CITIES COMMISSION

BRISBANE RIVER FLOOD INVESTIGATIONS

FINAL REPORT

Prepared by

SNOWY MOUNTAINS ENGINEERING CORPORATION

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BRISBANE RIVER FLOOD JANUARY 1974 — FLOODING IN BRISBANE CITY

(Photograph : Courtesy Austral Aerial Surveys)

CONTENTS

SUMMARY

In January 1974, widespread flooding occurred along the Brisbane River and thousands of residential, commercial, industrial and other buildings in the Cities of Brisbane and Ipswich suffered devastating damage. This flood reached a height of 5.45 m at the Brisbane City Gauge. The history of flooding in the Brisbane River shows that such events are not uncommon and, in fact, four other floods which occurred last century reached levels in excess of the January 1974 event. As a result of the widespread economic loss due to flood damage and because of the Cities Commission's interest in regional land use studies it offered assistance to the Queensland Government by way of a co-operative investigation into Brisbane River flooding. Specifically the Cities Commission undertook to finance a study to determine flood damage along the Brisbane River for floods of various magnitudes up to a maximum of 10 m (Australian Height Datum) at the Brisbane City Gauge and to assist in the preparation of maps showing the extent of flooding in the Cities of Brisbane and Ipswich. The Cities Commission engaged the Snowy Mountains Engineering Corporation (SMEC) to carry out the detailed work.

The flood damage estimates are described in Part 1 of this report and were based mainly on data collected after the January 1974 flood by Australian, State and Local Government authorities. Where additional data were required, these were collected by means of questionnaires or through the private valuing firm, Alex Overett Pty Ltd, which was engaged by SMEC to assess the cost of damage to selected samples of buildings. A most encouraging aspect of the study was the co-operation given by Australian, State and Local Government authorities and also private organisations and individuals in the supply of a wide range of information and basic data needed for the successful completion of the work.

To facilitate the rapid analysis of flood damages to buildings on the Brisbane River flood plain, a computer data bank for over 28 000 residential, commercial, industrial and other types of buildings was compiled. For each building affected by flooding, essential information such as floor area, floor level and river kilometres from the Brisbane River mouth was transferred from detail maps to the data bank. The buildings in the data bank were analysed for flood damage using general relationships

derived from the 1974 flood damage data and estimated flood profiles for the Brisbane River (see Part 3) provided by the Brisbane City Council. Flood damage to public utilities were gathered for the 1974 flood and to obtain the utilities damage for other floods the 1974 flood damage figure was multiplied by the ratio of the respective flooded areas.

The estimated river stage - damage curves are given in Figure 15 in Part 1 and it is recommended that these should be used in the economic studies for the proposed dam at Wivenhoe on the Brisbane River. Details of the estimated damage for various flood heights are given in the table below:

DETAILS OF FLOOD DAMAGE

FLOOD HEIGHT m	FLOODED AREA km^2	BUILDINGS AFFECTED	FLOOD DAMAGE MILLION $	
			Direct	Direct + Indirect
2	12	470	8	10
4	57	6 700	67	83
6	102	15 300	173	217
8	153	23 500	288	362
10	205	31 000	426	531

Because of the comprehensive nature of the data collected and the analysis of flood damage information, it is believed that the data and the results of this study could be used as a guide for estimating flood damage in other large Australian cities. For this reason many details of the data collected are included in the Appendices.

The other aspect of the Brisbane River flood investigations which involved the assistance of SMEC was the preparation of flood maps. These maps, which were being finalised at the time of printing of this report, were prepared at a scale of 1:10 000 and show the areas inundated for flood heights, of 2 m, 4 m, 6 m, 8 m and 10 m at the Brisbane City Gauge. In all 18 maps were required to cover the area from the Brisbane River mouth to the Bremer River upstream of Amberley Air Force Base. The details of the work involved are described in Parts 2 and 3 of this report. The preparation and production of the flood maps was very much a co-operative effort between various Australian, State and Local Government authorities.

Overall technical guidance was provided by the Flood Recording Co-ordination Committee which was convened after the 1974 flood by the Queensland Co-ordinator General's Department.

Part 2 of the report was contributed by the Survey Office of the Queensland Department of Lands and contains a description of the production of the flood maps. Part 3 contains a description of the derivation of the flood profiles and the flood frequency analysis for the Brisbane River. This work was carried out by the Brisbane City Council. A short addition to Part 3 describing flood profiles and flood frequencies studies for the Bremer River was included by SMEC.

PART I

FLOOD DAMAGE STUDY

PART 1 - FLOOD DAMAGE STUDY

CONTENTS

LIST OF TABLES

LIST OF FIGURES

PART 1 - FLOOD DAMAGE STUDY

1. INTRODUCTION, CONCLUSIONS AND RECOMMENDATIONS

1.1 INTRODUCTION

In late January 1974 a severe flood occurred in the Brisbane River when it reached its highest level this century, and extensive flood damage occurred in the Cities of Brisbane and Ipswich. At the Brisbane City Gauge the peak of the flood reached 5.45 m (Australian Height Datum).

Immediately following the flood, the Cities Commission requested the Snowy Mountains Engineering Corporation (SMEC) to visit Brisbane to determine what co-operative studies of Brisbane River flooding could be undertaken in conjunction with the Queensland Government and Local Government authorities. On 1 February 1974, two SMEC officers attended a meeting arranged by the Queensland Co-ordinator General's Department. This meeting was attended by officers from a number of Australian, State and Local Government authorities and details were discussed of the various flood investigations that should be put in hand.

The meeting concluded that the majority of the hydrological and meteorological studies required could be carried out by various departments in Brisbane. It was recommended, however, that the revision of the 1933 flood map prepared by the Queensland Bureau of Industry, and the preparation of a river stage - flood damage curve for the Brisbane River would be the most valuable form of assistance that the Cities Commission could provide. It was planned that this work would be carried out by using the services of SMEC.

Following this meeting, SMEC submitted an initial assessment of the work required for the above two items of assistance and on 18 February 1974, the Cities Commission requsted a final submission for the proposed study. In the week 18 to 22 March, two SMEC officers again visited Brisbane to determine in detail the information which would be made available for the study.

On 19 April 1974 SMEC submitted a detailed proposal for consulting services and this was accepted by the Commission in its letter of 8 May 1974.

The study was confined to Brisbane River flooding to ensure that the work already in progress by other consultants investigating tributary flooding was not duplicated. However, the areas along tributaries including the Bremer River at Ipswich were included wherever they were affected by backwater flooding from the Brisbane River. In the case of tributary flooding the backwater was assumed to be a horizontal surface.

The two main objectives were:

(a) To prepare maps* showing the extent of flooding along the Brisbane River for flood heights at the Brisbane City Gauge of 2 m, 4 m, 6 m, 8 m and 10 m (A.H.D.). These maps would be based on information collected from the January 1974 flood and flood profiles determined by the Brisbane City Council.

(b) To prepare a graph showing river stage versus the cost of flood damage for floods in the Brisbane River with gauge heights in the range of 2 m to 10 m, based on the assessment of damage along the Brisbane River during the 1974 flood. As with the determination of the extent of flooding, the flood damage in the tributary streams above the level backwater effects of the Brisbane River was excluded from the study.

1.2 CONCLUSIONS AND RECOMMENDATIONS

The principal conclusions and recommendations of this flood damage study were as follows:

(a) Flood damage due to flooding along the Brisbane River is very high. In the January 1974 flood the direct damage was estimated to be \$ 142 million and the total damage (direct plus indirect) \$ 178 million. The flood height at the Brisbane City Gauge in 1974 was 5.45 m. For floods of 2 m, 4 m, 6 m, 8 m and 10 m at the Brisbane City Gauge the total damage estimates would be \$ 10, \$ 83, \$ 217, \$ 362 and \$ 531 million respectively.

(b) The estimated number of buildings affected by the January 1974 flood was 13 000. Floods of 2 m, 4 m, 6 m, 8 m and 10 m at the Brisbane City Gauge would affect an estimated 470, 6 700, 15 300, 23 500 and 31 000 buildings respectively.

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* See Parts 2 and 3 of this report

(c) The accuracy of the derived flood damage curves is difficult to assess but it is believed that the results are more likely to be under estimations of actual flood damage than over-estimations. This is because it is extremely difficult to determine all possible sources of flood damage. However, it is also believed that an even more exhaustive investigation would not result in an increase in flood damage for a given flood height of more than another 5% to 10%.

(d) It is believed that most of the collected data, results of analysis and the procedures developed have direct relevance to flood damage studies in other Australian urban areas. It is therefore recommended that use could be made of this report in regional land use studies involving urban developments in other parts of Australia. In transferring results, however, care must be taken to make due allowance for significant differences in the flood characteristics of another river and the types of buildings in another urban centre.

(e) It is recommended that the river stage - damage curves given in Figure 15(a) and (b) be used for the benefit-cost analysis of the flood mitigation studies for the proposed Wivenhoe Dam.

2. GENERAL INFORMATION CONCERNING FLOODS

2.1 INTRODUCTION

The locality plan in Figure 1 shows the extremities of the area of greatest interest in this study of flood damage along the Brisbane River. The two main urban centres are the Cities of Brisbane and Ipswich, with populations of 713 000 and 65 000 respectively. Both cities have large industrial, commercial and residential developments on the flood plain. The Moreton Shire, which surrounds the City of Ipswich, also has substantial areas subject to flooding but as yet urbanisation has not progressed into these to any great extent. Hence in 1974 the flood damage in the Shire of Moreton below the Mt Crosby Weir was relatively small.

Figures 2, 3 and 4 show the estimated flood lines* for floods with a height of 2 m, 6 m and 10 m at the Brisbane City Gauge. These floods have average return periods* of 11, 60 and 200 years respectively. In the case of the 10 m flood an area of 153 km^2 in the City of Brisbane and 51 km^2 in the City of Ipswich would be inundated. These figures exclude the area covered by the river at normal times and represent about 15% and 42% respectively of the area of each city. Although flood affected areas in the Shire of Moreton below the Mt Crosby Weir are not urbanised to any great extent a further 10 km^2 of land lies below the 10 m flood line and the potential damage which could occur in this area if urbanisation is allowed to proceed should not be overlooked.

Figure 1 also shows the position of the damsite at Wivenhoe as proposed in the Co-ordinator General's Report (1971). It is located at a point on the Brisbane River where it could mitigate floods from the upper half of the total Brisbane River catchment.

2.2 BRISBANE RIVER FLOODING

The Brisbane River has a catchment area of 13 600 km^2 at the point where it discharges into Moreton Bay. The catchment is located at a latitude where tropical cyclones occur relatively frequently. Brunt (1968) states that on the average two cyclones affect the northeast coast of Australia each cyclone season. These cyclones are very efficient rain producers and can cause rainfall over areas in excess of 250 000 km^2

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* See Part 3 of this report for further details.

The history of flooding on the Brisbane River shows that large floods are not uncommon although, until the January 1974 flood, no major damaging floods had been recorded in this century. The Queensland Co-ordinator General's report (1971) described very accurately the position with the statement:

> 'It would be unrealistic to assume that the past seventy years of immunity from high floods comparable with perhaps four disastrous ones experienced in a much shorter preceding period has been other than fortuitous.'

Figure 5(a) which is taken from the report by Ward (1974) shows the important historical floods recorded since 1840.

The construction of Somerset Dam as a water supply and flood mitigation dam has had a significant effect on floods in recent years. Figure 5(a) shows with a dashed line the estimated height to which the floods since 1950 would have risen without Somerset Dam. Although Somerset Dam has a catchment of only 1 330 km^2 out of a total for the Brisbane River of 13 600 km^2, it is believed that flood producing rainfalls are frequently highest over this part of the catchment. The isohyetal maps originally presented by Shields (1974) and reproduced as Figures 5(b) and 5(c) show that this was the situation for the very high flood in 1893 but was not the case for the recent January 1974 flood.

2.3 BREMER RIVER FLOODING

As shown on Figure 5 the Bremer River has a catchment area of 2 020 km^2. The main urban areas are in the City of Ipswich, the centre of which is located on the Bremer River some 17 km from its confluence with the Brisbane River.

The Bremer River is capable of producing rapid runoff and high-velocity flooding and it was these characteristics which caused the greatest amount of damage in the January 1974 flood. A recurrence of the 1974 flood with a flood mitigation dam at Wivenhoe would only have a marginal effect on the flood damage in Ipswich. On the other hand, with a flood similar to that which occurred in 1893, the effect of the proposed dam would be much more dramatic in reducing damage in the City of Ipswich. The analysis of the flood damage in this study assumes a

level backwater from the Brisbane River up the Bremer River. The implications of this assumption are discussed in greater detail in Section 3.5(b).

2.4 METROPOLITAN CREEK FLOODING

In a similar manner to the Bremer River at Ipswich, a number of the metropolitan tributaries including the Bulimba, Enoggera, Norman and Oxley Creeks, cause problems due to floods originating in their own catchments. In recent years these tributaries have had a number of severe floods at times when no significant rise in water level has occurred in the Brisbane River.

In 1974, tributary flooding was very severe on 25 and 26 January when it preceded the flood in the main river. On 28 January the Brisbane River reached its peak in the upper suburban reaches and early on 29 January the river peaked at the Brisbane City Gauge and many areas on the tributaries suffered flooding for the second time. This study considers a level backwater from the Brisbane River for each tributary and has not been concerned with floods originating in the tributaries. This point is discussed further in Section 3.5(c).

2.5 FLOOD DAMAGE DATA

As this study followed the 1974 flood, it was possible to gain a large amount of flood damage data from a wide range of sources. The details of the data and their sources are described in the relevant parts of this report. The following is a summary of the principal sources:

(a) Residential Damage

- Queensland Flood Damaged Homes Committee
- Alex Overett Pty Ltd, Valuers
- Australian Department of Housing and Construction, Defence Service Homes
- Queensland Disaster Welfare Committee, Flood Unit

(b) Industrial and Commercial Damage

- Alex Overett Pty Ltd, Valuers
- SMEC questionnaires
- Queensland Department of Commercial and Industrial Development

- Queensland State Insurance Commissioner
- Queensland Disaster Welfare Committee
- Chamber of Manufacturers
- Metal Trades Industry Association
- Printing and Allied Trades Employers Association

(c) Damage to Utilities

- Brisbane City Council
- Ipswich City Council
- Shire of Moreton
- State Government Departments
- Australian Government Departments

2.6 FLOOD PROFILE INFORMATION

The Brisbane City Council accepted the responsibility for the preparation of flood profiles* for the Brisbane River for use in this study. As a basis for the calculation of these profiles the Queensland Harbours and Marine Department** completed a survey of the Brisbane River below high-tide level and the Queensland Survey Office** prepared contour maps for the Brisbane River and Bremer River flood plains. The estimated flood profiles for floods from 2 m up to 10 m at the Brisbane City Gauge are shown in Figure 6.

2.7 FLOOD MAPS

The flood maps used for the flood damage studies were prepared by SMEC from the flood profile information and contour maps referred to in Section 2.6. These maps differ from the maps to be published by the Survey Office (see Part 2 and 3) in as much as level backwater floods were shown extending up the Bremer River. The flood lines were drawn for flood heights at the Brisbane City Gauge of 2 m, 4 m, 6 m, 8 m and 10 m. Figures 2, 3 and 4 are a reduced version of these maps and show only the 2 m, 6 m and 10 m flood lines.

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* Details in Part 3 of this report

** Details in Part 2 of this report

The flood lines shown extending up each tributary in the metropolitan area of the Cities of Brisbane and Ipswich are also drawn on the basis of level backwaters from the Brisbane River. More information on tributary flooding is available from the Brisbane City Council and the Ipswich City Council.

3. BASIC CONSIDERATIONS FOR FLOOD DAMAGE STUDIES

3.1 FLOOD DAMAGE CATEGORIES

Many different breakdowns of flood damage have been used in flood damage studies. In this study it has been found convenient to use two main categories; tangible and intangible damage as shown in Figure 7.

The tangible flood damages are defined as either direct or indirect damages. The direct damages include all those damages which result from the physical contact of property or structures with flood-water, including damage due to sediment, debris or any other floating object in the flood-water. The principal direct damages are those which occur to the structures or contents of private buildings (residences, factories, offices, etc.) public buildings and public utilities. The indirect damages include all those damages which are an indirect result of the flood and may occur either during the flood or in the weeks or months following the flood. Included in this category is the value of lost business for flood affected commercial and industrial enterprises, the loss of revenue due to the disruption of public transport systems, removal of goods from flooded areas and their return after the flood, the construction or establishment of temporary facilities for families made homeless by the flood, the erection of temporary levees and the like.

The intangible damages of a flood or any natural disaster are many and varied. They may include increased marital stress, feelings of insecurity, depression and the like among the residents on the flood plain and depressed business activity in commercial and industrial areas. Police records show that 12 people lost their lives in the Brisbane and Ipswich area in the 1974 floods. This of course, is an intangible loss together with the loss of health suffered by either the flood plain dwellers or those who helped in flood operations during or after the flood.

Figure 7 summarises the various categories of flood damage and how they are further broken down in the course of this study.

3.2 COMMENTS ON DAMAGE CATEGORIES

Clearly a monetary value can be ascribed to all direct damages as described above. It is because this type of damage can be expressed in monetary terms without great difficulty that no debate arises to justify its inclusion as part of the total flood damage. It should be noted, as pointed out by Breaden (1973), that the direct damage is either the cost to restore a building or property to its pre-flood condition in order that it can again perform its pre-flood function or, where this action cannot be justified or fulfilled, the damage should be taken as the present worth of the expected future productivity if the flood had not occurred.

The assessment of indirect damages is not as straightforward as that of the direct damages. Their nature makes accurate estimates impossible and cases of double counting of losses may occur unless assessments are very carefully examined. Further, the justification for the inclusion of many indirect losses depends entirely on the viewpoint taken in the study. In the U.S.A., the study by Kates (1965) yielded some very pertinent points. Kates stated:

> 'The question as to what constitutes an admissible flood damage loss requires an extensive digression. The present social guides are clearly inadequate. The original Congressional imperative to consider 'losses to whomsoever they may accrue' reflects the concern of Congress to identify all losses. However, little guidance is offered as to what to do when the summation of losses leads to double counting or when losses conflict - one man's loss, another man's gain. The operational procedures of the Corps*, cited previously seems conceptually valid in theory but loosely enforced in practice. For example, Corps procedures provide for the elimination as flood losses, of sales made alternatively by competitors. Although competitive sales are the rule in our economy, the author has never found an actual example in the damage survey of the Lehigh (or in several others reviewed at various times) of exclusion of a sales loss due to alternative sales by competitors.

*Corps refers to the U.S. Corps of Engineers

> The confusion sometimes found in this area stems both from ambiguity in the stance of the analyst making the economic loss valuation and the difficulty in application of the analysis. Present Corps procedures favour a stance from a national point of view. Conceptually, this embodies the notion of economic efficiency or measuring flood damages as decreases in national income. A variety of other stances, valid for other purposes, might be considered as well. There is the traditional concept of the firm and this stance is appropriate for preparing the balance sheet of a firm's flood losses.
>
> A somewhat enlarged concept of a firm as a productive unit that is concerned with gains and losses to labor as well as to the owners of capital might be called an establishment. An aggregation of such units are found in the community or region. Analysis from the establishment point of view, would be appropriate for making flood damage estimates where the regional or local impact of a flood control program was being measured'.

On the basis of this discussion, the analysis of indirect damages included in this report falls into the category of an analysis from the 'establishment' point of view because no effort has been made to eliminate from the claimed losses, additional sales made by competitors. (It appears very doubtful whether, in fact, there is any way of really knowing when additional sales are made by competitors).

3.3 PREPAREDNESS OF FLOOD PLAIN OCCUPANTS AND AUTHORITIES

The January 1974 flood occurred on the 'Australia Day' long weekend when many residents of Brisbane were absent from their homes. Likewise owners, managers and staff who would normally operate or work in industrial and commercial buildings, public buildings and offices were also absent. As the flood situation was widespread even those who wished to heed the flood warnings could not, in many cases, return because access to Brisbane had been cut-off by flood-waters. In addition the January 1974 flood followed a very long period without any major flooding on the Brisbane River like that of the 1890's, although in 1931 a flood of 3.5 m at the Brisbane City Gauge occurred. This flood height corresponds to the lower limit for major flooding.

Given the above circumstances it is not altogether surprising that in 1974 neither the occupants of the flood plain nor the governmental authorities were well prepared for the occasion. The question then arises whether the substantial efforts of Australian, State and Local government authorities and the occupants of the flood plain since the 1974 flood can reduce the damage resulting from future floods. Unfortunately, experience elsewhere in Australia and overseas indicates that within a period of 20 or 30 years if no substantial flood has occurred the occupants become either apathetic, ignorant of the facts or falsely confident that a flood will not recur. Personnel in public offices change with time (as do the occupants of the flood plain) and the knowledge and experience which was gained from a prior flood is slowly eroded. It is on this basis that this study makes the important assumption that tangible flood damages from future floods are not likely to be reduced significantly because of lessons learnt from the January 1974 flood. More success is likely to be achieved by reducing intangible damages through the operation of improved techniques of flood forecasting and better communications generally.

3.4 GROWTH FACTOR FOR THE FLOOD PLAINS

Consideration was given to applying a growth factor to present day estimates of flood damage to allow for further development on the flood plain and hence increased risk of flood damage. In the Queensland Co-ordinator General's Department report (1971) it applied a growth factor of 10% to the industrial and commercial areas adjacent to Oxley Creek and to the areas upstream to the boundary of the City of Brisbane. This growth rate was expected over a 10-year period to 1980. For residential development the same report allowed a growth rate of 5% in existing rural areas. On the evidence available at the present time, it seems possible that considerable portions of the central business area of Brisbane and many areas such as South Brisbane could be redeveloped with the result that the potential for flood damage would also be considerably increased in these areas. There are also many locations subject to flooding from the Brisbane River in the outer parts of the City of Brisbane, the City of Ipswich and the Shire of Moreton, particularly between the Bremer and Brisbane Rivers which could be urbanised and would then add very substantially to residential losses. However, the projection of growth rates depends on many factors, including such matters as the implementation of flood-plain zoning and flood-plain insurance schemes. These considera-

ations were not within the scope of this study and, for this reason, it was decided to base all estimates of flood damage on the situation as it existed at the time of the January 1974 flood.

3.5 DISCOUNTING OF TRIBUTARY FLOOD DAMAGE

(a) General

The characteristics of the 1893 flood and the 1974 flood illustrate a most important point concerning the derivation of stage-damage curves for the Brisbane River. The 4-day isohyets for the rainfall which produced the first flood in February 1893 are shown on Figure 5(b). This figure shows the flood producing rainfall was centred near the northeastern divide where the rainfall reached 1 800 mm. For the metropolitan area the maximum rainfall was of the order of 400 mm, whilst the highest falls over the Bremer River catchment were about 200 mm. Heatherwick (1974) gives the average rainfall for the 1893 flood over the Stanley River catchment as 939 mm compared with 137 mm over the Bremer and 288 mm over the Brisbane metropolitan area. In these circumstances, tributary flooding in the city* would not have been very severe and the Bremer River too would have produced only a small flood. Backwater flooding in Ipswich, however, was very severe with the level at the Ipswich City Gauge reaching a maximum of 24.5 m (A.H.D.).

Backwater flooding in the metropolitan tributaries in 1893 would also have been very severe.

In January 1974 the rainfall was centred nearer the Brisbane metropolitan area as can be seen in Figure 5(c). Rainfalls of 800 mm to 1 000 mm were recorded in the downstream sections of the catchment. Heatherwick (1974) gives the average rainfall for the Stanley River catchment as 507 mm, the Bremer 461 mm and the Brisbane metropolitan area as 656 mm. As is well known, this rainfall distribution led to a situation in which many areas suffered flooding from local tributaries and later backwater flooding from the Brisbane River. In this report this effect is referred to as double flooding; the first flood

* An additional indication of the severity of tributary flooding in 1893 was that the recorded head of 1.1 m at Enoggera Dam was less than half that recorded in January 1974 when the flow depth was 2.5 m.

being due to tributary or local flooding and the second due to the Brisbane River flood. Double flooding also occurred in Ipswich where the first flood was caused by the Bremer River and a later flood by the backwater from the Brisbane River. The maximum water level reached at the Ipswich City Gauge was 20.7 m (A.H.D.) for both the Bremer River flood and later the Brisbane River backwater flood.

The question is, therefore, how to rationally treat a situation where double flooding can occur as the two flood situations are obviously not independent as they can be caused by the same meteorological event. In the Co-ordinator General's report (1971), areas which fell into this category were discounted by 30% and 70% for residential and for industrial and commercial areas respectively. The basis of the figures of 30% and 70% is not given, but it is stated that these reductions were included to make allowance for the fact that prior local flooding may have resulted in only incremental flood damage from Brisbane River backwater. An approach of this nature appears logical from the point of view that it removes the possibility of double counting of flood damages in evaluating cost-benefit results for flood mitigation works on both the tributaries and the main river.

(b) Discounting of Flood Damage along the Bremer River
The question of flood damage attributable to Brisbane River floods backing-up along the Bremer River is complex. It is apparent that in 1893 the flood damage would have been caused mainly by the Brisbane River backwater flood while in 1974 the Bremer River flood was responsible for most of the damage in the City of Ipswich. These two floods appear to illustrate the likely extremes of the flood situation along the Bremer River when there is significant flooding along the Brisbane River. It is also possible to have flooding along the Bremer River without flooding along the Brisbane River, such as occurred in January 1947, but this situation is of no direct consequence to this study.

A search was made of the records of flooding along the Brisbane and Bremer Rivers, supplied by the Bureau of Meteorology, and two more significant floods were noted. These occurred in 1931 and 1955, when the gauge reading at the Ipswich City Gauge reached 15.47 m (A.H.D.)

and 13.82 m (A.H.D.) respectively. The 1893, 1931, 1955 and 1974 floods appear to present a logical basis for calculating a discount factor for flood damage along the Bremer River in the City of Ipswich. For each of these floods, the area inundated by the Brisbane River backwater flood (A_{bw}) and also the area within the Brisbane River backwater which had prior flooding from the Bremer River (A_t) were measured. Assuming the following simple relationship, the values of the discount factor for the above four floods were determined and are presented in Table 1.

$$\text{Discount factor} = \left[1-(A_{bw} - A_t)/A_{bw}\right] 100 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where A_{bw} = area flooded by the Brisbane River backwater,

A_t = area flooded by tributary flooding (in this case the Bremer River) prior to the Brisbane River backwater flood.

TABLE 1 - BASIS OF DISCOUNT FACTOR FOR THE BREMER RIVER

FLOOD	A_t km^2	A_{bw} km^2	DISCOUNT FACTOR %
1893	0.0	42.4	0
1931	3.1	9.0	34
1955	0.0	5.1	0
1974	14.0	16.2	86
			Mean 30

The mean value of the discount factor for the four floods is 30%. Although this factor is based on an analysis of only four floods, it is believed that no better information is available to calculate the factor and, for this reason, it has been used in later sections to adjust the flood damage for Ipswich estimated on the basis of a level backwater.

(c) Discounting of Flood Damage for Metropolitan Tributaries
The question of the application of a discount factor to flooding in the Brisbane metropolitan tributaries is also complex. These tribu-

taries have caused severe flooding in recent years (1967 and twice in 1972) at times when no flooding occurred along the Brisbane River. On the other hand, in the January 1974 situation, flooding first occurred from the tributaries and later from the Brisbane River.

The relative catchment sizes of the Brisbane River catchment (13 600 km^2) the Bremer River catchment (2 020 km^2) and the metro-politan tributaries (up to 80 km^2) suggest that the correlation between the Brisbane and the Bremer floods will be much stronger than that between the Brisbane and metropolitan tributary floods. The small metropolitan catchments could even possibly respond to isolated thunderstorms, whereas floods from the larger catchments will almost certainly be caused by cyclones.

As a result of the floods prior to the January 1974 flood, the Co-ordinator General's Department had already engaged consultants to investigate methods of mitigating floods in the metropolitan tribu-taries. These works, however, will only mitigate local flooding along the tributaries and areas subject to backwater flooding from the Brisbane River will remain in that situation. As there exists a basic long-term objective to carry out flood mitigation works on both the tributaries and the main river, the argument to discount for areas of double flooding is less credible. This can be illus-trated by assuming that flood mitigation proposals for Enoggera Creek have been shown to be justifiable (on the basis of Enoggera Creek flooding only) and implemented. In these circumstances, a Brisbane River flood will still cause backwater flooding in Enoggera Creek, but instead of this causing incremental damage as may have occurred under the prior conditions of double flooding,* the full impact of the backwater flood will be felt by the occupants of the affected part of the flood plain. This same argument can be applied to each metropolitan tributary and it is on this basis that the stage-damage curves presented in this study make no allowance for discounting tributary flood damage.

* This statement only holds true when the tributary flood is contained within the flood mitigation works for the tributary.

4. ADOPTED APPROACH TO ASSESS FLOOD DAMAGE

4.1 INTRODUCTION

At the outset of this study it was realised that many thousands of buildings were affected by flooding from the Brisbane River. It was also realised that the task of adding together the estimated damages for all these buildings for floods of various magnitudes would require a vast number of relatively simple repetitive calculations. The problem was, therefore, ideally suited for solution by computer techniques provided a data bank containing basic details of all the buildings on the flood plain could be compiled.

4.2 DATA BANK

To compile the data bank certain essential information on all flood affected buildings had to be transferred from detail plans to punched cards. To carry out this task, dyeline prints of the detailed sewerage maps for the Cities of Brisbane and Ipswich were obtained. These maps were drawn at a scale of 40 feet to 1 inch or, in the case of some maps in Ipswich, at a scale of 50 feet to 1 inch. Over 1 000 of these detail maps were obtained through the Brisbane City Council and the Ipswich City Council.

Using the maps, the following data were transferred to punch cards for each residence on the flood plain which would be affected by a flood reaching 9 m (A.H.D.) at the Brisbane City Gauge:

(i) REFERENCE NO - a consecutive number for each postcode which was used for checking purposes

(ii) ADOPTED MIDDLE THREAD DISTANCE (A.M.T.D.) - the distance from the river mouth measured along the river to a position adjacent to the residence in question. Where backwater flooding occurred on tributaries, the A.M.T.D. was taken as that value occurring at the mouth of the creek or river entering the Brisbane River

(iii) POSTCODE - The Australian Post Office postal district number

(iv) ADDRESS - the street name

(v) BUILDING MATERIAL - the predominant type of building material

(vi) FLOOR LEVEL - the reduced level of the main floor in the resi-dence to A.H.D.

(vii) PROPERTY LEVEL - the approximate reduced level of the centre of the property to A.H.D.

(viii) FLOOR AREA - the area of the main floor of the residence in square metres

(ix). EXCEPTIONS - these were used to flag irregularities in the data; for example, a change in datum for reduced levels.

The task of transferring the residential data into the required form was far more difficult than initially anticipated. The main difficulties were:

- the necessity to transfer the outlines of many residences from recent aerial photos to detail maps at locations where floor levels had not been recorded. This assisted the collection of the missing data in the field
- the extensive use required of the 4 chain to 1 inch maps and aerial photographs in areas not covered by the detail maps
- confusion arising from two sets of detail maps in some areas and incorrectly designated postcode boundaries on commercially available street maps

Over 21 000 residences were transferred to the data bank of which approximately 70% of these had all the required data available on the maps. About 25% of the residences required floor level information to be picked up in the field. For the remaining 5% of the residences, a rough estimate was made of the floor level, either on the basis of surrounding houses, or evidence on the aerial photographs.

Similar data to that described above for residences was transferred to the data bank for all remaining buildings on the flood plain. It was decided to proceed on the basis of definable buildings, rather than on an ownership basis, because many of the larger commercial and industrial complexes consisted of several buildings at different floor levels. This section of the data bank was divided into three groups:

- commercial buildings
- industrial building
- miscellaneous buildings

In general, the separation of industrial and commercial buildings was not too difficult, although some buildings could have been placed in either category. Local knowledge gained during the course of the project and town planning maps supplied by the Brisbane City Council were both useful guides in defining commercial and industrial areas. Miscellaneous buildings included all those buildings not judged to be a residence, commercial or industrial building. It therefore included such buildings as churches, halls, sports pavilions, schools and the like. This part of the data bank presented similar difficulties in compilation to that described above for residences, although it was helped by the reduced number of buildings involved. On the other hand the larger size of the buildings permitted easier interpretation from aerial photographs. In all, data for over 7 000 commercial, industrial and miscellaneous buildings were transferred to the data bank.

Although at the outset greater accuracy had been sought with the compilation of the data bank than had finally been achieved, it is extremely unlikely that the inaccuracies introduced as a result of deficiencies in the data on the detail maps could have introduced a bias in the total flood damage estimates described later in this report. The accuracy of the individual values of the estimated damages to buildings would not be great but this is not a serious problem as it was not an objective of the study.

4.3 FLOOD DAMAGE COMPUTER PROGRAM

A relatively straight forward computer program was prepared to process the data after it had been transferred from punched card to magnetic tape. This program allowed the rapid calculation of most of the information required to prepare the flood stage-damage curves described in Section 11.

A flow chart for the flood damage program is shown in Figure 8. The program reads the estimated flood profile (see Section 2.6) and for each residence or building in the data bank it interpolates from the river profile the flood level corresponding to the A.M.T.D. for the building. If the building is subjected to flooding for the particular flood profile it computes the flood damage. All damages are summed in postcode areas and also along the river in kilometres from the mouth. Details of the procedures used in the computer program for the estimation of flood damages to commercial and industrial, residential and miscellaneous buildings are given in Sections 5, 6 and 7 respectively.

4.4 ITEMS NOT INCLUDED IN THE DATA BANK

It was not found to be convenient to include the damage to public utilities such as water mains, sewers, roads, bridges and the like in the data bank. Fortunately these data were not so numerous and could be conveniently handled and analysed by hand computations as described in Sections 8 and 9.

5. FLOOD DAMAGE TO COMMERCIAL AND INDUSTRIAL BUILDINGS

5.1 INTRODUCTION

In the initial stages of this study it was uncertain whether the greater amount of damage for the 1974 flood occurred in commercial and industrial areas or in residential areas. The approximate number of buildings affected in each category was known and, on this basis, it was decided to put a similar amount of effort into the assessment of damages for both categories.

With the benefit of hindsight, it was apparent at the conclusion of the study that a greater effort should have been directed to the estimation of commercial and industrial losses, because not only were they considerably higher than the residential losses, but their variability was also very much greater. In a similar manner to this study Homan & Waybur (1960) of the Stanford Research Unit in the U.S.A. found that the losses in industrial areas were very much more variable than those in the residential areas and in their study they concluded that a regression analysis of the industrial damage data was not warranted.

The commercial and industrial damages were studied in two main groups; direct and indirect damages as defined in Section 3.1. As with other parts of this study, the direct damages for the commercial and industrial buildings were relatively easy to substantiate, while the indirect damages were often vague and open to dispute.

5.2 DATA SOURCES

(a) Trade Organisations

Contacts were made with the various trade organisations in Brisbane to obtain any details of flood damages which may have been collected following the 1974 flood. The following trade organisations were able to provide helpful information:

- Metal Trades Industries Association
- Chamber of Manufacturers
- Printing and Allied Trades Employers Association

(b) Government Departments

The Department of Commercial and Industrial Development and the Ipswich District Development Board provided copies of their records of commercial and industrial buildings affected in the 1974 flood.

(c) SMEC Questionnaire

A questionnaire and covering letter (see Appendix A) was prepared and posted to 1 450 commercial and industrial firms with buildings either on or near the flood plain. The names and addresses for the posting of questionnaires were obtained either from categories (a) and (b) above or the electricity tariff records of the Brisbane City Council. A good response was received from the questionnaires with 825 returns, that is, 57% of the questionnaires posted. These returns were classified as follows:

580 useful
44 partly useful
201 not useful

825 total

In addition to the questionnaires posted by SMEC, the Chamber of Manufacturers assisted by posting SMEC's questionnaire directly to their members.

(d) Alex Overett Pty Ltd

Because it was not possible to collect all the desired information from the data sources (a), (b) and (c), it was decided to obtain some returns in greater detail using a qualified valuing firm. Discussions were held with a number of valuing firms to determine their suitability for the type of work envisaged. The firm most suitable and available to fit into the program of work was Alex Overett Pty Ltd and it was engaged to look closely at 50 of the larger damage claims. A further questionnaire and covering letter (see Appendix A) was prepared by SMEC for the guidance of the valuers and to ensure that the required information was systematically collected.

(e) Insurance Claims

The State Insurance Commissioner was able to provide a summary of the insurance claims arising as a result of the January and February 1974 weather conditions in Queensland. These figures were compiled by the Commissioner from questionnaires returned from all insurance companies operating in Queensland. The complete figures as supplied are shown in Table 2. They are broken into two main areas; the Brisbane metropolitan area and the remainder of Queensland. The figures are mainly of interest as a guide to damage in the commercial

TABLE 2 - GENERAL AND MARINE INSURANCE CLAIMS

TYPE OF POLICY	GROSS CLAIMS PAID TO 30.6.74		GROSS CLAIMS ESTIMATED AS OUTSTANDING ON 30.7.74		GRAND TOTAL
	Brisbane Metropolitan Area 1000 $	Remainder of Queensland 1000 $	Brisbane Metropolitan Area 1000 $	Remainder of Queensland 1000 $	Queensland 1000 $
Fire and House-holders:					
Storm and Tempest and Rainwater	2 048	479	2 347	463	5 337
Flood	21 356	1 186	23 442	2 454	48 438
Loss of profits:					
Storm and Tempest and Rainwater	11	1	122	12	146
Flood	1 893	608	7 666	1 273	11 441
Motor Vehicle:	1 742	376	1 413	261	3 792
Marine:	2 821	126	1 566	25	4 539
Other:	568	324	781	207	1 880
Total	30 439	3 101	37 337	4 696	75 571

NOTE: The table is based on claims made on insurance companies operating in Queensland. All the claims arose as a result of weather conditions during January and February 1974.

and industrial category as most buildings in this category carry flood insurance. Although separate figures were not available for commercial, industrial, and residential losses it is known, from the flood damage survey at Jindalee and other surveys, that few residential homes were insured against floods. In their study of Jindalee, Swanell & Issacs (1974) found that 8.6% of a sample of 280 homes had flood cover included in the insurance policies for their homes, and the bulk of these were Australian Government Defence Service Homes.

All the data obtained through sources (a) to (d) inclusive have been summarised, according to depth of flooding, in Appendix B. Because these data were provided on a confidential basis names and addresses of individual commercial and industrial buildings have been omitted.

5.3 ASSESSMENT OF ACCURACY OF DATA

It was realised that the estimates of flood damage given in some questionnaires were probably exaggerated. The valuers were therefore engaged to examine in detail the flood damages for mainly larger commercial and industrial buildings. A comparison of the two lots of estimates was carried out and this showed that:

- the differences between Overett's estimates and that of the other sources exceeded 50% on seven occasions for the direct damage and five occasions for the direct plus indirect damage
- the mean percentage difference for the direct damage was −19% and for the direct plus indirect damage was −18%. That is, the results from the questionnaires appeared to be on the average too high, perhaps by as much as 20%

It should be noted, however, that the figures of −19% and −18% were largely influenced by one apparently very exaggerated claim which was about 400% higher than that of Overett's estimate of damage. Omitting this value from the calculations, the percentages reduced to −10% and −8% respectively. Figure 9 shows two graphs with Overett's results plotted against the questionnaire results (sources (a), (b) and (c)) for both direct, and direct plus indirect damage. As the mean percentage difference referred to above and the results shown on the two graphs reveal a slight bias towards over-estimation in the questionnaires, it was decided to reduce the mean damage per unit area by a factor of −15% when applying these to estimate total flood damage in the computer program.

5.4 ANALYSIS OF THE DATA – METHOD 1

Various groupings of the data for commercial and industrial damage were considered but apart from the most obvious of grouping according to depth of flooding none proved to be helpful.

After subdividing the data for commercial buildings into 10 groups, according to the depth of flooding over the main floor level, a mean depth was calculated for each of the 10 groups. This mean depth was plotted against the 'overall mean flood damage per unit area' defined as the total damage within the group divided by the total floor area flooded in the group. These damages per unit area were calculated for both the direct and indirect damage and are shown plotted on Figure 10(a). For the direct damage the figure displays the generally expected type of relationship in as much as a roughly linear increase in the mean damage per unit area occurs up to about 3 m, after which further flooding does not increase the damage. The indirect damage suggests that there is only a very weak correlation with stage but this was not unexpected.

The same analysis as that described above was carried out for all the industrial buildings and this yielded similar results as shown in Figure 10(b).

In an attempt to smooth out some of the irregularities in the curves for direct damage in Figure 10(a) and (b), it was decided to combine all the industrial and commercial damage data together and plot the mean depths for each group against the overall mean damage per unit area for each group. These results are shown in Figure 10(c). In this case, the plotted points for the direct damage are a good representation of the expected stage damage relationship and it was therefore adopted in METHOD 1 for the estimation of the commercial and industrial damages. To allow for the over-estimation of damage in the questionnaires (referred to in Section 5.3) the computer program reduced all the damage per unit area values by 15%.

The relationship between stage and indirect damage shown in Figure 10(c) is obviously not strong and hence an alternative approach which is described in Section 5.7 was used to estimate these damages.

5.5 ANALYSIS OF THE DATA - METHOD 2

Because of the importance of the commercial and industrial component in the total damage, a further analysis of the combined data was made using a different method of analysis.

In METHOD 1, the damages were referred to as overall mean damage per unit area because they were computed by dividing the total damage in a group by the corresponding total area. This approach tends to give the greatest weight to the greatest values (either damages or areas). Alternatively, the mean of the individual damage per unit area could possibly be utilised with equal justification. These values were calculated for the combined commercial and industrial damage and the results are listed in Table 3 below together with their corresponding standard deviations. The mean depth

TABLE 3 - STATISTICS FOR THE COMMERCIAL AND INDUSTRIAL DAMAGES PER UNIT AREA

RANGE OF FLOOD DEPTH m	NUMBER IN SAMPLE	DAMAGE PER UNIT AREA - $/m^2$			
		Arithmetic mean	Arithmetic s.d.*	Mean of logarithms	s.d. of logarithms
0-0.30	21	14.2	25.0	0.705	0.675
0.31-0.90	78	26.2	80.9	0.862	0.662
0.91-1.36	84	33.9	51.7	1.24	0.605
1.37-1.82	62	39.7	46.8	1.29	0.593
1.83-2.28	63	70.8	106.3	1.54	0.549
2.29-2.73	44	66.3	62.6	1.60	0.519
2.74-3.65	52	74.5	124.5	1.59	0.533
3.66-4.56	66	85.8	103.7	1.65	0.511
4.57-5.49	35	92.6	95.0	1.70	0.555
> 5.49	57	60.4	69.9	1.53	0.495

s.d.* denotes standard deviation

for each range given in Table 3 was plotted against its corresponding mean damage per unit area as shown in Figure 11(a) to produce a graph similar to Figure 10(c). A comparison of the two figures reveals that the adoption of Figure 11(a) instead of Figure 10(c) would result in substantially higher estimates of flood damage.

Table 3 also lists the means and standard deviations of the logarithms of the damage per unit area. The logarithmic mean shows a gradual and smooth increase with increasing flood depth before coming to a reasonably steady value, while the standard deviations indicate a slight decrease in variability with increasing depth.

Examination of the arithmetic standard deviations of the mean damage per unit area in Table 3 shows that these data are positively skewed. This observation suggests that the logarithms of the damage per unit area may be approximately normally distributed. The nature of the damage data also suggests that it could be log-normally distributed because a few items have very high damage. A good example was a glass-fibre manufacturer which suffered estimated damage per unit area equal to $ 670/m² while, at the other extreme, the lowest values must always be slightly greater than zero. The positive skew in the data was normalised by taking the logarithms of the damage and the 'good fit' of the data to a log-normal distribution demonstrated by ranking the damage per unit area for three groups in descending order and determining a plotting position using the formula:

$$\text{Probability} = \frac{m}{N+1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

where m = rank number, and
N = number of samples

These data, for three ranges of flood depth, were plotted on logarithmic normal paper as shown in Figure 11(b). The fact that the data plots generally as a straight line strengthened the view that they could be approximated by a log-normal distribution. On this basis it was possible to preserve the logarithmic normal distribution of the data by using a Monte Carlo technique to generate damage per unit area for each building in the data bank for commercial and industrial buildings. The basic equation for the generation of the damage was:

$$\$D_g = \text{antilog}\ (\$D_m \pm \epsilon\ (\$SD)\ ) \quad \ldots\ldots\ldots\ldots \quad (3)$$

where $\$D_g$ = the generated value of the damage per unit area in $/m²,

$\$D_m$ = the mean value of the logarithms of the damage per unit area,

$\$SD$ = the standard deviation of the lorarithms of the damage per unit area,

ϵ = a random normal variate.

Implicit in using the generation equation is the assumption that the damage per unit area is statistically independent of the size of the building. In other words, a very high (or low) rate per unit area is equally likely to apply to either a large industrial (or commercial) building or to a small building. To investigate that this was actually the position, two different ranges of depth of flooding were selected from the data in Appendix B and, for each building, the floor area was plotted against its corresponding damage per unit area. As anticipated, the resulting graphs showed that no correlation existed between the two variables.

A comparison of the results for METHOD 1 (Section 5.4) and METHOD 2 are given in Section 5.6. Attention is drawn to the fact that METHOD 2 generated values of D_g which bear no direct relation to the actual damage which may occur at a particular commercial or industrial building. The method relies on the premise that by generating a great number of individual damage per unit area values in such a way that the observed statistical distribution of these damages are preserved, an unbiased estimate of the total damage will result.

5.6 COMPARISON OF RESULTS FROM METHODS 1 AND 2

The total industrial damages were computed for floods with heights at the Brisbane City Gauge of 2 m, 4 m, 6 m and 8 m using METHOD 2 and compared with the corresponding results from METHOD 1. As could be expected METHOD 2 gave consistently higher results with the mean for the four floods being 21% greater. The method would also give higher results for the commercial buildings. Although no reason could be found for rejecting the results from METHOD 2 it was decided to adopt the lower results by the more conventional approach used in METHOD 1. This was a conservative assumption from the viewpoint of the benefit-cost studies for the proposed dam at Wivenhoe as adoption of the results of METHOD 2 for estimation of commercial and industrial damage would increase the total flood damage for any stage.

5.7 INDIRECT DAMAGE

The indirect damage as defined in Section 3.1 were covered in all the questionnaires and in the work by the valuing firm Alex Overett Pty Ltd. Nevertheless, it remained a difficult part of the analysis because the

question of whether certain claimed damages were admissible frequently arose. As defined in Section 3.2, the analysis of indirect damage in this study is from the 'establishment' point of view and not from a 'national' point of view .

The results from all assessments of flood damage to commercial and industrial buildings were studied separately. It was found that for 206 commercial buildings which gave results of both direct and indirect losses, the indirect losses averaged 35% of the direct losses for commercial buildings while, for a sample of 236 industrial buildings, the corresponding figure was 65%. A separate analysis using only the returns from the valuing firm Alex Overett Pty Ltd, was made and this gave 45% for the commercial losses and 51% for the industrial losses. These sample sizes were small being 11 and 24 respectively. In the study by Kates (1965) the corresponding values for the U.S.A. ranged from 23% to 48% for commercial areas and 25% and 123% for industrial areas. Kates suggested the adoption of values of 37% for commercial and 45% for industrial establishments.

In flood mitigation studies for the metropolitan tributaries carried out in recent years for the Queensland Co-ordinator General's Department, the figures adopted for indirect damage were those suggested by Kates; that is, 37% for commercial areas and 45% for industrial. Although the industrial figure (45%) is substantially lower than that indicated by the data available for this study (65%) it was decided for consistency that the figures adopted in earlier allied studies for the Co-ordinator General's Department should also be adopted in this study.

5.8 TRANSFERABILITY OF RESULTS

A great deal of effort was put into the collection of data for commercial and industrial buildings and these data have been summarised in Appendix B of this report. These data should prove useful for flood mitigation studies in other cities where a wide range of commercial and industrial building are affected. In areas of more limited flooding the data could probably be used selectively to give a general guide to the likely losses per unit area due to flooding at various depths. Attention is drawn, however, to the conclusion of Section 5.3 that the information collected from questionnaires is probably inflated by an average of about 15%.

The results and methods of analyses of the data should also be of assistance to other flood damage studies either directly or as a basis of comparison.

6. FLOOD DAMAGE TO RESIDENCES

6.1 INTRODUCTION

Except for the estimation of indirect damage, the task of assessing damage to residences was relatively straight forward. This was largely attributable to the smaller variability of the data in comparison with the flood damage data for the commercial, industrial, and miscellaneous buildings (see Sections 5 and 7). Other studies have shown that a functional relationship can be derived by regression analysis between flood damage to a residence and a number of possible independent variables such as depth of flooding, floor area, type of building material, value of building, etc. This type of approach was used by Homan & Waybur (1960) of the Stanford Research Institute in their study of flood damage data collected in the U.S.A.

In this study it was found convenient to divide the direct damage to residences into structural and contents damage as defined below. Separate consideration was also given to:

- clean-up costs for residences
- flooding above property level but below floor level
- indirect damage

6.2 SAMPLE SIZE FOR ANALYSIS

At the commencement of the project it was thought that of the order of 10 000 to 15 000 residences were affected by the January 1974 flood in the Cities of Brisbane and Ipswich. Obviously it was impossible to analyse all these buildings for damage. Instead a selected representative sample of 500 residences was chosen for initial investigation.

6.3 STRUCTURAL DAMAGE

For the purposes of this study, structural damage to residences was taken as the cost of repairing the building to its former state. It included the cost of repainting where this was necessary. Fortunately for this study the Queensland Government established the Flood Damaged Homes Committee after the January 1974 flood, to investigate and allocate funds for the restoration of flood damaged residences. As part of the process of allocating funds, building inspectors were engaged to assess the cost of repairing structural damage to residences. Arrangements were made to extract selected details of these data on the condition that they be treated confidentially. (Recently, the Flood Damaged Homes Committee (1975) published its final report which contains a great amount of detail on structural damage to flood affected homes in Queensland in 1974).

At the time of collecting these data, the Committee had investigated of the order of 6 000 claims. A selection of these files were studied and the following details were extracted for a sample of 1 500 residences:

- applicants name
- address of property
- foundation height
- depth of flooding
- type of house (building material)
- estimated cost of structural damage
- indicators of data of a different character, e.g., double-storied homes were 'flagged'

This sample was selected to ensure that all the postcode districts affected by the flood were adequately represented and that all building types and flood depths were properly sampled. These data were transferred to punch cards for processing by regression analyses as described in Section 6.5.

6.4 CONTENTS DAMAGE

In this study, damage to the contents of residences was taken to be the cost of either restoring or replacing with a similar standard of article all flood damaged furnishings, clothing and other contents of residences. In the case of residences flooded above the main floor area, the damage to the garden, fences and other similar exterior parts of the residence were included in the contents damage. Where a property was flooded, but the water level did not exceed the main floor level, the cost of the damage was added as a separate item, as described in Section 6.6.

As the work involved in assessing contents damage was not within the range of activities normally undertaken by SMEC a Brisbane based property valuer was engaged to carry out this task. Discussions were held with a

number of valuers and it was apparent that the firm of Alex Overett Pty Ltd could most adequately carry out the work envisaged in the time required. SMEC engaged this firm to undertake initially an assessment of contents damage to the sample of 500 residences referred to in Section 6.2. A questionnaire (see Appendix A) was prepared to ensure that the valuers obtained the information required for the study. The questionnaire used was based on a questionnaire developed by Munro & Johnson (1973) for their flood mitigation studies of Kedron Brook, also in Brisbane.

The arrangement with the valuing firm proved to be most successful. The firm posted to each resident in the selected sample a letter from SMEC (see Appendix A) explaining the purpose of the study. After a few days a personal contact was made with the owner of the residence and, where possible, the assignment was completed. The valuers provided the results progressively in order that preliminary analyses could be put in hand immediately. In this way it was found possible to cut the sample back from 500 to 400 (see Section 6.5). In the majority of the cases the owners contacted were most co-operative and interested in the study. The only real difficulty encountered by the valuing firm was the comparatively high number of residences at which no occupants could be interviewed during the working week. Some of these were interviewed at weekends but, where this was not possible additional names and addresses were provided from the original sample of 1 500 until 400 assessments had been successfully completed.

6.5 REGRESSION ANALYSIS OF THE DAMAGE DATA

As mentioned above, the information on the assessment of contents damage was made available by the valuers in batches of 50. The questionnaires were examined carefully and the following additional data for each residence were transferred to the punched cards which contained the corresponding structural damage details:

- the estimated contents damage
- the estimated market value of the residence
- the estimated floor area of the building

A regression analysis of the accumulated data was carried out as each additional batch of data arrived from the valuers. By the time the total sample had reached 400 it was apparent that additional assessments would not add significantly to the accuracy of the derived regression equations. This is apparent from Tables 4 and 5 which show the standard errors and correlation coefficients for the initial regression analyses which developed both arithmetic and logarithmic equations for estimating contents and structural damage using the following independent variables:

- flood depth above the main floor level
- floor area of the building
- estimated market value of the building

For the final regression studies the estimated value of the building was omitted, because it was not readily available for all the residences on the flood plain for which the equation would have to be applied when using the flood damage computer program described in Section 4.3. However, as the estimated market value was closely correlated with the floor area ($r = 0.74$) the standard error of the final equations did not change appreciably from those indicated in the initial analysis. The final arithmetic and logarithmic equations were based on a sample of 375. This sample excluded all double-storied houses which were included in the original sample of 400 assessments. The final arithmetic and logarithmic equations were:

$$\$DR_s = 150 + 466\,(FH) + 10.0\,(FA) \quad \text{.......... (4)}$$

$$\$DR_c = 204 + 328\,(FH) + 13.5\,(FA) \quad \text{.......... (5)}$$

$$\$DR_s = 203FH^{0.581}FA^{0.413} \quad \text{.......... (6)}$$

$$\$DR_c = 56.4FH^{0.517}FA^{0.645} \quad \text{.......... (7)}$$

where:

$\$DR_s$ = the estimated January 1974 damage to the structure of the residence,

$\$DR_c$ = the estimated January 1974 damage to the contents of the residence,

FH = flood height above the main floor level in metres, and

FA = floor area of the main floor level in square metres.

The standard errors and correlation coefficients for these equations are given in Table 6 on page 46.

TABLE 4 - SUMMARY OF RESULTS OF ARITHMETIC REGRESSION ANALYSIS

SAMPLE SIZE	MEAN FLOOD HEIGHT	MEAN FLOOR AREA	MEAN MARKET VALUE	MEAN DAMAGE		STANDARD ERROR OF ESTIMATE		MULTIPLE CORRELATION COEFFICIENT	
				Structural	Contents	Structural	Contents	Structural Damage	Contents Damage
N	m	m²	$	$	$	$	$		
50	1.95	121	11 365	2 557	2 606	1 106	1 622	0.475	0.510
100	2.21	119	12 432	2 761	2 480	1 085	1 465	0.638	0.356
150	2.45	114	11 842	2 536	2 340	1 129	1 360	0.554	0.427
200	2.44	108	11 034	2 477	2 130	1 209	1 382	0.573	0.464
250	2.44	104	10 272	2 420	1 985	1 186	1 320	0.556	0.462
300	2.48	103	9 918	2 386	2 048	1 145	1 265	0.575	0.469
350	2.32	107	10 399	2 324	2 024	1 144	1 536	0.589	0.507
400	2.42	107	10 514	2 315	2 035	1 232	1 497	0.596	0.519
375*	2.48	104	10 136	2 351	2 021	1 229	1 216	0.588	0.516

* Excluding double-storied residences

TABLE 5 - SUMMARY OF RESULTS OF LOGARITHMIC REGRESSION ANALYSIS

SAMPLE SIZE N	PERCENTAGE STANDARD ERROR OF		MULTIPLE CORRELATION COEFFICIENTS	
	Structural Damage	Contents Damage	Structural Damage	Contents Damage
50	55.4,-35.6	118.9,-54.3	0.548	0.427
100	52.9,-34.6	97.5,-49.4	0.658	0.370
150	54.2,-35.2	92.7,-48.1	0.667	0.422
200	63.9,-39.0	93.1,-48.2	0.654	0.476
250	64.1,-39.0	90.3,-47.5	0.636	0.481
300	61.9,-38.2	86.2,-46.3	0.650	0.487
350	67.8,-40.4	99.7,-49.9	0.639	0.546
400	71.5,-41.7	102.6,-50.6	0.619	0.548
375*	63.9,-39.0	96.9,-49.2	0.652	0.517

* Excluding double-storied residences

TABLE 6 - STATISTICS OF FINAL REGRESSION EQUATIONS FOR RESIDENCES

TYPE OF EQUATION	STANDARD ERROR		CORRELATION COEFFICIENT	
	Structural	Contents	Structural	Contents
Arithmetic	$ 1 241	$ 1 217	0.578	0.515
Logarithmic	64.5%,-39.2%	96.9%,-49.2%	0.648	0.517

A decision was necessary on the type of regression equation to adopt for the flood damage computer program described in Section 4.3. The analysis described above revealed very little difference between the accuracy of the two equations based on their respective standard errors and correlation coefficients. The only similar study of this nature located in the literature was that by Homan & Waybur (1960) who developed the following arithmetic regression equation:

$$\$DH = -761 + 0.0255\ (\$MV_s) + 0.2325\ (\$MV_c) + 93.34\ (FH) \quad \ldots\ldots \quad (8)$$

where $\$DH$ = total damage to the residence,

$\$MV_s$ = market value of the structure,

$\$MV_c$ = market value of the contents,

FH = flood height above the main floor level (in inches).

The standard error and correlation coefficient for this equation was $ 987 and 0.735 respectively. The linear form of the equation has been criticised by Robinson (1970) as being inappropriate. To further examine this criticism the results from equations (4) to (7) were plotted in Figure 12 for the extremes of flood areas covered by the sample. Figure 12 shows that the (linear) arithmetic equation gives unrealistically high values for the flood damage for zero flood height while the non-linear equation gives zero damage at zero flood height and a decreasing rate of damage with increasing flood depth. On this basis, it appeared to be more logical to adopt the logarithmic regression equations for use in the flood damage program. The actual difference in overall flood damage estimates to residences by the two sets of equations was only 4%. This was determined by running the data bank for the residences through the flood damage computer program for an 8 m flood at the Brisbane City Gauge. The logarithmic regression equation yielded the lower result. It should be carefully noted that because the standard errors of equations (4) to (7) are all comparitively large the accuracy of estimates for individual residences would not be high.

For comparison purposes, Figure 12 also shows the assumed stage-damage curves for individual residences used in the flood mitigation reports by the Co-ordinator General's Department (1971) and Cameron, McNamara & Partners (1973). Both these curves are for total damages, that is the sum of contents plus structural damage.

6.6 BELOW FLOOR LEVEL FLOODING

In many instances in the January 1974 flood residences were affected by flooding but, because the water did not reach the main floor level, the amount of damage was not great. To provide for residences falling into this category, the information classified under external damage in the residential questionnaire (see Appendix A) was extracted. No relationship was apparent between the damage in this category and flood depth and, therefore, an average of all 400 returns was adopted for use in the flood-damage computer program. The average value was $ 135 and includes damage to fences, gardens, possessions stored under the house, lawns and outbuildings.

6.7 CLEAN-UP COSTS

The cleaning-up of residences after a flood is a task which involves much volunteer labour and many, usually unaccounted for, hours of work in the weeks following the flood. In the case of the Brisbane River flood, the streets of the affected areas were often crammed with the cars of volunteers who turned out to help in cleaning-up operations. These volunteers were, in many cases, on leave from their normal employment while others would have been unemployed members of the community such as housewives, students and school children. Although there is considerable difficulty in including this item as a flood-damage cost, other studies have chosen to do so, for example Breadon (1973) states:

> 'The property owners and their families, neighbours and friends invest long, hard hours in drying damp belongings and in removing the sediment and debris deposited by the flood. The sacrifice represented by these efforts may be a major damage item and can be estimated by man-hours of work at an appropriate wage'.

In this study the valuers were requested to obtain from each resident for which an assessment of contents damage was carried out, an estimate of the man-days of time occupied in cleaning-up activities. These estimates, which were much higher than expected, were classified into eight ranges in flood depth and a mean for each range determined. Figure 13 shows a graph of the mean number of man-days of clean-up time versus flood depth. Although the resulting graph is a smooth curve, the variability within each range of depth was very considerable. The curve on Figure 13 was described by the following equation in the flood damage computer program:

$$C = 16.5 \ln (F/0.023) \qquad (9)$$

where:

C = clean-up time in man-days for a house,

F = flood depth over floor level in metres.

The inclusion of this item as a damage requires a decision as to what 'appropriate wage' should be adopted for converting the man-days of work to a cost in dollars. The 1973 Official Year Book of Australia gives for Queensland average weekly earnings at nearly \$ 100. However, as many of those employed in the clean-up operation would not normally be employed, it was arbitrarily decided that the determination of total clean-up costs should be based on a figure of \$ 50/week of work.

6.8 INDIRECT DAMAGES

So far the residential damage considered all fall into the category of direct damage as defined in Section 3.1. Allowance for indirect damage (also defined in Section 3.1) is much more difficult. No assessment of this type of damage was included in the work undertaken by the valuers and, as no simple means existed for obtaining a reasonable estimate for indirect damage to residences, it was decided to adopt the results obtained from overseas studies. The work by Kates (1965) summarises figures compiled by the U.S. Corps of Engineers for four districts in the U.S.A. The adopted result was that the indirect damage was 15% of the direct. A similar result was obtained by Homan & Waybur (1960) of the Stanford Research Institute. On the basis of these two studies, the indirect residential damage was taken as 15% of the direct damage.

6.9 TRANSFERABILITY OF RESULTS

A considerable amount of work was required to collect the data for contents and structural damage, to derive the regression equations and to compile the curve for clean-up costs. These studies should, however, be a useful guide for estimating flood damage in other Australian cities both existing and proposed. Whenever transferring the data to other locations due consideration should be given to possible differences in the design and construction of residences. The sample used in these studies was selected to account for all types of residences found in the suburbs of Brisbane. These ranged from the typical timber, high-set residence built on 2.5 m posts in the older Brisbane suburbs, to the modern low-set brick veneer homes more typical of residences in Australia's southern cities.

7. FLOOD DAMAGE TO MISCELLANEOUS BUILDINGS

7.1 INTRODUCTION

In Sections 5 and 6 flood damage to commercial, industrial and residential buildings was considered. These buildings could be grouped relatively easily but there still remained other buildings which either could not be identified readily or could not be categorised in the above groups. These buildings were placed into a miscellanous group and included buildings such as churches, schools, sports pavilions, sports stadiums, railway stations, hospitals, etc.

7.2 ADOPTED DAMAGE PER UNIT AREA

Very little data were available on flood damage to buildings included in the miscellaneous category. This was not surprising as, of the estimated total of 13 000 buildings affected by the 1974 flood, only about 430 of these were placed in the miscellaneous category. The Queensland Department of Public Works provided costs for the restoration of structural damage to a number of public buildings. These buildings were mainly schools and were flooded to an average depth of 2.7 m. The mean damages were nearly \$ 40/m^2. Comparing this figure with the means (for contents plus structural damage) given in Table 3 would suggest that there was no reason to suspect that the mean damage to the miscellaneous buildings would differ greatly from that of the commercial and industrial buildings. Evidence was also available to show that the total damage per unit area to some public buildings could be great. The best example of this was where the loss of library books in one building resulted in total damage of the order of \$ 700/m^2.

On the basis of this limited information, it was decided to adopt the same values for direct damage per unit area as those adopted in Section 5 for commercial and industrial buildings. Because of the nature of these miscellaneous buildings no indirect damages were included in the estimates.

8. FLOOD DAMAGE TO PUBLIC UTILITIES

8.1 INTRODUCTION

In the 1974 flood the damage to public utilities such as sewers, roads, water mains, telephone and electricity services were all documented by the government authorities responsible for providing the service in the Cities of Brisbane and Ipswich. In most instances, all flood damage had been pooled irrespective of the source of flooding. Hence some judgment was required to separate damage due to tributary flooding and the Bremer River, and that due to flooding from the Brisbane River.

There were some outstanding examples of damage to public utilities such as the slip-circle failure which occurred in Coronation Drive, Auchenflower and the ramming of Centenary Bridge, Jindalee by a gravel barge. Although these particular damages are unlikely to happen again in a future flood the repair costs were lumped in with other flood damage to utilities on the basis that other types of freak failures would occur in future floods.

8.2 ANALYSIS OF DAMAGE TO UTILITIES

It appeared logical to relate many of the damages to public utilities to the area flooded. A relationship was derived between river stage and the area flooded in the Cities of Brisbane and Ipswich. This relationship is shown in Figure 14. The curve was derived by measuring the area flooded for the 2 m, 6 m and 10 m floods, less the area which is normally inundated by water below high-tide level. This curve was used as the basis for estimating flood damage to the types of utilities listed in Table 7. This table also shows the estimated damage to these public utilities in the January 1974 flood for the Cities of Brisbane and Ipswich.

In each case the flood damage in 1974 was obtained from the relevant authority and the damage for other gauge heights at the Brisbane City Gauge was estimated by simply assuming that the flood damage was directly proportional to the flooded area, that is:

$$\$DU_x = \$DU_{74}(A_x/A_{74}) \qquad (10)$$

where $\$DU_x$ = damage to utilities for a flood with a gauge height of x metres at the Brisbane City Gauge,

$\$DU_{74}$ = damage to utilities for the 1974 flood,

A_x = area flooded for a flood of x metres, and

A_{74} = area flooded in the January 1974 flood.

