6. HYDRAULIC MODEL DEVELOPMENT

6.1. Introduction

The availability of high quality LiDAR data and flow behaviour present means that the study area is suitable for two-dimensional (2D) hydraulic modelling of major flowpaths. 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 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 Tomaga River and Candlagan Creek catchments are largely rural and development is concentrated around four areas, namely Tomakin, Mossy Point, Broulee and Mogo. Typically, developed areas require a grid resolution of no more than 3 metres to capture the various flow mechanisms characteristic of a built-up environment. However, such a grid resolution over the 120 km^2 covered by the Tomaga River and Candlagan Creek would result in excessive runtime and while splitting the two catchments into separate models would reduce run-times, the proximity of the respective river mouths makes this difficult as flood levels from both catchment can be inter-dependent. A more elegant solution that is particularly suited to the study area is to take a nested approach to the hydraulic modelling. The upper parts of the catchment which are outside the study area are routed in the hydrologic model and applied as boundary inflows to the hydraulic model domain. The overall hydraulic model extent is shown on Figure 9 and has a 10 m x 10 m grid resolution which is refined to a resolution of 2.5 m x 2.5 m for the areas where development is concentrated.

6.3. Digital Elevation Model

The model grid was established by sampling from a 2 m x 2 m DEM. This DEM was generated from a triangulation of filtered ground points from the 2012 LiDAR dataset discussed in Section 2.1.1 and bathymetric survey discussed Section 2.1.2 was used to cut out the Tomaga River channel geometry. Figure 2 presents the two respective datasets as well as the cross-sectional information derived from bridge construction drawings implemented in the cutting out Candlagan Creek.

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 the 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 12 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 9.4.

Table 12: Manning's 'n' Values

Surface Type	Manning's 'n' Value
Concrete-lined pipes	0.015
Roads and paved surfaces	0.02
Waterways – Rivers, Lakes, Estuaries and Ocean	0.03
Dirt areas	0.03
Light density vegetation (very short grass or sparse vegetation)	0.04
Medium density vegetation	0.07
Heavy density vegetation	0.12
Swamp areas	0.06
Default	0.08

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 4. 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 (Figure 3), and previous experience modelling similar structures. The Roads and Maritime Services provided data on the dimensions of structures underneath the Princes Highway. Eurobodalla Shire Council provided data on the location of drainage structures within their jurisdiction and where details of dimensions were not available, larger asset dimensions were estimated during site inspection and less critical assets were assumed blocked. Sensitivity analysis of the effect of the hydraulic structure parameters is presented in Section 9.4.

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 effect of fences on flow behaviour is implicitly contained within the roughness parameter discussed in Section 6.4.

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, cars, and even houses in extreme cases as witnessed during the recent flooding of Dungog in April 2015. 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 25 percent blockage of pipes and culverts greater than or equal to 450 mm in diameter. Pipes less than 450 mm in diameter are conservatively assumed to be completely blocked. The sensitivity of model results to changes in the blockage assumptions is discussed in Section 9.4.

It is worth noting that for large structures upstream of Tomakin, Mossy Point, Broulee and Mogo, assuming blockage while increasing peak levels upstream of the structure, may potentially decrease peak flood levels in the areas of interest for this study. Therefore, assuming blockage is not always inherently conservative. As a result no blockage was assumed for large bridges.

7. HISTORIC FLOOD MODELLING

7.1. Introduction

Modelling of known historic flood events is carried out to calibrate and validate the hydrologic and hydraulic models. This process is important to ensure that the models are sufficiently representing flood behaviour within acceptable limits. Calibration involves modifying (within an acceptable range) the model parameter values to replicate observed flood behaviour or levels. Validation is undertaken to ensure that the model parameter values determined in the calibration phase are acceptable in other flood events with no need for additional alteration of values.

The model parameters that are typically adjusted include (as detailed within the *ARR Revision Project 15: Two Dimensional Modelling in Urban and Rural Floodplains Report*, 2012):

- Hydraulic roughness parameters;
- Energy losses at structures/bends;
- Inflow hydrographs (parameters involved include temporal rainfall patterns and spatial rainfall distribution);
- Downstream boundary location and assumptions, particularly stage-discharge boundaries; and
- Blockage of inlets and hydraulic structures.

Selection of calibration and validation events is based upon data availability and magnitude of the storm or flood event. Ideally, the rainfall calibration events span a range of magnitudes with a preference for the more significant events, such as those near the 1% AEP event.

It is ideal to have historical rainfall (daily and pluviographic) and historical streamflow (daily and instantaneous) data to calibrate the hydrologic model, independent of the hydraulic model. As streamflow data is not available within the study areas, the hydrologic model has been calibrated in tandem with the hydraulic model in this flood study. This is in accordance with guidelines produced by Engineers Australia (within the *AR&R Revision Project 15: Two Dimensional Modelling in Urban and Rural Floodplains Report*, 2012) that recommends that the two models be jointly calibrated.

To calibrate the hydrologic and hydraulic models it is necessary to have data on historical rainfall, historical boundary conditions and historical flood records or observations.

