Shoalhaven City Council

Environment - Growth - Community

ST GEORGES BASIN FLOOD STUDY

Badgee Bridge, Sussex Inlet, 1991

SEPTEMBER 2001

SHOALHAVEN CITY COUNCIL

ST GEORGES BASIN
FLOOD STUDY

October, 2001

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ST GEORGES BASIN
FLOOD STUDY

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FOREWORD

The State Government's Flood Policy is directed at providing solutions to existing flooding problems in developed areas and to ensuring that new development is compatible with the flood hazard and does not create additional flooding problems in other areas.

Under the Policy, the management of flood liable land remains the responsibility of local government. The State Government subsidises flood mitigation works to alleviate existing problems and provides specialist technical advice to assist Councils in the discharge of their floodplain management responsibilities.

The Policy provides for technical and financial support by the Government through the following four sequential stages:

1. *Flood Study*
   - determine the nature and extent of the flood problem.
2. *Floodplain Management Study*
   - evaluates management options for the floodplain in respect of both existing and proposed development.
3. *Floodplain Management Plan*
   - involves formal adoption by Council of a plan of management for the floodplain.
4. *Implementation of the Plan*
   - construction of flood mitigation works to protect existing development,
   - use of local Environmental Plans to ensure new development is compatible with the flood hazard.

The St Georges Basin Flood Study constitutes the first stage of the management process for St Georges Basin and its catchment area. It has been developed by the Shoalhaven Floodplain Management Advisory Committee and prepared for the Committee by Webb, McKeown & Associates to define flood behaviour under current conditions.

SUMMARY

St Georges Basin has a catchment area of approximately 327 square kilometres and lies wholly within the boundaries of Shoalhaven City Council. The area of the Basin itself is approximately 37 square kilometres. Flooding of roads and residential areas within the catchment has occurred on a number of occasions in the last 20 to 30 years corresponding with the storm events listed below. Several flood events are also known to have occurred in the 1950's.

This report was prepared by Webb, McKeown and Associates on behalf of Shoalhaven City Council, and details the hydrologic and hydraulic investigations carried out to determine the design flood levels. It represents the first step in the process to provide a formal Floodplain Management Plan for the area.

All available rainfall, flood and topographic data were collected and analysed as part of the study. It is known that significant storm events occurred in:
- March 1959,
- October 1959,
- February 1971,
- June 1991.

Minor storms also occurred in:
- May 1953,
- February 1958,
- March 1961,
- March 1975,
- March 1976,
- October 1976,
- February 1992,
- September 1993,
- April 1994.

Whilst there are some good flood data around the Basin and in Sussex Inlet for the 1971 and 1991 events, the historical flood record within the broader catchment is generally poor. The approach adopted for the hydrologic and hydraulic modelling of the catchment was influenced by the quality and quantity of data available.

The WBNM hydrologic model was set up to cover the entire catchment draining to St Georges Basin, as well as the Basin itself. The RUBICON hydraulic model was structured to model the main waterways within the designated study area, which encompasses the Sussex Inlet Channel, the Basin and its fringe area as well as the lower reaches of the major tributary creeks.

The hydrologic and hydraulic models were calibrated against available historical data to ensure that they accurately simulated recorded floods. For both models parameter values from established texts and those found to be applicable in previous studies were used in determining appropriate values for the present study.

Design rainfall data were determined from Australian Rainfall & Runoff 1987 and input to the hydrologic model to define flow inputs to the hydraulic model. Design flood levels were obtained by inputting the "still" water design flows and boundary conditions to the hydraulic model.

The accuracy of the design flood levels at any one location is largely dependent on the availability of reliable historical flood data, the survey data, and the accuracy of the design rainfall intensities. The accuracy of the "still" water design flood levels established in this study are considered to be ±0.3 m for Tomerong Creek, the Basin and low lying foreshore areas as well as Sussex Inlet. For the reaches of the other tributary creeks upstream of the Basin influence the accuracy is more likely to be in the order of ±0.5 m due to the paucity of data available for model calibration and verification.

The potential influence of wind wave effects on design flood levels around the Basin foreshore can be significant depending on the conditions prevailing for a particular location. In order to provide some quantification of the possible increase in flood levels due to wind wave effects, a preliminary analysis was undertaken for three alternative sites situated around the northern foreshore of the Basin. The results suggest a wave runup value of somewhere between 0.1 m and 0.6 m should be added to the "still" water design flood levels to minimise the risk of inundation for new foreshore development. Further consideration of this issue is required when establishing appropriate Flood Planning Levels during the Floodplain Management Study/Plan phase.

1. INTRODUCTION

St Georges Basin is a coastal lagoon within the Shoalhaven City Council area (Figure 1). The Basin has a surface area of approximately 37 square kilometres discharging through the Sussex Inlet Channel to the Pacific Ocean at Bherwerre Beach. The total catchment area to the Pacific Ocean is approximately 327 square kilometres. The Basin itself therefore represents approximately 10% of the total catchment.

A number of properties surrounding the Basin are very low lying and flooding has caused damage in the past. Many residents of the area provided flood level data for the more significant flood events in recent times, which were the floods of February 1971 and June 1991.

In the last 30 years the land usage around the Basin has undergone significant changes, from a predominantly rural community, to a community with significant pockets of urbanisation. The town of Sussex Inlet in particular has undergone massive changes, mainly due to the development of canal estates which commenced in 1971. There has been a significant increase in population and a heightened awareness of environmental issues. These changes have already affected the Basin and there is the potential for further changes. In view of these factors it is necessary to define the existing flood problem and carefully manage future development on the floodplain.

Shoalhaven City Council engaged Webb, McKeown & Associates to prepare a Flood Study as the first stage in the development of an overall Floodplain Management Plan for St Georges Basin.

The primary objectives of this Flood Study are:

- to determine the flood behaviour of St Georges Basin and its tributaries under existing conditions,
- set up numerical models of the catchment to determine flood flows, velocities and levels for the design and extreme flood events,
- to formulate the models such that the effects on flood behaviour of catchment development and future flood mitigation options can be investigated,
- provide increased “flood intelligence” to assist the SES in preparation of emergency plans.

This report details the results and findings of the Flood Study investigations. The key elements are:

- a summary of available data,
- reasons for the choice of hydrologic and hydraulic models,
- calibration of these models,
- establishment of design flood behaviour.

The Flood Study does not consider local flooding which may result from inadequate urban drainage provisions in more frequent rainfall events.

A number of related studies have been carried out in the local area and a compilation of previous reports is contained in Reference 1. All levels in this report are to Australian Height Datum (AHD), which is the common national plane of reference approximating mean sea level.

2. BACKGROUND

2.1 Study Limits

The study area for this investigation (refer Figure 1) incorporates the Sussex Inlet Channel, the fringe area around St Georges Basin, and the lower reaches of the major tributary creeks. The tributaries are bound approximately by Sussex Inlet Road, the Princes Highway, The Wool Road, Jervis Bay Road and Naval College Road. A breakdown of the total catchment area (indicated on Figure 9) is shown in Table 1.

Table 1: Catchment Areas [1]

Tributary	Catchment Area	
	(km^2)	%
Cow Creek	13.1	4.0
Tullarwalla Creek	18.3	5.6
Wandandian Creek	159.3	48.7
Pats Creek	6.4	2.0
Home Creek	4.6	1.4
Tomerong Creek (also referred to as Cockrow Creek)	42.8	13.1
Worrowing Waterway	5.9	1.8
Erowal Creek	2.6	0.8
Stony Creek	2.7	0.8
Basin and non-tributary Fringe Area [2]	56.5	17.3
Sussex Inlet Creek and Channel [3]	14.9	4.5
TOTAL	**327.1**	**100.0**

Notes:
(1) Catchment area contributing to the Basin except where noted below.
(2) Incorporates Actual Basin surface area to Sussex Inlet channel and smaller foreshore tributary areas surrounding the Basin.
(3) Residual area below Basin.

2.2 Catchment Description

A significant part of the upper catchment is rural land which has been largely cleared of natural vegetation. It is mainly used for hobby farm activities. The lower slopes in the vicinity of the Basin contain a number of centres of urban development, including Sussex Inlet, Wandandian, Bewong, Basin View, St Georges Basin, Sanctuary Point, Old Erowal Bay, Erowal Bay and Wrights Beach.

The Basin is an attractive feature in the local area and a major drawcard for the tourist industry. Any reduction in the aesthetic quality of the Basin would have a significant impact on the local area.

The average bed level of the Basin varies from approximately -6 mAHD west of Garden Island to approximately -10 mAHD east of Garden Island. The deepest bed level in the Basin is approximately -11 mAHD, occurring approximately 2 kilometres south of Sanctuary Point (refer Figure 8).

