LAND & WATER
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NSW Department of Land and Water Conservation

LOWER HAWKESBURY RIVER
FLOOD STUDY

Coastal and Floodplain Resources
Department of Land and Water Conservation
Report No. CFR97/06

FINAL DRAFT

prepared by

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Foreword

Australian Water and Coastal Studies Pty Ltd (AWACS) was commissioned by the Department of Land and Water Conservation (DLWC) to undertake investigations of the flooding characteristics of the Lower Hawkesbury River from Sackville to the ocean. These investigations form part of a series of studies being carried out to assist in the formulation of a floodplain management plan for the Hawkesbury Nepean Valley.

The State Government's Flood Policy is directed towards providing solutions to existing flood problems in developed areas and ensuring that new developments are compatible with the flood hazards whilst not causing additional flooding in other areas.

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

The policy provides for technical and financial support by the State Government through the following sequential stages:

1.	Flood Study	- Determines the nature and extent of flooding.
2.	Floodplain Management Study	- Evaluates management options for the floodplain in respect of both existing and proposed developments.
3.	Floodplain Management Plan	- Involves formal adoption by council of a plan of management for the floodplain.
4.	Implementation of the Plan	- Construction of flood mitigation works to protect existing developments.
		- Use of Local Environmental Plans to ensure new development is compatible with the flood hazard.

The Lower Hawkesbury Flood Study findings will provide the technical information to allow the formulation of a floodplain management plan for the Lower Hawkesbury Valley. This study has been prepared for the four councils that manage the local government areas of the Lower Hawkesbury River, namely Hawkesbury City, Baulkham Hills, Gosford City and Hornsby.

Summary

This report details the flood study of the Lower Hawkesbury River carried out to simulate the flood behaviour for a range of design flood conditions, including the 20% Annual Exceedance Probability (AEP), 5% AEP, 2% AEP, 1% AEP, 0.5% AEP and the Probable Maximum Flood (PMF) events. The study area comprised the section of the Lower Hawkesbury River from Sackville to the ocean at Broken Bay.

The total catchment area of the Hawkesbury River is about 22,000 square kilometres. A catchment area of 9,000 square kilometres or approximately 40 percent of the total catchment is below Sackville. The major tributary of the Hawkesbury River downstream of Sackville is the Colo River which has a catchment area of about 4,600 square kilometres. The Lower Hawkesbury River is characterised by relatively rugged terrain, with several small townships and communities along the river. The main communities are Brooklyn, Spencer, Gunderman, Wisemans Ferry, Leets Vale, Lower Portland and Sackville, and these were examined in detail in terms of design flood conditions.

The Hawkesbury River drains into the Pacific Ocean through Broken Bay. Broken Bay is exposed to the full range of oceanic conditions such as tides, storm surge and ocean waves. Examination of the interaction of catchment runoff and ocean levels was therefore an important part of the investigations.

The hydraulic flood model developed for the Lower Hawkesbury River from Sackville to Broken Bay was a two-dimensional depth averaged finite element model. The model was constructed using bathymetric data collected by hydrosurvey between 1978 and 1984 and topographic details from the 10 and 20 metre contours digitised from 1:25,000 scale topographic maps. The flood hydrographs and ocean level inputs to the model were based on information from Sydney Water (SW 1994) and Department of Public Works and Services (DPWS) (MHL 1992) studies.

The combination of flooding from Hawkesbury and Colo Rivers is an important factor in determining the design flood levels, particularly from Lower Portland to Spencer. As historical flood data provided a limited indication of trends, a statistical analysis approach as detailed in Appendix C was employed. The study results are based on the adoption of a 35-hour difference between the flood peaks at Sackville and Upper Colo, the historical coincidence of flood flows from the Colo and Hawkesbury Rivers and a derived relationship of Sackville flows, Upper Colo flows and flows at the confluence of the two rivers. The procedure for analysing joint coincidence of flows was outlined in Laurensen (1974). The results showed (Section 9) that the 1% AEP flood levels at Wisemans Ferry (Webbs Creek Ferry) could vary by 0.4 m depending on the combination of design floods originating for either the Colo or Hawkesbury Rivers.

The interaction of floods and ocean levels was examined, as detailed in Section 7. This showed that the flood levels at Brooklyn would be dominated by the design ocean levels and this impacts as far upstream as Gunderman.

The Sydney Water flood studies (SW 1994) were undertaken as part of a larger series of investigations for protecting Warragamba Dam against failure in extreme flood events. They were primarily focussed on assessing flooding of the larger centres of urban development, mainly surrounding Penrith, Emu Plains, Richmond and Windsor, as these areas would be subject to the severest impacts. This required simulating the complex hydrologic and hydraulic behaviour throughout the entire valley, to ensure the adoption of realistic inputs and boundary conditions, as far as possible, in the determination of potential flooding for the main areas of interest. The lower reaches of the Hawkesbury River were only modelled in sufficient detail for the purposes of establishing downstream boundary conditions for Sydney Water's RUBICON hydraulic model. As would be expected, some refinement and adjustment of Sydney Water's downstream outputs was found to be appropriate for these more detailed investigations of the lower Hawkesury Valley.

In setting up the hydraulic model for this study, it became apparent that the estimated inflows at Sackville and Upper Colo were critical to behaviour of the model. Sydney Water's discharge estimates for the 1990 and 1978 flood events, which were used for calibrating and validating the model, were reduced to improve the fit of modelling against the recorded levels. These adjustments took into account measured flow data at Sackville and at Webbs Creek Ferry, as well as a review of the rating curve at Upper Colo.

The significance of river bed scour, Manning's 'n' coefficients, the combination of Hawkesbury and Colo Rivers flows, the accuracy of the design flood hydrographs together with the combination of floods and ocean tides, were examined by undertaking sensitivity analyses. The results, presented in Section 9, should be taken into consideration when evaluating the effectiveness of various floodplain management strategies.

Table 10.1 presents the best estimate design flood levels at selected locations along the Lower Hawkesbury River. Figure 10.6 presents the flood profiles of the Lower Hawkesbury River for each design flood.

The previous estimates for the 1% AEP flood levels between Sackville and Spencer are generally 1 m lower than the 1% AEP best estimates recommended in this report (Table 10.2) and about 0.3 m higher at Brooklyn.

Further refinement of the flood model would only be worthwhile if additional flood level and flow data became available. In this regard, it is recommended that the operation of the monitoring stations continues and in the event of a major flood, flood flows be monitored to improve the present rating at Sackville and Upper Colo.

The model has the abilility to simulate the velocity distribution for various size floods on the floodplains and can be readily upgraded with more accurate survey information to examine development options in detail at selected locations along the river.

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1. Introduction

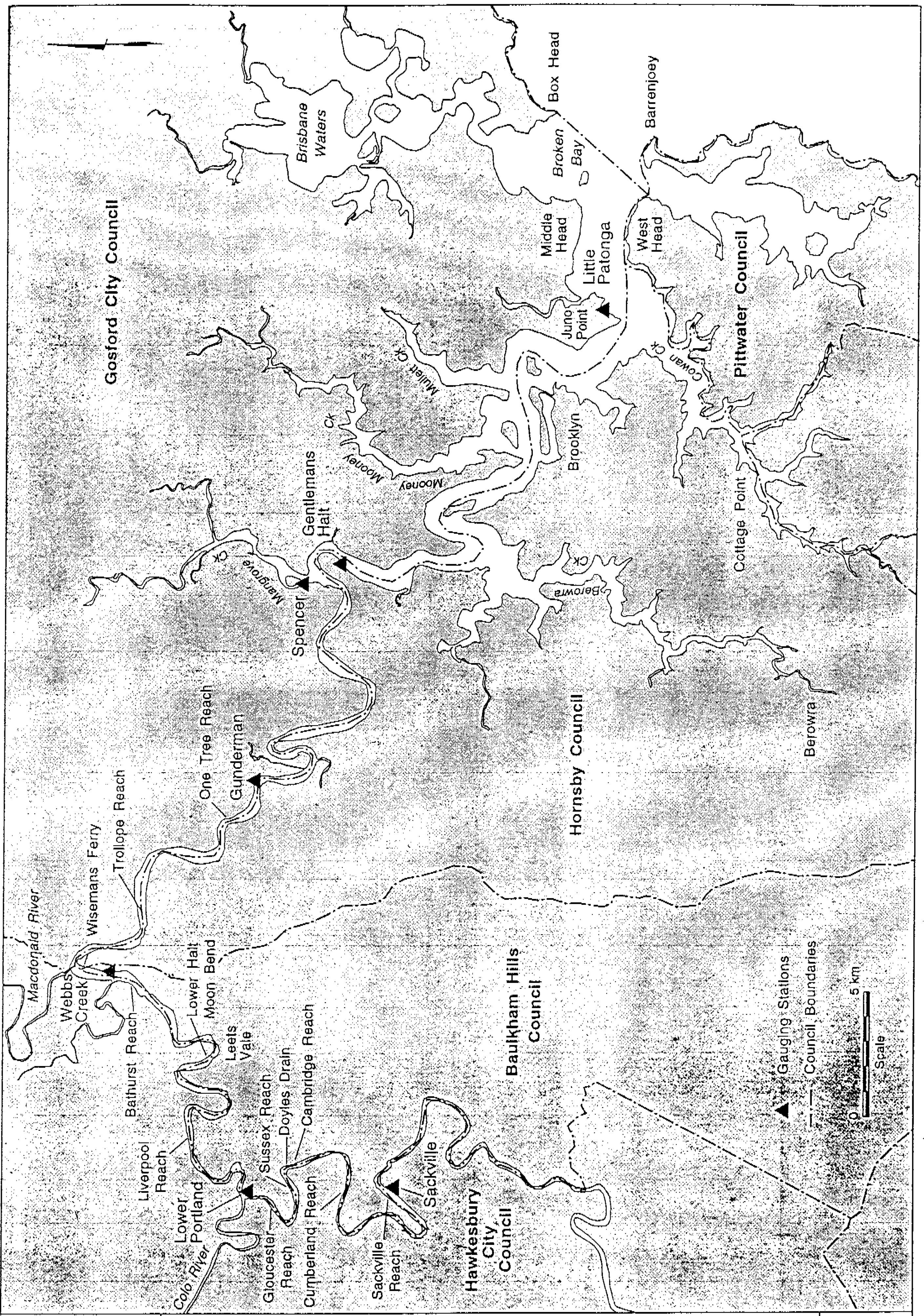
The Hawkesbury Nepean River and its tributaries form one of the largest coastal river systems in New South Wales. The total Hawkesbury Nepean River system is over 480 kilometres long from Goulburn to Broken Bay and the section within the study area (i.e. downstream of Sackville) is about 100 km long. The total catchment area is about 22,000 square kilometres above Brooklyn, with approximately 9,000 square kilometres or 40 percent of the catchment contribution downstream of Sackville. Figure 1.1 shows the extent of the Lower Hawkesbury River downstream of Sackville.

The Lower Hawkesbury River meanders through steep gorges downstream of Sackville to Spencer where the topography changes and the river widens into the drowned valley. The important feature of the Lower Hawkesbury River is the relatively narrow floodplains compared to the wide floodplains between Richmond and Windsor.

Historical flooding in the Lower Hawkesbury River has caused significant loss and damage to private properties, transport links, agriculture and other infrastructure along the river. During major floods, access to some main areas is restricted as roads and ferry crossings become flood-affected. Depending on the severity of the flood, this disruption can last for days, and in some areas safe evacuation of people from inundated areas can be critical.

The potential damage, losses, inconvenience and hazard to local residents have prompted the Government, in conjunction with local councils and the community, to recognise the need for an effective floodplain management plan for the entire area of the river. The Lower Hawkesbury River comes within the local government boundaries of four councils, namely Hawkesbury City, Baulkham Hills, Hornsby and Gosford City. Figure 1.1 shows the local government boundaries.

In the preparation of an Environmental Impact Statement for the upgrading of Warragamba Dam, Sydney Water carried out a major flood study (SW 1994) to determine the flood behaviour throughout the Hawkesbury Nepean floodplain. These investigations modelled design flood heights and flows for the river and floodplain downstream of the dam at Penrith, Richmond and Windsor, where flood impacts are potentially most severe. The river downstream of Windsor to the ocean was modelled to establish reasonable downstream boundary conditions. The coincidence of flood flows from the Colo and Hawkesbury Rivers and the range of ocean conditions were not examined in detail. Simulated flows generated by the Sydney Water study were reviewed and adopted where possible for this study.



AWACS

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LOCATION PLAN

Figure 1.1

2. Study Objectives

The objectives of the Lower Hawkesbury River Flood Study were:

- to construct, calibrate and validate a numerical two-dimensional hydraulic model of the Lower Hawkesbury River and floodplain from Sackville to Broken Bay;

- to estimate flows and heights for various design flood conditions; and

- to present the flooding characteristics for various design floods at larger communities located at Sackville, Lower Portland, Leets Vale, Wisemans Ferry, Gunderman, Spencer and Brooklyn.

The flood behaviour of the Lower Hawkesbury River can be influenced by the joint occurrence of floods from the Hawkesbury River upstream of Sackville and major tributaries below Sackville. This is potentially significant when large flood flows from the Colo and Hawkesbury Rivers combine. As well as this joint occurrence, flood conditions in the lower reaches can be influenced by the combination of rainfall runoff flooding and tidal variations. This study examined all these factors and recommends combinations for estimating design flood events.

The amount of flood flow and flood height data covering the river downstream of Sackville is limited. To calibrate and validate the model, this study utilised the available historical flood information for the main river combined with the flows estimated from the Sydney Water study (SW 1994).

3. Methodology

The main objective of this study was to simulate flood levels and flows along the lower reaches of the Hawkesbury River between Sackville and Broken Bay for the 20% AEP, 5% AEP, 2% AEP, 1% AEP, 0.5% AEP and PMF design floods. Particular attention was to be placed on the location of the larger communities as listed in Section 4.

There was insufficient flood level information from either historical flood data or the Sydney Water study (SW 1994) to derive reliable design flood levels using a flood frequency analysis approach. Therefore, design flood simulation techniques were employed. This required the estimation of design flood hydrographs and the development of an hydraulic model to characterise the hydrodynamics of the river. Sufficient flood level and flow data was available to allow the calibration and validation of the model to simulate the dynamic flood behaviour in the lower reaches of the Hawkesbury River.

The following steps outline the approach used to estimate design flood conditions:

- The rainfall runoff hydrographs derived from the Sydney Water study (SW 1994) were used as initial inputs in the study. Outputs were reviewed in light of the overall investigations and the initial hydrographs were modified as appropriate.

- Design ocean levels were selected, based on long-term observations.

- Joint probability studies based on the historical flood data were undertaken to determine the likely joint occurrence of floods in the Hawkesbury and Colo Rivers.

- A numerical hydraulic model was configured and then calibrated and validated against recorded flood levels and flows to simulate the flood behaviour of the Hawkesbury River between Sackville and Broken Bay. The lower river reaches are characterised by many acute bends and very confined floodplains. Flow paths and velocity distributions are therefore likely to be very complex and could differ markedly depending on the magnitude of the flood. To allow a better appreciation of the complex behaviour over the full range of potential floods, particularly in regard to evaluating hazards and future development proposals, a two-dimensional model was selected for this study.

- A range of design flood hydrographs was tested in the hydraulic model to examine the significance of joint occurrence, the likely range of flood levels and to allow recommendations on design flood levels.

- A sensitivity analysis was undertaken on a range of parameters to gauge their relative importance and to indicate the error band over the best estimate of design flood levels.

4. Existing Development

Flooding along the Hawkesbury River and its tributaries between Sackville and Broken Bay is confined within relatively deep and narrow river gorges. This results in steep flood gradients and very high flow velocities both within the river channel and the limited adjoining floodplains.

Located along these floodplains and river banks are a number of small communities. These can be significantly affected by major floods where the risk of damage to developments and the hazard to occupants can be very high. There are many locations which present significant hazards because emergency exit and/or safe refuge is made more difficult by the presence of high cliffs and rugged ridges immediately surrounding the floodplains.

The locations identified for detailed flood analysis were:

- Sackville
- Lower Portland
- Leets Vale
- Wisemans Ferry
- Gunderman
- Spencer
- Brooklyn

In addition to the above communities, there are a number of small dispersed developments sited along the river foreshores, often in the form of single dwellings on farms. Water skiing is also very popular in the area and several caravan parks and resorts have been established to cater for these recreational activities.

5. Catchment Description

5.1 General Description

The Hawkesbury River, known as the Nepean River upstream from its junction with the Grose River, drains an area of around 22,000 square kilometres. The total catchment lies south from the Hunter Valley, east from the Great Dividing Range and north from the Illawarra Range. The greater part of the northern and western areas of the Hawkesbury River Valley is rugged, mountainous terrain which is heavily timbered. The southern area of the catchment is undulating to hilly. A large alluvial plain lies between Richmond and Windsor and a series of small alluvial plains on the Colo and Macdonald Rivers and Webbs Creek punctuate what is essentially a drowned river valley meandering through steep sandstone ridges.

Table 5.1 details the catchment areas at selected locations along the Hawkesbury River and its tributaries

Table 5.1 Catchment Areas

River	Location	Catchment Area (km^2)
Hawkesbury	Sackville	12,950
	Lower Portland	13,450
	Wisemans Ferry	18,150
	Brooklyn	21,600
Colo	Upper Colo	4,350
	Morans Rock	4,640
Macdonald	St Albans	1,680
Nepean	Wallacia	1,760
Warragamba Dam	-	9,000
Mangrove Creek	-	414

Warragamba Dam, located on the upper Hawkesbury River some 100 km upstream of Sackville, came into operation in 1960 and since construction has impacted on the flood behaviour of the Hawkesbury River Valley. The dam controls about 40% of the total Hawkesbury Nepean catchment area. Floods that occur while the water level in Warragamba Dam is below full supply level would have a proportion of their flood waters stored in the dam and the peak outflow from the dam would be reduced. The impact and operation of Warragamba Dam was addressed in the Sydney Water study (SW 1994).

5.2 Tributaries of the Lower Hawkesbury River

The Lower Hawkesbury River Flood Study area is bounded upstream from Sackville and downstream to Broken Bay between Middle Head and West Head (see Figure 1.1). The following rivers and tributaries flow into the study area:

- Colo River
- Webbs Creek
- Macdonald River
- Mangrove Creek
- Berowra and Mooney Mooney Creeks
- Mullet and Cowan Creeks
- several other minor creeks.

5.3 Hawkesbury River Entrance

The Hawkesbury River drains into the Pacific Ocean through Broken Bay. Broken Bay is exposed to the full range of oceanographic processes: tides, storm surges and ocean waves. The water depths at the entrance of the Hawkesbury River are such that waves penetrate the river entrance unbroken. Therefore, wave setup is not a component of the elevated ocean levels at the model boundary.

Middle Head which is located 2 km inside Broken Bay was selected as the downstream boundary of this flood study. A large storage volume was included at the downstream boundary of the hydraulic model to simulate Brisbane Water, Pittwater and Broken Bay, however the model results showed no change in flood levels with and without the storage volumes included.

5.4 Hawkesbury River Floodplains

The floodplains of the Hawkesbury River between Sackville and Brooklyn are generally small and narrow and are confined between the river and the steep escarpment of the gorges. The geomorphological processes of the river have created these alluvial floodplains where in places small communities have been established. At some locations the level of the floodplains tends to be relatively low when compared to normal tide levels in the river. At Spencer the level of the floodplains is just above normal high tide level while at Sackville the floodplain level is about 4 to 6 m above high tide.

6. Existing Information

6.1 Warragamba Dam Environmental Impact Statement Flood Study

Sydney Water commissioned consultants Webb McKeown & Associates Pty. Ltd. to investigate flood behaviour in the Hawkesbury Nepean floodplain downstream of Warragamba Dam. The study (SW 1994) reviewed all available flood and rainfall information for the purposes of calibrating and verifying hydrologic and hydraulic models of the catchment and the Hawkesbury River. Hydrologic models (RORB) were set up to estimate runoff into Warragamba Dam and flows from the tributaries downstream of the dam. A hydraulic model (RUBICON) was set up between Camden and Broken Bay. The calibrated and verified models were used to estimate design flood conditions at Penrith, North Richmond and Windsor.

The numerical modelling focussed on the river and extensive floodplains upstream of Sackville, as the prime objective of the study was to address the impact of various options for upgrading Warragamba Dam on the major population centres. Detailed hydraulic examination of the lower reaches of the Hawkesbury River and ocean conditions were outside the scope of the Sydney Water study (SW 1994).

The following information was obtained from Sydney Water and initially incorporated in this study (Appendix D):

- design discharge hydrographs at Sackville;
- design discharge hydrographs for all of the tributaries below Sackville;
- flood travel times between Warragamba Dam and Sackville, and
- simulated flood discharge hydrographs for the 1978 and 1990 historical floods at Sackville and the Colo River.

The Sydney Water study (SW 1994) simulated flood behaviour under the following conditions at Warragamba Dam:

- the dam wall at the present level, that is, incorporating the interim works
- gates operating to H14 procedure, and
- dam full at the commencement of the design flood.

6.2 Topography

Maps at 1:25,000 scale were available for the section of the Lower Hawkesbury River between Sackville and Broken Bay. These maps gave the necessary wide coverage and because of the valley's steep escarpments, were ideal for providing information on important topographic features over the very large study area (e.g. the location of flood storage areas). However for more precise investigations over much smaller areas, such as a specific site, the 10 m contour interval would be a limitation which, at best, can only be considered indicative. The topographic maps would not have sufficient detail for defining the levels and extent of the floodplains accurately. At some locations the 10 m and/or 20 m Australian Height Datum (AHD) contour was the lowest contour plotted on the topographic maps. The 10 m and 20 m contours were digitised to define the extensive terrain in the hydraulic model.

Generally the terrain of the Lower Hawkesbury River falls into one of three categories: the river channel, the floodplain above the river banks and the steep valley escarpment. An inspection of the river was carried out and the average ground levels of the floodplains were estimated in relation to normal tide levels. The extent of the floodplains was checked against measurements from aerial photographs.

Given the size and physical character of the river and floodplain within the study area, as well as the confinement of floodwaters by the gorges, extensive ground survey could not be justified in terms of increased accuracy of the model results. Digitising of the topographic maps was considered appropriate to meet the objectives of this study. Detailed survey of the floodplains should, however, be undertaken to refine the model and improve accuracy when evaluating local flood conditions for individual developments. Any survey data that councils may have collected for site specific developments could be incorporated into the model.

6.3 Bathymetry

Hydrographic surveys of the river have been carried out since the turn of the century. The most complete and detailed survey was carried out between 1978 and 1984 and that survey was used for the construction of the model. Table 6.1 sets out the hydrographic survey details used in this study.

Table 6.1 Hydrographic Survey Details

River	Location	Description	Source	Date	Plan No.
Hawkesbury	Juno Point to Lowlands	199 cross-sections	DPWS	1978-1980	6343
Hawkesbury	Middle Head to Wisemans Ferry	cross-section profiles	DPWS	1983-1984	7630
Cowan Creek	Challenge Head to Bobbin Head	cross-section profiles	DPWS	1984	7636
Coal and Candle Creek, Smiths Creek and Jerusalem Bay	-	cross-section profiles	DPWS	1984	7637
Mooney Mooney Creek	Spectacle Island to upstream of old Pacific Highway bridge	cross-section profiles	DPWS	1985	7638
Berowra Creek	Mongamarra Point to upstream Berowra Waters Ferry	cross-section profiles	DPWS	1984	7635
Mullet Creek	Alison Point to upstream Wondabyne railway station	cross-section profiles	DPWS	1984	7633
Mangrove Creek	Mouth to pump station	cross-section profiles	DPWS	1984	7634
Macdonald River	Mouth to about Bakers Gully	cross-section profiles	DPWS	1983	7602
Colo River	Lower Portland Bridge to Wheeny Creek	cross-section profiles	DPWS	1978	5392

DPWS - Department of Public Works and Services

As part of the investigation of river bed scour in the Hawkesbury River (see Section 9.2.2) AWACS undertook some new bathymetric survey of the river at locations within proximity of the 1978-1984 survey cross-sections. The results were only used to assess the potential for changes in river profile.

6.4 Aerial Photography

Aerial photographs were used to estimate the extent of the Lower Hawkesbury River floodplains. The photographs were taken in August 1986 as part of the DLWC Estuary Management Program.

6.5 Flood and Tide Data

The Lower Hawkesbury River hydraulic model required historical ocean tide levels and flood flows at the downstream and upstream boundaries respectively. Flood levels recorded between Sackville and Brooklyn were used for the calibration and validation of the hydraulic model throughout the study area. Historical data confirmed that flooding due to the combination of river and tributary flows is dominant as far downstream as Wisemans Ferry, whereas in the lower reaches it was mainly the combination of river flows and ocean levels.

6.5.1 Water Level Monitoring Stations

Historical flood levels have been recorded in the Hawkesbury River Valley since early settlement in the 1800s. Most of the flood data collected prior to the 1960s was measured by local observers visually reading flood gauges over the rising and falling stages of the flood. Flood level hydrographs were measured at various locations during the 1978 flood (PW 1979) by timed pegging of the rising and falling flood levels and surveying these later. Debris surveys were undertaken following some floods, including the 1978 flood, to establish the peak flood levels. The floods observed by the local observers formed an important database on the history of flooding. For the Lower Hawkesbury River there was not a substantial number of widespread and accurately measured levels for historical floods, as was available for areas near the major upstream towns such as Penrith, Richmond and Windsor.

To improve the reliability of data gathering, a network of continuous automatic water level stations was established by DPWS along the Hawkesbury River from Castlereagh to the ocean. As well, the Department of Water Resources (DWR), Sydney Water and the Bureau of Meteorology (BOM) also collected continuous and daily read water levels. The Upper Colo, Morans Rock and Macdonald River stations were periodically gauged, thus providing sufficient flow data to generate ratings for these stations. Figure 1.1 shows the location of the stations below Sackville and Table 6.2 lists details of the stations in the Lower Hawkesbury study area. Figure 6.10 shows the flood and tide levels collected by the DPWS stations during the August 1990 flood.

Table 6.2 Gauging Stations

River	Station	Period of Record	Frequency of Recording	Authority
Colo	Upper Colo	1909 - 71 1971 -	Daily Continuous	DWR, BOM, SW
	Morans Rock	1971 -	Continuous	SW
Hawkesbury	Sackville	1962 - 69 1979 -	Daily Continuous	DPWS DPWS
	Lower Portland	1961 - 89 1989 -	Daily Continuous	DPWS DPWS
	Wisemans Ferry	1964 1981 -	Daily Continuous	DPWS DPWS
	Gunderman	1986 -	Continuous	DPWS
	Spencer	1992 -	Continuous	DPWS
	Gentlemans Halt	1986 -	Continuous	DPWS
	Little Patonga	1986 -	Continuous	DPWS
Macdonald	St Albans	1954 - 72 1972 -	Daily Continuous	DWR DWR

SW Sydney Water
BOM Bureau of Meteorology,
DWR Department of Water Resources
DPWS Department of Public Works and Services

To provide estimates of Hawkesbury River flood flows, DPWS set up two water level stations located 1500 metres apart along Sackville Reach to measure the flood profile. Sackville Reach is an ideal monitoring location in that it is a long straight channel of uniform cross-section and it conveys all of the floodwaters draining from the upstream floodplains to the ocean. This has been in operation since 1988, and flood slopes and flows have been measured for some recent floods and tidal flows. Flood flows were also gauged by Sydney Water, at Webbs Creek Ferry (upstream of Wisemans Ferry) in August 1990.

6.5.2 Flood Data

6.5.2.1 Hawkesbury River Flood Level Data

Prior to commencement of this study, a survey conducted amongst local councils and residents to seek additional historical flood level information provided a number of historical flood observations. Appendix A summarises the historical peak flood levels recorded in the Hawkesbury River and tributaries at and below Windsor from 1910 to present. This data was compiled from historical daily read data, continuous water level stations data and some flood observations from local residents and councils. All of the floods measured at the particular locations have been converted to m AHD.

The March 1978 flood was the largest flood event recorded in the Lower Hawkesbury River over the last 30 years. Flood levels for this event were recorded by staff gauge readers, and DPWS pegged and surveyed flood levels during the event and additional levels from flood debris marks. The results of this flood survey are presented in report PW 1979.

Since the commissioning of the continuous water level recorders, six flood events have occurred. The August 1990 flood was the largest measured, having a frequency of occurrence of around 5% AEP. Table 6.3 summarises the flood levels recorded for these six floods. Figure 6.1 shows the flood profiles for these events together with the 1978 and pegged 1867 flood levels. The origin of the data on the 1867 flood is unclear.

Table 6.3 Recorded Peak Flood Levels by Continuous Stations

Date	Windsor (m AHD)	Sackville (m AHD)	Lower Portland (m AHD)	Webbs Creek (m AHD)	Gunderman (m AHD)	Gentlemans Halt (m AHD)
July 88	10.96	NR	5.85	2.78	NR	NR
Apr 89	NR	5.36	4.55	2.14	NR	NR
Feb 90	7.69	4.59	NR	1.97	1.48	NR
Apr 90	8.72	5.65	5.1	2.58	1.61	1.48
Aug 90	13.46	9.97	7.46	**4.30	2.73	1.45
Feb 92	10.82	*7.51	5.77	NR	NR	NR

NR No record
* Flood flows gauged by DPWS
** Flood flows gauged by Sydney Water

6.5.2.2 Colo River Flood Data

A gauging station at Upper Colo has been maintained by various authorities from 1909 to the present, with the exception of the period between 1933 and 1942. The former DWR managed the station until 1970, whereupon numerous gaugings were undertaken on routine servicing visits and on specific flood events. The highest flow gauging was undertaken at gauge height 7.4 metres. The DWR developed a rating for this station extrapolating the curve to 15 metres, as shown in Figure 6.2.

Sydney Water gained responsibility for the station in 1970 and has managed this station since. The highest flow measured by Sydney Water was at a gauge height of 11.8 metres and a rating was developed and extrapolated to the 1978 flood gauge level of 19.2 metres (RL 20.7 metres AHD) as shown in Figure 6.2.

Since 1970 BOM has also maintained a gauge at Upper Colo, approximately 100 metres further upstream, as part of their flood warning system. As shown in Figure 6.3 the correlation between BOM and Sydney Water data sets is reasonable and for the occasions when the Sydney Water gauge was inoperative, the BOM peak height flood levels could be adopted.

6.5.2.3 Macdonald River Flood Data

DLWC (formerly DWR) has operated a gauging station on the Macdonald River at St Albans from 1955 to the present, for which gauge heights, flow records and a rating table could be obtained. The gauge heights were converted to flows using this rating table. Table B2 details the annual series.

6.5.2.4 Review of Upper Colo River Ratings

Figure 6.2 shows both the DWR and the Sydney Water rating curves and the level of the highest flood gauging. Both rating curves could only be considered accurate to the highest flood gauged flows. Any estimation of flood flows above these levels would be based on an extrapolation of those rating curves by these agencies. In particular, the estimation of the Colo River 1978 flood discharge and the design flood discharges, used later in this study, depended very much on the validity of the rating curve extensions.

It was considered that the discharges from the Sydney Water Upper Colo rating curve are overestimated because it suggests that major Colo flood flows at this station are similar in magnitude to major Hawkesbury River flows at Sackville. Given that the catchment area at Sackville is three times that at Upper Colo, major flood flows from the two separate river systems are unlikely to be similar.

To examine this more closely, an independent check on the validity of the rating curve extrapolation was undertaken by both AWACS and DLWC. Two methods were applied:

- a theoretical approach considering waterway characteristics of the river, and
- an approach which involved setting up a one-dimensional numerical model of the upper reaches of the Colo River.

Method 1

The theoretical approach firstly considered the waterway characteristics of the Upper Colo River. Cross-sectional details of the Upper Colo River gauging station were obtained from Sydney Water. Figure 6.4 shows a plot of discharge versus stage and $AR^{2/3}$ versus stage for the gauging site. As can be noted from the plots, the two lines are roughly parallel up to a discharge of about 1,000 m^3/s.

Using Figure 6.4 and Manning's equation, Figure 6.5 was derived by equating $n/S^{1/2}$ and relating this to stage. In Figure 6.5, the section of the curve below the highest gauging of 11.8 m shows an upward linear trend. However, for the section of the curve above the highest gauging there is a marked downward trend. There are no physical reasons why this change should occur. It is considered that the trend observed for the gauged portion of Figure 6.5 should extend to the ungauged portion. Based on this assumption a new rating curve was developed as shown in Figure 6.6. With this curve, the 1978 peak gauge height of 19.2 m represents a lower estimated discharge of 3,800 m^3/s rather than the 5,800 m^3/s estimated from the Sydney Water rating table.

Method 2

A one-dimensional numerical hydrodynamic model (MIKE-11) was set up to simulate the Upper Colo River hydrodynamics. Cross-section details were available for the Colo River from the Hawkesbury River confluence up to the Putty Road bridge, near the Morans Rock gauging station. Upstream of Putty Road bridge the Sydney Water Upper Colo gauging station cross-section was adopted as representative of the channel up to a point upstream of the Upper Colo gauging station. The bed slope and Manning's 'n' coefficient in the model were adjusted to calibrate the model. The model was calibrated by reproducing the differences in measured flood levels recorded during the 1978 flood between Upper Colo and Morans Rock. The calibrated model reproduced a similar rating at Upper Colo to that derived from Method 1.

These investigations provide a valid basis to use a revised rating curve, with this study proceeding on the basis of the extrapolation shown at Figure 6.6. The use of the extrapolated rating curve was agreed to by DLWC.

6.5.2.4.1 Flood Frequency Analysis of the Upper Colo River

The assessment of the historical flood data indicated that there was sufficient data available for the Colo River at Upper Colo to undertake an annual series flood frequency analysis. For the annual series flood frequency analysis the AWACS extrapolation curve was adopted to assign flows to the recorded levels. Appendix A details historical flood levels for the Colo River at Upper Colo.

Table B1 lists the annual series of peak gauge heights and discharges for the Colo River at Upper Colo using AWACS' extrapolation. As there were no flood records between 1934 and 1941, a comparison of the Windsor flood records gave an indication that this was a relatively flood-free period. It was assumed that only small floods occurred during these years of missing data. Therefore, the annual flood frequency analysis was undertaken for the records between 1909 and 1991, applying the guidelines for flood frequency analysis in AR&R (1987) and assuming very low flood flows between 1934 and 1941.

Figure 6.7 shows the annual series flood data plotted with the line of best fit using a Log Pearson Type III distribution as recommended in AR&R (1987). The Log Pearson Type III distribution was fitted to the top 72 of the 83 flood events. The results of the flood frequency analysis and estimation of confidence limits are shown below in Table 6.4. The confidence intervals give the range within which the actual distribution is expected to lie with a selected level of probability and is enclosed by confidence limits.

Table 6.4 Colo River at Upper Colo Flood Frequency Analysis

Frequency	5% confidence limit (m^3/s)	Discharge (m^3/s)	95% confidence limit (m^3/s)
20% AEP	1500	1200	1000
5% AEP	3300	2500	1900
2% AEP	5000	3400	2300
1% AEP	6400	4000	2500
0.5% AEP	8300	4700	2700

6.5.2.5 Flood Frequency Analysis of Macdonald River at St Albans

Based on the flood data presented in Appendix B (Table B2), an annual series flood frequency analysis was undertaken for the Macdonald River at St Albans.

There was no flood data for the years 1956 and 1957. An examination of the Colo River flood data suggests there may have been a significant flood event in 1956. Without any other supporting data on the magnitude of such a flood, it was decided to assume that only small floods occurred during 1956 and 1957 for the purposes of this study. Figure 6.8 shows the frequency curve plotted from this data using a Log Pearson Type III distribution as recommended in AR&R (1987). The Log Pearson Type III distribution was fitted to the top 21 of the 36 floods. The results of this flood frequency analysis indicate the following peak discharges and frequencies.

Table 6.5 Macdonald River at St Albans Flood Frequency Analysis

Frequency	Discharge (m^3/s)
50% AEP	170
20% AEP	430
10% AEP	600
5% AEP	740
2% AEP	950
1% AEP	1100
0.5% AEP	1300

6.5.3 Tide Data

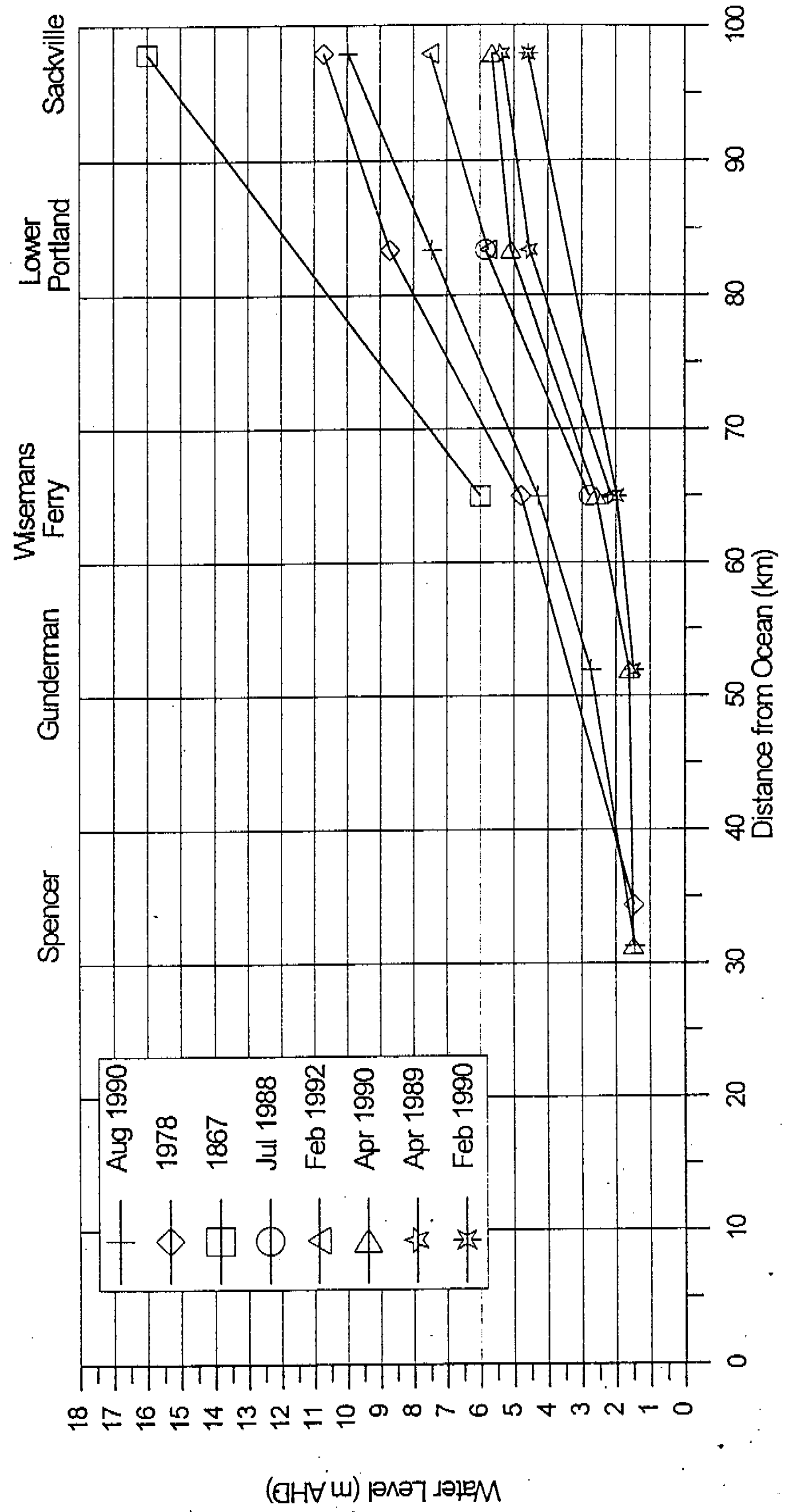
The Hawkesbury River flood flow interacts with the prevailing ocean tide at Broken Bay. The two processes can interact for some distance upstream of Broken Bay depending on the magnitude and phasing of the flood and tide. The August 1990 flood is the largest event comprehensively monitored by the continuous water level recorders in the Lower Hawkesbury from Brooklyn to Windsor. During that event, the tides were in a spring cycle with evidence of a storm surge. The influence of the ocean tide, as shown in Figure 6.9, was experienced as far upstream as Wisemans Ferry. During major flood events the magnitude and phasing of the tide and storm surge will be critical in determining flood conditions in the lower reaches of the river, while during relatively minor flood events, the tide may even dominate.

