4. COMMUNITY CONSULTATION

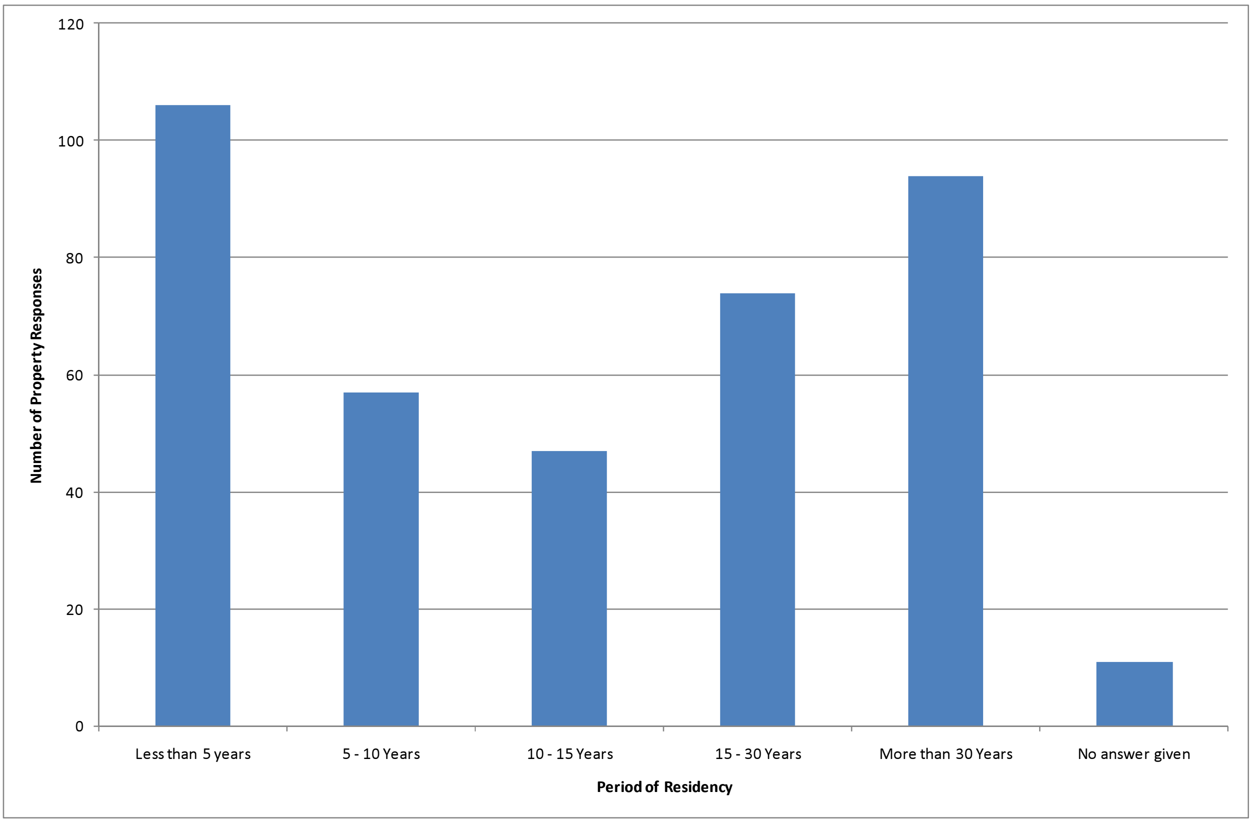
4.1. Community Questionnaire and Information Sheet

In collaboration with WCC, a questionnaire and information sheet were distributed to residents and owners of property within the study area catchment, describing the role of the Flood Study in the floodplain risk management process, and requesting records of historical flooding. A total of 3113 surveys were distributed with reply paid envelopes, and 393 responses were received (a return rate of 13%). Of these, 314 responses related to residential property, 59 related to commercial or industrial property, and 3 related to both residential and non-residential property. The remaining 17 responses did not indicate the type of property.

The information requested in the survey included details about length of residency in the catchment, descriptions of any experiences of flooding, and evidence of flood heights or extents such as photographs or flood marks.

The community members who responded to the questionnaire represented a balanced mix of short- and long-term experience in the catchment, as illustrated in Diagram 2. A total of 168 respondents indicated a period of residency of more than 15 years, many of whom would have experienced the March 1995 and August 1998 storms.

Diagram 2: Histogram of Residency Period for Community Survey Respondents



The occasions when respondents recalled being affected by flooding are summarised in Table 5 below. The most frequently recalled flood experiences in the survey responses related to the December 1990 storm, although other events were also mentioned by a significant number of respondents.

Table 5: Summary of Reported Incidents of Flooding

Flood Event	Total Responses	House Flooded (above floor)	Other Buildings Flooded (above floor)	Other Descriptions of Flooding
March 1978	21	4	3	11
March 1983	20	1	2	13
December 1990	35	5	3	23
March 1995	33	5	5	18
August 1998	18	1	5	11

The flood experiences described in the survey responses generally related to nuisance flooding, such as ponding of stormwater in roadways or gardens, although instances of above floor flooding in both residential and non-residential properties were also reported. December 1990 and March 1995 were the storms with the most reports of above floor inundation of residential property, with 5 instances apiece.

A copy of the questionnaire and flyer, as well as a summary of responses received, is provided in Appendix B.

4.2. Public Exhibition

Another community consultation phase was undertaken towards the end of the study, when all design modelling and mapping was completed. A draft Flood Study report was placed on public exhibition, with printed reports available for viewing at the Council library, and electronic versions available online.

Council distributed a letter including a questionnaire and feedback form to residents identified as being potentially affected by the Probable Maximum Flood (PMF). The criterion for properties included in the consultation was for more than 10% of the land to be inundated by greater than 0.15 m in the PMF. It should be noted that the PMF is an extremely rare event, and therefore there was a very low threshold of assessed total flood risk for a property to be included in the mail-out for this stage.

The report was placed on exhibition for a period of 4 weeks. During this period, two information sessions were held by staff from Council and WMAwater to listen to issues or comments identified by the community.

Approximately 80 written responses were received from the community providing feedback on the draft Flood Study report. The responses were primarily focussed each property owner's opinion as to whether the property should be "included in a flood zone," or similar. These responses could largely be grouped into three categories:

- those who identified a serious flood problem either at their own property (approximately 15% of respondents);
- those who provided a neutral response, indicating they had not observed flood issues, but not providing any indication of agreement or otherwise with the outcomes of the study (about 42% of respondents). Several of these responses contained a description of local overland flow behaviour in the vicinity of the property, which generally validated the modelled overland flow paths; and
- those who gave a response indicating displeasure or disagreement with the outcomes of the study (approximately 43% of respondents). Many of these responses included a comments related to whether the property should be labelled as flood prone. It should be noted that about half of the replies in this category were from owners of properties not tagged as within the Flood Planning Level extent (refer to Figure E-22). That is, the property would not be subject to flood-related development controls for normal residential/commercial development. Many of these responses also indicated a fear of rising insurance premiums relating to flood cover as a result of this Flood Study.

Some of responses displayed a lack of understanding of the objectives of the study, and/or a lack of clarity of some of the key terms used in the mail-out, such as “affected by flooding” and “Probable Maximum Flood”. Some additional discussion is provided below to attempt to clarify some of the issues and concerns raised by respondents.

4.2.1. Clarification of Flood Mechanisms

Several responses contained phrases like “my property is X m above sea level,” or “my property is on the high side of the road,” and indicated that flooding was consequently impossible at these locations. It should be noted that the study considered flooding from several mechanisms, including ocean effects but also including overland flow from rainfall/runoff at the top of the catchment.

It is quite common for properties to be flooded by overland flow from higher areas, even on very steep slopes (although not necessarily above floor level, if the house is raised above surrounding ground levels). This can even be true for properties on the high side of the street, if flow depths exceed the carrying capacity of the road gutters in the next street up the hill, or if the topography of areas above causes a concentration of flow towards the property.

Height above sea level is not the only indictor of flood risk for a property. The catastrophic flooding at Toowoomba in January 2011 resulted in four deaths and widespread damage, despite the towns positioning towards the crest of the Great Dividing Range, some 700 m above sea level. Steep hill gradients can in some places accentuate flood risk, due to potentially increased rainfall intensity resulting from orographic effects.

5. STUDY METHODOLOGY

5.1. General Approach

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.). High quality survey datasets were available for this study, which enabled a detailed topographic model of the catchment to be established. However the historical hydrologic data (such as rainfall patterns and stream-flows) were relatively limited.

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 overland 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.

There are no stream-flow records in the catchment, so the use of a flood frequency approach for the estimation of design floods is not possible.

