estimated peak water depths.

Significant flooding events were experienced on the peninsula. The major flood events identified by the community were reported in the previous study (DHI, 2010).

- August 1972
- 1st May 1974
- 1984
- March 1986
- 1st April 1988
- February 1990
- March 1991
- February 1992
- 1st May 1998
- April 1999
- 1st June 2007

In addition, Giammarco Engineering (1989) reported another major flooding event in January 1989 in the suburbs around Blackwall Mt.

However, the April 1988 flood event was the only event with enough data collected during the community consultation (DHI, 2010) for model calibration.

Figure 6.1 shows the rainfall timeseries for the April 1988 event derived in the previous study. It should be noted that the temporal patterns were taken from the Peats Ridge gauge which was the nearest available for this event The 6 min interval temporal pattern of Peats Ridge (BOM Gauge 61351) was scaled by the daily rainfall measured at Everglades Golf Course (BOM Gauge 61318) to produce this timeseries.

Note that 311mm of the total rainfall over 72 hours was recorded between the 27th at 12PM and the 30th at 12PM April 1988. This corresponds to a 6-7% AEP rainfall event.

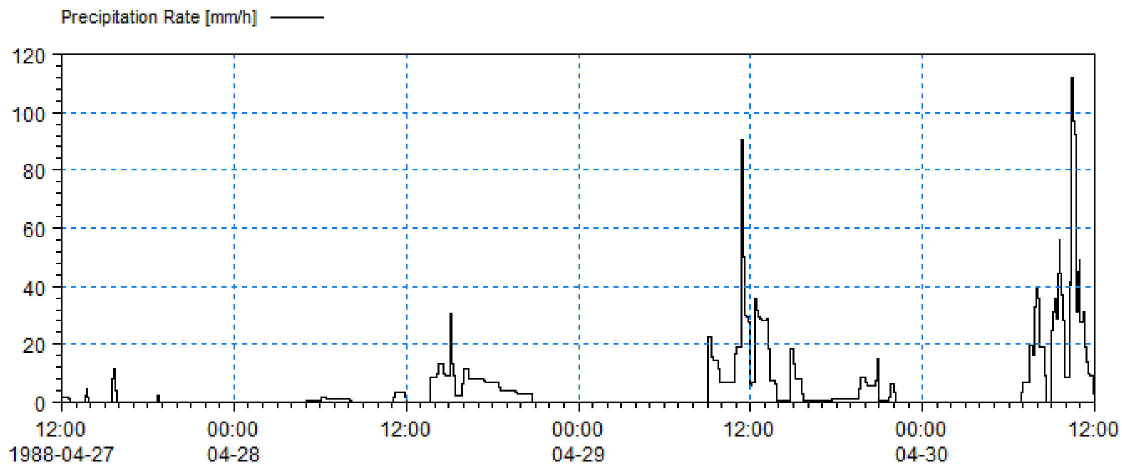

Figure 6.1 April 1988 Event, Rainfall intensity in mm/hr (Time interval is 6 min)

The sea boundary condition was taken from two of MHL's tidal gauge stations: Ettalong 212423 Station and Koolewong (Decommissioned in 2016) 212422 Station. The water levels along the coast between the two stations were linearly interpolated.

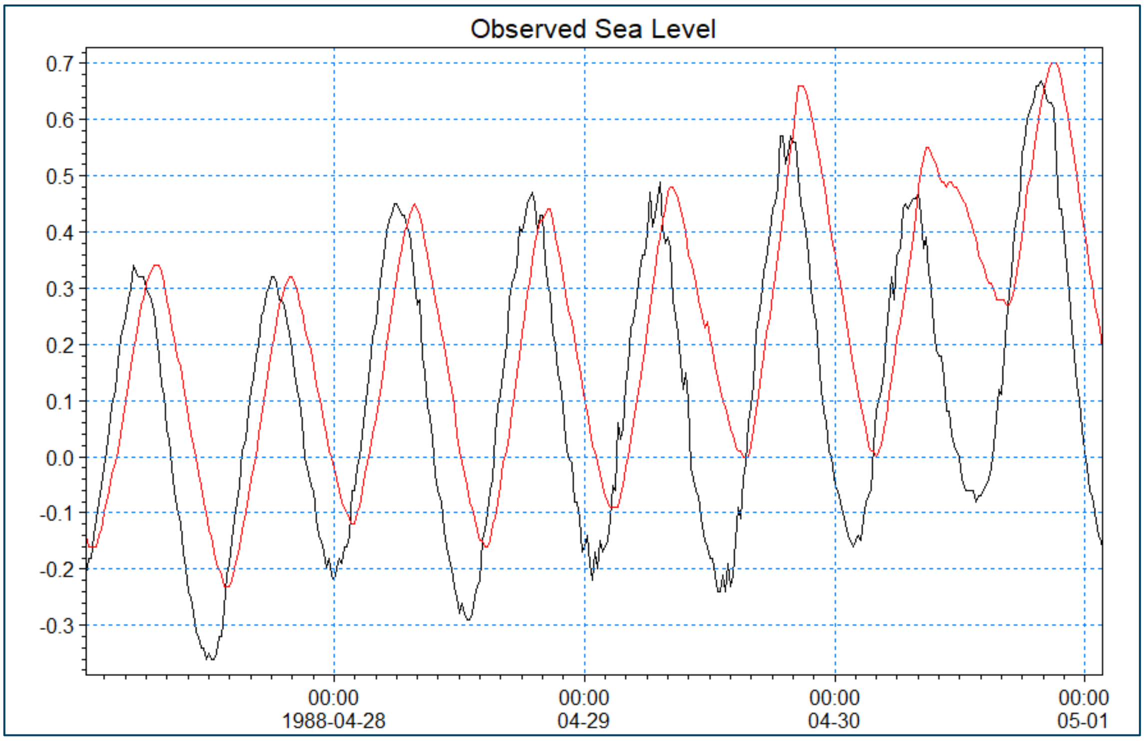

Figure 6.2 April 1988 event, Gauged Sea Water Levels; Black: Ettalong 212423 Station and Red: Koolewong 212422 Station

Table 6.1 summarises the collected estimated peak depths in the previous Woy Woy flood study (DHI, 2010) and in the Brisbane Water Foreshore Flood Study (Cardno, 2013). **Figure 6.3** shows the corresponding locations.

There were no corresponding reported water depths for the April 1988 calibration event at the Kahibah Creek system. The *Kahibah Creek Flood Study* (Willing & Partners, 1991) considered two large flood events January 1989 and 7 February 1990 in the Kahibah Creek System as calibration events. The Kahibah Creek system has been modified largely after the study in 1991, including widening and lining of the channel paths, replacement of major structures and clearance of weeds. These changes are so significant that a calibration of the model against either of these events would not be relevant for the current study.

Table 6.1 Estimated peak water depth in the Apr 1988 event from residents' report

Site Number	Location	Resident Comment	Estimated 1988 Peak Depth (mm)
1	Intersection, Wharf Rd and North Burge Rd	2002 flooding estimated 200 mm from photos	>200
2	78 Dunalban Ave	600mm in 1990 – in 1988 slightly less	500
3	30 Shepard St	Halfway up car window	800
4	20 Ridge St	Pooling only when stormwater inlets get clogged	<150
5	73 Lone Pine Ave	calf muscle depth between 73 Lone Pine avenue and Shepard st	400
6	306 Blackwall Rd	30cm over entire yard and adjoining properties Outside laundry & toilet which are at ground level approx. 75 to 100mm. (Cardno 2013)	300 100 (Cardno 2013)
7	12 Shepard St	46 to 50cm deep in road gutter	500
8	28 Ross St	photo provided	600
9	58 Watkin Ave	1990 worst flooding	500
10	140 Paton St	36cm deep over my block	360
11	132 Paton St	1ft under the house	300
12	18 Darley Rd	Houses flooded to window level	500
13	4 Cogra Rd	water over floor by 90mm	400
14	39-51 Karloo Rd	5ft deep from paling fence blocking flow	1500
15	28 Waratah Ave	150 mm 3 times yearly	>150
16	3 Forest Rd	30cm deep spread across the road	300
17	10 Dulkara Rd	Up to 0.75m in streets	750
18	61 Boronia Ave	30cm deep in back lane	300
19	10 Lalina Ave, Blackwall	Water reached to just above piers (Cardno, 2013).	Unknown
20	67 Lone Pine Ave	Deeoest point in street	400

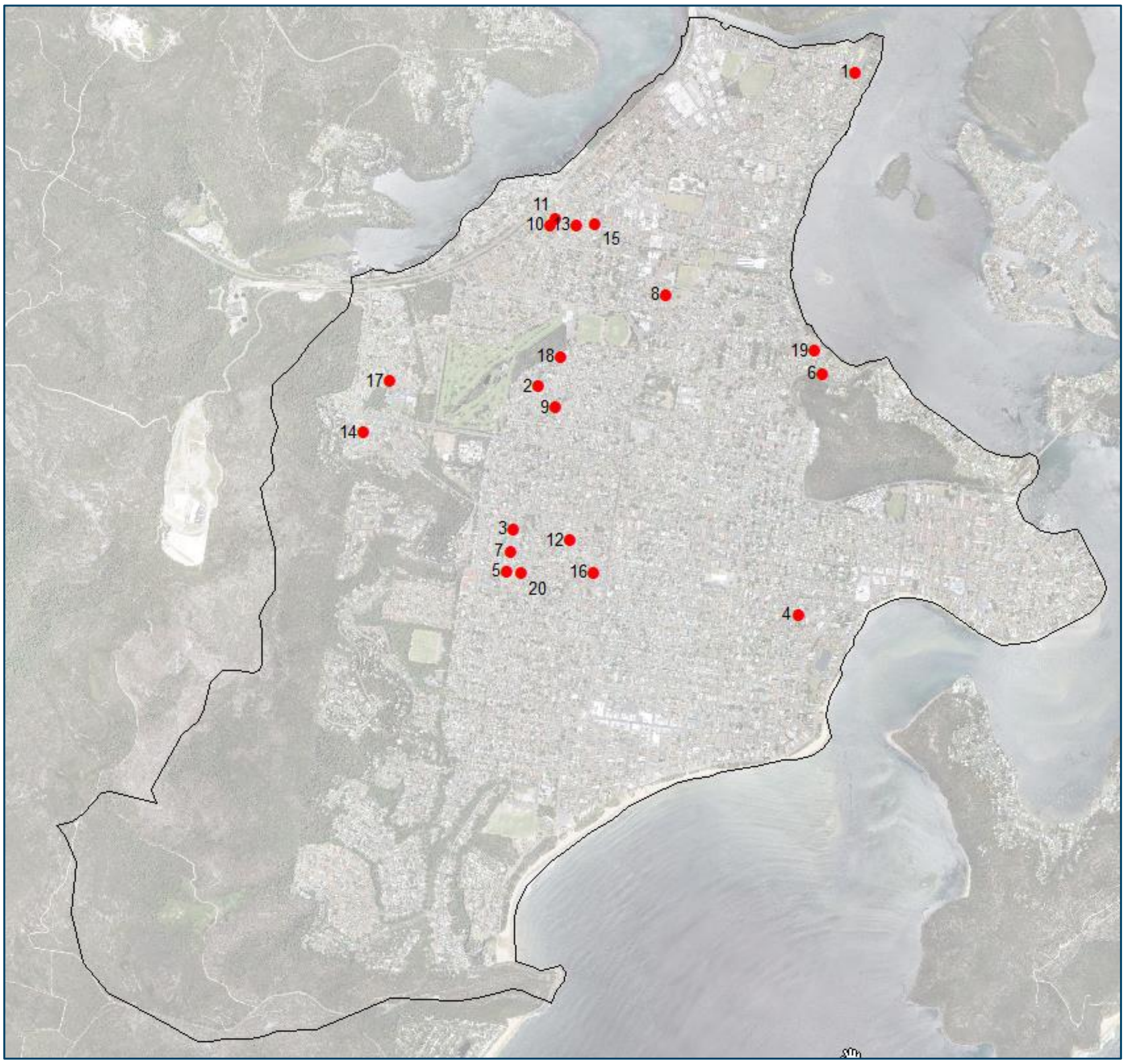

Figure 6.3 Location of water Depth collected for the 1988 flood event

6.2 Antecedent catchment condition

Antecedent rainfall for the event was relatively high. A total of 570mm of rainfall was observed between the 18th of March and the 19th of April at the BoM's Everglades Golf Course (BOM Gauge 61318). A relatively wet antecedent catchment condition is also indicated by the long-term simulation carried out in the groundwater study. The long-term groundwater simulation was run with an averaged coastal boundary of 0.1mAHD. **Figure 6.4** shows the simulated groundwater level at the observation bore WW43 located close to the intersection of Ryans Rd and Shepard Ave (DHI 2019). The groundwater mound is estimated to be located around this bore hole. As it can be seen from the graph, the groundwater level at the start of the event (27th of April 1988) was estimated to be relatively high and it rises quickly during the event.

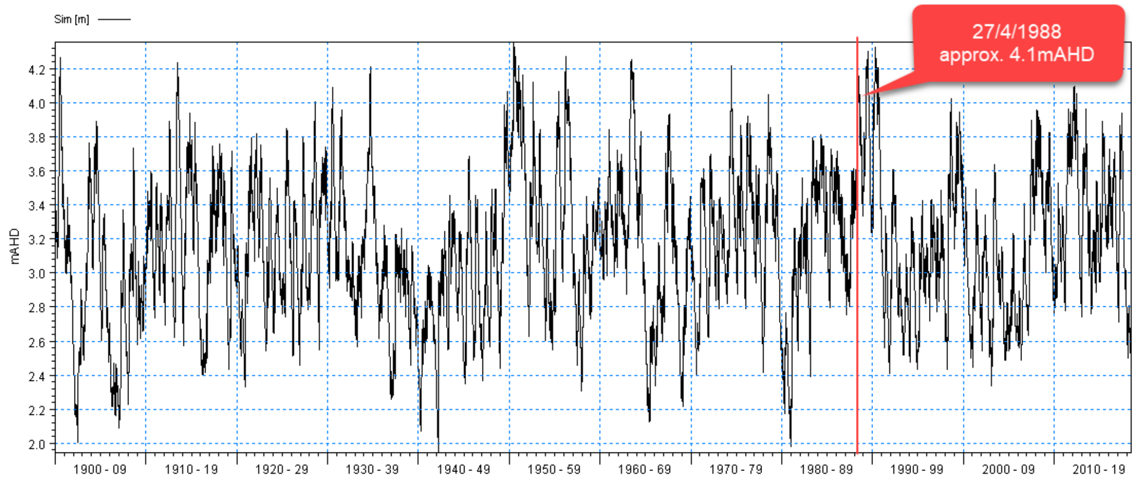

Figure 6.4 Simulated groundwater level at WW43 (near the intersection of Ryans Road and Shepard Street)

6.3 Effective drainage during the calibration event

The same assumption as in the previous *Woy Woy Peninsula Flood Study* (DHI, 2010) was applied; no effective drainage network existed during the calibration event of 1 April 1988. Storm drainage pipes were not included in the model except the outlet pipes from the Palmtree Grove Detention Basin and the pipes in the Everglades Catchment which interconnect the open drainage channels.

