WORORONORA RIVER FLOOD STUDY

PUBLIC WORKS

August 1991

WORONORA RIVER FLOOD STUDY

PWD REPORT NO.86011
ISBN NO. 724030190
August 1991

Foreword

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

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

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

1. Flood Study

- determine the nature and extent of the flood problem.

2. Floodplain Management Study

- evaluates management options for the floodplain in respect of both existing and proposed development.

3. Floodplain Management Plan

- involves formal adoption by Council of a plan of management for the floodplain.

4. Implementation of the Plan

- construction of flood mitigation works to protect existing development.
- use of Local Environmental Plans to ensure new development is compatible with the flood hazard.

The Woronora River Flood Study constitutes the first stage of the management process for the areas adjacent to the lower reaches of the river from the Needles to the Georges River confluence.

Contents

List of Tables

List of Figures

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 of being exceeded; it would be fairly rare but it would be relatively large.
Australian Height Datum (AHD)	a common national surface 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.
Extreme Flood Event	an approximation of the flood calculated to be the maximum which is likely to occur.
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 the standard 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 floodwater 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 high 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.
Hydrograph	a graph which shows how the discharge changes with time at any particular location.
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.

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 means and timing by which the plan will be implemented.

Mathematical/Computer Models

the mathematical representation of the physical processes involved in runoff and streamflow. These models are often run on computer 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.

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.

1. Summary

A flood study of the Woronora River was undertaken to determine the design flood levels for the 1%, 2% and 5% AEP events, as well as the levels for an extreme flood event. The defined study area extends from the Needles to the Georges River confluence, and involves a river length of approximately 10.8 km.

The Woronora River has a catchment area of about 174 sq km to the Georges River. Woronora Dam controls runoff from an area of 78.2 sq km within the catchment. The effect of the dam on flood flows into the study area was considered as part of the study.

The first phase of the study involved an extensive compilation of relevant data which have been presented separately in a Compendium of Data. The historical information included recorded flood levels from events between 1933 and 1988 (plus a single level for the highest known flood in 1898), rainfall records for these events, streamflow records on the Woronora River from 1924 to the present, tidal measurements in the river in 1985/86 and tidal records at Fort Denison, and topographic data, particularly from a detailed hydrographic survey of the river in 1984/85.

The flood study was carried out using a mathematical modelling approach. A hydrologic model was used to convert rainfall to runoff and a one-dimensional unsteady flow hydraulic model was used to convert runoff to flood levels. The models were calibrated and verified against information from historical floods.

Nine historical rainfall-runoff events between 1933 and 1988 were simulated using the hydrologic model. The results indicated that the model was generally able to achieve good agreement with recorded streamflow hydrographs.

Six historical flood events were simulated using the hydraulic model. The model was calibrated to the April 1988 event which is the largest flood since dredging of the Woronora River was carried out in the 1968 to 1976 period. The other events occurred before or during the dredging period and there is some uncertainty regarding the river channel topography at the time of those events. The results using adjusted cross-section data for these events provided general confirmation of the capability of the hydraulic model to reproduce observed flood behaviour in the river.

Design rainfall intensities and temporal patterns for the 1%, 2% and 5% AEP events were taken from the 1987 version of Australian Rainfall and Runoff. The rainfall for the extreme event was estimated from the Bureau of Meteorology's Bulletin 51. These rainfalls were input to the hydrologic model and the generated flows were then run through the hydraulic model. The Georges River flood levels adopted as the downstream boundary conditions for these events were defined by the PWD.

The 1%, 2% and 5% design flood levels at each modelled cross-section along the river are given in **Figure 9** and **Table 9**. The flow distributions between the left overbank, main channel and right overbank and the average velocities in each segment are given in **Tables 11** to **13**. Flood contours for the 1%, 2% and 5% events, together with velocities at representative locations, are presented in **Figures 11** to **13**.

Flood levels were also determined for an extreme rainfall event over the catchment. The modelled levels for this event, as given in **Table 14**, are from 2.0 to 3.3 m higher than the flood levels for the 1% AEP event.

Woronora Dam was most commonly full (over 90% full in 18 of the last 20 floods) at the start of historical flood events. Modelling of the 1% AEP event with different starting storage levels indicated that there would be negligible reduction in peak stream flows into the study area for initial storage depletion up to 10 per cent of capacity. Hence the 1% design flood levels are not sensitive to the variations in storage conditions commonly experienced in Woronora Dam.

Sensitivity runs were also carried out to determine the effect of changes in various parameters on the 1% AEP flood levels, namely: increased river channel roughness, higher Georges River flood levels and higher Woronora River bed levels. The results of these runs, as presented in **Figure 10**, indicate that flood levels could vary by as much as 0.3 to 0.5 m above adopted levels as a result of parameter variation.

2. Introduction

The Woronora River is a tributary of the Georges River, joining that river some 10 kilometres upstream of its entrance to Botany Bay. The catchment area of the Woronora River at its confluence with the Georges River is about 174 square kilometres. The catchment falls within the area administered by Sutherland Shire Council, Campbelltown City Council and Wollongong City Council. The study area falls within the Sutherland Shire Council area.

Woronora Dam, operated by the Sydney Water Board for water supply, is a major feature of the catchment. The catchment area at the dam is 78.2 square kilometres.

The location of the Woronora River catchment is shown in **Figure 1.** The area of interest for this study is the lower reach of the river from the Needles to the Georges River confluence. The study reach has a length of about 10.8 kilometres (measured from the Como Railway Bridge which is located just downstream of the confluence). The study area is shown in **Figure 2.**

The Woronora River flood study has three main phases as follows:

- Collection and review of all available data
- Development of computer models to predict the river's flood behaviour
- Determination of design flood levels along the river for the 1%, 2% and 5% Annual Exceedance Probability (AEP) flood events, as well as for an extreme flood event.

The results of the first phase are presented in a separate Compendium of Data (Ref.1). This report presents the findings of the second and third phases of the study.

3. Available Data

A comprehensive survey of available data is given in the Compendium of Data prepared as part of this study (Ref.1). This section summarises the available data, with emphasis on the data utilised directly in the study.

3.1 RAINFALL DATA

The sources of rainfall data for the study were:

- The records from an extensive network of daily rainfall stations in the vicinity of the study catchment, operated by the Bureau of Meteorology; and
- The records from eight pluviometer stations, including one pluviometer operated by the Bureau and seven pluviometers operated by the Sydney Water Board

The daily rainfall records were analysed in order to identify significant historical rainfall events. The records were also used to construct isohyetal diagrams for the major events. The pluviometer records were used to define the temporal patterns of rainfall over the catchment for these events.

The rainfall gauge and pluviometer records for the events which were subject to hydrologic modelling are presented in **Appendix A**.

3.2 STREAMFLOW DATA

The sources of streamflow data for the Woronora River were:

- The records for Woronora Dam, from 1937 to the present; and
- The records from a stream gauging station on the Woronora River at Engadine (downstream of the dam), from 1924 to 1951.

The streamflow records provided a good indication of the relative severity of historical flood events in the study area. The twenty highest outflows from Woronora Dam and the ten highest flows at Engadine are listed in **Tables 1 and 2** respectively. As discussed in the Compendium of Data, there is considerable concern about the reliability of the 'recorded' discharge of 601 m^3/s at Engadine in 1943. This peak flow record may be in error and should be treated with caution.

The recorded streamflow hydrographs represent important data for calibration of a hydrologic model of the Woronora River catchment.

3.3 TIDAL DATA

Tide levels were measured by the Public Works Department at three locations along the Woronora River from October 1985 to January 1986. In addition, tidal flows were measured at the Woronora road bridge on 14th November 1985. The information from these measurements is useful for calibration of a hydraulic model for tidal conditions; in particular, determination of in-bank roughness values.

TABLE 1 - RANKING OF RECORDED FLOODS, WORONORA DAM

Rank	Date	Peak Water Level (m AHD)	Peak Surcharge over Weir (m)	Peak Discharge (m^3/s)	Approx Depletion* (%)
1	18/11/1961	169.67	0.79	501	4.3
2	10/2/1956	169.62	0.74	454	33.0
3	18/6/1949	169.54	0.66	381	0
4	30/4/1988	169.52	0.64	361	0
5	4/3/1977	169.51	0.63	354	1.6
6	19/2/1956	169.50	0.62	345	0
7	15/6/1952	169.50	0.62	345	7.8
8	26/7/1952	169.49	0.61	336	1.4
9	23/7/1950	169.46	0.58	308	0
10	12/3/1974	169.46	0.58	308	1.4
11	16/6/1950	169.45	0.57	300	0
12	1/5/1955	169.44	0.56	291	1.6
13	14/11/1969	169.43	0.55	283	0
14	13/5/1962	169.43	0.55	283	2.1
15	11/6/1964	169.41	0.53	266	3.1
16	23/3/1963	169.38	0.50	242	0
17	18/5/1963	169.38	0.50	242	0
18	30/8/1963	169.38	0.50	242	4.0
19	19/1/1951	169.37	0.49	234	3.1
20	6/8/1986	169.36	0.48	230	16.0

Note: * refers to depletion of storage prior to flood as percentage of storage capacity.

TABLE 2 - RANKING OF RECORDED FLOODS, WORONORA RIVER AT ENGADINE (1924 TO 1951)

Rank	Year	Peak Flow at Engadine (m^3/s)	Peak Flow at Woronora Dam (m^3/s)
1	1933	682	ND
2	1943	601	Doubtful reading
3	1949	374	381
4	1950	285	308
5	1927	272	ND
6	1945	267	243
7	1932	194	ND
8	1930	193	ND
9	1938	192	NDR
10	1937	163	NDR

Note: ND - Woronora Dam not completed
NDR - Records available are given as a gauge height, which has an unknown datum and cannot be cross-referenced to the current spillway configuration.

Tide level data are also available from the recorder maintained at Fort Denison by the Maritime Services Board (MSB). This information was considered in conjunction with the flood characteristics of the Georges River to define the hydraulic conditions at the confluence of the Woronora River and the Georges River.

The high tide levels at Fort Denison during likely flood events in the Woronora River were extracted from MSB tide records for the dates of these events. The high tide levels are summarised in **Appendix A**.

3.4 FLOOD LEVEL DATA

Historical flood level data for the study area were obtained primarily from:

- flood mark surveys and other flood level records held by Sutherland Shire Council
- flood levels obtained from interviews with local residents in 1985 and 1990
- levels estimated from photographs taken during floods
- levels taken from a 1967 report by Professor Munro
- levels at the Woronora Bridge from RTA records.

The flood level data were plotted on profiles of the river and are presented in **Appendix A**. The levels showed considerable scatter, due largely to the length of time between the occurrence of the flood events and the collection of level data.

The most reliable data were collected for events in 1933, 1956, 1961, 1969, 1974, 1984 and 1988. Review of the rainfall and streamflow records indicated that these events were included in the major historical rainfall-runoff events in the catchment, except for the 1984 event.

The flood level data are the most important information for calibration and verification of a hydraulic model of the Woronora River.

3.5 TOPOGRAPHIC DATA

The topographic data included information from a survey of the Woronora River by the Public Works Department in 1984/85. Cross-sections of the river channel and overbank areas were surveyed at 48 locations between the Georges River and the Needles. The cross-section information is appropriate for use in a hydraulic model of the river.

The location and details of surveyed cross-sections are shown on Public Works Department's Plan Room Cat Nos 7304, 7305, 7399, 7400 and 7401.

Sutherland Shire Council re-surveyed 5 cross-sections of the river in August 1990. The results of this survey indicated that there had been little change in the river channel at these locations since the 1984/85 survey. Reference Council Plan No 12323.

The river bed profile determined from the 1984/85 survey is presented in **Appendix A**, together with the 1990 survey bed levels.

The study reach of the Woronora River has been extensively dredged, with most of the dredging reported to have occurred in the 1968/69 to 1976 period.

3.6 SUMMARY OF AVAILABLE DATA

The availability of flood level, rainfall and streamflow data for various historical flood events is summarised in **Table 3.**

TABLE 3 - SUMMARY OF AVAILABLE DATA

	FLOOD DATE											
	1933	1946	1949	1956	1961	1969	1974	1975	1977	1984	1986	1988
1. Flood levels from Resident Interviews	7	-	3	5	2	9	-	2	1	15	2	7
2. Flood levels from Council Survey	3	4	-	11	14	-	8	-	-	-	-	-
3. Flood levels from resident's photographs	-	-	-	-	-	1	1	-	-	-	-	-
4. Flood levels from previous studies	3	-	-	2	2	1	-	-	-	-	-	-
5. Flood levels from newspaper research	-	-	-	-	-	1	-	-	-	-	-	-
TOTAL FLOOD LEVELS	13	4	3	18	18	12	9	2	1	15	2	7
No. of pluviometer records available	-	-	2	1	1	3	4	4	3	4	6	6
Flow estimates available at Woronora Dam	No	No	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Flow estimates available at DWR gauging station, Engadine	Yes	Yes	Yes	No	No	No	No	No	No	No	No	No

Note: 1961 flood levels include six levels identified as 1967 in Council survey.

Examination of this data and other available information indicates that the ranking of the major floods on the Woronora River in this century is as follows:

1. January 1933
2. February 1956
3. June 1949
4. November 1961
5. November 1969

Sufficient flood levels were obtained for these events, except the 1949 event, to enable their use in calibration and verification of a hydraulic model of the river. More recent events with a reasonable amount of flood level data occurred in March 1974 and April 1988.

The flood level recorded on the old Menai Bridge plans indicate that the 1898 flood would have exceeded all the major floods of this century. There is no other flood level information available for this event.

4. Study Approach

4.1 GENERAL

The available information, as presented in the Compendium of Data (Ref.1) and summarised in **Section 3**, is clearly inadequate to define accurately the 1%, 2%, and 5% probability design flood levels using only the historical records.

The appropriate alternative approach is to estimate the design flood levels using computer-based mathematical modelling techniques. The mathematical modelling involves two main components:

- A hydrologic model to estimate runoff hydrographs resulting from rainfall over the Woronora River catchment, and including the effect of Woronora Dam.

- A hydraulic model to determine the relationship between streamflows from the catchment and flood levels in the study area, taking into account also the Georges River as a downstream boundary condition.

The models are calibrated to the catchment and study area using the historical data. Calibration involves determination of the appropriate values of the model parameters to give good agreement between the model results and recorded data for selected historical events.

The models are then verified by simulation of independent historical events without changing the model parameter values. If satisfactory agreement is achieved between modelled and recorded behaviour, the models can be accepted as reliable and can be used with confidence for design events.

4.2 HYDROLOGIC MODEL

The hydrologic model is required to estimate streamflow hydrographs in the Woronora River at the upstream limit of the study area, ie. The Needles. The model is also required to estimate the runoff hydrographs from the local catchments draining to the river between The Needles and the Georges River confluence.

The runoff-routing model RORB, developed by Laurenson and Mein (Ref.2), is suited to the requirements of the study and was adopted. The model can simulate the major hydrologic features of the Woronora River catchment, including:

- Runoff-routing from rainfall

- The river reaches drowned by Woronora Dam

- The flood routing effect of the dam, including the storage and spillway characteristics

- The channel routing effects in the river between the dam and The Needles.

In the RORB model, the catchment is divided into sub-areas and the stream channel system is represented by a network of conceptual storages similar to the actual network. The RORB model has several important advantages over the alternative unit hydrograph procedure, including:

- The model can accommodate variations in rainfall temporal patterns across the catchment.

- The model reproduces the non-linear rainfall-runoff response observed in many gauged catchments.

The RORB model has received widespread acceptance in Australia, and substantial research has led to a satisfactory database for estimation of model parameters for ungauged catchments.

4.3 HYDRAULIC MODEL

The hydraulic model is required to represent the flood behaviour of the study reach of the Woronora River; in particular, to provide information on flood levels, discharges and velocities.

The hydraulic model representation includes:

- the inflow hydrographs from the upstream Woronora River catchment and the local sub-catchments as upstream boundary conditions (as determined by the hydrologic model)

- the Georges River stage hydrograph at the Georges River - Woronora River confluence as a downstream boundary condition

- unsteady flow effects in the study reach of the Woronora River.

The MIKE-11 model developed by the Danish Hydraulics Institute (DHI) was adopted as the hydraulic model for the study. MIKE-11 is a one-dimensional unsteady flow modelling package, capable of modelling unsteady flows in open channel systems through a numerical solution of the one-dimensional St Venant Equations (Ref.3). The model is capable of representing the important hydraulic phenomena in the study reach, as outlined above.

5. Hydrologic Modelling

5.1 RORB MODEL STRUCTURE

The runoff-routing model of the Woronora River catchment of about 174 sq km was assembled according to the procedures recommended in the RORB Users Manual (Ref.2). The layout of the RORB model developed for the catchment is shown in **Figure 3.**

The catchment area to The Needles is about 142 sq km and contains 19 sub-areas (A to S). The remaining seven sub-areas (T to Z) take account of local catchment runoff between The Needles and the Georges River confluence.

The river reaches in Woronora Reservoir are modelled as drowned channels. The model incorporates the stage-storage relationship for the reservoir and the stage-discharge relationship for the dam spillway.

5.2 MODELLING OF HISTORICAL EVENTS

5.2.1 General

Modelling of historical events was undertaken for the purposes of calibration and verification of the RORB model; in particular, determination of the values of the parameters m and k_c in the storage-discharge relations in RORB (Ref.2).

The value of the exponent m is normally set to 0.8 , and this value was adopted for the Woronora River catchment. The calibration phase is therefore largely concerned with determination of the value of k_c, which is an empirical coefficient applicable to the catchment.

The calibration and verification process also involves determination of catchment losses, which are subtracted from the total rainfall to give the rainfall excess (runoff producing rainfall).

The streamflow information available for most of the historical events are the recorded hydrographs at Woronora Dam and the streamflow record at the DWR gauging station at Engadine.

The selection of events for calibration and verification was as follows:

- In the early stages of the study, the 1969, 1974 and 1977 events were the only events with good pluviometer records for definition of temporal patterns of rainfall. These events were adopted for model calibration.
- The 1933, 1949, 1956 and 1961 events were adopted for model verification. There were some limitations in the available pluviometer data for these events, particularly in the catchment upstream of Woronora Dam. Hence generation of synthetic temporal patterns was required.
- Information from recent events in 1986 and 1988 was available to provide testing of the parameter values obtained in the early stages of the study.

The set of parameter values and catchment losses derived from the modelling of the historical events is as follows:

- Model parameters $m = 0.8$, $k_c = 14.5$

- Catchment losses Initial loss = 0 mm
  Continuing loss = 1.2 mm/hr

The above k_c value compares reasonably well with the value derived from the recommended regional relationship for ungauged catchments, as given in Australian Rainfall & Runoff, 1987 (Ref.4). The relationship for eastern New South Wales, with $m = 0.8$, is:

$$k_c = 1.22\,A^{0.46}$$

Where A = catchment area in sq km

This relation would give $k_c = 13.1$ for the Woronora River catchment.

The comparison of recorded and modelled peak discharges using the adopted model parameter and loss values for these events is presented in **Table 4**.

TABLE 4 - COMPARISON OF RECORDED AND MODELLED PEAK DISCHARGES FOR HISTORICAL EVENTS - WORONORA DAM

Event Type	Date of Peak	Peak Discharge Recorded (m^3/s)	Peak Discharge Modelled (m^3/s)	Error (%)	Time of Peak Recorded (hrs)	Time of Peak Modelled (hrs)
Calib-ration	14/11/69	283	287	1	0800	0800
	12/3/74	308	333	8	0100	0320
	4/3/77	354	356	1	1200	0900
Verifi-cation	23/1/33	- (682)	- (686)	- (1)	-	-
	18/6/49	381 (374)	378 (347)	-1 (-9)	1300	1230
	10/2/56	454	454	0	1100	1100
	18/11/61	501	473	6	1700	1700
Testing	6/8/86	230	171	-26	1600	1600
	30/4/88	361	357	1	0700	0700

Notes: Peak timing data were not available for the 1933 event.
The 1986 event was modelled with initial loss = 144 mm and continuing loss = 0.
Figures shown ( ) are for Engadine gauging station.

The modelled events are discussed briefly in the following Sections.

5.2.2 Calibration Events

The general procedure for the calibration of RORB to the recorded flows was as follows:

- The total rainfall for each event was derived from the daily rainfall records (0900 hrs to 0900 hrs) which spanned the period of the event, as indicated by the pluviometer data. The total rainfall was distributed spatially according to the variation in daily rainfalls across the catchment.

- The temporal pattern of rainfall across the catchment was taken from the available pluviometer records.

- The water level in Woronora Dam at the beginning of simulation of each event was established from the dam level records, which are taken at 0800 hours every day.

- Initial catchment losses were assumed to be zero because most of the modelled events occurred in generally wet periods and minor rainfall occurred prior to the main rainfall events. Continuing loss rates were then determined from recorded data as part of the calibration process.

The calibration events are described briefly as follows:

1969 Event

Total rainfalls for this event varied from 180 mm in the northern part of the catchment to 300 mm in the south. Temporal patterns of rainfall were recorded by pluviometers at Penshurst Tanks, Woronora Dam and Reverces. Analysis of the records indicated that the temporal pattern from the Woronora Dam pluviometer was more likely to produce the recorded peak flow timing than was the Reverces pluviometer. Hence the Woronora Dam temporal pattern was applied across the catchment. This led to good results with regard to peak flow magnitude and timing, as indicated in **Table 4**.

1974 Event

The 1974 event was one of the most well-defined events in terms of available data. The total rainfall varied from 150 mm in the north of the catchment to 290 mm in the south. Pluviometer records were available for Reverces, Woronora Dam, Cronulla and Mortdale.Thus, the temporal pattern of rainfall was reasonably defined across the catchment. The model calibration indicated good agreement with the recorded peak flow at Woronora Dam (**Table 4**).

1977 Event

The total rainfall for the 1977 event varied from 120 mm to 250 mm. Pluviometer records were available for Reverces, Woronora Dam and Cronulla. Good agreement was achieved between the modelled and recorded peak flows, but the timing of the peak appeared to be in error by three hours. Analysis of the data indicated a possible discrepancy in the recorded time of peak, and the likely timing error resulting from the modelling is 45 minutes. This error was considered to be acceptable.

5.2.3 Verification Events

The modelling procedure for the verification events was generally similar to the procedure for calibration. However, synthesis of temporal patterns was required because of the lack of pluviometer data for these events.

1933 Event

The 1933 event was recorded only by daily rainfall stations, so there was no information on the temporal pattern. The temporal pattern used for the 1956 event was adopted for the 1933 event, because both events were of similar magnitude and occurred in late summer. The stream gauging station at Engadine was the only source of peak flow data. The modelled peak flow at Engadine was in good agreement with the recorded peak flow. The available data did not permit timing of the modelled peak to be checked.

1949 Event

Total rainfalls for this event varied from 70 mm in the north to 160 mm in the south. There were no pluviometer records available in the catchment for this event. The pluviometer records for Penshurst Tanks and Cronulla, which are located near the northern part of the catchment, were used to define temporal patterns in the catchment. The modelled peak flow is in good agreement with the recorded peak flow.

1956 Event

Total rainfalls for this event varied from 210 mm in the north of the catchment to 330 mm in the south. A temporal pattern was available from the pluviometer record at Penshurst Tanks, while a temporal pattern was synthesised for the sub-areas above Woronora Dam. The agreement achieved with peak timing is quite good considering the poor temporal data. It is possible that the storm was of relatively even distribution across the catchment. This hypothesis is supported by the reasonably constant total rainfall volumes from north to south.

1961 Event

Total rainfalls for this event varied from 120 mm in the north to 415 mm in the south. The pluviometer records at Penshurst Tanks provided information on the temporal pattern. This information was augmented by synthesising a temporal pattern above Woronora Dam. The pluviometer records at Woronora Dam and Reverces for the 1969 event, with some modification, were used for this purpose. The 1961 storm was of similar magnitude and duration as the 1969 storm, and both events occurred in November.

5.2.4 Additional Events for Testing

The most recent large rainfall events which occurred in August 1986 and April 1988 were simulated using the RORB model as a further test of the model parameters adopted from the calibration ($m = 0.8$, $k_c = 14.5$).

1986 Event

The rainfall for this event occurred over about a three day period from the 4th to 6th August 1986. The total rainfall over the Woronora River catchment varied from about 360 mm in the north to 420 mm in the south.

The storage in Woronora Dam at the start of the event was 60 400 ML; ie. the reservoir was depleted by about 16% below capacity. The maximum spillway discharge for this event was 230 m^3/s, occurring at 1600 hrs on 6th August 1986. This discharge is ranked 20th in the list of highest recorded flows since 1937.

The isohyetal diagram for this event was constructed using total rainfall records from all available rainfall stations within and adjacent to the catchment. The total rainfall inputs to each sub-area of the RORB model were derived from the isohyetal diagram.

The records from six pluviometer stations were available to define the temporal pattern of rainfall over the catchment. The stations at Reverces, Woronora Dam and Yarrawarrah are within the catchment and were adopted for use in the RORB model.

The model was run first with an initial loss of zero and continuing loss of 1.2 mm/hr, as for the previously modelled events. The model results for this case indicated a peak outflow from Woronora Dam of 287 m^3/s at 0600 hrs on 6th August 1986. This result compares poorly with the magnitude and timing of the recorded peak outflow.

Review of the Woronora Dam records indicated that there was negligible water level rise for about 18 hours after the start of rainfall and that spillway discharge commenced after about 48 hours.

The initial loss value in the RORB model was increased to provide better agreement with the observed rise in water level. The maximum possible initial loss to give agreement between recorded and modelled hydrograph volumes is 144 mm; in this case, the continuing loss is zero.

The model run with an initial loss of 144 mm and continuing loss of zero gave an improved fit to the rise of the recorded outflow hydrograph. The modelled peak discharge was 171 m^3/s at 1600 hours on 6th August. The timing of the peak was in good agreement with the timing of the recorded peak, but there was a substantial error in magnitude.