Previous investigations undertaken by AWACS (1991) showed a good correlation between Fort Denison and a tide recorder deployed at Newport, Pittwater. These investigations concluded that the Sydney historical tidal database could be adopted for Broken Bay, therefore any ocean water level either used for calibration or design conditions could be applied from Fort Denison Sydney data.

Tide levels have been accurately and continuously monitored at Fort Denison Sydney since 1914 with the May 1974 tide and associated surge being the highest on record (1.48 m AHD). Investigations undertaken by Foster et al (1975) reported water levels of 2.4 metres ISLW (1.48 m AHD) were also measured in Brisbane Water. A ranked listing of the highest 10 tide levels measured at Sydney are shown in Table 6.6.

Table 6.6 Highest Recorded Tide Levels at Fort Denison (1914-1990)

Peak Level (m AHD)	Date
1.48	May 1974
1.43	April 1990
1.40	June 1956
1.35	June 1984
1.34	July 1964
1.32	July 1978
1.31	August 1921
1.30	December 1950
1.29	June 1947
1.28	June 1973



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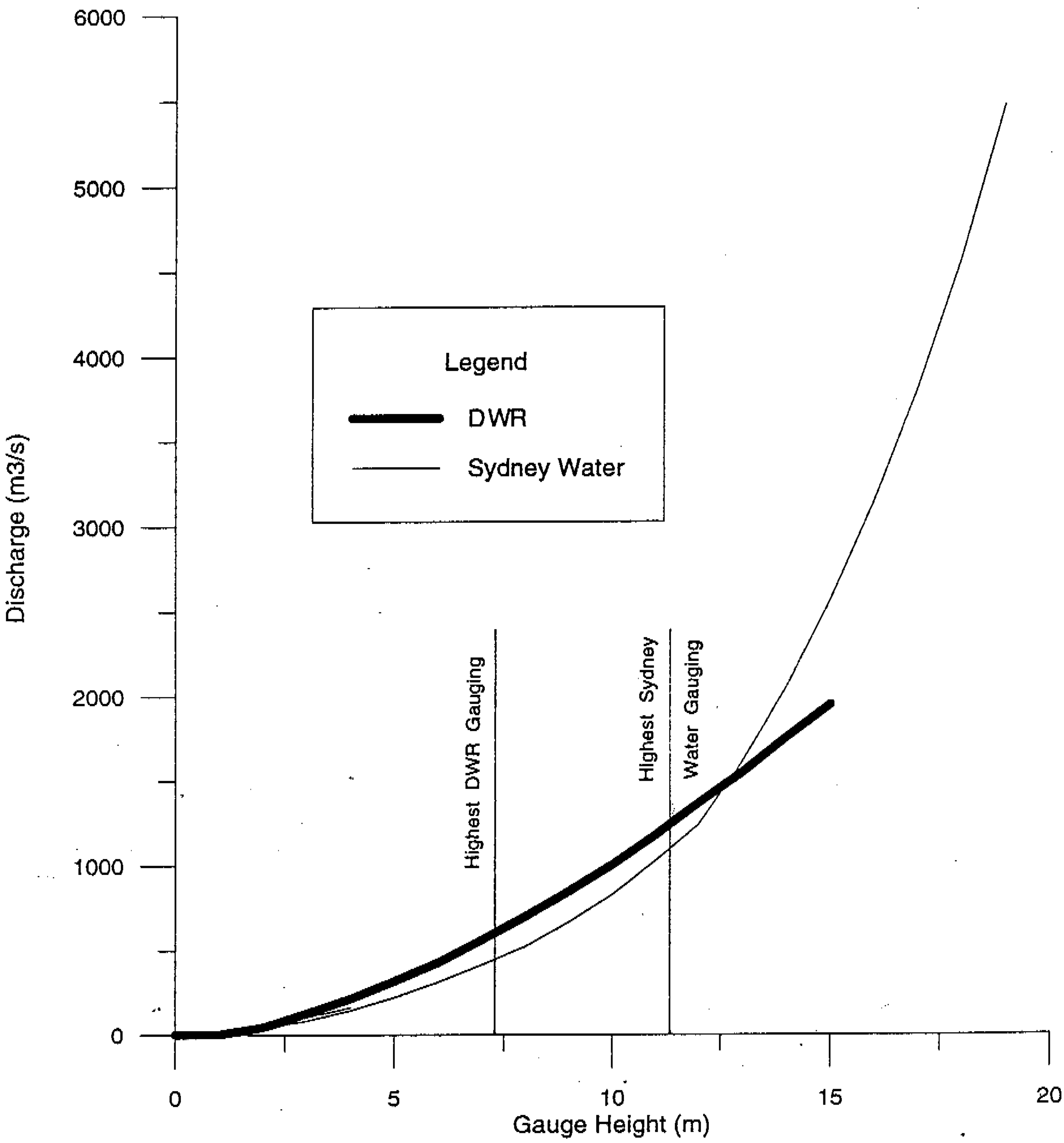
HISTORICAL FLOOD PROFILES

Figure 6.1



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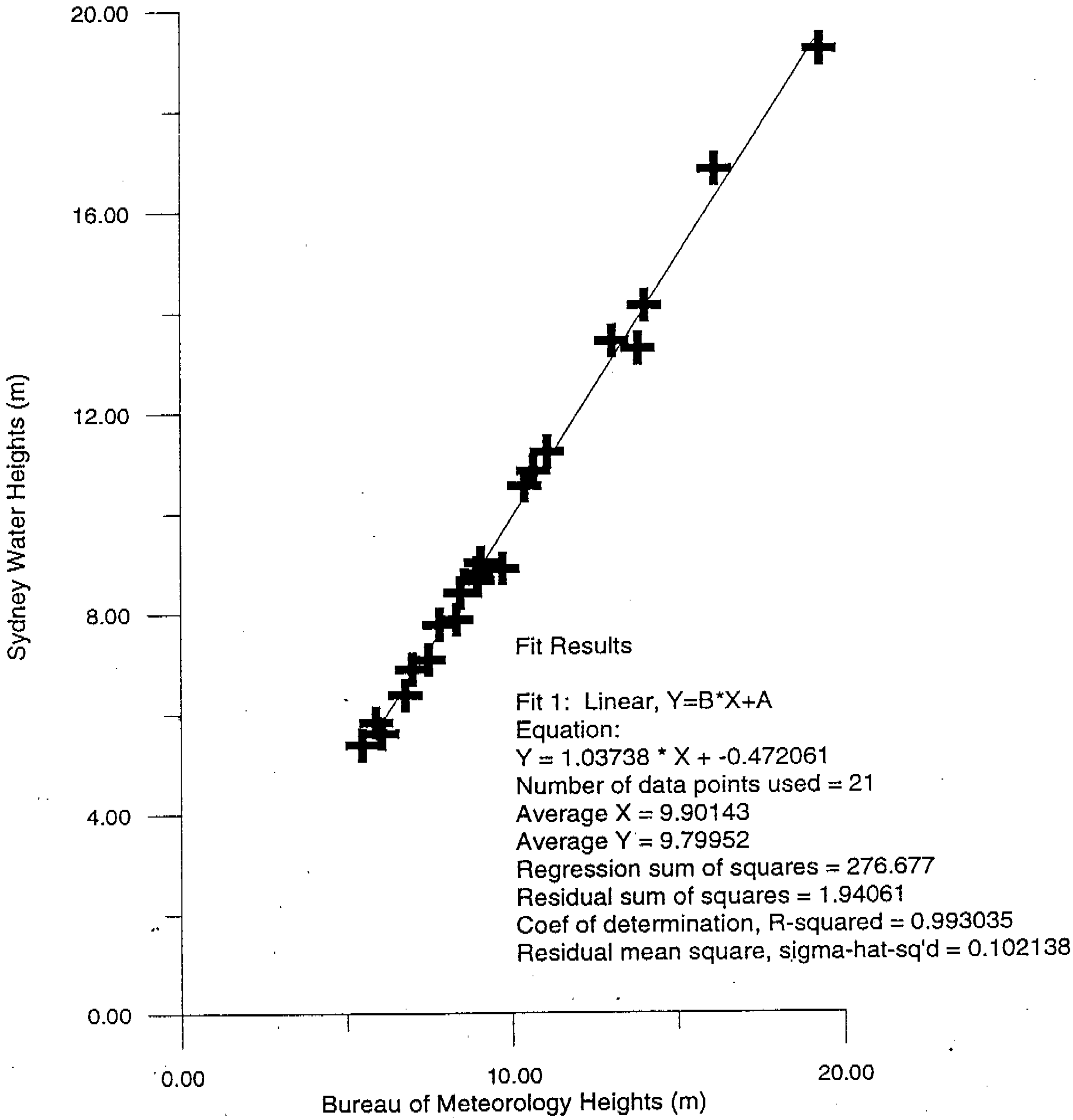
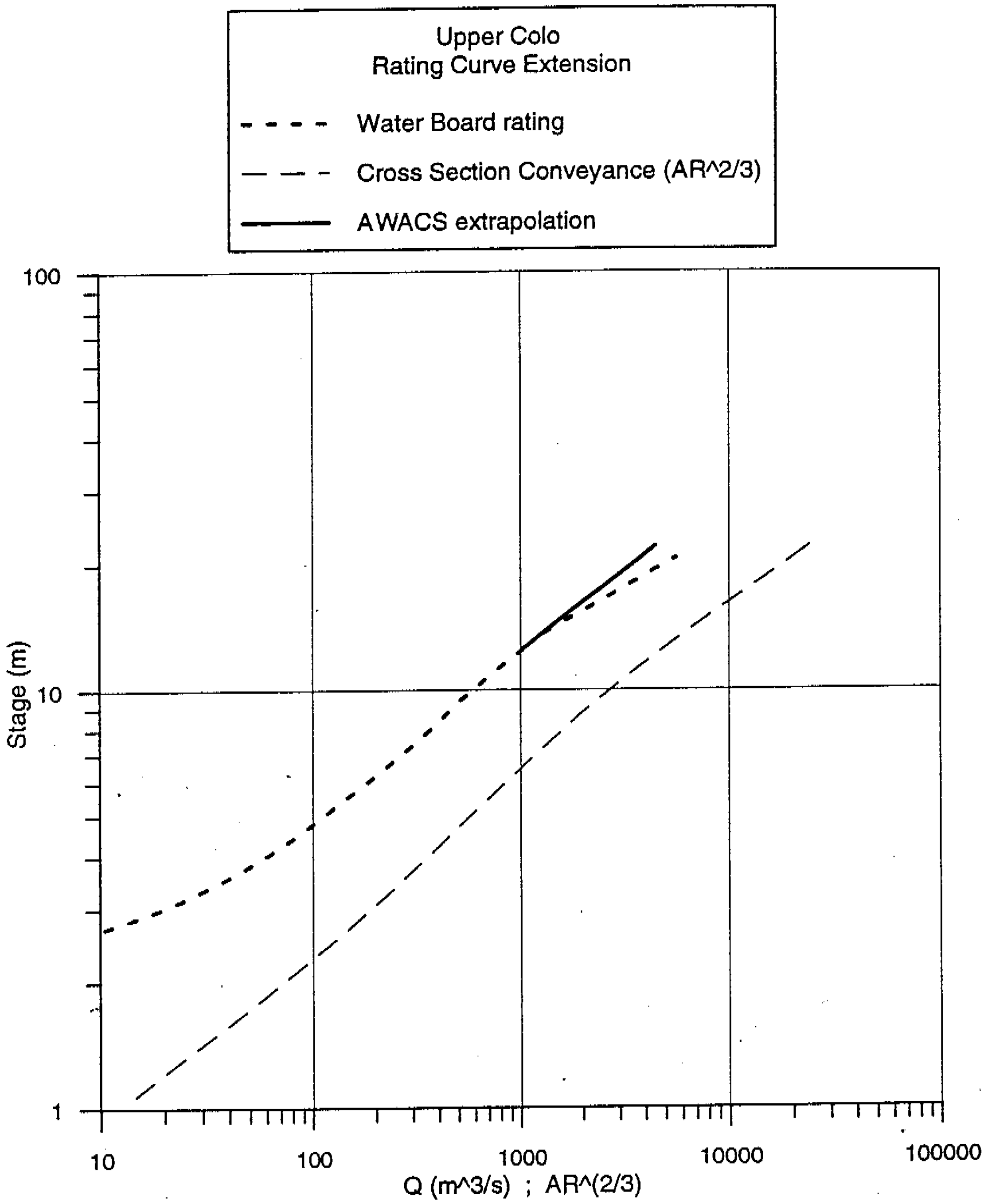
COLO RIVER AT UPPER COLO
RATING CURVES

Figure
6.2



AWACS Report No. 94/29	COLO RIVER AT UPPER COLO GAUGE HEIGHTS 1972 - 1990	Figure 6.3



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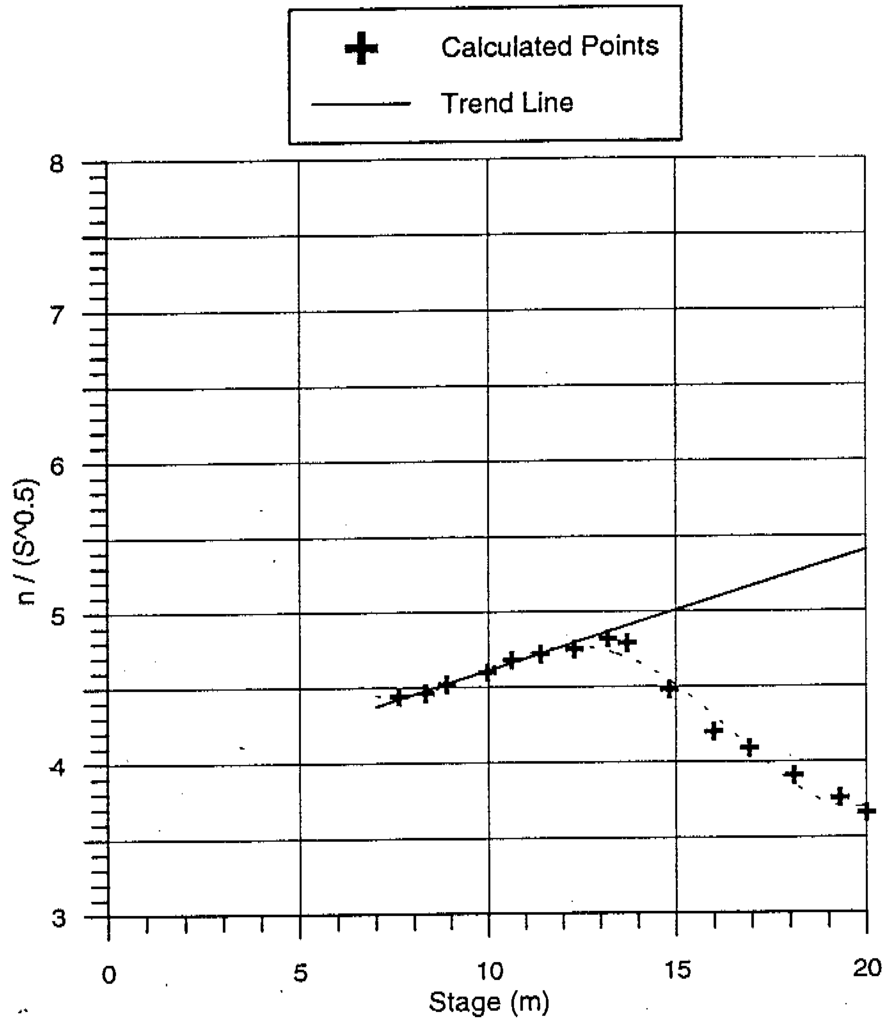
INVESTIGATION OF UPPER COLO RATING
Q AND $AR^{2/3}$ VERSUS STAGE

Figure
6.4



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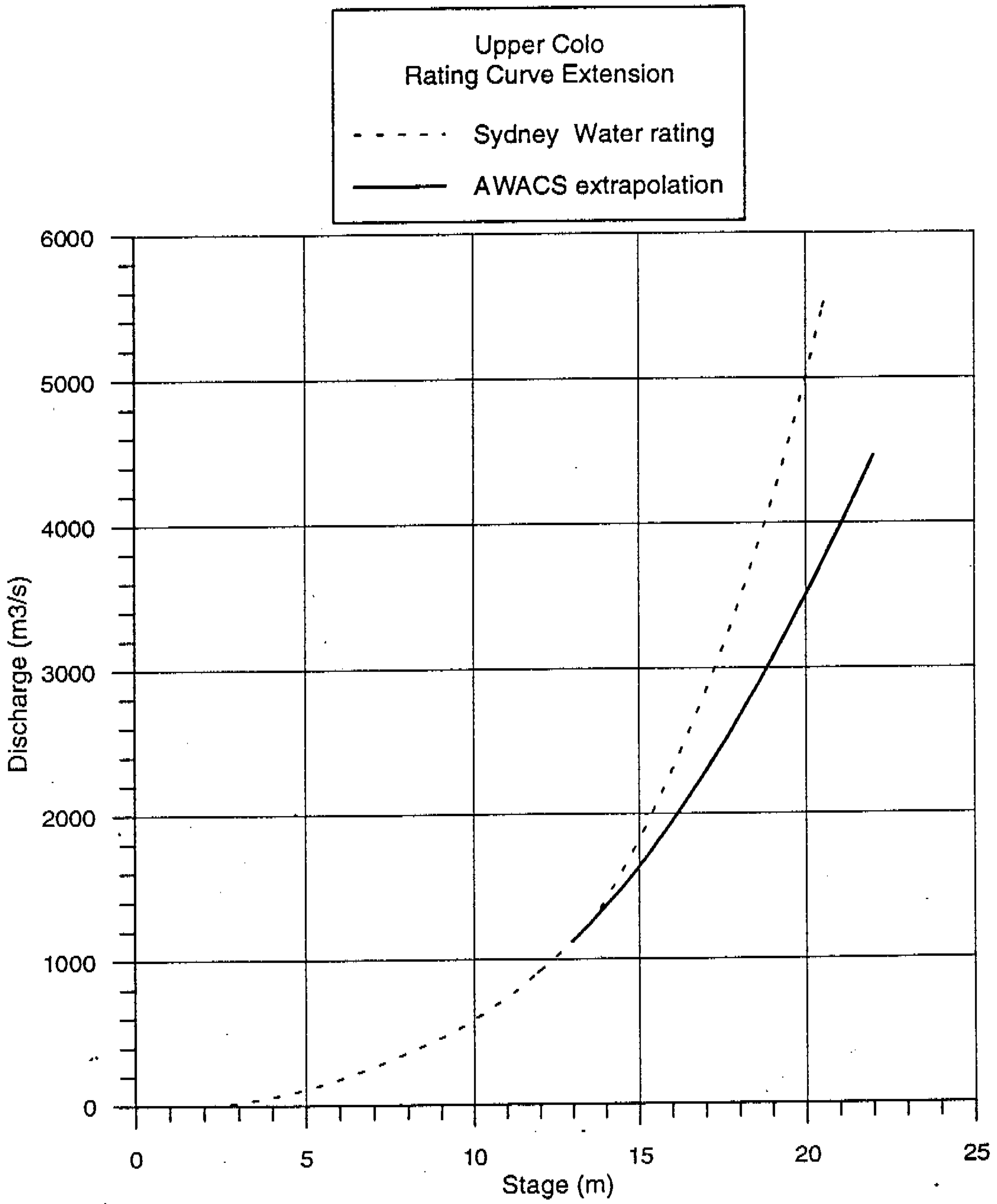
INVESTIGATION OF UPPER COLO RATING
$n/S^{0.5}$ TREND

Figure
6.5



AWACS
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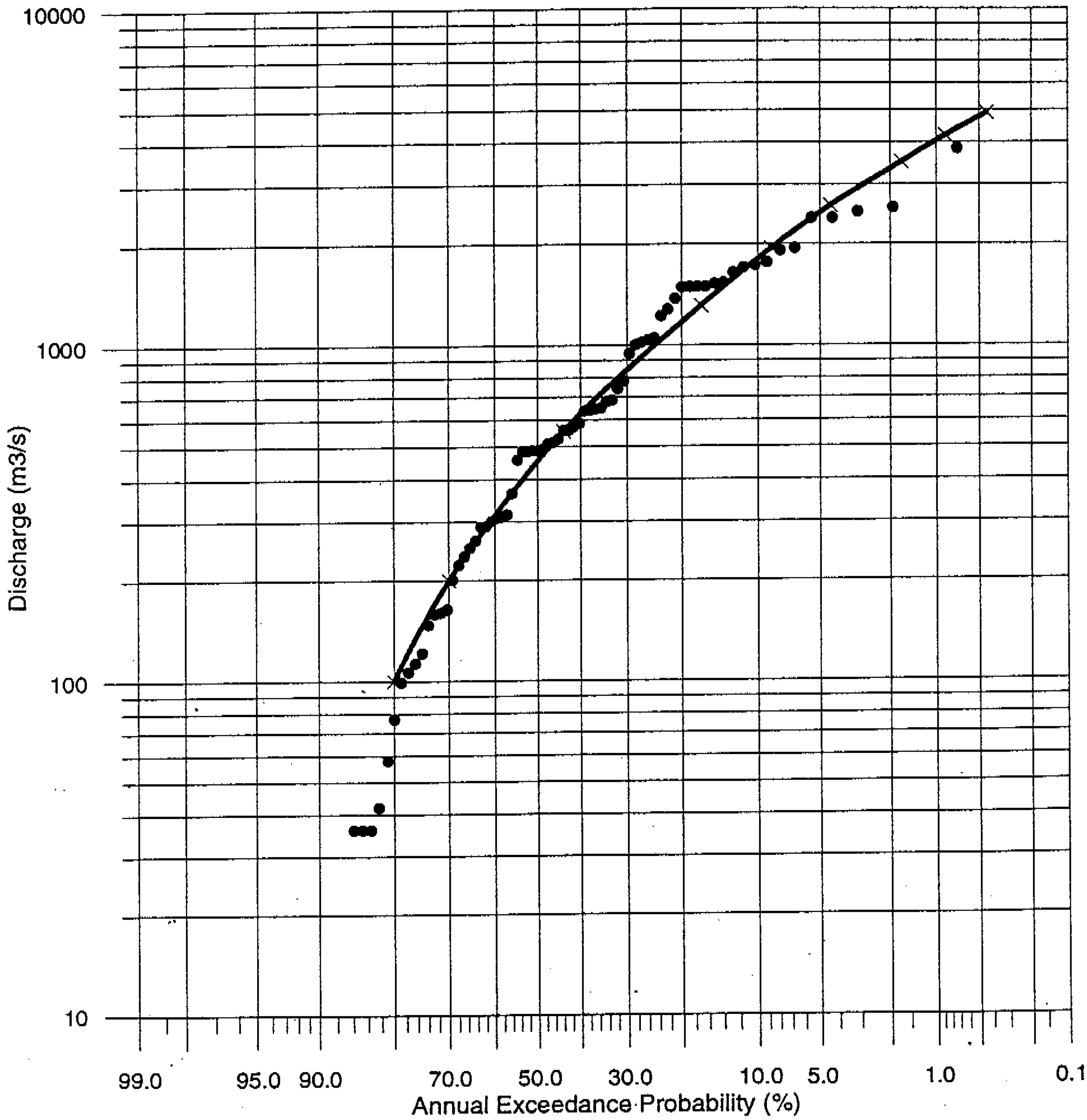
INVESTIGATION OF UPPER COLO RATING
PROPOSED NEW EXTRAPOLATION

Figure
6.6



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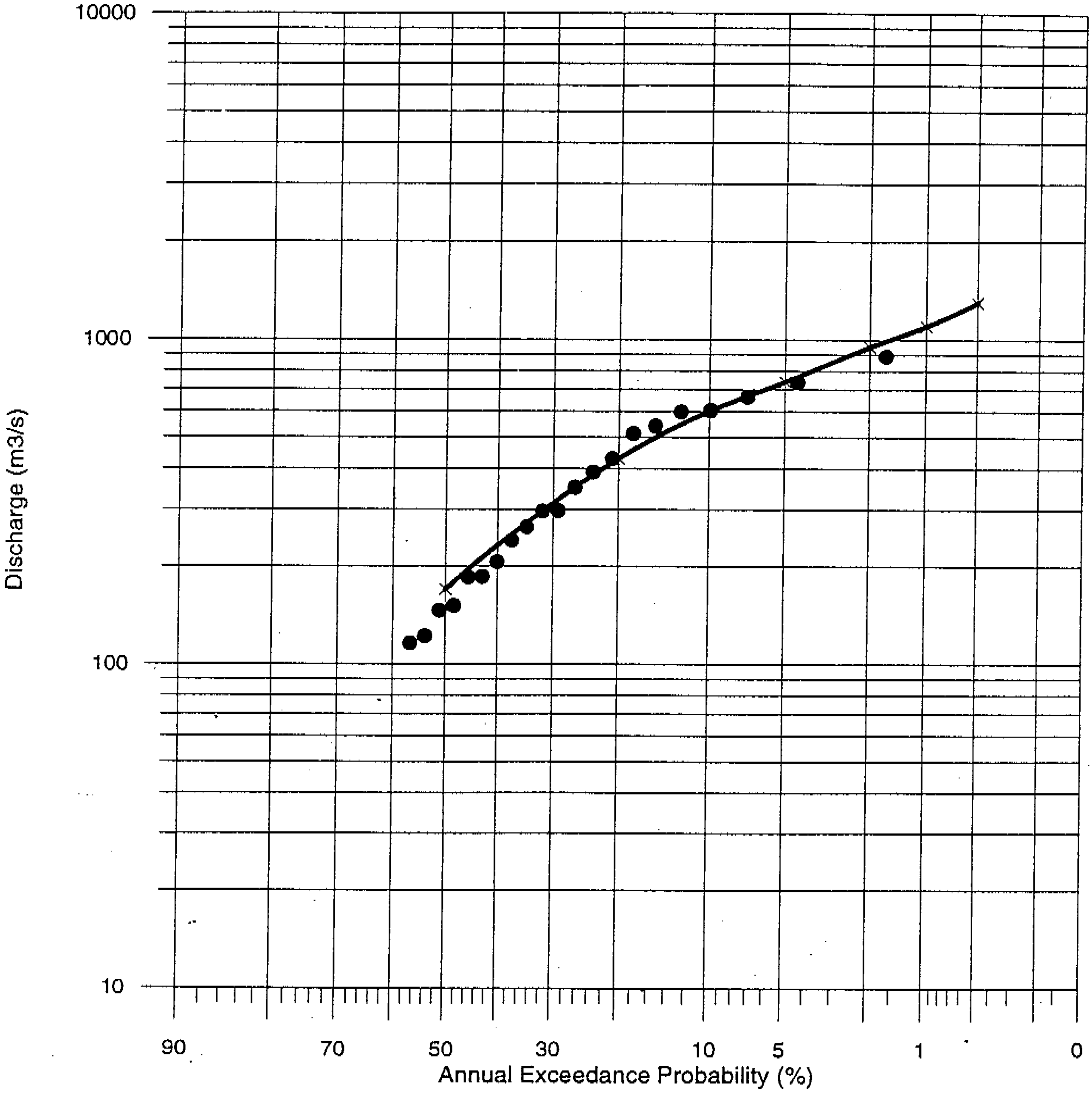
COLO RIVER AT UPPER COLO
FLOOD FREQUENCY CURVE

Figure
6.7



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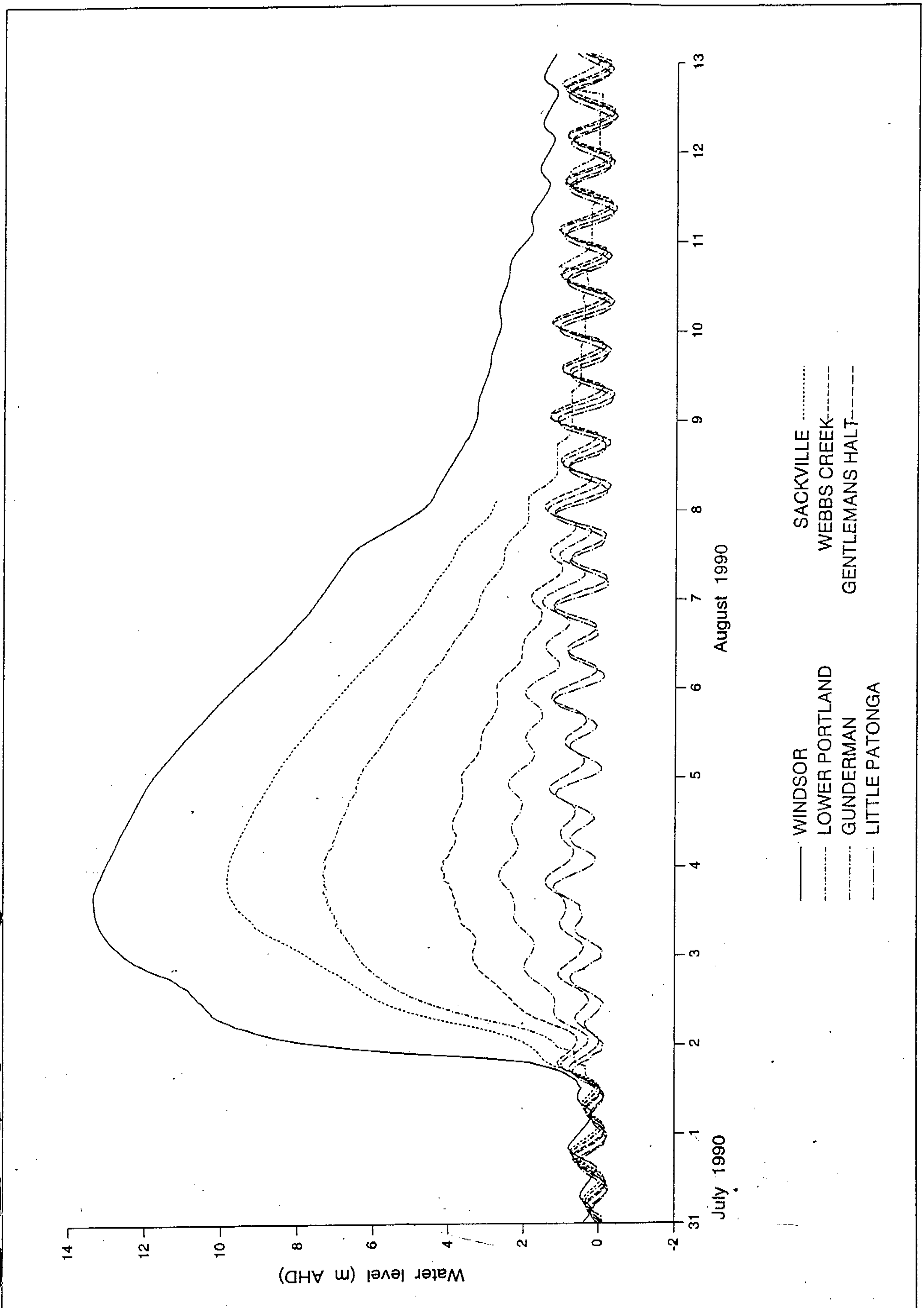
MACDONALD RIVER AT ST ALBANS
FLOOD FREQUENCY CURVE

Figure
6.8



AWACS Report No. 94/29	AUGUST 1990 STAGE HYDROGRAPHS	Figure 6.9

7. Hydrodynamic Model Calibration and Verification

7.1 Model Description

A two-dimensional hydraulic numerical model (RMA-2) was set up to simulate the complex hydrodynamic behaviour of the Lower Hawkesbury River and its tributaries from Sackville to Broken Bay. The behaviour of the river in a flood depends on the combination of the rainfall runoff hydrographs, river storage effects and ocean water levels. Each of these processes is dynamic in time and can act individually or in some combination. The hydraulic model selected needed to be able to simulate all these conditions and be capable of modelling future flood management options.

The following features outline the relevant characteristics of the model:

- the model is two-dimensional
- velocities are depth averaged
- friction losses are based on Manning's 'n' formula
- the model is schematised by a network of triangles and quadrilaterals
- within each element of the model the velocity and stage characteristics are calculated by quadratic approximations, and
- the solution is time dependent.

7.2 Hydraulic Model Schematisation

A detailed hydrosurvey of the Hawkesbury River was undertaken between 1979 and 1984 with cross-sections of the river approximately every one kilometre (Table 6.1 details the available survey plans). This survey defined the bed levels at the time of survey, however, whether the bed was in a state of scour or accretion could not be determined from this survey alone. This survey was the only comprehensive data available along the river and was therefore adopted and digitised for the hydraulic model schematisation. The hydrosurvey levels are approximately zero AHD.

Survey details above the normal water level were digitised into the model from the 1:25,000 topographic maps. The topographic maps detailed the 10 m and 20 m AHD contour levels. Details of the floodplain between the water level and the 10 m contour were determined by measurements from aerial photographs (areas) and by estimating floodplain levels from field inspections in relation to tide levels (see Section 6.2). The accuracy obtained from this method was suitable for the flood study, however, if during any later floodplain management studies, site specific localised flow conditions are required, accurate levels on the floodplain would need to be incorporated into the model for determining detailed flow characteristics for that area.

Figure 7.1 shows the layout, mesh and boundaries adopted for the numerical model. There are 10 boundary conditions in the model including runoff from tributaries, Hawkesbury River inflows at Sackville and ocean tide levels.

The flood profiles presented in this report required chainages to be assigned to the major locations. The following chainages were used in this study and are based on the hydrosurvey detailed in Section 6.3 with zero chainage at Barrenjoey Head.

Sackville (Sackville Ferry)	97.88 km
Dargle	88.53 km
Doyles Drain	87.99 km
Lower Portland	83.37 km
Wisemans Ferry (Webbs Creek Ferry)	64.97 km
Wisemans Ferry (Wisemans Ferry)	63.50 km
Gunderman Gauge	51.94 km
Spencer	34.34 km
Gentlemans Halt Gauge	31.29 km
Brooklyn	13.36 km

7.3 Hydraulic Model Calibration and Verification

Confidence in the model to accurately simulate design flood conditions is dependent on both adequate and accurate historical flood levels and flow data and the ability of the model to simulate these flood conditions, particularly levels and corresponding times. Floods monitored in 1990 and 1978 were selected for the calibration and verification of the model respectively, as these were the only significant floods comprehensively monitored in the Lower Hawkesbury and Colo Rivers.

7.3.1 Model Calibration - August 1990 Flood

The August 1990 flood in the Hawkesbury River (estimated at about a 5% AEP event at Windsor) was the only significant event where the continuous water level recorders measured the passage of the flood down the Hawkesbury River and its interaction with the tributary inflows and the ocean tide. The August 1990 flood event was also used for verifying the model used in the Sydney Water study (SW 1994). The flow hydrographs used as inputs at Sackville were derived from the Sydney Water study.

The Sydney Water study (SW 1994) simulated discharge hydrographs at Sackville were initially used in this study for the calibration of the hydraulic model. Figure 6.9 shows the recorded flood level hydrographs for this event at the seven gauging station locations. The recorded peak flood level at Sackville was 9.97 m AHD and according to the Sydney Water study, this corresponds to a peak discharge of 6,150 m^3/s.

The Upper Colo discharge hydrograph was originally based on that supplied by Sydney Water, however, as discussed in Section 6.5.2.4 an extrapolated rating curve was revised by AWACS for the station. The estimated peak discharge from the Colo River was estimated to be 2,300 m^3/s using the AWACS extrapolation.

After adjusting the Manning's 'n' coefficient within realistic values, the hydraulic model using the Sydney Water estimated hydrograph at Sackville still resulted in flood levels more than 1 m higher than those recorded. Bed scour in the Hawkesbury River was considered as a possible cause for this inability of the hydraulic model to reproduce the historical flood levels. However, the results could not be improved significantly by incorporating reasonable bed scour in the model. Therefore, the feasibility of lowering the simulated Sydney Water flows was investigated.

Figure 7.2 shows the DPWS gaugings at Sackville (see Section 6.5.1) together with the flows and water levels estimated in the Sydney Water study (SW 1994) for the floods of 1990 and 1978. Following the same procedure as used on the Colo River (Section 6.5.2.4 Method 1), a theoretical $AR^{2/3}$ curve was plotted through the DPWS gaugings. This theoretical approach suggests that the discharges estimated by the Sydney Water study for the 1990 and 1978 floods may have been overestimated.

Another check on the Sydney Water estimated discharges showed that the model results using the Sydney Water flows overestimated the discharge at Webbs Creek Ferry when compared to the gaugings measured by Sydney Water for the 1990 flood (Figure 7.3). Based on the likely overestimation of discharges shown in Figures 7.2 and 7.3, AWACS reduced the Sydney Water discharge hydrographs by 15%. As can be seen in Figure 7.3 the model results using the reduced discharge hydrograph produced a better match with the recorded data.

With the new estimated Sackville hydrograph and the following Manning's 'n' coefficients, the simulated hydrographs successfully reproduced the recorded level hydrographs at Sackville, Lower Portland, Webbs Creek, Gunderman and Gentlemans Halt (see Figure 7.4).

Manning's 'n' coefficients adopted were:

- within bank (about Lower Portland) = 0.026
- within bank (about Lower Portland to Half Moon Bend) = 0.020
- within bank (downstream of Half Moon Bend) = 0.018
- floodplain (throughout) = 0.038

Table 7.1 and Figure 7.5 show the simulated and recorded peak water levels for the 1990 event.

Table 7.1 Peak Water Levels - Calibration of 1990 Flood Event

Location	Recorded (m AHD)	Simulated (m AHD)	Difference m
Sackville	9.97	9.78	-0.19
Lower Portland	7.46	7.50	+0.04
Webbs Creek	4.30	4.45	+0.15
Gunderman	2.73	3.19	+0.46
Gentlemans Halt	1.46	1.35	-0.11

It is not unusual for fully two-dimensional models to have different values of roughness when compared against the one-dimensional models, such as the RUBICON model used in the Sydney Water study (SW 1994). The energy losses from abrupt changes to the channel and flow paths, such as sharp bends in the river, usually result in higher Manning's 'n' coefficients in the one dimensional models. In the lower reaches of the Hawkesbury River, these differences would be accentuated because of the relatively high degree of sinuosity of the river and the narrowness of the floodplain which confines the bulk of the flood flows to within the channel. This is particularly relevant along the river between Sackville and Bathurst reaches (near Half Moon Bend).

7.3.2 Model Verification - March 1978 Flood

The March 1978 flood levels on the Hawkesbury River are estimated to be around a 3% AEP flood event at Windsor and a 1% AEP flood at the Colo River. This event was significant in the Hawkesbury because of the magnitude of the flows from the Colo River. The Colo River flood peaked at about 3,800 m^3/s, and in the Hawkesbury River at Sackville the flood peak was about 6,100 m^3/s. Both of these estimated peak discharges are based on the adjusted AWACS flows.

Using the revised inflow hydrographs and the Manning's 'n' coefficient derived in the calibration of the 1990 event, the model was run and the result compared to recorded data. Figure 7.6 shows the recorded and simulated water levels at Sackville, Doyles Drain, Webbs Creek and Spencer for the flood event and Figure 7.7 shows the simulated and recorded peak water levels.

The following comments are made on the verification results:

- The timing of the simulated hydrograph peaks (Figure 7.6) along the Hawkesbury River are generally good except at Webbs Creek Ferry. The recorded Webbs Creek Ferry flood hydrograph (PW 1979) shows considerable variation around the peak and does not record the time of the peak.

- The 1978 flood levels were recorded by manual gauge readings and manual pegging of flood levels during the event. Data collected by this manner would be less accurate than collected by the continuous recorders.

- The 1978 flood was significantly different in nature to the 1990 calibration flood. The Colo River significantly impacted on the behaviour of the 1978 flood, whereas the 1990 flood and most other major floods in the Hawkesbury are mainly impacted by Hawkesbury River flooding.

- Considering the magnitude of the flows, the sensitivity of the model to changes in discharge and the method of estimating the inflow hydrograph, the 1978 flood verification confirmed that the adopted roughness and new rating curves for Sackville and Upper Colo were justified.