The historic rainfall conditions can be determined from daily and pluviometer gauging stations. The pluviometer data provides information on the temporal pattern of the rainfall (as in, the variation in the rainfall amount across a period of time). The combination of the daily and pluviometer data provides information on the possible spatial distribution of the rainfall (as in, the variation in the rainfall depth across the catchment area). Generally, historic boundary conditions may be a stage-discharge relationship or tidal data for catchments discharging into ocean-influenced waterways.

Historic records or observations that can be used to define historical flood behaviour, and thereby calibrate the model against, include:

- *Rain Gauges*: pluviometers provide rainfall intensities which permit modelling of the rainfall pattern for the event while daily rain gauges assist in mapping total rainfall over the event across the subcatchments;
- *Continuous Water Level Recorders*: gauges that record the complete hydrograph enable calibration of not just the peak flood level but also the timing of the rise and fall of the flood;
- *Maximum Height Gauges*: gauges that record the peak flood level reached during a specific event;
- *Peak Level Records*: markers placed (usually by government agencies) after the event to indicate the peak flood level or maximum flood extent reached;
- *Debris Marks*: where floating debris remains on an object from the receding flood waters, resulting in a line indicating the flood level reached;
- *Watermarks on Structures*: residual watermarks on structures can indicate the flood level reached; and
- *Anecdotal Information*: descriptions of flood levels or behaviour, as well as photographs or videos.

The study area contains very little of the abovementioned data recorders:

- No pluviometers are located within the Tomaga River or Candlagan Creek Catchments;
- No daily rain gauges are located within the Tomaga River or Candlagan Creek Catchments;
- The closest ocean tide station is Ulladulla Harbour located 70 km away;
- The continuous water level recorder located on the Tomaga River is tidally affected;
- No large events have occurred in recent years;
- Community consultation feedback mostly anecdotal.

The paucity of information does not facilitate an extensive calibration and validation exercise. Nevertheless, the study considers a tentative calibration based on the little data available within the catchment and supplemented with data from neighbouring catchments. Adjustment and checks of modelling inputs undertaken include:

- Tidal event hydraulic model calibration;
- Rainfall event hydraulic model calibration;
- Validation of modelled flows against those obtained using Regional Flood Frequency Estimates; and
- Verification of modelled flood extents against flood affected areas reported by the community.

7.2. Tidal Calibration

7.2.1. Description

In addition to rainfall-derived calibration events, it is recommended that tidal calibration be undertaken in catchments where the interaction between the tidal inundation and the rainfall runoff is important, as is the case in the catchments investigated in this flood study. Tidal calibration ensures that the model can reproduce tidal amplification and isolate the mechanisms that may be responsible for variations in the modelled and recorded hydrographs.

Tidal calibration is of particular significance for this study as the majority of the areas of interest are within the tidal affectation zone. Tomakin, Mossy Point and Broulee are situated on the coast and the tidal signature can be observed as far up as Mogo.

7.2.2. Methodology

Tidal calibration is undertaken by modelling the recorded hydrograph produced by a tide level gauge during a period with no recorded precipitation. The resulting hydraulic model hydrograph is compared against the recorded hydrograph produced by a continuous water level gauge.

The tide level recorded at Jervis Bay was applied to the downstream boundary of the hydraulic model, thereby prompting the filling and emptying of the model domain within the tidally affected zone. The George Bass Drive continuous water level recorder is 3 km upstream of the applied tailwater boundary and within the tidal zone.

Two tidal calibration events were selected, where gauges around the Tomaga River and Candlagan Creek catchments recorded no precipitation. The 3 day period between the 6th and 8th of May 2012 was selected due to the chronological proximity to the rainfall calibration event and the current catchment conditions. The 3 day period between the 22nd and 25th of December 1998 was selected due to the correlation with the bathymetric survey period (Section 2.1.2).

7.2.3. Calibration Results

Results for the tidal calibration of the hydraulic model for the 1998 and 2012 events are shown in Figure 10 and Figure 11. The timing of the tidal oscillations was replicated by the model across both events. The 1998 event displayed a stronger correlation between the hydraulic model behaviour and the recorded hydrograph; in terms of both the amplitude and the rate of the rise and fall. The hydraulic model results for 2012 event exhibited slightly higher water levels at the crests and lower water levels at the troughs. Both the ascending and descending limbs displayed some acceleration, with the latter somewhat more than the former. As there is a period of time separating the 2012 event from the 1998 bathymetric survey, the variation in hydraulic model behaviour could be attributed to variations in the morphology of the waterway (i.e. scouring).

Generally, from observation of the tidal levels comparative to the water levels recorded at

George Bass Drive, it can be seen that tidal flows are subject to attenuation such that:

- the water level crest at George Bass Drive is lower than the tidal crest and occurs on the descending limb of the tide; and
- the water level trough at George Bass Drive is higher than the tidal trough and occurs on the ascending limb of the tide.

7.3. Rainfall Calibration – February 2010 Event

7.3.1. Description

As discussed in Section 2.6.1, three continuous stations are situated close to the Tomaga River and Candlagan Creek catchments. The Moruya Airport and Araluen Lower pluviometers were established in January 1999 and January 1980 respectively while the Deep Creek Dam tipping bucket was established in December 1996. The highest rainfall total over 24 hours of any gauge was recorded at Moruya Airport on February 2010. The same event ranked sixth on the Deep Creek Dam gauge which is located north of the catchment (while Moruya Airport Gauge is located south). Araluen gauge is located to west of the catchment but it is substantially further away from the study area than the other two gauges and is on the other side of the low mountain range delimiting the Tomaga River/Candlagan Creek Catchments and the event ranked 138th.