The results indicated that the model did not perform well for the August 1986 event, largely because of the difficulty in representing the catchment response for high initial loss conditions. The catchment conditions were assessed by reviewing the rainfall records for Darkes Forest (Station No. 068024) in the upper catchment. The rainfalls in the months preceding the August 1986 event are compared with the long-term mean rainfalls as follows:

	Rainfall (mm)	
	1986	Mean (1894-1989)
July	21	103
June & July	45	247

This comparison indicates that a condition of extreme catchment dryness existed at the time of the August 1986 event. This condition did not exist for the other modelled events, and is not an appropriate condition for the design events.

Hence the relatively poor performance of the RORB model for the August 1986 event is not considered to be a major concern with regard to its applicability to estimation of design flood hydrographs.

1988 Event

The rainfall for this event occurred over about a three day period from the 28th to 30th April 1988. The total rainfall over the Woronora River catchment varied from about 260 mm in the north to 430 mm in the south. The records from the pluviometers at Reverces, Woronora Dam and Yarrawarrah were used to define the temporal pattern of rainfall over the catchment.

Woronora Dam was full at the start of the event. The maximum spillway discharge for this event was 361 m^3/s, occurring at 0700 hrs on 30th April 1988. This discharge is ranked 4th in the list of highest recorded flows since 1937.

The RORB model was run for this event with model parameter values of $m=0.8$ and $k_c=14.5$, zero initial loss and a continuing loss rate of 1.2 mm/hour. The model results indicated a good fit to the recorded peak discharge, in both timing and magnitude. The overall hydrograph shape was also reproduced well.

The isohyetal diagram for the April 1988 event and the recorded and modelled outflow hydrographs from Woronora Dam are presented in **Appendix B**.

The 1988 event is a higher magnitude flood than the 1986 event and is more representative of design conditions. The good results achieved for the 1988 event were considered to provide a satisfactory test of the RORB model for the Woronora River catchment.

5.2.5 Application of Model Results

The results summarised in **Table 4** indicate that the RORB model achieved good performance for a range of historical events (except the 1986 event), with a single set of model parameter values. The model was therefore adopted as the best technique for estimation of catchment runoff hydrographs to be input to the hydraulic model of the study reach of the Woronora River.

The most important inflow to the study area is from the Woronora River at Needles. The RORB model hydrographs at this location for selected historical events are presented in **Appendix B**.

5.3 DESIGN EVENTS

5.3.1 General

The RORB model was applied to estimate runoff hydrographs for design events with Annual Exceedance Probabilities (AEPs) of 1%, 2% and 5% respectively.

The adopted model parameter values were m = 0.8 and k_c = 14.5. The adopted catchment losses were initial loss = 0 and continuing loss = 1.2 mm/hour. These loss values are consistent with a high runoff condition for the catchment, as occurred in most of the historical events.

As a sensitivity analysis, the 1% design event was also run with the design loss rates given in Australian Rainfall & Runoff, 1987 (Ref.4), namely:

Initial loss	10 to 35 mm
Continuing loss	2.5 mm/hour

The results of these model runs indicated that the design discharges would not be significantly affected by the catchment loss rates. For example, use of the AR&R loss rates would reduce the 1% AEP peak discharge at the Needles by about 4 percent.

The storage conditions in Woronora Dam prior to historical flood events were assessed to provide a basis for adoption of dam conditions for the design events. The degree of storage depletion for the 20 largest floods events was as follows:

TABLE 5 - WORONORA DAM DEPLETION FOR HISTORICAL FLOOD EVENTS

Storage Depletion (% of Capacity)	No. of Events	Dates of Events
0	8	Jun 1949, Jun 1950, Jul 1950, Feb 1956, Mar 1963, May 1963, Nov 1969, Apr 1988
0.1 to 2.0	4	Jul 1952, May 1955, Mar 1974, Mar 1977
2.0 to 5.0	5	Jan 1951, Nov 1961, May 1962, Aug 1963, Jun 1964
> 5.0	3	Jun 1952, Feb 1956, Aug 1986

The dam was most commonly full at the start of historical flood events. Hence a dam-full condition was adopted for the design events. The effect of dam depletion on design flood levels was also tested, as a sensitivity analysis (see **Section 5.3.4**).

5.3.2 Design Rainfalls

Rainfall intensity-frequency-duration curves were derived for various points within and adjacent to the study catchment. The points corresponded to the locations of rainfall stations as indicated in **Table 6.**

TABLE 6 - STATIONS AT WHICH DESIGN RAINFALLS DERIVED

Station Number	Station Name
66040	Miranda Bowling Club
66049	Penshurst
66060	Sutherland
66090	Engadine
68024	Darkes Forest
68028	Helensburgh Post Office
68070	Woronora Dam
68159	Wedderburn (Booalbyn)
68160	Campbelltown (Kentlyn)
68177	Maddens Plains Golf Links
568069	Reverces

The rainfall curves were derived by the procedures outlined in Australian Rainfall & Runoff, 1987 (Ref.4).

Isohyetal diagrams were prepared for the design events using the point rainfall data. The diagrams were prepared for storm durations of 12, 18, 24, 36 and 48 hours for the 1%, 2% and 5% AEP events. The isohyetal diagram for the 1% AEP, 36 hour duration event is shown in **Figure 4.**

The total rainfalls in each sub-area of the RORB model for the design events were estimated from the isohyetal diagrams. The temporal patterns of rainfall were adopted according to the recommendations of Australian Rainfall & Runoff, 1987 (Ref.4).

5.3.3 Design Hydrographs

The RORB model was run for the range of storm durations, and the peak discharges at The Needles were compared in order to identify the critical duration. The modelled peak discharges at the Needles for the 1% AEP event are:

Storm duration (hrs)	12	18	24	36	48
Peak discharge (m^3/s)	1 228	1 160	1 173	1 282	1 234

From these results, the 36 hour duration was adopted as the critical storm duration for the study reach of the Woronora River. The modelled hydrographs at the Needles for the 1%, 2% and 5% AEP events for this duration are shown in **Figure 5.**

The hydrographs at the Needles, together with the runoff hydrographs from the local catchments between the Needles and the Georges Rivers confluence, were adopted as the design streamflow inputs to the hydraulic model of the Woronora River (see **Section 6.4**).

5.3.4 Effect of Woronora Dam Depletion

The RORB model was also run for a dam depletion of 10% (of storage capacity) at the start of the design events. The peak discharges at the Needles for the 1% and 5% AEP events with a 36 hour storm duration are summarised as follows:

Dam Depletion (%)	1% AEP Peak Discharge (m^3/s)	5% AEP Peak Discharge (m^3/s)
0	1 282	985
10	1 267	891

The volume corresponding to 10% depletion of Woronora Dam is 7.2 million m^3, whereas the total volumes of the design flood hydrographs at the Needles are:

1% AEP event	54.7 million m^3
5% AEP event	39.1 million m^3

The available volume due to depletion of Woronora Dam is small compared to the volumes of the design flood hydrographs, particularly for the 1% AEP event. Hence the 1% peak flows into the study area, and the 1% flood levels, will not be affected significantly by the initial storage conditions in Woronora Dam.

5.4 EXTREME EVENT

An extreme event was also modelled using RORB. For the purposes of this study, the rainfall for the extreme event was estimated as the maximum rainfall over the catchment in a 6 hour period, as derived from the Bureau of Meteorology Bulletin 51 (Ref 5).

The estimated maximum 6 hour rainfall over the catchment is 447 mm. The temporal pattern for this event was also taken from Bulletin 51. The extreme event was modelled with the same catchment losses and RORB parameter values as the design events. The modelled peak discharge in the Woronora River at the Needles was 2 720 m^3/s.

This peak discharge is approximately 2.1 times the peak discharge at the Needles for the 1% AEP event. Hence this extreme event was considered appropriate for assessing river flooding for conditions much more severe than the 1% AEP event.

6. Hydraulic Modelling

6.1 MIKE-11 MODEL STRUCTURE

6.1.1 General

The MIKE-11 model layout adopted for the study area is shown in **Figure 6**. The model incorporates 46 cross-sections along the Woronora River between the Needles and the Georges River and a cross-section of Forbes Creek upstream of the Woronora River confluence. The cross-sections, as shown by location and number in **Figure 6**, correspond to the river cross-sections surveyed in 1984/85.

The model also incorporates eight streamflow inputs which correspond to the runoff hydrographs derived by the RORB runoff-routing model. The primary input is the hydrograph in the Woronora River at the Needles, designated $Q_{U/S}$ in **Figure 6**. This hydrograph represents the runoff from sub-areas A to S in the RORB model. The remaining inputs represent the runoff from each of sub-areas T to Z.

The effect of Georges River flows is represented as a downstream boundary in the form of a stage hydrograph.

6.1.2 Woronora Bridge

The design plans of the Woronora Bridge crossing of the river at Menai Road were reviewed to determine the appropriate representation of the bridge in the model. The plans of the previous bridge which was in use until 1981 indicate that the soffit level of that bridge was above the highest known flood level, as recorded in 1898. The plans indicate a bridge soffit level of about RL 3.3 m AHD, compared with a 1898 flood level of RL 3.04 m AHD. Hence this bridge would have caused negligible afflux in the historical flood events.

The plans of the existing bridge, opened in 1981, indicate that this structure consists of five spans, with a total bridge length of 98.9 m. The soffit level ranges from RL 2.3 m AHD at the west abutment to RL 2.8 m AHD at the east (Sutherland) abutment. The bridge can be considered as a "perched" structure because overland flow around the west abutment will occur before the bridge waterway area is fully utilised. This overland flow is estimated to commence when water levels upstream of the bridge reach about RL 2.0 m AHD.

The existing bridge will cause a very small afflux in minor flood events in which the water level does not reach the bridge soffit level; eg the 1988 flood. This afflux is due to the small encroachment of the abutments into the river channel and the effect of the bridge piers. The encroachment is represented in model cross-section no. 15 and the effect of the bridge piers will not be significant.

For major events where the water level may reach the bridge soffit, the afflux will increase. Because of the major overland flow path, the overall flow conditions will be governed by friction effects, as well as by pressure flow through the bridge opening. Hence it was concluded that the effect of the bridge could be reasonably represented by adjustment of the resistance parameter (Manning n value) at cross-section no. 15.

To determine the appropriate Manning n value for design flood events, the HEC-2 program developed by the US Army Corps of Engineers was used for detailed bridge afflux computations (Ref. 6). A HEC-2 model was set up to represent the Woronora River in the vicinity of the bridge. The model was used to estimate the effect of the bridge corresponding to the peak water level - discharge conditions determined from initial MIKE-11 model runs. The MIKE-11 model was then re-run with the Manning n value at cross-section no. 15 increased so as to reproduce the calculated afflux. The afflux estimates were also checked using the procedure outlined in the NAASRA Technical Report "Bridge Waterways - Hydrology and Design"(Ref.7).

6.2 TIDAL CALIBRATION

The MIKE-11 model of the Woronora River was first calibrated for tidal conditions using records obtained on 14th November 1985. Tide recorders were installed at three locations along the river and tide levels were measured from 0800 hours to 1800 hours. In addition, water levels and velocities at the upstream side of the Woronora Bridge were measured over this period. The locations of the recorders are indicated in **Appendix C**. The results of the measurements are presented in a report by the Public Works Department (Ref.8).

The upstream boundary conditions for the 14th November 1985 event were modelled as zero discharge; there was negligible flow over the Woronora Dam spillway and no precipitation occurred in the catchment on that day or the previous day.

The downstream boundary condition, which is the Georges River stage hydrograph at the Woronora River mouth, was estimated from the Fort Denison tide records on that day. The Maritime Services Board indicated that tide levels in Botany Bay are similar to the levels in Fort Denison. A relationship between levels in Botany Bay and levels at the Woronora River mouth was developed using information from various reports (Refs.9,10 and 11) and involved amplification of Botany Bay levels by 4.7 percent, with a lag of 30 minutes.

With this relationship, the downstream boundary condition was determined by applying an amplification factor of 4.7 percent and a lag of 30 minutes to the Fort Denison tide levels on 14th November 1985.

The tidal modelling involved adjustment of Manning n values to achieve good agreement with recorded levels at the locations of the three recorders along the Woronora River (corresponding to model cross-section nos. 23, 17 and 7) and at the Woronora Bridge (cross-section no. 15).

A Manning n value of 0.035 gave good agreement with recorded levels, at the three tide recorder locations, as well as good agreement with the recorded levels and discharges at Woronora Bridge. The comparison between modelled and recorded behaviour for the tidal event is presented in **Appendix C**.

Manning n values of 0.025 and 0.05 were also tested, but the agreement with recorded data was less satisfactory. Hence n = 0.035 was adopted as a good estimate of in-bank roughness for the Woronora River.

6.3 FLOOD CALIBRATION AND VERIFICATION

The procedure for calibration and verification of the MIKE-11 model for flood behaviour involves two main steps:

- adjustment of Manning n values to achieve good agreement between modelled and recorded water levels for significant historical flood events (calibration)
- modelling of other flood events without changing the model representation or parameter values (verification)

As discussed in **Section 3.6** ,the historical events considered to be of value for modelling, based on the flood magnitude and the quantity and quality of available data, are the 1933, 1956, 1961, 1969, 1974 and 1988 events.

The upstream boundary conditions for these events -ie. the catchment runoff hydrographs - were derived from hydrologic modelling, as described in **Section 5**. The peak discharges from each sub-area for the events are given in **Table 7**. The major inflow to the study area is from the Woronora River at the Needles. The modelled hydrographs at this location are presented in **Appendix B**.

TABLE 7 - PEAK INFLOWS TO HYDRAULIC MODEL FOR HISTORICAL FLOOD EVENTS

Model Inflow	Peak Discharge (m^3/s) for Event in Year 1933	1956	1961	1969	1974	1988
U/S	835	576	549	360	448	539
T	24	21	12	38	26	28
U	16	14	8	24	17	18
V	81	73	37	134	67	90
W	18	16	8	32	14	19
X	35	28	14	50	32	34
Y	38	30	17	52	38	39
Z	23	17	9	31	23	21

Note: Model inflow corresponds to sub-area in RORB model.
U/S inflow at the Needles is from sub-areas A to S.

With regard to the downstream boundary conditions, there were no recorded flood level data at the Woronora River - Georges River confluence for these events. The adopted downstream levels were based on the Fort Denison tidal records (taken as indicative of Botany Bay levels), with addition of from 0.2 m to 0.4 m to account for possible coincident floods in the Georges River. This addition was based on advice provided by the PWD regarding design floods in the Georges River.

The MIKE-11 model representation is based on cross-sections surveyed in 1984/85 and following the extensive river dredging which is reported to have occurred in the 1968 to 1976 period (Ref. 12). The 1988 event is the only significant historical flood which occurred under river conditions similar to those represented in the model. The earlier events occurred before or during the stated dredging period, and there is some uncertainty regarding the actual river conditions for these events.

Hence the 1988 event was identified as the only reliable event for model calibration. The earlier floods were treated as verification events for which it was necessary to modify the cross-section topography to represent pre-existing conditions.

6.3.1 Calibration Event

The 1988 event was modelled with the RORB hydrographs as streamflow inputs. There were no water level records at the Woronora River -Georges River confluence to define the downstream boundary condition for the model. The stage hydrograph at this location was estimated by adding 0.2 m to the Fort Denison tidal records to allow for a flood rise in the Georges River.

The preliminary Manning n values for the model were n = 0.035 for the river channel, determined from the in-bank tidal calibration, and n = 0.05 to 0.08 for overbank areas. The higher value was used in overbank areas where there is urban development.

The effect of different roughness values was also tested by running the model with:

. lower n values (channel n = 0.030)
. higher n values (channel n = 0.040)

The modelled flood profiles for the three cases are shown in **Figure 7**. The flood profile for n = 0.035 provides good overall agreement with recorded flood levels. The modelled levels are within 0.2 m of recorded level at all locations. The flood profile for n = 0.030 is below all recorded levels, and the profile for n = 0.040 is significantly higher than the recorded levels, except at one location.

6.3.2 Verification Events

The 1933, 1956, 1961, 1969 and 1974 events were also modelled using MIKE-11. The downstream boundary condition was derived by adding 0.4 m to the Fort Denison tidal records. Sensitivity testing indicated that the model results at the locations with flood level records were not greatly affected by the downstream boundary condition. For example, a water level change of 100 mm at the downstream boundary resulted in a level change of about 30 mm at a distance five kilometres upstream.

The cross-section topography was modified for these events which occurred before or during the dredging period. The river bed profile as surveyed in 1984/85 provides a general indication of the extent of dredging (see **Figure A.3**). An estimate of the changes in bed level due to dredging was obtained from a previous report which assessed channel changes in the Woronora River (Ref.12). An indicative estimate of RL -3 m AHD for the pre-dredged bed level was derived from this report. The model was modified to incorporate this level as the minimum bed level between cross-section nos. 7 and 28.

The events were modelled initially with n = 0.035 for the river channel and n = 0.05 to 0.08 for overbank areas. There is considerable scatter in the recorded flood levels for most events, but the results indicated that the modelled flood levels are generally low relative to recorded levels. The events were modelled with higher n values, namely n = 0.040 for the river channel, and similar proportional increases for overbank areas. The resulting flood profiles indicated an improved fit, for all events.

The modelled flood profiles for the 1933, 1956, 1961, 1969 and 1974 events are presented in **Appendix C**. The results provide a general confirmation of the capability of the MIKE-11 model to simulate observed flood behaviour. There are some uncertainties associated with these events, particularly with regard to river topography and the reliability of the flood level records. Hence these events should have less weighting than the 1988 event for selection of Manning n values for existing conditions.

The results for the 1988 event were given primary weight in adoption of n values for the calibrated model. This event has the best definition of catchment rainfall and therefore the most reliable estimates of catchment runoff. The model representation of river topography is also most reliable for this event and the recorded flood levels are reasonably consistent.

Hence n values of 0.035 for the river channel and 0.05 to 0.08 for overbank areas were adopted as the calibration values for the MIKE-11 model. The effect of higher n values on design flood levels was considered as part of sensitivity analyses.

6.4 DESIGN EVENTS

6.4.1 Base Case

The 1%, 2% and 5% AEP events were modelled in MIKE-11 for design conditions as follows:

- Inflow hydrographs as derived from the RORB model for the critical storm duration of 36 hours, with Woronora Dam full at the start of the events. The peak discharges at each model inflow are listed in **Table 8.**

- Downstream boundary condition based on a 36 hour design storm occurring over the entire Georges River catchment, including the Woronora catchment, discharging into a mean high water spring level in Botany Bay. The Georges River stage hydrographs at the Woronora River mouth were derived by the PWD and are shown in **Figure 8.**

- River channel topography as determined from the 1984/85 survey. The 1990 check survey by Sutherland Shire Council indicated that there have not been significant changes in the river in the last 5 years.

- Manning n values of 0.035 for the river channel and 0.05 to 0.08 for overbank areas, as determined from the model calibration.

TABLE 8 - PEAK INFLOWS TO HYDRAULIC MODEL FOR DESIGN FLOOD EVENTS

Model Inflow	Peak Discharge (m^3/s) for Event with AEP: 1%	2%	5%
U/S	1282	1124	985
T	40	34	31
U	25	22	20
V	129	115	104
W	28	24	22
X	51	46	41
Y	58	50	44
Z	32	29	25

Note: Model inflow corresponds to sub-area in RORB model.
U/S inflow at the Needles is from sub-areas A to S.

The computed flood levels at each cross-section are given in **Table 9**, and the results are presented as design flood profiles in **Figure 9**.

In the lower reaches of the Woronora River, the flood levels resulting from a design storm tide event in Botany Bay will be higher than the levels resulting from a design rainfall-runoff event in both the Woronora and Georges River catchments. No detailed investigations on storm tide conditions in Botany Bay are available. In the absence of such investigations, the PWD provided the following preliminary estimates.

1% AEP event	1.7 m AHD
2% AEP event	1.6 m AHD
5% AEP event	1.5 m AHD

These levels are the appropriate design flood levels for cross-section nos. 1, 1A and 2 in the Woronora River.

TABLE 9 - DESIGN FLOOD LEVELS - WORONORA RIVER

Cross-Section No.	River Chainage (km)	MIKE-11 Chainage (km)	Peak Flood Level (m AHD) for Event with AEP: 1%	2%	5%
1	0.33	10.31	1.7*	1.6*	1.5*
1A	0.66	9.98	1.7*	1.6*	1.5*
2	0.97	9.67	1.7*	1.6*	1.5*
3	1.37	9.27	1.9	1.7	1.5
3A	1.65	8.99	2.1	1.9	1.7
5	1.97	8.67	2.3	2.0	1.8
6	2.43	8.21	2.6	2.3	2.1
7	2.71	7.93	2.6	2.4	2.1
8	2.99	7.65	2.8	2.6	2.3
9	3.19	7.45	2.8	2.6	2.3
10	3.39	7.25	2.9	2.6	2.4
11	3.62	7.02	3.0	2.7	2.4
12	3.85	6.79	3.0	2.8	2.5
13	4.03	6.61	3.1	2.8	2.5
14	4.24	6.40	3.1	2.8	2.6
15 D/S	4.45	6.19	3.2	2.9	2.6
U/S	4.47	6.17	3.4	3.1	2.8
16	4.64	6.00	3.5	3.2	2.9
17	4.85	5.79	3.6	3.3	3.0
18	5.07	5.57	3.6	3.3	3.0
19	5.31	5.33	3.6	3.3	3.0
20	5.49	5.15	3.7	3.4	3.1
21	5.69	4.95	3.8	3.5	3.2
22	5.90	4.74	3.9	3.6	3.3
23	6.10	4.54	4.0	3.7	3.4
24	6.30	4.34	4.1	3.7	3.4
25	6.52	4.12	4.2	3.9	3.6
25A	6.65	3.99	4.3	4.0	3.6
27	6.94	3.70	4.4	4.1	3.7
28	7.14	3.50	4.6	4.3	3.9
29	7.38	3.26	4.6	4.3	3.9
30	7.60	3.04	4.7	4.4	4.0
31	7.80	2.84	4.8	4.5	4.1
32	8.00	2.64	5.0	4.7	4.3
33	8.18	2.46	5.1	4.7	4.4
34	8.38	2.26	5.1	4.7	4.4
35	8.62	2.02	5.5	5.1	4.7
36	8.80	1.84	5.5	5.1	4.8
37	9.01	1.63	5.7	5.3	4.9
38	9.20	1.44	5.7	5.3	4.9
39	9.42	1.22	5.8	5.4	5.0
40	9.61	1.03	5.9	5.5	5.1
41	9.79	0.85	6.0	5.6	5.2
42	9.90	0.65	6.1	5.7	5.3
43	10.20	0.44	6.1	5.7	5.3
44	10.42	0.22	6.5	6.1	5.6
45	10.64	0.00	6.7	6.3	5.8

Notes: River chainage is measured upstream from Como Railway Bridge.
MIKE-11 model chainage is measured downstream from cross-section No 45.
Woronora Bridge is at river chainage 4.46 km.
Forbes Creek confluence is at river chainage 5.80 km.
* Flood levels due to storm tide conditions in Botany Bay.

6.4.2 Sensitivity Analyses

Sensitivity analyses were carried out to determine the effect on the 1% AEP flood levels of various changes, including:

Increased Roughness
The Manning n values were increased to 0.04 for the river channel and 0.06 to 0.09 for overbank areas.

Higher Georges River Flood Levels
The downstream boundary condition was modified to represent an elevated tailwater, comprising the 1% peak discharge in the Woronora River coincident with the peak of a 1% storm tide event in Botany Bay. A preliminary estimate of the downstream stage hydrograph for this case was supplied by the PWD and has a maximum water level of 1.7 m AHD. Although the inclusion of the 1% Georges River flood would elevate the tailwater by a further 0.3 m, the joint probability of all three occurrences would be significantly less than 1%.

Higher Woronora River Bed Levels
The minimum bed level between cross-section nos. 7 and 28 was raised to RL -3 m AHD, similar to the levels used in modelling of the 1933 to 1974 flood events (Section 6.3.2). This case represents an approximation to the pre-dredged conditions of the Woronora River.

The modelled flood levels for these cases are given in **Table 10**. The modelled flood profiles are compared with the profile for the base case in **Figure 10**. The flood level increases, compared to the base case for the 1% AEP event, would be up to:

- 0.56 m for increased roughness, with the greatest increase in the upper reaches
- 0.35 m for higher Georges River flood levels, with the greatest increase in the lower reaches
- 0.34 m for higher Woronora River bed levels, with the greatest increase in the middle reaches.

With the base case results shown in **Table 9**, the 1% AEP flood level at Woronora Bridge is 0.37 m higher than the 1898 recorded flood level at this location. Also the 5% AEP flood profile corresponds approximately to the 1933 recorded flood levels. Therefore the flood levels derived for the statistical design events are reasonably consistent with the historical flood levels.

The flood levels given in **Table 9** are considered to be the best estimates of design flood levels for the study area. The higher flood levels resulting from the sensitivity analyses, as given in **Table 10**, provide some guidance on the influence of other factors.

6.4.3 Flood Contours and Velocities

Figures 11, 12 and **13** show the flood contours along the Woronora River for the 1%, 2% and 5% design events. The average velocities are also indicated for selected locations.

The flow distributions and average velocities at the time of maximum flood levels are given for all cross-sections in **Tables 11, 12** and **13**. The average velocities at the time of peak discharge are similar to the velocities at peak water level, except near the Georges River confluence where higher velocities will occur.

It should be noted that the velocities represent average velocities across each segment of the flow. Localised velocities at particular sites may vary from the average velocity and may be influenced by factors such as constrictions, obstructions and proximity to the main river flow path.

