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Wyong Council

Tuggerah Lakes Flood Study

September, 1994

LAWSON AND TRELOAR PTY LTD

AUSTRALIA

Wyong Council

Tuggerah Lakes Flood Study

SEPTEMBER, 1994

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GLOSSARY

Annual Exceedance Probability (AEP)	refers to the probability or risk of a flood of a given size occurring or being exceeded in any given year. A 90% AEP flood has a high probability of occurring or being exceeded; it would occur quite often and would be relatively small. A 1% AEP flood has a low probability of occurrence or being exceeded; it would be fairly rare but it would be relatively large.
Australian Height Datum (AHD)	a common national plane of level corresponding approximately to mean sea level.
catchment	the area draining to a site. It always relates to a particular location and may include the catchments of tributary streams as well as the main stream.
designated flood	(see flood standard)
development	the erection of a building or the carrying out of work; or the use of land or of a building or work; or the subdivision of land.
discharge	the rate of flow of water measured in terms of volume over time. It is to be distinguished from the speed or velocity of flow which is a measure of how fast the water is moving rather than how much is moving.
flood	relatively high stream flow which overtops the natural or artificial banks in any part of a stream or river.
flood hazard	potential for damage to property or persons due to flooding.
flood liable land	land which would be inundated as a result of a designated flood.
floodplain	the portion of a river valley, adjacent to the river channel, which is covered with water when the river overflows during floods.
floodplain management measures	the full range of techniques available to floodplain managers.
floodplain management options	the measures which might be feasible for the management of a particular area.
flood standard (or designated flood)	the flood selected for planning purposes. The selection should be based on an understanding of flood behaviour and the associated flood risk. It should also take into account social, economic and ecological considerations.

flood storages	those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood.
floodways	those areas where a significant volume of water flows during floods. They are often aligned with obvious naturally defined channels. Floodways are areas which, even if only partially blocked, would cause a significant redistribution of flood flow, which may in turn adversely affect other areas. They are often, but not necessarily, the areas of deeper flow or the areas where higher velocities occur.
high hazard	possible danger to life and limb; evacuation by trucks difficult; potential for structural damage; social disruption and financial losses could be high.
hydraulics	the term given to the study of water flow in a river, in particular, the evaluation of flow parameters such as stage and velocity.
hydrograph	a graph which shows how the discharge changes with time at any particular location.
hydrology	the term given to the study of the rainfall and runoff process as it relates to the derivation of hydrographs for given floods.
management plan	a document including, as appropriate, both written and diagrammatic information describing how a particular area of land is to be used and managed to achieve defined objectives. It may also include description and discussion of various issues, problems, special features and values of the area, the specific management measures which are to apply and the means and timing by which the plan will be implemented.
mathematical/computer models	the mathematical representation of the physical processes involved in runoff and stream flow. These models are often run on computers due to the complexity of the mathematical relationships. In this report, the models referred to are mainly involved with rainfall, runoff and stream flow.
peak discharge	the maximum discharge occurring during a flood event.
probable maximum flood	the flood calculated to be the maximum which is likely to occur.

probability	a statistical measure of the expected frequency or occurrence of flooding. For a fuller explanation see Annual Exceedance Probability.
runoff	the amount of rainfall which actually ends up as streamflow, also known as rainfall excess.
stage	equivalent to 'water level'. Both are measured with reference to a specified datum.
stage hydrograph	a graph which shows how the water level changes with time. It must be referenced to a particular location and datum.

FOREWORD

The State Government's Flood Policy is directed towards providing solutions to existing flooding problems in developed areas and ensuring that new development is compatible with the flood hazard and does not create additional flooding problems in other areas. Policy and practice are defined in the Government's Floodplain Development Manual (ref 1).

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 State Government through the following four sequential stages:

1. Flood Study	Determines 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 developments.
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 environmental plans to ensure new development is compatible with the flood hazard.

The Tuggerah Lakes Flood Study constitutes the first stage of the management process for the floodplain and has been prepared for Wyong Council to define flood behaviour under current conditions.

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SUMMARY

The Tuggerah Lakes flood study was undertaken to determine flood behaviour for 1%, 5%, 20% and 50% annual exceedance probability (AEP) floods and an extreme flood event. The study area extends from Lake Munmorah in the north to Tuggerah Lake in the south. It is linked to the ocean through The Entrance at the southern end of Tuggerah Lake, see figure 1.

Data were compiled for historical flood events to enable the calibration of a mathematical hydrodynamic model of the lake system. Historical flood event water level and rainfall data was available in addition to recent continuous water level data and rainfall pluviographs. Hydrographic survey information of the three lakes and two survey runs for The Entrance channel were available.

A depth averaged looped and branched fully dynamic numerical computer model was applied to the lake system. Three historical flood events were used for calibration and verification of the model. Analysis of wind data and wind setup was undertaken to determine the importance of wind effects on lake levels. An entrance breach mechanism was developed for the calibration and design events.

The flood study results show that the 1% design flood level for Tuggerah Lakes is RL 2.23 mAHD. During floods there is a significant lag time between the flood peak upstream of Wyong and the peak water level in the lakes due to the retardation of flow through various road and railway crossings, and the substantial overbank flood storage areas in the lakes and flooplain downstream of Wyong.

The entrance condition is the single most important aspect controlling flood behaviour in the lakes. Little data on the development of The Entrance during floods is available, but predicted entrance behaviour has been based on available information at The Entrance, documented lagoon breakouts at other locations, and other flood studies where breakouts have been investigated. This is discussed in more detail in section 7.

The model as developed can be used as a basis for a Floodplain Management Study, and is used as the basis of a flood forecast system.

1 INTRODUCTION

1.1 BACKGROUND

The Tuggerah Lakes system is located on the New South Wales Central Coast approximately 80km north of Sydney. Tuggerah Lakes comprises the three inter-connected lakes of Tuggerah, Munmorah and Budgewoi. Tuggerah Lake is connected to the Pacific Ocean by a tidal channel at The Entrance. The lakes have a catchment area of approximately 790km^2 of which approximately 10% is lake area, see figure 1.

Recent increases in development pressures have resulted in a demand to develop areas close to the lakes with water views and cooling breezes. However, these are the areas most likely to be affected by flooding from the lakes. It is estimated that 3700 lots may be affected by the 1% AEP flood level that was adopted at the time of construction of the Munmorah Power Station. Because of the availability of updated rainfall and flood level information and potential for increased development around the Lakes, Council decided to review the current design flood levels for the Lakes. This flood study has been undertaken by Wyong Council to determine design flood levels and develop a model which will provide a basis for assessing future flood management options for the flood liable areas on the foreshores of the lakes.

1.2 STUDY AREA

The study area comprises Tuggerah Lakes (Tuggerah, Budgewoi and Munmorah, figure 2) and its catchment. It also extends seaward for the purpose of establishing ocean storm conditions to define ocean tailwater levels during flood events.

Tuggerah Lake is the largest of the three lakes and is connected to Budgewoi Lake and Lake Munmorah by narrow channels, both with road bridge crossings. The Wyong River and Ourimbah Creek catchments are the largest catchments contributing to the Lakes system, and drain into the southern end of Tuggerah Lake. The lakes system is connected to the ocean via a single tidal channel through the barrier dune at the southern end of Tuggerah Lake. The channel generally remains open but closes occasionally due to littoral processes unless it is maintained open by dredging programs which have existed since 1993.

1.3 STUDY OBJECTIVES

The primary objective of the flood study is to provide Council with design flood levels for Tuggerah Lakes. A second objective, to provide a flood forecasting capability, is described in a separate report (ref 18). A hydraulic model has been developed to assess the nature and extent of the flood hazard in the Tuggerah Lakes floodplain, and the model can be used to determine design flood levels and velocities for the purpose of evaluating floodplain management options.

2 DATA COLLECTION

2.1 GENERAL

The source for all flood level information used in this report is the Tuggerah Lakes Flood Study Compendium of Data (ref 2) which contains the most reliable flood information currently available. Various datums used for identifying flood levels in the past have been difficult to trace and were in some cases undefined, hence the reporting of historical flood levels was unreliable. The Compendium of Data clarifies the various datum shifts and converts the historical flood levels to Australian Height Datum(AHD). For the flood study, additional information required for the development of rainfall-runoff, hydraulic models and breach mechanism understanding included:

- Rainfall data
- Stream gaugings
- Peak historical flood levels
- Wind data
- Road and Rail Bridge information
- Hydrographic survey of the Lake system and Entrance
- Entrance photography bore hole sampling data
- Wave data
- Ocean level/tidal data

2.2 RAINFALL INFORMATION

All available daily rainfall records were obtained from stations controlled by the Bureau of Meteorology (BOM) in and near the catchment. The rainfall data was of reasonable quality with a long record of daily readings. Additional information was obtained from NSW Public Works who also operate rainfall stations in the catchment. The locations of available rainfall stations are shown in figure 3.

2.3 FLOOD LEVEL DATA

Water level recorders were installed by NSW Public Works (PWD) in 1985 at Toukley on the northern end of Tuggerah Lake, and Killarney Vale at the southern end of the lake to determine the impact of wind setup on lake levels. The Killarney Vale gauge was moved to Long Jetty in 1991, approximately two kilometers to the east. Three years of hourly records from 1990 to 1993 were obtained in order to investigate the correlation between wind setup and flooding events.

Peak historical flood levels were obtained from the Tuggerah Lakes Compendium of Data (ref 2).

2.4 STREAM GAUGING

The NSW Department of Water Resource (DWR) operates stream gauges at several locations, as shown in figure 3. Gauging information was obtained for the calibration events to assist in the calibration of the hydrologic models. Table 1 lists the stations and their period of records that were obtained for the study.

Table 1 DWR stream gauging data

STATION No.	STATION NAME	PERIOD OF RECORDS OBTAINED
211002	Wyong River @ Wyong	01/05/1959 - 01/06/1966
211004	Jilliby Creek @ Olney	01/02/1961 - 01/01/1989
211005	Ourimbah Creek @ Tuggerah	01/10/1965 - 25/05/1989
211006	Wallarah Creek @ Warnervale	01/01/1965 - 15/07/1976
211007	Wyong River @ Wyong Weir	01/06/1966 - 01/01/1972
211009	Wyong River @ Gracemere	01/12/1972 - 17/02/1993
211010	Jilliby Creek @ Durren Lane	01/12/1972 - 23/04/1991
211013	Ourimbah Creek Upstream Weir	01/01/1976 - 18/02/1993
211014	Wyong River @ Yarramolong	01/01/1976 - 04/05/1992

2.5 WIND DATA

Wind speed and direction data as three hourly 10-minute averages were obtained from the BOM for Norah Head for the period 1990 - 1992.

2.6 ROAD AND RAIL BRIDGE PLANS

Road bridge general layout drawings for crossings of Ourimbah Creek and Wyong River were obtained from the Roads and Traffic Authority (RTA) Bridge Department. Structural drawings were obtained for the Sydney-Newcastle Freeway crossings of Ourimbah Creek and Wyong River, and the Pacific Highway crossing of Wyong River.

The State Rail Authority (SRA) provided general layout drawings for the Great Northern Railway crossings of Ourimbah Creek and Wyong River.

2.7 HYDROGRAPHIC SURVEY DATA

Hydrosurvey for Tuggerah Lake, Budgewoi Lake, Lake Munmorah and The Entrance were provided by NSW Public Works. The lake and Entrance surveys were undertaken in 1975 with an additional Entrance survey in 1993.

Additional cross section information for Wyong River was obtained from hydraulic modelling work undertaken for the Lower Wyong River Flood Study Review (ref 3). Cross sections of Wallarah Creek were obtained from the Wallarah Creek Flood Investigation Stage 1 report (ref 4).

2.8 ENTRANCE PHOTOGRAPHY AND BEDROCK

A set of 51 aerial photographs of The Entrance for the period 1941-1993 were available from the NSW Public Works photographic library. Bore hole locations and depths of the bedrock below the entrance were obtained from Wyong Council (ref 5&6).

2.9 OCEAN TIDE AND WAVE DATA

Because no water level records have been taken offshore of Tuggerah Lakes, water level information was obtained from the NSW Public Works Stockton Bridge continuous water level recorder in the Hunter River. Because the Hunter River is deep, the site represents a good measure of the ocean water level conditions.

Wave data from the Offshore Botany Bay waverider buoy system was obtained from the Maritime Services Board. The 19 years of data represent the longest and most reliable records close to The Entrance.

3 STUDY METHODOLOGY

The flood study centres on the development and verification of hydrologic and hydraulic models of the lakes, incorporating the following key physical influences:

- Inflows to the lake from the surrounding catchments
- Lake stage/storage characteristics
- Lake/Ocean interface (entrance breach development)
- Ocean level conditions

The approach adopted was to use a rainfall-runoff model to describe the inflows to the lake system, with calibration of the models based on historical flooding information. Sensitivity work was undertaken to determine if flood levels were likely to be influenced by catchment runoff parameters. Design flood hydrographs were estimated using rainfall patterns from Australian Rainfall and Runoff (ref 10).

A calibrated WBNM (ref 8) rainfall-runoff routing model of the Wyong River catchment was available from a previous study (ref 9). The model was extended to include Ourimbah Creek, Wallarah Creek and other residual catchments draining to the lake system, including the lake area. The model was used to provide flood hydrographs for the three calibration events, the design events and the extreme flood estimate.

The lake stage/storage characteristics were simulated using an unsteady hydraulic model which was capable of describing the variations in bathymetry of the lake system and the dynamics of the ocean entrance (breach development). The choice of model depends on the complexity of the particular flooding situation. In the case of Tuggerah Lakes, a one dimensional flow model was considered appropriate to determine flooding behaviour because the response time of the lakes is very slow, and overall velocities would be generally low in the body of the lake. The MIKE-11 system (ref 7) was chosen as the most suitable model because it was able to accommodate all the important physical processes, including entrance dynamics and wind effects.

Ocean conditions near the Entrance were investigated by using a reverse-ray refraction model to bring information from the offshore wave climate in to the local region of The Entrance, and a surf zone model to provide an estimate of the local wave breaking and shoaling processes, see section 6.

Joint probability analysis was undertaken using recorded flood levels and recorded offshore wave event information.

The Tuggerah Lakes Flood Study Compendium of Data (ref 2) estimates severity ratings for known historical storms from 1867 to 1992. These severity ratings have been used as a basis for selecting storms for the calibration of the Tuggerah Lakes hydraulic and hydrologic models. Three categories of severity were used, from 1 for the most severe storms on record, decreasing to 2 and 3 for less severe storms. One storm from each category was chosen to represent each rating.

The three historical floods selected for calibration of the models on the basis of severity of flooding and availability of data are as follows:-

- February 2 - 08 1990 Category 1 Storm
- February 8 - 13 1992 Category 2 Storm
- August 1 - 05 1990 Category 3 Storm

Flood level information from the lake recorders were available for these events on an hourly basis to provide calibration information for the hydraulic model. Flood inflow hydrographs were estimated using the calibrated hydrologic model. During modelling of these floods, the width and depth of the entrance (breach development) was varied as part of the calibration process.

Sensitivity to hydraulic and hydrologic parameters, and the entrance breach mechanism was undertaken. Wind setup was investigated to determine if flood levels were likely to be influenced by high winds.

Design flood levels were predicted by applying design rainfall temporal patterns and volumes to the calibrated rainfall-runoff model, and applying those hydrographs to the calibrated hydraulic model. The breach mechanism used for the calibration events was used for determining the design flood behaviour. Design tailwater conditions were based on joint probability analysis and extreme value analysis of wave climate information.

The results of the design flood prediction process described in sections 4, 6, 7 and 8 were then compared with the frequency analysis described in section 5.

4 HYDROLOGIC MODELLING

4.1 RAINFALL INFORMATION

Table 2 shows the rainfall stations for which hourly recorded data was available for the selected calibration events.

Table 2 Available Rainfall Data – Calibration Events

	Rainfall Data Available		
Rainfall Station	**February 1990**	**August 1990**	**February 1992**
Whitemans Ridge	✓	✓	✓
Summerlees	✓	✓	✓
Wyong	✓	✓	
Mardi Dam	✓	✓	✓
Chittaway	✓	✓	
Warnervale			✓
Lisarow			✓
Bateau Bay			✓

Rainfall isohyetal maps for each event were developed based on recorded rainfall volumes at the above rainfall stations (see Fig 3). Rainfall volumes were for the WBNM model sub-areas were then determined from the isohyetal maps. Rainfall temporal information were assigned in the same pattern as used in Reference 9.

4.2 WBNM MODEL DEVELOPMENT

The Tuggerah Lakes system covers a catchment area of approximately 790 km^2 which was divided into a total of 43 sub-catchments for rainfall-runoff modelling. The Wyong River and Ourimbah Creek catchments provide the majority of the catchment area draining to the Lakes, with 160 km^2 contributing from Ourimbah Creek, and 447 km^2 from the Wyong River catchment. The WBNM model developed as part of the Upper Wyong River Flood Study (ref 9) had 25 sub-catchments. The Ourimbah Creek catchment consisted of 6 sub-catchments. The remaining area consists of 12 individual catchments each discharging directly into the Lakes system. The catchment layout is shown in figure 4.

Approximately 10% of the total catchment area consists of the three lakes area. This area was not included in the hydrological model because there would be no hydrological losses, and was included as an inflow directly into the hydraulic model.

4.3 HYDROLOGIC MODEL CALIBRATION

Where extensive reliable recorded flood data is available for historical storms, it is possible to calibrate both hydrological models and hydraulic models independently. Hydrological models are typically calibrated by matching recorded flows during an historical flood with the model-predicted flows. In this case however, no reliable flow records for significant floods are available in the Tuggerah Lakes area. In the Upper Wyong River Flood Study (ref 9) it was noted that due to the sparseness and poor quality of flow data it was not possible to obtain an accurate calibration of the hydrological model. As a result the calibration of models in the present study was achieved by linking the hydrologic and hydraulic lake models and calibrating to historically gauged levels in Tuggerah Lakes. While this process has less certainty in calibration then a complete data set, it is the only practical method of developing reliable prediction tools for the Tuggerah Lake system.

The WBNM model was originally calibrated (ref 9) using the following parameters.

C	=	1.15
n	=	0.23
Initial Loss	=	15 mm
Continuing Loss	=	2.5 mm/hr

The C parameter used in the WBNM model modifies the lag or storage effects within the model, while the n parameter is a non linearity parameter. A linear catchment is represented by an n value of zero. The initial and continuing losses are generally acceptable on a regional basis, and sensitivity analysis as part of the Upper Wyong Flood Study indicated that there was little sensitivity to the parameters.

Flow data from the hydrologic model for various C and n parameters was entered directly into the lake hydraulic model to produce a stage hydrograph that was compared to recorded lake levels at Toukley for the February 1992 storm event. The timing of the flood peak using the original calibration parameters was approximately 12 hours early. Varying the hydrologic model parameters significantly did not resolve this timing difference.

By investigation of hydraulic impacts of the various road and railway crossings of Wyong River, Ourimbah Creek and Wallarah Creek, the timing difference between the recorded and modelled hydrograph shapes was found to be caused by the hydraulic controls in the lower reaches of Wyong River and Ourimbah Creek. As a result the original set of calibrated parameters, as shown above, were adopted for the hydrological model and calibration was undertaken using the hydraulic model. Because the hydraulic routing was dominant in the lower part of the Wyong River, and the total volume of water draining from the catchments is not sensitive to C and n, the choice of WBNM parameters was not modified from the parameters calibrated in reference 9.

The detailed hydraulic model calibration process is described in detail in section 7.

4.4 RESULTS

The estimated peak flows for the historical storms are shown in Table 3. Figure 5 shows the February 1990 storm hydrographs, figure 6 shows the August 1990 storm runoff hydrographs, and figure 7 shows the February 1992 storm runoff hydrographs for Wyong River and Ourimbah Creek.

Table 3 Estimated Calibration Event Flood Peak Flows

Catchment	**Modelled Peak Flow (m^3/s)**		
	Severity 1 Storm February 1990	**Severity 2 Storm February 1992**	**Severity 3 Storm August 1990**
Wyong River	1439	1145	501
Ourimbah Creek	619	275	256
Total Lake Catchment	2205	1530	703

The flows of table 3 are peak flow rates from the hydrologic model, and do not contain allowances for any hydraulic effects. Because of the impact of hydraulic ratings care should be taken in direct comparison with quoted flows for similar events in other reports (ie ref 3).

5 FREQUENCY ANALYSIS

5.1 GENERAL

Historical peak flood level data were provided for the period 1927–1992 from the Compendium of Data (ref 2). A theoretical flood frequency distribution can be fitted to the data series either by mathematical techniques or by graphical methods. Both methods are described in Australian Rainfall and Runoff (ref 10). From 1927 to 1960 there are only 6 peak annual water level records available, but these are some of the largest floods on record. The remaining period of record from 1961 to 1992 contains 26 peak annual water level records.

Because a complete annual series could not be established for floods prior to 1961, the graphical technique was considered the most appropriate, particularly for a lake system where peak flood discharge is not directly related to the water level due to the influence of the entrance.

5.2 PEAK STAGE FREQUENCY CURVE

In a tidally-influenced lake system such as Tuggerah Lakes, water level frequency curves have strong upper and lower bounds. Typically the upper bound is the peak water level to which the lake would rise under Probable Maximum Flood (PMF) conditions and the frequency curve should asymptote to this level. The lower bound of the frequency curve represents the lowest level of a peak annual water level in the lake. For this analysis it was assumed that the lower bound would be close to the mean annual water level of approximately 0.2 to 0.3 mAHD, hence the frequency curve would be expected to asymptote to this lower bound. The shape of the frequency curve between the upper and lower limits was determined by a graphical technique based on the available data.

Graphical fitting involves estimating the probability of historical levels using a plotting position formula, and then plotting on probability paper and fitting a frequency curve by eye. For the analysis of the Tuggerah Lake levels, two methods were used to define approximate plotting positions of the historical peak flood data, both of which have merit at different frequency levels. In addition, an analysis was undertaken on the seven years of available data from continuous water level recorders to further assist in determining the 50% AEP flood level. The final line of best fit frequency curve was developed from these various plotting position considerations. The two main techniques are detailed below:-

<u>Method 1</u>

The first method is to assume that the 32 available records from 1927 to 1992 were a continuous record of 32 consecutive years and complete an annual series frequency analysis. This method would be expected to overestimate the frequency of the larger floods because there is an effective decrease of the length of record and an increase the number of large floods in this period. The smaller events will also be considered to occur with a higher probability than the data indicates, because if we consider that many of the smaller events are not recorded then the true probability of these smaller events will be higher than the data indicates Consequently this method will overestimate the frequency of occurrence of large flood events, but for smaller events will provide a better frequency estimate than method 2.

Method 2

The second method is to assume that there is a continuous record of 66 years and the 32 individual flood records represent the highest floods that occurred during this period. This assumption is not unreasonable because the higher floods are generally recorded while the smaller floods are not. If this method is used, the probability of the higher floods becomes much more reliable because we preserve the true length of the record over which these larger flood events occurred. However, the smaller flood events with much fewer records will become much less frequent when compared to the total length of record and consequently will have been plotted with a much lower probability than the true frequency would indicate. Consequently this method will underestimate the frequency of small events but provides a reliable estimate of the larger events.

The historical data are plotted by both methods on log-probability paper as shown in figure 8. Based on the above discussions, it is apparent that the adopted frequency curve should approximate the second method plotting positions at the more frequent events, while the adopted curve should approximate the first method plotting positions for larger flood events.

To define the frequency curve between these two extremes, further analysis was undertaken to determine the 50% AEP level. Using the longest available continuous annual series of recorded data, the best continuous sample available from the recorded data set is for the period 1983 to 1992. This length of record is sufficient to provide a reasonably accurate estimate of the 50% AEP level. The 50% AEP level derived was 0.95m AHD.

In order to develop the limits of the flood frequency curve, the PMF estimate from the hydraulic model (as described in Section 8) was also plotted. The PMF estimate is approximately 0.5m higher than the 1% AEP level which compares well with the estimate in reference 20 which estimates the PMF level to be approximately 2 feet (0.61m) higher than the 1% AEP level.

The overall shape of the curve was compared to that produced by the Cities Commission Report (1974) (ref 20). Based on the available information, a line of best fit was plotted, and is shown in figure 8. The levels derived from this curve are shown in table 4.

Table 4 Frequency Curve Based Design Flood Level Estimates

Frequency	Flood Level mAHD
1%	2.20
5%	1.90
20%	1.35
50%	0.90

6 ELEVATED OCEAN CONDITIONS

6.1 GENERAL

Ocean water levels vary continuously as a result of forcing by a range of physical phenomena. Some of these variations are of short period, some irregular and some related to longer term meteorological variations. The principal components of elevated ocean water levels are:-

- astronomical tide
- storm surge
- wave setup
- coastal trapped waves

The methodology adopted for the elevated ocean water levels was to investigate the contribution of each component, and to combine them together to provide an estimate of the appropriate tailwater conditions for the calibration and design events.

6.2 ASTRONOMICAL TIDE

Astronomical tide levels are deterministic and can be predicted using a range of harmonic constants. Because there is little offshore tidal variation along the NSW coast, tidal constants for Sydney (ref 11) are applicable to Tuggerah Lakes.

6.3 STORM SURGE

Storm tides occur when meteorological conditions cause an increase in water level above the predicted astronomical tide. The increment in water level is commonly called storm surge. Storm surge has two components. They are:-

- wind setup
- inverse barometer effect

Wind setup is caused by the shear stress between the wind and water surface which causes a net flow. When this flow is directed shoreward, the water can "pile up" against the shoreline causing setup. The impact of wind is proportional to the square of wind speed and is inversely proportional to water depth. Because the near shore water depth is relatively large, there will be minimal wind setup at the Entrance.

The inverse barometer effect is caused by a drop in local atmospheric pressure below "average" local pressure levels. A one hectopascal (HPa) drop in pressure corresponds to a one centimetre rise in average ocean level.

6.4 WAVE SETUP

Wave setup develops shoreward of the breaker zone where significant wave energy dissipation occurs. In order to maintain conservation of momentum, the still water level increases non-linearly in the shoreward direction. At river and lagoon entrances the situation becomes more complicated because waves often have not broken completely by the time they propagate to the entrance. Additionally, the waves themselves are affected by flood flows against the direction of wave propagation. For the full wave setup level to develop inside the entrance, a significant volume of water must be transported through the entrance to the lake system. Because the volume of storage in Tuggerah Lakes is quite large, wave setup impacts will be relatively small at The Entrance under normal opening conditions.

6.5 COASTAL TRAPPED WAVES

Coastal trapped waves are caused by the movement of large-scale meteorological systems over the southern oceans, and may cause changes in the mean tide level of up to 0.1 metres over a period of several days.

6.6 WAVE CLIMATE AT THE ENTRANCE

Details of the wave climate computations are included as Appendix B, and are described here in general terms only.

Estimation of the offshore wave climate at the Entrance was based on 19 years of offshore Botany Bay Waverider buoy data collated with wave directions derived from daily synoptic charts. The offshore wave climate was transposed to the nearshore area of the lake entrance using a wave refraction model (ref 12) which uses the reverse ray frequency-direction spectral wave refraction method. The sea bed bathymetry between offshore and onshore was developed from bathymetric charts.