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, and the approach does not lend itself well to estimating flood inundation in mid- to upper-catchment areas, as required for this study. 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 single modelling approach.

In view of the above, 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 (including runoff from roof drainage systems, gutters, etc.). The hydrologic model used design rainfall patterns specified in Reference 1, and the runoff hydrographs were then used in a hydraulic model to estimate flood depths, velocities and hazard in the study area.

The subcatchments in the hydrologic model were kept small (less than a typical residential block), such that the overland flow behaviour for the study was generally defined by the hydraulic model. This joint modelling approach was calibrated against observed historical flood levels. Additionally, the estimated flows at various points in the catchment were validated against alternative modelling techniques.

5.2. Hydrologic Model

Australian Rainfall and Runoff (Reference 1) 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 more complex rainfall-runoff routing models that estimate complete flow hydrographs.

In recent times, techniques have been developed to incorporate estimation of the rainfall-runoff relationship in a catchment directly into the hydraulic model. These techniques, often referred to as “direct rainfall on grid” approaches, remove the need for a hydrologic model entirely. However there is widespread uncertainty as to the validity of such methods, which can be highly sensitive to modelling assumptions, and therefore are highly dependent on good quality calibration data being available.

For the present study, the Watershed Bounded Network Model (WBNM) was used to estimate sub-catchment flows, with the maximum sub-catchment size in urban areas limited to about a quarter of a residential block. Using this approach, most of the flood routing of runoff is undertaken in the hydraulic model, with the hydrologic model only used to model the concentration phase of runoff. This concentration phase includes physical processes that occur on a scale too small to be adequately represented by the hydraulic model, such as runoff collecting on roofs, drainage through downpipes, and surface sheet flow from gardens etc.

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, in light of the lack of suitable calibration data. WBNM has been widely adopted in Australia for use in similar studies.

The flow estimates from the joint hydrologic/hydraulic modelling approach were verified against model results for peak catchment flow generated by hydrological modelling alone, at various locations in the catchment (refer to Section 6.3).

5.3. Hydraulic Model

The availability of high quality ALS data means that the study area is suitable for two-dimensional (2D) hydraulic modelling. Various 2D software packages are available (SOBEK, TUFLOW, Mike FLOOD) and the TUFLOW package (Reference 7) was adopted as it is widely used in Australia and WMAwater have extensive experience in the use of the TUFLOW model.

The Wollongong City catchment study area consists largely of low- to medium-density residential development, with some commercial, industrial and open space areas. Overland flood behaviour in the catchment is generally two-dimensional, with flooding along road reserves and large storage areas near the catchment outlet. In the upper catchment areas, the study objectives will require accurate representation of the overland flow system including kerbs and gutters and defined drainage channels.

The 2D model is capable of dynamically simulating complex overland flow regimes and interactions with sub-surface drainage systems. It is especially applicable to the hydraulic analysis of flooding in urban areas which is typically characterised by short-duration events and a combination of channelised and overland flow behaviour.

For the hydraulic analysis of complex overland flow paths (such as the present study area where overland flow occurs between and around buildings), 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 overland flow 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 result definition required, the dimensions of the river/creek channel (as a rough guide the channel should have over 3 cells widths in order to accurately define it) and the computer run time (which is largely determined by the total number of grid cells).

5.4. Design Flood Modelling

Following validation of the peak inflows from upper catchment areas through the use of alternative calculation methods, the following steps were undertaken:

- design inflows for localised sub-catchments were obtained from the WBNM hydrologic model and applied as inflows to the TUFLOW model;
- sensitivity analysis was undertaken to assess the relative effect of changing various TUFLOW modelling parameters; and
- design floods were modelled in TUFLOW using parameters selected to provide a sensible match between design flood levels and available recorded peak flood levels from historical events.

6. HYDROLOGIC MODELLING

6.1. WBNM

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 study area. The catchment layout for the model is shown on Figure 9.

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 channelised flows occurring through each catchment;
- rainfall initial and continuing losses to represent infiltration; and
- the percentage of catchment area with a pervious/impervious surface.

6.2. Design Event Modelling

Given the lack of calibration data for the hydrologic modelling, the input parameters were determined based on typical or default values, based where possible on experimental data for similar catchments.

6.2.1. Lag Parameter

Lag times for runoff depend on several physical catchment characteristics, including area, planform shape and steepness (among others) for natural catchments. Experimental data for natural catchments in Australia has demonstrated that the dominant factor 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. For WBNM, the adopted relationship is (Reference 8):

$$Lag\ time = Lag\ Parameter\ \times Area^{0.57} \times Discharge^{-0.23}$$

Since the relationship includes the effect of catchment area and flood magnitude, a similar value of the Lag Parameter (C) should apply to a wide range of catchment and flood sizes (Reference 8). 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. Similar data has been obtained for Queensland, Victoria, and South Australia, with estimated values of 1.47, 1.74, and 1.64 respectively. A value of 1.7 was adopted for design flood modelling in this study, in agreement with the NSW data. Note that previously the recommended value for the lag parameter was 1.29 (Reference 1), but this has now been superseded due to the availability of additional data.

6.2.2. Rainfall Losses

Methods for modelling the proportion of rainfall that is “lost” to infiltration are outlined in AR&R. 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 Reference 1 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 lower bound initial loss of 10 mm was adopted, as it is the most conservative estimate (i.e. it will lead to increased runoff estimates), and as it will result in the minimum reduction in total volume from the AR&R design burst rainfall patterns. The design storms do not contain antecedent rainfall, whereas real storm bursts are often preceded by a period of lower intensity rainfall, which would wet the catchment and reduce infiltration during the peak storm burst.

6.2.3. Impervious Areas

Runoff from connected impervious surfaces such as roads, gutters, roofs or concrete aprons occurs significantly faster than from natural surfaces, resulting in a faster concentration of flow at the bottom of a catchment, and increased peak flow in some situations. It is therefore necessary to estimate the proportion of a catchment area that is covered by such surfaces.

The residential areas of the Wollongong City catchment are generally low to medium density, with relatively large yards and grass verges adjacent to the roads. Based on a review of aerial photographs, these areas were assumed to be approximately 40% covered by impervious areas, mainly due to roads, pavements and roof-gutter systems. The assumed impervious area for each sub-catchment is indicated on Figure 9.

6.2.4. Embedded Design Storm Approach

The traditional AR&R87 approach to design storm hydrology is based on the estimation of a peak flow generated by a critical duration peak burst rainfall pattern. The method assumes that antecedent rainfall prior to the critical duration burst does not impact upon the peak flow estimates (Reference 13). Several other studies indicate that a failure to incorporate antecedent conditions prior to the critical duration peak burst may result in the underestimation of peak flows for some catchments (References 14 and 15). This is particularly the case where the critical burst durations are much shorter than the duration of historic flood-producing rainfall events. For the Wollongong City catchment, there is a significant chance that high-intensity short duration storm bursts likely to cause flooding at culverts and other structures in the smaller urbanised catchments will occur during the course of a broader, longer duration storm of reduced intensity, which may produce flood issues at trapped depressions or the major watercourses.

To address this issue, an alternative approach was adopted for this study whereby a shorter critical duration design storm burst is embedded within a longer duration storm of the same return interval. The shorter burst is used to replace the peak of the longer duration storm, and the intensity before and after the peak is adjusted so that the total rainfall depth is consistent with that of the original longer duration design storm. This procedure has been widely used since the publication of AR&R87.

Additionally, this approach has the benefit of allowing the design temporal patterns for multiple storm durations to be modelled as part of a single storm. In the Wollongong City catchment, the 2-hour and 9-hour design storms were found to be critical for peak flood levels across most of the catchment. Using the 2-hour burst embedded in the 9-hour storm allowed this behaviour to be modelled using a single temporal pattern (see Section 9.1 for additional discussion).

6.3. Validation of Methodology

Stream-flow records are required for calibration of a hydrologic model as they provide measurements that can be compared to the model outputs. There are no stream-flow records available to undertake a meaningful calibration of the hydrologic model.

The verification approach has involved comparing results using the WBNM model alone (with typical parameters for the region) against results from the TUFLOW model using two approaches:

- Direct rainfall-on-grid (“RF” approach); and
- Application of routed flows for each sub-catchment directly to road-reserves/open channels (“SA” approach).

Figure 8 shows the portion of the study area selected for verification. Purple polygons indicate sub-catchment extents, and black polygons indicating the area within which each sub-catchment flow was applied for the “SA” method. Preliminary maps of peak depth results of TUFLOW modelling for the “RF” and “SA” methods are shown in Figure D-1 and Figure D-2 respectively. Figure D-3 shows a comparison of outflows for the study area using the following methods:

- WBNM with stream routing factor of 1.0 (default for natural catchments);
- WBNM with stream routing factor of 0.67 (default for gravel surfaces);
- TUFLOW with “SA” method; and
- TUFLOW with “RF” method.