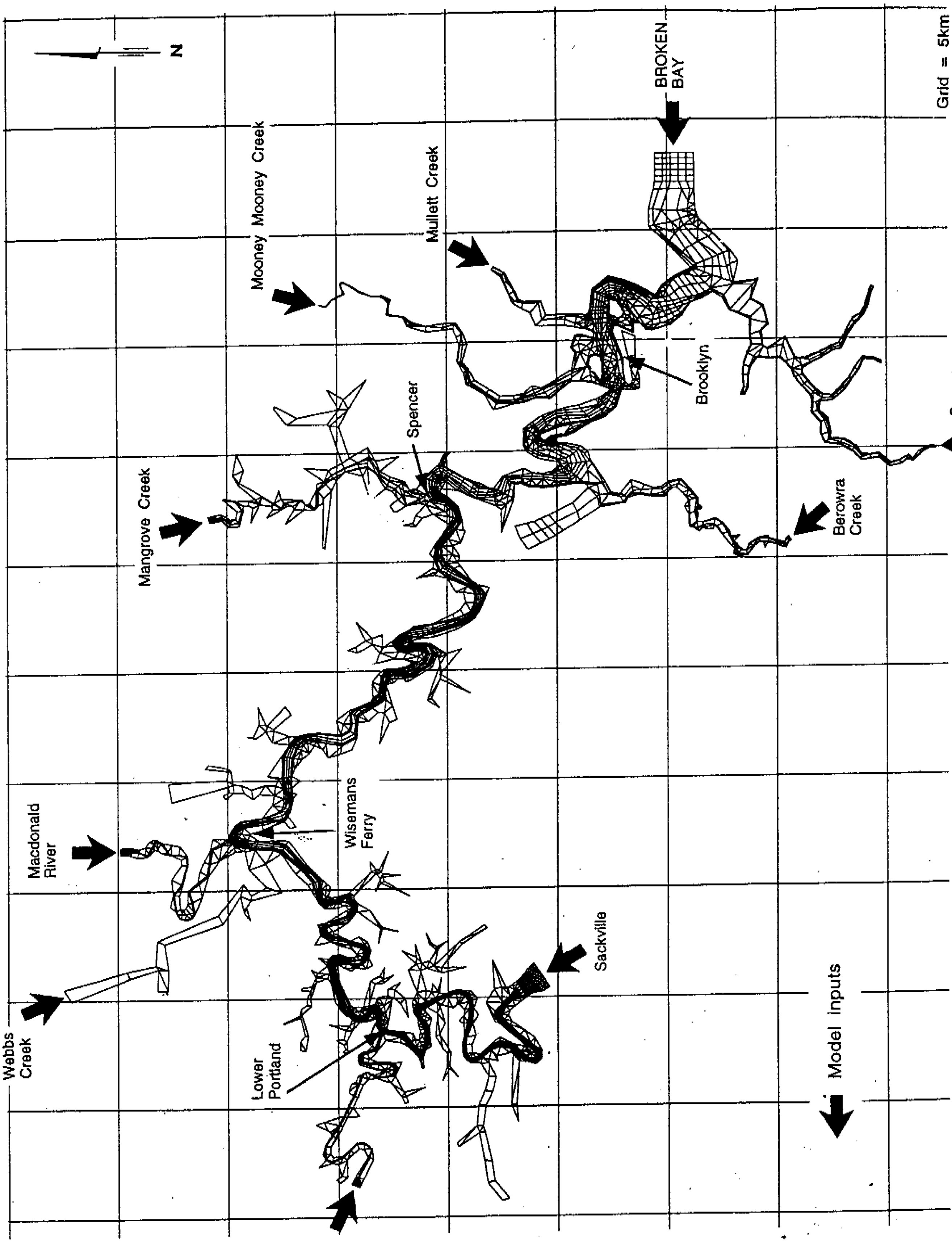
7.3.3 Discussion of Model Calibration and Verification

The results of the hydraulic model calibration and verification process highlighted the importance of data accuracy and estimated flows at Sackville and Upper Colo. It also demonstrated that flood behaviour in the Lower Hawkesbury can vary significantly from one event to another.

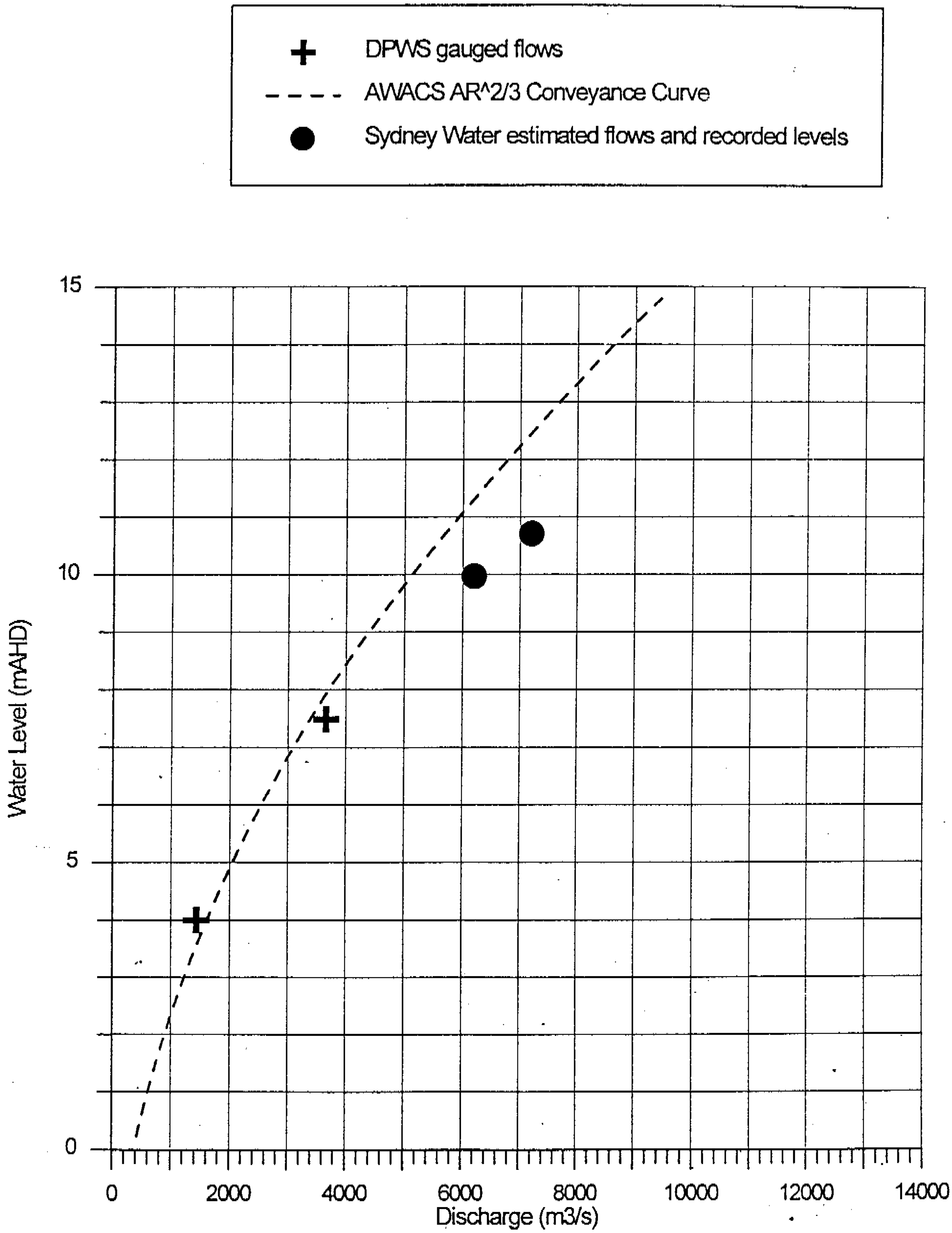
As a result of the findings described in Section 7.3.1, in regard to the difference in flow estimates, Sydney Water's Warragamba Dam EIS consultants, Webb McKeown and Associates (WMA) undertook further investigations to re-examine the hydraulic modelling around the interface of the two models. The Sydney Water model was established to provide flood information on the more developed floodplains upstream of Pitt Town and Wilberforce, whereas this study focused in greater detail on the areas downstream of Sackville. Given the different objectives, it was recognised that there would be differences in the approach of each study, the level of attention given to investigating various areas, as well as in the interpretation and application of the available flood data.

The major finding from the WMA study was that cross-sectional information used in their RUBICON model underestimated the storage volume in the tributary creeks between Pitt Town and Sackville, especially in Long Neck Lagoon, Cattai Creek and Little Cattai Creek. The distribution of additional storage volume over these areas resulted in a reduction of flows downstream. However, it was necessary to increase the main channel roughness, to compensate for a reduction in flood levels and hence restore the model calibration. The reduction in flow for the August 1990 flood was 9% and for the March 1978 flood 8%.

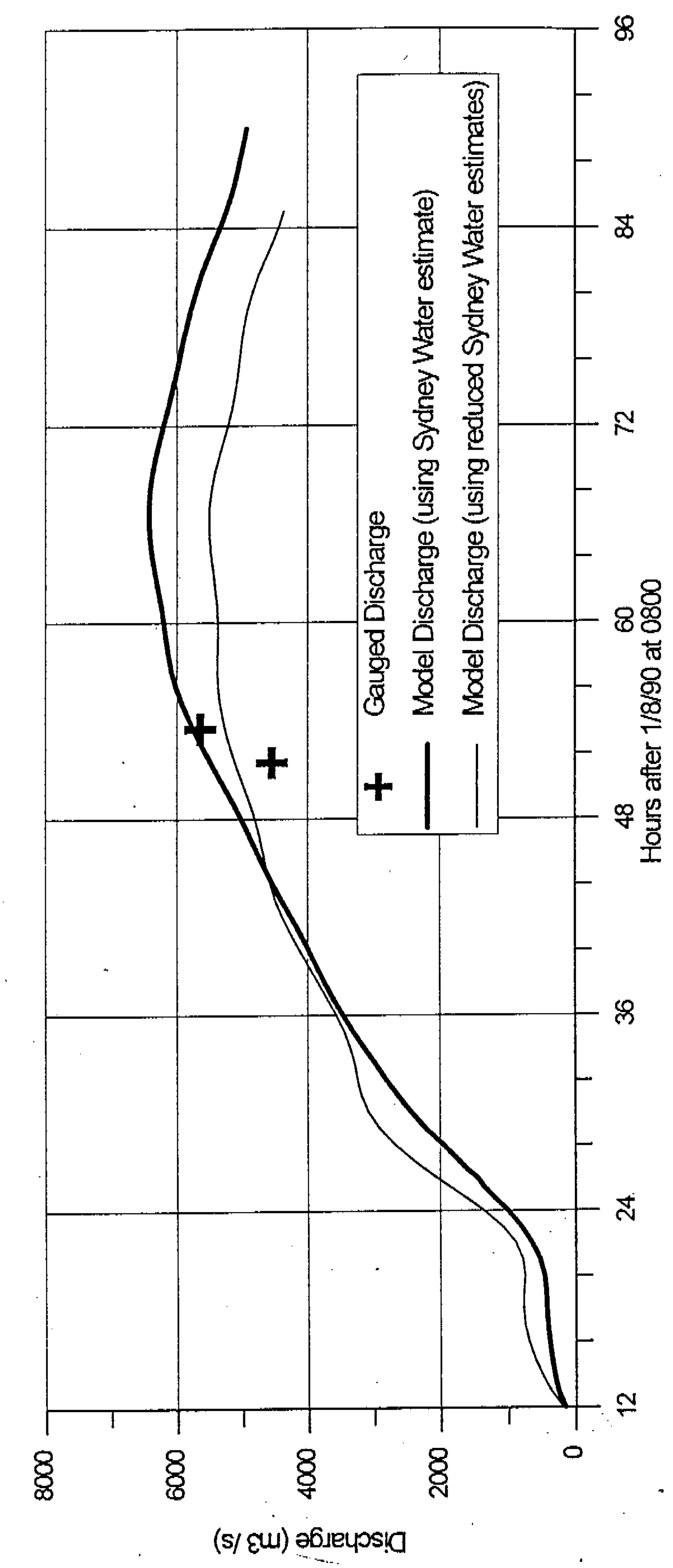
In this study the model calibration of the August 1990 flood and model verification using the March 1978 flood required the estimated Sydney Water flows at Sackville to be reduced to achieve calibration. The reduction in this study's flows when compared to the WMA study results above shows a net difference in the estimated flows of about 6%. Considering the complexity of the hydrology and hydraulic modelling and the accuracy of the data needed to estimate the flows at Sackville, the difference (i.e. less than 10%) is relatively small and well within the bounds of accuracy for flow estimation.



AWACS Report No. 94/29	HYDRAULIC MODEL SCHEMATISATION	Figure 7.1



AWACS Report No. 94/29	SACKVILLE $AR^{2/3}$ CURVE	Figure 7.2



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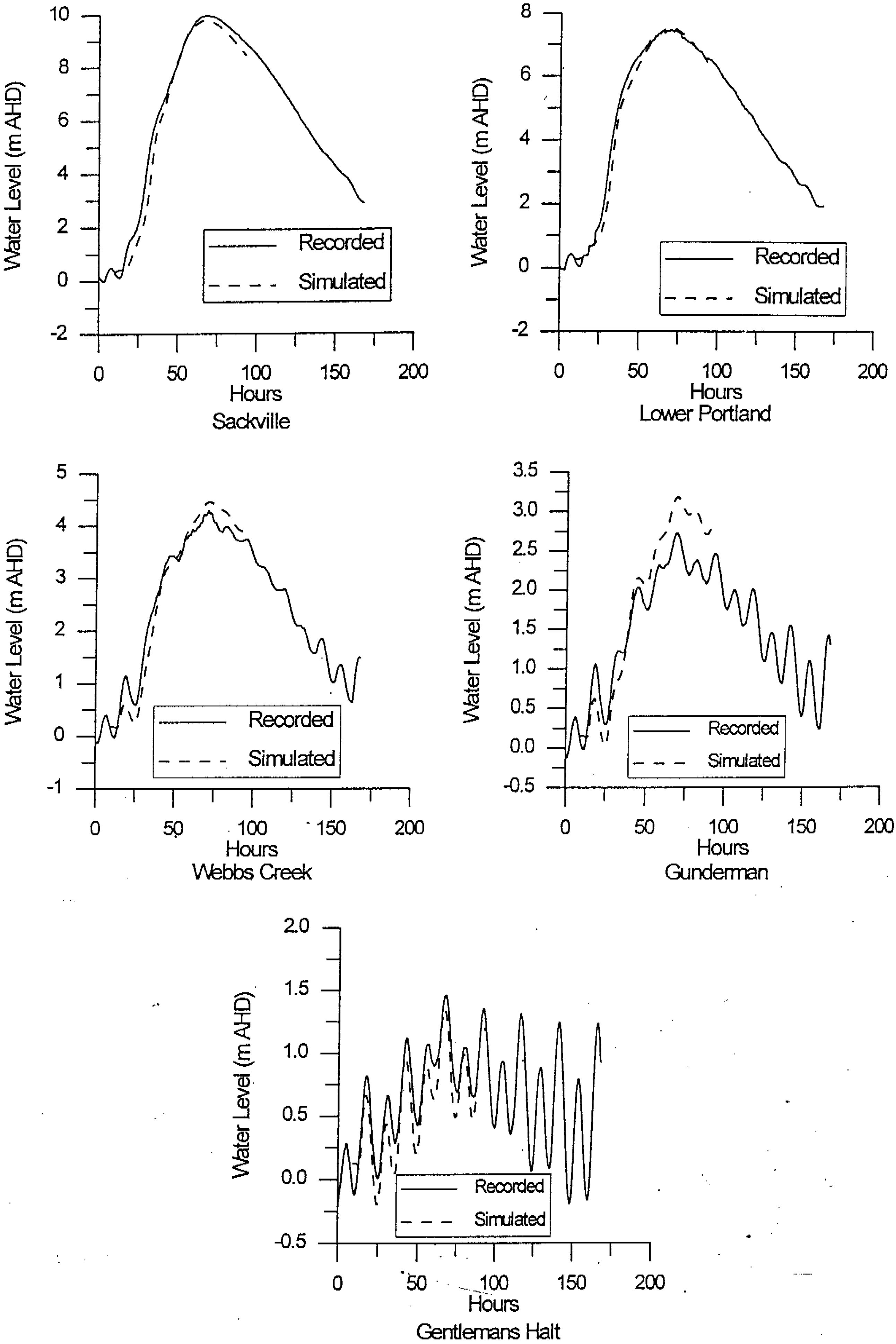
WEBBS CREEK MODELLED AND RECORDED DATA
AUGUST 1990

Figure
7.3



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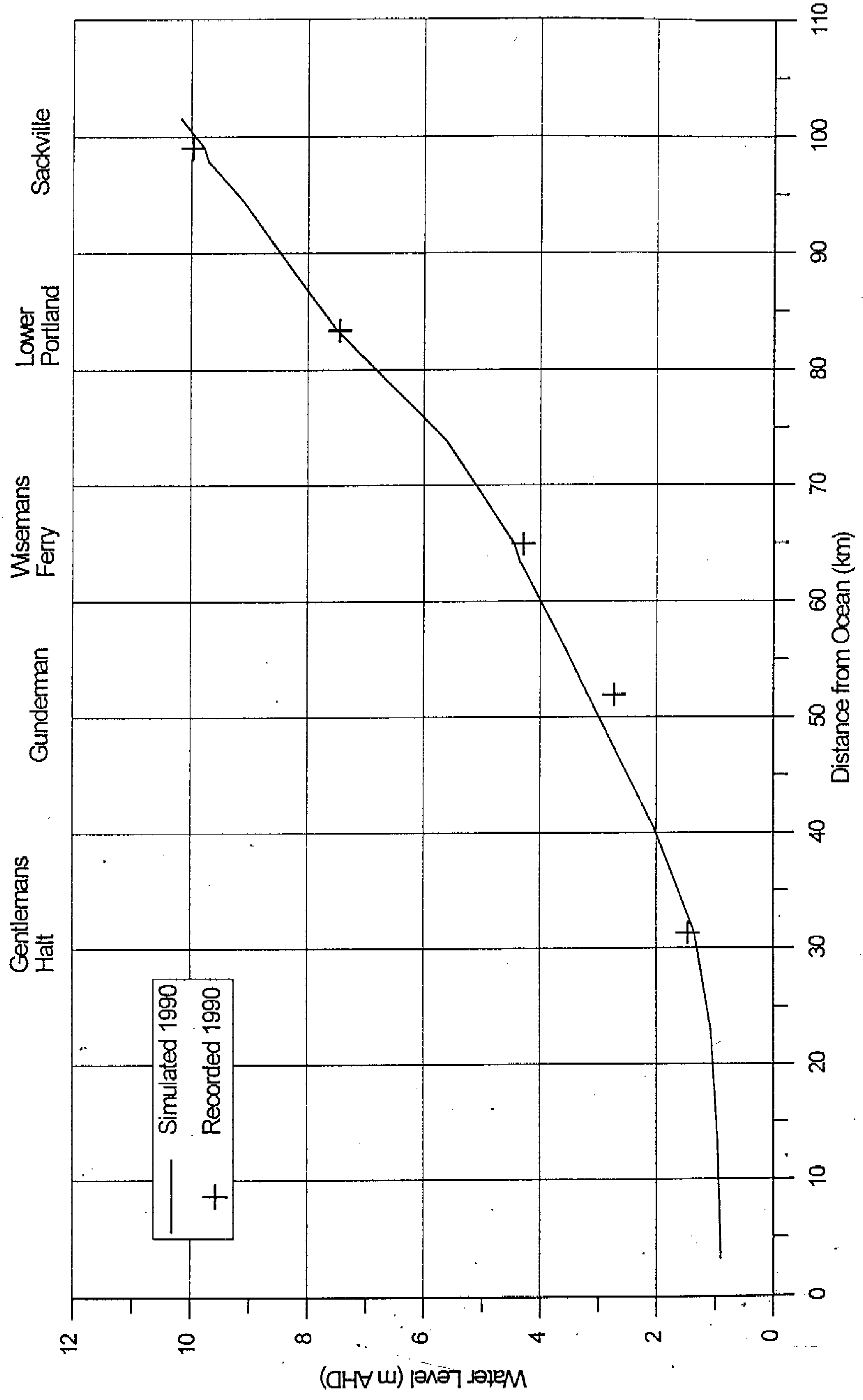
AUGUST 1990 CALIBRATION

Figure
7.4



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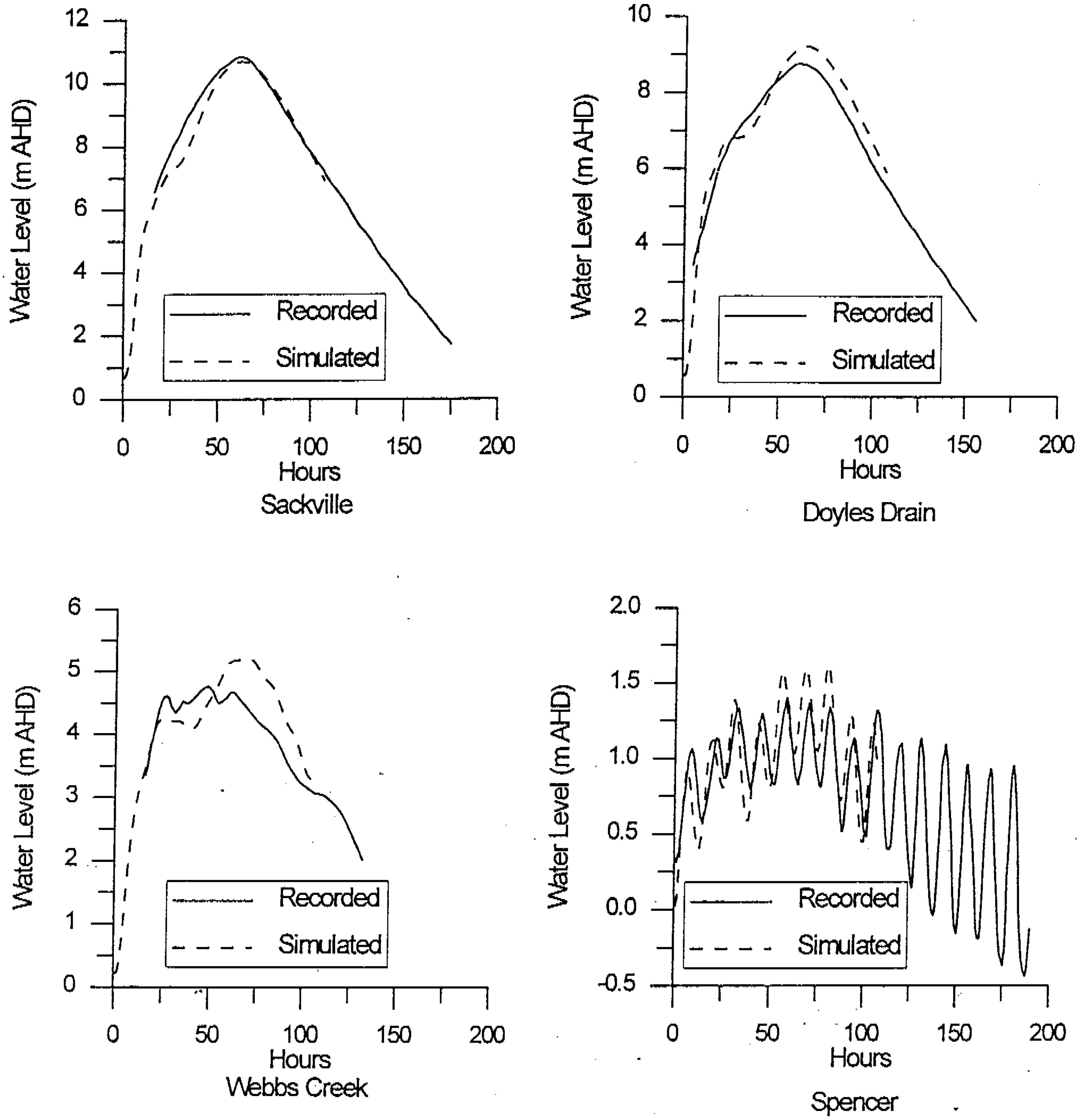
AUGUST 1990
FLOOD PROFILE

Figure
7.5



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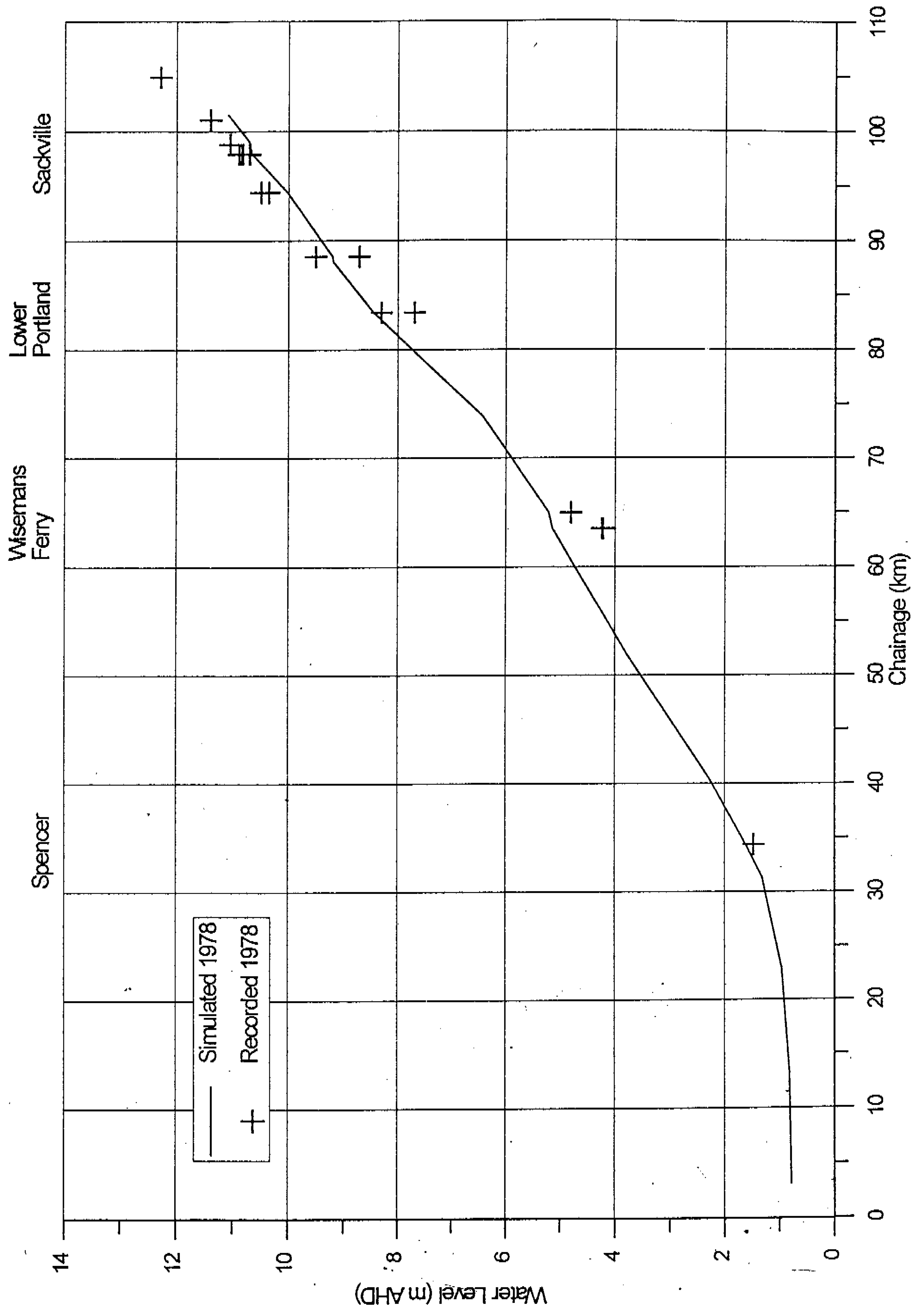
MARCH 1978 CALIBRATION

Figure
7.6



AWACS Report No. 94/29	MARCH 1978 FLOOD PROFILE	Figure 7.7

8. Design Input Data

8.1 Design Inflow Flood Hydrographs

As mentioned earlier, the Sydney Water study discharge hydrographs at Sackville for the 1990 and 1978 floods are slightly higher in comparison to that adopted for this study. Such an adjustment to the Hawkesbury RMA-2 model allowed much better fitting of these two historical events in calibrating and verifying the model. For consistency, it was decided that a reduction should be applied to all Sackville design flood discharge hydrographs derived from the Sydney Water study (SW 1994) and used as upstream inputs in this study. Figure 8.1 shows these design flood hydrographs adopted as input for this study and the peak discharges summarised in Table 8.1 below.

Table 8.1 Hawkesbury River at Sackville Design Peak Discharges

Frequency	Sackville Peak Discharge (m^3/s)
PMF	27,200
1% AEP	9,200
2% AEP	7,400
5% AEP	5,400
20% AEP	3,100

The design discharge hydrographs derived in the Sydney Water study (SW 1994) for the downstream tributaries were also reviewed. The Colo River was subject to particular attention as discussed in Section 6.5.2.4, regarding an extrapolation of the DWR and Sydney Water ratings curves for Upper Colo above the gauge height of 11 metres. This impacts on the Sydney Water calibration of the RORB model and therefore the estimated design outflow hydrographs from the Colo River.

A new set of design flood hydrographs for the Colo River and all other downstream tributaries was derived, following a review of the Sydney Water RORB modelling of the Colo River. In reviewing the RORB calibration the following points were taken into consideration:

- The Sydney Water RORB model was calibrated and the design hydrographs were estimated using the following parameters:

Kc	= 70
m	= 0.8
Initial Loss (IL)	= 70 mm
Continuing Loss (CL)	= 2.5 mm

- Based on these design parameters, the design rainfall and an area reduction factor of 0.95 the 1% AEP peak discharge was estimated to be 9,700 m^3/s.

- The Sydney Water calibration of the RORB model for the Colo catchment showed a range of IL from 60 mm to 165 mm. Sydney Water calibrated the RORB model to the Upper Colo rating which has now been revised with this investigation (see Section 7).

- The flood frequency analysis of the Upper Colo (Table 6.4) suggests a 1% AEP peak discharge of about 4,000 m^3/s.

- The Sydney Water study (SW 1994) discussed several methods for calculating the kc value used in RORB, namely:

  $kc = 4.23\ A^{0.34}$ (ungauged catchment)
  $kc = 1.17\ A^{0.558}$ (Boyd 1983), and
  $kc = 1.22\ A^{0.46}$ (AR&R 1987)

  These equations gave estimated kc values for the Colo ranging from 59 to 132.

- AR&R (1987) suggests an appropriate m value of 0.85 when estimating significant events.

- Recent work by Masters and Irish (1994) has suggested that an area reduction factor of 0.76 may be more applicable to the Hawkesbury River Valley.

- The 1978 flood event has a frequency of occurrence of around 1% AEP and the shape of the hydrograph was recorded.

AWACS recalibrated the RORB model, with the following results:

1. The peak of the 1% AEP flood was estimated at around 4,000 m^3/s and the shape of the hydrograph was similar to the 1978 event.

2. A rainfall area reduction factor of 0.76 was applied to the estimation of tributary flows to the Lower Hawkesbury River.

3. The four RORB coefficients used in the modelling exercise fitted within the accepted bounds.

Therefore, the following values were used to calibrate the RORB model to the 1% AEP flow in the Colo River:

kc = 75
m = 0.85
IL = 80 mm
CL = 2.5 mm

Figure 8.2 shows the estimated 1% AEP hydrograph for the Colo River compared to the 1978 recorded event.

For the Macdonald River catchment and other tributaries, the loss coefficients were maintained as selected in the Sydney Water study (SW 1994). The kc and m values were, however, modified in a manner similar to that used on the Colo River.

In summary, the parameters given in Table 8.2 were adopted for the RORB model used in this study, to calculate the design runoff hydrographs for the tributaries downstream of Sackville.

Table 8.2 RORB Design Parameters

Frequency	kc	m	IL	CL
Colo River				
PMF	75	0.85	20	1.0
1% AEP	75	0.85	80	2.5
2% AEP	110	0.8	80	2.5
5% AEP	110	0.8	80	2.5
20% AEP	110	0.8	80	2.5
Macdonald River				
PMF	60	0.85	20	1.0
1% AEP	60	0.85	70	2.5
2% AEP	85	0.8	70	2.5
5% AEP	85	0.8	80	2.5
20% AEP	85	0.8	70	2.5
Other tributaries				
PMF	75	0.85	20	1.0
1% AEP	75	0.85	70	2.5
2% AEP	110	0.8	70	2.5
5% AEP	110	0.8	70	2.5
20% AEP	110	0.8	70	2.5

Figures 8.3 and 8.4 show the design discharge hydrographs for the Colo and Macdonald Rivers. The design hydrographs were calculated for all the tributaries. The peak discharges for the three major contributing systems are listed in Table 8.3.

Table 8.3 Design Peak Discharge

Frequency	Sackville	Colo River	Macdonald River
PMF	27,200	12,200	3,700
1% AEP	9,200	4,100	1,600
2% AEP	7,400	3,200	1,200
5% AEP	5,400	2,200	800
20% AEP	3,100	1,000	300

8.2 Design Ocean Levels

As discussed in **Section** 6.5.3 the design ocean tide levels derived from the more extensive record at Fort Denison Sydney were adopted for Broken Bay because there is good correlation. The range of peak design ocean levels, adopted at the downstream boundary of the model are given in Table 8.4. The magnitude of tidal peaks for various return periods is shown in Figure 8.5.

Table 8.4 Design Ocean Levels

Frequency	Peak Ocean Levels (m AHD)
100% AEP	1.26
20% AEP	1.38
5% AEP	1.41
2% AEP	1.46
1% AEP	1.49
Extreme	1.78

In analysing the likely combination of tides with river flood conditions, the following factors were considered.

- The May 1974 peak water level of 1.5 m AHD, resulted from the concurrence of a 0.6 m storm surge with a 0.9 m tide. It is important to note that a storm surge, even at a large magnitude of 0.6 m (largest measured), can still result in a low ocean level if it does not occur on a spring tide. For example, if the 1974 surge had occurred on a neap tide cycle peaking at 0.5 m AHD, the combined level of 1.1 m would not even reach the level of the highest astronomical tide level (HAT) of 1.2 m. It should also be noted that storm surges generally last for around two to three days, and their impacts are most apparent at high tide over approximately two-hour periods. Consequently the spring tide cycle would need to be included in the model's input to get a level of the order of 1.5 m with the impact of the tide lasting only two hours with every high tide cycle.

- Pugh and Vassie (1986) have developed a method of statistically combining astronomical tides and surges. This method assumes that the two events are independent and simply multiplies the frequency distribution of the astronomical tide by the frequency distribution of surges, then adds the combined probabilities for equal levels. This method was applied to the Fort Denison Sydney data to derive design tide levels. Figure 8.5 presents the results of this method (MHL 1992). An interesting result of this analysis for Sydney is that the difference between the extreme ocean level and, say, the one-in-one-year levels was found to be only 0.5 metres. Section 9.2.3 of this report shows the impacts of different ocean levels on flooding along the river.

- The Public Works tide gauge network deployed along the coast has identified that tidal anomalies in the range of 0.1 metres to 0.3 metres occur frequently (MHL 1992). Results of the combined tide and storm surge analysis described above suggest that tide levels of approximately 1.1 m are relatively common. Further examination of these two observations show that frequent 0.1 to 0.3 m anomalies and associated tides are generated by isolated weather systems over large areas of the Tasman Sea, hence the frequency of these high levels is not unexpected considering the regularity of moderate to low pressure systems over such a large region of ocean. Broken Bay ocean levels can be affected by storms occurring between Bass Strait and Tweed Heads. The important point here is that tidal anomalies and associated high ocean levels do not necessarily require the presence of a local coastal storm, i.e. high ocean levels can occur without local catchment flooding.

- The May 1974 tidal levels recorded at Sydney statistically correspond to the 1% AEP event (MHL 1992). As such, the recorded tide levels for the May 1974 event, over a period of 100 hours, were adopted for the 1% AEP tailwater level. The same tide cycle was adopted for all the other frequency events with the peak level adjusted either upwards or downwards to correspond to the relevant peak surge levels shown in Table 8.4. Figure 8.6 shows the adopted tide and associated surge for the 1% AEP event.

8.3 Joint Occurrence of Rainfall Runoff Flooding for Design Conditions

The joint occurrence of flood events from the Colo and Hawkesbury Rivers was examined from two perspectives. Firstly, the historical data was examined to see if there were any evident trends and secondly a statistical approach was investigated to compare and broaden any interpretations from the historical data.

8.3.1 Historical Data

Comparison of the historical flood data presented in Table A1 do not show any particular trends which establish conclusively the joint occurrence of Colo and Hawkesbury River floods. Figures 8.7 to 8.10 show corresponding flood levels/flows at Windsor, Upper Colo and St Albans for the same flood events. The following comments are made on the results:

- The largest flood recorded in the Colo River occurred in March 1978 and was about a 1% AEP event. The 1978 event also caused a major flood in the Macdonald River with a frequency of occurrence of about 3% AEP. The same flood in the Hawkesbury River at Windsor had a frequency of occurrence of about 3% AEP.

- Comparing the flooding at Windsor with that occurring in the Colo River, over a similar period of record (1910 to 1994), there were four significant flood events with the following probability of occurrence:

1956	Windsor (13.83 m AHD) 4% AEP	Colo (15.26 m AHD) 3% AEP
1961	Windsor (14.95 m AHD) 2.5% AEP	Colo (9.3 m AHD) 50% AEP
1964	Windsor (14.57 m AHD) 3% AEP	Colo (14.61 m AHD) 7% AEP
1978	Windsor (14.46 m AHD) 3% AEP	Colo (20.72 m AHD) 1% AEP

  The AEPs for the Windsor levels were determined from the Sydney Water study (SW 1994) while the Colo AEPs were estimated from the flood frequency analysis undertaken as part of this study.

- Figure 8.10 indicates that when there is a flood in the Colo River, there is a reasonable chance of a flood occurring in the Macdonald River.

- While the historical analysis of the flood data does not conclusively suggest any particular joint occurrence of floods in the Colo and Hawkesbury Rivers, the results do suggest there is an envelope of joint occurrences.

8.3.2 Stochastic-Deterministic Method

The results from the historical data analysis did not provide conclusive evidence on joint occurrence of flooding. Consequently, a study of the joint occurrence of Colo River and Hawkesbury River flows from a statistical viewpoint was considered wothwhile. This employed an approach developed by Laurensen (1974). Laurensen suggests that hydrologic systems have both stochastic and deterministic components. Appendix C outlines in detail the method adopted for Colo and Hawkesbury.

From the joint occurrence study outlined in Appendix C it was found that time between the Colo River and Hawkesbury River flood peaks was a parameter in the relationship. This meant that there were too many unknowns to use the exact approach as recommended by Laurensen. To overcome this complication, an average travel time between peaks was used to allow the method to be employed. This was estimated from the information available from the Sydney Water study (SW 1994), that is:

- the peak of the Colo River flood occurred, on average, about 11 hours before the peak of the flood at Warragamba Dam, and

- the average travel time of a flood between Warragamba Dam and Sackville was about 24 hours.

An examination of this information and a review of the timing of historical floods suggested that 35 hours was an average time difference between Colo River and Sackville flood peaks. The 35 hours was adopted for the joint occurrence study and applied to all of the design flood events estimated later in this report.

The joint occurrence study identified the frequency of flood flows downstream of the Hawkesbury River and Colo River confluence. However, any given peak flood flow downstream of the confluence, could result from a range of different combinations of Hawkesbury River and Colo River flows. For example, the 1% AEP flow of 9,300 m^3/s downstream of the confluence, could result from any of the three scenarios described below as a, b, and c.

While each different scenario could produce the same peak flow downstream of the confluence, the shape or volume of the hydrograph downstream of the confluence could vary considerably, depending on the shape of the Colo River and Sackville discharge hydrographs. A hydrograph with the same peak flow but greater volume would also cause greater flooding and result in higher flood levels. From an inspection of the historical data, it appears more likely that the larger magnitude flood would occur in the Hawkesbury rather than in the Colo River.

