4. STUDY METHODOLOGY

The estimation of flood behaviour in a catchment is often conducted as a two-stage process, consisting of:

1. hydrologic modelling to convert rainfall estimates to overland flow and stream runoff; and
2. hydraulic modelling to estimate flow distributions, flood levels and velocities.

When historical flood data are available they can be used to allow calibration of the models, and increase confidence in the estimates. The calibration process is undertaken by altering model input parameters to improve the reproduction of observed catchment flooding. Recorded rainfall and stream-flow data are required for calibration of the hydrologic model, while historic records of flood levels, velocities and inundation extents can be used for the calibration of hydraulic model parameters.

Following model calibration the design rainfall is modelled. The approach adopted in flood studies to determine design flood levels largely depends upon the objectives of the study and the quantity and quality of the data (survey, flood, rainfall, flow etc.).

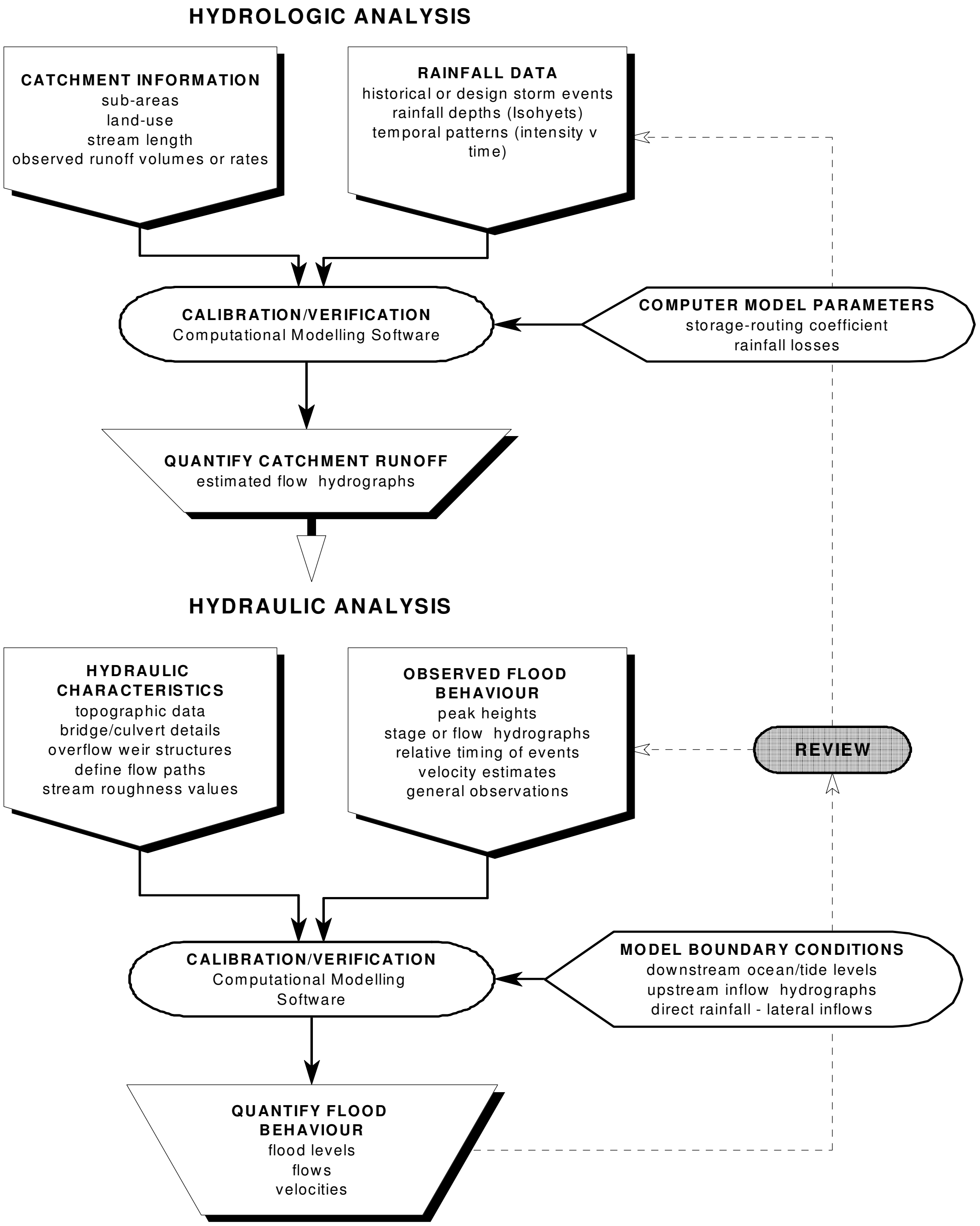
Flood estimation in urban catchments generally presents challenges for the integration of the hydrologic and hydraulic modelling approaches, which have been treated as two distinct tasks as part of traditional flood modelling methodologies. As the main output of a hydrologic model is the flow at the outlet of a catchment or sub-catchment, it is generally used to estimate inflows from catchment areas upstream of an area of interest. The hydrological model can also be useful to conceptually model hydrologic processes within the study area (such as runoff from roof and gutter systems, and On-site Stormwater Detention (OSD) systems). The aim of identifying the full extent of flood inundation can therefore be complicated by the separation of hydrologic and hydraulic processes into separate models, and these processes are increasingly being combined in a joint modelling approach.

The broad approach adopted for this study was to use a widely utilised and well-regarded hydrologic model to conceptually model the rainfall concentration phase, and for steep catchment areas upstream of the hydraulic model study area. The runoff hydrographs from the hydrologic model were then used in a hydraulic model to estimate flood depths, velocities and hazard in the study area. This joint modelling approach was calibrated against observed historical flood levels.

This approach reflects current engineering practice and is consistent with the quality and quantity of available data.

A diagrammatic representation of the Flood Study process is shown in Diagram 5.