This was supported by the fact that many of major pipe drainage works have taken place since this event (DHI, 2010) and large section of the drainage systems, both pits and pipes were found to be completely blocked with sand and silt in the drainage studies in 1990s (Patterson Britton & Partners, 1997).

6.4 Calibration Parameters

Calibration focused on the surface parameters such as surface-subsurface leakage coefficients and roughness. Surface-subsurface leakage coefficient is the parameter used in MIKE SHE to reduce infiltration rates at the ground surface. Conceptually this parameter is used to account for soil compaction on the surface or fine sediment deposits on floodplains. Here it was used to account for pavement and compacted soil along the streets.

Roughness coefficient in the 1D channel branches were not adjusted in the calibration exercise for the following reasons:

- Roughness coefficient in the river impacts a hydrograph shape and time to peak. It does not make sense to adjust channel roughness when there is no observation data of dynamic responses.
- Observation of flood depths for the calibration event are concentrated in a part of the peninsula where channel characteristics do not have significant influence on flood patterns.

6.5 Calibration results

Simulated maximum flood depths across the study area are shown in **Figure 6.5** and summarised in **Table 6.2** for specific locations where estimated water depths were reported. The resulting MIKE SHE parameters are given in **Table 6.3**.

The model shows reasonable agreement with the affected areas in the central and north-eastern parts of the study area. While overall, the peak water depths were reasonably reproduced, there were specific locations where the model significantly underestimated them. Examples are Location 14 on Karloo Road or Location 12 on Darley Road (**Figure 6.5**). There may have been localised features such as paling fences or building walls that could have resulted in localised impacts on the flooding depths.

Table 6.3 lists the adopted MIKE SHE parameters after the calibration exercise.

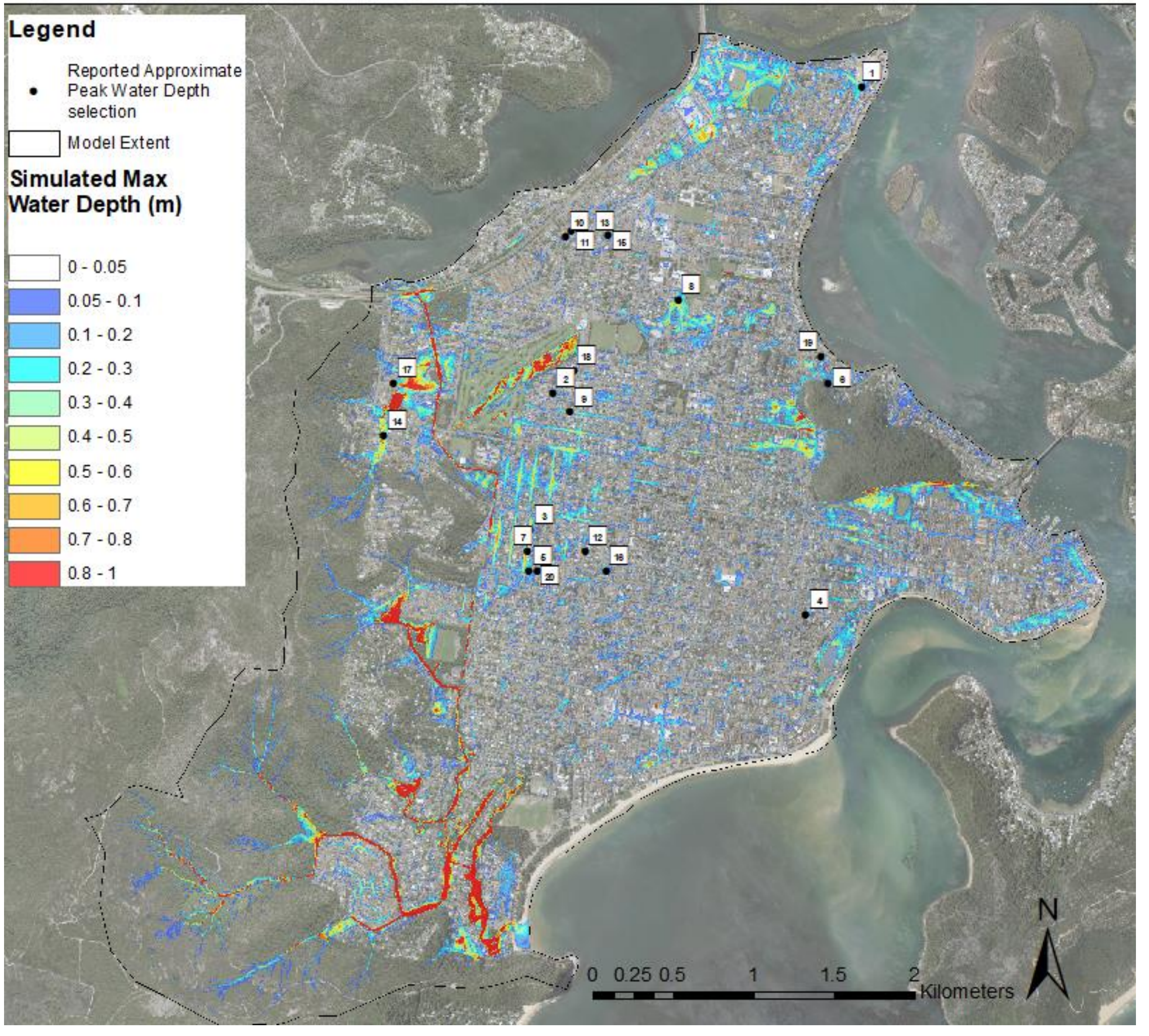

Figure 6.5 Simulated maximum water depth of April 1988.

Table 6.2 Estimated Peak Water Depth and Simulated Maximum Water Depth of Calibration Event (Apr 1988)

Site Number	Location	Resident Comment	Estimated 1988 Peak Depth (mm)	Simulated Max. Depth (mm)
1	Intersection, Wharf Rd and North Burge Rd	2002 flooding estimated 200 mm from photos	>200	320
2	78 Dunalban Ave	600mm in 1990 – in 1988 slightly less	500	210
3	30 Shepard St	Halfway up car window	800	729
4	20 Ridge St	Pooling only when stormwater inlets get clogged	<150	160
5	73 Lone Pine Ave	calf muscle depth between 73 Lone Pine avenue and Shepard st	400	416
6	306 Blackwall Rd	30cm over entire yard and adjoining properties Outside laundry & toilet which are at ground level approx. 75 to 100mm. (Cardno, 2013)	300 75-100 (Cardno)	180-200
7	12 Shepard St	46 to 50cm deep in road gutter	500	460-700
8	28 Ross St	photo provided	600	600
9	58 Watkin Ave	1990 worst flooding	500	406
10	140 Paton St	36cm deep over my block	360	250
11	132 Paton St	1ft under the house	300	89
12	18 Darley Rd	Houses flooded to window level	500	45
13	4 Cogra Rd	water over floor by 90mm	400	100
14	39-51 Karloo Rd	5ft deep from paling fence blocking flow	1500	790
15	28 Waratah Ave	150 mm 3 times yearly	>150	100-230
16	3 Forest Rd	30cm deep spread across the road	300	145
17	10 Dulkara Rd	Up to 0.75m in streets	750	1000
18	61 Boronia Ave	30cm deep in back lane	300	223
19	10 Lalina Ave, Blackwall	Water reached to just above piers (Cardno, 2013)	Unknown	13-57
20	67 Lone Pine Ave	Deepest point in street	400	409

Table 6.3 Adopted MIKE SHE parameters

Land use	Roughness (m1/3/sec) (1/n n is Manning's coefficient)	Surface-Subsurface Leakage Coefficient (/sec)
Grass/ Council reserves	10	Not used
Unsealed Roads	10	2e-5
Paved Roads	30	1e-10
Trees on Sand	8	Not used
Trees on Sandstone	8	Not used
Buildings	3	1e-10
Concrete/Asphalt parking	30	1e-10

6.6 Limitations in calibration and uncertainty of the model

The model does not represent paling walls or buildings at each dwelling. While this does not affect the overall flood patterns, local property scale may not be accurately reproduced.

As no gauged water level or discharge data is available at any of the creeks, storm drainage channels and pipes in the study area, calibration was limited to the approximate peak water depths reported by the residents. Although this means that a dynamic flood response of the model is not evaluated, two-dimensional flood models are known to reproduce flood extents much better than the conceptual model or the 1D models even without calibration as long as major flow paths are correctly represented.

Calibration of flow exchange between the overland flow and the storm drainage network was not carried out, as it was assumed that most drainage inlet structures were blocked/ or did not exist during the calibration event. Calibration of the model parameters controlling inlet flow to the drainage network or drainage flow is difficult without a flow record. In addition, uncertainty lies in drainage invert levels estimation as discussed in **Section 4.4**. While uncertainty exists in the drainage conditions, relevant parameters in the model were estimated using the best practice to have realistic drainage flows under the current condition.

The Kahibah Creek catchment has not experienced major flooding since major channel works and the maintenance program were undertaken after the large flooding in 1988, 1989 and 1990. It is recommended to reassess flood behaviours if it experiences flooding again.

As flooding in the Woy Woy peninsula is pluvial and not fluvial, the key factors affecting the flood extent and depth in the Woy Woy peninsula are topography and infiltration loss due to its high permeable soil, rather than accurate representation of the channel system. The model was calibrated against the bore hole data for an extended time period. This provides more confidence in the estimation of infiltration loss and a dynamic response of the catchment to rainfall.

7 Design Events

7.1 Rainfall

Design rainfalls are a probabilistic estimate of rainfall depth. ARR 2019 revised Intensity-Frequency-Duration (IFD) design rainfall using more extensive rainfall data compared to ARR 87. One of the major changes is the use of ensemble temporal patterns, while a single temporal pattern used to be used in ARR87.

Design rainfall depths and Probable Maximum Precipitation depths are summarised in **Table 7.1.** These depths account for aerial reduction factors.

Table 7.1 Design Rainfall Depth (mm)

Duration	Frequency (AEP)							
	50%	20%	10%	5%	2%	1%	0.50%	PMP
1 hr	27.9	39.0	46.9	55.2	66.6	75.9	95.5	320
2 hr	37.0	51.0	61.0	71.3	85.4	97.1	104.8	480
3 hr	43.5	59.5	70.9	82.7	99.0	112.0	120.4	570
6 hr	58.2	79.4	94.9	110.7	133.3	152.2	176.0	770
12 hr	78.8	107.8	128.8	151.6	182.7	208.0	224.6	870*/870**
24 hr	106.2	147.4	177.8	210.2	254.4	289.7	316.1	1030
48 hr	139.1	196.2	238.6	283.9	342.0	387.2	453.0	1220
72 hr	158.1	225.2	275.5	327.8	391.9	443.2	499.3	1270

* GSDM (Generalised Short-Duration Method)

* GSAM (Generalised Southeast Australia Method)

7.2 Ocean Boundary Conditions

The joint probability of simultaneous occurrence of rare catchment flooding and rare ocean flooding is low. It is not appropriate to use the design ocean level of a rare ocean flooding as being an ocean boundary condition for a rare catchment flooding such as 1% AEP.

The Brisbane Water Foreshore Flood Study (Cardno, 2013) analysed water level data at Koolewong and Ettalong and established downstream boundary conditions for any flood studies of creeks that drain to Brisbane Water. It recommends using the 1% probability of exceedance (PoE) water level, which corresponds to a 99% confidence level that the level will not be exceeded during any creek flood event, as downstream boundary conditions. These levels at the selected locations along the estuary are summarised in Table 6.3 of the study report. Five of these locations are located within the study area.

Table 7.2 summarises the 1% PoE water levels and the corresponding levels with mean sea level rise (MSLR) calculated by Cardno at the foreshore of the Woy Woy peninsula. The locations are shown in **Figure 7.1**. The Brisbane Water Foreshore Flood Study reported the water level labelled “Ettalong Creek” at approximately 2km east to the actual location of the Ettalong Creek outlet. We kept this location as reported in this study and this is reflected in **Figure 7.1**.

Table 7.2 1% Probability of Exceedance Levels at Selected Foreshore Locations (Cardno, 2013)

Location	1% PoE Level (mAHD)	1% PoE Level +0.2m MSLR (mAHD)	1% PoE Level +0.9m MSLR (mAHD)
Ettalong	0.85	1.05	1.75
Ettalong Creek	0.93	1.13	1.83
Woy Woy	0.68	0.88	1.58
Woy Woy Bay	0.74	0.94	1.64
Rip Bridge	0.66	0.86	1.56

Figure 7.1 Locations within the study area where 1% Probability of Exceedance Levels were reported These locations were extracted from Figure H1 in Brisbane Water Foreshore Flood Study (Cardno, 2013

7.3 Antecedent Catchment Condition for Design Events

The initial groundwater level profile over the study area is required as an input to the design event models. Experience shows that adopting median antecedent conditions (e.g. for soil moisture/rainfall losses or reservoir water levels) for flood risk modelling generally leads to an underestimation of flood risk. However, adopting a high antecedent condition (e.g. worst case) can lead to an overestimation of flood risk. The previous flood study (DHI, 2010) used analogous example selecting the antecedent water level in a large reservoir when determining the flood frequency of downstream flooding: an antecedent condition that approximates the 80th percentile occurrence gives similar flood risk estimates to a more thorough study involving complex joint probability analysis.

Based on this, the initial ground water level equivalent to the 80th percentile occurrence was adopted for the design event modelling. The 80th percentile of groundwater levels were determined based on long-term groundwater levels simulated with a coarser 100m groundwater model for Woy Woy (DHI, 2020). A statistical analysis was undertaken at 6 bore locations (WW8, WW32, WW39, WW40, WW43 and WW44) across the catchment (**Figure 7.2**). Based on this analysis, groundwater levels on 1/3/1984 were selected as antecedent conditions for the design models. The groundwater surface in the form of depth below ground is shown in **Figure 7.2**.

