Shell Garage, Tongara Road

Mr Harris has lived in Albion Park for sixty odd years and recalls the following details :

1. In 1978, flood waters were one foot over Manson's Bridge, Calderwood Road.
2. Flood waters have on several occasions reached the fence around the police station.
3. Flood waters have regularly entered the General Store and O'Gorman's Butcher Shop.
4. The cricket oval is regularly flooded and water has reached up tot eh concrete path behind where the new supermarket has opened at the show ground entrance.
5. Flood waters have reach the steps of the new 'Commercial Hotel'
6. The houses at the end of Hamilton Street have been flooded

.

7. Flood waters have been lessened since the Council removed the wooden bridge near Boles Farm 1965. The council put in concrete bank protection and raised the road and river to prevent the river from breaking tis banks and flowing into the swamp on the southern side of the road. This may have temporarily relieved the flooding caused by the Rivulet combining with the flood waters from Frazers Creek and flooding the Albion Park township. The swamp area south of Boles Farm acted as a retention basin but now the flood waters are forced north westward causing increased flooding on farms on the other side of the river. Eventually the Rivulet and Frazers Creek flood waters pool back to swamps.

8, In Croome Street, Greenmeadows, is subject to flooding. Flood waters have entered houses on two occasions since the opening of the Estate, and residents have had to remove fence palings to allow other passing of the flood waters.

9. Frazers Creek regularly floods, closing the road but the waters drop as quickly as they rose.

Mr Russell, Meadow View Farm, Illawarra Highway

Felt that the 1978 flood was the worst flood he had experienced since the 1960's. Flood waters reached between the two Railings of the front gate and the waters reached within 15 feet of his front door as well as reaching the cement slab in front of the dairy.

3.10 PREVIOUS STUDIES

The catchment of Macquarie Rivulet has been the subject of several flood related studies completed over the last quarter of a century. Those published prior to 1996 were reviewed and summarised in the report by the Water Research Foundation Illawarra Committee (refer Water Research Foundation (1996)). All studies reviewed in the course of this study are listed below, but those previously reviewed in the WRF(1996) report have not been further discussed in this report.

WATER RESOURCES COMMISION NSW

- *Albion Park Flood Study Report (1986)*

WARGON CHAPMAN & ASSOCIATES

- *Report on Investigation for New Railway Bridge Over Macquarie Rivulet at Albion Park 101.169 km (Illawarra Line) (1980)*

FORBES RIGBY PTY LTD

- *A Report on the Impact of Flooding on the Development of Land at Macquarie Rivulet, Yallah (1990*
- *A Supplementary Report on the Impact of Development of Land Adjacent to Macquarie Rivulet on Flooding in Macquarie Rivulet (1991)*
- *A Review of Flooding in the Lower Reaches of Macquarie Rivulet and the Potential for Cross Flows from Macquarie Rivulet into Albion Creek (1993)*
- *A Review of Flooding in the Lower Reaches of Macquarie Rivulet and the Impact of Development on Flooding (1994)*
- *A Review of Flooding at the Corner of Terry St and Tongarra Rd Albion Park, NSW (1995).*

KINHILL ENGINEERS PTY LTD

- *Extension to Albion Park Flood Study (1993)*

The studies completed post publication of the 1996 WRF report, together with the WRF report itself, are summarised as follows.

WATER RESEARCH FOUNDATION

A Report on Development of Macquarie Rivulet as a Reference Catchment for the Illawarra(1996)

This report, edited by Mr Rigby, was prepared for the Water Research Foundation by the Illawarra Regional Committee of the Water Research Foundation. This committee was set up after major flooding in Mullet Creek in February 1984, to share information on flooding in the region generally and to collate and analyse data for rainfall and flooding in Macquarie Rivulet, with a view to establishing Macquarie Rivulet as Reference Catchment for flood modellers practicing in the region.

This report and the report's companion 'Compendium of Data' are a highly useful source of information on the catchment and historic flooding within the catchment. The report in addition provides insight into appropriate hydrologic model calibration in the catchment and questions the voracity of the current rating curve for the Albion park (Sunnybank) continuous stage recorder. An alternate rating curve based on hydraulic modelling is proposed and a provisional rating curve was developed for the continuous stage recorder below the Princes Highway Bridge. Using these amended rating curves, a group of committee members were able to independently obtain good correlation between the discharges predicted by their various models and discharges recorded at the two gauges, lending weight to the reasonableness of the rating curves and calibrations adopted. Of particular value to the present study is the comprehensive dataset, collected at the time, on flooding in the June 1991 event.

LAWSON & TRELOAR

Lake Illawarra Flood Study (2001)

This study was undertaken for the Lake Illawarra Authority, Wollongong City Council and Shellharbour City Council. The study is confined to consideration of lake flood behaviour and does not address the behaviour of streams feeding into the Lake. With respect to Macquarie Rivulet, the study identifies limits of flooding due to elevated water levels in the lake for a range of events including the 1% AEP and PMF events. This flood behaviour, including a table documenting flood levels in the Lake near the Macquarie Rivulet mouth, is described in **Section 7**.

The study determined that the critical storm duration for the lake is 36 hours (using the standard Australian Rainfall & Runoff design flood estimation procedure). It was also found that flooding in the lake is controlled largely by the hydraulic constriction at the lake entrance near Windang, accordingly there is negligible spatial variation in lake flood level across the outlet of Macquarie Rivulet. The resulting peak 1% AEP lake flood level near the outlet of Macquarie Rivulet is reported as being 2.30 mAHD whilst the corresponding PMF flood level is 3.24 mAHD. Stage hydrographs are provided for the 1% and PMF design events.

Lake Illawarra Floodplain Management Study & Plan (2005)

This study was prepared for the Lake Illawarra Authority, Wollongong City Council and Shellharbour City Council to formulate and assess management options for reducing the impact of flooding in the vicinity of the lake. Hydraulic hazard was reassessed during this study and a Floodplain Management Plan prepared.

Riverine flooding due to flow in Macquarie Rivulet was outside the scope of this study, however options for dealing with flood problems due to lake flooding are considered. Flood levels derived from the Lake Illawarra Flood Study (Lawson & Treloar, 2001) are therefore of relevance to the current study and were incorporated as described in **Section 7**.

4 HYDROLOGIC MODEL ESTABLISHMENT

4.1 MODEL SELECTION

The Macquarie Rivulet catchment is a catchment of moderate complexity with some significant areas of substantial flood plain storage. There are several structures (bridges, culverts and basins) on streams within the catchment and diversions of flow can develop at several locations between sub-catchments. Accordingly, the hydrology model proposed must be able to simulate these real world complexities.

The hydrologic computer model selected for use in this study was the WBNM (Watershed Bounded Network Model) developed by Boyd, Rigby, and Van Drie. WBNM is a lumped parameter (runoff-routing) model that has been developed locally and used extensively in the Illawarra region. Detailed information with respect to the model can be obtained from the website http://www.uow.edu.au/eng/cme/research/wbnm.html

Successful calibration has been achieved using WBNM on many catchments similar to the study catchment. A high degree of confidence can therefore be ascribed to the models results. One of the model's particular strengths is its ability to model complex arrangements of storages and diversions, as exist in this catchment.

4.2 MODEL CONSTRUCTION

4.2.1 SUBAREA TOPOLOGY

WBNM model construction requires the overall catchment to be broken down into a number of sub-areas, each sub-area representing a discrete watershed region. Sub-areas were delineated using computer based GIS software in combination with the ALS ground surface data.

Initially the overall catchment was broken down into broad sub-catchments corresponding to the main arms of Macquarie Rivulet (as shown in **Appendix A.6**). Each of these arms was then further broken down into component sub-areas according to natural patterns of internal tributaries and the location of structures such as culverts, bridges, abnormal areas of floodplain storage and diversions. A Subareal plan of the model is reproduced in Figure 4.2.1 . Figure 4.2.1 is also reproduced at a larger scale together with more detailed layouts of sub-catchments in areas of greater complexity in **Appendix C.1**.

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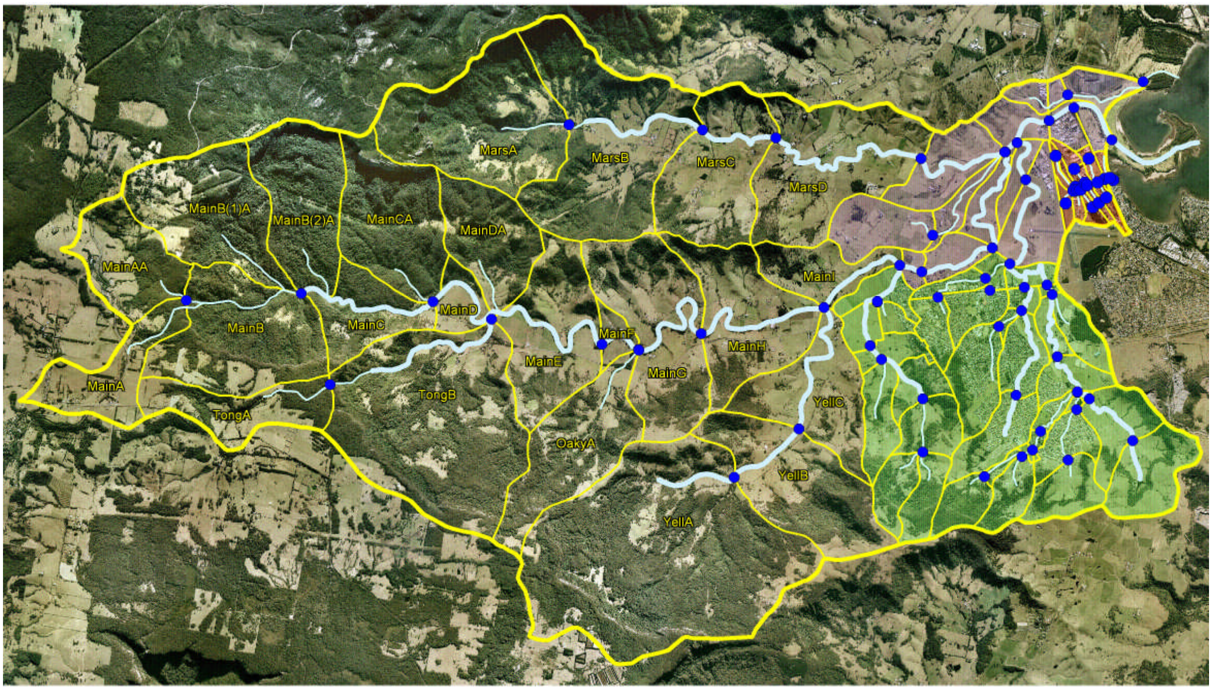

Figure 4.2.1` : WBNM Model Topology

In the final model, the catchment was broken down into a total of 95 sub-areas. In order to allow for identification of these sub-areas a node based naming convention was developed based on the Arm ID and an alphabetic character string.

Once naming was completed, the physical properties of each sub-area were calculated using MAPINFO. The area in hectares, co-ordinates of the outlet and co-ordinates of the sub-area centroid were each calculated in an automated manner using GIS techniques in a form suitable for direct input to WBNM.