For this study, the following three flooding scenarios (a, b and c) were considered for combining Colo and Hawkesbury River floods and tested using the hydraulic model:

- scenario 'a'- the X% AEP flood from Sackville and the corresponding flood out of the Colo River required to produce the X% AEP desired confluence peak flow (i.e. using Figure C3);

- scenario 'b'- the X% AEP flood from the Colo River and the corresponding flood from Sackville required to produce the desired X% AEP confluence peak flow (i.e. using Figure C3);

- scenario 'c'- examine the information presented in Figures C2 and C3 to determine the most likely combination of Sackville and Colo River floods to produce the desired X% AEP confluence peak flow (i.e. the combination plotted on the 50% probability line on Figure C2).

The results of the frequency study (detailed in Appendix C) and the three scenarios adopted to achieve corresponding peak flows at the confluence, are summarised in Table 8.5. Each scenario was examined using the hydraulic model.

Table 8.5
Probability of Flows at the Confluence of Colo and Hawkesbury Rivers

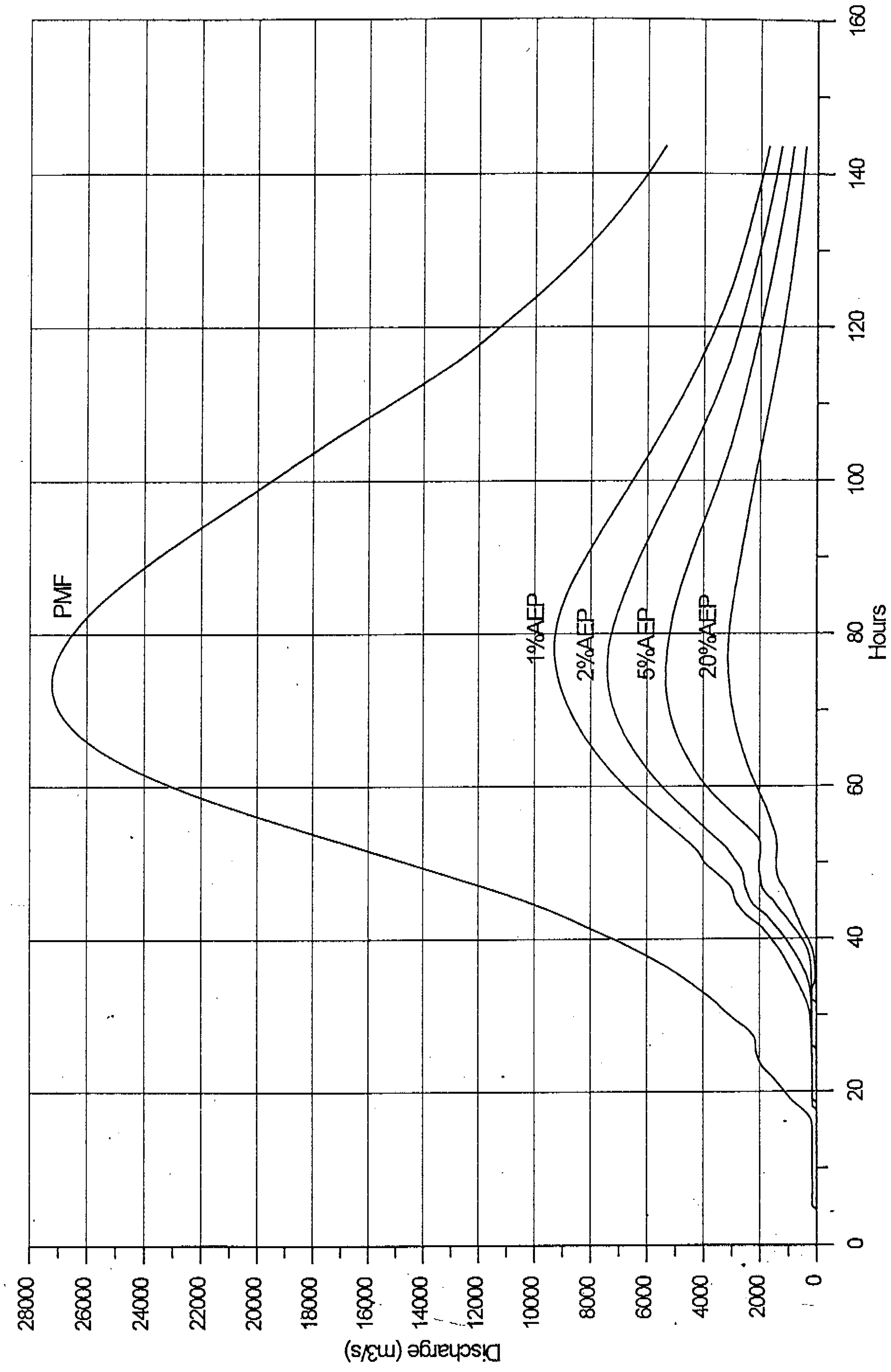
Probability of Occurrence	Flow at the confluence (m^3/s) (see Figure C4)	Scenario	Flow in the Hawkesbury River at Sackville (m^3/s) (see Figure C3)	Flow in the Colo River at Upper Colo (m^3/s) (see Figure C3)	Probability of flow at Sackville and Upper Colo (%) (see Figure C2)
20% AEP	3,700	'a'	20% (3,100 m^3/s)	2,200 m^3/s	25%
		'b'	3,500 m^3/s	20% (1,000 m^3/s)	55%
		'c'	3,600 m^3/s	1,200 m^3/s	50%
5% AEP	6,000	'a'	5% (5,400 m^3/s)	3,300 m^3/s	10%
		'b'	5,600 m^3/s	5% (2,200 m^3/s)	35%
		'c'	5,700 m^3/s	1,900 m^3/s	50%
2% AEP	7,800	'a'	2% (7,400 m^3/s)	3,400 m^3/s	25%
		'b'	7,500 m^3/s	2% (3,200 m^3/s)	25%
		'c'	7,600 m^3/s	2,400 m^3/s	50%
1% AEP	9,300	'a'	1% (9,200 m^3/s)	600 m^3/s	95%
		'b'	8,800 m^3/s	1% (4,100 m^3/s)	15%
		'c'	8,900 m^3/s	2,900 m^3/s	50%

8.4 Joint Occurrence of Flooding and Ocean Tides for Design Conditions

The interaction of catchment runoff and higher ocean levels at the entrance of the Hawkesbury River is a complex process. Without sufficient data it would be difficult to derive realistic design conditions. However, the fact that catchment flooding and ocean levels are independent events and the difference between extreme and yearly tide levels is only 0.5 metres, it would be reasonable to adopt the coincidence of design ocean level and flood peaks for design purposes. The various frequency combinations of ocean levels reviewed in the sensitivity analysis are presented in Section 9.2.3.

In the lower reaches of the Hawkesbury River around Brooklyn, the 1% AEP flood level will be determined by the 1% AEP ocean levels, while from Sackville to Gunderman the river 1% AEP flood conditions are likely to dominate. In the section of river between these two reaches, the flood levels will be a combination of ocean and river conditions.

For this study the 1% AEP flood was combined with the 1% AEP ocean level and phased such that the peak of the flood and the ocean peak coincided at around Brooklyn. Sensitivity analysis was undertaken in Section 9.2.3 to assess the effect of this combination.



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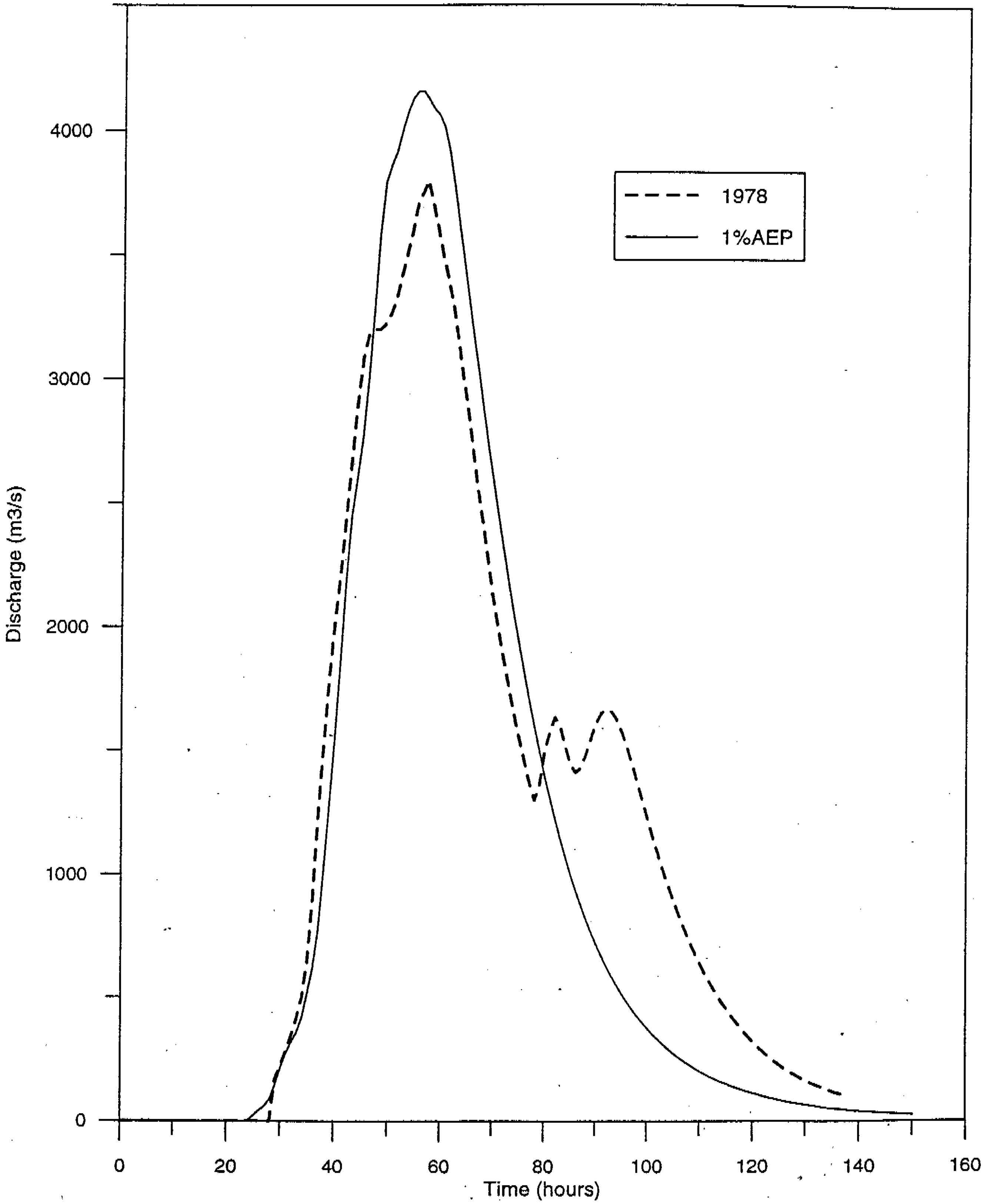
HAWKESBURY RIVER AT SACKVILLE
DESIGN FLOOD HYDROGRAPHS

Figure
8.1



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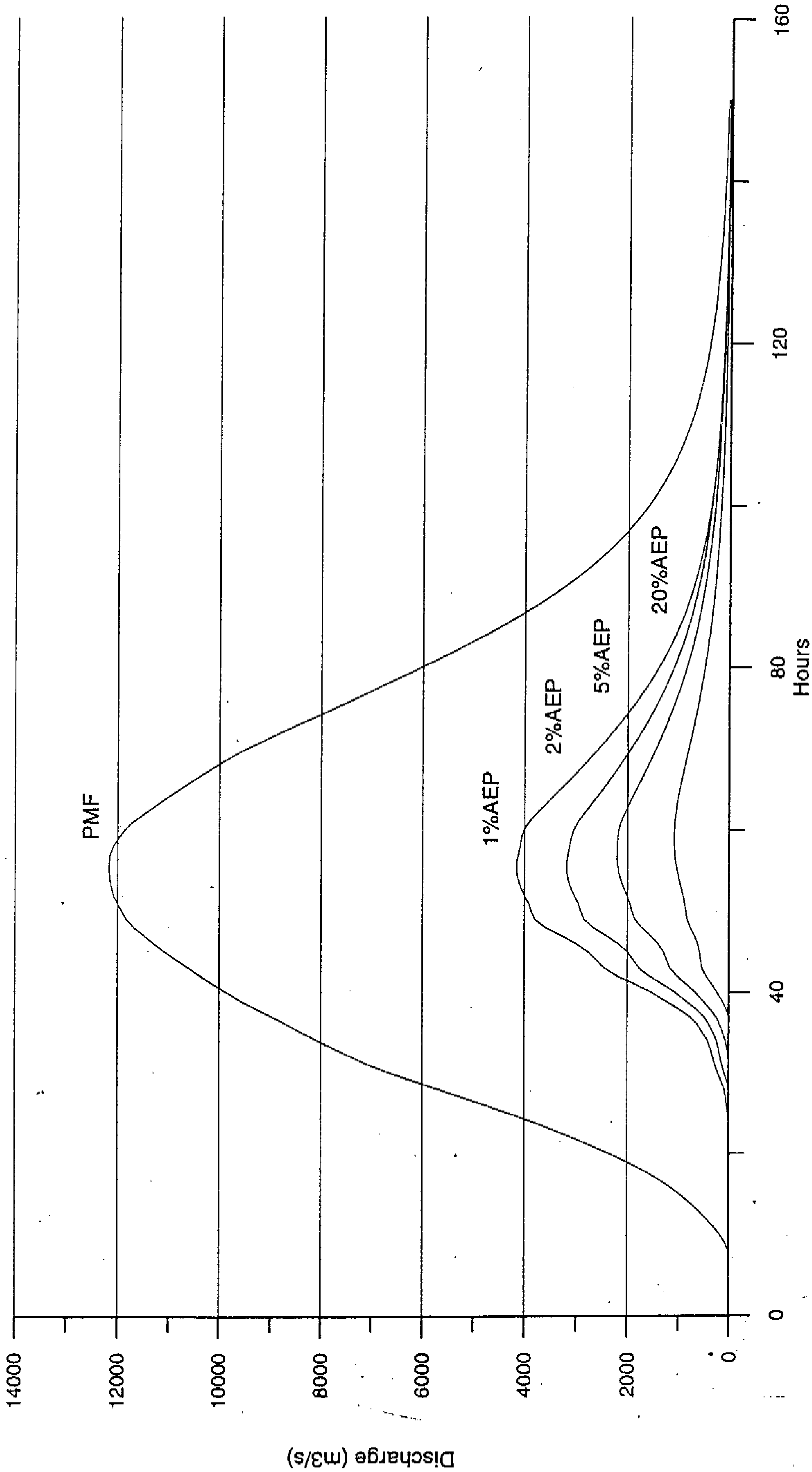
RE-CALIBRATION OF RORB MODEL TO 1978 FLOOD

Figure
8.2



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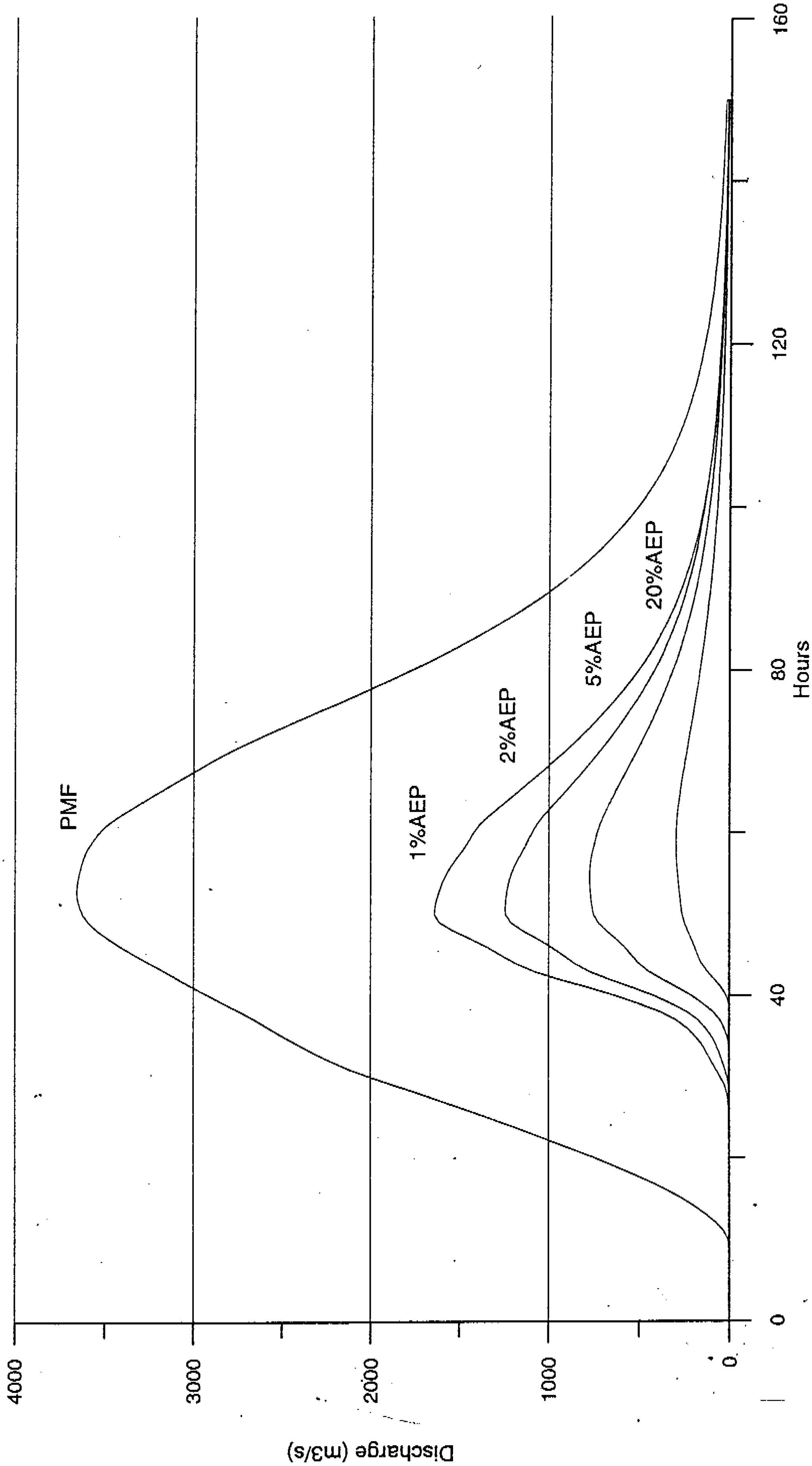
COLO RIVER
DESIGN FLOOD HYDROGRAPHS

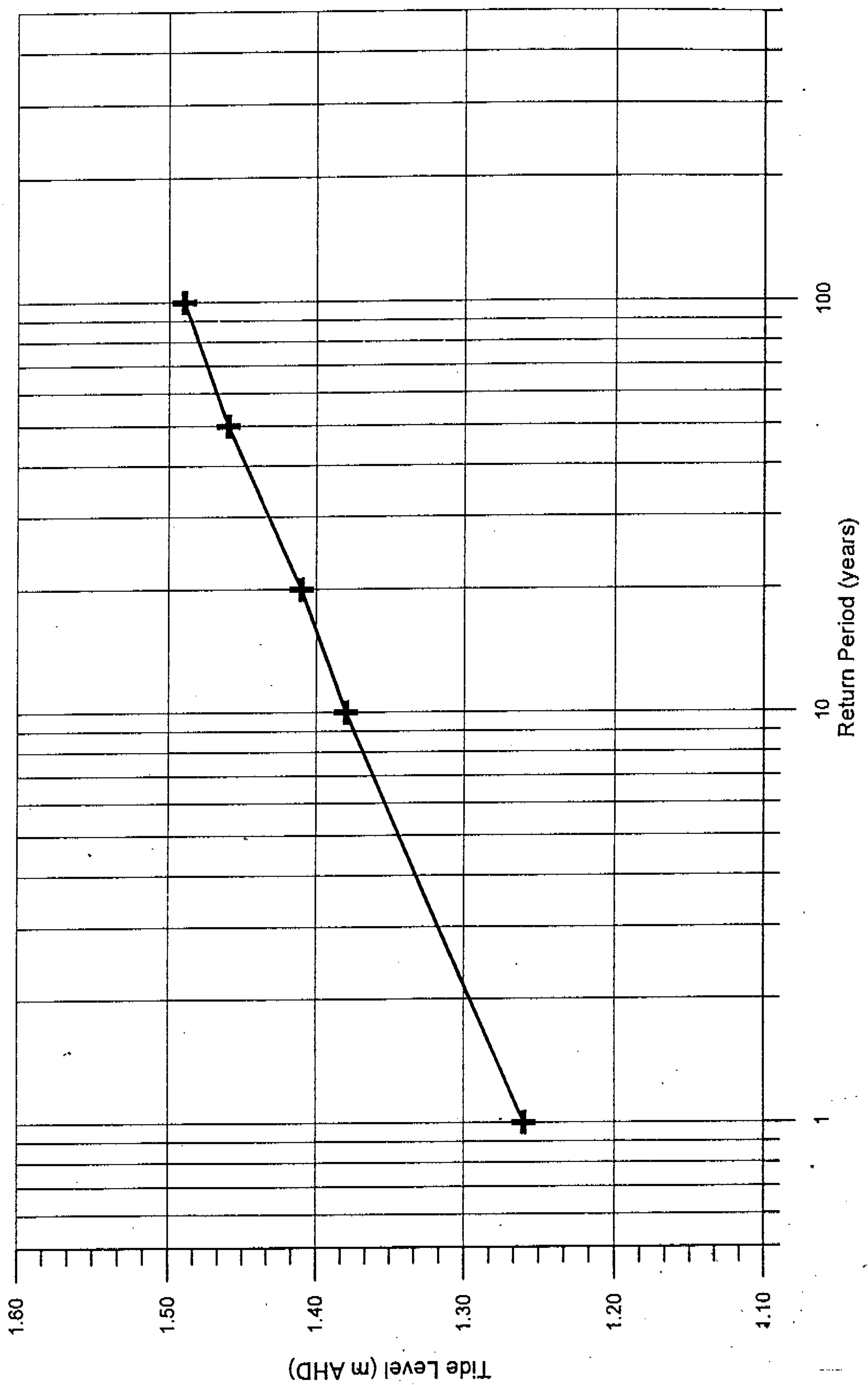
Figure
8.3



AWACS Report No. 94/29

MACDONALD RIVER
DESIGN FLOOD HYDROGRAPHS

Figure 8.4



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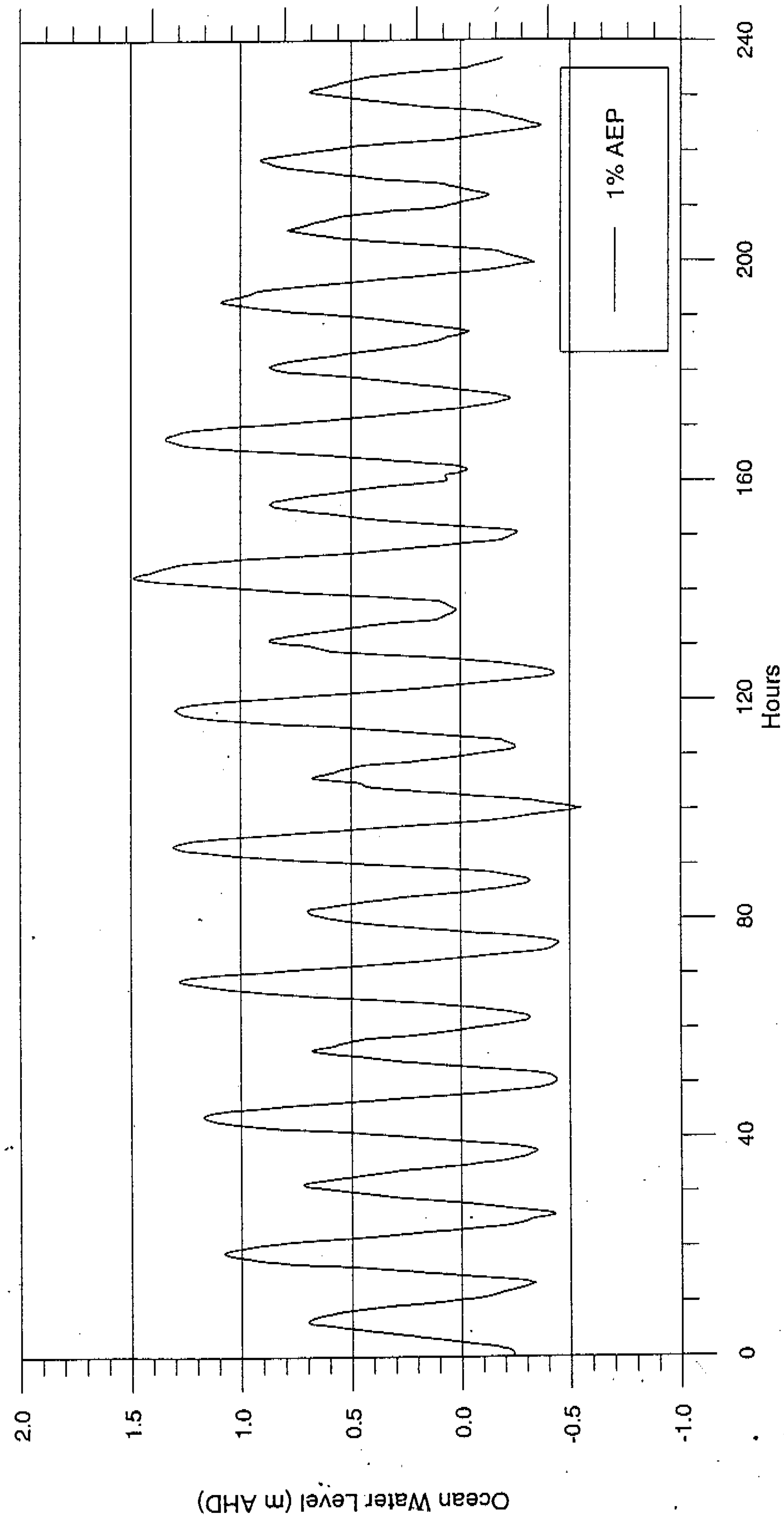
AWACS Report No. 94/29	SYDNEY TIDE LEVEL FREQUENCY (1914 - 1991)	Figure 8.5



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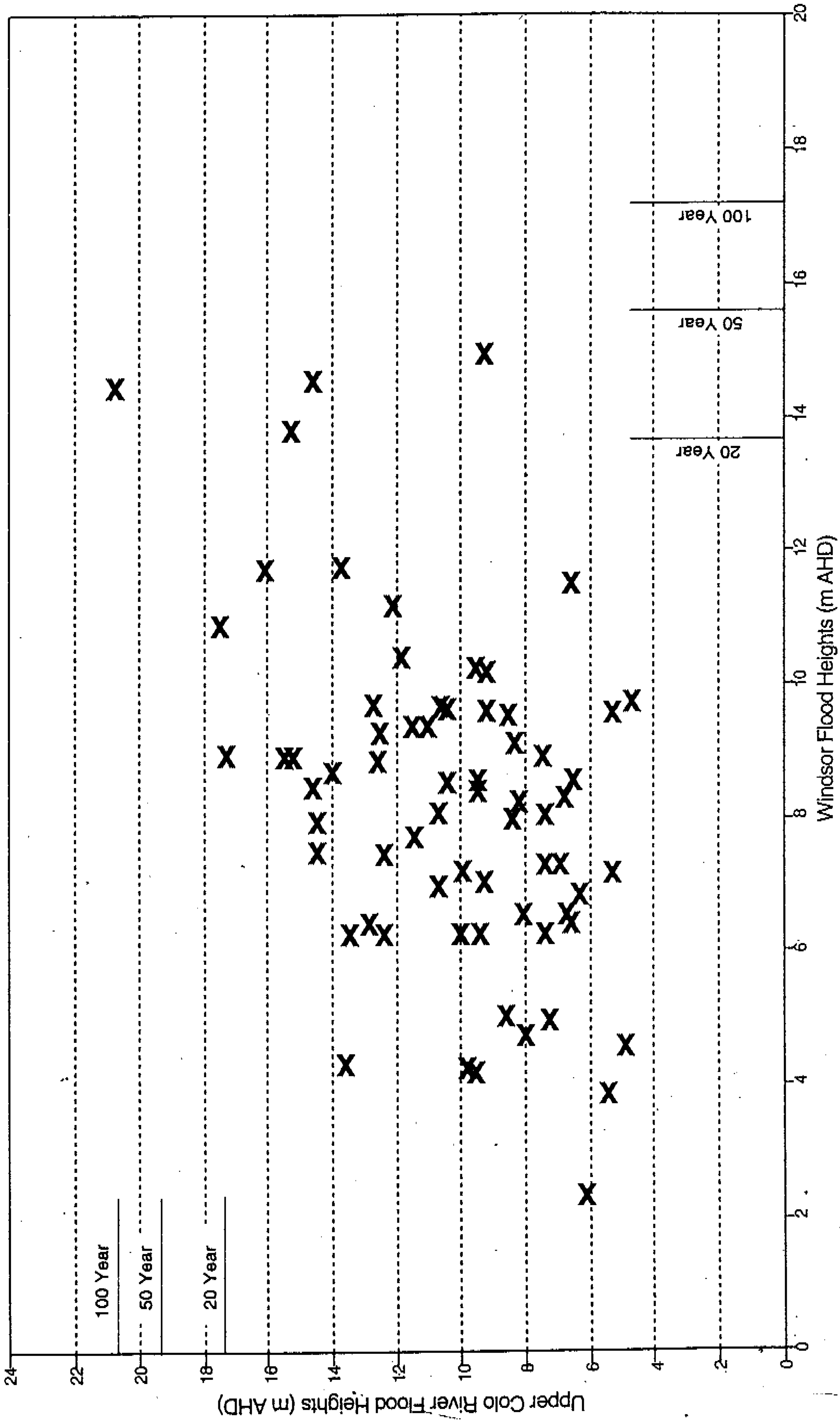
1% AEP
TAILWATER LEVELS

Figure
8.6



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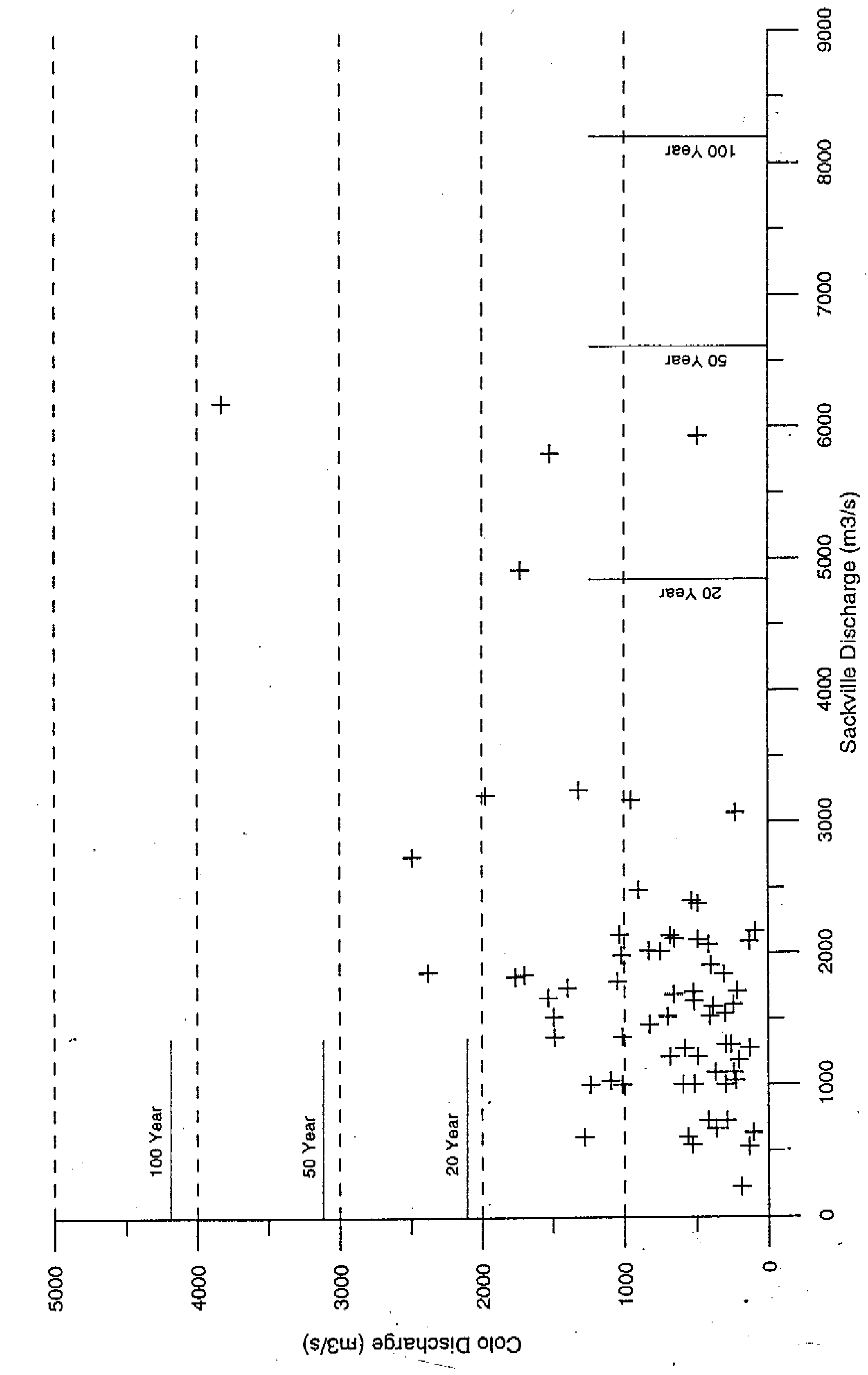
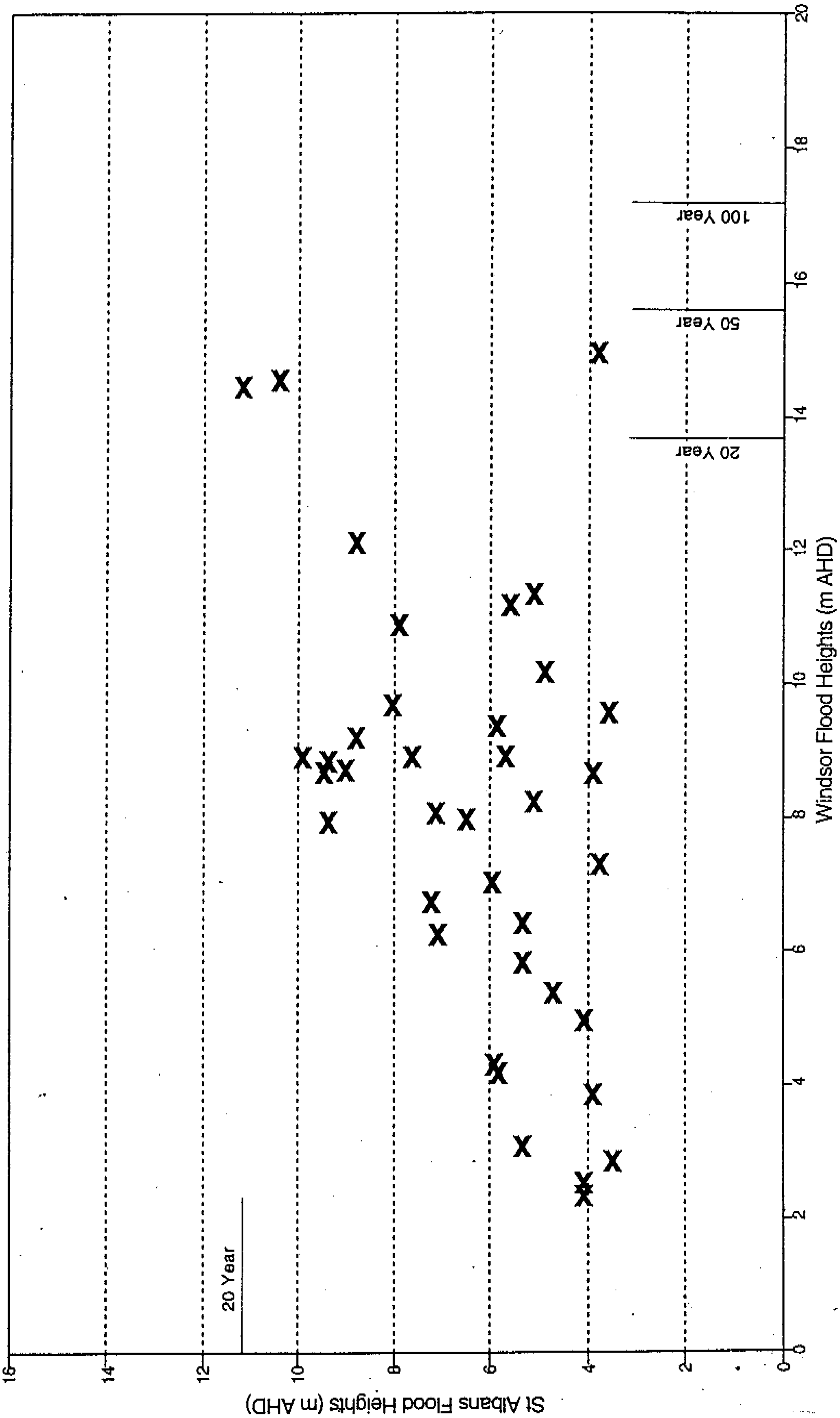
JOINT OCCURRENCE OF COLO RIVER AND WINDSOR FLOOD HEIGHTS

Figure 8.7



AWACS Report No. 94/29	JOINT OCCURRENCE OF UPPER COLO RIVER AND SACKVILLE FLOWS	Figure 8.8



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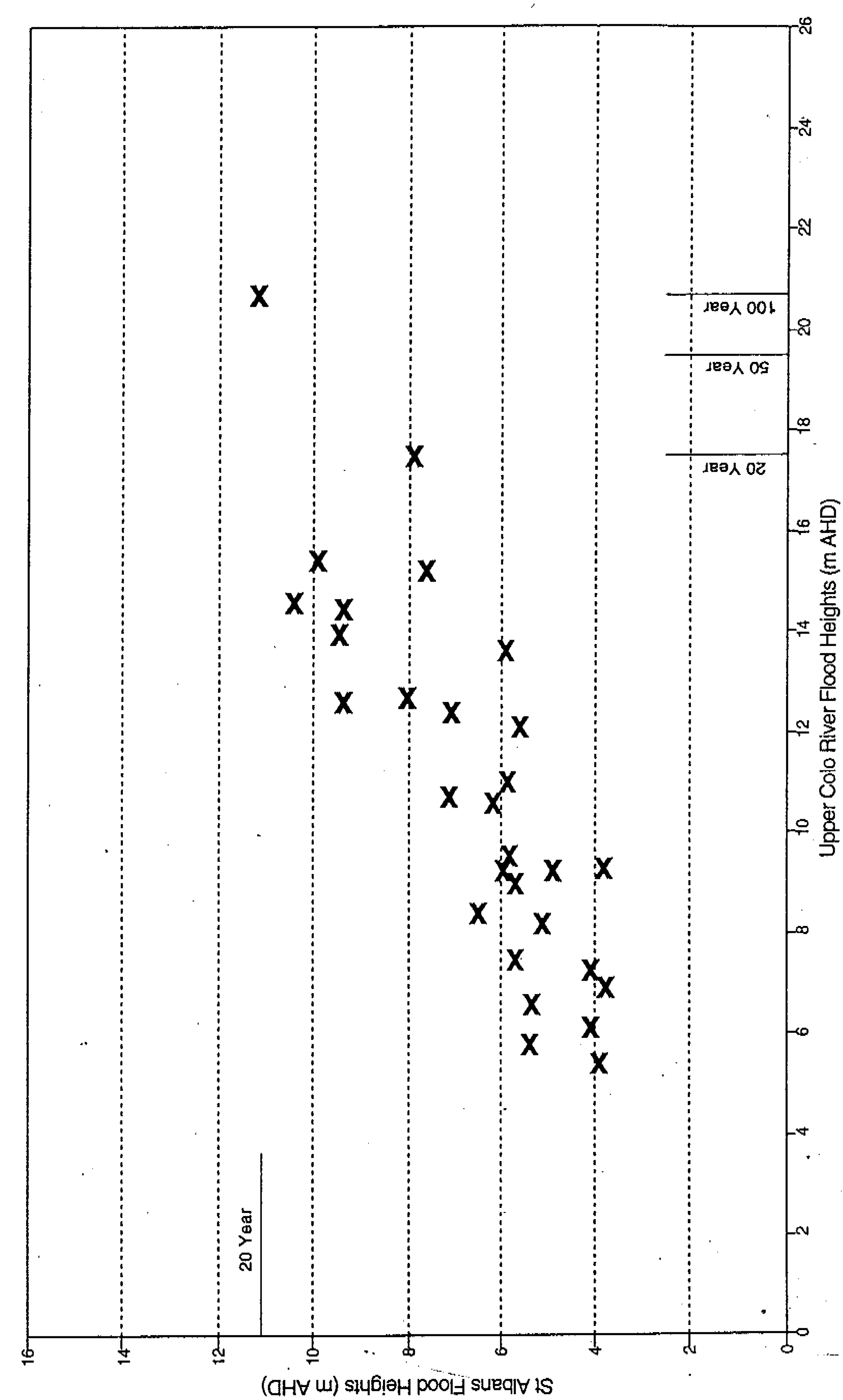
JOINT OCCURRENCE OF WINDSOR AND
ST ALBANS FLOOD HEIGHTS

Figure
8.9



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JOINT OCCURRENCE OF COLO RIVER AND MACDONALD RIVER FLOOD HEIGHTS

Figure 8.10

9. Sensitivity Analysis of Hydrodynamic Model

9.1 Sensitivity of Design Flood Levels to Combination of Hawkesbury and Colo Flows

The investigation into the joint occurrence of Hawkesbury and Colo River floods discussed in Section 8 provides the basis for selection of the procedure for estimation of the design flood levels. As discussed in Section 8.3, the joint probability analysis provided the peak flows for the various AEPs at the confluence of the Hawkesbury and Colo Rivers. However, different combinations of flows from the two rivers could produce the given confluence peak. As illustrated in Figure 9.1, although the peaks might be identical, the hydrograph shapes would be different and each combination would produce different flood profiles down the river. To provide the information required for the selection of the design flood levels, several different combinations of inflow hydrographs, each producing the same confluence peak, were run through the hydraulic model to compute the flood profiles.