While larger events have occurred in the past (Section 3.3), temporal patterns are required to undertake a thorough calibration exercise; consequently only events that occurred subsequent to the installation of pluviometers can be considered. Furthermore, the George Bass Drive water level gauge, which is the only source of recorded water levels within the catchment, was opened in 1998. Consequently the February 2010 event presents the only viable option for a potential rainfall calibration without widespread assumptions.

7.3.2. Methodology

Analysis of pluviometer data, the rainfall distribution derived from rainfall gauges proximate the catchments (no rainfall gauges were located within), and radar data originating from the Canberra (Captains Flat) radar station permits confirmation of the February 2010 storm behaviour.

The 2010 storm event occurred over a 24 hour period approximately; straddling two days of daily read rainfall data (hence the rainfall distribution is derived from the 48 hour period prior to 9am on the 16th February 2010). Linear triangulation of the total depths for the 48 hour period creates an interpolated surface from which total rainfall depths can be interrogated for each respective subcatchment shown in Figure 12. Applying the most relevant temporal pattern to each subcatchment and running the hydrologic model provides the flow hydrographs for each subcatchment which is then applied as an inflow into the hydraulic model as shown in Figure 9.

Tidal levels recorded at the Jervis Bay Gauge (216470) during the event are applied as the downstream tailwater level for the hydraulic model. Similarly, initial water level in both the 1D and 2D hydraulic model domain is taken as the recorded level at Jervis Bay at the time the event simulation is commenced.

7.3.3. Calibration Results

From observation of the tidal levels comparative to the water levels recorded at George Bass Drive, it can be seen that during the 2010 event:

- Runoff began arriving at George Bass Drive between 17:00 hrs and 20:00 hrs on the 14th February. This was determined from the crest of the water level occurring at a similar height to the crest of the tide level (rather than the water level recording a lower height to the tide level, discussed in Section 7.2.3). This runoff was found to originate from the rainfall that occurred prior to 15:00 hrs on the 14th February.
- Runoff arriving at George Bass Drive peaked during the low tide around 15:00 hrs on the 15th February. This resulted in a plateau of the water levels recorded at George Bass Drive and the largest difference between the tide level and the water level at George Bass Drive.
- Runoff continued to arrive at George Bass Drive until after 00:00 hrs on the 17th February. This was again determined from the crest of the water level occurring at a similar height to the crest of the tide level.

It should be noted that rainfall that descends on the upper areas of the catchment will result in runoff at a downstream location (such as George Bass Drive) several hours after the rainfall occurred due to the flow needing to traverse the distance between the upper and lower areas of the catchment.

Results for the rainfall calibration of the hydraulic model against the February 2010 event are shown in Figure 13. The peak water level modelled was found to be within 0.01 m of the peak water level recorded at George Bass Drive. However, the modelled results displayed a somewhat earlier peak than what was recorded. The water level peak that was recorded at around 22:00 hrs on the 15th February was found to occur within the model at around 17:00 hrs on the 15th February. This would likely indicate that the rainfall depths applied to the model are a reasonable representation of what occurred; however that the rainfall temporal pattern experienced within the catchment may have differed slightly from the temporal patterns recorded within the adjacent catchments. This is a constraint of not having any pluviometric data within the catchment. Nevertheless, replication of the peak water level means that some confidence can be derived for the hydraulic model schematisation, including but not limited to the selected roughness values within the hydraulic model domain.

7.4. Flow Validation – Regional Flood Frequency Estimates

7.4.1. Description

The paucity of streamflow data in the study area is typical of many small to medium sized catchments in Australia. In these cases, peak flow estimates can be obtained using a Regional Flood Frequency Estimation (RFFE) approach, which transfers flood frequency characteristics from a group of gauged catchments to the location of interest. Even in cases where there is recorded streamflow data it is beneficial to pool the information in the gauged record with the RFFE information. The RFFE technique used in this study is being developed as part of the ARR Project 5 (Reference 7) and information is derived from a national database consisting of 853 gauged catchments.

7.4.2. Methodology

Flood Frequency Analysis refers to procedures that use recorded and related flood data to identify the underlying probability model of flood peaks at a particular location in the catchment. While the methodology is designed to be robust and widely applicable, catchment attributes such as catchment storage and steep impervious gradients requires consideration as they can significantly affect discharge volumes. This is particularly applicable to the swamp area south of Candlagan Creek which is likely to significantly attenuate flood peaks. Consequently flood estimates from the RFFE method will overestimate discharge volumes downstream of that location.

7.4.3. Validation Results

The peak discharges obtained from the RFFE method were compared to that computed by the hydraulic model. For the upper parts of the Tomaga River catchment, discharge values obtained from the RFFE method are 15% - 20% lower than that computed from the hydraulic model. The discrepancy arises as a result of the steep gradient characteristic of the upper catchment not being explicitly contained in the RFFE method. The steeper upper part of the Candlagan Creek catchment led to a larger discrepancy.