4.2.2 CALCULATION OF IMPERVIOUS COVER

Impervious cover (i.e. the proportion of impermeable surfaces such as rooves, roads, bitumen and concrete in each sub-area), is a key physical parameter affecting both the quantum and timing of runoff from a catchment. Traditionally these values are estimated from zoning maps using typical percent impervious cover levels for each zone. While such an approach is quite straightforward once each zone is fully developed, mapping to a model subarea becomes difficult when zones are not fully developed, typically requiring further separation of the developed and undeveloped areas of each zone

In this study, GIS techniques were used to directly associate percent impervious cover with areas of different (hydrologic) land use identified on the aerial photographs and to allocate impervious cover to each model subarea. This involved a three step process as described below.

Step 1
This initial step involved the development of a base (hydrologic) landuse map from the high resolution aerial photography.

- In this step a hydrologic landuse plan was created identifying the spatial extents of each of the different hydrologic landuse classes. This is reproduced in Figure 4.2.2a and at a larger scale in **Appendix B.2**.

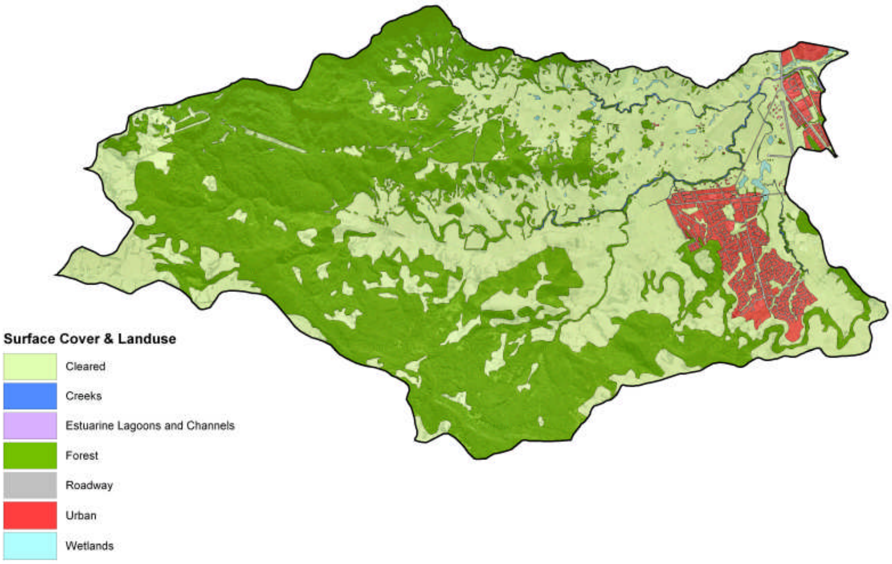

Figure 4.2.2a : Hydrologic Land Use Classes

Step 2
The next step involved the estimation of impervious cover values for each hydrologic landuse class.

- As a quite detailed review of impervious cover levels in the adjacent Horsley catchment had been recently completed involving both manual site sampling and several spectral analysis techniques, the values derived from this work were adopted in this study. These values are set out in Table 4.2.2 for each of the above classes. As commercial and industrial land uses were relatively limited in extent and vary across a wide range of impervious cover levels, they were lumped with residential land uses under a single 'urban' landuse.

Table 4.2.2: Adopted Impervious Cover Levels

Landuse Class	%Imp
Cleared grassland	0
Creeks (Ephemeral)	0
Estuarine lagoons/channels	100
Forested Areas	0
Roadway & paved areas	75
Urbanised areas	51
Wetlands (Ephemeral)	0

Step 3
This last step involved the use of GIS to allocate impervious covers to each subarea and to provide tabular input of subareal geometry and impervious cover to the hydrologic model WBNM.:

- In this step GIS software (MAPINFO) was first used to associate each land use with a particular attribute of impervious cover, as calculated/estimated in Step 2 above. The hydrologic model subarea's (as shown in **Appendix C.1**) were then overlaid and the impervious cover from each of the hydrologic landuses (contained within each subarea) used to calculate each subarea's overall impervious cover (from each component landuse). Tables were configured in the process to provide direct input of this information to WBNM.

The above methodology takes advantage of the excellent resolution of the aerial photography to reliably estimate different land uses and impervious cover for the catchment and removes any subjectivity from the estimation of the proportion of landuse within each subarea. A thematic map of the catchment is presented in .Figure 4.2.2b showing the calculated impervious cover levels for each subarea. The concentration of impervious cover in the eastern half of the catchment, around the Princes Highway and village of Albion Park, Is readily evident in this plot. .

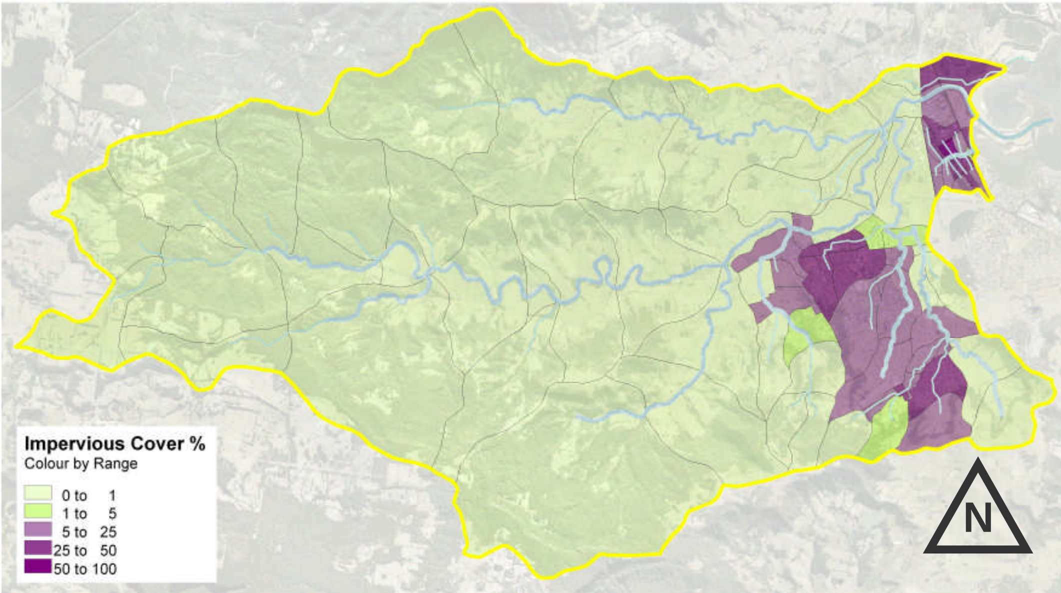

Figure 4.2.2b: Impervious Cover Within Each Macquarie Rivulet Sub-Catchment

4.2.3 BASINS

There are a number of small basins (retarding basins or ponds) in the Macquarie Rivulet catchment. Most are associated with recent development around the village of Albion Park, aimed at maintaining post development peak discharges to pre-development levels. As this study considers flooding in the mainstream of Macquarie Rivulet only, these small local basins have not been incorporated into present modelling.

4.2.4 AREAS OF (ABNORMALLY) HIGH FLOODPLAIN STORAGE

Most floodplains exhibit 'normal' storage characteristics and can be adequately described by implicit computation of their storage/discharge characteristics based on the overall catchment lag coefficient C. Occasionally areas of 'abnormally' high flood plain storage are encountered. These areas are normally visually obvious during a flood being typically much more extensive and deeper than floodplain storage elsewhere on the floodplain. If in doubt, the implicit storage associated with a given discharge can be compared with that determined explicitly (typically by hydraulic modelling). If there are significant differences, the hydrologic model should be built with explicit storages in the area of 'abnormal' storage.

Several sub-catchments were identified in the catchment of Macquarie Rivulet as containing abnormal' floodplain storages. These areas are shown in Figure 4.2.5 and at a larger scale in Appendix C3.

Since flow passing through these sub-areas is explicitly routed through their associated storages, no routing as provided implicitly by catchment lag is necessary. In sub areas with abnormally high flood storage, C is set to zero if the flood storage would cover most of the sub-area, eliminating both local overland and stream attenuation arising from implicit storage-lag. Where the abnormally high flood storage only covers a small part of a much larger sub-area, C is left at its default value and the watercourse factor reduced to zero. In this way only the implicit stream routing is eliminated.

4.2.5 DIVERSIONS

Potential for diversion of flow between various subareas was identified based on a combination of field inspection, detailed review of ALS/Aerial Photography, engineering judgement and feedback from preliminary hydraulic model runs. These potential diversion paths have been shown on the Hydrologic Model Diversions plan included in Figure 4.2.5 and at a larger scale in **Appendix C.3**.

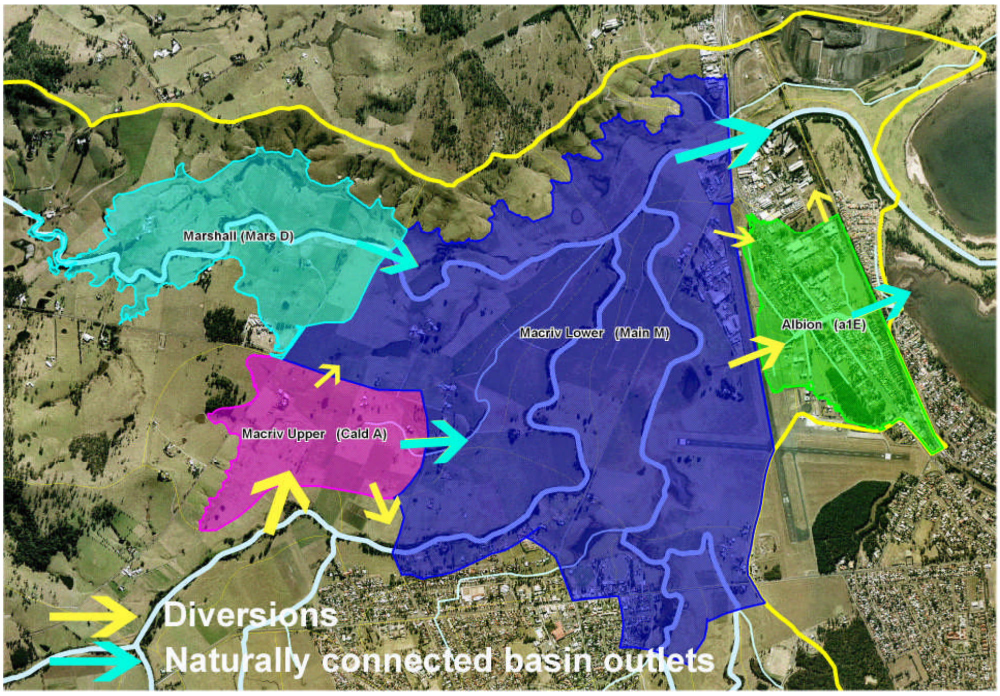

Figure 4.2.5: Areas of Abnormal Floodplain Storage and Diversions

The discharge characteristics of each diversion (variations in discharge to each downstream point with flood stage) were calculated using a combination of hand hydraulic calculations and hydraulic modelling. This resulted in stage/discharge relationships (rating curves) for each diversion that could be input to the hydrologic model's structure routines..

4.2.6 DESIGN RAINFALL

Three 'design' rainfall gauges were constructed for the catchment with names and locations as listed in Table 4.2. The locations of the three gauges were selected to describe the spatial distribution of rainfall across the catchment including, in particular, the significant east-west rainfall gradient. IFD characteristics for each gauge were derived using standard procedures as described in AR&R 1987.