TABLE 7 - SUMMARY OF PRINCIPLE DAMAGE TO PUBLIC UTILITIES IN THE 1974 FLOOD

ITEM	MILLION $
1. Immediate repairs including clean-up work, temporary road diversions, spraying, burying dead animals, etc.	2.39
2. Water supply, sewerage and stormwater facilities including repairs to water mains, sewers, pumping stations, etc.	0.84
3. Public transport facilities including restoration of highways, railways, roads, bridges, ferry terminals, wharves, etc.	3.16
4. Electricity facilities including repairs to power generation and transmission installations, sub-stations, meters and switchboards	2.53
5. River transport including principally dredging of the lower river	0.90
6. Telephone communications including damage to telephones, switchboards, cables, conduits, etc.	2.36
7. Public amenities including parks, gardens, sport grounds, etc.	0.28
TOTAL	12.46

The simple assumption of equation (10) was not considered satisfactory for estimating flood damage to the installations of the Australian Post Office, the Brisbane City Council's electrical installations and costs incurred by the Department of Harbours and Marine for the dredging of the riverbed after the flood. For the Australian Post Office and the Electricity Department of the Brisbane City Council, estimated damages were provided by each authority for damage to their installations for

floods 2 m greater and 2 m less than the 1974 flood. The amount of dredging of the Brisbane River required after a flood depends on the magnitude of the flood. Following the 1974 flood about 800 000 m^3 of material deposited by the flood in the lower reaches of the river were removed by dredging. In contrast, in 1957 after a minor flood in the Brisbane River, about twice the quantity of sediment had to be removed. It appears that some scouring of the lower channel occurs at high floods and hence an inverse relationship between flood magnitude and dredging costs was assumed to exist.

Consideration was also given to the loss of revenue from public utilities due to floods and whether this should be included as an indirect loss. Kates (1965) in his U.S.A. studies suggested a figure of 10% of the direct damage. However, as some of the public utilities mentioned in this section do not operate on a profit basis, it was decided not to include an indirect loss.

9. OTHER FLOOD DAMAGE

9.1 FLOOD DAMAGE TO AGRICULTURE

Enquiries through the Queensland Department of Primary Industries indicated that while flood damage occurred to agriculture in locations below the proposed damsite at Wivenhoe on the Brisbane River it was not significant when compared to the flood damage in the Cities of Brisbane and Ipswich. This was illustrated by the fact that although loss of crops and pastures including soya beans, lucerne, irrigated pastures and pumpkins in the Brisbane River catchment area were estimated to be about $ 1.6 million, only a small fraction of this damage would have occurred downstream of the damsite at Wivenhoe. On the basis of this evidence, it was decided that agricultural damage could be neglected without causing significant errors in the total damage estimates.

9.2 FLOOD DAMAGE IN THE MORETON SHIRE

Unlike the Cities of Brisbane and Ipswich, the Shire of Moreton does not contain large urban areas on the Brisbane River. For this reason, losses in urban areas in this shire were relatively insignificant. Further, from information that was available from the Shire of Moreton it was also apparent that the flood losses were spread over a wide area and that the task of separating losses due to the Brisbane River and that due to the tributary flooding would have proved to be long and tedious. In view of these circumstances, it was decided that these damages would be omitted from the stage-damage curves.

Careful note should be made of the fact that for the 1974 flood an area of about 10 km^2 below Mt Crosby Weir and within the Shire of Moreton was flooded. At that time this area contained very few houses and hence in comparison with urban areas in the Cities of Brisbane and Ipswich these damages were negligible. The situation would be markedly different in the future should the present urban expansion include these portions of the Brisbane River flood plain.

9.3 FLOOD DAMAGE TO MOTOR VEHICLES AND BOATS

As mentioned under data sources in Section 5.2, the State Insurance Commissioner provided information on various types of flood claims made upon insurance companies operating in Queensland. Included in this information, which was reproduced in Table 2, were the data on motor vehicle and marine claims. These claims cover flood damage to cars, trucks, boats, launches and the like. In contrast with residences, most cars and boats are insured in comprehensive policies which include flood damage. Therefore the figures supplied in this category by the Insurance Commissioner are likely to be reasonably close to the actual 1974 flood damage.

To apply the 1974 flood damage in this category to floods of greater and smaller magnitude it was assumed that Figure 14 showing the relationship between the flooded area and river stage could be utilised in a similar manner to its application in Section 8 for the estimation of utilities damage. An arbitrary allowance was made for damage attributable to the Bremer River flood and to the metropolitan tributaries by reducing the derived figures by 30%.

9.4 FLOOD DAMAGE TO THE IPSWICH COAL MINES

Unlike most of the flood damage described in this report, damage to coal mines results in a stepped stage-damage curve for this component. This is because as soon as the flood rises above a mine entrance a sudden increase in the damage occurs. There being comparitively few mines these steps are not 'smoothed out' in the same way as damage to other items.

In the 1974 flood the question of losses to the Ipswich coal mines due to the flood are quite complex. Five mines were affected but only one mine was considered to be economical to reinstate and this at a cost of $ 1.5 million. The other affected mines have been abandoned since the flood. At one of the flooded mines the washing plant has been resited at another mine site above the 1974 flood level.

The most substantial loss due to the flood was brought about by the need to import coal from other areas. This additional cost was estimated to be $ 5 million and includes extra freight costs, handling charges and the higher price of coal at the pithead.

For floods higher than the 1974 flood no increase in mine damage would occur (for present conditions) because no additional mines are within the flood profile of the 10 m flood (at the Brisbane City Gauge). Floods

do not have to be very much lower than the 1974 flood for the mines to be unaffected. On this basis it was assumed that for floods less than 4 m no damage occurs while for a flood equal to the 1974 flood at 5.5 m the damage would be $ 6.5 million ($ 1.5 + $ 5) and for floods greater than this no increase in damage occurs.

The question of future flood damage to the Ipswich coal mines is complicated by many factors. One mine has been fitted with flood gates since the 1974 flood and should these be efficient then they will minimise future flood damage to this mine. On the other hand the opening of additional entrances to existing mines or the development of new mines or other works could increase damage. As with other components of the damage estimates it is assumed that the 1974 condition persists.

Further details of the damage to the Ipswich coal mines in 1974 are given in a letter from the Queensland Coal Board which is included in Appendix B.

10. INTANGIBLE FLOOD DAMAGE

10.1 INTRODUCTION

In addition to the direct and indirect tangible damage caused by a flood there are numerous forms of intangible damage. In the case of the Brisbane River flood intangible damage occurred in industrial, commercial and residential areas. Some information on these damages was available from reports provided by the Queensland Disaster Welfare Committee, SMEC questionnaires and from assessments made by the valuing firm engaged by SMEC.

In flood mitigation studies it has been usual practice to draw attention to the nature and extent of intangible damage without in any way attempting to quantify these losses monetarily. This practice was followed in this report in accordance with SMEC's proposal for the work. However, attention is drawn to the fact that other related studies, such as environmental impact statements for water resources projects, do attempt to quantify intangible aspects of the effects of engineering works. In a similar manner, it may be possible to make more decisive statements about the intangible effects of future flooding in the Brisbane River Valley.

10.2 COMMERCIAL AND INDUSTRIAL AREAS

A wide variety of reactions, criticisms and suggestions were received from the commercial and industrial areas affected by the 1974 flood. Many of these were related to the carrying out of flood works such as the construction of Wivenhoe Dam and the dredging of the tributaries to the Brisbane River. Quite frequently the comments showed a lack of knowledge of the flood problems; a situation which may be improved by the dissemination of more literature of a semi-technical nature on flooding in the Brisbane River and its tributaries. Some occupants of the flood plain believe they could have reduced their damages substantially had they been given better warning of the flood danger, or perhaps if they had been able to interpret the warnings more precisely.

It was also apparent from a number of questionnaires that some industrial and commercial buildings which were not flooded incurred losses because they were either cut-off by flood waters, and therefore effectively put out of operation, or were without electric power. Another important point was that 70% of the commercial and industrial respondents directly affected by the flood indicated that other business activities were affected by their temporary closure or reduction in output.

Opinion was divided on whether the 1974 foood would affect future expansion of industrial and commercial buildings in the flood affected areas. About 50% of the returns indicated that, due to the 1974 flooding, further expansion of existing premises was unlikely to take place.

Another damage which is partly intangible and partly an indirect damage is that due to the time required to regain full production. Table 8 shows information on the time required for various commercial and industrial organisations to regain full production.

TABLE 8 - TIME REQUIRED TO REGAIN FULL PRODUCTION

TIME REQUIRED Weeks	NUMBER OF RESPONDENTS	
	Commercial	Industrial
0 - 4	126	150
4 - 8	35	48
8 - 12	17	35
> 12	27	28
Discontinued	2	7
Total	207	268

In some cases this may have resulted in retrenchment of employees, loss of contracts and loss of business confidence.

After the 1974 flood, Quinnell (1974) in a report to the Queensland Disaster Welfare Committee surveyed 35 flood affected small businesses typical of those which could be found in any suburban shopping centre. The survey showed that, in addition to the financial problems caused by the flood, family emotional and health problems were found to be either aggravated or created by the circumstances following the flood. Further, the community which utilised these small businesses suffered a loss beyond the direct provision of goods and associated services because these businesses were an integral part of the network of relationships in the urban community.

10.3 RESIDENTIAL AREAS

In answers to the question 'what affect did the flood have on your family', respondents to the Indooroopilly Flood Survey (see Davey et al, 1974) described the effect using words like shock, confusion, bewilderment, depression, disbelief, panic, fear, anxiety, etc. As not many of the thousands of residents affected by the January 1974 flood believed that they could be inundated, the reaction to the sudden devastation was not surprising.

Besides the immediate emotional problems caused by the flood devastation, physical injuries were inflicted on persons during the evacuation and 12 people lost their lives in the Brisbane area. In the weeks and months following the flood, studies of residents affected showed many cases of shock and mental illness which were attributed to the flood.

Other intangible damage may become apparent in the coming years. Typically, in the flood affected areas, land values become depressed unless flood mitigation works are carried out. The expectation of further flooding causes a feeling of insecurity among the residents and discourages property improvements. The long-term effect may be to produce areas with lower material living standards and a depressed social atmosphere.

A detailed statistical study after the Bristol (England) floods of 1968 by Bennet (1970) provided some interesting facts on the effect of a flood disaster on a residential community. Bennet states:

> 'An investigation into the health of people in Bristol flooded in July 1968 was made by means of a controlled survey and a study of mortality rates. There was a 50 per cent increase in the number of deaths among those whose homes had been flooded, with a conspicuous rise in deaths from cancer.
>
> Surgery attendances rose by 53 per cent, referrals to hospital and hospital admissions more than doubled. In all respects the men appeared less well able to cope with the experience of the disaster than the women.'

There is no reason to believe why these findings would not apply equally to the urban areas along the Brisbane River which were flooded in 1974.

11. ESTIMATED STAGE-DAMAGE CURVES

11.1 INTRODUCTION

To compile stage-damage curves for Brisbane River flooding, it was necessary to consider the flood profiles for five floods. These floods were the 2 m, 4 m, 6 m, 8 m, and 9 m floods at the Brisbane City Gauge. The adopted flood profiles (see Section 2.6) were defined by the Brisbane City Council and entered into the flood-damage computer program (see Section 4.3) to calculate damage to residences, including clean-up costs, commercial buildings, industrial buildings and miscellaneous buildings. Flood damage to public utilities, motor vehicles and boats and the Ipswich coal mines were calculated separately.

11.2 FLOOD DAMAGE TO BUILDINGS

The details of the methods employed to calculate the flood damage for commercial and industrial, residential and miscellaneous buildings have been described in Section 5, 6 and 7 respectively.

Table 9 summarises the number of buildings classified as commercial, industrial, residential and miscellaneous affected by floods of 2 m, 4 m, 6 m, 8 m, 9 m and 10 m at the Brisbane City Gauge while Table 10 summarises the cost of the direct, indirect and total damages for each of these groups of buildings. Full details of the computer printouts of the damage for these floods are available upon request from SMEC. These details include a breakdown of the damage on a postcode basis and on a river kilometre basis from the mouth of the Brisbane River.

TABLE 9 - NUMBER OF BUILDINGS AFFECTED BY VARIOUS HEIGHTS OF FLOODING

FLOOD HEIGHT m	COMMERCIAL BUILDINGS	INDUSTRIAL BUILDINGS	NUMBER OF RESIDENCES	MISCELLANEOUS BUILDINGS	TOTAL
2	165	64	208	32	469
4	708	861	4 941	206	6 716
6	1 230	1 925	11 614	515	15 284
8	1 664	2 615	18 461	786	23 526
9	1 883	2 879	21 403	889	27 054
10	(2 180)	(3 300)	(24 500)	(1 020)	(31 000)

NOTE: Figures in brackets obtained by extrapolation

11.3 OTHER FLOOD DAMAGE

Sections 8 and 9 describe the procedures used to calculate the flood damage to public utilities, motor vehicles and boats, and the Ipswich coal mines. These three items were estimated by hand methods. The results for floods of 2 m, 4 m, 6 m, 8 m, 9 m and 10 m are also summarised in Table 10.

11.4 STAGE-DAMAGE CURVE

Figure 15(a) shows the various components of the stage-damage curve for direct damage. These have been plotted from the information contained in Table 10 and include:

- commercial buildings
- industrial buildings
- residential buildings
- residential clean-up
- miscellaneous buildings
- public utilities
- motor vehicles and boats
- Ipswich coal mines

Figure 15(b) shows the stage-damage curves for direct, indirect and total damages. This curve was derived by the addition of 37%, 45% and 15% respectively to the direct damage to the commercial, industrial and residential components of Figure 15(a). Other components of the direct damage curve were assumed to have zero indirect damage as indicated in Table 10. Again it is stressed that the January 1974 development of the flood plain and costs have been assumed throughout the study.

11.5 PREVIOUS ESTIMATE OF THE STAGE-DAMAGE CURVE

In the flood investigation study for the proposed dam at Wivenhoe, the Co-ordinator General's Department (1971) estimated a stage-damage curve for the Brisbane River. Its estimate is shown on Figure 15(b). The earlier stage-damage curve gives very much smaller damage for any flood height than that estimated by SMEC. The main reason for the large difference is attributed to the fact that SMEC had very much better data on which to make its estimate than were available for the study by the Co-ordinator General's Department. This was, of course, due to the fact

8.15 65.5 165 278 339 408

Flood Height m	Commercial	Industrial	Residential	Residential Clean-Up	Miscellaneous	Public Utilities	Motor Vehicles and Boats	Ipswich Coal Mines	Total
	COMPONENTS OF DIRECT DAMAGE (million $)								
2	2.91	0.74	0.15	(0.00)	0.56	2.98	0.81	0.00	8.15
4	16.4	18.8	9.48	1.39	4.35	8.45	3.75	4.30	~~66.9~~
6	28.2	65.1	33.9	(4.80)	13.1	14.1	6.80	6.50	~~173~~
8	39.9	109	69.3	9.79	23.1	20.2	10.2	6.50	~~288~~
9	46.7	129	93.3	12.8	27.9	24.2	11.8	6.50	~~352~~
10	(55.2)	(149)	(123)	(18.0)	(33.2)	27.6	13.3	6.50	426
	COMPONENTS OF INDIRECT DAMAGE (million $)								
2	1.08	0.33	0.02	0	0	0	0	0	1.43
4	6.06	8.46	1.42	0	0	0	0	0	15.9
6	10.4	29.3	5.09	0	0	0	0	0	44.8
8	14.7	49.0	10.4	0	0	0	0	0	74.1
9	17.3	58.2	14.0	0	0	0	0	0	89.5
10	(20.4)	(67.0)	(18.5)	0	0	0	0	0	106
	TOTAL DAMAGE (million s)								
2	3.99	1.07	0.17	(0.00)	0.56	2.98	0.81	0.00	9.58
4	22.5	27.3	10.9	1.39	4.35	8.45	3.75	4.30	82.9
6	38.6	94.4	39.0	(4.80)	13.1	14.1	6.80	6.50	217
8	54.5	158	79.7	9.79	23.1	20.2	10.2	6.50	362
9	64.0	187	107	12.8	27.9	24.2	11.8	6.60	441
10	(75.6)	(216)	(141)	(18.0)	(33.2)	27.6	13.3	6.50	531

NOTE: Figures in brackets obtained either by interpolation or extrapolation.

that the Co-ordinator General's study was prior to the 1974 flood and SMEC's after that event. Another factor which helped SMEC was that a reasonable amount of money and time was allowed for the study.

11.6 ACCURACY OF FLOOD DAMAGE ESTIMATES

To nominate an accuracy for the stage-damage curves is difficult. Care has been taken to minimise any bias in the results of estimating the various components of the flood damage. On this basis it would appear that the accuracy of the direct damage estimate for any given flood height should be within ± 10% of the correct answer. However, flood damage studies tend to overlook certain direct damage through lack of knowledge of all possible sources of information. It was very apparent in this study that additional flood damage items kept appearing, often very unexpectedly, during the progress of the study. Overlooking flood damage items of course leads to an under-estimation of flood damage as also does the deliberate omission of relatively small items. The consideration of or inclusion of the following items would have increased the damage estimates:

- an allowance for basement flooding especially in the Brisbane City
- an allowance for uninsured motor cars, boats, etc.
- damage in the Shire of Moreton
- buildings (especially commercial and industrial) which were not flooded but suffered because of their dependency on flooded buildings
- the extraction of data from aerial photos which were a few years old instead of up to date photos
- an allowance for flooding of the first floor (in addition to the ground floor) level

Further, the finally adopted method of damage estimation for commercial and industrial buildings described in Section 5 gives lower results than an alternative less conventional approach which was also described in the same section. For all these reasons, it appears most unlikely that the estimates of direct flood damage should be lower than that indicated by the stage-damage curves. The error is, therefore, believed to be one of under-estimation and the true values could be 5% to 10% greater than those indicated by the curves.

The situation is somewhat different for the stage-damage curve which includes both direct and indirect damage. For this curve the accuracy is complicated by the somewhat arbitrary adoption of the percentages of 37%, 45% and 15% for the calculation of indirect damage for commercial, industrial and residential buildings respectively. However, it is again believed that any error in estimation would be an underestimate of the true damage.

In the consideration of the accuracy of the stage-damage curves it is again stressed that all the estimates of damage have been based on the assumption that a similar situation will prevail throughout the flooded community as that which existed in January 1974. It is believed that only by persistent and prolonged effort by both the authorities and the flood plain occupants, throughout every future cyclone season, could the potential damage for any future flood be reduced substantially below that indicated by the curve.

It is again pointed out that no allowance has been made for commercial, industrial and residential growth on the flood plain. The growth rate is determined largely by future government policies which could mean:

- a continuation of the existing growth rate
- a reduction in the growth rate due to flood plain zoning
- a negative growth rate due to the imposition of flood insurance on the flood plain occupants, or
- a reduction in the growth rate due to government decentralisation policies.

11.7 JANUARY 1974 FLOOD DAMAGE

As indicated on Figures 15(b) the 1974 flood damage for the Brisbane River is estimated to be $ 142 million for direct damage and $ 178 million for the total damage (direct + indirect). These estimates are for Brisbane River flooding only, that is, a flood at the Brisbane City Gauge 5.45 m high with level backwaters up all tributaries including the Bremer River. The actual total 1974 flood damage in the Cities of Brisbane and Ipswich would have been much greater than these two figures because of the severe flooding on metropolitan tributaries in both Brisbane and Ipswich and the damage caused by the Bremer River in the City of Ipswich.

The estimated number of commercial, industrial and residential buildings affected by the 1974 flood was 13 000. Again this figure is for flooding from the Brisbane River only.

11.8 TRANSFERABILITY OF RESULTS

In the sections of this report dealing with flood damage to commercial, industrial and residential buildings comments have been made in respect of the confidence with which the results of this study could be transferred to other Australian cities with flood problems. As a general comment it is believed that the data, estimation procedures and results can be transferred to other Australian cities. Obvious differences between the Brisbane situation and the city to which the transfer is taking place should of course be taken into account. These differences could relate to either the types of buildings in the city or the nature of the river flood.

A summary of the results for this study on an area basis is given in Table 11. These results are based on Figure 14 which gives the flooded areas for the Cities of Brisbane and Ipswich. Two points are made concerning the results given in the table. Firstly the costs per unit area increase with flood stage. This is because the average depth of flooding through buildings increases with increasing flood magnitude. The second point to note is that the total area subject to flooding is not fully urbanised. This is apparent from the flood maps (Figures 2, 3 and 4). The damage per unit area would be very much higher if the whole of the flooded area was urbanised.

TABLE 11 – SUMMARY OF FLOOD DAMAGE PER UNIT AREA

FLOOD HEIGHT m	AREA km^2	DAMAGE PER UNIT AREA (million $/$km^2$)	
		Direct	Direct + Indirect
2	12	0.65	0.77
4	57	1.17	1.45
6	102	1.70	2.13
8	153	1.88	2.37
9	179	1.97	2.46
10	205	2.08	2.59

12. ACKNOWLEDGMENTS

To carry out a study of this nature obviously required successful co-operation with all the local, State and Australian Government authorities concerned with the flood which occurred in January 1974. Without exception, the many authorities which were called upon to supply basic data, maps, information and advice did so willingly and their help is very gratefully acknowledged.

Information on flood damage was sought from many residents and commercial and industrial organisations affected by the 1974 flood. Their co-operation in supplying much of the essential basic data are also very gratefully acknowledged.

Although it is impossible to make specific mention of all the organisations which assisted, SMEC was particularly appreciative of:

- the assistance given by the valuers Alex Overett Pty Ltd
- the assistance of the Brisbane City Council and the Survey Office of the Queensland Department of Lands for the supply of plans, flood profiles and frequency data
- the general guidance and helpful contacts made by the Queensland Co-ordinator General's Department
- the helpful discussions held during the course of the study with the Cities Commission

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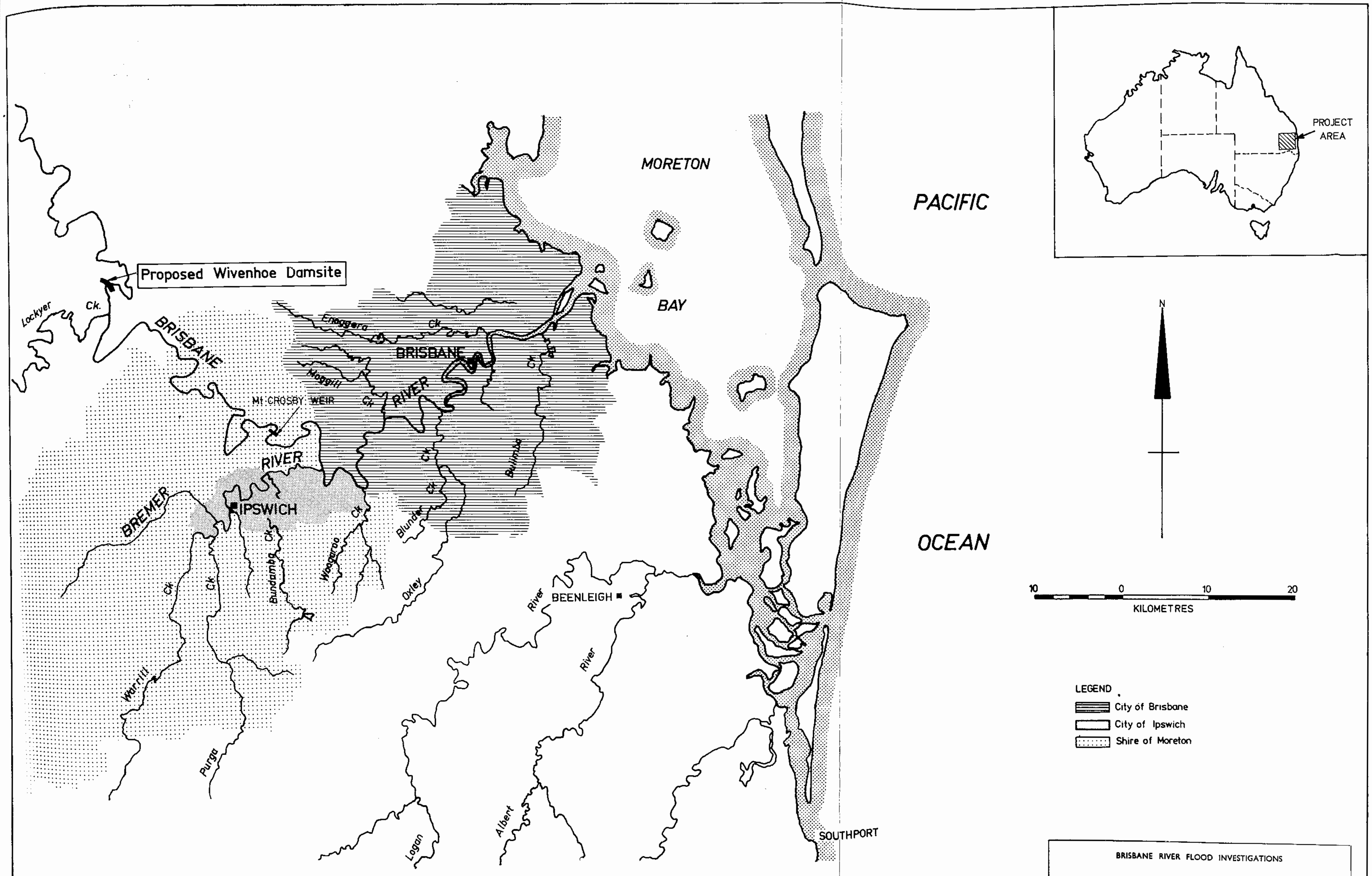
FIGURES

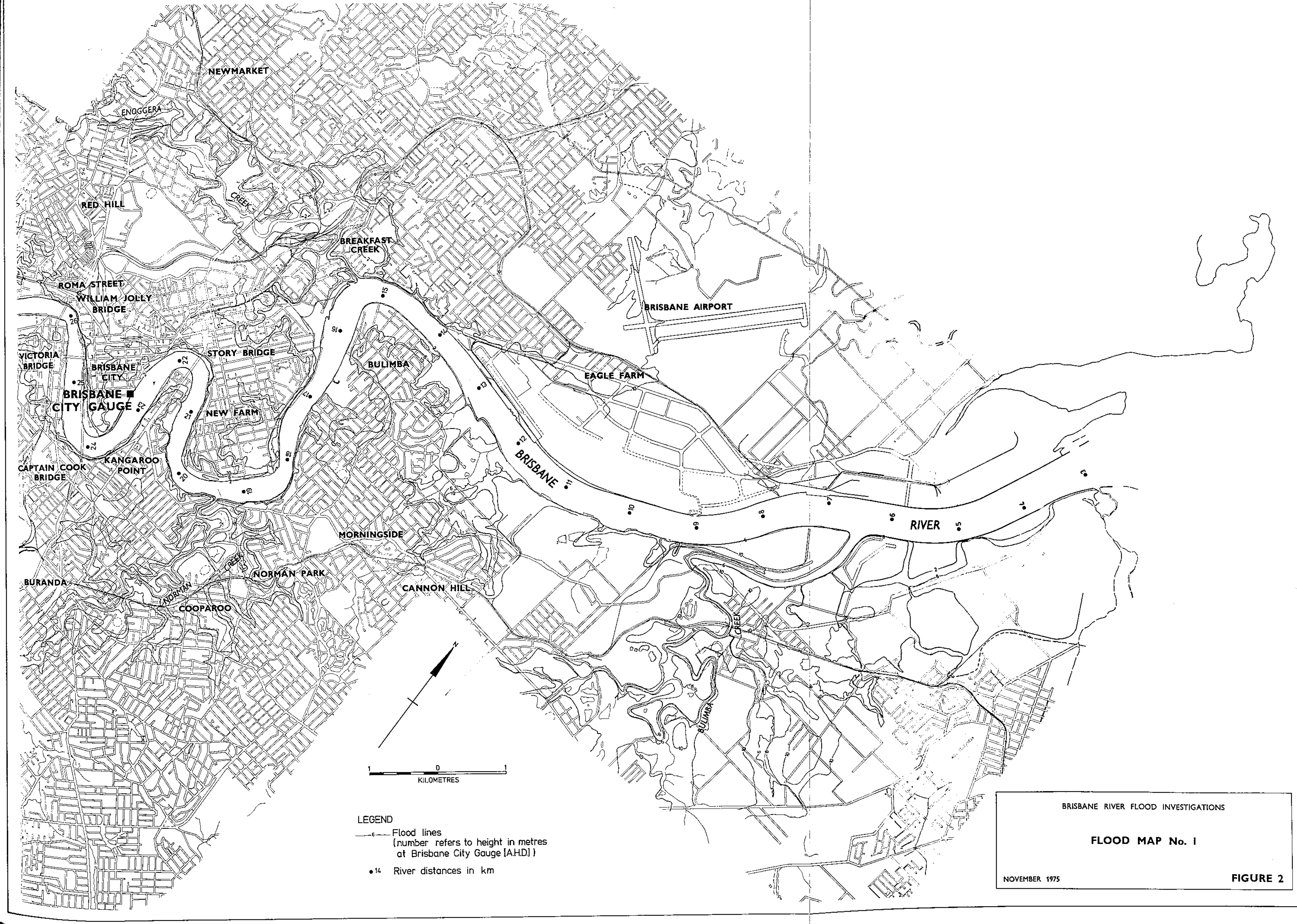


BRISBANE RIVER FLOOD INVESTIGATIONS

LOCALITY PLAN

NOVEMBER 1975

FIGURE 1



LEGEND

Flood lines (number refers to height in metres at Brisbane City Gauge [A.H.D.])

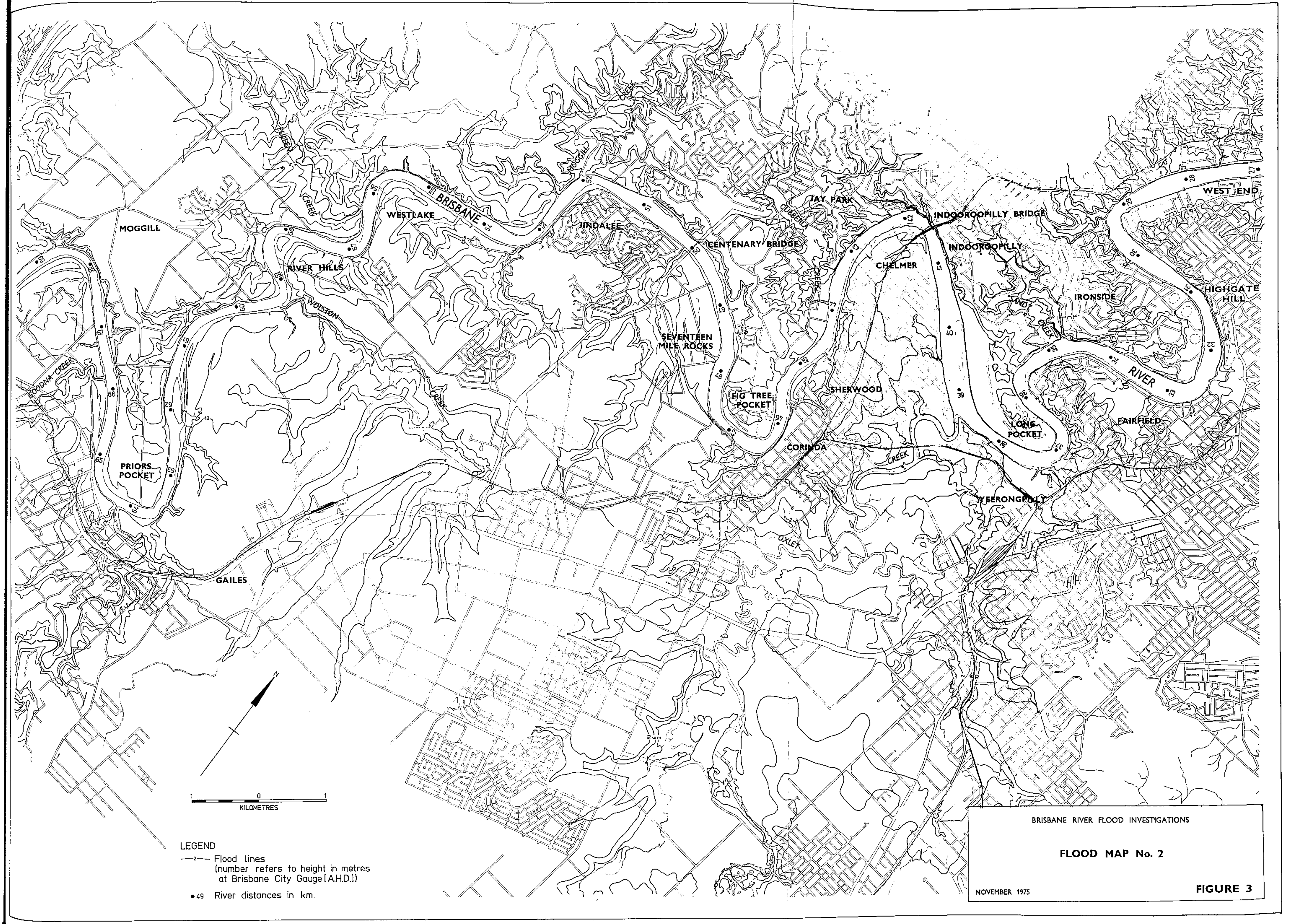
River distances in km

BRISBANE RIVER FLOOD INVESTIGATIONS

FLOOD MAP No. 1

NOVEMBER 1975

FIGURE 2

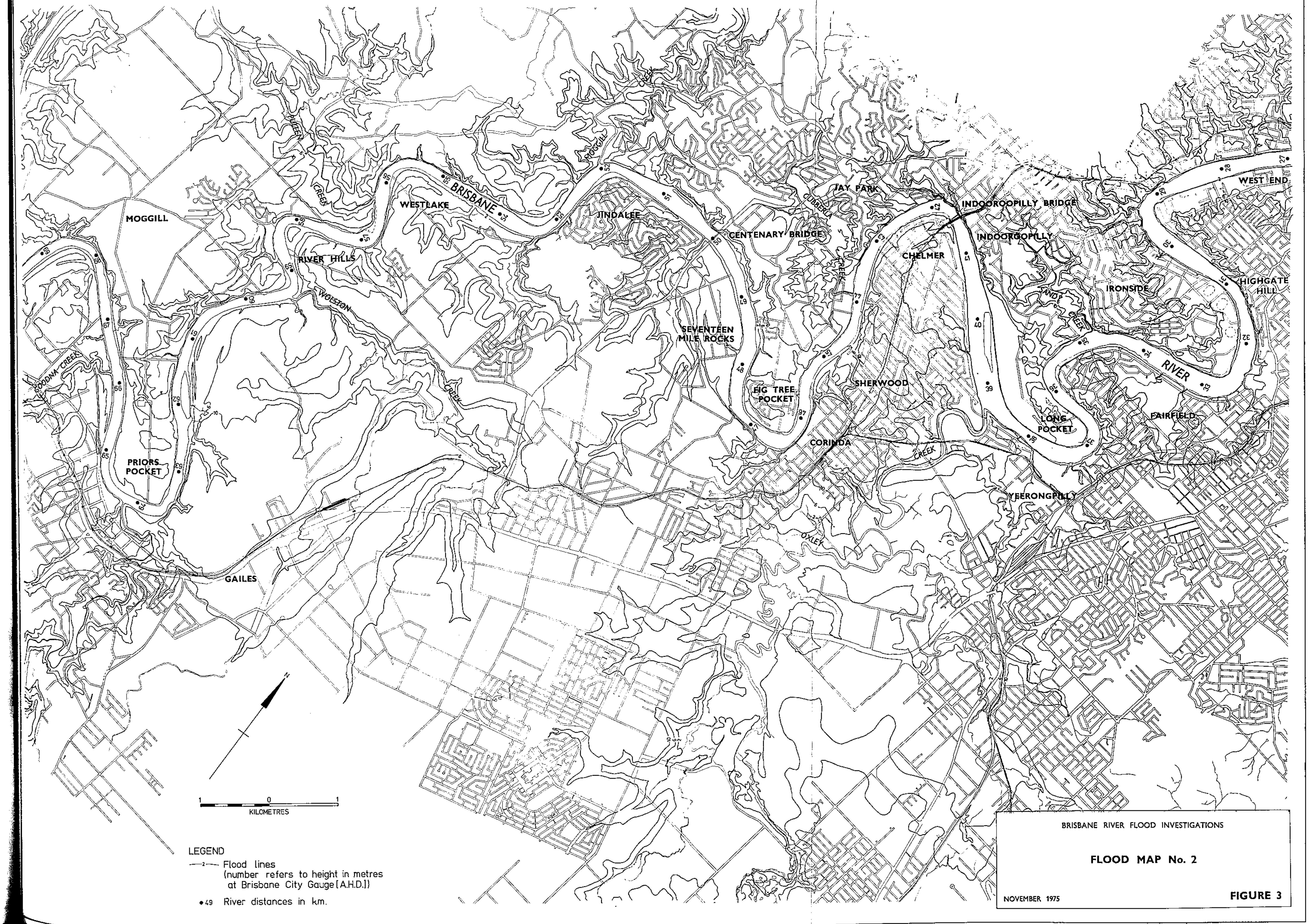


BRISBANE RIVER FLOOD INVESTIGATIONS

FLOOD MAP No. 2

NOVEMBER 1975

FIGURE 3

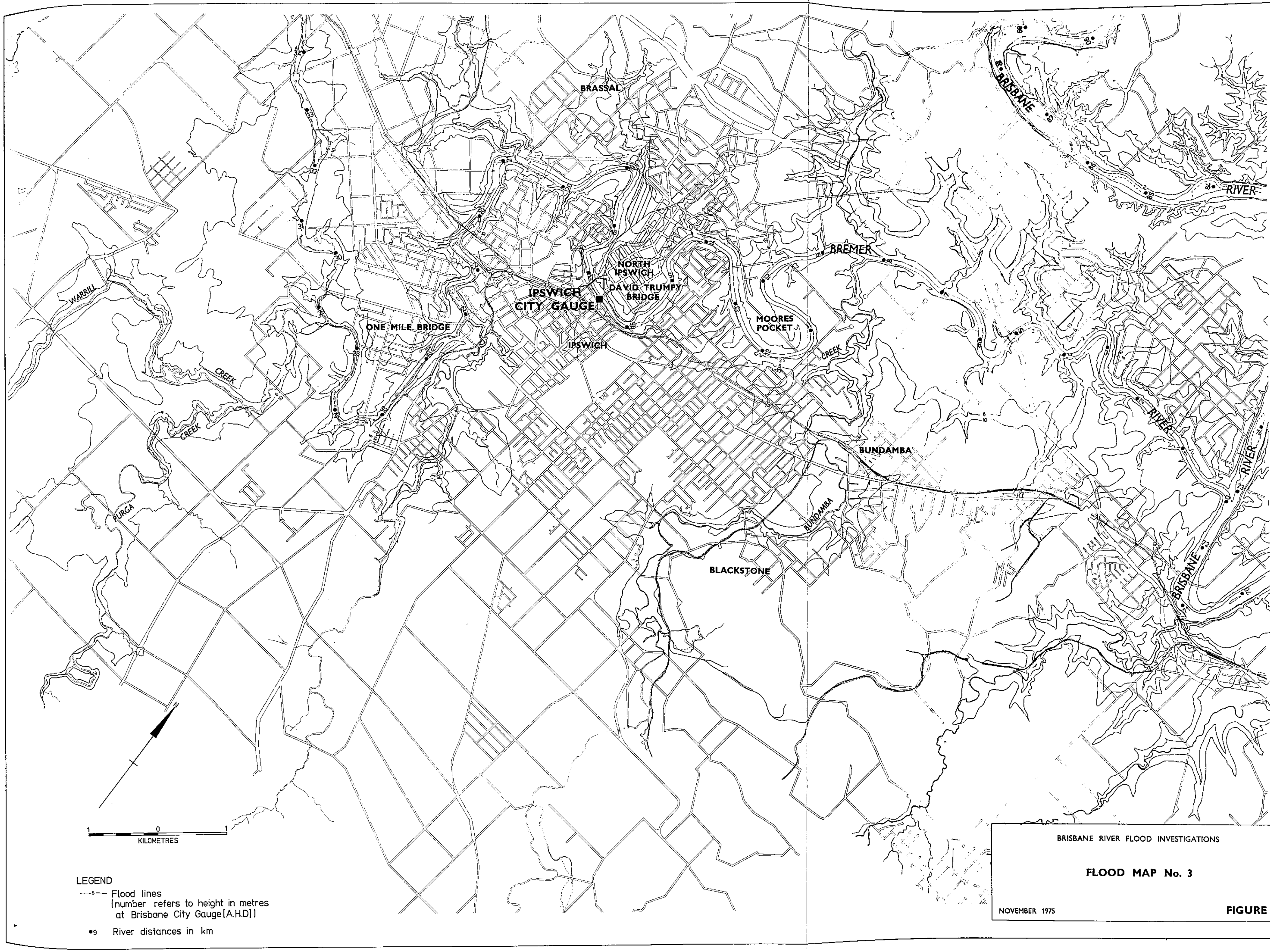


BRISBANE RIVER FLOOD INVESTIGATIONS

FLOOD MAP No. 2

NOVEMBER 1975

FIGURE 3

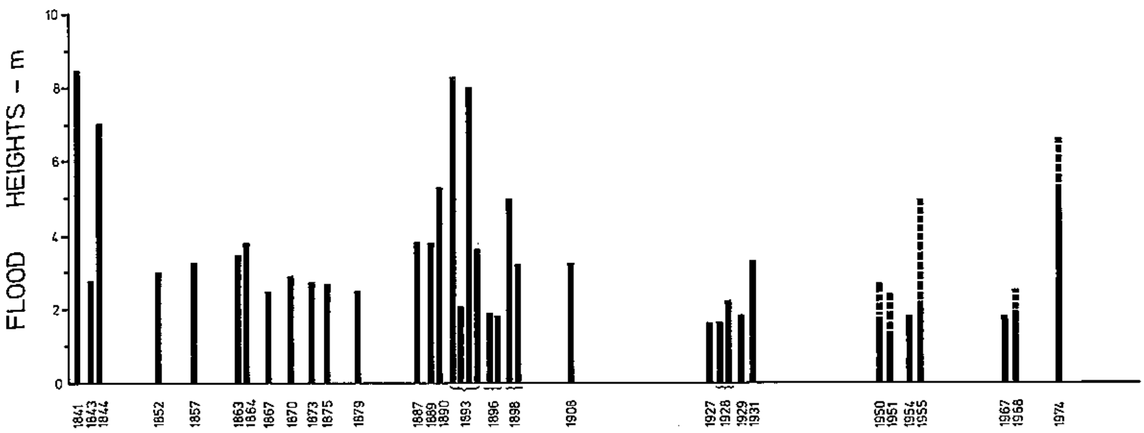


BRISBANE RIVER FLOOD INVESTIGATIONS

FLOOD MAP No. 3

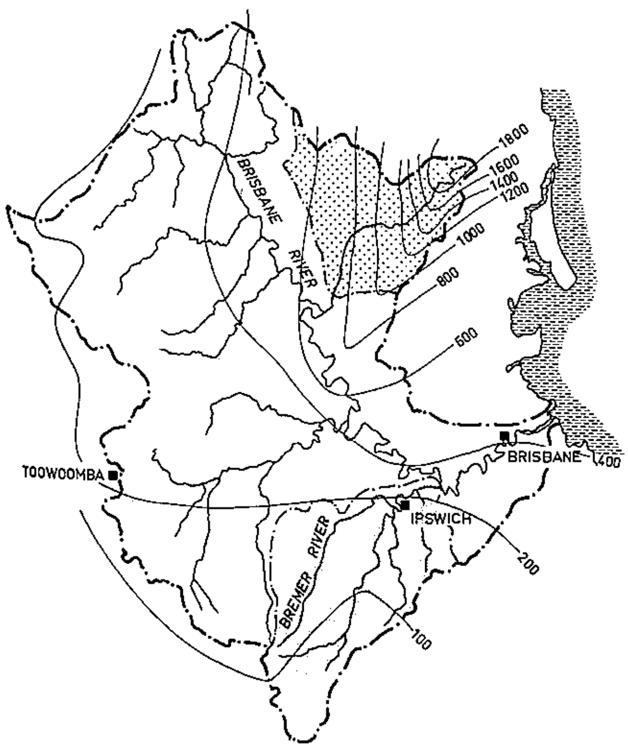
NOVEMBER 1975

FIGURE 4

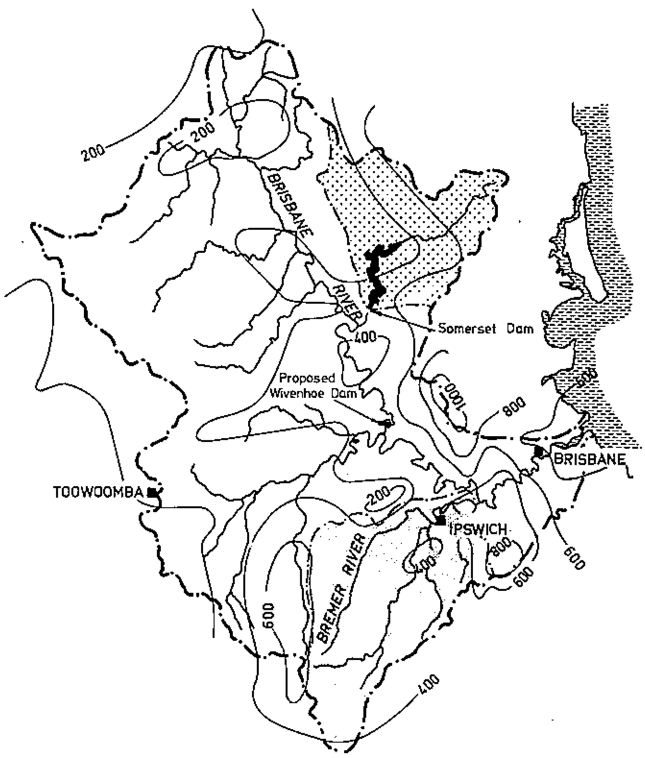


a. BRISBANE RIVER FLOOD HEIGHTS
1841 – 1974

NOTES
1. Flood heights are for the Brisbane city gauge
2. Estimated height without flood mitigating effect of Somerset Damshown thus ----



b. RAINFALL ISOHYETS FOR 1893 FLOOD



c. RAINFALL ISOHYETS FOR 1974 FLOOD

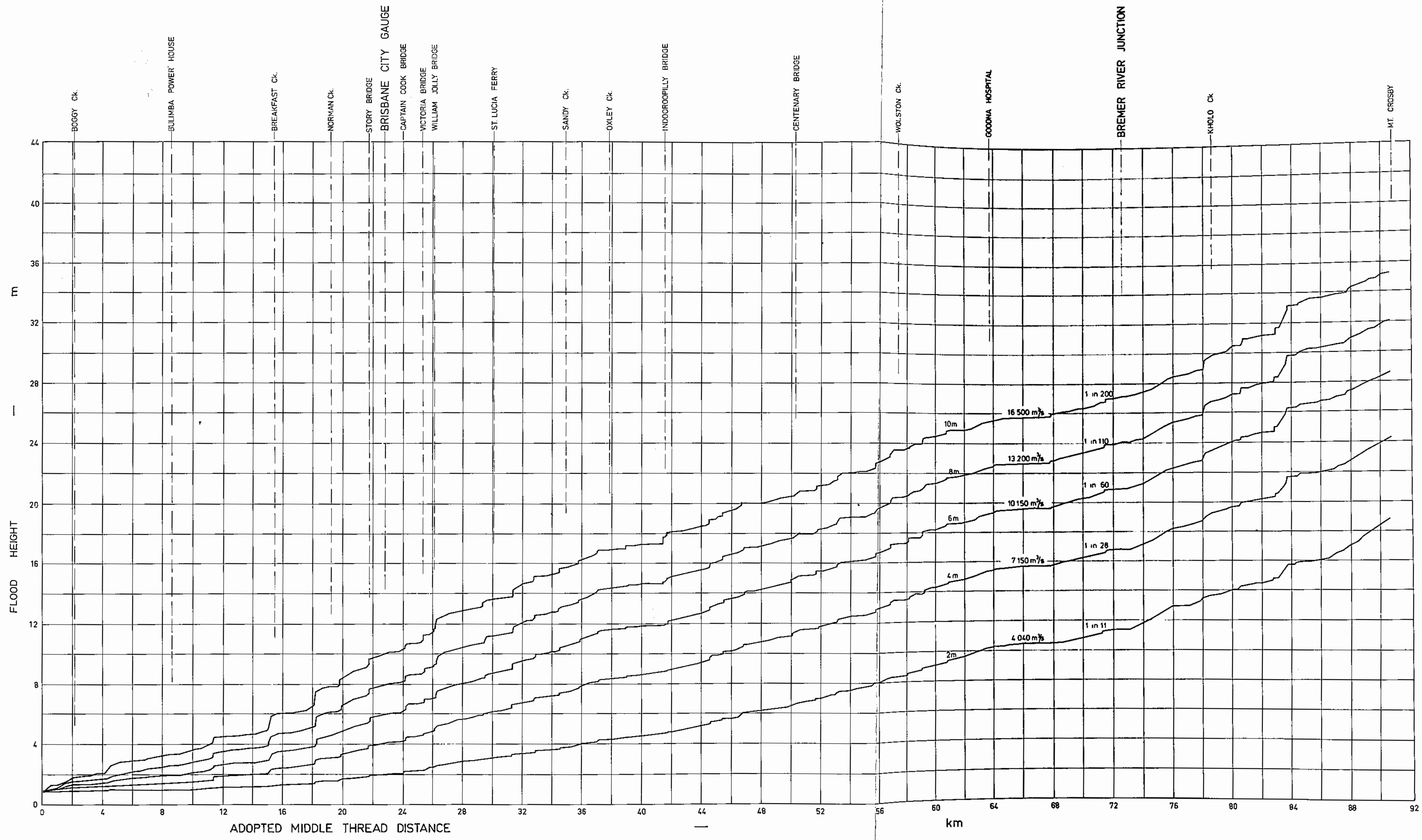
LEGEND
Somerset Dam catchment (1330 km²)
Bremer River catchment (2020 km²)
Brisbane River catchment (13 600 km²)
-200— Rainfall isohyets in mm

10 5 0 10 20 30 40 50
KILOMETRES

BRISBANE RIVER FLOOD INVESTIGATIONS

FLOOD HEIGHTS AND ISOHYETAL MAPS

NOVEMBER 1975 **FIGURE 5**

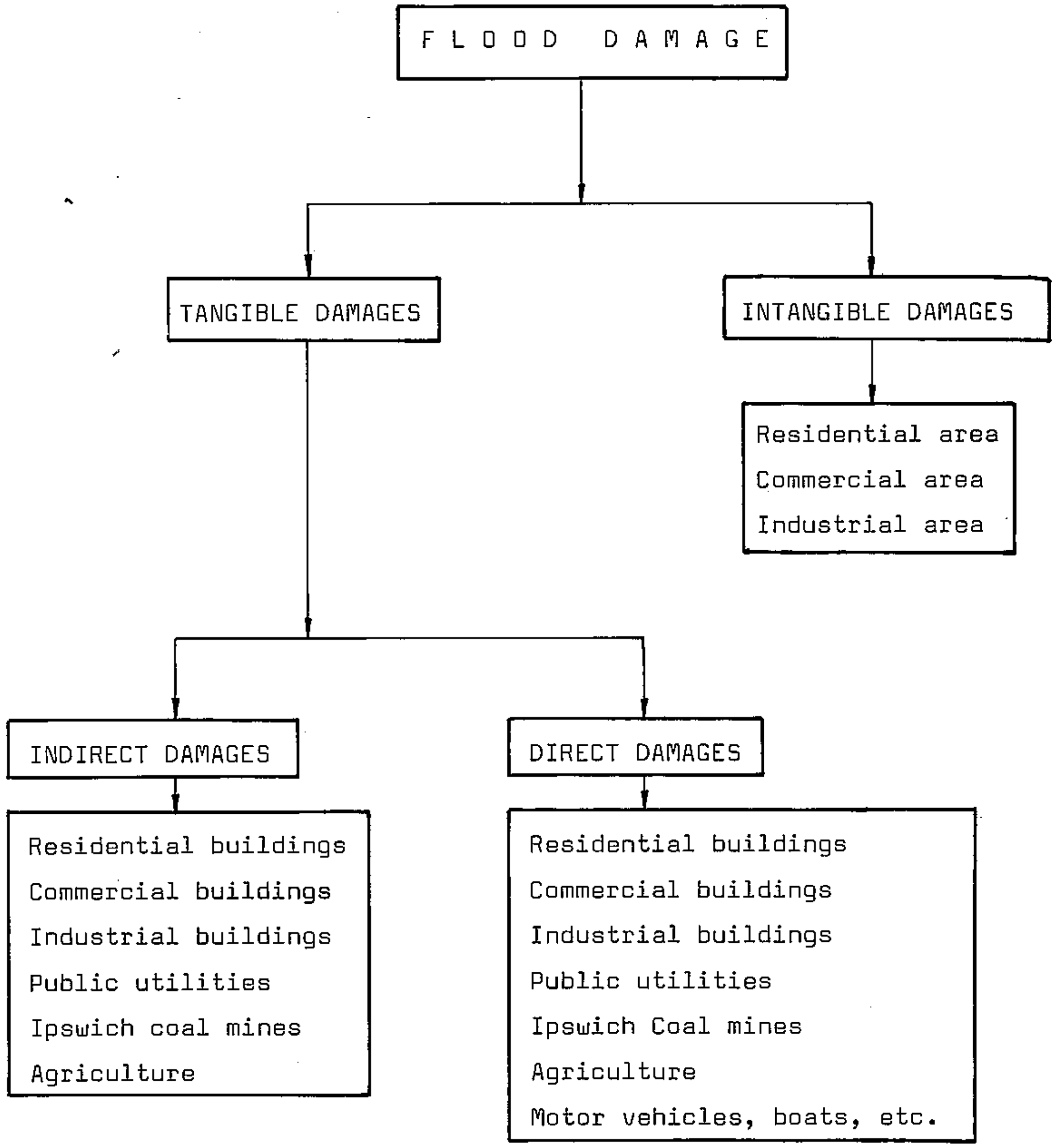


BRISBANE RIVER FLOOD INVESTIGATIONS

BRISBANE RIVER FLOOD PROFILES

NOVEMBER 1975

FIGURE 6

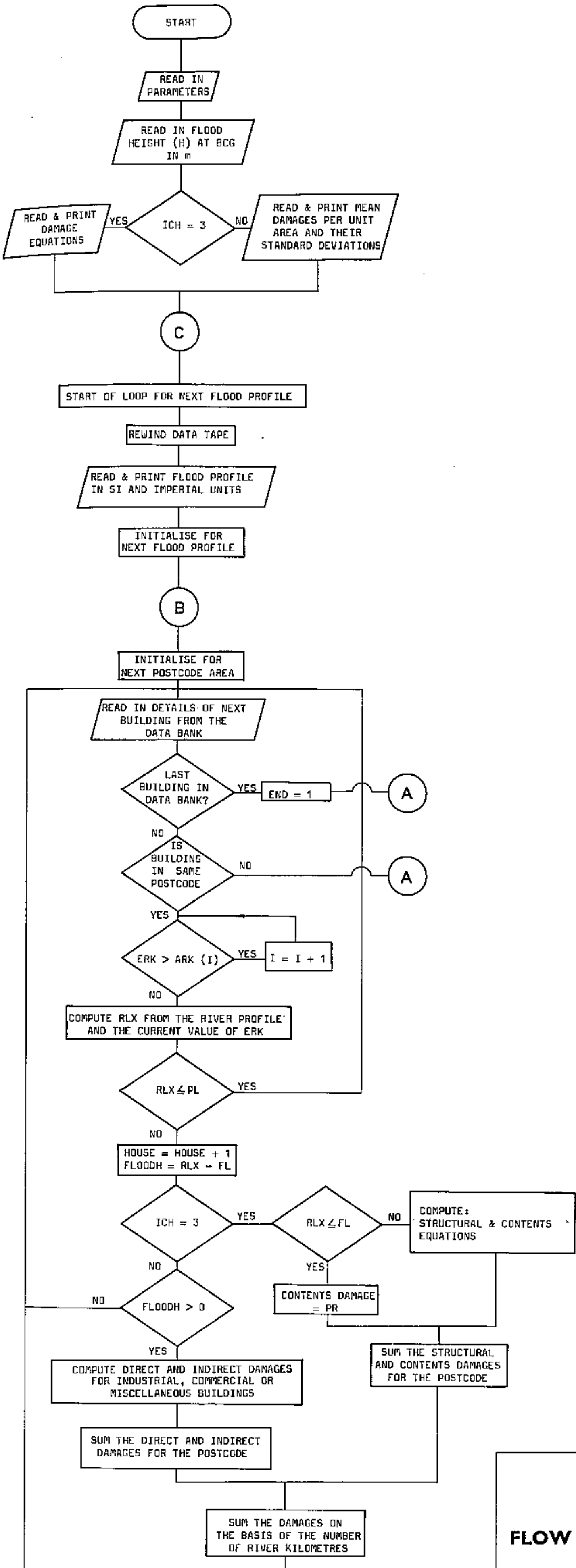


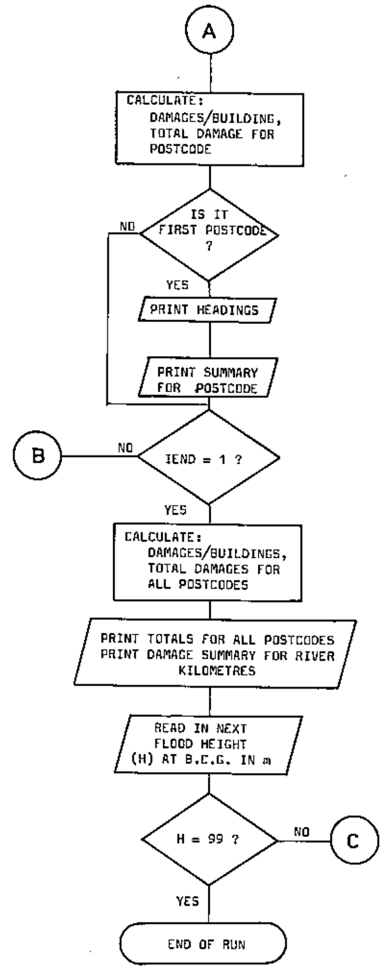
BRISBANE RIVER FLOOD INVESTIGATIONS

FLOOD DAMAGE CATEGORIES

NOVEMBER 1975

FIGURE 7





FLOW CHART SYMBOLS

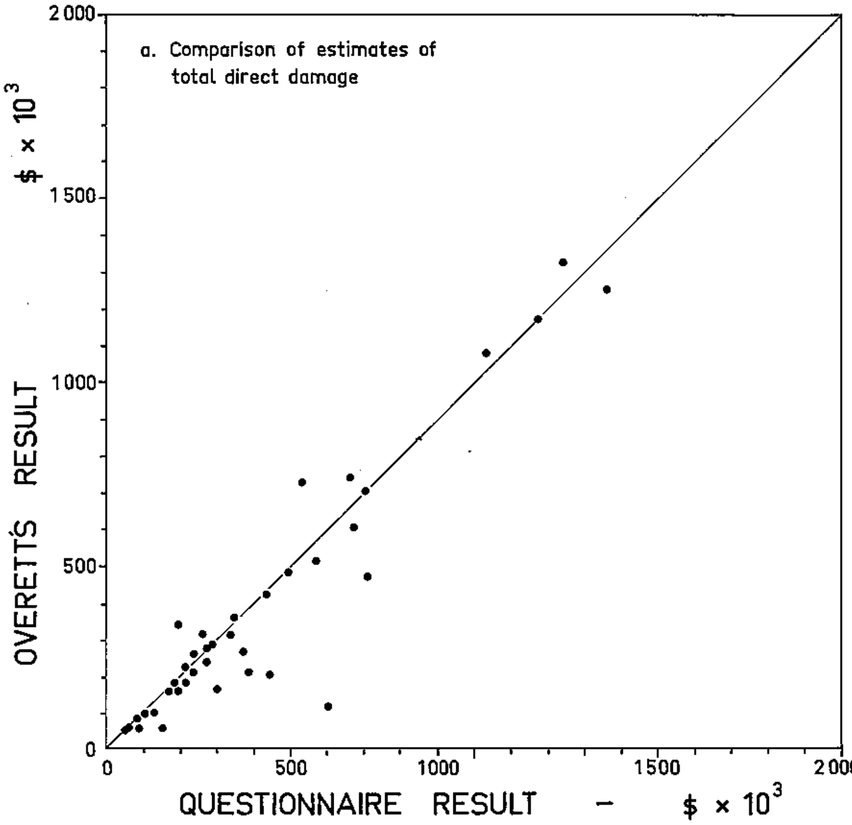
ARK	approximate river kilometres from the river mouth
ERK	the value of ARK in kilometres for the building currently under examination for damages
FL	floor level for building under examination
FLOODH	the height of flooding over the floor level
H	flood height at the Brisbane City Gauge in m (equals 99 for the end of a run)
HOUSE	counter for buildings affected by the flood
I	a counter
ICH	an indicator for building type (equal to 3 for residential damage)
IEND	an indicator
PL	property level for building at ERK kilometres
PR	damage to property where flooding does not exceed floor level
RLX	the flood level for a particular flood profile at ERK kilometres

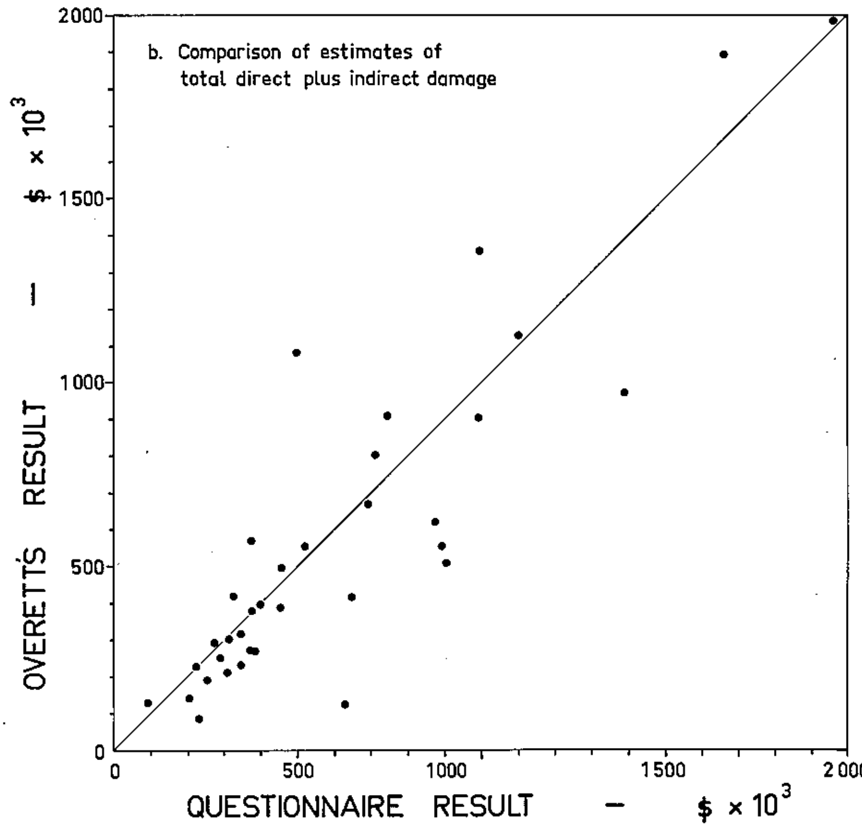
BRISBANE RIVER FLOOD INVESTIGATIONS

FLOW CHART FOR FLOOD DAMAGE PROGRAM

NOVEMBER 1975

FIGURE 8



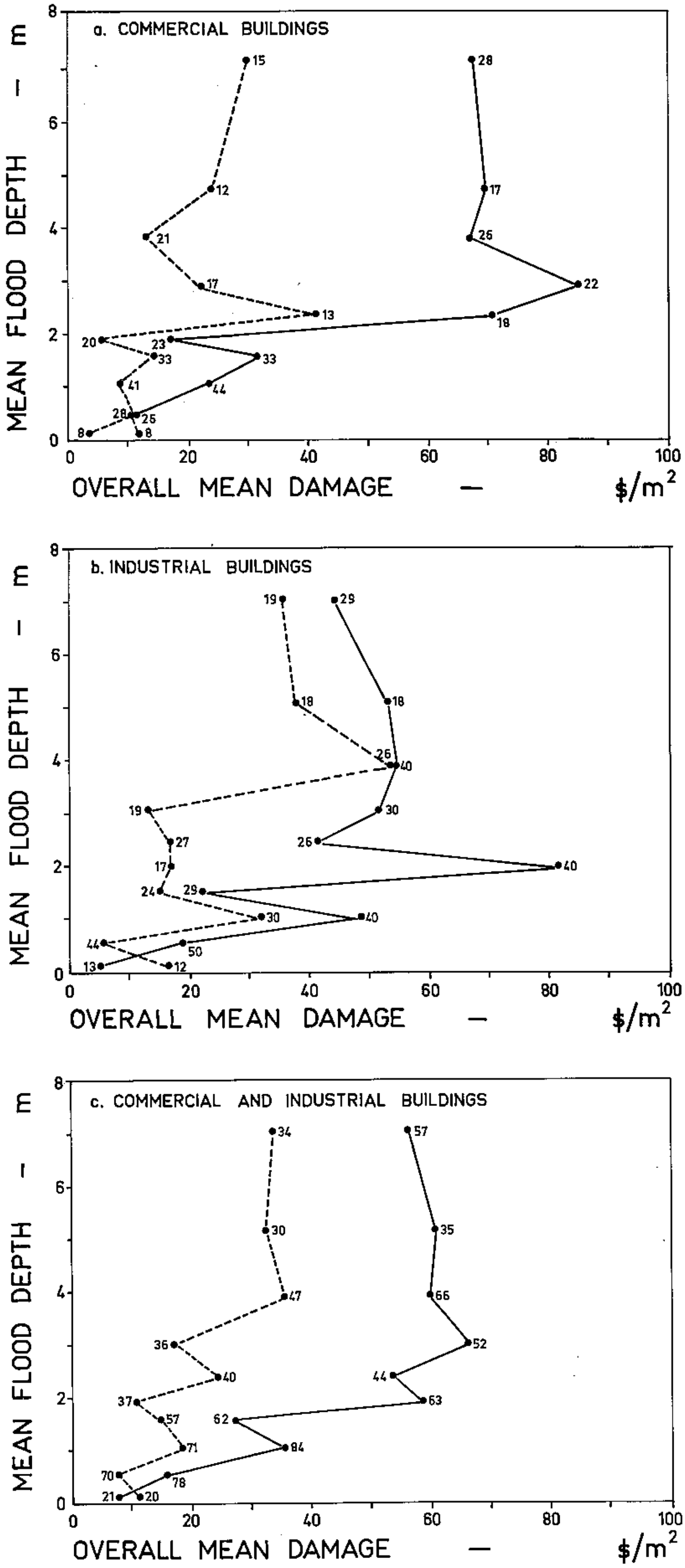


BRISBANE RIVER FLOOD INVESTIGATIONS

OVERETT'S RESULTS
VERSUS QUESTIONNAIRE RESULTS

NOVEMBER 1975

FIGURE 9



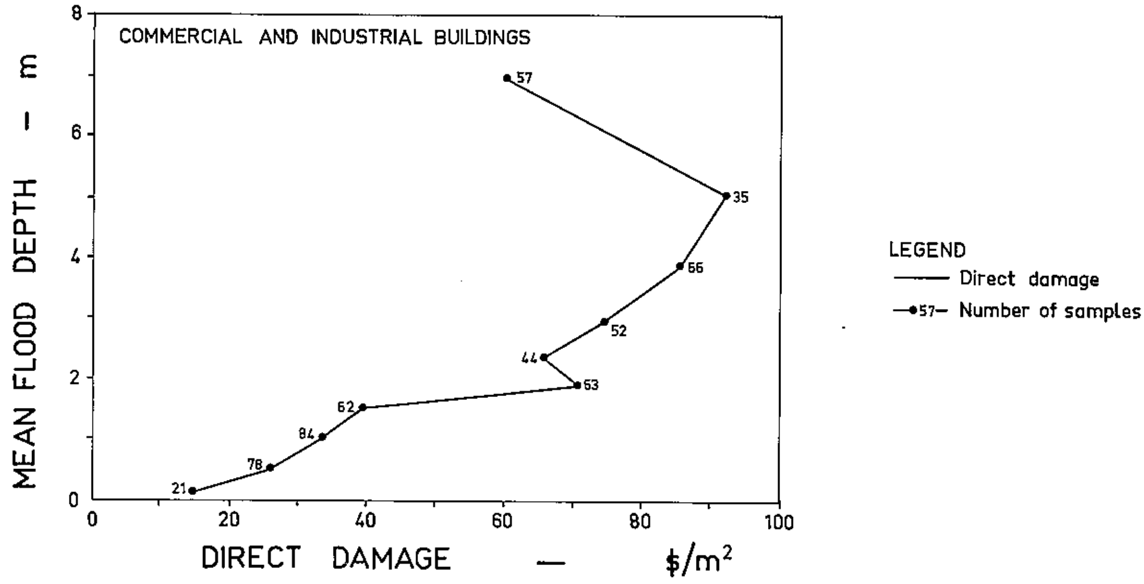
LEGEND
—— Direct damage
- - - - - Indirect damage
-57●— Number of samples