Values from the RFFE method are in good agreement with that obtained from the hydraulic model further downstream. For the catchment areas as far as the Princes Highway approximately, the bias introduced by the steep slopes is reduced by the more typical topography characterising the middle part of the catchment.

Downstream of the highway, the Tomaga River floodplains and swamp area north of Candlagan Creek significantly attenuate flows explaining some of the underestimation by the RFFE method to that obtained by the hydraulic model. Furthermore, the 1 m AHD downstream tailwater conservatively applied to the hydraulic model leads to backwatering of the flows therefore augmenting the difference between discharges obtained from the respective methods.

7.5. Flood Extent Verification –Reported Flooding

7.5.1. Description

As part of the community consultation a number of areas that get wet were reported (discussed in Section 3.3). While levels such as those shown in Figure 6 subsequent to the 1974 storm cannot be used in a full calibration due to the absence of rainfall data, they still provide information regarding flood mechanism and extents.

7.5.2. Verification Results

Figure 14 identifies areas reported as previously flood affected against the 1% AEP modelled extent. Areas identified as flood affected in Tomakin, Broulee and Mogo are shown to be wet in the 1% AEP event. It is worth noting that flood affectation in Tomakin and Broulee is characterised by overland flow whereas Mogo is more mainstream and consequently more likely to scale in larger events. No flooding has been reported in Mossy Point and modelled flood extents indicate that the elevation of the area leads to low flood risk there.

Table 13 approximates the depths from Figure 6 and compares these to the design flood depths (modelled as per Section 8). The equivalent design depths were found to be generally smaller than a 0.2 EY event (or 5 year ARI event), which corresponds to the ARI estimated from the daily read rainfall data (shown in Table 9).

Table 14 approximates the depths from anecdotal descriptions and compares these to the design flood depths (modelled as per Section 8). The equivalent design depths were found to be between the 10% AEP – 5% AEP event (or the 10 year – 20 year ARI event), which corresponds to the ARI estimated from the daily read rainfall data (shown in Table 10).

Table 13: Elizabeth Drive, Broulee – 1974 event flood marks compared to design depths (in meters)

Location	Approx. depth of floodmark minus 0.3 m	Average Equivalent Design Event	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	PMF
90	0.60	~ 0.2 EY	0.51	0.62	0.74	0.88	1.00	1.11	1.38
86	0.60		0.71	0.82	0.94	1.08	1.20	1.31	1.57
72	0.70		0.76	0.87	0.99	1.13	1.25	1.36	1.63
62	0.60		0.47	0.58	0.70	0.84	0.96	1.07	1.33
56	0.65		0.69	0.80	0.92	1.06	1.18	1.29	1.55
52	0.40		0.56	0.67	0.79	0.93	1.04	1.16	1.42
36	0.10		0.27	0.35	0.45	0.58	0.69	0.81	1.07
34	0.10		0.34	0.45	0.57	0.71	0.83	0.94	1.20
28	0.10		0.12	0.21	0.32	0.44	0.55	0.64	0.89

Table 14: Princes Highway, Mogo – 1991 event flood depths (approximate) compared to design depths (in meters)

Location	Approx. depth	Average Equivalent Design Event	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	PMF
52	1.60	10% AEP – 5% AEP	1.45	1.58	1.92	2.29	2.63	2.80	4.10
48	1.60		1.36	1.53	1.88	2.24	2.58	2.75	4.06
42	1.60		1.35	1.48	1.83	2.19	2.53	2.70	4.03

8. DESIGN FLOOD MODELLING

8.1. Introduction

There are two basic approaches to determining design flood levels, namely:

- *flood frequency analysis* – based upon a statistical analysis of the flood events, and
- *rainfall and runoff routing* – design rainfalls are processed by hydrologic and hydraulic computer models to produce estimates of design flood behaviour.

The *flood frequency* approach requires a reasonably complete homogenous record of flood levels and flows over a number of decades to give satisfactory results. The George Bass Drive gauge has was unsuitable for this purpose due to tidal affectation and insufficient length of operation (gauge established in 1996).

For this reason a *rainfall and runoff routing* approach was used. The rainfall intensities and patterns from AR&R 1987 were used in the computing of the design rainfall to be input into the WBNM model. The WBNM model derived inflow hydrographs that were input to the TUFLOW hydraulic model. The TUFLOW hydraulic model in turn determines design flood levels, flows and velocities.

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

- Lag Parameter (C) – 1.7
- Pervious Area Initial Rainfall Loss – 20 mm
- Pervious Area Continuing Rainfall Loss – 3.5 mm/hour
- Impervious Area Initial Rainfall Loss – 1 mm
- Impervious Area Continuing Rainfall Loss – 0 mm/hour

8.2. Oceanic Coincidence

Flooding in tidal waterways may occur due to a combination of oceanic inundation and catchment flooding derived from the same storm cell. The combined impact of these two sources on overall flood risk varies significantly with distance from the ocean and the degree of ocean influence, which is in turn affected by the estuary's entrance conditions. The *Development of Practical Guidance for Coincidence of Catchment Flooding and Oceanic Inundation*, hereon in referred to as the guide, presents a multivariate approach to translating the real-world environment for hydraulic modelling purposes. A sequential road-map is provided quantifying a number of parameters likely to affect flood mechanisms particularly in the context of peak flood levels and velocities. Parameters include the waterway entrance type, degree of accuracy required in the results and geographical location. The approach facilitates an optimum solution between the conflicting constraints of maintaining consistency in the modelling methodology while avoiding over-conservativeness in results.