The results from the TUFLOW “SA” method are more consistent with the WBNM results, both in terms of peak flow (magnitude and timing) and rate of rise. While the WBNM model results are not necessarily “correct”, the WBNM model has been verified against substantial empirical data, including data from the Wollongong region. It therefore provides a good indication of expected runoff hydrograph results for the area. These results highlight the problems that can be encountered when using the “RF” method with the concentration phase of the rainfall, with the response time, rate of rise and peak flow all reduced compared to other methods.

Based on these results, the “SA” method for modelling the runoff concentration phase was selected for this study.

7. HYDRAULIC MODELLING

7.1. Model Extents

A hydraulic model was established for the study using the TUFLOW package. The model extents cover the entire study area catchment. The model includes a small portion of Port Kembla Inner Harbour at the downstream end, to assess backwater effects from tidal influences. The model extent is indicated on Figure 10.

7.2. Terrain Model

A computational grid cell size of 2 m by 2 m was adopted, as it provided 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 triangulation of filtered ground points from the LiDAR dataset, with detail survey of cross-sections “stitched in” to provide increased accuracy for major waterways.

Permanent buildings and other 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 (i.e. they were removed from the model grid – refer to Figure 10).

Photograph 2: Looking east from Springhill Road 250m south of Masters Road – example location where dense vegetation returns a flase ground signal for aerial survey.

Vegetation density is an important factor for the estimation of flood behaviour, particularly in the riparian corridor. The extents of various vegetation groups were identified using aerial photography, and classified according to observations from site inspections. The floodplain within the study area has been largely cleared, except for areas subject to regular tidal inundation in the lower catchment, where there are areas of mangrove and reeds. The riparian vegetation in some of these areas is extremely thick, to the degree that some places it returns a false "ground" signal for the aerial survey dataset (Photograph 2). Based on site visits and aerial photographs, parts of the terrain model were reduced in areas of very dense riparian vegetation (Figure 10) to reflect actual ground levels. Comprehensive detail survey of these areas was not available, but survey of pipe inverts in the vicinity of the dense vegetation was used to inform estimates of the actual ground level.

7.3. Boundary Conditions

The model schematisation is illustrated on Figure 10, including the location of the subcatchment inflow boundary conditions.

The downstream boundary condition was located at Port Kembla Inner Harbour. The assumptions for the tailwater boundary are discussed in detail in Section 9.5.

7.4. 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, based on detailed survey undertaken in the study area (Appendix C). Culverts were generally modelled as 1D features embedded in the 2D model, since the majority of culverts of interest have dimensions smaller than the grid resolution. For some larger culverts and bridges, where the main flow width exceeds the grid resolution, modelling was undertaken in the 2D domain using a TUFLOW feature specifically designed for this purpose, whereby the energy losses and blockage caused by the piers and deck 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 detailed survey, photographs taken during site inspections, and previous experience modelling similar structures. Sensitivity analysis of the effect of the hydraulic structure parameters is presented in Section 10.3.

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 addressed by the hydrologic modelling approach and also partially via the roughness approach for residential areas.

8. MODEL CALIBRATION

As identified in the sensitivity analysis (Section 10), rainfall intensity and blockage are the model inputs which have the most influence on peak flood levels in the study area. Often these two factors are the most uncertain when attempting to re-produce historical flood behaviour with models. The catchment is subject to significant variability in rainfall (Section 3.6), and there is a high propensity for blockage of structures as evidenced by the August 1998 floods, and as recognised in the stormwater management policies of WCC (Reference 6). These factors were therefore primary considerations for the model calibration process.

The adopted calibration approach involved modelling of a wide variety of historical storms, and assessing the envelope of modelled flood levels against recorded historical flood levels. This approach was required partially because of the uncertainty surrounding some recorded flood marks about which flood event they may actually have applied to (due to several similarly-sized storms in the 1990s). Furthermore, substantial variation was observed in the rainfall records at different gauges for many of the large historical storms.

In particular, the closest long term pluviometer to the catchment (Port Kembla gauge) consistently showed substantially less rainfall than other gauges further inland (and at higher elevations). This strong orographic rainfall gradient introduces significant uncertainty into the estimation of an appropriate catchment average rainfall intensity for historical events. The assumed historical rainfall pattern has a significantly greater effect on modelled flood levels in the catchment than modification of model parameters, which would be the normal method used to adjust the model results to match observed levels.

The calibration results for the 2011 storm were considered the most reliable, mainly because:

- The relatively new rain gauge at Figtree provided good quality rainfall data that appears to be indicative of what fell in most of the catchment (whereas the Port Kembla gauge recorded far less rainfall, as with other historical events). The Figtree gauge was not operational for other major historical storm events on the catchment;
- Good quality recorded flood level information (collected by WCC staff) was available at key catchment locations, including in areas of deeper flow at open channels and at culverts. These recorded data were considered more reliable than many of the other historical flood marks, which were often vague in description and in locations where only shallow flooding was observed; and
- Catchment topography has changed in the last thirty to forty years since the flood events of the 1970s and 1980s. Increases to pervious catchment area and infill development are likely to have changed the catchment response significantly compared to present conditions. There would therefore be significant uncertainty associated with calibrating the model to the older events, as the flood levels produced by the same rainfall under current conditions would be different, but the extent of changes are unknown and difficult to quantify.

In light of the uncertainties with rainfall data highlighted above, the hydraulic model was schematised using typical parameters for similar catchments based on experience and theory. The calibration results matched reasonably well with historical data, suggesting that a deviation from these typical parameters was unwarranted. A summary of results at calibration locations is given in Table 6 below. Additionally, qualitative comparisons of modelled flood extents with photographs taken after the March 2011 storm are presented in Appendix D.

Table 6: Comparison of Modelled and Recorded Flood Levels for March 2011 Calibration Event

POINT ID	Recorded Level mAHD	Modelled Levels (mAHD)		
		At Peak	At 2:30PM	At 3:00PM
1	1.63	1.76	1.74	1.65
2	1.68	1.82	1.79	1.68
3a	1.80	1.93	1.89	1.75
4b	1.80	1.96	1.91	1.77
5c	2.00	1.99	1.95	1.80
6b	2.41	2.60	2.49	2.45
6a	2.41	2.46	2.45	2.45
5b	2.00	1.99	1.95	1.80
4a	2.20	2.17	2.14	2.12
3b	1.80	1.93	1.89	1.75
3c	1.80	1.92	1.88	1.74
5a	2.00	2.13	2.06	1.86

The exact time of capture is not known for the photographs used to estimate the recorded flood levels, but it is estimated that most were taken roughly between 2:30pm and 3pm, after the flood peak. The model results for these times are generally a good match for the levels estimated from the photographs (and structure survey).

The locations of calibration levels are indicated on Figure D-14 (March 2011), Figure D-15 (August 1998), and Figure D-16 (May 1995). Photographs were used to estimate the levels (Appendix D) in conjunction with survey data (Appendix C).

9. DESIGN FLOOD MODELLING

9.1. Critical Duration

To determine the critical storm duration for various parts of the catchment, modelling of the 1% AEP event was undertaken for a range of design storm durations from 15 minutes to 12 hours, using temporal patterns from Reference 1. 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 2-hour and 9-hour design storms are critical for a significant majority of the study area. It is a common occurrence for these two storm durations to feature as critical design storms, due to the nature of the temporal patterns for these durations provided in Reference 1. Each of these temporal patterns includes bursts of shorter duration that have a similar recurrence interval (i.e. they are representative of a consistent return interval across the entire storm duration).

Based on this outcome, it was considered appropriate to adopt an embedded design storm for the entire catchment, using the two-hour design storm burst within the 9-hour design storm, adjusted to maintain the correct 9-hour total rainfall depth (see Reference 13). The peak flood depths produced using the embedded design storm were generally found to be within ±0.15 m of the peak depths obtained from the envelope of multiple storm durations.

9.2. Model Parameters

The key modelling parameters adopted for the 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 Wetting Loss – 1 mm
- Impervious Area Percentage – varies by subcatchment (see Figure 9).

A grid cell size of 2 m by 2 m was used for the hydraulic modelling. In TUFLOW this means there is a calculation point with an assigned survey elevation every 1 m, with alternating water level (H) and velocity (V) calculation points, such that water level is calculated at 2 m spacing.

The adopted parameters for the design flood hydraulic modelling are given in Table 7 and Table 8. The extent of the surface roughness regions as listed in Table 7 are illustrated on Figure 11.

The adopted roughness values are consistent with typical values in the literature (References 17 and 18) and previous experience with modelling similar catchment conditions. The sensitivity of model results to changes in the roughness values is discussed in Section 10.1.