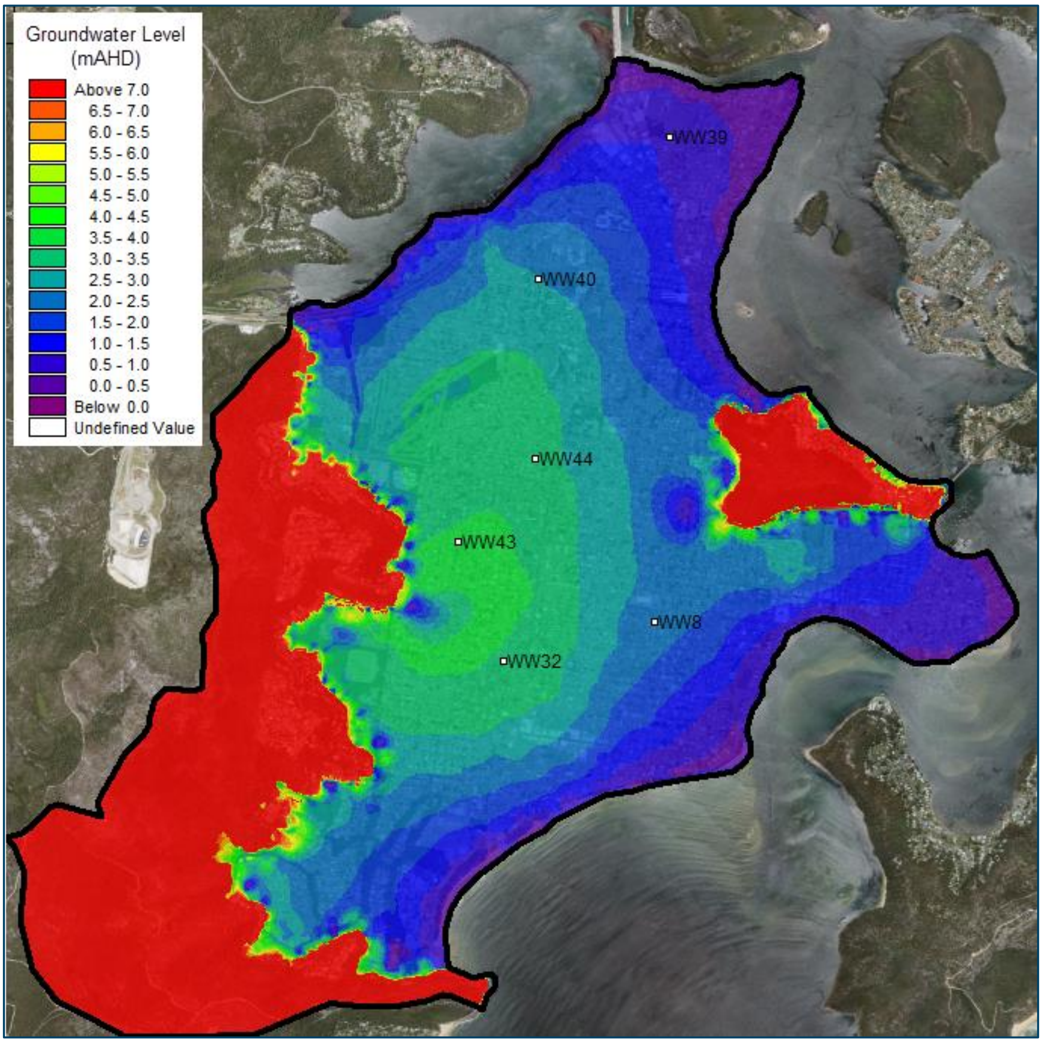

Figure 7.2 Adopted antecedent groundwater table from 1/3/1984 for design event modelling and monitoring well locations used for the statistical analysis

7.4 Evapotranspiration

The long-term average evapotranspiration (3.19 mm/d) is used for design events.

7.5 Other assumptions

All inlet structures and pipes are assumed to function for the existing condition simulation (Baseline).

7.6 Determination of Temporal Patterns for each duration and AEP

Traditionally a single burst temporal pattern was used to distribute rainfall over time. As outlined in **Section 3**, there have been major changes in the ARR guideline, and 10 temporal patterns are generated for each AEP, duration, and region. This means that total 80 rainfall timeseries (10 temporal patterns x 8 durations) are available for each AEP in ARR Data Hub (ARR Data Hub).

It should be noted that ensembles of temporal patterns were generated for each frequency (AEP group), duration and region, as **Figure 7.3** demonstrates. For example, the temporal patterns for 1-hour duration event differ from the ones for 24-hours duration event of 1%AEP. Similarly, temporal patterns for 1hour duration of 1%AEP (Rare frequency) differ from the ones of 1-hour duration of 10%AEP (Intermediate frequency).

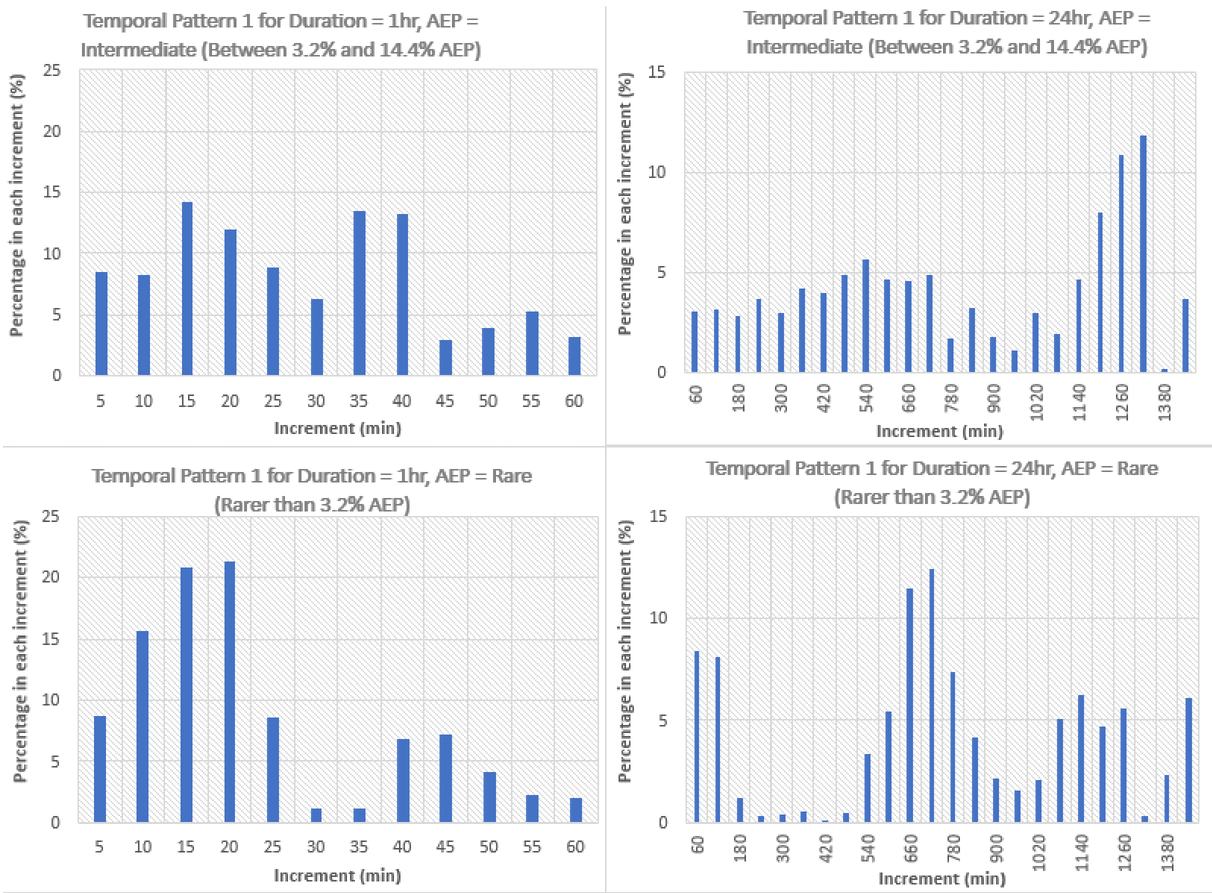

Figure 7.3 Example of temporal patterns for different AEPs and durations

It is not practical to run all 10 temporal patterns for each duration and each AEP in a two-dimensional model due to its computational time. Most other flood studies run a separate conceptual/ lumped hydrological model in order to determine the pattern which provide the average peak flow as the computational effort for running all design scenarios with the lumped model is significantly lower than with the 2D model. However, a typical conceptual/lumped type of hydrological model can be only applied to catchments where a single well-defined catchment outlet with a calibration data exists. This is not the case for the Woy Woy peninsula, except the Kahibah Creek Catchment.

Therefore, to select a temporal pattern which provides an average peak flow, it was decided to set up a conceptual hydrological model for the Kahibah Creek Catchment and then to run all rainfall timeseries with this model. The NAM rainfall-runoff model, which forms a part of rainfall-runoff modules in the MIKE HYDRO package (www.mikepoweredbydhi.com) was used as a conceptual model. As no flow observation data is available in the catchment, modelled outflows from the catchment from the MIKE SHE 2-D flood model for the 1988 event were used. Setting up the NAM model for the Kahibah Creek System is described in **Appendix D.**

The calibrated NAM model was used to run all design scenarios (7 AEPs x 8 durations x 10 temporal patterns = 560 scenarios) and to narrow down which temporal pattern should be applied in the 2D model (entire study area).

For each AEP and duration, the temporal pattern that produced the peak closest to the median peak flow was selected. For the Probable Maximum Precipitation (PMP) the temporal pattern that produced the highest peak was determined. The selected temporal patterns for each AEP and duration are presented in **Table 7.3**.

Table 7.3 Selected temporal pattern IDs for each duration and AEP

Duration	Frequency (AEPs)						
	50%	20%	10%	5%	2%	1%	0.50%
1 hr	1	3	8	7	4	9	7
2 hr	4	4	8	10	2	7	2
3 hr	6	6	4	4	5	9	9
6 hr	7	7	2	1	4	9	9
12 hr	3	2	7	3	10	2	1
24 hr	2	2	3	10	3	8	8
48 hr	9	9	8	5	7	2	2
72 hr	6	10	6	5	6	8	8

8 Design Runs

8.1 Existing Conditions (Baseline)

As summarised in 7.6, selection of temporal patterns for each duration (1, 2, 3, 6, 12, 24, 48 and 78 hrs) was carried out by running conceptual hydrological model for 560 scenarios (8 durations x 10 temporal patterns x 7 AEPs) to avoid extensive computational costs of running them with the 2D flood model. Initially we aimed to select the critical duration using the results of these conceptual hydrological model runs, as well as temporal patterns. However, the analysis of these results showed short duration events such as 2hr, 3hr, and 6hrs as critical durations, whereas the 2010 flood study adopted the 72hr event as the critical duration. This could be due to the characteristics of the Kahibah Creek catchment where the hydrological model was established. Therefore, it was decided to simulate all durations with the 2D flood model.

Derived Flood Depth, Water Levels and Velocity maps are provided in **Appendix E**. It should be noted that these maps are generated as maximum of all durations for each AEP.

8.1.1 Comparison to the 2010 study

The design runs show a smaller flood extent and shallower flood depths in general compared to the flood maps produced in the previous flood study in 2010.

There are various changes to modelling of design runs since the previous study:

- Design rainfalls were updated in ARR. Both the rainfall intensities and temporal patterns were updated.
- The antecedent groundwater condition for design runs have been updated using the updated groundwater model. The groundwater model for the Woy Woy peninsula have been updated with the new LiDAR and recalibrated for the longer records of the bore data.
- The model topography was updated with the LiDAR 2013 data and refined from 10m to 5m.
- Refinement of the model allowed a finer representation of landuse.
- The stormwater drainage network has been updated with the new database, although this should not differ much from the previous study.
- The model domain was extended to cover the entire escarpment and the Kahibah creek catchment.
- The updated model was recalibrated.

All these changes contribute to the different flood extents and depths. Refinement of the model spatial resolution represents streets which works as surface drainage paths and the impervious areas in lots more accurately based on LiDAR. For example, whereas the 2010 study applied low permeability to all residential blocks, the current study represents approximate outlines of buildings based on LiDAR. This change can allow more infiltration at parts of allotments and less ponding to be simulated at allotments.

Changes to the design rainfall is also substantial. Rainfall intensities are smaller in this study than the 2010 study where rainfall intensities in ARR 1987 were applied. This is particularly evident in short duration events. While only one temporal pattern was available for each duration of design event, 10 temporal patterns were examined and the one producing a mean peak flow was adopted in 2D modelling. Differences in rainfall intensities are further discussed in the sensitivity analysis in **Section 8.4.5**.

The adopted antecedent groundwater conditions for design runs also differ significantly as shown in **Figure 8.1**. Both the 2010 study and current study adopted the 80th percentile groundwater level as the antecedent groundwater condition using the long-term groundwater simulations. However, the current study's antecedent condition is based on the updated groundwater model which was well calibrated against much longer records of bore data in the recent *Woy Woy Integrated Water Management and Case Study Everglades Catchment Study* (DHI, 2021). As seen in **Figure 8.1**, the antecedent groundwater levels adopted in the 2010 study are approximately 1 to 2m higher than the one in the current study. This is much higher than the 90th percentile groundwater level derived in the current study (See the sensitivity analysis in **Section 8.4.1**), which is only 0.1 to 0.2m higher than the 80th percentile groundwater level. A higher antecedent groundwater level means more wet soil prior to a rain event and thus less infiltration loss. This could have resulted in overestimation of flooding, especially groundwater driven flooding at some locations and have had some consequences on the selection of critical durations in the previous study. Given the improved groundwater model, the prediction of the 80th percentile groundwater level and the antecedent catchment condition for design events in this study has a greater confidence.

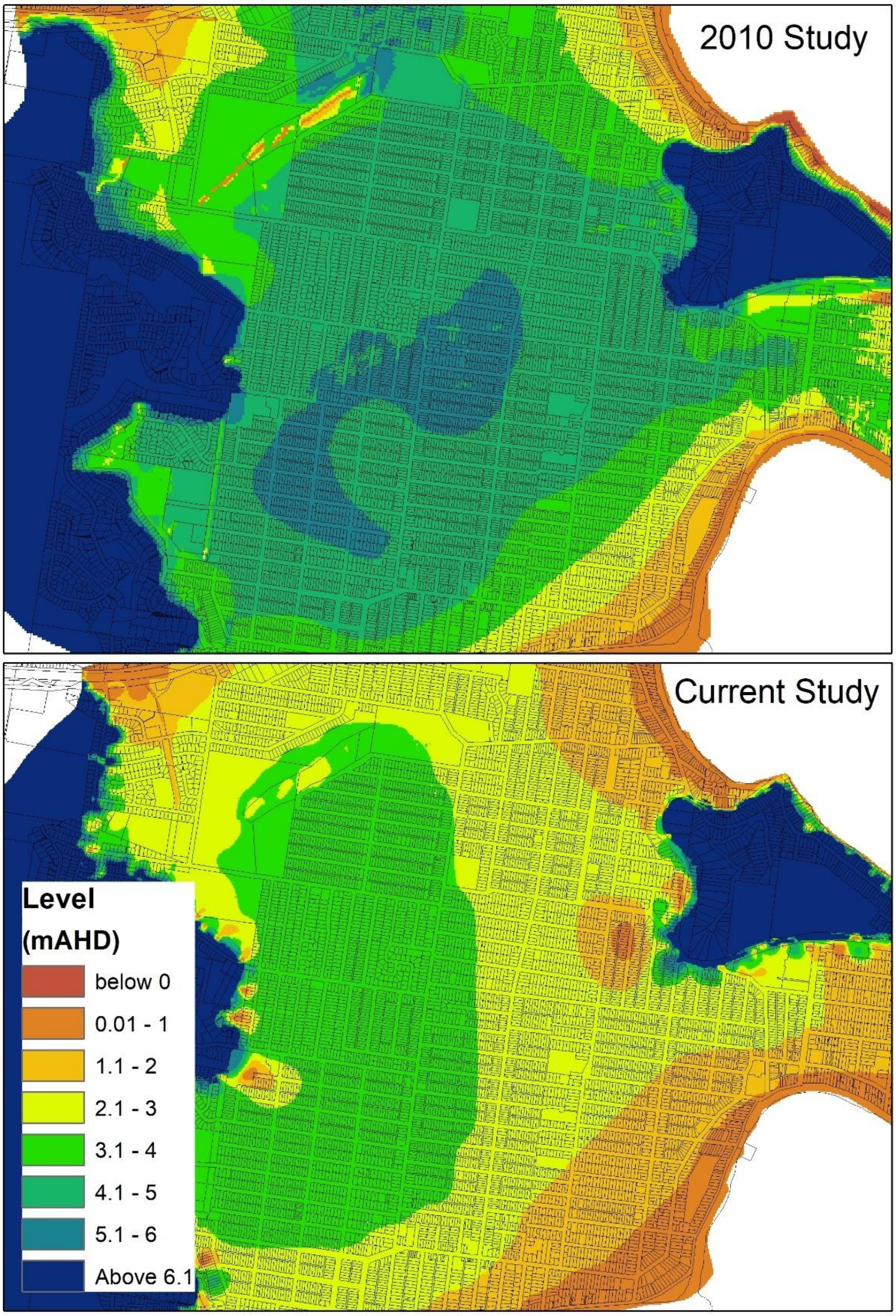

Figure 8.1 Adopted antecedent groundwater levels in the 2010 study (Top) and in the current study (Bottom)

8.2 Selection of Critical Durations

As per **Section 7.1**, events of 1,2,3,6,12,24,48 and 72 hrs durations which were assessed in the previous flood study (DHI, 2010) were analysed. Peak water depths were compared at selected locations that typically get flooded in the catchment as shown in **Figure 8.2**. The duration that returned the largest water depths are summarised in **Table 8.1** with the numbers in brackets being maximum and minimum peak depths (m) calculated by different durations.