BRISBANE RIVER FLOOD INVESTIGATIONS

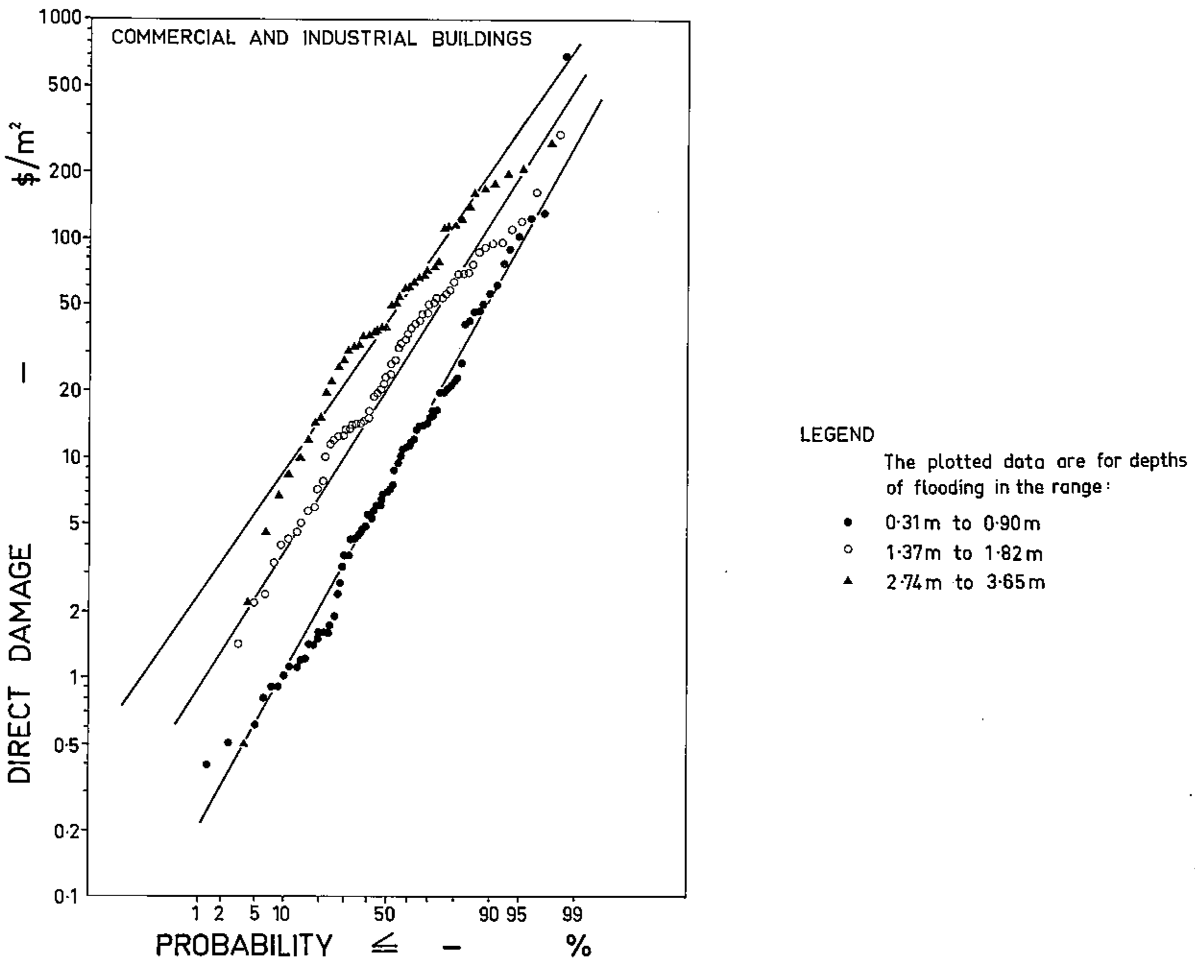
COMMERCIAL AND INDUSTRIAL DAMAGE
(METHOD I)

NOVEMBER 1975

FIGURE 10



a. MEAN OF INDIVIDUAL DAMAGES



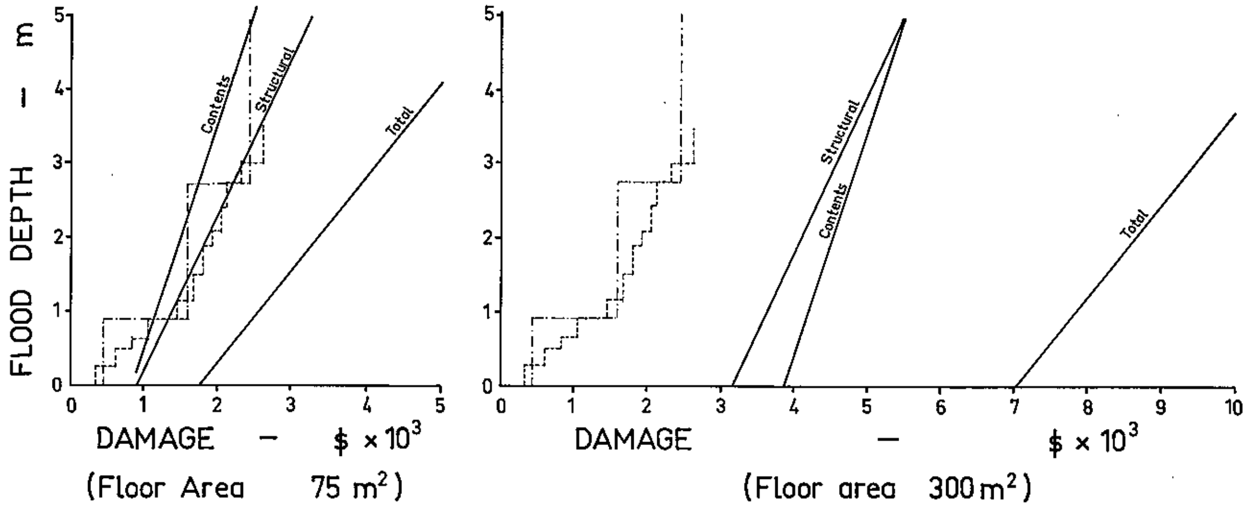
b. FREQUENCY DISTRIBUTION OF DIRECT DAMAGES

BRISBANE RIVER FLOOD INVESTIGATIONS

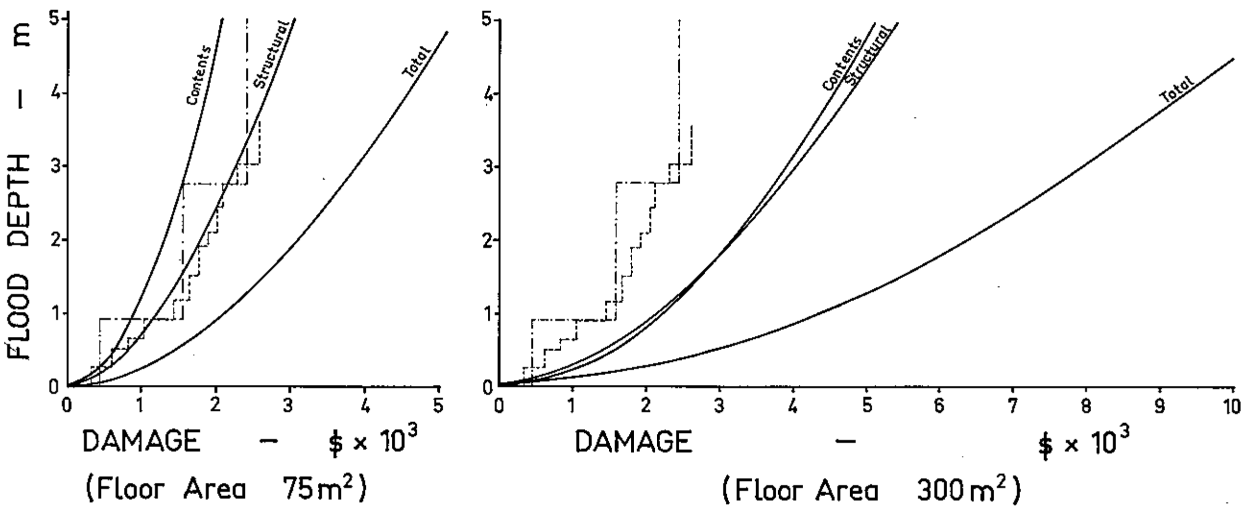
COMMERCIAL AND INDUSTRIAL DAMAGE (METHOD 2)

NOVEMBER 1975

FIGURE 11



a. RESULTS FROM ARITHMETIC REGRESSION



b. RESULTS FROM LOGARITHMIC REGRESSION

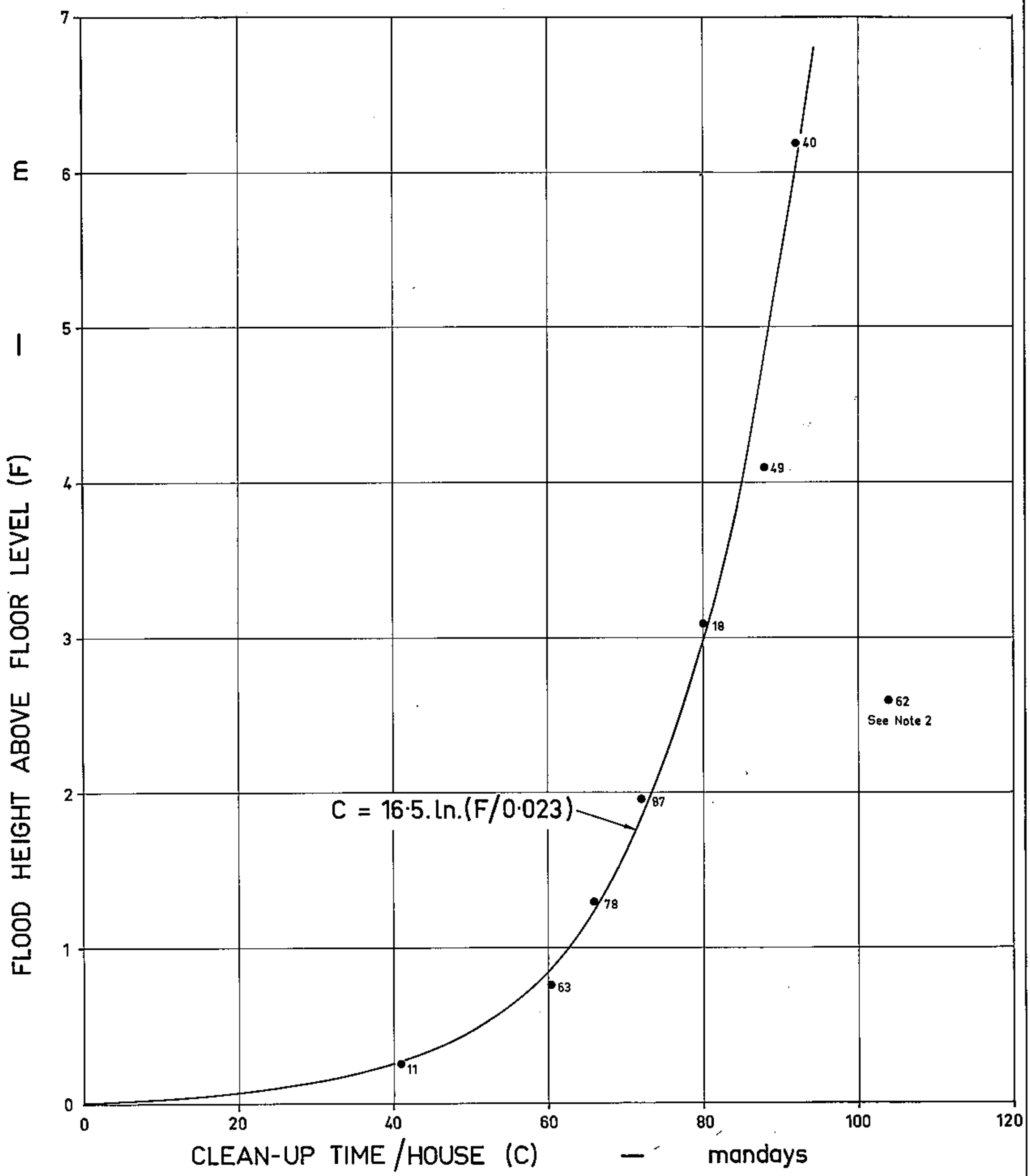
LEGEND
——— Typical results from regression analysis
- - - - Cameron, McNamara and Partners (1973)
-·-·-·- Co-ordinator Generals Report (1971)

BRISBANE RIVER FLOOD INVESTIGATIONS

RESIDENTIAL DAMAGE

NOVEMBER 1975

FIGURE 12



LEGEND

- ● Calculated mean clean-up time/house

NOTES

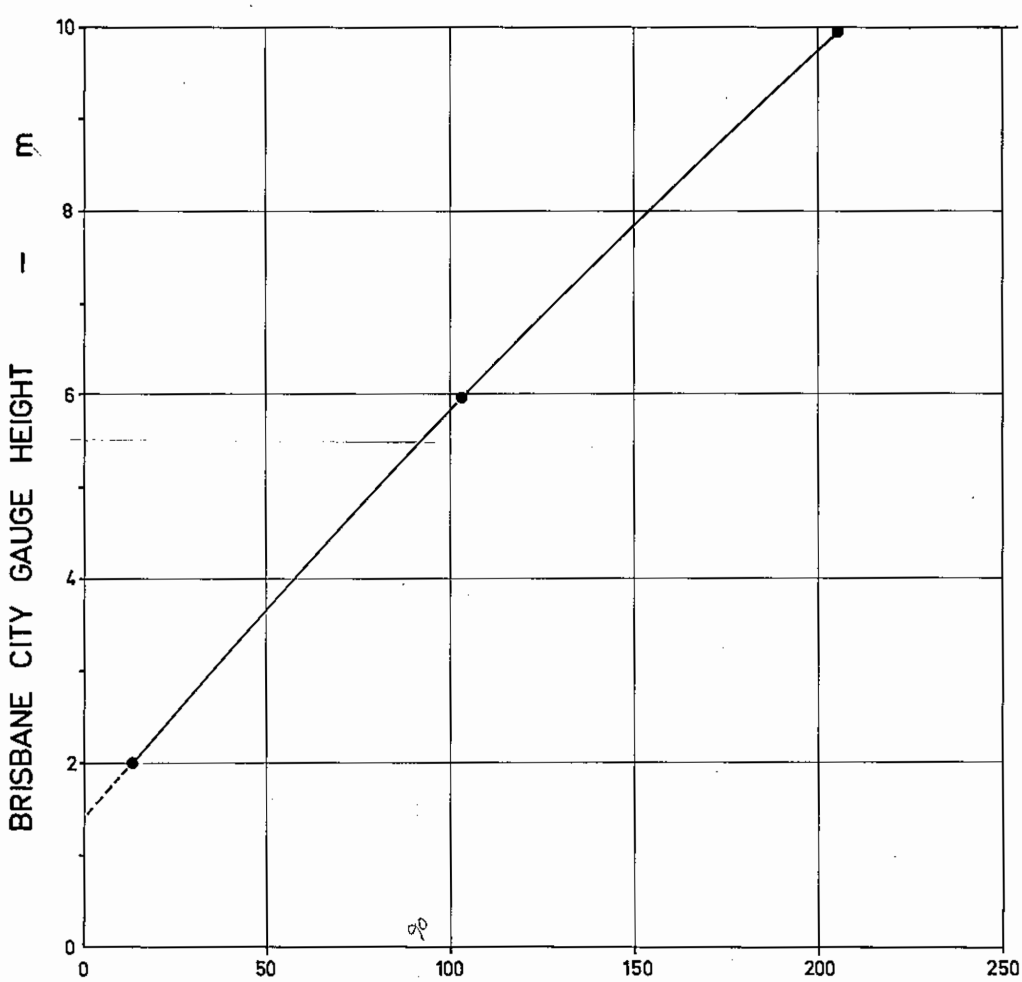
1. The number alongside the plotted point indicates number of values used for calculating the mean
2. This plotted point includes an estimate for 1 house with a clean-up time of 900 mandays and has been disregarded in drawing the general curve.

BRISBANE RIVER FLOOD INVESTIGATIONS

RESIDENTIAL CLEAN-UP TIME

NOVEMBER 1975

FIGURE 13



NOTES

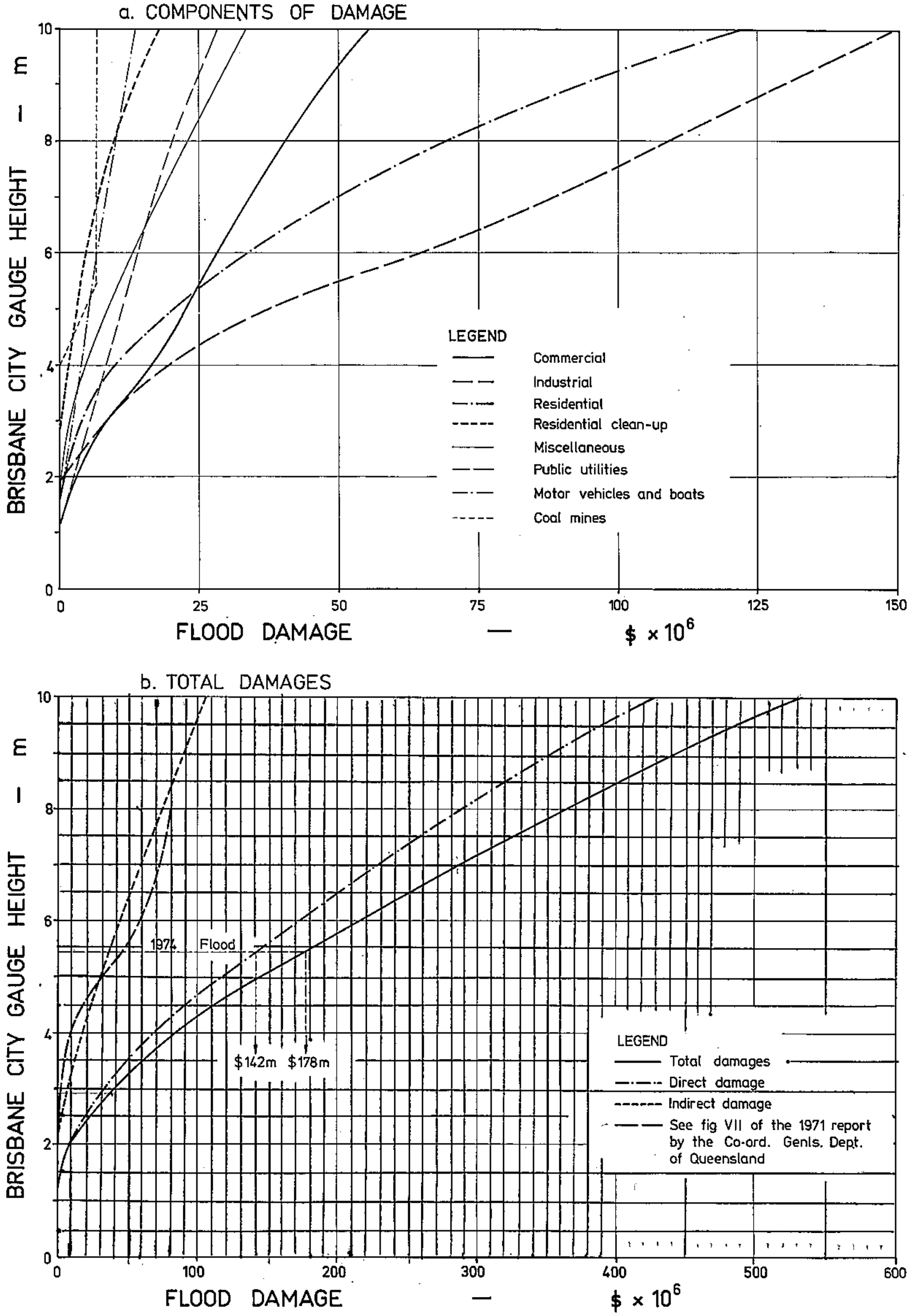
1. The area flooded refers to the area within the cities of Brisbane and Ipswich not normally covered by water
2. Points shown thus were measured from 1:40 000 maps showing the 2, 6 and 10 m flood lines

BRISBANE RIVER FLOOD INVESTIGATIONS

STAGE-FLOODED AREA CURVE

NOVEMBER 1975

FIGURE 14



NOTE

The curves are based on costs and conditions of development on the flood plains as existed in January 1974

BRISBANE RIVER FLOOD INVESTIGATIONS

STAGE-DAMAGE CURVES

NOVEMBER 1975

FIGURE 15

APPENDICES

APPENDIX A

CIRCULAR LETTERS AND QUESTIONNAIRES

CONTENTS

Postal Address : P.O. Box 356, Cooma North, 2630, Australia
Telephone : Cooma 21777 Telex : 61025 Cables : "Snowyconsult, Cooma"

Snowy Mountains Engineering Corporation

TELEPHONE ENQUIRIES

Dear

Following the disastrous flooding in the Cities of Brisbane and Ipswich early this year, the Cities Commission in conjunction with State Government and Local Government Departments, engaged the Snowy Mountains Engineering Corporation to carry out various flood investigations of the Brisbane River. Included in these investigations is an assessment of the total damage caused by the flood.

As part of the process of estimating damages we have selected a large sample of commercial and industrial enterprises from which we would like to receive approximate estimates and other details of the flood damage suffered as a result of the January 1974 flood. This information will be used in the estimation of the total flood damage in 1974 and also for estimating damages that could result from floods of higher and lower magnitude. Your co-operation in this aspect of our study by completing the attached questionnaire would be very much appreciated, as without this type of information it will be very difficult to make satisfactory flood damage estimates.

The Corporation wishes to stress that all information on damages supplied by individuals will be treated as strictly confidential. Further, when reporting on our investigations we will only include a summary of the damage caused and the names and addresses of individuals who supplied the information will be omitted.

Yours faithfully,

[redacted]

Project Manager,
Brisbane River
Flood Investigations

BRISBANE RIVER FLOOD DAMAGE INVESTIGATIONS

QUESTIONNAIRE

Please note that any information supplied will be treated as STRICTLY CONFIDENTIAL

1. NAME OF FIRM: ....

   ADDRESS: ....

2. TYPE OF INDUSTRY OR BUSINESS: ....

3. TOTAL FLOOR AREA FLOODED: (sq. feet) ....

4. DEPTH OF FLOOD WATERS OVER THE MAIN FLOOR LEVEL: ....

5. TYPE OF BUILDING MATERIAL: ....

6. ESTIMATED FINANCIAL LOSS: (Express in financial terms)

   (a) Building ....

   (b) Equipment ....

   (c) Stocks ....

   (d) Materials ....

   (e) Reduced Sales/Loss of profit ....

   (f) Damage to Grounds ....

   (g) Other ....

   (h) Total (a)+(b)+(c)+(d)+(e)+(f)+(g) ....

   (i) Estimated increase in total damage (h) for a flood 1 metre higher than that which occurred in January 1974. ....

   (j) Estimated decrease in total damage (h) for a flood 1 metre lower than that which occurred in January 1974. ....

7. DECREASE IN PRODUCTION DUE TO: (Express as a percentage of normal production)

   (a) Damage to plant and equipment ....

   (b) Damage to stocks and materials ....

   (c) Reduced availability of materials ....

   (d) Reduced availability of transportation ....

8. ESTIMATED TIME REQUIRED TO REGAIN FULL PRODUCTION: ....

9. TOTAL NUMBER OF PERSONS USUALLY EMPLOYED: ....

10. ANY FURTHER COMMENTS: (Please write on reverse side).

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Dear

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The assessment of the total damage includes that caused to commercial and industrial enterprises. This type of work is beyond the expertise of the Corporation and for this reason we have engaged Alex Overett Pty. Ltd. a Brisbane based valuing firm, to conduct this important aspect of our investigations.

In the next few days a member of the firm of Alex Overett Pty. Ltd., will contact you to request your assistance and if possible arrange a mutually convenient time for a valuer to discuss particulars of the flood damage caused to your business or industry. It is possible, that in the course of our data gathering you have already supplied us with data we are seeking either through a trade or commercial organization, or have obliged by returning our questionnaire form. For this we are most grateful. However we find that to achieve our objectives in the assessment of the total flood damage we require more information and your co-operation in this aspect of our study would be much appreciated.

The Corporation wishes to stress that all information on damages supplied by individuals will be treated as strictly confidential. Further, when reporting on our investigations we will only include a summary of the damage caused and the names and addresses of individuals who supplied the information will be omitted.

Yours faithfully,

[redacted]

Project Manager,
Brisbane River
Flood Investigations

SNOWY MOUNTAINS ENGINEERING CORPORATION

BRISBANE RIVER FLOOD INVESTIGATIONS

COMMERCIAL AND INDUSTRIAL DAMAGE ASSESSMENT

Date: .... Interviewer: ....

A. GENERAL

1. Name of Firm: ....
2. Location (a) Street: ....
   (b) Suburb and Postcode: ....
3. Number of Premises: .... (Give details, addresses, etc):
   ....
4. Type of Business or Industry: ....
5. Value of Rent (where applicable): ....
6. Lot No: .... Property Area: .... acres
   Property Area: .... squares

B. ESTIMATE OF FLOODWATER DAMAGE

1. Type of Building Material
   - Brick or Brick Veneer
   - Masonry Block
   - Concrete
   - Timber
   - Steel Frame and Metal Cladding
   - Other
2. Floor Area Flooded versus Maximum Depth of Flood Waters:
   (a) Basement: ....sq.ft ....ft .... in.
   (b) Main Floor: ....sq.ft ....ft .... in.
   (c) 1st Floor: ....sq.ft ....ft .... in.
   (d) Total: ....sq.ft
3. Depth of Flood Water over Property versus Percentage of Property Area:
   ....ft ....in ....% of Property Area
4. Damage to Grounds (describe): ....
   ....
   ....
5. Restoration Costs of Grounds: ....$ ....

6. Damage to Buildings (describe): ....
....
....

7. Restoration Costs of <u>Buildings</u>: ....$ ....

8. Damage to Plant and Equipment (describe): ....
....
....
....

9. Restoration Costs of <u>Plant</u>: ....$ ....
Restoration Costs of <u>Equipment</u>: ....$ ....

10. Damage to <u>Stocks</u>: ....$ ....
Allow for Salvage Value if applicable ....$ ....

11. Damage to <u>Raw Materials</u>: ....$ ....
Allow for Salvage Value if applicable: ....$ ....

12. Other <u>Direct Damages</u>:

	Basement	Main Floor
(a) Furniture:	$..............	$..............
(b) Fixtures:	$..............	$..............
(c) Wiring:	$..............	$..............
(d) Appliances:	$..............	$..............
(e) Furnishings:	$..............	$..............
(f) Office Equipment:	$..............	$..............
Total:	$..............	$..............
	TOTAL (12):	$..............

13. <u>Indirect Damages</u>:

(a) Damage to Business Records: ....
....

(b) Damage to Employee's Possessions
(in Factory or in Office): .... $..............

(c) Loss of Business Income/Reduced Production (describe): ....
....
....
....

Show Amount: $..............

(d) Loss of Employee's Wages:

Number: .... Weeks:.......... Days:....... $..........

(e) Other Indirect Loss:

Cost of Evacuation $................

Cost of Reoccupation $................

Flood Prevention $................

.... $................

.... $................

Q.(1): What percentage of your damages are recoverable by Insurance?

....

Q.(2) Are there any Industries or Commercial Businesses which would rely on your products?

....

....

Q.(3) What expansion is likely to occur with your business in the way of more buildings, more stocks or raw materials in the next 10-20 years?

....

....

Q.(4) If flood protection measures were introduced, what difference would it make to your answer to Q.(3) above?

....

....

S.1. Estimated time required to regain full production:

....

S.2 REMARKS: ....

....

....

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Snowy Mountains Engineering Corporation

TELEPHONE ENQUIRIES

Dear

Following the disastrous flooding in the Cities of Brisbane and Ipswich early this year, the Cities Commission in conjunction with State Government and Local Government Departments, engaged the Snowy Mountains Engineering Corporation to carry out various flood investigations of the Brisbane River. Included in these investigations is an assessment of the total damage caused by the flood.

The assessment of the total flood damage includes that caused to the contents of private homes. This type of work is beyond the expertise of the Corporation and for this reason we have engaged Alex Overett Pty. Ltd., a Brisbane based valuing firm, to conduct this important aspect of our investigations.

In the next few days a member of the firm of Alex Overett Pty. Ltd., will contact you to request your assistance and if possible arrange a mutually convenient time for a valuer to inspect and discuss the flood damage to the contents of your home. Your co-operation in this aspect of the study would be very much appreciated as without this information we cannot make a complete assessment of the total flood damage.

The Corporation wishes to stress that all information on damages supplied by individuals will be treated as strictly confidential. Further, when reporting on our investigations we will only include a summary of the damage caused and the names and addresses of individuals who supplied the information will be omitted.

Yours faithfully,

[redacted]

Brisbane River,
Flood Investigations

BRISBANE RIVER FLOOD INVESTIGATIONS

ASSESSMENT OF DAMAGE TO HOUSE CONTENTS

1.00 GENERAL

.01 Name: ....

.02 Address - Street: ....

.03 Suburb and Postcode: ....

.04 Type of House: High - Set; Low - Set; On Slab;

.05 Construction: Timber Frame; Brick Veneer; Cavity Brick;
Masonry Block;

.06 Age of House: (if not known, estimate) ....

.07 Approximate Floor Area: ....

.08 Approximate Market Value ....

.09 Any measures taken to limit flood damage ....
Moved car; mower; W/Machine? ....

.10 Any damaged items not restored
after flooding? ....

.11 Householder estimate of
damage to contents: ....

.12 Time taken to clean up
(in man-days): ....

2.00 EXTERNAL DAMAGE

.01 Fences: ....

.02 Lawn: ....

.03 Gardens: ....

.04 Pot Plants ....

.05 Retaining Walls: ....

.06 Garages: ....

.07 Other (indicate): ....

3.00 DAMAGE UNDER HOUSE

.01 Hot Water System: ....

.02 Motor Mowers: ....

.03 Car: ....

.04 Tools of Trade: ....

.05 Garden Tools: ....

.06 Other: ....

Name & Address

(to be repeated) ....

4.00 MAIN LEVEL

Furniture and floor coverings (indicate age)

.07 Kitchen – Table & Chairs: ....

.08 Kitchen – Cupboards (not built-in) ....

.09 Kitchen – contents of cupboards: ....

.10 Kitchen – Foods: ....

.11 Kitchen – Floor coverings: ....

.12 Other: ....

.13 Dining Room – Table: ....

.14 Dining Room – Chairs: ....

.15 Dining Room – Sideboard: ....

.16 Dining Room – Floor coverings: ....

.17 Dining Room – Contents of cupboards: ....

.18 Other: ....

.19 Lounge – Chairs: ....

.20 Lounge – Tables: ....

.21 Lounge – Books: ....

.22 Lounge – Paintings: ....

.23 Lounge – Clocks: ....

.24 Lounge – Piano & Musical Instruments: ....

.25 Lounge – Floor coverings: ....

.26 Lounge – Other: ....

Bedroom	1	2	3	4
.27 Double bed & Mattress:				
.28 Single bed & Mattress:				
.29 Pillows:				
.30 Blankets:				
.31 Bed Covers:				
.32 Dressing Table:				
.33 Wardrobes:				
.34 Clothing & personal effects:				
.35 Books & Toys:				
.36 Floor Coverings:				
.37 Other:				

SNOWY MOUNTAINS ENGINEERING CORPORATION

Name & Adress

(to be repeated) ....

5.00 BLINDS AND CURTAINS

.01 Kitchen ....

.02 Dining Room: ....

.03 Lounge Room: ....

.04 Bedroom: 1: ....

2: ....

3: ....

4: ....

.05 Other: ....

6.00 ELECTRICAL

.01 Telephone: ....

.02 Power Outlets: ....

.03 Television: ....

.04 Radiogram & Records: ....

.05 Radio: ....

.06 Tape Recorder & Tapes: ....

.07 Fans: ....

.08 Heater: ....

.09 Refrigerators: ....

.10 Stoves: ....

.11 Wall Oven: ....

.12 Vacuum Cleaners: ....

.13 Polishers: ....

.14 Sewing Machine: ....

.15 Washing Machine: ....

.16 Electrical Wall Switches: (No.) .... (No. damaged) ....

.17 Electrical Light Fittings: ....

.18 Other: ....

APPENDIX B

FLOOD DAMAGE DATA

CONTENTS

NOTE: The source of the flood damage data for commercial and industrial data is indicated by the number in the column headed source. These numbers have the following meaning:

1 – data from questionnaires posted for SMEC by the Chamber of Manufacturers

2 – data from questionnaires posted by SMEC

3 – data obtained by the valuing firm Alex Overett Pty Ltd

4 – data from various sources supplemented by telephone contacts

5 – data from Printing and Allied Trades Employers Association

DATA FOR FLOOD DAMAGED HOMES

POSTCODE NO.	FOUNDATION HEIGHT m	FLOOD DEPTH m	FLOOR AREA m^2	MARKET VALUE $	DAMAGE Structural $	DAMAGE Contents $	BUILDING MATERIAL	EXCEPTION
4068	2.74	0.91	162.6	9 500	820	1 603	WB	0
4068	0.30	3.66	120.8	10 000	7 270	3 615	F	0
4068	0.61	1.52	102.2	8 000	5 240	3 093	WB	0
4068	2.13	1.52	111.5	12 000	1 910	1 966	WB	0
4068	1.52	0.61	111.5	10 000	599	970	WB	0
4068	1.83	2.74	92.9	9 000	1 360	2 384	WB	0
4068	0.30	1.83	139.3	14 000	2 202	3 827	WB	0
4068	0.76	4.57	88.3	9 500	3 840	1 950	WB	0
4068	0.91	1.22	185.8	14 000	1 700	345	WB	0
4068	0.61	1.83	92.9	7 500	2 590	1 926	WB	0
4068	0.61	2.13	92.9	7 500	2 330	2 471	F	0
4068	0.76	2.29	185.8	12 000	1 820	4 855	F	0
4068	0.61	4.27	111.5	10 000	2 800	8 112	F	0
4068	0.91	1.68	111.5	9 000	3 845	2 613	F	0
4068	0.61	1.52	92.9	9 500	1 530	3 034	WB	0
4068	0.61	1.22	92.9	12 000	3 438	2 245	WB	0
4068	0.91	2.13	79.0	4 000	3 530	2 320	WB	0
4068	0.61	0.76	92.9	7 000	1 800	993	WB	0
4068	2.44	0.61	120.8	15 000	2 650	1 740	B	0
4068	2.44	2.44	67.4	5 000	4 330	1 280	WB	0
4068	2.13	1.83	102.2	10 500	2 205	305	WB	0
4068	2.13	1.37	88.3	8 500	1 815	2 302	WB	0
4068	0.30	1.98	92.9	9 000	3 970	4 238	WB	0
4068	0.61	0.61	69.7	6 750	950	650	WB	0
4068	2.13	1.22	120.8	12 000	1 250	3 149	WB	0
4068	0.61	2.13	232.2	15 000	4 070	2 035	WB	0
4068	1.52	3.66	111.5	7 000	2 510	9 099	F	0
4068	2.13	3.66	167.2	11 000	3 770	2 430	WB	0
4068	0.91	1.52	185.8	17 000	1 700	3 990	WB	0
4068	2.13	0.30	130.1	14 000	1 300	1 323	WB	0
4068	0.30	1.37	167.2	25 000	2 930	6 225	B	0
4068	1.07	2.44	74.3	7 000	2 170	1 985	WB	0
4068	1.98	4.88	83.6	6 000	2 430	4 297	WB	0
4068	2.44	2.44	92.9	11 000	3 765	5 900	B	2
4068	1.83	0.91	120.8	11 000	3 040	2 854		0

POSTCODE NO.	FOUNDATION HEIGHT m	FLOOD DEPTH m	FLOOR AREA m^2	MARKET VALUE $	DAMAGE Structural $	DAMAGE Contents $	BUILDING MATERIAL	EXCEPTION
4068	0.61	2.44	157.9	15 000	2 530	2 383	WB	0
4068	2.13	1.07	106.8	11 000	1 520	1 477	WB	0
4068	2.13	1.22	111.5	10 000	660	750	WB	0
4069	0.30	1.83	116.1	11 000	780	110	B	0
4069	0.30	1.83	139.3	26 000	2 400	1 465	B	0
4069	1.83	2.44	171.9	14 000	2 575	2 385	B	0
4069	0.30	1.37	116.1	11 000	2 950	1 385	WB	0
4069	0.30	2.59	130.1	16 000	3 070	3 453	B	0
4069	2.13	0.91	83.6	9 000	1 830	245	B	0
4069	0.30	1.52	153.3	13 000	1 650	825	B	0
4069	1.98	1.52	120.8	10 000	1 640	2 155	B	0
4069	0.61	3.05	148.6	15 000	3 955	1 242	B	0
4069	0.30	2.13	120.8	13 000	2 930	2 060	F	0
4069	0.30	1.83	139.3	14 000	3 000	2 570	B	0
4074	0.91	2.13	139.3	15 000	2 900	5 660	WB	0
4074	2.74	3.66	130.1	14 000	4 150	3 455	F	0
4074	2.13	2.59	130.1	15 000	2 095	1 580	F	3
4074	2.74	1.52	116.1	12 000	3 300	3 365	WB	0
4074	0.30	3.66	130.1	15 000	3 350	3 485	B	0
4074	2.13	0.91	120.8	12 000	945	2 184	WB	0
4074	2.74	3.66	223.0	26 000	4 550	2 055	B	3
4074	0.30	4.27	148.6	17 000	6 870	690	B	0
4074	2.44	1.37	139.3	23 000	2 090	280	B	3
4074	0.61	1.52	102.2	9 000	1 815	4 075	WB	0
4074	0.61	4.27	130.1	13 000	5 085	5 575	WB	0
4074	2.74	0.46	130.1	13 000	1 460	735	B	0
4074	2.44	0.91	130.1	14 000	1 527	940	B	0
4074	2.44	1.37	102.2	10 000	510	2 286	WB	0
4074	0.30	4.57	116.1	12 500	3 750	940	B	0
4067	0.91	0.61	92.9	11 000	2 230	2 175	WB	0
4067	0.15	2.74	92.9	20 000	3 720	3 460	WB	0
4067	2.44	2.74	83.6	10 000	3 225	1 735	WB	0
4067	2.44	2.74	92.9	12 000	2 850	2 710	WB	0
4066	2.44	1.22	111.5	6 500	405	1 620	WB	0
4066	0.61	4.88	130.1	14 000	2 860	3 725	B	0
4066	2.13	1.83	111.5	13 000	2 140	1 394	WB	0

POSTCODE NO.	FOUNDATION HEIGHT m	FLOOD DEPTH m	FLOOR AREA m^2	MARKET VALUE $	DAMAGE Structural $	DAMAGE Contents $	BUILDING MATERIAL	EXCEPTION
4066	2.74	0.91	111.5	12 500	1 520	1 180	WB	0
4066	0.30	1.22	120.8	5 500	1 630	2 070	WB	0
4066	0.91	2.74	74.3	7 500	3 040	1 734	WB	0
4066	0.30	1.83	120.8	35 000	1 940	3 527	B	0
4066	2.74	3.66	83.6	5 000	2 095	2 304	WB	0
4066	2.13	4.88	92.9	14 000	3 710	1 530	WB	0
4075	2.74	1.83	83.6	8 000	2 930	1 427	WB	0
4075	0.30	4.27	120.8	14 000	4 790	4 055	B	0
4075	2.74	1.07	92.9	8 500	1 795	1 264	WB	0
4075	2.74	5.18	111.5	14 000	5 650	1 402	WB	0
4075	0.30	4.88	139.3	15 000	3 005	3 426	WB	0
4075	0.91	2.74	111.5	13 000	5 825	2 315	B	0
4075	0.91	1.22	83.6	6 000	1 440	2 549	B	0
4075	0.30	2.44	125.4	12 000	3 290	2 110	WB	0
4075	0.91	2.74	111.5	11 000	5 485	2 040	0	0
4075	1.98	2.13	126.3	15 000	3 515	2 938	WB	0
4075	1.83	2.74	116.1	18 000	4 105	2 367	WB	0
4075	1.83	2.74	120.8	15 000	2 485	3 900	WB	0
4075	0.91	2.13	102.2	14 000	1 910	1 784	B	0
4075	2.44	3.05	92.9	12 000	3 745	3 292	B	0
4075	1.22	2.29	111.5	12 500	2 860	2 620	WB	0
4075	2.44	2.74	97.5	11 000	4 125	3 853	WB	0
4075	0.61	1.52	102.2	10 000	1 600	2 362	WB	0
4075	2.29	2.44	204.4	21 000	4 795	5 352	B	0
4075	2.13	1.83	74.3	10 000	1 600	2 278	WB	0
4075	0.61	1.22	120.8	12 000	1 435	795	0	0
4075	0.46	1.37	92.9	7 000	1 690	1 510	B	0
4075	2.74	2.74	213.7	28 000	6 150	1 128	B	0
4075	0.30	0.91	139.3	16 500	1 120	2 135	B	0
4075	0.61	2.59	92.9	7 500	3 000	4 517	0	0
4075	0.30	1.68	83.6	10 500	1 310	616	B	0
4075	0.61	6.10	92.9	9 500	4 775	5 160	WB	0
4075	2.13	6.10	106.8	11 000	2 520	2 698	WB	0
4075	2.13	2.74	46.4	3 000	3 425	970	WB	0
4075	2.44	0.91	69.7	6 000	1 220	1 236	WB	0
4075	0.61	1.83	120.8	13 000	2 415	825	WB	0

POSTCODE NO.	FOUNDATION HEIGHT	FLOOD DEPTH	FLOOR AREA	MARKET VALUE	DAMAGE		BUILDING MATERIAL	EXCEPTION
					Structural	Contents		
	m	m	m^2	$	$	$		
4075	1.52	6.10	83.6	8 000	2 960	2 769	WB	0
4076	0.30	4.27	116.1	10 000	4 360	3 040	F	2
4076	1.83	0.61	92.9	5 000	986	880	WB	0
4075	2.13	3.05	83.6	12 500	2 000	2 090	WB	0
4075	2.74	1.52	130.1	22 000	1 930	720	B	0
4075	2.44	1.83	79.0	11 000	1 561	949	WB	0
4075	2.44	1.83	97.5	12 000	1 885	1 907	WB	0
4075	1.83	2.13	130.1	10 000	1 570	2 215	WB	0
4075	0.91	2.44	92.9	10 000	1 210	1 675	WB	0
4075	2.13	2.13	144.0	16 000	2 600	2 790	B	0
4075	1.98	1.52	102.2	12 000	2 020	605	WB	0
4075	1.83	1.83	92.9	13 000	1 900	1 335	WB	0
4075	2.13	2.13	83.6	10 000	2 739	1 647	WB	0
4075	2.13	1.22	83.6	8 500	1 270	380	WB	0
4075	0.91	1.83	102.2	12 500	1 250	2 981	WB	0
4075	1.07	1.22	88.3	7 000	1 205	650	WB	0
4075	1.22	4.57	148.6	13 000	2 980	1 740	WB	0
4075	0.91	1.83	97.5	11 500	1 350	1 308	WB	0
4075	2.44	1.83	111.5	12 500	1 930	295	WB	0
4075	0.61	1.68	92.9	8 000	1 250	661	WB	0
4075	1.83	2.74	130.1	11 000	2 045	2 914	WB	0
4075	0.61	2.44	111.5	15 000	2 520	2 748		0
4075	1.22	2.13	116.1	10 000	1 883	2 634	WB	0
4075	0.91	2.13	97.5	12 000	1 100	2 040	WB	0
4075	2.74	0.91	92.9	9 000	420	703	WB	0
4075	0.61	1.22	120.8	12 000	1 265	2 476	O	0
4030	1.83	1.52	102.2	9 500	1 095	1 610		0
4030	0.91	1.83	92.9	8 000	1 165	1 060	F	0
4030	2.13	0.76	92.9	7 000	570	1 225	WB	0
4030	1.83	2.13	111.5	8 000	1 900	1 135	WB	0
4030	1.83	0.30	92.9	6 000	330	975	WB	0
4300	2.29	5.49	102.2	14 000	5 509	2 875	WB	0
4300	1.22	4.88	120.8	16 000	2 820	2 628	WB	0
4300	1.52	1.52	111.5	13 000	1 225	2 405	WB	0
4300	3.05	5.79	74.3	7 000	1 170	1 458	WB	0
4300	1.22	4.57	83.6	9 000	2 225	2 795		0

POSTCODE NO.	FOUNDATION HEIGHT	FLOOD DEPTH	FLOOR AREA	MARKET VALUE	DAMAGE		BUILDING MATERIAL	EXCEPTION
					Structural	Contents		
	m	m	m^2	$	$	$		
4300	1.83	7.92	130.1	12 000	4 510	6 280	WB	0
4300	0.61	0.30	83.6	9 000	1 150	4 100	WB	0
4300	0.61	8.53	134.7	13 000	2 660	4 390	WB	0
4300	1.83	15.24	139.3	8 500	4 030	4 684	F	0
4300	1.52	1.22	120.8	15 000	1 810	1 080	WB	0
4300	1.83	2.13	92.9	13 000	2 560	1 460	WB	0
4300	1.52	4.27	111.5	11 000	2 730	2 722	WB	0
4300	2.44	3.66	111.5	11 000	4 590	1 435	WB	0
4300	1.22	6.71	102.2	9 500	6 050	10 870	WB	0
4300	0.30	1.07	130.1	15 000	880	1 210	B	0
4300	1.22	1.07	51.1	3 000	4 580	2 823	F	0
4300	2.13	4.88	139.3	13 000	4 895	2 730	WB	0
4300	2.44	5.49	130.1	10 000	8 275	975	WB	0
4300	2.13	7.62	111.5	9 000	7 295	2 653	WB	0
4300	0.61	4.27	102.2	10 000	2 530	2 877	F	0
4300	2.44	7.01	111.5	15 000	2 000	1 325	B	0
4300	0.61	1.22	157.9	16 000	2 980	3 045	WB	0
4300	1.22	1.98	92.9	10 000	1 100	1 478	WB	0
4304	0.61	3.66	60.4	5 000	2 100	1 666	WB	0
4304	2.13	2.13	120.8	12 000	1 530	1 285	WB	0
4304	1.83	1.83	41.8	2 000	2 593	1 404	WB	0
4304	0.61	4.27	41.8	3 000	1 530	655	WB	0
4304	0.61	1.37	74.3	5 000	1 720	390	WB	0
4304	1.83	1.22	79.0	8 000	1 010	946	O	0
4304	1.83	3.05	55.7	4 500	1 525	715	F	0
4304	0.61	1.83	46.4	3 000	1 565	692	WB	0
4304	0.46	3.05	111.5	5 000	7 000	2 381	WB	0
4304	0.91	4.27	92.9	11 000	2 788	2 166	WB	0
4304	1.83	2.13	83.6	8 000	1 600	1 155	WB	0
4304	2.44	4.57	74.3	8 000	2 870	1 937	WB	0
4304	3.66	1.83	120.8	13 000	1 800	1 550	WB	0
4304	2.44	0.76	74.3	9 000	910	375	WB	0
4304	0.91	1.52	60.4	6 000	2 115	1 161	WB	0
4304	0.61	2.44	92.9	10 000	2 860	420	B	0
4304	2.13	1.83	65.0	4 000	1 045	815	WB	0
4304	1.22	1.83	148.6	11 000	1 980	1 314	WB	0

POSTCODE NO.	FOUNDATION HEIGHT m	FLOOD DEPTH m	FLOOR AREA m^2	MARKET VALUE $	DAMAGE Structural $	DAMAGE Contents $	BUILDING MATERIAL	EXCEPTION
4304	2.44	2.13	102.2	8 500	250	370	WB	0
4304	0.91	1.52	74.3	6 000	2 000	440	WB	0
4304	0.76	0.61	102.2	8 000	1 450	1 270	WB	0
4304	1.22	0.30	55.7	4 500	100	280	WB	0
4304	2.13	2.44	139.3	12 000	2 630	3 787	F	0
4304	0.61	2.59	111.5	11 500	2 465	1 368	WB	0
4304	0.61	0.91	92.9	10 000	660	1 160	WB	0
4304	0.61	1.98	111.5	10 000	1 730	1 945	WB	0
4304	0.61	1.68	74.3	6 500	3 235	600	WB	0
4304	0.61	0.91	106.8	11 500	810	1 185	WB	0
4304	1.83	2.13	111.5	8 000	600	850	WB	0
4304	0.91	1.52	116.1	8 000	1 020	1 290	WB	0
4304	0.91	1.22	74.3	8 000	1 970	675	WB	0
4304	2.13	0.46	74.3	11 000	910	1 210	WB	0
4304	0.91	0.91	54.8	5 500	2 000	1 237		0
4304	0.30	2.13	111.5	9 000	2 260	1 265	WB	0
4304	0.61	1.83	74.3	7 000	1 050	1 149	WB	0
4304	0.91	1.52	74.3	8 000	1 300	523	WB	0
4304	0.61	1.83	102.2	11 000	2 380	776	O	0
4304	0.91	1.22	92.9	7 500	2 085	650	WB	0
4304	0.76	0.61	92.9	10 000	430	430	WB	0
4304	2.13	3.66	111.5	7 000	2 460	2 686	F	0
4304	0.30	1.52	60.4	7 500	1 890	701	WB	0
4304	1.83	0.91	102.2	7 500	1 460	992	WB	0
4304	0.61	1.52	92.9	7 500	1 960	1 435	WB	0
4304	0.91	0.61	74.3	7 000	1 160	602	WB	0
4304	0.91	3.66	69.7	8 500	1 975	1 347	WB	0
4304	0.91	0.61	83.6	9 000	2 005	640	CR	0
4304	2.74	5.49	92.9	11 000	1 130	954	WB	0
4305	0.46	2.44	66.9	5 500	2 510	1 092	WB	0
4305	1.83	1.83	111.5	4 000	3 180	610	WB	0
4305	2.44	1.07	102.2	6 000	2 750	1 386	F	0
4305	2.74	6.10	102.2	4 000	1 100	482	WB	0
4305	0.61	0.61	55.7	3 000	770	770	WB	0
4305	2.13	3.66	79.0	5 000	1 480	1 400	WB	0
4305	1.98	3.05	60.4	4 500	1 810	1 932	WB	0

POSTCODE NO.	FOUNDATION HEIGHT m	FLOOD DEPTH m	FLOOR AREA m^2	MARKET VALUE $	DAMAGE Structural $	DAMAGE Contents $	BUILDING MATERIAL	EXCEPTION
4305	0.91	3.66	55.7	4 250	2 355	832	WB	0
4305	1.83	2.13	83.6	5 500	870	1 306	WB	0
4305	1.83	3.05	65.0	4 000	6 000	699	WB	0
4305	1.52	1.22	74.3	4 000	2 590	1 307	WB	0
4305	0.91	0.91	120.8	7 000	720	1 105	WB	0
4305	2.13	1.52	74.3	6 000	1 760	535	WB	0
4305	0.61	2.74	111.5	13 000	1 890	1 944	WB	0
4305	0.76	4.88	55.7	5 500	2 520	1 223	F	0
4305	1.22	2.13	65.0	6 500	1 540	1 170	WB	0
4305	1.22	1.83	74.3	7 000	2 605	1 241	WB	0
4305	0.91	1.37	69.7	5 500	2 140	666	WB	0
4305	0.76	6.10	69.7	4 000	2 495	777	F	0
4305	1.22	2.44	111.5	6 500	3 625	5 146	WB	0
4305	2.13	1.83	65.0	8 000	1 560	1 270	WB	0
4305	0.91	4.27	92.9	10 500	3 720	2 166	WB	0
4305	2.44	2.44	79.0	5 500	1 650	843	WB	0
4305	0.91	2.44	139.3	9 000	4 270	1 153	WB	0
4305	2.13	2.59	116.1	8 000	2 605	2 618	WB	0
4305	1.22	2.13	111.5	8 000	1 930	1 755	WB	0
4305	0.30	3.05	111.5	8 000	3 045	1 130	WB	0
4305	2.74	0.91	79.0	9 000	1 115	245	WB	0
4305	0.91	2.29	102.0	9 500	1 625	1 386	WB	0
4305	0.91	0.76	79.0	8 000	725	965	WB	0
4305	0.61	2.74	111.5	8 000	3 980	3 953	WB	0
4305	0.61	2.74	92.9	8 000	1 401	1 065	WB	0
4305	1.07	0.91	83.6	4 000	2 895	376	F	0
4305	3.66	1.37	92.9	7 000	1 790	3 645	WB	0
4305	0.91	4.57	74.3	6 000	1 860	780	WB	0
4305	0.30	6.10	139.3	16 000	6 130	3 150	B	0
4305	0.30	2.13	111.5	17 000	3 690	842	B	0
4305	2.13	1.83	92.9	8 000	1 320	2 349	O	0
4305	2.44	4.57	69.7	8 000	2 040	2 586	WB	0
4064	2.29	0.91	92.9	5 000	1 490	1 151	WB	0
4064	0.91	0.30	65.0	6 000	175	1 069	WB	0
4064	0.91	5.49	92.9	4 000	2 980	5 005	WB	0
4064	2.44	0.91	134.7	10 000	1 025	2 217	WB	0

POSTCODE NO.	FOUNDATION HEIGHT m	FLOOD DEPTH m	FLOOR AREA m^2	MARKET VALUE $	DAMAGE Structural $	DAMAGE Contents $	BUILDING MATERIAL	EXCEPTION
4064	0.61	3.66	116.1	10 000	2 390	2 900	B	0
4064	2.44	3.35	120.8	8 000	2 970	3 562	WB	0
4064	1.98	0.91	88.3	8 000	1 240	1 748	WB	0
4064	1.83	1.52	92.9	7 000	920	3 135	WB	0
4064	2.13	0.61	55.7	3 000	430	481	F	0
4064	2.13	1.83	88.3	7 000	1 755	1 051	WB	0
4064	2.13	1.52	148.6	15 000	1 510	1 410	WB	0
4101	2.44	0.61	130.1	7 000	1 300	2 220	WB	2
4101	1.68	4.27	55.7	4 000	3 020	2 082	WB	0
4101	1.83	0.61	74.3	5 500	270	1 232	WB	0
4101	2.13	1.52	111.5	7 500	2 600	1 797	WB	0
4101	2.44	0.91	92.9	7 500	1 730	1 359	WB	0
4101	2.13	0.91	102.2	5 000	1 545	2 250	WB	0
4101	2.13	1.52	92.9	4 000	2 000	532	WB	0
4103	1.98	4.57	92.9	8 500	1 775	1 908	WB	0
4103	2.44	0.91	102.2	8 500	570	1 785	WB	0
4103	1.07	6.10	92.9	8 000	2 450	2 340	WB	0
4103	2.13	1.98	97.5	8 000	1 730	2 055	WB	0
4103	0.91	2.44	88.3	9 000	1 900	4 237	WB	0
4103	1.52	4.57	111.5	8 500	5 515	3 023	WB	0
4103	2.74	1.98	79.0	7 000	1 910	1 876	WB	0
4105	1.68	5.49	113.8	8 000	4 490	4 140	WB	0
4105	0.91	2.29	88.3	9 000	2 230	4 678	WB	0
4105	0.91	3.05	88.3	7 000	2 650	1 003	WB	0
4105	0.46	1.52	74.3	8 500	1 585	2 520	WB	0
4105	0.61	2.13	83.6	8 000	1 610	1 931	WB	0
4105	0.61	2.44	97.5	9 500	3 880	3 198	WB	0
4105	0.61	2.74	79.0	5 000	935	1 002	WB	0
4106	0.91	1.22	130.1	8 000	2 525	2 411	WB	0
4106	0.61	4.57	83.6	7 000	1 100	1 924	WB	0
4106	0.30	2.59	111.5	8 000	2 460	3 144	WB	0
4106	0.30	3.35	83.6	7 000	2 655	2 140	WB	0
4106	0.61	1.83	74.3	8 000	2 510	2 285	WB	0
4106	0.00	4.27	111.5	9 500	3 960	3 460	WB	0
4106	0.76	2.13	102.2	8 500	2 135	2 985	WB	0
4106	0.91	4.88	102.2	8 500	2 285	2 195	WB	0

POSTCODE NO.	FOUNDATION HEIGHT m	FLOOD DEPTH m	FLOOR AREA m^2	MARKET VALUE $	DAMAGE Structural $	DAMAGE Contents $	BUILDING MATERIAL	EXCEPTION
4106	2.13	1.22	102.2	9 000	1 385	2 341	F	0
4106	1.98	6.71	102.2	10 000	4 320	3 670	WB	0
4106	0.61	5.49	74.3	7 000	3 060	2 391	F	0
4106	0.00	0.91	92.9	10 000	1 440	1 890	B	0
4106	1.22	4.27	37.2	2 500	1 460	732	WB	0
4104	1.52	1.22	111.5	12 000	2 620	1 136	F	0
4104	0.91	0.61	111.5	13 000	1 330	1 722	WB	0
4104	2.13	2.29	102.2	10 000	2 210	2 765	WB	0
4104	1.98	2.59	83.6	8 500	2 260	1 351	WB	0
4104	0.91	4.88	97.5	12 000	1 730	4 347	WB	0
4104	1.83	1.83	116.1	8 000	1 190	2 653	WB	0
4104	0.00	4.88	148.6	16 000	7 155	3 937	B	0
4104	2.44	1.07	97.5	12 000	1 630	1 329	B	0
4104	0.61	1.83	204.4	14 000	3 100	2 401	WB	0
4108	0.61	2.74	88.3	6 500	2 830	3 215	WB	0
4108	1.83	2.13	88.3	9 000	1 050	1 254	WB	0
4108	0.91	2.13	92.9	7 500	2 965	1 417	F	0
4170	0.46	1.52	92.9	6 000	1 340	1 199	F	0
4170	0.61	1.37	83.6	5 000	525	498	F	0
4170	0.61	0.61	120.8	10 500	615	486	B	1
4171	0.61	0.46	92.9	7 000	340	105	F	0
4066	2.13	1.22	111.5	15 000	1 760	654	WB	0
4075	0.91	0.30	102.2	17 500	495	790	CR	0
4304	0.91	2.90	74.3	5 000	2 250	1 456	WB	0
4305	1.83	0.91	83.6	6 000	1 195	721	B	0
4305	1.52	4.57	60.4	7 000	3 640	945	F	0
4305	0.61	3.20	83.6	8 500	3 682	2 180	WB	0
4103	2.44	2.13	97.5	8 000	1 165	1 630	WB	0
4068	2.44	3.96	130.1	15 000	2 082	2 238	B	2
4068	2.44	1.83	48.3	5 000	550	1 110	B	2
4068	2.13	0.61	92.9	11 000	1 100	1 133	WB	0
4068	2.13	2.44	162.6	17 500	3 385	3 868	WB	3
4068	2.44	2.44	278.7	50 000	5 660	22 360	B	0
4068	0.30	2.90	185.8	20 000	6 090	4 640	B	0
4068	2.74	2.13	334.4	40 000	2 670	1 715	B	0
4069	2.74	2.44	232.2	17 000	850	5 590	B	2

POSTCODE NO.	FOUNDATION HEIGHT m	FLOOD DEPTH m	FLOOR AREA m^2	MARKET VALUE $	DAMAGE Structural $	DAMAGE Contents $	BUILDING MATERIAL	EXCEPTION
4074	0.30	2.13	157.9	20 000	1 633	1 280	B	2
4074	2.74	0.30	185.8	14 000	270	200	B	2
4074	2.44	2.13	139.3	11 000	1 600	615		0
4074	2.13	3.05	185.8	20 000	2 950	1 084		0
4075	2.74	0.30	102.2	16 000	1 045	2 070	WB	0
4075	2.44	3.05	185.8	16 000	4 945	815	WB	0
4075	2.44	1.83	102.2	9 000	1 239	1 816	B	0
4075	2.13	0.91	139.3	15 000	730	330		0
4030	1.83	0.30	116.1	9 000	940	1 283	WB	0
4304	1.22	0.91	74.3	8 000	740	375	WB	3
4304	2.13	1.07	55.7	5 000	300	125	WB	0
4305	2.74	1.22	102.2	11 000	1 966	1 230	B	0
4105	2.44	0.15	97.5	9 000	355	300	WB	0
4075	2.44	0.91	232.2	25 000	2 220	1 167	B	0
4304	2.29	0.91	139.3	15 000	500	355	WB	0
4305	1.98	0.00	120.8	10 000	396	40	WB	0
4101	2.13	0.91	83.6	15 000	1 040	10	B	0
4068	2.44	1.22	111.5	12 000	2 960	1 038	WB	0
4068	0.91	1.83	92.9	10 000	3 530	1 877	WB	0
4068	0.61	2.13	185.8	20 000	2 985	2 990	B	0
4068	0.30	1.83	111.5	12 000	961	1 852	WB	0
4068	2.13	0.91	111.5	12 000	1 850	1 306	WB	0
4068	0.61	4.57	111.5	12 000	3 250	2 450		0
4068	1.98	1.52	111.5	12 000	1 800	1 475	WB	0
4068	2.74	3.05	111.5	12 000	5 270	3 267		0
4068	0.91	1.83	130.1	14 000	1 250	1 695		0
4068	0.30	3.66	148.6	16 000	2 590	4 781	B	0
4069	0.30	2.13	181.2	26 000	4 350	1 215	B	0
4069	3.66	4.88	204.4	30 000	6 100	3 050	B	0
4069	0.30	4.57	106.8	25 000	5 245	4 675	B	0
4069	2.44	4.88	325.1	40 000	3 240	8 776	B	3
4069	2.44	2.29	148.6	16 000	190	214	B	2
4069	0.30	0.61	130.1	18 000	1 090	215	B	0
4069	0.30	2.29	92.9	12 000	2 750	2 116	WB	2
4069	0.30	3.66	139.3	30 000	3 925	4 080	B	0
4030	1.83	2.44	92.9	6 500	915	2 145	WB	0