Table 7: Design Flood Mannings "n" Values

Surface type	Mannings "n" value
Channel beds	0.035
Very short grass or sparse vegetation	0.035
General overland areas, gardens, roadside verges, low density residential lots etc. (default)	0.045
Medium density vegetation	0.06
Heavy vegetation	0.1
Roads, paved surfaces	0.025
Concrete pipes	0.013

9.3. Hydraulic Structures

The key model parameters for modelling of hydraulic structures such as culverts and bridges are the assumed energy losses at the structure (from turbulence, expansion/contraction of flow etc.) and blockage of the structure waterway area from debris. Schematisation of structures and the method for estimating loss parameters are discussed in Section 7.4. The loss parameters for bridges are given in Table 8 below. For culverts, losses were adjusted based on approach and departure flow velocities at the culvert entrance/exit, up to a maximum entrance loss of K=0.5 and a maximum exit loss of K=1.0.

Table 8: Design Parameter Values for Hydraulic Losses at Structures

Structure	Cumulative Loss Parameter *K* (as factor of dynamic head $V^2/2g$)	Blockage*
Bridge (below deck obvert)	0.3	0% to 20%
Bridge deck	1.3	100%
Bridge handrails (where present)	1.6	20% to 25%

* Note this blockage is due to the estimated ratio of waterway area that is obstructed by the piers at each structure, and not an allowance for potential debris blockage at these locations. Values are based on inspection of survey and photographs.

9.4. Blockage for Design Modelling

An additional blockage factor of waterway area due to debris was implemented in accordance with Wollongong City Council's Conduit Blockage Policy from Reference 6, which requires the following blockage factors to be applied to structures across all watercourses when calculating design flood levels

i. 100% blockage for structures with a major diagonal opening width of less than 6 m;
ii. 25% bottom up blockage for structures with a major diagonal opening width of greater than 6 m;

For bridge structures involving piers or bracing, the major diagonal length is defined as the clear diagonal opening between piers/bracing, not the width of the channel at the cross-section.

iii. 100% blockage for handrails over structures covered in (i) and for structures covered in (ii) when overtopping occurs.

This study included a detailed blockage assessment in the determination of design flood levels. Specifically, the approach included consideration of the combination of blocked and unblocked structures that produces the "worst-case" design flood estimate at each key hydraulic structure in the catchment resulting from blockage.

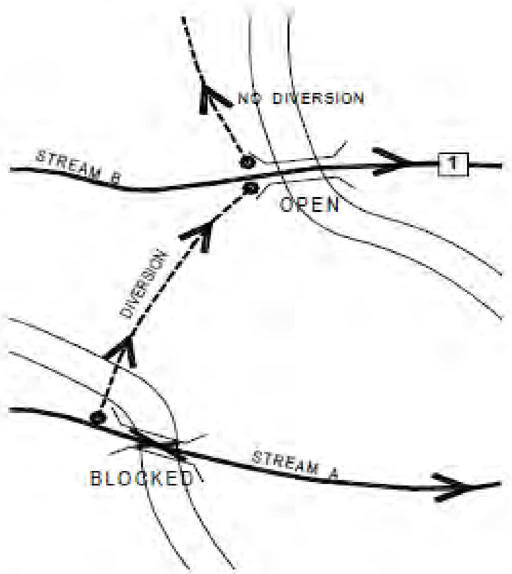


Figure 3.2 – Stream Diversions Occurring from Blockage

Note: Flow at location 1 is maximised in a scenario where the Stream B culvert is 'open' and the Stream A culvert is blocked causing a diversion of flow (Rigby and Silveri, 2001)

Diagram 3: Stream diversions occurring from blockage (Reference 22)

There are two major groups of culvert crossings where blockage would result in a significant influence on peak flood levels:

- the culverts under the Illawarra Railway Line and in the upper catchment (Upstream group); and
- the culverts under Springhill Road, Swan Street, and other locations where residential areas discharge into the main downstream waterways (Downstream group).

By modelling two scenarios, where each of these culvert groups was blocked according to Council's policy, and taking an envelope of peak flood levels, a scenario meeting Council's Blockage Policy was obtained including the full impact of blockage on design flood behaviour in the catchment.

Table 9 indicates the assumed blockage for larger pipes and bridges in the design flood modelling. The location of each structure corresponding to the ID labels in the table is illustrated in Figure 5, and detail survey information is provided in Appendix C. Other pipes not listed in the table have diagonal openings less than 6 m and were therefore assumed to be blocked.

The blockage assumption relating to significant bridges and culverts for the two blockage scenarios is given in Table 9. There are over 500 pipes in the hydraulic model, so only pipes for which detail survey was available are included in the table. The ID labels in the table correspond to those from Figure 5 and the survey information in Appendix C. Other pipes were blocked in either the Upstream (U/S) or Downstream (D/S) Railway scenarios according to their location.

Table 9: List of Blockage Assumptions for Design Flood Modelling

Culvert or Bridge ID	Blockage %	
	U/S Railway scenario	D/S Railway scenario
C9_01	0	100
C8	0	100
C7	0	100
C6	0	100
C5	0	100
C4_WOLLONGON	0	100
C4_CONISTON	100	0
C25_pipe	100	0
C24	100	0
C23	100	0
C22	100	0
C21	100	0
C20	0	100
C2	0	100
C19	0	100
C18	0	100
C17	0	100
C16	0	100
C15S	0	100
C12_01	0	100
C11	0	100
C10	100	0
B7	0	25
B6	0	25
B5	0	25
B4	0	25
B3_02	0	100
B2	0	25
B13	100	0
B10	25	0
B1	0	25

9.5. Tailwater Levels

Coastal flooding can have a significant influence on coastal waterways and can exacerbate the impacts of catchment flooding. These impacts are more likely to coincide in coastal catchments where the coastal and flood events are likely to be a result of the same storm event, as is the case for Wollongong City and Port Kembla.

The lower reaches of this estuary can be influenced by the loss of available storage in the estuary due to the coastal flooding and the backwater effects resulting from elevated ocean levels. Hence, the ocean level in the lead up to and coincident with peak catchment runoff can also have an influence on flood levels in the lower reaches. Consideration must therefore be given to accounting for the joint probability of coincident catchment and coastal flooding.

A full joint probability analysis is beyond the scope of the present study. Traditionally, it is common practice to estimate design flood levels in these situations using a 'peak envelope' approach that adopts the highest of the predicted levels from the two mechanisms. Reference 11 recommends an envelope of scenarios that includes:

1. 1% AEP ocean flooding with 5% AEP catchment flooding with coincident peaks;
2. 5% AEP ocean flooding with 1% AEP catchment flooding with coincident peaks; and
3. neap tide cycle with 1% AEP catchment flooding with coincident peaks.

However Council's Stormwater Management controls (Reference 6) stipulate an envelope of the following scenarios:

1. 1% AEP ocean level with 10% AEP catchment flooding; and
2. Ocean level of 1.9 mAHD (including sea level rise of 0.4 m by 2050) with 1% AEP catchment flooding.

For this study, scenarios were developed which were consistent with both Council policy and NSW government guidelines. As part of this process it was necessary to determine an appropriate tailwater condition for the 1% AEP ocean flood level.

NSW government guidelines (Reference 11) specify recommended approaches for setting the tailwater at an ocean water level boundary for flood risk assessment. The guideline provides three approaches to the development of appropriate tailwater levels for open entrances, for consideration in flood risk assessments. The first two approaches involve a fixed and dynamic boundary condition with a maximum level of 2.6 mAHD. The third requires a site specific assessment, and is recommended for major catchment studies where the 2.6 mAHD assumption is considered too conservative. The Consideration of Sea Level Rise in Flood and Coastal Risk Assessment paper presented at the NSW Floodplain Management Authorities Conference in 2011 (Reference 21) states:

> ***'Where the fixed approach is likely to be too conservative for the resultant decision, either the dynamic ocean boundary provided in the guideline or one specifically developed for the location and the associated conditions should be used to assess flood behaviour. Studies undertaken under the State's Floodplain Management***

Program are not to use the conservative fixed ocean boundary condition unless specifically agreed to by DECCW [now OEH].***"***

It was therefore considered appropriate to determine a site specific ocean water level boundary condition for this study. The Wollongong City catchment outlet at the Gurungaty Waterway is reasonably sheltered by the trained entrance and northern headland of Port Kembla Inner Harbour. Additionally, the volume of the harbour, while relatively small compared to other bays and harbours on the NSW coast, is still substantial enough to reduce the influence of wave setup on harbour water levels (as there is enough depth for wave inflows to flow back out through the entrance).