The results of the wave refraction computation were then used to estimate inshore wave climate using a surf zone model (ref 13).

Data from the Botany Bay waverider system provided the following wave information for the three calibration events.

Table 5 Calibration event recorded ocean wave conditions

Calibration Event	Peak Storm Hs(m)	Zero Crossing Period(s)	Offshore Direction
Feb 2 - 7 1990	3.7	6.7	62
Aug 1 - 5 1990	7.2	7.3	133
Feb 9 - 10 1992	4.5	5.9	148

Only the storm of August 1990 could be classified as a severe ocean storm. Wave setup in a range of water depths was calculated in order to predict the level of wave setup as the entrance scoured and tide level changed.

Table 6 Estimated wave setup (m) at the entrance for historical flood events

	Entrance Depth (m)			
	0.5m	1.0m	1.5m	2.0m
Calibration Event				
Feb 2-7 1990	0.21	0.1	0.05	0.00
Aug 1-5 1990	0.45	0.34	0.3	0.25
Feb 9-10 1992	0.25	0.14	0.1	0.05

The severe storm of August, 1990 would have contained wave setup impacts of approximately 0.45 metres when the entrance was not opened, and reducing to 0.25 metres as the entrance deepened and scoured under the influence of the flood flows. The wave setup for the February 1990 and October 1992 events would have been small because the entrance was scoured to a depth of 2m before the flood occurred.

The results given in Table 6 indicate that for larger design flood events, where the entrance could be expected to scour to depths greater than 2 metres, and would widen considerably, the impact of wave setup would likely be small. The actual values of wave setup would be related to wave heights adopted for the design flood events, and the depth of entrance scour predicted by the calibrated hydraulic model.

7 HYDRAULIC MODELLING

7.1 MODEL DEVELOPMENT

The hydraulic model was developed to include those areas of the lakes and foreshores that are subject to flooding.

Cross sections for the Lakes were obtained from hydrographic survey supplied on a 1:4000 orthophoto base. In view of the need to model areas above the high water line, the cross-sections were extended to the 4 metre AHD contour as shown on the orthophotos. The information provides a reasonable basis for predicting flood levels as the floodplain area above high water does not increase rapidly up to the 4m contour. Flood extents can be determined in local areas by combining the flood levels with specific ground survey. The lower reaches of the Wyong River provide substantial floodplain areas which may be inundated during a 1%AEP event, and these were included by using the relevant ground survey information from the Lower Wyong River Flood Study Review (ref 3).

The hydraulic model included Lake Munmorah, Budgewoi Lake, Tuggerah Lake and the Entrance channel (figure 9). Additional model branches were included to represent Ourimbah Creek, Wallarah Creek and Wyong River up to the Sydney-Newcastle Freeway. The model as developed would be suitable for investigating a second entrance by including an appropriate branch.

Wyong River cross-sections were derived from reference 9, while the Wallarah Creek Cross-sections were digitized from cross-section plots in reference 4. Cross-sectional information for Ourimbah Creek was available from hydrographic survey.

At Wyong, the model included the old Pacific Highway and Great Northern Railway bridges. Based on bridge drawings obtained from the RTA and SRA, the bridges were combined to represent a single hydraulic structure, with increased headloss factors to account for the complexity of the crossing. Upstream of the highway bridge, a storage area was included in the model to represent the volume of stored water that would be held upstream of the bridge through Wyong River inbank and overbank storage. The storage characteristics upstream of the road crossings over Ourimbah Creek were incorporated in a similar fashion. The volume of storage required was estimated by broad scale mapping and then varied as part of the calibration process.

7.2 ENTRANCE BREACH DYNAMICS

The entrance to Tuggerah Lake is formed by a narrow mobile dune barrier. Normally the entrance develops near the southern headland where the bedrock formation restricts the potential erosion depth of the entrance. During a flood the entrance scours vertically and horizontally. The scoured channel provides a regime which is conducive to influx of sediment during flood tides with less transport on ebb tide. This process leads to a gradual closure of the entrance over a period of several months following a flood event. It is unlikely that supercritical flow occurs in this entrance, as it does in others, because the water level difference between the ocean and Tuggerah Lake is not large enough. This implies that the ocean conditions are likely to have some impact on lake flood levels throughout flood events.

Inspection of historical aerial photographs of the entrance area shows that the entrance shoal system extends across the whole entrance upstream a distance of about 500m - almost to The Entrance Bridge, with braided tidal streams forming a patchwork of channels. Significant erosion during a flood appears to be restricted to an area near the entrance. During more severe floods, flow is likely to occur over the low sand barrier dune to the north of the incised channel, and to erode the barrier dune as the channel widens.

Recorded flood hydrographs show that floods rise relatively rapidly, but recede quite slowly and may take several days to return to normal. The combined incised channel and a long overflow barrier (or weir) entrance concept would be consistent with observed flooding behaviour. The breach model proposed was consistent with previous observed manual opening events (ref 16 and 17). Figure 11 shows the variation of water level, discharge, entrance width and depth and Froude number during a typical calibration event, and provides very similar behaviour to that proposed in reference 16. A detailed description of the MIKE-11 breach model and sediment transport erosion mechanism are described in Appendix E.

7.3 WIND SETUP

An analysis of likely wind setup effects during flood events on Tuggerah Lake was undertaken to determine the possible impacts of storm conditions. Perceptions that wind setup would cause a significant water level difference in the lake during flood events were investigated, but no firm evidence could be identified to support the theory during specific historical events. The analysis for the impact of wind was therefore based on recorded information at Killarney Vale (Long Jetty) and Toukely.

The differences in water level between Toukley and Killarney Vale (Long Jetty) and the wind data from Norah Head were analysed on an hourly basis to provide a continuous data set from 1/1/90 to 30/6/92. Lake setup was calculated as the difference in level between the Toukley and Long Jetty water level gauges. A negative setup indicated that the water level at Long Jetty in the south was higher than at Toukley in the north.

The data as plotted in figure 10 for lake level vs setup indicate that there is most likely to be a negative setup (Toukley lower than Long Jetty) during periods of high lake levels. The wind and setup data for a selection of lake levels greater than 1 mAHD are provided in table 7, and show that the negative setup is contradictory to the wind data in which the wind is generally from the south and east during these events. One reasonable explanation is that the majority of the catchments drain from the southern part of the lake system via Ourimbah Creek and Wyong River, and there would be a flood gradient from south to the north as the water flows in a northward direction. From the data, it was considered that wind setup has negligible impact on lake flood levels.

Table 7 Lake level and wind speed vs setup

Date	Lake Level mAHD	Wind Speed kph	Direction deg N	Setup m
1990/2/6	1.36	18.4	180	-0.09
1990/2/6	1.29	25.9	135	-0.17
1990/2/6	1.21	33.5	90	-0.10
1990/2/7	1.04	33.5	135	-0.04
1990/2/7	1.15	44.3	90	-0.07
1990/2/7	1.08	44.3	90	-0.16
1992/2/9	1.08	33.5	90	-0.05
1992/2/10	1.11	29.5	22.5	-0.06
1992/2/10	1.10	22.3	67.5	-0.06
1992/2/10	1.12	40.5	45	-0.12
1992/2/11	1.04	40.5	45	-0.07
1992/2/11	1.00	20.5	45	-0.05

Figure 10 also shows that the larger positive setups tend to occur during periods of very low lake levels (of the order of 0.3 meters), which is likely to be due to the increased friction at low depths. As water depths increase, bed friction on the circulating current reduces and setup is reduced.

To investigate the potential wind setup that could occur during a large flood, wind was applied to the hydraulic model with February 1990 flood inflow hydrographs and wind speeds up to 20 m/s (72 km/h). The results suggest that a maximum setup of approximately 0.2 metres is possible, but would require the wind to come from a specific direction for a prolonged period of time. The data do not indicate that the wind climate has the required characteristic to attain the theoretical setup.

As wind setup is small during flooding and hydraulic gradients due to flood flows are of the same order as setup gradients, the issue of wind setup is inconclusive and wind effects were not included in the design flood model runs.

7.4 CALIBRATION

Preliminary model runs indicated that peak flood level in the lake system was sensitive to the initial entrance breach width and level, while the recession limb behaviour was sensitive to the rate at which the breach changes during the flood. Calibration was achieved by producing an initial breach state and breach opening mechanism that would contain the physical elements described in section 7.2 and reproduce the observed water level time series for the three calibration events.

The typical breach mechanism for the calibration events comprised of a small incised channel with a long overflow weir stretching to the north of the entrance. As the flood event progresses, the incised channel gradually increases in depth and width. When the water level rises above the point where it is confined to the incised channel, the conveyance of the breach increases rapidly, producing the characteristic flat peak of the observed hydrographs.

February 1990 Event

The calibrated WBNM hydrographs were applied to the hydraulic model. The initial water level in the lakes was 0.1 metres AHD, which was the recorded level at the Toukley gauge at the start of the model run approximately 3 days before the flood peak. The tailwater condition was a water level time series from the recorded Stockton Bridge gauge at hourly increments. The initial breach specifications consisted of a channel 20m wide and with the bed at RL -2mAHD. The breach was eroded so that after 3 days, the width had increased to 250m. These characteristics provided a reasonable calibration of the rising limb of the hydrograph. It was found that if the breach was kept open past the peak stage, then modelled flood levels dropped too quickly on the recession limb. Since the entrance dynamics investigations indicated that there would likely be an infilling of the entrance due to a change in the sediment transport regime and littoral processes after the flood peak, the entrance was progressively reduced in size to calibrate the recession limb of the hydrograph, the results of which are shown in figure 12 .

August 1990 Event

The calibrated WBNM hydrographs were applied to the hydraulic model. The initial water level in the lakes was taken from the recorded level at the Toukley gauge. The August 1990 flood was characterised by a breach that was initially low with a relatively wide incised channel at the southern end. The recorded water level time series of the flood indicates two flood peaks (figure 13), but no physical basis for the timing of the water level shape could be determined . This flood was a result of a combination of runoff hydrographs, relatively large ocean waves and a comparatively high spring tide. The recorded water level at Stockton Bridge would contain the tide and inverse barometer components, and the only additional influence in the tailwater condition would be the wave setup, which was estimated in section 6. Wave setup would not be expected to cause a dip in the water level as is shown in the recorded water level at Toukley, so the observation is considered unlikely to be caused by the tailwater.

Sensitivity to lake inflows and ocean conditions were investigated to assess impacts on the hydrograph shape. This flood is unique in comparison to the February 1990 and February 1992 flood in that flood levels in Ourimbah Creek peak significantly earlier than those in Wyong River. Because both Creeks had similar flood peak magnitudes, this indicates that the storm event moved in from the South with significant rainfall over the Ourimbah catchment before moving into the Wyong catchment. In order to attempt to reproduce the observed behaviour, the Wyong River inflow hydrograph was lagged approximately 14 hours and the model was re-run. The results of this run showed that there was not enough volume in the Ourimbah Creek inflow hydrograph to reproduce the rising water level time series. The Ourimbah Creek inflow was doubled to compensate and the model re-run. Results indicate an improvement in the modelled water level time series but were not sufficient to provide a completely satisfactory result.

February 1992 Event

The calibrated WBNM hydrographs were applied to the hydraulic model. The initial water level in the lakes was taken as the recorded level at the Toukley gauge. The February 1992 flood was recorded by two water level recorders, Toukley and Long Jetty. The tailwater condition was a water level time series from the recorded Stockton Bridge gauge at hourly increments. The recorded data shows that flood levels at the Toukley gauge were lower than Long Jetty (figure 14). This level difference is typical for that observed in other smaller flood events, and represents an hydraulic gradient from the south to the north of Tuggerah Lake as the flood waters entering at the southern end of the lake travel north to Budgewoi Lake and Lake Munmorah.

May 1974 Ocean Storm Event

One of the most severe ocean storms to have affected the central coast of NSW in recent times is the storm which peaked on 26th May, 1974. The estimated peak water level at the Fort Denison (Sydney Harbour) tide gauge was 1.44m AHD. A similar peak storm water level of 1.37m AHD was observed at the Port Newcastle tide gauge.

The recorded peak flood level in Tuggerah Lake was between 1.2 and 1.3m AHD with an entrance width of about 40m (based on aerial photograph data). Thus it appears likely that the small entrance restricted inflow of ocean water, and limited the water level rise in the lake. While this event was not modelled in the investigations (because it would not provide any additional information for flood model development), it does show that the entrance can serve to limit the impact of ocean events, and any permanent widening of the entrance will result in an increase of the likelihood of ocean storms causing flooding. Even if a May 1974 storm event were to re-occur with the entrance open to 300m wide, and a small volume of rainfall, the resulting level would approximate only a 20%AEP flood on a frequency basis.

7.5 SENSITIVITY

Sensitivity of the hydraulic model to the roughness parameter, Mannings 'n' was undertaken. Mannings 'n' values in the range of 0.035 to 0.060 were tested on the main body of the lakes with no appreciable impact on water levels or discharges. The entrance roughness was tested, however the resultant changes are relatively small compared to the effects of alternative breach configurations. Because of the inclusion of the various road and railway bridge systems, and the substantial overbank storage areas of the lower Wyong River floodplain, the hydrologic model inputs were found to be relatively insensitive, as the total volume of inflow was unchanged by varying hydrologic model parameters. Sensitivity to breach configuration and tail water conditions is discussed in section 8.9.

8 DESIGN FLOOD ESTIMATION

8.1 GENERAL

Design flood levels were predicted by applying design rainfall temporal patterns and volumes to the calibrated rainfall-runoff model, and applying those hydrographs to the calibrated hydraulic model. A range of durations were tested at the 1%AEP level to determine the critical duration. The breach mechanism was retained as for the calibration events. Design tailwater conditions were based on joint probability analysis and extreme value analysis of water level and wave climate information.

8.2 JOINT PROBABILITY ANALYSIS

Flooding in Tuggerah Lake is influenced by both elevated ocean levels and catchment runoff. Elevated ocean levels are caused by storms which generate strong onshore winds, large waves and have low atmospheric pressure. These factors cause the ocean level at the shoreline or in entrances to estuaries or lakes to be elevated above normal tidal levels. The main components of this increased water level include wind setup, wave setup and inverse barometric setup. This abnormal elevation normally is characterised by a relatively rapid increase to a peak followed by a subsequent decline over periods up to 3 days. This elevation is super imposed upon the normal tidal variation. The probability of the peak elevated ocean level occurring at similar times to the peak lake level due to catchment runoff depends on the response time of the catchment, and the duration of the elevated ocean levels assuming that both are caused by the same storm. This assumption of a common storm is not always valid or if so, not necessarily representative of the most severe conditions.

For the February 1990 and 1992 storms examined in the model calibration, the lake water level peaked at about 48 to 72 hours after commencement of the rainfall while the peak discharge in the inflow hydrographs occurred after 15 to 20 hours. The lakes, as would be expected, fill relatively slowly with the catchment runoff. This relatively long response time of the lake compared to rainfall, limits the possibility for the coincidence of a high lake level due to rainfall runoff and severe elevated ocean levels.

This contention was examined by reviewing the records of lake water level, rainfall, recorded ocean levels, offshore wave heights and wind speeds. Twenty years of reliable records between 1972 and 1992 were available. The main forcing mechanisms were identified for each storm which caused a lake water level above normal levels of about RL 0.25 m AHD. The storms were classified in the following manner to identify the main forcing mechanisms:

1	rainfall only
2	offshore waves only
3	wind only
23	combination of waves and wind
13	combination of rainfall and wind
123	all three mechanisms

A list of all investigated storms and their ratings are included as Appendix C. The highest recorded lake level during the period of complete records occurred in February 1990 at which time there were high offshore wave heights *(Hs = 3.7 m)* and strong winds *(max vel > 100 kph)*. As discussed in Section 6, the penetration of elevated ocean levels into the entrance depends largely on the entrance conditions, especially the water depth. Entrance depths or widths during this storm are unknown, however on the basis of the general calculations in Section 6, the influence of elevated ocean levels on flood levels in the lake is likely to have been small.

For storms in which only waves and wind appeared to be the forcing mechanisms, the highest recorded lake level was 1.3m AHD (May 1974). Generally, those storms causing waves and winds (but not significant runoff) were at the lower end of recorded flood levels in the lake. This suggests that while elevated ocean levels can cause flooding in the lake, the primary cause of significantly elevated flood levels is rainfall runoff.

The dependence of lake flood levels on factors which are the main cause of elevated ocean levels, namely wave heights and wind speed have been examined in section 6.

There was no strong correlation between wind speed and lake flood levels. During the February 1990 flood, the average recorded wind speed was about 32 kph. At each wind speed between 30 to 70 kph, the recorded lake flood levels varied between 0.3 to 1.2 m AHD. However, these elevated lake levels were also associated with high rainfall which is likely to have been the main cause of flooding in the lake for these events.

Similarly, the correlation between recorded offshore wave heights and lake flood levels was weak. The maximum recorded offshore wave height occurring during a flood was about 7 m (Hs) and in combination with a relatively small volume of runoff resulted in a peak lake level of approximately 1.3 m AHD. The majority of the rainfall related floods occurred with offshore significant wave heights less than 4 m. This suggests that the storms causing the most severe wave heights do not cause significant rainfall on this catchment.

The recorded peak flood levels in the lake have typically occurred at times of higher high tide ocean levels *(as measured at Fort Denison)*. Nearly all occurred when recorded high tides were above a level of 0.6 mAHD. The maximum recorded lake level occurred in association with a high tide level of 1.1 mAHD. The maximum recorded high tide level of 1.4 mAHD occurred during high offshore waves *(7m Hs)* and wind speeds, with a resultant peak lake level of 1.3 m AHD which was also influenced by a relatively small volume of runoff. Therefore, during significant rainfall on the catchment it is likely that the storm will cause elevation of ocean levels such that they are typically near or above high water spring levels.

In summary, it would appear from records of the lake floods over the last 20 years that elevated ocean levels influence the more severe lake flood levels. However, it is not considered to be a significant factor compared to rainfall runoff. It is likely that the storms which produce severe rainfall on the catchment do not necessarily cause significant elevated ocean levels. Propagation of the elevated ocean level into the lakes depends upon storm duration and entrance conditions.

The adopted design ocean conditions were to assume a wave condition of 4.5m HS and spring tide coinciding with the flood for the 1% AEP event, and a lower tide of 0.6m AHD and 4.5m Hs work for the 20% AEP event, based on the data presented in Appendix C. The 50% AEP event was considered to be unlikely to coincide with a severe wave event, and a tailwater condition of a 0.6m AHD tide without wave action was adopted.

8.3 DESIGN CATCHMENT FLOWS

The calibrated WBNM model with design rainfall volumes and temporal patterns (ref 10) were used to generate the design flood hydrographs for 1%, 5%, 20% and 50% Annual Exceedance Probability (AEP) events. At the 1%AEP level, a range of durations were generated and run through the calibrated hydraulic model to determine the critical duration. The resulting impact on peak flood level is plotted in figure 15, and shows that the critical duration is the 48-hour storm at the 1% AEP event. This was adopted as the critical duration for all events.

The estimated peak flows for the design storms are shown in Table 8. Figure 16 shows the Tuggerah Lake total Inflow hydrographs for the 48 hour duration storm.

Table 8 Estimated Design Flows (1,5,20,50%AEP Event, 48 Hour)

Catchment	Peak WBNM Discharge (m^3/s) 48-Hour Storm			
	1% AEP event	**5% AEP event**	**20% AEP event**	**50% AEP event**
Wyong River	3055	2099	1366	873
Ourimbah Creek	1278	897	616	415
Total Lake Catchment	4858	3320	2197	1416

The figures given in Table 8 are directly from the WBNM model and do not include hydraulic routing impacts. Care should be taken when making comparisons with other reports (ie ref 3).

8.4 DESIGN BREACH MODEL

For flood calibration, a standard breach model was developed which reproduced the observed water level time series in Tuggerah Lake for each of the modelled events. Because of channel infilling on the falling limb of the hydrograph, it was necessary to define the shape of the breach through the falling limb. For the design events a standard erosion based breach model using the Engelund-Hansen sediment transport formula was developed (ref 7) which reproduced the behaviour of the calibration events on the rising limb. The erosion based breach is defined by initial and limiting breach geometry and a sediment grain size. The initial breach configuration was specified as a 20m wide incised channel and a long overflow weir, while the limiting section was defined as a fully open entrance constrained by the bedrock below the entrance (ref 6). This initial breach condition is consistent with the calibration events. The entrance breaches during each model time step according to the prevailing conditions in the breach.

8.5 DESIGN TAILWATER CONDITIONS

Based on the joint probability analysis, a tailwater condition was developed as a combination of the following components:

- Spring Tide
- Inverse Barometer effect
- Wind and Wave setup

The spring tide was developed using Port Newcastle tidal constants. The wave setup component was determined based on a Hs=4.5 wave offshore, which was determined to be the most likely wave to coincide with floods to be 0.07m based on the joint probability analysis (Section 8.2). The inverse barometer effect was estimated based on an average over a number of flood events in the data of Appendix C. These components were combined to provide a time-varying tailwater condition. Because of the relatively slow response of the lake system, the timing of the peak tide and flood was tested and found to be relatively insensitive by comparison with other parameters.

8.6 DESIGN FLOOD LEVELS

The design levels were produced using the design runoff hydrographs with a tailwater condition as described in section 8.5. The initial lake level was set at 0.3 metres AHD. The erosion-based breach model was incorporated which allowed the entrance to scour based on the hydraulic conditions.

8.7 EXTREME FLOOD

The PMF was determined by applying a PMP estimate to the calibrated WBNM model. The probable maximum precipitation (PMP) was determined in accordance with procedures described in Bulletin 51 (ref 15) as recommended in Australian Rainfall and Runoff (AR&R ref 10). These procedures predict the rainfall resulting from extreme convective cells (thunderstorms) which are considered the most efficient rainfall producing mechanism. The procedure estimates the maximum amount of moisture that is available in the atmospheric column at a maximum dew point derived for any location in Australia.

The method allows for a maximum 6 hour duration storm event for catchments up to 1000 km^2. It is assumed that the PMF for Tuggerah Lakes would result from the Probable Maximum Precipitation (PMP) centered in the catchment of Wyong River and Ourimbah Creek. For a six hour duration storm in rough terrain with a 70% reduction of the extreme moisture index, the total rainfall depth was calculated as 455 mm. The temporal pattern for this event was derived using AR&R. This storm is significantly less than the critical duration 1% AEP event, and produced a lower peak water level in the lake than in the 1% AEP storm when run through the hydraulic model. It was considered that a longer duration extreme flood was likely to be critical for Tuggerah Lakes as the volume of runoff is the principal factor in raising flood levels in the Lake system.

The transfer of PMP estimates for similar sized catchments with similar terrain is appropriate. NSW Public Works had conducted PMF estimates for the Georges River catchment, and was able to supply the information for the purposes of completing a PMP for the Tuggerah Lakes catchments. The Georges River has a catchment area of 890 km^2 (compare Wyong 790km^2) and is of broadly similar topography (RL 500m general top of catchment) and proximity to the ocean. On this basis, it was considered appropriate to transfer these estimates to the Wyong River catchment.

As a first estimate, the rainfall was assumed to be spatially uniform, and was routed through the calibrated WBNM model to obtain inflow hydrographs from the catchments draining to Tuggerah Lakes. The non-linearity calibration parameter of the WBNM model was varied to simulate a range of parameters from non-linear to linear PMF estimates. A zero initial loss and 2.5mm/hr continuing loss were assumed. It has been observed that extreme floods generally behave in a more linear fashion than more frequent events, and a linear catchment response may be more appropriate. The total volume of the hydrograph is relatively unchanged between the linear and non-linear models. The PMF generation parameters adopted were:

C	1.15
n	0.0
Initial Loss	0mm
Continuing Loss	2.5mm/hr

The 6, 24 and 48 hour PMF's were then produced using the above parameters. The 24 hour PMP was found to be critical, producing a PMF level of 2.7mAHD with a rainfall depth of 900mm. Although the lake system was found to have a critical duration of 48 hours for the 1%AEP event, the 24 hour duration PMP occurs so rapidly and is of such large volume that the flood peak arrives before the entrance can fully develop an opening, thereby giving a higher peak water level than the 48 hour PMF.

8.8 SENSITIVITY TO BREACH AND TAILWATER CONDITIONS

A range of sensitivity investigations were conducted to predict the impact of various hydraulic and hydrologic inputs, and are described in section 7.5.

Breach Conditions

The breach mechanism provides by far the most important influence on lake levels. A range of sensitivity runs were conducted by varying individually important breach parameters, namely:-

- Sediment grain size (for rate of erosion)
- Side erosion index (for speed of propagation of the breach)
- Head loss factors (for energy losses through the breach)
- Critical shear stress (for commencement of erosion)
- Initial dune barrier level (crest level)

The results of Sensitivity to breach model parameters are given in Table 9, and should be compared to the final values given in Table 11 for the relative impact.

Table 9 Sensitivity to Breach Model

Breach Parameter	cp. Adopted	Modelled Lake Level 1% AEP	Change
Grain Diameter	+10%	2.23	0
	−10%	2.23	0
Side Erosion Index	+10%	2.11	−0.12
	−10%	2.26	+0.03
Critical Shear Stress	+10%	2.23	0
	−10%	2.23	0
Head Loss Factors	+10%	2.31	+0.08
	−10%	2.13	−0.10
Crest Level	+10%	2.24	+0.01
	−10%	2.22	−0.01

Tailwater Conditions

Sensitivity to ocean water levels was assessed for the 1%, 20% and 50% AEP flood events. The following scenarios were investigated were:−

- 1.32 mAHD elevated ocean level with wave setup.
- 1.32 mAHD elevated ocean level without wave setup.
- 0.60 mAHD elevated ocean level with wave setup.
- 0.60 mAHD elevated ocean level without wave setup.

The results of these sensitivity runs for the 1% AEP event given in Table 10.

Table 10 Sensitivity to Ocean Tailwater Conditions

Design Flood Event	Elevated Ocean Level mAHD	Wave Conditions used for set-up	Maximum Lake Level mAHD
1% AEP	1.32	4.5Hs	2.23*
	1.32	none	2.11
	0.60	4.5Hs	2.22
	0.60	none	2.03
20% AEP	1.32	4.5Hs	1.43
	1.32	none	1.31
	0.60	4.5Hs	1.36*
	0.60	none	1.24
50% AEP	1.32	4.5Hs	1.09
	1.32	none	1.00
	0.60	4.5Hs	1.00
	0.60	none	0.91*

Note: * − adopted design run. Reasons for variation in wave set−up are discussed in Section 8.2.

Hs is the average of the highest one−third of waves observed in a wave record.

8.9 RESULTS

On the basis of the model runs and the frequency analysis work, the design and extreme flood levels are given in Table 11.