Table 4.2 Location and description of design rainfall gauges

Design Gauge	Gauge Location (E,N)	Description of Location
Macriv #1	290046.30 6172300.00	On Macquarie Rivulet, in center of catchment
Macriv #2	285724.60 6172576.80	On Macquarie Pass, in catchment headwaters
Macriv #3	295050.00 6172240.00	In Albion Park Village, in lower catchment

The locations of these three 'design' gauges are shown in red on the Catchment Gauges plan reproduced in **Appendix A.7**.

4.2.7 ESTABLISHMENT OF HYDROLOGIC PARAMETERS

Hydrologic parameters required for input into the WBNM model runfile are described in Table 4.2.7 below along with a description and explanation for the values adopted. It is noted that some of these values were subsequently amended during calibration (refer **Section 6**).

Table 4.2.7 Hydrologic Model Parameters

Hydrologic Parameter	Description	Value Adopted
Catchment Lag 'C'	This parameter governs the lag time taken for rainfall to be converted to runoff for the natural (pervious) portion of a particular subarea. A quick response natural subarea has a short lag time whilst a slow response natural subarea has a long lag time. Extensive research carried out at the University of Wollongong (Boyd *et al*) has found that the value of C used to calculate the lag time is insensitive to most natural catchment characteristics.other than area (refer WBNM user documentation for further description).	The value adopted by this study for Catchment Lag 'C' is 1.3. It is noted that the current WBNM user guide recommends a general value be applied in the vicinity of 1.6 based on analysis of historic flow gauge data (generally this data was sourced from catchments external to the Illawarra). However, when coupled with stream roughness values applicable to streams in the Illawarra, better overall calibration has been previously achieved by the authors using the lower value of 1.3. This value has therefore been adopted in the initial hydrologic establishment phase.

Impervious Lag	The Impervious Lag factor is derived from the value of C (described above) and applied to impervious portion of a catchment. It represents a percentage of C (i.e. an Impervious Lag factor 0.1 means that impervious surface lag is only 10% of that from a natural pervious surface catchment). This accounts for the much faster response time from the unnatural (impervious) portion of a particular subarea.	The recommended value of 0.1 was adopted for this study. This value has been used in previous Illawarra studies for which successful calibration has been achieved and is recommended by the authors of WBNM.
Stream Lag	The stream lag is also derived from the value of Catchment Lag 'C' and is used for non-linear routing of flow through subareas containing a natural watercourse. The value of Stream Lag reduces with increasing velocity as would result from channel straightening and lining. The user guide recommends a value of 1.0 for natural streams, 0.5 for earth lined channels, and 0.33 for concrete lined channels.	The recommended values have been adopted for the current study. A value of 1.0 was applied to the majority of catchments as most have natural or semi natural features. Time delay of 0.5 minutes (i.e. no stream routing) was used on all dummy nodes and those subareas with a very small area (say 0.1ha).
Initial Storage levels	WBNM permits storages to be modelled with a starting water level other than completely dry (i.e. part full). This may be relevant where storages have a slow drawdown rate and/o where antecedent rainfall has partially filled the basins.	All storages were assumed dry at the commencement of modelling, reflecting the conditions known to prevail in the calibration model and as would likely apply prior to a long duration 36 hour design storm.

Losses Initial Loss (IL)	Initial losses represent the amount of rainfall lost to groundwater at the beginning of the storm. They closely reflect the 'dryness' of the catchment prior to the storm being modelled and soil characteristics (permeability and storage potential in particular). (shallow highly impermeable clays have a low IL while deep loamy or sandy soils have a high IL).	For model establishment and calibration an Initial Loss of 15mm was initially proposed reflecting a typical NSW catchment with limited lead up rainfall as per measurements by Cordery Web *(1974) (This was later increased to 150mm for the calibration event to better match the early stage of the recorded hydrographs at Sunnybank and the Princes Highway.)*
Continuing Loss Rate (CLR)	Continuing losses represent the amount of rainfall lost to groundwater during the storm after abstraction of initial losses. They closely reflect the soil types within a catchment (shallow highly impermeable clays have a low CLR while deep loamy or sandy soils have a high CLR).	A Continuing Loss Rate of 2.5mm per hour was used in modelling both the calibration and design events, reflecting typical values used in the Illawarra. It is noted that the predominantly clay based soils in the catchment have low permeability therefore a CLR of the order of 2.5mm/hour is considered appropriate.

4.2.8 PRELIMINARY MODEL CHECKS

The WBNM model runfile was initially constructed using the catchment data and parameters described in the previous sections. Prior to model calibration (refer **Section 6**), the following preliminary model checks were undertaken using a 9 hour, 1% AEP design rainfall burst:

Overall volume conservation:
The total runoff volume (as calculated at the catchment outlet) was checked against total rainfall volume (i.e total amount of rain falling on the entire catchment surface). As expected from a correctly constructed model, these two values were the same once the volume of rainfall lost to groundwater (and therefore not converted to runoff) was accounted for.

Unit discharge – from the local subarea:
The unit discharge from each subarea was calculated by dividing the local runoff (sum of the pervious and impervious peak discharges) from each subarea by its area to give a discharge per hectare rate. These values should all lie within a typical range for a 100Yr storm of 9 hour duration, of between 0.1 and 0.3 m^3/s/ha, with variation inside this range being due to spatial differences in rainfall and differences in area and impervious cover (larger sub-areas having lower unit discharges and more impervious sub-areas having higher unit discharges). As is apparent in Figure 4.2.8a these local unit discharges do reduce generally to the east. reflecting the significant east-west rainfall gradient. Some increase in local discharge levels does however re-develop in the lower rainfall zone, due to the presence of impervious surfaces and to a lesser extent, the smaller sub-areas present in this zone.

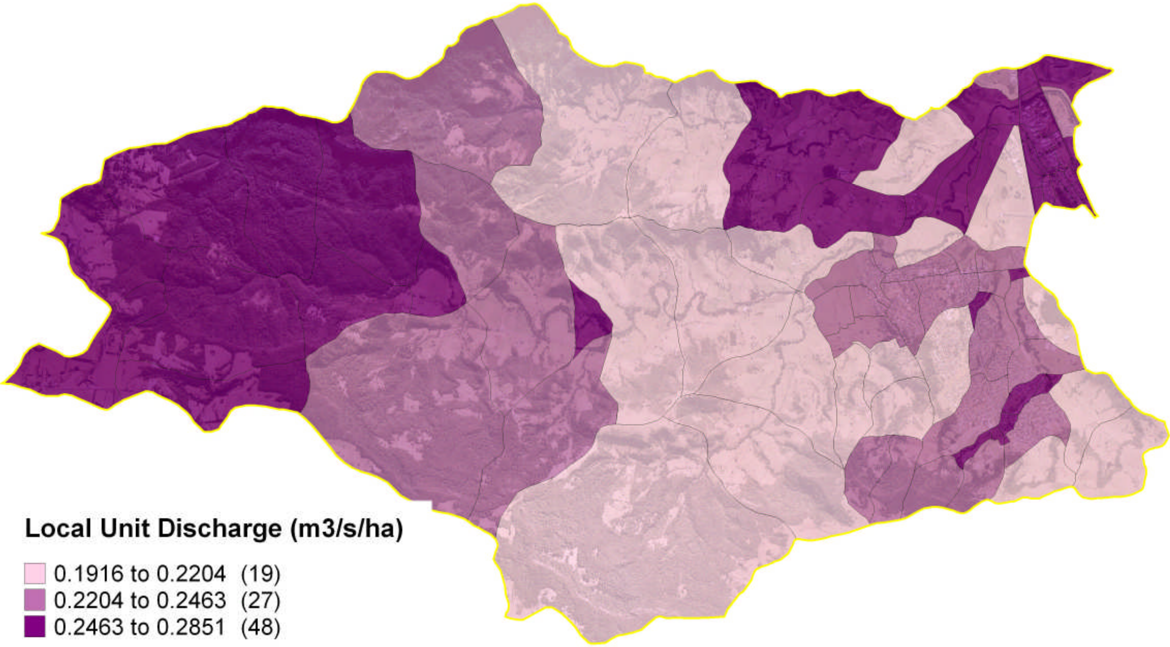

Figure 4.2.8a Peak Local Unit Discharge (m3/s/ha) 1% AEP Event

Unit discharge - cumulative

The cumulative unit discharge at each subarea's outlet was calculated by dividing the total flow at each outlet node by the cumulative contributing catchment area. These values must also lie within a typical range for a storm of this intensity and duration falling on a naturally connected catchment without abnormal storages (as previously, of the order of 0.1 to 0.3 m3/sec/ha).

Consideration of changes in the magnitude of cumulative unit discharge allow checks to be made on the modelled rainfall distribution, influence of stream routing, attenuation due to basin storage and increases in discharge due to diversion.

Subareas at the upstream (western) end of this catchment should (in a correctly constructed model) have higher cumulative discharge rates as rainfall intensities are higher and less stream routing has occurred. Subareas downstream of large storages should, all else being equal, have lower cumulative discharge rates due to attenuation of peak flow by these storages. Subareas receiving significant diversion should, all else being equal, have higher cumulative discharge rates (sometimes substantially above the natural range), since these subareas can receive a much greater inflow than the naturally connected upstream catchment would normally deliver.

The cumulative unit discharge patterns shown in Figure 4.2.8b reflect all these described factors, giving confidence to the model's construction. It is noted that the pattern is again dominated for the most part, by the rainfall gradient and areas of impervious cover. There are however some areas with very high cumulative unit discharges (dark blue colour). These are areas receiving diversions from adjoining catchments, producing higher discharges than would be expected in a natural topologically connected catchment.

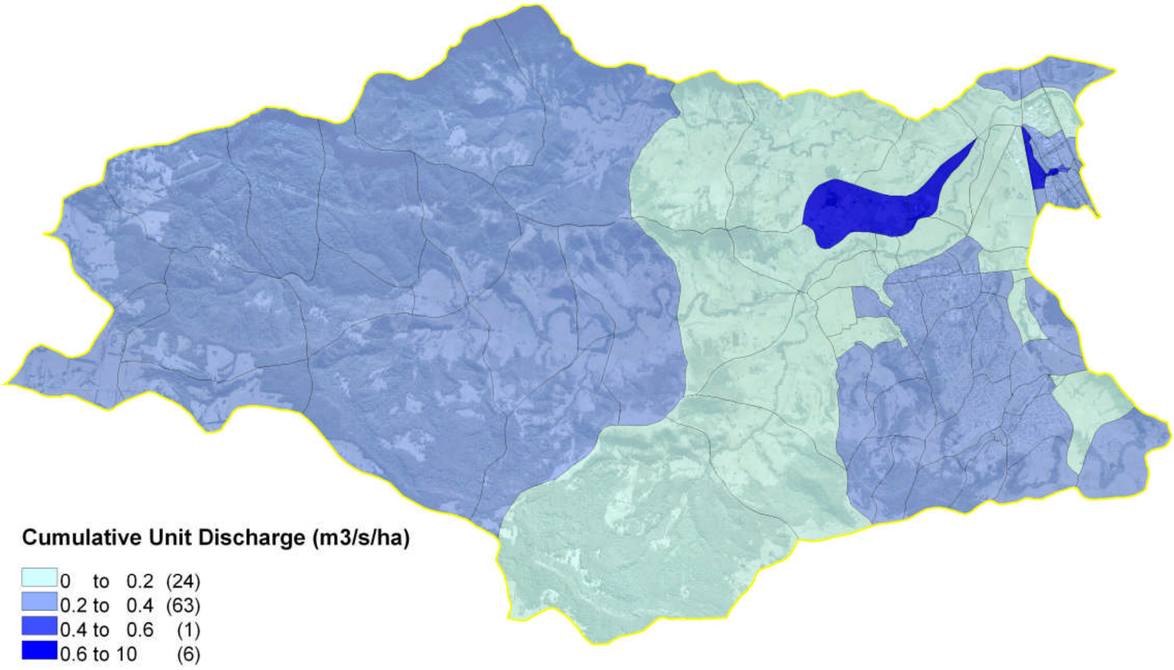

Figure 4.2.8b Peak Cumulative Unit Discharge (m3/s/ha) 1% AEP Event

5 HYDRAULIC MODEL ESTABLISHMENT

5.1 MODEL SELECTION

There are many computer based hydraulic models available today, that have been developed to specifically simulate surface flood behaviour. As noted by Rigby (2005), selection of an appropriate hydraulic model for a particular flooding problem is not trivial, as each of the available models has very distinct strengths and weaknesses. Selection is further complicated by the fact that selection must also consider user skills and possible future uses of the model.