POSTCODE NO.	FOUNDATION HEIGHT m	FLOOD DEPTH m	FLOOR AREA m^2	MARKET VALUE $	DAMAGE Structural $	DAMAGE Contents $	BUILDING MATERIAL	EXCEPTION
4030	2.74	2.13	92.9	7 000	3 360	2 698	WB	0
4030	2.44	2.29	106.8	12 000	340	925	WB	0
4300	2.44	2.13	102.2	13 500	3 895	1 565	WB	0
4300	0.61	4.57	46.4	3 500	1 940	1 866	WB	0
4300	1.22	0.91	83.6	8 000	2 200	770	B	0
4300	1.83	2.44	111.5	8 000	1 180	1 496	WB	0
4300	0.91	5.49	130.1	11 000	580	320	WB	0
4300	2.74	2.74	167.2	11 000	1 590	4 218	WB	0
4300	0.30	0.61	102.2	12 000	770	2 289	B	0
4300	1.83	4.57	65.0	5 000	1 915	1 856		0
4300	2.59	0.91	130.1	9 000	640	345	0	0
4304	1.83	1.83	60.4	3 000	2 147	1 387	WB	0
4304	1.83	1.22	65.0	5 000	1 030	711	WB	0
4304	0.61	4.57	92.9	8 000	2 320	2 130	WB	0
4305	1.22	2.44	74.3	7 500	1 290	950	WB	0
4064	2.29	2.74	83.6	4 500	2 175	2 320	WB	0
4064	2.13	0.91	111.5	12 000	555	1 185	WB	0
4064	1.83	3.66	92.9	5 000	1 490	3 540	WB	0
4051	2.74	2.13	83.6	3 000	1 400	1 380	WB	0
4103	1.37	1.98	79.0	6 000	1 360	1 335	WB	0
4103	1.83	1.83	130.1	14 000	1 560	1 695	WB	0
4103	0.91	2.74	111.5	13 000	2 735	2 569	WB	0
4105	0.91	3.35	102.2	7 500	2 210	2 980	B	0
4106	1.22	1.83	111.5	8 500	1 130	1 150	WB	0
4106	0.61	2.44	120.8	9 000	2 930	3 245	WB	0
4106	2.13	2.44	92.9	8 000	1 590	2 845	WB	0
4106	0.76	1.22	83.6	8 000	2 125	2 685	WB	0
4106	1.98	0.91	88.3	9 500	1 270	976	WB	0
4106	1.07	1.83	102.2	11 000	1 980	2 400	WB	0
4106	0.91	5.18	111.5	9 000	3 480	1 490	WB	0
4106	0.76	4.57	83.6	7 500	1 465	2 460	WB	0
4106	0.00	0.91	92.9	10 000	1 480	400	WB	0
4104	2.29	9.14	116.1	15 000	15 000	3 990	WB	0
4104	2.74	1.22	111.5	8 000	974	2 615	WB	0
4104	2.29	6.10	102.2	9 000	2 330	3 020	WB	0
4104	1.22	3.66	92.9	7 000	2 190	2 810	WB	0

POSTCODE NO.	FOUNDATION HEIGHT m	FLOOD DEPTH m	FLOOR AREA m^2	MARKET VALUE $	DAMAGE Structural $	DAMAGE Contents $	BUILDING MATERIAL	EXCEPTION
4104	1.52	0.00	176.5	20 000	270	30	WB	0
4300	0.61	1.07	111.5	2 000	1 860	824	WB	0
4106	0.61	4.57	92.9	7 000	2 210	1 265	WB	0
4106	0.76	1.37	88.3	9 500	1 270	1 575	WB	0

NOTE: WB denotes weather-board house
B denotes brick house
F denotes fibro-cement house
CR denotes cement-render house
0 denotes house
blank denotes house

Flood Depth Range: 0 m to 0.31 m

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m^2$	DAMAGE Indirect $/m^2$
2	Confectionery	0.03	0.9	8.9
2	Marine Retailer	0.2	26.9	134.6
2	Millers & Merchants	0.2	19.9	3.9
2	Electrical Retailer	0.1	107.6	10.8
2	Wholesale Distributor	0.2	3.1	0
2	Appliance Distributor	0.2	4.7	1.4
2	Motor Vehicle Sales	0.2	0	1.4
3	Machine Tool Supplies	0.1	0.3	0.1

Flood Depth Range: 0.31 m to 0.91 m

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m^2$	DAMAGE Indirect $/m^2$
2	Real Estate Agent	0.6	3.6	53.8
2	Chemicals & Plastics Sales	0.5	23.0	7.6
2	Wholesale Merchant	0.6	6.8	6.8
2	Engineering Supplies	0.4	9.4	35.8
1	Cleaning Supplies	0.3	4.6	0
1	Funeral Director	0.8	0.6	0
2	Electrical Equipment Distr.	0.4	1.2	0
2	Stationery Wholesaler	0.6	1.6	0
2	Floor Emulsion Wholesaler	0.6	1.4	0
3	Sausage Casing Merchant	0.3	0.9	0.9
2	Seed Merchant	0.5	99.6	26.9
2	Wholesaler	0.7	5.7	32.3
2	Butchers Supplier	0.8	21.1	0
2	Wine & Spirit Merchant	0.8	14.0	3.2
2	Appliance Sales	0.6	16.1	25.8
2	Steel & Cable Stockist	0.8	2.4	0
2	Residential Flats	0.6	6.1	1.1
2	Wholesale Distributor	0.6	41.0	0
2	Service Station	0.7	0.9	2.2
2	Wholesale Distributor	0.5	40.4	93.1
1	Appliance Merchant	0.3	89.1	33.4
2	Pharmaceutical Wholesaler	0.3	46.2	7.6
2	Footwear Retailer	0.3	0.5	0
2	Electronic Equipment Wholesaler	0.5	22.4	11.2
2	Ceramic Tile Wholesaler	0.6	8.9	3.6
2	Milk Bar	0.3	1.5	3.1
4	Retail Store	0.3	3.2	0
4	Snack Bar	0.5	19.4	0

Flood Depth Range: 0.91 m to 1.37 m

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m^2$	DAMAGE Indirect $/m^2$
1	Paint Wholesaler	1.2	34.4	0
2	Tea Merchant	1.0	136.9	143.5
2	Drawing Officer Supplier	1.2	17.8	10.8
2	Machinery Merchant	1.2	13.9	0
2	Wholesaler	1.0	40.9	0
2	Electrical Appliances Sales	1.2	15.9	2.5
2	Scale Sales	1.2	8.6	43.1
2	Storage Contractor	1.0	67.5	2.0
2	Tyre, Battery Supplier	0.9	18.1	29.8
2	Investment Company	0.9	1.8	0
2	Newsagent	0.9	10.0	2.9
2	Electrical Wholesaler	0.9	41.6	0
2	Industrial Land Lease	0.9	7.2	0
2	Electrical Goods Wholesaler	1.2	110.9	36.8
2	Wholesale Distributors	1.2	86.1	0
2	Wholesale Distributors	1.2	18.6	6.6
2	Carpet Retailer	1.2	43.1	0
2	Squash & Tennis Centre	1.2	16.9	5.4
2	Mfrs Agent	0.9	53.8	0
2	Marine Retailer	1.2	41.2	10.7
2	Marine Retailer	1.2	3.6	4.3
2	Motel	1.2	4.7	22.1
2	Wool Broker, Agent	1.2	44.9	39.5
2	Motor Vehicle Sales	1.3	71.9	0
2	Building Materials Supplier	1.2	20.9	29.9
2	Upholstery Supplies	1.2	2.7	2.7
2	Caravan Park	1.2	125.6	35.8
2	Machinery Merchant	0.9	1.0	0.6
2	Snack Bar	1.1	11.8	21.5
2	Shipping Company	1.0	2.5	5.7
2	Automotive Wholesaler	1.2	2.7	0
2	Car Park	1.0	29.6	15.4
2	Car Park	1.2	0.3	0.5
2	Wholesaler	1.1	0.8	0.9

Flood Depth Range: 0.91 m to 1.37 m - Continued

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m^2$	Indirect $/m^2$
2	Footwear Retailer	0.9	77.7	0
2	Retail Store	0.9	20.2	0
2	Refrig. Spares Wholesaler	1.1	51.1	13.5
2	Footwear Retailer	0.9	2.8	0
2	Wholesaler	1.2	57.1	21.5
3	College	1.2	10.9	0.2
2	Wholesale Merchant	1.0	14.9	4.7
4	Mixed Business	1.2	95.0	-
4	Motor Parts Retailer	1.1	26.3	-
4	Glassware Merchant	1.2	32.3	-

Flood Depth Range: 1.37 m to 1.83 m

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m^2$	Indirect $/m^2$
2	Service Station	1.7	12.5	38.8
2	Hotel	1.7	4.0	0
2	Office Equipment Supplies	1.5	62.2	7.2
2	Foodstuff Wholesaler	1.7	50.1	21.5
2	Brassware Wholesaler	1.8	94.3	17.8
2	Scientific Instrument Supplier	1.4	15.0	0
3	Steel Products Distributor	1.5	27.8	2.0
2	Machine Tool Dealer	1.4	49.5	0
2	Softgoods Retailer	1.5	161.5	107.6
2	Oil Company	1.5	5.9	8.1
2	Lighting Products Supplier	1.4	94.7	193.8
2	Textile Wholesaler	1.7	108.5	0
2	Importer	1.5	56.7	21.5
2	Electronic Computer Marketer	1.7	292.5	477.9
2	Motor Cycle Retailer	1.5	14.0	10.8
2	Food Wholesaler	1.5	69.6	1.8
2	Earthmoving Equip. Retailer	1.7	5.0	4.3
2	Motel	1.5	1.4	1.4
2	Hotel	1.7	3.3	4.7
2	Clothing Wholesaler	1.5	75.6	0
2	Insurance Broker	1.4	45.2	0
2	Wool Broker	1.5	13.3	0
2	Car Park	1.7	0.1	0.2
2	Warehousing	1.4	34.3	0
2	Footwear Retailing	1.5	89.9	0
2	Footwear Retailing	1.5	117.0	0
2	Florist	1.5	12.5	5.4
2	Forklift Hire	1.4	14.7	0
4	Interior Decorator	1.7	21.5	0
4	Produce Merchant	1.7	22.9	0
4	Veterinary Clinic	1.7	52.9	0
4	Mixed Business	1.4	24.0	0
2	Industrial Belts Wholesaler	1.6	18.9	0

Flood Depth Range: 1.83 m to 2.29 m

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m^2$	DAMAGE Indirect $/m^2$
2	Hotel	1.8	2.9	3.0
2	Wholesale Distributor	1.8	59.6	84.4
2	Property Leaser	2.0	6.8	0
2	Real Estate Valuers	2.1	64.6	0
2	Service Station	1.8	17.5	5.4
2	Timber Merchant	1.8	26.9	0
2	Wholesaler	2.1	47.1	19.8
2	Soft Furnishing Wholesaler	2.1	15.1	0
2	Wool Brokers	1.8	1.8	3.7
2	Food Wholesaler	1.8	51.2	1.2
2	Mfr Agent	1.8	47.4	0
2	Oil Sales	2.0	13.8	0
2	Wholesaler	2.1	51.0	0
2	Wholesaler	1.8	36.1	0
2	Motor Vehicle Sales	2.2	76.5	21.4
2	Wool Brokers	2.0	53.5	9.9
2	Agent, Importer	2.1	17.8	0
3	Wholesaler	2.2	448.0	20.5
4	Milk Bar	1.8	59.8	-
1	Building Society	2.0	6.1	0
4	Tailor	1.8	129.2	-
4	TV, Radio Sales	1.8	130.9	-
3	Tractor Distributor	2.1	137.9	32.9

Flood Depth Range: 2.29 m to 2.74 m

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m^2$	DAMAGE Indirect $/m^2$
2	Toy Wholesaler	2.6	49.8	70.2
2	Elect. Appliances Distr.	2.4	65.1	51.9
3	Building Supplier	2.4	177.5	218.5
3	Hardware Retailer	2.4	140.0	2.8
2	Wallpaper Wholesaler	2.4	35.8	0
2	Wine Distributor	2.4	39.0	8.8
2	Diesel Engines Distributor	2.4	6.7	4.8
2	Wholesale Merchant	2.4	157.7	59.3
2	Marine Sales	2.4	4.5	224.2
2	Motion Picture	2.4	58.3	8.9
2	Coach Operator	2.4	1.1	6.5
2	Soft Goods Wholesaler	2.3	199.1	0
4	Used Appliance Sales	2.4	32.3	-
2	Hardware Retailer	2.3	35.8	11.2
4	Newsagent	2.3	113.9	-
4	Customs Agent	2.4	37.4	-
4	Spirit Merchant	2.3	111.3	-
4	Beauty Salon	2.4	68.9	-

COMMERCIAL DAMAGE

Flood Depth Range: 2.74 m to 3.66 m

SOURCE	TYPE	FLOOD DEPTH	DAMAGE	
		m	Direct $/m^2$	Indirect $/m^2$
1	Chemicals Agent	3.0	43.4	6.2
2	Machinery Distributor	3.4	29.9	239.2
3	Farm Machinery Retailer	2.7	131.3	75.5
2	Metal Merchants	3.0	2.4	8.9
3	Paper Merchants	2.7	102.7	5.1
2	Wholesaler	3.0	34.2	29.4
1	Plant Hire	2.7	106.1	61.5
2	Food Wholesaler	2.7	89.9	23.6
2	Brushware Warehouse	2.7	78.0	4.5
2	Mixed Retail	3.0	55.2	20.2
2	Used Car Dealer	2.7	33.3	22.5
2	Electrical Wholesaler	2.7	100.4	14.3
2	Bank	3.0	1.8	0
3	Metal Distributor	3.6	92.5	26.8
2	Wine & Spirit Merchant	2.9	170.1	8.9
3	Warehouse	2.8	90.7	6.1
4	Mixed Retail	2.7	47.1	–
5	Canvas Goods Retailer	2.9	71.8	–
4	Carpet Retailer	3.4	86.1	–
4	Butcher	3.0	47.8	–
2	Hardware Retailer	2.7	53.8	–
2	Automotive Dealer	2.8	37.6	39.1

Flood Depth Range: 3.66 m to 4.57 m

SOURCE	TYPE	FLOOD DEPTH	DAMAGE	
		m	Direct $/m^2$	Indirect $/m^2$
2	Metal Distributors	4.0	92.6	5.9
2	Bottle Merchants	3.7	9.8	6.4
2	Veterinary Wholesaler	3.7	12.2	52.2
2	Electrical Wholesaler	4.0	141.2	26.9
2	Tyre Sales	4.0	10.1	6.0
3	Automotive Spare Parts Wholesaler	3.8	200.9	58.4
2	Machinery Sales	3.7	31.8	16.1
2	Grocery	4.0	55.7	53.8
1	Retailer & Wholesaler	4.0	308.6	53.8
2	Outboard Motor Retailer	3.7	25.1	7.2
2	Textile Importer	3.7	183.4	36.7
2	Tyre Wholesaler	3.8	25.3	10.4
2	Tyre Importer	3.7	32.3	77.5
2	Electrical Wholesaler	4.2	521.0	61.0
2	Produce Merchant	3.7	46.3	5.0
2	Food Wholesaler	3.7	64.0	2.7
2	Variety Wholesaler	3.7	88.4	7.6
2	Metal Retailer	3.7	70.4	43.1
2	Sales Consultants	4.3	28.4	0.8
3	Wholesale Market	4.4	318.1	8.5
4	Tyre Retreads	3.7	34.4	–
2	Food Wholesaler	3.7	49.7	1.2
4	Hotel	4.0	16.1	–
5	Swimming Pool Sales	3.7	44.9	–
4	Importer	4.4	36.7	–
4	Office Furniture Retailer	3.7	105.3	–

Flood Depth Range: 4.57 m to 5.49 m

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m^2$	DAMAGE Indirect $/m^2$
2	Importer	4.7	85.3	63.8
2	Pump Hire	4.6	33.7	134.6
2	Dolomite Sales	4.9	17.2	56.5
2	Automotive Parts Wholesaler	4.9	162.9	23.7
2	Hire Service	4.9	12.6	3.6
2	Tractor Sales	5.0	50.2	5.7
2	Food Wholesaler	4.9	235.3	3.1
3	Warehouse	4.7	146.2	1.9
2	Cordials Distributor	4.9	7.1	0.1
2	Insecticides Wholesaler	4.6	158.8	46.6
2	Bus Service	5.2	20.8	1.1
3	Timber Merchant	4.6	50.1	71.6
4	Hotel	4.6	32.7	-
4	Shop	4.6	144.2	-
4	Service Station	4.6	71.0	-
4	Boat Sales	4.6	7.2	-
4	Frock Salon	4.9	209.9	-

Flood Depth Range: 5.49 m and over

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m^2$	DAMAGE Indirect $/m^2$
2	Service Station	5.5	7.6	1.0
2	Chemical Importer	5.5	367.5	37.1
2	Steel Merchant	5.5	12.1	15.8
2	Motor Truck Sales	7.3	87.3	18.4
2	Plumbing Supplies	8.4	8.1	7.2
3	Caravan Sales	6.1	289.7	94.0
2	Car Sales	6.1	7.2	0
2	Food Wholesaler	5.8	28.6	33.7
2	Paint & Wallpaper Merchant	6.1	62.8	382.8
2	Timber Merchant	7.6	7.4	2.0
2	Electrical Appliances Wholesaler	5.5	180.9	80.0
2	Materials Handling	9.2	9.4	1.8
3	Tractor Sales	5.6	68.4	27.0
2	Electrical Retailer	6.7	134.6	67.3
2	Furniture Retailer	6.1	78.4	44.5
4	Car Parts Wholesaler	7.0	38.4	-
4	Shop	5.5	13.9	-
4	Skating Rink Operator	6.1	15.6	-
4	Truck Sales	8.8	126.5	-
4	Clothing Retailer	6.1	53.8	-
4	Gift Retailer	7.6	70.0	-
4	Service Station	6.1	8.5	-
4	Service Station	9.2	28.7	-
4	Clothing Retailer	6.1	68.9	-
4	Sporting Goods Retailer	15.9	72.8	-
4	Service Station	9.2	15.9	-
4	Food Ingredients Retailer	9.2	66.2	-
4	Supermarket Operator	7.0	218.8	-

Flood Depth Range: 0 m to 0.30 m

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m²	DAMAGE Indirect $/m²
1	Furniture Mfr	0.2	3.6	22.4
2	Printer	0.2	6.2	4.3
2	Film Production	0.1	21.5	71.8
2	Spring Mfr	0.2	0	1.0
2	Engineers & Mfr	0.1	1.2	114.9
2	Steel & Wire Products Mfr	0.2	19.2	9.6
2	TV Service	0.1	3.9	4.7
1	Chemical Mfr	0.1	2.4	0
2	Laminated Products Mfr	0.1	3.9	0
2	Fibrous Plaster Mfr	0.2	0.4	0
2	Truck Assembly	0.1	25.1	0
2	Safe Mfr	0.2	0	7.6
5	Printer	0.2	5.4	-

Flood Depth Range: 0.31 m to 0.91 m

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m²	DAMAGE Indirect $/m²
2	Tyre & Rubber Mfr	0.8	4.3	0
1	Furniture Mfr	0.8	15.3	35.8
2	Window Electronics Mfr	0.8	122.0	35.8
2	Toolmaker	0.8	6.9	16.0
2	Bakery	0.6	0.5	0
2	Motor Dealer	0.6	1.9	8.6
1	Sport Goods Mfr	0.5	16.4	10.2
2	Bitumen Emulsion Mfr	0.6	11.1	10.8
2	Valve Mfr	0.6	6.0	6.7
2	Panelbeater	0.8	3.6	0
1	Canvas Goods Mfr	0.8	1.7	0
2	Light Mfr	0.5	11.9	0
2	Wire & Nail Mfr	0.3	13.2	4.1
2	Flour Packer	0.6	1.0	0
2	Respiratory Equipment Mfr	0.8	2.7	8.1
2	Plaster Board Mfr	0.4	5.4	0
2	Laundry	0.5	1.1	0
2	Plumber	0.3	1.1	0.9
1	Fruit Juice Mfr	0.6	6.4	2.7
2	Furniture Mfr	0.5	5.4	10.8
2	Garage Door Mfr	0.7	1.6	9.8
1	Engineers & Mfr	0.4	7.2	0
2	Engineers, Pump Mfr	0.6	0.8	1.1
2	Panelbeater	0.5	1.6	0
2	Rubber Mfr	0.6	4.8	4.5
2	Saddlery	0.3	4.3	0
2	Food Mfr	0.7	10.8	0
2	Aluminium Fabricator	0.5	10.0	38.4
2	Plumbing Contractor	0.8	20.5	5.2
2	Panelbeater	0.4	0.4	2.4
2	Boat Builder	0.5	1.2	2.7
2	Electrical Engineers	0.5	12.2	6.8
2	Fibreglass Mfr	0.8	4.7	20.2
2	Concrete Products Mfr	0.3	7.3	0

Flood Depth Range: 0.31 m to 0.91 m - Continued

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m²	Indirect $/m²
3	Civil Engineers	0.8	129.0	0
2	Furniture Mfr	0.5	4.5	6.8
3	Shipwright	0.8	14.1	0
3	Interstate Transport	0.8	691.4	23.0
2	Shop Fitter	0.3	7.5	0
2	Gravel Producer	0.5	61.1	79.8
3	Ship Builder	0.6	19.5	1.5
5	Printer	0.6	76.0	-
4	Motor Parts Mfr	0.6	15.4	-
5	Printer	0.3	26.8	2.5
4	General Engineers	0.8	49.1	-
5	Printer	0.9	14.3	-
4	Clothing Mfr	0.6	56.0	-
5	Printer	0.8	46.6	-
2	Drilling Equipment Mfr	0.6	11.2	18.6
3	Heavy Engineering	0.6	1.4	0.5

Flood Depth Range: 0.91 m to 1.37 m

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m²	Indirect $/m²
2	Interstate Transport	1.0	8.1	4.5
1	Sawmill	1.1	9.1	40.4
2	Gear Mfr	0.9	9.5	0
1	French Polisher	0.9	22.6	0
2	Hot Mix Mfr	1.1	73.2	8.6
2	Mower Assembly Plant	1.1	17.9	10.8
2	Interstate Carrier	1.2	0.6	6.5
2	Carrier	0.9	2.7	179.4
2	Plastic Fabricator	1.0	12.7	16.6
2	Carrier	1.2	66.4	8.9
2	Recond. Commercial Refrig.	1.0	23.2	15.4
2	Canvas Goods Mfr	0.9	8.6	4.3
2	Builder	1.1	33.4	0
1	Furniture Mfr	0.9	20.2	0
2	Safety Equipment Mfr	0.9	65.2	10.8
2	Chemical Mfr	1.0	44.1	215.3
2	Water Treatment Engineers	0.9	79.7	36.9
2	Bait Processor	1.2	12.2	4.6
2	Petroleum Equipment Installer	0.9	11.2	0
2	Air Cargo Forwarders	1.2	17.9	215.3
2	Steel Fabricator	0.9	1.8	1.8
2	Spring Mfr	1.3	2.9	5.1
2	Engine Reconditioners	1.0	8.1	5.4
2	Engine Machining	1.1	12.6	25.6
2	Civil Engineers	1.2	119.6	38.0
2	Boat Builder	1.1	92.9	1.8
2	Fruit Juice Mfr	0.9	0.5	0.8
2	Signwriter	0.9	92.9	3.6
3	Refrig. Equipment Mfr	0.9	44.1	97.6
5	Typesetter	0.9	24.8	-
5	Printer	0.9	61.9	-
5	Printer	4.0	24.2	-
4	Wrought Iron Products Mfr	1.0	9.8	-
5	Electroplater	1.2	53.8	-

Flood Depth Range: 0.91 m to 1.37 m - Continued

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m^2$	Indirect $/m^2$
4	Stock Feed Mfr	1.2	21.5	-
5	Printer	1.1	64.6	-
4	Furniture Mfr	0.9	49.6	-
4	Panelbeater	1.2	11.8	-
4	Spring Mfr	0.9	5.2	-
3	Can Mfr	1.2	104.5	44.6

Flood Depth Range: 1.37 m to 1.83 m

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m^2$	Indirect $/m^2$
2	Motor Engineers	1.5	15.1	12.9
1	Hat Mfr	1.5	13.5	33.6
1	Fruit Juice Mfr	1.5	13.9	23.9
1	Furniture Mfr	1.5	54.6	46.1
2	Marine Instrument Mfr	1.7	86.4	29.1
2	General Engineers	1.4	5.7	20.1
2	Cabinetmaker	1.5	31.5	10.8
2	Signwriter	1.5	20.1	0
2	Lightning Specialist	1.4	19.7	7.2
1	Pharmaceutical Mfr	1.5	35.7	0
2	Plant & Equipment Mfr	1.6	2.4	0
2	Steel Products Mfr	1.5	4.6	0.5
2	Spring Mfr	1.4	2.2	2.7
2	Civil Engineers	1.5	67.8	129.2
2	Fastener Mfr	1.4	11.8	10.8
2	Panelbeater	1.5	7.8	3.6
2	Plumber	1.5	32.9	12.3
2	General Mfr	1.7	4.3	0
2	Cycle Mfr	1.5	40.4	32.3
2	Wheel Mfr	1.7	38.9	30.0
2	Panelbeater	1.5	14.3	10.8
3	Furniture Mfr	1.7	10.1	3.2
3	Chemicals Mfr	1.7	7.1	37.8
3	Glass Mfr	1.4	68.7	40.4
5	Paper Bag Mfr	1.7	41.4	-
5	Printer	1.5	11.5	-
5	Furniture Mfr	1.5	44.9	-
5	Printer	1.4	53.8	-
4	Furniture Mfr	1.5	26.9	-

Flood Depth Range: 1.83 m to 2.29 m

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m^2	DAMAGE Indirect $/m^2
2	Wire Weaver	1.9	8.9	0.4
2	Plumber	2.1	26.0	82.8
2	Motor Engineer	1.8	2.0	0
2	Security Systems	1.8	241.4	61.5
2	Umbrella Mfr	2.1	84.3	35.8
3	Chemical Mfr	2.1	56.8	19.9
2	Concrete Products	1.8	240.4	0
2	Paint Mfr	2.8	13.5	12.9
2	Panelbeater	2.8	4.0	7.2
2	Vehicle Repairs	2.8	183.0	30.8
2	Cooperage	1.8	11.6	2.5
1	Furniture Mfr	2.0	9.9	8.3
2	Mfr, Engineers	2.0	6.4	1.0
2	Transformer Mfr	2.1	23.4	18.0
2	Electrical Mfr	1.8	142.4	21.5
2	Panelbeater	1.8	4.1	6.8
2	Oil Field Service	2.1	29.4	10.8
2	Air Cond. Contractor	1.8	50.5	13.5
2	Wood Mouldings Mfr	1.8	83.5	10.7
3	Fibreboard Mfr	2.1	71.5	18.2
2	Sawmiller, Mfr	2.0	42.5	15.6
2	General Carrier	2.0	122.2	20.2
2	Motor Repairs	2.1	30.1	8.1
2	Concrete Placer	2.3	13.7	0
2	Welder	2.0	16.1	2.2
1	Boat Builder	2.8	7.2	10.8
3	Metal Processor	1.8	31.9	46.1
5	Printer	2.0	63.1	-
4	Furniture Mfr	2.0	24.6	-
5	Printer	2.1	19.6	-
4	General Engineers	1.8	37.2	-
5	Joiner	1.8	32.3	-
5	Wire Mfr	2.0	20.1	-
5	Panelbeater	1.8	35.8	-

Flood Depth Range: 1.83 m to 2.29 m - Continued

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m^2	DAMAGE Indirect $/m^2
5	Printer	1.8	161.9	-
4	Glass Fibre Mfr	2.0	667.9	-
5	Printer	2.1	123.8	-
5	Printer	2.2	89.7	-
4	Canvas Goods Mfr	2.0	83.3	-
4	Clock Repairs	2.0	40.2	-

Flood Depth Range: 2.29 m to 2.74 m

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m^2	DAMAGE Indirect $/m^2
2	Motor Repairs	2.6	14.9	22.4
2	Real Estate Developer	2.4	31.2	0
1	Lithographer	2.4	150.7	59.8
1	Furniture Mfr	2.4	30.7	0
1	Flour Miller	2.4	39.4	0
2	General Mfr	2.6	25.7	59.8
1	Furniture Mfr	2.4	48.4	64.6
1	Stainless Steel Mfr	2.4	9.8	27.6
2	General Mfr	2.4	207.0	276.0
2	Steel Fabricator	2.4	8.3	12.9
2	Automotive Engineer	2.4	38.5	57.4
2	Food Processor	2.4	53.0	6.5
2	Aluminium Fabricator	2.6	2.2	8.1
2	Building Contractor	2.4	22.3	14.3
2	Brake Repair Specialist	2.3	27.3	10.4
2	Engine Packing Mfr	2.4	121.1	53.8
2	Plywood Mfr	2.4	73.4	9.8
4	Electrical Contractor	2.4	75.3	–
4	Plastic Fabricator	2.4	12.2	–
5	Hardboard Mfr	2.6	19.7	–
5	Furniture Mfr	2.4	14.0	–
4	Kitchen Furniture Mfr	2.3	57.7	–
4	Printer	2.4	61.0	–
4	General Engineering	2.3	269.1	–
4	General Engineering	2.4	64.6	–
4	Designer & Printer	2.4	107.6	–

Flood Depth Range: 2.74 m to 3.66 m

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m^2	DAMAGE Indirect $/m^2
2	Freight Forwarder	2.9	5.3	3.1
1	Steel Tube Mfr	2.7	112.3	80.7
2	Mfrs & Engineers	3.0	3.9	19.4
2	Engineers	3.0	878.4	150.7
1	Stock Feed Mfr	2.7	35.1	33.9
1	Venetian Blind Mfr	3.0	11.1	31.6
2	Motor Repairs	2.9	9.3	2.5
2	Shop Fitter	3.4	17.9	5.4
2	Spectacle Case Mfr	3.4	13.0	7.6
2	Mechanical Engineers	3.4	22.4	16.6
3	Plastic Mfr	3.5	142.1	118.4
1	Plywood Mfr	3.0	3.3	3.1
2	Transport	2.9	9.9	0
2	Brewery	3.1	30.9	6.0
2	Asphalt Producer	2.8	54.5	23.0
3	Ship Repairs	3.0	278.5	0
3	Office Furniture Mfr	3.4	91.5	30.1
3	Biscuit Mfr	3.0	57.7	17.0
5	Printer	3.4	56.1	–
4	Earth Mover	2.7	26.2	–
4	Steel Fabricator	3.4	5.7	–
5	Joinery	3.0	67.3	–
5	Mineral Earth Processor	2.9	68.6	–
5	Furniture Mfr	2.7	66.0	–
5	Soft Drink Mfr	3.0	18.5	–
5	Steel Fabricator	3.4	35.8	–
2	Plastic Bag Mfr	3.4	124.9	21.5
4	General Engineers	2.7	10.8	–
5	Printer	3.4	64.0	–
4	Furniture Mfr	2.7	45.4	–

Flood Depth Range: 3.66 m to 4.57 m

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m^2	DAMAGE Indirect $/m^2
2	Mfrs & Engineers	4.0	35.7	51.0
2	Furniture Mfr	4.3	11.0	53.8
2	Automotive Engineer	4.0	12.1	16.1
3	Chemical Mfr	4.0	178.9	435.4
2	Trailer Mfr	4.0	17.2	5.4
1	Clothing Mfr	3.7	19.6	21.5
1	Tile Mfr	3.7	10.8	17.2
2	Muffler & Radiator Repairs	4.3	7.5	8.6
2	General Mfr	3.7	46.7	34.4
2	Shop Fitter	4.3	145.3	107.6
2	Steel Fabrication	3.8	3.8	3.9
1	Furniture Mfr	4.0	51.7	25.8
3	Builders Supplies Mfr	4.2	29.7	27.6
2	Auto Electrical Repairs	4.3	37.7	134.6
3	Fire Protection Engineers	4.0	220.0	180.5
2	Concrete Producer	4.0	322.9	0
2	Road Transport Specialists	3.7	13.7	6.9
2	Engine Reconditioners	4.1	15.4	12.6
2	Motor Repairs	4.3	17.9	2.8
3	Food Mfr	4.4	136.9	139.2
2	Ink Mfr	3.7	62.9	9.1
3	Removalist	4.1	262.0	1.3
4	Furniture Mfr	4.3	25.3	–
4	Paper Products Mfr	3.7	53.8	–
2	Plywood Mfr	4.3	228.2	464.6
2	Concrete Producer	4.0	331.2	331.2
5	Clothing Mfr	3.7	76.2	–
5	Pipe Mfr	3.7	94.2	–
4	TV Repairs	3.7	215.3	–
4	General Engineers	4.4	158.1	–
5	Tyre Retreaders	4.3	67.3	–
5	Patternmaker	3.7	20.1	–
5	Clothing Mfr	3.8	10.2	–
4	Roof Rack Mfr	3.7	29.1	–

Flood Depth Range: 3.66 m to 4.57 m – Continued

SOURCE	TYPE	FLOOD DEPTH	DAMAGE Direct $/m^2	DAMAGE Indirect $/m^2
4	Switchboard Mfr	4.3	15.8	–
4	Chemical Mfr	4.3	18.9	–
4	Printer	4.3	3.2	–
4	Caravan Mfr	4.1	11.6	–
3	Agr Machines Mfr	4.3	50.3	4.5
3	Laboratory Supplies Mfr	4.0	42.8	11.6

Flood Depth Range: 4.57 m to 5.49 m

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m^2	DAMAGE Indirect $/m^2
2	Semi-trailer Mfr	4.9	3.6	2.9
1	Rubber Products Mfr	4.6	271.3	53.8
3	Furniture Mfr	4.9	52.7	41.7
1	Interstate Haulier	5.0	217.3	13.5
2	Semi-trailer Mfr	4.9	83.2	49.0
2	Soap Mfr	4.9	8.4	16.1
2	Paper Converter	4.6	67.2	0
2	Paper & Cardboard Recycler	4.6	30.4	6.1
2	Earth Moving Contractor	4.9	430.6	403.7
2	Fibreglass Mfr	4.6	16.8	5.4
3	Transport	5.2	152.5	203.8
2	Sand Processor	5.0	25.9	13.3
2	Laboratory	5.0	114.5	13.9
2	Broom Mfr	4.6	12.7	4.7
3	Plumber & Mfr	5.1	56.5	30.0
3	Semi-trailer Mfr	5.3	3.9	5.8
1	Glass Fabricator	4.9	124.8	0
3	Heavy Engineering	5.5	122.1	105.2

Flood Depth Range: 5.49 m and over

SOURCE	TYPE	FLOOD DEPTH m	DAMAGE Direct $/m^2	DAMAGE Indirect $/m^2
2	Transport	5.5	137.9	430.6
2	Spring Mfr	6.1	13.5	26.9
1	Brassware Mfr	5.5	41.6	16.8
2	Motor Car Mfr	5.5	39.5	179.4
2	Transport	6.1	39.5	3.6
2	Refrigerator Mfr	5.5	17.2	6.4
1	Building Components Mfr	6.1	98.7	179.4
3	Mfrs & Engineers	7.2	63.9	18.7
2	Hose & Pulley Mfr	9.2	8.3	14.3
2	Metals Processor	11.7	30.5	107.6
2	Earthmoving Contractor	5.5	53.8	45.7
2	Diesel Pump Repairs	9.2	5.1	1.9
2	Contractors	6.1	79.7	21.5
2	Vegetable Packer	6.7	38.8	0
2	Refrigerator Mfr	6.1	119.9	91.9
2	Concrete Supplier	5.8	5.9	0
2	Transport Engineers	5.5	26.2	1.5
4	Displays Fabricator	6.1	5.9	-
1	Soft Drink Mfr	5.5	16.5	8.3
4	Panelbeater	5.5	14.6	-
4	General Engineers	6.1	36.2	-
4	House Renovator	18.3	18.8	-
5	Furniture Mfr	6.1	33.6	-
4	Food Processor	7.6	86.1	-
4	Panelbeater	6.1	2.2	-
4	Precast Concrete Mfr	6.1	15.4	-
4	General Engineers	5.5	47.8	-
4	Wire Products Mfr	6.1	75.3	-
3	Paint Mfr	6.5	121.4	10.9

SMALL BUSINESS DAMAGE

DATA FROM QUEENSLAND DISASTER WELFARE COMMITTEE

Business	Employees	Flood Depth	Direct $ Damage
Auto repairs	0	12	5 000
Fruit, vegetables, shop	0	12	580
T.V. Repairs	3	60	6 000
Washing m/c repairs	0	50	11 000
Supermarket	17	20	75 000
Nursery (plants)	0	10	8 900
Mixed business	0	20	6 700
Mixed business	2	8	8 300
Marking equipment	2	20	7 680
Soil plane etc. repairs	0	30	6 350
Musical instruments	2	7.5	32 500
Hairdresser	3	above floor	540
Snack bar, delicatessen	2	15	27 300
Pharmacy	1	20	11 500
Grocery	0	9	2 800
Pastry shop	0	11	1 500
Grocery	2	8	12 500
Mixed business	1	8	4 000
Electrical eng.	8	11	9 000
Panel beating	1	above floor	1 428
Milk bar, delicatessen	3	30	10 000
Pharmacy	2	8	10 000
Milk bar, delicatessen	3	abt 6	3 550
Pharmacy	4	above floor	31 000
Hairdresser	0	14	100
Service Station	–	above floor	6 000
Butcher	1	above floor	8 300

THE QUEENSLAND COAL BOARD

TELEGRAMS: "COALBOARD BRISBANE"
TELEPHONE: 24 0414

169 MARY STREET
BRISBANE, 4000

BOX 384, G.P.O., 4001

27th February, 1975.

Mr. S.K. Stephens,
Project Manager,
Brisbane River Flood Investigation,
Snowy Mountains Engineering Corporation,
Box 356, P.O.,
<u>COOMA NORTH, 2630.</u>

Dear Sir,

<u>Brisbane River Flood Damage</u>

In reply to your letter dated 28th January, 1975, your ref. A74/21, and further to my recent discussions with your Mr. Aitken relative to the Brisbane River Flood Damage, I now enclose a completed plan (with explanatory notes) indicating the sites of the coal mines affected by this flood.

The following information is given in response to the request on page 2 of your letter, i.e. –

(a) <u>For the highest flood</u> –

1. Little or no damage would result to the now remaining mines on the Ipswich field, with the exception of the Westfalen Mine at which flood gates are now being installed. In consequence, the estimated cost of flood damage would be minimal. However, the efficacy of these gates would need to be adequately tested.

(b) <u>For the 1974 flood</u> –

1. One mine only – Westfalen – is capable of being reinstated. This is now underway and will cost an estimated $1.5 million to bring it back to the same production level existing prior to the flood.

   The other three mines – Haighmoor, Rylance Abermain/Moreton and Aberdare No.8 – are considered incapable of being reinstated. However, as all of these three Companies are also operating opencut mines which were not affected, and in view of the fact that their underground mines which were lost were operating at a loss, no loss of profits has occurred.

   The Board, taking cognisance of the effect of the flood with regard to production and the inability to meet contractual commitments, cancelled all contracts and increased the price of coal accordingly with the result that the Companies so affected are now operating profitably.

.../2

Explanation of Information marked on The City of Ipswich Plan for The Snowy Mountain Authority

(1) Haigmoor (Tivoli) Colliery

Mine flooded through new entry made nearer to the Bremer River at a lower level. At this date no actual work has been carried out to dewater the mine for re-opening.

The washing plant (Jig) unaffected by the flood is in operation washing coal from an open cut mine in the Rosewood district (a joint venture with the Normanton Colliery Company).

(2) Rylance Abermain No. 1

The mine in this area had been abandoned many years prior to the flood. The washing plant (Jig) shown at this point has been moved to this site from the flooded surface area at Moreton Extended Colliery. The plant is not in operation at this date but most probably will be utilised to wash coal from an open cut mine in the Rosewood district when in operation.

(3) Rylance Abermain No. 2

Again in this case the flooding did not reach the entrance of this Colliery. This Company in recent years had taken over the Moreton Extended mine and had made an interconnection to this mine's workings and, with the flood waters entering the Moreton Extended Colliery, both mines were flooded. No reclamation work has been started at this time. The washing plant (tables) unaffected by the flood is still in operation washing coal produced at the Rylance No. 3 area.

(4) Moreton Extended

This mine was not in production during the flood but a connection recently made from the Rylance Abermain 2 mine was winning the coal from this area. The flood waters entering the Moreton Extended Mine flooded both Collieries and it has been considered that the reserves are not large enough to warrant the cost of dewatering and reclaiming the mine workings.

The washing plant shown was within the flood area and has since been re-sited in the Rylance Abermain No. 1 area.

(5) Aberdare 8

Prior to the 1974 flood this mine area had already experienced problems with underground roof movement causing flooding so that new workings were being driven to skirt these abandoned workings. The 1974 flooding entered these new workings and no move has been made by this Company todewater the mine.

(6) Rylance 3

No flooding at this mine. The underground workings had already been abandoned because of lack of reserves in this area.

A small washing plant (tables) operates at this site washing coal produced from a nearby open cut mine.

(7) Aberdare Whitwood

Also an abandoned underground mine site which was not affected by the flood.

The washing plant (Jig) on this site is at present washing coal produced from 3 open cut mines reasonably adjacent to the site.

-1-

-2-

(8) Rhondda No. 1

This old Colliery with limited future prospects was also clear of the 1974 flood waters.

The washing plant (Jig) washes coal from the Rhondda 1. mine, the W. Haenke mine and also coal from the Company's open cut mines.

(9) W. Haenke

The most recently opened Colliery by the Rhondda Company also unaffected by the 1974 flood.

(10) Sunrise

This old Colliery abandoned many years ago by a previously owned Company, was clear of flooding in 1974.

The washing plant has been retained in full working condition and is at present on lease to the Rylance Company who are washing coal produced from an open cut mine in the Rosewood district.

(11) Westfalen 2. & 3.

The No. 3 mine which first produced coal in 1968 was opened up to replace the No. 2 mine which has been abandoned. This new mine was completely inundated during the 1974 flood causing a great deal of damage to the tunnel entrance and also the air-shaft. The owners have since carried out a reclamation program for this mine by putting down a drill hole to the workings and installing in it a submersible pump to dewater the mine. It is expected that 12 months of pumping will be required to completely recover the mine. In the meantime, tunnels are being driven around the perimeter of the workings to maintain production and ensure that entries are available to the production area when the mine is dewatered.

The No. 3 washing plant was also in the flooded area and had only previously been brought into commission washing part of the production from the underground and a nearby open cut mine. The balance of the production was still being washed at the No. 2 mine area. Since the flood this same arrangement is still in operation regarding the two washing plants.

(12) The Southern Cross, Box Flat and New Hope Collieries and their respective washing plants were all clear of the 1974 floods but are outside of the City of Ipswich area.

- 2 -

2. As to the estimated additional cost in providing coal from other areas, this amounted to some $5,000,000 which figure includes extra freight costs, handling charges and the higher price of coal at pithead.

3. The effect on the work force by the closure of the affected mines amounted to 312 men of whom 212 were placed at other mines in the West Moreton region - Ipswich and Rosewood - with the balance of 100 being found positions in the Central Coalfields and in positions outside the industry.

Yours faithfully,

[redacted]

Chairman.

Encs.

PART 2

PRODUCTION OF FLOOD MAPS

PART 2 - PRODUCTION OF FLOOD MAPS

CONTENTS

PART 2 - PRODUCTION OF FLOOD MAPS

1. INTRODUCTION

The Survey Office of the Queensland Department of Lands was responsible for the preparation of the basic contour maps required for plotting of the flood inundation lines and for the production of the final flood maps for the Brisbane and Bremer Rivers. The following report describing this work was prepared by the Survey Office for inclusion in the overall report on the Brisbane River Flood Investigations. Since the preparation of the report by the Survey Office further work has been completed on the maps and this has been included in Section 6 which was prepared by Snowy Mountains Engineering Corporation (SMEC).

2. AERIAL PHOTOGRAPHY

When it became apparent on the 27 and 28 January 1974, that the Brisbane River was going to flood to its highest level this century, efforts were made to determine the area inundated by the flood. Arrangements were made for the Royal Australian Air Force Reconnaissance Squadron based at Amberley to photograph the flood from the air. A commercial aerial photography firm was also engaged to take air photographs of the flood. The commercial firm which is based at the Archerfield Aerodrome, started to photograph on the afternoon of Tuesday, 29 January, when the danger of the aerodrome being flooded diminished.

The photographs taken by the R.A.A.F. on the 28 January, when the flood was at its greatest height in Brisbane, have proved to be extremely useful. The R.A.A.F., because of cloud cover, had to fly at heights varying from 2 000 ft to 20 000 ft to get any worthwhile photography. The photography was effected as reconnaissance photography and as such it did not cover the whole of the Brisbane and Ipswich areas, although a large part of the area was covered by both high and low-level photography. The photography comprised 8 runs at 20 000 ft and 9 runs varying between 2 000 ft and 4 000 ft - in all a total of 388 prints. This photography was used as a complementary source of information to both the commercial photography and the various field reports on flood heights received by the Survey Office.

The commercial aircraft, in the few hours of suitable daylight left on the afternoon of the 29 January, flew 7 runs across the city. As weather permitted, the firm, over the next 10 days, completed the photographic coverage of virtually all of the built-up areas of Brisbane and Ipswich, and the more urbanised areas in between the two cities. This coverage comprised 56 runs of photography, totalling 2 871 prints. This photography, because of overcast skies, was flown at 2 500 ft to 3 000 ft. Although conditions for photography were far from ideal, and flying conditions were difficult, it provided a record of the flood. It showed the area inundated at the time of the photography and, because it was flown at low level, it recorded debris marks, especially on roadways. Since the photography was carried out after the peak of the flood had paased, the debris marks formed a good guide to the height reached by the flood.

The aerial photography carried out by the R.A.A.F. and the commercial firm proved invaluable in determining the area inundated by the flood.

3. REPRODUCTION OF OLD FLOOD MAP

The Survey Office decided to reprint the old flood map produced by the Bureau of Industry in 1933 for the then built-up area of Brisbane, with the 1974 flood limits superimposed on it. Work on the map started as soon as the flood receded. Surveyors checked flood limits in certain areas, in addition to air-photo information. Within one month the Survey Office produced a map entitled 'Flood Map of Brisbane and Suburbs'. This map shows:

(a) The extent of the area inundated during the 1974 Australia Day floods (superimposed in a full red line).

(b) Flood inundation lines for particular heights (in feet) on the Port Office Gauge, as embodied on the original 1933 map

The Port Office Gauge was until 1 January, 1975 the gauge to which all floods in the Brisbane River in Brisbane were referred for purposes of comparison.

A recent edition of the Brisbane Street and Road Map was used as a base for the flood map. While contour information in 1933 in the flood affected areas was not complete, inundation lines enclosing areas flooded at particular heights on the Port Office Gauge proved to be reasonably accurate.

The line enclosing the area inundated by the January 1974 flood was deduced from the air photography and shown on the map as a continuous red line. Field inspection of certain areas to clear up matters in doubt was made by the field staff to supplement the information from the air photography. The areas inundated by floods of a particular height on the Port Office Gauge were shown by colour bands. Besides the Cartographic Section, the preparation of the map involved the Lithographic and Photographic Sections of the Survey Office in such work as scribing, the preparation of colour masks, photography and colour proofing in readiness for plate making before being presented to the Queensland Government Printer for final printing. Special explanatory notes were printed on the side of the map - the Brisbane City Council and the Co-ordinator General's Department assisted in the preparation of the notes.

As stated earlier, copies of the map were available for sale about a month after the decision was made to produce it. The short time taken to produce this map was a result of the willingness of all concerned in its production to co-operate. A leaflet requesting advice about any errors found in the map was printed and issued with every map sold. Only two letters advising of errors were received and these indicated errors of a minor nature. The map had a ready sale to the public and the large number sold indicated that the public valued the information provided on the map.

4. INITIATION OF FLOOD INVESTIGATION AND ASSESSMENT

During the week following the flood, the Queensland Premier, the Honourable Mr Bjelke-Petersen, stated that the January 1974 flooding, in those streams where it reached a record or a near record height, would be recorded on maps. Very high floods were recorded in most streams in the Moreton Region from Lockyer Creek, a tributary of the Brisbane River, south to the State Border and west to the ranges. Gathering the data on these floods was considered to be a major task and the Co-ordinator General's Department set up a Flood Recording Co-ordination Committee. This Committee allocated the gathering of data in particular streams to various Government Departments, the Brisbane City Council and a firm of Consulting Engineers.

The Minister for Urban and Regional Development, Mr T. Uren, offered to make money available to Queensland through the Cities Commission to allow SMEC to help in the Brisbane River flood assessment. The Flood Recording Co-ordination Committee agreed that there were two problems that were beyond the resources of State Government Departments on which to start work quickly, if they were to be engaged on flood data collection.

One problem was the realistic assessment of the damage caused by the flood. In assessing the benefit which might accrue as a result of flood mitigation storage to be provided in the proposed Wivenhoe Dam, the Co-ordinator General's Department had used inundation information based on the 1893 flood, on which to derive damage estimates. The estimates derived were deliberately conservative. The January 1974 flood provided an opportunity to derive a stage-damage curve, at least up to the height of the flood, based on actual experiences. The determination of a stage-damage curve based on an actual flood would allow a more confident assessment of the economics and desirable size of the flood compartment to be built into the proposed Wivenhoe Dam.

The other problem was that the Brisbane and Suburbs Flood Map 1974 covered only a limited part of the total flooded area in Brisbane and did not include the City of Ipswich or the Shire of Moreton at all. It was based on inadequate level and other information, and was out of date. New flood maps were required for the 1975 wet season, covering the length of the Brisbane River to Mt Crosby, and the length of the Bremer River from its mouth to upstream of Ipswich. SMEC was engaged by the Cities Commission to produce inundation lines for floods of various heights on the reference gauge from basic mapping and flood profile information supplied by Queensland Departments and Local Authorities. These inundation lines were to be used by the Survey Office to produce new flood maps. It was proposed that the new maps should comprise three sheets at 1:20 000 scale covering the whole of the Brisbane-Ipswich area affected by Brisbane and Bremer River floods.

Because of delays in determining the necessary technical inputs to the drawing of inundation lines, it later proved necessary to issue interim provisional flood maps for the 1975 wet season. These maps were issued on a restricted list to Councils, Police and State Emergency Services for use in a flood emergency and they will be withdrawn when the final maps are available.

Also, during the course of the work, it became obvious that the flood maps at 1:20 000 scale would be produced from maps produced at 1:10 000 scale on which the flood inundation lines would be first marked. Maps at 1:10 000 scale can be effectively used by Police and State Emergency Services in a flood emergency, while maps at 1:20 000 scale are of more limited use for this purpose. For planning purposes maps at 1:10 000 scale could be used instead of the 1:20 000 scale originally proposed, although more maps would be required. Towards the end of 1974 the Committee decided not to produce three flood maps at 1:20 000 scale but to produce eighteen flood maps at 1:10 000 scale so as to increase the usefulness of the maps.

5. BASIC MAP PRODUCTION AND HYDROLOGICAL STUDIES

The Survey Office was required by the Co-ordination Committee to produce quickly the contour maps for the area to be flood mapped to allow the SMEC work on analysis and assessment to get under way. Contours were to be produced in metres.

The Brisbane City Council was required by the Committee to produce updated flood envelopes for various heights on the reference gauge. Discussion took place on which gauge should be the reference gauge, and since Australia was to convert to Australian Height Datum in metres on 1 January 1975, it was decided to refer all flood heights to A.H.D. on the new Brisbane City Gauge. The Council requested that cross-sectional information along the Brisbane River to Mt Crosby should be obtained to allow a more accurate updating of its flood envelope curves. Initially it was proposed that the Department of Harbours and Marine should take soundings below water level at each cross-section location, and field survey would extend these sections up the banks. It was later decided that above water level contours would be preferable to cross-sections and the feasibility of photography to determine 1 m contours was investigated. This was considered to be both too costly and too time-consuming to implement, and another solution had to be found.

The Survey Office, after consideration of a number of alternatives, concluded that the maps could best be produced in three stages:

(a) The preparation of base maps. These should show ground detail and the standard grid so that references could be consistent between the maps used by the R.A.A.F. and Army and the maps used by the Police and State Emergency Service.

(b) The production of overlays (separate sheets which can be superimposed on the base map) showing contours.

(c) The preparation of a second set of overlays showing flood inundation lines for 2 m, 4 m, 6 m, 8 m and 10 m as recorded on the Brisbane City Gauge. It was decided that upstream of the Moggill Gauge this would provide inundation lines with too great a height difference and hence for maps upstream of the Moggill Gauge the reference gauge for inundation lines is the Moggill Gauge with a 3 m interval.

The City Council provided the conversion table to convert heights on the Brisbane City Gauge to heights on the Moggill Gauge, and indeed for other gauges on the river.

The preparation of the base maps presented no great problem. The orthophoto process which allows for the correction of aerial photographs to show ground detail in its true map position was used. The maps would need to be at a scale large enough to show individual residences, but not so large that the number of map sheets would be unduly inconvenient. A scale of 1:10 000 was adopted and this required making 18 orthophotos from the available 1:50 000 aerial photography to cover the flood affected areas from the mouth of the Brisbane River up as far as Mt Crosby and Ipswich. The Brisbane area had already been mapped and control for mapping as part of the State's standard 1:25 000 topographic mapping program had been obtained in the Ipswich area. The production of orthophotos could therefore be undertaken immediately. Unfortunately the photography was some years old - 1967 for the Brisbane area and 1971/72 for the Ipswich area - and considerable development had taken place in the flood plain of the Brisbane and Bremer Rivers since then. There was no time available to re-fly the area. Sheet boundaries were selected to agree with the available air photographs. Unfortunately it was not possible to produce the orthophotos on the standard mapping sheet patterns because of the photos available and for other practical and technical reasons.

The production of contours was a more difficult and time consuming task and alternatives were investigated to produce them in the time available. Two separate methods were adopted for different areas.

The Brisbane City Council (B.C.C.) has produced over the years sewerage maps covering a good deal of the city at a scale of 40 feet to 1 inch (1:480). These maps showed a large number of surveyed heights in feet on B.C.C. datum and 5 ft contours derived from these heights. It was found that about one-third of the total flooded area was covered on 850 Brisbane City Council sewerage maps. The Brisbane City Council supplied dyeline copies of these 850 maps to the Survey Office. The Survey Office drew on these maps 1 m contours on A.H.D. using specially compiled tables which took into account the conversion from feet to metres and the change in datum. These 850 maps, now showing 1 m contours, were reduced

from 1:480 to 1:10 000, i.e., the scale of the orthophotos. This required a 21 times reduction, and two microfilm exposures were needed for each Brisbane City Council map. These 1 700 reduced maps were fitted to the orthophotos by matching street intersections and other identifiable features, and the 1 m contours traced off. It should be noted that no record has been kept of the amount of filling and excavation, particularly on private property, that has taken place since compilation of these maps. The plans derived from them are therefore likely to be in error in some areas.

Over the two-thirds of the flooded area not covered by the Brisbane City Council maps, 2.5 m contours were drawn from aerial photographs at a scale of 1:25 000 using stereoplotting instruments. The contours were plotted on to transparent plastic map base sheets at a scale of 1:10 000. The contours were then prepared as an overlay for the orthophotos. Figure 1 shows the areas covered by the different types of contour information.

The orthophotos with 1 m and 2.5 m contours were supplied to the Brisbane City Council to allow them to prepare the updated flood envelopes. Copies of the orthophotos and the Brisbane City Council maps were also supplied to SMEC.

It became apparent towards the end of 1974 that the updated flood envelopes would not be produced in time to allow flood inundation lines to be determined by SMEC before the next wet season. Flood maps were needed before the 1975 wet season for use by the Police and State Emergency Service, should a flood occur in the Brisbane River. It was decided that the Survey Office should produce flood maps on a provisional basis for restricted distribution. The contoured orthophotos were used in conjunction with the then existing Brisbane City Council river flood envelope lines. These flood envelopes were plotted as inundation lines on the flooded areas by interpolation with the ground contours. In the lower reaches of the Brisbane River the flood inundation lines show the limits of flooding for each 2 m reading on the Brisbane City Gauge. In the higher reaches of the Brisbane River and in the Bremer River the inundation lines show the limits of flooding for each 3 m reading on the Moggill Gauge.

The area covered by the flood inundation lines is equivalent to a reading of 10 m on the Brisbane City Gauge. The January 1974 flood rose to about 5.5 m and the highest 1893 flood rose to about 8 m, on the Brisbane City Gauge. Street and suburb names were added to the maps and gauge height readings were placed on the inundation lines. Title strips, which included notes on production, height datum, accuracy limits, a gauge conversion table and a warning on the use of the maps were added to the orthophoto map. Printing plates were prepared from the flood prediction overlays and the gridded orthophotos and a limited number of black and white lithographs were printed to form a Restricted Provisional edition of the flood map. This Restricted edition was issued to Police, Councils, Government Departments and the State Emergency Service for use in a flood emergency.

The production of all mapping described above, in the restricted time available, was a huge task requiring considerable cartographic skill, initiative and dedication. It took 8 months to complete the project - including the preparation of base maps - and up to 38 draftsmen were employed at one time.

6. PRODUCTION OF FINAL FLOOD MAPS

As with the provisional flood maps, the Survey Office supplied all the map information required for the plotting of the final flood inundation lines. The actual plotting was carried out by SMEC who also prepared a short technical description of Brisbane River flooding for printing on the reverse side of the flood maps.

The final flood maps, which were in the course of preparation for printing by the Survey Office at the time of finalising this report, will show flood inundation lines for the 2 m, 4 m, 6 m, 8 m and 10 m floods at the Brisbane City Gauge. Upstream of the Moggill Gauge intermediate flood lines will be shown as dashed lines where the primary flood lines are widely spaced. In addition to flood heights the flood return periods will also be shown on the maps. In the case of the Bremer River, only return periods will be shown on the flood lines as the gauge heights are complicated by the effect of inflows from the Bremer River increasing backwater levels from the Brisbane River in the City of Ipswich.

KEY TO SHEETS

BRISBANE RIVER FLOOD MAP

SCALE 1:253440

1m Contours derived from B.C.C 1:480 Sewerage Plans shown.

2·5m Contours from Survey Office Photogrammetric Plots shown.

PHOTOGRAPHY

Sheets	Date
1, 2, 3, 4, 5, 6	1967
7, 9, 12	1971
8, 10, 11, 13, 14, 15, 16, 17, 18	1972

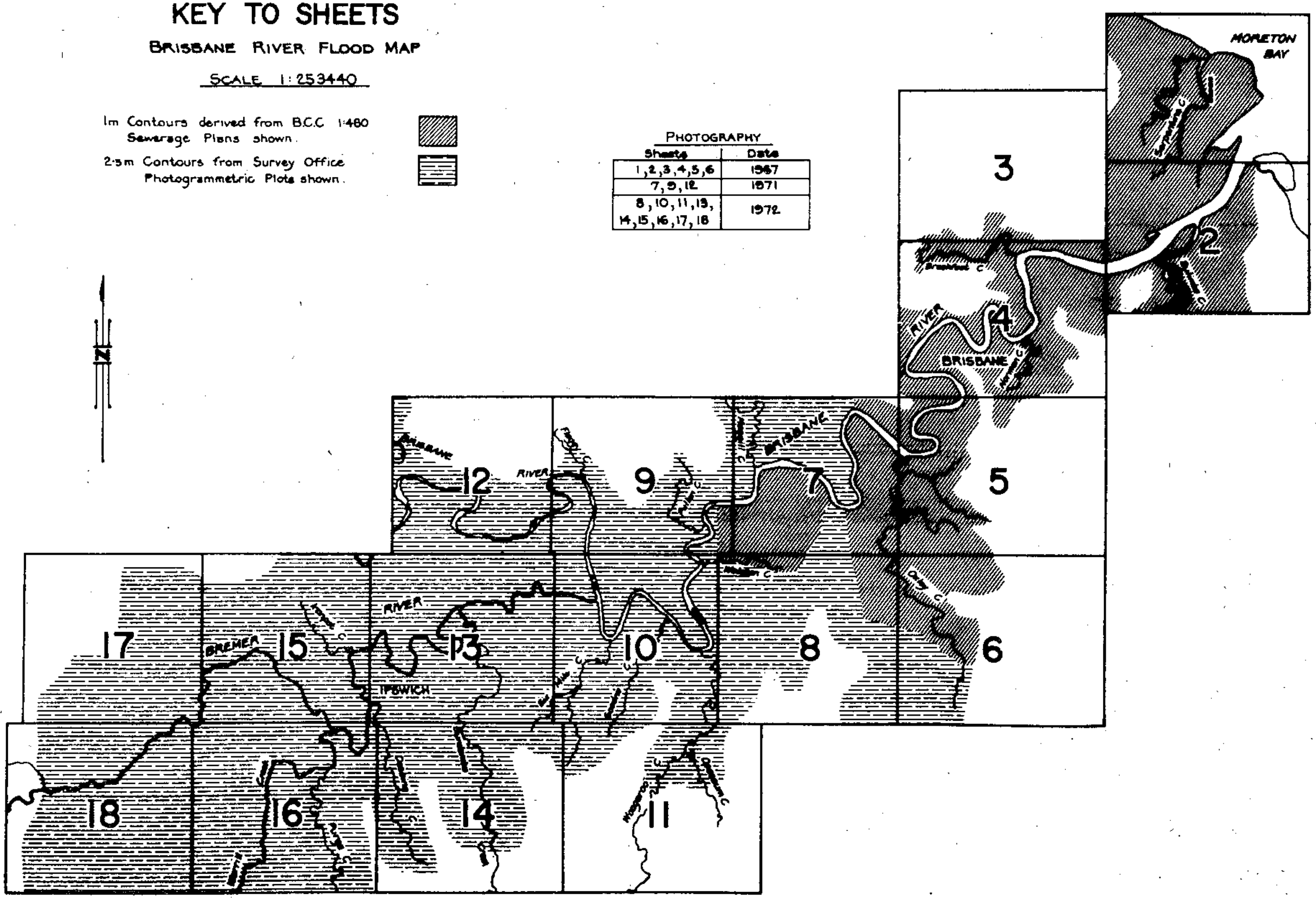

FIGURE 1

PART 3

FLOOD PROFILES AND FLOOD FREQUENCY

PART 3 - FLOOD PROFILES AND FLOOD FREQUENCY

CONTENTS

PART 3 - FLOOD PROFILES AND FLOOD FREQUENCY

1. INTRODUCTION

1.1 BRISBANE CITY COUNCIL CONTRIBUTION

The Water Supply and Sewerage Department of Brisbane City Council (B.C.C.) made available the master copies of the 40 feet to 1 inch sewerage detail plans on which much of the mapping for the overall study was based. These have been prepared using Imperial units on Brisbane City Council Datum, which is a low water datum. These contour plans had therefore to be converted to metric values on A.H.D. by the Survey Office for use in the flood investigations.

Apart from making the original contour plans available, the two main contributions by the B.C.C. to the flood investigations have been the derivation of the envelopes of maximum flood heights shown on Figure 6 of Part 1 of the Report and the flood frequency studies for the Brisbane River. The flood envelopes are referred to as flood profiles in Part 1 and are sometimes loosely called 'gradients'. They are not instantaneous flood gradients but indicate the maximum height to which the peak of the flood hydrograph rises as it progresses downstream from Mt Crosby through the City reaches of the Brisbane River at about 4 km/h.

The description of the work carried out by the B.C.C. (Sections 2, 3 and 4) on the Brisbane River flood envelopes and flood frequency studies was provided by the B.C.C. for inclusion in this part of the report.

1.2 SNOWY MOUNTAINS ENGINEERING CORPORATION CONTRIBUTION

Flood envelopes and flood frequency studies were also required for the Bremer River to produce the flood maps in the City of Ipswich. These studies were similar to the corresponding studies carried out for the Brisbane River by the B.C.C. and are described in Sections 5 and 6 of this part of the report.

2. EARLY ESTIMATES OF FLOOD ENVELOPES FOR THE BRISBANE RIVER

The derivation of flood envelopes for the Brisbane River has a history stretching back to the record flood of 1893; levels of this flood being surveyed in some detail between 1894 and 1896. The envelopes of successive floods were added as they occurred but this technique failed to produce a consistent diagram of the style of Figure 6 in Part 1 of this report due to the variable effects of the continued dredging in the lower reaches of the river for the improvement of navigation.

Following the 1931 flood for which an improved set of levels was obtained and for which a discharge measurement was made from Indooroopilly Bridge, serious attempts were made to simulate flood envelopes for the Brisbane River by applying Manning's equation to successive half mile reaches of the river. The simulation of maximum water level envelopes was started by the Special Committee of the Bureau of Industry in 1933. This Committee was set up to investigate dams for the water supply and flood mitigation of Brisbane and Ipswich and subsequently recommended the construction of Somerset Dam on the Stanley River, the major tributary of the Brisbane River, for this purpose. A backwater approach was adopted using the Harbours and Marine soundings of the first 26 km of the River. The investigation showed what effects the widening and deepening of the river for navigation purposes had on reducing flood levels. Friction and bend losses were taken into account but the bend losses were generally under-estimated. This had the effect of overstating the bed friction losses and so under-estimating the peak flows. The calculations had, necessarily to be done by hand which severely curtailed the number of feasible trials. The calculations were revised in 1947 using more detailed data and this process was repeated in 1955 following a small flood in the Brisbane River.

No further improvement was made, except in detail, until the 1968 flood was gauged at Centenary Bridge under very favourable conditions by the Irrigation and Water Supply Commission. Prior to this time the only gaugings had been made in 1931, 1951 and 1955 from Indooroopilly Bridge just downstream of a major bend in this river. The reversed flow on the inside of the bend made the interpretation of the results uncertain and it was finally decided to disregard them in favour of the more reliable measurements made at Centenary Bridge.

The 1968 flood was simulated, this time using a computer. Computers had by this time become commonplace, reliable, easy to program and economical to use. A package program based on Manning's equation was used. This program took velocity head into account and could account for bend losses. A major difficulty in this simulation, as in the earlier attempts, was the lack of river bed data above William Jolly Bridge at A.M.T.D. 26.5 km from which reliable cross-sections could be derived. The only river soundings available came from odd cable and pipeline crossings, together with some systematic soundings at river shoals and a vague centre of navigable channel sounding carried out by a commercial firm for its barges. A good deal of imagination had necessarily to be used.