Figure 8.2 Locations of Critical Duration Spot Assessment

In addition, the duration which returned the largest peak depth was extracted at each computational grid with a simulated water depth of more than 0.1m and statistics calculated. The *All Grids* column shows the duration which produced the largest peak depth at most computational grids.

The *Selected critical durations* column shows the selected durations for each design event. Selection was based on the following criteria.

1. Check which duration is most critical at the selected 11 locations. E.g. for 1%AEP, 6hrs is the critical duration at 9 out of 11 locations.
2. Check which duration is most critical at all grids. E.g. for 1% AEP 6hrs is critical at most grids.
3. If durations selected by 1 and 2 are different, compare the peak depths at the selected 11 locations.

For example, for PMF, 1hr duration returned the largest flood depth at 6 locations out of 11 selected locations but 2hrs duration is critical over all grids. The figure below shows the flood depths produced by PMF 1hr and PMF 2hr simulations. The difference in flood depths between PMF1hr and PMF2hr is generally small except at Neera Road, where the 2hr duration produces a significantly higher peak depth. Therefore, 2hrs was chosen as the critical duration for the PMF.

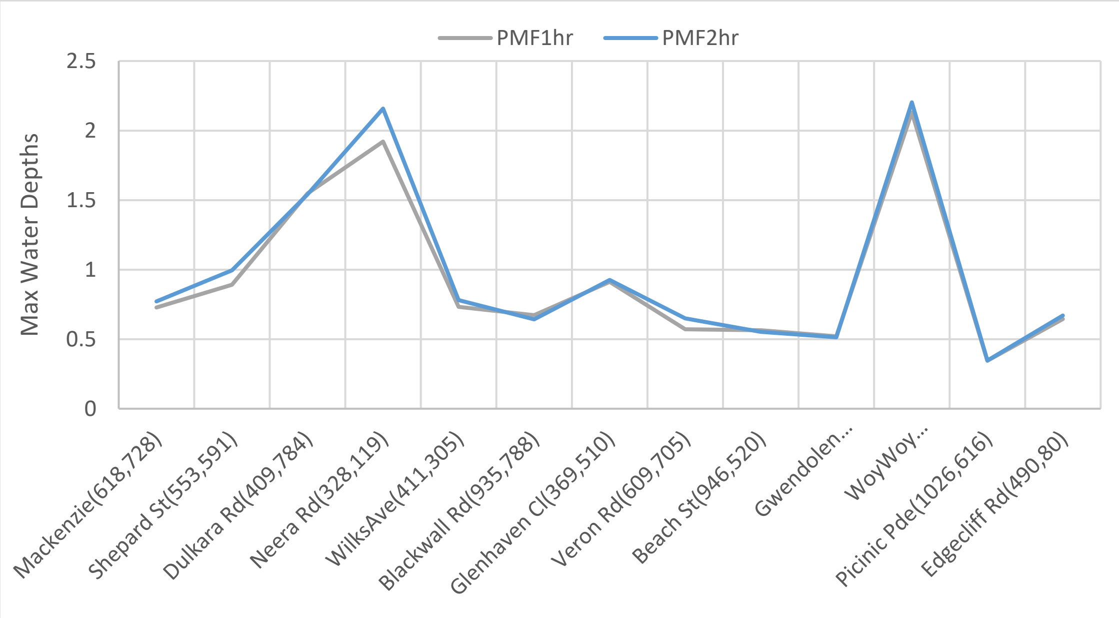

Figure 8.3 Comparison of peak water depths of 1hr and 2hr duration events of the PMF

For the 50% AEP, 2hrs is most critical at 6 out of 11 selected locations while 6hrs is critical at most grids. The comparison of peak water depths simulated at 11 locations are shown below. Difference in peak water depths is generally small between 50% AEP 2hrs and 50% AEP 6hrs except Woy Woy Centre where 6hrs produced significantly larger peak depth. Therefore, 6hrs was selected as the most critical duration for 50% AEP.

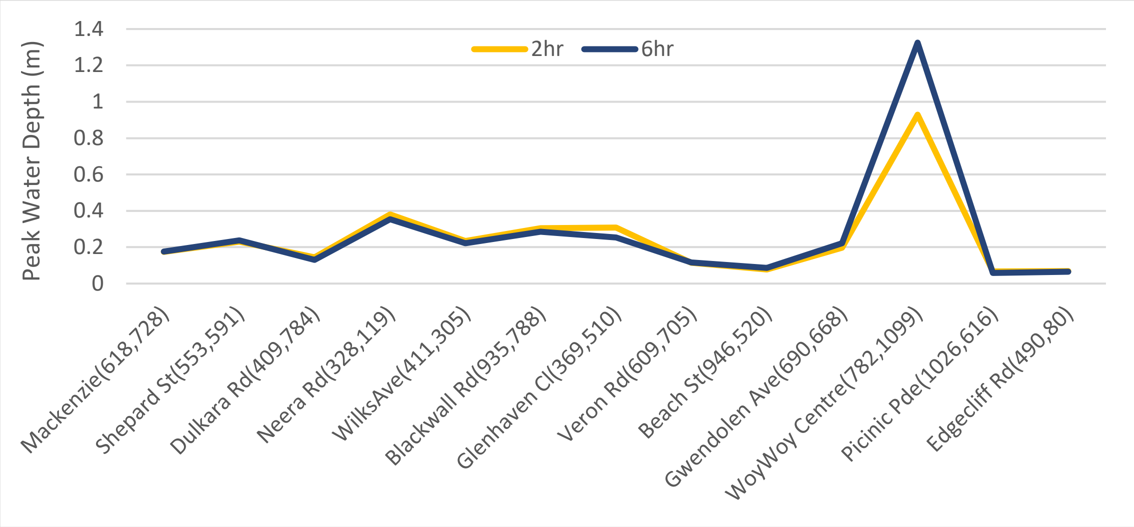

Figure 8.4 Comparison of peak water depths of 2hr and 6hr duration events of the 50% AEP

The selected critical duration for each AEP and PMF is used for sensitivity analysis and option assessment.

Shorter durations such as 2hrs and 6hrs were selected as critical durations, contrary to 72hrs adopted in the previous flood study (DHI, 2010). This difference in critical durations was closely examined. The main reason was the much higher antecedent groundwater condition adopted in the previous study, as discussed in **Section 8.1.1**. Due to the high antecedent groundwater level, the catchment had no infiltration capacities at the start of a rainfall event and during the event. This resulted in that a longer duration event with a larger total volume was more dominant for high water depth. Meanwhile, the better calibrated antecedent groundwater condition in this study provides more realistic infiltration capacities and higher water depths were simulated in a shorter duration event with a larger peak rainfall intensity.

Table 8.1 Most critical durations at the selected locations (the numbers in the brackets are maximum and minimum water depths simulated with different durations)

	Mackenzie	Shepard St	Dulkara Rd	Neera Rd	WilksAve	Blackwall d	Glenhaven Cl	Veron Rd	Beach St	Gwendolen Ave	WoyWoy Centre	All Grids (D>0.1m)	Selected Critical Durations
PMF	12hr_GSDM (0.9 - 0.73)	6hr (1.14 - 0.89)	1hr (1.55 - 1.1)	2hr (2.16 - 0.52)	2hr (0.78 - 0.35)	1hr (0.67 - 0.39)	6hr (0.98 - 0.39)	6hr (0.78 - 0.57)	1hr (0.56 - 0.46)	12hr_GSDM (0.59 - 0.48)	2hr (2.2 - 1.91)	1hr	2hr
0.5%AEP	6hr (0.46 - 0.35)	48hr (0.51 - 0.35)	6hr (0.9 - 0.61)	2hr (0.65 - 0.4)	2hr (0.39 - 0.28)	2hr (0.41 - 0.3)	2hr (0.52 - 0.28)	3hr (0.28 - 0.15)	6hr (0.24 - 0.15)	6hr (0.3 - 0.27)	48hr (1.73 - 1.56)	2hr	2hr
1%AEP	6hr (0.44 - 0.31)	6hr (0.43 - 0.32)	6hr (0.9 - 0.59)	1hr (0.63 - 0.41)	6hr (0.37 - 0.27)	6hr (0.39 - 0.29)	1hr (0.49 - 0.27)	6hr (0.25 - 0.14)	6hr (0.23 - 0.14)	6hr (0.3 - 0.27)	48hr (1.69 - 1.53)	48hr	6hr
2%AEP	6hr (0.41 - 0.3)	2hr (0.4 - 0.3)	12hr (0.57 - 0.25)	2hr (0.6 - 0.36)	2hr (0.36 - 0.27)	2hr (0.38 - 0.29)	2hr (0.48 - 0.25)	2hr (0.23 - 0.13)	2hr (0.2 - 0.13)	2hr (0.28 - 0.26)	12hr (1.66 - 1.47)	2hr	2hr
5%AEP	6hr (0.39 - 0.25)	3hr (0.36 - 0.27)	6hr (0.34 - 0.18)	2hr (0.59 - 0.34)	2hr (0.34 - 0.25)	2hr (0.38 - 0.28)	2hr (0.49 - 0.23)	2hr (0.21 - 0.12)	6hr (0.18 - 0.1)	6hr (0.27 - 0.24)	24hr (1.59 - 1.33)	2hr	2hr
10%AEP	6hr (0.35 - 0.16)	6hr (0.32 - 0.24)	3hr (0.24 - 0.17)	3hr (0.48 - 0.32)	3hr (0.28 - 0.24)	3hr (0.33 - 0.27)	1hr (0.39 - 0.22)	6hr (0.17 - 0.11)	3hr (0.15 - 0.09)	12hr (0.26 - 0.22)	48hr (1.58 - 1.21)	3hr	3hr
20%AEP	6hr (0.27 - 0.13)	6hr (0.28 - 0.21)	12hr (0.2 - 0.14)	2hr (0.46 - 0.23)	1hr (0.28 - 0.23)	1hr (0.33 - 0.26)	1hr (0.4 - 0.18)	12hr (0.15 - 0.09)	6hr (0.12 - 0.07)	48hr (0.25 - 0.2)	48hr (1.58 - 1.03)	1hr	1hr
50%AEP	6hr (0.18 - 0.13)	6hr (0.24 - 0.18)	2hr (0.14 - 0.09)	2hr (0.38 - 0.21)	2hr (0.24 - 0.17)	2hr (0.3 - 0.26)	1hr (0.35 - 0.18)	6hr (0.12 - 0.09)	6hr (0.09 - 0.07)	48hr (0.24 - 0.17)	48hr (1.56 - 0.74)	6hr	6hr

8.3 Climate Change Scenarios

Council adopts Representative Concentration Pathway Scenarios (RCP) 8.5 as the climate change projection. RCP8.5 is a scenario with greenhouse gas emissions continuing to rise throughout the 21st century and considered as the worst-case climate change scenario.

Interim Climate Change Factor or the projected rainfall intensity increase are available through ARR Data Hub (arr-software.org).

Council Policy (March 2015) defines the projected medium local sea rise for 2050, 2070, and 2100; 0.20, 0.39 and 0.74m.

Three climate change scenarios were considered for the projected year 2050, 2070 and 2090. **Table 8.2** summarises the climate change scenarios and the projected sea level rise (SLR) in conjunction with rainfall intensity increase (RII) conditions.

Table 8.2 Climate change scenarios

Scenario	Project Year	Sea level rise	RCP8.5 Interim Climate Change Factor (%)
Climate Change 1	2050	0.20 m	9.0%
Climate Change 2	2070	0.39 m	14.2%
Climate Change 3	2090	0.74 m	19.7%

A flood study typically considers the sea level rise and rainfall intensity change only during a flood event and ignores the impact of these on the antecedent catchment conditions. However, the sea level rise and rainfall intensity increase will likely elevate the average groundwater level in the study area and flooding in a part of the Woy Woy peninsula is sensitive to the antecedent groundwater levels. Therefore, it was decided to test how much the flood extent is affected by the antecedent catchment conditions.

1. Adopt the Climate Change 3 scenario (worst projected) and rerun the long-term peninsula groundwater model.
2. Reselect the 80percentile groundwater level of the long-term model with the Climate Change 3 conditions and use it as the antecedent catchment condition. This corresponded to the groundwater level on 23 March 1984. The initial groundwater level is 0.2-0.7m higher than the one adopted in the Baseline design run.

Figure 8.5 shows the increase of flood depth from Baseline under Climate Change 3 scenario with the updated antecedent catchment conditions. **Figure 8.6** shows the increase of flood depth from Baseline under Climate Change 3 scenario, discarding the climate change impact on the antecedent catchment conditions.

A comparison of the two figures highlights that the difference of 0.1-0.2m is primarily found at the bottom of the escarpment and in the Everglades catchment where the groundwater mound is located. These areas are known to have relatively shallow sand layers and the flooding is affected by the groundwater level. Besides these areas, the antecedent catchment conditions generally do not have a large impact on the flood extent in the 1%AEP design event.

Therefore, Climate Change 1 and 2 scenarios were run disregarding the impact of climate change on the antecedent conditions. These results are summarised in the Woy Woy Floodplain Risk Management Study (DHI, 2022).