The annual high astronomical tide (due to gravitational effects of celestial bodies) on the NSW coast is around 1.1 mAHD to 1.2 mAHD. The highest recorded tide at Fort Denison in Sydney Harbour is 1.5 mAHD, which included barometric effects (storm surge) from a low pressure cell, and the 1% AEP level at Fort Denison is 1.45mAHD.

Port Kembla Inner Harbour has a trained entrance that will significantly reduce the influence of some of the ocean level components mentioned above, particularly the wave action and associated potential for wave setup, which are effects with a relatively short-term duration.

Based upon 10% of the significant wave height (1 hour to 6 hour storm duration) at Port Kembla wave setup would be between 0.88 and 0.79m. This does not consider the impacts of the harbour which would be expected to reduce this significantly.

Research has shown that the effects of wave setup in trained, deep entrances are minimal (Hanslow & Nielsen, Reference 23). Therefore a more appropriate allowance for wave setup might be 0.2m – 0.4m. Considering the 1% AEP level at Fort Denison is 1.5m AHD, together with the advice of Hanslow and Neilsen, tailwater levels of between 1.7m AHD and 1.9m AHD, excluding climate change impacts were considered appropriate. A 1% AEP peak level of 1.8 mAHD was adopted for this study.

In addition to the above it is not unreasonable to expect that the effects of a severe storm in terms of ocean levels and runoff could be coincident for a catchment of this size. Hence to establish the design flood levels in the present study, the relative phasing of the ocean levels was adjusted such that the peak of the tidal hydrograph would approximately coincide with the peak of the catchment runoff.

For smaller design events, a constant tailwater boundary of 1.5 mAHD was adopted. This level corresponds to the 1% AEP tidal level (including storm surge) based on records at Sydney harbour, and is therefore conservative for the smaller design events. However, the results for events smaller than the 1% AEP are highly insensitive to the assumed tailwater level of 1.5 mAHD, primarily due to the following factors:

- The Council blockage assumptions result in full blockage of the causeway structure within the Bluescope site, approximately 350 m upstream of the outlet to Port Kembla Inner Harbour (structure B3, Figure 5). This blockage assumption requires that all flow out of the Gurungaty Waterway must overtop the road crest at a level of 1.23 mAHD. The effect of this structure on conveyance in the channel dominates the effect assumed tailwater level, as can be seen from the design flood profiles shown on Figure E-1.

- The removal of temporary flood storage within the catchment, due to the relatively high initial 1.5 mAHD level, does not significantly affect the design flood results for small events. This is because the embedded storm approach provides significant antecedent rainfall prior to the peak intensity burst. In combination with the Council blockage policy, this antecedent rainfall would fill the low lying flood storage areas regardless of the assumed initial water level from the tailwater assumption.

Sea level rise of 0.4 m by 2050 and 0.9 m was considered as part of the climate change assessment, in accordance with NSW government guidelines (Reference 11).

A full matrix of design tailwater scenarios is given in Table 10 (Section 9.6). The influence of these varying tailwater assumptions were generally confined to the lower reaches below the main residential and commercial areas of the catchment.

9.6. Design Flood Model Scenarios

As previously discussed, the design flood results for this study were obtained from an envelope of various modelling scenarios (Table 10).

Table 10: Matrix of Design Flood Modelling Scenarios

<table>
<tr><th>Design Flood Event</th><th>Rainfall Event</th><th>Port Kembla Tailwater</th><th>Blockage</th></tr>
<tr><td></td><td colspan="3">Envelope of scenarios</td></tr>
<tr><td rowspan="2">20% AEP (current)</td><td rowspan="2">20% AEP 2-hour in 9-hour embedded storm</td><td rowspan="2">Constant 1.5 mAHD</td><td>U/S Railway</td></tr>
<tr><td>D/S Railway</td></tr>
<tr><td rowspan="2">10% AEP (current)</td><td rowspan="2">10% AEP 2-hour in 9-hour embedded storm</td><td rowspan="2">Constant 1.5 mAHD</td><td>U/S Railway</td></tr>
<tr><td>D/S Railway</td></tr>
<tr><td rowspan="2">5% AEP (current)</td><td rowspan="2">5% AEP 2-hour in 9-hour embedded storm</td><td rowspan="2">Constant 1.5 mAHD</td><td>U/S Railway</td></tr>
<tr><td>D/S Railway</td></tr>
<tr><td rowspan="2">2% AEP (current)</td><td rowspan="2">2% AEP 2-hour in 9-hour embedded storm</td><td rowspan="2">Constant 1.5 mAHD</td><td>U/S Railway</td></tr>
<tr><td>D/S Railway</td></tr>
<tr><td rowspan="3">1% AEP (current)</td><td>1% AEP 2-hour in 9-hour embedded storm</td><td>Constant 1.5 mAHD</td><td>U/S Railway</td></tr>
<tr><td>1% AEP 2-hour in 9-hour embedded storm</td><td>Constant 1.5 mAHD</td><td>D/S Railway</td></tr>
<tr><td>10% AEP 2-hour in 9-hour embedded storm</td><td>100 year dynamic tide with storm surge, wave setup etc. (peak 1.8 mAHD)</td><td>None</td></tr>
<tr><td rowspan="3">1% AEP with 0.4 m Sea Level Rise (2050)</td><td>1% AEP 2-hour in 9-hour embedded storm</td><td>Constant 1.9 mAHD</td><td>U/S Railway</td></tr>
<tr><td>1% AEP 2-hour in 9-hour embedded storm</td><td>Constant 1.9 mAHD</td><td>D/S Railway</td></tr>
<tr><td>10% AEP 2-hour in 9-hour embedded storm</td><td>100 year dynamic tide with storm surge, wave setup etc. (peak 2.2 mAHD)</td><td>None</td></tr>
<tr><td rowspan="3">1% AEP with 0.9 m Sea Level Rise (2100)</td><td>1% AEP 2-hour in 9-hour embedded storm</td><td>Constant 2.4 mAHD</td><td>U/S Railway</td></tr>
<tr><td>1% AEP 2-hour in 9-hour embedded storm</td><td>Constant 2.4 mAHD</td><td>D/S Railway</td></tr>
<tr><td>10% AEP 2-hour in 9-hour embedded storm</td><td>100 year dynamic tide with storm surge, wave setup etc. (peak 2.7 mAHD)</td><td>None</td></tr>
<tr><td rowspan="4">PMF (current)</td><td>30-minute Probable Maximum Precipitation</td><td rowspan="4">100 year dynamic tide with storm surge, wave runup etc, without reduction from guideline due to entrance conditions.
(peak 2.6 mAHD)</td><td rowspan="4">All blocked</td></tr>
<tr><td>60-minute Probable Maximum Precipitation</td></tr>
<tr><td>120-minute Probable Maximum Precipitation</td></tr>
<tr><td>180-minute Probable Maximum Precipitation</td></tr>
</table>

Two different blockage scenarios were modelled for each return interval, as discussed in

Section 9.4, except for the PMF where a scenario with all structures blocked was adopted.

The 1% AEP results included an envelope of higher ocean flood levels with lower catchment rainfall as per Council's Stormwater Management Guidelines (Reference 6). No blockage was included in this scenario as blockage would lower flood levels in this instance by preventing the elevated ocean level from causing backflow up the Gurungaty waterway.

9.7. Results

Maps of estimated peak flood depths and flood level contours from the design modelling process are presented in Appendix E on Figure E-5 to Figure E-10, for a range of flood magnitudes.

Peak velocities for the PMF and 1% AEP events are shown on Figure E-11 and Figure E-12 respectively.

Peak flood level profiles for the main waterway branches in the lower catchment are presented on Figure E-1 to Figure E-3. The chainage for each branch is measured from the downstream extent of the Gurungaty waterway at the outlet to Port Kembla Inner Harbour. Flood hydrographs (flood level and flow) at selected locations are shown on Figure E-4A to Figure E-4D. Refer to Figure 12 for the locations of profiles and hydrographs in these figures.

Results have been provided to Council in digital format compatible with Council's Geographic Information System (GIS).

9.7.1. Sea Level Rise

Estimated impacts on flood levels resulting from projected sea level rise are relatively low, with impacts confined to the lower catchment areas. Interestingly, modelling indicates that a rise of 0.4 m would have a negligible impact on peak flood levels for the 1% AEP event, even in the lower catchment. This appears to be due to the influence of the multiple bridge and causeway crossings in the lower Gurungaty Waterway, and particularly south of Tom Thumb lagoon.

Sea level rise of 0.9 m would have a more noticeable impact on 1% AEP peak flood levels, with increases of around 0.2 m in waterway downstream of Springhill Road, and increases of around 0.1 m upstream of Springhill Road.