Table 11 Design Flood Levels

Frequency	Flood Level mAHD
PMF	2.70
1%	2.23
5%	1.80
20%	1.36
50%	0.91

The results are shown as a single level which is applicable to all areas of the three lakes. It was found that lake levels in the three lakes do vary as the flood rises and falls, but that the peak level is the same in all three. Figure 17 shows the variation in water level between Tuggerah Lake and Lake Munmorah for the 1%AEP event. The figure shows that Tuggerah Lake rises faster than Lake Munmorah, as the majority of inflow is to Tuggerah lake, and that Lake Munmorah drains more slowly, but the peak water levels are very similar and occur at roughly the same time.

On the basis of the overall sensitivity analysis, it is considered likely that the 1% AEP level should be considered to have an accuracy of ± 0.15m.

8.10 RECOMMENDATIONS

In view of the sensitivity of the results to the mechanism of the entrance opening during flood events, the model could be improved by undertaking a long term data collection programme to better define the breach processes.

The gauging stations used to calibrate the hydrologic models were not gauged to flow levels suitable for accurate calibration, and the reliability of the study would be further enhanced by gauging teams being available to capture data during future flood events.

9 ACKNOWLEDGEMENTS

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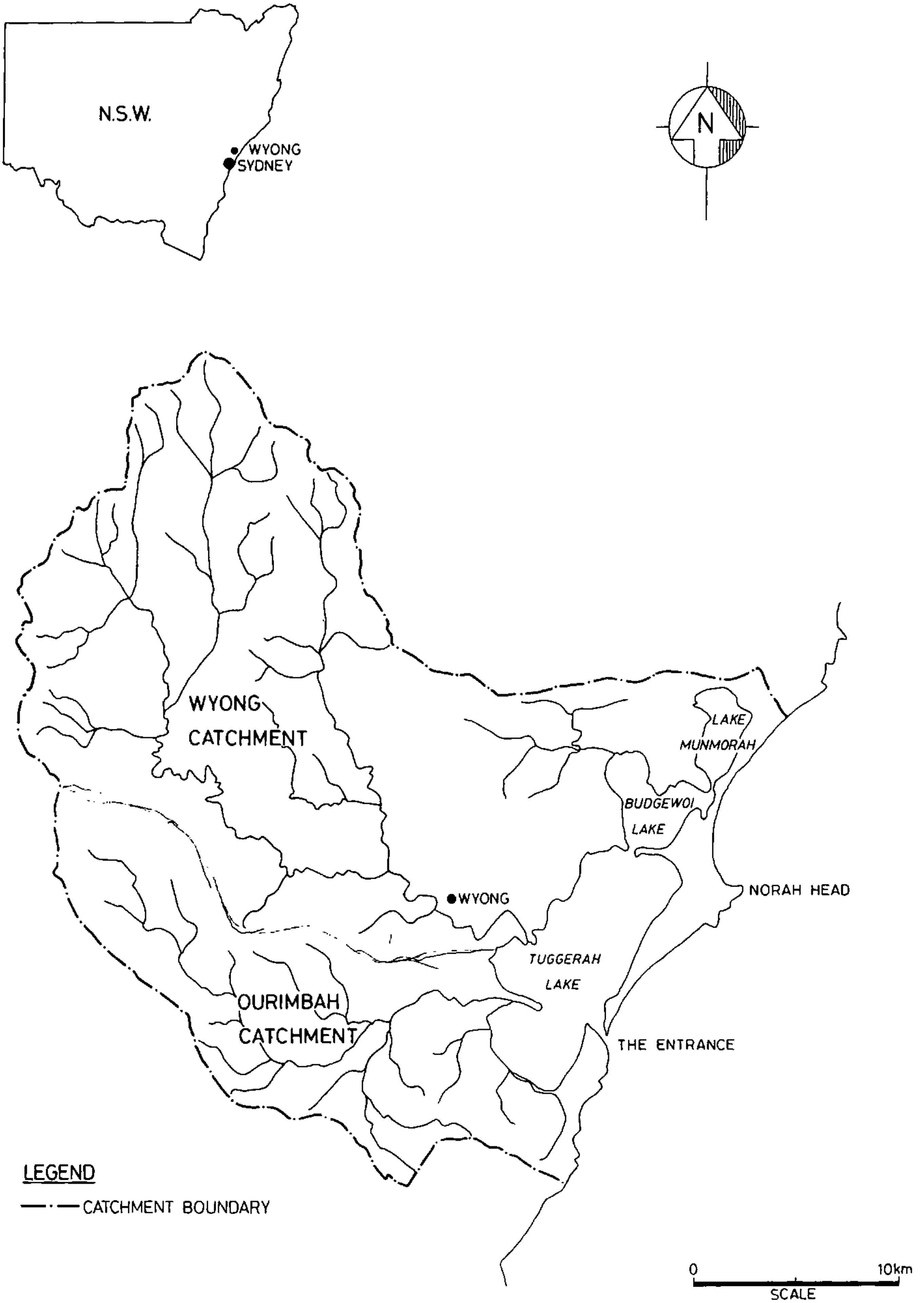
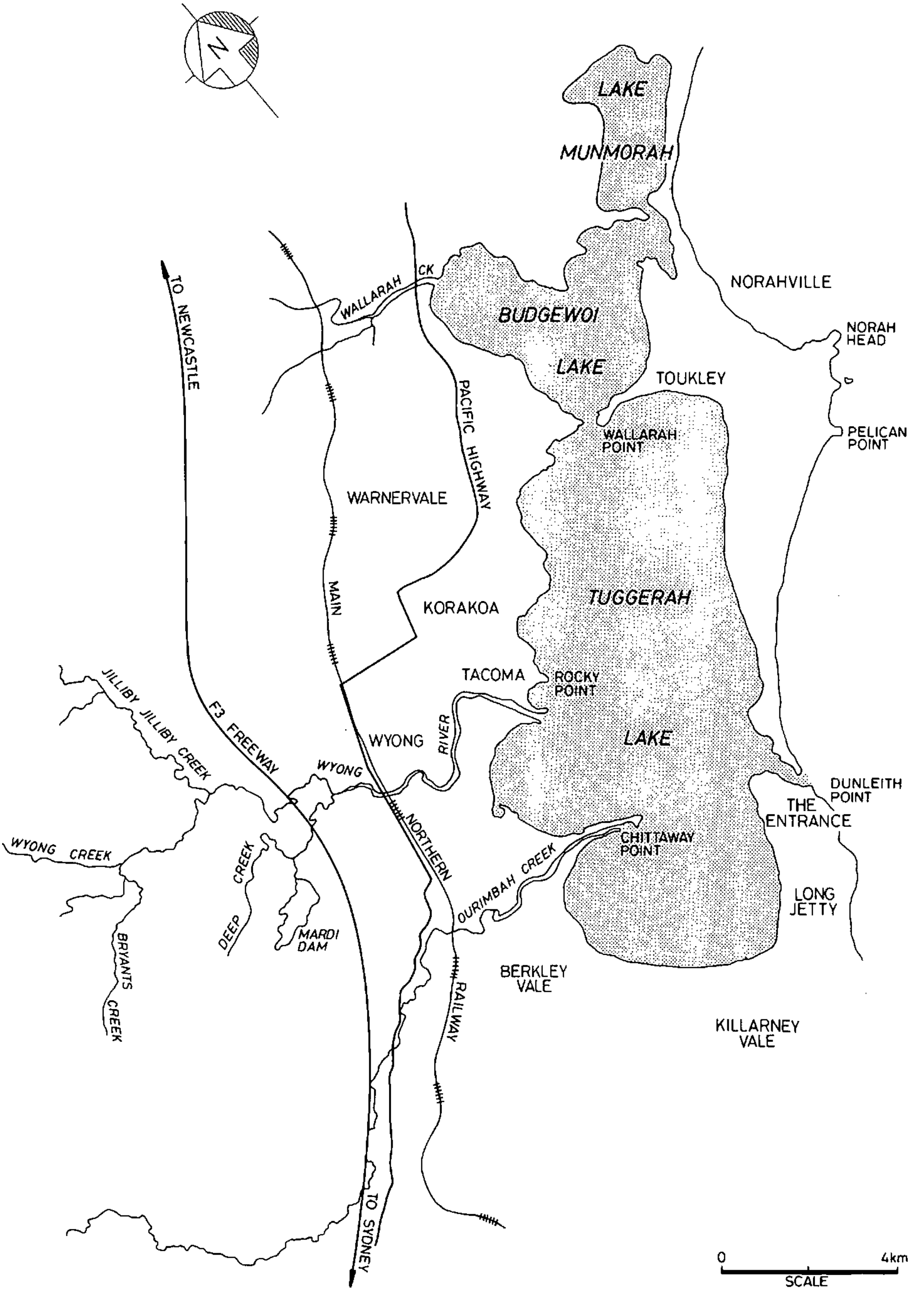
FIGURES



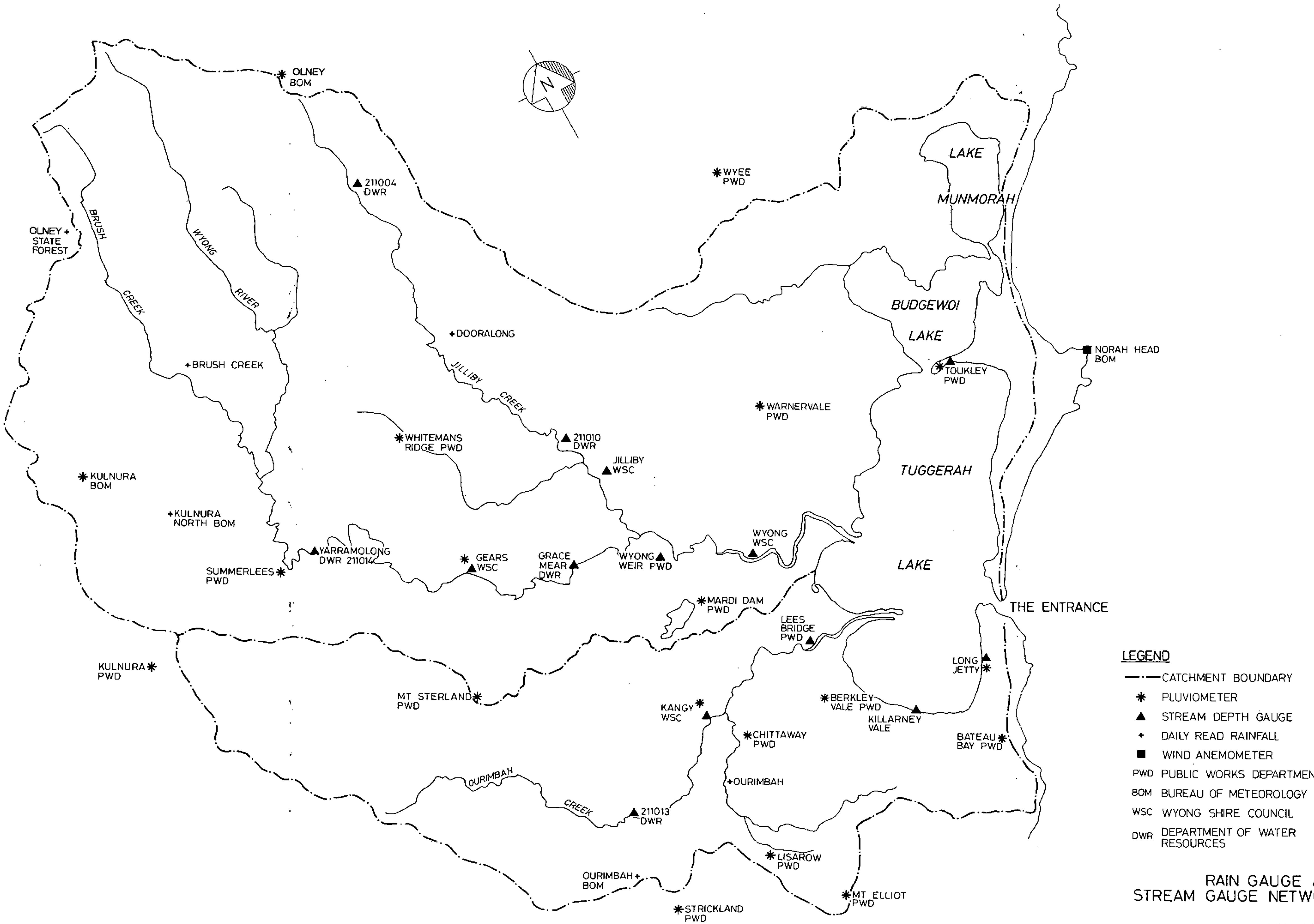
LOCALITY PLAN

FIGURE 1



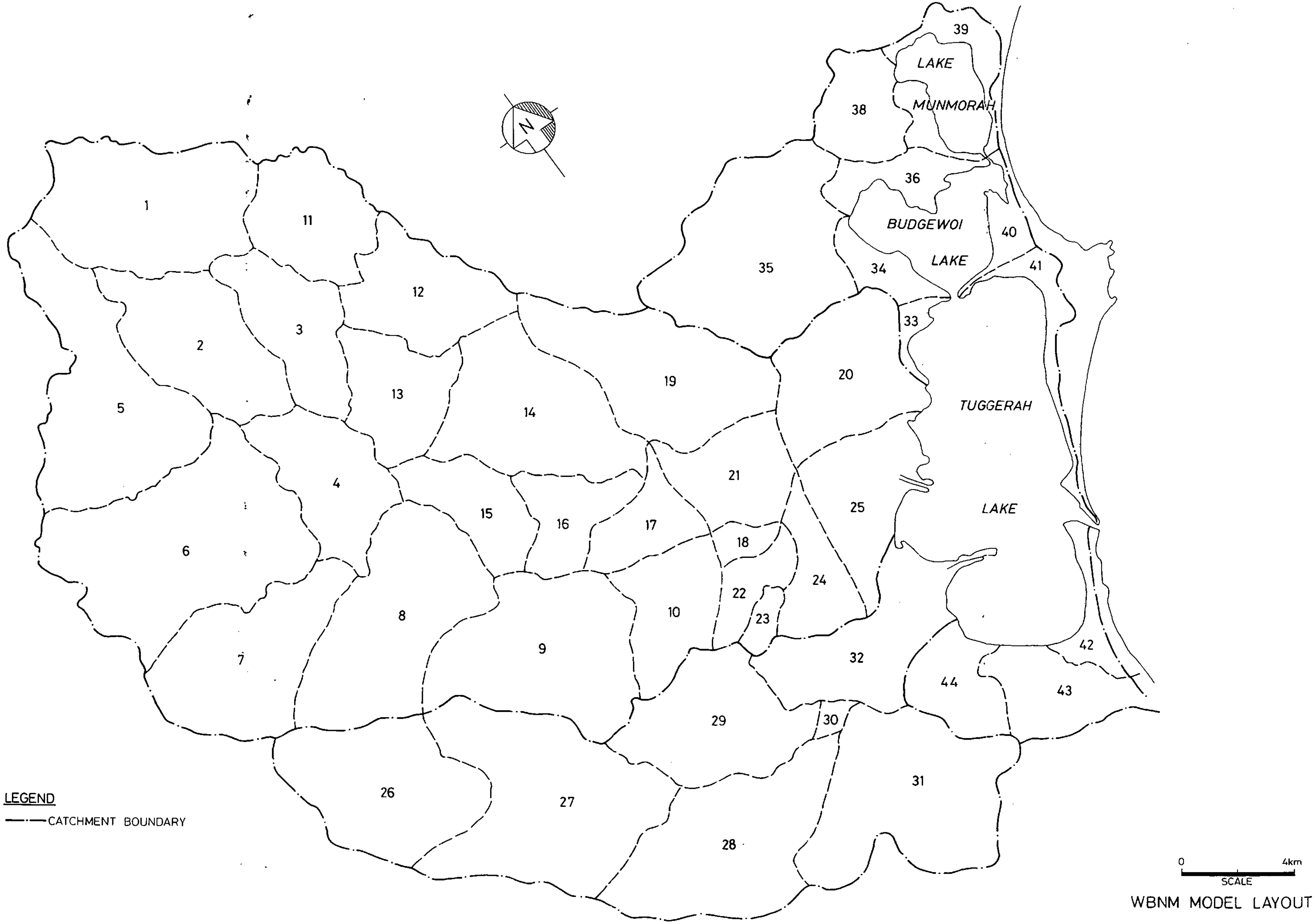
STUDY AREA

FIGURE 2



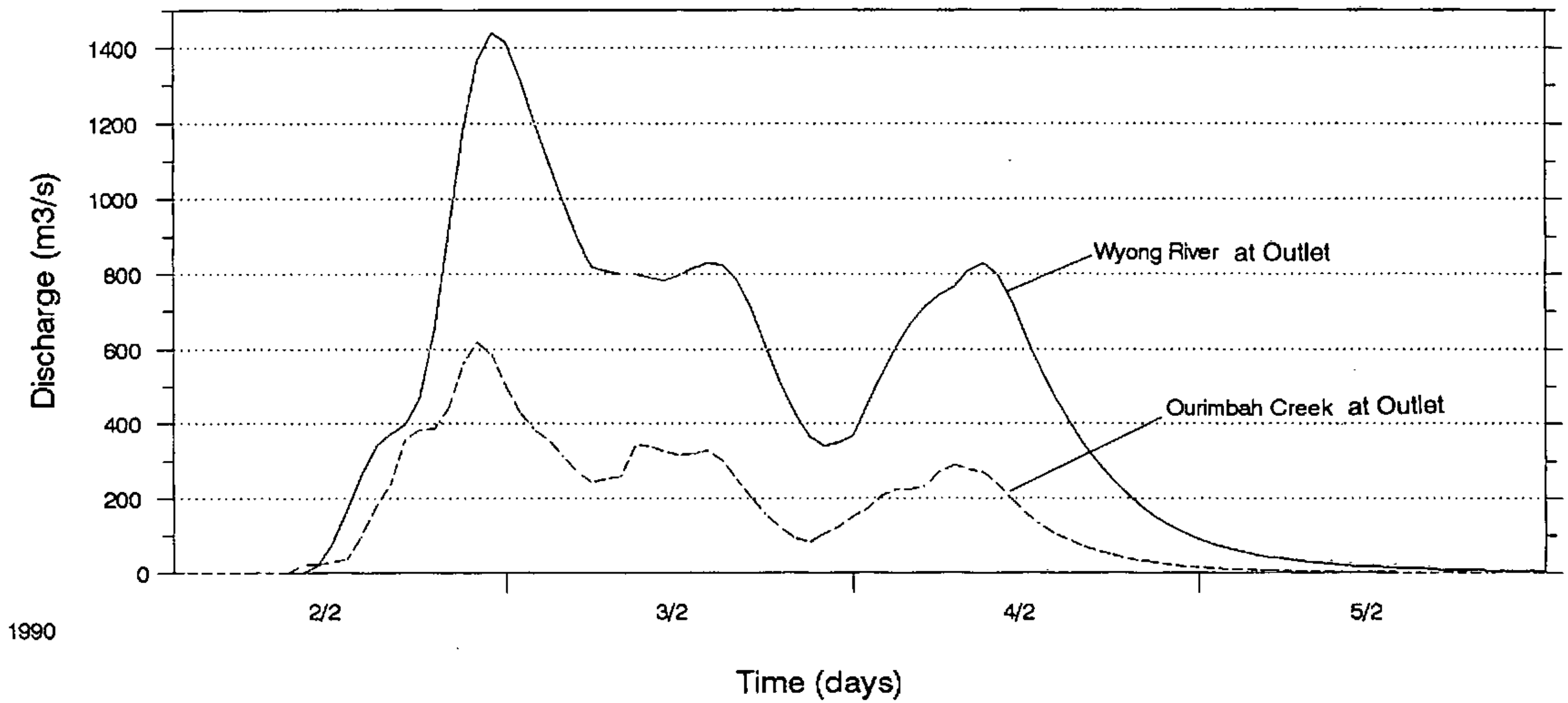
RAIN GAUGE AND STREAM GAUGE NETWORK

FIGURE 3



WBNM MODEL LAYOUT

FIGURE 4



FEBRUARY 1990
FLOOD INFLOW HYDROGRAPH

FIGURE 5

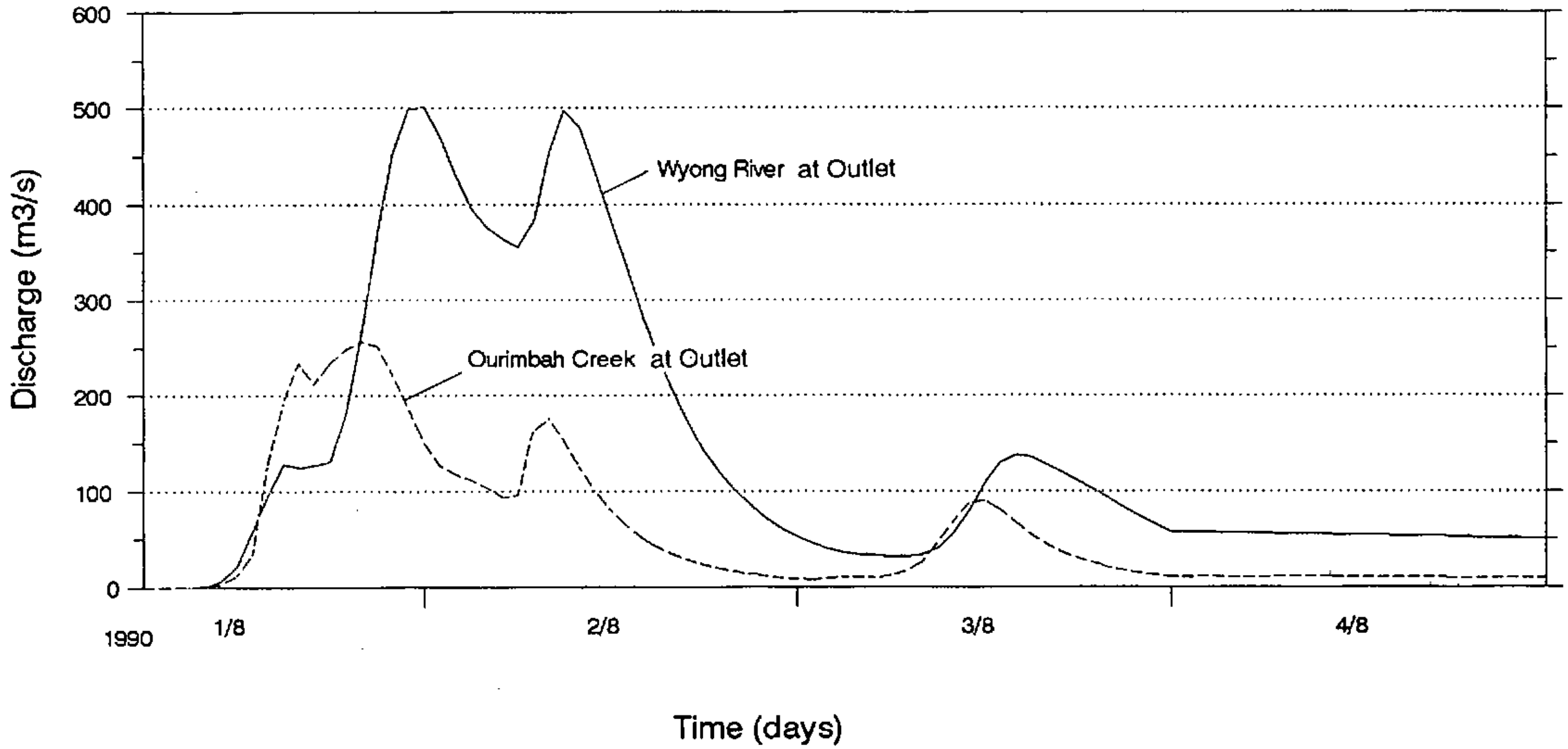
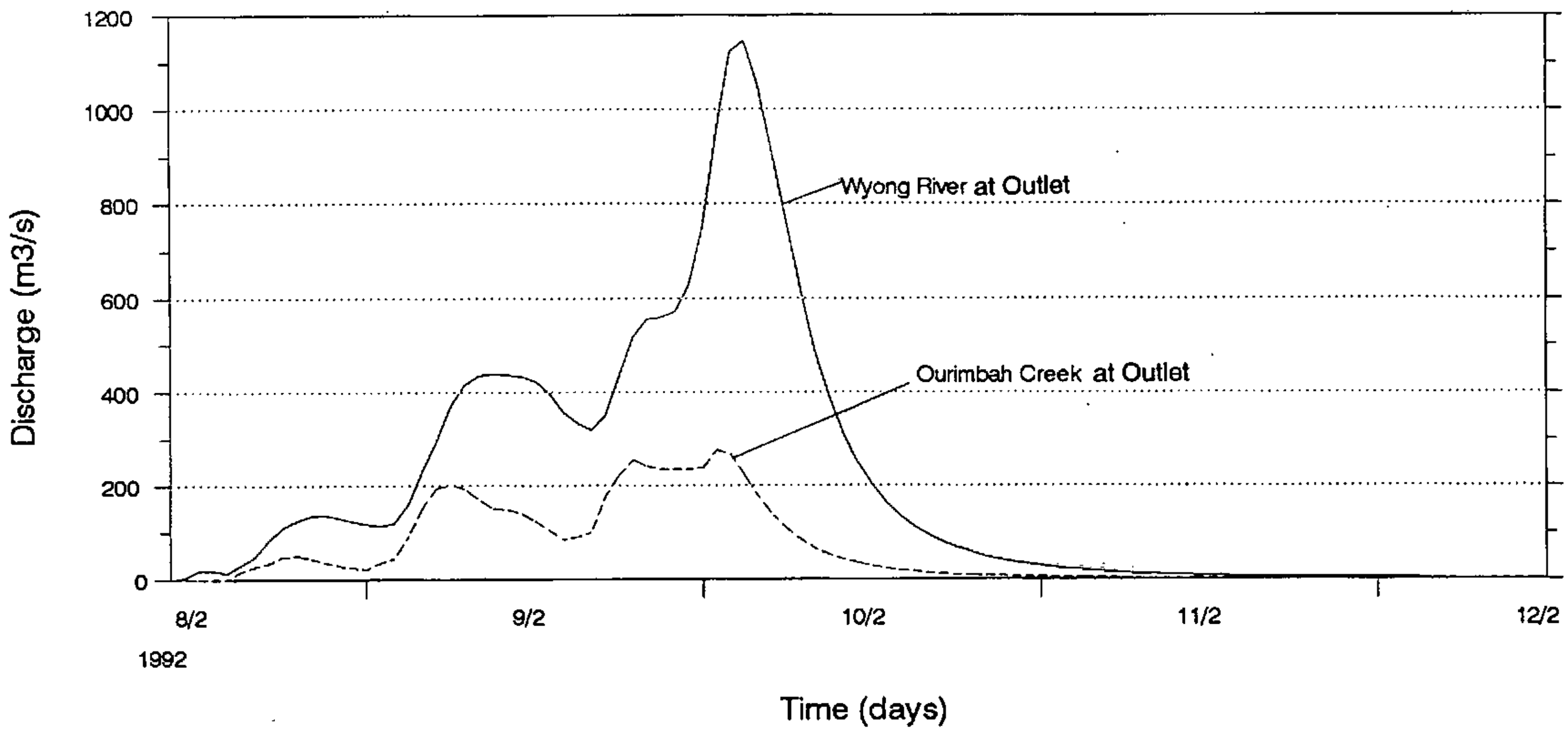