The selection of a design flood profile for the 1% AEP involves two scenarios:

- adopting the joint occurrence of the flood peak at the confluence of the Hawkesbury and Colo Rivers, and
- the combination of flows from those two rivers.

The approach used for the 1% AEP flood was to adopt the modelled profile for the 1% AEP peak at the confluence (i.e. 9,300 m^3/s), itself derived from the joint probability analysis in Section 8.3, and to adopt the most probable combination of flows from the two rivers. In its simplest form, the most probable combination of input flows would lie on the 50% probability line on Figure C2. This is listed as combination 'c' in Table 8.5. This combination lies on the best estimate of the relation between Colo and Sackville peak flows.

Flood profiles for the three combinations of 1% AEP floods are listed in Table 9.1 and plotted on Figure 9.2.

Table 9.1 1%AEP Flood Level (m AHD) for Different Combinations of Inflow Floods

Combination from Table 8.5	'a'	'b'	'c'
AEP of Sackville Flood (Figure C1)	1%	1.5%	1.5%
AEP of Colo River Flood (Figure 6.8)	40%	1%	2%
Probability of Colo and Sackville Flood combination (Figure C2)	95%	15%	50%
Sackville	12.9	13.0	12.9
Lower Portland	10.2	10.5	10.3
Leets Vale	7.4	7.8	7.6
Wisemans Ferry	6.5	6.9	6.7
Gunderman	5.0	5.4	5.2
Spencer	2.6	2.8	2.7
Brooklyn	1.7	1.7	1.7

The 1% AEP flood profiles in Figure 9.2 show that the most probable scenario 'c' results lie in the middle of the results from scenarios 'a' and 'b'. Combination 'c' was selected as the design profile, as it approximates the most probable and covers more of the possible range. This procedure was adopted for all the other frequency floods.

9.2 Sensitivity Analyses

Sensitivity analyses were undertaken to examine the effects of the following parameters on peak flood profiles:

- tributary inflows
- bed scour
- ocean levels
- flood hydrographs, and
- bed roughness.

These analyses were aimed at reviewing the effects of varying the parameters to within reasonable bounds to satisfy some of the uncertainties in the selected parameter and combination assumptions. These results were examined in relation to the design flood profiles. The Hawkesbury is a highly complex and interactive river system and the results of sensitivity studies highlight these processes. The sensitivity studies also serve to highlight the need to improve understanding of the system by continuing data collection on future floods.

9.2.1 Sensitivity to Tributary Inflows

In the estimation of the 1% AEP design flood levels, it was necessary to make assumptions on the combination of river and tributary flows. These estimates were considered appropriate considering the available data. However, to gauge the effects of assuming different tributary conditions, the following sensitivity studies were undertaken. Table 9.2 shows the sensitivity of the 1% AEP design flood levels to contributions from the Colo and Macdonald Rivers.

Table 9.2 1% AEP Design Flood Levels Sensitivity to Tributary Flows

Location	1%AEP with Colo and Macdonald Rivers flows (m AHD)	Change in Water Level for 1% AEP No Colo and Macdonald Rivers flows (m)
Sackville	12.9	-0.1
Lower Portland	10.3	-0.2
Leets Vale	7.6	-0.2
Wisemans Ferry	6.7	-0.2
Gunderman	5.2	-0.2
Spencer	2.7	-0.1
Brooklyn	1.7	0.0

These results show a relatively small change in flood levels for other combinations of the Hawkesbury River, Colo River and Macdonald River flows. If the Colo, Macdonald and all other tributaries did not flood, then the 1% AEP peak flood level at say Wisemans Ferry, reduces by about 0.2 m.

9.2.2 Sensitivity to River Bed Scour

River bed scour, although not addressed in the calibration and verification models, is considered to be a process that should be examined in the sensitivity analysis.

Detailed hydrosurveys prior to and after any of the major floods to indicate the amount of scour during a flood are generally not available. The degree to which the Hawkesbury River bed scours during floods is, therefore, not reliably known. Surveys undertaken in 1971 from Sackville to Pumpkin Point (28 km from the ocean) by Public Works, when compared to the 1872 survey by Lieutenant Gowlland, showed no significant overall changes in channel depth (Scholer 1974). This comparison indicates that the river has experienced no net change in bed level. This suggests that during a flood the river bed is likely to scour on the rising stages and accrete on the falling stages. These areas of scour and accretion may also be reworked by the tidal currents.

To provide further insight into the potential bed scour during floods, modelling of bed scour and specific field studies were undertaken.

9.2.2.1 Bed Scour Numerical Modelling

An investigation on potential bed scour was undertaken using the US Army Corps of Engineers HEC-6 numerical model. HEC-6 is a one-dimensional movable boundary open channel flow and sediment model designed to simulate changes in river profile due to scour and deposition over long periods. Although there was no data with which to calibrate the model, it was anticipated that in developing the model to reproduce the expected characteristics of the Hawkesbury River, some quantitative assessment of bed scour would be obtained.

The preliminary results of the HEC-6 investigation were found to be inconclusive and did not provide any worthwhile guidelines on scour and deposition in the Hawkesbury River. Given that the HEC-6 model was developed mainly to simulate long-term changes to river bed profiles, the model was not able to produce useful results for the purposes sought in this investigation. Consequently further investigations using this approach were not pursued.

9.2.2.2 Field Investigations of Bed Scour

To access the potential river bed scour, a range of field investigations was undertaken to examine the nature of the Hawkesbury River sediments and to identify any changes to the bed profiles that have occurred over time. The following field measurements were undertaken in April 1995:

- River bed cores were collected at sites adjacent to some of the developed areas, that is in the vicinity of Sackville, Leets Vale and Wisemans Ferry. Coastal and Marine Geosciences were engaged to examine the cores. Location of cores and surveys is shown in Figure 9.3.

- Long sections of the Hawkesbury River were surveyed along a number of suitable long straight sections of the river, namely at the Sackville reach, downstream of Lower Portland and near Wisemans Ferry as shown in Figure 9.3.

- Sixteen cross-sections were measured between Sackville and Wisemans Ferry. These sections were positioned as close as possible to the sections previously surveyed (1978 and 1984) and used in the numerical model.

The following summarises the results of these investigations.

- The bed cores collected between Sackville and Wisemans Ferry, to depths of 3 metres, consisted of fluvial sediment, medium to coarse grain with charcoal and shell. Layers of charcoal within the sediment probably represent surfaces upon which fluvial sediment has been deposited. Charcoal layers at around 1 m below the bed may represent distinct historical changes in the river bed. No consolidated materials were found at the core sites, which could give an indication of the limit of scour.

- Measurements of long sections identified regular series of asymmetric bedforms (sand dunes with an amplitude of about 1 m and wavelength of 16 m) orientated downstream. These bedforms were located between Sackville and Wisemans Ferry, generally in the straight reaches of the river. These bedforms may be related to flood event(s) where river bed velocities are sufficiently high to mobilise the bed sediments. Charcoal layers at around 1 m below the bed in the cores at Sackville may be related to the episodic migration of these bedforms.

- Of the 16 cross-sections surveyed some sections showed minor accretion while the remainder showed similar cross-sections to those previously surveyed in 1984. This confirms the findings of Scholer (1974), which concluded that there has been no net change of the bed levels.

In summary, the above collected field data indicated active bed formations and sediment movement of the river bed. This movement of the river bed is continuous over the area surveyed. There is potential for significant scouring of the river bed which would only be limited by the magnitude of the velocity and the duration of the flood. The evidence from the cores shows bed scour of at least 1 m occurring, however, the scour profiles tend to eventually recover whereby no net changes in bed levels result.

Based on the field survey results a 1.0 m scour from Sackville to the ocean was tested in the model. The results are shown in Table 9.3.

Table 9.3 1% AEP Design Flood Levels Sensitivity to Bed Scour

Location	1% AEP with Bed Levels as Surveyed (m AHD)	Change in Water Level for 1% AEP and 1.0m scour (m)
Sackville	12.9	-0.3
Lower Portland	10.3	-0.3
Leets Vale	7.6	-0.3
Wisemans Ferry	6.7	-0.3
Gunderman	5.2	-0.3
Spencer	2.7	-0.2
Brooklyn	1.7	0.0

The results indicate that peak flood levels would be generally reduced by about 0.3 m upstream of Gunderman if river bed scour was assumed.

9.2.3 Sensitivity to Ocean Levels

Table 9.4 indicates the 1% AEP flood levels using the 1% AEP tide level and an average recurrence interval tide level (ie a one-year tide with a peak ocean level of about 0.8 m AHD).

Table 9.4 1% AEP Design Flood Levels Sensitivity to Ocean Levels

Location	1% AEP with 1% AEP tide (m AHD)	Change in Water Level for 1% AEP and a 1-year tide (m)
Sackville	12.9	0.0
Lower Portland	10.3	0.0
Leets Vale	7.6	-0.1
Wisemans Ferry	6.7	-0.1
Gunderman	5.2	-0.4
Spencer	2.7	-0.6
Brooklyn	1.7	-0.7
Ocean	1.7	-0.7

The flood levels below Gunderman are sensitive to the ocean levels. When the 1% AEP flood is combined with the 1% AEP tide level the level at Brooklyn is 0.4 m higher than combining the 1% AEP flood with the average recurrence interval tide. While there is limited data available to confidently identify the most likely combination of flood and tide events, it is clear that the flood levels along the downstream end of the Hawkesbury River are determined by high ocean levels rather than from catchment runoff..

9.2.4 *Sensitivity to Input Flood Hydrographs*

The adopted flood flows at Sackville were based on the revised Sydney Water study (SW 1994). The sensitivity of the modelling results to the overall magnitude of the input flood hydrographs was examined by increasing and decreasing all inflow flood hydrographs in the model by 10%. Table 9.5 shows the results.

Table 9.5 1% AEP Design Flood Levels Sensitivity to Flood Hydrographs

Location	1% AEP with flood hydrographs as estimated (m AHD)	Change in Water Level for a +10% change to flood hydrographs (m)	Change in Water Level for a-10% change to flood hydrographs (m)
Sackville	12.9	+0.7	-0.7
Lower Portland	10.3	+0.6	-0.6
Leets Vale	7.6	+0.5	-0.5
Wisemans Ferry	6.7	+0.5	-0.5
Gunderman	5.2	+0.4	-0.4
Spencer	2.7	+0.2	-0.2
Brooklyn	1.7	+0.1	0.0

Flood levels at Wisemans Ferry varied by about ±0.5 metres highlighting the model sensitivity to flows and emphasising the need for accurate flow measurements of major flood events at Sackville.

9.2.5 *Sensitivity to Bed Roughness*

The bed friction of the Hawkesbury River is simulated in the hydraulic model by the Mannings 'n' roughness coefficient. In the calibration stage of the model the Mannings 'n' values calculated were 0.026, 0.020 and 0.018 for the river and 0.038 for the floodplain. To examine the sensitivity of the hydraulic model to bed roughness the Mannings 'n' values for the main channel were altered ±10%. The results are tabulated in Table 9.6.

Table 9.6 1%AEP Design Flood Levels Sensitivity to Bed Roughness

Location	1%AEP with n values as calibrated (m AHD)	1%AEP Changes in Water Level for a +10% 'n' (m)	1%AEP Changes in Water Level for a -10% 'n' (m)
Sackville	12.9	+0.3	-0.3
Lower Portland	10.3	+0.3	-0.3
Leets Vale	7.6	+0.3	-0.3
Wisemans Ferry	6.7	+0.2	-0.2
Gunderman	5.2	+0.2	-0.2
Spencer	2.7	+0.1	-0.10
Brooklyn	1.7	0.0	0.0

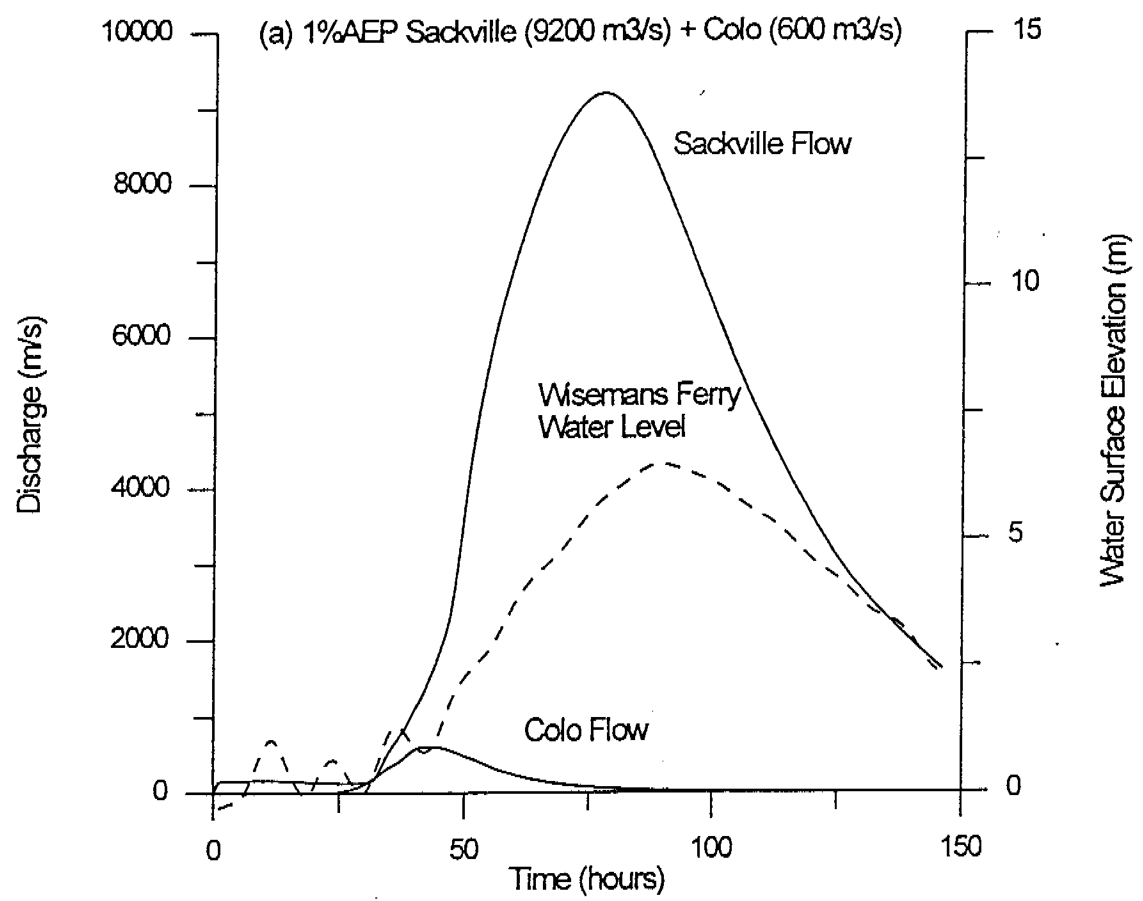
These results show that the resultant flood levels vary by up to ± 0.3m with a ± 10% change in bed roughness. Importantly, however, variations to the adopted bed roughness also cause changes to the phasing or relative timing of the flood peaks.

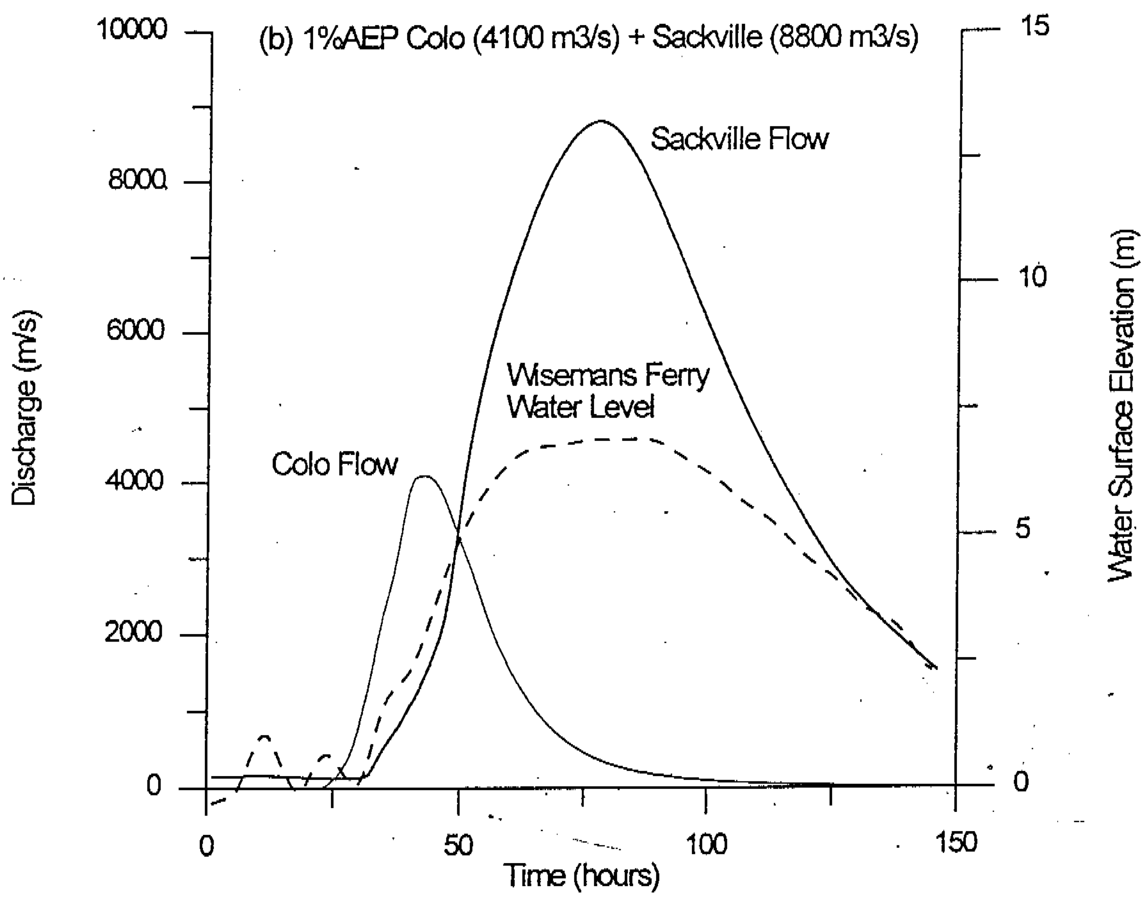
9.2.6 *Summary of Sensitivity Studies*

Table 9.7 summarises the range of changes to water level from the sensitivity study.

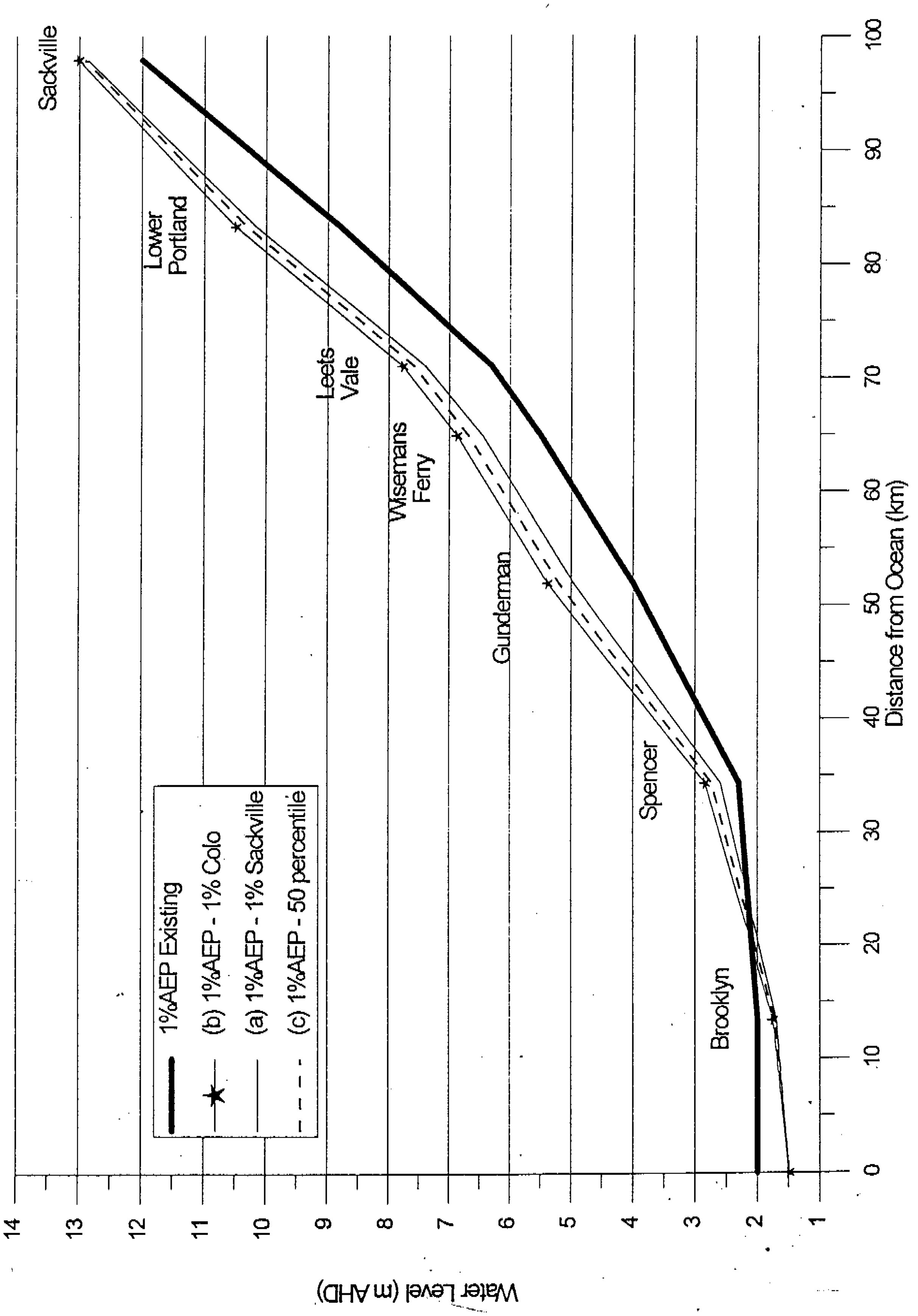
Table 9.7 Summary of Water Level Changes

Location	No Tributary Flow (m)	Bed Scour (m)	Ocean Level (m)	Increased Flow (m)	Decreased Flow (m)	Increased Roughness (m)	Decreased Roughness (m)
Sackville	-0.1	-0.3	0.0	+0.7	-0.7	+0.3	-0.3
Lower Portland	-0.2	-0.3	0.0	+0.6	-0.6	+0.3	-0.3
Leets Vale	-0.2	-0.3	-0.1	+0.5	-0.5	+0.3	-0.3
Wisemans Ferry	-0.2	-0.3	-0.1	+0.5	-0.5	+0.2	-0.2
Gunderman	-0.2	-0.3	-0.4	+0.4	-0.4	+0.2	-0.2
Spencer	-0.1	-0.2	-0.6	+0.2	-0.2	+0.1	-0.1
Brooklyn	0.0	0.0	-0.8	+0.1	0.0	0.0	0.0

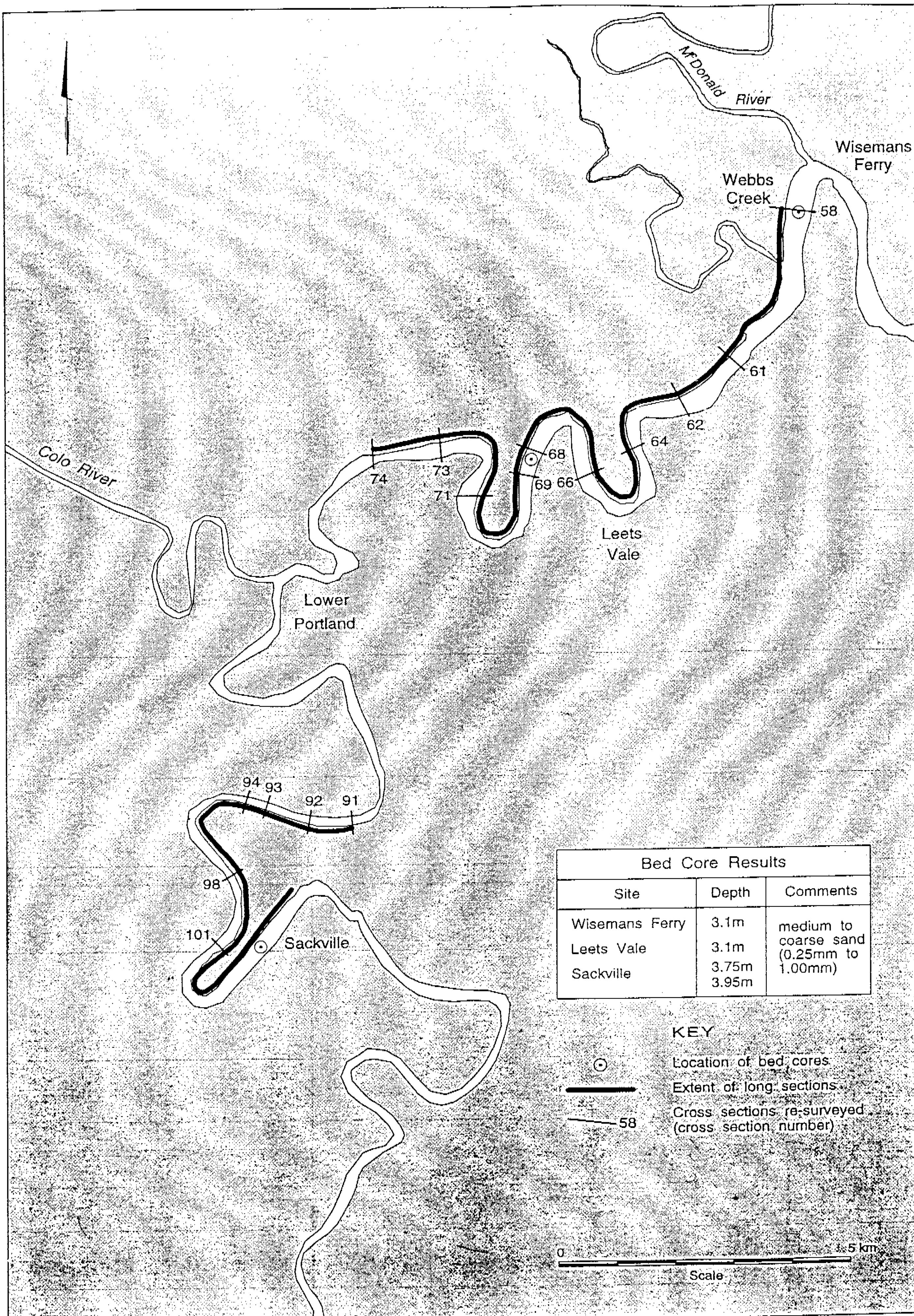




AWACS Report No. 94/29	1% AEP COMPARISON OF FLOOD HYDROGRAPHS	Figure 9.1



AWACS Report No. 94/29	1% AEP IMPACT OF PEAK FLOW FROM COLO OR SACKVILLE	Figure 9.2



Bed Core Results

Site	Depth	Comments
Wisemans Ferry	3.1m	medium to coarse sand (0.25mm to 1.00mm)
Leets Vale	3.1m	
Sackville	3.75m 3.95m	

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FIELD INVESTIGATION OF BED SCOUR

Figure 9.3

10. Design Flood Conditions

10.1 Best Estimate Design Flood Levels

The two-dimensional numerical model has the ability of simulating complex flood behaviour, allowing the estimation of flood levels and flow velocities for design events, along the lower reaches of the Hawkesbury River. This provides local councils and their floodplain management committees with sufficient information to enable them to examine flood hazards in managing the floodplain. However, more detailed information on flows and velocities across specific locations on the floodplain can be obtained by increasing the level of survey data incorporated in the model. This might be worthwhile when assessing various types of future development options.

The numerical model was calibrated and verified using the most recent information currently available. Design rainfall runoff hydrographs were inputed into the model to simulate design flood conditions. The best estimate of flood levels for a range of design events and selected locations are shown in **Table** 10.1 and **Figures 10.1 to 10.6.**

Table 10.1 Best Estimate Design Flood Levels

Location	1% AEP (m AHD)	2% AEP (m AHD)	5% AEP (m AHD)	20% AEP (m AHD)	PMF (m AHD)
Sackville (Sackville Ferry)	12.9	11.7	9.9	7.6	25.4
Lower Portland	10.3	9.1	7.5	5.5	22.3
Leets Vale	7.6	6.5	5.2	3.8	17.9
Wisemans Ferry (Webbs Creek Ferry)	6.7	5.6	4.4	3.2	16.3
Gunderman	5.2	4.3	3.3	2.4	14.0
Spencer	2.7	2.3	1.9	1.7	8.0
Brooklyn	1.7	1.6	1.5	1.5	3.3

The sensitivity studies also highlighted a number of important factors that could influence flood behaviour and design flood levels. The major factors were found to include:

- magnitude of the design flood flow hydrographs at Sackville
- degree of bed scour associated with various size Hawkesbury River floods, and
- ocean tailwater levels (from Gunderman to Brooklyn).

The first two factors can only be addressed to a better degree through further information on river scour and flow measurements being available.

10.2 Comparison of the Estimates of the 1% AEP Flood Profile and Historical Flood Levels

The design flood profiles for the 1% AEP as presented in Table 10.2 and shown in Figure 10.7 are higher than the previous estimates adopted by councils for planning controls in flood risk areas.

Table 10.2 1%AEP Design Levels Compared to Historical Flood Levels

Location	1% AEP Best Estimate (m AHD)	1978 (m AHD)	*Δ 1% (m)	1990 (m AHD)	Δ 1% (m)	Existing 1% AEP (m AHD)	Δ 1% (m)
Sackville	12.9	10.7	2.2	10.0	2.9	12.0	0.9
Lower Portland	10.3	8.7	1.6	7.5	2.8	8.8	1.5
Leets Vale	7.6					6.3	1.3
Wisemans Ferry	6.7	4.8	1.9	4.3	2.4	5.5	1.2
Gunderman	5.2			2.7	2.5	4.0	1.2
Spencer	2.7	1.5	1.2			2.3	0.4
Brooklyn	1.7					2.0	-0.3

*Δ Differences between 1% AEP and nominated flood

Information on Hawkesbury flooding is generally extensive at Windsor, in terms of period of record and reliability. It was also far more comprehensive than that available for downstream locations, consequently earlier estimates of design flood events relied on a frequency analysis of historical flood level records at Windsor. The Public Works estimates which have been widely adopted by local councils for more than a decade were based on applying the flood freqency relationship at Windsor to downstream locations by correlating the recorded flood levels at Windsor with levels recorded at these locations. The number of flood events where flood levels had been recorded simultaneously at Windsor and the downstream locations was limited.

With regard to the 1% AEP flood, the downstream estimates would have been based on a 1% AEP flood level of 16.0 m AHD at Windsor. As such the Sackville 1% AEP flood estimate was 12 m AHD.

More recently, Sydney Water recommended the 1% AEP flood level at Windsor be revised upwards to 17.2 m AHD in light of new rainfall predictions and flood data. The Sydney Water study (SW 1994) included the reaches downstream to Sackville and corresponding to the upward revision at Windsor, the 1% AEP flood level at Sackville was found to be 13.2 m AHD.

The estimated 1% AEP design flood profiles compare favourably with those recorded for the 1978 and 1990 events (Figure 10.7 and Table 10.2). Both these events were surveyed to known datums and are a good record of the profile from Sackville to the ocean.

The difference between the modelled 1% AEP levels and the earlier Public Works estimates is more pronounced downstream of the Colo and Hawkesbury Rivers confluence. This illustrates the influence of the Colo River on flood levels along the Hawkesbury River.

10.3 0.5% AEP Design Flood Levels

As there is a need to take into consideration the consequences of flooding larger than those events usually applied in controlling development, this study provides estimates of the 0.5% AEP design event at a number of locations. Stage frequency curves were plotted for each site based on the results of the hydraulic model results. An AEP of 1 in 100,000 was assigned to the stage of the PMF as was adopted in the Sydney Water study (SW 1994). AR&R (1987) provides guidelines on fitting curves to flood discharge frequency distributions and recommends that the frequency line should become horizontal towards the PMF. For this study, a straight line was fitted to the stage frequency data points. Straight lines were fitted between the 1% AEP level and the PMF level for the Gunderman, Spencer and Brooklyn stage frequency curves. More correctly a curve should be fitted to these distributions to take account of the ocean effects, however this makes very little difference to the resultant 0.5% AEP flood level. Figures 10.8 to 10.11 present the frequency curves. Table 10.3 summarises the results.

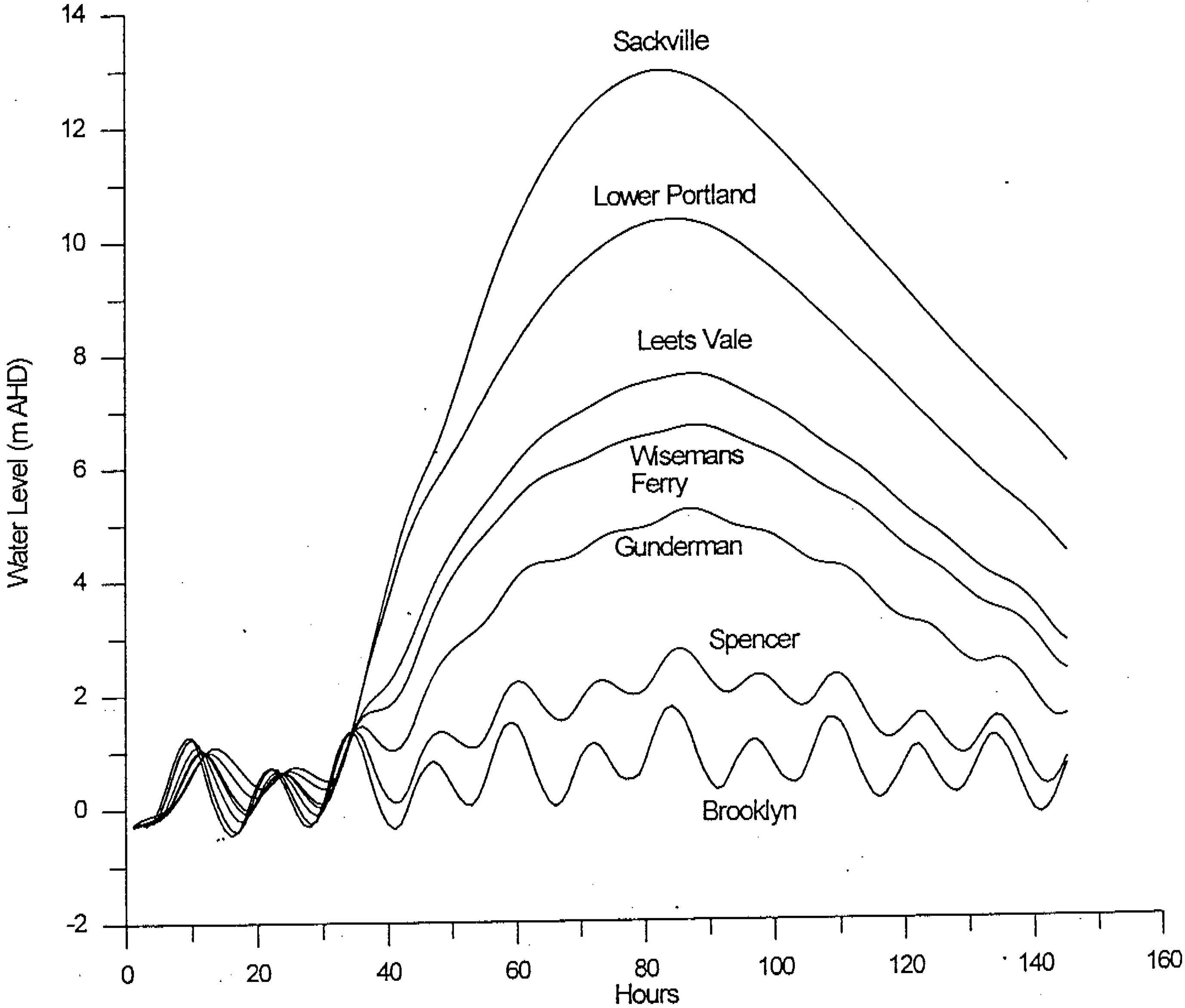
Table 10.3 0.5% AEP Design Flood Levels

Location	Flood Level (m AHD)
Sackville	14.5
Lower Portland	11.4
Leets Vale	8.6
Wisemans Ferry	7.5
Gunderman	5.9
Spencer	3.3
Brooklyn	1.9

10.4 Design Flood Velocities

The major advantage of using the two-dimensional model was the ability to examine the flood velocity distribution across the floodplain. A sample of the distribution is shown in Figure 10.12.

These velocities are mean values over the depth of river; surface velocities would be expected to be slightly higher. It should be noted that simulated velocity distributions on the floodplains are based on topography digitised from 1:25,000 scale maps. As these maps are only accurate to the interpolated 10 metre contours, the velocity distributions across the floodplains for each of the nominated development areas are inaccurate for the purposes of deriving velocity depth ratios at any selected site, however adequate for simulation of flood levels. The velocity distribution for various design floods across the floodplains can be produced when accurate survey information to AHD is available and incorporated into the model.