The Sussex Inlet channel links St Georges Basin to the ocean at Bherwerre Beach. It is approximately 6 km long, and ranges between 50 m and 300 m wide. The tidal range varies by approximately 1.8 m at the ocean entrance end of the channel and is then dampened to only 0.2 m within the Basin.

There are some areas where sand has built up and is impeding flow through the channel, particularly on the eastern side between the Basin and Neilson Road, on the western side between Neilson Road and Jacobs Drive, and in the middle of the channel near the ocean entrance. According to some of the local residents, the sand bar at the entrance can make boat access difficult and is believed to have been worsened by boats dumping ballast at this point many years ago.

2.3 Previous Studies

Previous investigations of the area include the St Georges Basin Estuarine Management Study and the Sanctuary Point Flood Study.

St Georges Basin Management Study: Stage 1 - Estuarine Processes (1993)
This study by Webb McKeown (Reference 1), represents the first stage of a three stage process to determine a Management Plan for St Georges Basin. It incorporates a comprehensive appraisal of hydrodynamic processes, sedimentation, water quality, ecology and management issues.

St Georges Basin Draft Estuary Management Plan (1996)
The series of Estuary Processes and Management Study investigations culminated in the development of an overall Estuary Management Plan for the St Georges Basin (Reference 8). Extensive community consultation was involved to assist in identifying the various estuary management issues and objectives to be addressed. Management strategies including planning controls, physical works, monitoring and education were developed, costed and prioritised for implementation.

Sanctuary Point Flood Study (1992)
This flood study by Lyall and Macoun (Reference 2), investigated flooding for the portion of Sanctuary Point village bordering Cockrow Creek (Tomerong Creek catchment). The study area extended upstream along Cockrow Creek from its outlet at St Georges Basin as far as Council's Sewage Treatment Plant. The study objectives were to:

- identify the causes of flooding,
- establish the effects of existing filling activities and other works on flooding and drainage,
- assess the feasibility of measures to improve drainage of the local catchments and to mitigate flooding from the direction of Cockrow Creek,

- in the catchment upstream of Robinsville Place, consider the use of retarding Basins to improve the situation in the short term and to cater for future possible development,
- establish within the limits of available hydrologic data, the frequencies of the major flood of February 1971 and the more recent floods of June 1991 and February 1992,
- establish peak water surface levels along Cockrow Creek for a range of flood frequencies up to the 100 year ARI, as interim values until a major flood study of the whole St Georges Basin catchment has been carried out.

Following the establishment of flood behaviour within the study area, several preliminary mitigation measures were investigated. Provision of a box culvert and drainage improvements for Sanctuary Point Road were recommended for immediate action with other measures to be investigated further.

2.4 Causes of Flooding

Flooding within the study area may occur as a result of a combination of the following factors:

- an elevated Basin level due to intense rain over the catchment. The Basin level rises when the rate of inflow to the Basin is greater than the outflow to the ocean. The Sussex Inlet channel can act as a significant constriction to outflows,
- elevated water levels within the individual creeks as a result of intense rain over the local catchments. The levels in the creeks may also be affected by an elevated Basin level or by constrictions along their lengths,
- local runoff over a small area accumulating in low spots. Generally this occurs in areas which are relatively flat with little ground slope to facilitate drainage. The problem may be compounded by inadequate local drainage provisions and elevated Basin levels at the downstream outlet of the urban drainage (pipe, road drainage) system,
- elevated ocean levels. Generally elevated ocean levels occur as a result of storm surge (from a low pressure system) in combination with increased wave activity,
- local wind conditions generating waves and setup across the fetch of the Basin.

These factors may occur in isolation or in combination with each other. In particular, the combination of high tides, strong winds (typically onshore easterly to south-easterly but also westerly) and peak inflows to the Basin are considered to be significant. Some local residents have reported that during the 1971 flood, levels at the eastern end of the Basin were 0.5 m higher than at the western end due to the effects of wind stress across the fetch of the water body. It should be noted that the investigation of wind wave runup for the purposes of this study was of a preliminary nature sufficient to gauge the potential magnitude of the problem.

3. DATA

3.1 Historical Flood Occurrences

A data search was carried out to identify the dates and magnitudes of historical floods. The search concentrated on the period since 1970, as it was considered that data prior to this date would generally be of insufficient quality and quantity for model calibration. The following sources of data were investigated:

- Shoalhaven City Council,
- Department of Land and Water Conservation,
- newspaper articles (refer Appendix A, for selected articles),
- previous reports,
- local residents.

The water level recorder at the Island Point Jetty indicated that since it was installed in July 1991, the only significant rises in water level within the Basin occurred in February 1992 (1.18 mAHD) and April 1994 (0.57 mAHD). During a significant flood event for Tomerong (Cockrow) Creek in September 1993, the Basin level only reached 0.23 mAHD which is nominally just above the high tide level.

Daily rainfall records dating from 1952 for Sussex Inlet were examined, and it was determined that significant storm events occurred in:

- March 1959,
- October 1959,
- February 1971,
- June 1991.

Minor storm events also occurred in:

- May 1953,
- February 1958,
- March 1961,
- March 1975,
- March 1976,
- October 1976,
- February 1992,
- September 1993,
- April 1994.

One or more flood levels are available for the following events:

- 6 February 1971,
- 11 June 1991,
- 11 February 1992,
- 18 September 1993,
- 14 April 1994.

Note: In some cases the flood peak may have occurred a day either side of the date shown.

3.2 Rainfall

3.2.1 Historical Rainfalls

The known sources of rainfall data within the area are listed in Table 2 together with their approximate distance from the catchment and their period of establishment. Pluviograph and daily read rainfall data from these sources were collected (where available) for the storm events in 1971, 1991, 1992 and 1993 (refer Appendix B). Figures 4, 5 and 6 show the locations of the relevant rainfall stations with the rainfall isohyets derived for the 1971, 1991 and 1992 events respectively. The combined locations of rainfall stations relevant to this study are indicated on Figure 9.

Table 2: Rainfall Station Information

Station Name/No	Type	Owner	Location	First Year	Last Year	Distance from Catchment Boundary (km)
069068	DR	BOM	Sussex Inlet Golf Club	1952 - Jan	1976	0
068204	DR	BOM	Sussex Inlet Bowling Club	1976	current (1996)	0
068205	DR	BOM	St Georges Basin MO	1944	1945	0
068078	DR	BOM	Hyams Beach Cyrus Street	1960	1974	2
068222	DR	BOM	Wandandian Post Office	1985	current (1996)	0
069011	DR	BOM	Ettrema	1954	1974	7
068203	DR	BOM	Ettrema (Hollerith)	1954	1992	7
069081	DR	BOM	Sassafras	1901	1918	7
068034	DR	BOM	Jervis Bay Lighthouse	1899	current (1996)	5
068079	DR	BOM	Jervis Bay Forest	1958	current (1996)	5
069025	DR	BOM	Bendalong	1939	1989	10
068229	DR	BOM	Bendalong (Jacaranda)	1989	current (1996)	10
068076	DR	BOM	Nowra RAN Air Station	1942	current (1996)	20
068076	P	BOM	Nowra RAN Air Station	1964 - Sept	current (1996)	20
PR1	PR	Private	Sussex Inlet	1991 - Jan	current (1996)	0

NOTES:
DR - Official daily read rain gauge (9am readings).
P - Pluviograph recorder.
BOM - Bureau of Meteorology.
PR - Private daily read rain gauge.

3.2.2 Design Rainfalls

Design rainfall intensities and patterns were taken from the 1987 version of Australian Rainfall and Runoff (AR&R - Reference 3). The total depths of rainfall for various storm durations and frequencies are given in Table 3.

Table 3: Design Rainfall Depths (mm)

Duration (h)	Frequency (AEP)							
	50%	20%	10%	5%	2%	1%	0.5%	0.2%
1	46	61	70	82	98	109	122	138
1.5	54	72	83	97	115	129	144	164
2	60	80	92	108	129	145	162	184
3	69	93	107	126	151	170	190	217
4.5	80	108	125	147	176	199	223	255
6	88	120	139	164	197	223	249	286
9	102	139	162	191	231	261	293	336
12	113	155	181	214	258	293	328	378
24	152	207	241	284	342	388	434	498
36	180	244	283	333	401	453	507	580
48	201	272	315	370	444	501	561	641
72	230	310	359	421	504	568	634	725

Note: Design rainfall depths derived from AR&R 1987 for the centroid of the catchment.

3.3 Streamflows and Flood Heights

3.3.1 Streamflow Records

There has been one streamflow gauging station operating within the study catchment. The gauge (#216006) is located on Wandandian Creek just upstream of the Princes Highway (refer Figure 1). Streamflow records for the period November 1971 to December 1981 are available from the Department of Land and Water Conservation (DLWC). There was no significant flooding during this period, only minor events in 1975 and 1976.