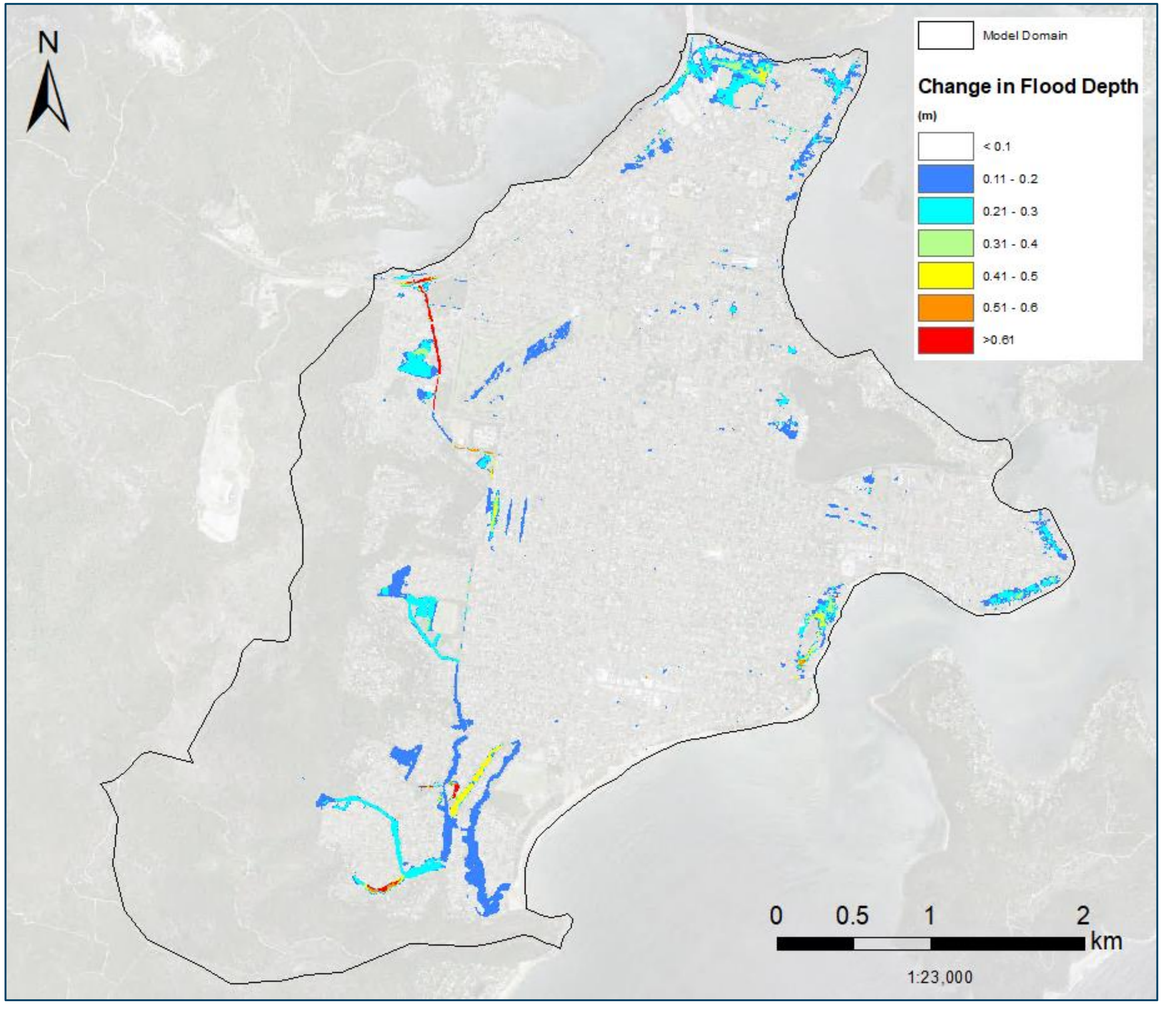

Figure 8.5 Change in flood depth from Baseline (Climate Change 3 with the updated antecedent conditions minus Baseline)

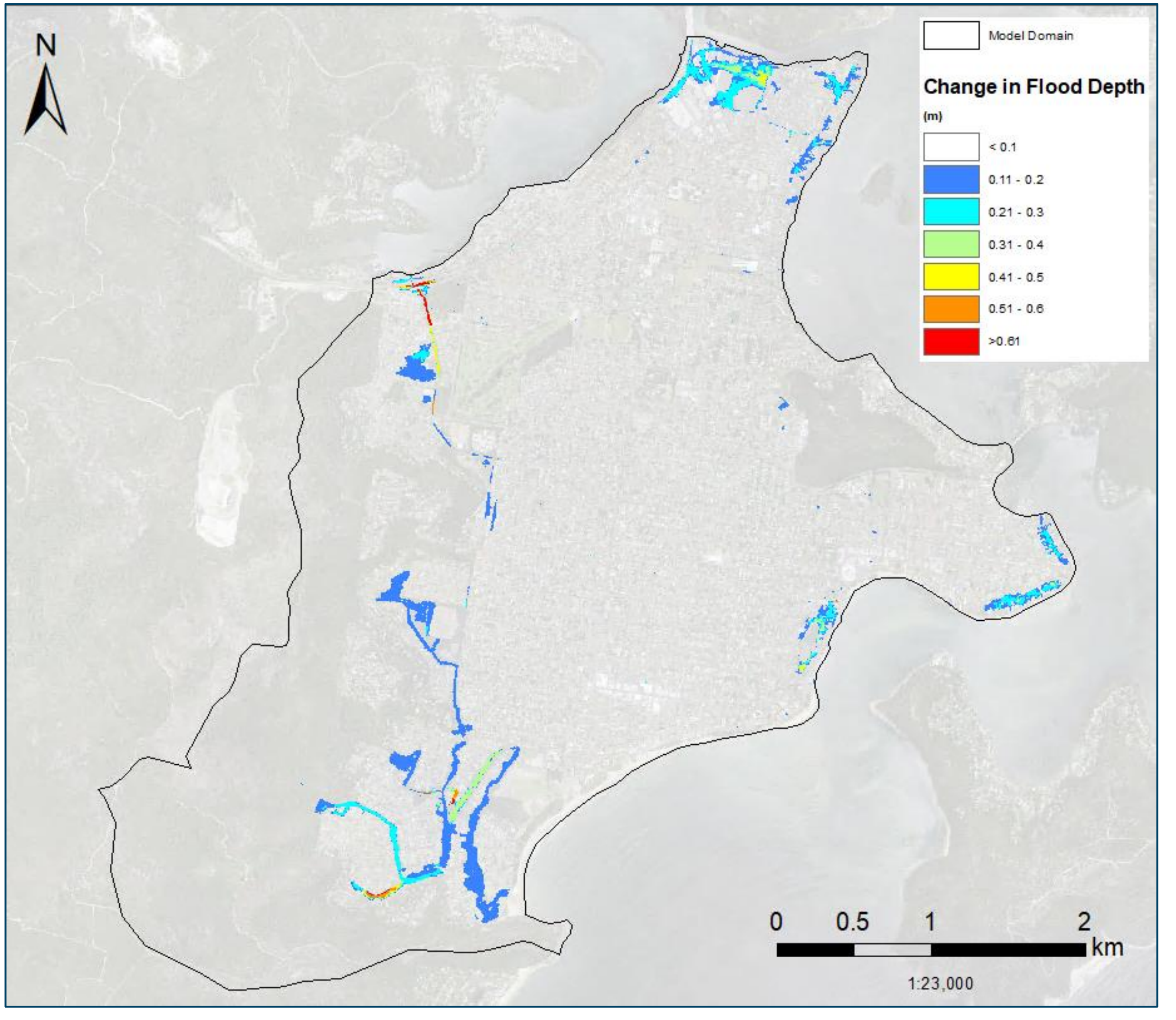

Figure 8.6 Change in flood depth from Baseline (Climate Change 3 minus Baseline)

8.4 Model Parameter Sensitivity

Sensitivity analysis was undertaken for the key model parameters summarised in **Table 8.3**.

The assessment was carried out based on the 1% AEP design run (and 5% AEP for the ARR 1987 rainfall scenario) for the critical duration (6hrs).

Table 8.3 Summary of Sensitivity Analysis

Sensitivity parameter	Description	AEPs
Infiltration losses	The 90th percentile antecedent groundwater levels derived from the long-term groundwater model will be applied as the initial condition.	1%
Hydraulic roughness	Manning's n roughness will be increased by 20%.	
Blockage	Hydraulic structures blockage will be applied	
Downstream Oceanic Boundary Conditions	Sensitivity to ocean boundary condition will be tested by increasing by 0.2m.	
ARR 1987 design rainfall	Design rainfall was developed based on ARR 1987 and the model were run for the same critical durations as in the base runs. The results will be compared to ARR 2019 results.	1%, 5%

8.4.1 Infiltration losses

The antecedent groundwater condition affects the initial and continuous infiltration loss of a rainfall event, in particular in the area where the groundwater table is close to the ground surface, i.e. near the groundwater mound in the Everglades catchment.

Woy Woy Integrated Water Management and Case Study Everglades Catchment (DHI, 2021) run the groundwater model over 100 years. This demonstrated that there is a significant seasonal and annual variation in shallow groundwater tables depending on the location. The difference between the minimum and the maximum groundwater tables over 100 years is up to 3.4m at inland locations, while the groundwater table close to the coastal area have little variations, being tied up with the sea level.

The Baseline design events were run with the antecedent groundwater condition that approximates the 80th percentile condition, as described in 6.2. The 80th percentile was proposed following the methodology adopted in the previous flood study (DHI, 2010) due to the lack of a clear guideline for determination of an antecedent groundwater condition for flood studies. The previous study adopted an analogous example of selecting the antecedent water level in a large reservoir when determining the flood frequency of downstream flooding to select an antecedent groundwater level. It states "Joint probability analysis generally shows that adopting Full Supply Level (i.e. assuming the reservoir is always full prior to an event) is conservative, resulting in an overestimation of flood levels for a certain level of risk (e.g. 1% AEP). Adopting the median water level (occurs 50% of the time) results in underestimation of flood levels. This is due to the nonlinear relationship between antecedent conditions and flood risk and the significance is dependent on having a relatively large reservoir capacity relative to the flood flows.

Previous flood studies by DHI generally show that an antecedent condition that approximates the 80thpercentile occurrence gives similar flood risk estimates to a more thorough study involving complex joint probability analysis. Based on this, for the design event modelling, the initial ground water level equivalent to 80thpercentile occurrence has been adopted.”

For the sensitivity analysis, the 90th percentile equivalent condition was extracted from the long-term groundwater model and used as the antecedent groundwater condition. 5/5/1984 was selected as the date approximating the 90th percentile condition. In the low-lying flat part of the peninsula, the 90th percentile groundwater level is generally 0.1 to 0.2m higher than the 80^{th} percentile one.

Figure 8.7 shows the difference of the flood depth from the Baseline. Major differences of 0.1m up to 0.5m were observed along the bottom of the escarpment. This is due to a thin layer of the sand aquifer along the bottom of the escarpment resulting the groundwater table hits or reaches close to the ground surface at low-lying spots. Other minor differences may be a result of interpolation of antecedent conditions from the 100m coarser groundwater model to the 5m finer flood model.

While sensitivity testing was carried out for the 1% AEP 6hrs duration event, it is expected that the antecedent catchment condition will affect the shorter duration event more significantly.

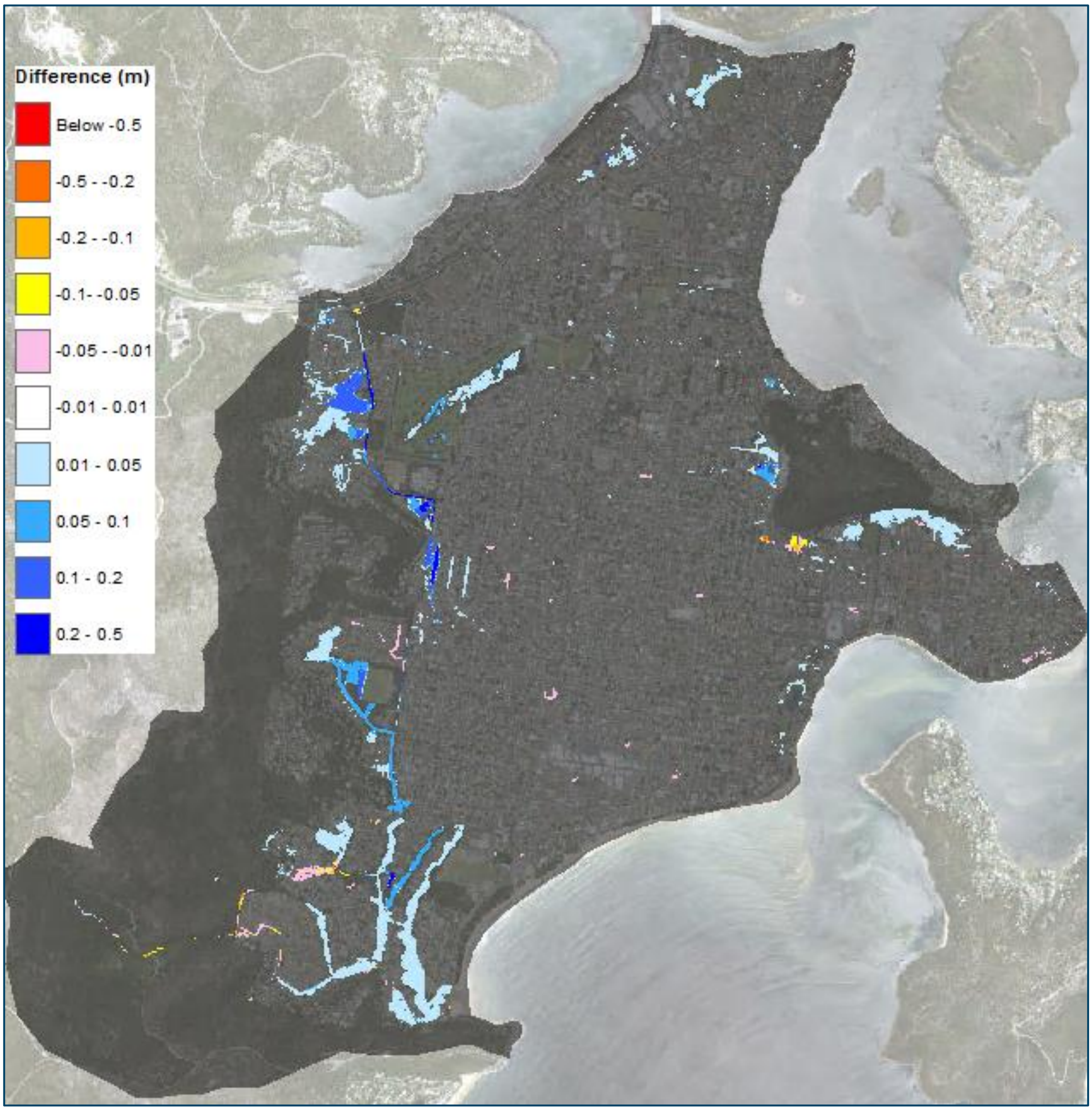

Figure 8.7 Difference in maximum water depth (higher antecedent groundwater minus Baseline)

8.4.2 Hydraulic Roughness

Sensitivity to the roughness parameter was tested in two ways:

1. Manning's roughness was increased by 20% only in the 2D domain (Roughness Scenario1)
2. Manning's roughness was increased by 20% both in the 2D domain and the 1D model including drainage pipes (Roughness Scenario2)

Figure 8.8 and **Figure 8.9** show differences in flood depth from the Baseline for 1) and 2), respectively.

At the escarpment, the flood depth increases slightly as surface water accumulates before draining down the slope. It is slightly lowered at the streets in the middle of the peninsula as water is retained longer at properties. Difference is generally smaller than 0.1m.

Increased roughness in the 1D model results in increased flood depths along the Kahibah Creek by up to 0.25m. Roughness increase can be caused by high vegetation in the channel or blockages of the drainage reserve.