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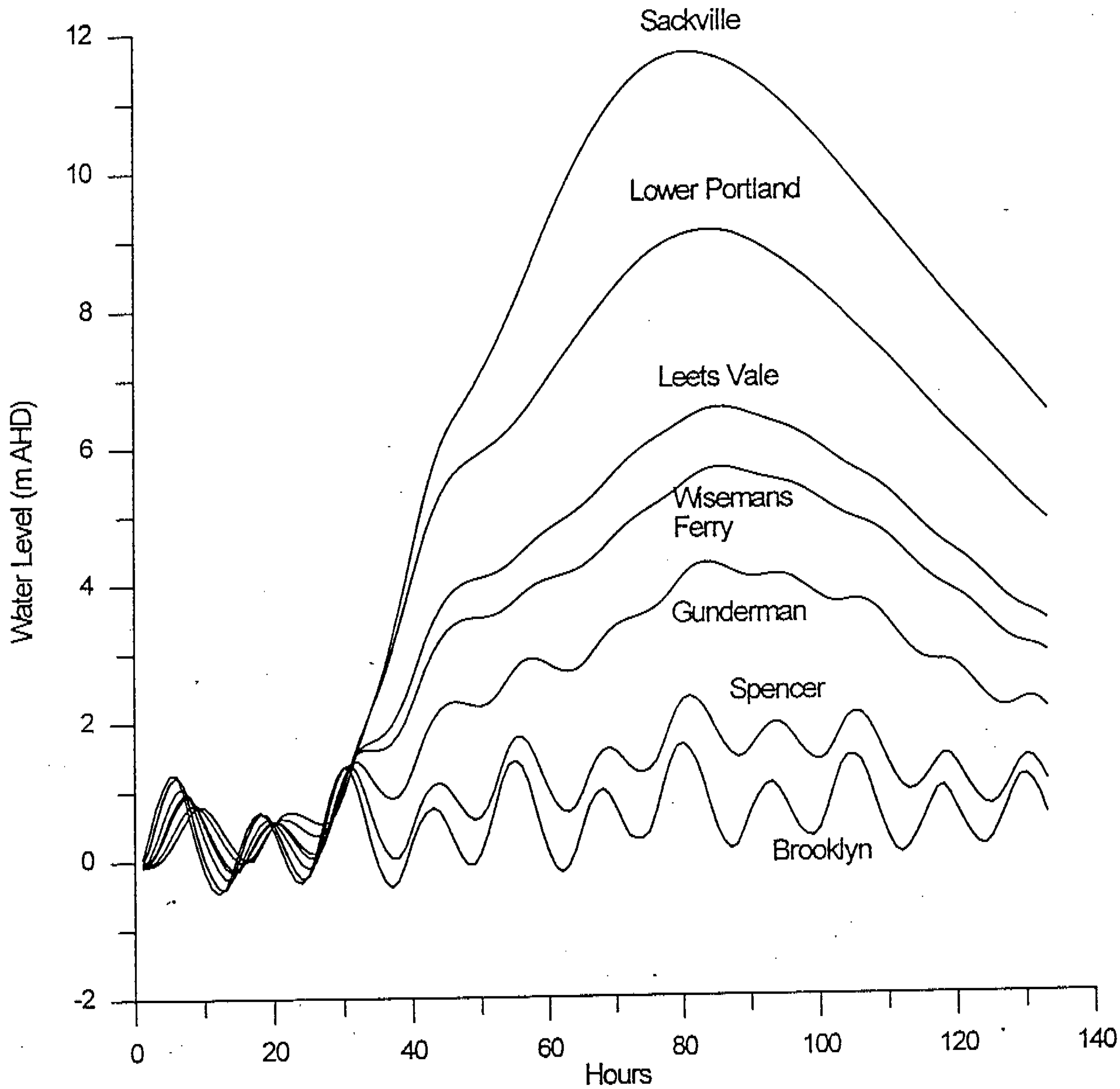
1% AEP
DESIGN STAGE HYDROGRAPH

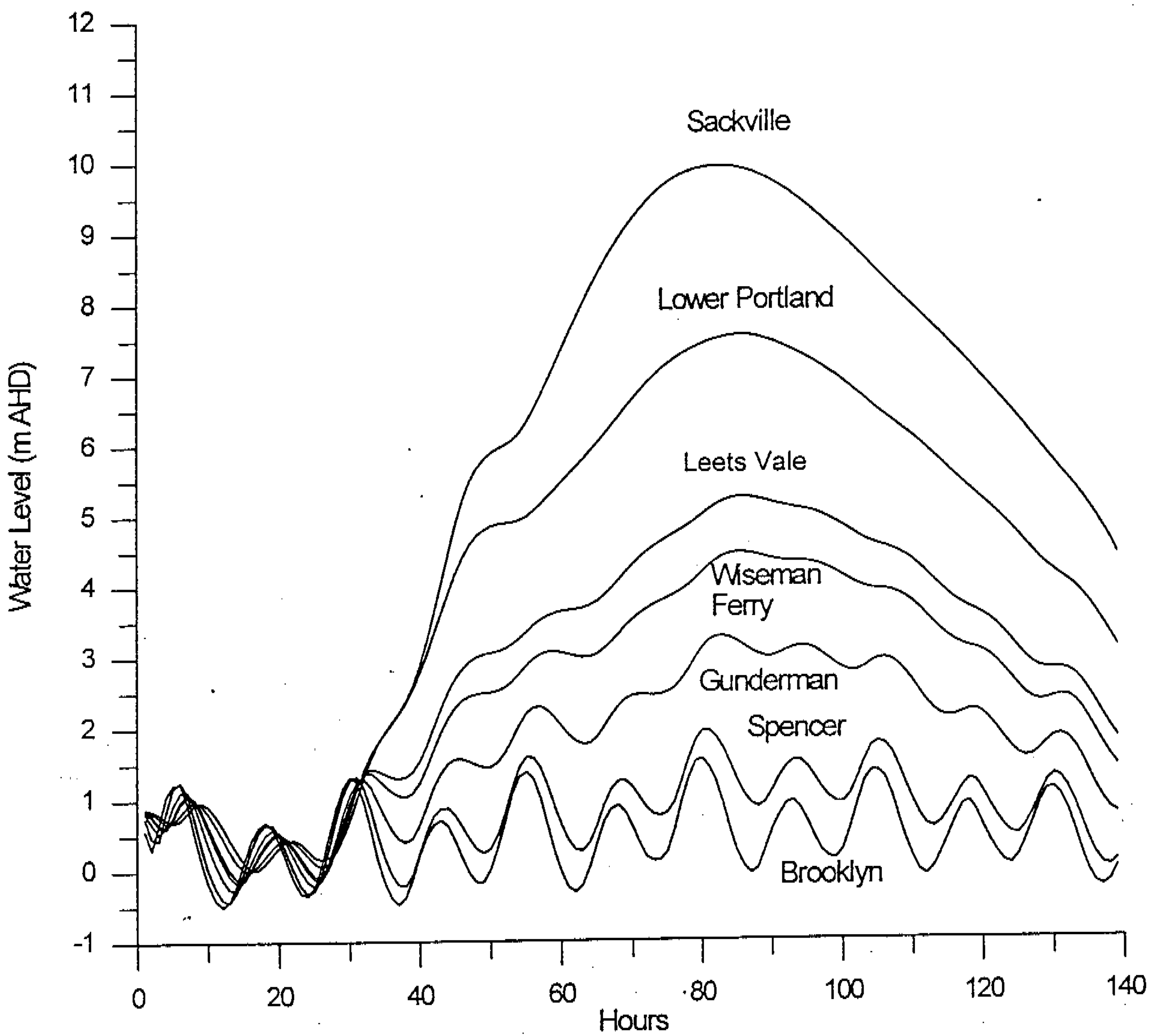
Figure
10.1



AWACS Report No. 94/29

2% AEP
DESIGN STAGE HYDROGRAPH

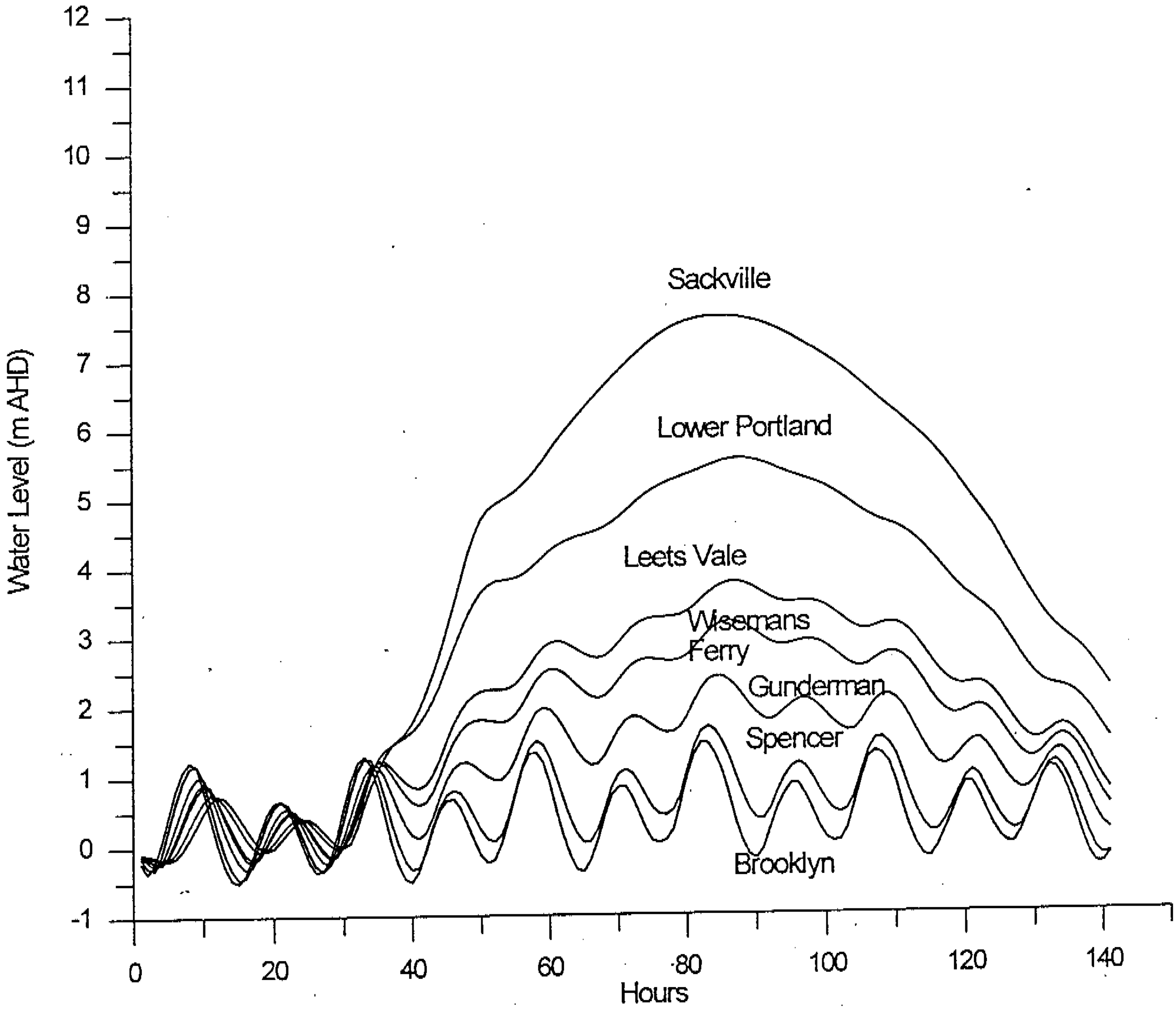
Figure 10.2



AWACS
Report No. 94/29

5% AEP
DESIGN STAGE HYDROGRAPH

Figure
10.3



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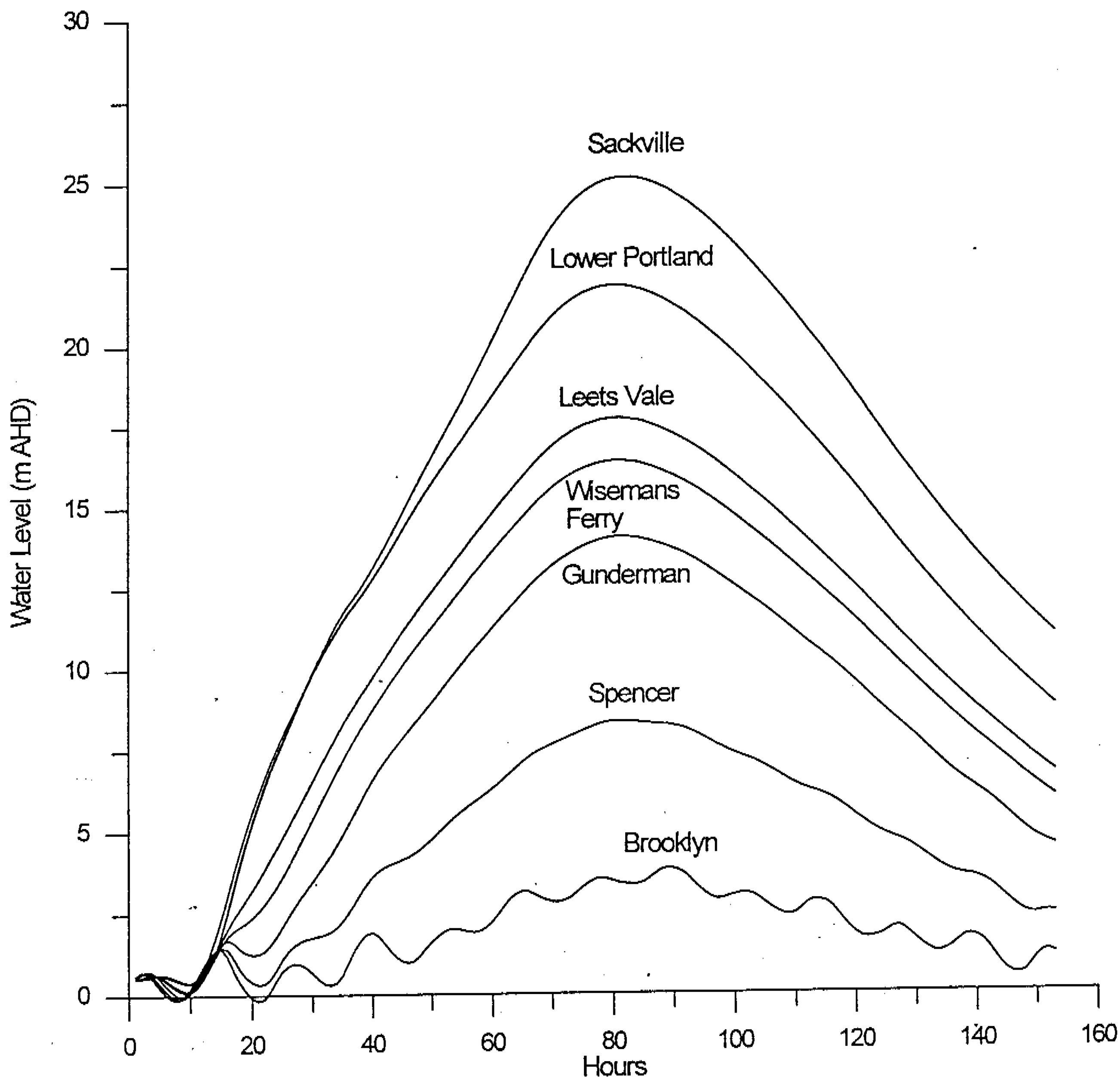
20% AEP

DESIGN STAGE HYDROGRAPH

Figure

10.4



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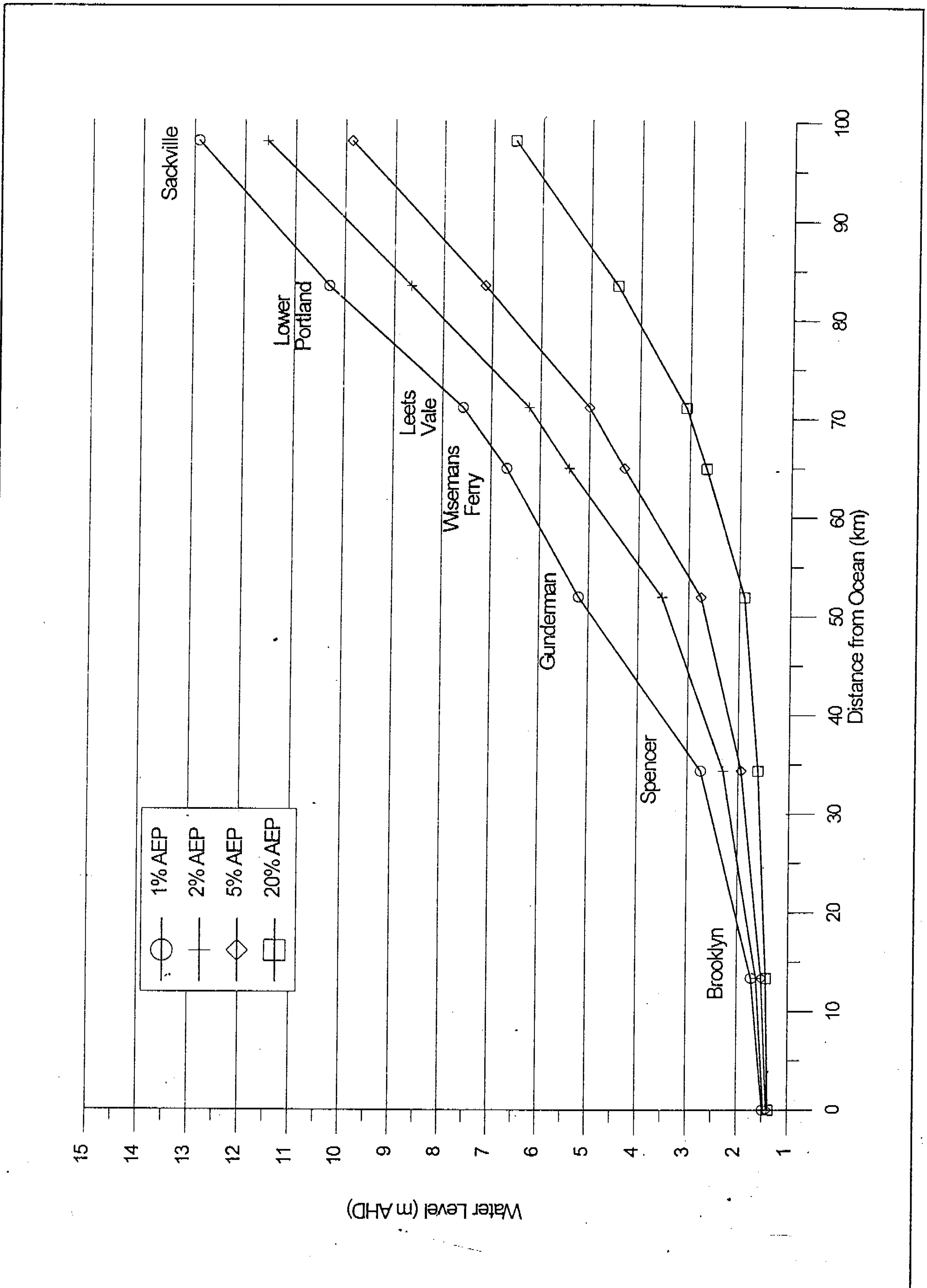
PMF
DESIGN STAGE HYDROGRAPH

Figure 10.5



AWACS
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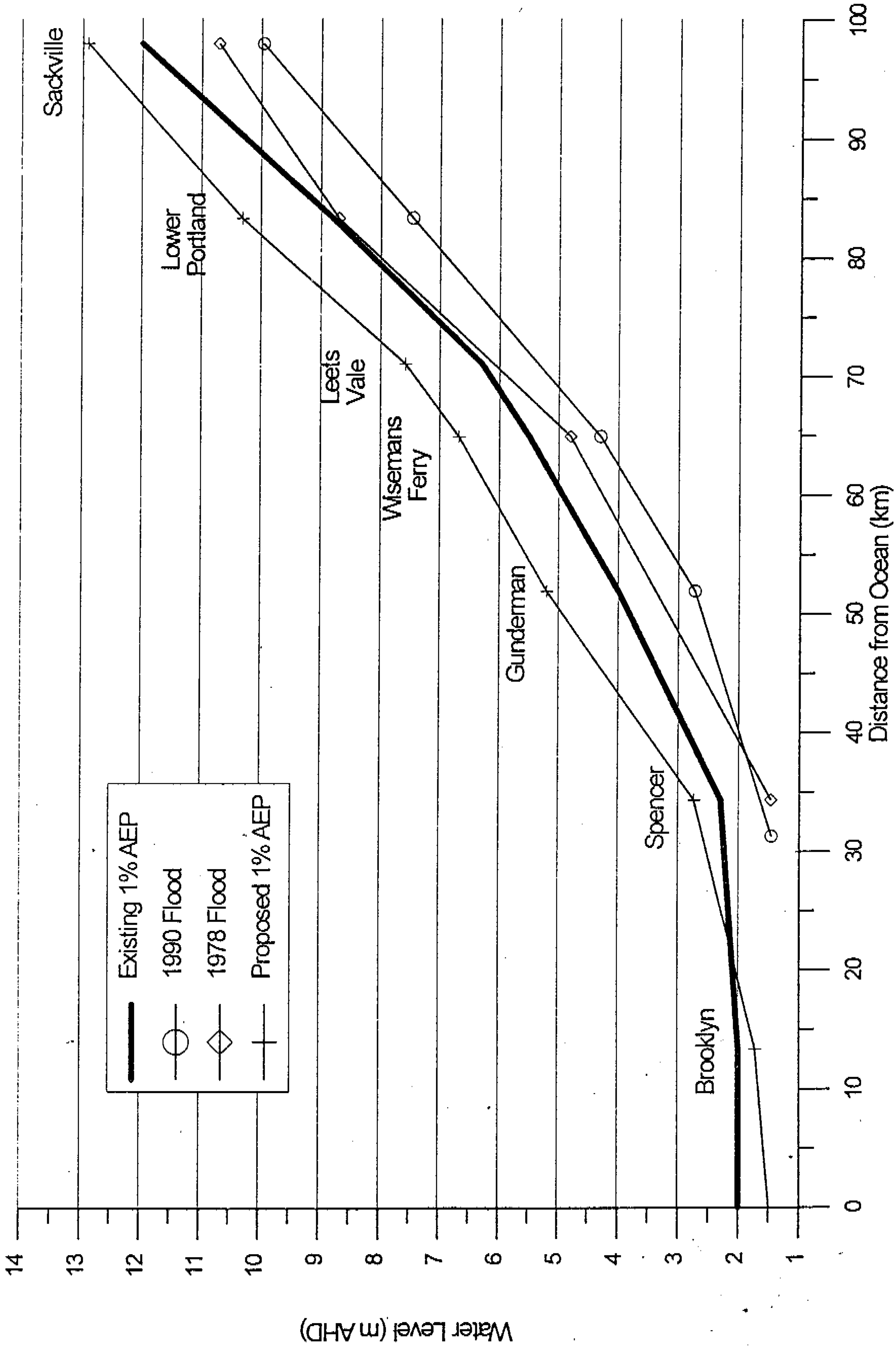
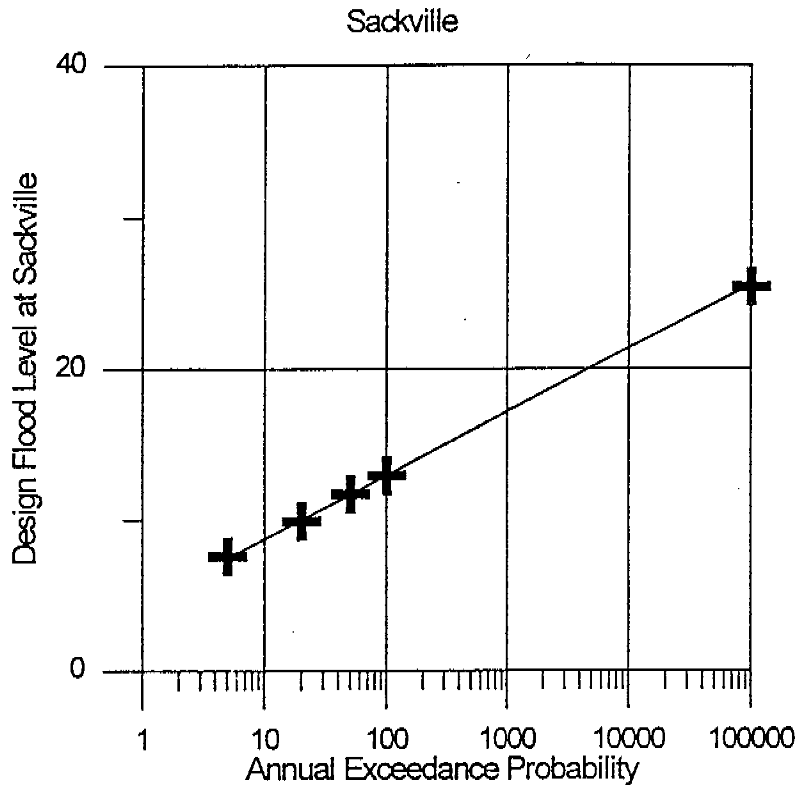
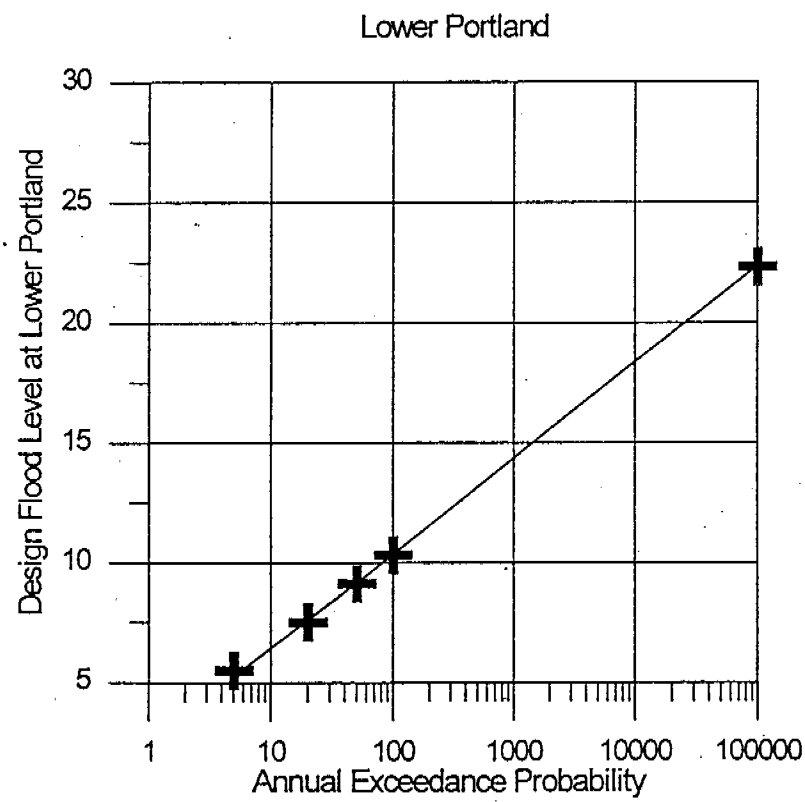
DESIGN FLOOD PROFILES

Figure
10.6



AWACS Report No. 94/29	1% AEP COMPARISON TO HISTORICAL PROFILE	Figure 10.7





AWACS
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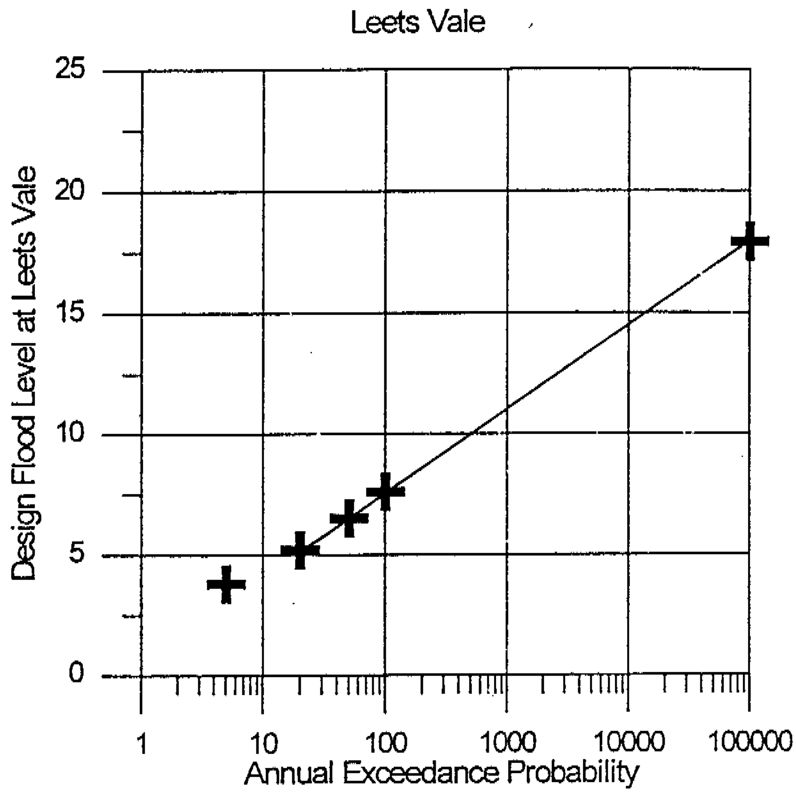
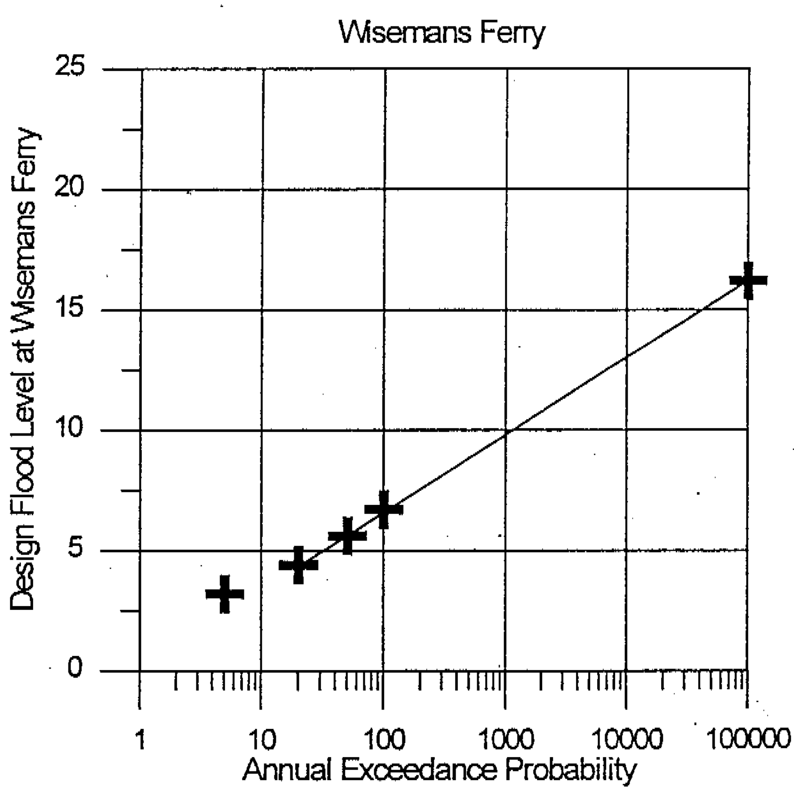
DESIGN FLOOD LEVELS
SACKVILLE AND LOWER PORTLAND

Figure
10.8





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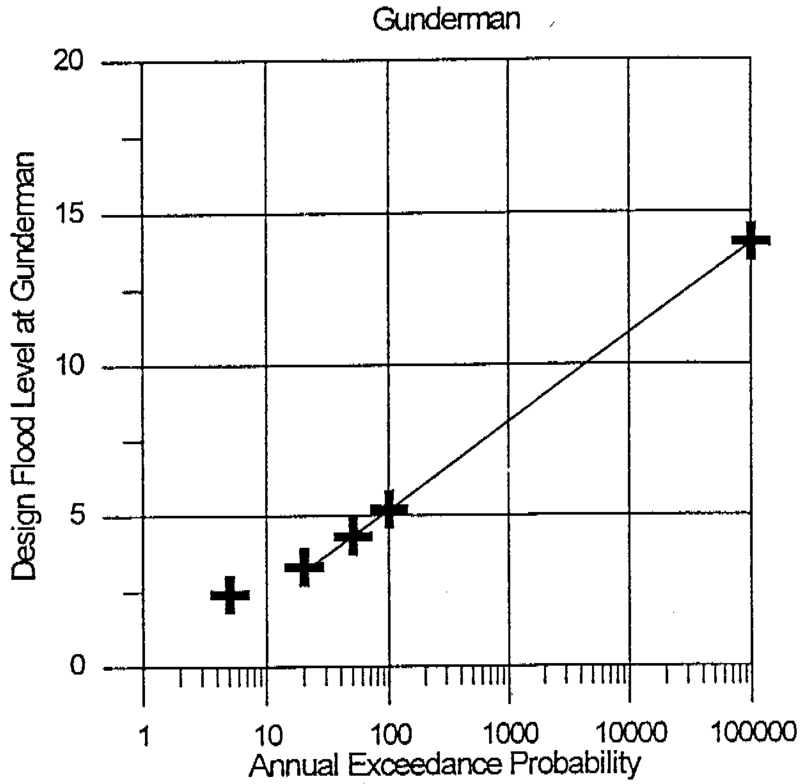
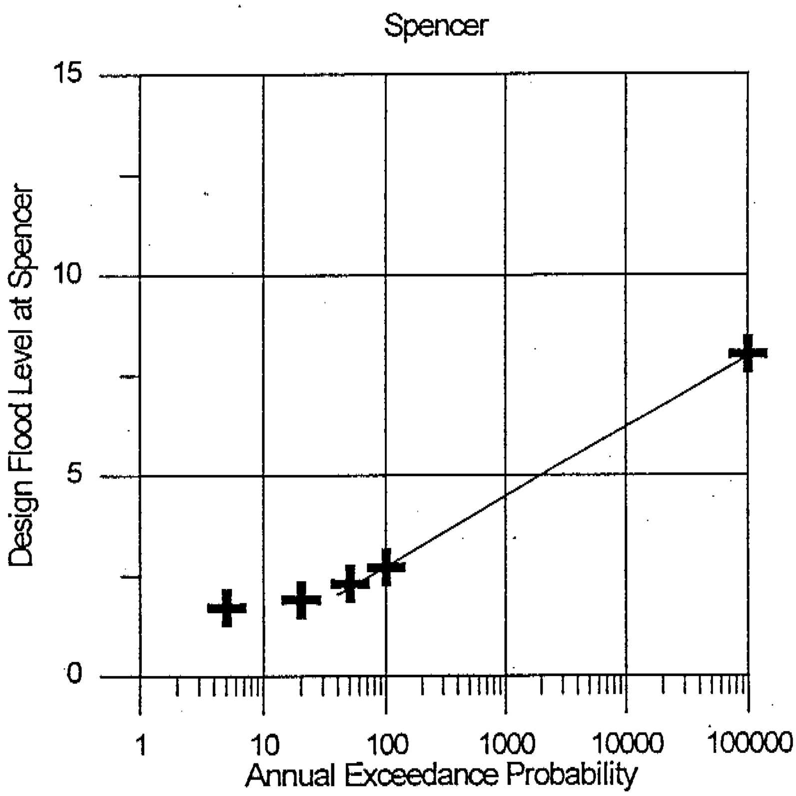
DESIGN FLOOD LEVELS
LEETS VALE AND WISEMANS FERRY

Figure
10.9





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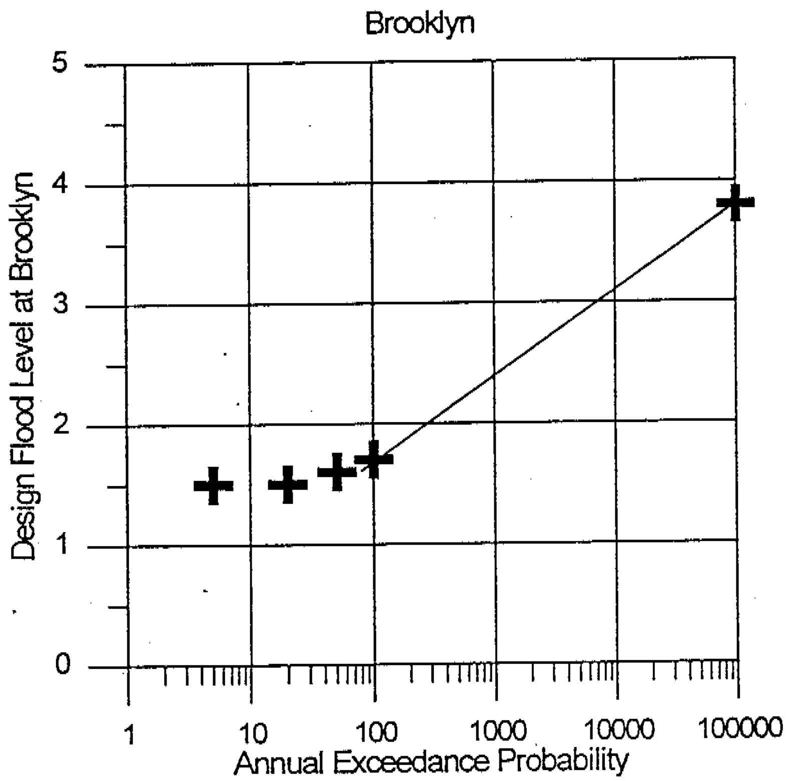
DESIGN FLOOD LEVELS
GUNDERMAN AND SPENCER

Figure
10.10



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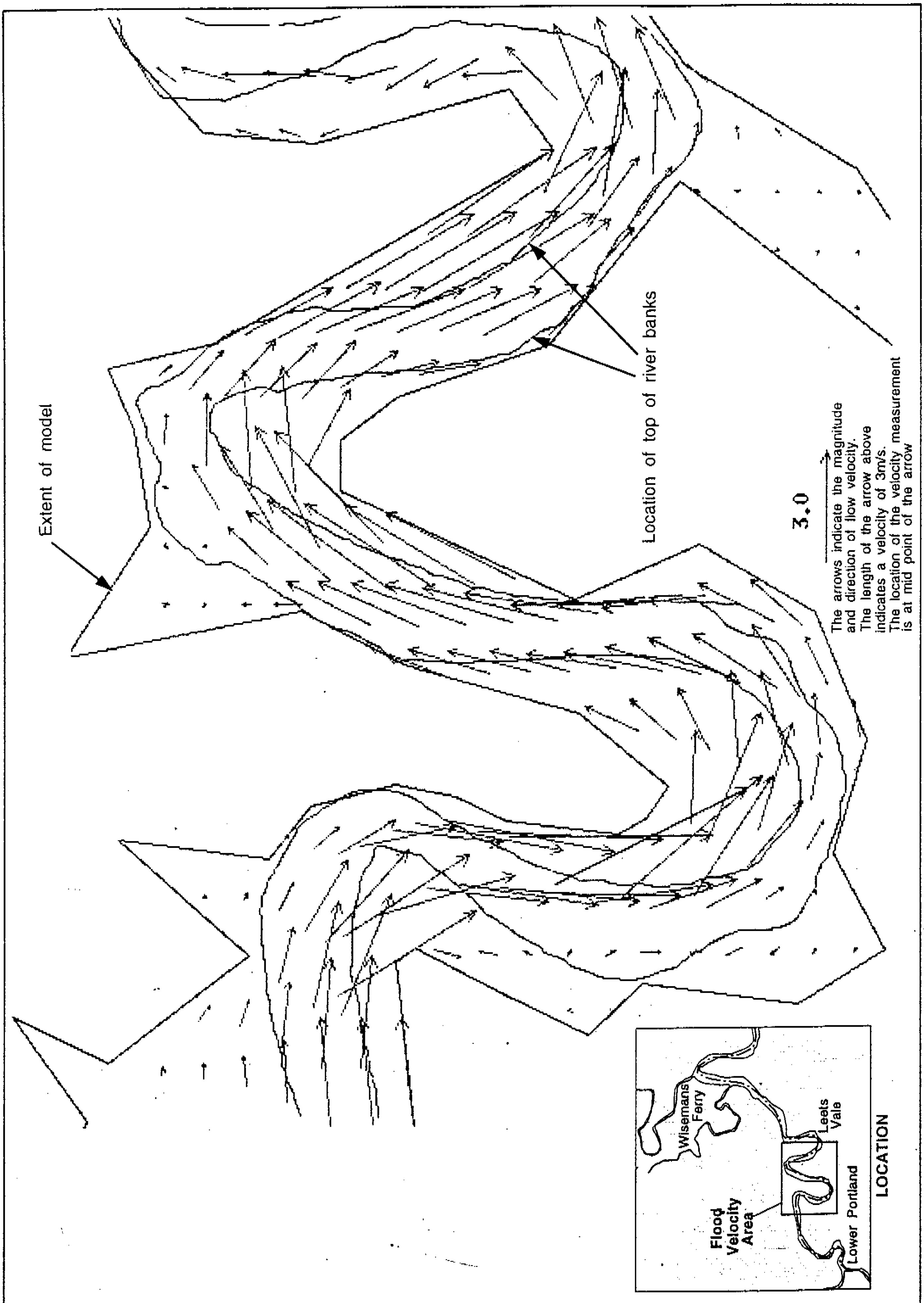
DESIGN FLOOD LEVELS
BROOKLYN

Figure
10.11



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TYPICAL FLOOD VELOCITY DISTRIBUTION

Figure 10.12

11. Recommendations

The results of this study have derived the best estimates of the flood profiles in the Lower Hawkesbury River given the information that was available. There are a number of processes that occur within the Hawkesbury River during flood that could be monitored to improve the calibration of the hydraulic model.