The guide recognises the differing requirements of studies. Consequently, it accommodates three approaches to deriving ocean boundary conditions and design flood levels for flood modelling investigations in coastal waterways. A simplistic approach, a general approach and a detailed approach are proposed. The simplistic approach is considered suitable for analysis of small scale site specific developments where a cost effective but conservative method is warranted. The guide recommends either the general or detailed approaches for strategic studies undertaken for local government or with state government funding unless agreed to in writing by the local council and the funding provider, if state government. For general or detailed approaches, the combination of catchment flooding and ocean inundation scenarios is shown in Table 15.

Table 15: Combinations of Catchment Flooding and Oceanic Inundation Scenarios (Table 8.1 within *Modelling the interaction of Catchment Flooding and Oceanic Inundation in Coastal Waterways – OEH Draft 2014)*

Design AEP for peak levels/velocities	Catchment Flood Scenario	Ocean Water Level Boundary Scenario
50% AEP	50% AEP	HHWS
20% AEP	20% AEP	HHWS
10% AEP	10% AEP	HHWS
5% AEP	5% AEP	HHWS
2% AEP	2% AEP	5% AEP
1% AEP Envelope Level	5% AEP	1% AEP
1% AEP Envelope Level	1% AEP	5% AEP
1% AEP Envelope Velocity	1% AEP	Neap
0.5% AEP	0.5% AEP	1% AEP
0.2% AEP	0.2% AEP	1% AEP
PMF	PMF	1% AEP

Report No. MHL 1881 (*NSW Ocean Water Levels – Manly Hydraulics Laboratory, 2011*) documents a consistent tidal water level increase from south to north along the NSW coastline. Consequently, the guide splits the coastline into two regions based on whether the study area is north or south of Crowdy Head. Design ocean still water levels are obtained from the Fort Denison gauge in Sydney Harbour. This provides peak elevated ocean levels for design purposes (rounded up to nearest 0.05 m) and these levels are adjusted with an additional 0.1 m for regions situated north of Crowdy Head. The study area is located to the south of Crowdy Head.

The guide provides a framework within which the interaction of catchment flooding and oceanic inundation for the various classes of estuary waterways found in NSW (as well as associated ocean boundary conditions) can be assessed. The degree of influence of coastal processes on flooding within a waterway depends on the connectivity of the waterway to the ocean. This in turn depends on the type of estuary linked to the coastal waterway, the morphology and training of the waterway entrance and any management intervention. The guide classifies waterways into five Groups which are in turn simplified in three types, namely: Type A, Type B and Type C. Type A includes open oceanic embayments, tide dominated estuaries and trained entrances draining directly to the ocean or to bays. Type B includes fully trained wave dominated entrances and Type C includes ICOLLS and estuaries with untrained entrances. The categorisation is catchment specific and can be guided by the NSW Government 'Estuaries of NSW' website (http://www.environment.nsw.gov.au/estuaries/list.htm), which provided classifications based on Roy *et al* (2001) (Reference 18).

Tomago River and Candlagan Creek were classified as wave-dominated, barrier estuaries with open entrance conditions. However, the entrances are untrained and the study area was therefore determined to be Type C, as summarised in the table below.

Table 16: Summary of Decision Making

Name of Waterway	Tomaga River and Candlagan Creek	
Location		
Purpose of Assessment	Flood Study	
Local Council	Eurobodalla Shire Council	
1. Available Information Informing this assessment		
	Adopted Methodology / Figures	**Reasoning / Reference / Source of Information**
2. Waterway Entrance Type	C	Group 3 Wave Dominated Estuaries
3. Selected Approach	General	Develop downstream boundary for catchment wide flood study
4. Entrance Condition and Management	N/A	Waterway Entrance Type C
5. Modelling the Ocean Water Level Boundary		
North or South of Crowdy Head	South	Eurobodalla – location south of Crowdy Head
Peak Design Ocean Boundary Water Level	1% AEP – 2.55 m AHD 5% AEP – 2.35 m AHD	Eurobodalla – location south of Crowdy Head for Type C Waterway
Static or Dynamic Analysis	Dynamic	
Initial water level conditions in estuary	Based upon dynamic ocean boundary water levels aligned to start of the simulation	
6. Translating the Ocean Boundary to Study Boundary		
Adjustment	N/A	
Method Used / Source	N/A	
7. Relative timing of catchment flooding and oceanic inundation		
Peak Catchment with Static / Dynamic Ocean	Dynamic catchment flooding and oceanic inundation – peaks aligned	Aligned at downstream boundary of study area
8. Determining design flood levels		
Design AEP	PMF 0.5% AEP 1% AEP 2% AEP 5% AEP 10% AEP 0.2 EY	Project Brief