Diagram 5: Flood Study Process



5. HYDROLOGIC MODEL DEVELOPMENT

5.1. Introduction

AR&R (1987) describes various techniques suitable for design flood estimation in rural and urban catchments. These techniques range from simple procedures to estimate peak flows (such as the Probabilistic Rational Method), to flood frequency analysis and more complex rainfall-runoff routing models that estimate complete flow hydrographs. Determination of which technique to employ is often based on the availability of data. For the present study, the rainfall and runoff routing approach was adopted. In current Australian engineering practice, examples of the more commonly used runoff routing models include RORB, RAFTS and WBNM. These models allow the rainfall depth to vary both spatially and temporally over the catchment, and have parameters governing runoff volume/shape that can be calibrated against recorded data.

For the present study, the Watershed Bounded Network Model (WBNM) was used. The WBNM model is an event-based, lumped-catchment conceptual model that is based on an extensive empirical dataset of rainfall-runoff relationships for Australian catchments. The model requires very few parameters to describe the physical aspects of the catchment, and is therefore less sensitive than other models to assumptions about catchment characteristics such as shape, steepness, and ground cover. WBNM was therefore considered a suitable tool for this study. WBNM has been widely adopted in Australia for use in similar studies.

5.2. Sub-catchment Delineation

The catchment boundary was determined by the ridges that create the natural drainage division. Precipitation falling on the other side of these boundaries would flow into other catchments and so was not modelled within these study areas.

The ridge bounding the Mummuga Lake catchment coincides with Brou Lake Road and Mitchells Ridge Road to the north and Tobacco Pinch Road, Box Cutting Road, Kianga Forest Road and Bell Ridge Road to the south. Part of the south boundary of this catchment forms the north boundary of the adjacent Kianga Lake catchment. Upon the ridge that bounds the Kianga Lake catchment is Bell Ridge Road and Kianga Forest Road to the north, Kianga Forest Road to the west and Appleby Road to the south. The ridge that defines the Wagonga Inlet catchment coincides with the south boundary of the Mummuga Lake and Kianga Lake catchments, along Applyby Road, Box Cutting Road and Tobacco Pinch Road. The ridge that defines the Wagonga Inlet catchment to the west coincides with Morts Folly Road. These boundaries are shown on Figure 1.

Within these catchments, smaller sub-catchment areas were delineated based on ALS survey and contours where ALS survey was not available. The sub-catchment layout ensures that where hydraulic controls exist that these are accounted for and able to be appropriately incorporated into hydraulic routing. The catchment layout for the hydrologic model is shown on Figure 6.

5.3. Model Parameters

The WBNM hydrologic runoff-routing model was used to determine hydraulic model inflows, both from catchment areas upstream of the hydraulic model extent, and for the local sub-catchments within the hydraulic model domain of the study.

The model input parameters for each sub-catchment are:

- a lag factor (termed C), which can be used to accelerate or delay the runoff response to rainfall;
- a stream-flow routing factor, which can speed up or slow down concentrated flows occurring through each catchment;
- rainfall initial and continuing losses to represent infiltration and filling of depression storage; and
- the percentage of catchment area with a pervious/impervious surface.

5.3.1. Lag Parameter

Lag times for runoff depend on several physical catchment characteristics, including area, shape and steepness (among others) for natural catchments. Experimental data for natural catchments in Australia has demonstrated that the dominant factor affecting lag is catchment area, with other characteristics showing strong correlation with area such that there is a strong case for catchment lag to be determined on area alone.

Experimental derivation of the Lag Parameter for 129 storms on 10 catchments in eastern NSW found that a value of 1.68 gave a good fit to all the data. A value of 1.7 was adopted for historical and design flood modelling in this study, in agreement with the NSW data.

5.3.2. Stream-flow Routing Parameter

WBNM provides the option to route upstream flows to the bottom of a sub-catchment via nonlinear routing, time-delay routing and Muskingum routing. This routing is required to estimate the attenuation and timing of flows from sub-catchments in the steep upper catchment areas that are not included in the hydraulic model extent. The nonlinear method was adopted for this study. For this method, Boyd et. al. (2007) recommends values of 1.0 for natural channels and 0.67 for gravel beds. Therefore, for this study, a value of 1.0 was adopted.

Where the hydrologic sub-catchment area coincided with the hydraulic sub-catchment area, these were applied as local inflows with no routing of upstream flows.

5.3.3. Rainfall Losses

Methods for modelling the proportion of rainfall that is “lost” to infiltration are outlined in AR&R (1987). The methods are of varying complexity, with the more complex options only suitable if sufficient data are available (such as detailed soil properties). The method most typically used for design flood estimation is to apply an initial and continuing loss to the rainfall. The initial loss represents the wetting of the catchment prior to runoff starting to occur, and the continuing loss represents the ongoing infiltration of water into the saturated soils while rainfall continues.

Initial and continuing losses are often used as the primary parameters for calibrating hydrologic models when observational data are available. For this study, typical values were adopted based on available data in similar catchments. Table 6.2 of AR&R (1987) recommends that for catchments east of the dividing range in New South Wales, an initial loss of 10 mm to 35 mm is appropriate, with a continuing loss of 2.5 mm/hr.

For this study, the initial loss of 10 mm was adopted, which is at the lower end of values recommended in AR&R.

5.3.4. Impervious Areas

Runoff from connected impervious surfaces such as roads, gutters, roofs or concrete surfaces occur significantly faster than from vegetated surfaces. This results in a faster concentration of flow within the downstream area of the catchment, and increased peak flow in some situations. It is therefore necessary to estimate the proportion of the catchment area that is covered by such surfaces.

The impervious surfaces within the study areas were determined through digitisation of the road surfaces (used in the hydraulic model to specify Manning’s ‘n’ roughness coefficients, see Section 6.4) and building footprints (used in the hydraulic model to simulate impermeable obstructions to the flood flow, see Section 6.3) through visual inspection of aerial photography. The proportion of these impervious surfaces within the sub-catchment area was adopted as the impervious percentage of the sub-catchment area.