3.3.2 Automatic Water Level Recorders

Flood height gauge records are available for 2 gauges within the catchment. The “Wandandian” gauge just upstream of the Princes Highway and the “St Georges Basin” gauge which is located on the western side of the Island Point Jetty (refer Figure 1). The latter gauge was installed by NSW Public Works in July 1991 and is now operated by Manly Hydraulics Laboratory. Its datum is 0 mAHD. The data (included in Appendix C) show that the average Basin water level is approximately 0.1 mAHD with tidal range variations of up to 0.2 m in a day reasonably common. Further variations can be caused by rainfall events.

The Island Point Jetty gauge recorded the following levels in the Basin for the floods noted above:

- February 1992: 1.18 mAHD,
- September 1993: 0.23 mAHD,
- April 1994: 0.57 mAHD.

3.3.3 Other Flood Height Data

The most comprehensive source of data was provided by local residents. A questionnaire (a copy of which is included as Appendix D) was distributed to residents who were considered to have potentially experienced flooding, and 71 responses were received. Residents were generally only able to recollect levels resulting from the two most significant storms, in 1971 and 1991. Some reported levels were given for storms in 1992 and 1993, but these were relatively minor events. Many of the responses were verified by field interview and reported flood levels then surveyed to AHD. In some areas, where it was considered that adequate or more reliable data were available, the additional information has been recorded and considered in carrying out our assessment, but detail survey was not requested. The observed levels presented in this report (refer Appendix E) are considered to provide a good indication of actual flood levels attained in the areas where data were available.

A summary of flood level data included in Appendix E is shown on Figures 2a, 2b and 3. The Sanctuary Point Flood Study (Reference 2) provided several additional levels in the vicinity of Tomerong Creek (referred to as Cockrow Creek).

3.4 Ocean Levels

3.4.1 Historical Ocean Levels

Tidal data (refer Appendix C) were obtained from the three gauges listed in Table 4. The Sydney data set was the most complete of the three gauges, and although further away from the catchment, was verified as having levels and timing similar to data from the Jervis Bay and Batemans Bay gauges. This study mainly used tidal data from the Sydney gauge and the small amount of data that was missing was estimated. The following peak ocean levels were identified:

- February 1971 1.69 mAHD,
- June 1991 0.92 mAHD,
- February 1992 0.66 mAHD,
- September 1993 0.86 mAHD.

Table 4: Tide Gauge Data

Station	Owner	Start Date	Finish Date
Sydney ZWARTS	DLWC	1914-May	Current
Jervis Bay CRESWELL	DLWC	1987-June	Current
Batemans Bay #216410	DLWC	1985-December	Current

3.4.2 Design Ocean Levels

Water levels in St Georges Basin are affected by elevated ocean levels. These occur along the NSW coastline as a result of a large number of factors, the most significant of which are associated with storm conditions. Design ocean levels are assessed as a combination of:

- *Astronomical Tide*. This varies depending upon the type of tide, e.g. Spring, Neap.
- *Barometric Effect* (or storm surge). This results from a drop in barometric pressure causing the sea level to rise. Levels can be raised by up to 0.3 m in this manner.
- *Wind Stress*. As wind associated with a storm moves across the ocean it creates a water surface current which moves in the same direction as the wind. When this current reaches the coastline the water piles up increasing the adjacent water levels. Still water levels may be raised by up to 0.3 m in this manner.
- *Wave Setup*. This occurs as a result of transport of water from offshore by wave action. Elevation of standing water levels by up to 1.5 m has been known to occur along the NSW Coast.

For a large coastal catchment it is reasonable to assume that a single meteorological condition (e.g. East Coast Low Pressure System, Cyclone) will produce both the elevated ocean level and the flood producing (long duration) rainfall. However, for a smaller catchment such as St Georges Basin, the short duration rainfall is likely to be produced from a local thunderstorm which may not be associated with ocean activity. In this study, both design tides and design rainfalls have been assumed to occur independently to produce envelopes of design flood levels.

The entrance to Sussex Inlet at the southern end of Bherwerre Beach is protected from swells by Farnham Headland and the island formation at the southern end of Bherwerre Beach.

The storm tide adopted for the design events in this study is based on the storm tide derived in Reference 1. This was determined by analysing elevated water levels and using a modified Monte Carlo modelling technique. The approximate composition of the 1% ocean level is a 0.7 m astronomical tide together with a barometric pressure and wind stress effect of around 0.5 m and a wave setup of 0.8 m. The peak ocean levels used for design events in this study were, therefore:

- Extreme Event 2.1 mAHD,
- 1% Event 2.0 mAHD,
- 2% Event 1.9 mAHD,
- 5% Event 1.8 mAHD.

3.5 Topographic Survey

The majority of the survey data for creeks flowing into St Georges Basin and the Sussex Inlet channel were provided by Phillip Brown (Registered Surveyor) in September 1995, as part of this study. The survey provided data for 41 cross-sections representing Wandandian, Cow, Tullarwalla, Home, Pats, Worrowing and Erowal Creeks and through the Sussex Inlet Canal Estate.

Shoalhaven City Council initially provided data for 19 cross-sections along Tomerong (Cockrow) Creek. This survey was carried out by Council in December 1992 for the Sanctuary Point Flood Study (Reference 2). Additional survey of this area was undertaken when finalising this study (June 1999) in order to extend the cross-sections and define the full width of the floodplain up to the PMF level.

Public Works provided data for 29 cross-sections along the Sussex Inlet Channel, and hydrosurvey data for the bed levels of the Basin. This survey was obtained in February 1992 for the Estuary Management Study (Reference 1) and shows that the average bed level of the Basin varies from approximately -6 mAHD west of Garden Island to approximately -10 mAHD east of Garden Island. The deepest bed level in the Basin is approximately -11 mAHD, occurring approximately 2 kilometres south of Sanctuary Point.

Figures 7a to 7d show the locations of the cross-section data used for this study and Figure 8 shows the Basin hydrosurvey data. Appendix F includes the source (distance, height) data for the 41 cross-sections obtained for this study and the 19 cross-sections provided by Council.

In addition to the above sources, some use was made of orthophoto maps and topographic maps where appropriate to define the overall shape of the catchment topography. Orthophoto maps at a scale of 1:2000 were available for the residential areas around the northern shore of the Basin while 1:10000 topographic maps were available for the southern shore and 1:25000 topographic maps were available for the entire catchment. The 1:2000 orthophoto maps have a theoretical contour accuracy of at best ±1.0 m, with potential for some errors as large as ±2 m, and should therefore be used with caution. The topographic maps would be less accurate.

In recent times the contour information from the available mapping has been transposed into electronic form for use with Council's geographic information system (GIS). These data have been utilised to assist with the finalisation and presentation of output for this study.

3.6 Wind Data

To assist with the analysis of wind wave setup and runup, summary wind frequency data (in the form of analysis tables and wind roses) were obtained from the BOM for the following meteorological weather stations:

- Jervis Bay (Point Perpendicular lighthouse - No. 068034 opened 1899, frequency analysis data 1957-2000),
- Ulladulla AWS (No. 069138 opened 1989, frequency analysis data 1990-2000),
- Nowra RAN Air Station (No. 068076 opened 1942).

4. APPROACH ADOPTED

4.1 General

The approach adopted for this study has been influenced by the quality and quantity of the available data and by accepted practice. There are two basic approaches to determining design flood levels, namely:

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

The flood frequency approach requires many decades of reasonably complete records to give satisfactory results. Such records were not available at any point within the catchment and the second approach was therefore adopted for St Georges Basin. A diagrammatic representation of the Flood Study process is shown on Diagram 1.

A hydrologic model was set up for the entire catchment and used to convert rainfall data into streamflows for input to the hydraulic model. The hydraulic model covered the lake and tributaries indicated by the extent of the study area as shown on Figure 1. To ensure confidence in the results, both models require calibration and verification against observed historical events. With the limited amount of rainfall and flood data available, and the lack of any stream gaugings, calibration of the models proceeded in tandem to make the best use of the available data.

The value of a particular flood for calibration of the models largely depends on the following:

- quantity and quality of daily rainfall data,
- quantity and quality of pluviograph data and the proximity of the gauge to the catchment,
- quantity and quality of the flood height data and location within the study area,
- the magnitude of the event and knowledge of flood behaviour (timing, flow patterns and other influencing factors).