In spite of these difficulties, the level data gathered from the 1968 flood were simulated closely but it was realised, subsequently, that it is relatively easy to manipulate roughness and bend loss coefficients to simulate the 20 or so flood levels collected from the 1968 flood over a river length of 90 km. This simulation, however, did allow the conditions of the first 26 km of the river, as disclosed by Harbours and Marine Department surveys, to be taken into account.

The envelopes of maximum flood levels were then extrapolated for higher and lower flood flows, using the constants derived from the simulation of the 1968 flood. This study suggested that the peak flow of the record 1893 flood had previously been under-estimated.

This suspicion was confirmed by further gaugings made by the Irrigation Commission from Centenary Bridge in the 1974 flood. Although the river velocity was so high that the current meters would not sink more than a few metres, the measurements clearly showed that previous ratings of high floods had been too low.

3. RECENT ESTIMATES OF FLOOD ENVELOPES FOR THE BRISBANE RIVER

In view of the magnitude of the damage caused in January 1974, it was desirable to simulate the flood closely to derive more reliable constants for use in extrapolating the higher and lower flood envelopes necessary for a thorough study of the damage potential of flooding in Brisbane and Ipswich. To facilitate the simulation, arrangements were made through the Co-ordinator General's Department for the Department of Harbours and Marine to carry out a detailed survey of the bed of the Brisbane River as far towards Mt Crosby as practicable and for the Survey Office of the Queensland Department of Lands to complete contour plans of the river banks and flooded areas, whilst the Brisbane City Council carried out a detailed survey of flood levels from Mt Crosby to the mouth of the Brisbane River.

It became apparent that this work would not all be completed in time for the flood simulation to be completed before the 1975 flood season. A preliminary version of Figure 6 was therefore drawn up on the basis of historical floods corrected by the simulations derived from the 1968 flood for the modern conditions of the dredged sections of the river and related to a standard ?, see p191 tide of 2.5 m from tide tables. This provisional version of Figure 6 was used in conjunction with the contours derived by the Survey Office to determine the inundation lines shown on the provisional 1:10 000 orthophotos produced by the Survey Office in December 1974 in time for the 1975 flood season.

When the depth survey of the Brisbane River was completed by the Department of Harbours and Marine in 1975, the Water Supply and Sewerage Department of the Brisbane City Council commenced the simulation of the 1974 flood. This river bed contour survey is the most complete, accurate and extensive available to date for the Brisbane River and allows cross-sections to be drawn at 100 m horizontal intervals if required. The only problem is the extent to which the river bed was altered during the flood. This factor is not known and the post flood bed configuration was necessarily adopted for the simulation.

More than 400 reliable flood levels were surveyed by Brisbane City Council along the banks of the Brisbane River in the 90 km length from Mt Crosby to the mouth of the river. The majority of levels were obtained from the Brisbane suburban areas between A.M.T.D. 14 km and 54 km. In addition, upwards of another hundred levels were obtained on the flood plain and along the suburban creeks in the area of the Brisbane River backwater. From a careful study of these levels, it is possible to deduce the peak level of the 1974 flood in all but a few special locations with an accuracy of ± 0.1 m.

The levels from the 1974 flood provided, for the first time on the Brisbane River, sufficient data to determine bend losses reliably. In particular, the levels at the Toowong bend were determined in considerable detail and the levels at 8 other bends were well defined. Sufficient levels were obtained from a further 8 bends to determine the bend losses with sufficient accuracy. The data from the remaining bends were sketchy due to the lack of buildings on which the levels could be detected. The losses at these bends are therefore less certain.

Bend losses were assumed to be of the form $kV^2/2g$ and the coefficient k was derived for each river bend from the measured data.

Basically the same package program was used as for the simulation of the 1968 flood but, in the meantime, the package program had been slightly modified. Manning's formula is used:

$$Q = \frac{1}{n} AR^{2/3}S^{\frac{1}{2}} \qquad (1)$$

$$= KS^{\frac{1}{2}} \qquad (2)$$

where Q = flow

A = area of section

R = hydraulic radius

S = friction slope

n = roughness coefficient

K = conveyance factor

The input and the output are in metric units and the program converts these to work in imperial units. The program also allows the width of the stream to be divided into six portions to allow different values of roughness to be applied to the bed, the banks and the flood plain of a river and to allow Manning's equation to be applied to each portion separately as follows:

For N portions, let:

$v_1, v_2 \dots v_N$ = mean velocities

$K_1, K_2 \dots K_N$ = conveyance factors

$A_1, A_2 \dots A_N$ = cross-sectional area

V = mean velocity of area

Thus:

$$v_1 = \frac{K_1}{A_1} S^{\frac{1}{2}} \qquad 3(a)$$

$$v_2 = \frac{K_2}{A_2} S^{\frac{1}{2}} \qquad 3(b)$$

$$v_3 = \frac{K_3}{A_3} S^{\frac{1}{2}} \qquad 3(c)$$

and $$Q = VA \qquad (4)$$

$$= \sum_{i=1}^{N} v_i A_i \qquad (5)$$

$$= (\sum_{i=1}^{N} K_i) S^{\frac{1}{2}} \qquad (6)$$

and $$S = Q^2/(\sum_{i=1}^{N} K_i)^2 \qquad (7)$$

The following energy balance equation is then applied to every reach (Figure 1) of the river in turn, starting from the mouth and progessing upstream:

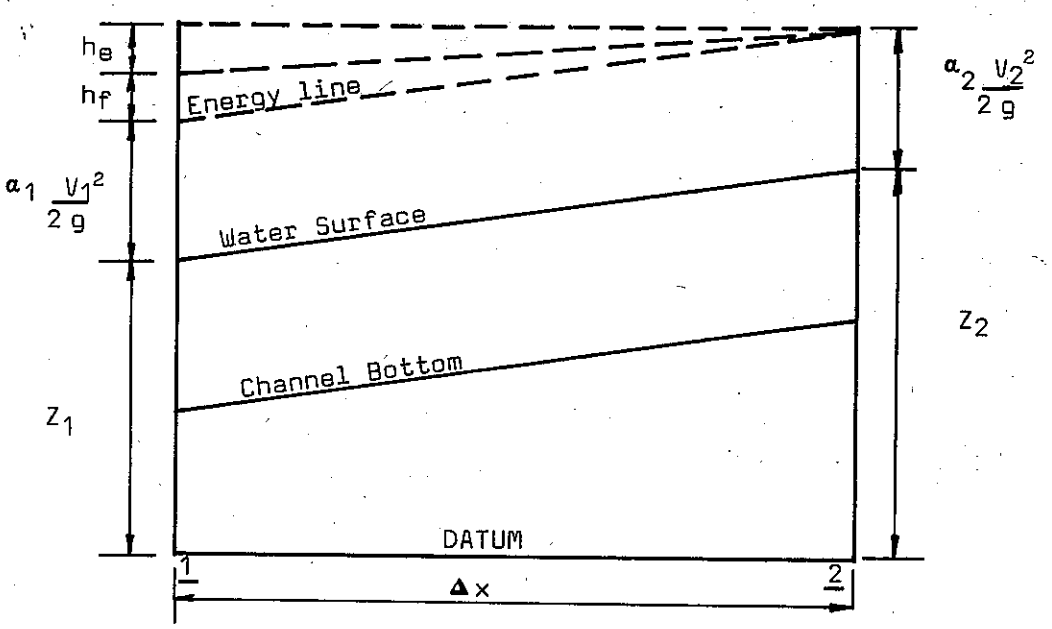

Figure 1 – Definition sketch of terms in total energy equation

Equating the total heads at the two end sections:

$$Z_2 + \frac{\alpha_2 V_2^2}{2g} = Z_1 + \frac{\alpha_1 k V_1^2}{2g} + h_f + h_e \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (8)$$

Where Z = water surface elevation

v = velocity for individual portions of sections

V = mean velocity of section

A = area of section

= Coriolis coefficient

$$\alpha = \frac{\sum_{i=1}^{N} (v_i^3 A_i)}{V^3 A} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

h_f = friction loss

= $S_f \Delta x$ where S_f is taken as the average of the friction slopes at the two end stations

h_e = bend loss which may be expressed as a proportion of the velocity head

= $k \left(\alpha \frac{V^2}{2g}\right)$ where k is the bend loss coefficient

Δx = length of reach

Starting from a known water surface level at Station 1, a trial water surface level at Station 2 is assumed and the total heads at the two stations compared. If the difference between these two heads is within the acceptance error, a solution has been obtained, otherwise a new trial water surface level at Station 2 is assumed and the process repeated. A maximum of thirty trials is made after which the program is terminated. This is normally enough trials for a solution to be obtained but if too small a closing error is specified by the user, more trials may be required.

This program worked well for the simulation of the 1968 flood for which no more than 20 flood levels were available over the 90 km of river between Mt Crosby and the mouth. However, the very much more detailed level data available for the 1974 flood caused difficulties in three areas of simulation, i.e., bend losses, bridge pier losses and shock losses due to deceleration; these being very sensitive to small trial changes in the roughness coefficients. These problems were overcome by increasing the number of river reaches in the vicinity of the areas of difficulty, with particular attention being given to those bends for which a good deal of level data was collected.

The 1974 flood and the 1968 flood were simulated in parallel to determine the different coefficients and, as an additional check, an intermediate stage of the 1974 flood was also simulated to check the effect of varying coefficients for different flood stages. In the process it was found necessary to reduce the peak flow of the 1974 flood slightly to obtain an accurate simulation; this flood flow being modified in preference to the 1968 peak flow because of the less thorough gauging in 1974.

When the simulation of the known profiles of the above floods was considered to be as satisfactory as the data would permit the derived coefficients were then used to simulate the profiles for higher and lower flood flows. The profiles for floods rising to heights of 2 m, 4 m, 6 m, 8 m and 10 m (Australian Height Datum) at the Brisbane City Gauge for a standard tide of 2 m (Port Office Datum) were then interpolated from these data. The adopted standard tide corresponds to 'High Water Mark' for the Port of Brisbane as defined in the Department of Harbours and Marine Tide Tables.

4. FLOOD FREQUENCY STUDY FOR THE BRISBANE RIVER

The other major contribution by the Water Supply and Sewerage Department of the Brisbane City Council was the revision of flood probabilities for the Brisbane River. Several estimates were made in the past but were all on different bases using a variety of analytical methods. Needless to say the probabilities assigned by different investigators to the same historical flood differ by as much as a factor of two. A major problem in this study has always been the non-uniform nature of the flood population available for analysis.

The levels of significant floods have been recorded at the Port Office in Brisbane since 1836. Some of the early levels are not known within a range of feet and many minor floods were not recorded. The Brisbane Port Office is affected by tides which have the greatest influence on small floods. Furthermore, the dredging of the metropolitan reaches of the Brisbane River for the improvement of navigation started in 1879 and reached its peak in about 1940. Since this date the central city reaches of the river have been abandoned for port purposes and dredging of them has been discontinued with the consequent silting up of the river.

In addition to all this some deliberate widening was carried out in the metropolitan reaches of the river in the late 1930's for flood mitigation and Somerset Dam on the Stanley River, a major tributary of the Brisbane River, became effective for flood mitigation from 1943. All these factors have resulted in a very mixed population of flood levels available for analysis from the Port Office in Brisbane. Some attempts were made in the past to produce a uniform population of floods for frequency analysis but the methods adopted to do this are questionable.

Annual flood levels are available from 1887 for the Mt Crosby gauge just above the tidal regime of the river at A.M.T.D. 90 km whilst daily river levels for this gauge date from 1894. Major flood levels date from 1893 with daily levels starting in 1909 at Lowood on the Brisbane River and daily readings started in 1915 at the site of Somerset Dam on the Stanley River and in 1920 at Fulham Vale on the Upper Brisbane River. The levels of moderate and major floods have been recorded at Caboonbah and Murrumba on the Brisbane River just below the Stanley junction since

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1890 and at Goodna on the Lower Brisbane River since the same date. Rainfall records, for other than Brisbane and Ipswich, begin to give a reasonable coverage for the significant parts of the Brisbane River catchment only from the late 1880's.

The main problems to be overcome in the re-evaluation of flood probabilities in the Brisbane River are, firstly, the effect that the operation of Somerset Dam would have had on the mitigation of historical floods prior to 1943 and, secondly the effects of the dredging and widening of the navigable part of the river on flood levels.

Somerset Dam mitigates floods by storing the flow of the Stanley River when the peak of the flood in the Upper Brisbane River is passing the Stanley junction. In small and moderate floods it is possible to store the whole of the Stanley River flow in Somerset Dam and release nothing on to the peak of the Upper Brisbane River. In this way the dam achieves the maximum possible degree of flood mitigation in the Brisbane River. However, in major floods, the limited flood pondage available in Somerset Dam requires some water to be discharged on to the peak of the Upper Brisbane flood to prevent the dam from being filled and overtopped.

The available data limited the period of the investigation from 1887 to 1974 and also required that different methods be adopted for the periods 1887 to the establishment of daily measurements at Lowood in 1909, from 1909 to the start of daily measurements of flow at Fulham Vale and Mt Tarampa in 1920 and finally from 1920 to 1943 when Somerset Dam first became effective for which period more or less complete hydrographs are available from a number of stations. From 1943 to date the dam has been in operation and the downstream flood levels are the result of the deliberate operation of the dam.

The effect of the operation of Somerset Dam on floods before 1943 was assessed by first calculating the hydrograph that would have occurred in the Stanley River had Somerset Dam not been built. This hydrograph was then routed downstream to Lowood and subtracted from the Lowood hydrograph to give the hydrograph of the Brisbane River catchment less the Stanley River catchment. The Stanley River flood was then routed through Somerset Dam with the dam being operated in the manner determined from 30 years of

practical experience. The discharge from the dam, if any, was then routed to Lowood and added to the previous subtraction hydrograph to give the hydrograph of the Brisbane River flood as mitigated by Somerset Dam.

A study of the records for Somerset Dam showed that a simple relationship holds between the peak day rainfall in a storm and the peak flood flow. This relationship was used to find the peak of the Stanley River hydrograph. The shape of the hydrograph was then found from factors deduced from a study of the response of the catchment. A continuing loss of 25 mm/day was adopted for the catchment and the hydrograph was then adjusted to equate the volume of runoff to the volume of effective rainfall.

The above is basically the same technique adopted by the Stanley River Works Board 40 years ago to examine the flood mitigation effects of the proposed Somerset Dam on a limited range of floods. In the last 40 years the accumulation of data on the rainfall response of the Brisbane River catchment and on the modification of flood hydrographs by valley storage along the Brisbane River has resulted in improved accuracy in the derivation and routing of hydrographs. In spite of these improvements little reason was found to modify the original work done by the Board on the representative floods of 1893, 1898, 1931 and 1928. However, the accumulated data made it possible to attempt the hydrology of small floods and freshes with confidence. This was not possible when the Stanley River Works Board commenced the process.

For the period 1894 to 1909 the Lowood hydrographs were obtained by extending the Mt Crosby readings back to Lowood. All calculations were centered on Lowood, firstly, because of its lengthy record of river levels and secondly, because it is only a short distance downstream of the site of the proposed Wivenhoe Dam. The Lowood derivations can be used to provide a population of floods into Wivenhoe Dam for assessing the flood mitigation effect of the dam provided the contribution of Lockyer Creek which enters the Brisbane River between Wivenhoe and Lowood is assessed.

Having obtained a population of floods at Lowood modified by the operation of Somerset Dam the floods were routed to Mt Crosby by height relationships derived from many years of records. Mt Crosby is located at the head of the tidal section of the Brisbane River and also marks the limit of urban development. The Bremer River with a catchment of 2 020 km^2 out of a total catchment of 13 560 km^2 at Brisbane enters the Brisbane River downstream of Mt Crosby. A study of the available records show that the Bremer River provides a steady proportion of the peak flow in the majority of the Brisbane River floods. However, on a number of occasions the Bremer River contribution to the peak flood flow has been non-typical. Peak levels are available from Goodna, downstream of the Bremer Junction for the majority of moderate and major floods since 1890 and it was possible to route these floods to account for the flood mitigation effect of Somerset Dam. The main problem lies with the minor floods and freshes for which very little data has been gathered. The variable contribution of the Bremer River to these floods is difficult to assess from rainfall. Fortunately, however, the effect of such small floods upon the high-tide level at the Port Office in Brisbane is small and, furthermore, is largely independent of the height of the tide. Except for 1947 in which a significant independent flood occurred in the Bremer River, the established relationship between the flow at Mt Crosby and the raising of the level at the Brisbane City Gauge for a standard tide of 2 m (from Tide Tables and therefore on Port Office Datum) was used for all minor floods. In general, the relationship from Figure 6 of Part 1 of this report was used between the Bremer Junction and Brisbane City Gauge.

In this way a uniform population of floods was obtained for the years 1887 to 1974 inclusive for the 1974 conditions of the Metropolitan reaches of the Brisbane River, for a standard tide of 2.0 m (Port Office Datum) and with Somerset Dam in operation to mitigate flooding. The return periods for floods of various heights at the Brisbane City Gauge were derived by fitting four different flood frequency distributions to the annual series for the period 1887 to 1974.

The results for a given flood height vary considerably as can be seen from Table 1. Three of the distributions in the table have gained wide acceptance in flood hydrology while Boughton (1975) has shown that the 'Empirical distribution' is a good fit to Queensland data. After consideration of the information in the table and graphical plots of the four distributions, the Flood Co-ordination Recording Committee decided the results from the Log Pearson Type III distribution were the most appropriate for the purpose of indicating return periods of flooding on the flood maps.

TABLE 1 – RETURN PERIODS FOR FLOODS ON THE BRISBANE RIVER

BRISBANE CITY GAUGE HEIGHT m on A.H.D.	RETURN PERIOD IN YEARS			
	EMPIRICAL DISTRIBUTION*	PEARSON TYPE III	LOG PEARSON TYPE III	LOG – NORMAL
1.5	8	8	8	8
2.0	12	11	11	10
2.5	18	16	14	12
3.0	25	25	18	16
3.5	33	35	23	20
4.0	40	50	28	22
4.5	60	70	34	25
5.0	80	100	40	30
5.5	110	140	50	33
6.0	140	185	60	37
6.5	200	260	70	40
7.0	250	350	80	50
7.5	350	500	90	55
8.0	600	700	110	60
8.5	800	1 000	130	70
9.0	1 000	1 400	150	75
9.5	1 700	2 000	170	85
10.0	2 500	2 500	200	90
11.0	6 000	5 000	250	100
12.0	$>$ 10 000	9 000	320	125
13.0	$\gg$ 10 000	$>$ 10 000	450	160
14.0	$\gg$ 10 000	$\gg$ 10 000	550	190
15.0	$\gg$ 10 000	$\gg$ 10 000	700	220

* Boughton, W.C. (1975). A study of Queensland floods, Symposium, the frequency of floods in Queensland, Inst. of Engs., Aust., Queens. Branch, Aug 1975.

5. FLOOD FREQUENCY FOR THE BREMER RIVER

A number of difficulties were involved in the determination of flood frequencies in the Bremer River from its junction with the Brisbane River to a point upstream of the Warrill Creek - Bremer River junction. The main difficulty was that no long-term record of flood discharges was available for the Bremer River in the City of Ipswich and although flood gauge readings were available for the Bremer River at the David Trumpy Bridge, their analysis was complicated by the fact that during most floods this gauge is affected by backwater from the Brisbane River.

A regional flood frequency approach was therefore adopted for the Bremer River using information available from 10 stream gauging stations in the Brisbane River catchment. For consistency with the B.C.C.'s work on the Brisbane River the individual stations analysed in the regional approach were assumed to fit the Log Pearson Type III distribution. The results of the regional study for floods of the same return period as those used to derive the Brisbane River flood profiles are as shown in Table 2.

TABLE 2 - RETURN PERIODS FOR FLOODS ON THE BREMER RIVER

RETURN PERIOD YEARS	DISCHARGE m^3/s
11	1 100
28	1 620
60	2 100
110	2 540
200	3 050

6. ESTIMATES OF FLOOD ENVELOPES FOR THE BREMER RIVER

The flood envelopes for the Bremer River were estimated by allowing the peak flood discharges indicated in Table 2 above to recede over a period of 28 hours. The reduced discharges resulting were then assumed to occur at the time of the Brisbane River flood peak and a backwater curve was computed along the Bremer River for this situation.

Where the backwater condition intercepted the normal flow depth for the peak discharges given in Table 2, these were assumed to form the remainder of the flood envelope. Because of the complicated nature of the analysis of the flood situation along the Bremer River and the limited data the computed flood envelopes are not considered to be as accurate as those provided by the B.C.C. for the Brisbane River.

Queensland Water Resources Commission

Surface Water Branch
Hydrology Section

Wivenhoe Dam
(Brisbane River at 150.2 km)

Report on

Downstream Flooding

By
[redacted]

Record – For Commission Use Only

Hydrology Report No :143005.PR/4
November 1984

Hydrology Report 143005.PR/4

QUEENSLAND WATER RESOURCES COMMISSION

SURFACE WATER BRANCH

BRISBANE RIVER

FLOODING DOWNSTREAM OF WIVENHOE DAM

[redacted]

NOVEMBER 1984

RECORD FOR COMMISSION USE ONLY

This report has been prepared by Surface Water Branch to provide hydrologic input for Commission investigations. Any views and conclusions expressed in this report may not necessarily be adopted by the Queensland Water Resources Commission.

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ABSTRACT

Flooding downstream of Wivenhoe Dam has been analysed as a continuation of previous studies of Wivenhoe Dam design floods (Weeks, 1983). The runoff routing model (Mein, Laurenson, and McMahon, 1974; Weeks, 1978) was calibrated for the catchment downstream of the dam. Design rainfalls of 2% to 0.01% annual exceedence probabilities were established in two ways - by analysis of individual station rainfall records and by analysis of combined catchment rainfalls. The Probable Maximum Precipitation was obtained from the Bureau of Meterology. Design floods of annual exceedence probabilities from 1% to Probable Maximum were calculated for cases with and without Wivenhoe Dam in the catchment by using the design rainfalls as input to the calibrated runoff routing model.

Wivenhoe Dam has a significant effect on reducing floods up to 0.01% annual exceedence probability, but the flood which would result from the Probable Maximum Precipitation would overtop the dam. This overtopping would almost certainly cause failure of the dam.

Some studies of a preliminary nature were done which showed that failure of the dam would cause devastating floods in the lower Brisbane River, considerably higher than any which could have occurred without the dam.

Operating rules for Wivenhoe and Somerset Dams were chosen from a consideration of the effects of various trial rules on the design floods. This part of the investigation was carried out by the Brisbane City Council.

TABLE OF CONTENTS

LIST OF FIGURES

1. INTRODUCTION

Wivenhoe Dam is a major dam in the final phase of construction on the Brisbane River at AMTD 150.2 km at the outlet of a catchment with an area of 7 020 km^2. When complete, the dam will provide urban water supplies to supplement those already available from Somerset Dam and North Pine Dam. It will also provide for flood mitigation on the lower reaches of the Brisbane River.

A study of design floods as inflows to Wivenhoe Dam has recently been completed (Weeks, 1983). This study derived low probabililty (1% to PMP) floods for design purposes.

Following preparation of this report, further analyses were required in two main areas:

1. Analysis of further design storms in the catchment of Wivenhoe Dam to provide additional data for the testing of flood operation policies.

2. Study of the effects of Wivenhoe Dam on downstream flooding.

The current report describes these studies.

2. CATCHMENT DESCRIPTION

Wivenhoe Dam is situated on the middle reaches of the Brisbane River with almost equal catchment areas above and below. The catchment above the dam is described in the previous report (Section 2 of Weeks, 1983).

The catchment area downstream of Wivenhoe Dam is about 7 000 km^2 in area with three major tributaries - Lockyer Creek, Bremer River and Warrill Creek. The major features of the catchment are shown in Figure 1.

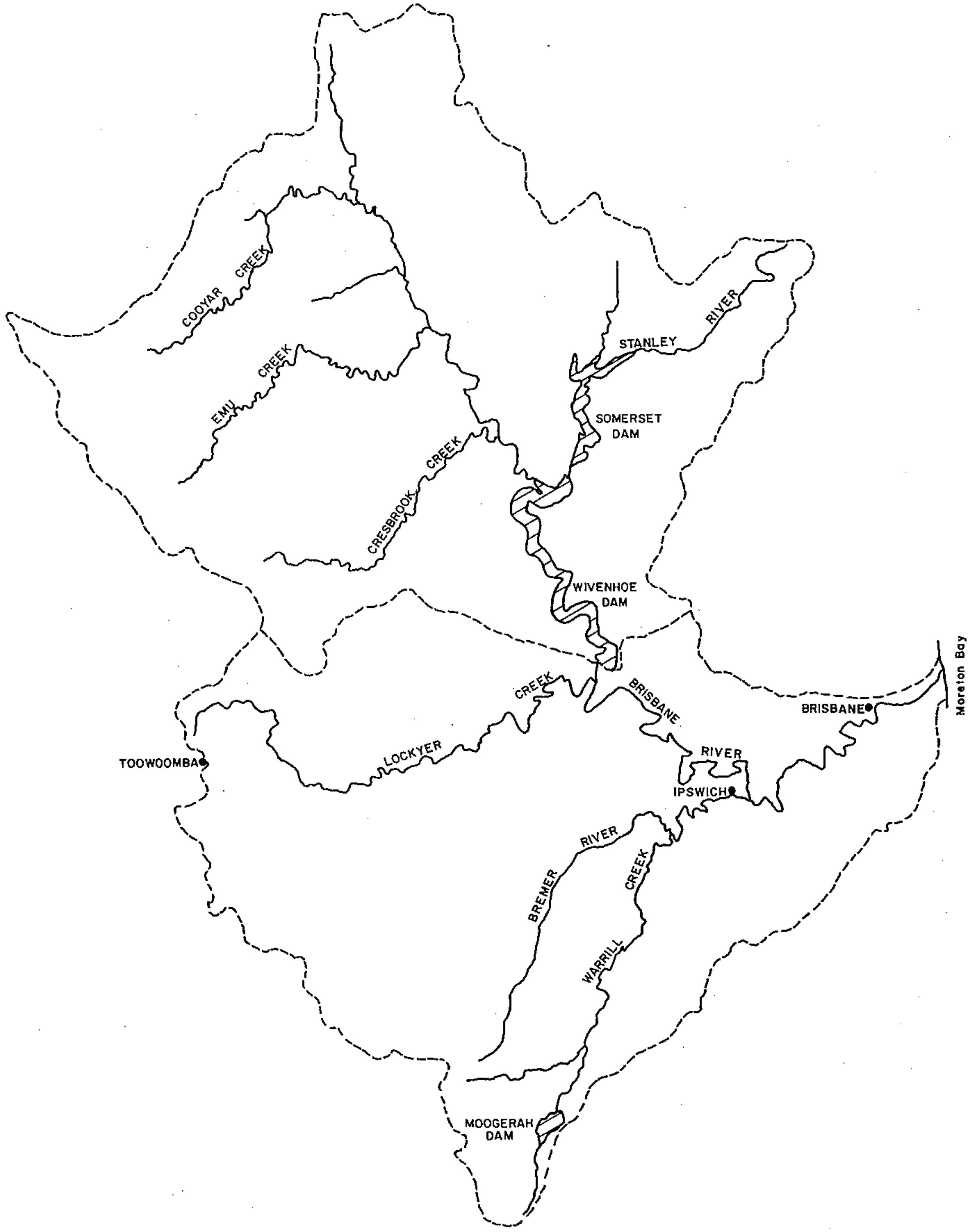
Lockyer Creek has a catchment area of about 3 000 km^2 and flows from the west joining the Brisbane River just below Wivenhoe Dam. The catchment is quite variable with the main stream flowing through a wide flat floodplain of alluvium but with steep mountains to the south and west. The floodplain is used for extensive areas of irrigated agriculture while much of the mountainous area is forested. There are a few small towns and weirs. There is one dam, Atkinson Dam, an offstream storage, that was not considered in this analysis.

The Bremer River has a smaller catchment of about 600 km^2 and joins the Brisbane River in the lower reaches near Ipswich. The catchment is generally hilly and lightly forested with grazing and agriculture the major land use.

Warrill Creek has a catchment area of about 900 km^2 and joins the Bremer River in its lower reaches. This catchment has extensive areas of irrigated agriculture on flat alluvial plains. More mountainous forested areas exist to the south. Moogerah Dam is situated on Reynolds Creek, a tributary in the upper reaches of Warrill Creek, and was not considered in this analysis.

The lower reaches of the Brisbane River extend for about 120 km through an incised channel from Wivenhoe Dam to the mouth of the river near Brisbane. There is considerable urban development of both Ipswich and Brisbane in the lower areas. The remainder of the area has mixed land use of forest, grazing and agriculture.

FIGURE I



LOCATION MAP

3. SUMMARY OF PREVIOUS STUDY

The previous design flood study for Wivenhoe Dam calibrated the runoff routing model using computer program WT87/WT87C (Weeks, 1978) on seven gauging stations in the catchment of Wivenhoe Dam (including recorded inflows to Somerset Dam). Seven different floods were used for calibration at all of the available stations. Models were developed for the catchment above Somerset Dam and for the remainder of the catchment excluding Somerset Dam. The routing of floods through Somerset Dam was considered independently.

Design rainfall depths were obtained from a frequency analysis of annual series of rainfall data in the catchment carried out by the Commission and from a study of Probable Maximum Precipitation (PMP) carried out by the Bureau of Meteorology.

Using the design rainfall data for various annual exceedance probabilities as input to the calibrated rainfall-runoff model, design floods were calculated for two conditions, namely as flow at Wivenhoe Damsite, assuming that the dam does not exist and also as inflow to the dam assuming that the dam was in existence and full. In all cases, Somerset Dam was assumed to be in existence and full. Table 1 summarises results.

A frequency analysis of the annual series of synthesised flood discharges for Wivenhoe Dam was also carried out as an independent test on the rainfall analysis.

TABLE 1: SUMMARY OF WIVENHOE DAM DESIGN FLOODS

Annual Exceedence Probability	Somerset Dam Inflow (m^3/s)	Wivenhoe Dam Inflow (m^3/s)	
		No Dam	With Dam
1	3 250	8 300	8 700
0.1	5 300	12 900	13 400
0.01	7 780	19 100	19 600
PMP	14 200	44 500	47 800

4. AVAILABLE DATA

The lower catchment of the Brisbane River has a reasonable coverage of hydrologic data, though not as good as the upper catchment for a flood study of this type.

Four Queensland Water Resources Commission stream gauging stations were used for model calibration. These stations are listed in Table 2.

TABLE 2: Q.W.R.C. GAUGING STATIONS USED IN FLOOD STUDY

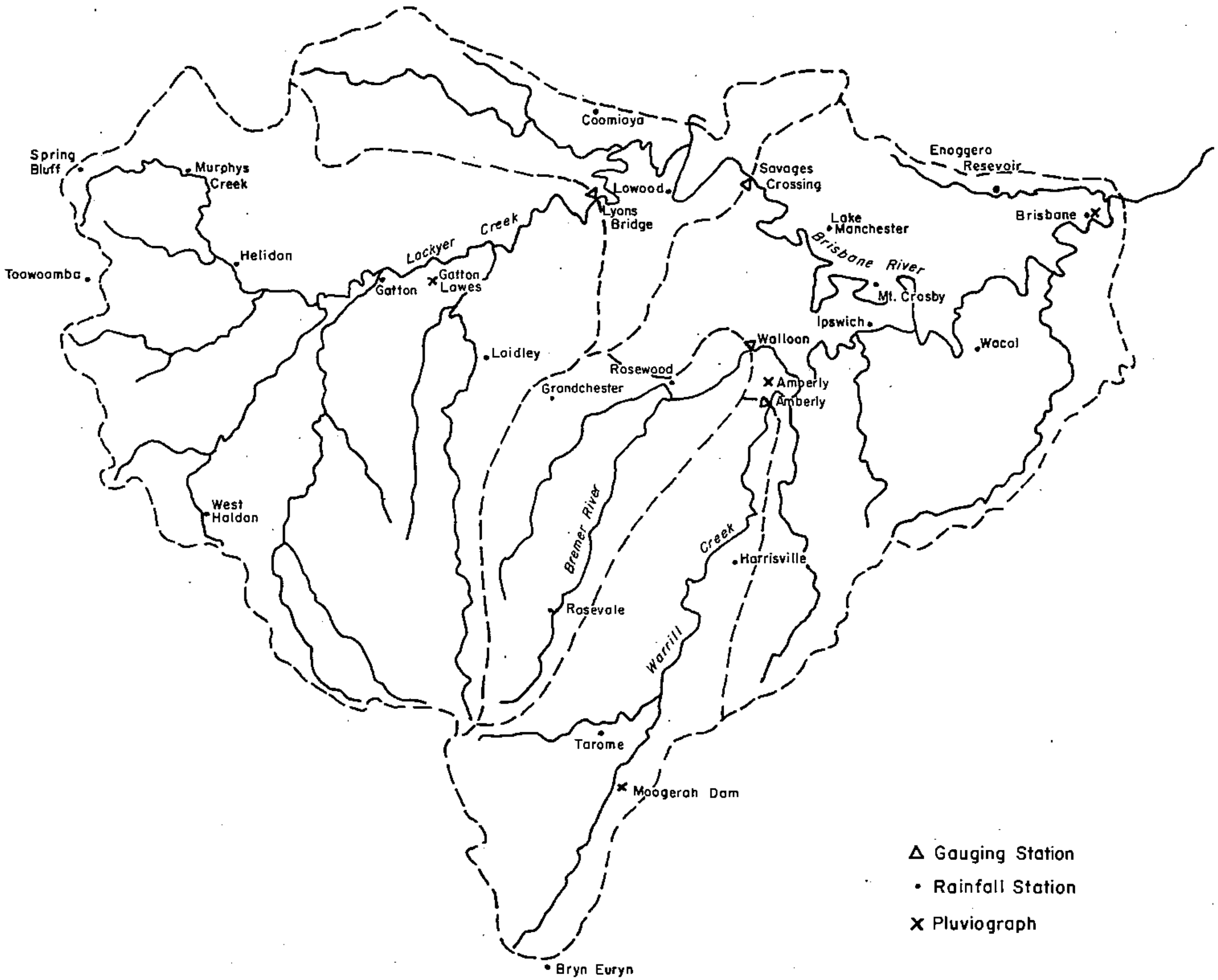
Number	Stream	Site	AMTD (km)	Catchment Area (km^2)	Period of Record
143001	Brisbane River	Savages Crossing	130.8	10 180	1909-
143107	Bremer River	Walloon	37.2	620	1961-
143108	Warrill Creek	Amberley	8.7	920	1961-
143210	Lockyer Creek	Lyons Bridge	27.2	2 540	1909-

These gauging stations are shown on the location map for the lower catchment, Figure 2. Appendix A contains comments on the quality of gauging station records by A. Seabrook (Senior Hydrographer).

In addition to the records from these stations, records of floods in the lower reaches of the Brisbane River, in particular at Moggill, were available from the Brisbane City Council.

A number of rainfall stations with long term records were available and were used in the analysis. There were many other stations in the catchment, but those used in the analysis had sufficiently long records for a frequency analysis and also gave an adequate coverage of the catchment. These stations are shown on Figure 2 and are listed in Table 3.

In addition, there were four pluviographs in the catchment with readily available data which were used. These are listed in Table 4 and are also shown on Figure 2.



CATCHMENT AREA BELOW WIVENHOE DAM

6. FIG. 2

TABLE 3: DAILY READ RAINFALL STATIONS USED IN FLOOD STUDY

NUMBER	STATION	YEARS OF RECORD
040030	Bryn Euryn	1918-1972
040083	Gatton	1895-
040091	Grandchester	1895-
040094	Harrisville	1897-
040096	Helidon	1894-
040101	Ipswich	1887-
040114	Laidley	1890-
040115	Lake Manchester	1918-
040120	Lowood	1888-
040142	Mt.Crosby	1895-
040153	Murphys Creek	1896-
040183	Rosevale	1910-
040184	Rosewood	1895-
040198	Tarome	1912-
040214	Brisbane	1861-
040225	Enoggera Reservoir	1888-
040227/040457	Wacol	1894-
040421	Spring Bluff	1896-
040424	West Haldon	1916-
041103	Toowoomba	1888-
040056	Coominya	1916-

TABLE 4: PLUVIOGRAPHS USED IN FLOOD STUDY

NUMBER	STATION	YEARS OF RECORD
040004	Amberley Aero	1961-
040082	Gatton (Lawes)	1963-
040135	Moogerah Dam	1958-
040214	Brisbane Regional Office	1911-

5. RUNOFF-ROUTING MODEL CALIBRATION

5.1 Introduction

The runoff routing model developed by Mein, Laurenson and McMahon (1974), and implemented as computer programs WT87 and WT87C (Weeks, 1978) was used to relate rainfall to runoff in the catchment downstream of Wivenhoe Dam. Flood hydrographs at Wivenhoe Dam as estimated in the previous report were input to the model.

This model is a simple conceptual representation of the catchment storage effects and provides for the routing of rainfall excess to produce a hydrograph of surface runoff. The model is distributed by division of the catchment into subareas, thereby allowing rainfall and loss input to vary from subarea to subarea. In addition, since the hydrograph is built up as the flood moves down the catchment, the hydrograph is available at any point in the catchment.

Two model parameters must be estimated by calibration using recorded data or by the use of regional formulae before the model can be used to calculate design floods. These model parameters are called K and m, and are constants for the catchment. The value of K is adjusted for each river reach in the catchment by multiplying it by a constant computed from the length of the reach.

The runoff routing model is the well known method for calculating hydrographs of surface runoff from rainfall excess. It has a number of advantages over the unit hydrograph method, as discussed by Mein and Laurenson (1975), but in this case, the major advantages are as follows:

1. The hydrograph is calculated by the model at all points in the catchment (at least at all of the nodes). This permits the model to be calibrated at several points, in this case at the gauging stations. Also points where design floods are calculated can be varied, in this case in the Brisbane city area, and also at Mt. Crosby and other points on the lower reaches of the river.

2. The effects of different operating policies of the dams can be investigated easily by the provision of upstream hydrograph input.

The model was primarily calibrated using flood peak discharges, but the general hydrograph shape was also considered. The model was calibrated with a single parameter pair for the whole catchment. If this calibration had not been successful, it would then have been necessary to derive separate parameter pairs for each of the gauged subcatchments. This would then have led to uncertainty about what values to use for the area downstream of these gauged catchments. Having a single parameter pair therefore led to much simpler design flood calculations.

A flood study had not previously been completed for this catchment downstream of Wivenhoe Dam so it was not possible to compare model parameters estimated in this study with any estimated previously.

5.2 Calibration Data

The seven major floods that were used for calibration in the catchment upstream of Wivenhoe Dam were again used for calibration. Data were available at one or more of the Commission gauging stations and also for the lower reaches of the Brisbane River from the Brisbane City Council in some cases. The hydrographs calculated at Wivenhoe Dam in the previous study were used as input to the model in all cases.

Baseflow was separated from the recorded hydrographs at Commission gauging stations using the same approach as previously. The baseflow hydrograph was assumed to follow a straight line from the beginning to the end of surface runoff, that is the baseflow was assumed to be below the straight line joining the point on the hydrograph of total runoff where the hydrograph begins to rise and the point on the falling limb, selected by semilog plots, marking the end of surface runoff. Flood data for the lower reaches of the Brisbane River were of much lower quality and no baseflow was assumed. The peak discharges of flood hydrographs at each of the available gauging stations, after deduction of baseflow are listed in Table 5.

TABLE 5: PEAK DISCHARGES OF RECORDED FLOODS (m^3/s)

FLOOD	WIVENHOE	LOCKYER CREEK	BREMER RIVER	WARRILL CREEK	SAVAGES CROSSING
Jul 1965	1030	545	502	-	1593
Mar 1967	1177	122	123	111	1361
Jun 1967	1568	565	471	322	2479
Jan 1968	2358	789	522	373	3496
Dec 1971	611	57	-	-	530
Jan 1974	6946	1504	-	2062	7487
Jan 1976	1104	468	-	96	1595

Rainfall data were available for these floods at the twenty-one stations in the catchment and depths for the storms are listed in Table 6.

TABLE 6: RAINFALL DEPTHS (mm) FOR EACH RAINFALL STATION

STATION	JUL 1965	MAR 1967	JUN 1967	JAN 1968	DEC 1971	JAN 1974	JAN 1976
Bryn Euryn	221.5	113.8	136.6	526.8	80.3	-	-
Gatton	234.5	74.7	100.9	286.8	89.7	319.6	41.4
Grandchester	252.0	63.7	148.6	352.2	73.7	460.2	88.0
Harrisville	199.4	42.7	120.2	202.3	73.2	444.2	58.2
Helidon	191.8	64.0	95.8	239.4	124.9	267.0	81.0
Ipswich	233.4	81.8	198.4	273.6	90.2	603.0	69.0
Laidley	230.5	62.0	108.9	242.6	86.7	497.0	69.2
Lake Manchester	295.1	102.3	254.2	300.4	86.9	518.1	111.2
Lowood	259.3	69.3	140.2	323.9	94.0	385.0	77.4
Mt. Crosby	246.9	99.1	246.6	328.4	94.5	684.6	117.0
Murphys Creek	204.0	61.0	121.6	168.3	119.9	309.2	92.8
Rosevale	212.9	53.4	106.0	269.9	60.2	208.8	80.4
Rosewood	206.7	68.6	161.8	342.0	89.9	572.6	70.2
Tarome	224.3	58.0	105.1	298.0	68.3	471.0	61.8
Brisbane	227.8	88.7	341.6	360.2	68.1	648.1	87.4
Enoggera Reservoir	269.0	94.7	292.1	413.3	78.5	898.4	121.2
Wacol	203.9	86.4	224.1	317.7	76.7	618.4	80.6
Spring Bluff	207.8	42.4	103.4	280.1	142.0	414.8	179.4
West Haldon	152.4	59.7	90.2	220.9	73.6	162.9	34.0
Toowoomba	181.6	39.6	117.6	285.8	99.8	291.8	136.8
Coominya	209.7	77.2	146.0	350.9	102.7	391.0	52.6

5.3 Catchment Data

The catchment below Wivenhoe Dam downstream to a point just below the junction of Breakfast Creek with the Brisbane River was divided into 69 subareas. Figure 3 is a catchment map showing subarea distribution. The subarea numbers for each subcatchment (between gauging stations) are listed on Table 7.

TABLE 7: SUBAREA NUMBERS FOR EACH SUBCATCHMENT

SUBCATCHMENT	AREA (km^2)	SUBAREA NUMBERS
Lockyer Creek	2540	45,46,47,48,49,50,51,52,53, 54,55,56,57,58,59,60,61,62, 63,64,65,66,67,68,69.
Bremer River	620	27,28,29,30,31,32,33,34.
Warrill Creek	920	18,19,20,21,22,23,24,25,26.
Savages Crossing	620	38,39,40,41,42,43,44.
Brisbane City	2160	4,5,6,7,8,9,10,11,12,13,14, 15,16,17,35,36,37.
Downstream Breakfast Ck.	121	1,2,3.

The subareas and distances from the catchment outlet for each subarea are listed in Table 8. The area weighted mean travel distance d_c for the catchment was calculated as 145 km.

Subarea relative delay times for each reach were then calculated as being proportional to reach length.

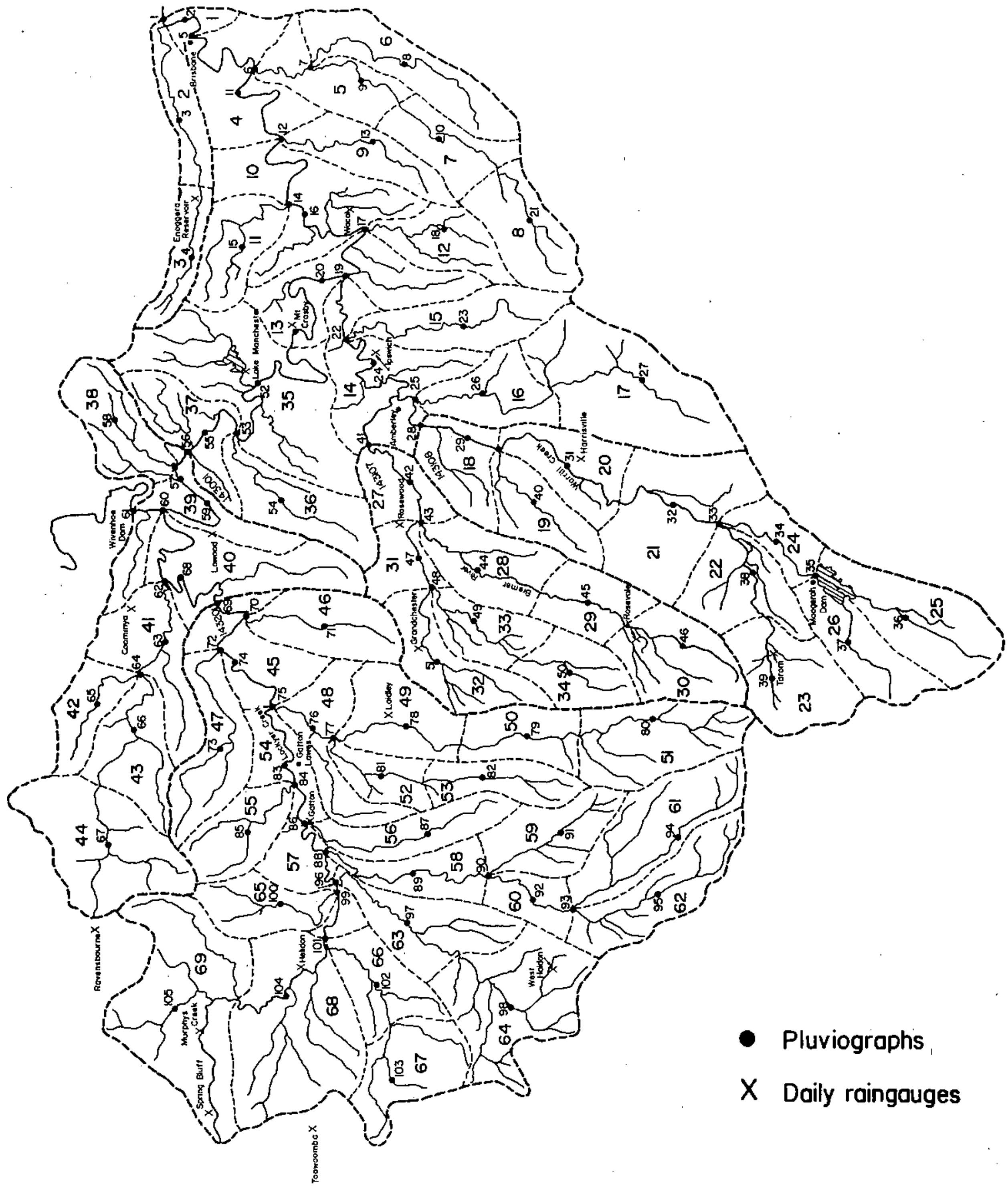
A listing of the catchment data file used in the runoff routing model is given in Appendix B.

5.4 Rainfall Data

Rainfall station data were assumed to apply to the model subareas as shown in Table 9, and the pluviograph data as shown in Table 10. The time increment used for model operation was two hours.

In the runoff routing model, the total rainfall depth for the storm is provided by the daily read rainfall stations, while the distribution in time is given by the pluviographs. In this catchment, as in most others, there are many more daily read rainfall stations than pluviographs. The total rainfall depths were therefore taken for each subarea from the stations listed in Table 9, while these volumes were distributed according to the distribution of rainflal at the four pluviographs.

FIGURE 3. 12.



CATCHMENT SUBDIVISION

BRISBANE RIVER BELOW WIVENHOE DAM

TABLE 8

BRISBANE RIVER DOWNSTREAM OF WIVENHOE DAM

SUBAREA NUMBER	AREA (km^2)	DISTANCE TO OUTLET (km)	SUBAREA NUMBER	AREA (km^2)	DISTANCE TO OUTLET (km)
1	19.4	1.5	36	109.2	119.3
2	53.8	12.5	37	90.0	115.5
3	48.1	27.5	38	73.7	130.5
4	134.7	21.6	39	75.9	125.5
5	90.7	31.6	40	110.4	149.3
6	129.0	40.3	41	87.9	154.3
7	79.4	42.9	42	50.0	166.8
8	104.2	57.9	43	79.1	168.1
9	81.5	42.9	44	142.9	183.1
10	172.2	45.4	45	93.0	171.9
11	87.2	55.4	46	94.3	171.9
12	81.5	65.5	47	103.5	183.2
13	144.5	64.2	48	94.9	185.7
14	159.4	75.5	49	97.6	199.5
15	112.0	85.5	50	71.4	214.5
16	90.7	99.3	51	132.3	229.5
17	232.4	119.3	52	50.4	197.0
18	76.0	99.3	53	68.1	210.8
19	78.0	110.6	54	59.0	188.2
20	88.8	113.1	55	74.7	202.0
21	182.9	128.1	56	80.6	210.7
22	98.9	140.6	57	60.9	207.0
23	116.3	153.1	58	68.7	214.5
24	47.8	140.6	59	102.2	234.5
25	140.5	156.9	60	48.5	233.3
26	90.8	155.7	61	142.1	250.8
27	60.1	100.5	62	111.4	248.3
28	97.7	114.3	63	104.2	219.5
29	70.0	128.1	64	172.3	237.0
30	105.6	139.4	65	70.1	217.1
31	47.5	110.5	66	79.2	222.1
32	96.4	123.0	67	99.5	232.1
33	66.6	120.5	68	222.0	223.4
34	75.7	134.3	69	239.0	239.7
35	261.4	96.7			
				6980.9	

$\sum ad = 1011516.93$

$d_c = 144.9$

TABLE 9: DISTRIBUTION OF RAINFALL STATIONS

NAME	SUBAREAS
Bryn Euryn	25,26.
Gatton	47,52,54,55,56,57,58.
Grandchester	32,33,46.
Harrisville	16,17,19,20,21.
Helidon	65,68.
Ipswich	14,15.
Laidley	48,49,50,53.
Lake Manchester	35,36,37.
Lowood	38,39,40.
Mt. Crosby	13.
Murphys Creek	69.
Rosevale	29,30,34,51.
Rosewood	18,27,28,31.
Tarome	22,23,24.
Brisbane	1,2,4,5,6.
Enoggera Reservoir	3,11.
Wacol	7,8,9,10,12.
Ravensbourne	43,44.
West Haldon	59,60,61,62,63,64,66.
Toowoomba	67.
Coominya	41,42,45.

TABLE 10: DISTRIBUTION OF PLUVIOGRAPHS

NAME	SUBAREAS
Brisbane	1,2,3,4,5,6,7,8,9,10,11.
Amberley	12,13,14,15,16,17,18,27, 31,32,35,36,37,38,39.
Moogerah	19,20,21,22,23,24,25,26, 28,29,30,33,34,51,61,62.
Gatton	40,41,42,43,44,45,46,47, 48,49,50,52,53,54,55,56, 57,58,59,60,63,64,65,66, 67,68,69.

5.5 Model Calibration

The model was calibrated using all the available recorded data. To ensure correct volumes at each gauging station, losses for each subcatchment and each calibration flood were computed from a water balance. The losses so calculated are given in Table 11.

TABLE 11: LOSSES USED IN MODEL CALIBRATION

(INITIAL LOSS - mm, CONTINUING LOSS - $mm.h^{-1}$)

		LOCKYER CREEK	BREMER RIVER	WARRILL CREEK	SAVAGES CROSSING	REMAINDER
Jul 1965	Initial	0	70	70	0	0
	Continuing	8.7	7.0	7.0	5.5	5.5
Mar 1967	Initial	30	40	30	0	0
	Continuing	4.0	0.4	2.5	0.0	0.0
Jun 1967	Initial	10	20	50	0	50
	Continuing	1.5	0.5	0.5	0.0	6.0
Jan 1968	Initial	100	50	150	0	0
	Continuing	2.5	1.8	3.5	0.0	0.0
Dec 1971	Initial	100	120	120	120	120
	Continuing	2.0	4.0	4.0	4.0	4.0
Jan 1974	Initial	100	0	0	0	0
	Continuing	1.5	2.5	2.5	1.5	1.5
Jan 1976	Initial	35	15	15	10	10
	Continuing	0.0	3.5	3.5	3.0	3.0

The best set of parameters was found to be $K = 270$, $m = 0.75$ and using these parameter values and the losses given in Table 11, the hydrographs at each gauging station were calculated. The calculated, and recorded if available, hydrograph details are given in Table 12.

TABLE 12: CATCHMENT MODEL CALIBRATION

(PEAK DISCHARGE - m^3/s, VOLUME - ML)

FLOOD	LOCKYER CREEK		BREMER RIVER		WARRILL CREEK		SAVAGES CROSSING	
	PEAK	VOLUME	PEAK	VOLUME	PEAK	VOLUME	PEAK	VOLUME
July 1965								
Recorded	545	66 000	502	27 000	-	-	1593	279 000
Calculated	475	65 000	261	23 000	304	35 000	1749	241 000
Mar 1967								
Recorded	122	18 000	123	13 000	111	13 000	1361	281 000
Calculated	107	20 000	99	11 000	108	15 000	1465	229 000
Jun 1967								
Recorded	565	114 000	471	57 000	322	52 000	2479	553 000
Calculated	607	122 000	434	59 000	408	54 000	1881	419 000
Jan 1968								
Recorded	789	163 000	522	92 000	373	74 000	3496	1162 000
Calculated	861	165 000	548	99 000	523	77 000	3025	951 000
Dec 1971								
Recorded	57	4 900	-	-	-	-	530	56 000
Calculated	34	5 000	0	0	0	0	574	70 000
Jan 1974								
Recorded	1504	356 000	-	-	2062	292 000	7487	1992 000
Calculated	3078	343 000	1597	155 000	2681	288 000	7965	1653 000
Jan 1976								
Recorded	468	66 000	-	-	96	11 200	1595	312 000
Calculated	460	64 000	111	11 400	66	11 300	1586	351 000

91

Flood data were not available for the lower reaches of the Brisbane River from Commission gauging stations but were available from the Brisbane City Council for some floods. In general, peak discharge data only were available and even when hydrographs were provided, the peaks only were used. A comparison of peak discharges is given in Table 13.

TABLE 13: BRISBANE RIVER MODEL CALIBRATION

(CALCULATED PEAK DISCHARGE AT CITY)

FLOOD	SITE	RECORDED PEAK (m^3/s)	CALCULATED PEAK (m^3/s)
Jul 1965	Moggill	1870	1688
Mar 1967	Moggill	1473	1535
Jun 1967	Moggill	3626	3369
Jan 1968	Jindalee	3740	4183
Dec 1971		-	310
Jan 1974	Port Office	9631	12371
Jan 1976		-	1598

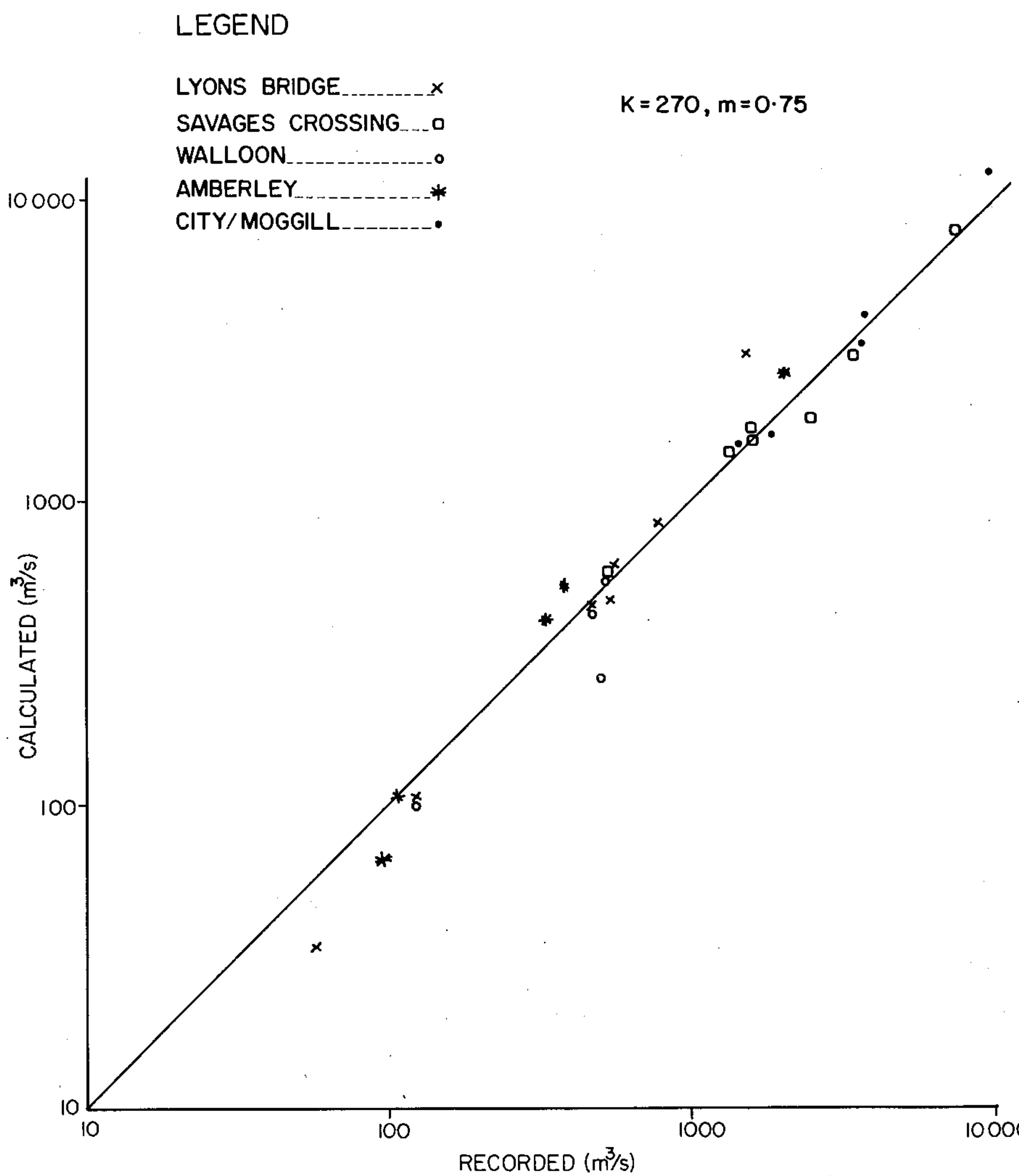
Figure 4 shows a plot of peak discharges and Figure 5 a plot of flood volumes. There are too many floods to include hydrographs for all floods in this report, but as an example the recorded and calculated flood hydrographs for the floods of January 1974 and June 1967 are given in Appendix C.

5.6 Discussion

The model parameters derived in this study for the lower part of the catchment are quite different from those derived in the previous study for the upper part. Since the catchment areas are almost the same, it could be expected that the K value should be the same, but it was considerably higher. The bulk of the streamflow in the lower part of the catchment however came from upstream of Wivenhoe Dam and the catchment in the lower part is much flatter.

The performance of the model calibration is not nearly as good as for the upper catchment, but is still acceptable. Savages Crossing, in particular, has a reasonably poor calibration, but most of the flow into this gauging station comes directly from upstream and varying model calibration would have little effect.

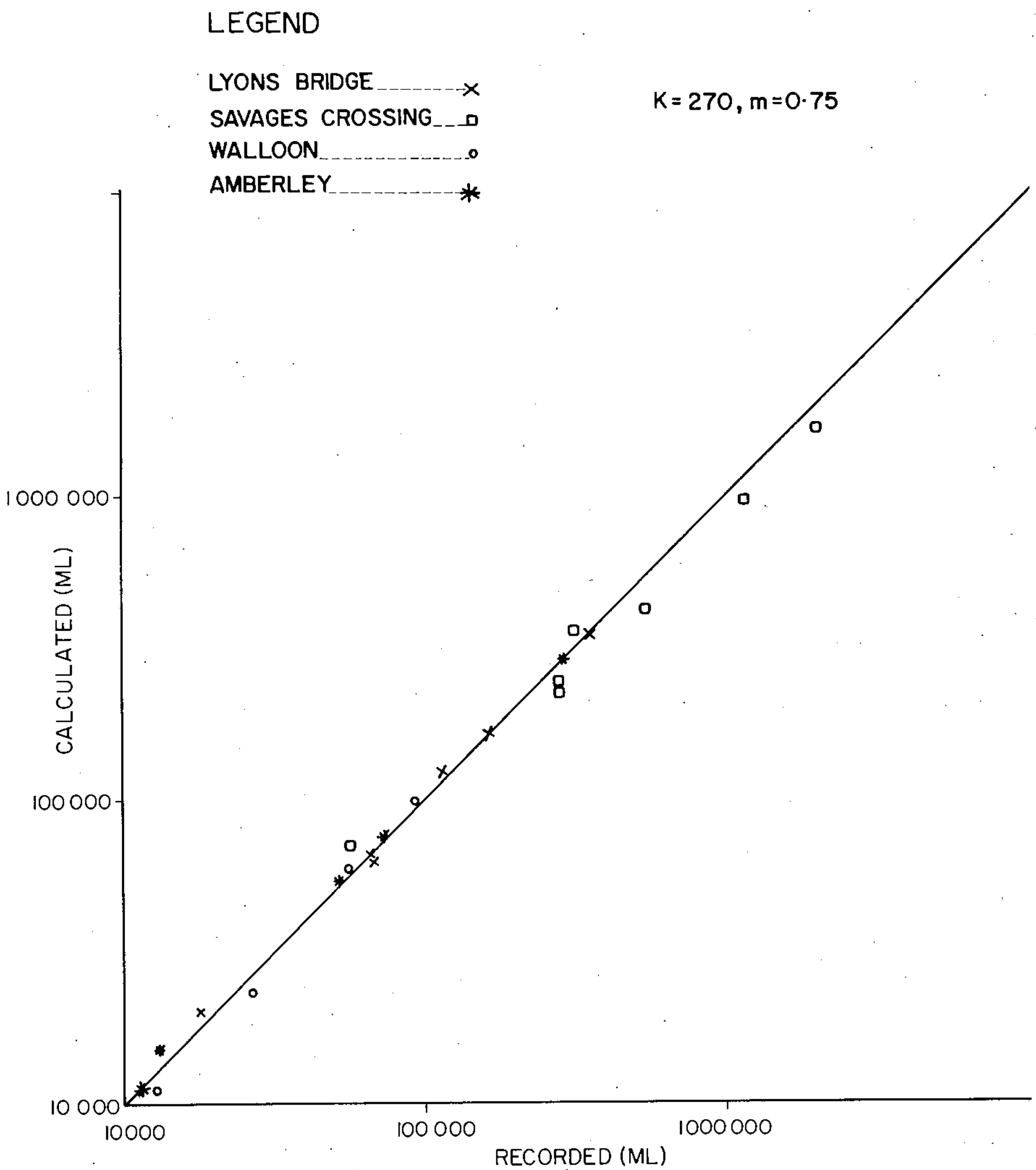
FIGURE 4



MODEL CALIBRATION

COMPARISON OF PEAK DISCHARGES

FIGURE 5



MODEL CALIBRATION

COMPARISON OF FLOOD VOLUMES

6. RAINFALL ANALYSIS

6.1 Introduction

As in the design flood study for Wivenhoe Dam, the primary approach for the calculation of design floods involved the use of a rainfall-runoff model used with design rainfall depths. Flood frequency analyses could not be used in this case, because the objective of the study was to determine the effects of Wivenhoe Dam on the downstream catchment. There were three different rainfall analyses performed, namely a frequency analysis on the annual rainfall series for individual stations (carried out by the Commission), a frequency analysis on the annual series of catchment rainfalls (carried out by the Brisbane City Council), and a Probable Maximum Precipitation (PMP) study (carried out by the Bureau of Meteorology (1983)).

6.2 Frequency Analysis of Rainfalls from Individual Stations

There were twenty one stations in the catchment that had sufficient data for a frequency analysis. The analysis was carried out for each station separately using computer program WS06 (Hadgraft, 1981). The annualseries of one day, two day and three day maxima were extracted and the log normal distribution fitted as is the normal practice with annual series of rainfall data. The log-normal distribution has been found to be the best for using with annual series of rainfall data. The 1%,0.1% and 0.01% annual exceedence probability rainfall depths were calculated from the fitted probability distributions.

The results are listed in Table 14.

The rainfall values vary significantly over the catchment, with high values near the coast and in the mountainous areas in the south. This spatial distribution was used in the flood studies, since this is what is found in the actual data. Figures 6, 7 and 8 show the isohyets drawn for the two day storms of probability 1%, 0.1% and 0.01%, for both above and below Wivenhoe Dam.

It was assumed that the rainfall for the catchment of a given probability was made up of the individual catchment rainfalls of that probability all occurring together.

The design rainfalls for these stations were assumed to apply to the model subareas as shown on Table 15.

The catchment rainfalls for the various durations and exceedence probabilities, using this distribution, were calculated, even though they were not used in any analysis. These catchment rainfalls are listed in Table 16.