It should be noted that the model was only calibrated against the reported approximate flood depths at the limited locations, since there are no stream gauges in any of the surface flow paths such as Ettalong Creek or Main Drain. While theoretical values of the roughness parameters were adopted in the model, the roughness of streams is usually estimated through calibration against the water level or discharge records. This sensitivity analysis indicates that there is relatively higher uncertainty in flood depths at Main Drain and the Kahibah Creek system. This uncertainty represents not only the calibration of the roughness parameters but also the channel conditions at the time of a flooding.

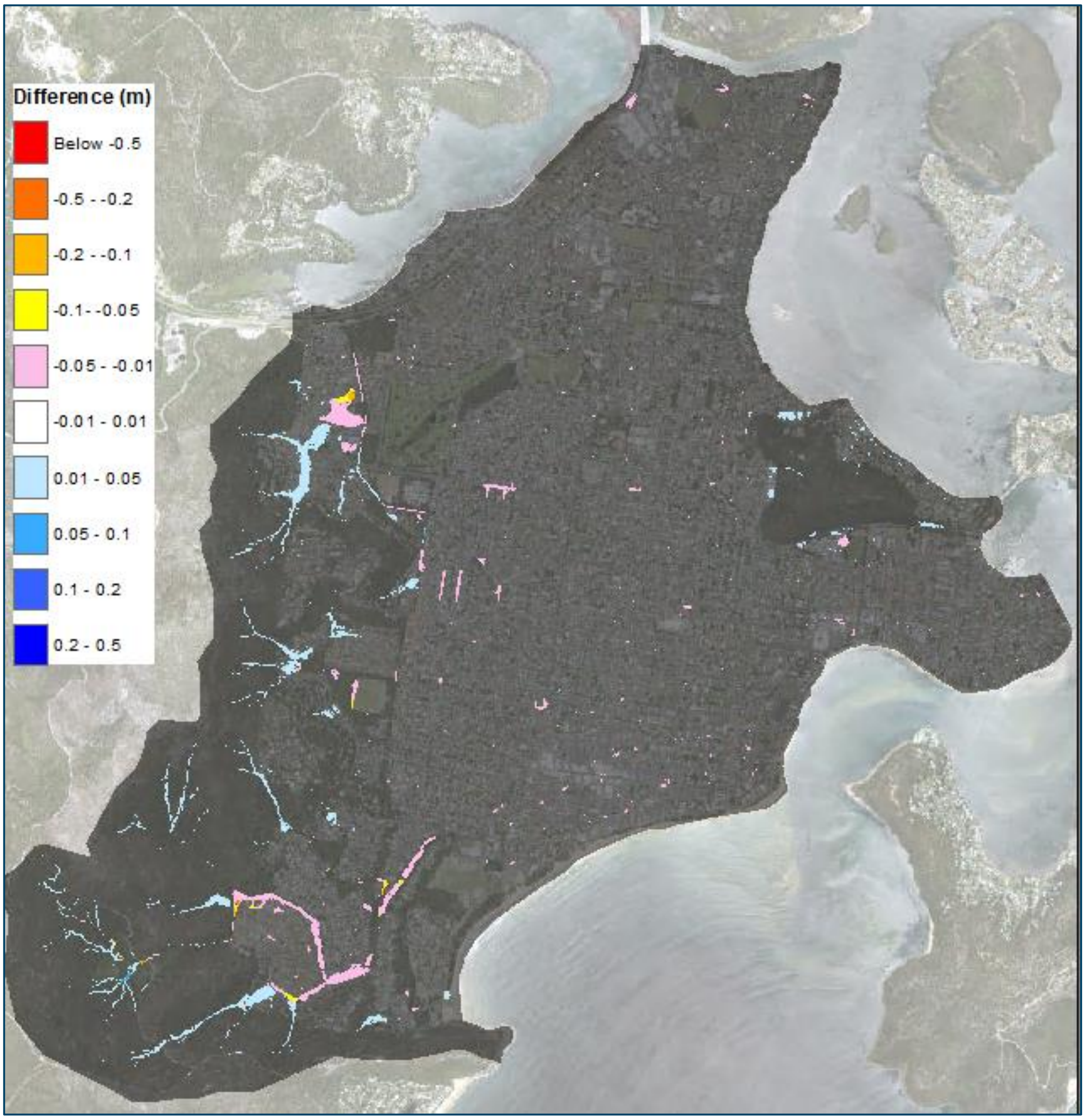

Figure 8.8 Difference in maximum water depth (Roughness Scenario 1 minus Baseline)

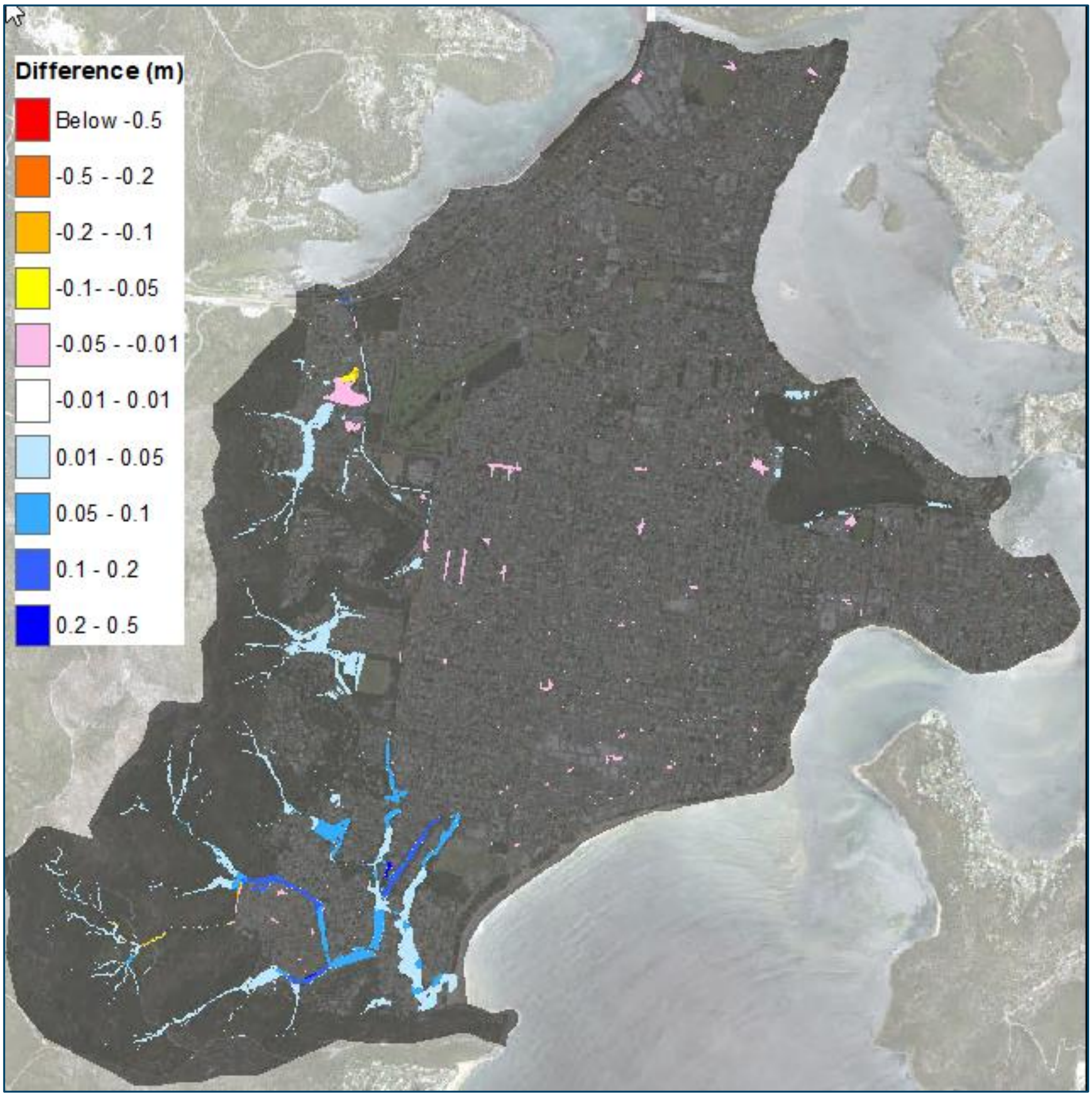

Figure 8.9 Difference in maximum water depth (Roughness Scenario 2 minus Baseline)

8.4.3 Blockage

The baseline design runs were run assuming that all inlets to the drainage systems and structures are cleared. This represent the condition with maximally utilised drainage capacities in the catchment.

ARR2019 states “The design blockage is the blockage condition that is most likely to occur during a given design storm and needs to be an “average” of all potential blockage conditions to ensure that the calculated design flood levels reflect the defined probability”. ARR does not provide a definitive approach for assessment of design blockage levels.

Reliable long-term records of availability, quantity and type of debris at a structure is rarely available. ARR recommends estimating the potential quantity of debris arriving at a structure from a contributing source area in a 1% AEP event by assessing debris availability in a source area (**Table 8.4**), debris mobility characteristics (**Table 8.5)** and debris transportability (**Table 8.6**). The 1% debris potentials based on the combination of availability, mobility and transportability are shown in **Table 8.7**. L10 is the average length of the longest 10% of the debris that could arrive at the site. In the absence of debris data, at least 1.5m is recommended as L10 in an urban area.

Table 8.8 summarises the assessment of the debris potential in different areas in the Woy Woy peninsula. Structures in Main Drain and the Kahibah system are in medium potential while structures in the remaining catchment are in low potential.

The ratio of the opening width of the structure to L10 is used to determine the likelihood of this material causing blockage at this structure.

Table 8.4 Debris Availability Classification (ARR 2019)

Classification	Typical Source Area Characteristics (1% AEP Event)
High	• Natural forested areas with thick vegetation and extensive canopy cover, difficult to walk through with considerable fallen limbs, leaves and high levels of floor litter. • Streams with boulder/cobble beds and steep bed slopes and steep banks showing signs of substantial past bed/bank movements. • Arid areas, where loose vegetation and exposed loose soils occur and vegetation is sparse. • Urban areas that are not well maintained and/or where old paling fences, sheds, cars and/or stored loose material etc., are present on the floodplain close to the water course.
Medium	• State forest areas with clear understory, grazing land with stands of trees. • Source areas generally falling between the High and Low categories.
Low	• Well maintained rural lands and paddocks with minimal outbuildings or stored materials in the source area. • Streams with moderate to flat slopes and stable bed and banks. • Arid areas where vegetation is deep rooted and soils are resistant to scour. • Urban areas that are well maintained with limited debris present in the source area.

Table 8.5 Debris Mobility Classification (ARR 2019)

Classification	Typical Source Area Characteristics (1% AEP Event)
High	• Steep source areas with fast response times and high annual rainfall and/or storm intensities and/or source areas subject to high rainfall intensities with sparse vegetation cover. • Receiving streams that frequently overtop their banks. • Main debris source areas close to streams.
Medium	• Source areas generally falling between the High and Low mobility categories.
Low	• Low rainfall intensities and large, flat source areas. • Receiving streams infrequently overtops their banks. • Main debris source areas well away from streams.

Table 8.6 Debris Transportability Classification (ARR 2019)

Classification	Typical Transporting Stream Characteristics (1% AEP Event)
High	• Steep bed slopes (> 3%) and/or high stream velocity (V > 2.5 m/s) • Deep stream relative to vertical debris dimension (D > 0.5L10) • Wide stream relative to horizontal debris dimension. (W > L_{10}) • Stream relatively straight and free of major constrictions or snag points. • High temporal variability in maximum stream flows.
Medium	• Stream generally falling between High and Low categories.
Low	• Flat bed slopes (< 1%) and/or low stream velocity (V < 1m/s). • Shallow depth relative to vertical debris dimension (D < 0.5*L_{10}). • Narrow stream relative to horizontal debris dimension (W < L_{10}). • Stream meanders with frequent constrictions/snag points. • Low temporal variability in maximum stream flows.

Table 8.7 Classification of the 1% AEP Debris Potential (ARR 2019)

Classification	Availability	Mobility	Transportability
High	High	High	High
High	High	High	Medium
Medium	Medium	Medium	Medium
Medium	High	Medium	Low
Medium	High	Medium	Medium
Medium	High	Low	Low
Low	Low	Low	Low
Low	Medium	Medium	Low
Low	Medium	Low	Low

Table 8.8 Assessment of Debris Potentials in the Woy Woy peninsula

Area	Source of debris	Availability	Mobility	Transportability	Debris Potential
Main Drain	National Park	Medium	Medium	Medium	Medium
Kahibah system	National Park	Medium	Medium	Medium	Medium
Remaining areas	Urban	Low	Low	Low	Low

8.4.3.1 Inlet blockage (Floating debris)

One of the mechanisms of the blockage is that a floating object bridges across the inlet of a structure. **Table 8.9** summarises ARR's inlet blockage levels based on the structure width at different debris potentials.

Based on this table, the blockage levels are estimated at each structure based on the debris potential as per **Table 8.8** and comparison of the width of the structure to L10. Without any data to validate debris dimensions, L10 was assumed to be 1.5m for the urban area and 2.5m for Main Drain and the Kahibah system. 1.5m was the recommended minimum value in an urban area in ARR and slightly larger value 2.5m was assumed for the Main Drain and the Kahibah system where large trees exist along the channels. **Figure 8.10** shows the inlet blockage levels calculated for each structure.