In lesser flood events, Macquarie Rivulet behaves as a 1D system (i.e. flow is generally parallel to the stream centreline and the majority of flow within or in close proximity to the channel). Several diversions exist for larger flood events, but these diversion flowpaths are mostly well defined and also mostly 1D in nature (such as the diversion along the upstream edge of the railway embankment from Albion Creek back into Macquarie Rivulet). Flood behaviour in larger events, above the Princes Highway and below Albion park village, is however much more complex, involving flow patterns that are far from 1D in nature. In addition there are significant timing differences in peak flows between the various arms that merge on the floodplain above the Princes Highway road bridge, requiring hydrodynamic 2D modelling capabilities to properly quantify the impact of these temporal differences on flood behaviour.

On the basis of the above, It was concluded that a hydrodynamic 2D model would be needed to properly simulate flood behaviour in the lower reaches of Macquarie Rivulet. 2D models are inherently dissimilar to 1D models in that they do not pre-empt flood behaviour by confining flow to individual pre-defined stream 'arms', but instead calculate flow behaviour based on boundary conditions and a three-dimensional representation of the ground surface. Provided close attention is paid to development of an accurate surface representation, (an inherently strong feature of most ALS based 2D models), then very high quality can be achieved in flood modelling. Diversion flowpaths can then be identified directly from 2D flow surface, rather than requiring the modeller to infer behaviour based on a comparison of predicted flood surface with ground elevations.

The model selected as being most suited to the present hydraulic modelling needs, was Tuflow, a grid based two dimensional hydrodynamic model developed by Mr W Symes of WBM Oceanics, an Australian company. Tuflow has been successfully applied on many similar studies in New South Wales and has significant application both in Australia generally and in several overseas countries, particularly the UK.. Tuflow permits incorporation of a wide range of hydraulic structures such as culverts, bridges, weirs and pipe systems into a model and can handle both subcritical and supercritical flows . Further more specific details of the model can be obtained from the website http://www.tuflow.com .

Tuflow is in addition supported by the SMS 9.2 modelling environment, providing powerful support for both construction of models and viewing of results from a Tuflow simulation. The Environmental Modelling Systems Inc website www.ems-i.com provides further information on this product's capabilities.

5.2 MODEL CONSTRUCTION

5.2.1 GENERALLY

Both Mapinfo and the Surface water Modelling System v9.2 (SMS) were used to facilitate construction of the Tuflow hydraulic model. Mapinfo is a spatial systems modelling tool produced by the Mapinfo Corporation (USA) and SMS is a hydraulic model building environment produced by the Environmental Modelling Research Laboratory (EMRL) at Brigham Young University. The Tuflow interface was developed by the EMRL in cooperation with the developer of Tuflow, Mr W Symes.

A 7km by 6km 2D grid (domain) based on a 10m cell was chosen to simulate flooding in the area of interest, this being a compromise between model resolution and simulation run time. Since the chosen cell resolution provided a good representation of the waterway of Macquarie Rivulet in larger events, within the study area, the linked 1D (Estry) capabilities of Tuflow were not incorporated.

An active zone for the model (a subarea of the 2D grid, within which cells would be included in computations) was initially established by creating a boundary outside of the physical floodplain using the ALS data and aerial photography to define this boundary. This initial active zone boundary was further refined once a Tuflow PMF run had confirmed the 'active' model extents. The model domain boundary and active zone boundary are reproduced in Figure 5.2.1 and at a larger scale in **Appendix D1**.

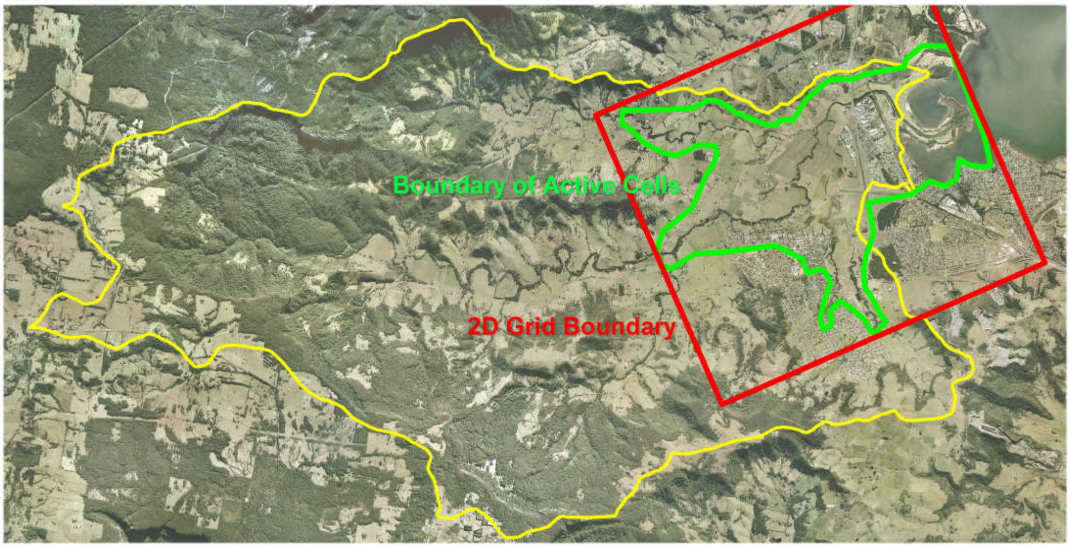

Figure 5.2.1: Model Grid & Active Zone Extents

5.2.2 SURFACE ROUGHNESS

Areas having different properties with respect to impedance of flow (roughness zones) for channel and overbank floodplain areas were identified using Council's zoning data in conjunction with the 2005 high resolution (orthorectified) aerial photography.

Zoning data provides a logical basis for discretization of roughness zones since the physical properties of urban land surfaces are correlated with their usage. For example roads (smooth) and residential areas (rough) have very different roughness properties nevertheless the boundary between them is well defined in the zoning data..

It is noted however that within each Council zone there may be areas of non-typical roughness for that zone. For example, open space zoned land can comprise mown grass (low roughness) or dense forest (high roughness). In addition, roughness can vary within a given zone (for example a mix of grassed areas and forest in an open space zone Council's zoning data therefore required further segmentation to account for these differences. This was achieved by reference to aerial photography and digitization of non-typical areas using tools available in SMS. The various surface roughness zones applying within the active zone of the model are reproduced below in Figure 5.2.2.

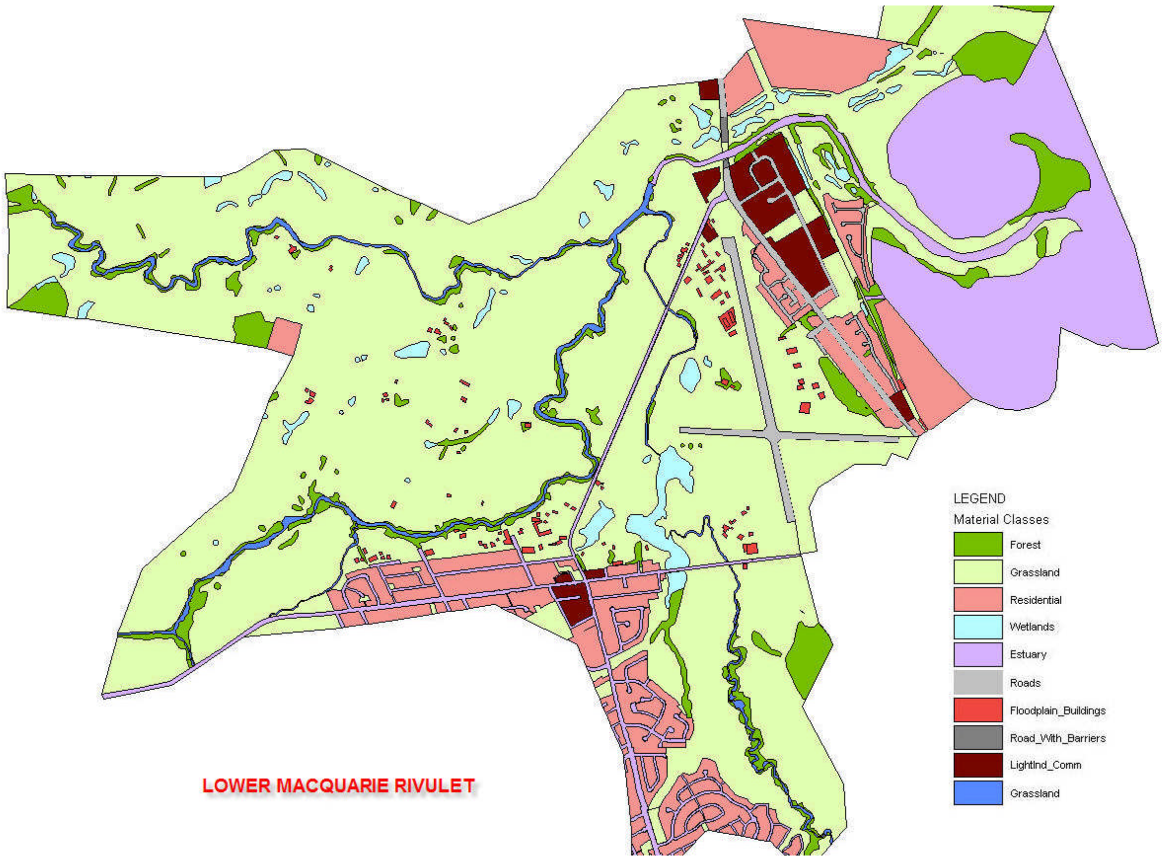

Figure 5.2.2 Model Roughness Zones

Once the surface roughness zones were categorised and mapped, each of the roughness zones was ascribed roughness characteristics. The values initially used for model establishment were derived from consideration of various industry reccommendations (including Chow (1959), Hicks *et al* (1991) and Arcement *et al* (1984)), further adjusted to account for local conditions and RIENCO's experience in previous 2D modelling in the Illawarra. For roughness zones such as residential and light industrial where industry consensus values are difficult to find, field inspection and hand calculation of the percentage of flow path blocked by large roughness elements (buildings and fences etc) were used to scale values of roughness for these partly-blocked zones.