5.3.5. Summary of Model Parameters

The key modelling parameters adopted for the historic and design hydrologic modelling are summarised as follows:

- Lag Parameter (C) – 1.7
- Pervious Area Initial Rainfall Loss – 10 mm
- Pervious Area Continuing Rainfall Loss – 2.5 mm/hour
- Impervious Area Initial Rainfall Loss – 1 mm
- Impervious Area Continuing Rainfall Loss – 0 mm/hour

6. HYDRAULIC MODEL DEVELOPMENT

6.1. Introduction

The availability of high quality ALS data means that the study area is suitable for two-dimensional (2D) hydraulic modelling of major flowpaths and lake areas. Various 2D software packages are available, such as SOBEK, TUFLOW and Mike FLOOD, among others. The TUFLOW package was adopted for this study as it is widely used in Australia and WMAwater have extensive experience in the use of the TUFLOW model.

The TUFLOW modelling software is produced by BMT WBM. The modelling package includes a finite difference numerical model for the solution of the depth averaged shallow water flow equations in two dimensions. The 2D model is capable of dynamically simulating complex overland flow regimes and interactions with sub-surface drainage systems.

For the hydraulic analysis of complex overland flow paths an integrated 1D/2D model such as TUFLOW provides several key advantages when compared to a 1D only model. For example, a 2D approach can:

- provide localised detail of any topographic and /or structural features that may influence flood behaviour,
- better facilitate the identification of the potential overland flow paths and flood problem areas,
- dynamically model the interaction between hydraulic structures such as culverts and complex overland flowpaths, and
- inherently represent the available flood storage within the 2D model geometry.

Importantly, a 2D hydraulic model can better define the spatial variations in flood behaviour across the study area. Information such as flow velocity, flood levels and hydraulic hazard can be readily mapped across the model extent. This information can then be easily integrated into a GIS based environment enabling the outcomes to be readily incorporated into Council’s planning activities. The model developed for the present study provides a flexible modelling platform to properly assess the impacts of any management strategies within the floodplain (as part of the ongoing floodplain management process).

In TUFLOW the ground topography is represented as a uniformly-spaced grid with a ground elevation and a Manning’s “n” roughness value assigned to each grid cell. The grid cell size is determined as a balance between the model definition required and the computer run time (which is largely determined by the total number of grid cells).

6.2. Model Extent

The hydraulic model extent for the Kianga and Duck Pond catchments coincided with the catchment boundaries in the hydrologic model extent. The upper reaches to the west of the Mummuga Lake and Wagonga Inlet catchment (along Cowdroy Creek, Billa Bilba Creek, Burrimbidgee Creek and Punkally Creek) were solely modelled within the hydrologic model, which provided inflows at the hydraulic model boundary for the catchment. The hydraulic model extents are shown on Figure 5.

6.3. Digital Elevation Model

The model uses a regularly spaced computational grid. The Wagonga Inlet catchment was simulated in two distinct hydraulic models. The Inlet model had a grid cell size of 12 m by 12 m and the Narooma Flat model had a grid cell size of 3 m by 3 m. The Mummuga Lake catchment was simulated in two hydraulic models, with a grid cell size of 6 m by 6 m in the Lake model and a grid cell size of 3 m by 3 m in the Dalmeny Township model. The Kianga Lake catchment had a grid cell size of 6 m by 6 m and the Duck Pond catchment had a grid cell size of 3 m by 3 m. This resolution was adopted as it provides an appropriate balance between providing sufficient detail for roads and overland flow paths, while still resulting in workable computational run-times.

The model grid was established by sampling from a 1 m by 1 m DEM. This DEM was generated from a triangulation of filtered ground points from the 2012 LiDAR dataset and 2002 bathymetric survey, discussed in Section 2.1 and shown on Figure 3. Locations where the topography differed from this were in the vicinity of the ICOLL sand berms, discussed in Section 7.1.2.

Permanent buildings and other significant structures likely to act as significant flow obstructions were incorporated into the terrain model. These features were identified from the available aerial photography and modelled as impermeable obstructions to the flood flow.

6.4. Roughness Coefficient

The TUFLOW model used for this study utilises the Manning's formulation to determine the energy loss from friction and other sources. The roughness coefficient, '*n*', is an empirically derived parameter which represents the retarding force applied to flowing water by the channel bed or ground surface. In practice, in computational modelling of real systems, this parameter often also incorporates other sources of energy loss such as turbulence and flow expansion/contraction from non-uniform cross sections.

The value of ‘*n*’ represents the resistance to flow in a given channel which depends on a number of factors such as:

- surface roughness;
- vegetation;
- channel irregularity and alignment;
- obstructions;
- silting and scouring;
- the size and shape of the channel; and
- the stage and discharge.

Inspection of the aerial photography was used to classify various land-uses categories, such as urban areas and vegetated areas. From this, spatially varying roughness values were applied to the model, based upon these differing categories. The roughness values adopted for the hydraulic model are shown in Table 18 and Figure 8.

The values are consistent with typical values in the literature (Chow, 1959 and Henderson, 1966), industry guidelines (*AR&R Revision Project 15: Two Dimensional Modelling in Urban and Rural Floodplains Report*, Engineers Australia, 2012) and previous experience with modelling similar catchment conditions. The sensitivity of model results to changes in the roughness values is discussed in Section 14.