Table 8.9 Most likely Inlet Blockage Levels (ARR2019)

Control Dimension Inlet Clear Width (W) (m)	Debris Potential at the structure		
	High	Medium	Low
$W < L_{10}$	100%	50%	25%
$L_{10} \le W \le 3{*}L_{10}$	20%	10%	0%
$W > 3{*}L_{10}$	10%	0%	0%

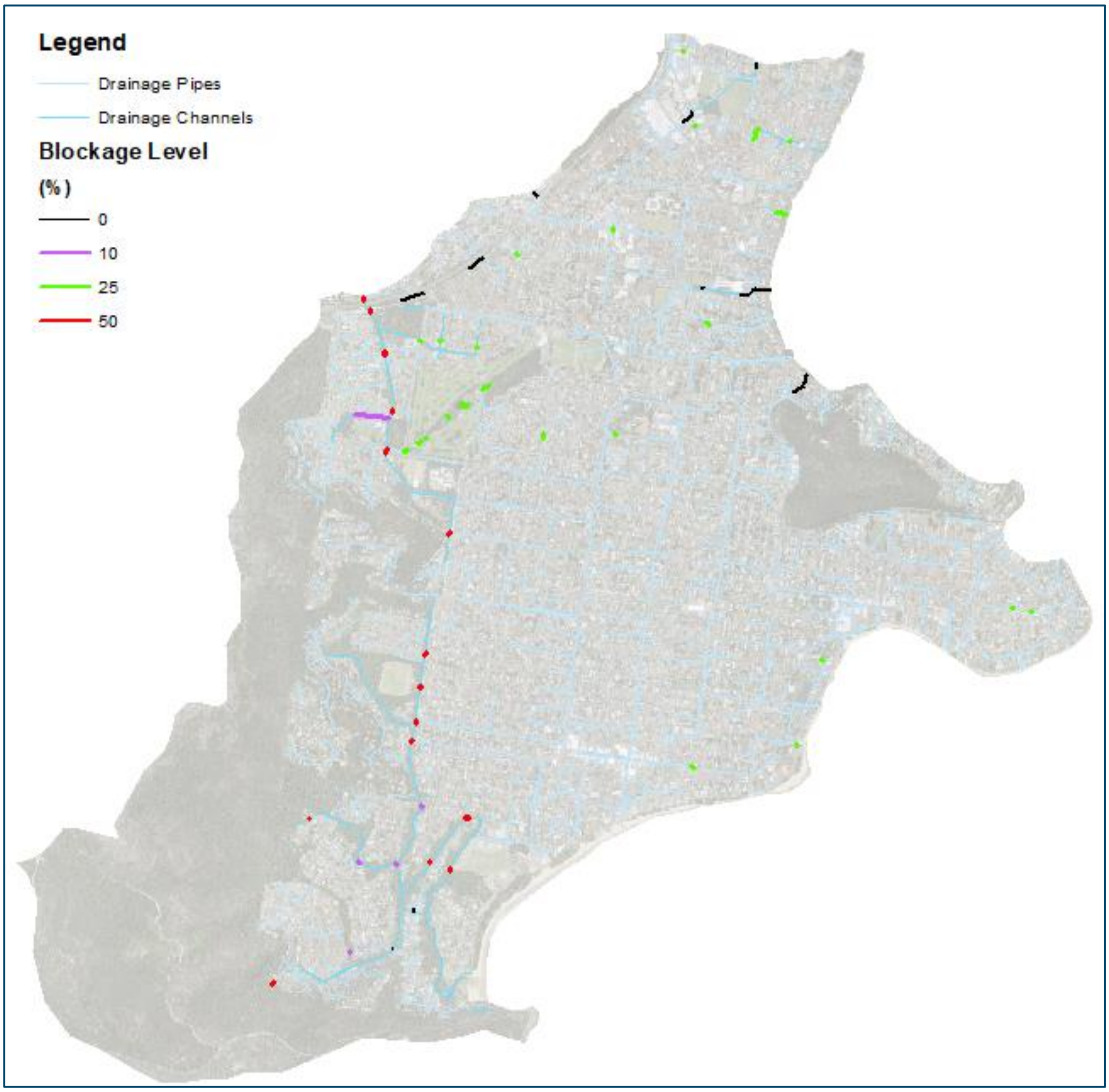

Figure 8.10 Inlet drainage blockage levels assumed for culvert structures

8.4.3.2 Barrel blockage (Non Floating debris)

ARR classifies the likelihood of sediment being deposited in the barrel or waterway of a structure based on the peak velocity through the structure and the mean sediment size. **Table 8.10** summarises the likelihood of sediment being deposited in barrel or waterway based on a peak velocity through structure when the mean sediment size is in the sand range (0.04-2mm). Table 8.11 summarises the corresponding likely depositional blockage levels based on the likelihood and the debris potential.

The peak velocities of most structures are above 0.1m/s. As the major sediment material is sand, the likelihood of sediment deposition is low according to **Table 8.10**. Then, most likely depositional blockage levels are 0%.

Table 8.10 Likelihood of sediment deposition in Barrel/Waterway (ARR 2019)

Peak Velocity Through Structure (m/s)	Sand 0.04 to 2 mm
>= 3	Low
1.0 to < 3.0	Low
0.5 to < 1.0	Low
0.1 to < 0.5	Low
< 0.1	Medium

Table 8.11 Most Likely Depositional Blockage Levels (ARR 2019)

Likelihood that deposition will occur	Debris Potential at the structure		
	High	Medium	Low
High	100%	60%	25%
Medium	60%	40%	15%
Low	25%	15%	0%

8.4.3.3 Drainage pipe blockage

ARR does not cover blockages of drainage inlets and pipes. For the sensitivity test, we assumed the pipe diameter is reduced to 100mm.

8.4.3.4 Modelling summary

Summarising 8.4.3.1, 8.4.3.2 and 8.4.3.3, the following blockage was applied for sensitivity analysis:

- All drainage pipes larger than 100mm were reduced to 100mm diameter.
- Culvert/Bridge structures are reduced by the blockage levels shown in **Figure 8.10**.

Figure 8.11 shows differences in flood depth from the Baseline. Due to blockage in drainage pipes, much more flow is retained on the streets and generally in upstream catchment areas. Flood depths in the receiving open channels are sometimes reduced despite blockage applied to culverts/bridges.

This demonstrates that flooding at low points on streets are highly sensitive to blockage of the drainage pipes. The baseline design runs assumed the drainage system are maintained at its capacity by the regular maintenance program. It is important to clear the blockage of inlets and pipes at a regular basis for flood risk management on streets.

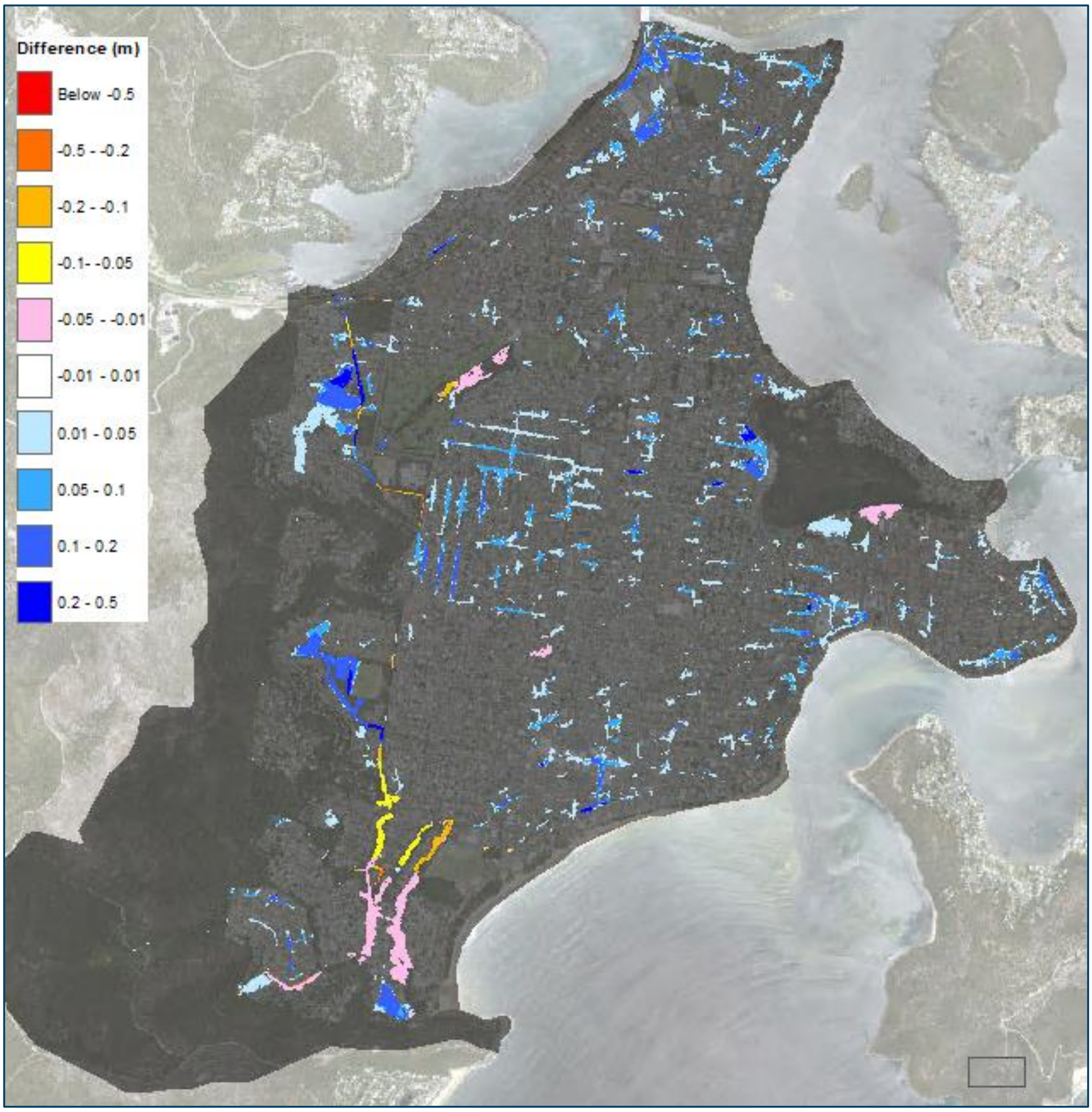

Figure 8.11 Difference in flood depth (Blockage minus Baseline)

8.4.4 Downstream Oceanic Boundary Condition

Sensitivity to the downstream ocean boundary condition was tested by raising all the sea boundary level by 0.2m. It should be noted that the ocean level was raised only during the design event and the antecedent groundwater condition was not affected.

Figure 8.12 shows the difference in flood depth from the Baseline. The flood depths affected by the sea level rise are along Main Drain at the mouth of Ettalong Creek and at the low-lying area (near the Woy Woy centre and Brick Wharf Rd). The differences are mostly less than 0.1m. The remaining area above the sea level was not affected.

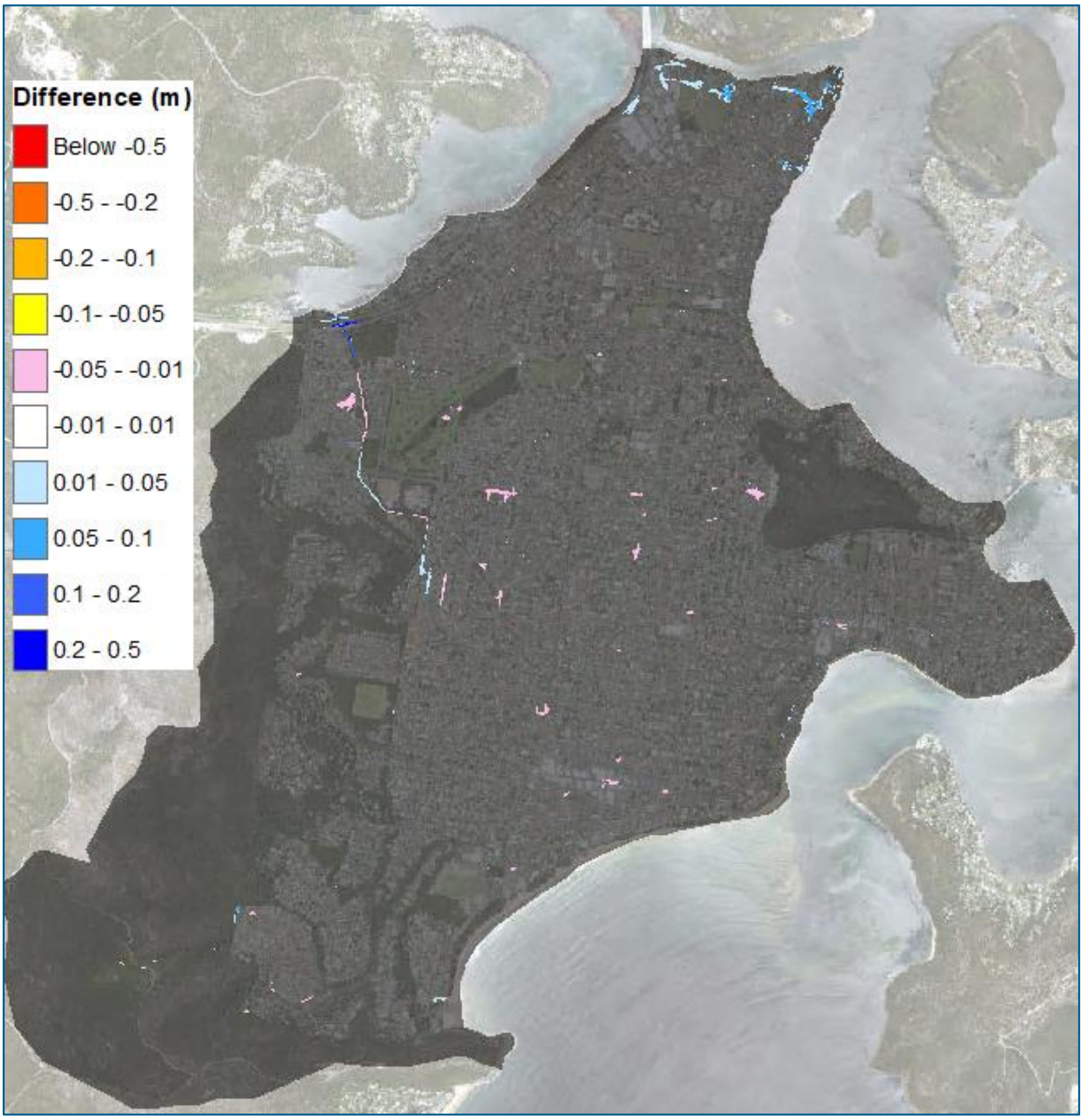

Figure 8.12 Difference in flood depth (Sea Level Increase minus Baseline)

8.4.5 ARR 1987

The Intensity-Frequency-Duration (IFD) design rainfalls were updated in 2016. **Table 8.12** compares the design rainfall intensity (Point) for 1% AEP and 5% AEP of ARR 1987 and ARR 2019. Generally, the intensities are higher in ARR 1987 especially in short durations. Rainfall intensity of 1%AEP 6hrs duration (critical duration) in ARR 1987 is approximately 14% higher than ARR 2019, while rainfall intensity of 5% AEP 2hrs duration (critical duration) in ARR 1987 is approximately 20% higher than in ARR 2019.

In addition, 10 temporal patterns are available for each frequency and each duration in ARR 2019. For the 2D flood modelling, one temporal was selected as described in **Section 7.6**.

Figure 8.13 compares the selected temporal pattern for 1% AEP 6hrs duration (critical duration) in ARR 2019 and the temporal pattern used in ARR 1987. Similarly, **Figure 8.14** compares the selected temporal pattern for 5% AEP 2hrs duration (critical duration) in ARR 2019 and the corresponding temporal pattern used in ARR 1987.

In this sensitivity test, the old design rainfall timeseries from ARR 1987 used in the 2010 flood study (DHI) were applied to the model, while maintaining the same initial and boundary conditions as the baseline.

Figure 8.15 shows the difference in flood depth between the run with ARR 1987 1% AEP 6hrs rainfall and the baseline. While higher flood depths were simulated at the low-lying areas due to the 14% higher rainfall intensity, a small decrease of less than 5cm is seen mainly at the bottom of the escarpment. This is likely due to the difference in temporal patterns. Significantly higher flood depths are simulated at the lower Kahibah creek system.

Figure 8.16 shows the difference in flood depth between the run with ARR 1987 5% AEP 2hrs rainfall and the baseline. A small increase (<5cm) in flood depth was produced. Significantly higher flood depth is produced at the lower Kahibah system.