Figure E-21 shows flood level hydrographs for the sea level rise scenarios, in the Gurungaty Waterway near Wollongong Greenhouse Park, at location E4-D (Figure 12).

Impacts of sea level rise on peak flood levels in the 1% AEP event are shown in Table 12. It should be noted that for smaller floods events, and regular high tides, the impact of sea level rise would be more significant. However, inundation of residential properties from purely tidal mechanisms would be unlikely to occur even with sea level rise of 0.9 m.

9.8. Provisional Flood Hazard and Preliminary True Hazard

Maps of provisional hydraulic hazard are presented on Figure E-17 and Figure E-18. Hazard categories were determined in accordance with Appendix L of the NSW Floodplain Development Manual (Reference 2).

The provisional hazards were reviewed in this study to consider other factors such as rate of rise of floodwaters, duration, threat to life, danger and difficulty in evacuating people and possessions and the potential for damage, social disruption and loss of production. These factors and related comments are given in Table 11.

Table 11: Weightings for Assessment of True Hazard

Criteria	Weight [1]	Comment
Rate of Rise of Floodwaters	High	The rate of rise in the creek channels and onset of overland flow along roads would be very rapid, which would not allow time for residents to prepare for the onset of flooding.
Duration of Flooding	Low	The duration for local catchment flooding will generally be less than around 6 hours, resulting in inconvenience to affected residents but not generally a significant increase in hazard.
Effective Flood Access	High	Roads within the catchment will generally be inundated prior to property inundation, which may restrict vehicular access during a flood.
Size of the Flood	Moderate	The hazard can change significantly at some locations with the magnitude of the flood, particularly in the residential areas near the golf course and JJ Kelly Park. However, these higher hazard areas are generally captured by mapping a range of events using the provisional hazard criteria.
Effective Warning and Evacuation Times	High	There is very little, if any, warning time. During the day residents will be aware of the heavy rain but at night (if asleep) residential and non-residential building floors may be inundated with no prior warning.
Additional Concerns such as Bank Erosion, Debris, Wind Wave Action	Low	The main concern would be debris blocking culverts or bridges. This is considered to have a high probability of occurrence and will significantly increase the hazard. There is also the possibility of vehicles being swept into the main channels (as occurred in Newcastle in June 2007) causing blockage. However design modelling for this study includes significant blockage and the provisional hazard classification therefore includes this factor. Wind wave action is unlikely to be an issue but waves from traffic may be, due to the proximity of flood prone properties to main traffic routes.
Evacuation Difficulties	Low	Given the quick response of the catchment evacuation is not considered to be necessary (it is safer to remain than to cross fast flowing floodwaters) except in a few instances and therefore was not given significant weight for assessing true hazard.
Flood Awareness of the Community	Low	The flood awareness of the community is quite high due to the frequency of recent flood events. As a result of this awareness of problem flood areas, this factor is assigned a low weight in assessing true flood hazard.
Depth and Velocity of Floodwaters	High	In areas of overland flow roads are subject to fast flowing water. In the main creek channels velocities and depth would be high. There is always a risk of a car or pedestrian being swept into the open channel while attempting to cross swiftly flowing waters at major creek crossings. However this factor is largely included in the provisional hydraulic hazard calculation metrics.

Note: [1] Relative weighting in assessing the preliminary true hazard.

For the Wollongong City catchment, the factors with high weighting in relation to assessment of true hazard are generally related to the lack of flood warning, and the potential for flooding of access to residential properties prior to above-floor flooding of buildings occurring. In most cases, it is likely that remaining inside the property will present less risk to life than attempting evacuation via flooded routes, as refuge can generally be taken on furniture etc. There may be some properties where remaining inside would present a high risk to life due to very high flood depths, but these properties will generally already be classified as high hazard using provisional hazard criteria. Further investigation of the risks of sheltering in place should be undertaken as part of the Floodplain Risk Management Study using surveyed floor level information.

In general it was found that areas where a high flood hazard would be justified based on consideration of the high-weight criteria in Table 11, the area was already designated high hazard as a result of the depth/velocity criteria used to develop the provisional hazard. However, additional information (particularly detailed floor level survey) may warrant revision of the true hazard categories at various properties during the Floodplain Risk Management Study phase.

9.9. Provisional Hydraulic Categorisation

Provisional hydraulic categorisations for the 1% AEP and PMF events are provided on Figure E-19 and Figure E-20. Reference 19 contains guidance on the delineation of floodways and flood storage areas. There is no technical definition of hydraulic categorisation that would be suitable for all catchments, and different approaches must be used in different areas, based on the specific features of the study catchment in question.

For this study, hydraulic categories were defined by the following criteria:

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

These criteria were selected based on sensitivity analysis of multiple combinations of depth/velocity conditions. The values adopted were considered by the Technical Sub-committee to produce the most reasonable categorisation with regard to the definitions provided in Reference 2.

9.10. Preliminary Flood Planning Extents

Preliminary flood planning areas were developed for the 1% AEP flood under current conditions, and for 2050 and 2100 conditions incorporating projected sea level rise increases from Reference 11.

The flood planning extents were obtained using the following methodology:

a. Peak design flood levels were filtered to remove areas of flooding shallower than 0.15 m.
b. Isolated patches of flooding smaller than 300 m^2 (approximately half the average residential property)
c. A freeboard of 0.5 m was added to the filtered peak levels;

d. The extent was increased to reflect additional areas within the 1% AEP plus 0.5 m level, based on the digital terrain model. This step was undertaken using the waterRIDE software package. In some areas of the catchment, particularly the steeper overland flow areas, it results in significant increases in flood extent. These increases in extent do not reflect physically realistic flow behaviour, as the additional runoff volume required to extend over these areas would be significantly higher than the design storm volume;
e. Parts of the extent outside the PMF flood extent were removed.
f. Properties more than 10% inundated by this extent were tagged.

Properties tagged using this process are shown on Figure E-22 to Figure E-24.

It may be necessary to consider the separation of overland flow and mainstream flood risk areas for development control in the Wollongong City Catchment, for example by specifying different freeboard requirements for overland flow areas. However differentiation of overland flow areas can require subjective assessment, due to the lack of formal creeks and drainage easements particularly in the upper catchment. This issue should therefore be investigated as part of the Floodplain Risk Management Study.

9.11. Preliminary Flood ERP Classification of Communities

The implications of the design flood modelling were assessed in accordance with guidelines for Emergency Response Planning (ERP) outlined in Reference 20. The criteria for classification of floodplain communities outlined in Reference 20 are generally more applicable to riverine flooding where significant flood warning time is available and emergency response action can be taken prior to the flood.

In urban areas like the Wollongong City catchment, flash flooding from local catchment and overland flow will generally occur as a direct response to intense rainfall without significant warning. For most flood affected properties in the catchment, remaining inside the home or building is likely to present less risk to life than attempting to drive or wade through floodwaters, as flow velocities and depths are likely to be greater in the roadway.

The design modelling indicates that in the PMF event some properties will be subject to high hazard flooding with depths greater than 1.0 m covering access routes, prior to potential flooding of buildings (Figure E-17). As floor level survey is unavailable for these properties, it is unclear what the depth of above-floor inundation of buildings on these properties may be. If estimated depths were life-threatening, these properties would need to be classified as “Low Flood Island” according to Reference 20, as by the time above-floor inundation occurs the roadways at the property frontages would already be inundated with high hazard flooding.

If a property is unaffected by above floor flooding but nearby streets are flooded, vehicular access from the area may be blocked, causing inconvenience or potentially threatening life if emergency medical care is required during a flood. This issue of flood isolation is less critical for urban flash flooding than for rural flooding as it is unlikely that access will be cut for more than a few hours. For example it is unlikely that provision of food or other supplies to isolated areas will be required in Wollongong City catchment. For this preliminary assessment, some areas have been classified as potential “High Flood Island” where flooding of roads may prevent access to emergency medical treatment. The SES does not provide definitive guidance on flood depth or velocity threshold before a road is “cut,” or on “acceptable” isolation times. For this study, roads have been assessed as potentially cut if the significant majority of the road is flooded by depths greater than 250 mm.

In light of these considerations, preliminary classification for the majority of the study area catchment is as “Indirectly Affected Area,” or “Not Flood Affected,” with some areas marked as “Potential Low Flood Island,” “Potential High Flood Island,” or as “Overland Escape” or “Overland Refuge” areas. Classification has been undertaken for the PMF, 1% AEP and 5% AEP events (Figure E-25 to Figure E-27).

Properties surrounded by high hazard flooding should be identified as priorities for any emergency response in this catchment (refer to Figure E-17 and Figure E-18).