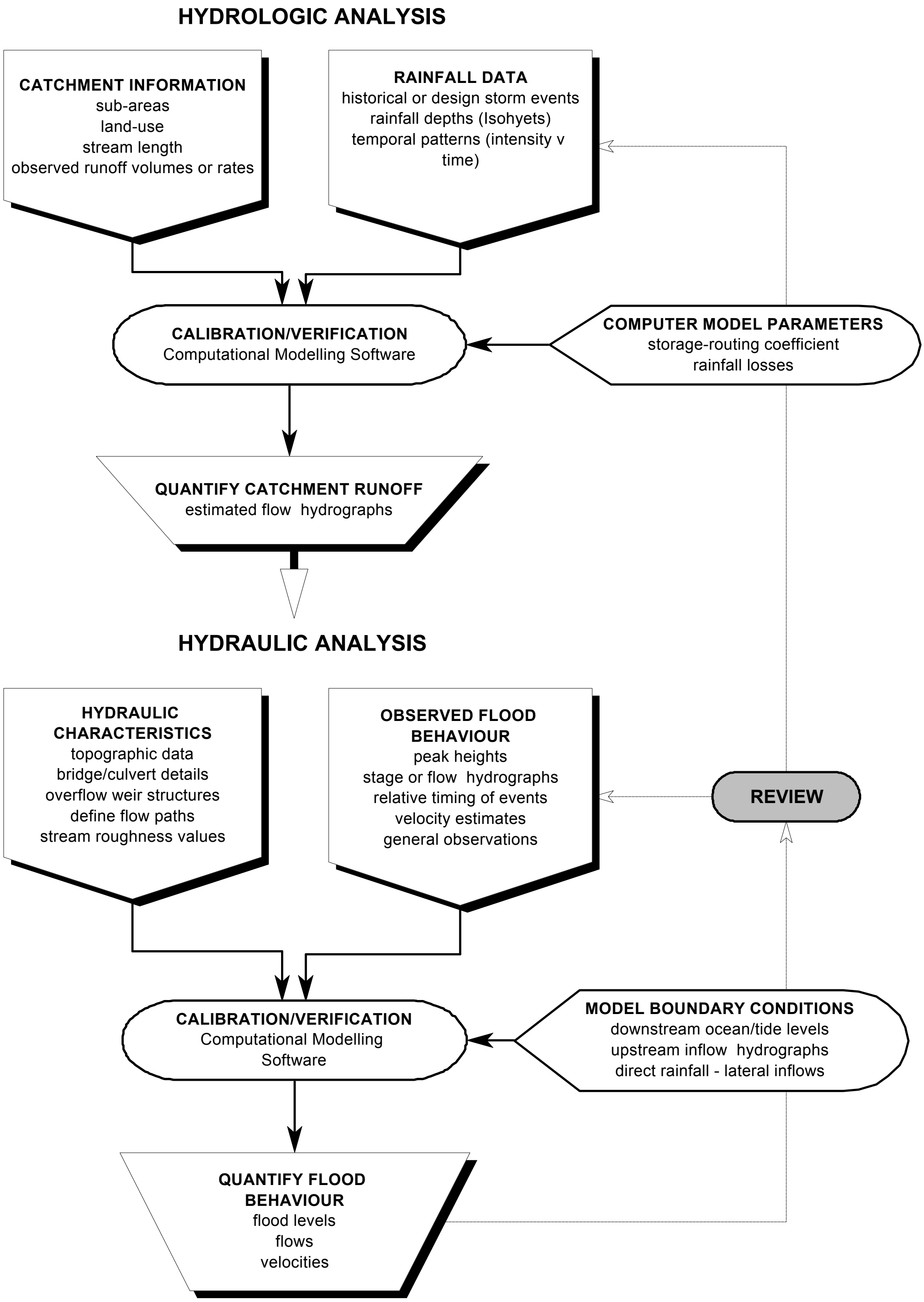
Taking these factors into account five storm events were initially selected for further analysis. Subsequently most of these events were largely discounted because:

- ground levels have changed significantly since 1971, particularly with the development of canal estates in Sussex Inlet which commenced in 1971,
- limited flood levels are available for the events of 1992 and 1993, and only one level was discovered for the 1994 event,
- the limited flood level data which are available for the 1993 event appear inconsistent with observations from other floods.

The final calibration was principally based on the flood event of June 1991, with some verification provided by the floods of February 1971 and February 1992. It should be noted that while the June 1991 and February 1992 events were relatively small (approximately equivalent to 10% and 50%

AEP respectively), the February 1971 flood is the highest recorded in over 100 years according to a pioneering family at Sussex Inlet (Reference 2).

Diagram 1: Flood Study Process



4.2 Hydrologic Modelling

The hydrologic models suitable for design flood estimation are described in AR&R (Reference 3). Of those readily available, the Watershed Bounded Network Model (WBNM) (Reference 5) and the RORB (Reference 6) and RAFTS (Reference 9) runoff-routing models allow the rainfall depth to vary over the catchment area and readily lend themselves to calibration against recorded data. Details of these models are provided in AR&R. References 3 and 5 compare the merits of WBNM and RORB and conclude that there is little difference between them. Reference 5 suggests that WBNM should be preferred as it is easier to use and requires less data. It also has the advantage over RORB of using different approaches to modelling overland runoff and streamflow. WBNM was therefore adopted as the hydrologic model.

4.3 Hydraulic Modelling

AR&R also provides a summary of available hydraulic models. The choice of model depends on many factors, including topography, available data, the aims of the study and availability of the model.

For a catchment such as St Georges Basin, an unsteady state model is necessary to enable adequate simulation of ocean and entrance dynamics as well as storage volume effects. A quasi two-dimensional model is required for simulation of the interaction between the main stream and the overbank floodplain areas. Such a model can also simulate the overbank storage more adequately than a one-dimensional model.

A fully two-dimensional model could not be justified for this study because of the additional expense and survey requirements. Additionally, because of the limited quantity and quality of calibration data, such a model would offer no significant advantages over a quasi two-dimensional model. Thus it was determined that the quasi two-dimensional model, RUBICON, be adopted.

The RUBICON model (Reference 7) has the following features:

- it is mathematically rigorous,
- it is easy to establish and then modify to reflect topographic changes,
- it is a technologically advanced model which is well structured and can accurately model hydraulic controls providing high quality output.

Further details of the model are provided in Appendix G. The general layout of the model structure established for St Georges Basin is shown on Figure 10.

4.4 Wind Wave Climate

The design flood (water) level applicable to a given location around the St Georges Basin foreshore is potentially influenced by a combination of Basin “still” water level (comprising normal tidal and/or ocean influences as well as any increases due to rainfall-runoff processes) and the interaction of waves created by the wind prevailing at the time.

The waves are typically generated by wind forces blowing across the surface of the Basin. The size of the waves depends on the length of fetch, depth of water and the speed and duration of wind (refer Appendix J). As the waves build up and approach the shallower shoreline the wave height increases to reach a limiting value before breaking and advancing up the shoreline as wave runup. The level of wave runup is primarily influenced by the underlying foreshore profile and the presence of any structures. A schematic representation of the wave runup process is shown in Diagram 2.

In order to provide a preliminary quantification of the possible increase in flood levels around the Basin foreshore due to wave action, values of wave runup have been calculated using the methods outlined in References 10 and 11, applied in a similar manner as in Reference 12. It should be noted that in the calculation of wave runup, Reference 12 applied a roughness co-efficient of 1.2 or 0.6 (depending on bank type), in accordance with the information presented in Reference 13. Based on the more recent information in Reference 11, a roughness co-efficient of 0.9 has been adopted for the purposes of this study. As a rigorous assessment was beyond the scope of this present study, only three sites were chosen for analysis:

Site 1: Paradise Beach at Sanctuary Point - primary risk of westerly wind exposure
Site 2: Loralyn Avenue, St Georges Basin East - primary risk of southerly wind exposure
Site 3: Watersedge Avenue, Basin View - primary risk of south-easterly wind exposure

These sites are situated around the northern foreshore (refer Figure 1) and were considered to represent the worst possible wind frequency/fetch combinations for the existing development areas. Details of the calculations are included in Appendix J.

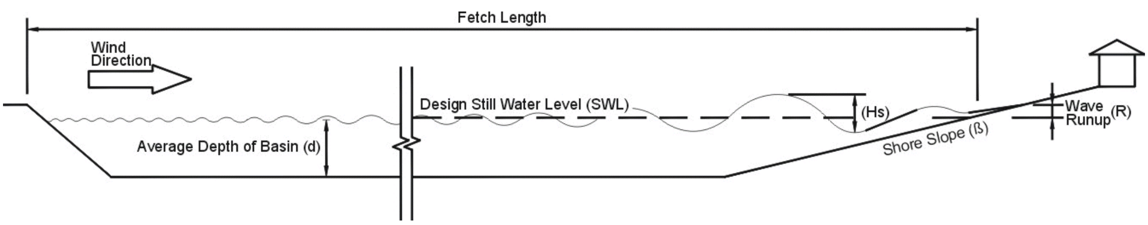


Diagram 2: Schematic Representation of Wave Runup

5. HYDROLOGIC MODELLING

5.1 General

As noted in Section 4.2, WBNM was adopted for use in this study. The model simulates the catchment as a series of sub-catchments based on watershed boundaries linked together to replicate the natural stream network. The adopted sub-catchment division is shown on Figure 9.