The values initially used by the model were adjusted as part of the calibration process described in **Section 6**. Adjusted values have been tabulated along with other adopted hydraulic design parameters in **Section 7**.

It is noted that slightly lower roughness values from those used in 1D modelling are appropriate in a 2D model, as the 2D model directly accounts for viscosity, whereas 1D models only incorporate viscosity effects by including them into a slightly higher 'n' value.

5.2.3 TOPOGRAPHY

The topography of the model was built by projecting the merged (ALS and bathymetry) elevation scatterset onto the model grid using tools available in Vertical Mapper (a companion product to Mapinfo).

Once the grid had been elevated in this manner, the model surface was contoured and inspected to confirm that the contoured model surface was comparable with that of the merged scatterset from which it was derived.

The contoured model surface, shown reproduced in Figure 5.2.3 and at a larger scale in **Appendix D2**, was found to be a good representation of the original merged scatterset surface within the study area.

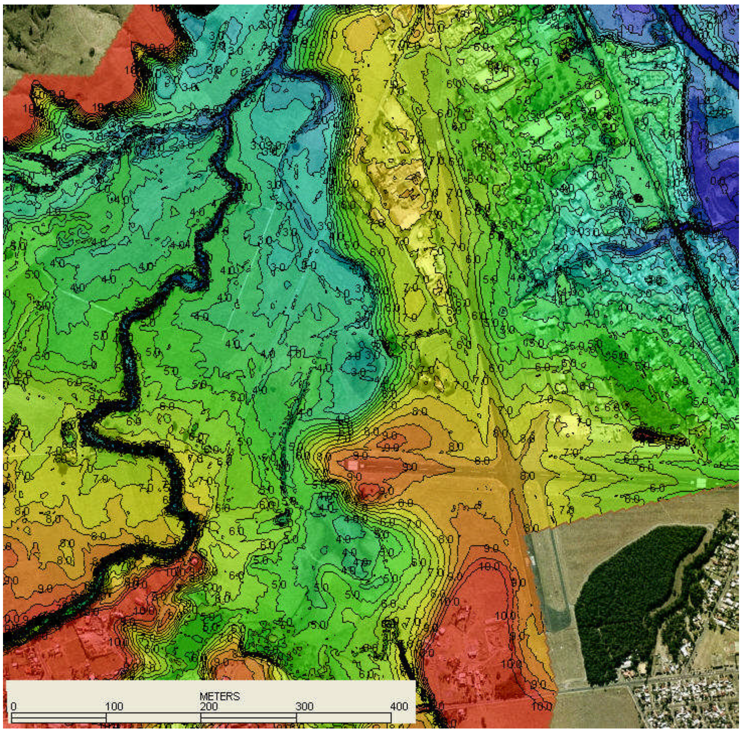

Figure 5.2.3: Model Topography

5.2.4 STRUCTURES

All significant structures, including road and railway culverts and bridges, within the study area were incorporated into the model. Geometric details of structures were obtained from existing survey data, augmented by survey undertaken by registered surveyor David Yates, as described in **Section 3**.

The Rail and Princes Highway bridges were modelled as 2D structures. In this form, the model surface through the structure is that of the creek and overbank areas between abutments. Additional data is provided for the bridge obvert level (at which pressure flow commences) and the overtopping geometry (at which weir flows commence over the bridge) No expansion or contraction losses are required as the 2D model explicitly accounts for losses associated with flow transitions and misalignment. Some minor form losses are however typically required to reflect pier losses. The recommendations of the User Manual were followed in this regard.

All other structures were modelled as 1D elements in which the surface through the structure is that of the embankment or road over, with flow between cells upstream and downstream of the embankment calculated using empirical equations as described in the User Manual.

All structures were initially modelled fully clear (i.e. no blockage).

5.2.5 TRUNK DRAINAGE NETWORK

There is no trunk drainage network in the study area, As such a linked 1D/2D model of the underground network was not required in this model.

5.2.6 BOUNDARY CONDITIONS

Two classes of temporally variable boundary conditions were applied in this model.

1. Inflow hydrographs (both on the upstream boundary and internally to the model)
2. Tailwater levels (at the models downstream boundary (Lake Illawarra))

The locations at which these various boundary conditions were applied are shown graphically in **Appendix D5**.

These are discussed further in **Section 6** (Model Calibration) and **Section 7** (Design Flood Hydrology)

5.2.7 PRELIMINARY MODEL CHECKS

Once constructed, the preliminary hydraulic model was run with 1% AEP and PMF flows assuming all structures to be clear. The primary purpose of these initial runs was to confirm model stability and general numerical sensibility. Where data was available, a comparison was made with previous model results to confirm that the preliminary model for the current study was providing results within the expected range.

Once preliminary model checks were completed the model was then calibrated in accordance with the methodology described in **Section 6**.

6 MODEL CALIBRATION AND VALIDATION

6.1 CALIBRATION & VALIDATION METHODOLOGY

Model calibration is required to confirm the ability of a hydrologic/hydraulic model to simulate recorded flood behaviour from recorded rainfall. Calibration involves building a model using recorded data from a historic storm and comparing predicted results with recorded results, then making adjustments to the model(s) (if necessary) to minimise differences. It was identified during the data collection phase of the current study that reliable data for such calibration was available for the June 1991 event. This information included;

- Historic flood levels from the event of June 11th 1991 in the form of continuous stage records (CSRs) at two gauges, Maximum Height Indicator (MHI) readings at two locations and a number of peak flood levels obtained by survey of debris and flood marks left after the event.

- Water levels for lake Illawarra over the relevant time period

- Coincident June 1991 rainfall data at five minute intervals from five rainfall gauges within the catchment,

- Co-incident information with respect to the model catchment conditions, level of development and presence of infrastructure in the floodplain at the time of the event

Using this data, hydrology and hydraulic models were constructed, run and their results recorded. Where justifiable, model parameters were reviewed and adjusted to ensure closeness of fit between modelled and recorded discharges and flood levels. It is of importance to note that the presence of CSRs in the catchment permitted the calibration of the hydrologic and hydraulic models to proceed independently. This (rather rare in the Illawarra) circumstance all but eliminates concerns as to whether the coupled hydrologic/hydraulic models are producing a good event simulation as a consequence of opposing errors in the calibration of each model.

6.2 CALIBRATION EVENT OF JUNE 1991

6.2.1 GENERALLY

Over the period 6th to 14th June 1991, a low pressure system moving south from Queensland, and a high pressure system fairly stationary in the Tasman Sea directed humid easterly air onto the South Coast of NSW, causing heaving rainfall over a 6 day period. This heavy rainfall caused minor to moderate flooding throughout the Wollongong area, particularly in the Lake Illawarra/Macquarie Rivulet catchments.

As previously noted, rainfall during this event was recorded at five gauges (5minute resolution), flood levels were recorded at two CSRs , six MHIs and at several points from survey of flood marks left after the event. While the CSRs now operating in Lake Illawarra had not been installed at this time, several readings were made of lake stage by the SES at different times, providing reasonable definition of the variation in Lake stage during the event. Flood marks, surveyed around the Lake after the event, confirm the sensibility of these observed Lake levels.

6.2.2 AVAILABLE DATA

The rainfall event was recorded by 5 gauges located within the catchment.

- Macquarie Pass(Glover Hill) (MHL)
- North Macquarie (MHL)
- Yellow Rock (MHL)
- Upper Calderwood (MHL)
- Albion park (SW)

A hyetograph of rain falling during this event at Macquarie pass and an analysis of burst IFD for the same gauge is reproduced in Figure 6.2.2a. Comparable plots for all gauges operating during event in the catchment, are reproduced in **Appendix B5**.

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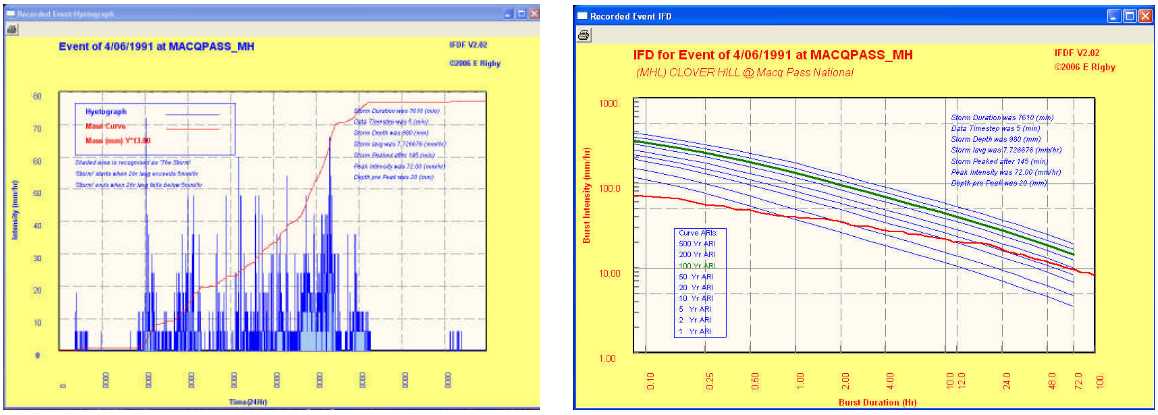

Figure 6.2.2a: Hyetograph & IFD –Macquarie Pass - June 1991

The variation in flood stage during this event was captured at the Albion Park (Sunnybank) CSR and also by the Princes Highway CSR. Traces for both gauges are reproduced below in Figure 6.2.2b and at a larger (full page) scale in **Appendix B6** and **Appendix B7**. The lake stage hydrograph based on SES observations is also included on the Princes Highway Gauge trace. The discharge hydrograph at each gauge, based on the WRF rating curves, is also included on each plot.

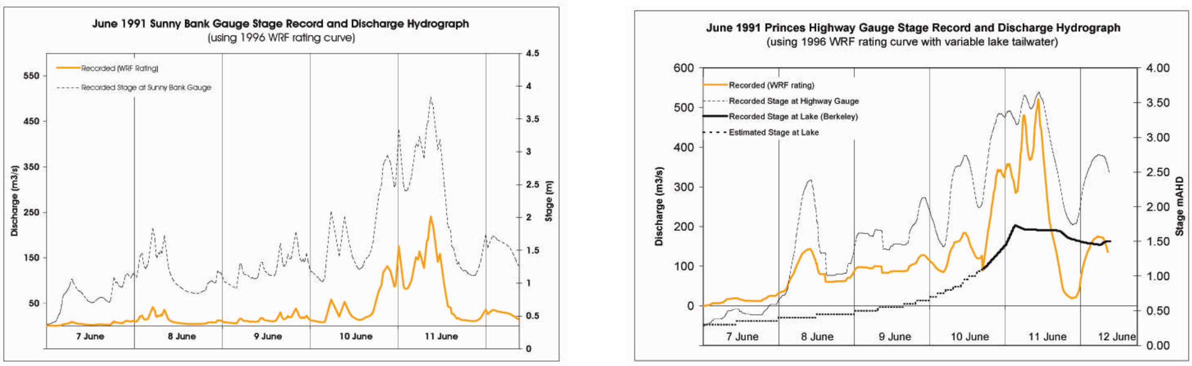

Figure 6.2.3b: Albion Park and Princes Highway CSRs – June 1991

6.2.3 MODEL CALIBRATION

Using the collected data, hydrologic and hydraulic models were constructed in order to represent flood behaviour within the study area during the June 1991 event.