Given that there will not be sufficient flood warning for evacuation prior to flooding occurring, it is likely that remaining indoors will generally present the lowest risk to life during flood events for people in the study area. However some of the properties in high hazard floodway and storage areas may potentially be “Low Flood Island” areas, depending on the height of the habitable areas above surrounding ground levels. Floor level survey is required to confirm the ERP classification of these properties.

It is recommended that detailed analysis including floor level survey be undertaken at the Floodplain Risk Management Study stage, with input from the SES, to complete classification in accordance with the ERP guideline.

10. SENSITIVITY ANALYSIS

Due to lack of historical data suitable for undertaking a formal model calibration, a number of assumptions have been made for the selection of the design approach/parameters, primarily relying on default parameter values or values used in similar studies. The following sensitivity analyses were undertaken for the 1% AEP event to establish the variation in design flood level that may occur if different assumptions were made:

- Mannings "n": The roughness values were increased and decreased by 20% at all locations;
- Structure losses: The energy loss at the bridge structures as a factor of dynamic head was increased/reduced by 20%;
- Intensity-Frequency-Duration: Different IFD design rainfall intensities can be obtained in different parts of the catchment. IFD values producing the highest and lowest rainfalls for the catchment (from Reference 1) were modelled and compared to the design case, which used values from the catchment centre.
- Inflows / Climate Change: Sensitivity to rainfall/runoff estimates was assessed by increasing the rainfall intensities by 10%, 20% and 30% as recommended under current guidelines. Refer to Section 10.4 below for discussion;
- Sea Level Rise: Sea level rise scenarios of 0.4 m and 0.9 m were tested, in accordance with the guidelines in References 10 and 11 (although these are not technically "sensitivity" analyses, as they are stipulated in policy for future planning); and
- Blockage The effect of leaving all structures unblocked was investigated.

It should be noted that the parameters are not independent and adjustment of one parameter (Manning's "n") would generally require adjustment of other values (such as inflows) in order for the model to produce the same level at a given location.

Table 12 on the following page provides a summary of peak flood level changes at various locations for the sensitivity scenarios. Yellow shading indicates that the magnitude of the change is greater than 0.1 m, while red shading indicates changes greater than 0.2m in magnitude.

10.1. Variation of Manning's "n"

Flood levels are relatively insensitive to the adopted "n" value compared to other parameters. An increase of 20% to roughness values increases flood levels by less than 0.1m generally for both mainstream and overland flow flood locations. A reduction of roughness values by 20% produces a reduction in flood levels of similar magnitude.

Given that there were insufficient calibration data to enable fine-tuning of roughness values, the relatively small change to flood levels resulting from changes to Manning's "n" reduces the uncertainty arising from this limited model calibration. Recommended values for Manning's "n" based on land-use/ground cover are widely available, and suitable ranges have been verified through practice and experience in hydrodynamic modelling. The values adopted were based on aerial photography and site inspection.

Table 12: Sensitivity of 1% AEP Peak Flood Levels at Indicative Locations

Refer to Figure 13 for map of sensitivity comparison locations.

ID	Location	1% AEP Peak Flood Depth	Sea Level +0.4m	Sea level +0.9m	Structure Losses –20%	Structure Losses +20%	Upper catchment IFD	Lower catchment IFD
1	Gurungaty Waterway US of private railway line, near Port Kembla Rd	2.56	-0.01	0.22	0.00	0.00	0.08	-0.07
2	Pedestrian bridge between Wollongong Greenhouse Park and Port Kembla Rd, crossing Gurungaty Waterway	2.95	0.01	0.20	-0.01	0.00	0.09	-0.08
3	Gurungaty Waterway US of Springhill Rd	3.31	0.00	0.11	-0.01	0.01	0.08	-0.08
4	JJ Kelly Park, near the corner of Swan St and Church St	2.66	0.00	0.10	-0.01	0.01	0.08	-0.08
5	JJ Kelly Park, DS of C1, near the corner of Swan St and Kembla St	2.93	0.01	0.10	-0.01	0.01	0.08	-0.08
6	D/S of Corrimal St	2.03	0.00	0.06	0.00	0.00	0.07	-0.07
7	Discharging into the Golf Course	1.69	0.01	0.06	0.00	0.00	0.06	-0.06
8	South of Springhill Rd, east of Tom Thumb Rd and west of the private railway line	3.60	0.00	0.00	0.00	0.00	0.11	-0.09
9	U/S of Springhill Rd near Masters Rd	3.17	0.00	0.05	-0.01	0.00	0.19	-0.18
10	U/S of Masters Rd	2.96	0.00	0.00	0.00	0.00	0.19	-0.11
11	U/S of pedestrian railway underpass near the junction of Gladstone Ave and South St Ln	1.54	0.00	0.00	0.00	0.00	0.03	-0.02
12	Near the junction of Gladstone Ave and Robertson St	6.34	0.00	0.00	0.00	0.00	0.15	-0.19
13	Gladstone Ave between Vale St and Grasmere St	2.62	0.00	0.00	0.00	0.00	0.05	-0.06
14	Gladstone Ave between Osborne St and Rowland Ave	0.84	0.00	0.00	0.00	0.00	0.06	-0.04
15	Cnr Myuna Way and Mangerton Road	0.74	0.00	0.00	0.00	0.00	0.03	-0.03

Yellow highlighting indicates change of 0.1m to 0.2m
Red highlighting indicates change of greater than 0.2m

ID	Location	Manning's –20%	Manning's +20%	Rainfall +10%	Rainfall +20%	Rainfall +30%	No Blockage
1	Gurungaty Waterway US of private railway line, near Port Kembla Rd	-0.04	0.03	0.08	0.14	0.20	-0.02
2	Pedestrian bridge between Wollongong Greenhouse Park and Port Kembla Rd, crossing Gurungaty Waterway	-0.03	0.02	0.10	0.18	0.26	0.00
3	Gurungaty Waterway US of Springhill Rd	-0.02	0.02	0.08	0.15	0.21	-0.06
4	JJ Kelly Park, near the corner of Swan St and Church St	-0.03	0.02	0.08	0.15	0.21	-0.04
5	JJ Kelly Park, DS of C1, near the corner of Swan St and Kembla St	-0.02	0.02	0.08	0.14	0.21	-0.05
6	D/S of Corrimal St	-0.03	0.02	0.07	0.13	0.18	-0.04
7	Discharging into the Golf Course	-0.03	0.03	0.06	0.12	0.18	-0.04
8	South of Springhill Rd, east of Tom Thumb Rd and west of the private railway line	-0.03	0.03	0.11	0.21	0.30	-0.87
9	U/S of Springhill Rd near Masters Rd	-0.04	0.04	0.19	0.36	0.54	-1.02
10	U/S of Masters Rd	-0.05	0.11	0.21	0.39	0.63	-1.30
11	U/S of pedestrian railway underpass near the junction of Gladstone Ave and South St Ln	-0.01	0.02	0.03	0.06	0.09	-0.14
12	Near the junction of Gladstone Ave and Robertson St	-0.08	0.09	0.15	0.25	0.32	-0.42
13	Gladstone Ave between Vale St and Grasmere St	-0.02	0.01	0.06	0.12	0.17	-0.21
14	Gladstone Ave between Osborne St and Rowland Ave	-0.03	0.02	0.06	0.12	0.18	-0.30
15	Cnr Myuna Way and Mangerton Road	-0.01	0.01	0.04	0.07	0.10	-0.07

10.2. Variation in Design Rainfall Assumptions

As indicated in Section 3.6.3, Intensity-Frequency-Duration (IFD) rainfall information from Reference 1 varies across the study area. Values from the centre of the catchment were used for design flood modelling, but values up to 11.3% higher or 10.4% lower can be obtained for other parts of the catchment.

The impacts of using these upper and lower bounds of design rainfall depths were modelled for the 1% AEP event. The change in flood levels for these scenarios was similar in magnitude to the 10% climate change sensitivity scenario, which is to be expected as the variation in rainfall depth is similar.

While rainfall depths for a given return interval may be slightly higher in the upper steep parts of the catchment, this will be balanced by slightly lower depths in the flatter parts of the catchment. However the total catchment rainfall, which is the most relevant factor for peak flood flows and depths in lower areas of the catchment, is likely to be well captured by the approach for this study of using a uniform rainfall depth based on information for the centre of the catchment (which is a standard approach for Flood Studies).

It should be noted that the design rainfalls from Reference 1, and the rainfall gradient due to the Illawarra escarpment, are likely to change as part of the upcoming revision of Australian Rainfall and Runoff. However the timeframe when the new data will be available from the Bureau of Meteorology is not yet finalised and it is not possible to predict the impacts of the revisions for design flood behaviour at Wollongong at this stage.