Ideally WBNM is calibrated by adjusting parameters in order to match observed streamflow hydrographs. Because there were no observed flow data for the calibration floods, this process was not possible. The parameters chosen were based on recommended values in AR&R and previous experience on the south coast and Shoalhaven Council area. The process of calibration was then achieved by adjusting the Manning's 'n' values in the hydraulic model (within reasonable limits) to match the recorded flood heights.

Design flows were obtained using the design rainfall intensities and patterns from AR&R (see Section 3.2.2) and the adopted hydrologic model parameters.

5.2 Model Parameters

In calibrating WBNM, four parameters can be varied to achieve a fit to the observed data. One of these, "C", modifies the storage routing functions in the model. The non-linearity exponent of the storage routing equation is generally set at the recommended value (0.77), and is only adjusted if there are sufficient streamflow data to justify choosing a different value. The other two parameters, initial loss and continuing loss, modify the amount of excess rainfall input for routing through the model storages.

The value of C was adopted as 1.29 and the non-linearity as 0.77, these being the current recommended (AR&R) design values for ungauged catchments. Initial loss was set at zero and continuing loss at 2.5 mm/h. AR&R suggests values for initial loss ranging from 0 mm to 35 mm for eastern NSW catchments. The use of zero initial loss was justified in that prior to the flood producing rains of each historical storm, the catchment was wet from preceding rain. The adopted value of 2.5 mm/h for continuing loss has been found to be applicable over a wide range of catchments in Eastern Australia.

5.3 Design Flows

WBNM was used to derive design flows for input to the hydraulic model. Hydrographs were obtained for the 5%, 2% and 1% probability events.

Design rainfall intensities and temporal patterns were derived from AR&R. Uniform depths of rainfall, with depth-reduction factors ranging between 0.84 and 0.95 for different durations, were applied across the whole catchment.

The model was run for a range of storm durations (1 hour to 72 hours) for the 1% design event to determine the critical storm duration, i.e. the duration which produced the highest peak flow. It was found that the different tributary sub-catchments had different critical durations, as shown in Table 6. The 2h, 4.5h, 9h and 48h storms were adopted for the range of critical events, and were run through the hydraulic model for the other design flood frequencies. The design peak at each location was then determined by adopting the maximum value obtained from the four storm durations.

An approximation to the extreme flood was obtained by multiplying the 1% design inflow hydrographs by three. This approach is consistent with other studies in the area and provides an indication of the effects of a flood much rarer than the 1% event.

The design inflow hydrographs for the various tributary catchments are included as Appendix H.

6. HYDRAULIC MODELLING

As discussed in Section 4.3, the RUBICON unsteady flow model was used for simulating flood behaviour in St Georges Basin. The model layout is shown on Figure 10.

The hydraulic efficiency of the creeks is represented in the model by the stream roughness or friction factor known as Manning's 'n'. This factor describes:

- channel roughness,
- channel sinuosity,
- vegetation and other debris in the channel,
- bed forms and shapes.

The model was calibrated by adjusting Manning's 'n' to match available historical flood height data. As discussed in Section 4.1, the flood of June 1991 was used as the principal calibration tool. The floods of February 1971 and February 1992 were used to provide a measure of verification of the adopted 'n' values and resulting hydraulic gradients.

Figures 11 to 13 show modelled peak height profiles for the 1991 flood along Sussex Inlet channel, Badgee Lagoon, Sussex Inlet Canal Estate, Tomerong Creek and Wandandian Creek. The fit to observed levels is good throughout (typically within +/- 0.3 m), except for some observed levels in the Basin itself where there is a considerable scatter of observations and several are 0.5 m to 0.8 m higher than the model results. It is possible that many of the reported levels are higher than the "mean" level in the Basin because of the effects of wave action. The modelled levels are more a function of the definition of Basin storage volume rather than entrance roughness (Manning's 'n') values along the inlet channel.

Figures 14 to 17 compare the observed data available for the floods of 1971 and 1992 with the modelled peak flood profiles. Both of these events provide reasonable verification of the adopted 'n' values and consequent flood gradients produced by the model. The best indication is provided by the inlet channel profile (Figure 14) where a large number of observed data points are available. Although there is some scatter in levels (up to 0.8 m in absolute difference), the modelled profile typically conforms to the weighted mean of observations.

In the absence of calibration data along the tributary creeks, Manning's 'n' values were based on surrounding "calibrated" tributaries of similar characteristics, field inspections, experience and recognised texts.

7. DESIGN FLOOD RESULTS

7.1 Results of Rainfall-Runoff Event Modelling

Flow hydrographs for the 1% flood were obtained from WBNM for durations ranging from 1 hour to 72 hours. The RUBICON model was run for each of these storm durations to determine the critical durations throughout the catchment. The critical duration was found to vary between 2, 4.5, 9 and 48 hours for the different tributary sub-catchments. The peak results for each design frequency were determined from the envelope of these four storm durations. Tables 5 to 7 summarise the modelled peak flood level, flow and velocity results for the historical and design events at selected locations indicated on Figure 1. The resulting peak height profiles for the design events are shown on Figures 18 to 23. Design flood contours for the four main areas of interest are shown on Figures 24(a-d) to 26(a-d) for the 1%, 2% and 5% AEP events respectively. Figures 27(a-d) to 29(a-d) present the corresponding flow distributions with indicative velocities.

Table 5: Comparison of Modelled Historical and Design Flood Levels

No.	Location	Creek	Level (mAHD)						
			1971	1991	1992	Ext.	1%	2%	5%
1	Basin	Inlet	2.23	1.49	1.07	5.1	2.35	2.09	1.78
2	Badgee Lagoon Jtn	Inlet	2.17	1.44	1.00	5.1	2.30	2.05	1.74
3	Jacobs Drive	Inlet	2.09	1.39	0.93	5.0	2.26	2.01	1.72
4	Cater Canal	Inlet	1.95	1.33	0.89	4.8	2.18	1.95	1.68
5	Coastal Patrol	Inlet	1.69	1.20	0.79	4.4	2.05	1.85	1.63
6	The Haven	Inlet	1.13	1.01	0.66	3.1	1.96	1.86	1.75
7	D/s Sussex Inlet Rd Bridge	Badgee	2.16	1.44	1.31	5.0	2.30	2.04	1.74
8	U/s Badgee Bridge	Badgee	2.16	1.44	1.00	5.0	2.30	2.05	1.74
9	Jacobs Dr Bridge	Cater	2.16	1.44	0.99	5.0	2.30	2.04	1.74
10	U/s Cater Bridge	Cater	2.10	1.40	0.95	5.0	2.26	2.02	1.72
11	1 km D/s Sussex Inlet Rd	Cow	3.92	2.36	2.33	5.1	3.47	3.29	3.07
12	200 m D/s Princes Highway	Wandandian	6.98	4.56	4.72	10.2	6.66	6.29	5.81
13	Sawmill U/s	Wandandian	5.08	2.73	2.79	8.4	4.70	4.32	3.82
14	Sawmill D/s	Wandandian	4.88	2.59	2.64	8.2	4.50	4.12	3.63
15	Bewong	Wandandian	4.68	2.45	2.49	7.9	4.29	3.92	3.43
16	Wool Rd	Pats	4.33	4.00	4.04	5.1	4.26	4.22	4.18
17	U/s Wool Rd	Home	2.56	1.81	1.87	5.1	2.54	2.45	2.33
18	Wool Rd	Tomerong	3.62	2.11	2.47	5.1	3.44	3.26	3.01
19	Boronia Ave	Tomerong	3.19	1.79	2.14	5.1	3.02	2.86	2.64
20a	U/s Larma Ave	Tomerong	2.31	1.50	1.48	5.1	2.36	2.10	1.86
20b	D/s Larma Ave	Tomerong	2.24	1.49	1.40	5.1	2.36	2.10	1.78
21a	U/s Wool Rd	Worrowing	7.75	7.34	7.44	8.2	7.75	7.70	7.65
21b	D/s Wool Rd	Worrowing	5.64	4.74	4.91	6.9	5.66	5.53	5.40
22	Fitzpatrick St	Worrowing	2.56	1.71	1.86	5.1	2.56	2.44	2.32
23	Kallaroo Rd	Erowal	7.01	6.53	6.64	7.6	7.08	7.01	6.94
24	Killarney Rd	Erowal	4.01	3.50	3.78	5.1	4.06	4.01	3.97

Note: Refer Figure 1 for the result locations. Design flood contours are presented on Figures 24(a-d) to Figures 26(a-d).