Table 18: Manning's 'n' Values

Surface Type	**Manning’s ‘n’ Value**
Concrete-lined pipes	0.015
Roads and paved surfaces	0.025
Urban areas – general overland areas, gardens, roadside verges, low density residential lots etc	0.05
Light density vegetation (very short grass or sparse vegetation)	0.04
Medium density vegetation	0.07
Heavy density vegetation	0.10
Waterways, such as Lakes, Estuaries and Ocean areas	0.03
Default	0.05

6.5. Hydraulic Structures

The behaviour of hydraulic structures like culverts, fences, channels and bridges can have a significant influence on flood behaviour. When culverts are flowing near capacity or become blocked, backwater upstream of the culvert can flood properties or cause the road to be overtopped. The piers and deck of bridges over creeks can present an obstruction to flow, resulting in afflux (increased water level) upstream of the structure. It is therefore important to pay particular attention to the modelling of these features.

Key hydraulic structures were included in the hydraulic model, as shown on Figure 9. Culverts were generally modelled as 1D features embedded in the 2D model, since the majority of the culverts of interest have dimensions smaller than the grid resolution. For the bridges, where the main flow width exceeds the grid resolution, modelling was undertaken in the 2D domain using a TUFLOW software feature specifically designed for this purpose, whereby the energy losses and blockage caused by the piers, deck and above deck structure can be applied directly to the grid cells.

The modelling parameter values for the culverts and bridges were based on the geometrical properties of the structures, which were obtained from records of structures held by the authorities responsible for them, photographs taken during site inspections, and previous experience modelling similar structures. The Roads and Maritime Services provided data on the dimensions of structures underneath the Princes Highway. This included the bridge over Wagonga Inlet, the bridge over Kianga Creek and the bridge over Lawler's Creek (within the Mummuga Lake catchment). Eurobodalla Shire Council provided data on the dimensions of other structures within their jurisdiction. Sensitivity analysis of the effect of the hydraulic structure parameters is presented in Section 14.

Smaller localised obstructions within private property, such as fences, were not explicitly represented within the hydraulic model, due to the difficulty of identifying and characterising these structures from aerial photographs, and the relative impermanence of these features. The cumulative effects of fences on flow behaviour were assumed to be partially addressed via the roughness adopted for residential areas.

6.6. Blockage Assumptions

Blockage of hydraulic structures can occur with the transportation of a number of materials by flood waters. This includes vegetation, garbage bins, building materials and cars, the latter of which has been seen post-flood in Newcastle. However, the disparity in materials that may be mobilised within a catchment can vary greatly.

Debris availability and mobility can be influenced by factors such as channel shear stress, height of floodwaters, severity of winds, storm duration and seasonal factors relating to vegetation. The channel shear stress and height of floodwaters that influence the initial dislodgment of blockage materials are also related to the average exceedance probability (AEP) of the event. Storm duration is another influencing factor, with the mobilisation of blockage materials generally increasing with increasing storm duration (Barthelmess and Rigby 2009, cited in Engineers Australia 2013).

The potential effects of blockage include:

- decreased conveyance of flood waters through the blocked hydraulic structure or drainage system;
- variation in peak flood levels;
- variation in flood extent due to flows diverting into adjoining flow paths; and
- overtopping of hydraulic structures.

Existing practices and guidance on the application of blockage can be found in:

- the Queensland Urban Drainage Manual (Department of Natural Resources and Water, 2008);
- AR&R Revision Project 11 Blockage of Hydraulic Structures (Engineers Australia, 2013); and
- the policies of various local authorities and infrastructure agencies.

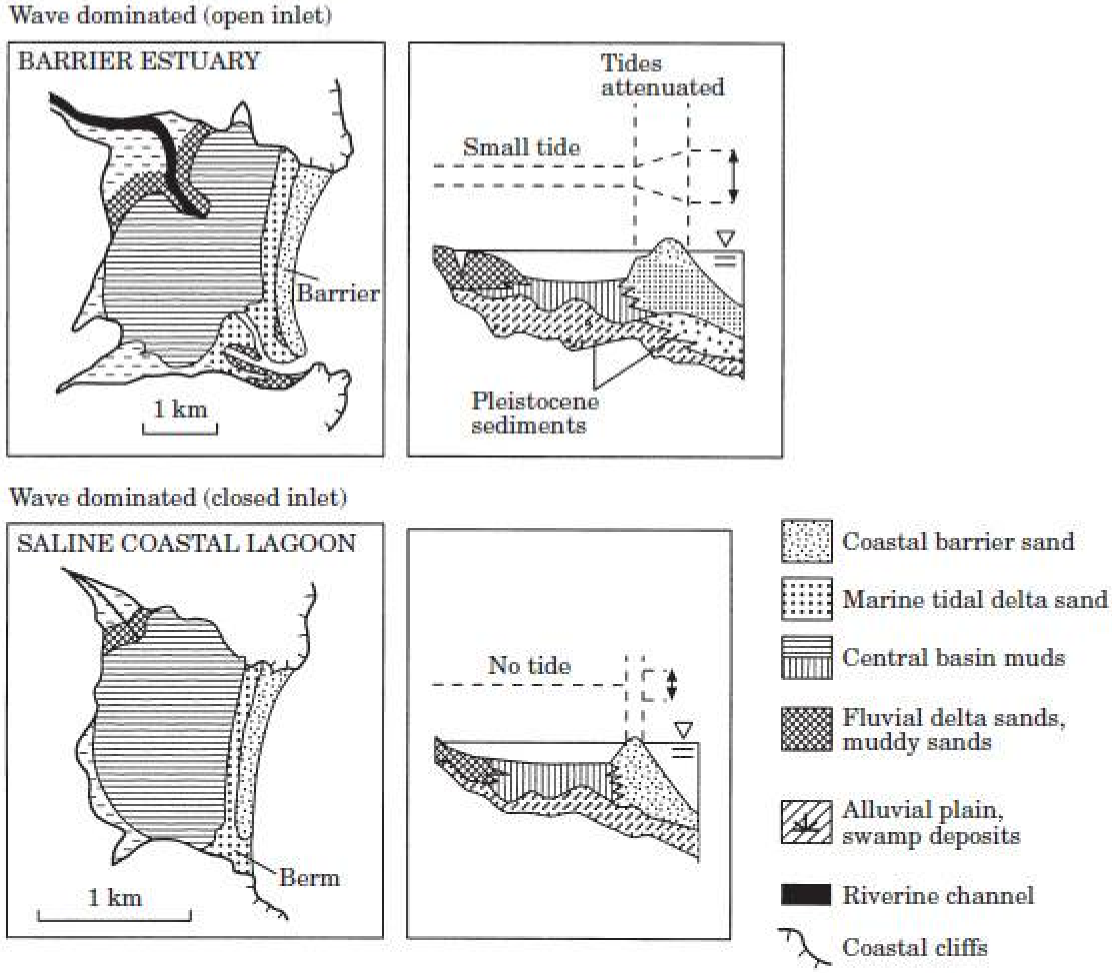
Current modelling has been undertaken assuming no blockage of pipes, culverts and bridges greater than 450 mm in diameter. Pipes less than 450 mm in diameter were conservatively assumed to be completely blocked. The sensitivity of peak flood levels to blockage will be considered in design sensitivity.