10.3. Variation of Hydraulic Energy Losses at Structures

The abutments, piers, decks and railings of bridges cause an obstruction to flow which results in afflux upstream of the bridge, and potentially a change in floodplain flow distribution. The influence of structures on channel flow, particularly in the Gurungaty Waterway, was included in the model through the use of energy loss parameters. The values used were based on typical bridge performance, and reflect assumptions about the hydraulic efficiency of the bridges.

The sensitivity of the model results to the assumed energy loss at bridges was tested by reducing and increasing the assumed loss parameters by 20%. The peak flood levels were found not to vary significantly with the changes, with a change of 0.01 m or less generally observed over the floodplain, including immediately upstream of the bridges. This is because the influence of the losses on flood levels is dependent on velocities, which are relatively low in the flatter areas of floodplain where the majority of the bridges are located.

10.4. Climate Change

10.4.1. Background

Intensive scientific investigation is ongoing to estimate the effects that increasing amounts of greenhouse gases (water vapour, carbon dioxide, methane, nitrous oxide, ozone) may be having on the average earth surface temperature. Changes to surface and atmospheric temperatures may affect climate and sea levels. The extent of any permanent climatic or sea level change can only be established through scientific observations over several decades. Nevertheless, it is prudent to consider the possible range of impacts with regard to flooding and the level of flood protection provided by any mitigation works.

Based on the latest research by the United Nations Intergovernmental Panel on Climate Change evidence is emerging on the likelihood of climate change and sea level rise as a result of increasing greenhouse gasses. In this regard, the following points can be made:

- greenhouse gas concentrations continue to increase,
- the balance of evidence suggests human activity has resulted in climate change over the past century,
- global sea level has risen about 0.1 m to 0.25 m in the past century,
- many uncertainties limit the accuracy to which future climate change and sea level rises can be projected and predicted.

The best available estimate of the projected sea level rise (including ice melt) along the NSW coast is from 0.18 m to 0.91 m by around the year 2100.

10.4.2. Discussion

The Bureau of Meteorology has indicated that there is no intention at present to revise design rainfalls to take account of the potential climate change, as the implications of temperature changes on extreme rainfall intensities are presently unclear, and there is no certainty that the changes would in fact increase design rainfalls for major flood producing storms. There is some recent literature by CSIRO that suggests extreme rainfalls may increase by up to 30% in parts of NSW (in other places the projected increases are much less or even decrease); however this information is not of sufficient accuracy for use as yet (Reference 9).

Any change in design flood rainfall intensities will increase the frequency, depth and extent of inundation across the catchment. It has also been suggested that the cyclone belt may move further southwards. The possible impacts of this on design rainfalls cannot be ascertained at this time as little is known about the mechanisms that determine the movement of cyclones under existing conditions.

Projected increases to evaporation are also an important consideration because increased evaporation would lead to generally dryer catchment conditions, resulting in lower runoff from rainfall. Mean annual rainfall is projected to decrease, which will also result in generally dryer catchment conditions. The influence of dry catchment conditions on river runoff is observable in climate variability using the Indian Pacific Oscillation (IPO) index (Reference 12). Although mean daily rainfall intensity is not observed to differ significantly between IPO phases, runoff is significantly reduced during periods with fewer rain days.

The combination of uncertainty about projected changes in rainfall and evaporation makes it extremely difficult to predict with confidence the likely changes to runoff and peak flows for large flood events at Wollongong under warmer climate scenarios.

In light of this uncertainty, the NSW State Government advice (Reference 9) recommends sensitivity analysis on flood modelling should be undertaken to develop an understanding of the effect of various levels of change in the hydrologic regime on the project at hand. Specifically, it is suggested that increases of 10%, 20% and 30% to rainfall intensity be considered.

10.4.3. Rainfall Increases

The effect of increasing the design rainfalls by 10%, 20% and 30% has been evaluated for the 1% AEP event, resulting in a relatively significant impact on peak flood levels in the study area. Generally speaking, each incremental 10% increase in flow results in a 0.06 m to 0.1 m increase in peak flood levels at most of the locations analysed.

There are some notable exceptions among the locations analysed where flood levels are more highly sensitive to rainfall increases, particularly at Springhill Road in the vicinity of Masters Road, and near Tom Thumb Road. Ponding at Gladstone Avenue near Robertson St immediately upstream of the railway line is also relatively sensitive to rainfall increases. This is because these locations are in trunk drainage areas of the lower catchment, where the catchment flows are concentrated, and therefore the rainfall increases result in more significant localised peak flood level increases (due to increases in both flood volume and peak flow).

10.5. Summary

From the sensitivity analysis, it was concluded that the principal factors that influence the modelled flood behaviour are the magnitude and timing of runoff flows, and to a lesser extent the effect of variation in design rainfall estimates across the catchment. The Manning's "n" roughness parameter and energy losses at structures have a relatively minor influence on peak flood levels in comparison.

It is considered that the design flood levels adopted reflect reasonable estimates of the model input parameters based on available information, and based on experience with other studies. However it is noted that variation of some of the above assumptions could result in localised changes to the estimated flood levels.

As mentioned above, there are limitless combinations of parameters, and a considerable effort was made to verify that the values used brought about a reasonable match of modelled design flood behaviour with the general flood behaviour from available historical flood records. The Manning's "n" values adopted were supported by historical aerial photography.

There is some uncertainty in the design rainfall depths adopted due to the significant orographic rainfall effects experienced in the Wollongong region. The results of the sensitivity analysis suggest that the impact of this uncertainty on peak flood levels is roughly equivalent to the impact that would be caused by a ±10% change in design rainfalls.

11. DISCUSSION

The hydrological and hydraulic modelling undertaken for this study has defined flood behaviour for the 5 year, 10 year, 20 year, 50 year and 1% AEP design floods, as well as the Probable Maximum Flood (PMF). The modelling process included a calibration/validation process that was somewhat limited by the availability of suitable hydrological data. Extensive sensitivity analyses were undertaken to assess the influences of modelling assumptions on key outputs, and the potential impacts of future climate change.

Based on the responses to the community consultation process, residents of the Wollongong City catchment are reasonably aware of flood issues in the area, due to a number of moderate flood events in the last 20 years, however, they may not be aware of the risks of a larger flood closer to the magnitude of the 1% AEP event. Based on inspection of the provisional hazard maps, some particularly high risk areas, in terms of risk to property and risk to life, include:

- Swan Street, and the southern parts of Kembla, Evans and Church Streets;
- Springhill Road, particularly between Swan Street and Keira Street;
- Springhill Road, 200 m to 300 m south of Masters Road;
- Trapped depressions along Gladstone Avenue upstream of the railway line, particularly near Osborne Street, Union Street, Robertson Street, and Vale/Grasmere Streets;
- Auburn Street, just south of Swan Street;
- Vacant land at the northern end of Gregory Street; and
- Wollongong Golf Club.

The design flood levels determined in this study report incorporate an envelope of blockage scenarios to give a “worst-case” for blockage, in accordance with Council’s Conduit Blockage Policy.

The design flood modelling indicates that significant flood depths may occur on the western side of the railway embankment, and shallow overtopping of the railway line at some locations, even without blockage of drainage systems under the railway. These potential issues were raised with RailCorp, who provided a response stating “there are no [railway] embankments with a potential for failure (meaning catastrophic collapse) under design flooding loading within the Wollongong City flood Study area.” (see Appendix B).

Roads & Maritime Services made comments during the course of the study regarding responsibility for management of Tom Thumb Lagoon and trash collectors in the vicinity of Bridge Street and Springhill Road intersection. It was also noted that significant silt build-up had occurred under the Springhill Road bridge. Responsibility for maintenance at these location could be addressed in the Floodplain Risk Management Study.

11.1. Recommendations for Floodplain Risk Management Study

During the course of the study, some issues were raised that may warrant further investigation in the Floodplain Risk Management Study, including:

- analysis of February 2012 rainfall that produced flooding in the lower catchment, and validation of the TUFLOW model against observed flood marks from that event;
- existing development controls relating to minimum floor levels and other flood planning levels may be unsuitable for overland flow areas in the Wollongong City catchment. The Floodplain Risk Management Study should review freeboard and flood planning level requirements in the catchment, and determine whether alternative development controls for overland flow areas can be implemented.
- more detailed assessment of the risks of “shelter in place” strategies for houses in high hazard flood areas, using detailed floor level survey, for a full range of flood events. Input from the SES regarding Emergency Risk Planning Classifications should be sought to clarify definitions for urban flash flood areas with little effective warning time; and
- clarification of maintenance responsibilities for structures and trash/debris racks, particularly in the vicinity of Springhill Road and in tidal areas where significant silt build-up has been observed.

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- Residents of Wollongong within the study area;
- Manly Hydraulics Laboratory;
- Bureau of Meteorology; and
- Sydney Catchment Authority.