TABLE 10 - SENSITIVITY ANALYSES OF DESIGN FLOOD LEVELS

Cross-Section No		River Chainage (km)	MIKE-11 Chainage (km)	Peak Flood Level (m AHD) for 1% AEP Event Increased Roughness	Higher Georges River Levels	Higher River Bed Levels
1		0.33	10.31	1.7*	1.7	1.7*
2		0.66	9.98	1.7*	1.8	1.7*
2		0.97	9.67	1.7*	2.0	1.7*
3		1.37	9.27	2.1	2.2	1.9
3A		1.65	8.99	2.3	2.4	2.1
5		1.97	8.67	2.5	2.5	2.3
6		2.43	8.21	2.8	2.8	2.6
7		2.71	7.93	2.9	2.8	2.7
8		2.99	7.65	3.1	3.0	2.9
9		3.19	7.45	3.1	3.0	2.9
10		3.39	7.25	3.1	3.0	3.0
11		3.62	7.02	3.3	3.1	3.1
12		3.85	6.79	3.3	3.2	3.2
13		4.03	6.61	3.3	3.2	3.2
14		4.24	6.40	3.4	3.2	3.3
15	D/S	4.45	6.19	3.4	3.3	3.3
	U/S	4.47	6.17	3.6	3.5	3.6
16		4.64	6.00	3.8	3.6	3.8
17		4.85	5.79	3.8	3.7	3.8
18		5.07	5.57	3.8	3.7	3.8
19		5.31	5.33	3.9	3.7	3.9
20		5.49	5.15	4.0	3.8	4.1
21		5.69	4.95	4.1	3.9	4.2
22		5.90	4.74	4.2	4.0	4.2
23		6.10	4.54	4.3	4.1	4.3
24		6.30	4.34	4.4	4.1	4.4
25		6.52	4.12	4.5	4.3	4.5
25A		6.65	3.99	4.6	4.4	4.6
27		6.94	3.70	4.8	4.5	4.7
28		7.14	3.50	5.0	4.7	4.9
29		7.38	3.26	5.0	4.7	4.9
30		7.60	3.04	5.1	4.8	5.0
31		7.80	2.84	5.2	4.9	5.1
32		8.00	2.64	5.4	5.1	5.3
33		8.18	2.46	5.5	5.1	5.3
34		8.38	2.26	5.5	5.1	5.3
35		8.62	2.02	5.9	5.5	5.7
36		8.80	1.84	5.9	5.5	5.7
37		9.01	1.63	6.1	5.7	5.9
38		9.20	1.44	6.1	5.7	5.8
39		9.42	1.22	6.2	5.8	5.9
40		9.61	1.03	6.4	5.9	6.1
41		9.79	0.85	6.5	6.0	6.1
42		9.90	0.65	6.6	6.1	6.2
43		10.20	0.44	6.7	6.2	6.3
44		10.42	0.22	7.0	6.5	6.6
45		10.64	0.00	7.3	6.8	6.8

Notes: River chainage is measured upstream from Como Railway Bridge.
MIKE-11 model chainage is measured downstream from cross-section No 45.
Woronora Bridge is at river chainage 4.46 km.
Forbes Creek confluence is at river chainage 5.80 km.
* Flood levels due to storm tide conditions in Botany Bay.

TABLE 11 - FLOW DISTRIBUTION AND AVERAGE VELOCITIES FOR 1% AEP EVENT

Cross-Section No.	Flow Distribution (%) Left O/Bank	Flow Distribution (%) Channel	Flow Distribution (%) Right O/Bank	Average Velocity (m/s) Left O/Bank	Average Velocity (m/s) Channel	Average Velocity (m/s) Right O/Bank
1	0	100	0	0	0.7 (1.8)	0
1A	0	88	12	0	1.0 (2.5)	0.4 (0.8)
2	0	91	9	0	1.9	0.9
3	0	100	0	0	1.6	0
3A	27	73	0	1.0	2.1	0
5	0	98	2	0.2	1.9	0.8
6	0	100	0	0	1.4	0
7	0	98	2	0.2	2.4	0.3
8	0	99	1	0	1.1	0.2
9	0	100	0	0.5	1.6	0
10	8	92	0	0.7	1.8	0.5
11	2	96	2	0.4	1.2	0.3
12	0	76	24	0.2	1.4	0.6
13	0	78	22	0	1.3	0.7
14	0	98	2	0.2	1.5	0.3
15	12	88	0	0.9	2.4	0
16	5	94	1	0.3	1.6	0.2
17	0	97	3	0.2	1.4	0.2
18	5	95	0	0.3	1.9	0.6
19	5	94	1	0.4	2.8	0.9
20	1	98	1	0.2	1.8	0.5
21	2	82	16	0.4	2.1	0.9
22	0	90	10	0	2.4	0.5
23	0	96	4	0	2.0	0.5
24	12	88	0	1.0	2.4	0
25	1	96	3	0	2.2	0.6
25A	0	87	13	0.7	2.7	0.8
27	6	93	1	0.6	2.7	0.9
28	0	68	32	0	2.0	0.5
29	1	99	0	0.7	2.3	0.4
30	0	95	5	0	2.6	0.7
31	0	96	4	0.1	2.7	0.9
32	8	91	1	1.2	2.4	0.6
33	2	98	0	1.2	2.5	0.6
34	0	100	0	0.1	3.2	0.9
35	0	96	4	0.5	2.3	0.9
36	1	99	0	0.8	2.4	0
37	1	99	0	0.5	2.0	0.5
38	0	92	8	0.1	2.8	1.1
39	1	99	0	0.9	2.8	0.2
40	0	99	1	0.5	2.8	0.7
41	0	99	1	0	2.9	0.9
42	0	98	2	0	3.2	1.2
43	2	94	4	1.4	4.2	1.6
44	1	99	0	1.2	3.3	0.7
45	3	80	17	1.7	4.5	2.0

Notes: Flow distributions are rounded to nearest one percent. Overbank flows of less than 0.5 percent are shown as zero.
Average velocities apply at time of maximum water level.
Average velocities at time of peak discharge are similar, except at cross-section Nos 1 and 14. The velocities at these cross-sections at peak discharge are indicated in brackets ( ).

TABLE 12 - FLOW DISTRIBUTION AND AVERAGE VELOCITIES FOR 2% AEP EVENT

Cross-Section No.	Flow Distribution (%) Left O/Bank	Flow Distribution (%) Channel	Flow Distribution (%) Right O/Bank	Average Velocity (m/s) Left O/Bank	Average Velocity (m/s) Channel	Average Velocity (m/s) Right O/Bank
1	0	100	0	0	0.6 (1.7)	0
1A	0	89	11	0	1.0 (2.4)	0.3 (0.7)
2	0	92	8	0	1.8	0.8
3	0	100	0	0	1.5	0
3A	26	74	0	0.9	2.0	0
5	0	98	2	0.1	1.8	0.7
6	0	100	0	0	1.3	0
7	0	99	1	0	2.3	0.2
8	0	99	1	0	1.0	0.2
9	0	100	0	0.5	1.5	0
10	7	93	0	0.6	1.7	0.5
11	1	97	2	0.3	1.1	0.3
12	0	77	23	0.1	1.3	0.6
13	0	79	21	0	1.2	0.7
14	0	98	2	0.1	1.4	0.3
15	8	92	0	0.7	2.1	0
16	3	96	1	0.2	1.5	0.2
17	0	98	2	0.2	1.3	0.2
18	4	96	0	0.3	1.7	0.6
19	3	96	1	0.3	2.6	0.8
20	0	99	1	0.1	1.7	0.4
21	2	84	14	0.3	2.0	0.8
22	0	92	8	0	2.2	0.4
23	0	97	3	0	1.9	0.4
24	11	89	0	0.9	2.3	0
25	0	97	3	0.4	2.1	0.5
25A	0	89	11	0.9	2.5	0.7
27	5	94	1	0.5	2.5	0.8
28	0	71	29	0	1.9	0.5
29	1	99	0	0.6	2.2	0.3
30	0	97	3	0	2.5	0.5
31	0	97	3	0	2.6	0.8
32	7	92	1	1.1	2.3	0.5
33	2	98	0	1.1	2.3	0.5
34	0	100	0	0.1	3.0	0.7
35	0	97	3	0.4	2.2	0.7
36	1	99	0	0.7	2.3	0
37	1	99	0	0.4	1.8	0.5
38	0	93	7	0.1	2.6	1.0
39	1	99	0	0.8	2.6	0
40	0	99	1	0.3	2.6	0.6
41	0	99	1	0	2.7	0.9
42	0	98	2	0	3.0	1.0
43	2	94	4	1.3	3.9	1.5
44	1	99	0	1.0	3.1	0.6
45	3	82	15	1.6	4.4	1.9

Note: Flow distributions are rounded to nearest one percent. Overbank flows of less than 0.5 percent are shown as zero.
Average velocities apply at time of maximum water level.
Average velocities at time of peak discharge are similar, except at cross-section Nos 1 and 1A. The velocities at these cross-sections at peak discharge are indicated in brackets ( ).

TABLE 13 - FLOW DISTRIBUTION AND AVERAGE VELOCITIES FOR 5% AEP EVENT

Cross-Section No.	Flow Distribution (%) Left O/Bank	Flow Distribution (%) Channel	Flow Distribution (%) Right O/Bank	Average Velocity (m/s) Left O/Bank	Average Velocity (m/s) Channel	Average Velocity (m/s) Right O/Bank
1	0	100	0	0	0.5 (1.7)	0
1A	0	91	9	0	0.8 (2.3)	0.3 (0.6)
2	0	93	7	0	1.7	0.8
3	0	100	0	0	1.4	0
3A	24	76	0	0.8	1.9	0
5	0	99	1	0	1.7	0.6
6	0	100	0	0	1.2	0
7	0	100	0	0	2.1	0.1
8	0	99	1	0	1.0	0.1
9	0	100	0	0.3	1.4	0
10	6	94	0	0.6	1.6	0.4
11	1	98	1	0.2	1.0	0.3
12	0	79	21	0	1.2	0.6
13	0	80	20	0	1.2	0.6
14	0	99	1	0	1.4	0.3
15	6	94	0	0.6	1.9	0
16	2	97	1	0.2	1.4	0.2
17	0	99	1	0.2	1.2	0.1
18	3	97	0	0.2	1.6	0.5
19	2	97	1	0.2	2.4	0.7
20	0	100	0	0	1.6	0.3
21	1	86	13	0.3	2.0	0.7
22	0	93	7	0	2.1	0.4
23	0	97	3	0	1.8	0.4
24	10	90	0	0.8	2.1	0
25	0	97	3	0.3	1.9	0.5
25A	0	90	10	0	2.4	0.7
27	4	95	1	0.5	2.4	0.7
28	0	74	26	0	1.9	0.5
29	1	99	0	0.5	2.0	0.1
30	0	98	2	0	2.4	0.4
31	0	98	2	0	2.4	0.6
32	7	93	0	1.0	2.1	0.4
33	2	98	0	1.0	2.2	0.4
34	0	100	0	0	2.8	0.5
35	0	98	2	0.3	2.0	0.6
36	0	100	0	0.6	2.1	0
37	0	100	0	0.3	1.7	0.3
38	0	94	6	0	2.4	0.9
39	1	99	0	0.6	2.4	0
40	0	100	0	0.2	2.4	0.4
41	0	99	1	0	2.5	1.4
42	0	98	2	0	2.8	1.0
43	2	95	3	1.2	3.6	1.4
44	1	99	0	0.9	2.9	0.5
45	2	85	13	1.6	4.2	1.7

Note: Flow distributions are rounded nearest one percent. Overbank flows of less than 0.5 percent are shown as zero.
Average velocities apply at time of maximum water level.
Average velocities at time of peak discharge are similar, except at cross-section Nos 1 and 1A. The velocities at these cross-sections at peak discharge are indicated in brackets ( ).

6.5 EXTREME EVENT

The MIKE-11 model was also run for the extreme rainfall event, as defined in **Section 5.4**. The runoff hydrographs generated using RORB were input to the model as upstream boundary conditions. The downstream boundary condition for this event was supplied by the PWD and involved an extreme flood on the Georges River coinciding with a 1% storm tide in Botany Bay, which resulted in a peak flood level of 3.7 m AHD at the confluence with Woronora River. The peak downstream water level was assumed to be coincident with the peak discharge in the Woronora River.

The modelled peak flood levels for the extreme event are given in **Table 14**. The levels range from 2.0 m higher than the 1% design flood levels at the Georges River to 3.3 m higher at the Needles.

TABLE 14 - EXTREME EVENT FLOOD LEVELS

Cross-Section No.	River Chainage (km)	MIKE-11 Chainage (km)	Peak Flood Level (m AHD) Extreme Event
1	0.33	10.31	3.7
1A	0.66	9.98	3.8
2	0.97	9.67	4.0
3	1.37	9.27	4.2
3A	1.65	8.99	4.4
5	1.97	8.67	4.5
6	2.43	8.21	4.9
7	2.71	7.93	5.0
8	2.99	7.65	5.2
9	3.19	7.45	5.2
10	3.39	7.25	5.2
11	3.62	7.02	5.4
12	3.85	6.79	5.5
13	4.03	6.61	5.5
14	4.24	6.40	5.5
15 D/S	4.45	6.19	5.6
U/S	4.47	6.17	5.9
16	4.64	6.00	6.1
17	4.85	5.79	6.1
18	5.07	5.57	6.1
19	5.31	5.33	6.2
20	5.49	5.15	6.4
21	5.69	4.95	6.4
22	5.90	4.74	6.6
23	6.10	4.54	6.6
24	6.30	4.34	6.6
25	6.52	4.12	6.8
25A	6.65	3.99	6.9
27	6.94	3.70	7.0
28	7.14	3.50	7.4
29	7.38	3.26	7.2
30	7.60	3.04	7.5
31	7.80	2.84	7.5
32	8.00	2.64	7.8
33	8.18	2.46	7.8
34	8.38	2.26	7.6
35	8.62	2.02	8.3
36	8.80	1.84	8.2
37	9.01	1.63	8.6
38	9.20	1.44	8.5
39	9.42	1.22	8.6
40	9.61	1.03	8.8
41	9.79	0.85	8.9
42	9.90	0.65	9.0
43	10.20	0.44	9.1
44	10.42	0.22	9.6
45	10.64	0.00	10.1

Notes: River chainage is measured upstream from Como Railway Bridge.
MIKE-11 model chainage is measured downstream from cross-section No 45.
Woronora Bridge is at river chainage 4.46 km.
Forbes Creek confluence is at river chainage 5.80 km.

7. Acknowledgements

This study was funded by the Sutherland Shire Council and the State Government and was undertaken by Sinclair Knight Consulting Engineers for the Public Works Department.

The study is an extension of earlier work which was carried out by Cameron McNamara Pty Ltd in 1985.

In compiling this report the Consultant has been assisted by advice and information from the Public Works Department (PWD), Sutherland Shire Council, various public authorities and local residents.

8. References

1. Public Works Department of NSW
Woronora River Flood Study - Compendium of Data
August 1991

2. E.M. Laurenson and R.G. Mein
RORB - Version 4 Runoff Routing Program User Manual
Monash University Department of Civil Engineering, May 1988

3. Danish Hydraulics Institute
MIKE-11 Hydraulic Modelling System
December 1989

4. Institution of Engineers, Australia
Australian Rainfall and Runoff - A Guide to Flood Estimation
1987

5. Bureau of Meteorology
The Estimation of Probable Maximum Precipitation in Australia for Short Durations and Small Areas
Bulletin 51, August 1984

6. US Army Corps of Engineers, Hydrologic Engineering Center
HEC-2 Water Surface Profiles Users Manual
1982

7. National Association of Australian State Road Authorities
Bridge Waterways Hydrology and Design
NAASRA Technical Report, January 1989

8. Public Works Department, Manly Hydraulics Laboratory, Data Collection Section
Woronora River: Tidal Measurement to Determine In-Bank Roughness
Report No. 447 (Draft), January 1986

9. K.Higgs, P.D.Treloar, D.N.Foster, B.S.Jenkins and N.V.Lawson
Tidal Hydraulics of Botany Bay
Water Research Laboratory, University of New South Wales Report No. 155, Vol 1, 1980

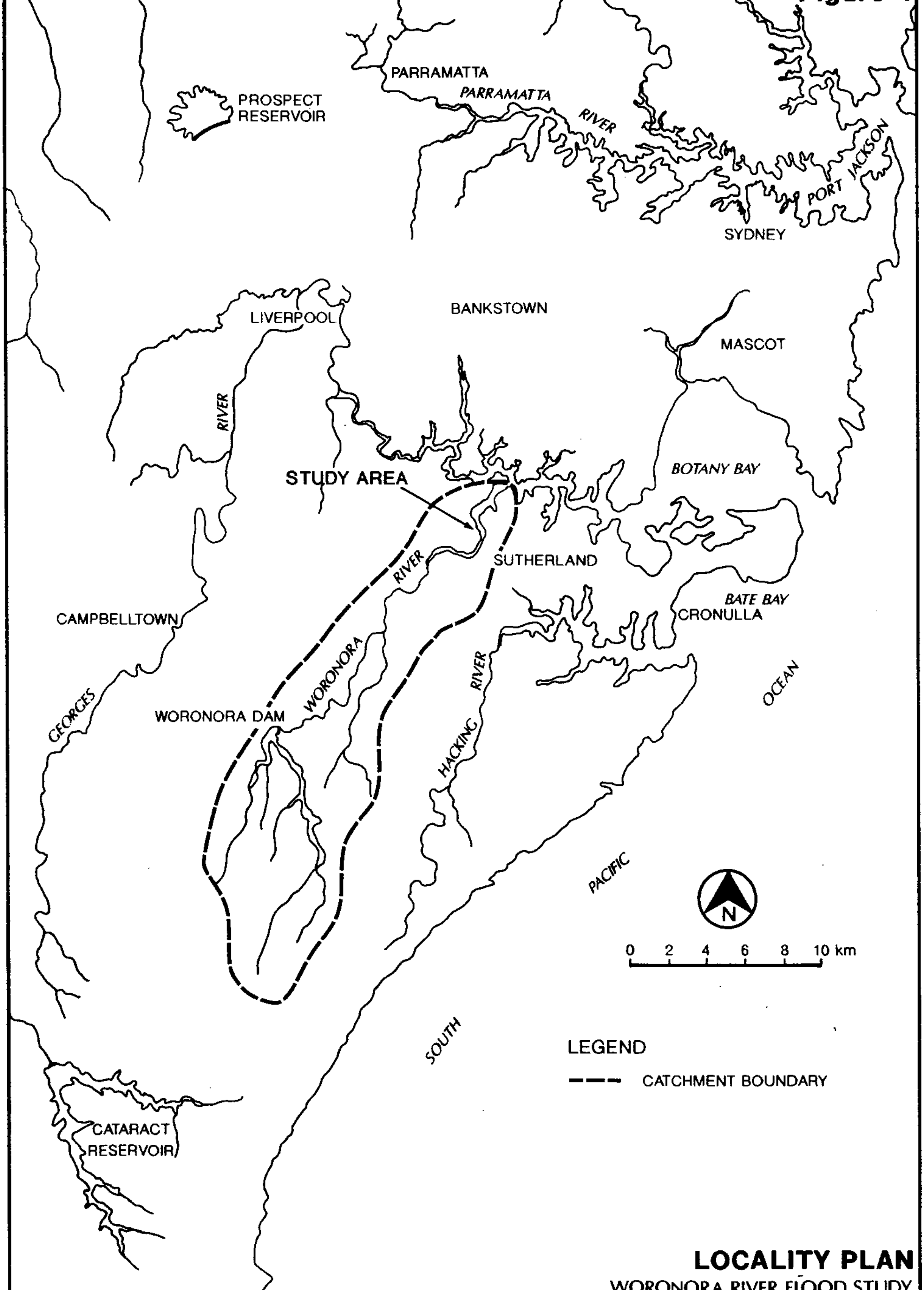
10. P.A. Douglas
Short Term Sediment Movement in the Woronora Estuary
Geology Society of Australia, Abstracts No. 13, 1985

11. Public Works Department of NSW
Georges River - General Plan and Tide Gradients
1961

12. R.F.Warner and G.Pickup
Channel Changes in the Georges River between 1959 and 1973/76 and their Implications
Bankstown Municipal Council, 1978