Design Flood Envelope	1%	Envelope derived from combinations as per Table 8.1 in guide
9. Sensitivity Testing		
Ocean boundary condition	Ocean boundary level increased by 0.3 m	
Peak Timing	Offset of peak ocean boundary to flood peaks by +/- 3hrs	Time of concentration 6-24hrs
Efficiency of Entrance	N/A	
10. Incorporating sea level rise		
Councils adopted Projections	Available	Council's adopted SLR projections
Adjustment made to: Boundary condtions Initial water levels Starting entrance conditions	Add Council's SLR projection to these factors.	Project Brief

8.3. Rainfall Critical Duration

To determine the critical storm duration for various parts of the catchment and inform the adopted design flood modelling, modelling of the 1% AEP rainfall event with a constant 2.35 m AHD ocean level was undertaken for a range of design storm durations from 25 minutes to 72 hours, using temporal patterns from AR&R (1987). An envelope of the model results was created, and the storm duration producing the maximum flood depth was determined for each grid point within the study area.

It was found that the 72 hour, 48 hour, 36 hour, 9 hour and 4.5 hour design storm durations were critical across the whole catchment for the 1% AEP event. The 36 hour storm duration was critical across a larger area of the catchment than the other storm durations; covering the Tomago River and Candlagan Creek, from the ocean outlet up to Tomakin Road and the Princes Highway. The 72 hour storm duration was critical within the township of Broulee, bounded by George Bass Drive to the west, Candlagan Creek to the north and the ocean to the east. The peak flood level difference between the two durations was 0.03 m in favour of the 72 hour duration (within Broulee) and 0.62 m in favour of the 36 hour duration. The 48 hour storm duration was critical along Mogo Creek (a tributary to Tomakin River) from downstream of the Princes Highway up to Burkes Lane. The peak flood level difference between the two durations was 0.03 m in favour of the 48 hour duration (within Mogo) and 0.10 m in favour of the 36 hour duration. The 9 hour storm duration was critical along the tributaries into Tomago River and Candlagan Creek, and within the township of Tomakin. The peak flood level difference between the two durations was 0.1 m in favour of the 9 hour duration (along Tomago River from Tomakin Road up to Dunns Creek Road) and 0.4 m in favour of the 36 hour duration. Within Tomakin, the peak flood level difference between the 9 hour and 36 hour durations was 0.02 m in favour of the 9 hour duration. The 4.5 hour storm duration was critical along the tributaries upstream of Dunns Creek Road. The peak flood level difference between the two durations was 0.1 m in favour of the 4.5 hour duration and 0.7 m in favour of the 36 hour duration. Therefore it was determined appropriate to adopt the 36 hour design storm duration for the design storm events ranging from the 20% AEP to the 0.5% AEP event.

Additionally, the critical storm duration was determined for the PMF event for a range of storm durations, ranging from 30 minutes to 6 hours using the Generalised Short-Duration Method (GSDM) and from 24 hours to 96 hours using the Generalised South-East Australia Method (GSAM). Similarly, an envelope of the model results was created, and the storm duration producing the maximum flood depth was determined for each grid point within the study area.

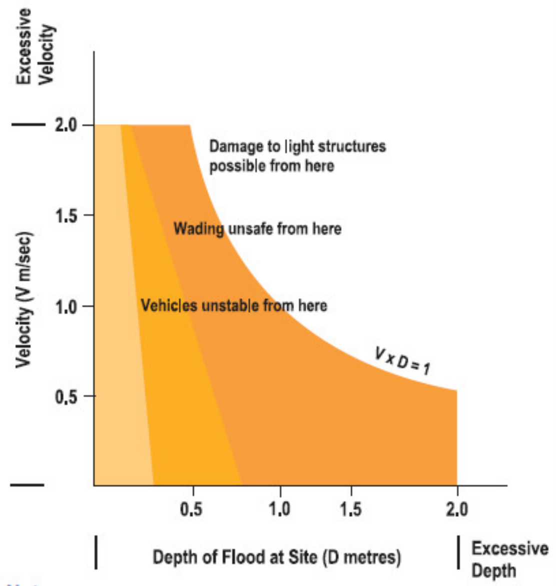
It was found that the 36 hour, 6 hour, 3 hour, 2 hour and 1.5 hour design storm durations were critical across the catchment for the PMF event. The 6 hour storm duration was critical across a larger area of the catchment than the other storm durations; covering the township of Tomakin, the Tomago River and Candlagan Creek, from the ocean outlet up to Tomakin Road and the Princes Highway. The 36 hour storm duration was critical within the township of Broulee; with the peak flood level difference of 0.4 m in favour of the 36 hour duration and 1.7 m in favour of the 6 hour duration. The 3 hour storm duration was critical along the Jeremadra Creek and Mogo Creek, including the township of Mogo. The peak flood level difference was 0.3 m in favour of the 3 hour duration and 0.6 m in favour of the 6 hour duration. The 2 hour storm duration was critical along Tomago River from Tomakin Road up to Dunns Creek Road; with the peak flood level difference of 0.2 m in favour of the 2 hour duration and 1.2 m in favour of the 6 hour duration. The 1.5 hour storm duration was critical along the tributaries upstream of Dunns Creek Road; with the peak flood level difference of 0.03 m in favour of the 1.5 hour duration and 1.6 m in favour of the 6 hour duration. Therefore it was determined appropriate to adopt the 6 hour design storm duration for the PMF event.