Table 8.12 1% AEP and 5% AEP Design Rainfall Comparison between ARR 1987 and ARR 2019

	1% AEP		5% AEP	
Duration	**ARR 1987 (mm)**	**ARR 2019 (mm)**	**ARR 1987 (mm)**	**ARR 2019 (mm)**
1 hr	84.83	75.86	64.44	55.16
2 hr	112.50	97.10	**85.56**	**71.28**
3 hr	131.97	112.02	100.44	82.65
6 hr	**173.04**	**152.15**	131.88	110.68
12 hr	227.28	208.01	173.40	151.59
24 hr	302.40	289.67	228.00	210.21
48 hr	393.12	387.23	292.8	283.88
72 hr	447.12	443.15	330.48	327.8

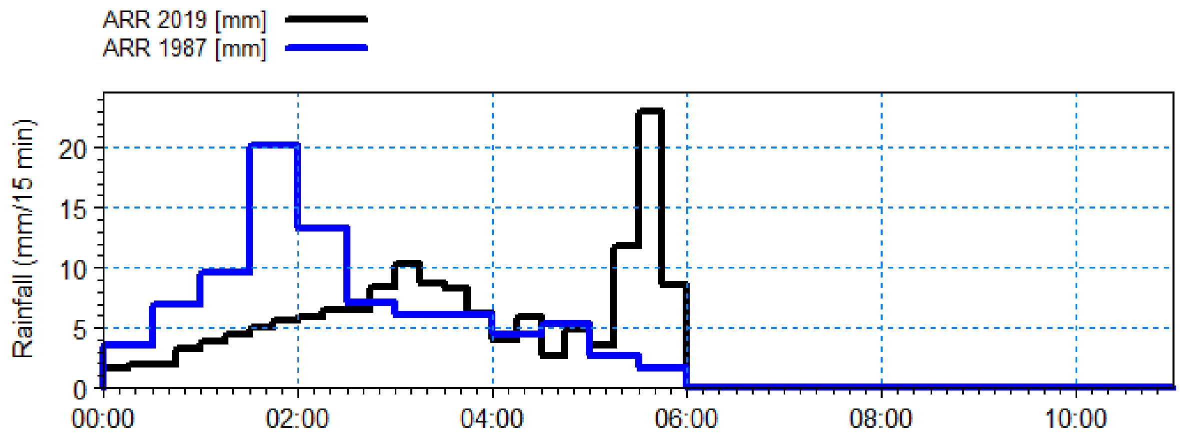

Figure 8.13 Comparison of 1% Design Rainfall between ARR 1987 (Duration 6hrs) in blue and ARR 2019 (Duration 6hrs, Temporal Pattern 9) in black (Simulation was run for additional 5 hours after the rain ceased)

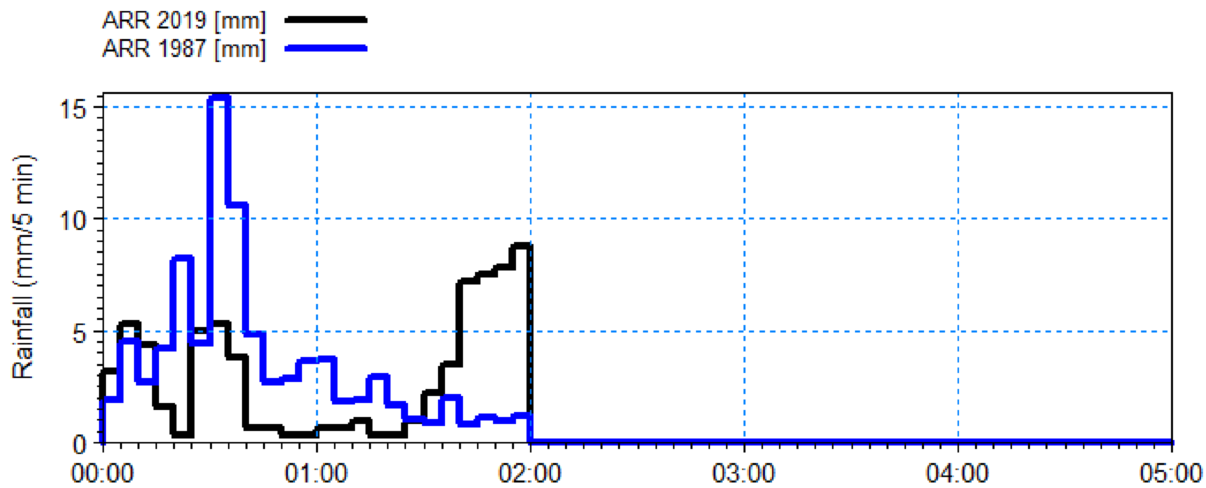

Figure 8.14 Comparison of 5% Design Rainfall between ARR 1987 (Duration 2hrs) in blue and ARR 2019 (Duration 2hrs, Temporal Pattern 10) in black (Simulation was run for additional 3 hours after the rain ceased)

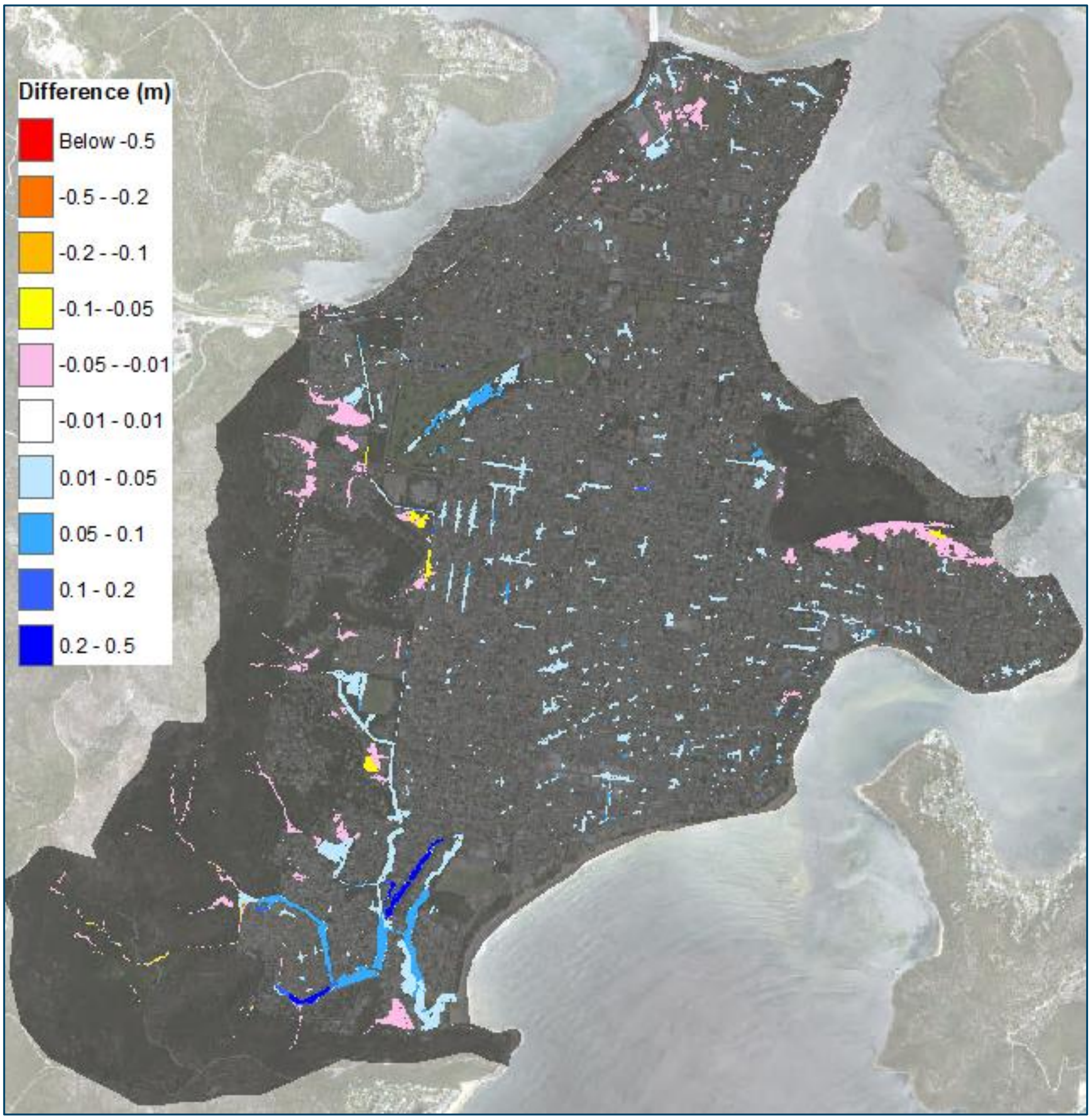

Figure 8.15 Difference in flood depth of 1% AEP 6hrs (ARR 1987 rainfall minus Baseline)

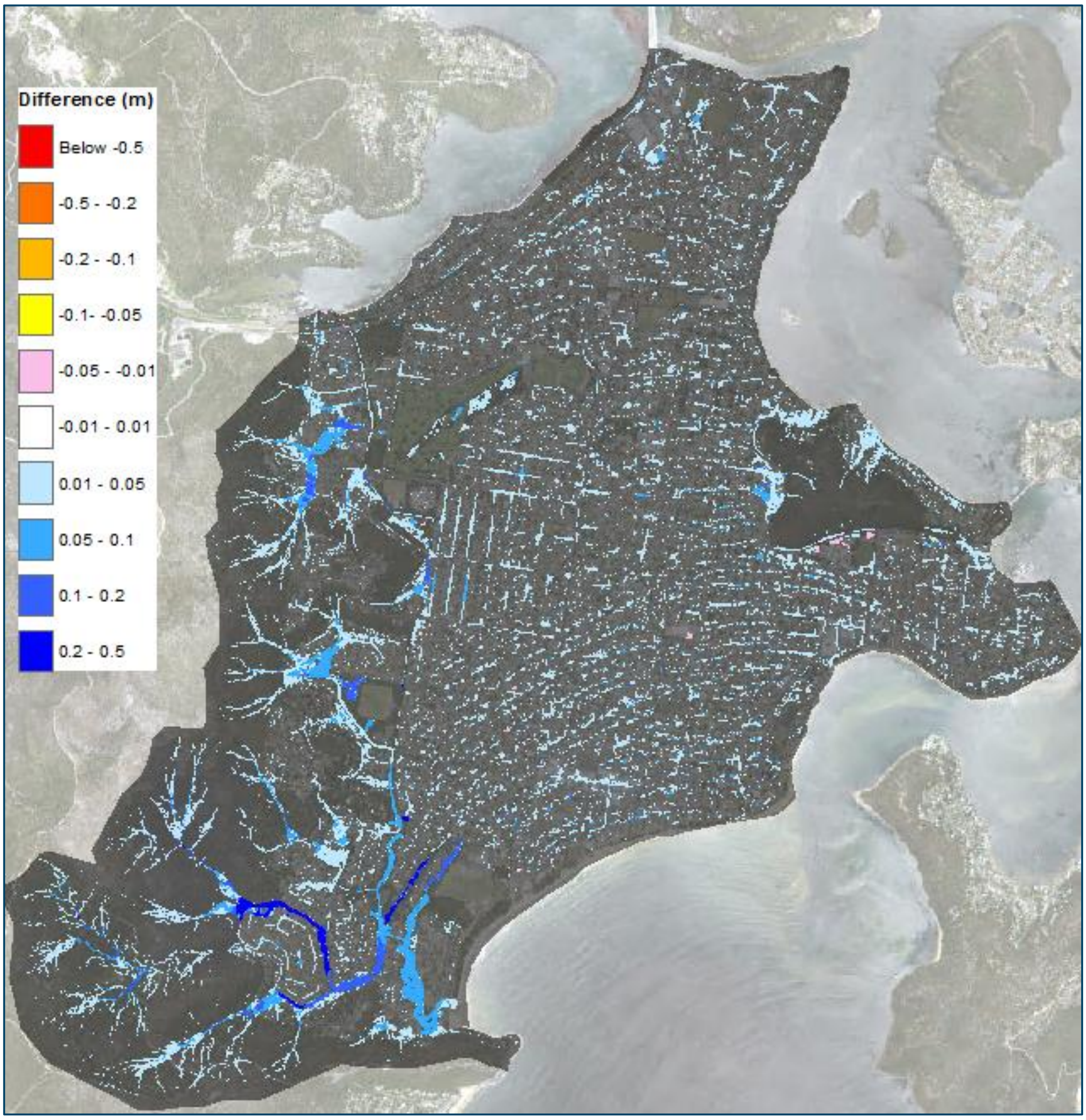

Figure 8.16 Difference in flood depth of 5% AEP 2hrs (ARR 1987 rainfall minus Baseline)

References

/1/ DHI (2010) Woy Woy Peninsula Flood Study

/2/ DHI (2021) Woy Woy Integrated Water Management and Case Study Everglades Catchment

/3/ Freeze R, Cherry J (1979) Groundwater. Prentice-Hall, Inc., Englewood Cliffs.

/4/ Office of Environment and Heritage (2015) Floodplain Risk Management Guide, Modelling the Interaction of Catchment Flooding and Oceanic Inundation in Coastal Waterways

/5/ GHD (2013) Palmtree Grove Basin – Umina Dam Break Study

/6/ Willing & Partners (1989) Ettymalong Creek Flood Study and Environmental Effects Assessment

/7/ Willing & Partners (1991) Kahibah Creek Flood Study

/8/ Willing & Partners (1991) Kahibah Creek Floodplain Management Study

/9/ Willing & Partners (1996) Kahibah Creek Floodplain Management Plan

/10/ Willing & Partners (2001) Kahibah Creek Review of Floodplain Management Measures

/11/ Australia Institute for Disaster Resilience (2017) Managing the Floodplain Handbook

/12/ New South Wales Government / Department of Infrastructure, Planning and Natural Resources (2005) Floodplain Development Manual

/13/ Cardno Lawson Treloar (2013) Brisbane Water Foreshore Flood Study

/14/ Cardno Lawson Treloar (2015) Brisbane Water Foreshore Floodplain Risk Management Plan

/15/ Cardno Lawson Treloar (2015) Brisbane Water Foreshore Floodplain Risk Management Study

/16/ Rhelm (2021) Woy Woy Climate Change Adaptation Study

/17/ Giammarco Engineering (1989) Woy Woy Peninsula Catchments Report Stormwater Investigations to Catchment Blackwall Mountain, Springwood Street, Waitangi Street, Warrigal Street, Wyalong Street, Memorial Avenue, Umina

/18/ Webb McKeown & Associates (1992) Woy Woy, Umina, Ettalong Peninsula, Drainage Strategy Study

/19/ Webb McKeown & Associates (1993) Rowan Catchment, Woy Woy Channel to Ocean Beach Road, Trunk Drainage Management Study and Management Plan

/20/ Webb McKeown & Associates (1996) Woy Woy Peninsula Catchments 'B' and 'C' Drainage Study

/21/ Patterson Britton & Partners (1997) Woy Woy Peninsula - Catchments 'P' and 'O' Drainage Investigation - Draft Report (Issue 1)

/22/ Kinhill Engineers (1999) Drainage Investigation Veron Road / Dulkara Road Catchment Umina / South Woy Woy

/23/ Ivan Tye and Associates (2000) Woy Woy Peninsula Catchments 'D' and 'E' Drainage Study

/24/ Kellogg Brown & Root (2005) Everglades Lagoon System Precinct, Plan of Management

/25/ Manly Hydraulics Laboratory (2017) Southern Central Coast Storm and Flood Forecasting Study