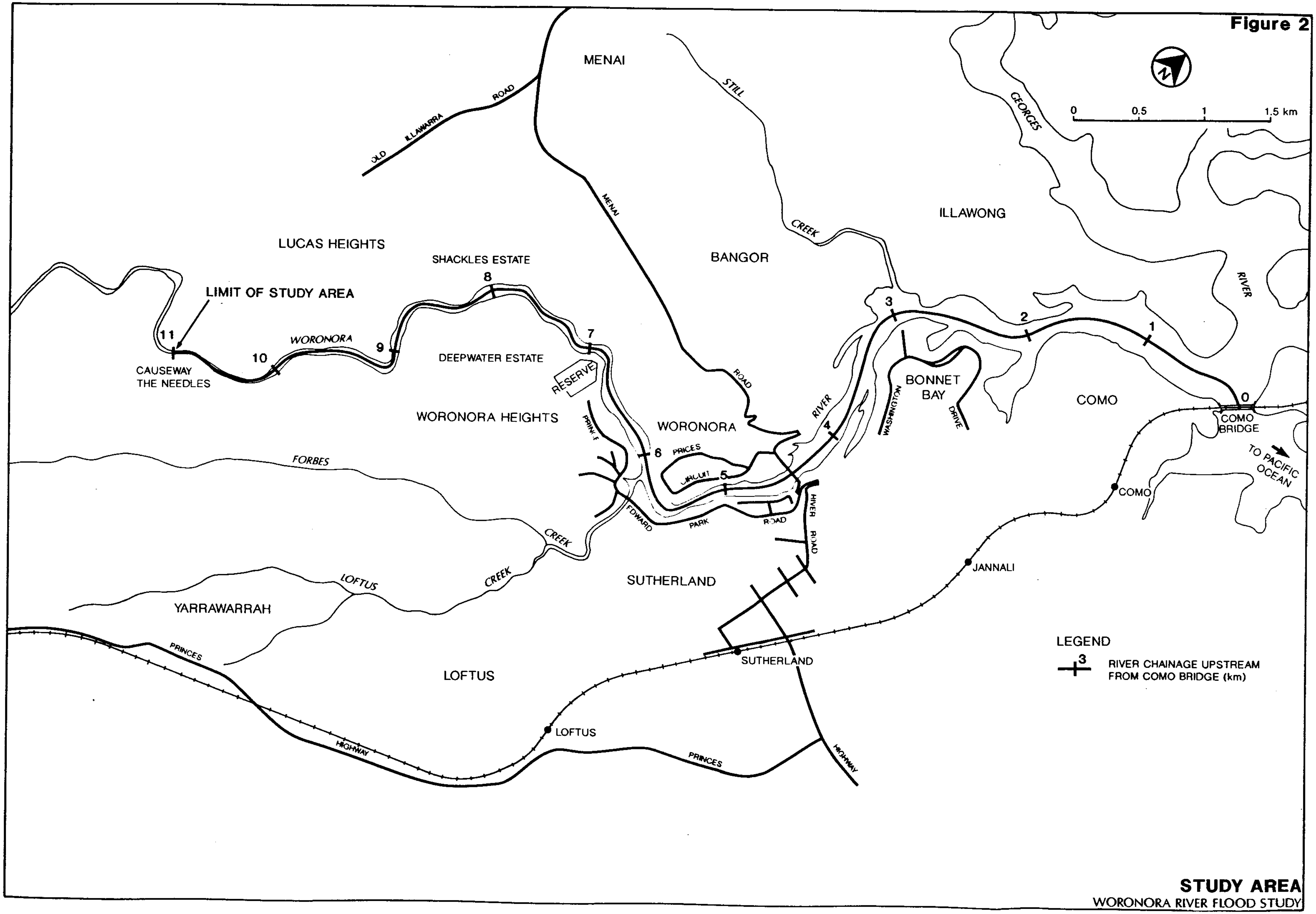
Figures

Figure 1



LOCALITY PLAN

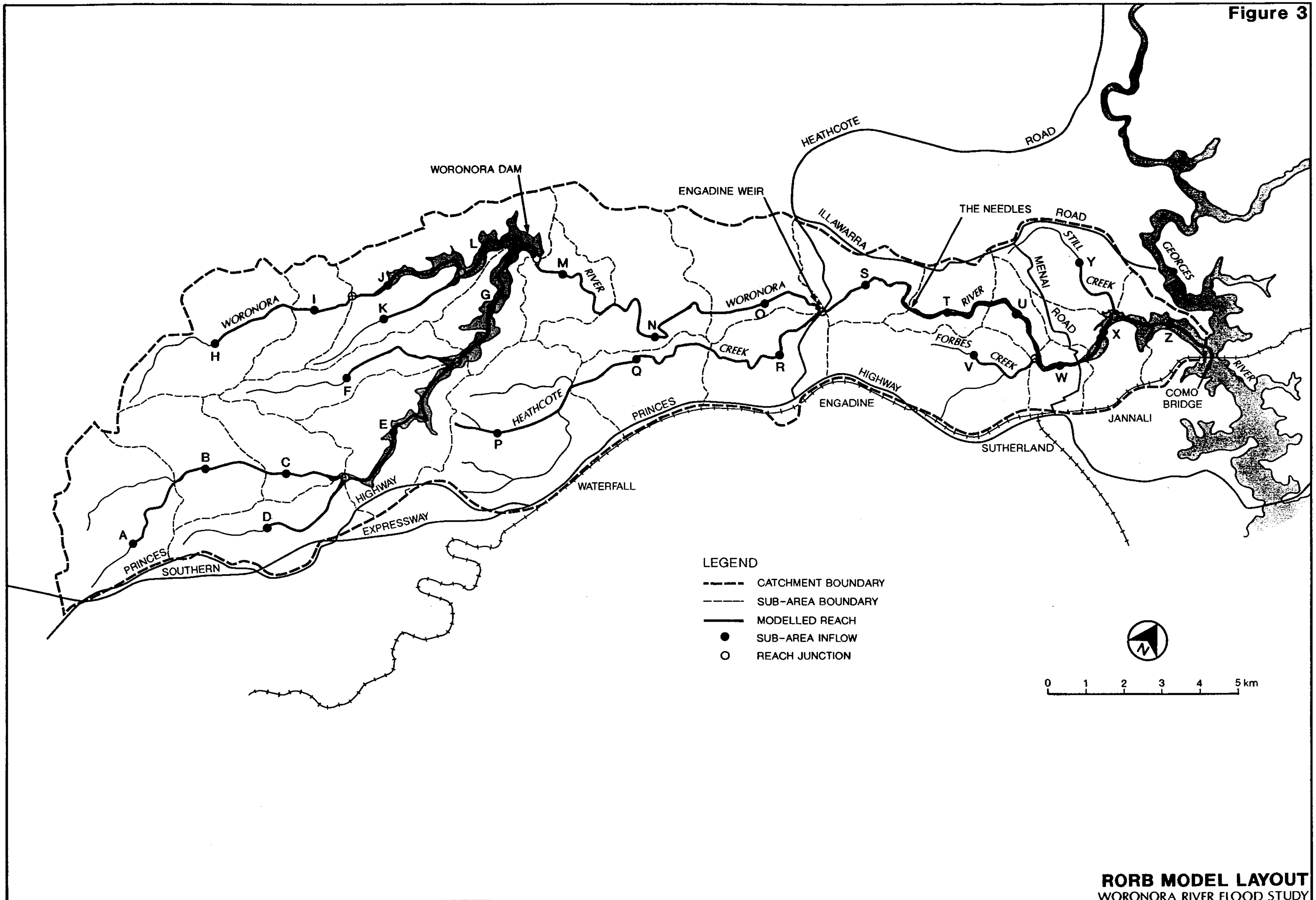
WORONORA RIVER FLOOD STUDY



STUDY AREA

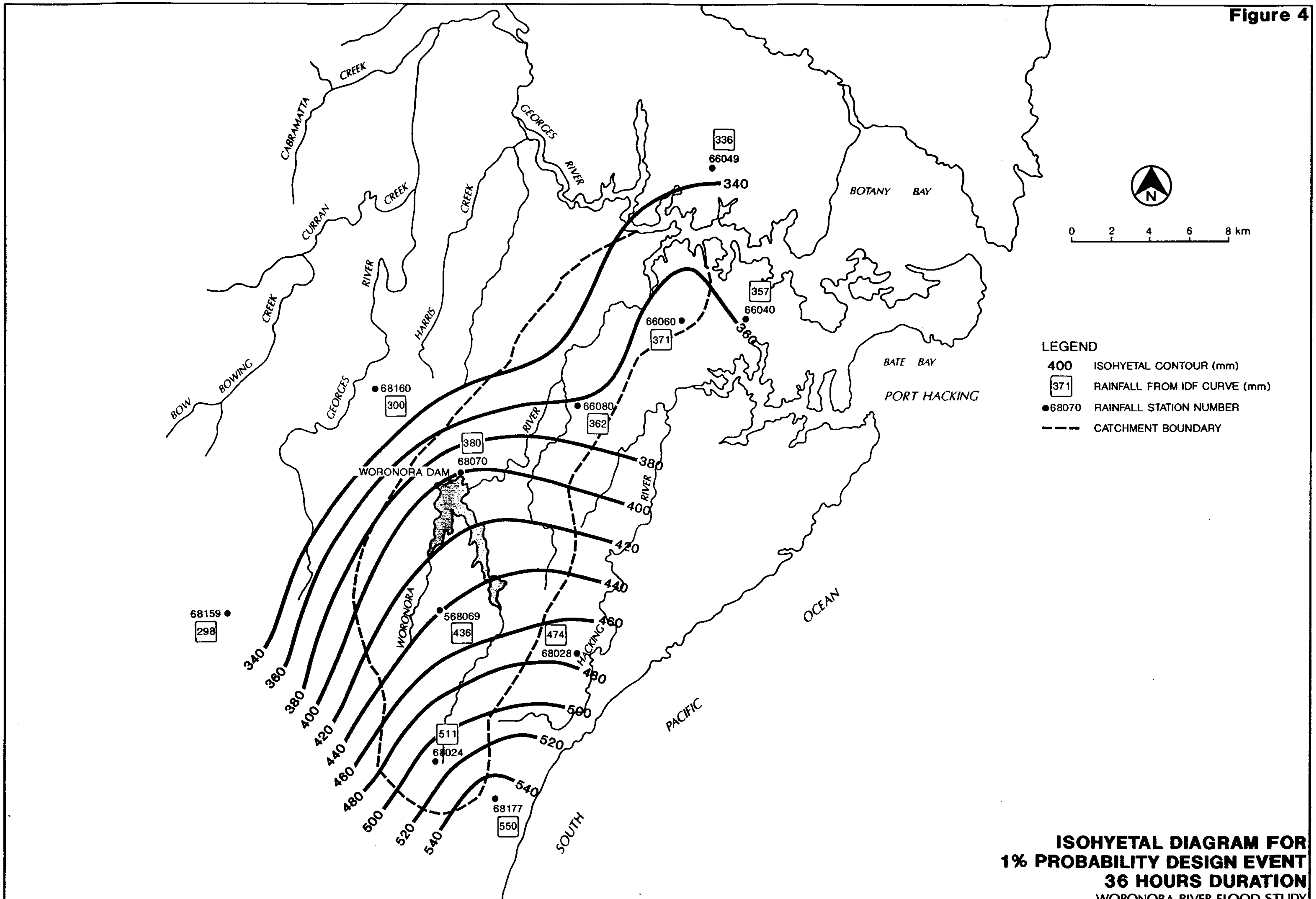
WORONORA RIVER FLOOD STUDY

Figure 3



RORB MODEL LAYOUT
WORONORA RIVER FLOOD STUDY

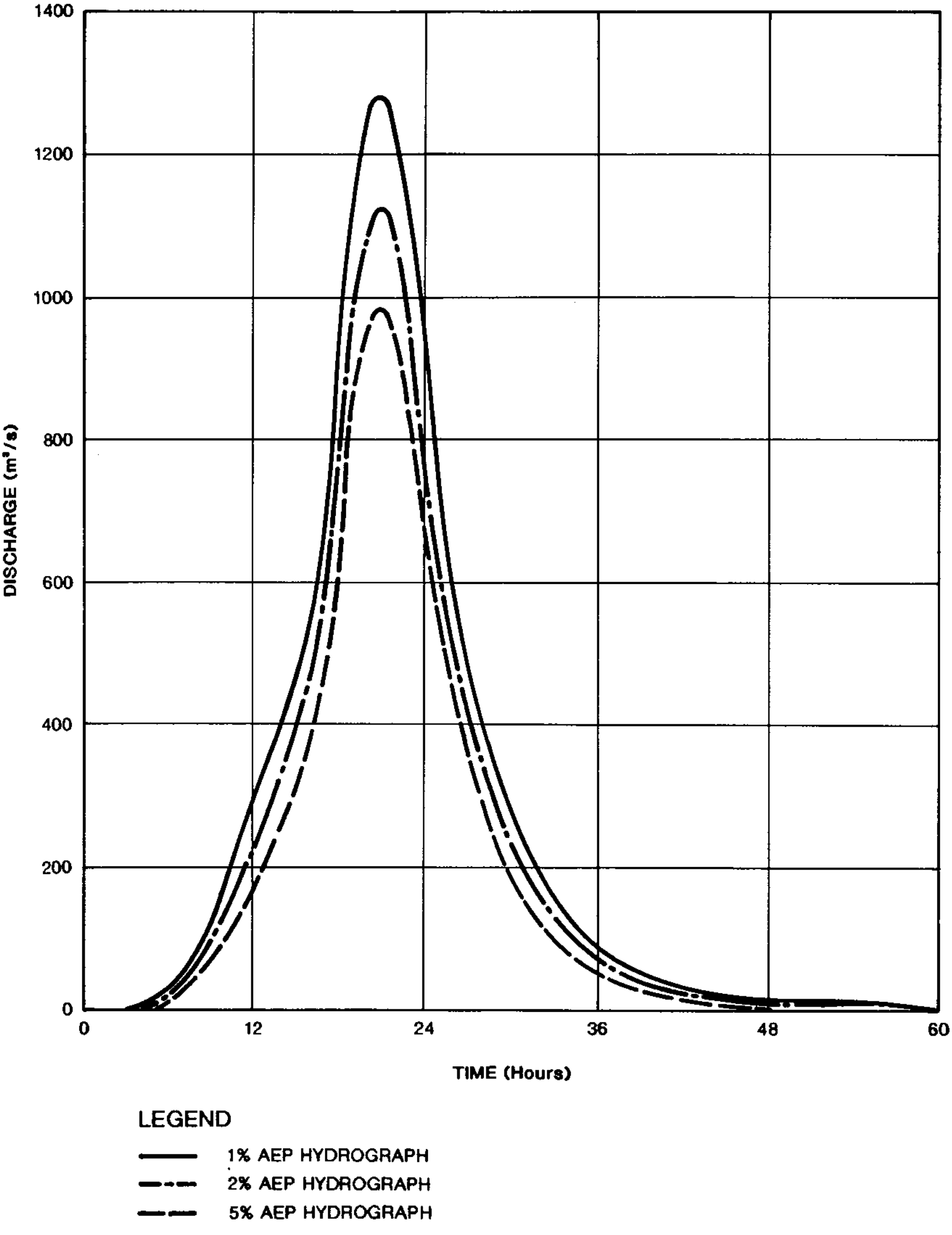
Figure 4



ISOHYETAL DIAGRAM FOR 1% PROBABILITY DESIGN EVENT 36 HOURS DURATION

WORONORA RIVER FLOOD STUDY

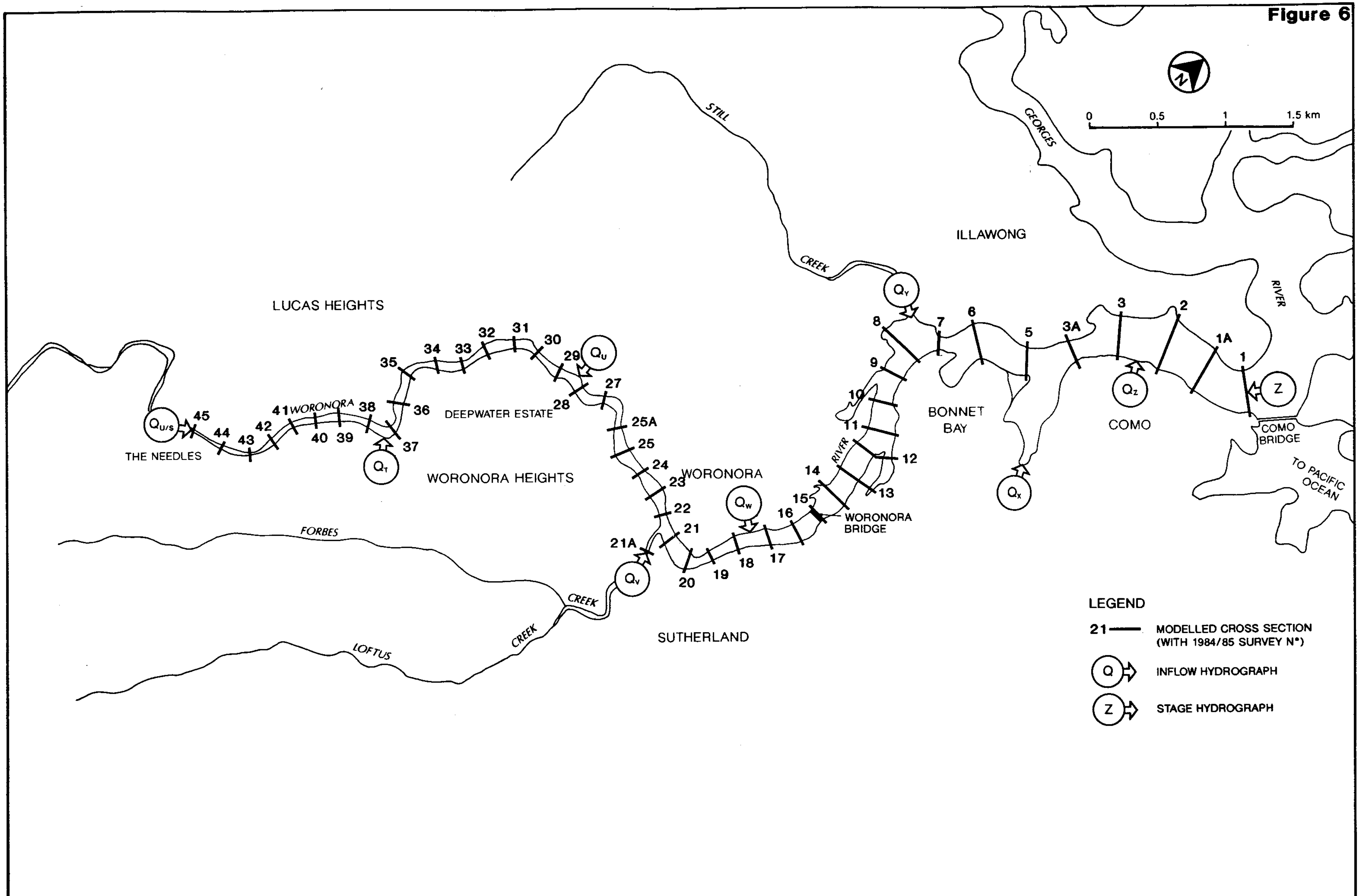
Figure 5



DESIGN HYDROGRAPHS FOR WORONORA RIVER AT THE NEEDLES 36 HOUR STORM DURATION

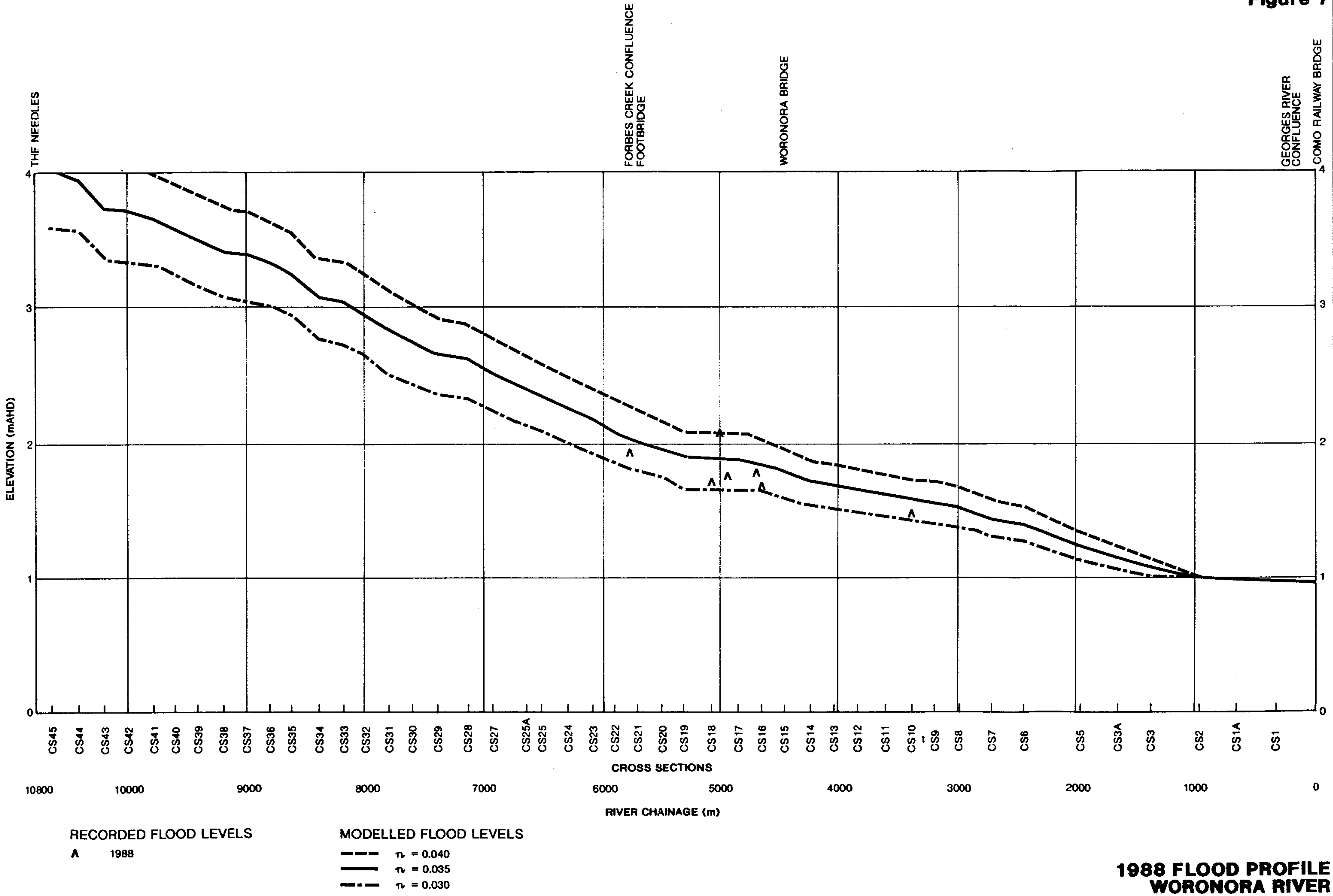
WORONORA RIVER FLOOD STUDY

Figure 6



MIKE 11 MODEL LAYOUT

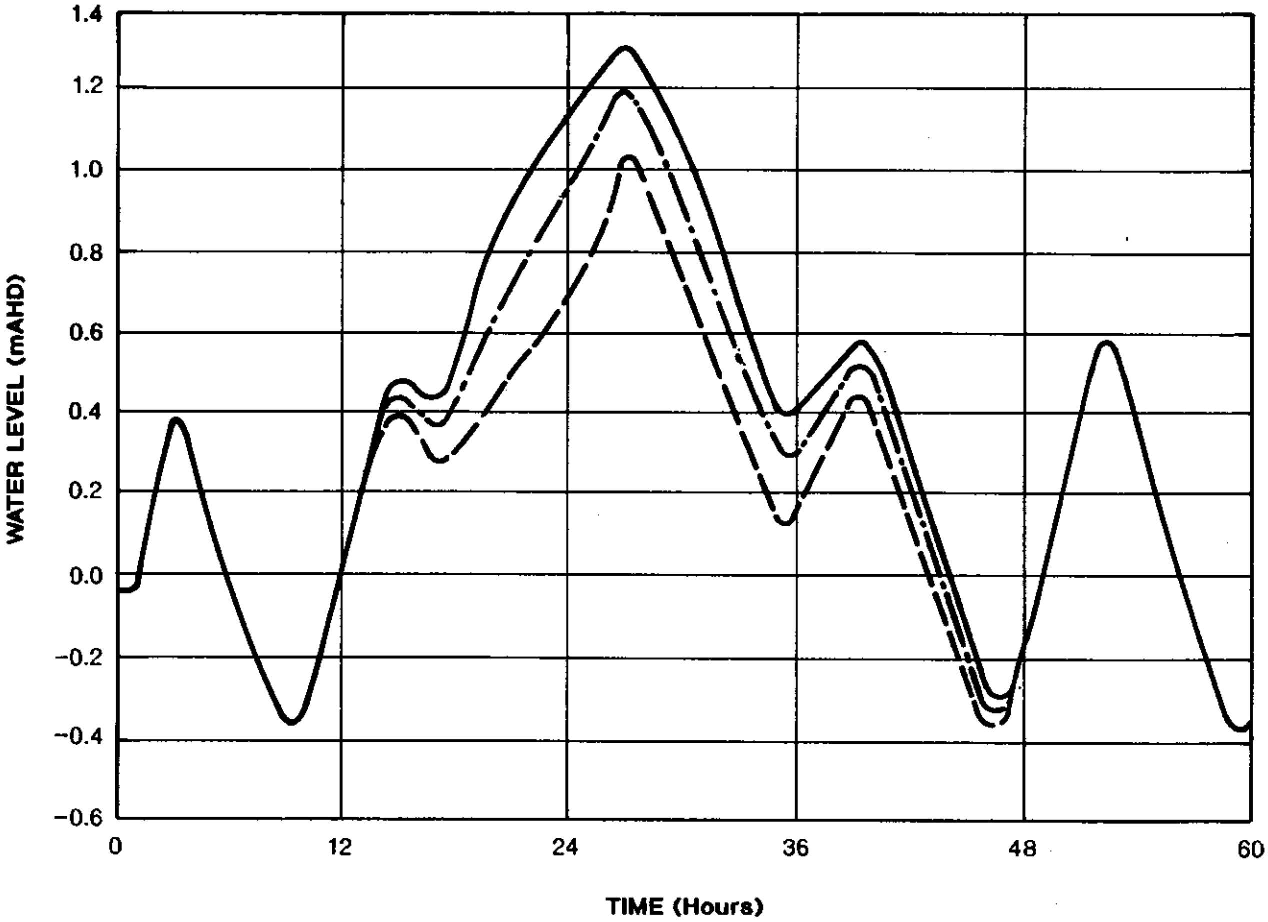
WORONORA RIVER FLOOD STUDY



1988 FLOOD PROFILE
WORONORA RIVER

WORONORA RIVER FLOOD STUDY

Figure 8



LEGEND

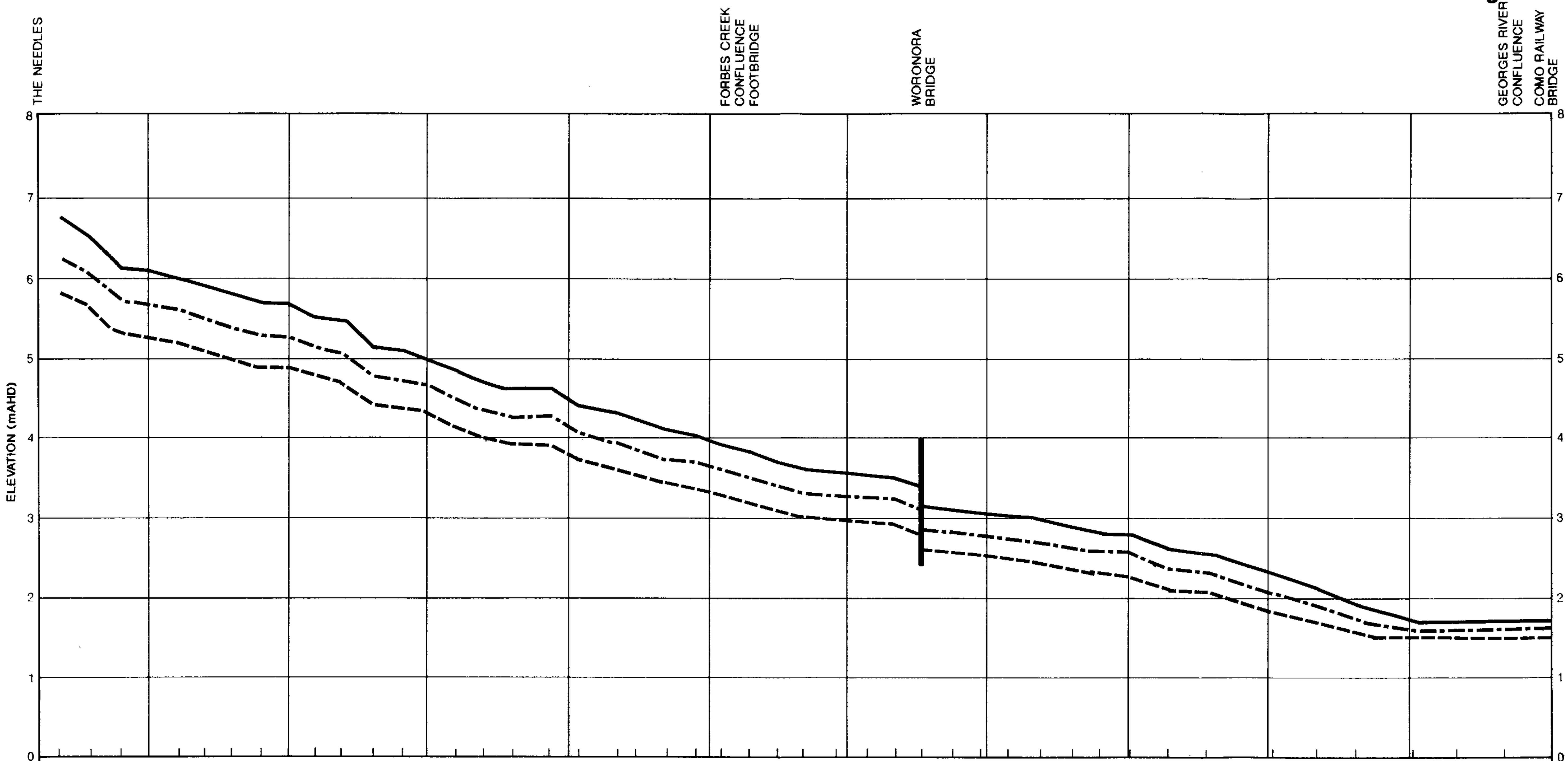
1% AEP HYDROGRAPH
2% AEP HYDROGRAPH
5% AEP HYDROGRAPH

NOTE: HYDROGRAPHS ARE FOR GEORGES RIVER AT CONFLUENCE WITH WORONORA RIVER

SOURCE: PUBLIC WORKS DEPARTMENT (1990)