The 'core' hydrologic model was modified to incorporate rainfall data recorded during the event as measured at the five gauges. Model parameters and losses were initially as described in **Section 4**. During the calibration process it was found that while the hydrologic model was simulating peak discharges well, predicted flows at the commencement of the event were far too high. Initial loss was therefore increased until the early discharges matched those recorded. This required an increase in initial loss for the June 1991 event to 150mm.

A comparative plot of the hydrographs as derived from stage records and simulated at each of the two CSRs is reproduced in Figure 6.2.3a. and at a larger scale in **Appendix E1** and **Appendix E2**. Given the excellent correlation in magnitude, timing and slope of the rising and falling limbs of the hydrographs at the two stations, no change was made to any of the other model parameters.

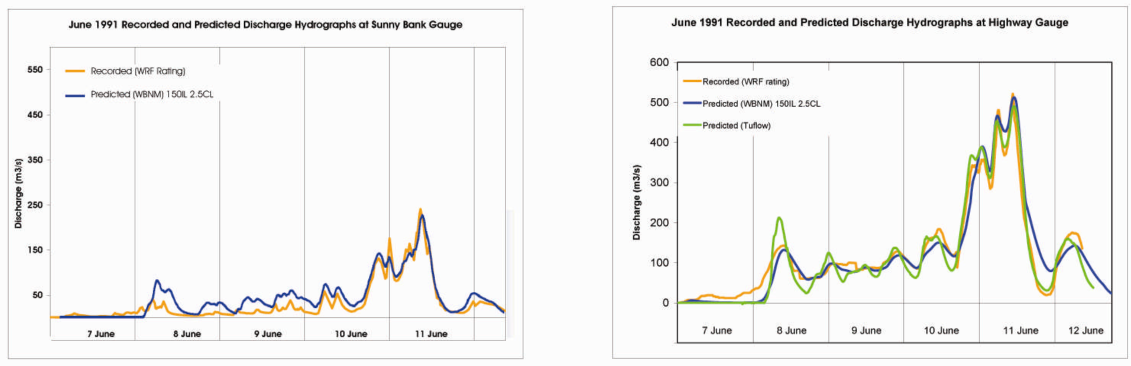

Figure 6.2.3a : June '91 Hydrographs – Albion Park and Princes Highway CSRs

The adopted June 91 hydrologic model parameters were then;

Initial Loss (IL perv)	150mm
Initial Loss(Il Imp)	1mm
Cont Loss (CL:R)	2.5mm/hr
Catcment Lag (C)	1.3
Imp Lag Factor	0.1
Stream Lag Factor	1.0
Init Storage Levels	Dry

Peak discharges for the June 1991 event extracted from the calibrated hydrologic model at previously modelled locations are then;

Table 6.2.3a: Peak Discharges at Selected Locations –June 1991event

Location (WBNM 2007 Subarea)	Qp PSxRM WRF 1996	Qp WBNM WRF 1996	Qp WBNM Rienco 2007	Gauged WRF rating
Sunnybank Gauge Site (MainE -Top)	290	270	245	241
Macriv @Mansons Bridge (MainJ)	420	370	253[1]	-
Marshal @ Grey Meadows (MarsD)	120	90	103	-
Frazers @ Tongarra Rd (FrasG -Top)	90	80	97	-
Princes Highway Gauge Site (MainM)	575	516	513	520

1. *Older models did not incorporate a diversion from the floodplain above Manson's bridge to the north back into Macquarie Rivulet downstream of the bridge (of the order of 180m3/sec).*

Following calibration of the hydrology model, the 'core' (present conditions) hydraulic model was modified to incorporate known changes to the surface topography and roughness between June 1991 and the present. These modifications to the 'core' (present conditions) hydraulic model included;

- Lowering of the airport runway back to the levels present in 1991.
- Removal of fill and reconstruction of an area on the northern bank of Macquarie Rivulet, downstream of the Road and Rail bridges, to reflect conditions prevailing in June 1991 (altered post 1991 during construction of a new subdivision in this area).
- Removal of a road embankment and culvert on Albion Creek, downstream of the Illawarra Rail crossing, to reflect conditions in June 1991 (altered post 1991 during construction of a new subdivision in this area).
- Reclassification of some residential areas to open space, that were not yet developed in 1991.
- Reclassification of a land parcel on the south bank of Macquarie Rivulet upstream of the Princes Highway to reflect conditions prevailing in June 1991 (now a developed nursery)

No other modifications to the hydrodynamic model were made in the initial run.

The simulated event stage/discharge hydrograph at the Princes Highway CSR, is presented in Figure 6.2.3a . In general, good agreement was achieved throughout the study area in the first run, except for a few locations where simulated levels were consistently higher or lower than the recorded levels. Minor adjustments were then made to the model surface roughness values, in these areas, to improve the fit.

The surface roughness values finally adopted are summarised in Table 6.2.3b. In this table n1 and n2 are the roughness values at depths D1 and D2 respectively. Below a depth of D1, n1 applies and above a depth D2, n2 applies. Between D1 and D2 the surface roughness varies linearly between n1 and n2.

Table 6.2.3b Adopted Surface Roughness Values

Roughness Zone	D1(m)	n1	D2(m)	n2
Grassland: Slashed or grazed 0.1m stubble	0.10	0.075	0.50	0.020
Treed Areas: Equal resistance at ground & depth	1.00	0.075	5.00	0.075
Paved Areas: Open areas of roads & runway	0.05	0.030	0.25	0.015
Urban Res: Residential areas with solid fences	0.90	0.150	4.50	0.100
Creeks: Flat to moderate grade some vegetation	0.50	0.075	2.50	0.040
Estuary: Flat grade sandy bed minimal veg	0.05	0.025	0.25	0.015
Wetlands: Ponded shallow reedy area few trees	0.50	0.075	2.50	0.020
Urban LIC: Urban Light industrial & commercial	1.50	0.150	7.50	0.100
Road Bars: Roads with barriers to flow over	0.70	0.050	3.50	0.020
Rock Weir: Section of stream with rock weir	1.00	0.200	5.00	0.050
Buildings: Individual buildings on floodplain	1.00	10.00*	5.00	10.00*

** A nominally large number to all but block flow through such elements.*

Simulated peak levels and differences from recorded levels for the 'calibrated' hydraulic model in the study area are shown in Figure 6.2.3b and at larger scale in **Appendix E.3**. This plot shows the difference between the simulated flood level and recorded flood level on the aerial photo underlay as a coloured bar, located in a calibration target in which the upper and lower limits of the target represent the recorded data confidence interval. Simulated levels falling within one confidence interval are shown in green. Simulated levels falling beyond one confidence interval are shown in orange.

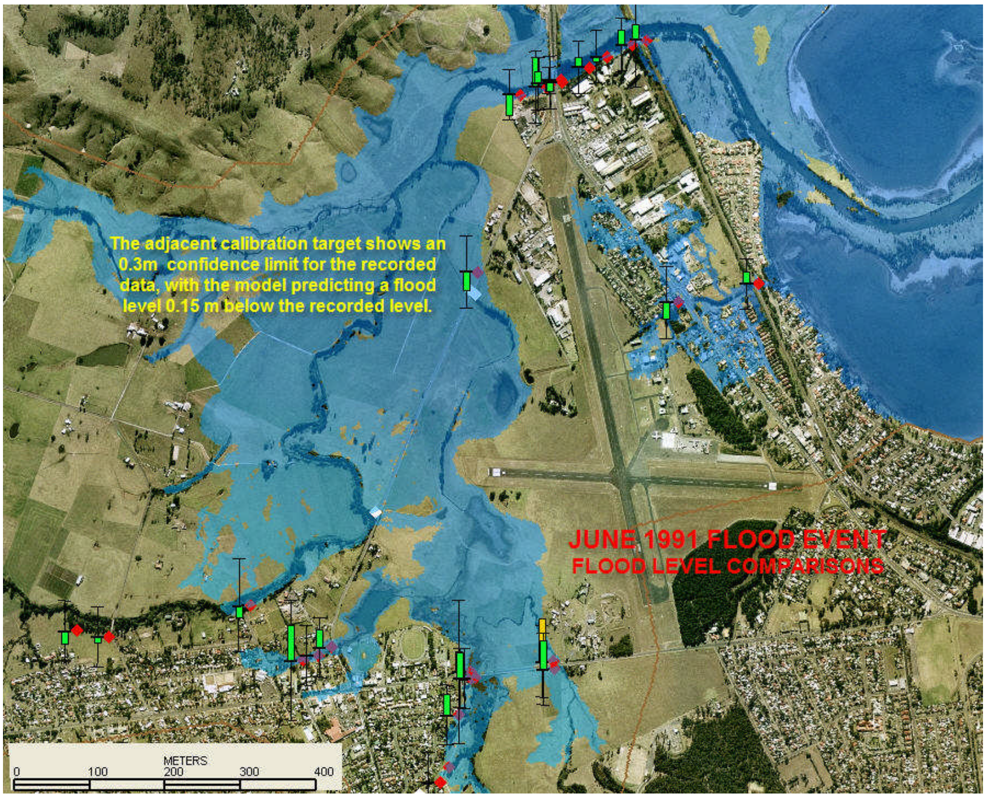

Figure 6.2.3b : Recorded –v- Simulated Peak Flood Levels – June 1991

In setting these confidence limits, two factors in particular were considered;

- The likely difference between the actual flood level and level of the point surveyed
- The likely difference between the simulated level and actual level due to positional differences

With respect to likely error due to the type of data record, three record classes were defined;

- CSR and MHI records - assigned a base confidence limit of 0.1m
- Wall Stains and debris on the flood fringe – assigned a base confidence limit of 0.2m
- Debris in trees or on fences- assigned a base confidence limit of 0.3m

Likely differences due positional misalignment of the model and record dataset were set at;

- 2m if a fixed, well defined visible location
- 10m elsewhere although directionally limited by the description if related to a particular feature (such as a fence line).

In setting individual confidence limits for each recorded flood level, the confidence limit was increased from the base 'class' value by the difference in flood surface level applicable due to likely positional differences. As is apparent in the target heights, this resulted in a variation in confidence limits for the different data classes and locations that varied from about 0.1 to 0.5m.

Given that simulated flood surface levels are all within one confidence interval of the recorded data with no higher or lower trends present, overall calibration against the June 1991 flood event is considered very good, giving confidence to the hydrologic/hydrodynamic models' ability to predict flood behaviour in a moderate flood event. Although this does not guarantee predictive accuracy for much larger events, both the hydrologic and hydrodynamic models are based on widely accepted parameter values (used for other similar catchments in the Illawarra), therefore reasonable results for larger events can be expected.

6.3 MODEL VALIDATION

As only one event dataset was considered sufficiently comprehensive and reliable for calibration/validation purposes, an independent validation run was not possible.

Given that the event did however engage most of the lower floodplain of Macquarie Rivulet and its tributaries and the hydrologic and hydraulic models were able to be independently calibrated, the concerns normally associated with extrapolation of event magnitude in coupled hydrologic/hydraulic models are not as significant as they might otherwise be.

We are therefore confident that the model can and will provide an accurate simulation of flood behaviour in a 1% AEP event and be as accurate as is ever the case in a PMF event.