The overall influence of the adopted ocean level or tide on the tributary profiles and Basin level in a 1% flood was found to be minimal. This was established by running the 1% inflow hydrographs with the peak ocean level reduced from 2 mAHD to 1 mAHD. It was found that the peak level achieved in the Basin dropped only 0.2 m. Basin levels are influenced more by the volume of runoff from the contributing catchment area as is evidenced by the longer critical duration of 48 hours for the Basin and Inlet channel downstream. The extended storm duration of some 5 days for the 1971 historical event, with a peak ocean level less than the 5% AEP design level, provides an actual demonstration of Basin levels being primarily influenced by volume effects.

Table 6: Comparison of Estimated Historical and Design Flood Flows

No.	Location	Creek	Flow (m^3/s)							Critical Duration
			1971	1991	1992	Ext.	1%	2%	5%	
1	Basin	Inlet	466	266	192	789	456	408	325	48
2	Badgee Lagoon Jtn	Inlet	384	235	171	513	384	348	279	48
3	Jacobs Drive	Inlet	376	235	171	419	379	345	277	48
4	Cater Canal	Inlet	486	273	194	1769	442	361	283	48
5	Coastal Patrol	Inlet	488	275	195	2204	510	416	-330	48
6	The Haven	Inlet	496	287	196	2229	512	427	338	48
7	D/s Sussex Inlet Rd Br'ge	Badgee	102	19	20	195	66	57	48	4.5
8	U/s Badgee Bridge	Badgee	128	-33	-21	-400	126	86	81	48*
9	Jacobs Dr Bridge	Cater	71	35	23	435	75	60	44	48*
10	U/s Cater Bridge	Cater	69	35	23	433	74	59	44	48*
11	1 km D/s Sussex Inlet Rd	Cow	169	41	40	366	123	106	89	9
12	200 m D/s Princes Hwy	Wandandian	1560	526	584	4132	1378	1189	976	9
13	Sawmill U/s	Wandandian	1506	515	555	4147	1336	1148	931	9
14	Sawmill D/s	Wandandian	1612	550	587	4519	1429	1224	988	9
15	Bewong	Wandandian	1660	560	593	4664	1460	1249	1005	9
16	Wool Rd	Pats	90	21	26	210	70	60	51	4.5
17	U/s Wool Rd	Home	21	6	7	54	21	18	15	2
18	Wool Rd	Tomerong	477	131	189	1247	414	358	295	9
19	Boronia Ave	Tomerong	476	130	188	1247	413	357	294	9
20a	U/s Larma Ave	Tomerong	538	144	208	1426	468	403	330	9
20b	D/s Larma Ave	Tomerong	537	144	208	1426	467	403	330	9
21a	U/s Wool Rd	Worrowing	50	12	17	153	51	44	37	4.5
21b	D/s Wool Rd	Worrowing	50	12	17	153	51	44	37	4.5
22	Fitzpatrick St	Worrowing	50	12	17	152	51	44	37	4.5
23	Kallaroo Rd	Erowal	38	10	14	135	45	38	33	4.5
24	Killarney Rd	Erowal	38	9	14	133	44	38	32	4.5

Note: Refer Figure 1 for the result locations. Design flows are presented on Figures 27(a-d) to Figures 29(a-d).
* peak flow from Inlet through Canal.
-ve values indicate flow is the opposite direction to that assumed in the model setup.

The variations in flood magnitudes between different tributary sub-catchments for the historical floods is related to the estimated rainfall distribution across the catchment, the different rainfall temporal patterns and to some extent, ocean or tidal effects. It should be noted that no observed flow data and only limited rainfall data were available for the catchment. The above historical flow estimates are therefore considered to be indicative only and are provided to assist comparison of relative flood magnitudes with design events. Negative flow results indicate that the peak absolute value occurs when flow is in the opposite direction to that assumed in the model setup. A similar (but slightly lesser) return value in the positive direction is also usually experienced as the floodwaters fill and then drain storage areas of the floodplain via the flow path branches modelled.

The results for the design floods suggest that the 1971 flood was generally a little larger than a 1% flood. Most of the tributary creeks flowing into the Basin experienced greater than 1% flooding, whilst flooding in the Basin itself and through the Sussex Inlet area was closer to a 2% event.

The floods of 1991 and 1992 were both smaller than a 5% flood. The 1991 flood was greater than the 1992 flood for the Basin, channel and canal estate areas, but smaller than the 1992 flood in most of the tributary creeks flowing into the Basin.

Table 7: Comparison of Historical and Design Flood Velocities

	Location	Creek	Velocity (m/s)						
			1971	1991	1992	Ext.	1%	2%	5%
1	Basin	Inlet	0.4	0.3	0.3	0.3	0.4	0.4	0.3
2	Badgee Lagoon Jtn	Inlet	0.4	0.3	0.3	0.4	0.3	0.3	0.3
3	Jacobs Drive	Inlet	0.5	0.5	0.4	0.5	0.4	0.5	0.5
4	Cater Canal	Inlet	0.7	0.6	0.5	0.8	0.5	0.5	0.5
5	Coastal Patrol	Inlet	0.6	0.5	0.6	0.9	0.5	0.5	-0.4
6	The Haven	Inlet	1.2	1.0	0.9	2.2	1.0	0.9	0.8
7	Sussex Inlet Rd Bridge	Badgee	0.8	0.6	0.6	1.0	0.7	0.7	0.6
8	Badgee Bridge	Badgee	1.0	-0.2	-0.2	-0.3	0.8	0.4	0.5
9	Jacobs Dr Bridge	Cater	0.2	0.5	0.7	0.2	0.2	0.2	0.3
10	Cater Bridge	Cater	0.9	0.9	0.8	0.6	0.8	1.0	1.0
11	1 km D/s Sussex Inlet Rd	Cow	3.3	2.1	2.1	2.2	3.1	3.0	2.8
12	200 m D/s Princes Highway	Wandandian	2.4	2.2	2.3	2.7	2.4	2.4	2.4
13	Sawmill U/s	Wandandian	0.7	0.6	0.7	0.9	0.7	0.7	0.7
14	Sawmill D/s	Wandandian	0.8	0.9	0.9	1.1	0.8	0.8	0.8
15	Bewong	Wandandian	1.1	0.9	0.9	1.6	1.1	1.1	1.0
16	Wool Rd	Pats	0.9	0.4	0.5	1.2	0.8	0.7	0.7
17	Wool Rd	Home	1.7	1.0	1.0	0.7	1.7	1.6	1.5
18	Wool Rd	Tomerong	0.8	0.7	0.7	1.0	0.7	0.7	0.7
19	Boronia Ave	Tomerong	0.6	0.6	0.5	0.8	0.6	0.5	0.5
20a	U/s Larma Ave	Tomerong	0.7	1.8	2.0	0.7	0.7	0.8	1.0
20b	D/s Larma Ave	Tomerong	0.8	1.8	2.4	0.8	0.9	1.0	1.3
21a	U/s Wool Rd	Worrowing	0.4	0.1	0.2	0.8	0.4	0.3	0.3
21b	D/s Wool Rd	Worrowing	1.9	1.4	1.5	2.1	1.9	1.8	1.8
22	Fitzpatrick St	Worrowing	1.5	1.0	1.1	2.1	1.5	1.5	1.4
23	Kallaroo Rd	Erowal	0.3	0.2	0.2	0.6	0.3	0.3	0.3
24	Killarney Rd	Erowal	0.5	0.3	0.3	1.0	0.6	0.5	0.5

Note: Refer Figure 1 for the result locations. Design velocities are presented on Figures 27(a-d) to Figures 29(a-d). The velocity values tabulated above represent the average velocity at peak flow for the overall cross-section at any given location. A breakdown of values for the individual cross-section segments (left bank, main flow area and right bank) is presented on Figures 27-29. Localised velocities may be higher than indicated by the model results. Negative values indicate the peak absolute value occurs when flow is in the opposite direction to that assumed in the model setup.

7.2 Comparison of Tomerong Creek Results with Previous Study

As outlined in Section 2.3 a flood study for Sanctuary Point incorporating the Tomerong Creek catchment (known as Cockrow Creek in the lower reaches) was carried out in 1992 (Reference 2). A comparison of design results is included in Table 8.