DESIGN STAGE HYDROGRAPHS FOR GEORGES RIVER 36 HOUR STORM DURATION

Figure 9



FLOOD LEVEL

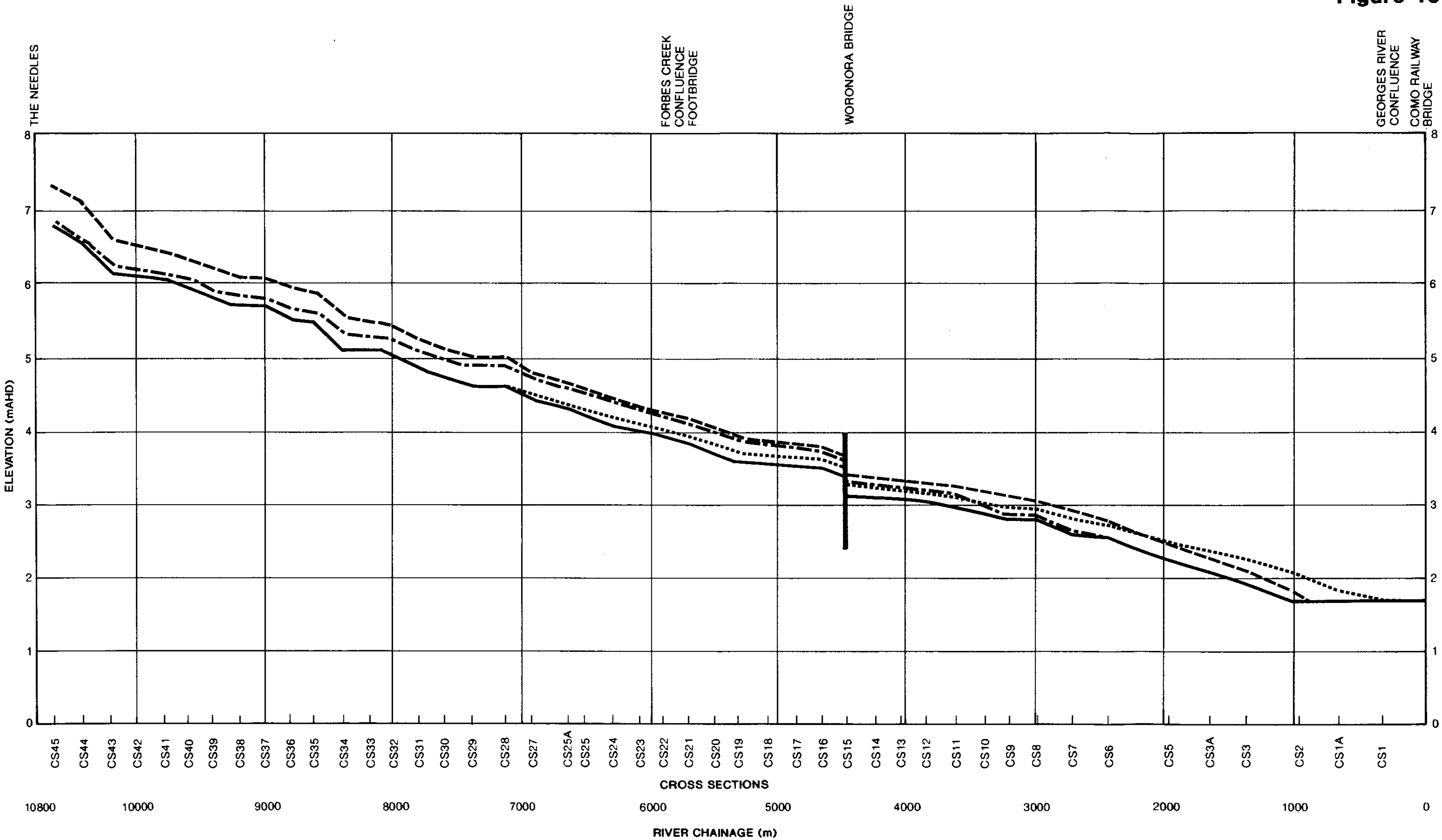
Cross section	CS45	CS44	CS43	CS42	CS41	CS40	CS39	CS38	CS37	CS36	CS35	CS34	CS33	CS32	CS31	CS30
1%	6.7	6.5	6.1	6.1	6.0	5.9	5.8	5.7	5.7	5.5	5.5	5.1	5.1	5.0	4.8	4.7
2%	6.3	6.1	5.7	5.7	5.6	5.5	5.4	5.3	5.3	5.1	5.1	4.7	4.7	4.7	4.5	4.4
5%	5.8	5.6	5.3	5.3	5.2	5.1	5.0	4.9	4.9	4.8	4.7	4.4	4.4	4.3	4.1	4.0

Cross section	CS29	CS28	CS27	CS25A	CS25	CS24	CS23	CS22	CS21	CS20	CS19	CS18	CS17	CS16	CS15
1%	4.6	4.6	4.4	4.3	4.2	4.1	4.0	3.9	3.8	3.7	3.6	3.6	3.6	3.5	3.4
2%	4.3	4.3	4.1	4.0	3.9	3.7	3.7	3.6	3.5	3.4	3.3	3.3	3.3	3.2	3.1
5%	3.9	3.9	3.7	3.6	3.6	3.4	3.4	3.3	3.2	3.1	3.0	3.0	3.0	2.9	2.8

Cross section	CS14	CS13	CS12	CS11	CS10	CS9	CS8	CS7	CS6	CS5	CS3A	CS3	CS2	CS1A	CS1
1%	3.1	3.1	3.0	3.0	2.9	2.8	2.8	2.6	2.6	2.3	2.1	1.9	1.7	1.7	1.7
2%	2.8	2.8	2.8	2.7	2.6	2.6	2.6	2.4	2.3	2.0	1.9	1.7	1.6	1.6	1.6
5%	2.6	2.5	2.5	2.4	2.4	2.3	2.3	2.1	2.1	1.8	1.7	1.5	1.5	1.5	1.5

DESIGN FLOOD PROFILES
WORONORA RIVER
WORONORA RIVER FLOOD STUDY

Figure 10



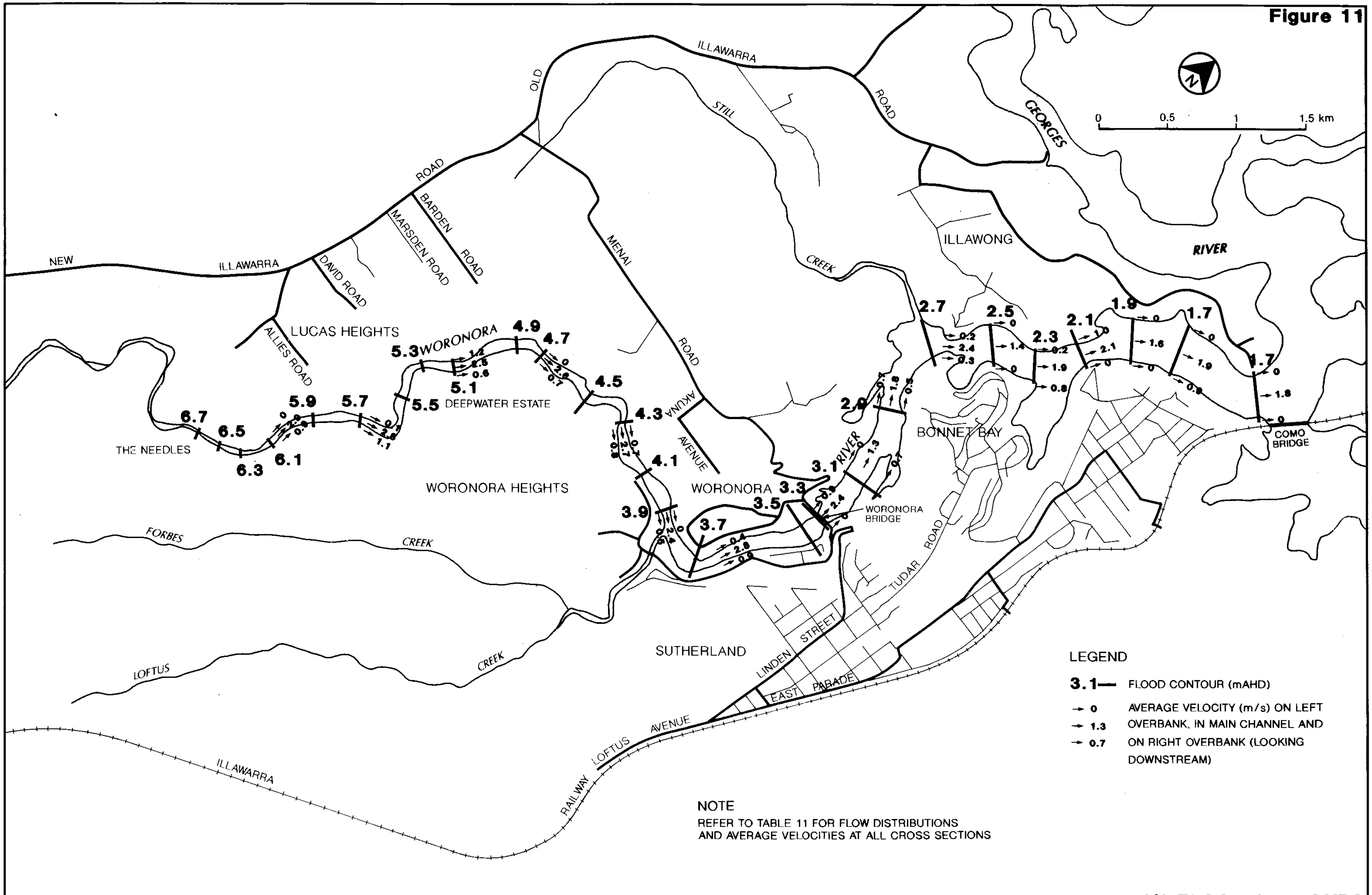
LEGEND

- 1% AEP FLOOD LEVELS ADOPTED (BASE CASE)
- INCREASED ROUGHNESS
- HIGHER GEORGES RIVER FLOOD LEVELS
- HIGHER WORONORA RIVER BED LEVELS

SENSITIVITY ANALYSES FOR 1% DESIGN FLOOD PROFILES WORONORA RIVER

WORONORA RIVER FLOOD STUDY

Figure 11



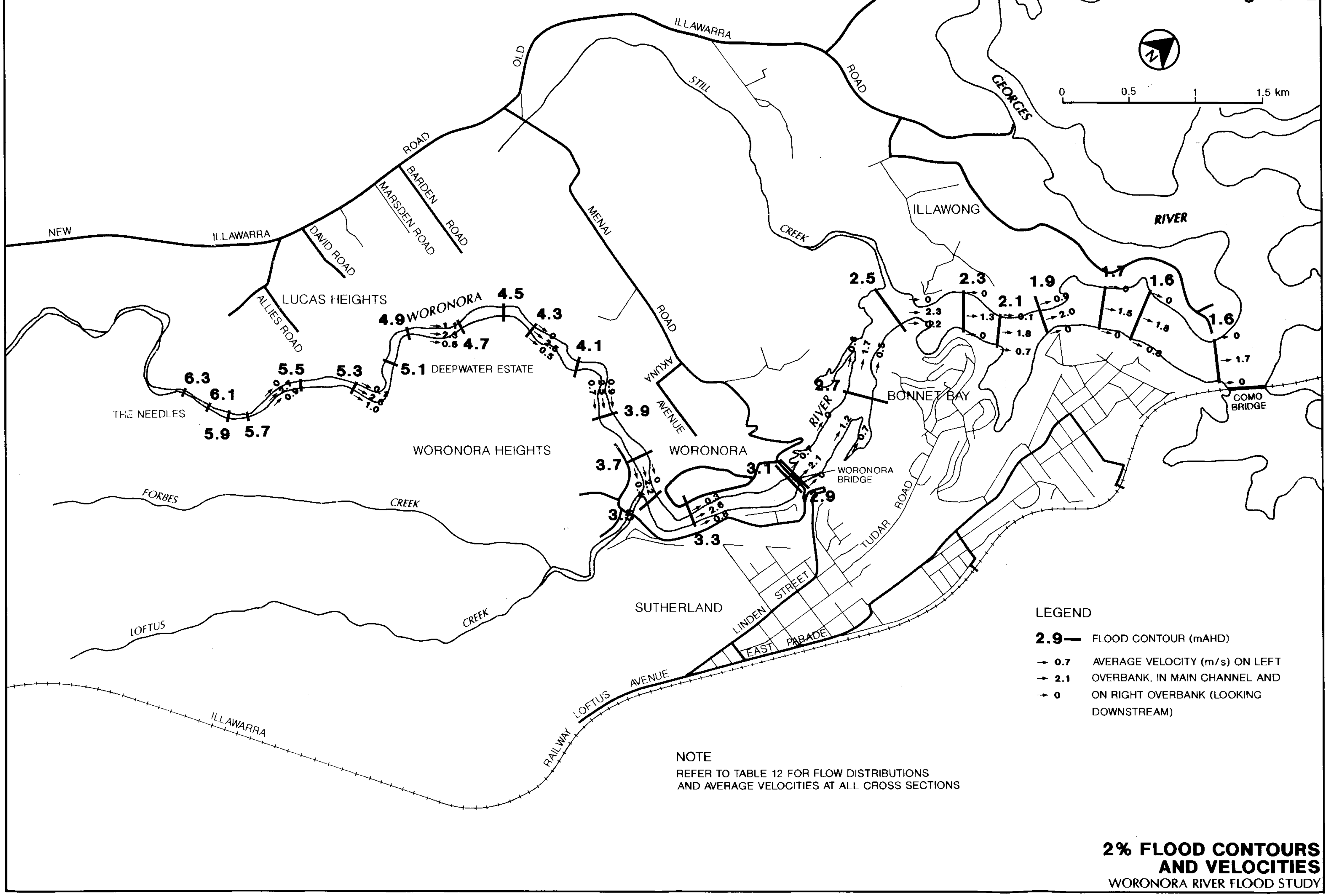
NOTE

REFER TO TABLE 11 FOR FLOW DISTRIBUTIONS AND AVERAGE VELOCITIES AT ALL CROSS SECTIONS

1% FLOOD CONTOURS AND VELOCITIES

WORONORA RIVER FLOOD STUDY

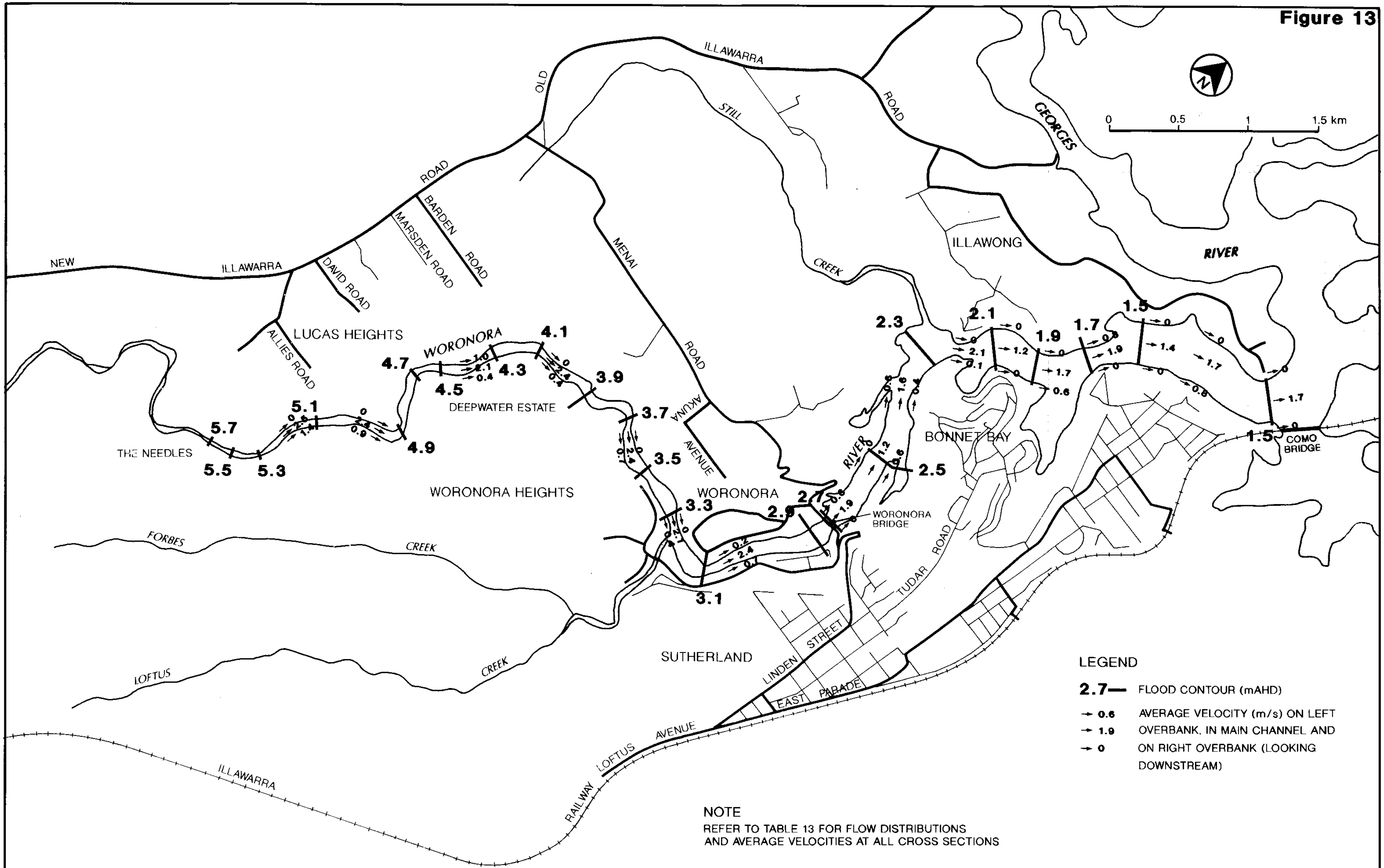
Figure 12



2% FLOOD CONTOURS AND VELOCITIES

WORONORA RIVER FLOOD STUDY

Figure 13



LEGEND

2.7 FLOOD CONTOUR (mAHD)

→ 0.6 AVERAGE VELOCITY (m/s) ON LEFT
→ 1.9 OVERBANK, IN MAIN CHANNEL AND
→ 0 ON RIGHT OVERBANK (LOOKING DOWNSTREAM)

NOTE

REFER TO TABLE 13 FOR FLOW DISTRIBUTIONS AND AVERAGE VELOCITIES AT ALL CROSS SECTIONS

5% FLOOD CONTOURS AND VELOCITIES

WORONORA RIVER FLOOD STUDY

Appendices

APPENDIX A - AVAILABLE DATA

APPENDIX B - HYDROLOGICAL MODELLING

APPENDIX C - HYDRAULIC MODELLING

Appendix A - Available Data

This appendix provides supporting information regarding the available data used in the study.

The rainfalls used in hydrologic modelling of historical rainfall-runoff events are presented in **Tables A.1** to **A.9**. These tables show both total rainfalls and pluviometer records. The total rainfalls for each sub-area of the catchment were derived from the daily rainfall records. The division of the catchment into sub-areas is shown in **Figure 3**.

The pluviometer records are presented as hourly rainfalls from the start of the modelled event. For events with more than one pluviometer record, the assignment of pluviometers to sub-areas is indicated. This assignment was made for the purposes of defining the temporal pattern of rainfall in each sub-area for the hydrologic modelling.

The available pluviometer records for some events were inadequate for modelling purposes. Synthetic pluviographs were developed for these events, and are also given in the tables.

Table A.10 shows the recorded high tide levels at Fort Denison for the dates of likely flood events in the Woronora River (as determined from rainfall and flood level records). The tidal records provided the basis for derivation of the hydraulic conditions at the Woronora River - Georges River confluence for historical flood events.

Figures A.1 and A.2 shows the recorded flood level data from 1898 to 1988, presented on a longitudinal section of the study reach.

Figure A.3 shows the Woronora River bed profile, as derived from hydrographic survey in 1984/85. The bed levels obtained in a check survey at five locations in 1990 are also indicated.

TABLE A.1 - RAINFALLS FOR JANUARY 1933 EVENT

TOTAL RAINFALLS

Sub-Area	Rainfall (mm)	Sub-Area	Rainfall (mm)
A	335	N	130
B	275	O	140
C	250	P	190
D	275	Q	160
E	200	R	160
F	240	S	155
G	175	T	160
H	260	U	170
I	225	V	175
J	190	W	180
K	200	X	185
L	150	Y	175
M	140	Z	190

PLUVIOMETER RECORDS

There are no pluviometer records available for this event. A temporal pattern was synthesized for the January 1933 event, based on the pluviometer record at Penshurst for the February 1956 event (**Table A.3**). This was adopted because both events were of similar magnitude and occurred at similar times of the year.

TABLE A.2 - RAINFALLS FOR JUNE 1949 EVENT

TOTAL RAINFALLS

Sub-Area	Rainfall (mm)	Pluvio Ref No	Sub-Area	Rainfall (mm)	Pluvio Ref No
A	159	2	N	90	1
B	150	2	O	90	1
C	140	2	P	140	1
D	160	1	Q	130	1
E	140	1	R	120	1
F	130	1	S	75	1
G	110	1	T	75	1
H	130	1	U	75	1
I	110	1	V	100	1
J	100	1	W	75	1
K	110	1	X	70	1
L	90	1	Y	70	1
M	100	1	Z	70	1

PLUVIOMETER RECORDS: HOURLY RAINFALLS FROM 2300 HRS, 17/6/1949

Pluvio No 1: Penshurst

3	3	3	4	4	2	2.3	2	2.5	2.4	8.3	10.4
12	3.3	0.7	0.7	1.7	0.7						

Pluvio No 2: Cronulla

3	3	3	4	4	2	1.3	1.4	1.3	5	11	14
7.7	1.3										

TABLE A.3 - RAINFALLS FOR FEBRUARY 1956 EVENT

TOTAL RAINFALLS

Sub-Area	Rainfall (mm)	Pluvio Ref No	Sub-Area	Rainfall (mm)	Pluvio Ref No
A	300	1	N	225	2
B	310	1	O	225	2
C	310	1	P	300	2
D	330	1	Q	275	2
E	300	1	R	250	2
F	285	1	S	215	2
G	275	1	T	215	2
H	270	1	U	215	2
I	265	1	V	235	2
J	265	1	W	240	2
K	275	1	X	225	2
L	260	1	Y	210	2
M	265	2	Z	210	2

PLUVIOMETER RECORDS: HOURLY RAINFALLS FROM 0700 HRS, 9/2/1956
Pluvio No 1: Synthetic

0	0	4	4.4	4.5	4.5	6	7	10	6	4.5	4
6	7	8	12	2	1.5	2	1.5	1.5	5	7	7
26	24	26	14	14	14	4	4	4	7	1	

Pluvio No 2: Penshurst

2	2	4.5	5	5	8	5	5	5	5
7.5	7.5	6.5	6.5	6.5	3	6	6	15.5	15.5
24.5	24.5	13.5	8.5	8.5	4	4	4		

TABLE A.4 - RAINFALLS FOR NOVEMBER 1961 EVENT

TOTAL RAINFALLS

Sub-Area	Rainfall (mm)	Pluvio Ref No	Sub-Area	Rainfall (mm)	Pluvio Ref No
A	415	1	N	160	2
B	300	1	O	125	2
C	215	1	P	185	2
D	215	1	Q	175	2
E	190	1	R	150	2
F	250	1	S	130	2
G	185	2	T	125	3
H	325	1	U	125	3
I	350	1	V	125	3
J	200	2	W	120	3
K	225	1	X	115	3
L	180	2	Y	125	3
M	185	2	Z	115	3

PLUVIOMETER RECORDS: HOURLY RAINFALLS FROM 1400 HRS, 17/11/1961

Pluvio No 1: Synthetic

0	2	0	0	1	1	0	1	1	1	2	2
3	3	2	2	3	4	3	8.4	6	5.2	4.8	5.2
22	29.2	10	5.2	2	1.2	2	2	2	4.4	2	2
4	6	5.2	7.2	6	5.2	4.8	1				

Pluvio No 2: Synthetic

0	0	0	0	0	0	1	0	0	1	1	1
1	2	2	1	1	4.8	13	5.2	6.4	6	5.2	6.4
6.4	10	10.4	10	10	6	4	6	4	4	3.6	4
3.2	4	4	5.2	6	8	10	9.2	8	23.2	26	

Pluvio No 3: Penshurst

0	0	0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	1.4	2	2	2	1	1
2	2	3	4	5.4	10	15	10	9	8	5.4	5
5	5	5	7.5	9	10	4	2.8	2	0.7	0	0
2	4.7	8	17	1.4	0	0.3	0.3	0.4	3	0.4	0.3
1	2.3	8.6	20	16.3	10						

TABLE A.5 - RAINFALLS FOR NOVEMBER 1969 EVENT

TOTAL RAINFALLS

Sub-Area	Rainfall (mm)	Pluvio Ref No	Sub-Area	Rainfall (mm)	Pluvio Ref No
A	300	1	N	220	2
B	255	1	O	210	2
C	275	1	P	260	2
D	285	1	Q	240	2
E	265	1	R	235	2
F	255	1	S	190	2
G	245	1	T	190	3
H	275	1	U	190	3
I	260	1	V	215	3
J	245	1	W	240	3
K	255	1	X	200	3
L	215	1	Y	180	3
M	240	2	Z	200	3

PLUVIOMETER RECORDS: HOURLY RAINFALLS FROM 0800 HRS, 12/11/1969

Pluvio No 1: Woronora Dam

0	0	0	0	4	5	5	0	0	2	2	1
1	1	1	1	1	2	2	2	3	2	1	6
6	3	3	5	5	8	8	11.5	11.5	8	8	3
3	2	2	3	3	4	4	8.5	8.5	15	15	3
3	1										

Pluvio No 2: Reverces

0	0	0	0	0	3	4	0	0	0	0	0
0	0	1	1	1	1	1	1	2	1	0	9
9	8	8	5	5	13	13	2	2	9	9	9
8	7	6	4	2	3	3	4	4	4	4	4
4	13	13	17	17							

Pluvio No 3: Penshurst

0	0	0	1	1	2	2	1	1	2	2	1
0	0	0	0	0	0	0	0	1	3	5	2
2.4	3.2	2	0.4	0.4	3.6	7.2	7.6	10	2.4	2	2
4.8	2.8	1.2	0.8	0	3.2	4.8	10	24	5.2	0	2
2	1.2	0	0.8								

TABLE A.6 - RAINFALLS FOR MARCH 1974 EVENT

TOTAL RAINFALLS

Sub-Area	Rainfall (mm)	Pluvio Ref No	Sub-Area	Rainfall (mm)	Pluvio Ref No
A	290	4	N	150	3
B	275	4	O	150	3
C	265	4	P	200	3
D	265	4	Q	175	3
E	225	4	R	150	3
F	225	4	S	150	1
G	190	3	T	150	1
H	215	4	U	150	1
I	190	4	V	150	2
J	180	4	W	150	2
K	190	4	X	150	1
L	170	3	Y	150	1
M	175	3	Z	165	1