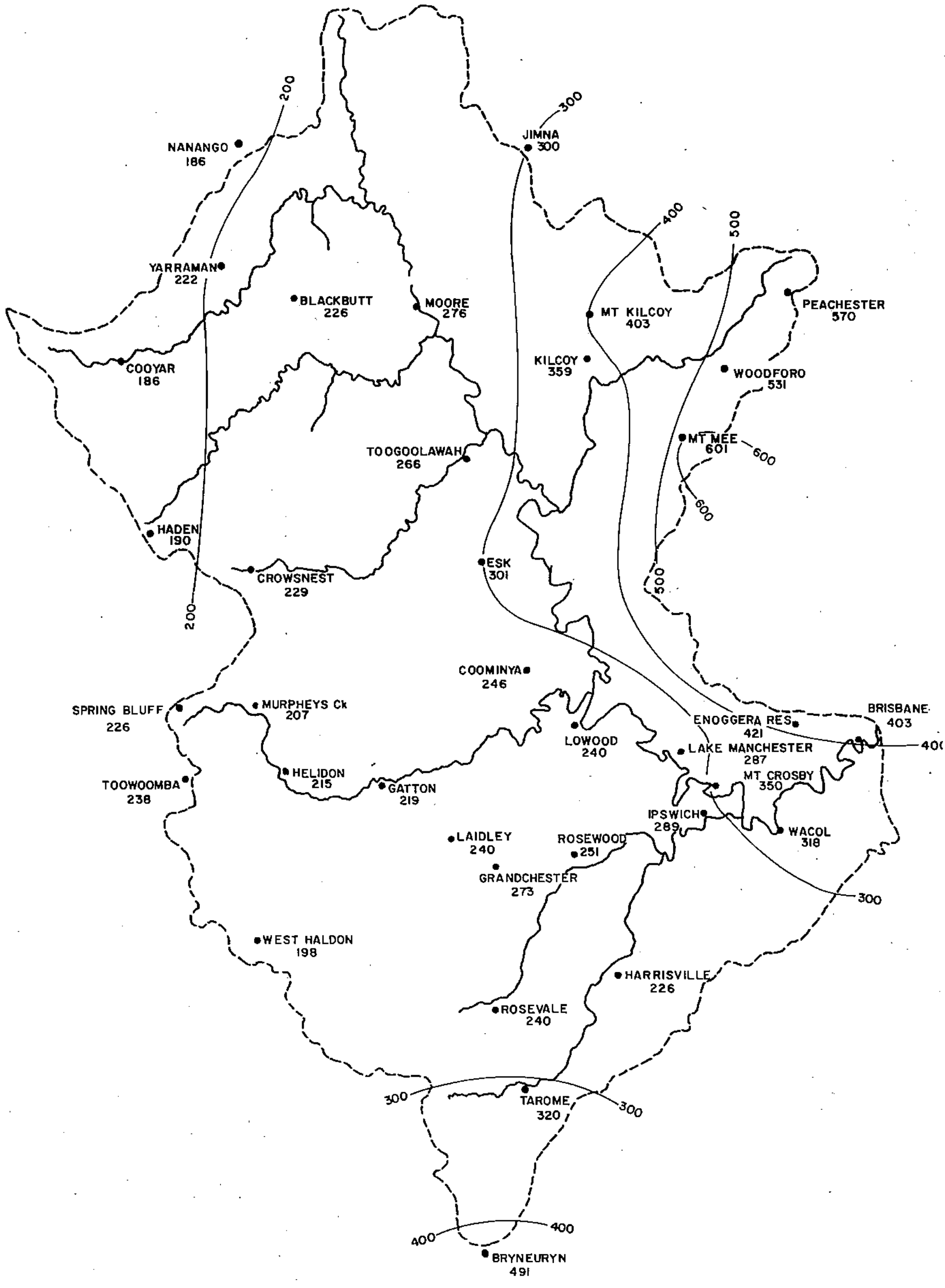
TABLE 14: RAINFALL FREQUENCY ANALYSIS

RAINFALL DEPTH (mm)

STATION		1 DAY			2 DAY			3 DAY	
	1%	0.1%	0.01%	1%	0.1%	0.01%	1%	0.1%	0.01
Bryn Euryn	340	485	650	491	709	960	596	872	1193
Gatton	174	233	297	219	292	372	254	342	437
Grandchester	204	283	370	273	376	489	312	433	566
Harrisville	169	222	279	226	298	374	262	350	444
Helidon	175	236	301	215	287	365	245	328	417
Ipswich	212	295	387	289	407	540	342	489	657
Laidley	178	243	314	240	330	428	281	391	512
Lake Manchester	216	304	403	287	399	521	358	509	680
Lowood	169	225	286	240	329	426	276	381	496
Mt. Crosby	250	358	482	350	512	700	418	621	860
Murphys Creek	157	207	259	207	274	344	237	314	396
Rosevale	180	237	296	240	318	399	269	353	442
Rosewood	184	245	310	251	339	434	304	418	543
Tarome	218	293	373	320	442	577	379	528	694
Brisbane	284	401	533	403	585	794	472	690	942
Enoggera Reservoir	315	464	639	421	619	850	500	741	1025
Wacol	241	333	435	318	443	581	371	520	688
Spring Bluff	162	212	264	226	300	380	277	379	491
Toowoomba	167	221	278	238	319	408	273	369	473
Coominya	177	235	297	246	335	433	285	396	520
West Haldon	128	163	200	198	266	339	226	304	389

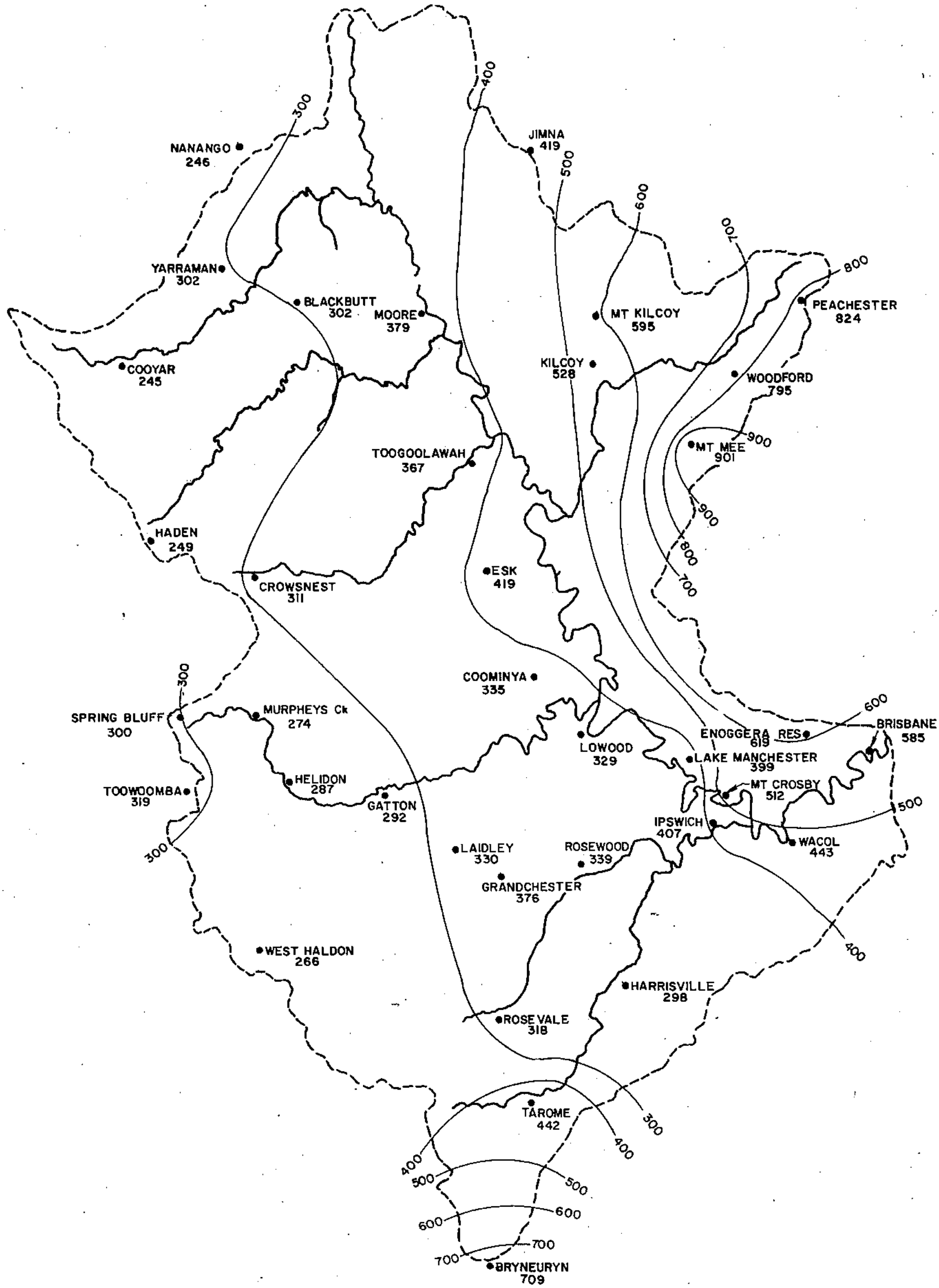
2

FIGURE 6



2 DAY 1·0%

FIGURE 7



2 DAY 0·1%

6.3 Frequency Analysis of Catchment Rainfalls

After considering the previous flood study (Weeks, 1983) the Brisbane City Council adopted a different approach to the estimation of design rainfall depths, by considering the frequency analysis of the annual series of catchment rainfalls. This is in contrast to the separate analyses of station rainfalls considered in the previous report.

Firstly the Council calculated new catchment design rainfalls for Wivenhoe Dam catchment.

Only two day rainfall depths were considered by the Brisbane City Council since two days was found to be the critical storm duration for the catchment in the previous study by the Commission. Mean catchment rainfalls calculated using the two approaches are listed in Table 17.

TABLE 17: TWO DAY WIVENHOE CATCHMENT RAINFALL

(LOG - NORMAL DISTRIBUTION) - FROM BRISBANE CITY COUNCIL

ANNUAL EXCEEDENCE PROBABILITY	USING MEAN CATCHMENT DATA	USING STATION DATA
1	304	298
0.1	486	419
0.01	716	556

This table shows that the two approaches give almost the same rainfall for the 1% probability, but the mean catchment data gives higher rainfalls for the rarer events.

For these catchment rainfalls, the rain was assumed to originate in either the Stanley River catchment or the upper Brisbane River catchment. Factors were calculated from generalised depth-area rainfall data that were used to modify the mean catchment rainfall depth to give subcatchment rainfall depths. These factors are listed in Table 18. The rainfalls downstream of Wivenhoe Dam are not affected by the assumed position of the storm centre.

TABLE 18: RAINFALL DEPTH FACTORS

STORM CENTRE	STANLEY RIVER	UPPER BRISBANE RIVER
Stanley River	1.21	0.86
Upper Brisbane and Upper Middle Reaches	0.95	1.03
Wivenhoe	1.00	1.00
Lockyer Creek	0.83	0.83
Lower Middle Reaches	0.76	0.76

When these factors were applied to the rainfall depths in Table 17, rainfalls for the subcatchments were obtained. These are listed in Table 19.

TABLE 19: TWO DAY RAINFALL DEPTHS (mm) FOR VARIOUS CATCHMENTS

SUBCATCHMENTS / ANNUAL EXCEEDENCE PROBABILITY	STANLEY RIVER	UPPER BRISBANE AND UPPER MIDDLE REACHES	LOCKYER	LOWER MIDDLE REACHES
2% (a)	312	222	214	196
1% (a)	368	261	252	231
0.1% (a)	588	418	403	369
0.01% (a)	866	616	594	544
2% (b)	245	266	214	196
1% (b)	289	313	252	231
0.1% (b)	462	500	403	369
0.01% (b)	680	737	594	544

(a) Stanley River storm centre
(b) Upper Brisbane River storm centre

The Brisbane City Council later added further design storms for analysis and testing of different operating procedures. A large range of thirty different design storms, all of two days' duration, were derived based on the rainfall analyses described above but using the Pearson Type 3 distribution for probability. The revised rainfall modification factors are listed in Table 20.

The Brisbane City Council did not say how these factors were derived but they considered them to be "more realistic".

These rainfall data were not used to calculate design floods, they were analysed to calculate a range of different flood types for the investigation of different operating procedures.

TABLE 20: RAINFALL DEPTH FACTORS (REVISED)

STORM CENTRE	STANLEY RIVER		UPPER BRISBANE RIVER
1. Stanley mean catchment rain	1.52		0.88
2. U/Brisbane & UMR catchment rain	0.66		1.08
3. Wivenhoe Dam catchment rain		1.00	
4. Lockyer catchment rain		0.58	
5. LMR catchment rain		0.67	
6. Bremer catchment rain		0.47	
7. Metropolitan catchment rain		0.24	

TABLE 21: BCC REVISED SUB CATCHMENT RAINFALLS

RAIN	CATCHMENT \ CASE	1	2	3	4	PMP
Standard	Stanley	366 (212)	420 (243)	594 (344)	766 (444)	1 380 (666)
	U/Bris & UMR	159 (260)	182 (298)	258 (422)	333 (544)	910 (1080)
	Lockyer	140	160	227	292	440
	LMR	161	185	262	338	510
	Bremer	113	130	184	237	360
	Metropolitan	58	66	94	121	180
Lower limit	Stanley	292 (207)	334 (237)	473 (336)	610 (433)	1 210 (860)
	U/Bris & UMR	229 (248)	262 (284)	371 (403)	479 (519)	950 (1030)
	Lockyer	200	229	325	418	830
	LMR	183	210	297	383	760
	Bremer	181	207	293	378	750
	Metropolitan	176	201	285	368	730
Upper limit	Stanley	446 (193)	510 (220)	723 (312)	932 (403)	1 380 (660)
	U/Bris &UMR	106 (272)	121 (312)	172 (442)	222 (569)	910 (1080)
	Lockyer	75	85	121	156	310
	LMR	65	75	106	136	270
	Bremer	25	28	39	50	100
	Metropolitan	25	28	39	50	100

29

TABLE 22: PROBABLE MAXIMUM PRECIPITATION (mm)

DURATION	BRISBANE RIVER	WIVENHOE DAM		SOMERSET DAM	
		OVER CATCHMENT	GENERAL DOWNSTREAM	OVER CATCHMENT	GENERAL DOWNSTREAM
6 hrs	220	260	20	400	140
12 hrs	380	380	30	560	200
1 day	560	600	50	840	300
2 days	700	1000	90	1380	500
3 days	880	1260	110	1760	620
4 days	1040	1460	130	2040	720
5 days	1080	1520	130	2120	760
6 days	1100	1560	140	2160	780
7 days	1200	1700	150	2340	840

7. FLOOD OPERATION PROCEDURES FOR WIVENHOE AND SOMERSET DAMS

(This section was provided by Mr. K. Hegerty from the Brisbane City Council.)

Since Wivenhoe Dam and Somerset Dam are to be operated in series to mitigate flooding in the lower catchment, it was important that the most effective combination of procedures for the two dams should be determined. Five procedures for each of Somerset and Wivenhoe Dams, giving a total of twenty-five combinations, were tested. The procedures are described briefly below.

(a) Somerset Dam Procedures

1. Store all water from Stanley River catchment until the inflow to Wivenhoe Dam has peaked and then release water from Somerset Dam at a rate such that the inflow to Wivenhoe Dam is not increased above that peak.

2. Route the Stanley River flood through Somerset Dam with minimal mitigation. Owing to the storage-discharge characteristics of the dam, a significant degree of mitigation can be achieved for some floods using this procedure.

3. Release at 75% of the inflow rate until the upper Brisbane flood peak reaches Gregors Creek gauge and then shut down the releases until the inflow to Wivenhoe Dam has peaked. Releases then begin again at a rate that does not increase the inflow rate to Wivenhoe Dam beyond that peak.

4. Release at 75% of the inflow rate until the upper Brisbane River flood peak reaches Gregors Creek gauge. Releases are then held constant until the inflow to Wivenhoe Dam has peaked and they are then increased without increasing the inflow to Wivenhoe Dam beyond that peak.

5. Raise Somerset Dam spillway gates to permit uncontrolled discharge over the spillway once the flood storage between Full Supply Level and spillway crest level has filled. Low level outlets remain closed until the inflow to Wivenhoe Dam has peaked or the level in Somerset Dam exceeds 102.2 m

(b) Wivenhoe Dam Procedures

The operation of Wivenhoe Dam, for all procedures, begins by storing the whole of the inflow until the lake level exceeds EL 67.250 m. Different procedures are then used, as follows.

1. Release water from Wivenhoe Dam onto the flow from Lockyer Creek at a rate to ensure that Fernvale Bridge is not submerged unncessarily.

2. Release from Wivenhoe Dam onto the flow from Lockyer Creek so that Fernvale Bridge is not submerged prematurely and once Fernvale Bridge is submerged by Lockyer Creek flows, continue the release at a rate such that Mt. Crosby Weir bridge is not submerged.

3. Release from Wivenhoe Dam onto the flow from Lockyer Creek at a rate such that Fernvale and Mt. Crosby Weir bridges are not submerged prematurely. The maximum flow at Lowood is to be the lesser of 3500 m^3/s and either the peak flow of Lockyer Creek or the peak flow of the Bremer River, whichever is the greater.

4. Release from Wivenhoe Dam onto the flow from Lockyer Creek at a rate such that Fernvale and Mt. Crosby Weir bridges are not submerged prematurely. The maximum level on the Lowood flood gauge is not to exceed 14 m.

5. Release from Wivenhoe Dam onto the flow from Lockyer Creek at a rate such that Fernvale and Mt. Crosby Weir bridges are not submerged prematurely. Once it is evident that Mt. Crosby Weir bridge will be submerged, releases can be increased so that the level of the Lowood flood gauge does not exceed 14 m until either (a) the Lockyer Creek flood peak flow passes the junction with the Brisbane River, or (b) the lake level in Wivenhoe Dam exceeds EL 73.5 m. The releases are then increased until the level in Wivenhoe Dam begins to fall.

For procedures 3, 4, and 5 and when the Lockyer Creek peak flow causes the level on the Lowood flood gauge to exceed 14 m, the level on the gauge should return to 14 m as quickly as practicable and is to remain at this level until the flood storages of Wivenhoe and Somerset Dams are emptied.

The opening of the Wivenhoe Dam spillway gates is restricted to one 500 mm increment of one gate per 10 minute time interval for lake levels below EL 71.5 and per 6 minute time interval for lake levels above EL 71.5. For simultaneous operation of two gates, it was assumed that either the gate increments were reduced to 250 mm or that the time intervals were increased to 20 and 12 minutes respectively.

The operating procedures for Wivenhoe Dam were designed so that initial release operations did not adversely affect later operations in the event of the flood magnitude's being larger than originally estimated.

8. DESIGN FLOODS

8.1 Introduction

Design floods were calculated for the catchment downstream of Wivenhoe Dam using three separate conditions, as follows:

(1) No dam in catchment
(2) No releases from dam
(3) Tests of different operating rules, described in Section 7.
(4) Dam operated according to rules defined in Section 10.

This chapter will discuss all cases except that for the Probable Maximum Precipitation case with Wivenhoe Dam in existence. The dam would be overtopped in such an event and that event will be considered in Section 9.

8.2 Results

8.2.1 Inflows to Wivenhoe Dam

Inflows to Wivenhoe Dam were calculated in the previous study (Weeks, 1983) for the Probable Maximum Precipitation and for the design storms based on frequency analysis of records from individuals stations. The peaks of these inflows for 2 day storms are reproduced in Table 29.

For the design rainfalls based on frequency analysis of catchment rainfalls, the same model and the same design rainfall losses as before, i.e. zero initial loss and 2.5 $mm.h^{-1}$ continuing loss, were used. Design flood inflows were calculated for Wivenhoe Dam, both assuming the dam to be in existence and full and with the catchment in its natural condition. Somerset Dam was assumed to be full at the beginning of the storm and operated as described in Section 7 of the previous report for the larger floods (of exceedence probability 0.1% and 0.01%). For the smaller floods, an arbitrary rule that limited outflow to 800 m^3/s was assumed since the floods were too small to use the design flood rule and a simple rule was required because of the large number of floods to be considered. For the design rainfalls estimated using the rainfalls of Table 19, the computed design floods at Wivenhoe Dam (including estimated Somerset Dam release described above) and Somerset Dam are given in Table 23, while Table 24 has the discharges of these floods at other gauging stations in the catchment.

These design floods should be compared with those calculated using the design rainfalls estimated from the station rainfall data. They are listed in Table 25.

TABLE 23: DESIGN FLOODS - USING B.C.C. RAINFALL DATA

ANNUAL EXCEEDENCE PROBABILITY		SOMERSET DAM FLOOD			WIVENHOE DAM FLOOD INFLOW		
		VOLUME (ML)	INFLOW (m^3/s)	OUTFLOW (m^3/s)	VOLUME (ML)	NO DAM (m^3/s)	WITH DAM (m^3/s)
2%	a	252 000	1 820	800	829 000	5 290	5 680
1%	a	327 000	2 340	800	1 123 000	6 970	7 380
0.1%	a	620 000	4 440	2 980	2 319 000	15 590	16 040
0.01%	a	991 000	7 140	4 510	3 814 000	25 150	25 810
2%	b	163 000	1 190	800	984 000	7 190	7 590
1%	b	222 000	1 600	800	1 348 000	9 160	9 610
0.1%	b	452 000	3 230	2 380	2 615 000	18 580	19 150
0.01%	b	742 000	5 330	3 461	4 248 000	29 540	30 210

34

TABLE 24: DESIGN FLOODS USING B.C.C. RAINFALL DATA
CONTRIBUTIONS FROM SUBAREAS ABOVE WIVENHOE DAM

ANNUAL EXCEEDENCE PROBABILITY	2% a	1% a	0.1% a	0.01% a	2%b	1% b	0.1%b	0.01% b
Cooyar Creek								
peak	870	1 150	2 260	3 730	1 180	1 520	2 840	4 560
volume	98 000	135 000	286 000	476 000	140 000	185 000	365 000	592 000
Linville								
peak	1 810	2 400	4 770	7 840	2 470	3 170	6 040	9 730
volume	204 000	282 000	59 700	994 000	292 000	386 000	761 000	1 236 000
Emu Creek								
peak	820	1 090	2 170	3 540	1 130	1 450	2 740	4 370
volume	94 000	129 000	274 000	456 000	134 000	177 000	349 000	567 000
Gregors Creek								
peak	3 430	4 590	9 160	14 950	4 740	6 120	11 560	18 470
volume	395 000	546 000	1 156 000	1 925 000	566 000	748 000	1 475 000	2 395 000
Cressbrook Creek								
peak	310	410	810	1 330	420	540	1 030	1 650
volume	33 000	46 000	97 000	161 000	47 000	63 000	123 000	200 000

35

TABLE 25: DESIGN FLOOD PEAKS USING STATION RAINFALL DATA - FROM WEEKS (1983) FOR 2 DAYS

ANNUAL EXCEEDENCE PROBABILITY	SOMERSET DAM FLOOD VOLUME (ML)	SOMERSET DAM FLOOD INFLOW (m^3/s)	SOMERSET DAM FLOOD OUTFLOW (m^3/s)	WIVENHOE DAM FLOOD INFLOW VOLUME (ML)	WIVENHOE DAM FLOOD INFLOW NO DAM (m^3/s)	WIVENHOE DAM FLOOD INFLOW WITH DAM (m^3/s)
1%	448 000	3 210	2 360	1 234 000	8 300	8 720
0.1%	738 000	5 300	3 440	2 044 000	12 910	13 410
0.01%	1 077 000	7 780	4 900	3 045 000	19 060	19 610
PMP	1 670 000	12 160	8 440	6 170 000	42 170	42 970

It can be noted from a comparison of Tables 23 and 25 that the use of catchment average rainfall analysis has dramatically reduced floods for Somerset Dam, but that those for Wivenhoe Dam have been increased. This is because the high rainfall areas of the Somerset Dam catchment have been averaged out with the lower rainfall areas of the remainder of the catchment.

Both are reasonable approaches to the estimation of design floods, but for the purposes of design floods for Wivenhoe Dam, but it is considered that the originally adopted design rainfall using the individual station analysis are more realistic so it is not suggested that the new floods replace these previously obtained.

For the design rainfalls estimated using the factors of Table 20, inflows to Somerset and Wivenhoe Dams were calculated for each of the thirty cases listed in Table 21. In each case, the dams were assumed to be in existence and full and for Wivenhoe Dam, only the inflow from the upper Brisbane and upper middle reaches catchments was calculated. These design peak inflows are listed in Table 26.

TABLE 26: BRISBANE CITY COUNCIL DESIGN STORMS
PEAK INFLOWS OF FLOODS (m^3/s)

	CASE	SOMERSET DAM	WIVENHOE DAM
1.	U/Bris. average curve - case 1	883	6 532
2.	Stanley average curve - case 1	2 324	2 046
3.	U/Bris. average curve - case 2	1 171	8 166
4.	Stanley average curve - case 2	2 834	3 093
5.	U/Bris. average curve - case 3	2 117	13 518
6.	Stanley average curve - case 3	4 498	6 446
7.	U/Bris. average curve - case 4	3 062	18 805
8.	Stanley average curve - case 4	6 164	9 661
9.	U/Bris. average curve - PMP	5 136	41 870
10.	Stanley average curve - PMP	12 161	34 583
11.	U/Bris. upper limit curve - case 1	837	6 013
12.	Stanley upper limit curve - case 1	1 629	5 185
13.	U/Bris. upper limit curve - case 2	1 115	7 566
14.	Stanley upper limit curve - case 2	2 023	6 619
15.	U/Bris. upper limit curve - case 3	2 042	12 693
16.	Stanley upper limit curve - case 3	3 337	11 303
17.	U/Bris. upper limit curve - case 4	2 958	17 723
18.	Stanley upper limit curve - case 4	4 653	15 990
19.	U/Bris. upper limit curve - PMP	7 077	39 730
20.	Stanley upper limit curve - PMP	10 495	36 300
21.	U/Bris. lower limit curve - case 1	708	7 050
22.	Stanley lower limit curve - case 1	3 081	186
23.	U/Bris. lower limit curve - case 2	957	8 765
24.	Stanley lower limit curve - case 2	3 690	503
25.	U/Bris. lower limit curve - case 3	1 816	14 386
26.	Stanley lower limit curve - case 3	5 746	2 633
27.	U/Bris. lower limit curve - case 4	2 674	19 887
28.	Stanley lower limit curve - case 4	7 779	4 878
29.	U/Bris. lower limit curve - PMP	5 136	41 870
30.	Stanley lower limit curve - PMP	12 161	34 583

8.2.2 Flooding Downstream of Wivenhoe Dam

The runoff routing model was used to calculate design floods using the parameter values K = 270, m = 0.75. In all cases, design losses were initial loss of 0 mm and continuing loss of 2.5 mm/h.

The continuing loss value was that used for all design flood calculations in this branch and its original derivation by Laurenson and Pilgrim (1963). The results for calibration floods in this case though do not appear to contradict the assumptions which seem to be reasonable.

Initially, the case without Wivenhoe Dam was considered for the probabilities 1%, 0.1% and 0.01% for 1, 2 and 3 day durations obtained from analysis of individual stations (Section 6.2) and for the PMP for 1 to 7 day durations (Section 6.4). Peak discharges and flood volumes for the city are listed in Table 27.

TABLE 27: DESIGN FLOODS AT CITY (WITHOUT WIVENHOE DAM)

QWRC DESIGN RAINFALLS

PROBABILITY OF EXCEEDENCE	DURATION (DAYS)	PATTERN	FLOOD VOLUME (ML)	PEAK DISCHARGE (m^3/s)
1%	1		1 970 000	10 600
	2		2 160 000	11 500
	3		1 960 000	9 580
0.1%	1		3 040 000	17 700
	2		3 590 000	19 400
	3		3 660 000	17 500
0.01%	1		4 220 000	26 000
	2		5 270 000	29 600
	3		5 610 000	26 600
PMP	1		6 850 000	53 200
	2		7 870 000	49 400
	3		9 360 000	46 700
	4		11 070 000	52 000
	5	early	11 190 000	53 600
	5	late	11 130 000	52 500
	6	early	10 920 000	51 200
	6	mid	10 610 000	54 400
	6	late	10 870 000	51 600

For the situation with no releases permitted from Wivenhoe Dam, only the two day durations were considered. These results are shown in Table 28.

This case could not be regarded as a practical operating policy since the dam would not be capable of holding the large amount of water required in the low probability cases, but was analysed to help in study the effects of Wivenhoe Dam on Brisbane flooding.

TABLE 28: DESIGN FLOODS AT CITY (NO RELEASE FROM WIVENHOE DAM)

QWRC DESIGN RAINFALL

PROBABILITY OF EXCEEDENCE	FLOOD VOLUME (ML)	PEAK DISCHARGE (m^3/s)
1%	1 000 000	4 480
0.1%	1 690 000	8 520
0.01%	2 450 000	13 400

The flood operation of Wivenhoe Dam was then considered, and the design floods in the downstream catchment were calculated with the dam operated. These results are listed in Table 29. It should be noted that the inflows to Wivenhoe Dam listed in this table are somewhat different from these given as design floods in the previous report because of the use of a different operation procedure for Somerset Dam.

TABLE 29: DESIGN FLOODS (WIVENHOE DAM OPERATED)

QWRC DESIGN RAINFALL

PROBABILITY OF EXCEEDENCE	DURATION (DAYS)	WIVENHOE DAM PEAK INFLOW (m^3/s)	WIVENHOE DAM PEAK OUTFLOW (m^3/s)	AT CITY VOLUME (ML)	AT CITY PEAK FLOW (m^3/s)
1%	1	9 130	3 470	2 320 000	5 190
	2	8 610	3 500	2 720 000	5 510
	3	6 670	3 500	2 610 000	4 740
0.1%	1	14 800	7 200	3 270 000	8 590
	2	13 100	8 090	4 040 000	10 300
	3	10 900	8 370	4 620 000	11 400
0.01%	1	20 600	8 640	4 320 000	12 400
	2	19 300	12 400	5 500 000	17 100
	3	15 800	11 900	6 030 000	19 100

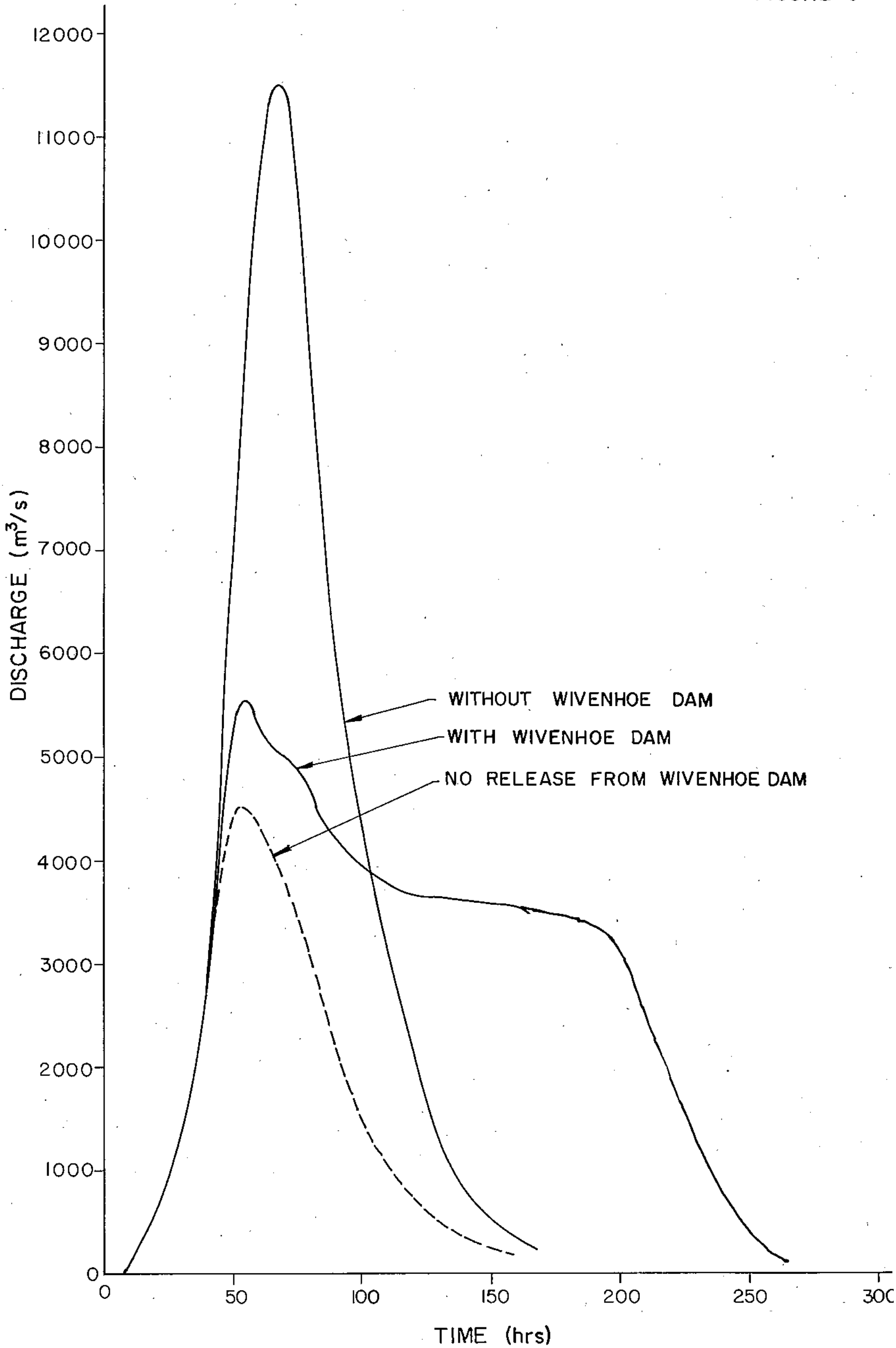
Figures 9, 10 and 11 plot hydrographs at the city for the three cases for 2 day duration storms for 1%, 0.1% and 0.01% annual exceedence probabilities.

Comparison of Tables 27 and 29 shows that operation of Wivenhoe Dam (and Somerset Dam) can provide a significant reduction in peak flood discharge in the city. However it should be remembered that the floods considered here are design floods; there are none of the unusual features that occur in floods that are actually recorded. These design floods have all of the rainfall occurring at the one time while, in practice, storms may move from one end of the catchment to the other, so causing the timing of releases to be different. Some unusual conditions could cause quite different results.

The Brisbane City Council routed the inflows for its design floods through Wivenhoe Dam using the operating rules defined in Section 7. The outflows from the dam were then routed through the downstream catchment to Brisbane city using the runoff routing model described in Section 5.

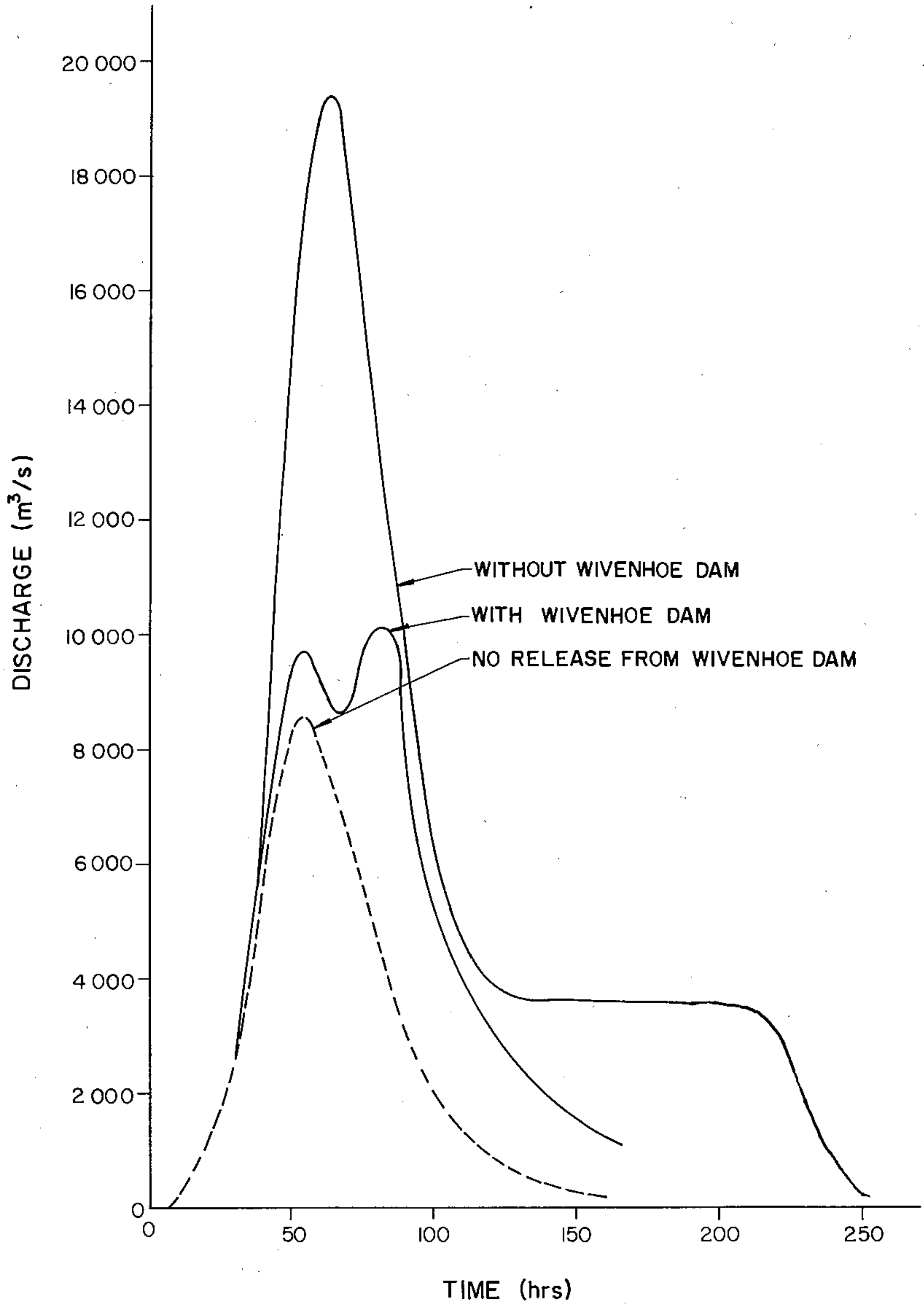
As a first step in the analysis of the Brisbane City Council storms, the cases without any release from Wivenhoe Dam were considered. Since, with no release from Wivenhoe Dam, there were no differences between cases with storms centred on the Stanley and Upper Brisbane River catchments, there were only fifteen cases to be considered for the floods based on the rainfalls of Table 21. The peak discharges in the city reach for these cases are listed in Table 30.

FIGURE 9



BRISBANE RIVER AT CITY
1·0% 2 DAY STORM

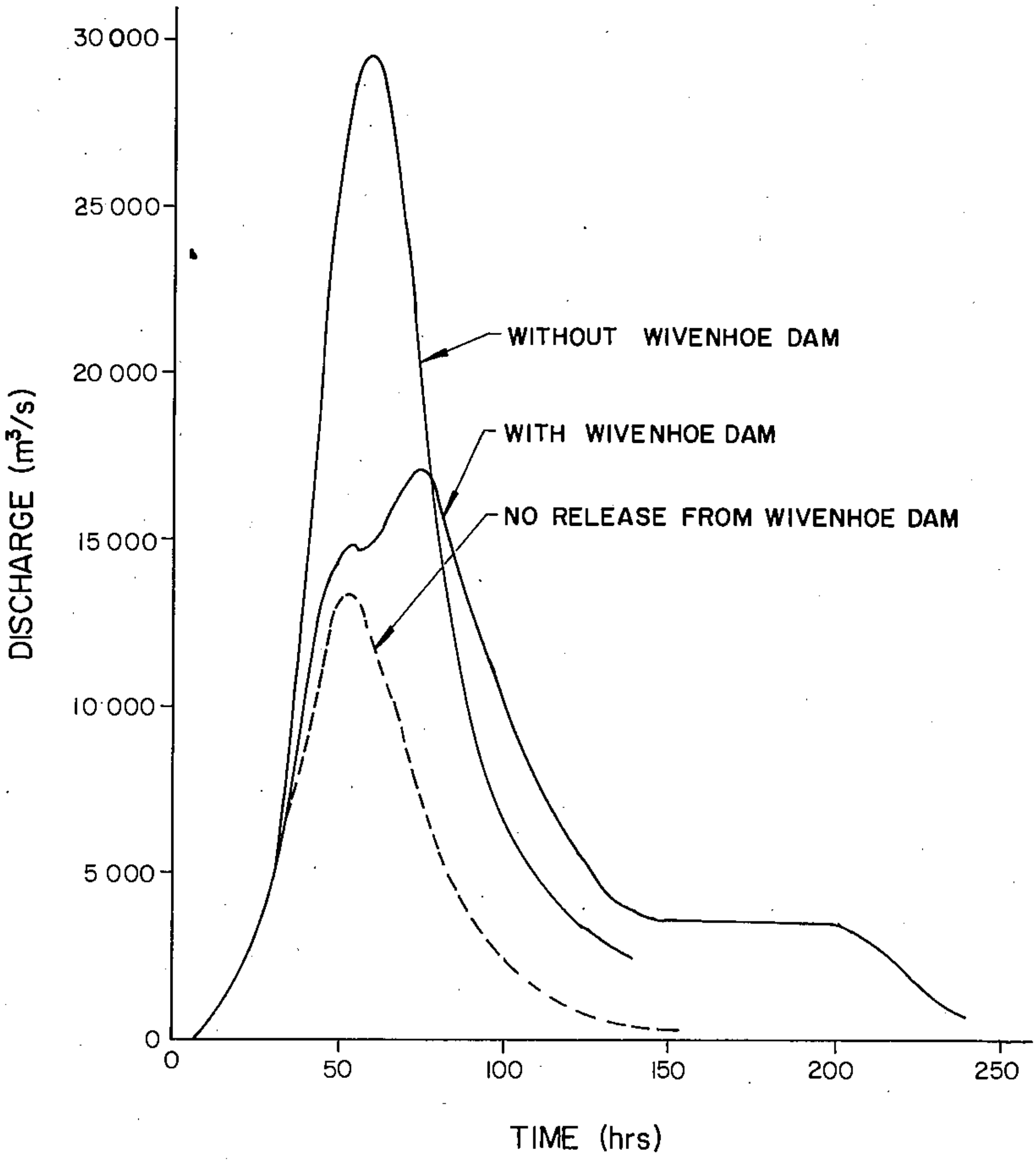
FIGURE 10



BRISBANE RIVER AT CITY

0·1% 2 DAY STORM

FIGURE 11



BRISBANE RIVER AT CITY

0·01% 2 DAY STORM

TABLE 30: BCC DESIGN STORMS PEAK DISCHARGE IN CITY (m^3/s)
NO RELEASE FROM WIVENHOE DAM

CASE	PEAK DISCHARGE IN CITY
1. average curve - case 1	405
2. average curve - case 2	797
3. average curve - case 3	2 665
4. average curve - case 4	4 869
5. average curve - PMP	10 470
6. upper limit curve - case 1	1 938
7. upper limit curve - case 2	2 868
8. upper limit curve - case 3	6 308
9. upper limit curve - case 4	10 024
10. upper limit curve - PMP	28 031
11. lower limit curve - case 1	0
12. lower limit curve - case 2	0
13. lower limit curve - case 3	124
14. lower limit curve - case 4	550
15. lower limit curve - PMP	3 702

Design floods were calculated using various operating policies for the two dams using the inflows and storm distributions provided by the Brisbane City Council. Firstly the original rainfall patterns given in Table 19 were used and when routed through the dams, gave the results shown in Table 31. Then the rainfall patterns given in Table 21 were used to produce the results shown in Table 32. In each case, there are two figures inside brackets separated by a slash. The first refers to the number of the operation rule used to operate Somerset Dam and the second to the operation rule used to operate Wivenhoe Dam.

All cases analysed here were selected by the Brisbane City Council and the results were then supplied to them for their consideration.

TABLE 31: PEAK DISCHARGE AT CITY FOR DIFFERENT BCC STORMS

CASE	PEAK DISCHARGE (m^3/s)
1. Stanley - case 1 (3/2)	1 656
2. Stanley - case 1 (5/2)	1 656
3. U/Brisbane - case 1 (3/4)	3 542
4. U/Brisbane - case 1 (5/4)	3 543
5. Stanley - case 2 (3/4)	3 523
6. Stanley - case 2 (5/4)	3 437
7. U/Brisbane - case 2 (3/4)	3 562
8. U/Brisbane - case 2 (5/4)	3 562
9. Stanley - case 3 (3/4)	6 776
10. Stanley - case 3 (5/4)	6 775
11. U/Brisbane - case 3 (3/4)	8 837
12. U/Brisbane - case 3 (5/4)	8 430
13. Stanley - case 4 (3/4)	12 017
14. Stanley - case 4 (5/4)	11 777
15. U/Brisbane - case 4 (5/4)	14 816

TABLE 32: PEAK DISCHARGE AT THE CITY FOR DIFFERENT BCC STORMS

	CASE	PEAK DISCHARGE (m^3/s)
1.	standard - case 1 Stanley (5/2)	1 645
2.	standard - case 1 U/Brisbane (5/4)	3 085
3.	standard - case 2 Stanley (5/4)	3 580
4.	standard - case 2 U/Brisbane (5/4)	3 581
5.	standard - case 3 Stanley (5/4)	4 019
6.	standard - case 3 U/Brisbane (5/5)	9 099
7.	standard - case 4 Stanley (5/5)	9 700
8.	standard - case 4 U/Brisbane (5/5)	13 750
9.	upper limit - case 1 Stanley (5/4)	3 659
10.	upper limit - case 1 U/Brisbane (5/4)	3 659
11.	upper limit - case 2 Stanley (5/4)	4 179
12.	upper limit - case 2 U/Brisbane (5/4)	4 184
13.	upper limit - case 3 Stanley (5/5)	9 949
14.	upper limit - case 3 U/Brisbane (5/5)	11 289
15.	upper limit - case 4 Stanley (5/5)	15 858
16.	upper limit - case 4 U/Brisbane (5/5)	17 341
17.	lower limit - case 1 Stanley (5/2)	1 599
18.	lower limit - case 1 U/Brisbane (5/4)	3 499
19.	lower limit - case 2 Stanley (5/4)	3 251
20.	lower limit - case 2 U/Brisbane (5/4)	3 498
21.	lower limit - case 3 Stanley (5/4)	3 514
22.	lower limit - case 3 U/Brisbane (5/5)	7 612
23.	lower limit - case 4 Stanley (5/4)	3 529
24.	lower limit - case 4 U/Brisbane (5/5)	11 516

All of these simulations were given to the Brisbane City Council where they provided the basis for the establishment of operating rules.

After these results had been completed, the Council advised that two of the depth factors in Table 20 had been accidentally transposed, and therefore the design rainfalls in Table 21 were incorrect. Since the operating rules for the dams had already beeen evaluated using these design floods and the Council advised that the discharge hydrographs for Wivenhoe Dam for the corrected design floods were not significantly different, it was decided not to route the corrected design floods down the river.

8.3 Discussion

This analysis shows that the operation of Wivenhoe Dam can have a significant effect on the magnitude of floods in the city reaches of the Brisbane River. Wivenhoe Dam stores the flood during the time of its highest discharge and then releases it after the peak has passed, thereby increasing the duration of low level flooding. The peak discharge with Wivenhoe Dam operated is not much higher than the peak discharge when there are no releases from the dam.

However the floods considered here are design cases, and do not consider any of the possible unusual situations that could occur. These unusual situations could produce much smaller reduction in flood peaks than what has been indicated here. Since only a very limited range of flood types have been investigated, many others, such as those with a longer duration or with the storms in the different subcatchments not at the same time, could cause quite different results. These design floods do not completely describe the actual flood mitigation effects that may occur in any particular storm that may occur.

9. WIVENHOE DAMBREAK ANALYSIS

Designs Branch commissioned Dr. C. Joy, an engineering consultant, to investigate the process of failure of Wivenhoe Dam in floods caused by the Probable Maximum Precipitation. Nine floods were provided which gave results for different failure conditions for the two day and seven day duration storms for the Probable Maximum Precipitation over the Wivenhoe Dam catchment.

Details on how these floods were derived or details on the parameters considered have not been provided by the consultant, so they cannot be interpreted here.

The very limited information that was provided on the failure floods is listed in Table 33.

TABLE 33: WIVENHOE DAMBREAK CASES CONSIDERED

RUN NO.	DURATION (days)
1A-2	2
1B-2	7
2A-2	2
2B-2	7
3A-2	2
3B-2	7
4A-2	2
4B-2	7
6A-2	2

The peak discharges of the inflow hydrographs for these two floods were:

2 day : 43 000 m^3/s
7 day : 48 000 m^3/s

The peak discharges of the outflow hydrographs for the nine cases are listed in Table 34.

TABLE 34: WIVENHOE DAMBREAK PEAK OUTFLOW DISCHARGE

CASE	PEAK DISCHARGE (m^3/s)
1A-2	61 356
1B-2	61 627
2A-2	114 810
2B-2	114 900
3A-2	164 950
3B-2	165 310
4A-2	213 550
4B-2	213 940
6A-2	70 431

Even though these floods were far higher than those used for model calibration, it was decided to use the runoff routing model to route the floods downstream. The Bureau of Meteorology had given estimates of the general rainfall that could be expected downstream when the Probable Maximum Precipitation occurred over the Wivenhoe Dam Catchment. These rainfalls were:

2 day : 90 mm
7 day : 150 mm

These floods were routed downstream and resulted in the peak discharges in the city listed in Table 35.

TABLE 35: PEAK DISCHARGE IN CITY - WIVENHOE DAMBREAK CASES

CASE	PEAK DISCHARGE (m^3/s)
1A-2	51 800
1B-2	53 300
2A-2	79 700
2B-2	83 800
3A-2	101 700
3B-2	107 000
4A-2	120 600
4B-2	112 700
6A-2	53 600

These flood peaks must be considered with extreme caution since the assumptions of the analysis would not hold at these extreme discharges, but they are included for comparison with the results for cases with no dam break.

APPENDIX A

QUALITY OF GAUGING STATION RECORDS

BY A. SEABROOK

1. BRISBANE RIVER AT SAVAGES CROSSING (143001C)

In November, 1958, the first of the Commission type GE transducers was installed on the left bank upstream of the bridge at Savages Crossing. This site is 10.2 km downstream of Vernors and immediately upstream of Banks Creek at AMTD 130.8 km.

As a gauging station, the site is superior to Lowood and Vernors. The reach is of adequate length, the banks comparatively symmetrical and floods are confined to a significantly smaller section than they were at the other stations. The 1893 flood was confined within a width of 400 m, although I suspect that the river broke out upstream and spread behind the confinement of the right bank.

The control is stable gravel, but the low flow rating has a tendency to fluctuate because of excess weed growth during drier periods.

The cableway has a span of 225 m but the footings are about 4.5 m below the 1893 flood level. I am not aware of why the cableway was constructed below the maximum recorded flood height, but the chance of being on site to measure a flood of such magnitude is remote.

Flood measurements have been obtained to 3 360 m^3/s, at 20 m gauge height. The 1974 flood peaked at 23.79 m with an extrapolated discharge of 7 700 m^3/s. The 1893 flood reached an estimated height of 27 m and discharge of 10 000 m^3/s.

The station is well rated up to the peak measurement and the extension of the rating curve appears reliable. Although the station is measured only to about 35% of the 1893 flood discharge, it is unlikely that higher measurements will be obtained.

2. LOCKYER CREEK AT LYONS BRIDGE (143210)

In 1964, a GE installation and cableway was established 2.4 km downstream of Tarampa to register flow in the lower Lockyer Creek.

The record of this station is reliable although there are significant periods of derived record, but fortunately the close proximity of the Bureau's flood warning station enables record to be derived with accuracy.

Fifteen rating curves have been used to rate the 23 years of record. Two high flow rating curves have been drawn, the first is applicable to November, 1979; I doubt the necessity for two curves, the second curve gives less discharge below 7 m and more above; the mean velocities that I've referred to were obtained from the second curve. All other curves embrace the low flow variation which at three m^3/s may vary by 50%.

All stream flow measurements have been compared with the daily volume report and the majority of measurements fall within $\pm$ 5% of the published record. The more excessive variations are in the extreme low flow range and generally when the flow has been estimated.