7 DESIGN FLOOD ESTIMATION

7.1 DESIGN FLOODING CONSIDERATIONS

7.1.1 DESIGN RAINFALL

Standard hydrologic modelling procedure as set out in AR&R (1987) calls for the selection of a critical storm 'burst' duration that maximises discharge at a particular location. This procedure assumes that a critical 'burst' of given ARI will, all else being average, produce a flood peak discharge of comparable ARI. This approach does not account for two major differences between real storms (from which this procedure was derived) and design bursts:

1) Initial losses applicable to a short duration design burst are unknown (often assumed zero)

2) Short duration design bursts can not account for the effects of lead up rainfall on stream flows and accumulated flood storage, at commencement of the design event.

To overcome these concerns, the approach proposed and adopted for this study is to embed a design burst within a much longer 'envelope storm'. A 36 hour AR&R design storm was selected as an envelope storm since this is within the duration range of typical flood producing events in the region and also corresponds to the duration of the design storm event used in modelling carried out for Lake Illawarra as described in the Lake Illawarra Flood Study (Lawson & Treloar, 2001). This then provides a logical basis for selecting an appropriate tailwater condition where Macquarie Rivulet enters the lake, since the same overall design storm duration is used on both the lake and Macquarie Rivulet catchments (refer **Section 7.1.4** for further discussion).

The Embedded Design Storm (EDS) procedure for embedment of a burst within an envelope storm has been described by Rigby et al (1996, 2003, 2006). The EDS procedure described involves positioning the burst within the storm such that the peak of the burst and peak of the envelope storm coincide. The burst is used to replace the storm where they overlap. In order to maintain the overall volume of the envelope storm, the wings of the envelope storm outside the burst are lowered on a pro-rata basis.

The paper by Rigby *et al* (2003) points out that where a catchment's critical duration is much less than the duration of typical flood producing storms in the area, and a catchment's critical burst duration is less than about six catchment lags, then the standard AR&R 'burst only' design flood estimation procedure will begin to underestimate peak flows when compared with historical storms of similar frequency and peak intensity..

The EDS procedure "'warms up' a catchment in a logically derived manner, eliminating for the most part, potential errors introduced by the AR&R design procedures lack of recognition of the influence of lead rainfall on burst response" (Rigby *et al*, 2003).

Application of the EDS procedure will also generate a more realistic hydrograph with a duration similar to that associated with historic storms, This provides more realistic estimates of periods of overtopping or inundation than those obtained from the AR&R 'burst only' based procedure.

Also the following should be noted with respect to the EDS procedure:

- The burst duration producing minimum error when embedded into a storm is often shorter than the critical burst duration that would normally be derived using the standard AR&R approach. This occurs since the embedded storm lead rainfall is sufficient to create part full storages prior to the burst. In the standard AR&R approach, burst duration often needs to be extended before flow can be maximised as runoff is still filling storages.

- PMP bursts cannot be embedded within PMP envelope storms of duration greater than 6 hours since the techniques for Probable Maximum Precipitation using the Bulletin 53 Generalised Short Duration Method (GSDM) extend only to 6 hours. Therefore the PMF embedded design storm used for the current study was a 36 hour, 500 year ARI storm built from standard AR&R IFD and temporal pattern data. It is noted that the effect of this envelope of smaller size (greater frequency) with respect to the burst can be ignored since the results are relatively insensitive to the volume of the envelope storm once a storm is sufficiently large to fill storages prior to the peak.

- It is acknowledged that the overall embedded design storm concept does not fall within current standard industry procedures and is not documented in the current version of Australian Rainfall & Runoff which is essentially a re-printed version of the Institution of Engineers 1987 publication. However, AR&R is currently undergoing major revision and it is understood that recommendations with respect to embedded design storms will be included in the next release (expected some time around 2008). It was however felt that given the considerable benefits of using the embedded design storm procedure in any catchment, that the current study should be proactive and apply the procedure to ensure its currency and validity is maintained after the imminent AR&R revision.

To establish the 'critical' burst duration to embed, a series of 1% AEP bursts of varying duration were embedded in a 1% AEP 36 hour envelope and peak flows in the vicinity of the study area compared. A burst of 9 hrs, embedded in the 36 hour envelope was found to maximise flows within the study area.

7.1.2 ANTECEDENT CONDITIONS AND STORM LOSSES

It is noted that when using the AR&R bursts based procedure for design flood estimation, a value of 0 mm is typically used as an initial loss. However the use of an embedded design storm requires use of higher (realistic) initial losses. For the purpose of hydrologic modelling of 'design' events a 15mm initial loss of rainfall was adopted based on initial losses recorded by Cordery & Webb (1974) in real events across NSW.

A 2.5mm per hour continuing loss was applied to rainfall for the duration of the storm, reflecting both a typical value for continuing loss used in design modelling in the Illawarra and the value obtained from calibration of the June 1991 storm event.

7.1.3 BLOCKAGES AND DIVERSIONS

The current study has incorporated the effect of culvert and bridge blockage on design flooding by directly modelling the obstruction caused by blockage at key structures. This required the establishment of blockage levels (percentage of opening) to be applied to the study. In consultation with Shellharbour Council and the Horsley Creek Floodplain Risk Management Committee the blockage levels set out in Table 7.1.3 were agreed as applicable to the Horsley Catchment. As Shellharbour Council has not yet developed a formal policy applicable to catchments within its LGA, The Horsley Creek Design Blockage levels have been adopted in this study.

Table 7.1.3: Blockage levels applied to this study

Type of Structure	Blockage Level Assumed
Multiple inlet trunk systems where combined street drainage inlet capacity >>> barrel capacity	assumed clear (0% blocked) throughout the event
Smaller culvert/bridge systems (<1500mm diagonal opening or with either height or width less than 1000m)	assumed 100% blocked at peak of storm
Larger culvert bridge systems (>= 1500mm diagonal opening)	assumed 20% blocked at peak of storm

As well as the level of blockage, the modelled pattern of blocked structures across a catchment must also be considered. Where blockage of a particular structure influences the size of a diversion occurring at the structure, blockage will increase flood levels along the diversion flowpath (and any receiving downstream reach), whilst at the same time decreasing flood levels in the directly connected reach immediately downstream of the structure. Two alternate blockage patterns must therefore be modelled as a minimum: 'all clear'; and 'all blocked'. Where there are two or more structures within adjoining catchments linked by diversion, a combination of clear and blocked structures or 'mixed' blockage pattern may also need to be considered. In highly complex catchment systems, it is possible that more than one mixed blockage pattern will need to be investigated since some patterns cannot be modelled concurrently. However when establishing the mixed blockage patterns to be modelled, consideration needs to be given towards the likelihood of the particular pattern being considered to avoid improbable combinations.

For design event modelling in the present study, the 'all clear' and 'all blocked' patterns only were modelled.

Results from the all clear and all blocked condition were used to establishing envelope values within the catchment. Flood levels, velocities, depths, unit flow and the NSW FPDM hazard surfaces presented in this report represent the maximum modelled results from these two blockage models, for each characteristic.

7.1.4 LAKE LEVEL

Flooding can occur in the lower reaches of Macquarie Rivulet (the area generally downstream of the weir in Macquarie Rivulet) via two mechanisms:

- Lake flooding due to elevated water surface levels in Lake Illawarra
- Riverine flooding due to elevated water surface levels in Macquarie Rivulet

Whilst both modes of flooding (lake and riverine) can and will occur to some extent during a single storm, it is unlikely that:

1) Both the lake and creek would experience maximum flood levels of the same ARI during a single event
2) That these maximums would occur at the some point in time.

This reflects the disparate hydrologic characteristics of Lake Illawarra (large catchment, slow response) and Macquarie Rivulet (smaller catchment with faster response).

For the purpose of design event modelling, a temporally variable tailwater condition needs to be established for the downstream boundary of the hydrodynamic model. Given the low probability that lake and riverine flood levels would peak at the same instant in time, a methodology is needed that reflects this temporal separation.

A tailwater condition in Lake Illawarra was therefore adopted that could be 'expected' to occur simultaneously with flooding in Macquarie Rivulet. This involved a procedure in which;

1) The Lake Illawarra stage hydrographs for the 1% AEP and PMF events (as modelled by Lawson & Treloar), were obtained. These differ slightly from those used in the Lake Illawarra Flood Study, as the lake model was upgraded by Lawson & Treloar following completion of the flood study.

2) For each design event modelled, the lake hydrograph, of equivalent ARI to the modelled stream event, was applied as a dynamic boundary condition along the downstream boundary of the hydraulic model domain. The temporal relationship between the lake and stream hydrographs was established by aligning the peak of the (36hr) storm causing Lake Flooding with the peak of the (36hr) Embedded Design Storm being applied to the Macquarie Rivulet catchment. This is considered a conservative approach as differences in the rainfall mechanisms causing peak lake flooding of a given ARI and stream flooding of the same ARI would probably not lead to events of comparable ARI simultaneously in both systems. It is however likely that heavy rainfall on the Lake and its contributing catchments would coincide with similarly heavy rainfall on the Macquarie Rivulet (itself part of the contributing catchment to the Lake).

In Figure 7.1.4, the supplied lake stage hydrograph is plotted on the same base as the Macquarie Rivulet stage and discharge hydrograph for the 1%AEP event at the Princes Highway road bridge. This plot shows the relationship of peak runoff in Macquarie Rivulet with Lake stage, highlighting the temporal separation of flood peaks between the two systems.

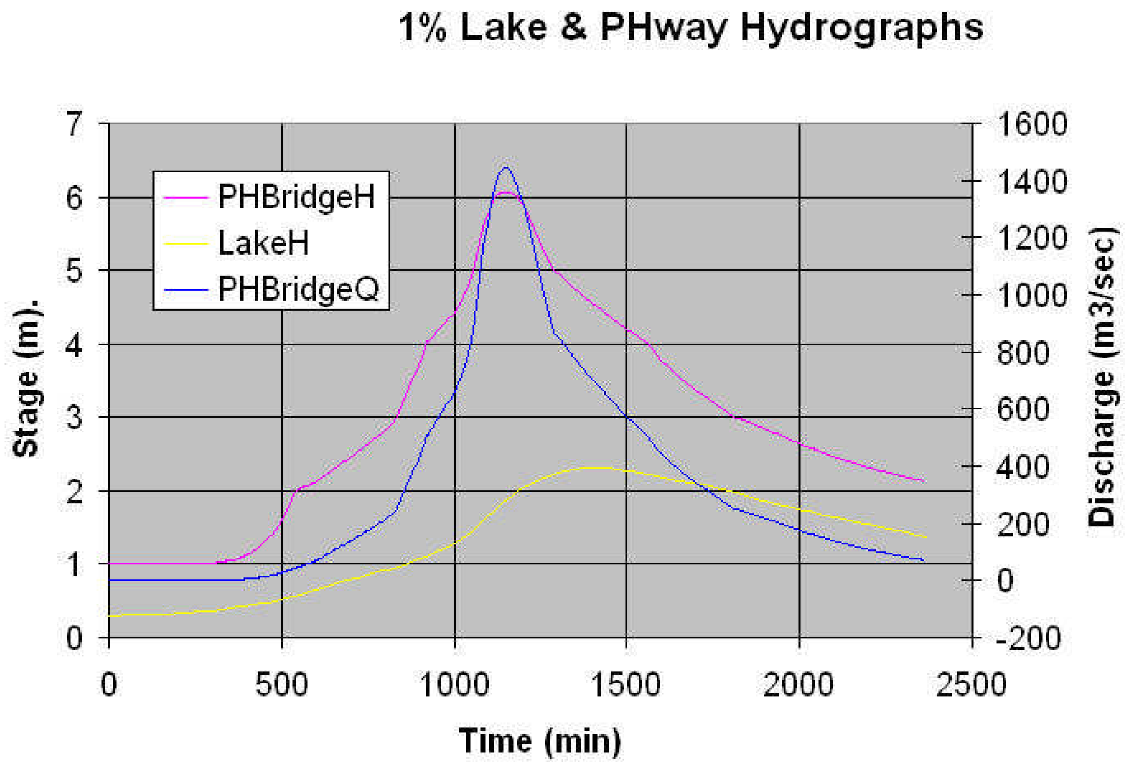

Figure 7.1.4: Lake Illawarra and Macquarie Rivulet 1%AEP Hydrographs

It is noted that the positioning of the Macquarie Rivulet hydrograph relative to the Lake Illawarra hydrograph was facilitated by the use of the same envelope storm duration as the critical storm applied to Lake Illawarra (i.e. 36 hours). It was simply assumed that the two storm hyetographs commenced at the same time, allowing the appropriate lake level at any point in time to be directly read from the Lake hydrograph. This is a transparent and logical approach which can be repeated for studies in other creeks affected by lake flooding. This approach is analogous to both the lake and inflowing catchments responding to a storm with similar temporal patterns.(a likely occurrence).