Table 8: Comparison of Results with Previous Study

Description	Units	5% AEP (20y ARI)		1% AEP (100y ARI)	
		1992 Sanctuary Point Study (Ref 2)	**2001 St Geoqrges Basin Flood Study**	**1992 Sanctuary Point Study (Ref 2)**	**2001 St Georges Basin Flood Study**
Basin Level	m	2.1	1.8	2.3	2.4
Peak 9h Discharge (u/s inflow to hydraulic model from Tomerong Creek catchment)	m^3/s	335	300	510	420
Flood level at: • cross-section 7	m	2.18	1.78	2.47	2.36
• cross-section 11 (u/s Larma Avenue)	m	2.43	2.03	2.80	2.36
• cross-section 14 (The Park Drive)	m	3.05	2.64	3.54	3.02
• cross-section 18	m	4.17	3.92	4.56	4.20

The design flood levels presented in this study are 0.1 m to 0.5 m lower than those of the previous study (Reference 2). The two factors primarily influencing these results are the peak discharge input to the hydraulic model and the Basin (downstream) water level assumed to be prevailing at the time of the flood peak. By way of explaining this difference the following points are noted:

- the peak flow adopted for the earlier study was based on calibrating a RORB hydrologic model to a value estimated using the Probabilistic Rational Method (PRM). The present study applied typical or recommended hydrologic model parameters to produce flow inputs which were then tested or verified by comparing observed flood height data with results obtained from the hydraulic model,
- a fixed Basin starting level was previously adopted (Reference 2) and assumed to coincide with the peak discharge from the catchment. In this study the Basin level takes into account the influence and timing of ocean conditions as well as inflows from the remainder of the catchment. The peak Basin level of 2.4 m is derived from the 48 hour storm whereas in the 9 hour event critical for the local catchment the Basin level attained for the 1% event is only 1.7 m,
- the hydraulic modelling approach for the previous study adopted a standard step (peak flow) backwater analysis. The effects of storage routing (particularly relevant to the expansive overbank areas) and hydrograph timing are better represented using the unsteady RUBICON model. In finalising this study, additional survey of the overbank

areas was obtained to ensure better definition of the available floodplain storage, and thus more accurate results.

7.3 Wind Wave Climate Results

Based on the general approach outlined in Section 4.4, the potential wind wave climate was calculated for three (3) alternative sites located around the northern foreshore of the Basin. Details of the analysis are included in Appendix J and the results are summarised in Table 9.

Table 9: Wind Wave Results

		Site 1 Paradise Beach Sanctuary Point	Site 2 Loralyn Avenue St Georges Basin	Site 3 Basin View
Fetch Length	km	6.2	5.2	9.3
Shoreline Slope	m/m	0.05	0.2	0.25
1% AEP Flood Event with 2.4 m Stillwater Basin Level and 1% Exceedance Recorded Wind Speed of 35km/h:				
Average Depth of Water	m	8.4	10.4	8.4
Wave Runup	m	0.09	0.34	0.56
Design Water Level	mAHD	2.5	2.7	3.0
Storm Tide with 0.8 m Stillwater Basin Level and 1% AEP Design Wind Speed of 165km/h:				
Average Depth	m	6.8	8.8	6.8
Wave Runup	m	0.41	1.56	2.42
Design Water Level	mAHD	1.2	2.4	3.2

Note: Locations of Sites is shown on Figure 1

It is evident from the results for the 1% AEP flood event scenario that for winds exceeded 1% of the time, a wave runup component of 0.1 m, 0.3 m or 0.6 m could be added to the estimated base design stillwater level of 2.4 mAHD at Sites 1, 2 or 3 respectively. In a worst case scenario, assuming the 1% AEP design wind gust of 165 km/h was sustained over a sufficiently prolonged duration to generate the wind waves, the provision of an additional 0.2 m (0.8 m total) may be considered appropriate for Site 3.

A sensitivity analysis of the 1% AEP flood event results for Site 2 indicated the variation of estimated runup to be contained within ±0.15 m even when the adopted base parameters were doubled or halved.

7.4 Accuracy of Results

The order of accuracy of design flood results is primarily influenced by the quantity and quality of recorded historical data (rainfall, flow, peak height) available for model calibration and verification.

For the purpose of this study it should be noted that there was a distinct lack of suitable data with which to confidently establish appropriate model parameters through rigorous calibration and verification. For the few events available for analysis, there is a distinct paucity of rainfall data, no observed runoff (flow) data, and with the exception of the Sussex Inlet area, only limited peak flood height information.

Typically where historical height data are available, such as within Tomerong Creek, the Basin and surrounding foreshore areas as well as Sussex Inlet, flood levels are considered to be accurate within the range of ±0.3 m. There is a greater uncertainty for the other tributary creeks due to the paucity of flow and flood height data and the order of accuracy is more likely to be within the range of ±0.5 m.

In order to improve this accuracy it is recommended that as much data as possible be obtained immediately following any future flood events. Such data would include rainfalls, ocean conditions and peak flood heights.

It should be noted that the design flood level results provided in this report are representative of the volumetric ("still") water levels based on rainfall/runoff processes from the contributing catchment area. Wind wave setup/runup and their potential implications for design flood levels (and hence Flood Planning Levels for new development) can be a problem in their own right, but vary considerably depending on the prevailing wind conditions and approach fetch across the water body in a particular event. Recorded data and observations from the historical flood events (refer Sections 2.4 and 3.3.3) would tend to support this theory.

A preliminary analysis of wind wave effects was undertaken for the purposes of this study in order to ascertain the potential magnitude of the problem and hence possible implications for design flood levels around the Basin foreshore. The results of the analysis suggest that wave runup could add somewhere between 0.1 m and 0.6 m to the estimated design "still" water flood levels depending on the site location and the prevailing conditions (including the joint probability of flooding and wind occurrence). The adopted bed slope and wind speed are the dominant factors affecting the results obtained. A sensitivity analysis undertaken for Site 2 indicated the adopted wave runup component fell within a range of ±0.15 m. A detailed assessment of the implications of wind wave effects on design flood levels was beyond the scope of this present study, and further consideration of the issue during the Floodplain Management Study is recommended to ensure appropriate allowance is incorporated when establishing Flood Planning Levels for new development.

8. ACKNOWLEDGMENTS

The study was carried out by Webb, McKeown & Associates Pty Ltd. It was funded by Shoalhaven City Council and DLWC. The assistance of the following in providing data and guidance to the study is gratefully acknowledged:

- Shoalhaven City Council,
- Department of Land and Water Conservation,
- Residents in the St Georges Basin Catchment area.

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ASCE Proceeding, 23rd ICCE, Venice Italy, 1992

APPENDIX A: PRESS ARTICLES

APPENDIX B: RAINFALL DATA

APPENDIX C: TIDE AND LAKE LEVEL DATA

APPENDIX D: QUESTIONNAIRE

QUESTIONNAIRE - AUGUST 1995
ST GEORGES BASIN FLOOD STUDY

Webb, McKeown & Associates have been engaged by Shoalhaven City Council to carry out a Flood Study of St Georges Basin and its catchment. Historical flood data are required to ensure the accuracy of the study and we require your assistance in this regard. If you have any information which may be of value please complete the form and return to the Reply Paid address (no stamp required) shown overleaf. Please return this form as soon as possible.

Resident's name: ____________________________ Date: ______________

Address: ____________________________ Telephone No: ______________

____

1. How long have you been a resident?
   .... Years

2. Have you ever been affected by flooding or drainage problems in your local area? ” Yes ” No

3. What was the month/year of the worst flooding?

   . . . .

4. Major flooding occurred on the dates listed below. Can you show us where the floods came to?

   6 February 1971 ” Yes ” No — 10 February 1992 ” Yes ” No
   10 June 1991 ” Yes ” No — 13 September 1993 ” Yes ” No

   Have you any information for any other flood events? ” Yes ” No

   If yes what information?

   . . . .

   . . . .

   . . . .

5. Do you have photographs of floods or a diary describing floods or any other information, (e.g. newspaper cuttings) which may assist us? If so may we examine them? ” Yes ” No

6. Do you have rainfall data for any flood event? ” Yes ” No

7. Do you know anyone in the area who might have flood or rainfall data? ” Yes ” No

8. Has water ever entered your home or building? ” Yes ” No

   If so when?

   . . . .

9. Has water ever entered your yard? ” Yes ” No

   If so when?

   . . . .

10. Have you ever had a flood warning (radio, T.V., etc.) ” Yes ” No

11. Has flooding ever caused you to miss work? ” Yes ” No

    If so when?

    . . . .

12. Has Council/SES/friends ever offered to provide assistance during or after a flood? ” Yes ” No

    If so when?

    . . . .

13. May we enter your property to undertake a survey of floor levels, flood levels, or yard levels? ” Yes ” No

14. Have you ever experienced flooding from local drainage rather than the creek overtopping its banks? ” Yes ” No

Thank you for your help. Should you have any further questions please contact:

Shoalhaven City Council PO Box 42 NOWRA NSW 2541	OR	Reply Paid 1752 Webb, McKeown & Associates Pty Ltd Level 8, 35 York Street SYDNEY NSW 2000
Telephone (044) 29 3111		Telephone (02) 299 2855
Attention: Mr J Downey		Attention: Mrs Valerie Tulk

Please use the remainder of this sheet to provide any further comments. You may wish to provide suggestions to reduce flood levels such as to clear the creeks or you may wish to comment on other aspects of the management of the creek system. Your comments are of great value in assessing the future management of the creeks and we welcome your input.