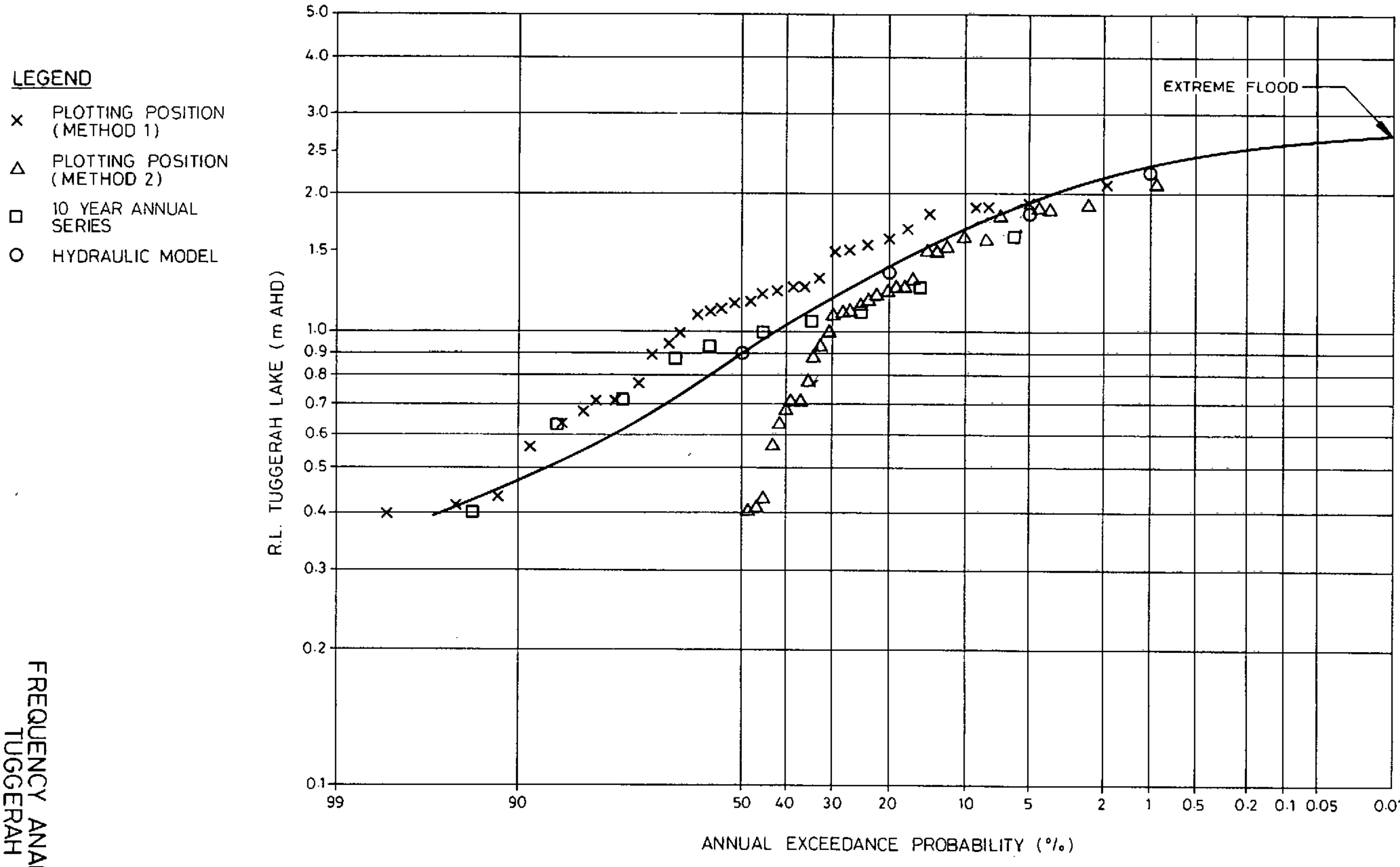
FIGURE 6

AUGUST 1990 FLOOD INFLOW HYDROGRAPH



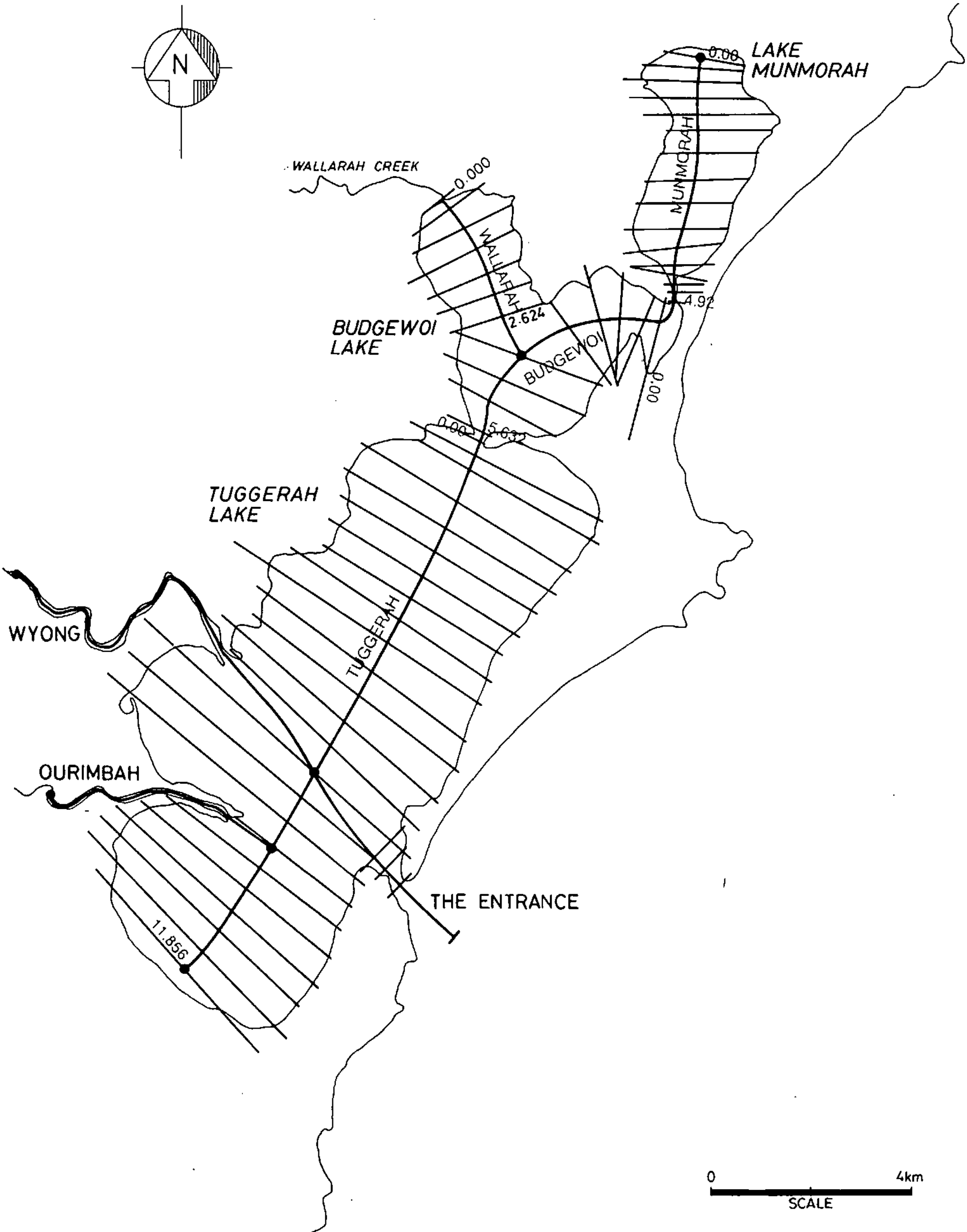
FEBRUARY 1992
FLOOD INFLOW HYDROGRAPH

FIGURE 7



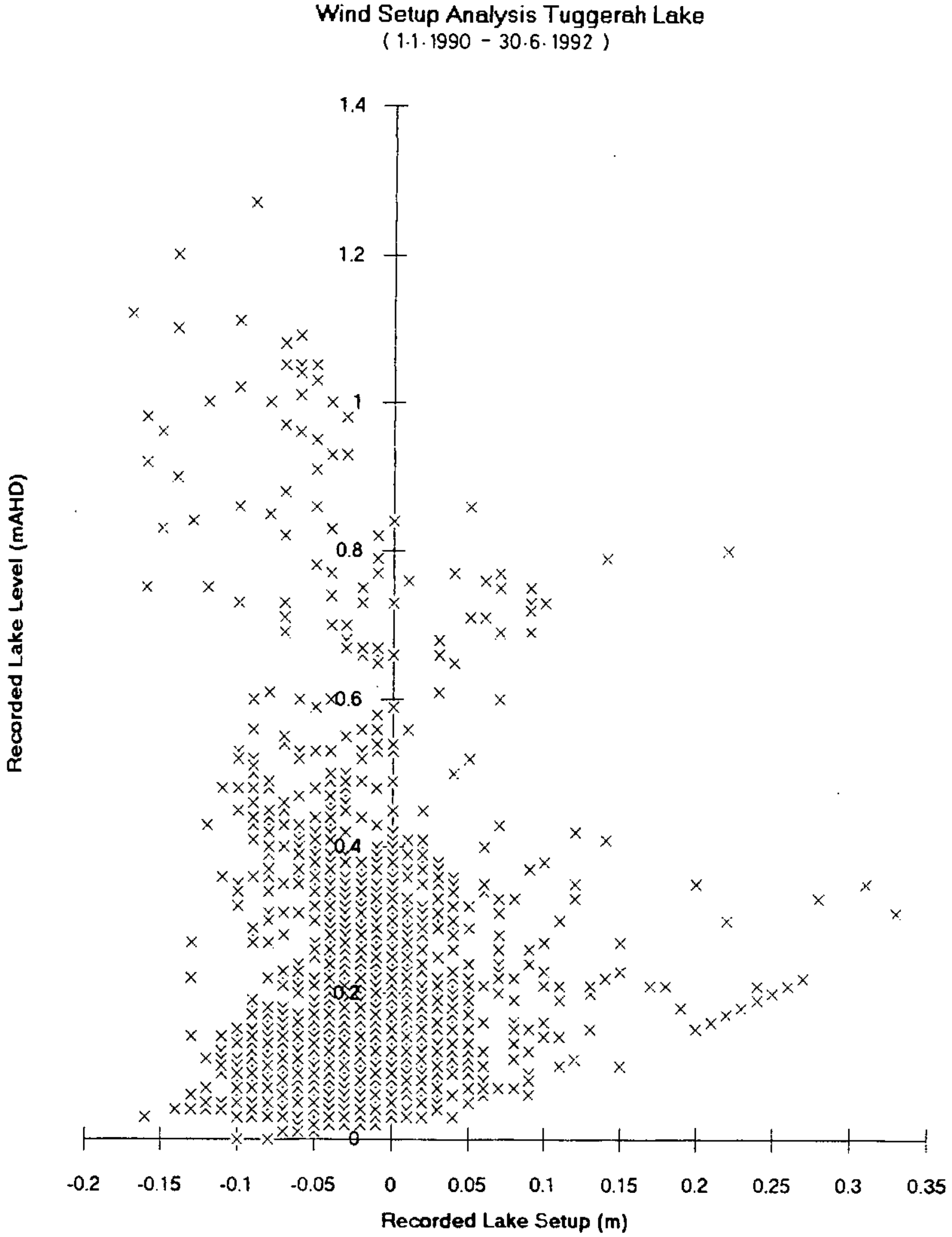
FREQUENCY ANALYSIS
TUGGERAH LAKE

FIGURE 8



HYDRAULIC MODEL LAYOUTS AND
CROSS-SECTION LOCATIONS

FIGURE 9

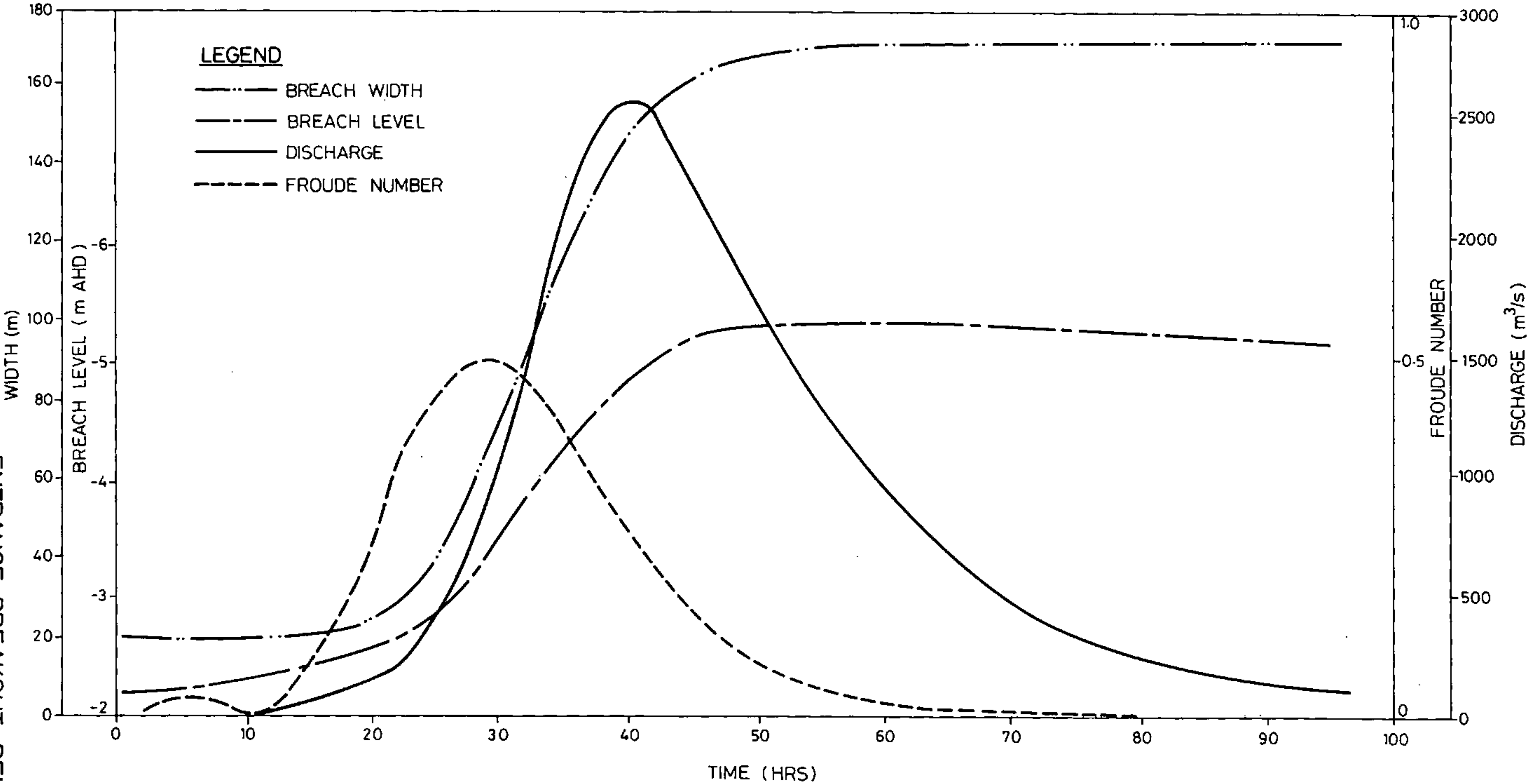


NOTE

- SEE SECTION 7·3 FOR SETUP DEFINITION.