It is again noted that it would be unlikely that in an event that maximises flow in Macquarie Rivulet, that lake flooding would also be maximised. The lake stage hydrograph in Figure 7.1.4 for example would most likely peak at a level significantly lower than that shown in a 1% AEP event in Macquarie Rivulet.. The approach adopted, which assumes the lake's hydrograph is at least on its way towards an event of comparable severity is therefore considered conservative.

7.2 DESIGN FLOOD HYDROLOGY

7.2.1 ADOPTED HYDROLOGIC DESIGN PARAMETERS

This section provides a summary of adopted hydrologic design parameters used to establish design peak discharge estimates within the catchment. Equivalent details for the hydraulic model are presented in **Section 7.3**.

As described in **Section 7.1** an Embedded Design Storm (EDS) approach was adopted for the current study. A 'critical duration' burst was embedded within an envelope storm, with both the burst and envelope storms being derived from standard AR&R IFD data and temporal patterns.

The 'critical' burst duration was selected as 540 minutes based on an analysis of all standard AR&R durations between 1 hour and 12 hours. The 9 hour (540 minute) burst embedded within the 36 hour envelope storm was found to generate maximal discharge in the mainstream of Macquarie Rivulet adjacent to and downstream of the proposed development site.

This 540 minute burst was embedded within a 36 hour envelope storm as described by Rigby et al (1996). The 36 hour storm envelope was selected since this event is the same duration as the event used for modelling flooding in Lake Illawarra and is within the range of typical flood producing events in the region and can therefore provide a realistic lead up rainfall condition.

A total of five design gauges were used, with locations as shown on the plan included in **Appendix C.1**. Figure 7.2.1 below shows the 1% AEP design EDS (9hr in 36 hr) hyetograph at the design gauge named "Macriv#1" located in the central part of the catchments. Other gauges have the same temporal pattern but are based on slightly different IFD data reflecting spatial variation in this data across the catchment.

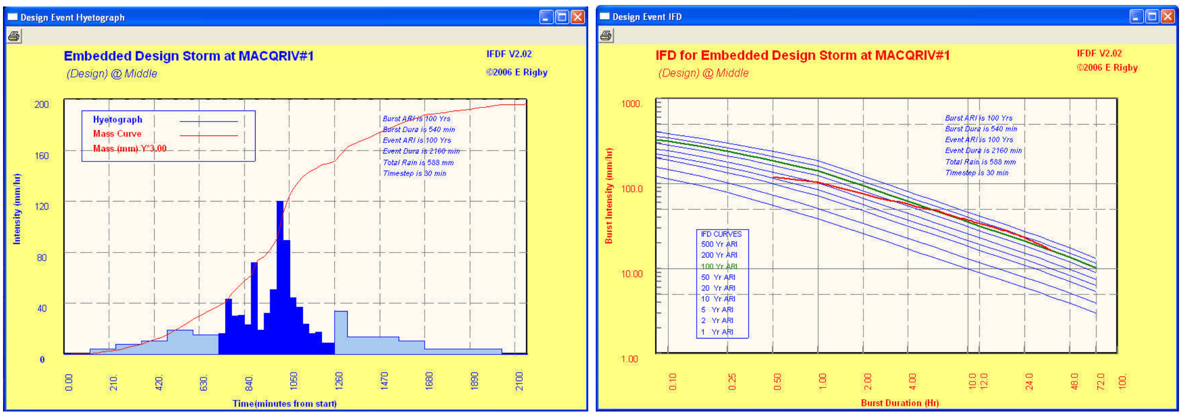

Figure 7.2.1: Design EDS Hyetograph & IFD at Design Gauge "Macriv#1"

All storages in the model were assumed to be empty at the beginning of the event. It is noted however that through use of the EDS design rainfall procedure, storages will contain some flood water prior to the AR&R burst. The exact volume of floodwater will have been calculated in a logical manner by the model and will account for the specific hydrologic and hydraulic characteristics of each storage and their contributing catchments.

The following Table 7.2.1 provides a summary of other adopted hydrologic design parameters used to establish design peak discharge estimates within the catchment. Further description of each parameter and its role is provided in **Section 4.2.6**.

Table 7.2.1 Adopted Design Hydrologic Model Parameters

Hydrologic Parameter	**Value Adopted**
Catchment Lag ‘C’	1.3 globally
Impervious Lag	0.1 globally
Stream Lag	1.0 Natural Stream condition 0.5 Grass Swales 0.33 Roads and Concrete Channels Time delay (no routing) of 0.5 minutes for all small sub-areas (~0.1ha)
Losses Initial Loss (IL)	15 mm
Continuing Loss Rate (CLR)	2.5 mm per hour

7.2.2 HYDROLOGIC MODELLING RESULTS

The following Table 7.2.2 provides a summary of design peak discharges for the 1% AEP and PMF events at locations of interest within the study area. These peak discharges are not coincident in time therefore the times at which these peaks occur (relative to the commencement of rainfall) have also been provided.

Table 7.2.2: Summary of Design Peak Discharges at Key Locations

Location	**Peak Discharge (m^3/s)**		**Time to Peak (min)**	
	1%AEP	**PMF**	**1%AEP**	**PMF**
Sunnybank Gauge Site (MainE -Top)	713	1402	1018	1121
Macriv @Mansons Bridge (MainJ)	818	1712	1106	1149
Marshal @ Grey Meadows (MarsD)	280	623	1074	1120
Frazers @ Tongarra Rd (FrasG -Top)	191	423	1055	1090
Princes Highway Gauge Site (MainM)	1443	1443	1101	1145

7.3 DESIGN FLOOD HYDRAULICS

7.3.1 HYDRAULIC DESIGN PARAMETERS

While the topographic and roughness data are those applicable in 2005 (not those applying in 1991), all other hydraulic model parameters were as discussed in previous sections and applied in calibration modelling.

7.3.2 HYDRAULIC BOUNDARY CONDITIONS

Boundary conditions applied in design event modelling were;

- The downstream lake stage hydrograph for a 36 hour storm event

- The various total and local inflows to the model from a 36 hour storm event

7.3.3 HYDRODYNAMIC MODELLING RESULTS

The following Table 7.3.3 provides a summary of design flood levels for the 1% AEP and PMF events at locations of interest within the study area. The values included in the table represent the maximum levels occurring at each location, from the two blockage scenarios considered, 'all clear' and 'all policy blocked'.

Table 7.3.3: Design Peak Flood Levels at Selected Locations

Location	**Flood Level (m AHD)**	
	1%AEP	**PMF**
Macquarie Rivulet DS of the Illawarra Rail Bridge	4.15	4.70
Macquarie Rivulet US of the Illawarra Rail Bridge	4.35	5.00
Macquarie Rivulet DS of the Princes Hway Road Bridge	5.45	6.50
Macquarie Rivulet US of the Princes Hway Road Bridge	6.00	7.50
Macquarie Rivulet @ Frazers Creek/ Junction	6.45	8.10
Macquarie Rivulet @ Rear Green Meadows Dairy	6.50	8.10
Frazers CK @ West End E/W Runway	6.50	8.10
Frazers Ck 300m DS of Tongarra Rd (at U bend)	6.90	8.15
Frazers Ck DS of Tongarra Rd	8.50	8.50

Note Unless noted otherwise the tabulated US/DS levels are immediately US or DS of the nominated structure, on the centreline of the stream.

Contours of peak flood elevation, flood depth, flood velocity and flood unit flow (V*D), throughout the study area, are reproduced in **Appendices G1 to G8**.

8 FLOOD BEHAVIOUR

8.1 GENERALLY

As shown in **Appendix G2**, the western half of Lot 6, south of the east-west runway and most of Lot 6 north of the east-west runway would be inundated in a PMF flood. The northern corner of Lot B would also be inundated in a PMF flood. Both lots are therefore partly 'Flood Prone' as defined in the Flood Plain development Manual (FPDM 2005). AS indicated on **Appendix G1**, Most of Lot 6 north of the runway remains inundated in a 1% AEP event. South of the runway, Lot 6 remains inundated in the western section, but to a lesser plan extent. Lot B is free of flooding in a 1%AEP event.

8.2 FLOOD HYDRAULICS

As is readily apparent in the graphics describing the distribution of unit flow (VxD) across the study area (**Appendix G7** and **Appendix G8**) flow patterns through the floodplain to the west of the proposed development sites are complex, varying in velocity, depth and direction across the floodplain and from event to event. In general however, flows bordering the proposed development sites are relatively low in velocity (<1m/sec with adjacent peak flood levels relatively constant along this border RL 6.5m AHD in 1% AEP event, RL 8.0m AHD in a PMF event). Areas of recirculating flow occur along this border. north and south of the end of the east-west runway. Some concentration of flow from Frazers Creek occurs where it is forced to flow out and around the western end of the east-west runway, slightly raising velocities in this area. In a 1%AEP event the flood surface adjacent to the low point in the runway is at or about the level of the runway. In a PMF event however, the runway would be substantially overtopped (approx 1.7 m deep at the sag), with large quantities of flow leaving Macquarie Rivulet at this location to flow into the headwaters of Albion Creek.

8.3 FLOOD HAZARD

An assessment of the distribution of provisional hydraulic hazard across the study area is presented in **Appendix G9** (1%AEP) and **Appendix G10** (PMF). These plots reflect the FPDM 2005 provisional hydraulic hazard categories set out in Figure L2 of the manual. The plotted provisional hazard categories are based on the most severe hazard category (from the combination of instantaneous velocity and depth consideration), occurring throughout the event.

As shown in **Appendix G9** (1% AEP), the relatively pronounced ground slopes at the edge of the floodplain adjacent to the proposed development sites prevents the development of a zone of low and/or transitional hazard of any significance. Most of the inundated portion of the sites therefore presents as a 'high' provisional hydraulic hazard.

It is of relevance to note that both Tongarra Road and the Princes Highway become inundated by floodwater and impassable for a period of a few hours, at least once every few years. .In such events the proposed development sites and Airport become an island of higher ground, effectively surrounded by floodwater, with implication for effective access and egress.

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