8.4. Analysis

8.4.1. Provisional Hydraulic Hazard

Hazard categories were determined in accordance with Appendix L of the NSW Floodplain Development Manual, the relevant section of which is shown in Diagram 3. For the purposes of this report, the transition zone presented in Diagram 3 (L2) was considered to be high hazard.

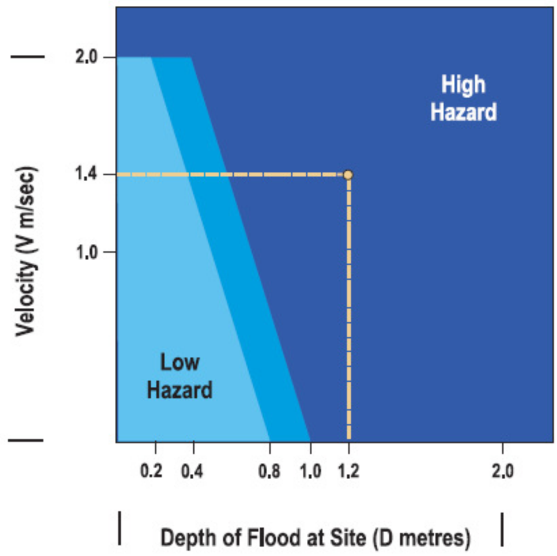
Diagram 3: (L1) Velocity and Depth Relationship; (L2) Provisional Hydraulic Hazard Categories (NSW State Government, 2005)



Notes

1. At velocities in excess of 2.0 m/s, the stability of foundations and poles can be affected by scour. Also, grass and earth surfaces begin to scour and can become rough and unstable
2. The velocity of floodwaters passing between buildings can produce a hazard, which may not be apparent if only the average velocity is considered. For instance, the velocity of floodwaters in a model test has risen from an average of 1 m/sec to 3 m/sec between houses.
3. Vehicle instability is initially by buoyancy.
4. At floodwater depths in excess of 2.0 meters and even at low velocities, there can be damage to light-framed buildings from water pressure, flotation and debris impact.

Derived from laboratory testing and flood conditions which caused damage.



Notes

The degree of hazard may be either -

- reduced by establishment of an effective flood evacuation procedure.
- increased if evacuation difficulties exist.

In the transition zone highlight by the median colour, the degree of hazard is dependant on site conditions and the nature of the proposed development.

Example:

If the depth of flood water is **1.2 m**
and the velocity of floodwater is **1.4 m/sec**
then the provisional hazard is **high**

8.4.2. Provisional Hydraulic Categorisation

The hydraulic categories, namely floodway, flood storage and flood fringe, are described in the Floodplain Development Manual (NSW State Government, 2005). However, there is no technical definition of hydraulic categorisation that would be suitable for all catchments, and different approaches are used by different consultants and authorities, based on the specific features of the study area.

For this study, hydraulic categories were defined by the following criteria, which correspond in part with the criteria proposed by Howells et. al. (2003):

- Floodway is defined as areas where:
  - the peak value of velocity multiplied by depth (V x D) > 0.25 m^2/s **AND** peak velocity > 0.25 m/s, **OR**
  - peak velocity > 1.0 m/s **AND** peak depth > 0.15 m

  The remainder of the floodplain is either Flood Storage or Flood Fringe:
- Flood Storage comprises areas outside the floodway where peak depth > 0.5 m; and
- Flood Fringe comprises areas outside the Floodway where peak depth < 0.5 m.

8.4.3. Preliminary Flood Emergency Response Classification of Communities

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

Criteria for determining flood ERP classifications and an indication of the emergency response required for these classifications are provided in the Floodplain Risk Management Guideline, 2007 (Flood Emergency Response Planning: Classification of Communities). Table 17 summarises the response required for areas of different classification. However, these may vary depending on local flood characteristics and resultant flood behaviour, i.e. in flash flooding or overland flood areas.

Table 17: Response Required for Different Flood ERP Classifications

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

8.5. Results

8.5.1. Peak Flood Depths and Levels

The peak flood depths and peak flood levels are summarised in the table below. In the 0.2 EY event, inundation occurs on approximately a dozen properties on the north side of Connells Close in Mossy Point, roadways in Broulee and properties along Veitch Street, Creek Street, Charles Street and the Princes Highway in Mogo. In the 2% AEP event, the affectation increases to include more properties on Connells Close (to the south and east of the roadway), River Road and Hilmer Avenue in Mossy Point and properties in Broulee from Heath Street to Train Street.