COMMENTS:

APPENDIX E: HISTORICAL FLOOD HEIGHT DATA

APPENDIX F: SURVEY DATA

APPENDIX G: THE RUBICON MODEL

APPENDIX G: THE RUBICON MODEL

G1. INTRODUCTION

HD-system RUBICON was developed by Haskoning BV and Delft Engineering Software. It can be used for studying a wide range of hydraulic engineering problems, such as:

- flood wave propagation through channels, rivers, floodplains and reservoirs,
- tidal flow in rivers and estuaries,
- effects of structures in channel systems,
- sediment movement in rivers and estuaries,
- optimum design and operation of irrigation and drainage systems,
- entrance breakout through an erodible beach berm,
- wave propagation in hydropower systems,
- wave propagation resulting from dam failures,
- hydraulic parameters in water quality studies.

Modelling is based on the full de Saint-Venant equations solved with a highly accurate and efficient modification of Preissmann's implicit finite difference scheme. It is very flexible in specifying external and internal boundary conditions. The user can select from a number of system elements to simulate complex flow over floodplains or define structures at any point of the channel system, such as weirs, gates, culverts, siphons, spillways, sluices, storm surge barriers, dykes, etc.

Limitations are the accurate simulation of super-critical flow and two-dimensional flow situations where the convective momentum terms play a significant role.

Important objectives during the design of the program were to make it a user-friendly system, which would minimise the time required for data preparation, and formulate the system in a modular way to facilitate addition of enhancements.

This is exemplified by the following features:

- programs written in Fortran with the source code made available,
- separate processing for input, execution and output sub-systems,
- extended free format data input, including comments,
- possible to add user defined sub-routines and functions,
- all user-defined model elements (channels, structures, etc.) addressed by names,
- automatic generation of computational grid and element numbers following user's directives,
- use of special information symbols to minimise input effort,
- extensive checking of input data,
- continuation of input processing after detection of errors,
- restart facilities in model execution,
- possible generation of output at any point of the channel system.

The original suite of programs has been extensively modified by Webb, McKeown & Associates. A layout of the current RUBICON modelling system is given as Figure G1.

G2. SYSTEM ELEMENTS

The following range of model elements are available:

- branches,
- cross-sections,
- nodes,
- structures,
- gridpoints,
- lateral flows.

Branches are used as schematised elements for:

- rivers,
- channels,
- estuaries,
- ocean entrances,
- connections between floodplain cells,
- closed conduits.

At the branch limits, *Nodes* are included to provide for:

- free branch ends,
- branch connections,
- floodplain cells.

A single node can connect any number of branches. A boundary condition can be applied at a free branch as a function of:

- height,
- flow,
- critical outflow,
- or any user defined parameter.

Gridpoints are located along branches and have an associated *Cross-Section* which defines the topography. *Structures* can be defined at any place along a branch and basically they provide a relationship between the discharge and upstream/downstream water levels. The definitions of structures are extremely flexible and includes culverts, free overflow structures, submerged structures, underflow structures or local loss structures. It is also possible to simulate complex gate opening/closing structures or pumps. The model also provides for a user-defined structure to be included as a sub-routine. This allows complete freedom in defining the structure.

Culverts are modelled using the approach adopted by Boyd (Reference G1). Culverts are checked for outlet and inlet control and the lesser flow is adopted. Box or pipe culverts can be modelled as well as the shape of the wing-wall, Manning's 'n', slope and other culvert characteristics. Weirs are input as a weir type formulae and are generally represented as a series of horizontal steps with appropriate C values. Ocean entrance berms are represented as trapezoidal sections which erode using the weir formula and appropriate sediment movement equations.

Inflows are generally input at the upstream nodes as a flow versus time function. However *Lateral inflows* can also be included as a flow versus time function at any location along a branch.

G3. OUTPUT

Output from RUBICON is extremely comprehensive including:

- maximum profiles - height, flow or velocity. The time of the peak is provided as well as the value of the other parameters,
- output at every time step of height, flow and velocity which can be represented as a dynamic profile,
- time functions of height, flow, velocity, area, width and a large number of other hydraulic parameters,
- the above data can be used to provide rating curves, hazard diagrams and a number of other plots.

The output can be provided on a screen, disk, hard copy or plot.

The output is output in a format compatible with the HGRAPH (or other) graphics package. This permits extremely high quality output to be provided on a range of devices.

G4. REFERENCES

G1. Boyd M J, et al
PC Programs for Flooding and Stormwater Drainage
Watercomp 1989, Institution of Engineers, May 1989.

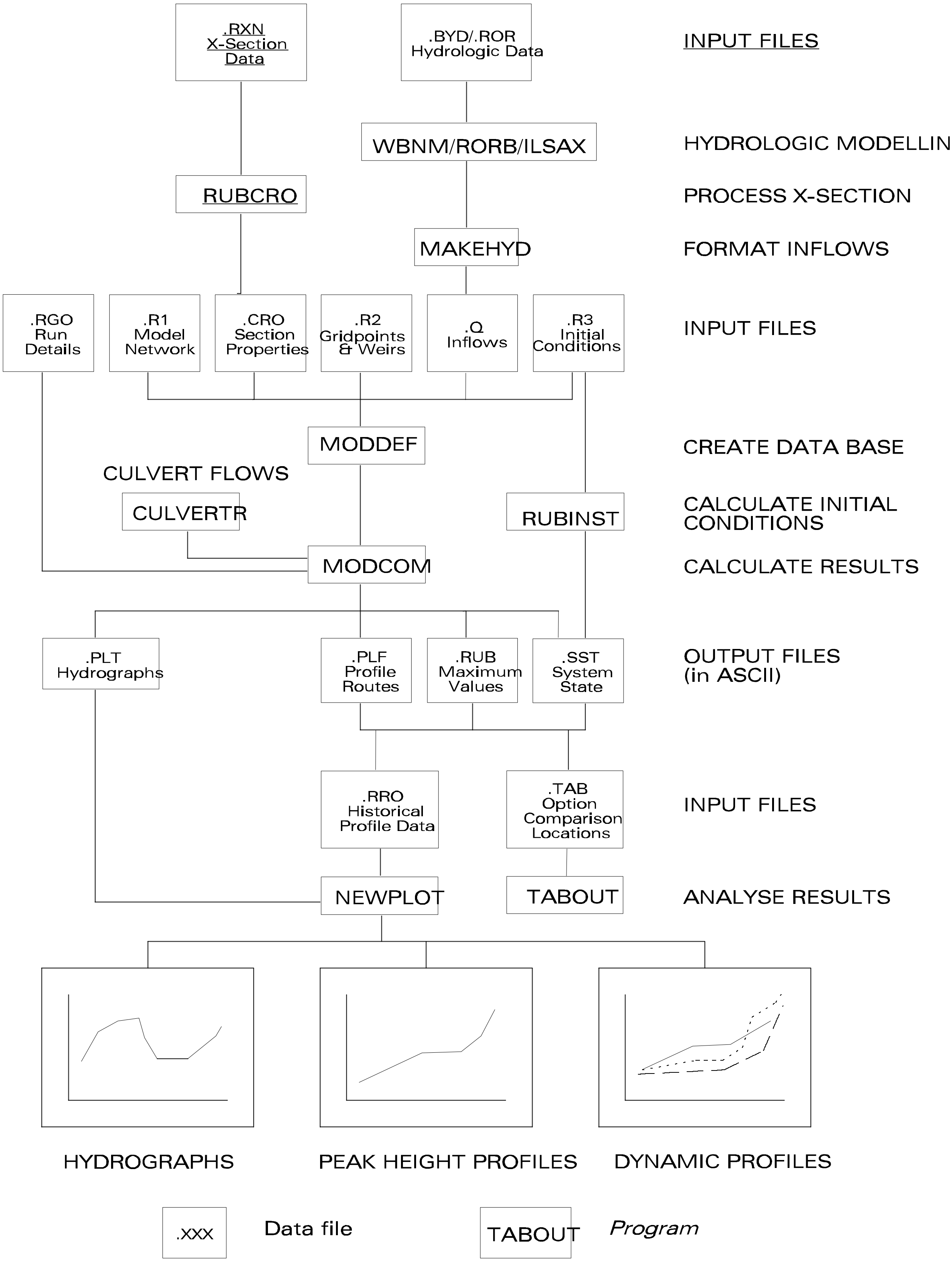

Figure G1: RUBICON System Flow Diagram

APPENDIX H: DESIGN INFLOW HYDROGRAPHS

APPENDIX I: MANNING'S 'n' VALUES

APPENDIX J: WIND WAVE ANALYSIS