The issues of bed scour, combinations of Hawkesbury and Colo Rivers flood flows and the estimates of Sackville flows were sensitivity tested to examine their impacts on the best estimate design flood levels. Their impacts on the flood levels ranged from ±0.7 m at some locations to ±0.1 m at other sites. To improve the accuracy and confidence in the estimated design flood levels, the following flood characteristics should be monitored and in the future used to refine the calibrated hydraulic model:

i. continuation of the water level monitoring stations
ii. gauge highs and flows at Upper Colo River station
iii. gauge highs and flows at Sackville station, and
iv. monitoring of flood scour and cross-sections at Sackville.

The first recommendation would ensure that future flood levels are captured and used to expand the existing database on Lower Hawkesbury River floods. The second recommendation would improve the AWACS extrapolated rating and better quantify how Colo River flood flows are impacting on the Hawkesbury River. The third recommendation would ensure all flood flows are captured, providing a database of low to medium flows to be included in the Sackville rating curve. The fourth recommendation would improve the Public Works' rating at Sackville and monitor bed scour and accretion processes over the various stages of a flood.

The simulated velocities distribution for the 1% AEP flood as shown in Figure 10.12 are an example of the output from the hydraulic model. These velocity distributions can be generated for any location along the river, however, to accurately represent the magnitude of the velocities, detailed land surveys of the floodplain would be required. It is therefore recommended that in areas where site specific details of flow velocity are required, additional survey details on the floodplain arrangement be collected and incorporated in the flood model to generate detailed velocity regimes across the floodplains.

12. Acknowledgments

This study was funded by the Department of Urban Affairs and Planning and managed by DLWC. The four councils of Hornsby, Baulkham Hills, Gosford City and Hawkesbury City and local residents provided data on flood conditions within their respective areas.

AWACS would like to acknowledge the technical assistance and guidance of Professor David Pilgrim and Doctor James Ball during this study. The numerical modelling for this project was shared between Ari Roizenblit and Peter Horton.

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14. Glossary of Terms

annual exceedance probability	The chance or likelihood that an event of a nominated size or greater (eg flood discharge) will occur in any year.
Australian Height Datum (AHD)	A common national plane of level corresponding approximately to mean sea level.
bathymetry	The measurement of depths of water; also information derived from such measurements.
bed load	That portion of the total sediment load that flowing water moves along the bed by the rolling sediment particles .
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.
damage potential	The susceptibility of coastline development to damage by coastline hazards.
design floor level (DFL)	A minimum floor level specified as part of a building control program.
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. **infill development:** refers to the development of vacant blocks of land that are generally surrounded by developed properties. **new development:** refers to development of a completely different nature to that associated with the former land use. For example, the urban subdivision of an area previously used for rural purposes. New developments typically require major extensions of existing urban services, such as roads, water supply, sewerage and electric power. **redevelopment:** refers to the rebuilding of an area. For example, as urban areas age, it may become necessary to demolish and reconstruct buildings on a relatively large scale. Redevelopment generally does not require major extensions to urban services.

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.

ebb tide The outflow of coastal waters from bays and estuaries caused by the falling tide.

effective warning time Is equal to the available warning time, less the time taken to alert flood-affected people (by radio, television, loud-hailer or word of mouth), and have them commence effective evacuation procedures.

estuary An enclosed or semi-enclosed body of water having an open or intermittently open connection to coastal waters in which water levels vary in a periodic fashion in response to ocean tides.

flood awareness An appreciation of the likely effects of flooding and a knowledge of the relevant flood warning and evacuation procedures. In communities with a high degree of flood awareness, the response to flood warnings is prompt and efficient. In communities with a low degree of flood awareness, flood warnings are liable to be ignored and residents are often confused about when to evacuate, what to take and where it should be taken.

flood fringe The remaining area of land affected by flooding, after floodway and flood storage areas have been defined.

flood hazard Potential for damage to property or persons due to flooding.

flooding The State Emergency Service uses the following definitions in flood warnings:

minor flooding: causes inconvenience such as closing of minor roads and the submergence of low level bridges. The lower limit of this class of flooding, on the reference gauge, is the initial flood level and the upper limit is determined by local conditions.

moderate flooding: low-lying areas are inundated requiring removal of stock and/or evacuation of some houses. Main traffic bridges may be covered. The range on the reference gauge is determined by local conditions.

	major flooding: extensive rural areas are flooded with properties, villages and towns isolated and/or appreciable urban areas are flooded. The threshold for this class of flooding is the upper limit of moderate flooding .
flood-liable land	Land which would be inundated as a result of the standard flood.
flood mitigation works	Structures that are designed to manage floodwaters (eg. levees, retarding basins).
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 management.
floodplain management options	The measures which might be feasible for the management of a particular area.
flood proofing	A combination of measures incorporated in the design and/or construction and alteration of individual buildings or structures subject to flooding, for the reduction or elimination of flood damages.
flood standard (or designated flood)	should be based on an understanding of flood behaviour and the associated flood risk. It should also take into account social, economic and ecological considerations.
flood storages	Those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood.
flood tide	The inflow of coastal waters into bays and estuaries caused by the rising tide.
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, areas of deeper flow or the areas where higher velocities occur

fluvial	Pertaining to non-tidal flows.
foreshore	The area of shore between low and high tide marks and land adjacent thereto.
freeboard	A factor of safety usually expressed as a height above the designated flood. Freeboard tends to compensate for factors such as wave action, localised hydraulic effects etc.
geomorphology	The study of the origin, characteristics and development of land forms.
high hazard	Possible danger to life and limb; evacuation by trucks difficult; potential for structural damage; social disruption and financial losses could be high.
hydraulic	The term given to the study of water flow in a river, in particular the evaluation of flow parameters such as stage and velocity.
hydrograph	A graph which shows how the discharge changes with time at any particular location.
hydrology	The term given to the study of the rainfall and runoff process as it relates to the derivation of hydrographs for given floods.
low hazard	Should it be necessary, people and their possessions could be evacuated by trucks. Able-bodied adults would have little difficulty wading.
mainstream flooding	Inundation of normally dry land occurring when water conveyed to the locality from further upstream overflows the natural or artificial banks of the principal watercourse in the catchment. It generally excludes any watercourses constructed with pipes or artificial channels or considered as stormwater channels.
management plan	A document including, as appropriate, both written and diagrammatic information describing how a particular area of land is to be used and managed to achieve defined objectives. It may also include description and discussion of various issues, problems, special features and values of the area, the specific management measures which are to apply and the means and timing by which the plan will be implemented.

mathematical/computer models	The mathematical representation of the physical processes involved in runoff and streamflow. These models are often run on computers due to the complexity of the mathematical relationships. In this manual, the models referred to are mainly involved with rainfall, runoff and stream flow.
neap tides	Tides with the smallest range in a monthly cycle. Neap tides occur when the sun and moon lie at right angles relative to the earth (the gravitational effects of the moon and sun act in opposition on the ocean).
numerical model	A mathematical representation of a physical, chemical or biological process of interest. Computers are often required to solve the underlying equations.
peak discharge	The maximum discharge occurring during a flood event.
probable maximum flood	The flood calculated to be the maximum which is likely to occur.
probability	A statistical measure of the expected frequency or occurrence of flooding.
runoff	The amount of rainfall which actually ends up as streamflow, also known as rainfall excess.
sediment load	The quantity of sediment moved past a particular cross-section in a specified time.
significant wave height	The average height of the highest one third of waves recorded in a given monitoring period. Also referred to as $H_{1/3}$ or H_s
spring tides	Tides with the greatest range in a monthly cycle, which occur when the sun, moon and earth are in alignment (the gravitational effects of the moon and sun act in concert on the ocean) .
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.
storm surge	The increase in coastal water levels caused by the barometric and wind setup effects of storms. Barometric setup refers to the increase in coastal water levels associated with the lower atmospheric pressures characteristic of storms. Wind setup refers to the increase in coastal water levels caused by an onshore wind driving water shorewards and piling it up against the coast.
tides	The regular rise and fall of sea level in response to the gravitational attraction of the sun, moon and planets. Tides along the New South Wales coastline are semi-diurnal in nature, i.e. they have a period of about 12.5 hours.
total catchment management (in the context of the Catchment Management Act, 1989)	"The coordinated and sustainable use of land, water, vegetation and other natural resources on a water catchment basis so as to balance resource utilisation and conservation".
water surface profile	A longitudinal plot showing the flood stage at any given location along a watercourse.
wave setup	The increase in water level within the surf zone above mean still water level caused by the breaking action of waves.

Appendix A
Historical Flood Data

Table A1
Lower Hawkesbury River
Recorded Peak Heights

Date of Flood at Windsor	Windsor (m AHD)	Colo River (m AHD)	Macdonald River (m AHD)	Sackville (m AHD)	Lower Portland (m AHD)	Wisemans Ferry (m AHD)
23 Jun 1867	19.68			16.1		6.0
25 Dec 1909		15.1				
15 Jan 1910		9.2				
22 Jan 1910		4.3				
20 Jul 1910	6.56	6.7				
13 Jan 1911	8.31	6.8				
5 Feb 1911		7.0				
14 Feb 1911		6.1				
22 Feb 1911		5.5				
27 Jul 1912	6.86	6.3				
Aug 1912	7.47					
7 April 1913		6.1				
14 May 1913	8.47	14.6				
20 May 1913		9.4				
24 Mar 1914	7.01					
19 Oct 1914		6.2				
31 Dec 1914		6.7				
Jan 1915	8.04					
8 May 1915		3.5				
5 Oct 1916	10.98					
7 Oct 1916		5.1				
17 Nov 1917		4.4				
4 Jan 1918		6.9				
13 Jan 1918	6.40					
3 Mar 1919		4.0				
13 Dec 1920	7.39					
22 Dec 1920		5.2				
7 Apr 1921		7.7				
20 May 1921	2.44					
2 Jul 1921		9.8				
16 Aug 1921		5.5				
27 Dec 1921		6.1				
15 Jan 1922		5.5				
25 Jul 1922	9.61					

Table A1
Lower Hawkesbury River
Recorded Peak Heights

Date of Flood at Windsor	Windsor (m AHD)	Colo River (m AHD)	Macdonald River (m AHD)	Sackville (m AHD)	Lower Portland (m AHD)	Wisemans Ferry (m AHD)
27 Jul 1922		11.3				
9 Jul 1923		3.1				
22 Nov 1924		3.7				
11 May 1925	8.62					
22 Jun 1925	11.53	6.6				
25 Mar 1926		10.4				
17 Apr 1927	7.23	9.9				
29 Nov 1927		6.9				
17 Feb 1928	6.57	8.1				
26 Jul 1928	5.66					
13 Feb 1929	8.07	7.4				
11 Sep 1929		5.2				
16 Oct 1929	8.58	9.5				
20 Jun 1930	6.27	10.0				
24 Apr 1931	7.01	10.67				
29 May 1931		5.3				
7 Jul 1931		6.5				
19 Sep 1932		3.5				
26 Jul 1933		4.9				
22 Feb 1934	9.30					
26 Aug 1938	7.42					
28 Mar 1942	7.53	14.49				
15 Oct 1942	6.44	12.83				
7 Nov 1942		7.5				
15 May 1943	10.28	9.58				
3 Sep 1943		4.76				
25 Aug 1944		3.5				
14 Jun 1945	8.55	10.44				
16 Apr 1946		14.5				
15 Dec 1947	5.96					
3 May 1948		6.1				
Jan 1949	7.49					
May 1949	5.04					
22 Jun 1949	12.14	17.4	8.8	8.4	6.8	3.1
19 Jan 1950	9.12	8.36				
6 Feb 1950		14.5				

Table A1
Lower Hawkesbury River
Recorded Peak Heights

Date of Flood at Windsor	Windsor (m AHD)	Colo River (m AHD)	Macdonald River (m AHD)	Sackville (m AHD)	Lower Portland (m AHD)	Wisemans Ferry (m AHD)
31 Mar 1950	9.36					
3 Apr 1950	9.36	11.48				
Apr 1950	8.69					
May 1950	7.47					
9 Jun 1950	9.61	9.24				
16 Jun 1950	7.47	12.42				
25 Jun 1950	7.75	11.46				
24 Jul 1950	8.39	9.5				
22 Oct 1950	9.76	4.7				
21 Jan 1951	9.30	12.51				
9 Jun 1951	5.04	8.59				
16 Jun 1951	7.32	7.39				
Sep 1951	6.86					
14 Apr 1952	4.59	4.86				
16 Jun 1952	9.55	8.51				
28 Jul 1952	11.78	13.69				
6 Aug 1952	9.64	10.43				
14 Aug 1952	8.97	17.33				
7 May 1953	6.25	13.47				
22 Feb 1954	8.86	12.61	9.4			
25 Feb 1955		15.9				
2 May 1955	9.93					
11 Feb 1956	13.83	15.26		9.4	7.0	
20 Feb 1956	11.73	16.06				
2 Mar 1956	9.11					
14 Mar 1956	9.98					
3 May 1956	7.18	5.31				
25 Jun 1956	9.68	10.59				
20 Feb 1957		4.8				
31 Jan 1958		7.6				
19 Feb 1959		9.5				
20 Nov 1961	14.95	9.3	3.8	10.4	7.2	3.2
13 Jan 1962	8.58	6.48				
14 May 1962	6.27	12.42	7.07			
18 Jan 1963	4.18	9.55	5.82			
24 Mar 1963	4.75	8.0				
30 Apr 1963	8.70	13.97	9.5	4.6	3.4	

Table A1
Lower Hawkesbury River
Recorded Peak Heights

Date of Flood at Windsor	Windsor (m AHD)	Colo River (m AHD)	Macdonald River (m AHD)	Sackville (m AHD)	Lower Portland (m AHD)	Wisemans Ferry (m AHD)
6 Jun 1963	8.94	7.48	5.7	4.44		
8 May 1963	8.08	10.72	7.11	4.22	2.7	
30 Sep 1963	9.58	5.28		4.88		
13 Jun 1964	14.57	14.61	10.4	10.97	7.7	4.2
10 Nov 1966	4.97	7.29	4.10			
27 Jan 1967	2.66					
7 Mar 1967	4.33					
9 Mar 1967	3.11		5.34			
8 Aug 1967	8.94	15.22	7.6	4.8	3.5	
7 Sept 1967	5.40		4.7			
14 Jan 1968		5.7				
11 Feb 1969	3.88	5.41	3.88			
16 Apr 1969	6.44	6.6	5.34			
26 Aug 1969	2.35	6.13	4.1			
16 Nov 1969	10.2	9.24	4.89	5.56		
27 Jan 1970		5.6				
2 Sep 1970	2.55		4.08			
9 Dec 1970	4.41					
1 Feb 1971	4.33	13.64	5.91			
11 Feb 1971	5.86		5.34			
15 Jan 1972	6.29	7.34				
26 Jan 1972	7.05	9.26	5.98			
2 Feb 1973	2.88		3.5			
12 Feb 1973		10.4				
12 Jan 1974	6.76		7.2			
12 Mar 1974	7.29	6.92	3.75			
12 Apr 1974	8.66		3.88			
28 May 1974	10.43	11.84				
5 Jun 1974	7.95	14.48	9.4			
29 Aug 1974	9.6		3.55			
22 Jun 1975	11.2	12.11	5.63			
5 Jul 1975	6.63					
24 Jan 1976	9.37	11.02	5.88			
5 Mar 1976	8.0	8.42	6.5			
4 Mar 1977	8.91	15.46	9.91	5.4	4.6	
21 Mar 1978	14.46	20.72	11.21	10.71	7.8	4.8
2 Jun 1978	9.70	12.72	8.01	5.6	3.7	

Table A1
Lower Hawkesbury River
Recorded Peak Heights

Date of Flood at Windsor	Windsor (m AHD)	Colo River (m AHD)	Macdonald River (m AHD)	Sackville (m AHD)	Lower Portland (m AHD	Wisemans Ferry (m AHD)
23 May 1981			7.1			
15 Mar 1982		5.8	5.4			
21 Mar 1983	4.25	9.8				
29 Jul 1984	8.25	8.2	5.1	3.0	0.8	
7 Nov 1984		10.6	6.2	2.4		
15 Oct 1985		9.0	5.7			
7 Aug 1986	11.35		5.1			
1 May 1988	12.8					
6 Jul 1988	10.96	17.5	7.9		5.85	2.78
5 Apr 1989	9.22		8.8	5.36	4.55	2.14
27 Apr 1989		9.2				
4 Feb 1990	7.69			4.59		1.97
21 Apr 1990	8.72		9.0	5.65	5.10	2.58
1 Aug 1990	13.46			9.97	7.46	4.3
10 Feb 1992	10.82			7.51	5.77	
1 Aug 1990	13.46			9.97	7.46	4.3

Notes on Flood levels: The above list of flood peaks was compiled from:

Windsor: The objective of this study was not to derive a detailed flood list for Windsor, however, the study did need to obtain data on flooding below Sackville and relate this to Hawkesbury flooding. The primary source of flood data listed in this report was collated from a PW list of floods. The data includes references to J.P. Josephson, the Windsor Richmond Gazette, H.C. Russell, John Tebbutt and PW reports. The Sydney Water report (SW 1994) lists Windsor floods above 8 m from 1857 to present. For consistency the Sydney Water flood levels have been adopted in this study.

Colo River: These flood levels were obtained from DWR and BOM listings of flood levels at Upper Colo No 1.

Macdonald River: These flood levels were obtained from DWR listings of flood levels at St Albans.

Sackville, Lower Portland and Wisemans Ferry: These flood levels were obtained from DPWS records of flooding at these locations.

Appendix B
Flood Frequency Analysis

Table B1
Annual Flood Series Colo River at Upper Colo
(AHD=GH+1.5 m)

Year	Recorded peak gauge height (m)	Discharge (m^3/s) AWACS Extrapolation
1909	13.53	1643
1910	7.70	483
1911	5.48	263
1912	4.77	201
1913	13.11	1533
1914	5.18	236
1915	1.98	36
1916	3.53	113
1917	2.87	77
1918	5.34	250
1919	2.49	58
1920	3.66	121
1921	8.28	557
1922	9.76	782
1923	1.57	21
1924	2.13	42
1925	5.03	223
1926	8.84	637
1927	8.38	571
1928	6.53	365
1928	7.95	513
1930	8.48	585
1931	9.15	684
1932	1.98	36
1933	3.40	106
1934	no record	no record
1935	no record	no record
1936	no record	no record
1937	no record	no record
1938	no record	no record
1939	no record	no record
1940	no record	no record
1941	no record	no record
1942	12.95	1491

Year	Recorded peak gauge height (m)	Discharge (m^3/s) AWACS Extrapolation
1943	8.05	526
1944	1.98	36
1945	8.92	649
1946	12.96	1493
1947	8.89	645
1948	5.95	307
1949	15.85	2390
1950	12.96	1493
1951	10.97	1016
1952	15.80	2380
1953	11.94	1224
1954	11.07	1037
1955	14.41	1910
1956	14.53	1942
1957	3.28	99
1958	6.02	314
1959	7.93	510
1960	no flooding	no flooding
1961	7.77	490
1962	10.90	1002
1963	12.45	1377
1964	13.08	1522
1965	no flooding	no flooding
1966	5.76	289
1967	13.69	1700
1968	4.22	158
1969	7.72	485
1970	4.06	147
1971	12.12	1278
1972	7.72	485
1973	8.86	640
1974	12.96	1493
1975	10.59	941
1976	9.50	739
1977	13.94	1763
1978	19.20	3830

Year	Recorded peak gauge height (m)	Discharge (m³/s) AWACS Extrapolation
1979	-	160
1980	-	310
1981	-	1050
1982	4.30	164
1983	8.30	560
1984	9.10	677
1985	7.45	457
1986	Morans Rock	2550
1987	-	300
1988	16.10	2490
1989	7.70	483
1990	13.80	1729
1991	-	290

Table B1: Note: the 1986 peak discharge was observed at Morans Rock.

Table B2
Annual Flood Series Macdonald River at St Albans

Year	Discharge (m³/s)	Year	Discharge (m³/s)
1955	600	1974	440
1956	no record	1975	150
1957	no record	1976	270
1958	240	1977	660
1959	90	1978	890
1960	30	1979	70
1961	20	1980	5
1962	300	1981	300
1963	600	1982	120
1964	740	1983	30
1965	30	1984	210
1966	20	1985	150
1967	350	1986	100
1968	90	1987	50
1969	120	1988	390
1970	20	1989	510
1971	190	1990	540
1972	40		
1973	190		

Appendix C
Stochastic-Deterministic Joint Occurrence of Hawkesbury and Colo River Flows

Joint Occurrence of Hawkesbury River and Colo River Floods

A paper by E. M. Laurensen (1974) outlines a procedure for examining statistically the combined probability of flows from two hydrologic systems. Details of the procedure are outlined in the paper; this report only provides general outlines and details the data used to arrive at the results.

As stated by Laurensen, "Most hydrologic systems have both stochastic and deterministic components. The stochastic components are parameters defined by means of probability distributions, whereas the deterministic components are processes that can be modelled mathematically or graphically without probabilistic statements".

In the case of the Hawkesbury River system it was essential to determine the flood flow frequency curve for the Hawkesbury River below the confluence of the Colo River. There are flood frequency curves available on the Hawkesbury River relating stage at Windsor to discharge at Sackville and on the Colo River at Upper Colo. Also, flooding on the Colo River is correlated with flooding on the Hawkesbury River because both can be affected by the same storm event.

The following steps were undertaken in line with the procedure outlined by Laurensen.

Step 1

Based on the results from the Sydney Water study (SW 1994) it was possible to derive a frequency curve of flows at Sackville (Figure C1). This was derived from design flood heights at Windsor and the rating curve relating peak height at Windsor against discharge at Sackville (Figure C5). The flows were modified according to the AWACS ratings at Upper Colo and Sackville.

Step 2

Next the historical flood levels at Windsor and Upper Colo (1909 - 1991) were used to derive a conditional probability distribution of flows (Figure C2). The Windsor flood levels were converted to flow using the Windsor/Sackville rating curve in the Sydney Water study (SW 1994) and the Upper Colo flood levels were converted to flow using the AWACS rating table. Again the AWACS ratings at Upper Colo and Sackville were used to modify flow rates. The principal component line was fitted to the data as the best estimate of the relation between the variables. There is a 50% probability of the data for a given event to plot above this line. The other lines on Figure C2 are for other probabilities of events plotting above the particular line.

Step 3

A graphic relationship was prepared relating Sackville flow, Upper Colo flow and flow below the confluence. Very little historical data existed (i.e. only seven floods) to develop this relationship. The historical data showed that the time difference between the Colo and Sackville peaks was in fact a fourth variable in the relationship. To overcome this problem a constant time difference between peaks was adopted. From an examination of the historical data and the calibration and design simulations in the RUBICON model it was decided to adopt 35 hours as the time difference between Colo River and Sackville peaks for all flow frequencies.

The RUBICON model was used to estimate the downstream confluence flows. The design inflow hydrographs at Sackville and Upper Colo derived in this study were used in the RUBICON model. Figure C3 shows the graphic relationship adopted to relate Sackville, Colo and confluence flows.

Step 4

From the information presented in Figures C1, C2 and C3, Table C1 was prepared detailing the matrices of probability distributions. Multiplication of the conditional probability matrix by the column matrix of Sackville probability densities gave the probability distribution of confluence flows as illustrated in Table C1.

Example:

- The confluence and Sackville flows were divided into class intervals. For example Sackville flow interval of 3,000 to 4,000 m^3/s, adopts Sackville flow of 3,500 m^3/s as the value to read.

- For a Sackville flow of 3,500 m^3/s, the Colo flow that is required to produce various confluence flows is derived from Figure C3 and listed in Column 1 below. For each of these particular combinations of Sackville and Colo flows, the probability of exceedance of each combination can be read from Figure C2 and is listed in Column 2 below. The conditional probabilities listed in Column 3 were obtained as the differences between the exceedance probabilities.

For a Sackville flow of 3,500 m³/s

	Column 1	Column 2	Column 3
Confluence Flow (m³/s)	Colo Flow (m³/s)	Exceedance Probability	Conditional Probability
infinity	infinity		
10,000	infinity		
8,000	infinity		
7,000	infinity	0	
			0
6,000	6,000	0	
			0.035
5,000	3,700	0.035	
			0.175
4,000	2,300	0.21	
			0.79
3,000	0	1.0	
2,000	0		
1,000	0		

- This step is repeated for each Sackville interval and Table C1 is created.
- By matrix multiplication of the conditional probabilities of Sackville flows with the Sackville probability distribution the confluence probability distribution was calculated.

Confluence peak flow (m³/s)	Conditional Probability of Sackville Flows (Sackville Flow m³/s)														Sackville probability distribution	Confluence probability distribution	Cumulative confluence probability
	9500	8500	7500	6500	5500	4500	3500	2500	1500	900	700	500	300	100			
															0.0000	0.0000	0
∞ - 9000	1.000	0.075	0.004												0.01	0.0104	0.0104
8000 - 9000		0.925	0.026												0.005	0.0049	0.0153
7000 - 8000			0.97	0.075											0.01	0.0108	0.0261
6000 - 7000				0.925	0.2	0.005									0.015	0.0201	0.0462
5000 - 6000					0.8	0.245	0.035	0.003							0.03	0.0400	0.0862
4000 - 5000						0.75	0.175	0.042	0.005	0.002	0.001				0.05	0.0633	0.1495
3000 - 4000							0.79	0.315	0.06	0.023	0.018	0.015	0.009	0.005	0.09	0.1573	0.3069
2000 - 3000								0.64	0.53	0.18	0.15	0.10	0.08	0.08	0.19	0.3477	0.6546
1000 - 2000									0.41	0.37	0.29	0.28	0.22	0.21	0.37	0.2228	0.8774
800 - 1000										0.43	0.12	0.05	0.09	0.07	0.08	0.0476	0.9249
600 - 800											0.42	0.08	0.07	0.04	0.07	0.0356	0.9605
400 - 600												0.48	0.07	0.06	0.06	0.0302	0.9907
200 - 400													0.46	0.06	0.0185	0.0086	0.9993
0 - 200														0.48	0.015	0.0007	1.0000

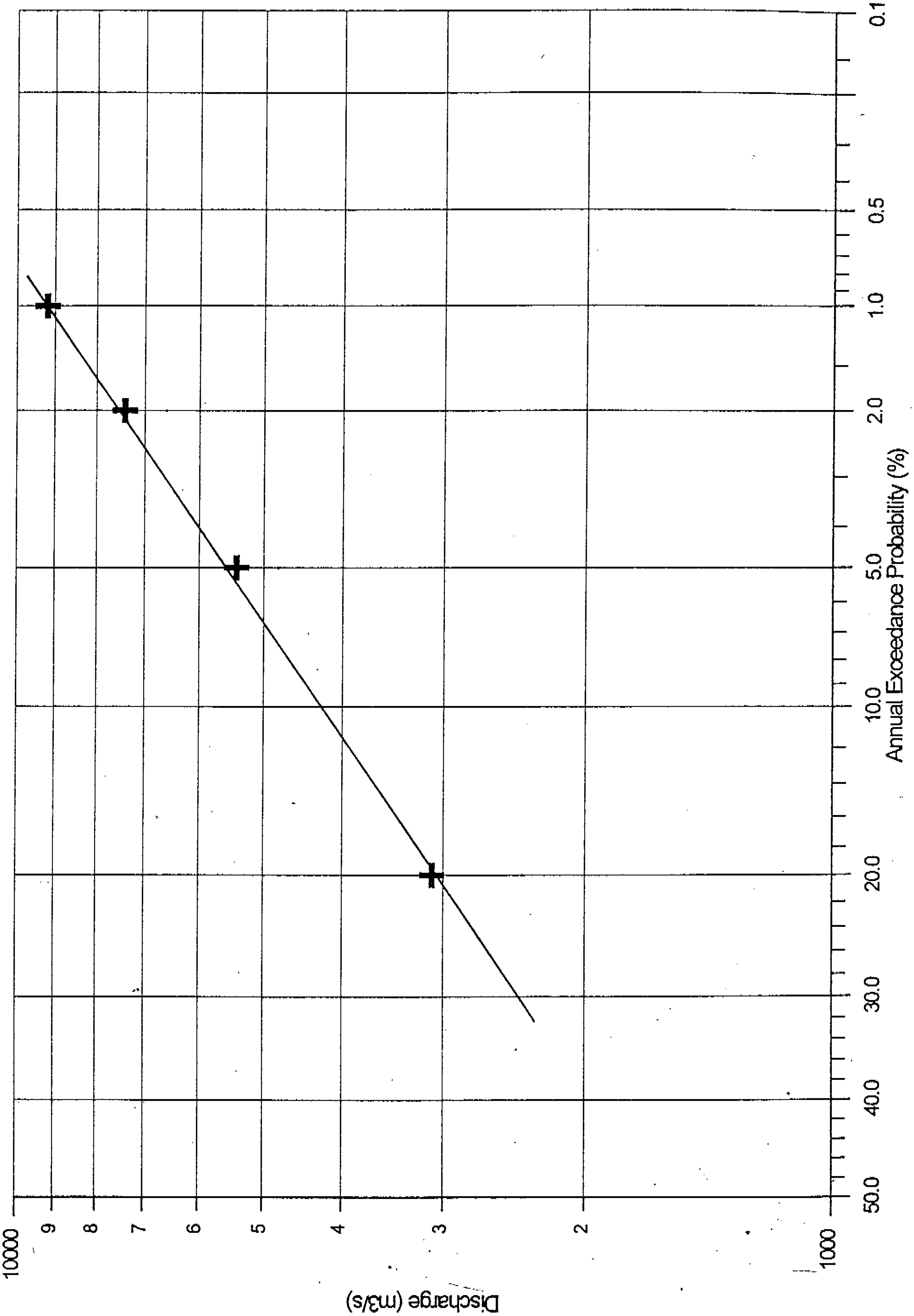
Table C1
Matrix of Probability

Step 5 Results

The confluence probability distribution was plotted and for each frequency desired the magnitude of the confluence flow was estimated (**Figure** C4). The results are summarised in **Table** C2.

Table C2
Frequency of Confluence Flows

Frequency of confluence flow	Discharge (m^3/s)
20%AEP	3,700
5%AEP	6,000
2%AEP	7,800
1%AEP	9,300



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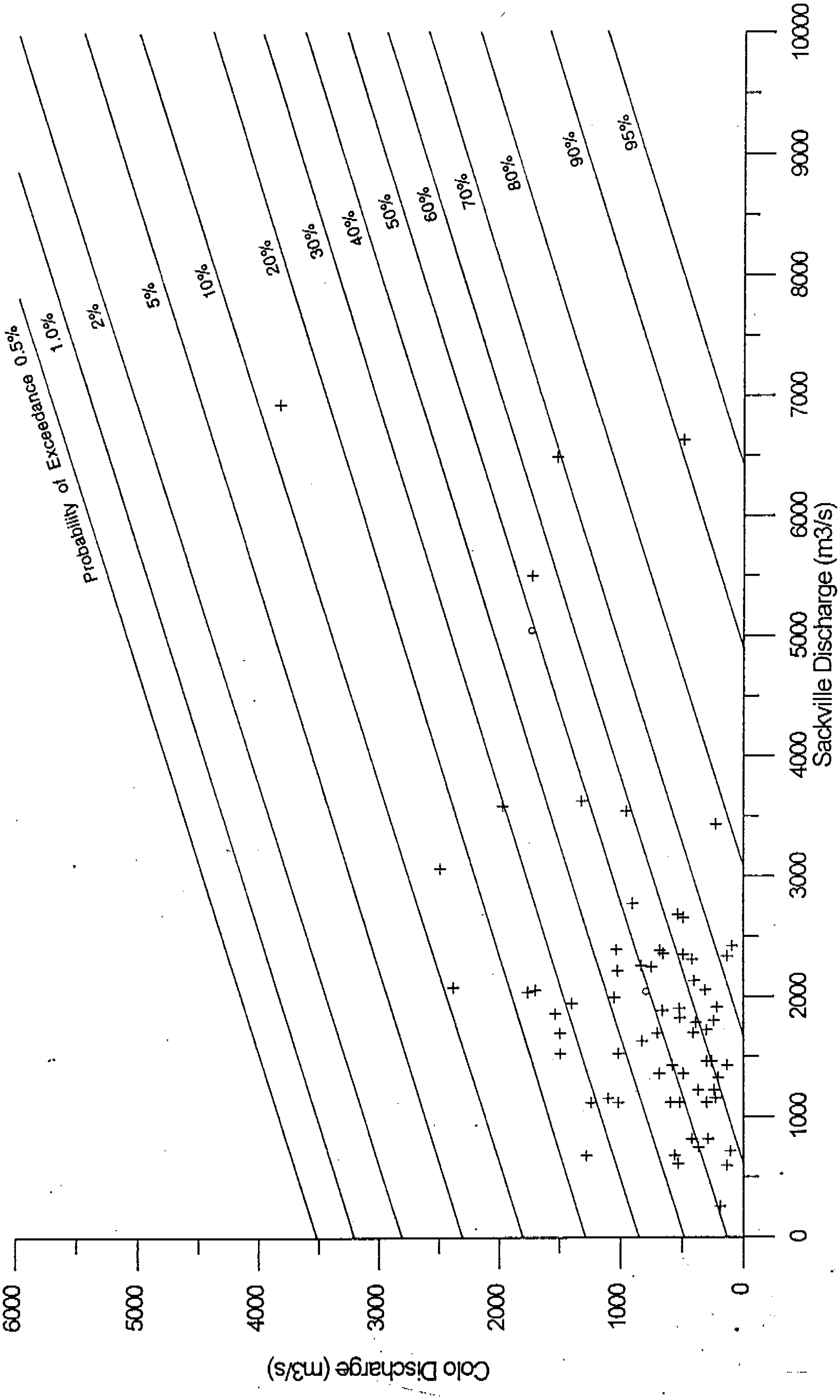
HAWKESBURY RIVER AT SACKVILLE
FLOOD FREQUENCY CURVE

Figure
C1



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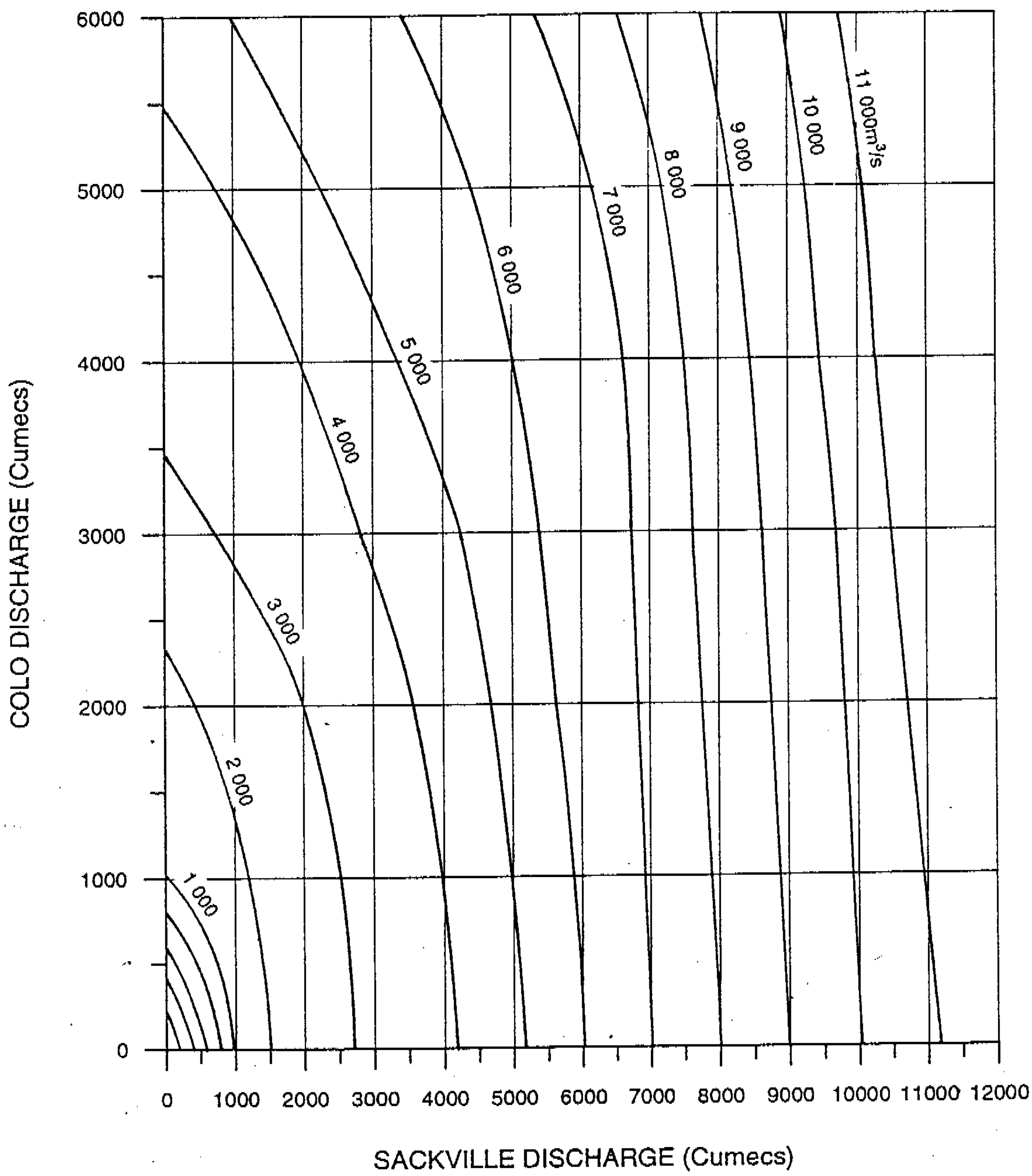
CONDITIONAL PROBABILITY
COLO AND HAWKESBURY RIVER DISCHARGES