PLUVIOMETER RECORDS: HOURLY RAINFALLS FROM 0800 HRS, 10/3/1974

Pluvio No 1: Mortdale

3.2	0	0	0	0	0	0	7.2	11.2	2.8	6.0	3.2
8.0	2.8	0	0	0.8	0	0	2	0	0	2	4
2.8	2	5.2	2.8	5.2	2	0	1	0	0	1	3
3	16	25.2	24.8	4	1						

Pluvio No 2: Cronulla

3	0	0	0	0	0	1	4	1	3	4	3
5	8	2	0	1	0	0	1	4	5.2	2	4
2	4	3	4	10	9.2	4	2	2	0	1	0
3	10	9	20	13	1						

Pluvio No 3: Woronora Dam

3	0	0	0	0	0	0	0	5	11	3	5
13	8	6	2	2	2	1	0	1	1	1	3
4	3	2	4	2	8	3	5	4	0	2	2
4	5	16	21	14	1						

Pluvio No 4: Reverces

5	0	0	0	0	0	6	10	6	10	10	8
10	5	0	0	0	4	5	4	3	5	2	2
4	5	10	3	6	5	5	3	2	0	2	5
10	13	21	20								

TABLE A.7 - RAINFALLS FOR MARCH 1977 EVENT

TOTAL RAINFALLS

Sub-Area	Rainfall (mm)	Pluvio Ref No	Sub-Area	Rainfall (mm)	Pluvio Ref No
A	250	1	N	130	2
B	175	1	O	165	2
C	172	1	P	160	2
D	170	1	Q	150	2
E	168	1	R	135	2
F	170	1	S	140	3
G	165	2	T	145	3
H	160	1	U	155	3
I	140	1	V	165	3
J	130	1	W	165	3
K	160	1	X	165	3
L	120	2	Y	145	3
M	140	2	Z	160	3

PLUVIOMETER RECORDS: HOURLY RAINFALLS FROM 1200 HRS, 3/3/1977

Pluvio No 1: Reverces

0	3	1	0	0	0	0	0.3	0.3	0.9	2.2	10
8	15	11	9	12.5	17	9	17.5	15	2	8.3	0.3

Pluvio No 2: Woronora Dam

0	1.9	0	0	0	0	0	1.2	1.3	0.7	2.2	4
5	9	7	7.3	11.5	13	10.5	15	9	2.3	2	0.1
0.1											

Pluvio No 3: Cronulla

0	2.5	0	0	0	0.5	0.5	0	0	1	0.5	0
0.5	1	0.5	1.5	2.5	4.5	3.5	2	4.5	4	7	12.5
15											

TABLE A.8 - RAINFALLS FOR AUGUST 1986 EVENT

TOTAL RAINFALLS

Sub-Area	Rainfall (mm)	Pluvio Ref No	Sub-Area	Rainfall (mm)	Pluvio Ref No
A	420	1	N	380	2
B	420	1	O	400	2
C	400	1	P	380	2
D	370	1	Q	380	2
E	390	1	R	380	3
F	410	1	S	400	3
G	390	2	T	390	3
H	430	1	U	370	3
I	425	1	V	400	3
J	410	1	W	390	3
K	400	1	X	370	3
L	390	2	Y	360	3
M	390	2	Z	360	3

PLUVIOMETER RECORDS: HOURLY RAINFALLS FROM 0900 HRS, 4/8/1986

Pluvio No 1: Reverces

1	2	3	10.5	6.5	4.5	11	11.5	6.5	4	4.5	2
2.5	2	6	1.5	3	9	10	13	14	7	2	5
10	8	8	6	8	6	11.5	6.5	9	6	8	6
7.5	7	15.5	5	7	15.5	17.5	23.5	12	2	3.5	3.5
3	4	7	12.5	11.5	6	7	5	2	2	0.5	0
0	0	1	0	3	1.5	0	0	0	1.5	1.5	0.5

Pluvio No 2: Woronora Dam

2.5	3	4	5	6.5	14	7	8	4	5	4	2
3	2.5	5	3.5	3.5	8.5	10	9.5	7.5	5.5	5	6
9	12	10	9	11	11	11	10	9.5	6	5.5	8
8	8	10.5	4.5	7	7.5	7.5	4.5	5	3	3.5	4.5
2	4	10	5	6.5	4	12	4.5	8.5	2.5	0.5	0
0.5	0	0	0	0	1	0	0	0	1	1	0

Pluvio No 3: Yarrawarrah

5.5	2	8.5	12	10.5	8	6.5	6.5	3	5.5	2	2.5
3.5	2.5	3.5	3	4.5	10.5	11	6	4.5	6.5	7	4.5
7.5	9.5	10	9.5	7	11	12.5	10	11.5	7.5	6.5	9.5
15.5	24	4.5	4	7.5	26.5	7.5	2.5	1.5	2	3	1.5
2	2.5	9.5	3	4.5	4.5	5	0	5	5	1	0.5
0	1	0	0	0	0	0	0	0	2	1.5	1

TABLE A.9 - RAINFALLS FOR APRIL 1988 EVENT

TOTAL RAINFALLS

Sub-Area	Rainfall (mm)	Pluvio Ref No	Sub-Area	Rainfall (mm)	Pluvio Ref No
A	425	1	N	305	2
B	390	1	O	270	2
C	370	1	P	340	2
D	370	1	Q	315	2
E	350	1	R	290	3
F	350	1	S	275	3
G	330	2	T	280	3
H	370	1	U	280	3
I	340	1	V	290	3
J	325	1	W	285	3
K	330	1	X	275	3
L	310	2	Y	270	3
M	315	2	Z	265	3

PLUVIOMETER RECORDS: HOURLY RAINFALLS FROM 0600 HRS, 28/4/1988

Pluvio No 1: Reverces

6.5	15.5	3.5	0	0.5	1	0	5	6.5	9	7	4.5
15	3	0	0	0.5	3	3.5	0.5	0	1	4.5	3
2	1.5	1	5.5	3	3	2.5	6	6	0.5	3.5	3.5
3	1.5	0.5	7	4.5	4.5	6	5.5	7.5	19	25	13.5
14.5	10.5	14	11	11.5	6	9	1.5	1.5	5.5	4.5	23.5
15											

Pluvio No 2: Woronora Dam

1.5	2.5	2	0	0	1.5	2.5	5	4	5	5	6
15.5	3	0	0	1	1.5	2.5	0.5	0.5	1	4	3
2.5	2.5	2	6.5	4.5	2	3	5.5	5.5	1	5	4.5
4.5	1.5	0.5	5.5	0.5	4	10	9.5	5	7.5	33.5	10.5
18	6.5	8.5	13	3.5	2.5	7	1.5	2	11	7.5	16.5
0.5	0.5										

Pluvio No 3: Yarrawarrah

0.5	0.5	1	0	1	1.5	3	6	4.5	3	4	6.5
14	2	0.5	0	0.5	3.5	0.5	1	1	1.5	2.5	2
3.5	2.5	3.5	7	4.5	3	3	7	4	1.5	7.5	4.5
9.5	0.5	1	1	0.5	2	14.5	6	4.5	16.5	13.5	27
6.5	9.5	15.5	5.5	4	6	5.5	0.5	1.5	18.5	4	3.5

TABLE A.10 - EXTRACTS FROM MSB TIDAL RECORDS AT FORT DENISON

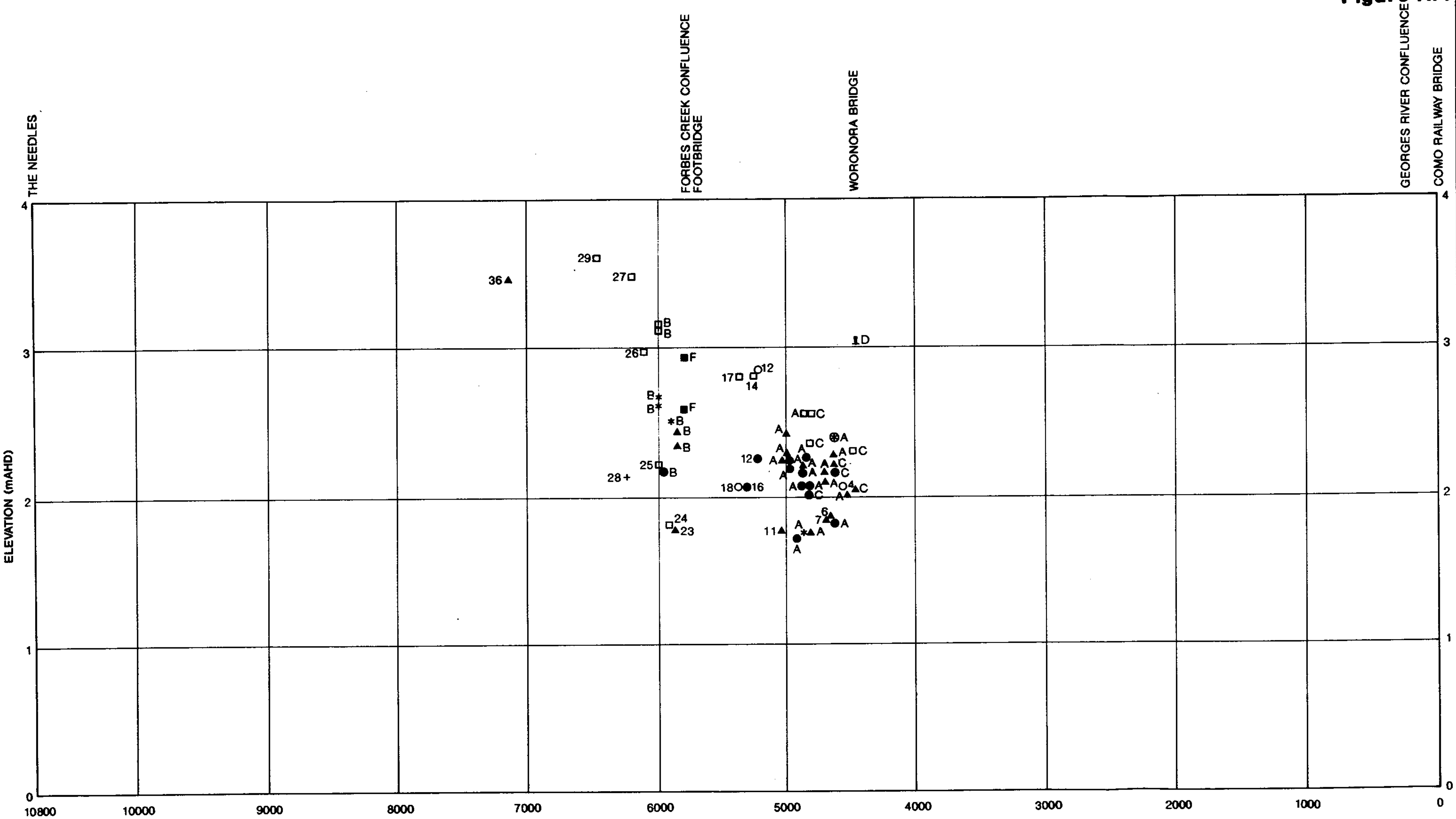
Date	High Water (Morning) (mAHD)		High Water (Afternoon) (mAHD)		Flood Data Sources
25/3/1926	4'11.5"	0.71	4'0.5"	0.43	S
25/1/1933	5'8.5"	0.94	3'9.5"	0.36	R,S
18/6/1949	4'5.5"	0.56	4'3.5"	0.51	W
14/11/1949	3'3.5"	0.19	4'5"	0.55	R
17/6/1959	4'1"	0.45	5'10"	0.98	R
23/7/1950	4'3"	0.50	4'11.5"	0.71	W
20/1/1951	5'3"	0.80	3'8"	0.31	R,W
15/6/1952	5'1"	0.75	4'7"	0.60	W
26/7/1952	4'4.5"	0.54	4'9"	0.65	W
1/5/1955	5'2"	0.65	4'8.5"	0.51	W
10/2/1956	5'9"	0.83	4'11.5"	0.59	R,W,S
11/2/1956	6'0.5"	0.92	5'3"	0.68	R,W
11/3/1958	6'8.25"	1.11	4'4.75"	0.41	R
19/11/1961	5'6.5"	0.76	5'1.5"	0.64	R,W,S
30/8/1963	4'10"	0.55	5'8.5"	0.81	R
11/6/1964	5'3.5"	0.69	7'1"	1.23	W
16/4/1969	5'4"	0.70	6'0.5"	0.92	R
14/11/1969	6'1"	0.93	NR		R,W
12/3/1974	1.36m	0.44	1.63m	0.71	W
10/3/1975	1.55m	0.63	1.35m	0.43	R,W
4/3/1977	1.74m	0.82	1.48m	0.56	W
16/3/1983	1.61m	0.69	1.54m	0.62	R
8/11/1984	1.68m	0.76	1.37m	0.45	R
6/8/1986	1.51m	0.59	1.99m	1.07	R
30/4/1988	1.52m	0.60	1.72m	0.80	R

Key: Flood Date Source: R - Rainfall records
W - Sydney Water Board records
S -Sutherland Shire Council

The conversion of the above readings to AHD is as follows:

- Prior to 31/12/1953
  Level (mAHD) = Tide reading (m) - 0.80
- Post 31/12/1953
  Level (mAHD) = Tide reading (m) - 0.925

Figure A.1



RECORDED FLOODS

- 1898
- 1926
- 1933
- 1940
- 1946
- 1949
- 1950
- 1954
- 1956
- 1961

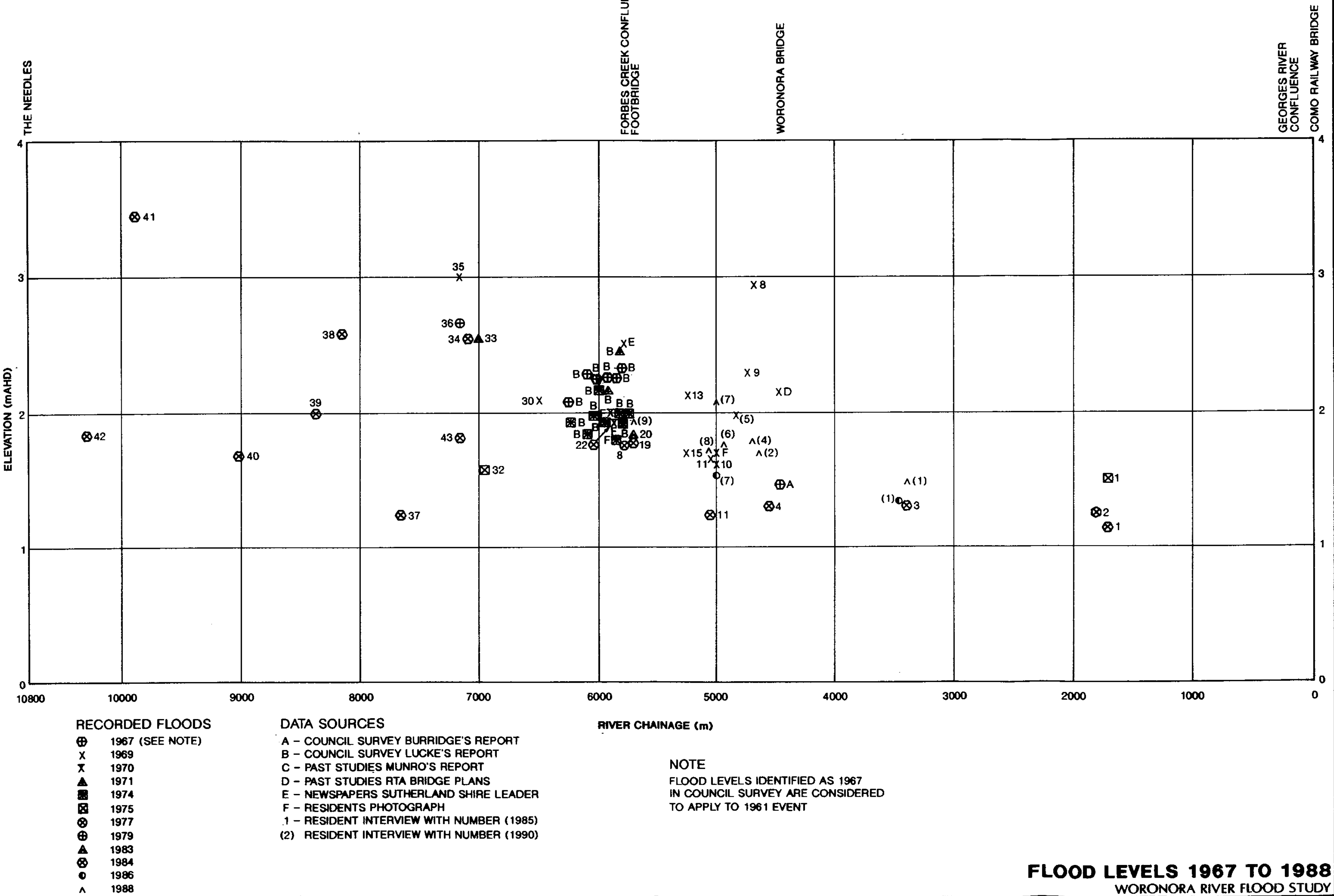
DATA SOURCES

A – COUNCIL SURVEY BURRIDGE'S REPORT
B – COUNCIL SURVEY LUCKE'S REPORT
C – PAST STUDIES MUNRO'S REPORT
D – PAST STUDIES RTA BRIDGE PLANS
E – NEWSPAPERS SUTHERLAND SHIRE LEADER
F – RESIDENTS PHOTOGRAPH
1 – RESIDENT INTERVIEW WITH NUMBER (1985)

FLOOD LEVELS 1898 TO 1966

WORONORA RIVER FLOOD STUDY

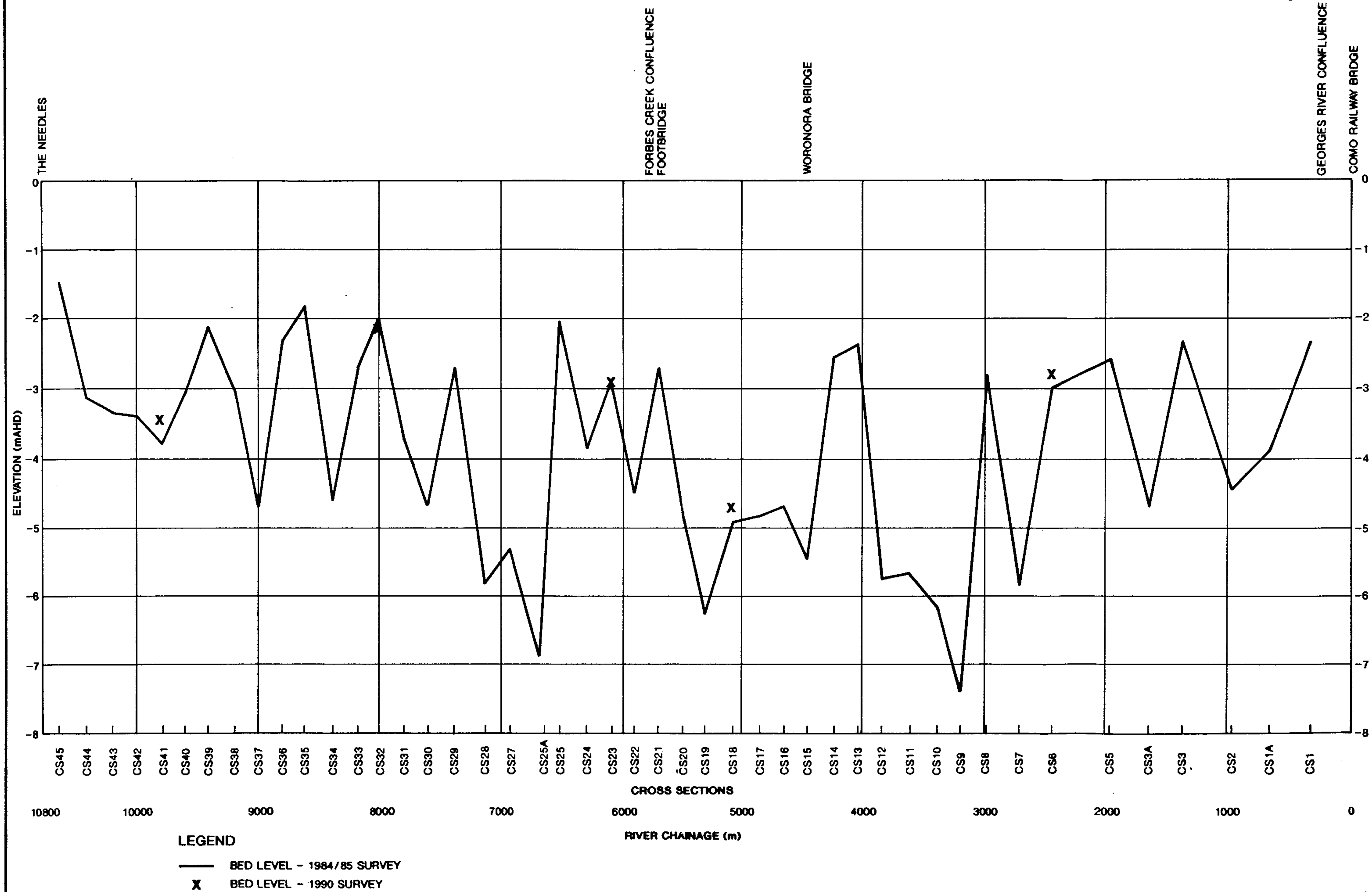
Figure A.2



FLOOD LEVELS 1967 TO 1988

WORONORA RIVER FLOOD STUDY

Figure A.3



WORONORA RIVER BED LEVELS

WORONORA RIVER FLOOD STUDY

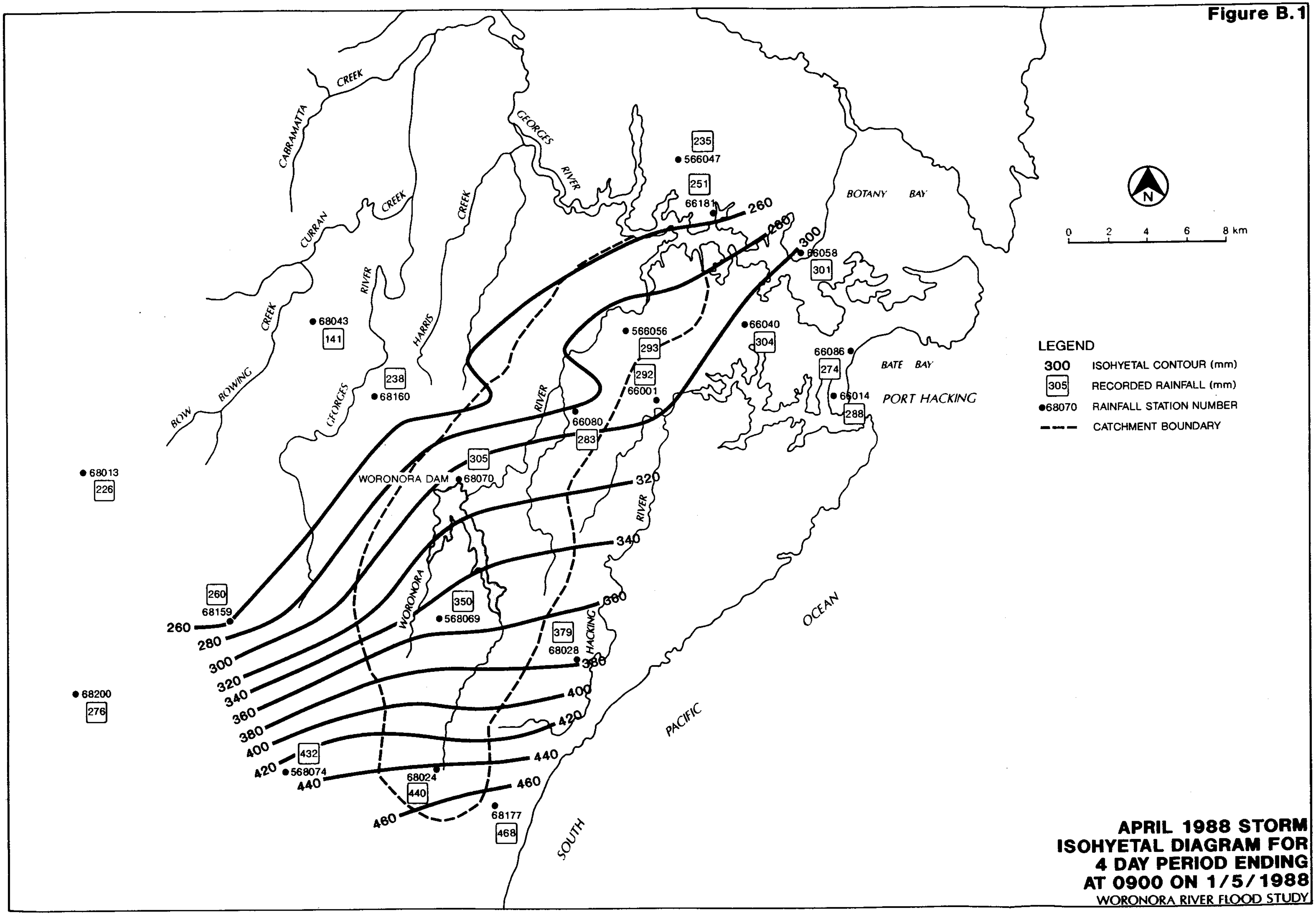
Appendix B - Hydrologic Modelling

This appendix provides supporting information on the hydrologic modelling undertaken as part of the study.

Figure B.1 shows the isohyetal diagram prepared for the April 1988 flood event. The total rainfalls for each sub-area of the RORB model were derived from this diagram.

Figure B.2 shows the recorded and modelled outflow hydrographs from Woronora Dam for the 1988 event. The good agreement between the two hydrographs, particularly in the vicinity of the peak discharge, indicate the validity of the RORB model developed for the catchment.