7. ENTRANCE CONDITIONS

7.1. Introduction

The entrance conditions of the catchments are separated by type into two categories; barrier estuary and ICOLL's (shown in Diagram 6). The Wagonga Inlet catchment is classified as a barrier estuary. The Dalmeny (consisting of both the Mummuga Lake and Duck Pond) catchments and the Kianga Lake catchments are categorised as ICOLL's.

Diagram 6: Estuary Types Extracted from Roy et. al. (2001)



General information regarding the entrance conditions, the use of software features in the hydraulic model to broadly represent these types of entrance conditions, and the catchment specific application, is provided below.

7.1.1. Barrier Estuary

Barrier estuaries are open inlets with a constricted entrance. The river discharges from barrier estuaries "tend to counteract the flux of wave-transported beach sand in the estuary mouths" (Roy et. al., 2001).

The tides between the ocean outlet and the estuary basin are subject to attenuation in barrier estuaries. This attenuation results in a spatially varying water level within the Wagonga Inlet basin, as characterised by the difference in water level hydrographs recorded at Narooma Public Wharf (located in the entrance channel) and Barlows Bay (located upstream of Narooma Public Wharf within the estuary basin) in Figure E 2. Also, given that the Barlows Bay station is a greater distance from the ocean outlet than the Narooma Public Wharf station, it also shows that the low tide does not propagate as great a distance into the Inlet as the high tide does.

7.1.2. ICOLL's

The modelled conditions of ICOLL entrances can considerably affect the flooding behaviour in the lower reaches of the catchments, in both the historic flood modelling and design flood modelling. Closed entrance conditions would likely produce higher flood levels during events with rainfall-derived inflows and lower flood levels during events with ocean-derived inflows. Open entrances conditions would likely produce the converse of this.

There are a number of approaches to simulating the entrance conditions of ICOLL's in the hydraulic model, including:

- A constant closed entrance: The sand berm would be elevated. When the level required to initiate an artificial opening of the ICOLL is reached, flow would occur over the sand berm, with no change in sand berm topography;
- A constant open entrance: This approach considers the entrance to be open prior to the event occurring, with no change in entrance topography during the simulation;
- A variable entrance: The sand berm would be elevated prior to the commencement of the event. During the simulation, the sand berm dimensions would gradually lower until the open entrance topography was achieved based upon specified initiation criteria. After the entrance reached open conditions, no further topographic changes would occur.

The constant closed or constant open entrance condition can be simulated in the 2D domain of the TUFLOW hydraulic model using the DEM. The variable entrance condition requires the schematisation of a variable shape element to be incorporated into the hydraulic model using a 2D feature provided in the TUFLOW software package. This feature is a simplified representation of the more complex processes that occur.

In the variable entrance scenario, the initial closed entrance dimensions are simulated in the DEM. The variable shape specifies the dimensions of the final open entrance conditions. Between the initial dimensions and final dimensions, the TUFLOW software adjusts the dimensions incrementally based upon the duration of change specified. The software provides various options for the initialisation of the varying topography including: at a specified time, when the water level reaches a stipulated height at a specified (trigger) location, or the water level difference between two specified locations exceeds a stipulated amount.

The approach adopted in this Flood Study for the simulation of the ICOLL entrance conditions, varies according to the different scenarios, such as the historic flood modelling, the design flood modelling and the sensitivity flood modelling. For each of the ICOLL catchments, the topographic dimensions (closed entrance and open entrance conditions) used in the three approaches discussed above is specified in the following.

7.2. Wagonga Inlet

In the case of Wagonga Inlet, the open entrance is characterised by twin training walls within the estuary channel and two breakwaters at the ocean outlet. The breakwaters are located at Wagonga Head. The twin training walls extend from Narooma Public Wharves on Bluewater Drive to the Princes Highway Bridge. The Wagonga Inlet is a wave dominated estuary (Roy et. al., 2001). The tidal prism of the Wagonga Inlet is provided in Table 19.

Table 19: 380m from the entrance on 3 December 1986 (OEH, 2012)

Tide State	Wagonga Inlet Flow (10^6 m^3)	Wagonga Inlet Tidal Range (m)	Sydney Harbour Tidal Range (m)
Ebb Flow	6.34	1.49	1.89
Flood Flow	6.64	1.24	1.57

The two breakwaters at Wagonga Heads were defined in the hydraulic model's DEM. The height and width of the structures enabled this schematisation, as it was of a sufficient width as to be greater than the computational grid cell size used in the 2D domain (discussed in Section 6.3).

The parallel training walls were of a width less than the computational grid cell size, and so the structure was not consistently identified within the DEM. The training walls were schematised in the hydraulic model using a 2D feature provided in the TUFLOW software package. The variables that may be specified in the feature include average height of the structure and percentage impervious through the structure. The unprocessed ALS data contained greater resolution than the DEM and was used to define the average height of the training walls. The percentage impervious parameter was used to represent the ratio of the area that was impervious (due to the rocks that the training walls are constructed from) and pervious (due to the gaps between the rocks facilitating marginal flow from one side of the training wall to the other). It was assumed that 10% of the lateral area may be considered pervious.