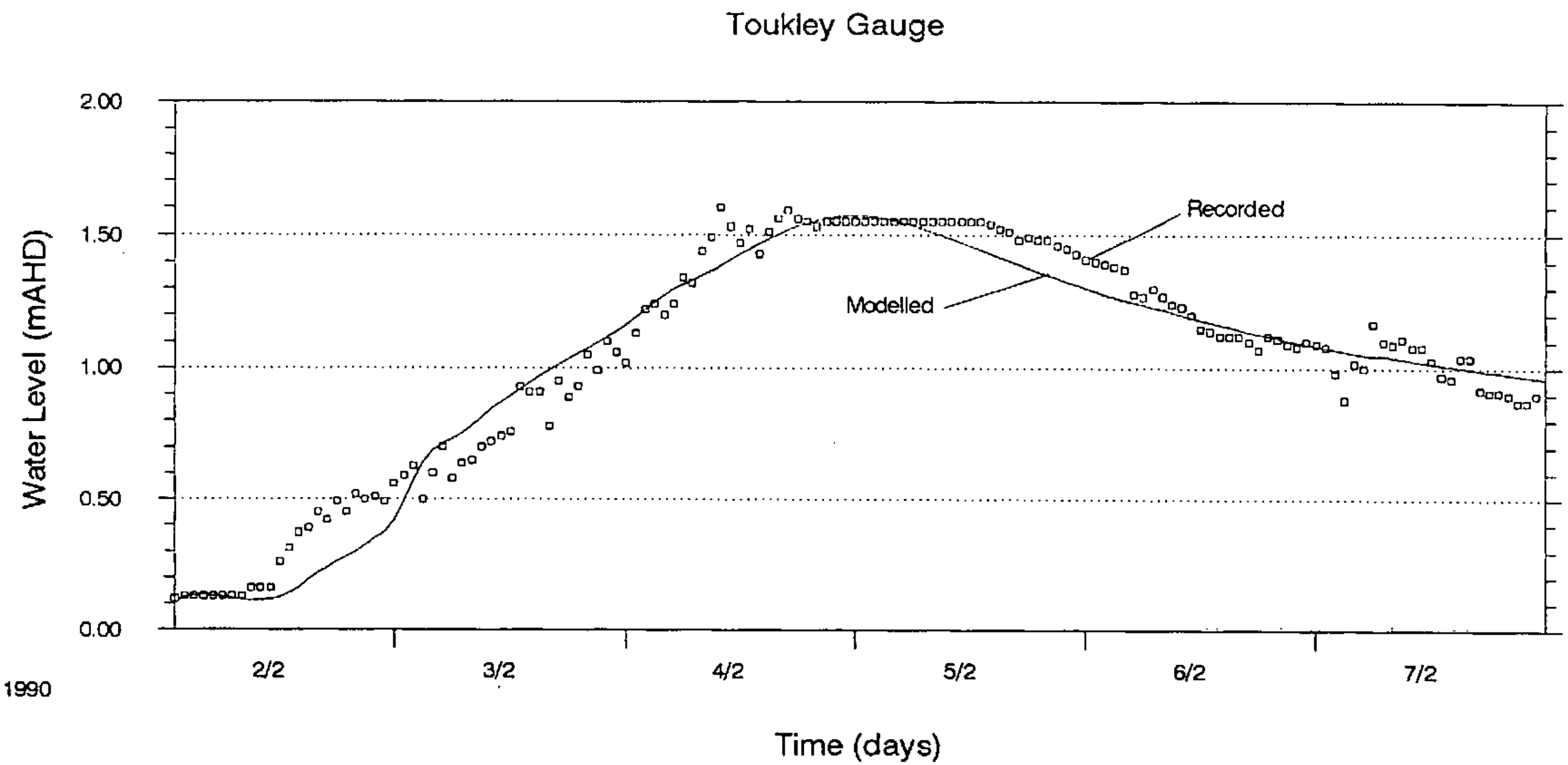
WIND SETUP ANALYSIS
TUGGERAH LAKE

FIGURE 10



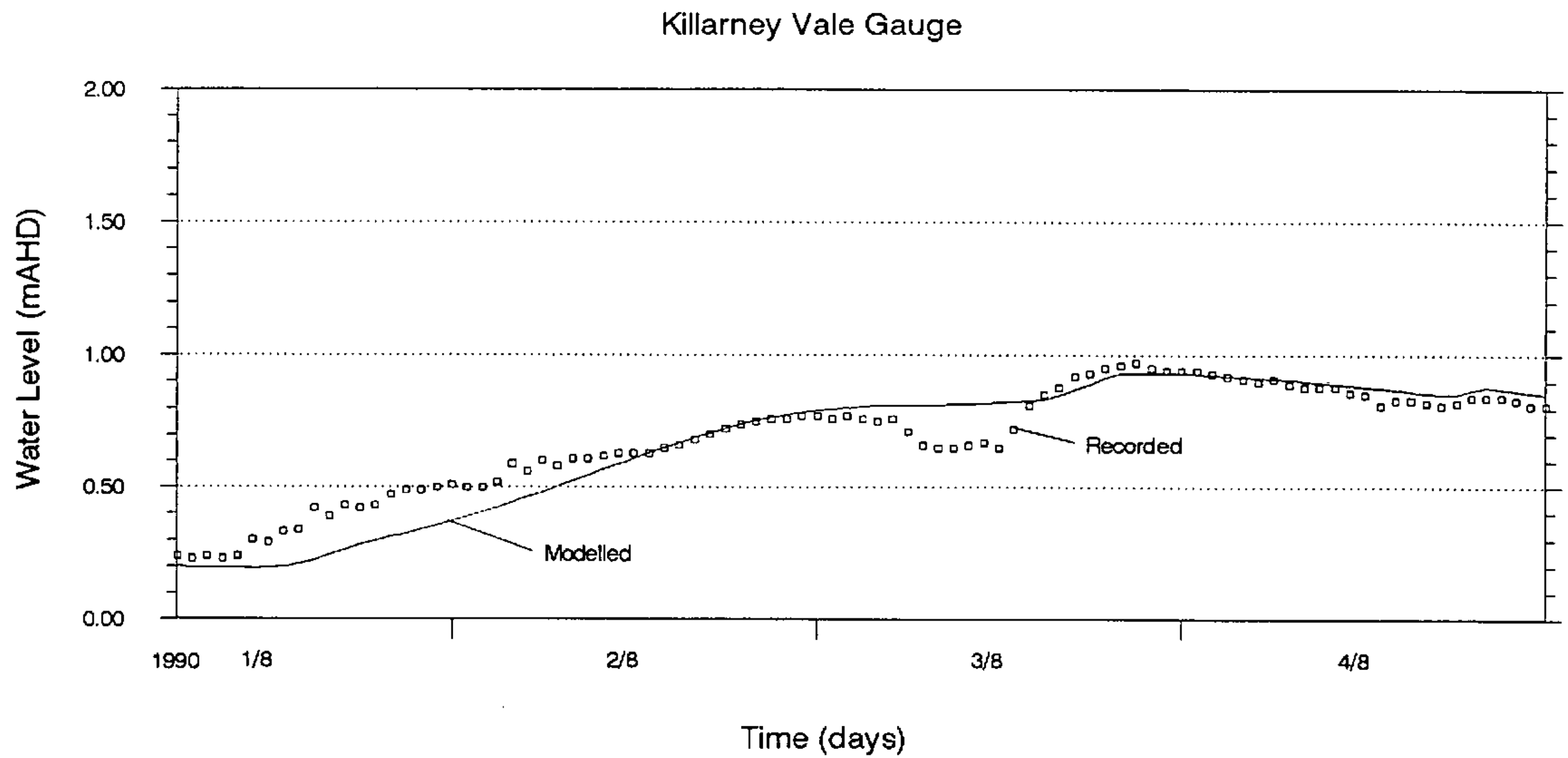
ENTRANCE BREAKOUT BEHAVIOUR

FIGURE 11

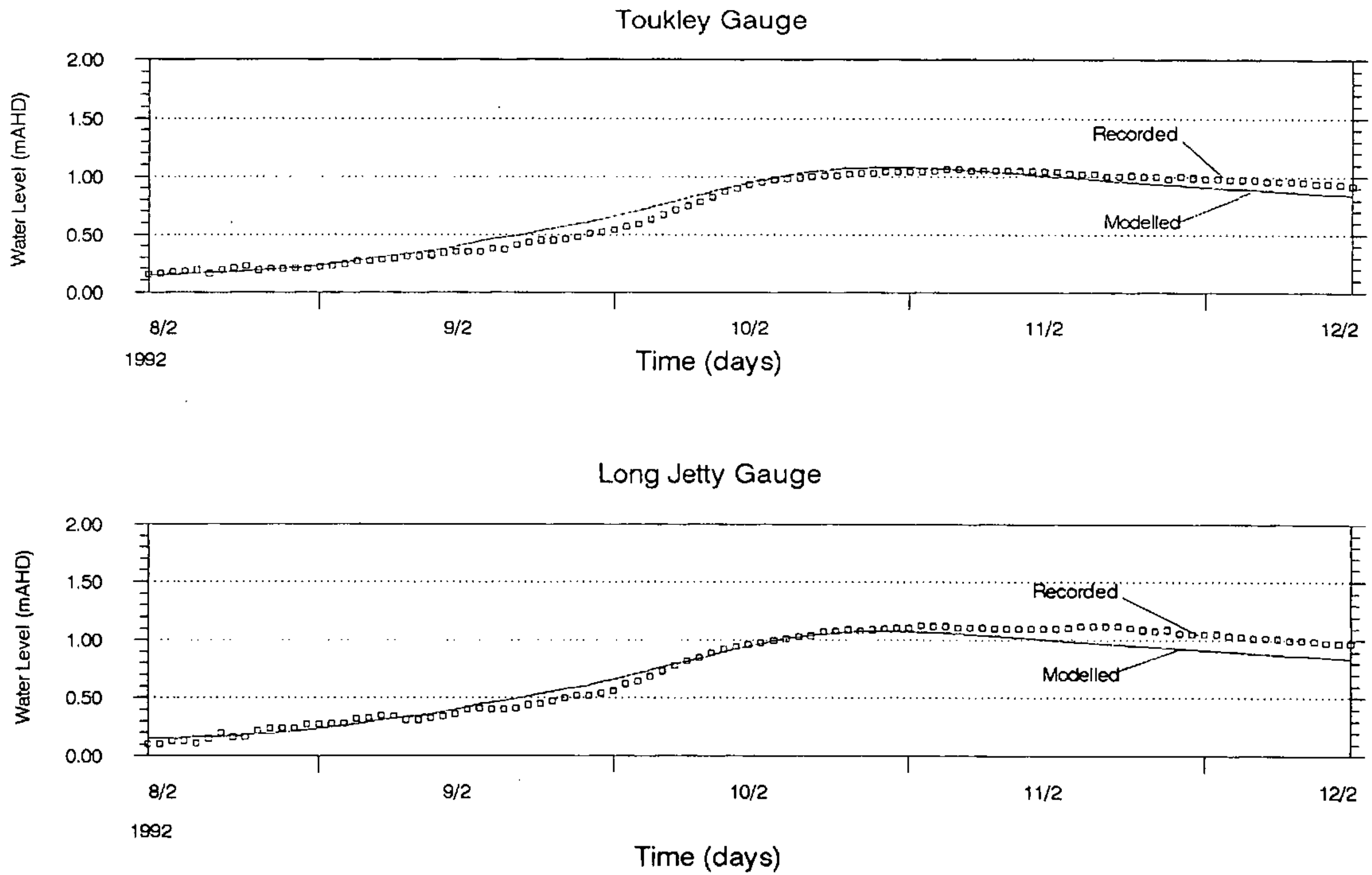


FEBRUARY 1990
FLOOD CALIBRATION

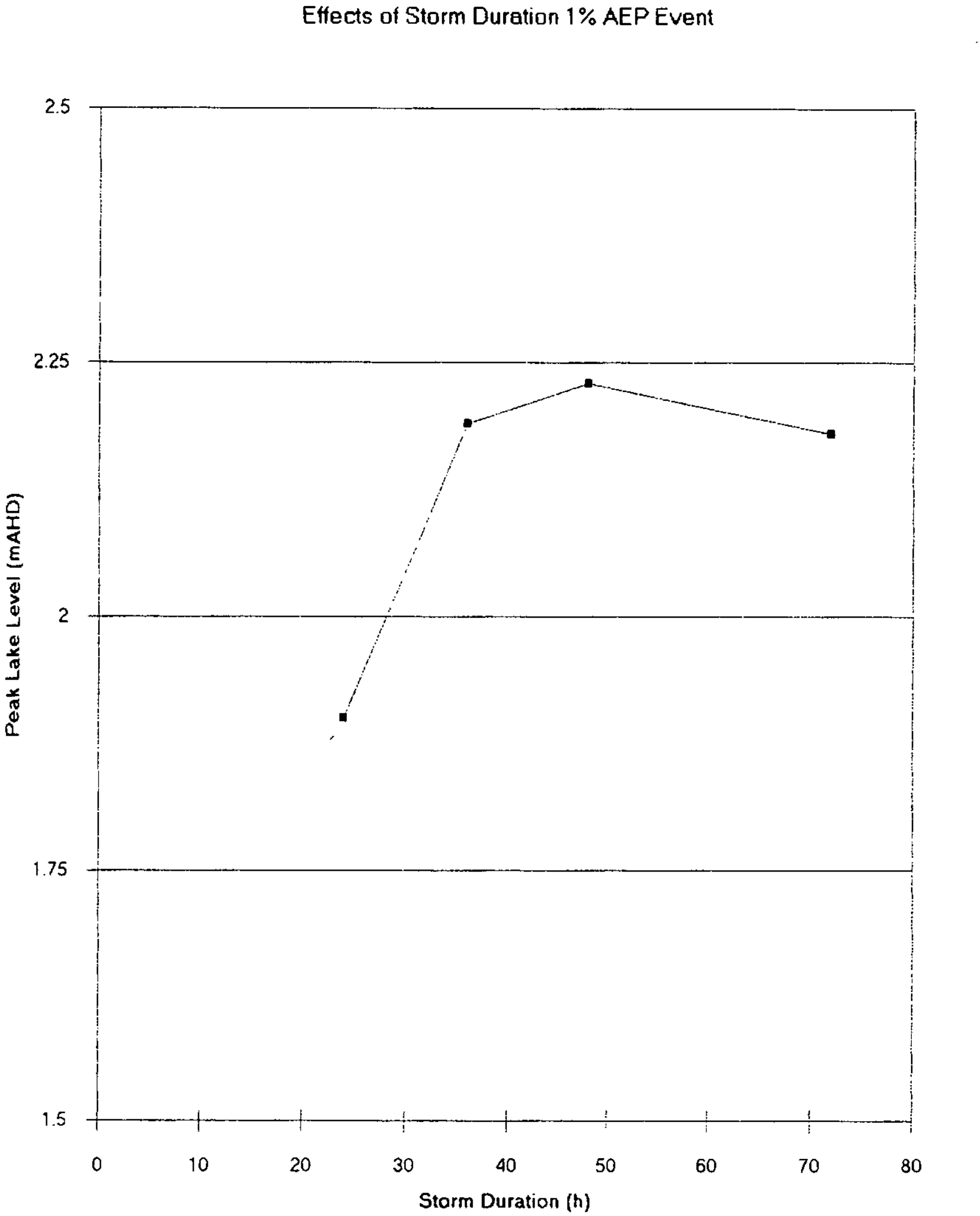
FIGURE 12



AUGUST 1990
FLOOD CALIBRATION
FIGURE 13

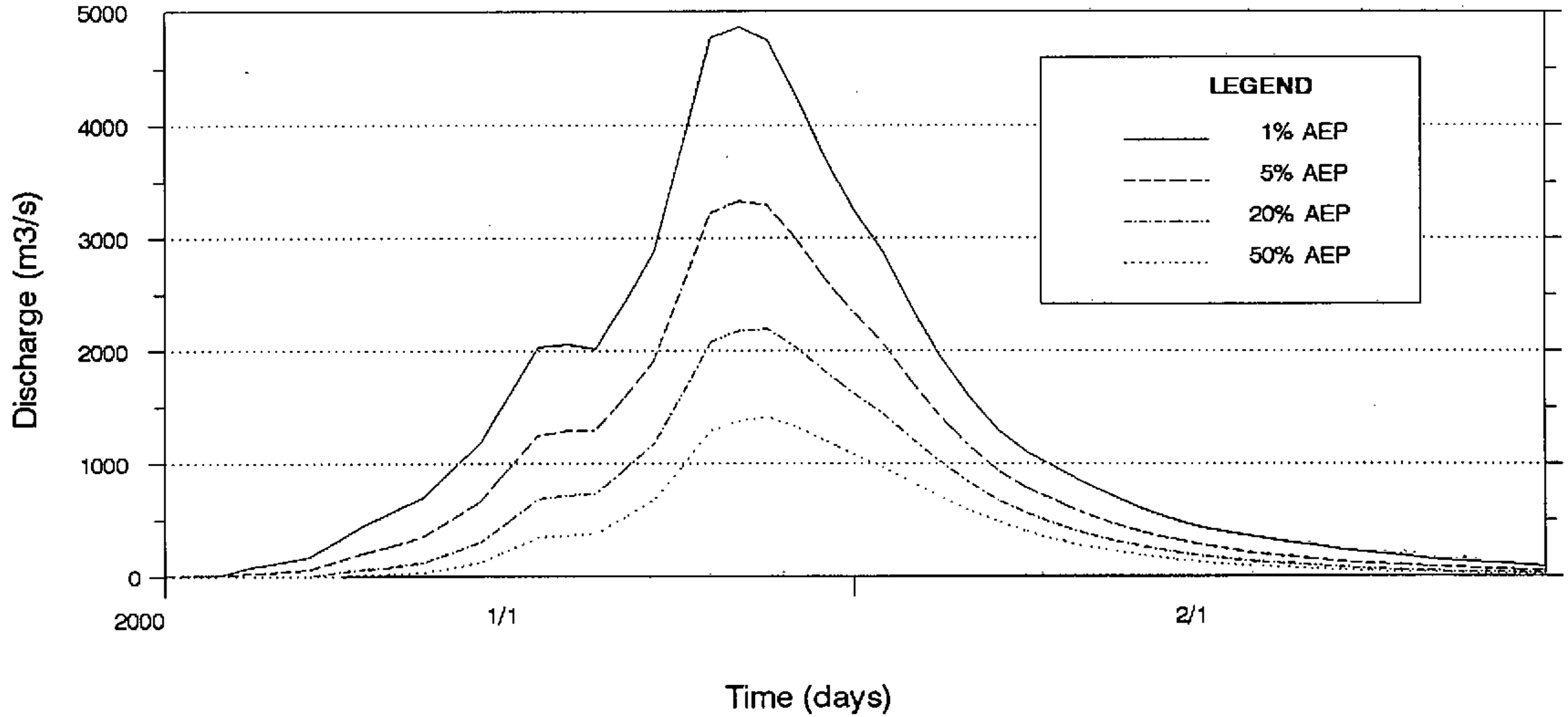


FEBRUARY 1992
FLOOD CALIBRATION
FIGURE 14



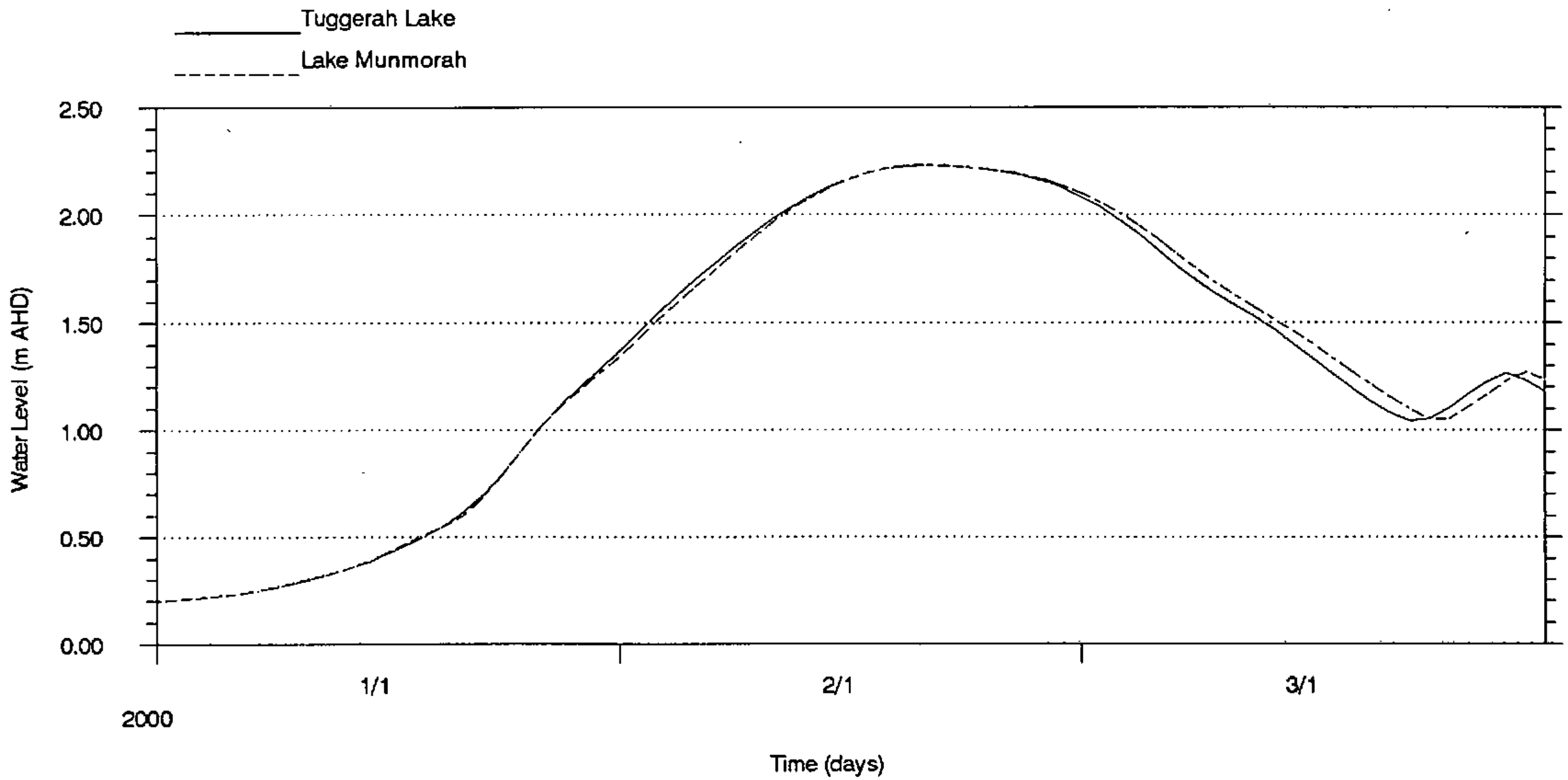
EFFECTS OF STORM DURATION
1% AEP EVENT

FIGURE 15



TUGGERAH LAKE
TOTAL INFLOW HYDROGRAPH

FIGURE 16



COMPARISON OF LAKE LEVEL
(TUGGERAH vs MUNMORAH)
1% 48 HOUR STORM

FIGURE 17

APPENDIX A – FLOOD FREQUENCY

RANK	YEAR	PEAK FLOOD LEVEL	PLOTTING POSITION METHOD 1	PLOTTING POSITION METHOD 2
1	1949	2.10	0.019	0.009
2	1946	1.88	0.050	0.024
3	1964	1.87	0.081	0.039
4	1927	1.81	0.112	0.054
5	1931	1.81	0.143	0.069
6	1990	1.60	0.174	0.085
7	1977	1.59	0.205	0.100
8	1963	1.53	0.236	0.115
9	1953	1.49	0.267	0.130
10	1941	1.48	0.298	0.145
11	1974	1.30	0.329	0.160
12	1975	1.25	0.360	0.175
13	1988	1.25	0.391	0.190
14	1967	1.23	0.422	0.205
15	1978	1.18	0.453	0.221
16	1962	1.17	0.484	0.236
17	1976	1.16	0.516	0.251
18	1992	1.12	0.547	0.266
19	1985	1.09	0.578	0.281
20	1981	1.08	0.609	0.296
21	1984	0.99	0.640	0.311
22	1986	0.93	0.671	0.326
23	1989	0.88	0.702	0.341
24	1973	0.77	0.733	0.356
25	1961	0.71	0.764	0.372
26	1987	0.71	0.795	0.387
27	1979	0.68	0.826	0.402
28	1983	0.63	0.857	0.417
29	1966	0.56	0.888	0.432
30	1965	0.43	0.919	0.447
31	1972	0.42	0.950	0.462
32	1991	0.40	0.981	0.477

TABLE A1 FLOOD FREQUENCY. DATA SOURCE: REFERENCE 2

APPENDIX B – EXTREME WATER LEVELS

B.1 GENERAL

The following sections describe in detail the approach taken to define the extreme water level condition outside the entrance to Tuggerah Lakes. The principal components of the ocean conditions are analysed individually, and then combined to form the design ocean condition.

B.2 ASTRONOMICAL TIDE

The astronomical tide is caused by the relative movement of the Earth, Moon and Sun and the gravitational forces between them. The structure of the tides is affected by the ocean basins and the distribution of the continents. The principal tidal constituents are the solar and lunar diurnal and semidiurnal components which cause one high and one low tide (diurnal) and two high and two low tides (semi-diurnal) in each twenty five hour period (approximately). Tide levels are deterministic and can be predicted using a range of harmonic constants.

Along the NSW coast there is little variation in tide level and phase between Eden in the south and Coffs Harbour in the north. Tides are described as mixed mainly semi-diurnal with the two high tides and two low tides being of different level in the central coast region. Tidal plane data relative to AHD presented in the 1993 Australian National Tide Tables for Sydney are:-

Table B.1 Tidal Plane Data

TIDE	LEVEL m AHD
HAT	1.2m
MHWS	0.7m
MHWN	0.4m
MSL	0.1m
MLWN	-0.3m
MLWS	-0.6m
LAT	-0.9m

These levels are based on a datum shift of 0.93m between LAT and AHD at Sydney. A similar datum shift is appropriate for the Tuggerah Lakes area.

B.3 STORM SURGE

Storm tides occur when meteorological conditions cause an increase in water level above the predicted astronomical tide. The increment in water level is commonly called storm surge. Storm surge has two components. They are:-

- wind set-up
- inverse barometer effect

Wind set-up is caused by the shear stress between the wind and water surface which causes an ocean current. When this current is directed shoreward the water can "pile-up" against the shoreline causing set-up. The effect of wind is described in the following relationship

$$\eta \propto \frac{w^2}{h} \qquad (1)$$

η is the wind setup (m)
w W is wind speed (m/s)
h h is water depth (m)

Wind set-up depends on the square of wind speed and inversely on water depth. At this site the deep near shore water area leads to minimal wind set-up. Wind set-down can occur also.

The inverse barometer effect is caused by a drop in local atmospheric pressure below "average" local pressure levels. In order to maintain equilibrium of total pressure, sea water flows towards areas of low atmospheric pressure leading to an elevation of the water surface. In a similar manner an increase in local atmospheric pressure leads to a depression of the local sea water level. The magnitude of the change is generally 1cm for each hPa of pressure change, but depends on the rate of change of atmospheric pressure.

B.4 WAVE SETUP

Wave set-up develops shoreward of the breaker zone where significant wave energy dissipation occurs. In order to maintain conservation of momentum, the still water level increases non-linearly in the shoreward direction. Computation of wave set-up proceeds iteratively from offshore to the shoreline where maximum set-up occurs - in the order of 10% to 15% of the effective offshore significant wave-height. At river and lagoon entrances the situation becomes more complicated because wave heights have not broken completely by the time they propagate to the entrance. Additionally, the waves themselves are affected by adverse currents and inertia of the outflow can reduce the penetration of wave set-up into the lagoon or estuary. For the full wave set-up level to develop in the water body, a significant volume of water must be transported in through the entrance - this happens also during periods of low atmospheric pressure, or a water level gradient, decreasing from the entrance, established. Because it requires more time to fill large lakes than small coastal lagoons, wave set-up caused water level fluctuations will be relatively small at this site.

B.5 WAVE CLIMATE

In order to estimate wave set-up in the entrance to Tuggerah Lake it is necessary to define wave conditions in the nearshore region of the entrance. The offshore wave climate was based on 19 years of offshore Botany Bay Waverider buoy data collated with wave directions derived from daily synoptic charts. This data has been reduced to parametric form, see Table B.2, in terms of significant wave-height, zero crossing period, offshore wave direction and probability of occurrence. Wave period bands of 1 second and directional bands of 22.5 degrees were used to define this climate. Therefore all offshore waves propagating from 11.25 degrees either side of east with a zero crossing period lying between 4.5 and 5.5 seconds were classified as East-5 seconds. Long term wave height exceedance was defined in terms of the log-normal distribution, Reference B1, using the parameters H_{10} and H_{90} which are the significant wave heights exceeded for 10% and 90% of the time, respectively.

θ		Tz			P1	δ_y
N (348.75–11.25)						
	3.0	0.000	–	–	–	–
	4.0	0.001	1.45	0.92	0.59	0.35
	5.0	0.003	1.77	1.06	0.63	0.40
	6.0	0.001	1.98	1.34	0.91	0.30
	7.0	0.000	–	–	–	–
	8.0	0.000	–	–	–	–
	9.0	0.000	–	–	–	–
	10.0	0.000	–	–	–	–
	11.0	0.000	–	–	–	–
NNE (11.25–33.75)						
	3.0	0.000	–		–	–
	4.0	0.003	1.32	0.99	0.75	0.22
	5.0	0.003	1.73	1.28	0.95	0.23
	6.0	0.007	2.30	1.45	0.91	0.36
	7.0	0.003	2.89	1.72	1.02	0.41
	8.0	0.001	3.26	1.87	1.07	0.43
	9.0	0.000	–	–	–	–
	10.0	0.000	–	–	–	–
	11.0	0.000	–	–	–	–
NE (33.75–56.25)						
	3.0	0.000	–	–	–	–
	4.0	0.020	1.29	0.94	0.69	0.24
	5.0	0.048	1.63	1.17	0.84	0.26
	6.0	0.034	2.05	1.36	0.90	0.32
	7.0	0.010	2.18	1.37	0.86	0.36
	8.0	0.003	2.73	1.56	0.89	0.44
	9.0	0.000	–	–	–	–
	10.0	0.000	–	–	–	–
	11.0	0.000	–	–	–	–
ENE (56.25–78.75)						
	3.0	0.000		–	–	–
	4.0	0.008	1.35	0.97	0.70	0.26
	5.0	0.027	1.73	1.19	0.84	0.28
	6.0	0.033	2.14	1.44	0.97	0.31
	7.0	0.012	3.07	1.87	1.14	0.39
	8.0	0.007	3.33	2.20	1.46	0.32
	9.0	0.001	4.20	2.53	1.53	0.39
	10.0	0.001	3.96	2.80	1.98	0.27
	11.0	0.000	–		–	–
E (78.75–101.25)						
	3.0	0.000	–	–	–	–
	4.0	0.009	1.23	0.92	0.69	0.23
	5.0	0.034	1.70	1.19	0.84	0.28
	6.0	0.040	2.21	1.43	0.92	0.34
	7.0	0.021	2.85	1.73	1.05	0.39
	8.0	0.005	3.77	2.16	1.24	0.43
	9.0	0.001	4.62	2.73	1.61	0.41
	10.0	0.000	–	–	–	–
	11.0	0.000	–	–	–	–

θ		H10(m)				δ_y	H50(m
ESE (101.25–123.75)							
	3.0	0.000	–	–	–	–	
	4.0	0.004	1.36	1.00	0.73	0.24	
	5.0	0.021	1.84	1.22	0.81	0.32	
	6.0	0.022	2.55	1.56	0.95	0.39	
	7.0	0.014	3.39	1.99	1.17	0.42	
	8.0	0.007	4.27	2.36	1.30	0.46	
	9.0	0.001	5.65	2.94	1.53	0.51	
	10.0	0.000	–	–	–	–	
	11.0	0.000	–	–	–	–	
SE (123.75–146.25)							
	3.0	0.000	–	–	–	–	
	4.0	0.009	1.36	0.91	0.61	0.31	
	5.0	0.039	1.84	1.20	0.78	0.33	
	6.0	0.053	2.58	1.61	1.00	0.37	
	7.0	0.038	3.08	1.97	1.26	0.35	
	8.0	0.014	4.06	2.39	1.41	0.41	
	9.0	0.004	4.30	2.48	1.43	0.43	
	10.0	0.001	5.01	2.80	1.56	0.46	
	11.0	0.000	–	–	–	–	
SSE (146.25–168.75)							
	3.0	0.000	–	–	–	–	
	4.0	0.008	1.18	0.84	0.60	0.26	
	5.0	0.033	1.85	1.25	0.85	0.30	
	6.0	0.059	2.58	1.68	1.09	0.34	
	7.0	0.046	3.20	1.96	1.20	0.38	
	8.0	0.018	4.08	2.61	1.67	0.35	
	9.0	0.005	4.81	2.83	1.67	0.41	
	10.0	0.001	5.37	3.13	1.83	0.42	
	11.0	0.000	–	–	–	–	
S (168.75–191.25)							
	3.0	0.000					
	4.0	0.014	1.17	0.86	0.63	0.24	
	5.0	0.059	1.77	1.18	0.79	0.31	
	6.0	0.083	2.47	1.57	1.00	0.35	
	7.0	0.063	3.21	2.07	1.33	0.34	
	8.0	0.026	3.80	2.43	1.56	0.28	
	9.0	0.007	4.14	2.68	1.73	0.34	
	10.0	0.001	5.77	2.70	1.26	0.59	
	11.0	0.000	–	–	–	–	

θ –is offshore dominant wave direction.
Tz–is average zero upcrossing period.
P1–is the probability that a particular offshore direction–wave period (θ –Tz) combination occurs
H10, H50, H90 - significant wave heights exceed 10% 50% and 90% of the time based on a log normal distribution.
δ_y –is standard deviation of y:y=lnH.

TABLE B2

Wave Climate Study
Tuggerah Lakes Entrance Study

Inshore Wave Conditions

Wave Coefficients
Offshore Direction (degrees)

Zero Crossing Period (Sec)	N .0	22.5	45.0	67.5	E 90.0	112.5	135.0	157.5	S 180.0
3.0	.24	.55	.73	.80	.82	.75	.67	.47	.17
4.0	.15	.36	.52	.64	.69	.63	.51	.42	.25
5.0	.10	.25	.39	.53	.61	.60	.51	.42	.34
6.0	.07	.18	.32	.47	.55	.58	.56	.46	.37
7.0	.05	.14	.28	.43	.52	.56	.60	.54	.41
8.0	.04	.12	.26	.41	.51	.55	.62	.61	.46
9.0	.03	.10	.24	.40	.52	.54	.63	.67	.53
10.0	.03	.09	.22	.40	.54	.54	.63	.70	.60
11.0	.02	.08	.20	.40	.56	.54	.63	.72	.65

INSHORE DIRECTIONS

3.0	25.18	32.33	48.43	67.58	85.84	106.65	127.31	131.19	120.60
4.0	25.53	34.28	51.91	67.66	80.98	94.78	113.78	110.51	96.19
5.0	25.94	36.23	54.69	67.73	77.72	86.34	96.18	97.41	92.07
6.0	26.25	38.04	57.32	67.77	75.39	82.12	86.80	88.50	89.95
7.0	26.71	40.92	59.98	67.80	73.54	79.63	83.11	83.60	86.17
8.0	27.57	44.91	62.08	67.85	72.13	77.91	81.54	81.53	82.98
9.0	28.62	48.43	63.43	67.94	71.06	76.56	80.73	80.72	81.13
10.0	29.57	50.89	64.32	68.05	70.32	75.49	80.22	80.41	80.20
11.0	30.32	52.51	64.93	68.13	69.90	74.79	79.92	80.30	79.79

Note: Standard deviation of offshore wave direction is 15 degrees.

Table B3. Inshore Wave Coefficients and Directions at The Entrance.
(Refraction + Shoaling – MSL)

The offshore wave climate was transposed to the nearshore area of the lake entrance using a wave refraction model, RAYTRK (see Reference B2). This model uses the reverse ray frequency-direction spectral wave refraction method which overcomes the computational difficulties which arise in forward ray computations when wave rays cross. Wave rays are propagated seaward at small directional increments for each of eleven frequencies - 0.06, 0.08,..., 0.26Hz from selected nearshore locations. The sea bed bathymetry between these points and the offshore region is described as a series of equilateral triangles within distinct parallelogram zones. Grid size may vary from one zone to another according to depth and bathymetric detail change. For this study ten zones were used with grid sizes varying from 187.5m to 1,500m.

The results of the wave refraction computation were than used to estimate inshore wave coefficients (refraction + shoaling) and weighted mean nearshore wave direction for each offshore wave direction/period case. Table B.3 presents these parameters. Note that they refer only to linear wave transformation and wave breaking is therefore excluded. Nevertheless, the results can be used to estimate the effective offshore wave height, Ho', (which is the refracted, unshoaled, significant height) and is used as input to a surf zone model.

Wave set-up near the lake entrance was estimated using a surf zone model based on wave breaking, random waves, non-linear shoaling, wave set-up and radiation stress algorithms following the technique of Goda, (ref B3) Three flood events and associated ocean wave conditions were selected.

Table B.4 Flood events and associated ocean wave conditions

Date	Peak Storm Hs (m)	Zero Crossing Period(s)	Offshore Direction
02-07/02/90	3.7	6.7	62
01-05/08/90	7.2	7.3	133
09-10/02/92	4.5	5.9	148

Wave data was provided by the Manly Hydraulics Laboratory, PWD. Only the storm of August, 1990 could be classified as a severe ocean storm. Using the wave coefficients presented in Table B.3 and the surf zone model, wave set-up in a range of water depths was calculated. Water depth would vary in the entrance as it scoured and tide level changed.