In conclusion I find the published record to be acceptable. The periods of missing record from 1974 to 1980 are too extensive to be satisfactorily derived and must remain as lost record.

```
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        METRIC.
        69   SUBAREAS OF AREA:
        19.4 53.8 48.1 134.7 90.7 129 79.4 104.2 81.5 172.2 87.2 81.5 144.5
        159.4 112 90.7 232.4 76 78 88.8 182.9 98.9 116.3 47.8 140.5 90.8 60.
        97.7 70 105.6 47.5 96.4 66.6 75.9 261.4 109.2 90 73.7 75.9 110.4 87.
        50 79.1 142.9 93 94.3 103.5 94.9 97.6 71.4 132.3 50.4 68.1 59 74.7
        80.6 60.9 68.7 102.2 48.5 142.1 111.4 104.2 172.3 70.1 79.2 99.5
        222 239
        INPUT HYDROGRAPH AT WIVENHOE DAM.
        PRINT WIVENHOE DAM OUTFLOW.
        STORE.
        RAIN ON AREA # 69  K1=.112
        ADD RAIN ON AREA # 68  K1=.061
        STORE.
        RAIN ON AREA # 67  K1=.069
        ADD RAIN ON AREA # 66  K1=.052
        GET.
        ROUTE K1=.043
        STORE.
        RAIN AREA # 65  K1=.061
        GET.
        ROUTE K1=.009
        STORE.
        RAIN # 64  K1=.121
        ADD RAIN ON AREA # 63  K1=.086
        GET.
        ADD RAIN ON AREA # 57  K1=.035
        STORE.
        RAIN ON AREA # 62  K1=.069
        STORE.
        RAIN ON AREA # 61  K1=.086
        GET.
        ROUTE K1=.035
        ADD RAIN ON AREA # 60  K1=.061
        STORE.
        RAIN ON AREA # 59  K1=.069
        GET.
        ROUTE K1=.069
        ADD RAIN ON AREA # 58  K1=.086
        GET.
        ROUTE K1=.043
        STORE.
        RAIN ON AREA # 56  K1=.104
        GET.
        ROUTE K1=.035
        STORE.
        RAIN ON AREA # 55 K1=.078
        GET.
        ROUTE K1=.017
        ADD RAIN ON AREA # 54  K1=.061
        STORE.
        RAIN ON AREA # 51  K1=.104
        ADD RAIN ON AREA # 50  K1=.104
        ADD RAIN ON AREA # 49  K1=.069
        STORE.
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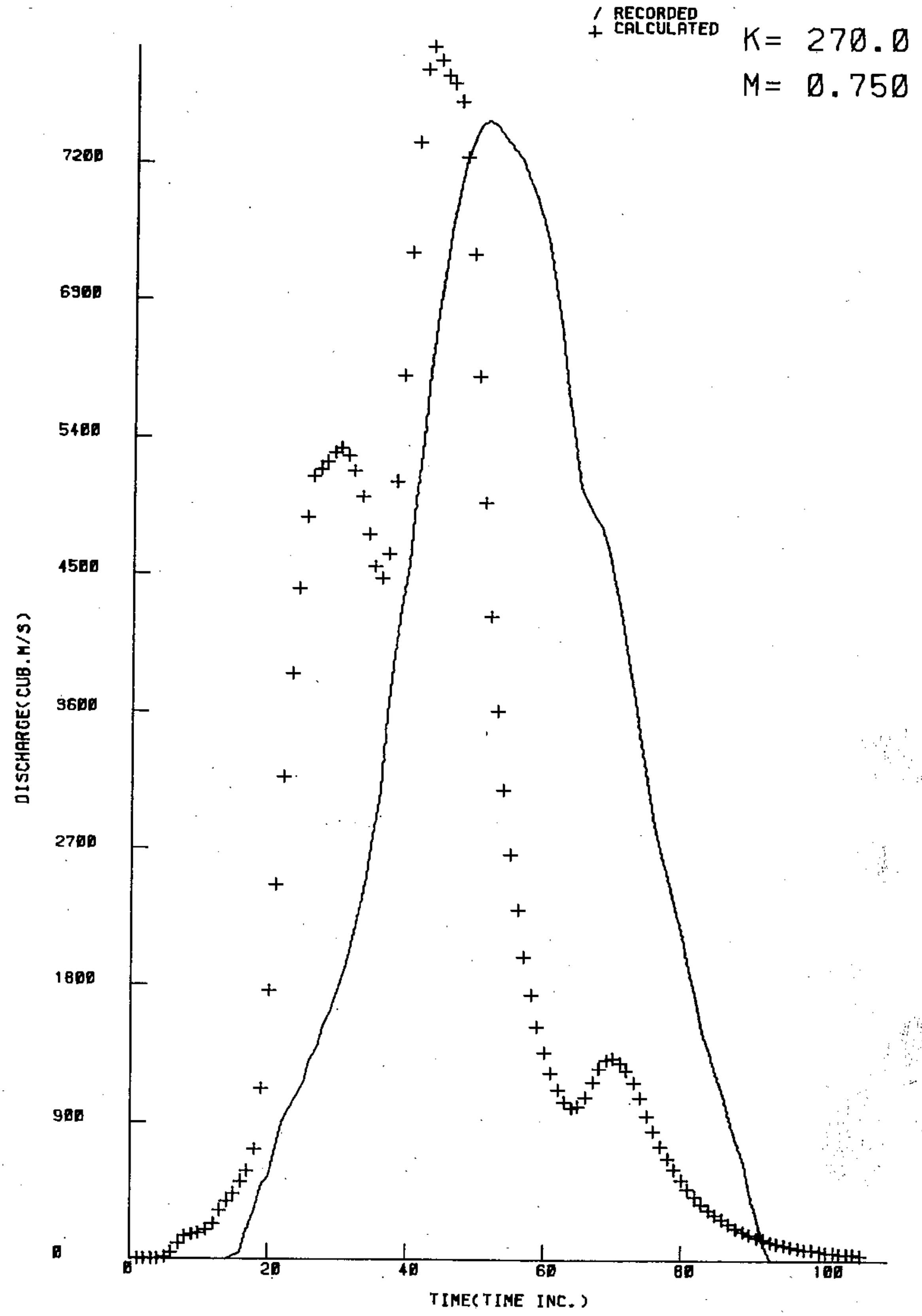
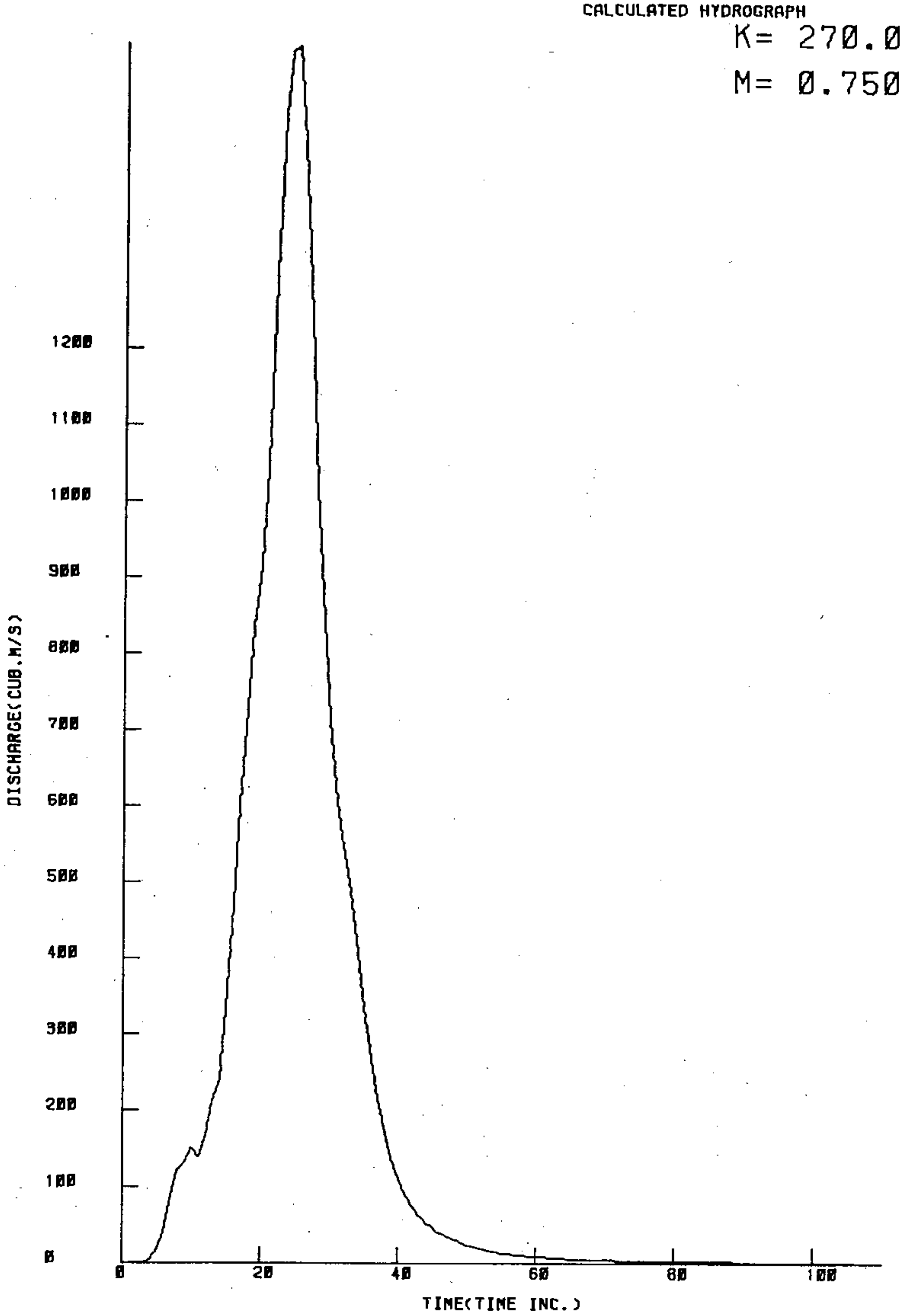
BRISBANE R AT SAVAGES XING - JAN 1974.

RUNOFF ROUTING (WT87) MODEL



BREMER RIVER AT WALLOON - JAN 1974

RUNOFF ROUTING (WT87) MODEL



WARRILL CREEK AT AMBERLEY - JAN 1974

RUNOFF ROUTING (WT87) MODEL

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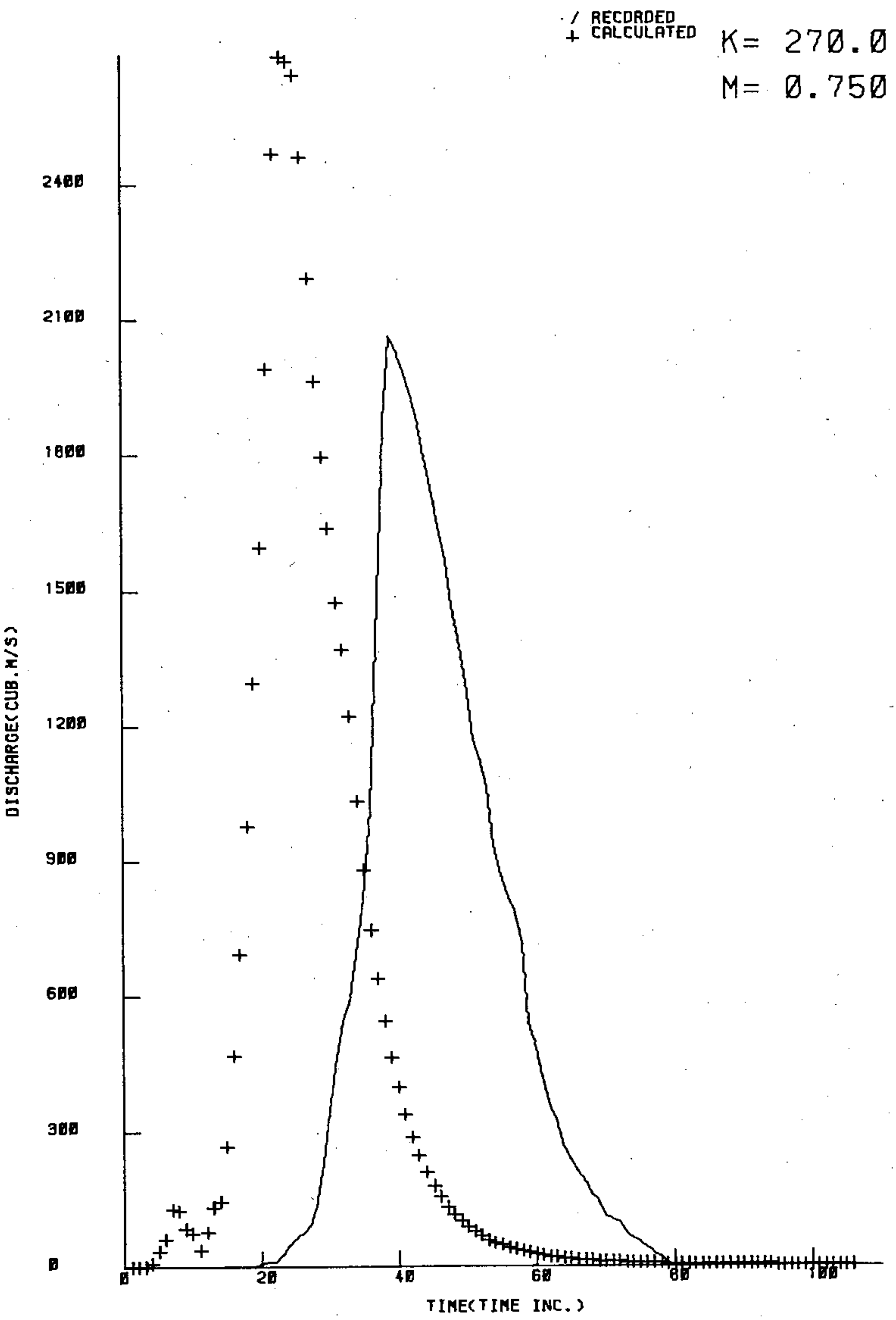
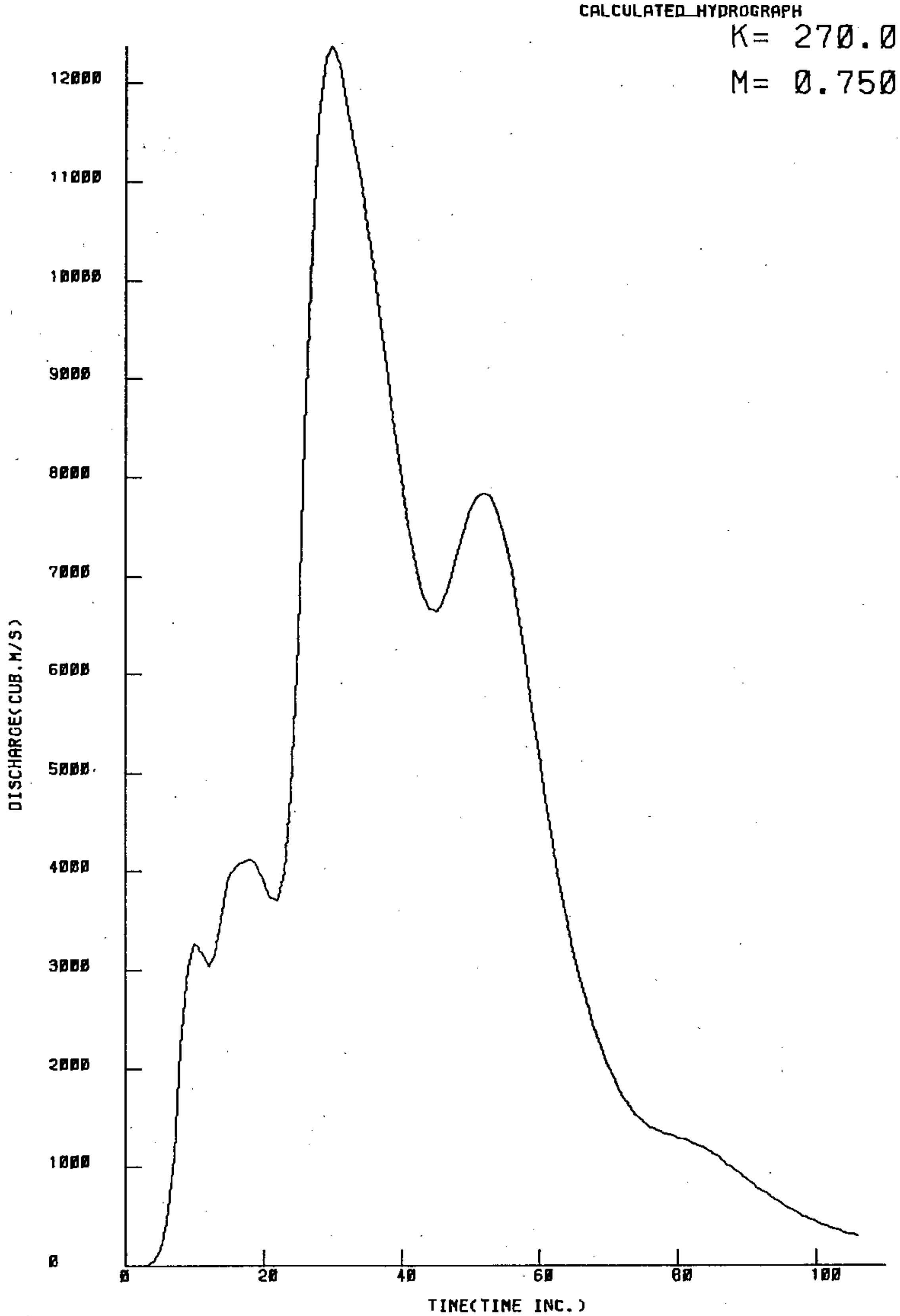
M= 0.750



BRISBANE R AT CITY - JAN 1974

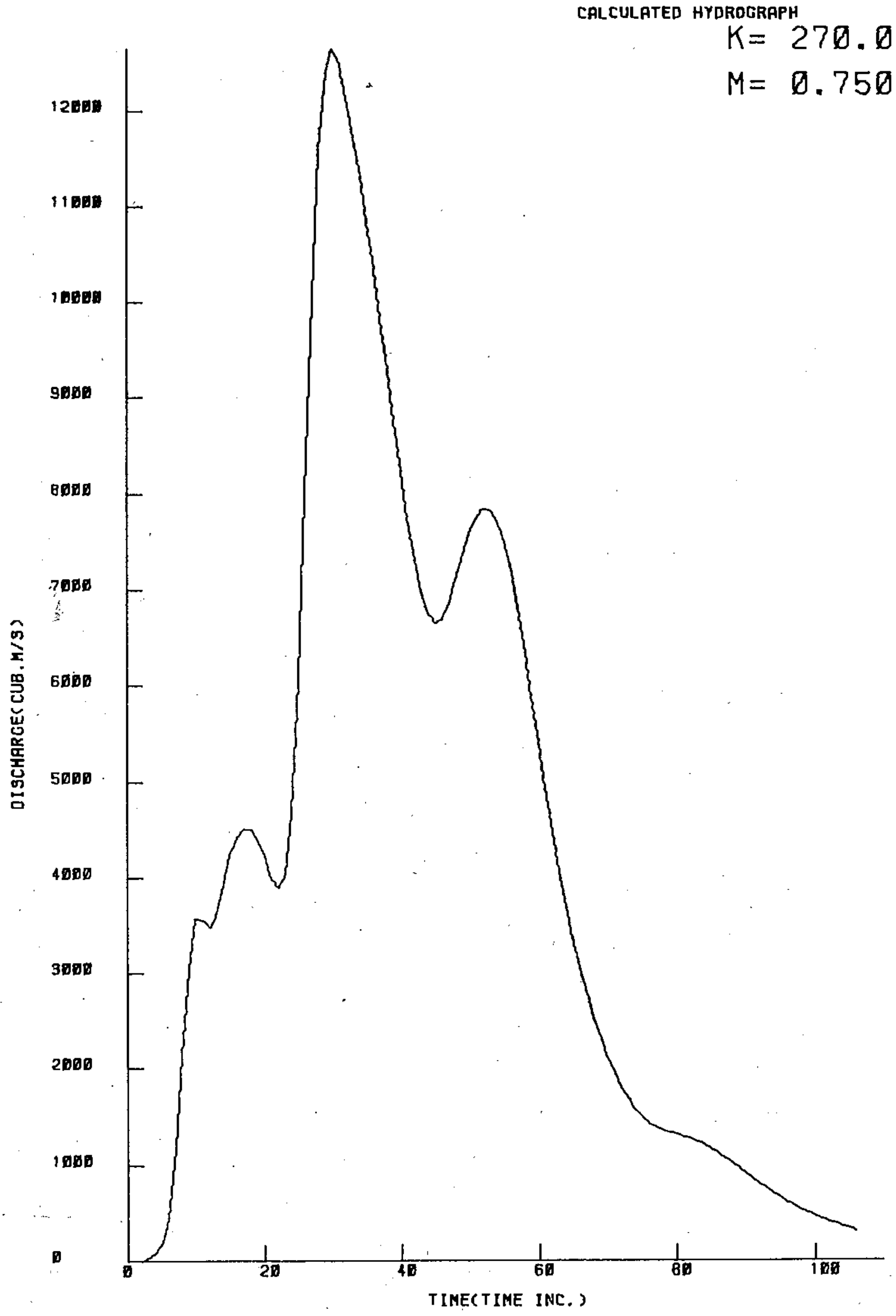
RUNOFF ROUTING (WT87) MODEL

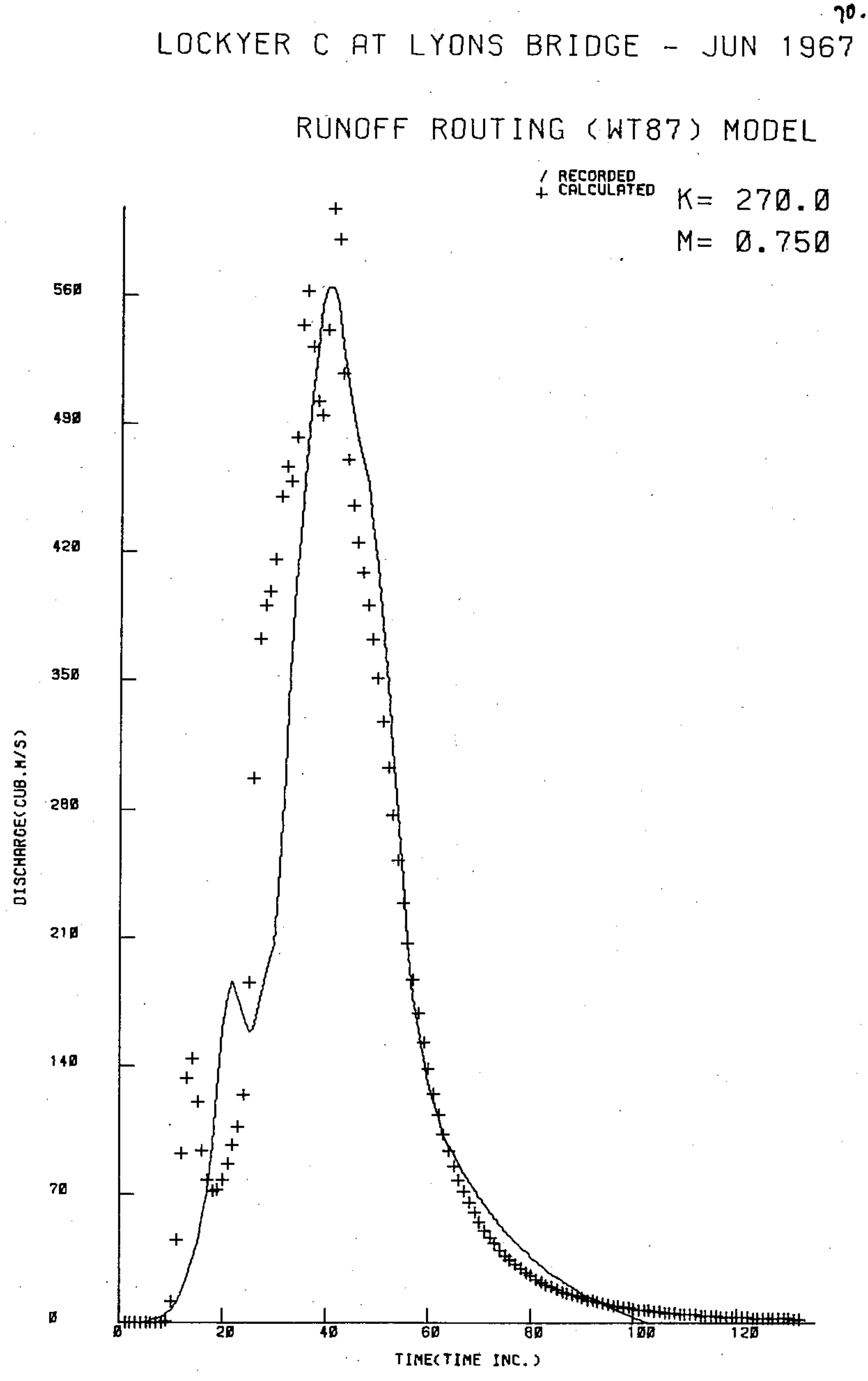


69.

BRISBANE R BELOW BREAKFAST C - JAN 1974

RUNOFF ROUTING (WT87) MODEL

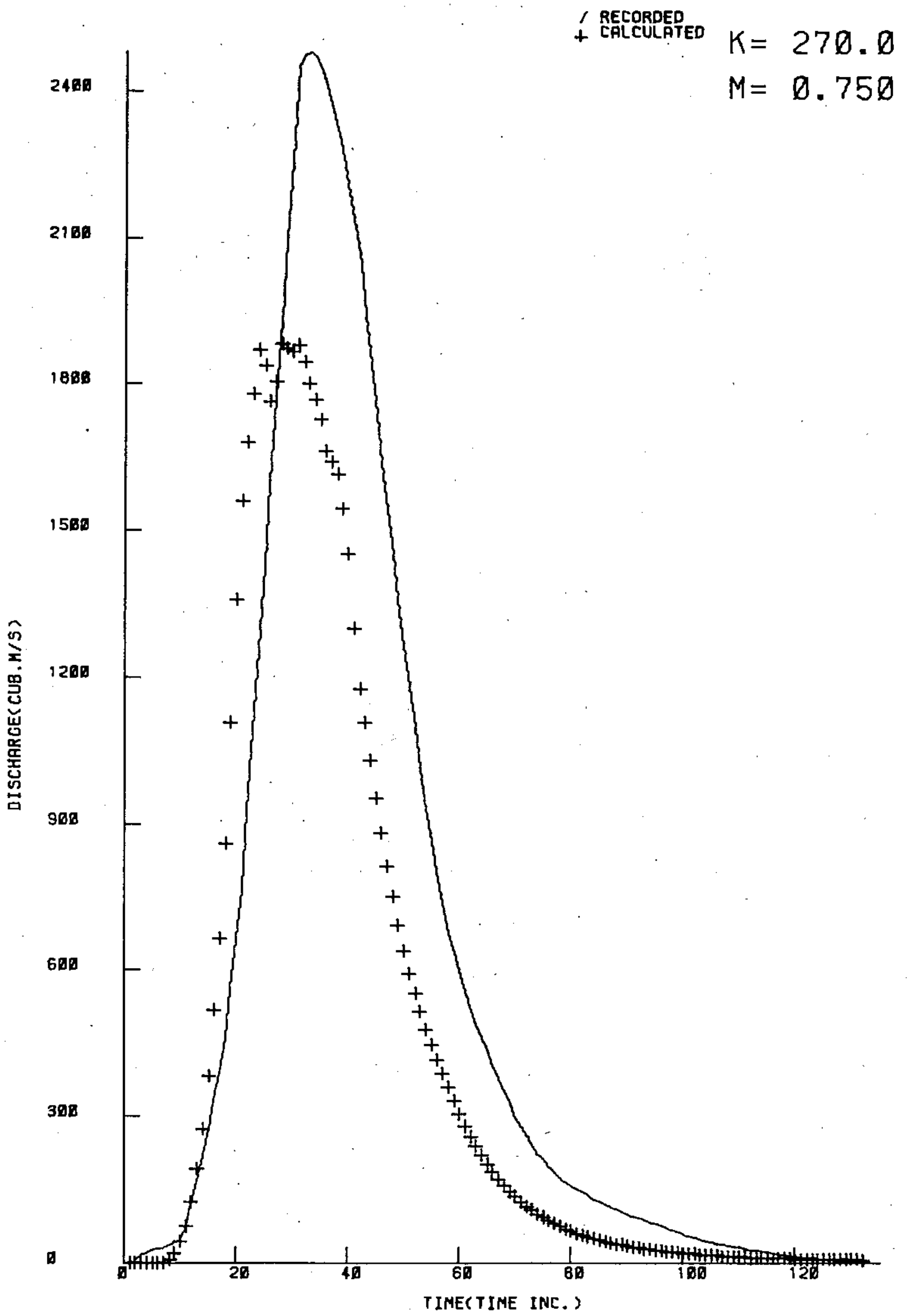




LOCKYER C AT LYONS BRIDGE - JUN 1967
RUNOFF ROUTING (WT87) MODEL
/ RECORDED
+ CALCULATED
K= 270.0
M= 0.750
DISCHARGE(CUB.M/S)
0
70
140
210
280
350
420
490
560
0
20
40
60
80
100
120
TIME(TIME INC.)

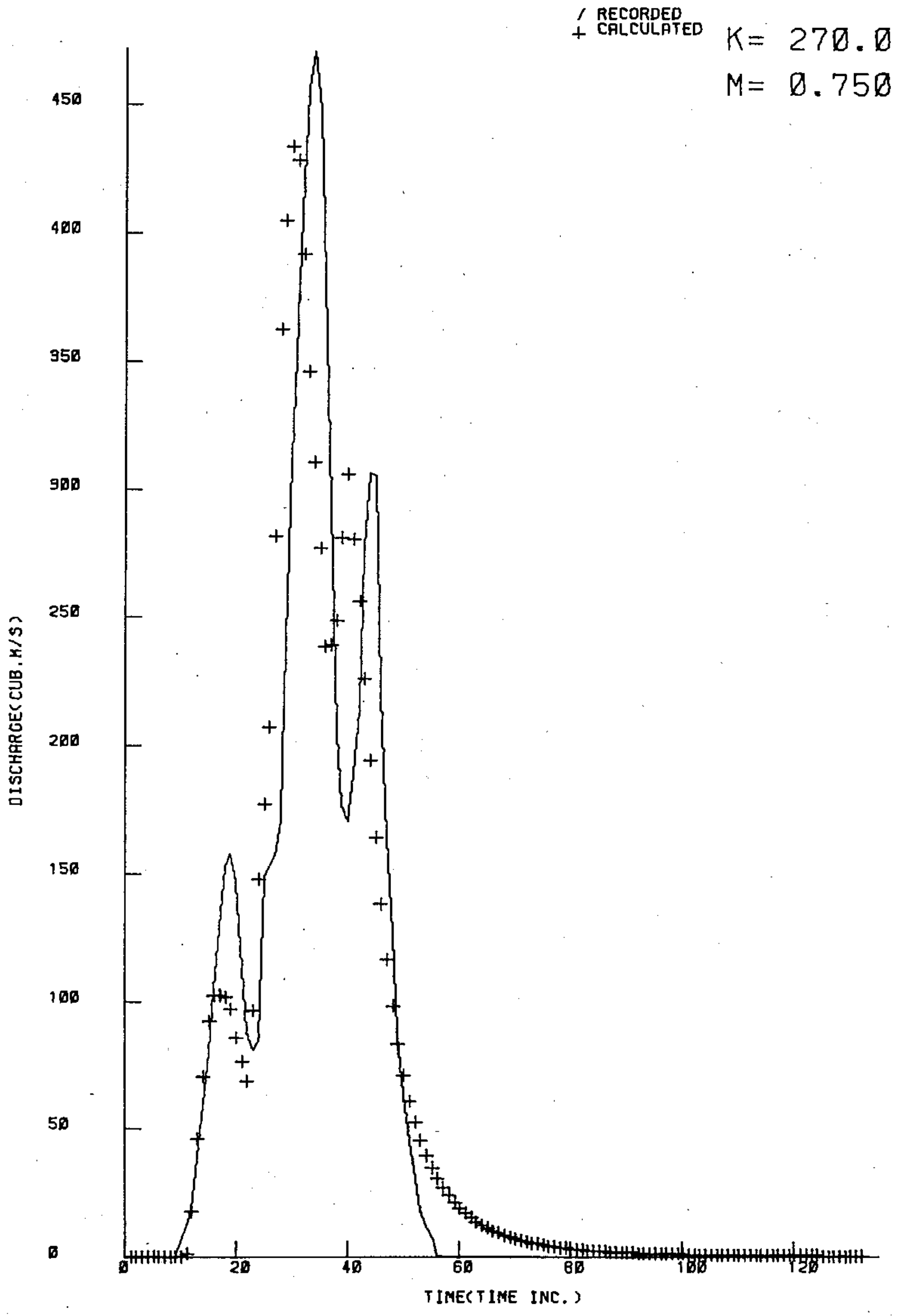
BRISBANE R AT SAVAGES XING - JUN 1967

RUNOFF ROUTING (WT87) MODEL



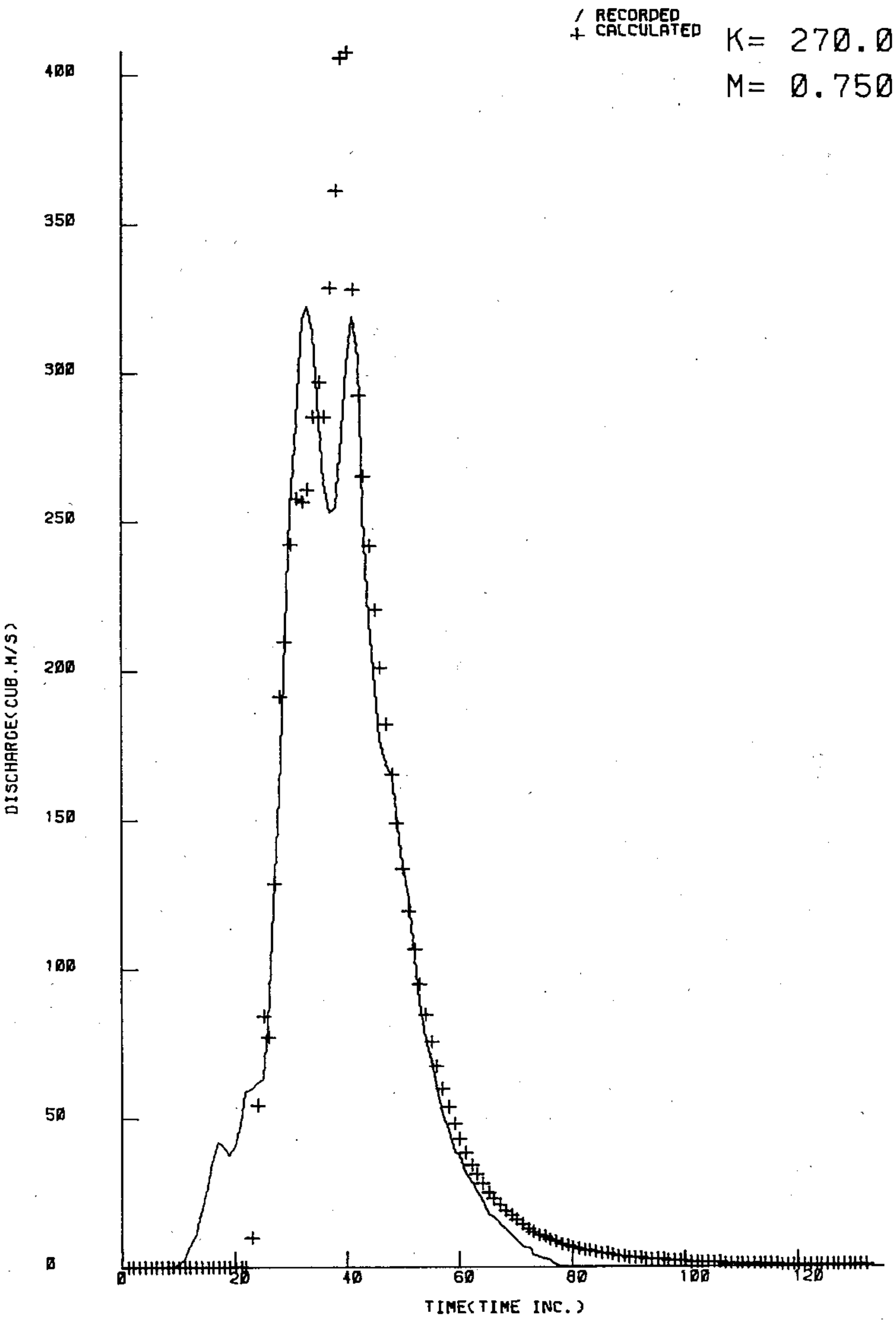
BREMER RIVER AT WALLOON - JUNE 1967

RUNOFF ROUTING (WT87) MODEL



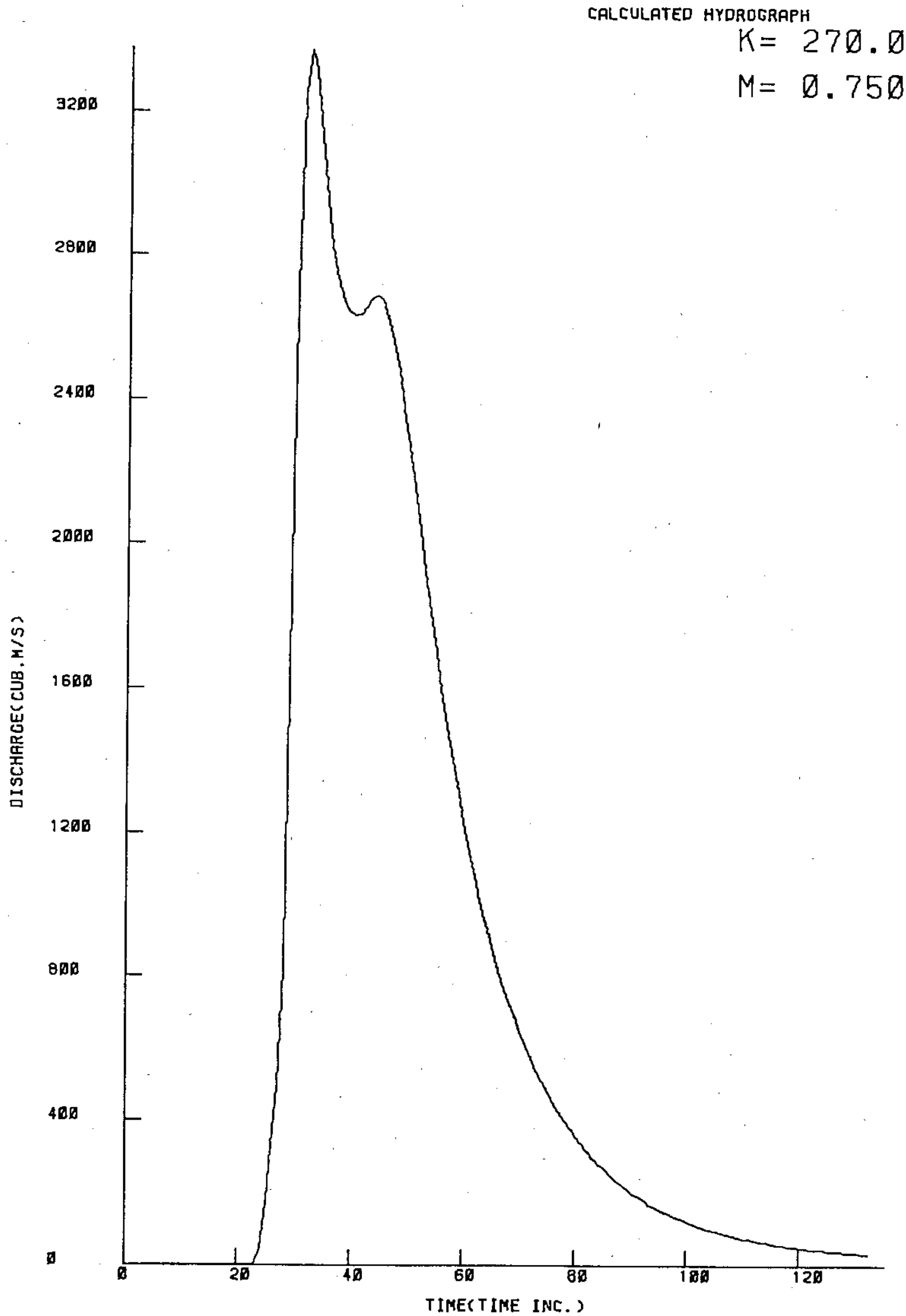
WARRILL CREEK AT AMBERLEY - JUN 1967

RUNOFF ROUTING (WT87) MODEL



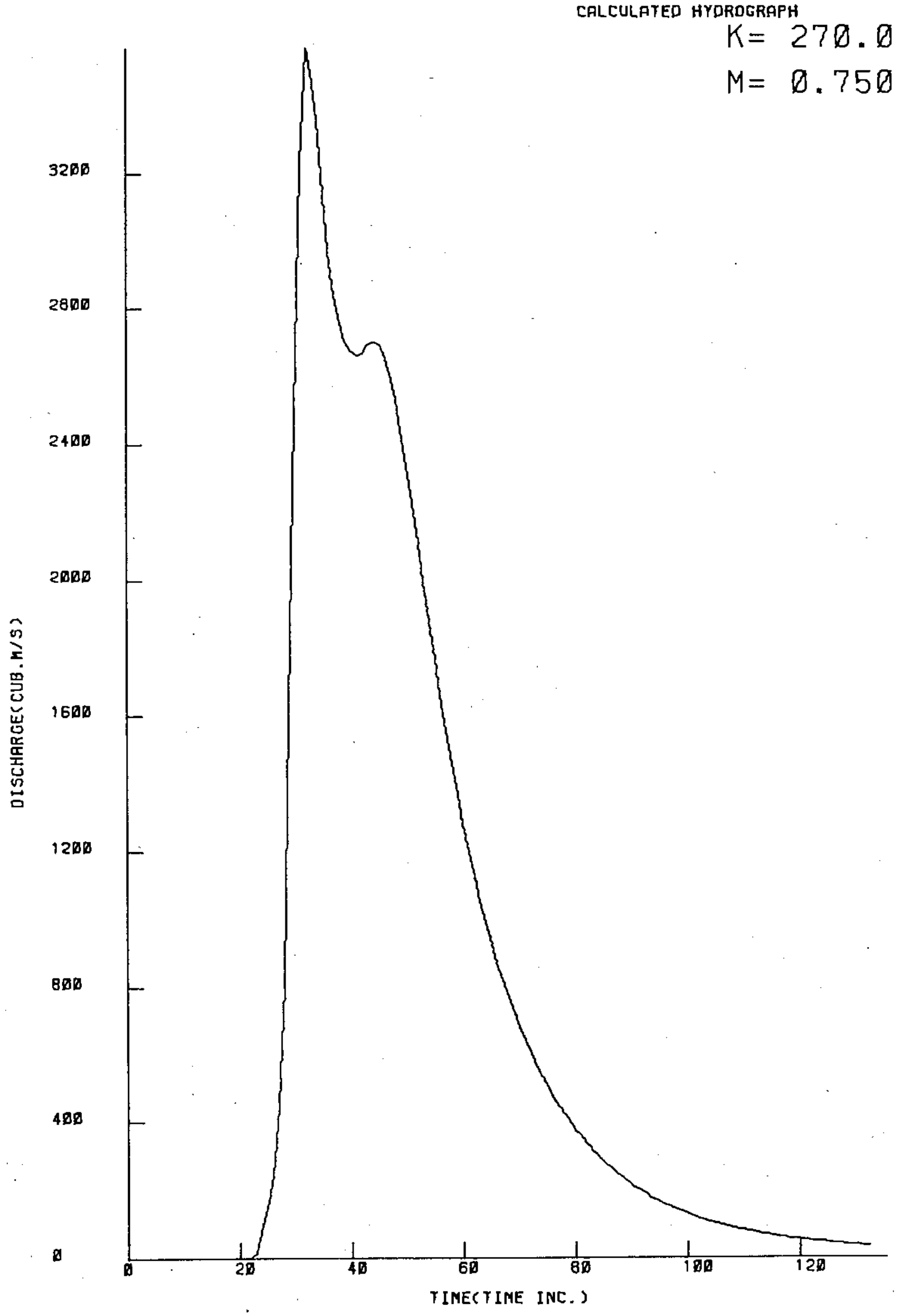
BRISBANE R AT CITY - JUN 1967

RUNOFF ROUTING (WT87) MODEL



BRISBANE R DS OF BREAKFAST C - JUN 1967

RUNOFF ROUTING (WT87) MODEL



AUGUST 1991

File was subject to a "FOI" application refer CHQ/ 2034

BRISBANE RIVER AND PINE RIVER FLOOD STUDY:

Report No. 4a

PINE RIVER FLOOD HYDROLOGY REPORT VOLUME I

Runoff-Routing Model Calibration

South East Queensland WATER BOARD

NATURAL RESOURCES Queensland

Brisbane River and Pine River Flood Studies

PINE RIVER FLOOD HYDROLOGY REPORT

REPORT ON RUNOFF – ROUTING MODEL CALIBRATION

Volume I
August 1991

TABLE OF CONTENTS

APPENDICES

A. RUNOFF-ROUTING CATCHMENT MODEL DATA FILES

B. DESIGN RAINFALL DEPTHS

C. BUREAU OF METEOROLOGY REPORT - PMP ESTIMATES

D. SUMMARY OF HISTORICAL FLOOD ESTIMATES

LIST OF TABLES

LIST OF FIGURES

PINE RIVER FLOOD STUDY

FLOOD HYDROLOGY

1.0 INTRODUCTION

A review of the flood hydrology for South East Queensland Water Board Storages was commissioned by the South East Queensland Water Board, with the study being undertaken by the Water Resources Commission. The study included revision of design floods for the storages, dambreak flood modelling downstream of the storages, and development of a flood management model for flood operations of the storages.

This report describes the development and calibration of runoff routing models for North Pine River and South Pine River. Estimates of design flood hydrographs are presented for North Pine Dam, Lake Kurwongbah and for the South Pine River at Cash's Crossing.

These design flood estimates are to be used in conjunction with a numerical hydraulic model to perform dambreak analyses of North Pine Dam and Lake Kurwongbah and to investigate the nature of flooding in the lower reaches of the North and South Pine Rivers.

In addition to the design flood estimates, estimates of historical flood events including the January 1974, April 1989, and the June 1967 event are presented. These hydrographs are to be used in the calibration of the numerical hydraulic model.

2.0 CATCHMENT DESCRIPTION

The Pine River catchment drains in a generally easterly direction from the relatively steep D'Aguilar Ranges towards the flat coastal plains of Bramble Bay. Refer to the Locality plan shown in Figure 2.1.

North Pine River and South Pine River join some 7 km upstream from Bramble Bay to form Pine River. Pine River at this point is a well developed estuary.

Pine River and the lower reaches of North Pine River and South Pine River are tidal. In North Pine River, the tidal influence extends to Young's Crossing, whilst in South Pine River, it extends to around the Bald Hills Railway Bridge.

North Pine River rises near Mt. Pleasant (522 m) which is located towards the north-west corner of the catchment. Laceys Creek, one of the five major tributaries of North Pine River, flows from the south where the range is extremely rugged and is over 700 m high. The predominant soil types in the upper parts of the catchment are shallow, stoney leached loams. Approximately 18 percent of the North Pine catchment area is covered by State Forest, with a little over 1 percent classified as National Park. The remainder is devoted to agricultural practices such as grazing and hobby farming.

In the lower part of the North Pine catchment the main soil type is a hard setting loamy soil with mottled yellow clayey subsoils. Several of the major tributaries of North Pine River join it in this region. These include Baxters Creek, Terrors Creek, Kobble Creek and Sideling Creek.

Sideling Creek Dam, which forms Lake Kurwongbah is located 1.6 km upstream of the confluence of Sideling Creek and North Pine River.

Sideling Creek Dam or Lake Kurwongbah as it is more usually known, provides water supply and recreational facilities for Pine Rivers Shire and was constructed in 1957. The storage was enlarged from 1966 to 1969, and now has a capacity of 15 450 ML.

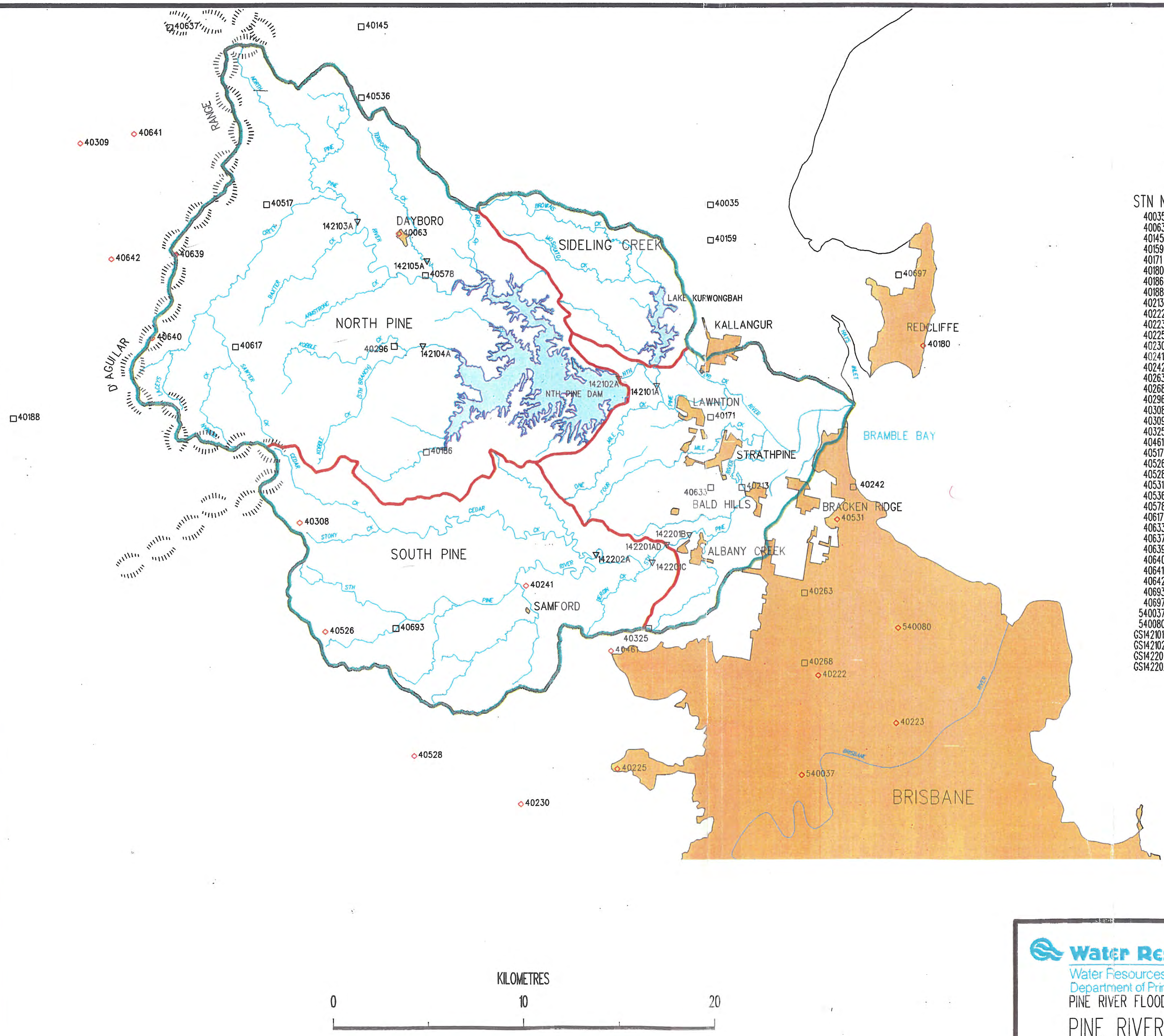
North Pine Dam is located on North Pine River, some 20 kilometres upstream from its confluence with South Pine River. North Pine Dam forms Lake Samsonvale which provides water supply to Brisbane, Pine Rivers and Redcliffe. The dam also has minor flood mitigation capability. Construction of North Pine Dam commenced in 1969 and was completed in 1976. The dam has a capacity of 202 000 ML.

The main urban areas of the North Pine catchment are located downstream of North Pine Dam. These areas include Strathpine, Lawnton and Petrie. Dayboro is the only urban area located above Lake Samsonvale. Refer to the locality plan in Figure 2.1.

South Pine River rises in the rugged D'Aguilar Ranges in the south and flows in an easterly and north-easterly direction towards Bramble Bay. South Pine River has one major tributary, Cedar Creek, that also rises in the D'Aguilar Ranges. The confluence of Cedar Creek and South Pine River is some 4 km upstream from Cash's Crossing. At this point the valley opens out into extensive floodplain areas.

Samford is the only urban area located above Cash's Crossing on South Pine River, although urban areas downstream include Albany Creek and Bald Hills. Grazing and rural residential are the major land uses in the South Pine River catchment.

FIGURE 2.1



LEGEND

◇ PLUVIO STATION

□ RAINFALL STATION

▽ GAUGING STATION

STN NO	STN NAME
40035	BURPENGARY
40063	DAYBORO
40145	MOUNT MEE
40159	NARANGBA
40171	PETRIE AUST. PAPER MILL
40180	MARGATE
40186	SAMSONVALE COMPOSITE
40188	SIM JUES CREEK
40213	BALD HILLS
40222	KALINGA BOWLING CLUB
40223	BRISBANE AIRPORT
40225	ENOGGERA RESERVOIR
40230	GOLD CREEK RESERVOIR
40241	SAMFORD CSIRO
40242	SANDGATE
40263	ZILLMERE POST OFFICE
40268	CHERMSIDE POST OFFICE
40296	DAYBORO (KOBBLESTONE)
40308	MT GLORIOUS
40309	MT BYRON
40325	FERNY GROVE
40461	FERNY GROVE (MUSEUM)
40517	DAYBORO (MACKENZIE CREEK)
40526	MT NEBO (WAS 040147)
40528	BRISBANE (3 CATCHMENTS)
40531	SANDGATE (DEAGON BCC)
40536	DAYBORO (OCEAN VIEW)
40578	DAYBORO (ARMSTRONG CREEK)
40617	DAYBORO (RAYNBIRD CREEK)
40633	STRATHPINE
40637	DAYBORO (MOUNT MEE FOREST STATION)
40639	DAYBORO (BYRON & REEDY CKS NO 2)
40640	DAYBOBO (BYRON & REEDY CKS NO 3)
40641	DAYBORO (BYRON & REEDY CKS NO 4)
40642	DAYBORO (BYRON & REEDY CKS NO 5)
40693	SAMFORD (HIGHVALE)
40697	REDCLIFFE AIRPORT
540037	WATER STREET DRAIN STEVENS STORAGE
540080	BANYO (ARMY DEPOT)
GS142101A	NTH PINE RIVER AT YOUNG'S CROSSING
GS142102A	NTH PINE RIVER AT DAMSITE
GS142201AD	STH PINE RIVER AT CASH'S CROSSING
GS142202A	STH PINE RIVER AT DRAPER'S CROSSING

Water Resources
Water Resources Commission
Department of Primary Industries
PINE RIVER FLOOD STUDY

PINE RIVER
LOCALITY PLAN

A3-98882

3.0 CLIMATE

The Pine River catchment has a moist sub-tropical climate typical of South-East Queensland. This type of climate features hot wet summers and cooler dry winters, with the majority of rainfall falling in the period from December to March.

The mean annual catchment rainfall is 1 250 mm, although a substantial spatial variation exists across the catchment. Catchment rainfalls are generally heavier around the western ranges as comparisons between mean annual rainfalls at daily rainfall stations indicate: 1 500 mm at Ocean View to the north-west and 1 150 mm at Samsonvale, located centrally within the Pine River catchment.

4.0 PREVIOUS STUDIES

4.1 INTRODUCTION

The Pine River system has been the subject of many hydrologic and hydraulic studies because of its proximity to the urban development of Brisbane and surrounding environs.

A report by the Department of Local Government entitled 'Development of Water Resources of North Pine River' in 1954, attributes the Department of the Co-ordinator General of Public Works as the first body responsible for initiating investigations into the potential water resource of the Pine River system.

The Co-ordinator General recommended approval for investigations to be conducted by the Local Government Department in 1951. The objective of this investigation was to determine the maximum extent to which the resources of the North Pine basin could be made use of to augment the water supply of Greater Brisbane.

Sideling Creek, a tributary of North Pine River was the subject of another investigation into the supply of water for Pine Rivers Shire Council and Australian Paper Mills. Design of a dam on this creek commenced in 1953 and construction was completed in 1957. The embankment was raised in 1966 to its present level.

4.2 DEPARTMENT OF LOCAL GOVERNMENT STUDIES

The methods adopted in the determination of net yield and flood discharge for the North Pine River were outlined in a paper by Webber, (1958). Synthetic unit hydrograph methods allied with flood frequency techniques were utilised in determining spillway design floods. A three hour synthetic unit hydrograph was developed using McCarthy's method.

Walpole's values of maximum possible precipitation were used to estimate the maximum possible flood. A mass curve of rainfall was drawn parallel to an enveloping depth-time curve for world record rainfalls. No losses were considered as infiltration rates were regarded as low.

The estimate of maximum possible flood was 2950 m^3/s for a dam on North Pine River with a catchment area of 347 km^2. The characteristics of the storm were assumed to be low initial precipitation rates finishing with higher intensities. Total effective rainfall for the seventy-two hour duration storm was 1530 mm.

Webber compared this result with a frequency distribution on Gumbel's skewed probability paper. The maximum possible flood was assigned a plotting position corresponding to a return period of 10 000 years.

In 1962 the Department of Local Government (DLG) published a design report setting out the relative merits of alternate proposals for a storage on North Pine River. The report concluded that a mass concrete gravity dam with a gated spillway was the most appropriate structure for the site located at AMTM 12.4 miles (20.3 kilometres AMTD).

The design inflow hydrograph was determined by application of design rainfalls estimated by the Bureau of Meteorology to a unit hydrograph derived for the catchment. Both transposed and maximised 1893 storm rainfalls and the US derived 'Thunderstorm' model rainfalls gave similar peak discharges. A design inflow hydrograph of 4300 m^3/s was determined which when routed through the reservoir yielded a spillway design flood of 3650 m^3/s.

The 'Thunderstorm' model rainfall estimates gave depths of 430 mm for a storm duration of six hours and 550 mm for a twelve hour duration event. The transposed and maximised rainfall estimates of the February 1893 event resulted in 730 mm in twenty-four hours; 1100 mm in forty-eight hours; and 1400 mm in seventy-two hours.

The DLG report also recommended physical model studies be conducted on the spillway gates to further refine the hydraulic design of the spillway. This study was conducted by the University of Queensland in the late 1960's, but the model study report was compiled some years later in 1977, (Brown)

A re-analysis of the yield available from North Pine Dam was performed by the Department of Local Government in 1973 during the construction of the dam. The dam was completed in 1976. Another re-assessment of the yield available from North Pine Dam was performed in 1990 by the Water Resources Commission. This re-assessment was conducted as part of an investigation into water supply sources in South-East Queensland.

The 1973 estimate of net yield from North Pine Dam with Full Supply Level at EL 39.6 mAHD was 58 600 ML/year. This was based on an analysis of historic streamflows extending from 1915 to 1970. For estimates of various risks of depletion, Barnes' statistical analysis of streamflows was used.

By comparison the 1990 re-estimates of yield indicated a historic safe yield of 59 250 ML/year with monthly probability of failure estimates ranging from 62 730 ML/year for 1% up to 76 000 ML/year for a 5% probability of failure. These estimates were obtained from a single storage yield analysis with monthly streamflows generated from a calibrated rainfall runoff model (Sacramento Model) for the period 1889 to 1988.

4.3 CAMERON MCNAMARA STUDIES

A number of other studies have been conducted on the Pine River system prior to and since the construction of North Pine Dam. Consulting Engineers, Cameron McNamara and Partners, investigated extraction of sand and gravel from both North and South Pine Rivers for the Co-ordinator General's Department in 1978. This study examined whether extraction should be permitted and if so what strategy should be adopted. Several aspects of the report relate to the hydrology and hydraulics of the Pine River system.

Flood frequency analyses were used to determine design flood discharges at a number of locations along North Pine River and South Pine River. These analyses were performed for both pre and post-North Pine Dam for a range of design floods with average recurrence intervals of up to 100 years.

A summary of these estimates is provided in Table 4.1

TABLE 4.1
PINE RIVER DESIGN FLOOD ESTIMATES

LOCATION		CATCHMENT AREA (km^2)	PEAK DISCHARGE (m^3/s) AVERAGE RECURRENCE INTERVAL (YRS)		
			10	50	100
North Pine River					
Young's Crossing	Pre	350	710	1275	1475
	Post		530	1050	1275
Petrie Rail Bridge	Pre	437	818	1469	1699
	Post		630	1245	1495
Confluence Sth Pine River	Pre	689	1125	2020	2337
	Post		935	1800	2130
South Pine River					
Draper's Crossing		158	285	473	553
Cash's Crossing		179	450	748	875
Confluence Nth Pine River		243	557	926	1084

The estimates for Young's Crossing, Draper's Crossing and Cash's Crossing are directly comparable with estimates in Section 7.2 of this report.

A steady state or backwater model was also utilised in this study to relate design flood estimates to predicted flood levels throughout North and South Pine River. The impact of extraction activities were investigated by this model.

4.4 JOHN WILSON AND PARTNERS STUDIES

John Wilson and Partners prepared a report for the Main Roads Department in 1975 on a new crossing of the South Pine River at Bald Hills by the Bruce Highway. Estimates of discharge for design events of recurrence intervals of up to 100 years were derived for the South Pine River catchment using the Cordery Synthetic Unitgraph. The effects of urbanisation were included in this analysis. Results of the ultimate development estimates at the Bald Hill bridge are presented in Table 4.2

TABLE 4.2

BALD HILLS BRIDGE DESIGN FLOOD ESTIMATES

LOCATION	CATCHMENT AREA (km^2)	PEAK DISCHARGE (m^3/s) AVERAGE RECURRENCE INTERVAL (YRS)		
		10	50	100
South Pine River Bald Hills Bridge	217	870	1220	1440

The difference in the John Wilson and Partners estimate and the Cameron McNamara estimate can be attributed to the effect of urbanisation.

John Wilson and Partners have also been involved in assessing a dam site on South Pine River at Greenwood in 1990 for the Water Resources Commission. Estimates of design flood inflows were derived using both flood frequency techniques and a runoff-routing model. (RORB). Peak inflow discharges at the dam site have been estimated for a range of average recurrence intervals from the 1 in 10 year up to the probable maximum flood. The dam site is situated just downstream of Draper's Crossing and has a catchment area of around 160 km^2.

Pine Rivers Shire Council also engaged John Wilson and Partners to conduct a review of the hydraulic design of the spillway of Sideling Creek Dam (also known as Lake Kurwongbah) in 1989. This study involved the re-estimation of design rainfall estimates for events up to probable maximum and the re-derivation of design flood estimates so that the adequacy of the spillway could be determined. Estimates of the Probable Maximum Precipitation were obtained from the Bureau of Meteorology. A runoff-routing model (RAFTS) was utilised to derive design flood estimates. The John Wilson and Partners report concludes the peak inflow into Lake Kurwongbah for the PMF is 1830 m^3/s for a storm duration of three hours, whilst the peak spillway discharge is 884 m^3/s, (associated with a storm duration of twelve hours and 920 mm rainfall depth). The spillway has sufficient capacity to pass this discharge without the storage overtopping and as a consequence the spillway is deemed by ANCOLD Guidelines to be of adequate standard.

4.5 SINCLAIR KNIGHT AND PARTNERS

Sinclair Knight and Partners have also completed hydrologic and hydraulic model studies of the North and South Pine Rivers on behalf of developers Ariadne Australia, for a proposed residential development known as 'Pine Waters'. The studies commenced in the mid 1980's and the latest report was published in November 1990. The land re-zoning application and subsequent approval from Council is the subject of a Land Court decision and as a result access to this report is limited at this time.

Runoff-routing models (RORB) for North Pine River above North Pine Dam and South Pine River to Leitch's Crossing have been developed and calibrated. These models were utilised to derive historic and design flood estimates for a numerical hydraulic model of the North Pine River below Young's Crossing and the South Pine River below Leitch's Crossing. The numerical hydraulic model adopted in the study is a MIKE 11 model. The calibration events of January 1974 and April 1989 were investigated and design floods of up to 100 year return periods were examined.

It should be noted that North Pine Dam was treated as having a spillway rating corresponding to normal gate operation as indicated in the Manual of Operational Procedures for Flood Releases from North Pine Dam, (1986).

5.0 AVAILABLE DATA

5.1 STREAMFLOW DATA

Streamflow data provides the basis of calibration of a runoff routing model. Basic data, consisting of water level and discharge versus time, is required for the calibration of historic flood events. The extent and reliability of a rating curve at a gauging station influences the confidence that can be placed in the results of the modelling process.

There were a number of stream gauging stations located on North Pine River and South Pine River and other creeks in the Pine catchment. A summary of the most pertinent stations is presented in Table 5.1. The locations of gauging stations are shown in Figure 2.1.

TABLE 5.1
STREAM GAUGING STATIONS

STATION NUMBER	STREAM	CATCHMENT				PERIOD OF RECORD
		NAME	AMTD (KM)	AREA (KM^2)	TYPE	
142101A	North Pine River	Young's Crossing	17.1	353	SF	1956-1978
142102A	North Pine River	Damsite	20.3	345	SF	1956-1969
142102B	North Pine River	YMCA Camp	18.8	345	PR	1969-1972
142210D	South Pine River	Cash's Crossing	14.0	179	S	1951-1965
142202A	South Pine River	Draper's Crossing	20.6	158	SG	1965-

Where: S: Manually read staff gauge
SF: Floatwell with Leupold and Stevens Recorder
PR: Foxboro Type pressure recorder
SG: Gas Electric Vactric/Transducer

Whilst the above table provides details of stations that were utilised in this study, streamflow stations have operated on North Pine River and South Pine River since 1915 and 1908 respectively. However, for a majority of this period the method of recording was by manually reading the staff gauge which is not necessarily as reliable as more recent automated recording methods. The streamgauge at Cash's Crossing relied on this method.

During the mid 1970's three streamgauging stations were established in the catchment of North Pine Dam on Laceys's Creek Kobble Creek and Terrors Creek. These stations were used in a catchment use study of North Pine Dam and were closed after 3 years of operation. The stations were not utilised in this study because of the small amount of record available and because they were poorly rated.

The Pine River catchment was not instrumented with pluviographs until about 1956, hence there is insufficient rainfall data available prior to this time for runoff routing techniques to be applied.

5.2.3 **GS 142102B - North Pine River at YMCA Camp**

In 1969, when construction commenced on North Pine Dam, the gauging station at the Damsite was moved 1.5 km downstream to near the Warrawee YMCA Camp.

The highest streamgauging measurement at this site is only 5 m^3/s recorded in October 1969. The station was closed in 1972, before the completion of the dam in 1976.

Comparisons between monthly volumes of this site and Young's Crossing show that the volumes at the YMCA Camp are significantly higher than the corresponding values at Young's Crossing. It is felt the adopted rating curve for the YMCA Camp overestimates flows to a large extent and as such, these flows should be treated with caution.

5.2.4 **GS 142101D - South Pine River at Cash's Crossing**

The quality of the rating curve at Cash's Crossing is poor because of the sand control that exists at this site. Three substantially different rating curves were derived for the high flows at this location for the period 1951 to 1965. Moderate to large floods altered the rating section in February, 1953, and March, 1955.

The highest streamgauging measurement during the period 1951 to 1953 is 280 m^3/s recorded in January, 1951, whereas the highest gauging during 1953 to 1955 is only 48 m^3/s recorded in February 1954.

The various rating curves are presented in Figure 5.3.

The station closed in 1965 when it was replaced by the station at Draper's Crossing.

5.2.5 **GS 142202A - South Pine River at Draper's Crossing**

Draper's Crossing has a highest streamgauging measurement of 283 m^3/s, recorded in June, 1967. It is evident that between 1965 and 1975 the rating section changed substantially. A large amount of scour on the left bank caused a substantial increase in waterway area.

The highest rating since that time is 194 m^3/s recorded in May, 1989. The rating curve of Draper's Crossing is shown in Figure 5.4. This station is the only gauging station still operational in the Pine River system. It is equipped with a travellerway for high flow measurements but the control remains as sand and gravel. Consequently, the rating curve is subject to change, particularly at low to moderate flows.

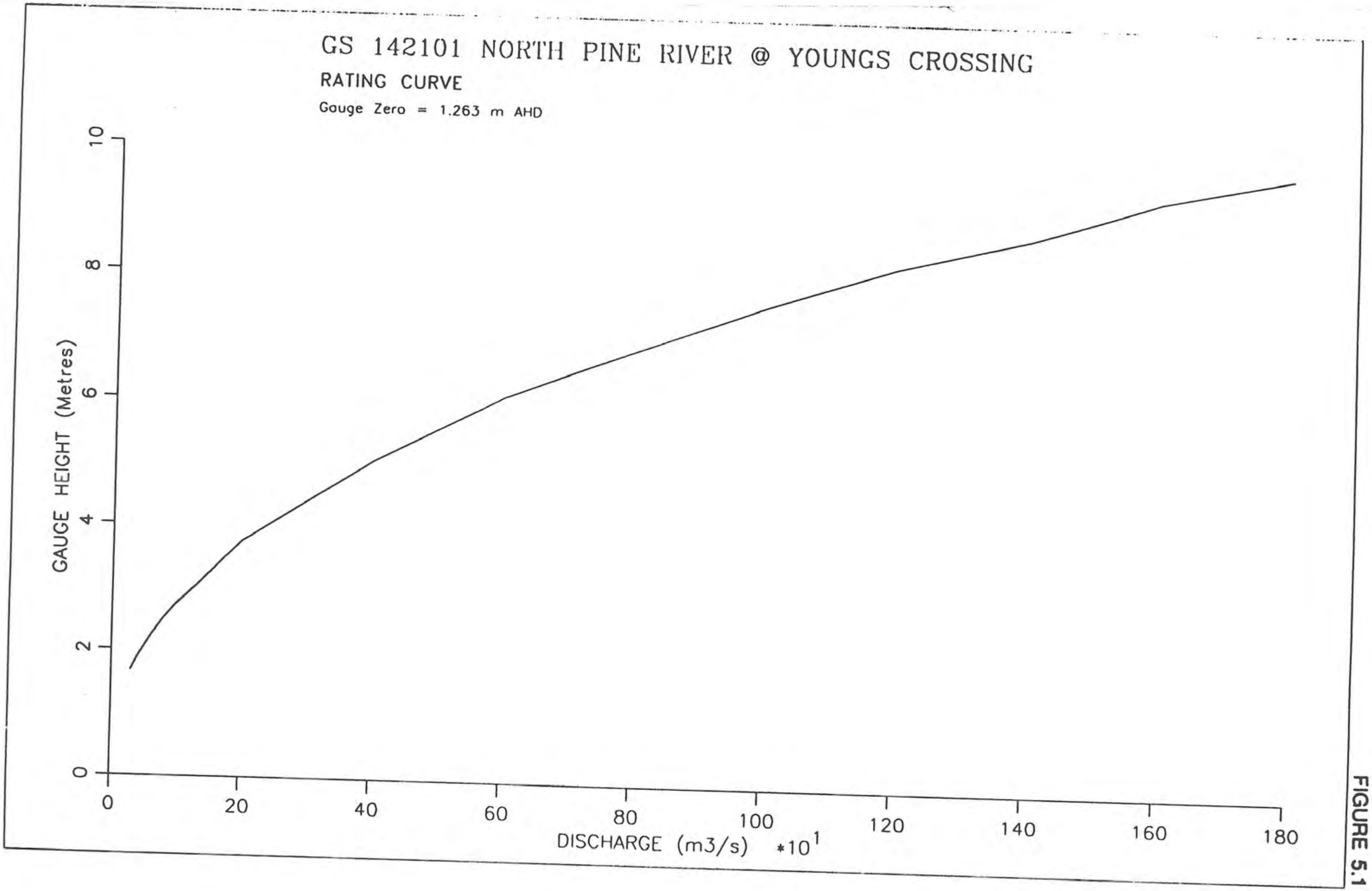
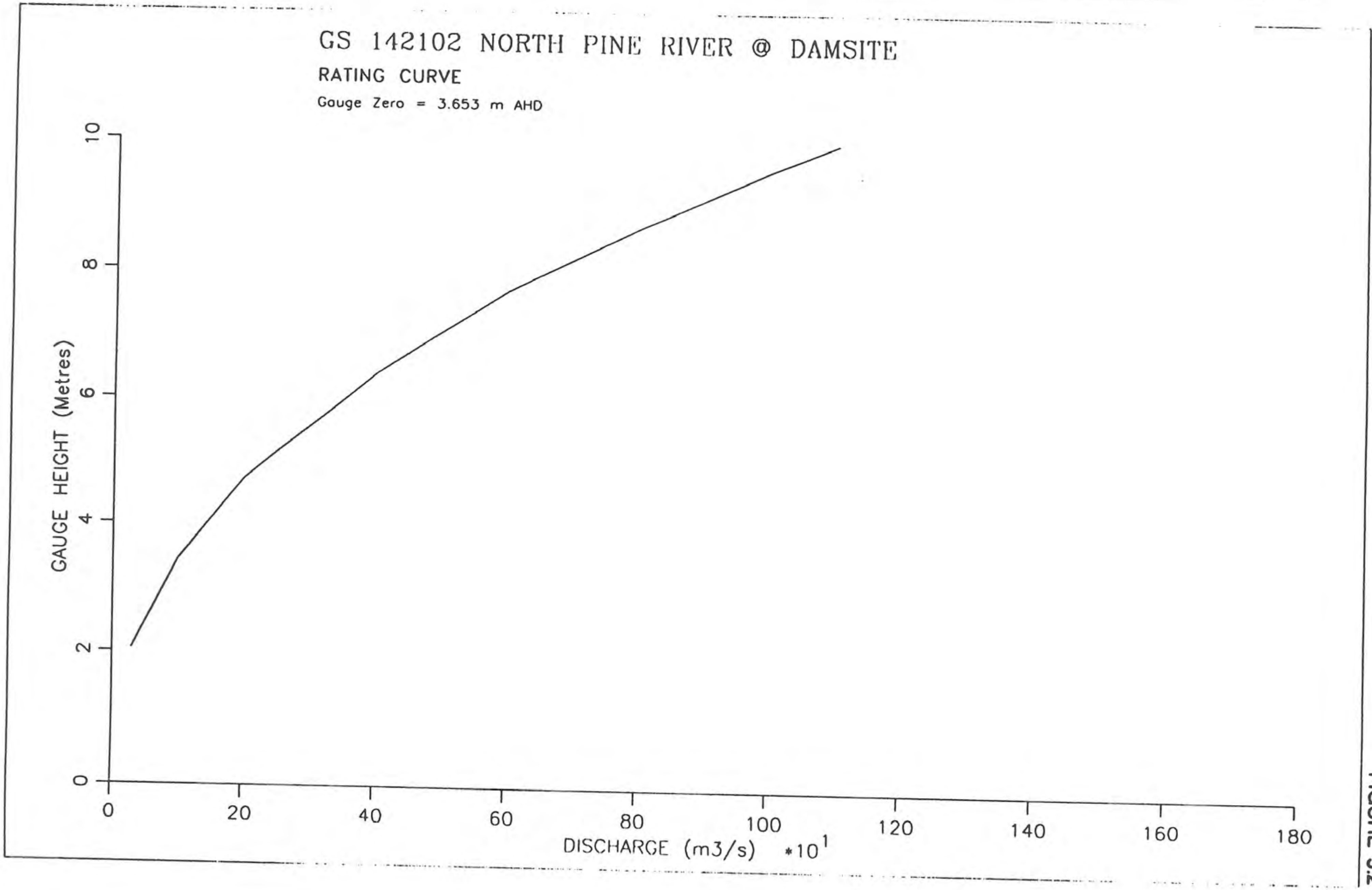

GS 142101 NORTH PINE RIVER @ YOUNGS CROSSING
RATING CURVE
Gauge Zero = 1.263 m AHD
GAUGE HEIGHT (Metres)
0
2
4
6
8
10
0
20
40
60
80
100
120
140
160
180
DISCHARGE (m3/s) *10^1

FIGURE 5.1



GS 142102 NORTH PINE RIVER @ DAMSITE
RATING CURVE
Gauge Zero = 3.653 m AHD
GAUGE HEIGHT (Metres)
0
2
4
6
8
10
0
20
40
60
80
100
120
140
160
180
DISCHARGE (m3/s) *10^1

FIGURE 5.2

6.0 RUNOFF ROUTING MODEL

6.1 INTRODUCTION

The runoff routing model, developed by Mein, Laurenson and McMahon (1974) and implemented as computer program WT42PC (Shallcross, 1987) was used to perform the simulation. This model is a simple conceptual representation of catchment storage effects that provides for the routing of rainfall excess to produce a surface runoff hydrograph.

The model describes a distribution of concentrated conceptual storages of the catchment, which allows rainfall and losses to vary throughout the catchment. Each storage has a non-linear storage-discharge relation of the form:

$$S = 3600.k.k_1Q^m$$

where: S = Storage (m^3)
Q = Discharge (m^3/s)
k = Dimensional Model Parameter
m = Dimensionless Model Parameter
k_1 = Dimensionless Model Parameter related to Travel Time in a Reach

The two model parameters k and m may be estimated by calibration using recorded data or they may be estimated from regional formulae appropriate for the area of interest. Calibration techniques were used in this case, and checked against regional formulae estimates.

6.2 CALIBRATION DATA

6.2.1 Streamflow

North Pine River

Calibration data for North Pine River consisting of basic streamflow data, daily rainfall and pluviograph data is available for the period from June 1967 to 1978, when the last stream gauge closed. Large flood events (for which streamflow records are available) occurred in fourteen months during that period. Prior to 1967, insufficient pluviograph data is available.

From these events, only four were considered for calibration:

June 1967
December 1970
July 1973
January 1974

The records for the January 1974 and July 1973 flood events are only available at Young's Crossing. It should also be noted that the recorder at Young's Crossing was not operational for the whole of the January 1974 flood event due to vandal damage, although peak height estimates near the station provide some indication of the magnitude of the flow. North Pine Dam was partially constructed and as a result it had some mitigating effect on the flow.

Because of a major discrepancy between volumes of flood hydrographs concerning the December 1970 event at both the YMCA Camp and Young's Crossing, two smaller events that occurred in June 1967 were also considered for calibration. The events that occurred in June 1967 have been designated as June 1967A, June 1967B and June 1967C, since the respective peak flows occurred on the 11th, 20th and 25th of the month.

Table 6.1 summarises the peak discharges and flood volumes of the selected calibration events for North Pine River.

TABLE 6.1

PEAK DISCHARGE AND FLOOD VOLUME
NORTH PINE RIVER

	GS 142101 - YOUNG'S CROSSING		GS 142102 - DAMSITE	
EVENT	PEAK DISCHARGE (m^3/s)	VOLUME (ML)	PEAK DISCHARGE (m^3/s)	VOLUME (ML)
June 1967A (a) (b)	283 419	65 980	258 385	56 400
June 1967B (a)	255	28 475	223	23 845
June 1967C (a)	271	34 352	229	28 870
December 1970 (a) (b)	470 280	66 660	530 312	84 540
July 1973 (a)	669	74 790	-	-
January 1974 (a)	821*	-	-	-

* Estimated Record

(a) and (b) refer to various peaks that occurred in a multi-peak event.

The values quoted in Table 6.1 have had baseflow separated and hence these values represent surface runoff estimates.

Baseflow was separated from each flood hydrograph to produce a surface runoff hydrograph. Baseflow separation points were obtained for each station from plots of hydrograph recessions on log-linear paper. The baseflow hydrograph was assumed to follow a straight line from the beginning to the end of surface runoff.

South Pine River

Streamflow calibration data is available at Draper's Crossing streamgauging station from 1965 to the present date. Events coinciding with the data available from the North Pine River catchment have been selected because of the ease of data gathering. Storage data is also available for a number of events after construction of North Pine Dam. As a result, some of these events were also considered for calibration of the South Pine River catchment.

Events considered for calibration of the South Pine River model include:

June 1967
December 1970
July 1973
June 1983

The events of January 1974, April 1988 and April 1989 have been used for model verification purposes since they are multi-peaked events.

Table 6.2 summarises the peak discharge and flood volumes (after baseflow separation) of the calibration and verification events.

TABLE 6.2

PEAK DISCHARGE AND FLOOD VOLUME
SOUTH PINE RIVER

EVENT		PEAK DISCHARGE (m^3/s)	FLOOD VOLUME (ML)
June 1967A	(a)	466	35 130
December 1970	(a)	213	20 820
	(b)	156	
July 1973	(a)	115*	8 930
January 1974	(a)	1 238	156 270
	(b)	1 050	
	(c)	1 239	
	(d)	973	
June 1983	(a)	356	21 670
April 1988	(a)	248	46 410
	(b)	520	
	(c)	174	
	(d)	378	
	(e)	282	
April 1989	(a)	339	45 980
	(b)	923	
	(c)	88	

* Estimated Record

(a) and (b), etc, refer to various peaks that occurred in a multi-peaked event.