To simulate the varying water levels and flow velocities within Wagonga Inlet that is probable to have occurred just prior to the various events, the hydraulic model simulation was started 3 hours prior to the commencement of the applied rainfall. During this part of the simulation the initial water level applied to the inlet is singularly the subject of the ocean tide. As such, the inlet condition at the commencement of the rainfall is the product of the relationship between the varying inlet water level and the ocean tide level. The values applied as the initial water levels within the inlet in the hydraulic model were dependent upon the event being modelled. This is discussed in Section 8 for the calibration events, Section 10 for the design events and Section 14 for the sensitivity analysis events.

7.3. Kianga

The location of the sand berm at the Kianga Lake entrance is shown in the 2010 aerial images (provided in Diagram 7) and the LiDAR survey that was obtained in 2005 and 2012 (provided in Diagram 8 and Diagram 9).

Both of the LiDAR sets defined the sand berm as having an elevation of approximately 2 m AHD. This elevation corresponds with that required to initiate an artificial entrance opening. The DEM produced from the 2012 LiDAR survey was adopted as the closed entrance topography.

However, no survey was available to define the open entrance topography. As such, the open entrance topography was assumed to be an interpolation of elevations from downstream of the sand berm to the channel inverts located upstream of the sand berm. These elevations were obtained from the bathymetric survey undertaken in 2002.

Diagram 7: Kianga Lake ICOLL Entrance – 2010 Aerial Images

Diagram 8: Kianga Lake ICOLL Entrance – DEM of 2005 LiDAR and 2002 bathymetric survey

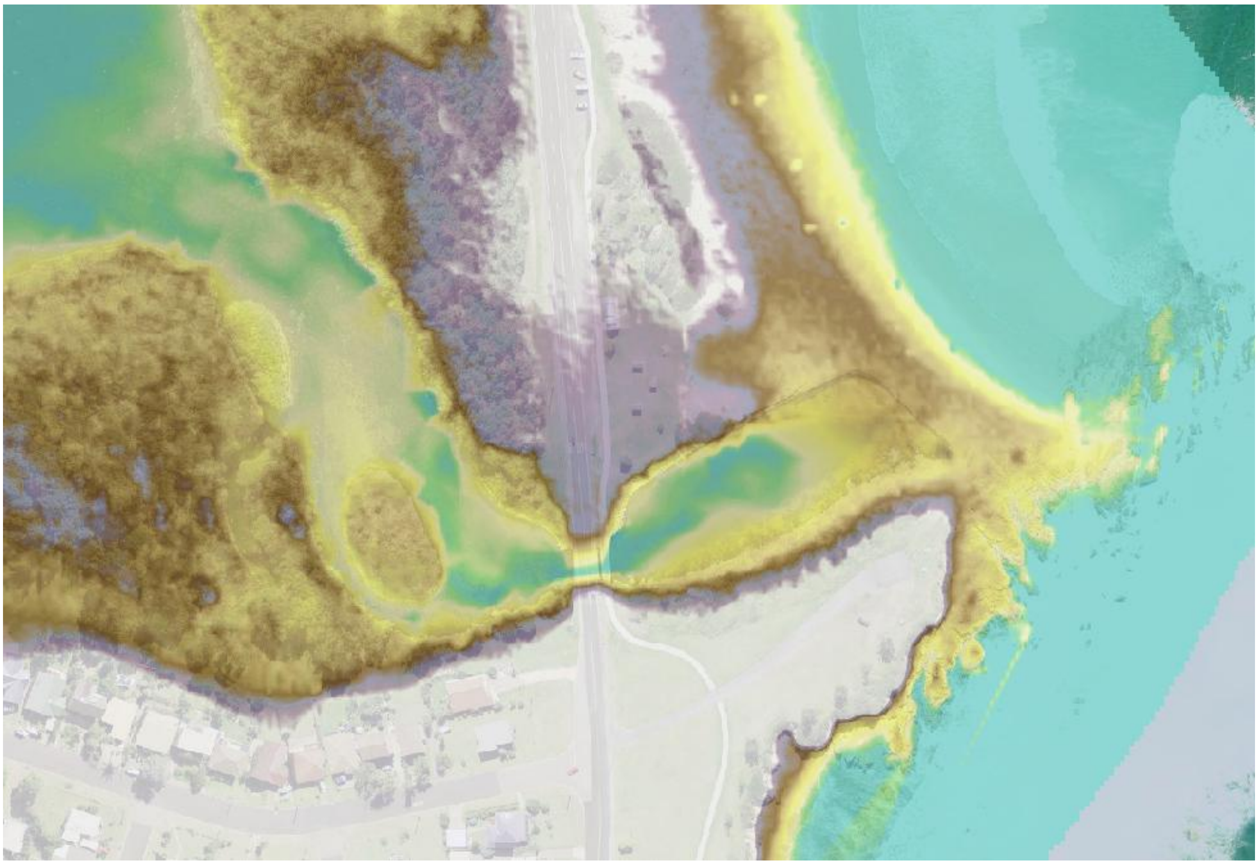
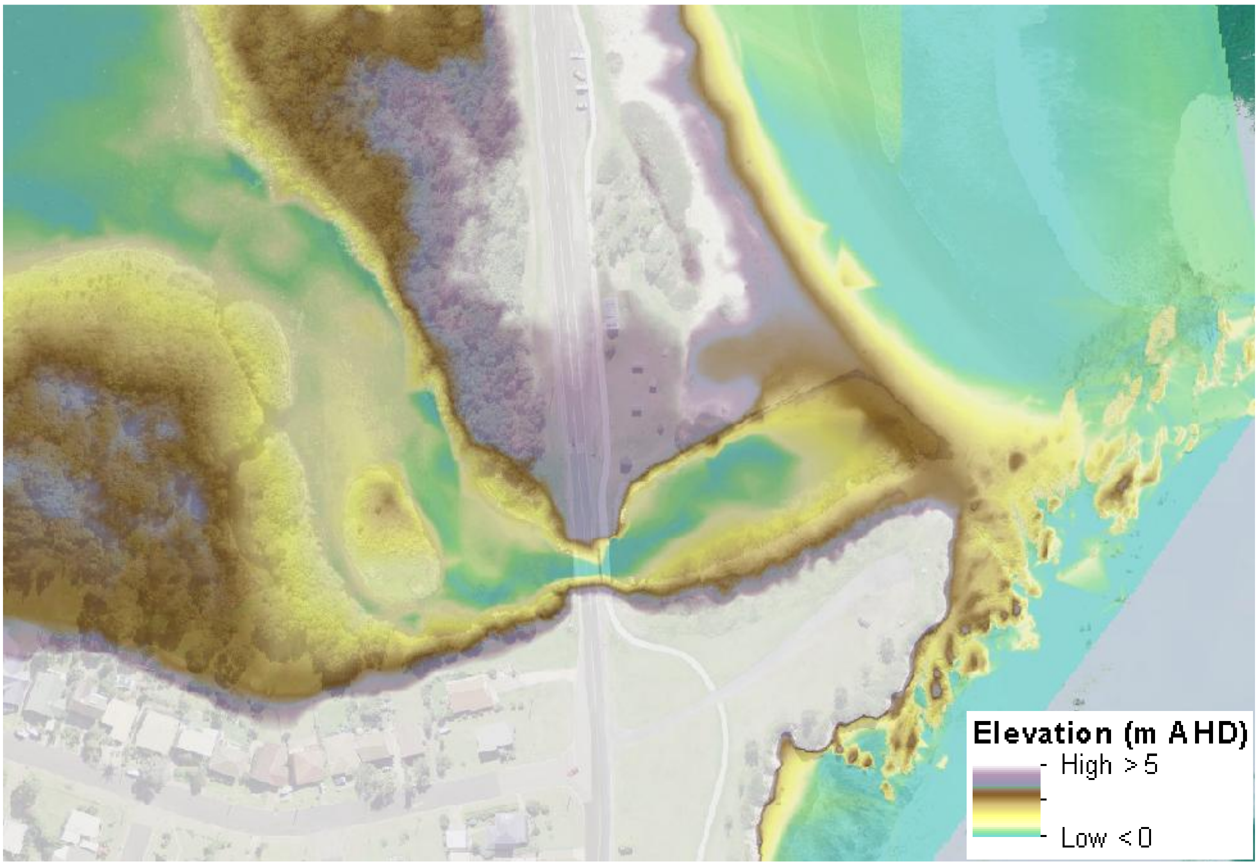