Table B.5 Estimated setup associated with historical flood events

DATE	Estimated wave setup (m)			
Entrance Depth (m)	0.5	1.0	1.5	2.0
02-07/02/90	0.21	0.1	0.05	0.00
01-05/08/90	0.45	0.34	0.3	0.25
09-10/02/92	0.25	0.14	0.1	0.05

These results show that other than for the severe storm of August, 1990, wave set-up is small in a depth of 2m which is typical for this entrance. Furthermore, these are peak storm wave set-ups and smaller values would have occurred over most of each flood event when waves were smaller. Therefore, model calibration was undertaken firstly excluding wave set-up, but with the understanding that water level discrepancies could result from omission of this parameter and/or the inverse barometer effect. For the August 1990 storm hourly wave data refracted to the region of the entrance was used to estimate wave set-up which was added to the astronomical tide.

The occurrences of ocean storms and astronomical tide events are not physically related and the phases between high tide and peak storm conditions are random. Other analyses have shown that there is some correlation between ocean storms and high lake levels. Based on the data available, the offshore significant wave height is likely to be between 4m and 5m when lake levels exceed 1m AHD, with a weighting towards 4m. There is no trend for higher wave heights associated with the higher water levels in the lake. Therefore a design significant offshore wave height of 4.5m has been adopted for all flood cases.

The wave set-up processes relate to spilling breakers which expend their energy continuously to the still water line. When waves propagate towards a river or lake with a finite depth of water in the entrance, this condition does not develop and so the full shoreline wave set-up can not develop.

This condition can be appreciated when the entrance is wide, but when it is narrow, the nearshore flow structure is complicated by the presence of the full wave set-up levels on the beach either side of the entrance. The situation may be further complicated by the inertia of flood flow leaving the entrance, super elevation of mean lake level by friction affected tidal flow, the size of the water body landward of the entrance, the variation of entrance depth over a tide cycle and the temporal variation of wave height.

When discharge through the entrance is supercritical, ocean level has no impact on lake level. Strong sub-critical currents may cause the waves to steepen and break. The radiation stress gradients associated with the breaking waves will cause a flow potential directed into the lake, which if small may, with low outflow, fill quickly leading to full ocean wave set-up in the lake entrance. On the other hand large lake systems with small entrances and reduced tidal range are likely to be affected minimally by wave set-up, unless it persists for many hours.

Wave set-up may be reduced from that on the shoreline to the set-up computed in the entrance depth, Reference 20. Field studies undertaken by the PWD, Reference B4, have tended to show that little if any wave set-up may penetrate river entrances. However, further physical model studies undertaken by the NSW Public Works have shown that the processes are quite complex, and, depending upon entrance/shoreline form, discharge and waterway, there may be some penetration of wave set-up into an estuary.

Transfer of an offshore significant wave height of 4.5m to the entrance area requires the selection of an appropriate offshore wave direction and zero crossing period. Table B.2 shows that the dominant offshore direction sector is from south-east to south-south-east. Table B.3 also shows that a realistic wave coefficient is then about 0.65 which transforms the 4.5m offshore height to an effective offshore wave height of 3m. Based on wave data presented in Reference B5, an appropriate wave period, Tz, is then 7.5 seconds. Computation of wave set-up at the shoreline near the entrance was then 0.5m based on the surf zone processes described by Goda, Reference B3. These processes include non-linear shoaling and adjustment of the random wave distribution as the higher waves break progressively. Radiation stress variation is then used to solve

$$\frac{\partial \eta}{\partial x} + \frac{1}{\rho g D}\frac{\partial S}{\partial x} = 0 \qquad (2)$$

where x is shore normal direction
η is wave set-up
ρ is water density
g is gravitational acceleration
D is water depth
S is the shore normal radiation stress

In the breaker zone

$$S = \frac{3}{16}\rho g H^2 \qquad (3)$$

where H is "wave height"

Equation (2) is solved iteratively leading to a profile of wave set-up. The solution of equation (2) where the shoreline includes a lake entrance can not be undertaken analytically and was therefore undertaken using the two-dimensional model MIKE-21. An idealized case was investigated in which a 400m length of coast was incised by a 30m wide entrance of depth 3m. This is representative of the Tuggerah Lakes entrance, at least in the early stages of flood and when the tide and lake level are high. Simplification of the X direction momentum equation from MIKE-21 in steady state leads to

$$gD\frac{\partial \eta}{\partial x} - fw^2 = 0 \qquad (4)$$

where f is wind friction factor
w is wind speed (m/s)

Equilibration of equations (2) and (4), and introduction of equation (3), leads to a relationship between wave height, wave height gradient in the shore normal direction and an equivalent wave/wind speed - wave height in terms of root-mean-square wave height. These parameters were extracted from the surf zone model and used to produce the wave/wind input for a two-dimensional model.

Set-up in the entrance will depend on lake volume and flood flow. Maximum wave set-up on the ocean side of the entrance occurs when the lake is small and flood flow is zero and so this case was modelled. Other cases were investigated also to test model sensitivity and response, which cases provided confidence in the physical reality of the model results.

For the ocean boundary, a realistic astronomic tide level of 0.8m AHD with a storm surge of 0.4m leading to an ocean level of 1.2m AHD was selected for preliminary investigations. This is close to the 5% AEP ocean level defined from extreme value analyses of the Newcastle, Reference B6, and Sydney, Reference B7, long term tidal gauge data. Other studies, References B8, have shown that a 1% AEP flood may typically occur with a 5% AEP ocean storm. Typical model procedure would then be to add the 0.5m of shoreline wave set-up whereas the MIKE-21 result shows that wave set-up in the modelled entrance is 0.3m. Other studies, have adopted the wave set-up calculated by the surf zone model in a depth of 3m. For this site and test conditions, this set-up was calculated to be 0.2m, a little lower than the two-dimensional model result, but most likely appropriate for a wider entrance. Note that the above procedures do not include the ocean storm hydrograph and storm-flood phasing. Because little is known of this phasing, peak wave conditions persist for no longer than 6 hours, typically, and Tuggerah Lake floods persist at a high level for 1 to 3 days, it is most likely that peak ocean storm conditions will occur during the flood.

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APPENDIX C STORM CLASSIFICATION

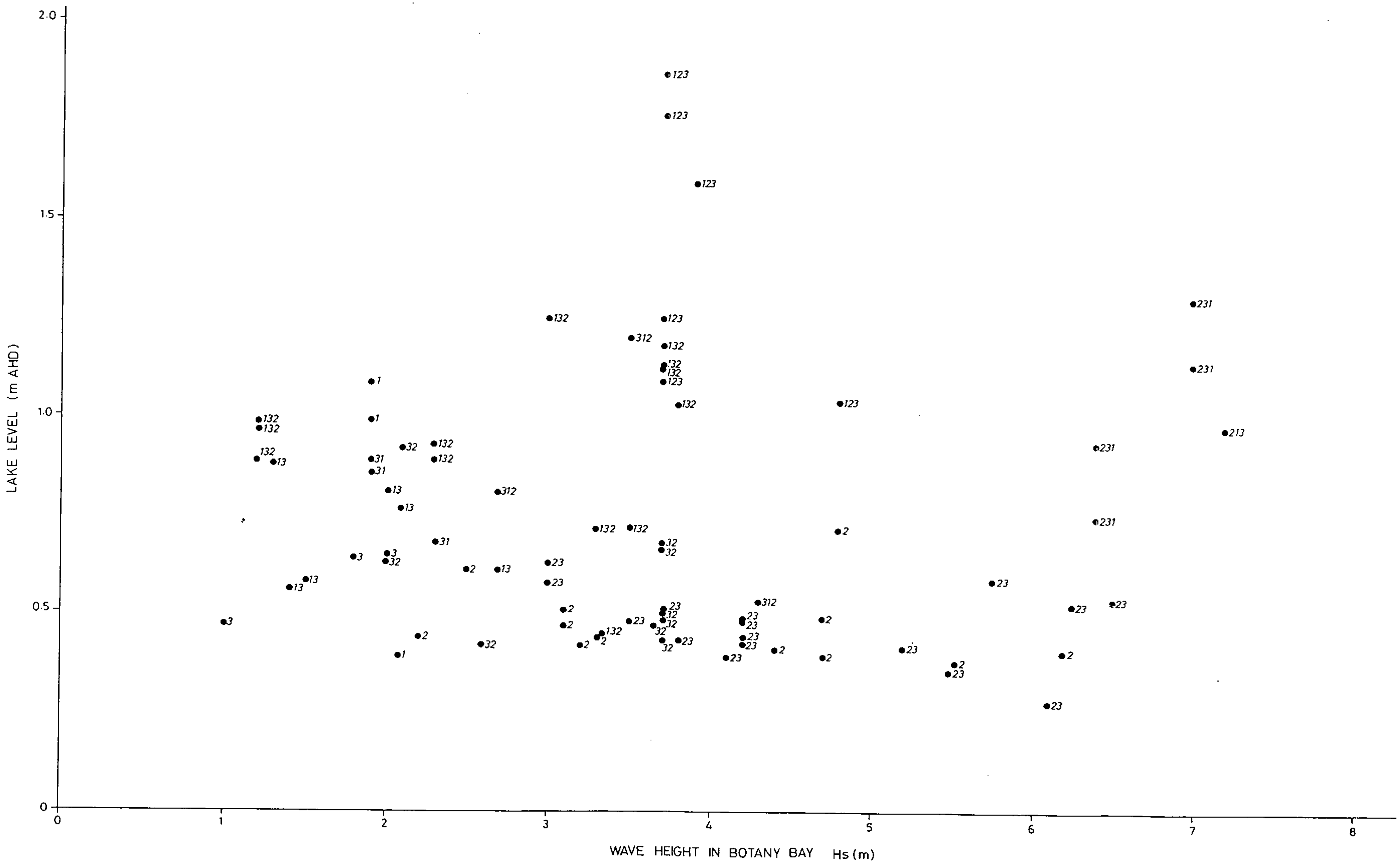
		Rainfall Stn 061083						Lake			Gauged Rising		Flood					M.	Wind Speed					
				Wyong				Flood Levels			Lake Levels		Level					Newcast	Norah	Nobby's				
Year	Month	Day	Peak	Rain	Total	Sever	Upper	Middle	Lower	Upper	Middle	Lower	used in	otany Ba	Ocean	Storm		High Wat	Head	Head		Comments		
			Rain	Days	Rain	-ity	Lake	Lake	Lake	Lake	Lake	Lake	Analysi	Peak Hs	Hs	Ts	am	pm	9am	max	Rain	Storm	Wind	Type
			(mm)	(no.)	(mm)		m AHD	m AHD	m AHD	m AHD	m AHD	m AHD	m AHD	(m)	(m)	(s)	(m)	(m)	km/h	(km/h)	1	2	3	
1885	6	24	165.1			3																		
1886	7	26	53.3																					
1887	1	21	74.7																					
1887	4	17	33.8																					
1887	5	30	72.9																					
1887	8	19	48.5																					
1889	5	27	143.5			1																		
1889	7	26	83.3																					
1890	2	20	42.2																					
1890	3	25	85.6																					
1891	3	24	58.9																					
1891	6	10	66.3																					
1891	6	22	74.9																					
1891	11	23	75.7																					
1892	3	19	82.0																					
1892	9	24	118.6																					
1892	12	3	115.8																					
1893	3	9	217.9			3																		
1893	6	3	73.9																					
1893	6	12	69.9																					
1894	3	7	80.5																					
1895	1	23	127.8																					
1895	2	8	167.6																					
1897	6	1	118.1																					
1898	1	25	102.4																					
1899	8	24	79.5																					
1900	5	23	94.0																					
1900	6	7	90.4																					
1900	7	5	70.6																					
1902	10	13	97.8																					
1905	4	2	195.6			2																		
1907	3	15	130.8																					
1908	2	24	63.5																					
1909	2	20	52.1																					
1909	9	28	54.1																					
1911	1	13	66.5																					
1911	7	13	56.4																					
1912	7	26	67.3																					
1913	4	7	125.2																					
1913	5	14	61.5																					
1913	5	26	65.0																					
1913	6	12	54.1																					
1915	5	7	72.6																					
1917	9	20	83.1																					

			Rainfall Stn 061083					Lake		Gauged Rising			Flood				M.		Wind Speed					
				Wyong			Flood Levels			Lake Levels			Level				Newcast		Norah	Nobby's				
Year	Month	Day	Peak	Rai:	Total	Sever	Upper	Middle	Lower	Upper	Middle	Lower	used in	otany Ba	Ocean	Storm	High Wat		Head	Head	Comments			
			Rain	Days	Rain	-ity	Lake	Lake	Lake	Lake	Lake	Lake	Analysi	Peak Hs	Hs	Ts	am	pm	9am	max	Rain	Storm	Wind	Type
			(mm)	(no.)	(mm)		m AHD	m AHD	m AHD	m AHD	m AHD	m AHD	m AHD	(m)	(m)	(s)	(m)	(m)	km/h	(km/h)	1	2	3	
1917	11	10	54.9																					
1919	5	15	48.8																					
1920	12	10	144.8																					
1921	4	2-9.	95.0																					
1921	5	12	99.8																					
1921	7	2	119.1																					
1921	12	31	74.2			2																		
1922	1	3	79.2																					
1923	4	27	50.0																					
1924	1	23	135.6																					
1924	4	3	79.8																					
1924	6	3	157.2																					
1925	5	11	112.5																					
1926	3	25	120.9																					
1926	12	29	162.3																					
1927	4	16	204.5	6	403.8	1	.2-1.7	1.81					1.81											
1927	11	25	60.7	4	149.6																			
1929	2	5-20	96.8	14	355.7																			
1929	10	9-16	84.3	7	291.2																			
1930	3	11	127.5	3	148.1																			
1930	6?	18?	127.5	7	292.3	1		1.81																
1931	3		no record																					
1931	4		no record			2	.2-1.7	.2-1.77					1.81											
1931	7		no record																					
1934	7	29	73.9	4	169.3																			
1937	6	5-9.	103.9	5	136.6																			
1938	1	19	113.8	6	180.4																			
1939	3	14	82.8	4	113																			
1941	2?	7?	40.1	5	96.6		1.2-1.7	.2-1.77					ave=1.48											
1942	3	23	212.1	5	303.5																			
1942	10	13	75.7	5	174.9																			
1943	5	7-29	82.8	12	376.1A																			
1945	6	7-15	95.5	8	311.5A																			
1946	4	16	238.5	6	560.9	1	.2-1.8	.2-1.88					1.88											
1947	2	26	67.1	3	109.8																			
1947	12	10	63.8	5	186.2																			
1948	1	15	124.7	6	208																			
1949	1	11	200.9	3	2964																			
1949	2	25	103.6	5	163.6																			
1949	5	21	158.0	3	220.5																			
1949	6	18	111.0	6	328.5A	1		1.82-1.8	.92-2.1				2.1											
1950	2	4-9.	94.0	5	168.7																			
1950	6	2-30	100.3	19	403.9A	2											1.63	1.93						
1950	7	0-20	60.7	10	141.4A																			

			Rainfall Stn 061083					Lake			Gauged Rising		Flood					M.	Wind Speed					
				Wyong				Flood Levels			Lake Levels		Level					Newcast	Norah	Nobby's				
Year	Month	Day	Peak	Rain	Total	Sever	Upper	Middle	Lower	Upper	Middle	Lower	used in	otany Ba	Ocean	Storm		High Wat	Head	Head		Comments		
			Rain	Days	Rain	-ity	Lake	Lake	Lake	Lake	Lake	Lake	Analysi	Peak Hs	Hs	Ts	am	pm	9am	max	Rain	Storm	Wind	Type
			(mm)	(no.)	(mm)		m AHD	m AHD	m AHD	m AHD	m AHD	m AHD	m AHD	(m)	(m)	(s)	(m)	(m)	km/h	(km/h)	1	2	3	
1950	9	12	96.5	2	185.9																			
1951	1	1	94.2	2	99.5												1.3	1.37						
1951	6	1-16	48.8	(	98.4												1.4	1.4						
1952	7	4-30	146.8	7	290.7																			
1952	8	13	71.1	4	134												1.24	1.68						
1953	5	9	133.9	[illegible]	248	2	.2-1.77						ave=1.49											
1954	2	23	109.7A														1.68	1.19						
1955	2	16	68.6	3	120.9												1.55	1.19						
1956	2	10	158.8	3	215																			
1956	3	2	69.3														1.4	1.6						
1956	6	25	158.8	2	165.7																			
1958	3	10	144.8	4	201.2																			
1960	12	19	67.8A	6	169.3A																			
1961	8	28	82.6A	5	143.3.A		.65-0.7	0.65					0.71											
1961	11	6-27	61.7	10	187.4A		0.57	0.48																
1962	5	14	63.5	6	278.3A	3	1.17	1.17					1.17				1.55	1.52						
1963	3	8-25	147.3A	7	159.5A		.7-.73	0.73					0.73											
1963	4	30	43.7	8	272.3A	2	.41-1.5	.38-1.52					1.53											
1963	5	7	195.3A	6	151.7		.88-.97	0.88					0.97											
1963	6	26	57.4	6	114.3		0.56	0.4																
1963	9	2	64.8	3	103.6		.48-.67	0.67																
1964	6	10	123.7	[illegible]	432.4A	1	.62-1.	.58-1.8	1.83				1.87											
1967	2	20	no record				.74-.86	0.83	0.84				0.86											
1967	3	9	no record				0.9-1.0	0.91	0.86				1.04											
1967	6	23	94.0	4	175.2	3	98-1.2	1.18	1.11				1.23											
1967	8	10	no record				.95-1.0	1.11	0.86				1.11											
1969	2	12	82.3	6	209.8																			
1970	12	9	109.7	5	170.4																			
1971	1	20	83.8	5	162.3												1.68	1.47						
1971	7	25	8.4	1	8.4										5.4	8								
1972	1	15	106.7	7	240.6									4.1			1.87	1.33		89	y	y	y	1 2 3
1972	10	1	80.8	3	105.2									2.1						63	y	x	y	1 3
1972	11	7	30.0	2	50.6					0.42				2.6						89	x	y	y	3 2
1973	2	8-18	85.9	10	231.5					0.77			0.77	2.1						78	y	x	y	1 3
1973	6	14	9.9	2	19.6					0.41				5.2	5.5	8.8				83	x	y	y	2 3
1973	10	16	80.5	5	121.8					0.58				1.5						78	y	x	y	1 3
1974	1	11	103.6	8	223					0.81			0.81	2						91	y	x	y	1 3
1974	3	24	62.8	3	92.4									?						74	x	?	y	3
1974	5	26-2	83.0	4	122	2			1.2-1.3	>1.19		>1.13	1.3	7			2.2	1.87		169	y	y	y	2 3 1
1974	6	5	80.6	6	160				1.2	1.19			1.2	3.5			1.65	2.07		85	y	y	y	1 2 3
1974	8	5-7.	0.0	0	0					0.39		0.49		4.7	6.4	11.9				59	x	y	x	2
1974	10	14	0.0	0	0									3.7						63	x	y	y	2 3
1975	2	2-28	75.8	7	203.2									2.8						82	y	x	y	1 3
1975	3	3	29.8	2	30.4					0.63				2						83	x	x	y	3 2

Year	Month	Day	Rainfall Stn 061083 Peak Rain (mm)	Wyong Rain Days (no.)	Total Rain (mm)	Severity	Lake Flood Levels Upper Lake m AHD	Lake Flood Levels Middle Lake m AHD	Lake Flood Levels Lower Lake m AHD	Gauged Rising Lake Levels Upper Lake m AHD	Gauged Rising Lake Levels Middle Lake m AHD	Gauged Rising Lake Levels Lower Lake m AHD	Flood Level used in Analysi m AHD	otany Ba Peak Hs (m)	Ocean Storm Hs (m)	Ocean Storm Ts (s)	M. Newcast High Wat am (m)	M. Newcast High Wat pm (m)	Wind Speed Norah Head 9am km/h	Wind Speed Nobby's Head max (km/h)	Comments Rain 1	Comments Storm 2	Comments Wind 3	Comments Type
1975	4	25	46.4	4	72.4					0.59	0.63	0.58		3						80	x	y	y	2 3
1975	6	12	0.0	0	0									5.3	5.3	7.7	1.43	2.14		148	x	y	y	2 3
1975	6	13-2	102.6	8	230.6					>1.2	>1.25	>1.1	1.25	3			1.57	2.1		145	y	y	y	1 3 2
1976	1	23	55.4	5	171.8									1.8					26		y	x	x	1
1976	2	12	65.8	4	119.2									2.2			1.7	1.28	26		y	x	x	1
1976	3	5	75.8	4	107.8					1.16			1.16	?					48		y	?	y	1 3
1976	3	26	101.8	4	124.4					0.71			0.71	3.3					48		y	y	y	1 3 2
1976	4	9-19	21.6	8	70.4					0.42				3.2					18		x	y	x	2
1976	6	8-23	41.2	6	86					0.44				4.2					48		x	y	y	2 3
1976	8	15	7.4	2	8					0.43				4.2					36		x	y	y	2 3
1976	10	19	80.4	4	95					0.51				3.7					36		x	y	y	2 3
1977	2-3.	2-11	168.2	17	411	2		1.59			1.59		not use	3.9			1.85	1.61	36		y	y	y	1 2 3
1977	4	9	6.2	2	8.4									5.1	5.1	8.1			48		x	y	y	2 3
1977	5	15	73.2	6	214									4.5					60		y	y	y	3 2 1
1978	1	4-29	155.0	5	207	3				0.89		0.93	0.93	2.3	5.2	7.5			60		y	y	y	1 3 2
1978	3	20	109.0	8	190.9	3					0.89	0.86	0.89	1.9			1.75	1.5	78		y	x	y	3 1
1978	4	9-16	19.2	7	49.2					0.43	0.48	0.5		3.7					60		x	y	y	3 2
1978	5	6-24	39.0	8	70					0.39				4.1			1.47	1.9	50		x	y	y	2 3
1978	6	2	135.4	3	217.2					1.12	1.13	1.18	1.18	3.7					74		y	y	y	1 3 2
1978	6	22	17.6	3	41.8					0.61				2.5					18		x	y	x	2
1978	7	23	4.4	1	4.4					0.44				2.2					26		x	y	x	2
1978	8	23	36.0	4	59.6						0.41			4.4					18		x	y	x	2
1979	5	11	52.8	6	134.2					0.47		0.51		3.1			1.7	1.97	20		x	y	x	2
1979	6	22	51.8	4	154.4					0.66		0.68		3.7					50		x	y	y	3 2
1981	2	7	164.8	3	262.6	3			1.08				1.08	?			1.97	1.58	60		y	?	y	1 3
1981	7	10	0.0	0	0									5.8	5.8	9.7	1.48	1.55	16		x	y	x	2
1982	3	5	85.0	4	110.6									2.1			1.56	1.23	20		y	x	x	1
1982	9	20	112.2	6	168									3			1.53	1.46	36		y	y	y	1 3 2
1983	3	15-2	73.6	9	230.2					0.53				4.3			1.62	1.6	64		y	x	y	3 1 2
1983	4-5.	24-3	58.2	9	145					0.63				1.8			1.46	1.18	36		x	x	y	3
1983	5	21-2	29.8	8	93.6					0.47				1			1.29	1.74	36		x	x	y	3
1983	7	9	8.6	4	18.2					0.4				6.2	6.2	9.2	1.5	2	26		x	y	x	2
1983	10	6	49.0	3	73.2					0.49				4.2			1.75	2.01	48		x	y	y	2 3
1983	12	17	116.4	1	116.4									?			1.63	1.31	4		y	x	x	1
1984	1	30	81.8	1	81.8									?			1.76	1.3	16		y	x	x	1
1984	2	24	37.6	6	65.8					0.44				?			1.65	1.24	50		x	x	y	3
1984	3	0-26	55.8	7	181.8					0.68				2.3			1.51	1.15	50		y	x	y	3 1
1984	5	10	14.6	4	29.8					0.44				3.3			gauge malf		26		x	y	x	2
1984	6	21	27.4	2	54					0.43				3.8			gauge malf		48		x	y	y	2 3
1984	6	30	0.0	0	0									5.2	4.9	8	gauge malf		18		x	y	x	2
1984	7	5	29.0	4	33.8									?	6.2	8.1	gauge malf		36		x	y	y	2 3
1984	7	30	55.6	3	94.8					0.65				2			gauge malf		40		x	x	y	3
1984	11	8	94.6	4	203.4	3		.9-.99	.93-.97	0.89			0.99	1.2	5.4	8.6	1.79	1.53	60		y	x	y	1 3 2
1985	4-5.	21-5	53.6	15	238							0.61		2.7			1.56	1.92	50		y	x	y	1 3

		Rainfall Stn 061083						Lake		Gauged Rising			Flood					M.	Wind Speed					
				Wyong			Flood Levels			Lake Levels			Level				Newcast		Norah	Nobby's				
Year	Month	Day	Peak	Rain	Total	Sever	Upper	Middle	Lower	Upper	Middle	Lower	used in	otany Ba	Ocean	Storm	High Wat		Head	Head	Comments			
			Rain	Days	Rain	-ity	Lake	Lake	Lake	Lake	Lake	Lake	Analysi	Peak Hs	Hs	Ts	am	pm	9am	max	Rain	Storm	Wind	Type
			(mm)	(no.)	(mm)		m AHD	m AHD	m AHD	m AHD	m AHD	m AHD	m AHD	(m)	(m)	(s)	(m)	(m)	km/h	(km/h)	1	2	3	
1985	5	20	11.4	3	22.8									5.8	5.8	8.1	1.55	2.01	80		x	y	y	3 2
1985	6	6	21.6	2	39.4									6.2	6.2	9.3	1.45	1.92	40		x	y	y	2 3
1985	7	10	22.2	4	53.2							0.35		5.5	5.5	8.4	1.47	1.48	48		x	y	y	2 3
1985	9	3	8.0	1	8							0.27		6.1	6.1	9.5	1.64	1.54	36		x	y	y	2 3
1985	10	2-20	170.0	8	242.8	3		1.09	0.99		0.92	0.99	1.09	1.9			1.92	1.8	30		y	x	x	1
1985	10	25	61.0	4	83.8						0.53	0.53		6.5	6.5	9.3	1.42	1.55	68		x	y	y	2 3
1986	1	23	98.8	3	119.2							0.45		3.3			1.69	1.26	50	93	y	y	y	1 3 2
1986	8	5	99.2	7	185.2						0.74	0.93	0.93	6.4	6.4	8.3	1.37	1.92	65	96	y	y	y	2 3 1
1986	11	13	89.0	3	167.4							0.39		2.1			1.63	1.45	28	59	y	x	x	1
1986	11	20	12.0	5	31.2						0.52	0.38		6.2	5.1	8.1	2	1.49	41	104	x	y	y	2 3
1987	8	3	3.8	3	6.8						0.58	0.55		5.7	5.7	8.5	1.38	1.65		78	x	y	y	2 3
1987	8	20	53.8	3	85.4						0.71	0.71	0.71	4.8			1.23	1.67		59	x	y	x	2
1987	10	26	3.0	1	3						0.48			3.5			1.87	1.34	20	83	x	y	y	2 3
1987	11	12	111.8	4	156.2									5.4	5	7	1.33	1.72	48	115	y	y	y	1 3 2
1988	1	17	142.4	2	147.8							0.57		1.4			2	1.46	56		y	x	y	1 3
1988	2	9	20.0	2	25.4						0.37	0.25		5.6	5.6	9.2	1.58	1.52	20		x	y	x	2
1988	2	14	28.0	5	53.2						0.47			3.6			2	1.61	56		x	y	y	3 2
1988	4	1-13	47.0	14	308.4						1.04		1.04	4.8			1.72	1.25	37		y	y	y	1 2 3
1988	4	30	90.2	3	157.8	3		.8-1.09	89-1.25		0.93		1.25	3.7	5	8.5	1.6	1.76	70		y	y	y	3 1 2
1988	6	19	23.0	3	39						0.51			2.1			1.42	1.7	19	63	x	y	y	3 2
1988	7	6	148.0	3	204.8	3		0.92	1.03			1.03	1.03	3.8			1.63	1.57	41	57	y	y	y	1 3 2
1988	9	16	132.4	5	192.4						0.66	0.72	0.72	3.5			1.71	1.49	30	48	y	y	y	1 3 2
1989	1	7	86.6	6	178.2							0.88	0.88	1.3			1.9	1.32	33	70	y	x	y	1 3
1989	4	2	60.2	10	203.2							0.81	0.81	2.7			1.5	1.22	39	61	y	y	y	3 1 2
1989	6	0	75.8	4	135						0.81		0.81	?			1.49	2.03	44	93	y	y	y	3 2 1
1989	12	6	266.4	7	307.8						0.63			?			1.31	1.6	24	70	y	x	x	1
1990	2	2-12	279.0	11	545.2	1		1.74-1.7	.69-1.87		1.6	1.32	1.87	3.7			1.99	1.5	32	106	y	y	y	1 2 3
1990	3	19	56.2	10	121						0.32	0.24		?					32	65	x	?	y	1
1990	4	20	90.4	7	187.4						0.69	0.71	0.71	?			1.62	1.49	24	46	y	x	x	1
1990	6	17-2	0.8	4	15.8							0.8	0.8	?					37	81	x	?	y	3
1990	8	2	101.2	5	195	3			0.97			0.97	0.97	7.2			1.2	1.65	33	61	y	y	y	2 1 3
1991	6	11	106.2	7	221							0.49		?					35	87	y	?	y	1 3
1991	12	12	66.8	5	211.2							0.34		?					48	70	y	?	y	1 3
1992	2	9	118.4	6	239.4	2			1.12		1.06	1.13	1.12	?						81	y	?	y	1 3
1992	10	23	0	0	0.0						0.24	0.25		?	5.5	8					x	y	y	2 3
Minimum							0	0.92	0.97	0.39	0.24	0.24		1	4.9	7	1.2	1.15	4	46				
Maximum			279.0	17.0	545.2		0	1.59	1.2	1.19	1.6	1.32		7.2	6.5	11.9	2.2	2.14	80	169				
Average			70.7	5.1	133.7														39.56	82.63				