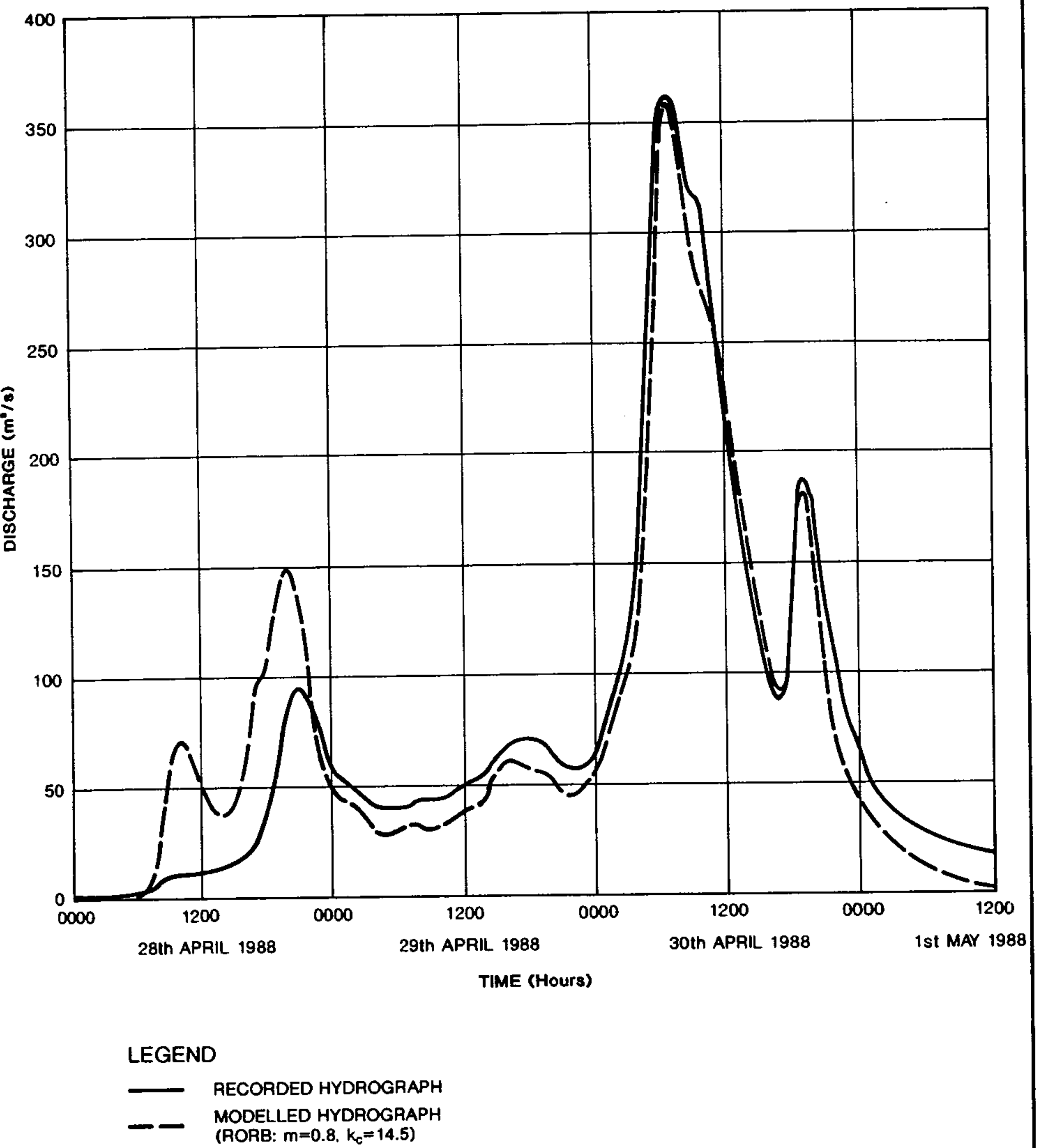
Figure B.3 shows the modelled hydrographs in the Woronora River at the Needles for six historical flood events. These hydrographs, together with the runoff hydrographs from local catchments downstream of the Needles, provided the hydrologic inputs for the hydraulic modelling of historical flood events.



APRIL 1988 STORM
ISOHYETAL DIAGRAM FOR
4 DAY PERIOD ENDING
AT 0900 ON 1/5/1988

WORONORA RIVER FLOOD STUDY

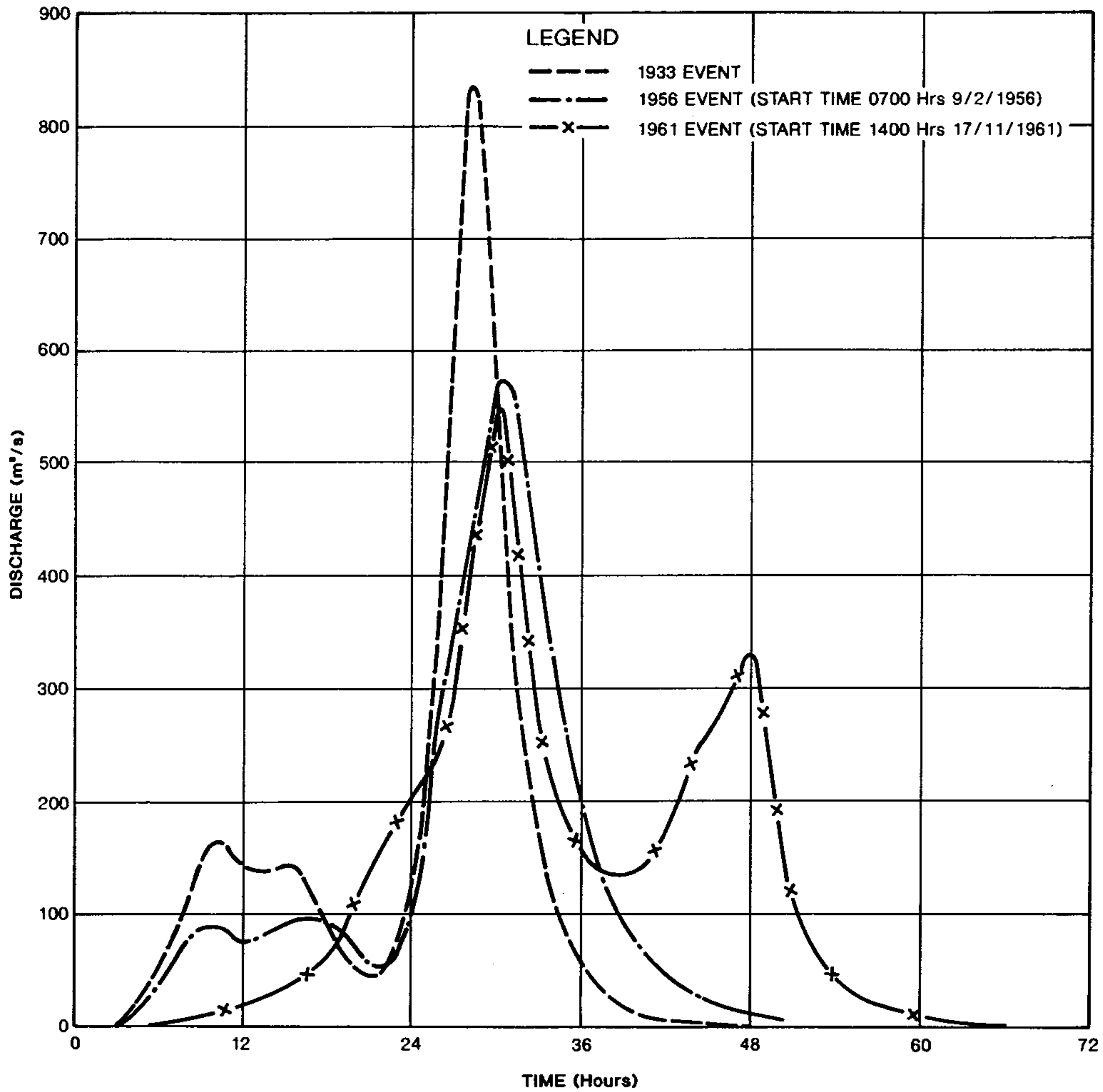
Figure B.2

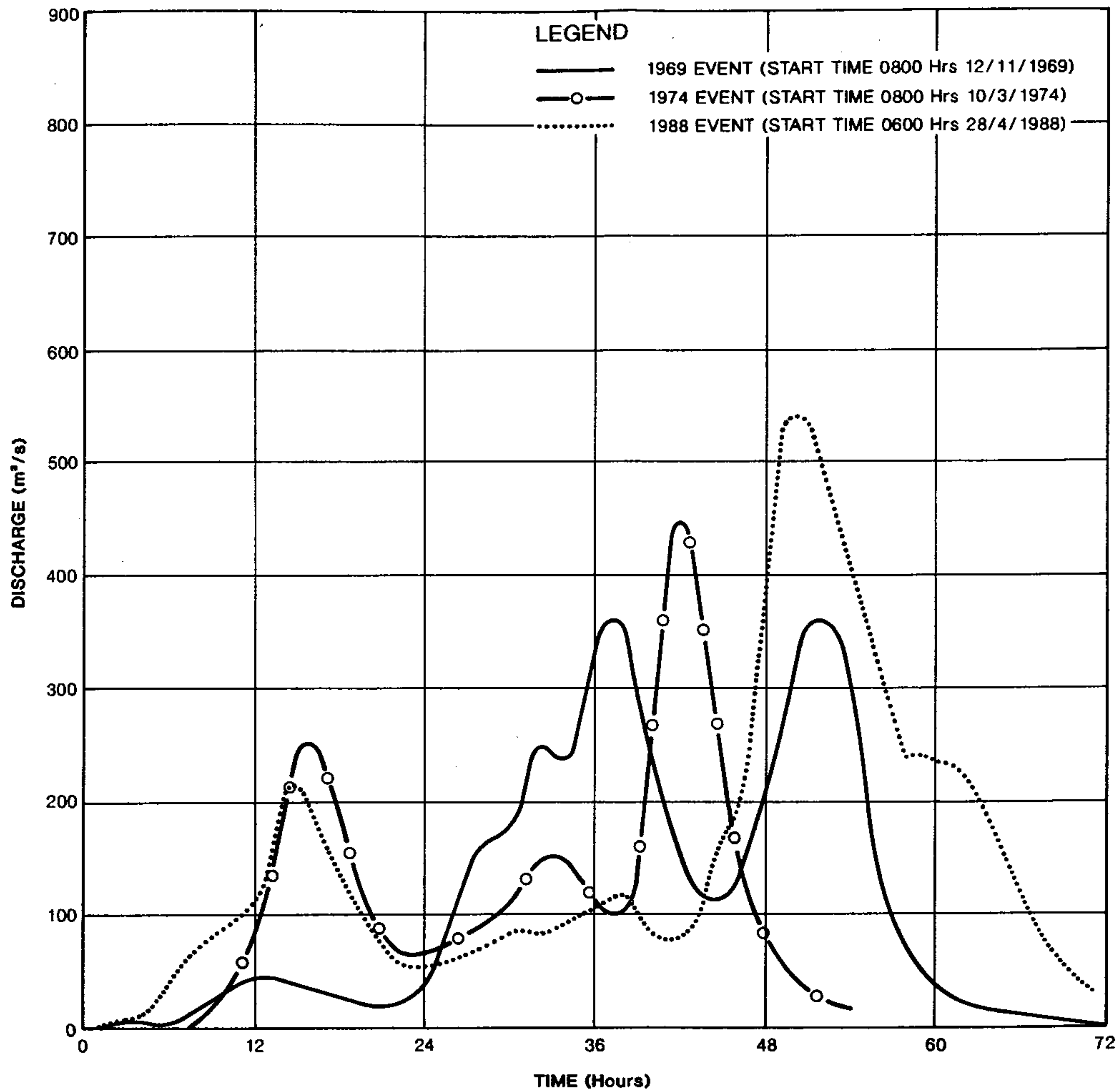


WORONORA DAM
OUTFLOW HYDROGRAPH FOR
APRIL 1988 EVENT

WORONORA RIVER FLOOD STUDY

Figure B.3





NOTE
HYDROGRAPHS ARE FOR WORONORA RIVER AT THE NEEDLES

MODELLED HYDROGRAPHS FOR WORONORA RIVER HISTORICAL FLOOD EVENTS

WORONORA RIVER FLOOD STUDY

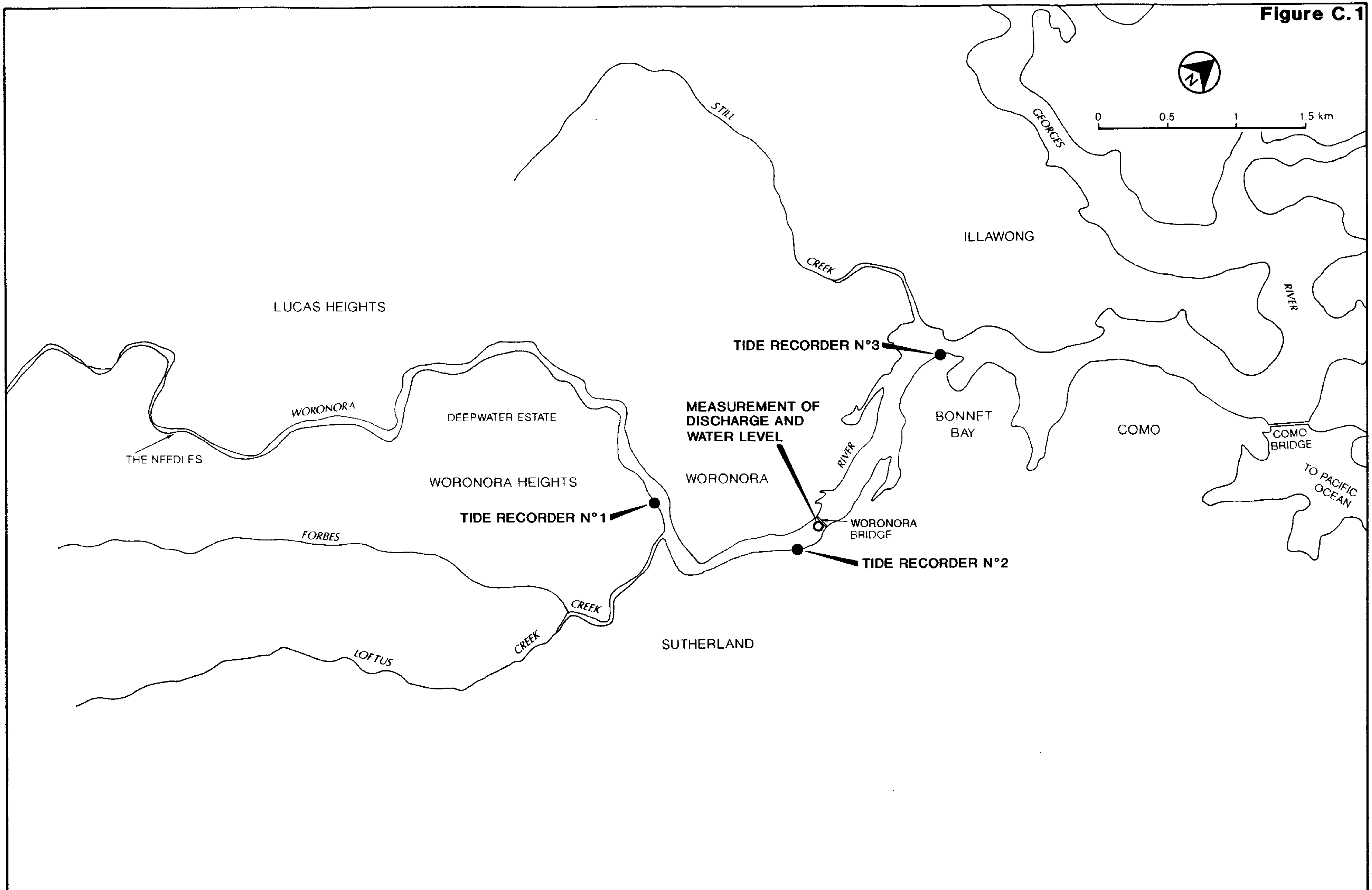
Appendix C - Hydraulic Modelling

This appendix provides supporting information on the hydraulic modelling undertaken as part of the study.

Figure C.1 shows the locations of three recorders installed to measure tide level variations in the Woronora River. The location of Woronora bridge, where tide levels and velocities were measured, is also shown.

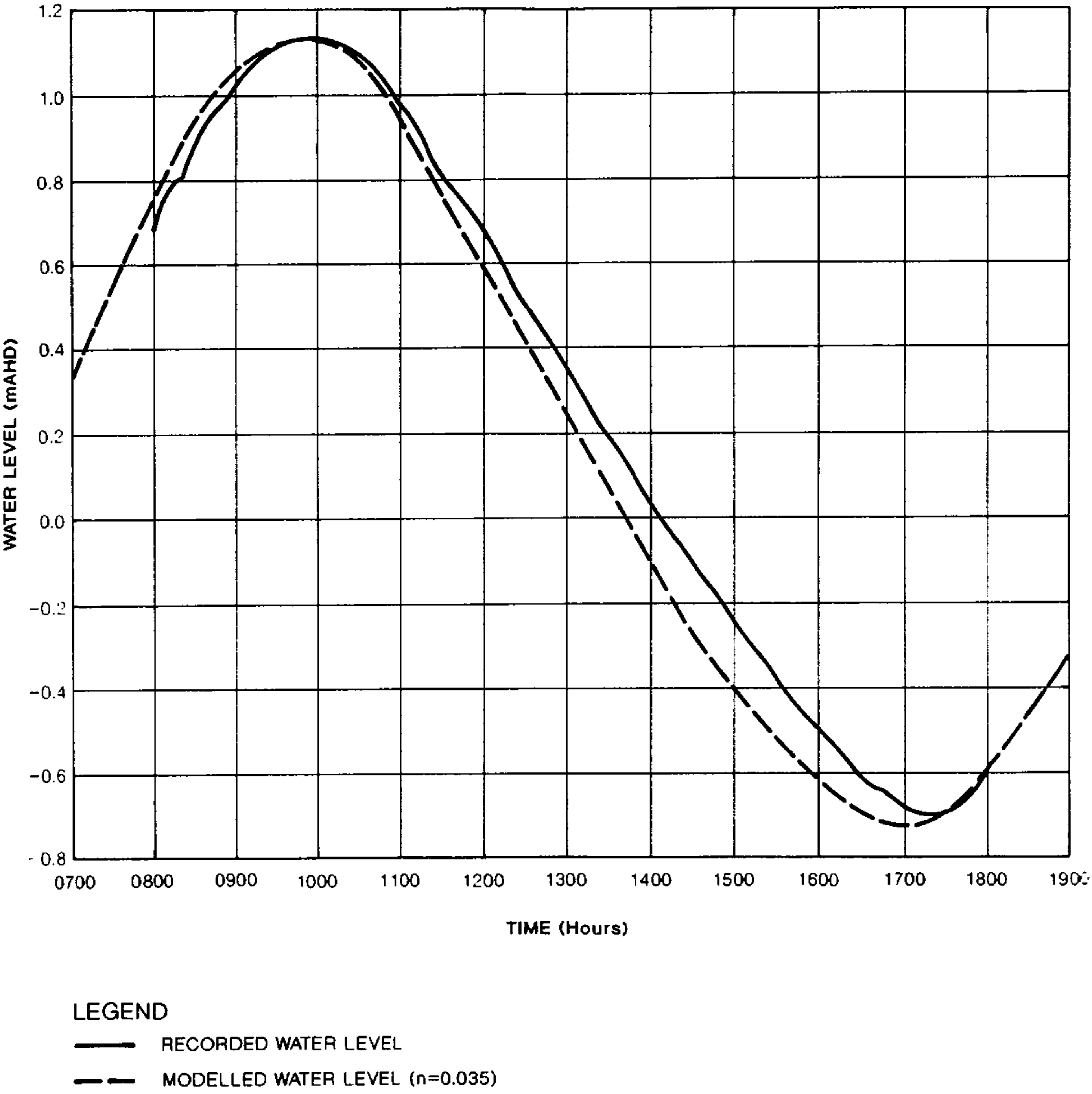
Figures C.2, C.3 and C.4 compare the measured and modelled levels at the three recorder locations from 0800 to 1800 hours on 14 November 1985. **Figure C.5** shows the recorded and modelled discharges, as well as water levels at Woronora bridge, over this period.

Figures C.6 to C.10 show the recorded flood levels and modelled flood level profiles for historical events in 1933, 1956, 1961, 1969 and 1974. The model results for those events provided general confirmation of the capability of the MIKE-11 model to simulate observed flood behaviour in the Woronora River.



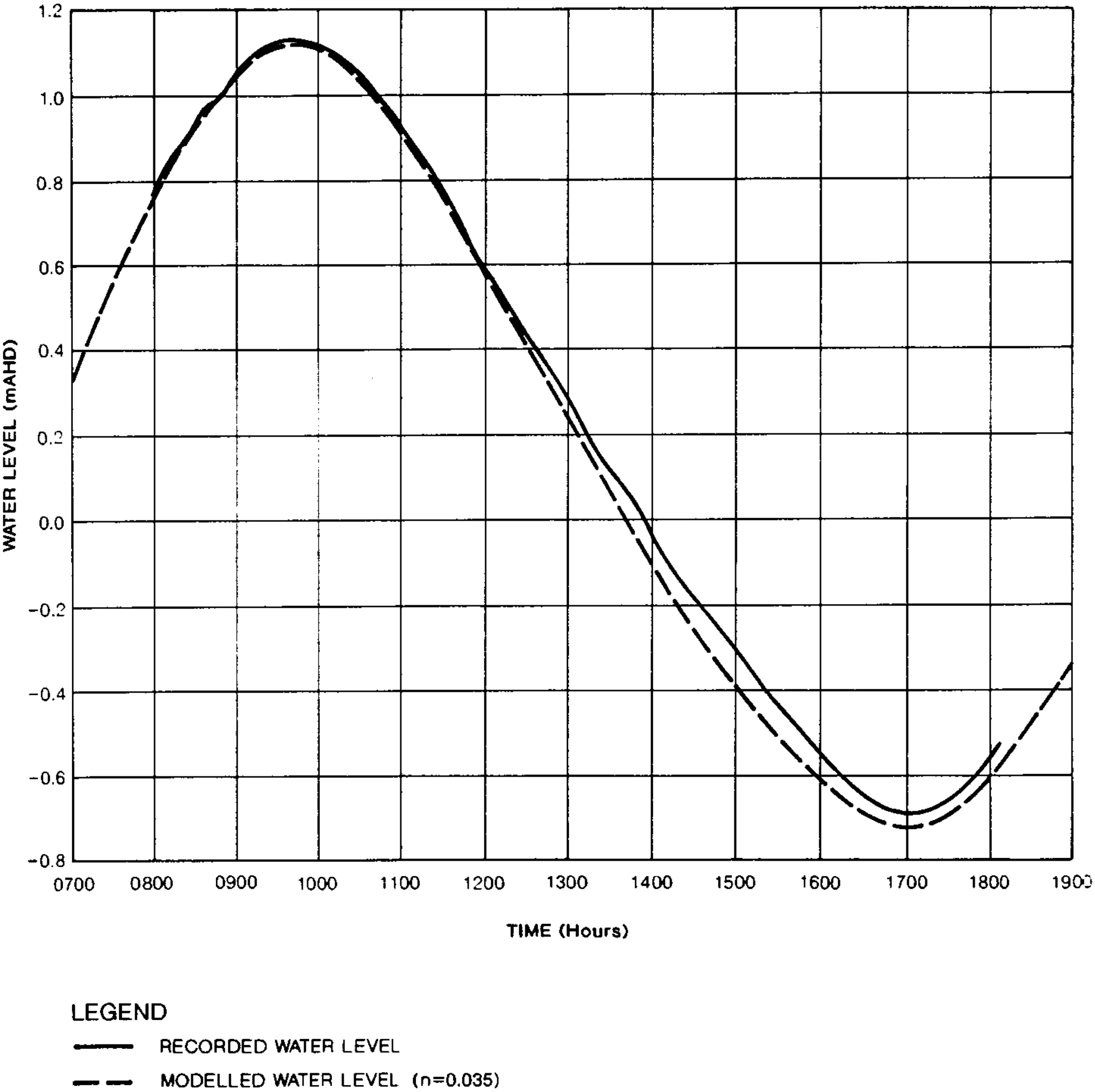
LOCATION OF TIDE RECORDERS
14 NOVEMBER 1985