Diagram 9: Kianga Lake ICOLL Entrance – DEM of 2012 LiDAR and 2002 bathymetric survey



7.4. Dalmeny

The location of the sand berm at the Mummuga Lake entrance is shown in the LiDAR survey that was obtained in 2005 (provided in Diagram 11). At the time of the survey, the sand berm was found to have an elevation of approximately 1.4 m AHD at its peak. This is higher in elevation than the water level height required at the footbridge to initiate an artificial entrance breakout (specified as 1.175 m AHD). As such, the closed entrance topography was defined by the 2005 LiDAR survey.

The open entrance conditions at the Mummuga Lake entrance is shown in the 2010 aerial images (provided in Diagram 10) and the LiDAR survey that was obtained in 2012 (provided in Diagram 12). The combination of the 2012 LiDAR survey and the 2013 bathymetric survey was adopted as the open entrance topography.

Diagram 10: Mummuga Lake ICOLL Entrance – 2010 Aerial Images

Diagram 11: Mummuga Lake ICOLL Entrance – DEM of 2005 LiDAR

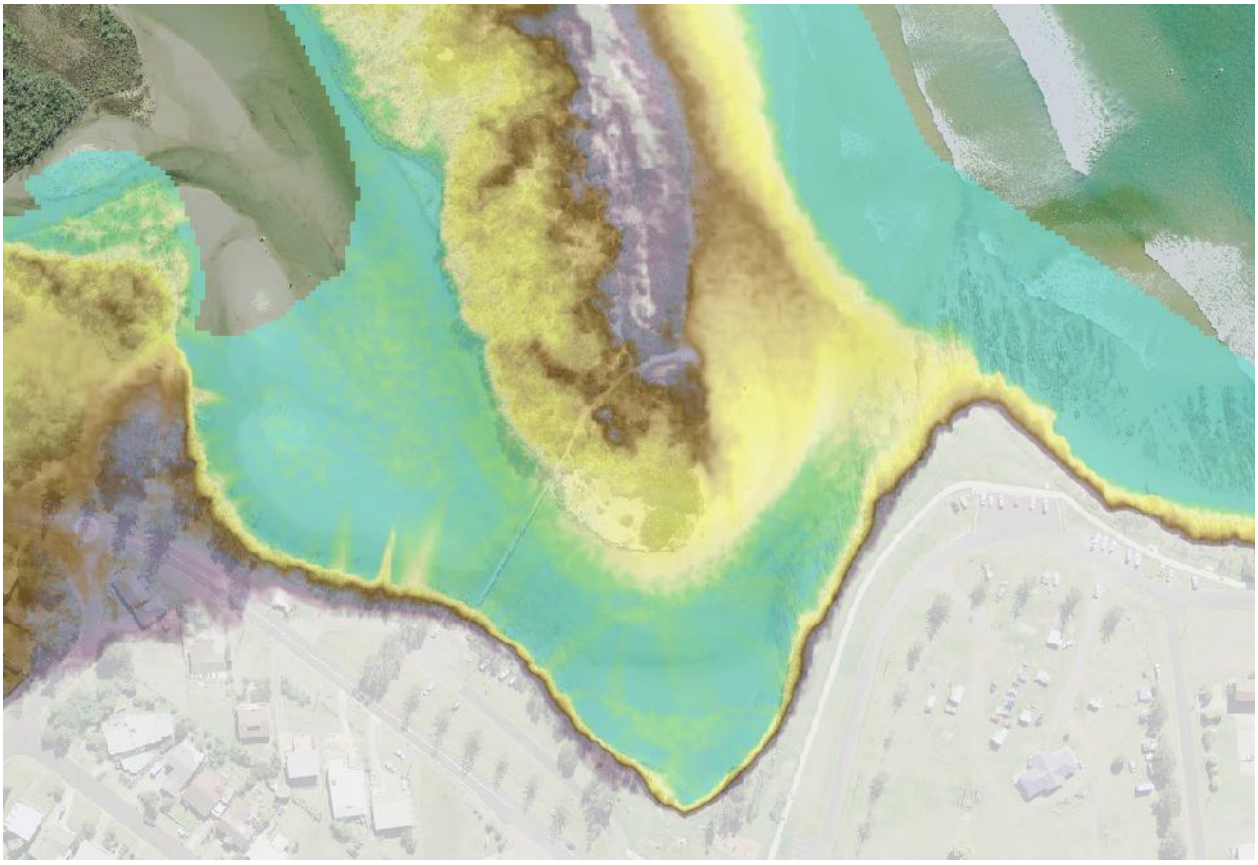
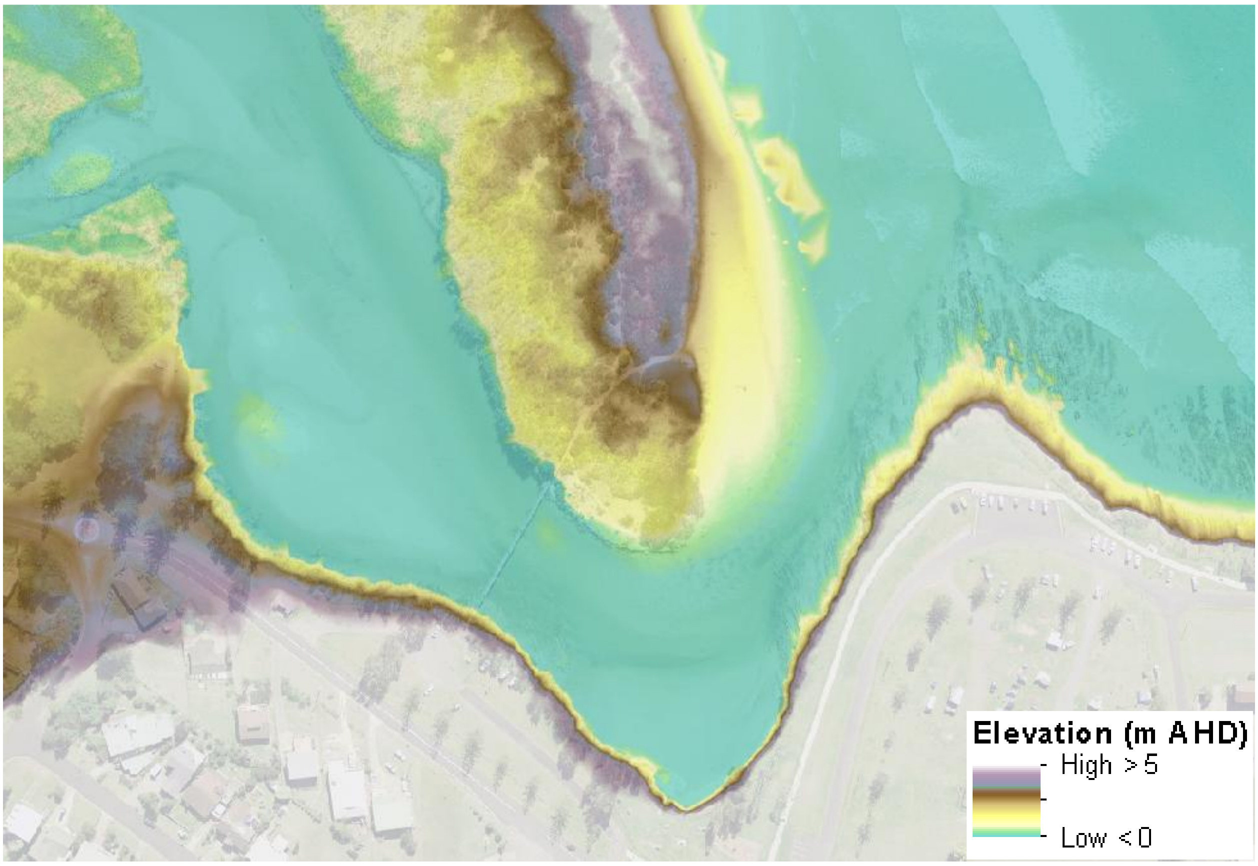


Diagram 12: Mummuga Lake ICOLL Entrance – DEM of 2012 LiDAR and 2013 bathymetric survey



The Duck Pond entrance is not identified on the OEH online estuary summary that details physical characteristics (such as ICOLL status) due to its relatively small size, however the features of the entrance indicate that it is an ICOLL. The location of the sand berm at the Duck Pond entrance is shown in the 2010 aerial images (provided in Diagram 13) and the LiDAR survey that was obtained in 2005 (shown in Diagram 14). However, as very little data is available concerning this ICOLL (including entrance breakout criteria or conditions), no variation in topography was adopted for this ICOLL to represent variations in open or closed entrance conditions.

Diagram 13: Duck Pond ICOLL Entrance – 2010 Aerial Images

Diagram 14: Duck Pond ICOLL Entrance – DEM of 2005 LiDAR