LAKE LEVEL vs WAVE HEIGHT
1972 - 1992

FIGURE C1

APPENDIX D – LIST OF INFORMATION

PLANS AND MAPPING

AUTHORITY	PLAN No.	Date	Description
WSC	10230	29/1/93	Water Level Recorders.(pwd & wsc not dwr)
WSC	10413	Sept 93	Hydrological Instruments Tuggerah Lakes Catchment
WSC	10047 (16 Sheets)		Bathymetric Survey of Saltwater, Tumbi-Umbi, Ourimbah & Wyong & Wallarah Creeks
Dept. of Rail.	158 - 33	14/3/60	Ourimbah Creek Renewal of Underbridge
Dept of Rail.	1283-35,726		Sydney-Newcastle Wyong Ck Bridge Detail Survey
Dept of Rail.	183-3 183-1	13/8/48	Wyong Ck 62m. 34.3c.N. Details of Concrete Bridge
Dept of Rail.	952/35,745	11/9/45	Replacement of Bridge North Wyong
DMR	0010 505BC 0423 0010 505BC 5025 0335 505BC 0105	16/2/60 21/6/72 16/1/79	Bridge over Ourimbah Creek
DMR.	6003 505 BC 0064a	24/1/86	Twin Bridges over Wyong River
DMR	0010 505BC 0469	2/9/63	Conversion of Disused Rail Bridge at Wyong Creek
DMR	335B908-1 335B908-4	20/1/67	Bridge Over Tuggerah Lake @ the Entrance
PWD	FILE: R 1069/46	25/9/90	Tuggerah Lake • The Entrance -Hydrographic Survey (3 Sheets) • East Budgewoi Lake • North Budgewoi • Wyong Creek • Tumbi Umbi & North Entrance • Munmorah Lake (2 sheets) • Ourimbah Creek
PWD	FILE: R 1069/42		Tuggerah Lakes Hydrographic Survey (28 Sheets)

REPORTS

Report For:	Report By:	TITLE
WSC	Webb, McKeown	Sensitivity Analysis The Entrance Channel Tuggerah Lake
WSC	GHD	Wallarah Creek Flood Investigations Stage 1
WSC	SKP	Wyong Shire Flood Mapping Summary Report
WSC	Webb, McKeown	Lower Wyong River Flood Study Review 1991
PWD	PWD	Jet Pump Systems for Maintaining Tidal Entrances
PWD	SKP	Lower Wyong River Flood Study (January 1984)
PWD	MHL	Central Coast Region Flood Data Survey (DRAFT) Flood Event February 1990
PWD	MHL	Buff Point Flooding with Revised Subsidence Levels
PWD	PWD	Tuggerah Lakes Flood Study Compendium of Data
PWD	PWD	Upper Wyong River Flood Study February 1988
PWD	PWD	Upper Wyong River Flood Study February 1988 Compendium of Data

DATA COLLECTED

Authority	Data Type	Station Name/No.	Description
BOM	Wind	Norah Head /061055	3 Years of 3 hourly wind Speed and Direction data
BOM	Rainfall	061029, 061057 061074, 061083 061093, 061137 061143, 061152 061164, 061165 061219, 061220 061236, 061262 061273, 061319 061351, 061362	All available daily rainfall records
BOM	Evaporation	Peats Ridge / 061351	Average Monthly Class A pan evaporation 1981 - 1992
PWD-MHL	Rainfall	Kulnura, Summerlees Whitemans Ridge Sterland, Wyee Warnervale, Mardi Dam Berkley Vale, Toukley Long Jetty, Bateau Bay Lisarow, Strickland Narara	Daily Rainfall Records 1 January 1990 to June 30 1992
PWD-MHL	Water Level	Stockton Bridge (AWRC 210456) Toukley (awrc 211401) Long Jetty (awrc 211402) Killarney Vale (awrc 211402	Hourly water level records 1 January 1990 to 30 June 1992
PWD-MHL	Water Level	Killarney Vale Toukley Chittaway Pt Long Jetty Tacoma	All annual maximum water level records
DWR	Discharge	211001, 211002 211003, 211003 211004, 211005 211006, 211007 211008, 211009 211010, 211013 211014	All daily average discharge records

APPENDIX E

MIKE-11 BREACH MODEL

Dambreak Structure

A dam break structure is a dam in which a breach can develop. The flow at the dam break structure is quite similar to a broad-crested weir, but there are two differences:

(i) the shape of the dam changes with time, i.e. the breach increases and the dam crest is shortened. As a consequence the critical flow characteristics (Q-h) relationship of the crest and of the breach cannot be calculated beforehand;

(ii) the Q-h relationship for the dam crest and the breach are different therefore the flow over the crest and the flow through the breach are calculated separately (see Figure 1). For each of these two flows the equations used are the same as for the broadcrested weir.

It is practical to calculate the crest flow and the breach flow at a dam in one structure since they depend on the same geometry.

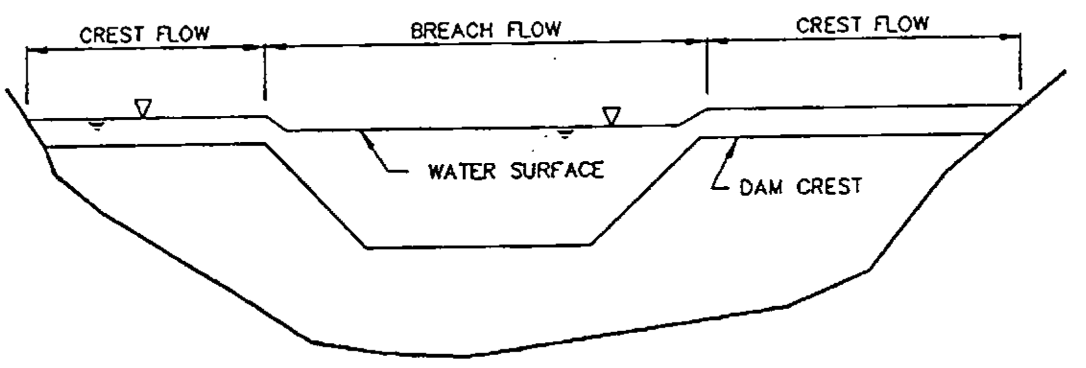


Figure 1 Combined Flow over Breach and Crest

Breach Geometry

The breach is initially trapezoidal. During the development of the breach the trapezoid increases in size and changes shape.

The initial breach shape is described by three parameters (as shown in Figure 2):

- level of the breach bottom (HB)
- width of the breach bottom (WB)
- side slope of the breach (SS) (horizontal : vertical). The left side slope and the right side slope are equal.

The development of the breach can either be specified as a function of time, or it can be simulated from the sediment transport capacity of the breach flow.

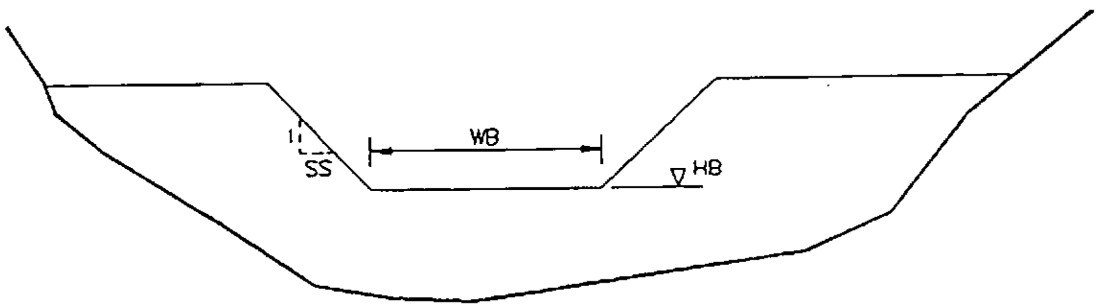


Figure 2 The parameters of the breach geometry.

Time Dependent Breach Development

The parameters of the breach are given as a time series in the boundary data base (B menus). The time is relative to the start of the breach. Between the specified times the parameters are linearly interpolated as shown in Figure 3. The breach bottom level must not be above the crest level.

Erosion Based Breach Development

If this mode is chosen the initial and the final breach shape must be specified. The increase of the breach during a time step is calculated from the actual prevailing conditions in the breach itself. The Engelund-Hansen sediment transport formula is used (see **NST Reference Manual, Engelund-Hansen Model, Equations 1 and 2**) to calculate the sediment transport in the breach.

As a first step, the flow resistance in terms of the total dimensionless shear stress is calculated, based on the Engelund formulation, (see **NST Reference Manual, Flow Resistance - θ-θ' Relationship**). This is then compared to the critical shear stress (the Shield's Criterion, defined by the user). If the computed shear stress is greater than the critical one, then the

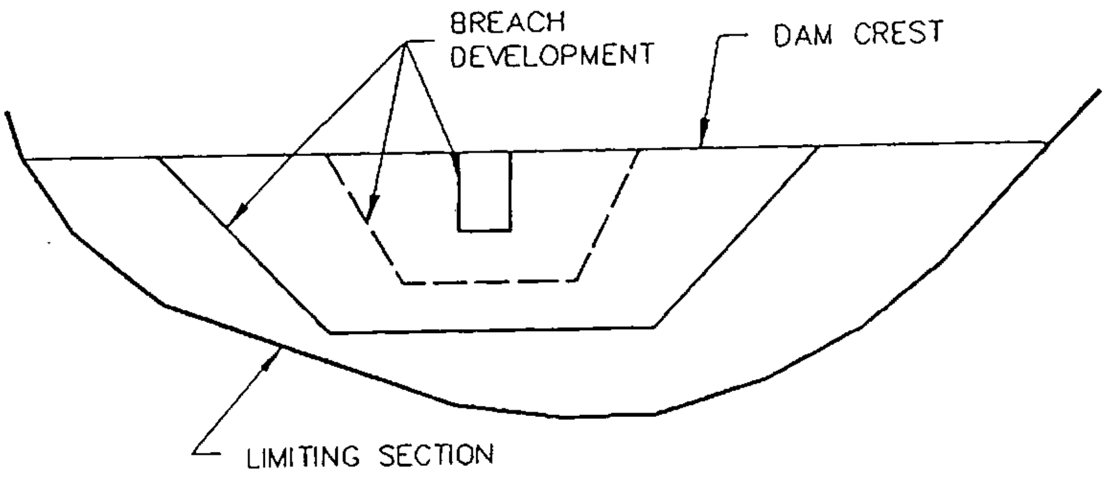


Figure 3 The breach is specified for example at times 0h and 1 h. The dashed line would be the linear interpolation for time 0.5 h.

sediment transport is calculated, otherwise it is set to zero.

The sediment transport rate, q_t, calculated from the Engelund-Hansen formula is in terms of m^3/s per metre-width of pure sediment only, and this must then be related to a change in bed (i.e. breach) level. It is assumed that the breach remains horizontal. From the given upstream and downstream slopes, the length of the breach in the flow direction, L_b, may be calculated. By application of the sediment continuity equation in the breach, the change in breach level dH_b in a time interval dt is given as:

$$\frac{dH_b}{dt} = \frac{q_t}{L_b(1-\epsilon)} \quad \text{or} \quad \Delta H_b = \frac{q_t}{L_b(1-\epsilon)} \Delta t \tag{1}$$

where

H_b is the breach level
q_t is the sediment transport rate (m^2/s)
ϵ is the porosity of the sediment
L_b is the breach length in the direction of flow
t is time

Modelling the variation of the width of the breach perpendicular to the flow direction is more difficult to relate to the classical theories of sediment transport. This is because the development of a wall boundary layer along the often very steep side walls of the breach, the theories for bed load and suspended load do not apply. As an approximation, the sediment transport at the sloping walls is assumed to be proportional to that in the central part of the breach. The coefficient of proportionality, x, (side erosion index) thus relates the increase in breach width, W_b, to depth as:

$$dW_b = 2 \cdot x \cdot dH_b \tag{2}$$

The side erosion index is generally of the order 0.5-1.0.

Piping Failure

An erosion based failure of a dam may be initially started by piping failure. In this case a flow forms through the dam wall. Erosion of the soil along the "pipe" causes the pipe to enlarge and the flow to increase. Eventually the dam will collapse into the void created by the piping failure erosion.

This failure scenario is modelled as an option in the erosion based breach development, ie. there are two options for the initial failure process:

(i) initial failure by overtopping;
(ii) initial failure by piping (after collapsing the erosion process returns to that of overtopping as described above).

The assumptions adopted for the piping failure are:

(i) the shape of the pipe is a circle;
(ii) the pipe is horizontal;
(iii) the pipe must always be running full;

The location of the pipe centre in the dam cross-section must be within the final breach shape and, if specified, the limiting section shape. When the circumference of the pipe extends outside these shapes, the hydraulic parameters are based on a reduced cross-section as illustrated in Figure 4.

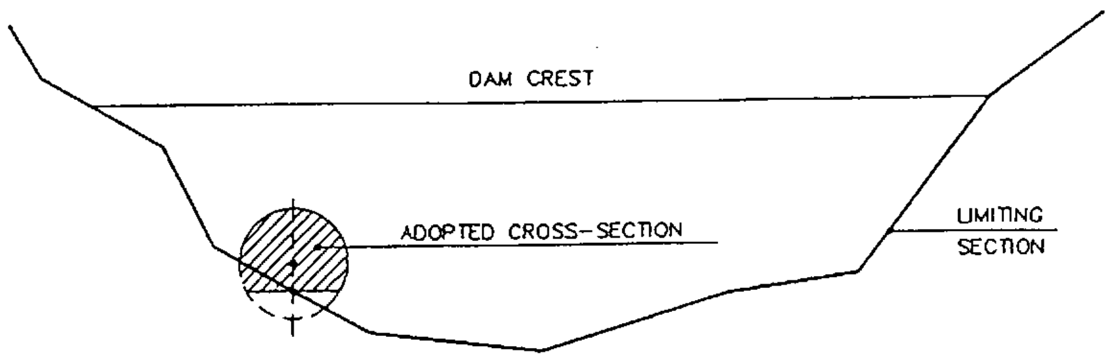


Figure 4 Adjustment of Pipe Cross-section.

The point of collapse is defined by a ratio, which is specified by the user, of the pipe diameter to the distance between the dam crest and the pipe obvert as illustrated in Figure 5. Once this ratio is exceeded part of the dam is collapsed to form a breach.

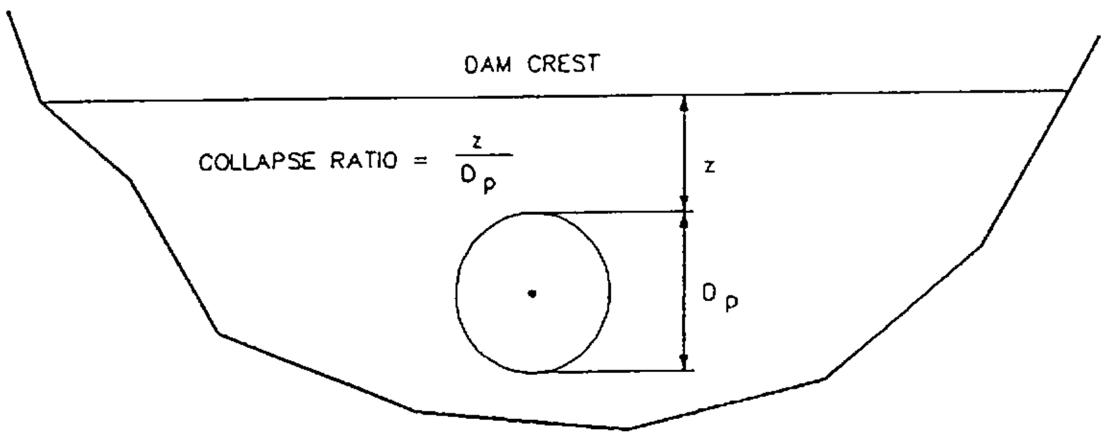


Figure 5 Piping Failure Collapse Ratio.

The shape of the breach is illustrated in Figure 6. The bottom width is first set equal to the pipe diameter and the breach level to the pipe invert. Not all of the collapsed material will typically be carried out without depositing on the bed of the breach. This is accounted for by a volume loss coefficient, f_{lost}, which is the fraction of material assumed to be washed out immediately after collapse. The remaining material is evenly distributed on the breach bed as shown in Figure 6. Note: these calculations are made using cross-section areas, not volumes, therefore, the length of the breach and consequently the upstream and downstream slopes of the dam faces are not taken into account.

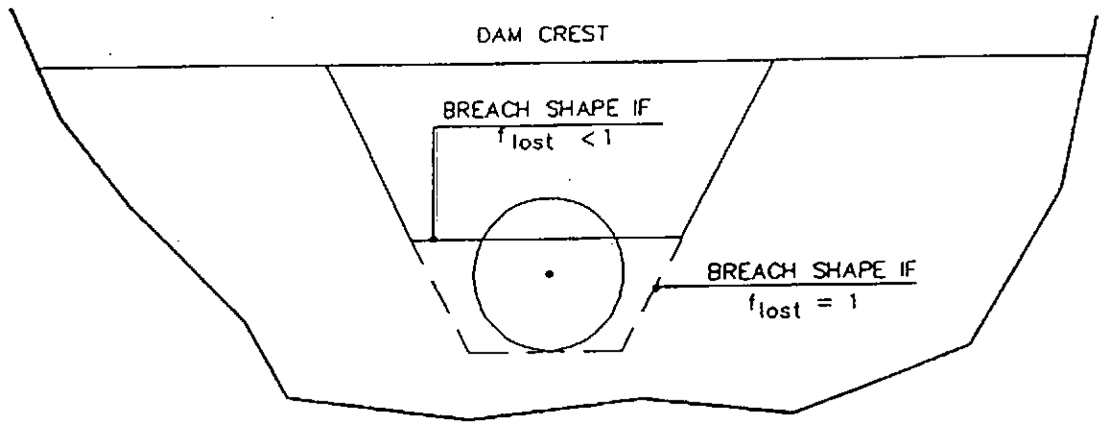


Figure 6 Breach Shape after Collapse.

Other parameters required are the initial diameter of the pipe at the start of failure and the pipe roughness, k_s, which is used to calculate the Darcy friction factor, f, according to

$$\frac{1}{\sqrt{f}} = 2 \log_{10}\left(\frac{12R}{k_s}\right) \tag{3}$$

where R is the hydraulic radius.

The discharge, Q_p, through the pipe is given by

$$Q_p = A \sqrt{\frac{2g\,\Delta H}{\left(1.5 + \frac{fL}{4R}\right)}} \tag{4}$$

where A is the flow area, L is the length of the pipe and ΔH is given by

$$\Delta H = h_1 - \max(h_2, z_{obv}) \tag{5}$$

where h_1 is the upstream water level, h_2 is the downstream water level and z_{obv} is the obvert of the pipe.

In Equation 4 an entrance head loss of half the velocity head in the pipe and, when there is a submerged outlet, an exit loss of one velocity head have been incorporated. Adopting a full velocity head loss in the latter case gives the transition between submerged flow and free outflow to be when the downstream water level, h_2, equals the pipe obvert, z_{obv}. (See Culverts, Q-h Relationship Calculated for a more detailed discussion.)

The enlargement of the pipe is based on the sediment transport relationship described above for breach erosion. The depth of water, y_p, used in calculating q_t is given by

$$y_p = \frac{\Delta H}{2} + D \tag{6}$$

where D is the pipe diameter.

By relating the volume of material eroded to the "effective" transport area (adopted here as the pipe length times half the circumference) the change in the pipe radius, ΔR_p, is defined as

$$\Delta R_p = C_{cal} \frac{q_t}{2L_p(1-\epsilon)} \Delta t \tag{7}$$

where C_{cal} is a "calibration" coefficient entered in Menu A.5.A.1.1. No sound recommendation can be given for the value of C_{cal}. It has been provided so the user can adjust the rate of change of R_p if so desired, or some data becomes available for calibrating the rate of formation of the pipe.

Limiting Cross-Section

There may be a boundary of an irregular shape beyond which the breach will not develop. This limit may be the surface of hard rock or it may be the natural bottom of the valley. To allow for a boundary of this type a cross section can be specified. In this case only the part of the trapezoid inside the cross-section is used for calculation of the flow. This is illustrated in Figure 7.

Flow Calculations

Once the breach geometry has been computed the flow can be calculated.

The flow over the dam crest and the flow through the breach are often quite different with respect to water depth, velocity, and flow state. At the breach the water level is lower and the velocity is higher than at the dam crest. When the flow is

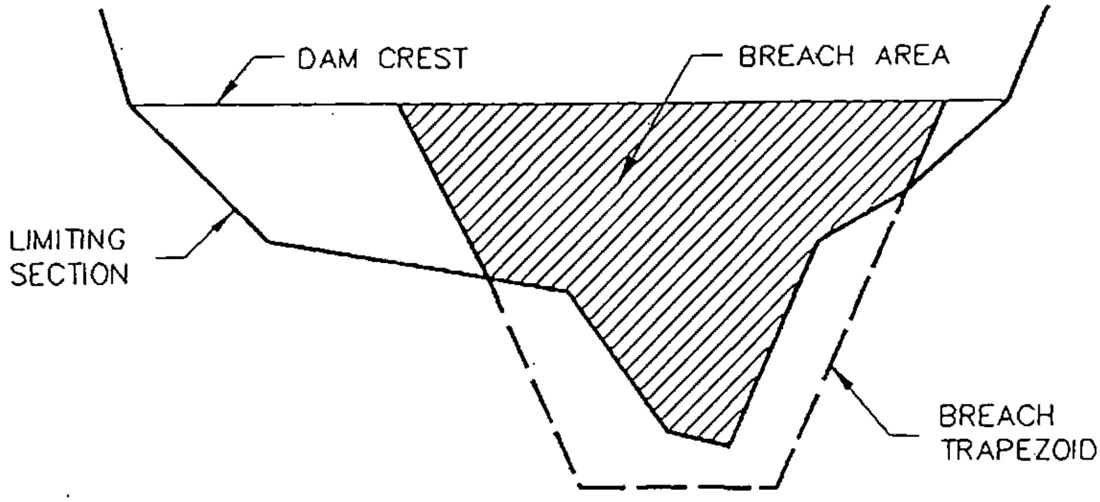


Figure 7 The breach (shaded area) is the area of the trapezoid inside the limiting section.

drowned at the breach the flow at the crest may be either free overflow or drowned. These two flows are therefore calculated separately.

The upstream water level (h_1), the downstream water level (h_2) and the total flow (Q) from the last iteration are used. (See **Appendix A: Scientific Background, Structures**).

The flow through the breach is calculated as follows:

Zero Flow

The water level at both sides of the structure is below the breach bottom level. No water can pass through the breach. Therefore the coefficients in the momentum equation are:

$$\alpha = 0$$
$$\beta = 1$$
$$\gamma = 0$$
$$\delta = 0$$

Free Flow or Drowned Flow?

The inflow head loss (ΔH) can be described as:

$$\Delta H = \zeta \frac{V_{bc}^2}{2g} = \left(h_1 + \frac{V_1^2}{2g}\right) - \left(h_{bc} + \frac{V_{bc}^2}{2g}\right) \tag{8}$$

h_1 : upstream water level
V_1 : velocity at h_1
V_{bc} : critical velocity in breach
h_{bc} : water level at the breach at critical flow
g : acceleration due to gravity
ζ : head loss coefficient (see **Appendix A: Scientific Background, Structures**)

In Equation (8) only h_{bc} and V_{bc} are unknown. The equation is solved iteratively. From h_{bc} the flow area A_{bc} can be calculated. The critical flow is then calculated according to Equation (9).

$$Q_{bc} = A_{bc} \, V_{bc} \tag{9}$$

The next step is to check if the flow is in fact critical and not drowned.

Inequality (10) gives the conditions for free flow.

$$\left(h_{bc} + \frac{V_{bc}^2}{2g}\right) - \left(h_2 + \frac{V_2^2}{2g}\right) \geq \zeta_2 \frac{V_{bc}^2}{2g} \tag{10}$$

$$\textit{Energy head upstream} - \textit{Energy head downstream} \geq \textit{outflow loss}$$

If (10) is not valid the flow is drowned and the breach water level, h_b, and the breach velocity, V_b, must be calculated from Equation (11).

$$\left(h_1 + \frac{V_1^2}{2g}\right) - \left(h_2 + \frac{V_2^2}{2g}\right) = \zeta_1 \frac{V_b^2}{2g} + \zeta_2 \frac{V_b^2}{2g} \qquad (11)$$

Energy head upstream − Energy head downstream = Inflow loss + Outflow loss

In Equation (11) only V_b is unknown and can be explicitly solved. The water level in the breach (h_b) is calculated from Equation (12):

$$h_b = \left(h_1 + \frac{V_1^2}{2g}\right) - \zeta_1 \frac{V_b^2}{2g} - \frac{V_b^2}{2g} \qquad (12)$$

Water level at structure = Energy head upstream − Inflow loss − Veloc.head in breach

Now the drowned flow, Q_b, can be calculated from V_b and A_b.