Figure C.2



WATER LEVEL AT TIDE RECORDER 1
14 NOVEMBER 1985
WORONORA RIVER FLOOD STUDY

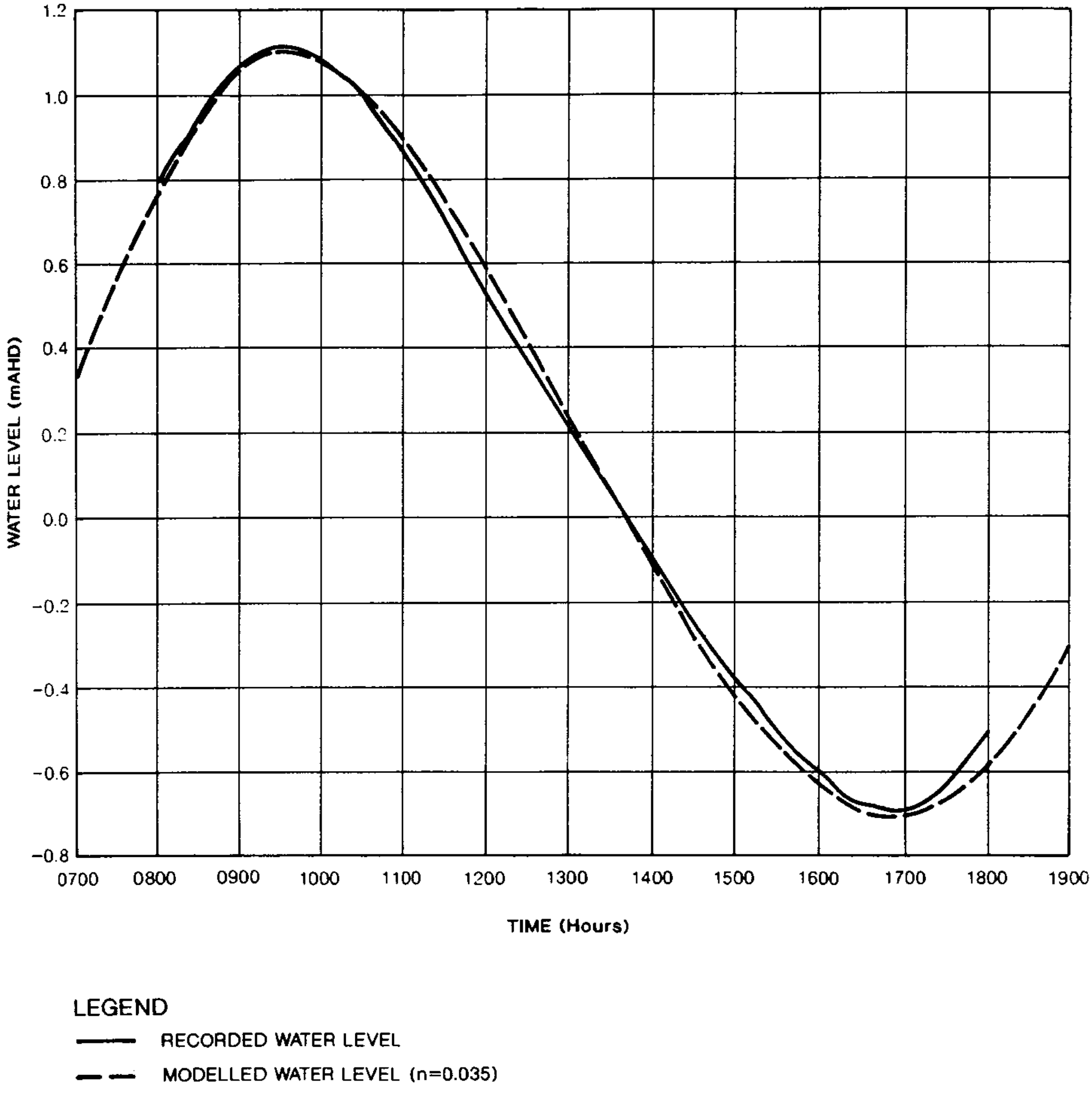
Figure C.3



WATER LEVEL AT TIDE RECORDER 2
14 NOVEMBER 1985

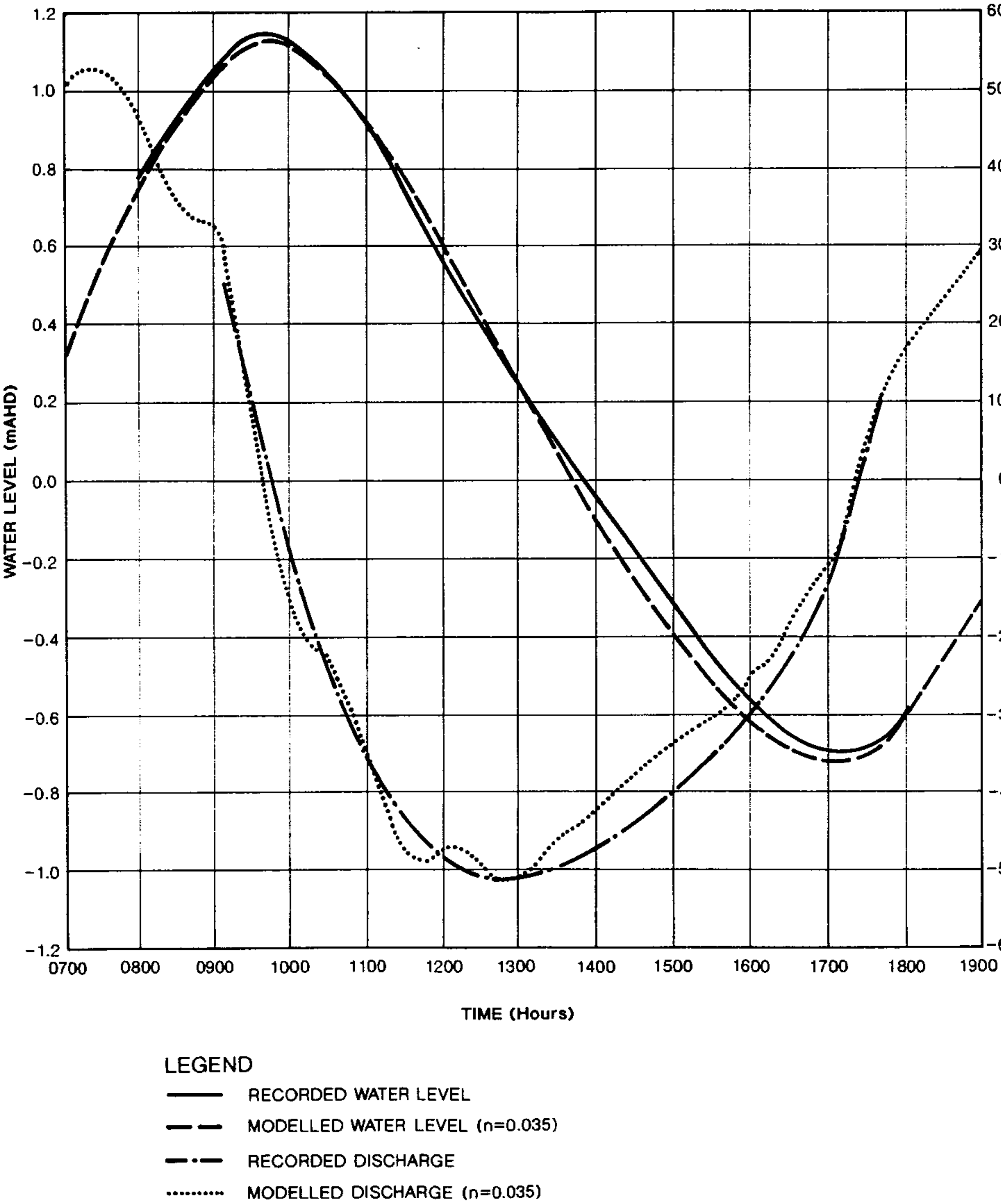
WORONORA RIVER FLOOD STUDY

Figure C.4



WATER LEVEL AT TIDE RECORDER 3
14 NOVEMBER 1985
WORONORA RIVER FLOOD STUDY

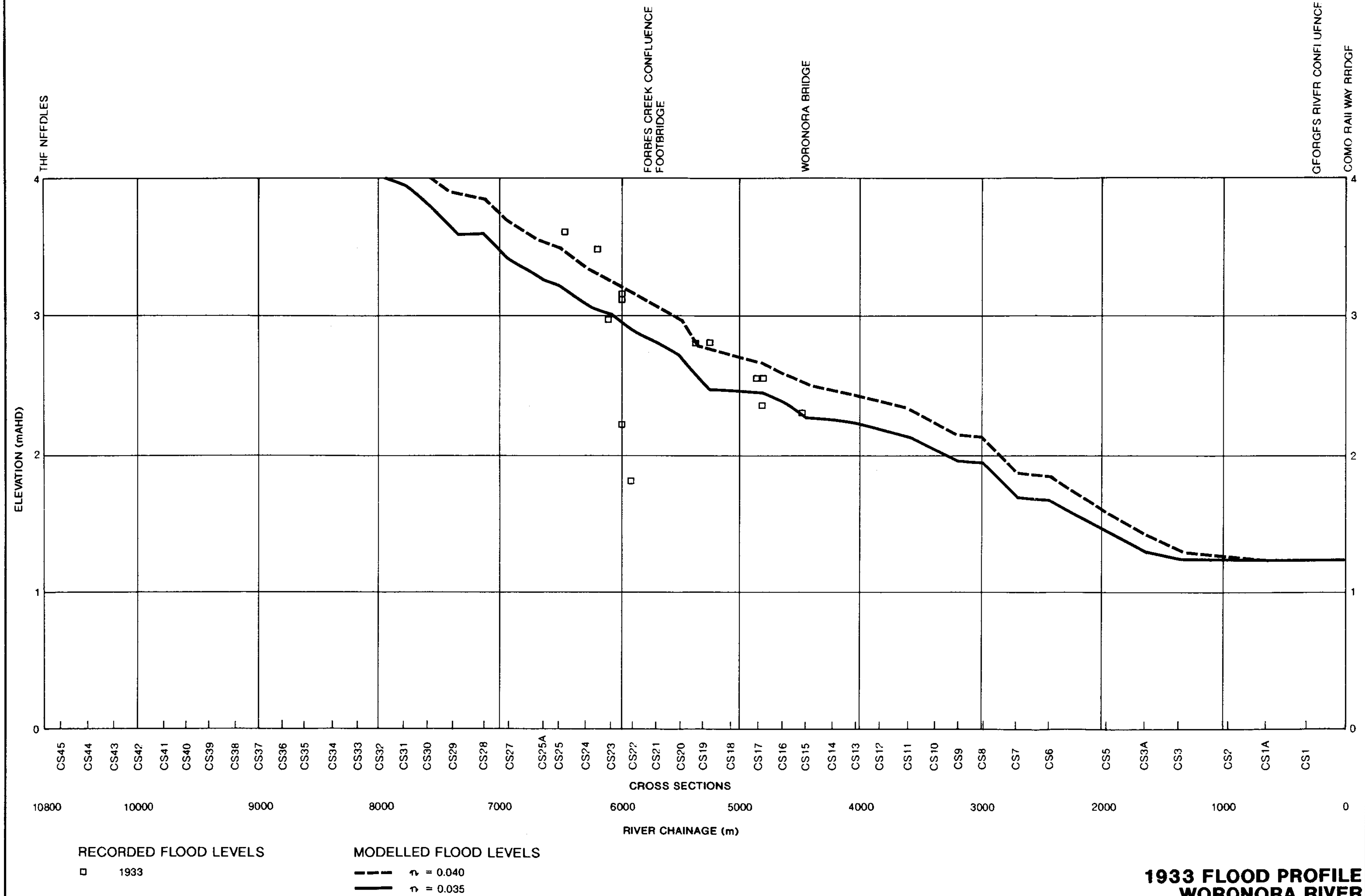
Figure C.5



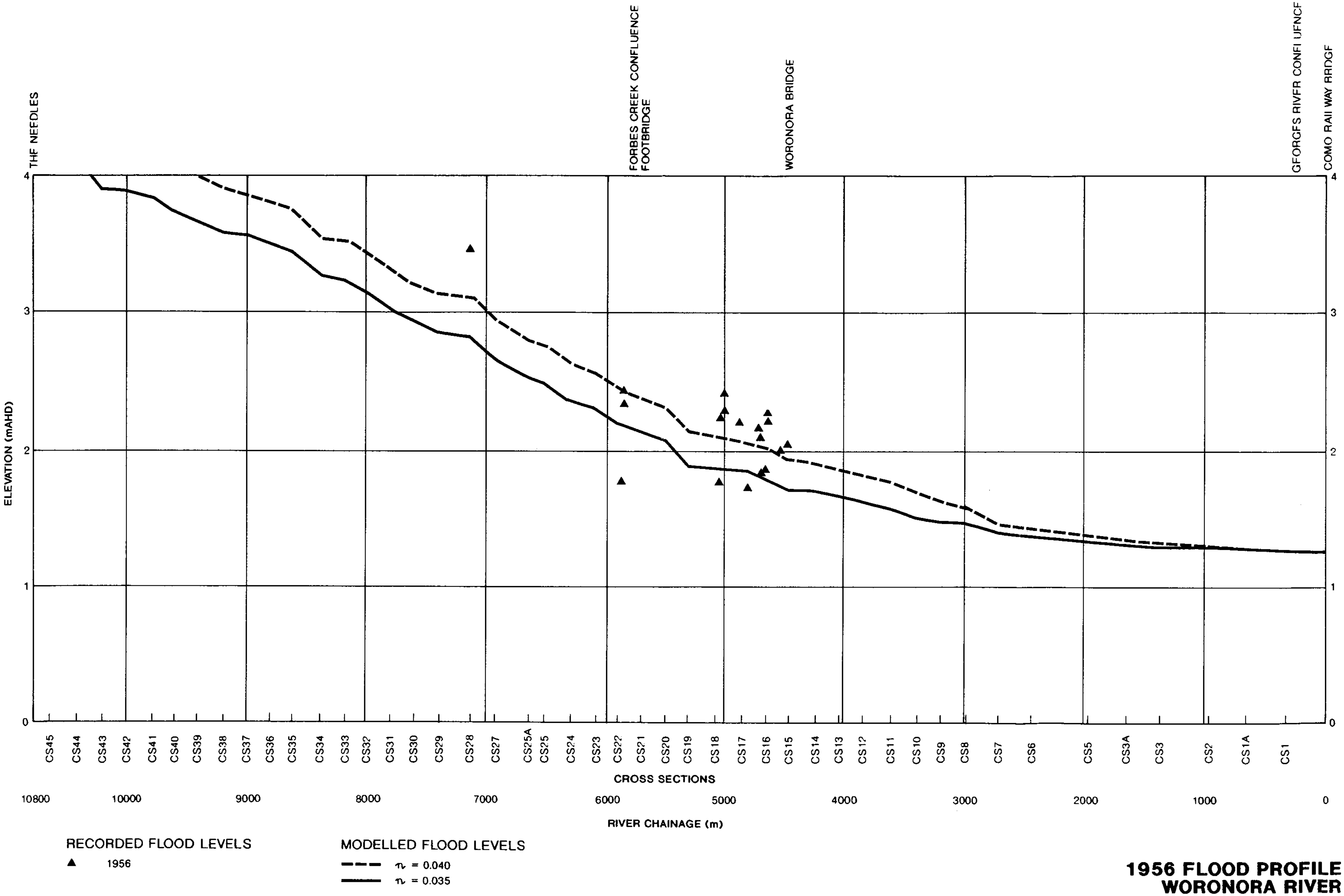
WATER LEVEL AND DISCHARGE AT WORONORA BRIDGE 14 NOVEMBER 1985

WORONORA RIVER FLOOD STUDY

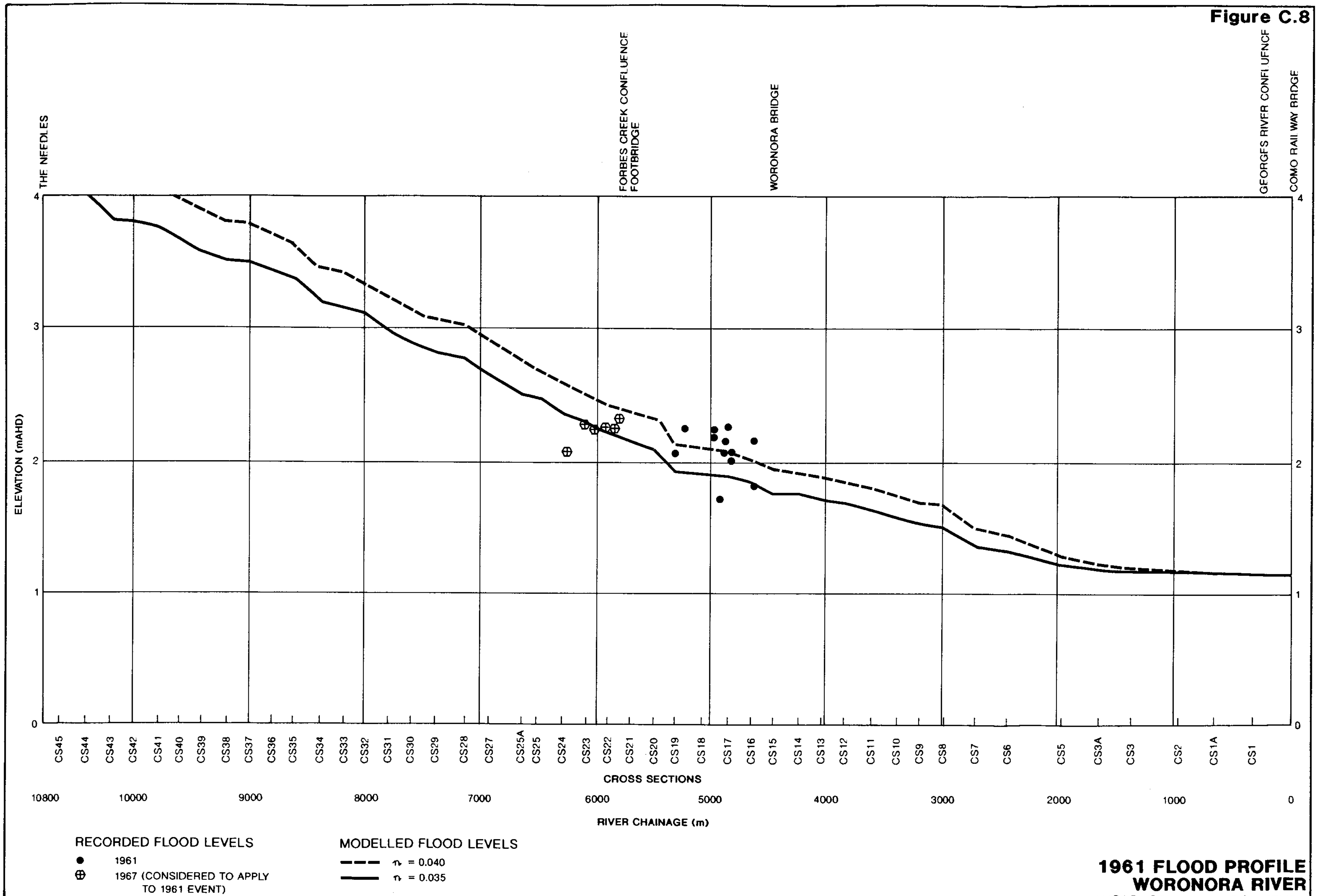
Figure C.6



1933 FLOOD PROFILE
WORONORA RIVER
WORONORA RIVER FLOOD STUDY

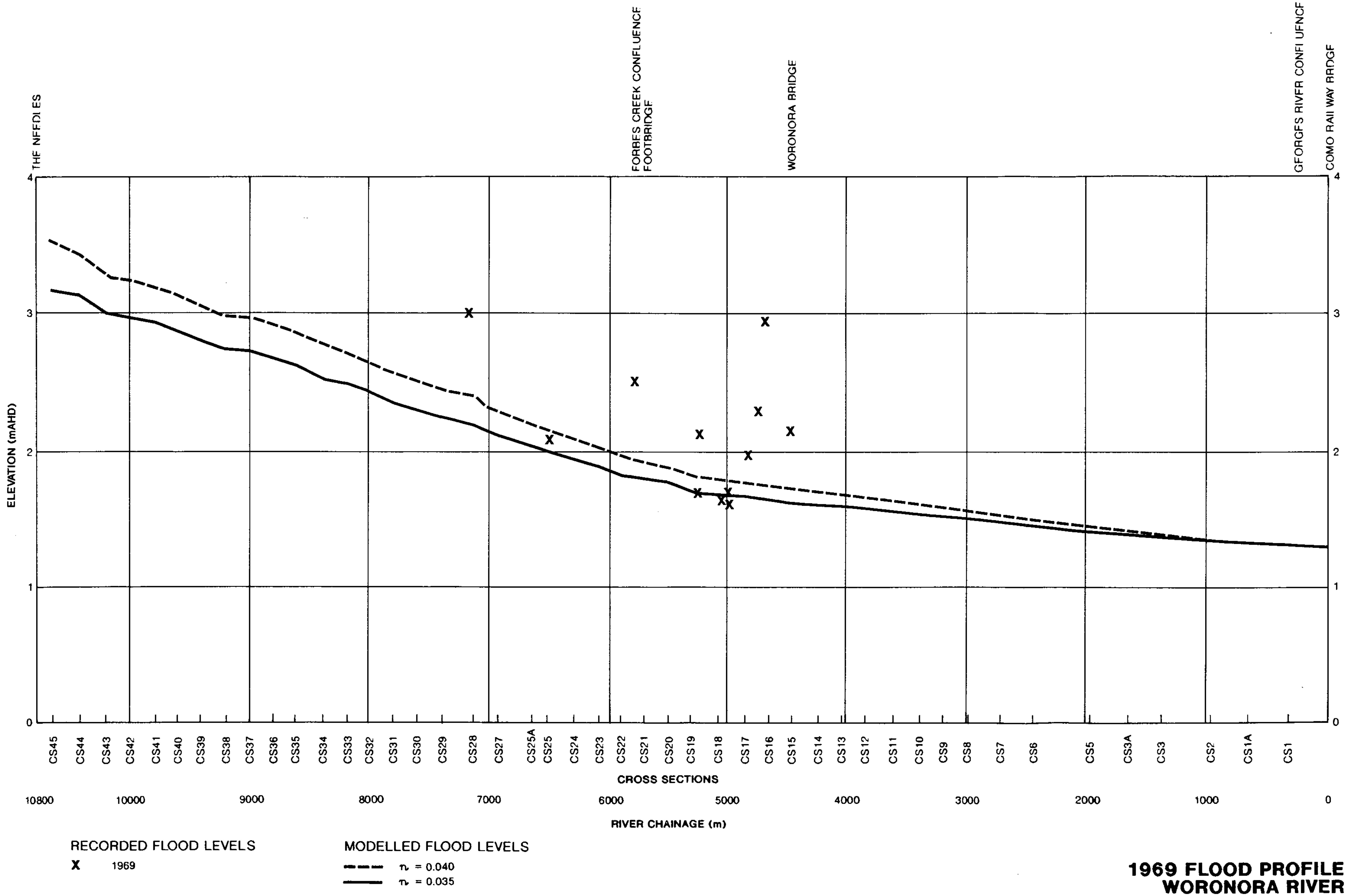


1956 FLOOD PROFILE
WORONORA RIVER
WORONORA RIVER FLOOD STUDY



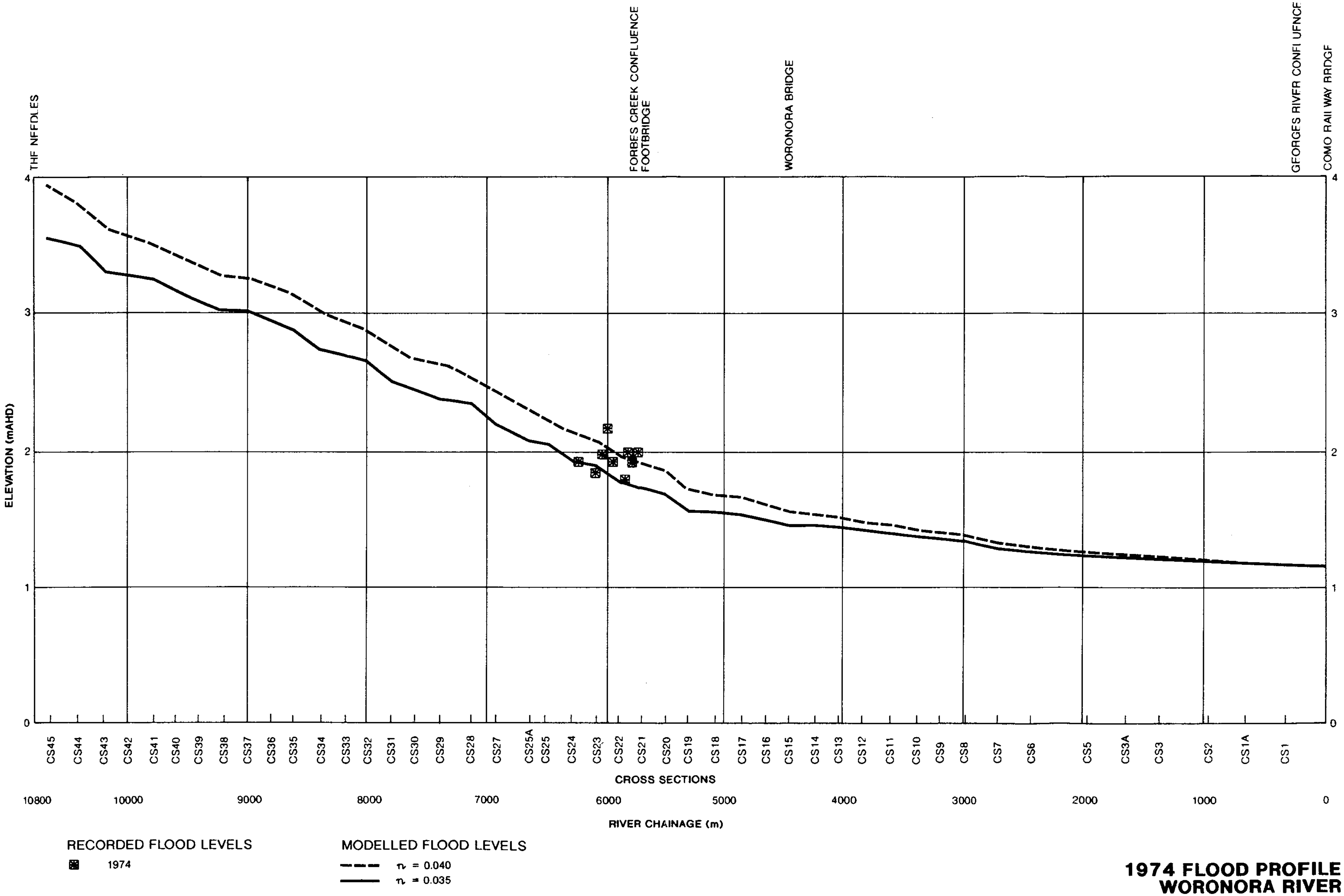
1961 FLOOD PROFILE
WORONORA RIVER
WORONORA RIVER FLOOD STUDY

Figure C.9



1969 FLOOD PROFILE
WORONORA RIVER
WORONORA RIVER FLOOD STUDY

Figure C.10



1974 FLOOD PROFILE
WORONORA RIVER
WORONORA RIVER FLOOD STUDY