Table 18: Peak Flood Depths (m) and Peak Flood Levels (m AHD) at Key Locations

Location	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	PMF
Depth (m)							
Candlagan Creek - Upstream of Coronation Dr	1.89	2.11	2.34	2.75	2.88	3.08	4.02
Candlagan Creek - Upstream of George Bass Dr	1.74	1.97	2.22	2.62	2.80	3.02	4.01
Broulee - Crn of Angle St and Elizabeth Dr	0.61	0.72	0.84	0.98	1.10	1.21	1.48
Tomago River - at the mouth	2.96	3.01	3.11	4.32	4.50	4.56	5.14
Mossy Point - Connells Cl	0.28	0.38	0.60	1.49	1.63	1.91	3.61
Tomakin - Crn of Ainslie Pde and Parks Pde	0.29	0.32	0.36	0.41	0.44	0.66	2.37
Tomago River - Downstream of George Bass Dr	3.68	3.91	4.18	4.95	5.12	5.40	7.12
Jeremadra Creek - Upstream of the Princes Hwy	2.68	2.98	3.32	3.64	4.03	4.63	5.59
Mogo Creek - Upstream of the Princes Hwy	2.37	2.69	3.06	3.43	3.79	3.96	5.28
Mogo Creek - Goba St	2.66	2.91	3.20	3.44	3.69	3.88	5.34
Level (m AHD)							
Candlagan Creek - Upstream of Coronation Dr	1.70	1.92	2.15	2.56	2.69	2.89	3.83
Candlagan Creek - Upstream of George Bass Dr	1.80	2.03	2.27	2.68	2.85	3.08	4.07
Broulee - Crn of Angle St and Elizabeth Dr	4.65	4.76	4.88	5.02	5.14	5.25	5.51
Tomago River - at the mouth	1.02	1.08	1.17	2.39	2.57	2.63	3.21
Mossy Point - Connells Cl	1.43	1.54	1.76	2.64	2.79	3.07	4.76
Tomakin - Crn of Ainslie Pde and Parks Pde	2.76	2.79	2.83	2.88	2.91	3.13	4.84
Tomago River - Downstream of George Bass Dr	1.46	1.69	1.96	2.73	2.90	3.18	4.90
Jeremadra Creek - Upstream of the Princes Hwy	5.74	6.03	6.37	6.69	7.09	7.69	8.65
Mogo Creek - Upstream of the Princes Hwy	7.68	8.00	8.37	8.74	9.10	9.27	10.58
Mogo Creek - Goba St	8.79	9.03	9.32	9.56	9.82	10.01	11.46

8.5.2. Peak Flow

The peak flood flows are summarised in the table below. In the smaller events, such as the 0.2 EY and 10% AEP event, the tidal flats between George Bass Drive and Coronation Drive attenuates the flow resulting in lower peak flow at Coronation Drive. In larger events, the attenuation properties of the tidal flats are exceeded by the flow along Candlagan Creek. Along Tomago River between George Bass Drive and the confluence with the ocean, the flow exceeds the attenuation capacity of the tidal flats in all events investigated; with the flow width across George Bass Drive extending further to the east in the PMF event.

Table 19: Peak Flows (m^3/s) at Key Locations

Location	0.2 EY	10% AEP	5% AEP	2% AEP	1% AEP	0.5% AEP	PMF
Flow (m^3/s)							
Candlagan Creek - Crossing Coronation Dr	27.91	37.54	49.99	73.55	85.62	100.73	249.44
Candlagan Creek - Crossing George Bass Dr	28.31	38.02	49.64	68.71	80.48	95.00	248.40
Tomago River - at the mouth	212.51	265.65	337.18	452.28	522.00	618.45	1558.90
Mossy Point - Connells Cl	0.82	1.03	1.52	2.12	2.40	3.12	6.84
Tomago River - Crossing George Bass Dr (between Annetts Pde and Tomakin Rd)	203.37	260.12	335.51	415.69	505.72	604.08	1588.28
Jeremadra Creek - Crossing the Princes Hwy	114.87	145.81	184.53	221.29	260.03	300.58	857.59
Mogo Creek - Goba St	74.11	93.58	117.94	142.19	168.58	196.30	507.54

8.5.3. Provisional Hydraulic Hazard

During the 5% AEP event, the high hazard areas are mostly contained within the creeks and river; including the creek through Mogo and small sections of roadway in Broulee. In the 1% AEP event, the high hazard areas extend out from the creeks and rivers more. The township of Tomakin was determined to be mostly low hazard in the 5% AEP and 1% AEP event, transitioning to mostly high hazard in the PMF event.

8.5.4. Provisional Hydraulic Categorisation

During the 5% AEP event, the floodway areas are mostly contained within the creeks and rivers, including the creek through Mogo. The flood storage areas in the tidal flats area are adjacent to the creeks and river, and small sections of roadway in Broulee. In the 1% AEP event, the floodway areas extend out from the creeks and rivers more. The township of Tomakin was determined to be mostly flood fringe in the 5% AEP and 1% AEP event, transitioning to mostly flood storage in the PMF event.

8.5.5. Preliminary Flood Emergency Response Classification of Communities

Mapping of the preliminary flood emergency response classification of communities is shown on Figure 31. Mogo is classified as Rising Road Access as the properties are inundated from the creek at the rear of the properties, allowing evacuation by road. Many parts of Broulee and Tomakin were classified as Low Flood Island, with roads cut before properties are inundated. Mossy Point was classified as either Overland Refuge Area on High Flood Island (to the north) or High Flood Island. The former classification allows evacuation to the adjacent High Flood Island area, which is above the PMF extent but isolated by floodwater.