Figure
C2



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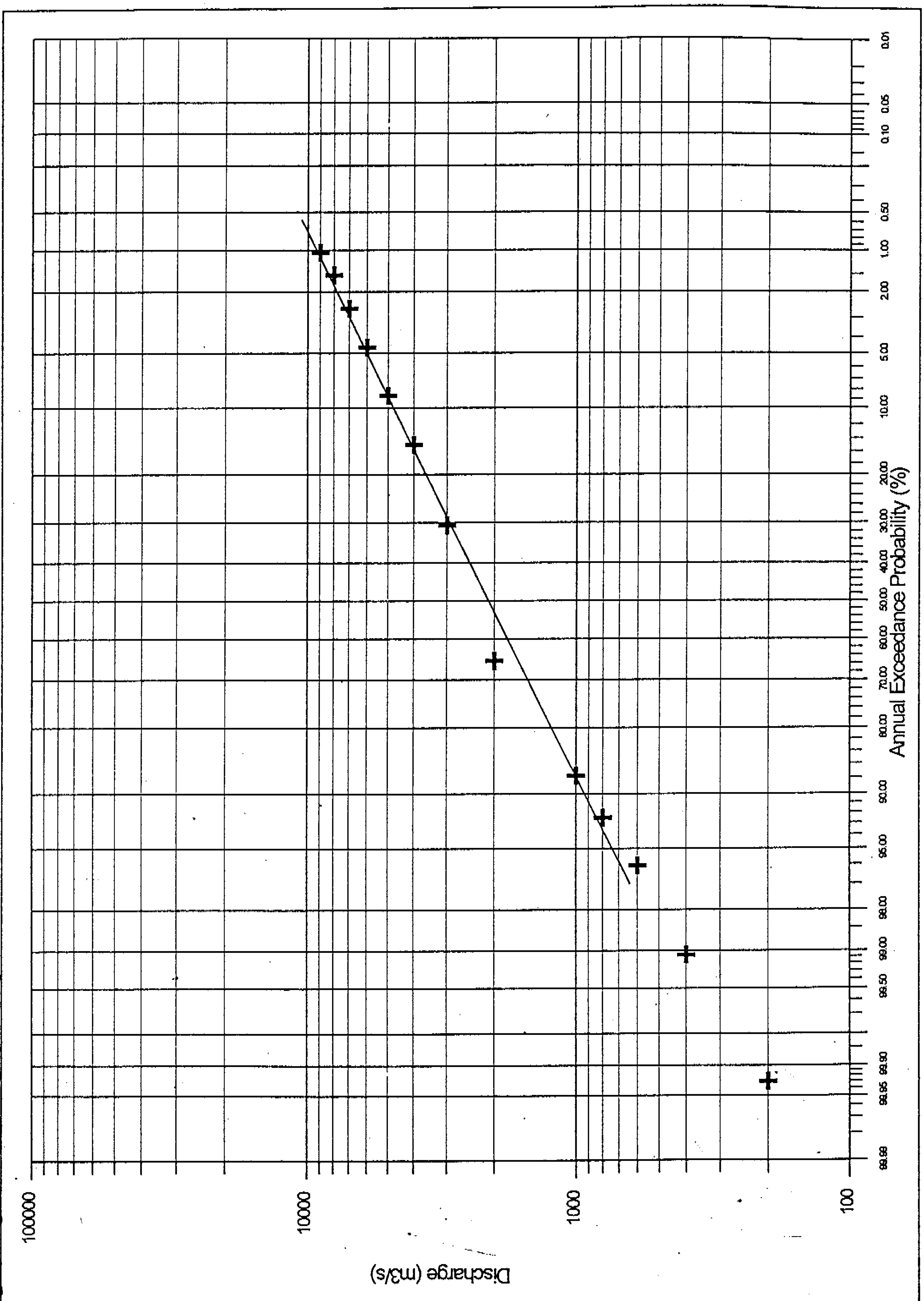
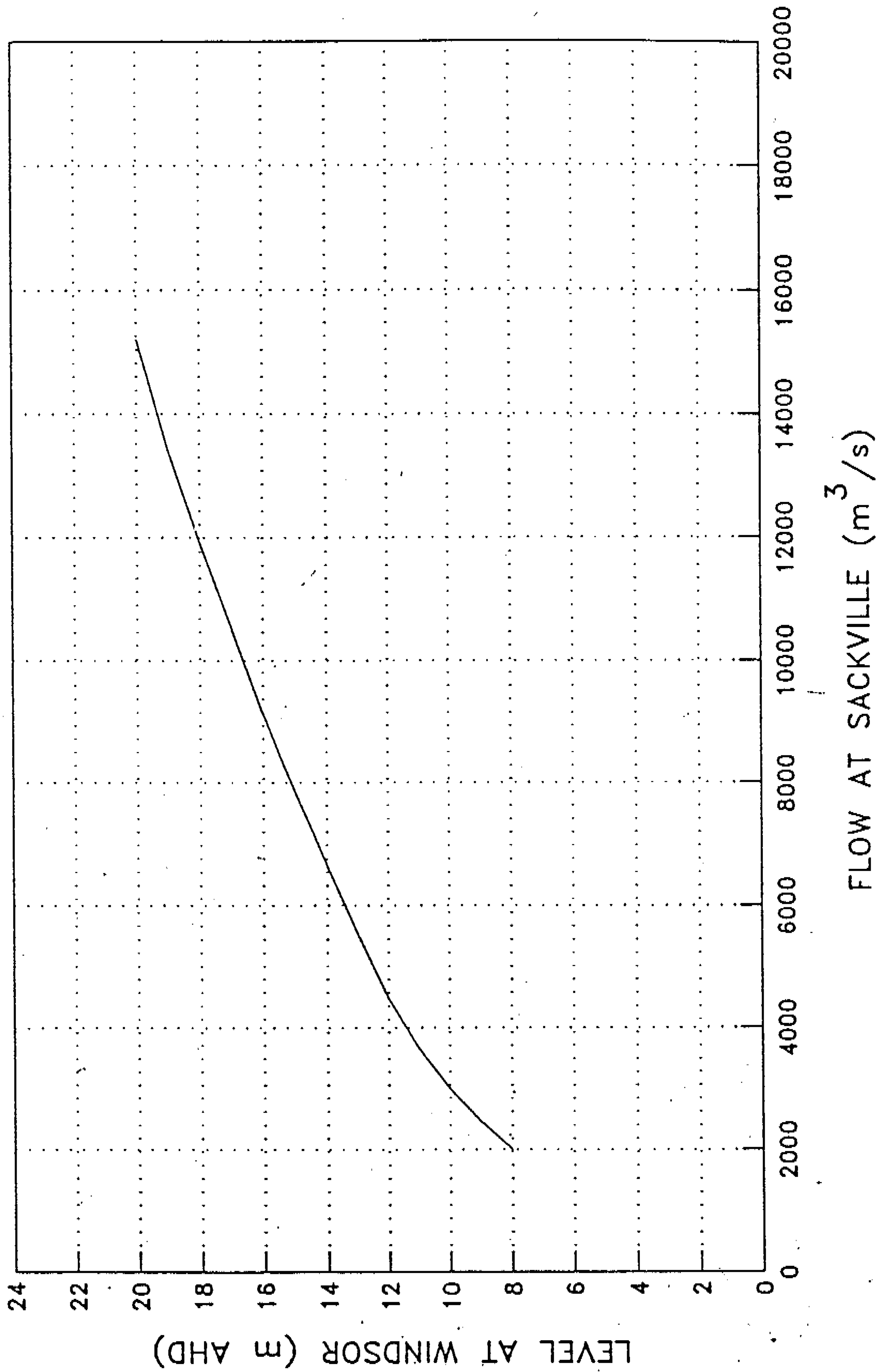
GRAPHIC RELATION OF PEAK FLOWS AT SACKVILLE, UPPER COLO AND AT THE CONFLUENCE

Figure C3



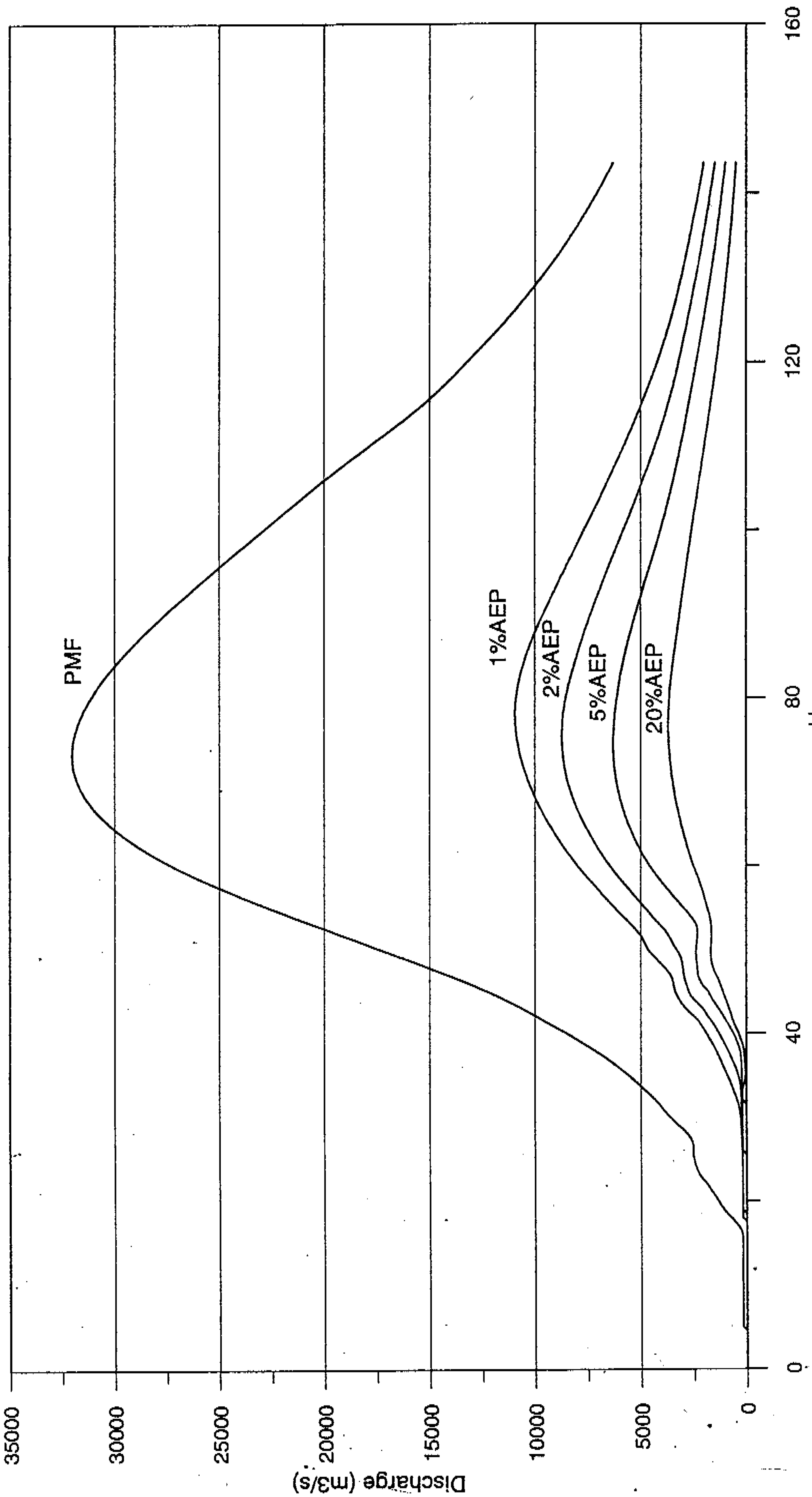
AWACS Report No. 94/29	HAWKESBURY RIVER AT CONFLUENCE FLOOD FREQUENCY CURVE	Figure C4



Source: SW (1994)

AWACS Report No. 94/29	WINDSOR - SACKVILLE RATING CURVE	Figure C5

Appendix D
Data Supplied by Sydney Water



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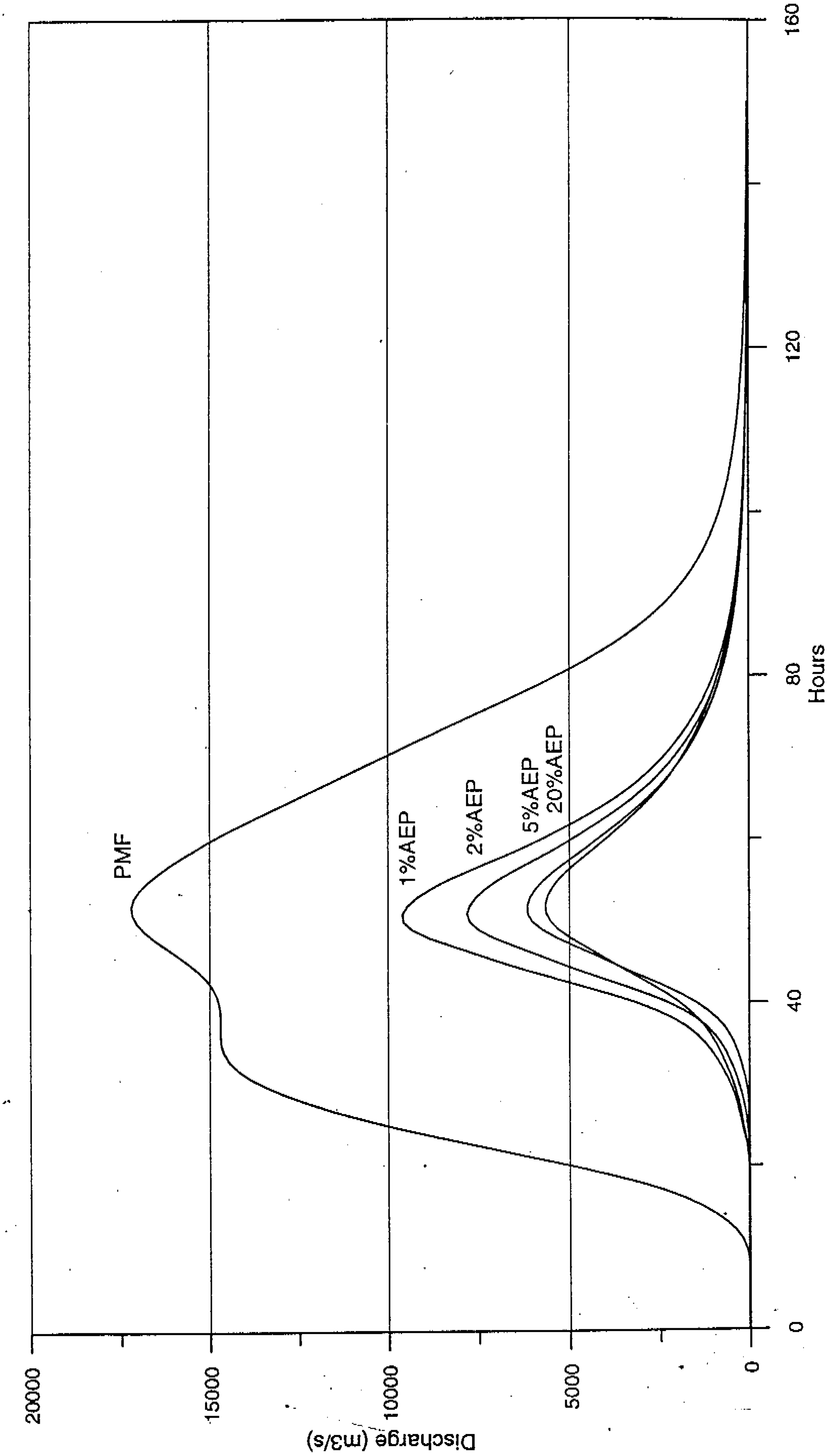
SYDNEY WATER (SW1994)

DESIGN HYDROGRAPHS AT SACKVILLE

Figure

D1



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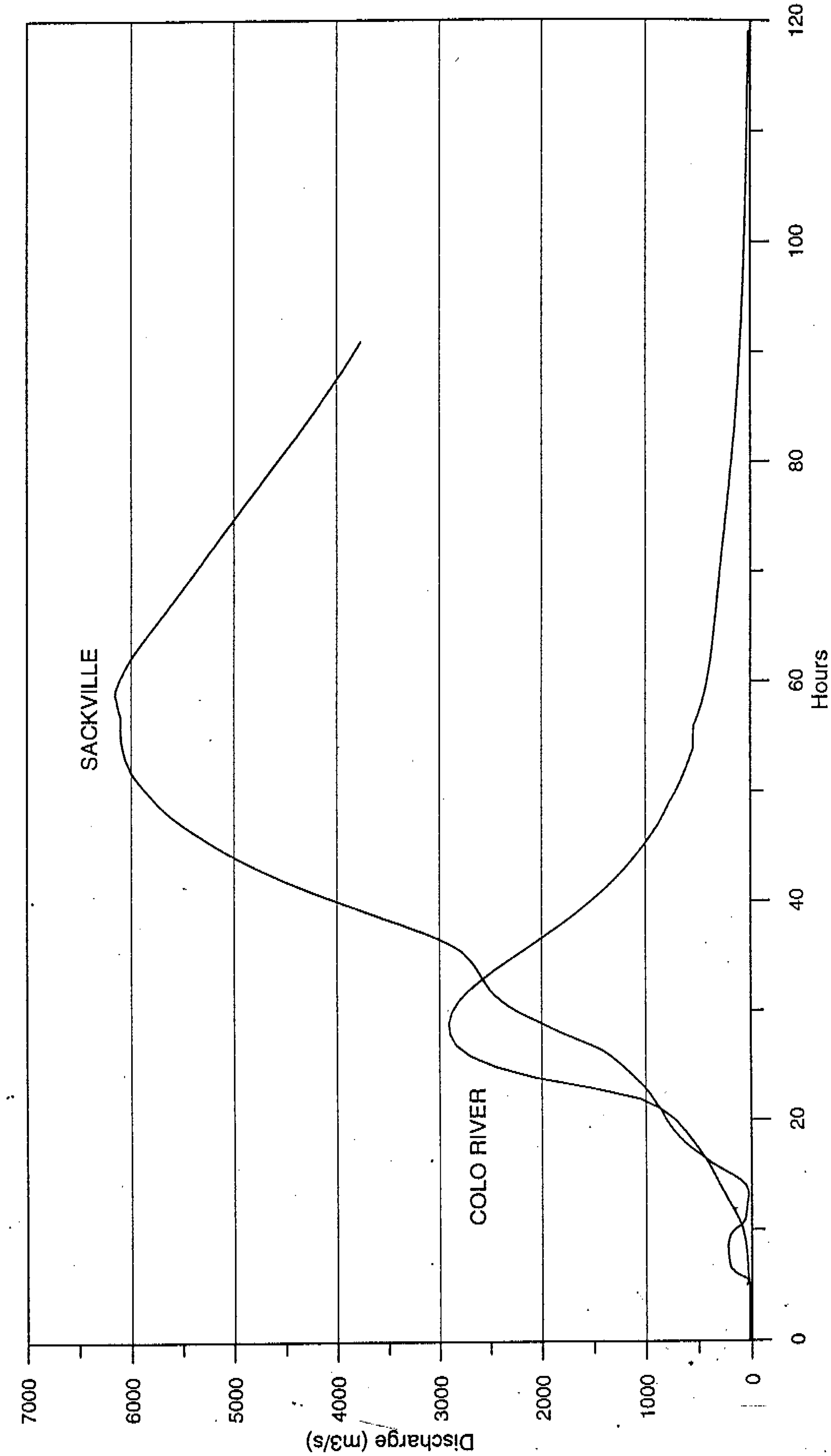
SYDNEY WATER (SW1994)

DESIGN HYDROGRAPHS FOR COLO RIVER

Figure

D2



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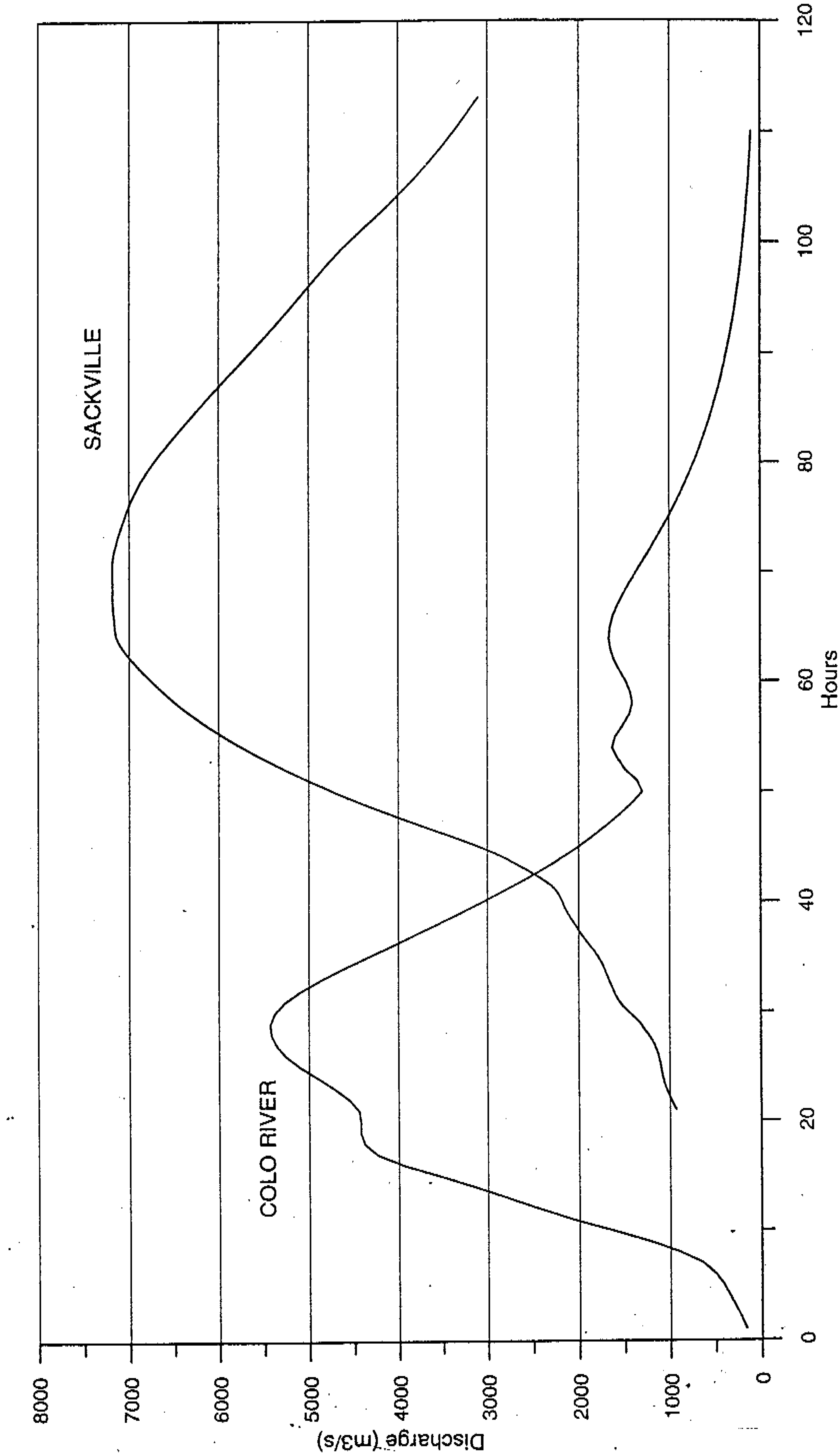
SYDNEY WATER (SW1994)
AUGUST 1990 HYDROGRAPHS

Figure
D3



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Report No. 94/29

SYDNEY WATER (SW1994)

MARCH 1978 HYDROGRAPHS

Figure

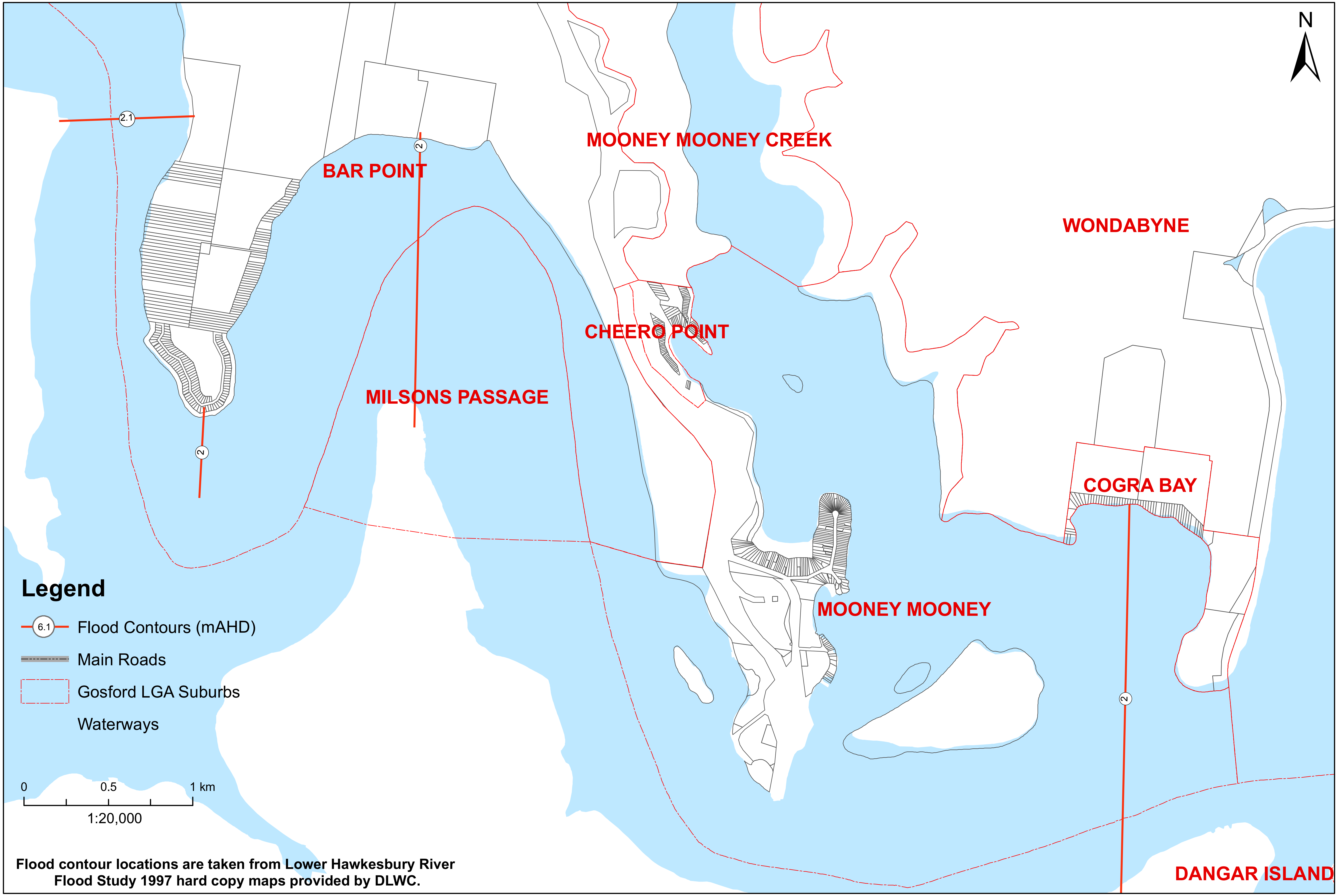
D4

End of Report

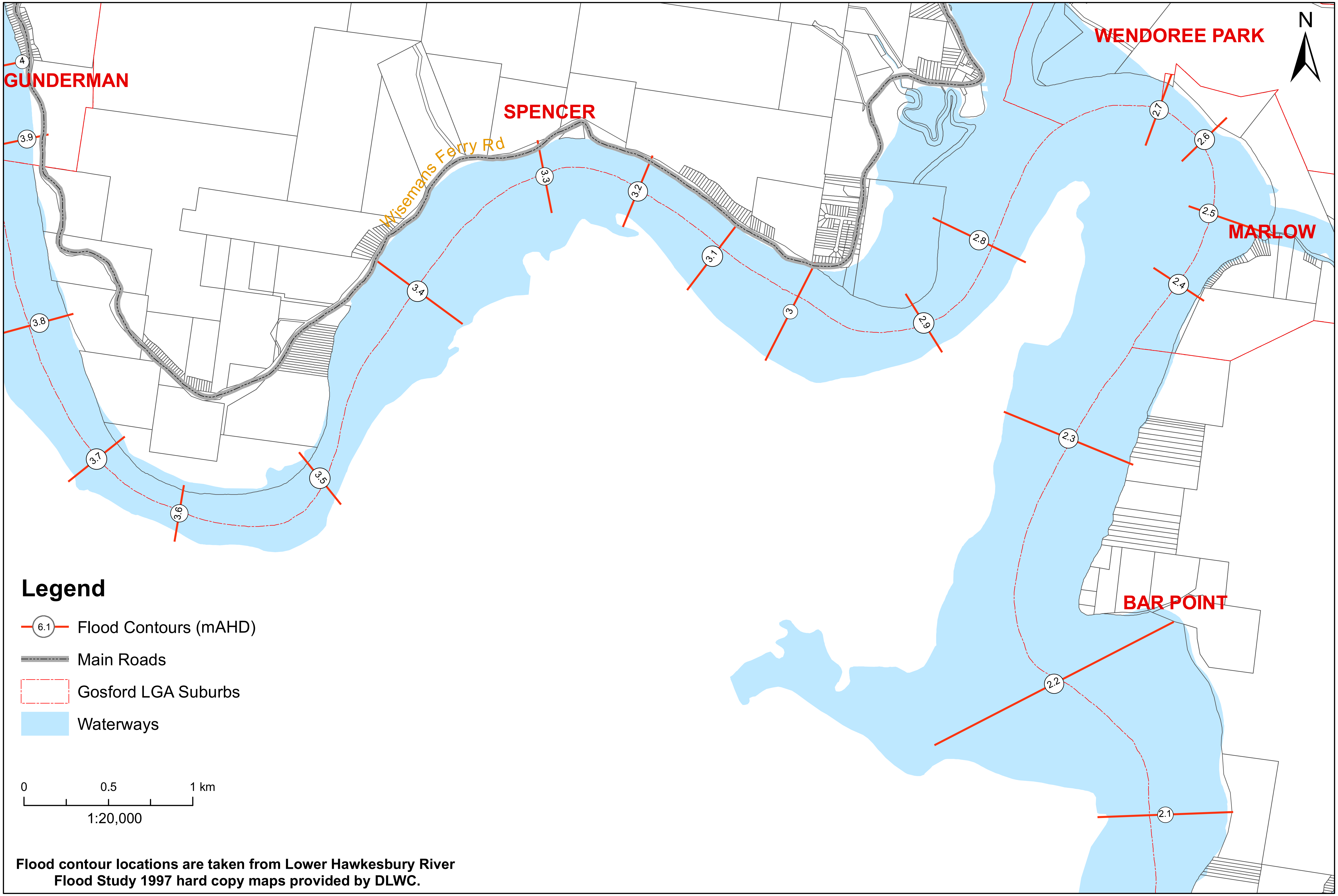
LAND & WATER CONSERVATION

NSW Department of Land and Water Conservation

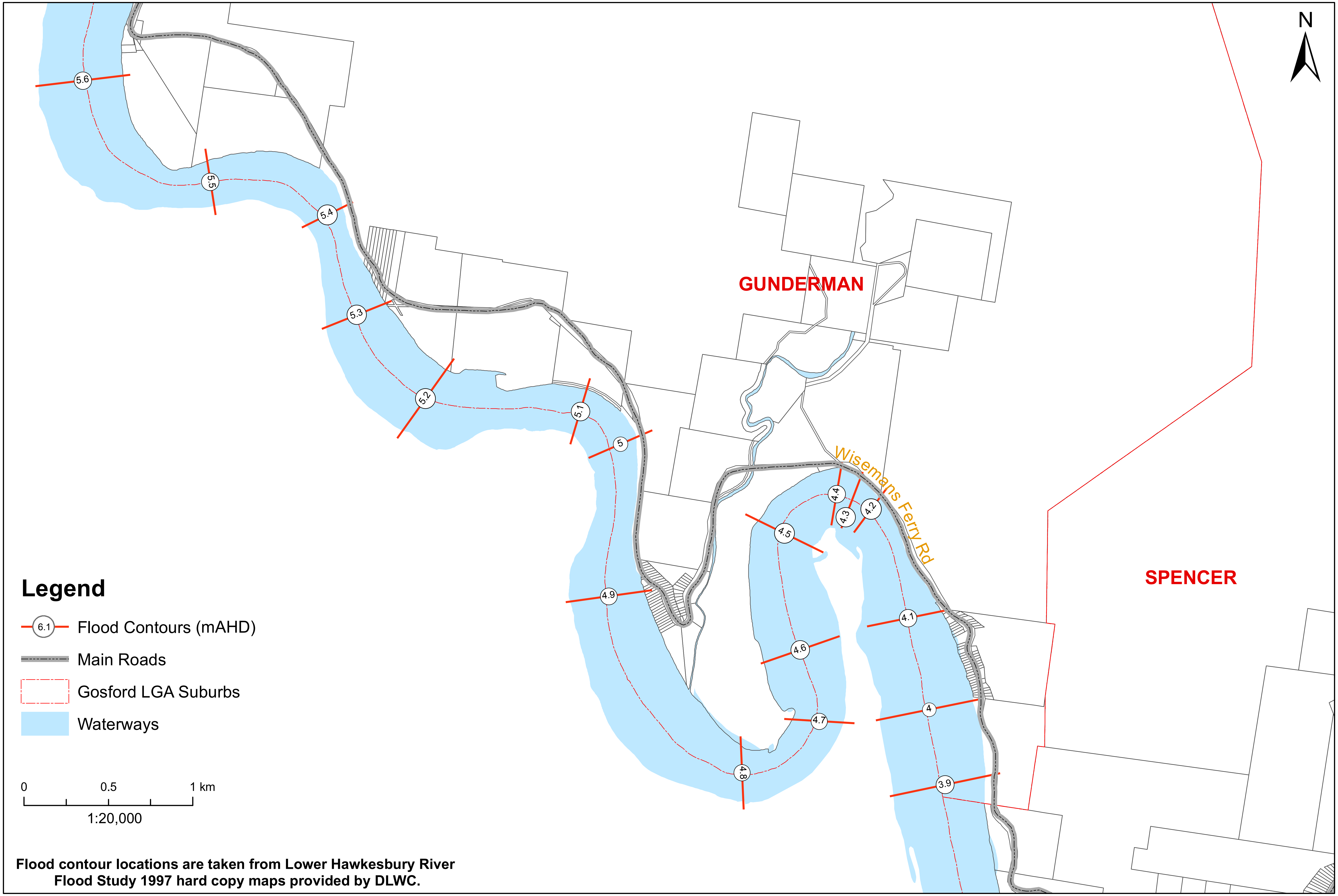
The following maps show the location of flood contours as depicted on the hard copy maps provided by DLWC in conjunction with the Flood Study.



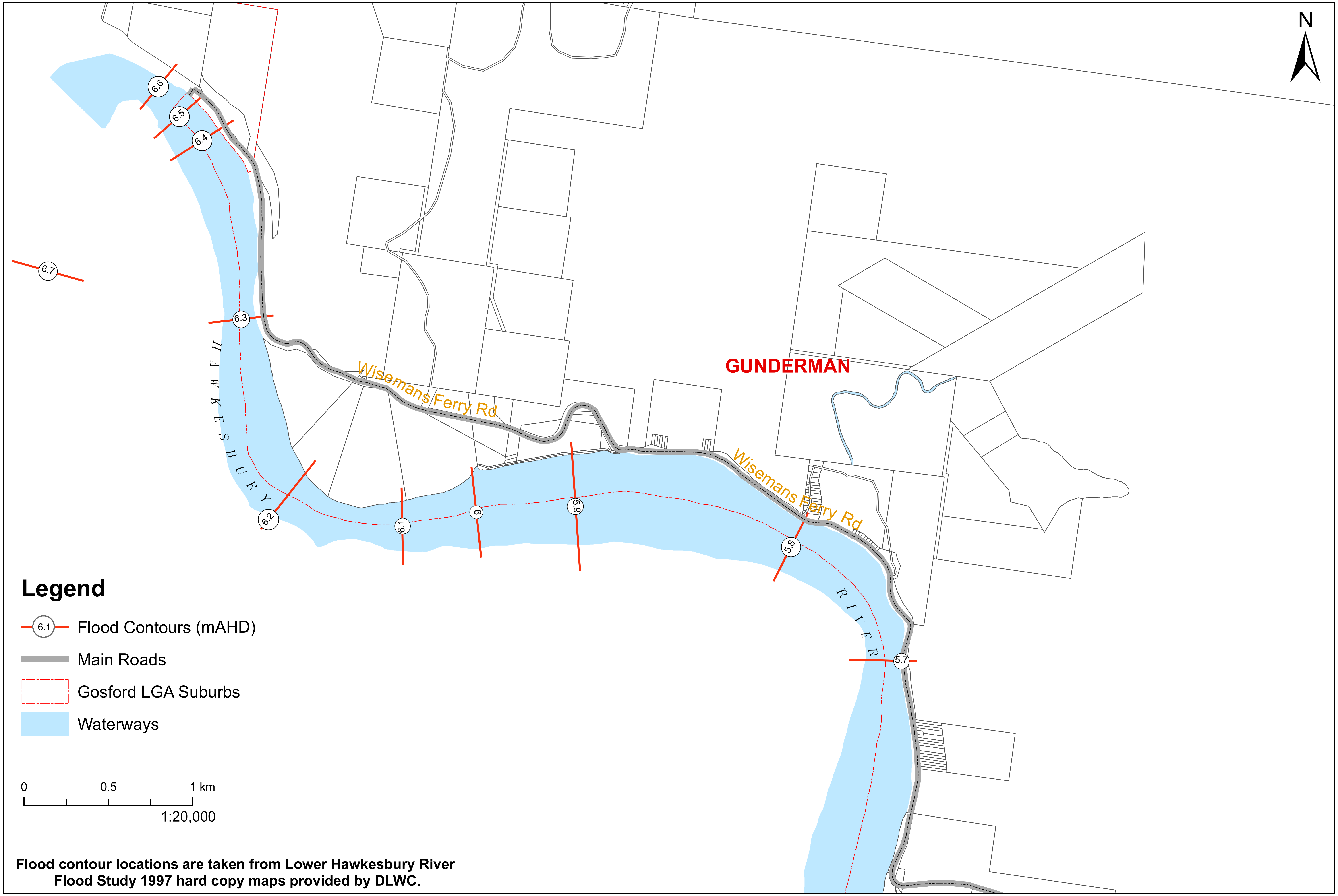
Hawkesbury River Flood Contours



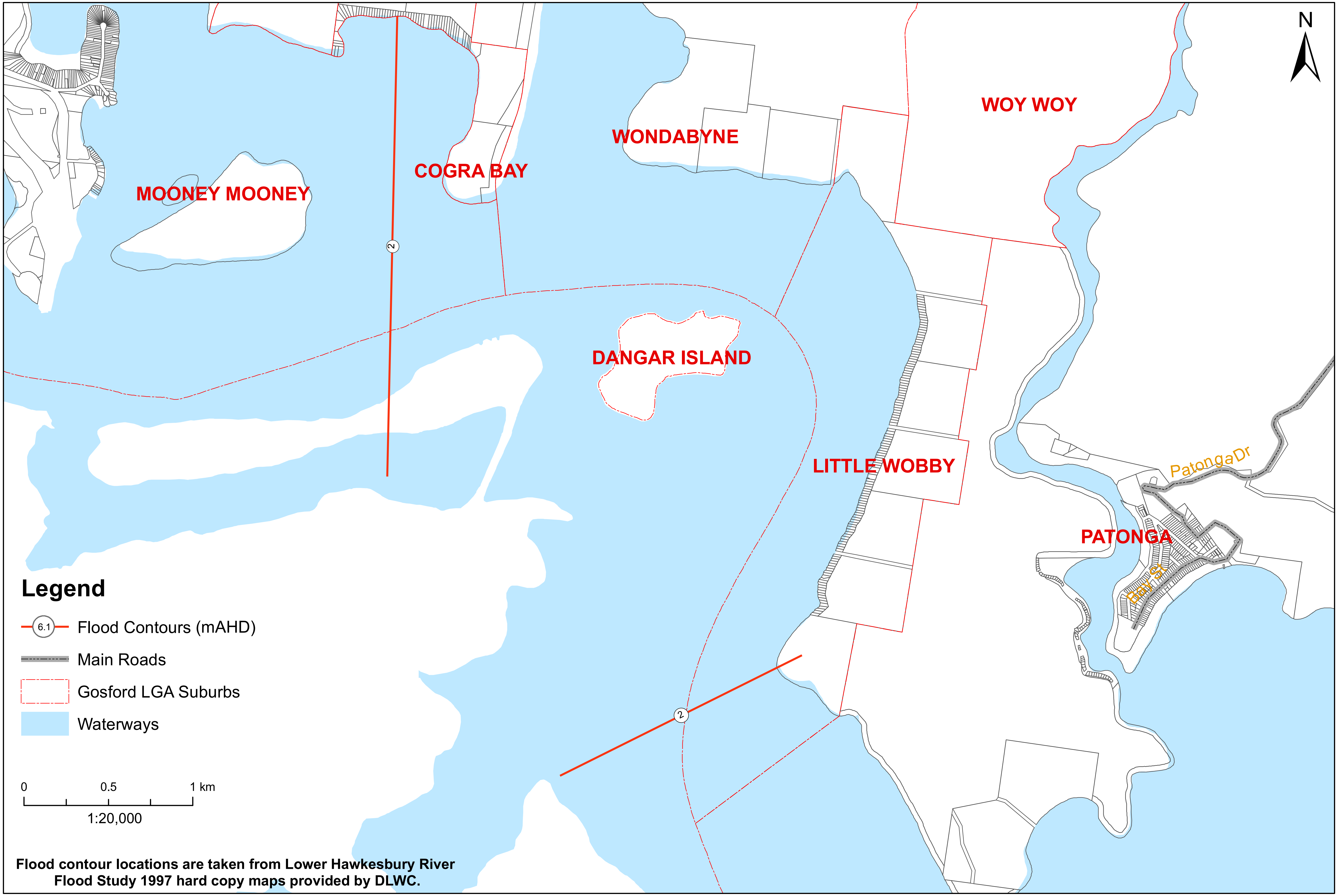
Hawkesbury River Flood Contours



Hawkesbury River Flood Contours



Hawkesbury River Flood Contours



Hawkesbury River Flood Contours