A last check is made to decide if the flow is drowned or free. If $Q_b > Q_{bc}$ the flow is free, otherwise it is drowned.

Before the coefficients α, β, γ and δ can be calculated a similar calculation must be made for the crest to obtain the flow (Q_c), the water level (h_c) and the flow area (A_c) at the crest.

The two flows are then added to get the total flow (Q_t). The coefficients are then calculated separately for the breach flow and the crest flow.

Coefficients for free overflow

Equation (13) can be derived from Equation (8) in a similar way as Equation (9) was derived from Equation (6) in **Appendix A: Scientific Background, Structures.**

$$Q_p = (h_a - h_p)\sqrt{\frac{2g}{(h_1-h_p)\left(\frac{1+\zeta_1}{A_p^2} - \left(\frac{Q}{Q_p}\right)^2 \cdot \frac{1}{A_1^2}\right)}} \qquad (13)$$

In this equation, the subscript p is substituted by b for breach flow and by c for crest flow. Q is the total flow. The coefficients can now be calculated:

$\alpha_p = -a/\alpha_q$
$\beta_p = 1$
$\gamma_p = 0$
$\delta_p = -a \cdot h_p/\alpha_q$

where

$$a = \sqrt{\frac{2g}{(h_1-h_p)\left(\frac{1+\zeta_1}{A_p^2} - \left(\frac{Q}{Q_p}\right)^2 \cdot \frac{1}{A_1^2}\right)}}$$

and α_q is the "head loss factor" for the free flow.

Coefficients for drowned flow

From Equation (6) the following equation can be derived:

$$Q_p = (h_1 - h_2)\sqrt{\frac{2g}{(h_1-h_2)\left[\frac{\zeta}{A_p^2} - \left(\frac{Q}{Q_p}\right)^2\left(\frac{1}{A_1^2} - \frac{1}{A_2^2}\right)\right]}} \qquad (15)$$

The coefficients become:

$\alpha_p = -b$
$\beta_p = 1$
$\gamma_p = b$
$\delta_p = 0$

where

$$b = \sqrt{\frac{2g}{(h_1 - h_2)\left[\frac{\zeta}{A_p^2} - \left(\frac{Q}{Q_p}\right)^2\left(\frac{1}{A_1^2} - \frac{1}{A_2^2}\right)\right]}}$$

To avoid instabilities the difference (h_1-h_2) is never allowed to become less than DELHS specified in Menu G.5.5.

Coefficients for the Total Flow

The coefficients for the total flow are calculated as

$$\begin{aligned} \alpha &= \alpha_c + \alpha_b \\ \beta &= 1 \\ \gamma &= \gamma_c + \gamma_b \\ \delta &= \delta_c + \delta_b \end{aligned}$$

in which the subscript b represents the breach and c the crest.

14.0 ADD-ON MODULE: DAMBREAK

14.1 Dambreak Setup

14.1.1 General

Most dambreak setups consist of a single or several channels, a reservoir, the dam structure and perhaps auxiliary dam structures such as spillways, bottom outlets etc. Further downstream the river may be crossed by bridges, culverts etc. It is important to describe the river setup accurately in order to obtain reasonable results. There is no limit to the number of dam structures in a MIKE 11 model.

14.1.2 River channel setup

Setting up the river channel description in the cross section database is the same for dambreak models as it is for other types of models. However, due to the highly unsteady nature of dambreak flood propagation, it is advisable that the river course be described as accurately as possible through the use of as many cross-sections as necessary, particularly where the cross-sections are changing rapidly.

Another consideration is that the cross-sections themselves should extend as far as the highest modelled water level, which will normally be in excess of the highest recorded flood level. If the modelled water level exceeds the highest level in the cross-section database for a particular location, MIKE 11 will extrapolate the <u>PROCESSED</u> data, i.e. A(h), R(h), b(h) etc.

14.1.3 Reservoir description and appurtenant structures

Reservoir

In order to obtain an accurate description of the reservoir storage characteristics, the reservoir may be modelled as a single h-point in the model. This point may also correspond to the upstream boundary of the model where inflow hydrographs are specified.

The description of the reservoir storage is carried out directly in the processed data. Figure 14.1 shows the description for a typical setup (Teton Dam, USA). The only columns which contain 'real' data are those containing the water level and the additional flooded area.

In this way the surface storage area of the dam is described as a function of water level. The lowest water level should be somewhere below the final breach elevation of the dam, and should be associated with some finite flooded area. (This first value, hence, describes a type of 'slot' in the reservoir).

The cross-sectional area is set to a large finite value. It is only used in calculating the inflow headloss into the breach as:

$$\Delta H = \frac{V_S^2}{2g} \cdot C_i \left(1 - \frac{A_S}{A_{res}}\right)$$

where V_S = velocity through breach
C_i = inflow headloss coefficient
A_S = flow area through breach
A_{res} = cross-section area in reservoir

A.6.5.P PROCESSED RIVER CROSS-SECTION DATA

River name : POND Chainage : 0.000 km	Coor Type: 1 X-Coor: -5.000 Y-Coor: 0.000	Section Ends (Type 2) X-Left : Y-Left : X-Right: Y-Right:
Resistance Radius Number of Levels : 4	Dir Type: 0 Dir:	

Level (m)	Cross Sec Area (m2)	Radius (m)	Storage Width (m)	Add Storage Area (m2)	Resistance Factor	Conveyance AR^(1/2)*Rf
1532.20	99999.0	1.000	0.000	1000	1.00	99999.0
1543.83	99999.0	1.000	0.000	2360000	1.00	99999.0
1593.23	99999.0	1.000	0.000	4740000	1.00	99999.0
1612.23	99999.0	1.000	0.000	7940000	1.00	99999.0

<I> Insert row (before cursor) <D> Delete row <P> Plot Processed Data
<Esc> end and update <'K'S> update <'Q> end WITHOUT update <F1> Help.

Figure 14.1 Processed data for Teton dam reservoir.

Hence, in order to obtain a reasonable headloss description, it is only necessary that $A_{res} >> A_S$ such that $(1-A_S/A_{res}) = 1$.

The hydraulic radius is not used at all in the calculation and hence any <u>non-zero</u> value may be specified.

The width b should be set to zero.

The total surface area of the reservoir is calculated as:

$$A_{TOTAL} = (B \times 2\Delta) + \text{Add. flooded area}$$

Since the total surface area is already described by the additional flooded area, the first term should equal zero.

It may be practical to locate the dambreak structure on a separate branch which contains 3 calculations points only, as shown in Figure 14.2.

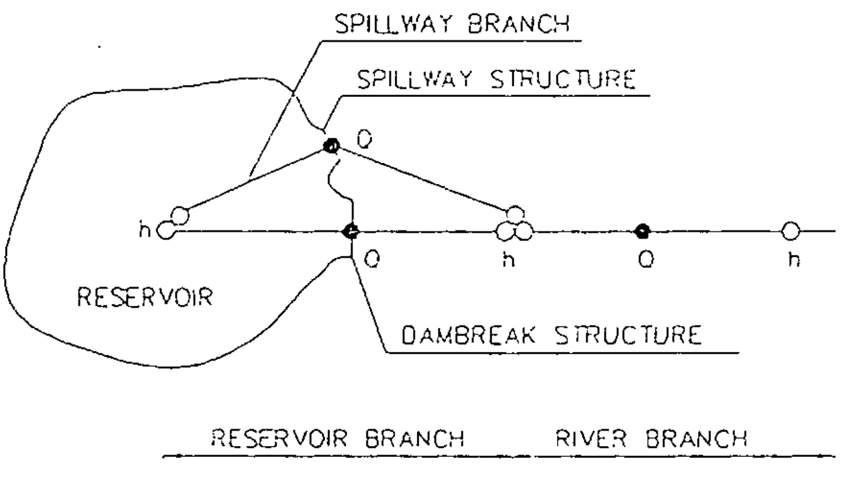


Figure 14.2 Typical setup for dambreak simulation.

The dam

At the Q point where the dambreak structure is located, the momentum equation is replaced by an equation which describes the flow through the structure. This may be either critical or sub-critical. A check on the energy levels at the structure and at the next downstream h-point is first carried out to determine which description is applicable. Refer to the MIKE 11 HD Reference Manual, Dambreak section.

As the momentum equation is not used at the Q-point, the ΔX step used between the adjoining h-points is of no consequence. The maximum ΔX step specified in Menu A.5.1 should, however, be greater than the difference between given chainages to prevent the insertion of interpolated cross sections.

Spillways and other structures

If a spillway is added to the dam itself, it should be described as a separate branch, as shown in Figure 14.3.

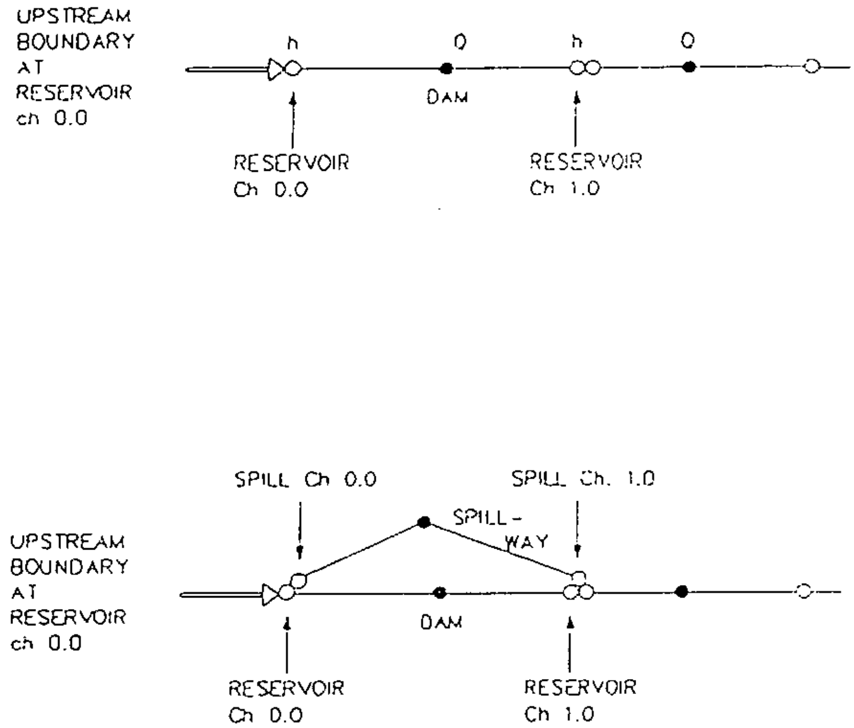


Figure 14.3 Setup with dam and spillway structures.

At the node where the two branches meet, the surface flooded area is taken as the sum of the individual flooded areas specified at each h-point.

Hence, if the reservoir storage has already been described in the reservoir h-point, the spillway h-point should contain no additional surface areas. In this case both 'b' and 'Additional flooded areas' should be set to zero. The cross sectional area, hydraulic radii etc. may be given as for the reservoir branch.

14.1.4 Boundary conditions

For the running of the dambreak, boundary conditions must be specified at both the upstream and downstream limits of the model.

The upstream boundary will generally be an inflow into the reservoir. Where the reservoir consists only of the 'dam' branch, this boundary may be specified (in Menu B.6) precisely at the chainage of the reservoir h-point (see Figure 14.4 (a)). If, however, a spillway branch is also added, it will not be possible to locate the boundary at this point, as it already constitutes an 'internal' boundary in the form of a node. Hence, any inflow may be specified as a lateral inflow (e.g. in the dam branch) at a point 1 metre downstream of the reservoir h-point (see Figure 14.4 (b)).

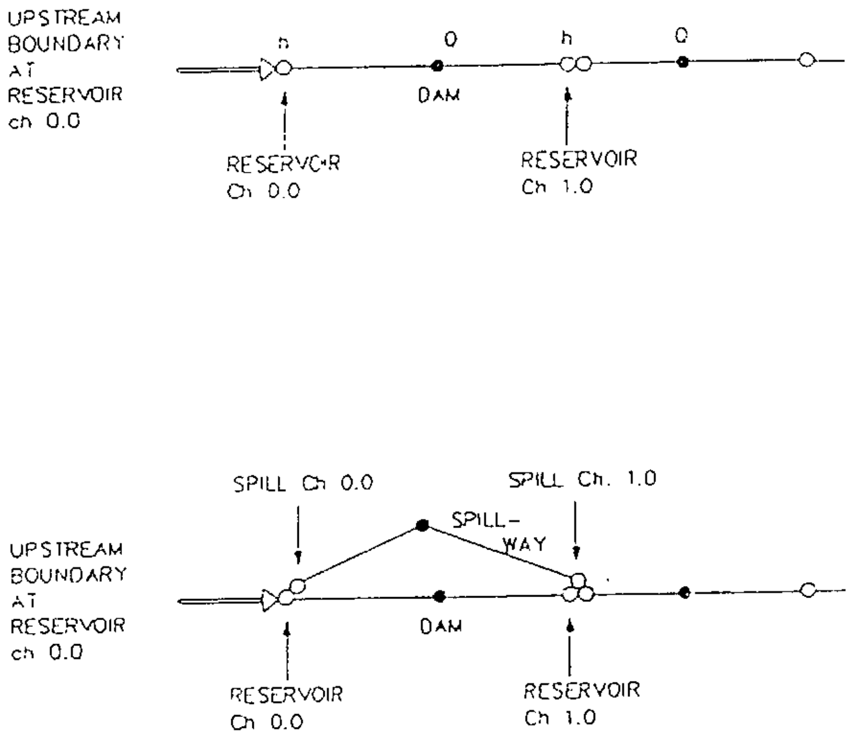


Figure 14.4 Upstream boundary specification.

14.2 Specifying the Dambreak

The specification of a dambreak requires the user to define relevant information from each of the following categories via Menu A.5.A.1 (see Figure 14.5).

```
A.5.A.1                        DAM BREAK STRUCTURES

 STRUCTURE no.  1 at River name  : RESERVOIR     Chainage (km):     0.500
 Head loss factor            |  Inflow    |  Outflow | Free overflow
 Positive flow .........        0.50         1.00        1.00
 Negative flow .........        0.50         1.00        1.00
 Crest level (m)        :  325.60        Crest length (m)       :  900.00
 Failure moment (1/2/3): 2
  1: Hours after start :
  2: Date and time      : 1992  10  3  11  0
  3: Reservoir water level :
 Failure mode (1/2)     : 2
  1: Time dependent  2 : Erosion based  (Engelund-Hansen formula)
 After time :     0.00   use :   1  time(s) the start timestep.

 Create dambreak text file (Y/N) : Y

 Limiting section (Y/N) : Y
  River name : RES LIM SEC        Topo ID : 1992
  Chainage :         0.500        Breach starts at x-coord. :    20.00
<Esc> end and update  <'K'S> update , <'Q> end WITHOUT update    <F1> Help.
```

Figure 14.5 Menu A.5.A.1.

a) Geometric specification
The dam height, crest length (perpendicular to river flow) and level should be specified.

b) Breach characteristics
The breach may be rectangular, triangular or trapezoidal in shape. For a known geometry failure (see below) the shape of the breach is specified as a function of time. For erosion failure the breach slope remains constant.

In either case a limiting section shape can be specified in Menu A.5.A.1. The limiting section can be of any shape and its dimensions are stored as raw data in the cross-section database using Menu A.6.5.R.

The limiting section allows an irregular shape to be used to define the breach limits. This is a useful feature which allows the natural shape of the river section at the dam site to be modelled. Only the shape of the dam breach lying inside the limiting section is used for calculating the hydraulic parameters. Refer to the MIKE 11 HD Reference Manual, Dambreak section for details.

c) Failure Moment
The manner in which the dam failure is to commence is specified as one of the following:

- A given number of hours after the start of the simulation.
- At a specified time (YEAR MONTH DAY HOUR MINUTE).
- Reservoir Water Level. In this case the dam failure will occur when the reservoir water level reaches the specified level.

d) Failure Mode

The failure mode may be defined as either

- Known geometry - the increase in breach dimensions as a function of time is specified in Menu B.6.5.E (option 15 in Menu B.6.5) in the time series database and in Menu B.5 for the BSF file.
- Erosion based - the increase in the breach depth is calculated from a sediment transport formula (Engelund-Hansen). The increase in breach width is calculated as the increase in breach depth multiplied by the side erosion index. If an erosion based failure is specified, further information is necessary. This information is entered into Menu A.5.A.1.1 and is described below.

1. Slopes of the upstream and downstream faces of the dam (horiz:vert)
2. Top width of the dam crest (i.e. parallel to flow direction) (m)
3. Representative grain diameter of dam core (mm)
4. Relative density of dam core material (specific gravity). Typical values: 2.50-2.70
5. Porosity of dam core material. Typical values: 0.3-0.5
6. Critical shear stress of dam material for transport (Shields criteria). Typical values: 0.03-0.06
7. The side erosion index as mentioned above
8. Final bottom level of breach (m)
9. Final bottom width of breach (m)
10. The breach slope (horiz:vert)

11. Initial failure made where
    1 - Breach failure
    2 - Piping failure

If the initial failure is by breaching the following data are required:

12a. Initial level of breach (m)
12b. Initial width of breach (m)

If initial failure is by piping (development of a pipe followed by collapse and formation of a breach which continues to erode) the following data are required:

13a. Starting level of pipe (m)
13b. Initial diameter (m)
13c. Roughness of the pipe wall (m)
13d. Collapse ratio
13e. Volume loss ratio
13f. Calibration coefficient

For details of the above data items and of the collapse mechanism refer to the MIKE 11 HD Reference Manual, Dambreak section.

```
A.5.A.1.1          DAM BREAK STRUCTURE  -  EROSION FAILURE

STRUCTURE no.   1 at :                 | Limits of breach geometry
  River name    : RESERVOIR            |   Final bottom level     :   320.00 m
  Chainage      :       0.500 km       |   Final bottom width     :    50.00 m
  Crest level   :   325.60 m           |   Breach slope           :    1.000
  Crest length  :   900.00 m           |
                                       | Initial Failure Mode     : 1 (1/2)
Dam geometry                           | 1 - Breach failure initially
  Upstream slope        :   2.000      |   Initial level          :   325.60 m
  Downstream slope      :   2.000      |   Initial width          :     0.00 m
  Top width             :   30.00 m    | 2 - Piping failure initially
                                       |   Starting level         :     0.00 m
Material properties                    |   Initial diameter       :    0.10 m
  Grain diam            :   0.010 mm   |   Roughness              :   0.010 m
  Rel. density          :   2.650      |   Collapse ratio (D/y)   :    1.00
  Porosity              :    0.30      |   Volume loss ratio      :    0.00
  Crit. shear stress    :   0.050      |   Calibration coef       :    1.00
  Side erosion index    :    0.50      |

<Esc> end and update  <^K^S> update , <^Q> end WITHOUT update    <F1> Help.
```

Figure 14.6 Menu A.5.A.1.1.

14.3 Initial Conditions

In many cases dam failures may occur on a dry river bed downstream. However, such initial conditions are not possible in MIKE 11, which requires a finite depth of water to be present throughout the entire model in order to ensure the 'connectivity' of the finite difference algorithm, and allow 'wetting' of several grid points within a timestep.

Hence, before a dambreak is actually simulated, it is expedient to create a steady state 'hotstart' file which can be used for all subsequent dambreak simulations.

The easiest way to create such a file is to make a setup identical to that used for the dambreak with the following exceptions.

1) A small lateral inflow is added at the first h-point in the river downstream of the dam. This will ensure some depth of water in the river from which a steady state may be reached.

2) The inflow into the reservoir may be non-zero if desired.

3) The dambreak structure should be specified not to fail, i.e. to ensure that the maximum calculated reservoir level is greater than the specified failure reservoir level (i.e. failure will not occur during the generation of the steady state hotstart file).

Initial conditions (h and Q) for this 'hotstart' simulation must be specified in the supplementary data (Menu G), including the reservoir level.

This setup should be run until a steady state condition is reached (Q = constant = lateral inflow at the downstream boundary). If this file (.RRF) is very large, a further simulation may be carried out using this as a hotstart and run for a few time steps using the same boundary conditions as previously. This short file may then be used for all future hotstarts and the large file discarded.

14.4 Running the Dambreak

With the hotstart file ready, the dambreak simulation may now be carried out using the correct .RRF file. It is suggested a DELTA value of slightly more than the default of 0.5 be used in Menu G.5.5 to damp out short waves which may lead to numerical instabilities. A timestep of the order 1-10 minutes is suggested. Breach development information is printed on a text file named DAMBRKn.TXT, where n = 1,2....

DWG NO. 10667
SHEET 1

Plan No.	Authority	Drawing No.	Date	Description
HS1/1	Wyong SC	10047-1	10/6/92	Bathymetric Survey of saltwater Tumbi-Umbi, Wyong & Wallarah Cks
HS1/2	Wyong SC	10047-2	10/6/92	"" ""
HS1/3	Wyong SC	10047-3	10/6/92	"" ""
HS1/4	Wyong SC	10047-4	10/6/92	"" ""
HS1/5	Wyong SC	10047-5	10/6/92	"" ""
HS1/6	Wyong SC	10047-6	10/6/92	"" ""
HS1/7	Wyong SC	10047-7	10/6/92	"" ""
HS1/8	Wyong SC	10047-8	10/6/92	"" ""
HS1/9	Wyong SC	10047-9	10/6/92	"" ""
HS1/10	Wyong SC	10047-10	10/6/92	"" ""
HS1/11	Wyong SC	10047-11	10/6/92	"" ""
HS1/12	Wyong SC	10047-12	10/6/92	"" ""
HS1/13	Wyong SC	10047-13	10/6/92	"" ""
HS1/14	Wyong SC	10047-14	10/6/92	"" ""
HS1/15	Wyong SC	10047-15	10/6/92	"" ""
HS1/16	Wyong SC	10047-16	10/6/92	"" ""
HS1/17	DPW	R1069/46	Sept 79	Tuggerah Lakes Ourimbah Ck
HS1/18	DPW	R1069/46	Aug 79	Tuggerah Lakes The Entrance
HS1/19	DPW	R1069/46	Oct 78	Tuggerah Lakes The Entrance
HS1/20	DPW	R1069/46	Oct 78	Tuggerah Lakes The Entrance
HS1/21	DPW	R1069/46	Aug 79	Tuggerah Lkes East Budgewoi Lake
HS1/22	DPW	R1069/46	Sept 79	Tuggerah Lakes Nth Budgewoi Lake
HS1/23	DPW	R1069/46	Sept 79	Tugergah Lahes Wyong Ck
HS1/24	DPW	R1069/46	Sept 79	Tuggerah Lakes Tumbi-Umbi Ck Nth Entrance
HS1/25	DPW	R1069/46	Sept 79	Tuggerah Lakes Munmorah Lake
HS1/26	DPW	R1069/46	Sept 79	Tuggerah Lakes Munmorah Lake
HS1/27				

DWG 10667
SHEET 2

Plan No.	Authority	Drawing No.	Date	Description
HS2/1	DPW	1069/42	Sept 75	Tuggerah Lakes Hydrographic Survey
HS2/2	DPW	1069/42	Sept 75	"" ""
HS2/3	DPW	1069/42	Aug-Sept 75	"" ""
HS2/4	DPW	1069/42	Sept 75	"" ""
HS2/5	DPW	1069/42	Sept 75	"" ""
HS2/6	DPW	1069/42	Aug-Sept 75	"" ""
HS2/7	DPW	1069/42	June-Sept 75	"" ""
HS2/8	DPW	1069/42	June-Sept 75	"" ""
HS2/9	DPW	1069/42	June-sept-Oct 75,Sept 76	"" ""
HS2/10	DPW	1069/42	April-Oct 75	"" ""
HS2/11	DPW	1069/42	April-June-July-Aug-Sept-Oct 75	"" ""
HS2/12	DPW	1069/42	July 75 Sept 76	"" ""
HS2/13	DPW	1069/42	Oct 75	"" ""
HS2/14	DPW	1069/42	May 75	"" ""
HS2/15	DPW	1069/42	April-May 75	"" ""
HS2/16	DPW	1069/42	April,June,Oct 75	"" ""
HS2/17	DPW	1069/42		"" ""
HS2/18	DPW	1069/42	May-june 75	"" ""
HS2/19	DPW	1069/42	May-June-Oct 75	"" ""
HS2/20	DPW	1069/42	May 75	"" ""
HS2/21	DPW	1069/42	April-May 75	"" ""
HS2/22	DPW	1069/42		"" ""
HS2/23	DPW	1069/42	June,Oct 75	"" ""
HS2/24	DPW	1069/42	May-June-Oct 75	"" ""
HS2/25	DPW	1069/42	March-May-Aug 75, Sept 76	"" ""
HS2/26	DPW	1069/42	Sept 76	"" ""
HS2/27	DPW	1069/42	June 75	"" ""
HS2/28	DPW	1069/42	May-June-Sept 75	"" ""
HS2/29	DPW	1069/42	March-May-Aug-Sept 75 Sept 76	"" ""
HS2/30	DPW	1069/42	June-Sept 75	"" ""

DWG NO. 10667
SHEET 3

Plan.No.	Authority	Drawing No.	Date	Description
MS1	DMR	0335505BC0105	16/1/79	Bridge over Ourimbah Ck
MS2	PWD	85230-9X		Ourimbah Sewage Rising main WS12 Railway Bridge Crossing
MS3	Dept of rail	1377-29,588		Main Nth-hornsby to Newcastle-bridge Alterations
MS4	DMR	355B908	20/1/67	Bridge over Tuggerah Lake
MS5	DMR	335B908	20/1/67	Bridge over Tuggerah Lake
MS6	DMR	6003505 BC0064A	24/1/86	Twin Bridges over Wyong River
MS7	Dept of Rail	183-3	4/5/48	Wyong Ck
MS8	Dept of Rail	183-1	4/8/48	Wyong Ck
MS9	NSW R		16/4/31	Hornsby-Newcastle
MS10	Dept of Rail	96-112	30/11/34	Nth Renewal of 8 Ft opening
MS11	DMR	10B420	26/10/59	Bridge over Ourmibah Ck
MS12	DMR	0010505B05025	21/6/72	Bridge over Ourmibah Ck
MS13	DMR	4266	16/1/79	Bridge over Ourmibah Ck
MS14	DMR	0010505BC0469	2/9/68	Conversion of disused railway bridge at Wyong
MS15		1263-35726	2/9/68	Wyong Ck bridge
MS16	Dept of Rail	952/35795	10/9/45	Proposed repalcement of existing bridge
MS17		10421-5	12/11/29	Poster St