6.2.2 **Rainfall Data**

Rainfall data available for the calibration storms is presented in Table 6.3. The rainfall depths are storm totals for the duration of the events. All values shown in Table 6.3 are millimetres. The duration of each rainfall event is listed in Table 6.4.

TABLE 6.3

CALIBRATION EVENT RAINFALL DEPTHS

STATION	EVENT								
	JUN 1967A	JUN 1967B	JUN 1967C	DEC 1970	JUL 1973	JAN 1974	JUN 1983	APR 1988	APR 1989
035 Burpengary	207	72	136	386	263	-	-	-	278
063 Dayboro	198	98	98	250	362	762	169	423	344
145 Mt Mee	231	97	121	355	475	961	197	375	348
159 Narangba	149	112	142	365	408	-	-	-	-
171 Petrie	209	108	144	299	342	709	207	330	265
180 Margate	200	98	175	330	342	787	195	246	-
186 Samsonvale	225	109	94	211	301	716	141	406	341
188 Sim Jue Ck	142	75	118	91	312	449	194	263	77
213 Bald Hills	224	130	145	346	373	611	186	276	263
223 Brisbane AO	397	121	134	290	271	617	-	207	186
225 Enoggera Res	318	127	96	226	252	898	-	-	289
241 Samford	235	127	99	246	285	983	157	294	289
242 Sandgate	147	108	199	403	299	628	147	267	205
263 Zillmere	253	100	157	319	361	787	172	231	224
268 Chermside	258	126	179	276	-	-	176	234	-
296 Kobblestone	200	138	96	242	358	816	-	-	-
308 Mt Glorious	263	165	88	265	302	1305	194	526	393
309 Mt Byron	-	-	-	203	333	425	-	-	-
325 Ferny Grove	339	128	97	286	-	-	180	274	-
517 Mackenzie Ck	173	93	99	253	370	689	156	386	322
526 Mt Nebo	293	-	-	215	206	1118	176	-	393
528 3 Way Catchment	-	-	-	176	232	-	-	-	-
531 Deagon BCC	-	-	-	-	329	625	161	276	185
536 Ocean View	241	111	117	-	-	918	235	465	350
617 Raynbird Ck	205	119	90	-	-	-	-	-	-
633 Strathpine	-	-	-	-	-	-	176	287	268
637 Mt Mee Forestry	-	-	-	312	315	-	221	381	300
693 Highvale	225	126	90	185	237	960	132	-	-
697 Redcliffe AO	-	-	-	-	-	-	188	220	-
- North Pine Dam	-	-	-	-	-	-	190	-	-

Isohyetal diagrams of each of the storm events showing the spatial variability of rainfall across the catchment are presented in Figures 6.1 to 6.9.

TABLE 6.4

DURATION OF RAINFALL EVENTS

EVENT	DURATION (HRS)
11th June 1967A	66
20th June 1967B	42
25th June 1967C	52
December 1970	84
July 1973	59
January 1974	96
June 1983	42
April 1988	84
April 1989	72

The availability of pluviograph data (which provides an indication of the temporal variability of rainfall of each storm) is summarised in Table 6.5. Although 12 stations are listed, data is generally only available from a number of them for any particular event. In some cases, data is only available for a part of the event.

TABLE 6.5

AVAILABILITY OF PLUVIOGRAPHS

STATION	EVENT								
	JUN 1967A	JUN 1967B	JUN 1967C	DEC 1970	JUL 1973	JAN 1974	JUN 1983	APR 1988	APR 1989
063 Dayboro	-	-	-	*	*	*	x	*	*
180 Margate	-	-	-	-	-	-	x	x	-
222 Kalinga	x	-	-	-	-	x	x	-	-
223 Brisbane AO	*	*	*	x	x	x	x	x	-
225 Enoggera Res	*	*	*	x	x	x	-	-	-
241 Samford CSIRO	*	*	*	*	*	*	*	x	*
308 Mt Glorious	-	-	-	*	*	*	*	x	*
461 Ferny Hills	-	-	-	-	-	-	x	*	-
526 Mt Nebo	*	x	x	*	*	-	*	-	-
538 3 Way Catchment	-	-	-	x	x	x	-	x	x
531 Deagon	-	-	-	-	*	*	x	x	x
- North Pine Dam	-	-	-	-	-	-	x	-	-

x Record Available

* Pluviograph record utilised during calibration of runoff routing model.

Figures 6.10 to 6.18 provide a summary of the temporal variation for each event, based upon the pluviograph stations that were adopted for use in the model calibration.

The distribution of pluviograph stations to each sub-area for each event is shown in Table 6.6. The sub-areas referred to in this table are illustrated in the RORB model layout shown in Figure 8.1.

The composite record indicated in Table 6.6 for the June 1967A event is based upon daily rainfall station proportions in conjunction with Brisbane Airport. The daily rainfall proportions were very different from the other pluviograph temporal patterns, which accounts for the difference in hydrograph shapes in the North and South Pine Rivers.

TABLE 6.6

DISTRIBUTION OF PLUVIOGRAPH STATIONS

STATION	SUB-AREAS
Event: June 1967A	
North Pine River	
Samford CSIRO	11 12 15 18 19 20
Mt. Nebo	13 14 16 17
Composite Record	1 2 3 4 5 6 7 8 9 10
South Pine River	
Samford CSIRO	3 4 5 6 7 10 11 12
Mt. Nebo	1 2 8 9
Event: June 1967B	
North Pine River	
Brisbane Airport	1 8 13 14 15 18 19
Samford CSIRO	2 3 4 5 6 7 20
Enoggera Reservoir	9 10 11 12 16 17
Event: June 1967C	
North Pine River	
Brisbane Airport	4 5 6 7 8 9 10 11 12 15 18
Samford CSIRO	1 2 3 14 16 17 19 20
Enoggera Reservoir	13
Event: December 1970	
North Pine River	
Dayboro	3 4 5 6 7 8 9 10 11 12 13 15 18
Mt. Glorious	1 2 14 16 17 19 20
Samford CSIRO	
South Pine River	
Mt. Nebo	1 2 3
Mt. Glorious	8 9 10
Samford CSIRO	4 5 6 7 11 12
Event: July 1973	
North Pine River	

STATION	SUB-AREAS
Dayboro	3 4 5 6 7 8 9 10 11 12 13 15
Mt. Glorious	18
Samford CSIRO	1 2 14 16 17
Deagon	19 20
South Pine River	
Mt. Nebo	1 2 3
Mt. Glorious	8 9 10
Samford CSIRO	4 5 6 7 11 12
Event: January 1974	
North Pine River	
Mt. Glorious	1 2 14 16
Dayboro	3 4 5 6 7 8 9 10 11 13 15 17
Deagon	12 18 19 20
South Pine River	
Mt. Glorious	1 8 9
Samford CSIRO	2 3 4 5 10
Deagon	6 7 11 12
Event: June 1983	
South Pine River	
Mt. Nebo	1 2
Mt. Glorious	8 9
Samford CSIRO	3 4 5 6 7 10 11 12
Event: April 1988	
South Pine River	
Dayboro	8 9 10 11 12
Ferny Hills	1 2 3 4 5 6 7
Event: April 1989	
South Pine River	
Dayboro	7 9 10 11 12
Mt Glorious	1 8
Samford CSIRO	2 3 4 5 6

6.3 CATCHMENT DATA

6.3.1 North Pine River

Two independent calibration runoff routing models have been developed for North Pine River; one extending to the Damsite and the other to Young's Crossing. This was done because of the discrepancy between hydrograph volumes at each of the gauging stations.

The runoff routing model extending to the Damsite consists of 19 sub-areas. The catchment was divided in its natural state, prior to the storage of North Pine Dam being incorporated. The layout of the runoff routing model for North Pine River to the Damsite in its natural state (pre-North Pine Dam) is shown in Figure 6.19. Catchment characteristics for this model are presented in Tables 6.7 and 6.8.

TABLE 6.7

NORTH PINE RIVER AT DAMSITE

SUB-AREA DETAILS

SUB-AREA	AREA (km^2)	DISTANCE TO OUTLET (km)
1	25.8	38.0
2	14.7	37.0
3	19.6	31.6
4	22.1	27.0
5	18.5	31.6
6	20.0	30.0
7	19.2	24.0
8	19.4	18.4
9	12.8	19.8
10	15.8	15.8
11	19.4	10.4
12	12.8	6.4
13	20.1	18.4
14	20.1	17.6
15	17.3	10.0
16	17.7	14.2
17	13.9	9.4
18	13.6	8.0
19	22.2	4.8

Catchment Area $A = 345.1\ km^2$
Distance to Centroid of Area $dc = 20.4\ km$

TABLE 6.8
NORTH PINE RIVER AT DAMSITE
REACH DETAILS

REACH	REACH LENGTH (km)	REACH	REACH LENGTH (km)	REACH	REACH LENGTH (km)
A-B	4.0	J-L	5.0	V-X	3.2
C-B	3.0	M-L	2.4	X-Y	2.8
B-D	2.4	L-N	2.2	Z-AA	4.8
D-E	4.6	O-P	4.0	AA-Y	2.2
E-F	3.8	P-N	2.0	Y-BB	2.0
G-H	4.6	N-R	3.4	CC-BB	2.8
I-H	3.0	R-S	4.0	BB-T	1.8
H-F	3.8	S-T	3.0	T-DD	1.6
F-J	2.2	U-V	5.2	EE-DD	3.0
K-J	3.0	W-V	4.4	DD-FF	1.8

Note: All reach types natural.

Sub-area relative delay times for each reach were calculated as being proportional to reach length for all models. Listings of all catchment data files used with the runoff routing model are given in Appendix A.

The calibration runoff routing model that extends to Young's Crossing is very similar to the Damsite model, but it consists of one extra sub-area.

Catchment characteristics of this model are presented in Tables 6.9 and 6.10.

TABLE 6.9

NORTH PINE RIVER AT YOUNG'S CROSSING

SUB-AREA DETAILS

SUB-AREA	AREA (km^2)	DISTANCE TO OUTLET (km)
1	25.8	41.6
2	14.7	40.6
3	19.6	35.2
4	22.1	30.6
5	18.5	35.2
6	20.0	33.6
7	19.2	27.6
8	19.4	22.0
9	12.8	23.4
10	15.8	19.4
11	19.4	14.0
12	12.8	10.0
13	20.1	22.0
14	17.3	21.2
15	17.7	13.6
16	13.9	17.8
17	13.6	13.0
18	13.6	11.6
19	22.2	8.4
20	7.6	2.6

Catchment Area $A = 352.7\ km^2$
Distance to Centroid of Area $dc = 23.5\ km$

TABLE 6.10

NORTH PINE RIVER AT YOUNG'S CROSSING
REACH DETAILS

REACH	REACH LENGTH (km)	REACH	REACH LENGTH (km)	REACH	REACH LENGTH (km)
A-B	4.0	J-L	5.0	V-X	3.2
C-B	3.0	M-L	2.4	X-Y	2.8
B-D	2.4	L-N	2.2	Z-AA	4.8
D-E	4.6	O-P	4.0	AA-Y	2.2
E-F	3.8	P-N	2.0	Y-BB	2.0
G-H	4.6	N-R	3.4	CC-BB	2.8
I-H	3.0	R-S	4.0	BB-T	1.8
H-F	3.8	S-T	3.0	T-DD	1.6
F-J	2.2	U-V	5.2	EE-DD	3.0
K-J	3.0	W-V	4.4	DD-FF	1.8
				FF-GG	1.0
				GG-HH	2.6

Note: All reach types natural.

The layout of the calibration runoff routing model for North Pine River to Young's Crossing is shown in Figure 6.19.

6.3.2 **South Pine River**

The calibration runoff routing model extending to Draper's Crossing comprises 12 sub-areas. The layout of the runoff routing model for South Pine River to Draper's Crossing is shown in Figure 6.19. Catchment characteristics for this model are presented in Tables 6.11 and 6.12.

TABLE 6.11

SOUTH PINE RIVER AT DRAPER'S CROSSING

SUB-AREA DETAILS

SUB-AREA	AREA (km^2)	DISTANCE TO OUTLET (km)
1	14.1	20.1
2	10.2	18.3
3	20.2	15.3
4	8.9	10.9
5	12.9	16.1
6	12.5	11.9
7	13.5	5.1
8	12.9	22.3
9	11.5	19.9
10	13.0	15.5
11	14.4	7.9
12	14.7	5.7

Catchment Area A = 158.9 km^2
Distance to Centroid of Area dc = 14.0 km

TABLE 6.12

SOUTH PINE RIVER AT DRAPER'S CROSSING

REACH DETAILS

REACH	REACH LENGTH (km)	REACH	REACH LENGTH (km)
A-B	4.2	J-K	4.6
C-B	2.4	L-M	4.6
B-D	3.6	N-M	2.0
E-D	3.0	M-O	2.4
D-F	1.4	O-P	7.6
F-G	2.2	P-Q	5.4
H-I	4.2	R-Q	3.2
I-G	3.2	Q-K	2.0
G-J	3.6	K-S	0.5

Note: All reach types natural.

6.4 MODEL CALIBRATION

For the purpose of model calibration the North Pine River models are considered independently, so that the most appropriate parameters can be obtained for each site (i.e. Damsite and Young's Crossing).

Calibration of the models were based on matching peak discharge and flood volumes by means of varying loss rates and model parameters k and m. A range of m values was considered for each site and each event and the most appropriate value of k selected for each different m value considered. Parameter interaction diagrams were constructed for each gauging station location to enable a unique pair of model parameters to be selected for that site. These diagrams are presented in Figures 6.20, 6.21 and 6.22.

The diagrams do not reveal a unique set of model parameters for any of the sites because the lines do not intersect at or close to a common point. A possible reason for this is that the rating curves may change between events at each station because of the sand and gravel controls, thus changing appropriate k and m values. It was noted during the calibration that lower m values produced 'better' looking fits, particularly on hydrograph recessions. The interaction diagrams, however, show greater disparity for lower m values.

In view of the above, an m value of 0.8 was adopted for all models. The adoption of this value is in line with the recommendations of Australian Rainfall and Runoff (1987).

6.4.1 Runoff-Routing Model Parameters

The following runoff routing model parameters were adopted for each model:

North Pine River at Damsite
$k = 46.1 \quad m = 0.8$

North Pine River at Young's Crossing
$k = 51.4 \quad m = 0.8$

South Pine River at Draper's Crossing
$k = 15.2 \quad m = 0.8$

The value of k for South Pine River to Draper's Crossing compares well with regional formulae proposed by Weeks and McMahon and Muller (refer Australian Rainfall and Runoff, 1987). These formulae estimate the k value for this catchment to be the following:

Weeks	$k = 12.9$	$m = 0.8$
McMahon and Muller	$k = 16.8$	$m = 0.8$

Since the calibration model parameters lie between the regional formulae estimates the adopted values appear appropriate.

John Wilson and Partners (1990) who investigated a possible dam site on South Pine River at Greenwood (one kilometre downstream from Draper's Crossing) also calibrated a runoff routing model to this station. The John Wilson and Partners report indicates a k = 10 and m = 0.8 was derived for a RORB catchment model that consisted on eight sub-areas. This report was part of a preliminary investigation into the damsite and only three calibration events were considered.

This independent study does show that the magnitude of the model parameters is similar to the values obtained above.

The values of k for the North Pine River catchments appear quite different from the regional formulae estimates. These estimates are as follows:

<u>North Pine River at Damsite</u>

Weeks	k = 19.5	m = 0.8
McMahon and Muller	k = 24.4	m = 0.8

<u>North Pine River at Young's Crossing</u>

Weeks	k = 21.0	m = 0.8
McMahon and Muller	k = 26.4	m = 0.8

It is apparent from catchment characteristics of area and distance to centroid of area, that South Pine River and North Pine River are quite different. This is because the mean travel time through South Pine River is some three quarters of that of North Pine River, whilst South Pine River has only half the catchment area. It is also evident from topographic plans of the two catchments that North Pine River has more 'storage' than South Pine River. That is, North Pine River is less confined and has much larger floodplains than South Pine River. This factor implies that the k value should be larger than otherwise expected.

The fact that both the Damsite and Young's Crossing produce similar results reinforces confidence in the rating curves of both stations. Proportioning the model parameters obtained for Young's Crossing by a ratio of distance to centroid of area produces a k value at the Damsite of 44.5. This agrees reasonably well with the adopted value at this site.

6.4.2 <u>**Rainfall Loss Parameters**</u>

The loss model adopted for all models is an initial loss, continuing loss type. Tables 6.13 to 6.15 summarise the losses that were adopted for each event for each model.

TABLE 6.13

NORTH PINE RIVER AT DAMSITE

CALIBRATION LOSSES

EVENT	INITIAL LOSS (mm)	CONTINUING LOSS (mm/hr)
June 1967A	0	0.71
June 1967B	20	1.03
June 1967C	5	0.43
December 1970	7	0.06

TABLE 6.14

NORTH PINE RIVER AT YOUNG'S CROSSING

CALIBRATION LOSSES

EVENT	INITIAL LOSS (mm)	CONTINUING LOSS (mm/hr)
June 1967A	0	0.24
June 1967B	15	0.71
June 1967C	0	0.15
December 1970	7	1.19
July 1973	65	2.12

TABLE 6.15

SOUTH PINE RIVER AT DRAPER'S CROSSING

CALIBRATION LOSSES

EVENT	INITIAL LOSS (mm)	CONTINUING LOSS (mm/hr)
June 1967A	5	0.37
December 1970	25	1.71
July 1973	100	4.71
January 1974	0	0.02
June 1983	5	0.54
April 1988	20	1.29
April 1989	30	0.36

Initial losses adopted for the North Pine River models are very similar, as to be expected. The associated continuing losses however, are reasonably different because of inconsistencies between the rating curves of the two stations. This is especially so for the December 1970 event, when the damsite gauge had been relocated to the YMCA Camp. The rating curve for this site appears to overestimate flows and as a consequence the volume of the hydrograph at the Damsite is much larger than the corresponding volume at Young's Crossing. The continuing loss for this event is therefore a lot lower for the damsite model than the Young's Crossing model.

In general the continuing losses for the North Pine River catchment appear small, being mostly below the 1 mm/hr range. These values appear consistent with the predominant soil types. They indicate low infiltration rates which is characteristic of the shallow stoney soils encountered in the catchment. The initial losses in turn reflect antecedent wetness of the catchment and the values obtained are consistent with preceding rainfalls for the events considered. For example, the first flood event that occurred in June 1967 was preceeded by between 20 to 90 millimetres of rainfall. This total fell in the twelve hours prior to the event. The second event of June 1967 was preceeded by seven days of no rainfall.

South Pine River loss rates are of the same order as the North Pine catchment values. This is to be expected, given the similarity of the soil types of the catchments.

The only noticeably different values are those associated with the July 1973 event. A high initial loss and corresponding continuing loss were required for the calibration of this event at the Draper's Crossing gauging station. The peak discharge of this event was some 115 m^3/s which was an estimated record. It would appear likely the estimated hydrograph may well be underestimated for this particular event which would reduce the loss rates to more consistent values. (Adopting the North Pine River model loss parameters for this event results in an estimated peak discharge of 243 m^3/s and a flood volume of 21030 megalitres).

It should be noted all events were treated as single rainfall bursts even though a number of the events were multi-peaked in nature. This is because WT42PC does not have the capability to handle multi-burst rainfalls. The calibration of some of the multi-peaked events could possibly be improved if this facility was available.

6.5 CALIBRATION RESULTS

6.5.1 North Pine River at Damsite

Results of model calibration for the four events considered at this site are presented in Table 6.16 and Figures 6.23 to 6.26. The adopted m and k values of 0.8 and 46.1 were used in conjunction with the rainfall loss parameters listed in Table 6.13 to achieve the results shown.

TABLE 6.16

CALIBRATION RESULTS

NORTH PINE RIVER AT DAMSITE

EVENT	PEAK DISCHARGE (m^3/s)			VOLUME (ML)		
	RECORDED	MODELLED	% ERROR	RECORDED	MODELLED	% ERROR
June 1967A (a) (b)	258 385	228 387	-11.6 0.5	56400	56215	-0.3
June 1967B (a)	223	197	-11.7	23845	23395	-1.9
June 1967C (a)	229	244	6.6	28870	28705	-0.6
December (a) 1970 (b)	530 312	537 282	1.3 -9.6	84540	84355	-0.2

(a) and (b) refer to various peaks that occurred in a multi-peaked event.

The table and comparisons in hydrographs indicate a reasonable calibration for North Pine River at the Damsite. Peak discharges and flood volumes are generally within 10% of the recorded values except for the smallest event of June 1967B. The general shape of the hydrographs are reasonably well matched, indicating the pluviograph records are adequate for the events considered. The only consistently occurring discrepancy that is evident from the hydrograph plots concerns the recession limb. To better match the recession limb the model parameter k could be increased (which will also decrease the peak discharge) or the model parameter m could also be increased (increasing the m value also delays the peak). Since the model calibration was most concerned with reproducing peak discharge and flood volume, neither of the above actions appear warranted.

6.5.2 North Pine River at Young's Crossing

Five events were considered during calibration of this model. Results from these events are presented in Table 6.17 and Figures 6.27 to 6.31. The adopted values of m and k of 0.8 and 51.4 were used along with rainfall loss parameters listed in Table 6.14 for the modelling of flood hydrographs.

TABLE 6.17

NORTH PINE RIVER AT YOUNG'S CROSSING

CALIBRATION RESULTS

EVENT	PEAK DISCHARGE (m^3/s)			VOLUME (ML)		
	RECORDED	MODELLED	% ERROR	RECORDED	MODELLED	% ERROR
June 1967A (a) (b)	283 419	266 413	- 6.0 - 1.4	65980	65765	-0.3
June 1967B (a)	255	234	- 8.2	28475	27950	-1.8
June 1967C (a)	271	282	4.1	34350	34155	-0.6
December 1970 (a) (b)	470 280	435 208	- 7.4 -25.7	66660	66465	-0.3
July 1973 (a)	669	727	8.7	74790	74620	-0.2

(a) and (b) refer to various peaks that occurred in a multi-peaked event.

The calibration of North Pine River at Young's Crossing model is perhaps not as good as the model for the damsite. The December 1970 event is underestimated, whilst the July 1973 event is overestimated and the small secondary peak of this event was not matched. Generally, the peak discharges were all within 10% of the recorded values, although some of the secondary peaks are underestimated by a substantial amount.

These problems may be overcome by the use of a multi-burst rainfall loss model.

6.5.3 **South Pine River at Draper's Crossing**

Table 6.18 and Figures 6.32 to 6.38 show the results of the South Pine River catchment model, using the adopted parameters of m = 0.8 and k = 15.2 together with the loss parameters listed in Table 6.15.

TABLE 6.18

SOUTH PINE RIVER AT DRAPER'S CROSSING

CALIBRATION RESULTS

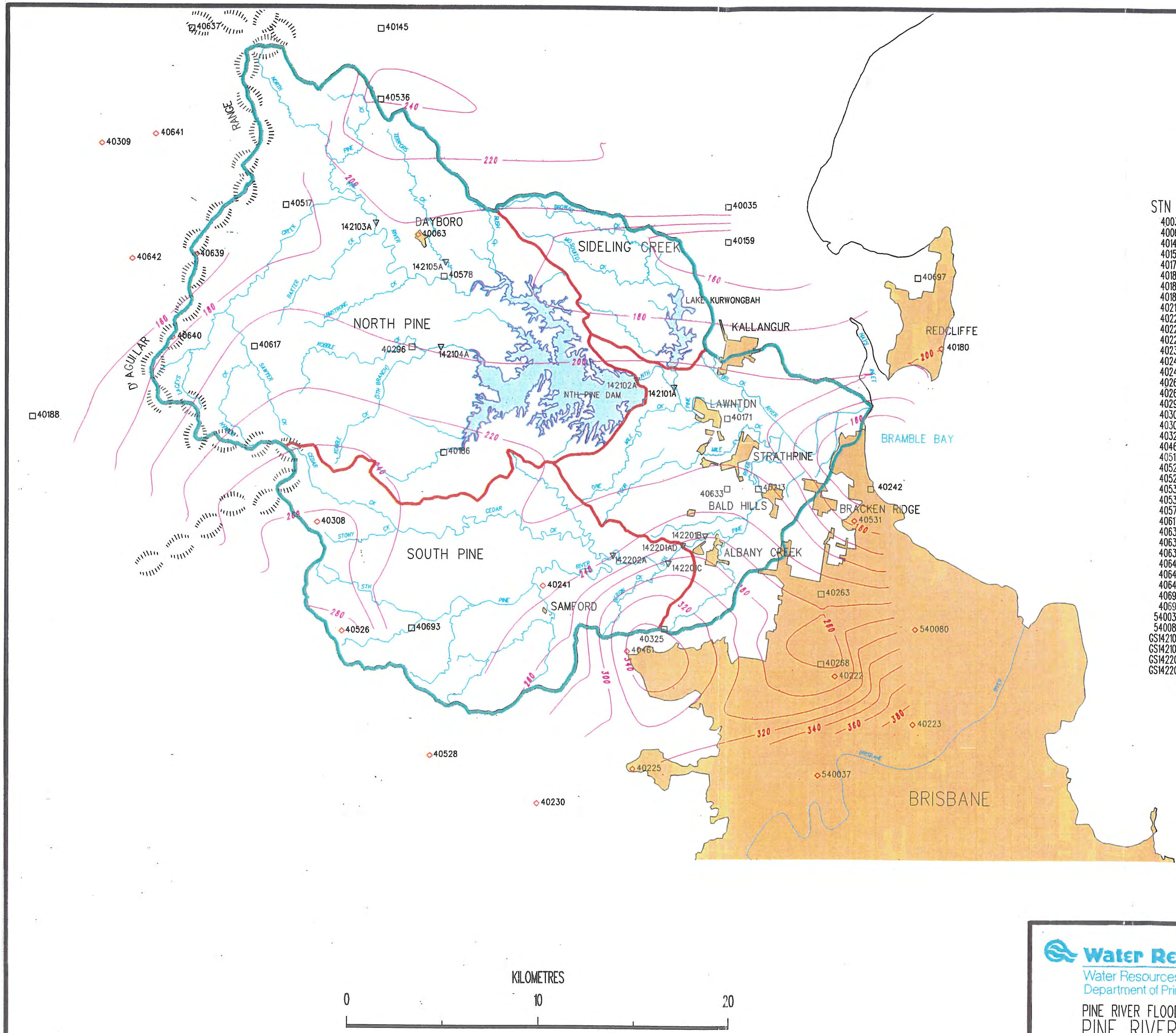
EVENT	PEAK DISCHARGE (m^3/s)			VOLUME (ML)		
	RECORDED	MODELLED	% ERROR	RECORDED	MODELLED	% ERROR
June 1967A (a) (b)	124 466	131 475	5.6 1.9	35130	35125	0
December (a) 1970 (b)	213 151	211 197	-0.9 30.5	20820	20820	0
July 1973 (a)	115	134	16.5	8930	8925	-0.1
January (a) 1974 (b) (c) (d)	1238 1050 1239 972	1295 - 1090 1005	4.6 - -11.7 3.4	156270	156265	0
June 1983 (a)	356	335	-5.9	21670	21650	-0.1
April 1988 (a) (b) (c) (d) (e)	248 520 174 378 282	244 455 152 215 234	-1.6 -12.5 -12.6 -43.1 -17.0	46415	46250	-0.4
April 1989 (a) (b) (c)	339 923 88	311 770 74	-8.3 -16.6 -15.9	45980	45260	-1.6

(a) and (b), etc, refer to various peaks that occurred during a multi-peaked event.

The results of the calibration of the South Pine River model are quite reasonable, especially considering the multi-peaked events of April 1988 and January 1974.

Most of the peak discharges are again within 10% of the recorded values except for the July 1973, April 1988 and April 1989 events. As was indicated earlier in the discussion concerning rainfall losses, the recorded peak discharge for the July 1973 event was estimated and as a consequence it may be lower than the actual value. The April 1988 event is multi-peaked with the highest peak discharge occurring as the second of five peaks. The fit of this event could be improved if the initial peak was considered independently.

FIGURE 6.1



LEGEND

◇ PLUVIO STATION

□ RAINFALL STATION

▽ GAUGING STATION

STN NO	STN NAME
40035	BURPENGARY
40063	DAYBORO
40145	MOUNT MEE
40159	NARANGBA
40171	PETRIE AUST. PAPER MILL
40180	MARGATE
40186	SAMSONVALE COMPOSITE
40188	SIM JUES CREEK
40213	BALD HILLS
40222	KALINGA BOWLING CLUB
40223	BRISBANE AIRPORT
40225	ENOGGERA RESERVOIR
40230	GOLD CREEK RESERVOIR
40241	SAMFORD CSIRO
40242	SANDGATE
40263	ZILLMERE POST OFFICE
40268	CHERMSIDE POST OFFICE
40296	DAYBORO (KOBBLESTONE)
40308	MT GLORIOUS
40309	MT BYRON
40325	FERNY GROVE
40461	FERNY GROVE (MUSEUM)
40517	DAYBORO (MACKENZIE CREEK)
40526	MT NEBO (WAS 040147)
40528	BRISBANE (3 CATCHMENTS)
40531	SANDGATE (DEAGON BCC)
40536	DAYBORO (OCEAN VIEW)
40578	DAYBORO (ARMSTRONG CREEK)
40617	DAYBORO (RAYNBIRD CREEK)
40633	STRATHPINE
40637	DAYBORO (MOUNT MEE FOREST STATION)
40639	DAYBORO (BYRON & REEDY CKS NO 2)
40640	DAYBOBO (BYRON & REEDY CKS NO 3)
40641	DAYBORO (BYRON & REEDY CKS NO 4)
40642	DAYBORO (BYRON & REEDY CKS NO 5)
40693	SAMFORD (HIGHVALE)
40697	REDCLIFFE AIRPORT
540037	WATER STREET DRAIN STEVENS STORAGE
540080	BANYO (ARMY DEPOT)
GS142101A	NTH PINE RIVER AT YOUNG'S CROSSING
GS142102A	NTH PINE RIVER AT DAMSITE
GS142201AD	STH PINE RIVER AT CASH'S CROSSING
GS142202A	STH PINE RIVER AT DRAPER'S CROSSING

START TIME: 0800 hrs
DATE: 9/6/67
DURATION: 66 hrs

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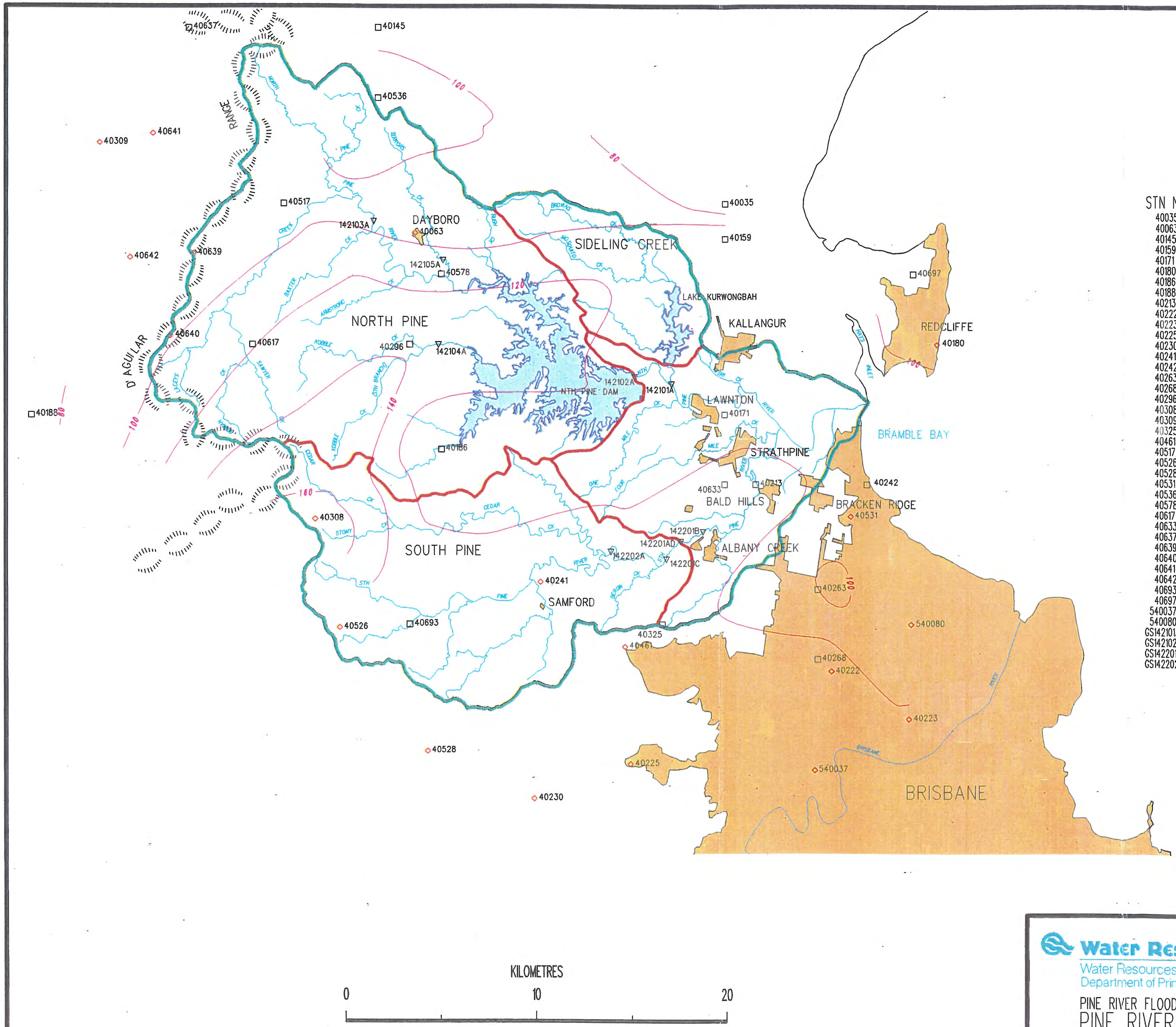
PINE RIVER FLOOD STUDY
PINE RIVER
ISOHYETS – JUNE 1967 (A)

A3 08887

FIGURE 6.2



LEGEND

PLUVIO STATION

RAINFALL STATION

GAUGING STATION

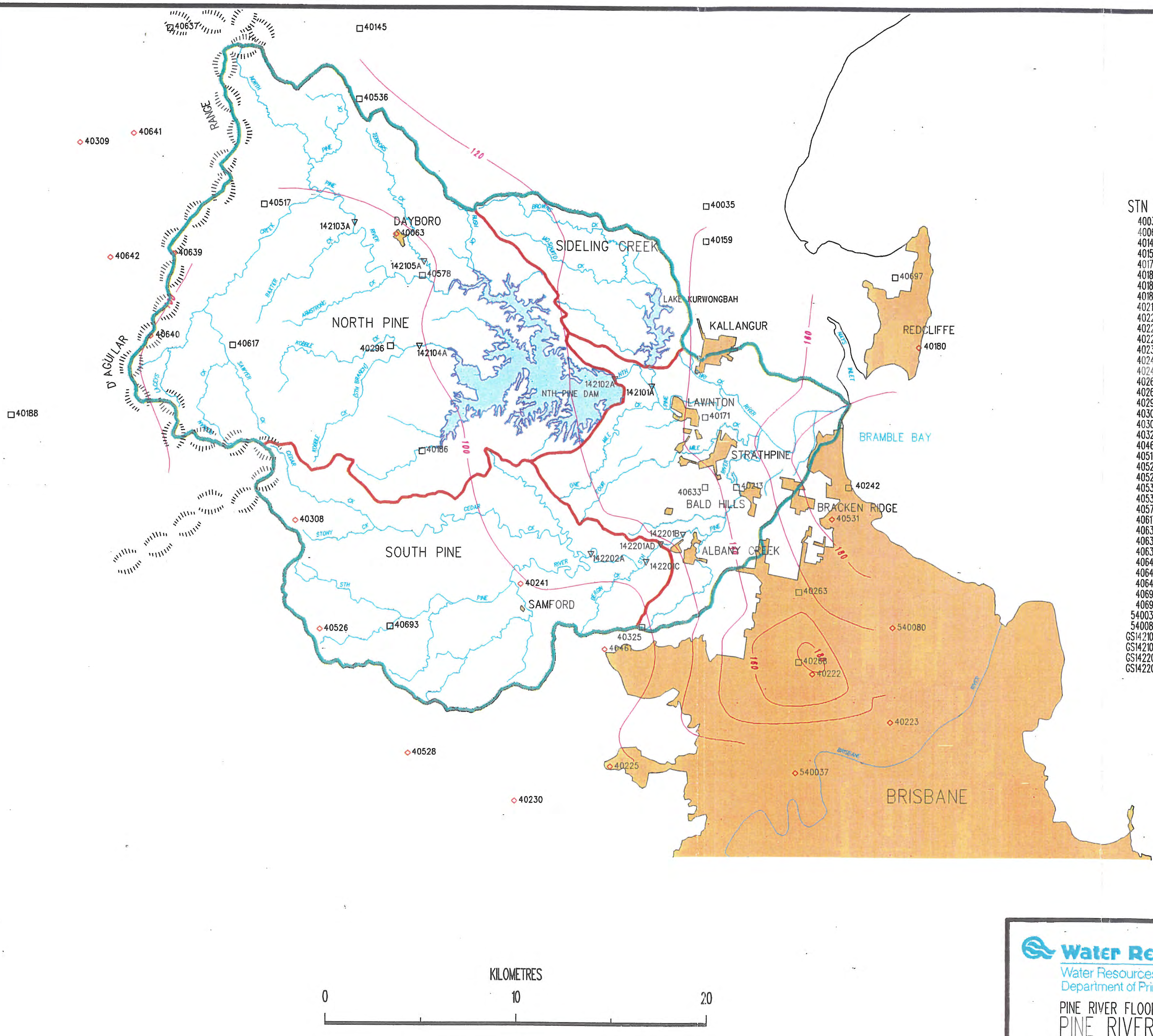
STN NO	STN NAME
40035	BURPENGARY
40063	DAYBORO
40145	MOUNT MEE
40159	NARANGBA
40171	PETRIE AUST. PAPER MILL
40180	MARGATE
40186	SAMSONVALE COMPOSITE
40188	SIM JUES CREEK
40213	BALD HILLS
40222	KALINGA BOWLING CLUB
40223	BRISBANE AIRPORT
40225	ENOGGERA RESERVOIR
40230	GOLD CREEK RESERVOIR
40241	SAMFORD CSIRO
40242	SANDGATE
40263	ZILLMERE POST OFFICE
40268	CHERMSIDE POST OFFICE
40296	DAYBORO (KOBBLESTONE)
40308	MT GLORIOUS
40309	MT BYRON
40325	FERNY GROVE
40461	FERNY GROVE (MUSEUM)
40517	DAYBORO (MACKENZIE CREEK)
40526	MT NEBO (WAS 040147)
40528	BRISBANE (3 CATCHMENTS)
40531	SANDGATE (DEAGON BCC)
40536	DAYBORO (OCEAN VIEW)
40578	DAYBORO (ARMSTRONG CREEK)
40617	DAYBORO (RAYNBIRD CREEK)
40633	STRATHPINE
40637	DAYBORO (MOUNT MEE FOREST STATION)
40639	DAYBORO (BYRON & REEDY CKS NO 2)
40640	DAYBOBO (BYRON & REEDY CKS NO 3)
40641	DAYBORO (BYRON & REEDY CKS NO 4)
40642	DAYBORO (BYRON & REEDY CKS NO 5)
40693	SAMFORD (HIGHVALE)
40697	REDCLIFFE AIRPORT
540037	WATER STREET DRAIN STEVENS STORAGE
540080	BANYO (ARMY DEPOT)
GS142101A	NTH PINE RIVER AT YOUNG'S CROSSING
GS142102A	NTH PINE RIVER AT DAMSITE
GS142201AD	STH PINE RIVER AT CASH'S CROSSING
GS142202A	STH PINE RIVER AT DRAPER'S CROSSING

START TIME: 1900 hrs
DATE: 20/6/67
DURATION: 42 hrs

Water Resources
Water Resources Commission
Department of Primary Industries

PINE RIVER FLOOD STUDY
PINE RIVER
ISOHYETS – JUNE 1967 (B)

FIGURE 6.3



LEGEND

◇ PLUVIO STATION

□ RAINFALL STATION

▽ GAUGING STATION

STN NO	STN NAME
40035	BURPENGARY
40063	DAYBORO
40145	MOUNT MEE
40159	NARANGBA
40171	PETRIE AUST. PAPER MILL
40180	MARGATE
40186	SAMSONVALE COMPOSITE
40188	SIM JUES CREEK
40213	BALD HILLS
40222	KALINGA BOWLING CLUB
40223	BRISBANE AIRPORT
40225	ENOGGERA RESERVOIR
40230	GOLD CREEK RESERVOIR
40241	SAMFORD CSIRO
40242	SANDGATE
40263	ZILLMERE POST OFFICE
40268	CHERMSIDE POST OFFICE
40296	DAYBORO (KOBBLESTONE)
40308	MT GLORIOUS
40309	MT BYRON
40325	FERNY GROVE
40461	FERNY GROVE (MUSEUM)
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40526	MT NEBO (WAS 040147)
40528	BRISBANE (3 CATCHMENTS)
40531	SANDGATE (DEAGON BCC)
40536	DAYBORO (OCEAN VIEW)
40578	DAYBORO (ARMSTRONG CREEK)
40617	DAYBORO (RAYNBIRD CREEK)
40633	STRATHPINE
40637	DAYBORO (MOUNT MEE FOREST STATION)
40639	DAYBORO (BYRON & REEDY CKS NO 2)
40640	DAYBOBO (BYRON & REEDY CKS NO 3)
40641	DAYBORO (BYRON & REEDY CKS NO 4)
40642	DAYBORO (BYRON & REEDY CKS NO 5)
40693	SAMFORD (HIGHVALE)
40697	REDCLIFFE AIRPORT
540037	WATER STREET DRAIN STEVENS STORAGE
540080	BANYO (ARMY DEPOT)
GS142101A	NTH PINE RIVER AT YOUNG'S CROSSING
GS142102A	NTH PINE RIVER AT DAMSITE
GS142201AD	STH PINE RIVER AT CASH'S CROSSING
GS142202A	STH PINE RIVER AT DRAPER'S CROSSING

START TIME: 0100 hrs
DATE: 25/6/67
DURATION: 52 hrs

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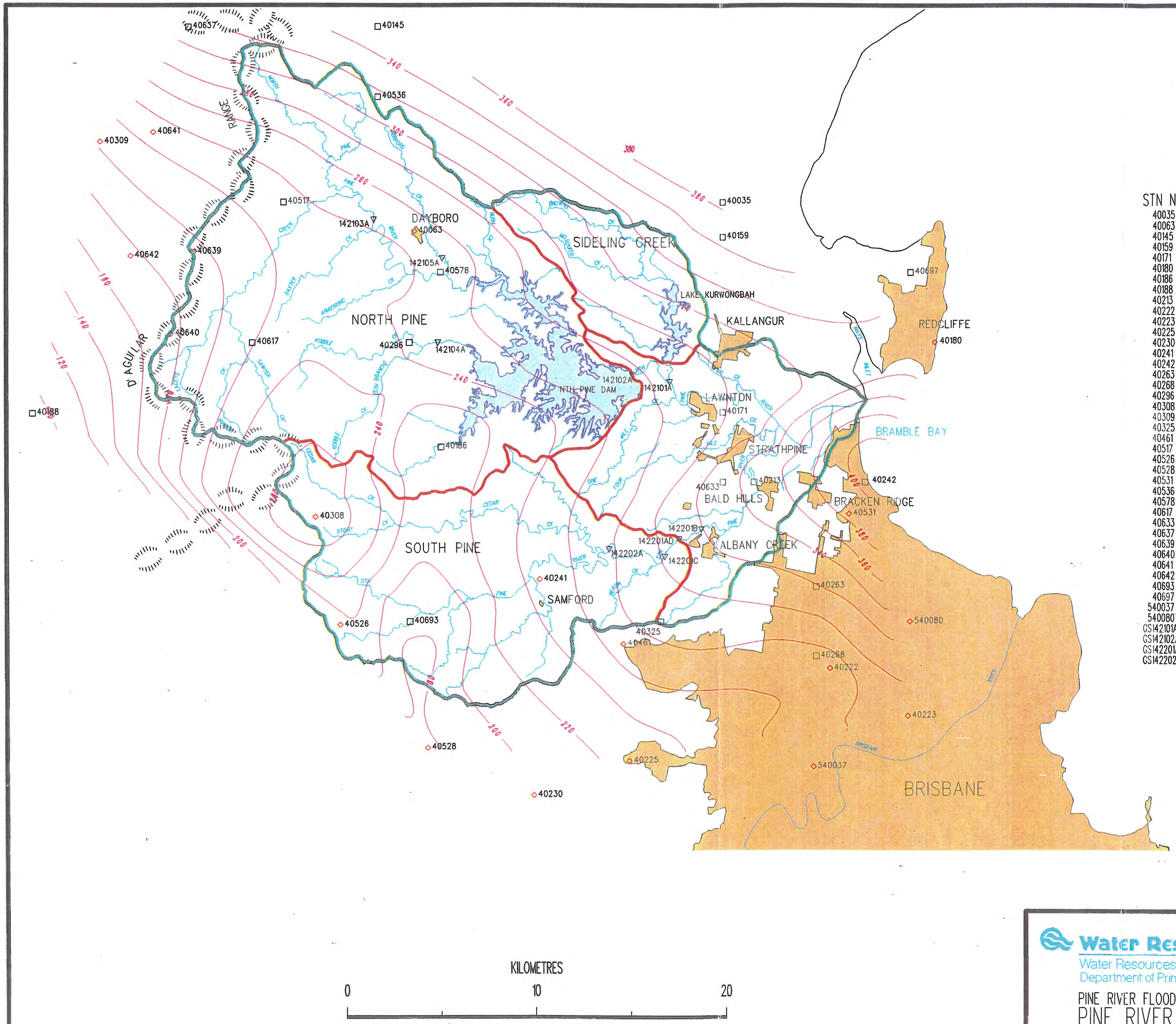
Department of Primary Industries

PINE RIVER FLOOD STUDY
PINE RIVER
ISOHYETS – JUNE 1967 (C)

A3-09885

FIGURE 6.4



LEGEND

◇ PLUVIO STATION

□ RAINFALL STATION

▽ GAUGING STATION

STN NO	STN NAME
40035	BURPENGARY
40063	DAYBORO
40145	MOUNT MEE
40159	NARANGBA
40171	PETRIE AUST. PAPER MILL
40180	MARGATE
40186	SAMSONVALE COMPOSITE
40188	SIM JUES CREEK
40213	BALD HILLS
40222	KALINGA BOWLING CLUB
40223	BRISBANE AIRPORT
40225	ENOGGERA RESERVOIR
40230	GOLD CREEK RESERVOIR
40241	SAMFORD CSIRO
40242	SANDGATE
40263	ZILLMERE POST OFFICE
40268	CHERMSIDE POST OFFICE
40296	DAYBORO (KOBBLESTONE)
40308	MT GLORIOUS
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40461	FERNY GROVE (MUSEUM)
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40526	MT NEBO (WAS 040147)
40528	BRISBANE (3 CATCHMENTS)
40531	SANDGATE (DEAGON BCC)
40536	DAYBORO (OCEAN VIEW)
40578	DAYBORO (ARMSTRONG CREEK)
40617	DAYBORO (RAYNBIRD CREEK)
40633	STRATHPINE
40637	DAYBORO (MOUNT MEE FOREST STATION)
40639	DAYBORO (BYRON & REEDY CKS NO 2)
40640	DAYBOBO (BYRON & REEDY CKS NO 3)
40641	DAYBORO (BYRON & REEDY CKS NO 4)
40642	DAYBORO (BYRON & REEDY CKS NO 5)
40693	SAMFORD (HIGHVALE)
40697	REDCLIFFE AIRPORT
540037	WATER STREET DRAIN STEVENS STORAGE
540080	BANYO (ARMY DEPOT)
GS142101A	NTH PINE RIVER AT YOUNG'S CROSSING
GS142102A	NTH PINE RIVER AT DAMSITE
GS142201AD	STH PINE RIVER AT CASH'S CROSSING
GS142202A	STH PINE RIVER AT DRAPER'S CROSSING

START TIME: 0900 hrs
DATE: 6/12/70
DURATION: 84 hrs

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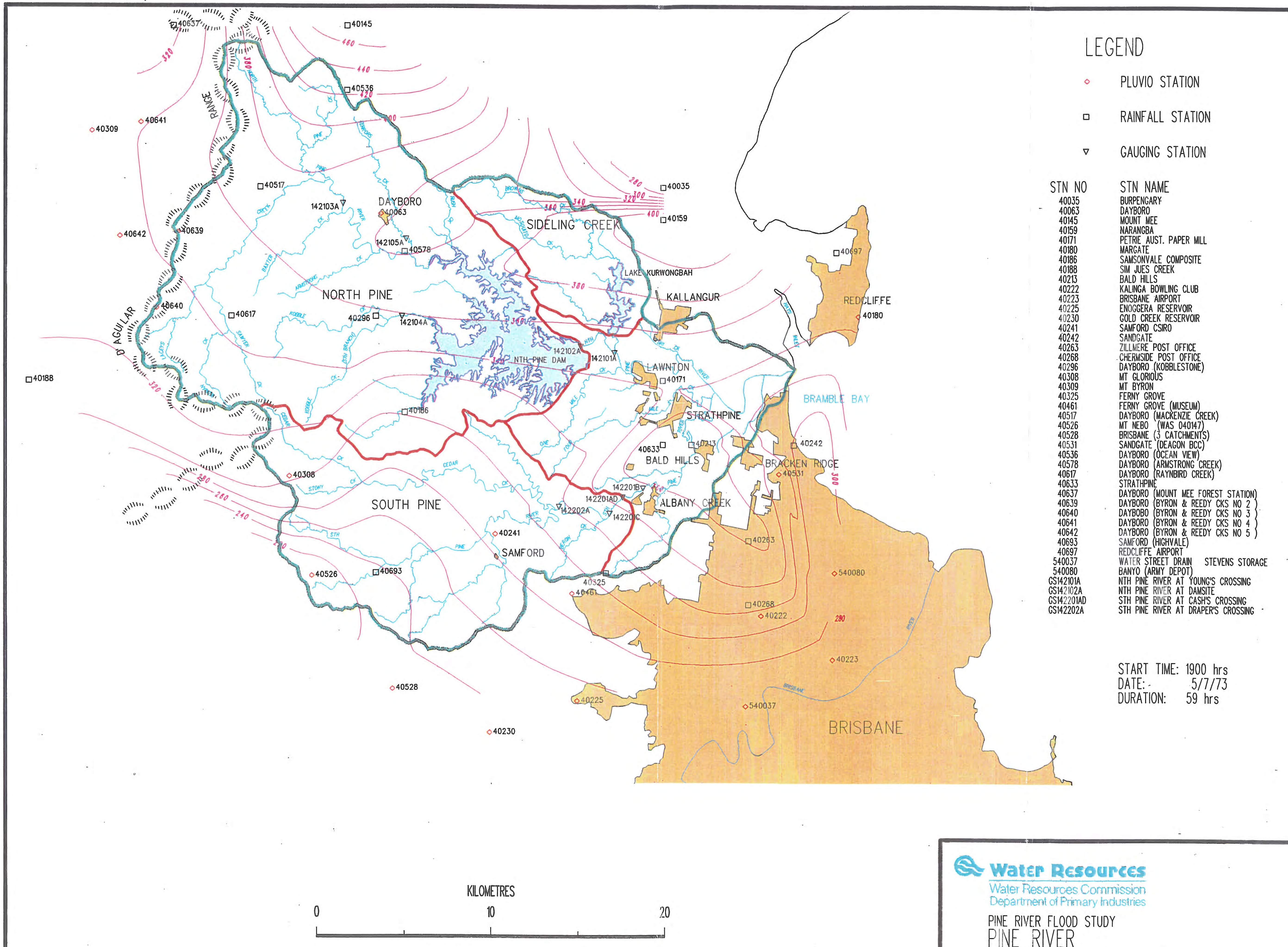
Department of Primary Industries

PINE RIVER FLOOD STUDY
PINE RIVER
ISOHYETS - DECEMBER 1970



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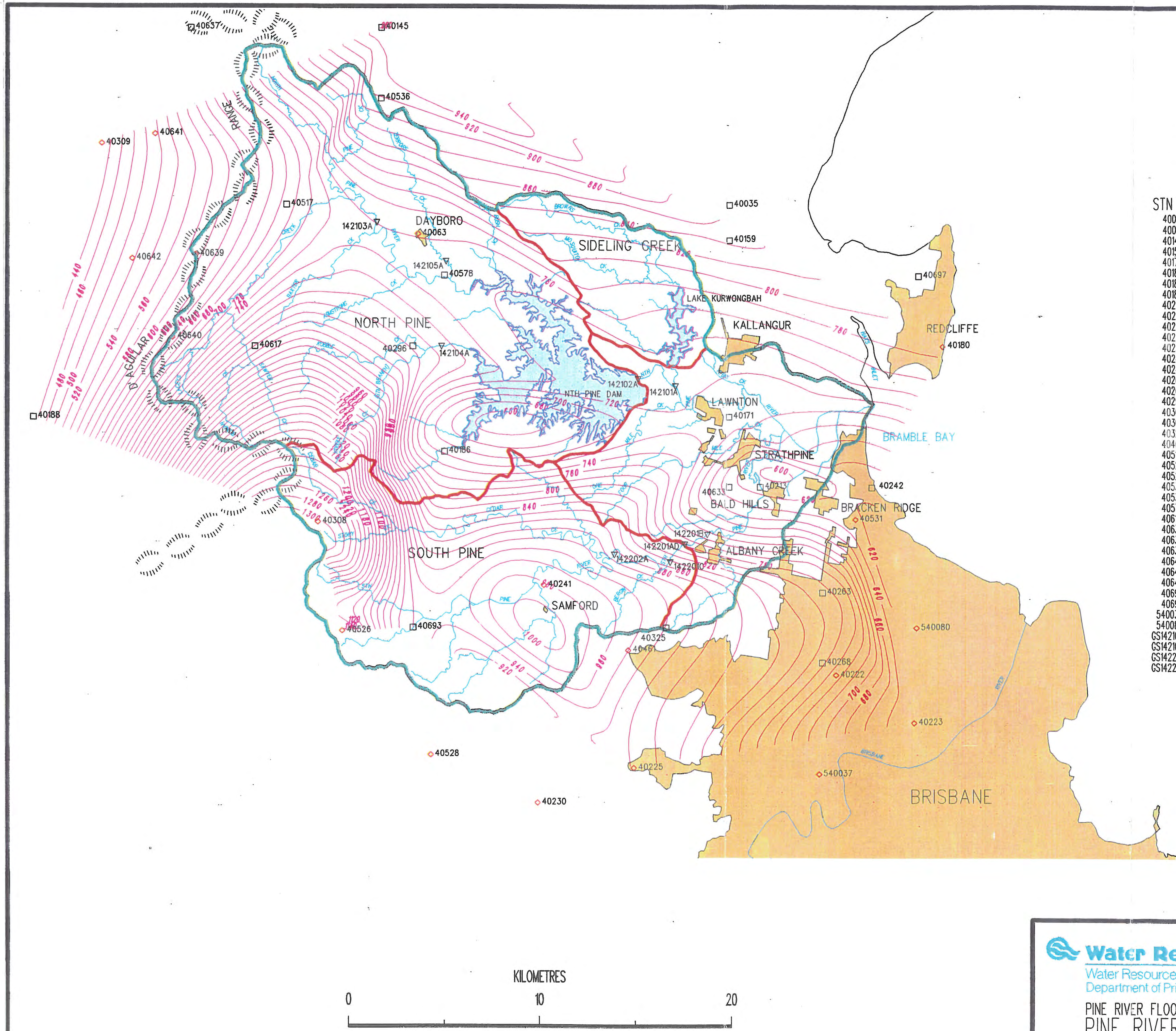
PINE RIVER FLOOD STUDY
PINE RIVER
ISOHYETS – JULY 1973

FIGURE 6.6



LEGEND

◇ PLUVIO STATION

□ RAINFALL STATION

▽ GAUGING STATION

STN NO	STN NAME
40035	BURPENGARY
40063	DAYBORO
40145	MOUNT MEE
40159	NARANGBA
40171	PETRIE AUST. PAPER MILL
40180	MARGATE
40186	SAMSONVALE COMPOSITE
40188	SIM JUES CREEK
40213	BALD HILLS
40222	KALINGA BOWLING CLUB
40223	BRISBANE AIRPORT
40225	ENOGGERA RESERVOIR
40230	GOLD CREEK RESERVOIR
40241	SAMFORD CSIRO
40242	SANDGATE
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40268	CHERMSIDE POST OFFICE
40296	DAYBORO (KOBBLESTONE)
40308	MT GLORIOUS
40309	MT BYRON
40325	FERNY GROVE
40461	FERNY GROVE (MUSEUM)
40517	DAYBORO (MACKENZIE CREEK)
40526	MT NEBO (WAS 040147)
40528	BRISBANE (3 CATCHMENTS)
40531	SANDGATE (DEAGON BCC)
40536	DAYBORO (OCEAN VIEW)
40578	DAYBORO (ARMSTRONG CREEK)
40617	DAYBORO (RAYNBIRD CREEK)
40633	STRATHPINE
40637	DAYBORO (MOUNT MEE FOREST STATION)
40639	DAYBORO (BYRON & REEDY CKS NO 2)
40640	DAYBOBO (BYRON & REEDY CKS NO 3)
40641	DAYBORO (BYRON & REEDY CKS NO 4)
40642	DAYBORO (BYRON & REEDY CKS NO 5)
40693	SAMFORD (HIGHVALE)
40697	REDCLIFFE AIRPORT
540037	WATER STREET DRAIN STEVENS STORAGE
540080	BANYO (ARMY DEPOT)
GS142101A	NTH PINE RIVER AT YOUNG'S CROSSING
GS142102A	NTH PINE RIVER AT DAMSITE
GS142201AD	STH PINE RIVER AT CASH'S CROSSING
GS142202A	STH PINE RIVER AT DRAPER'S CROSSING

START TIME: 0900 hrs
DATE: 24/1/74
DURATION: 96 hrs

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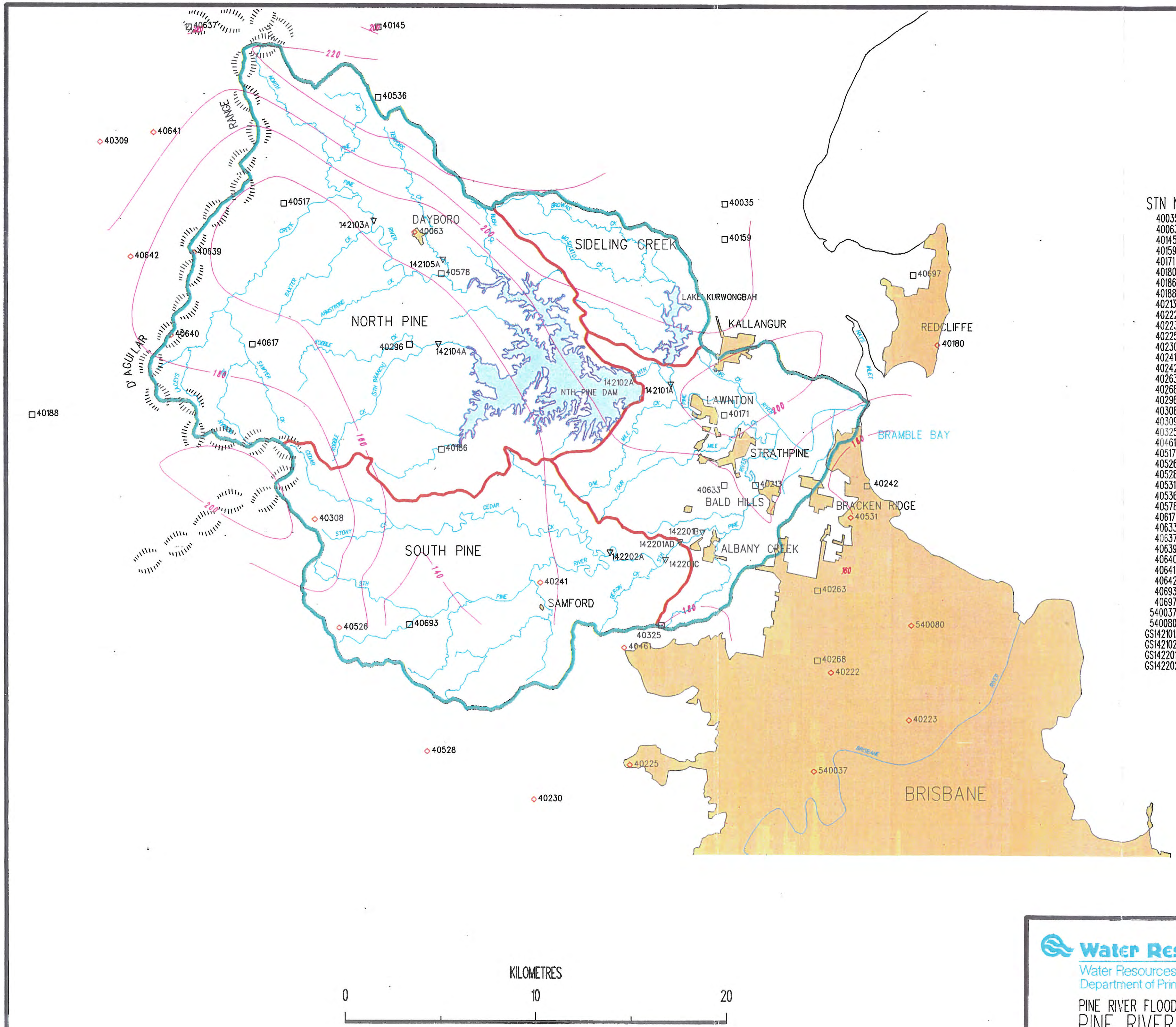
Department of Primary Industries

PINE RIVER FLOOD STUDY
PINE RIVER
ISOHYETS – JANUARY 1974

AZ 08888

FIGURE 6.7



KILOMETRES

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LEGEND

◇ PLUVIO STATION

□ RAINFALL STATION

▽ GAUGING STATION

STN NO	STN NAME
40035	BURPENGARY
40063	DAYBORO
40145	MOUNT MEE
40159	NARANGBA
40171	PETRIE AUST. PAPER MILL
40180	MARGATE
40186	SAMSONVALE COMPOSITE
40188	SIM JUES CREEK
40213	BALD HILLS
40222	KALINGA BOWLING CLUB
40223	BRISBANE AIRPORT
40225	ENOGGERA RESERVOIR
40230	GOLD CREEK RESERVOIR
40241	SAMFORD CSIRO
40242	SANDGATE
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40268	CHERMSIDE POST OFFICE
40296	DAYBORO (KOBBLESTONE)
40308	MT GLORIOUS
40309	MT BYRON
40325	FERNY GROVE
40461	FERNY GROVE (MUSEUM)
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40526	MT NEBO (WAS 040147)
40528	BRISBANE (3 CATCHMENTS)
40531	SANDGATE (DEAGON BCC)
40536	DAYBORO (OCEAN VIEW)
40578	DAYBORO (ARMSTRONG CREEK)
40617	DAYBORO (RAYNBIRD CREEK)
40633	STRATHPINE
40637	DAYBORO (MOUNT MEE FOREST STATION)
40639	DAYBORO (BYRON & REEDY CKS NO 2)
40640	DAYBOBO (BYRON & REEDY CKS NO 3)
40641	DAYBOBO (BYRON & REEDY CKS NO 4)
40642	DAYBORO (BYRON & REEDY CKS NO 5)
40693	SAMFORD (HIGHVALE)
40697	REDCLIFFE AIRPORT
540037	WATER STREET DRAIN STEVENS STORAGE
540080	BANYO (ARMY DEPOT)
GS142101A	NTH PINE RIVER AT YOUNG'S CROSSING
GS142102A	NTH PINE RIVER AT DAMSITE
GS142201AD	STH PINE RIVER AT CASH'S CROSSING
GS142202A	STH PINE RIVER AT DRAPER'S CROSSING

START TIME: 0600 hrs
DATE: 21/6/83
DURATION: 42 hrs

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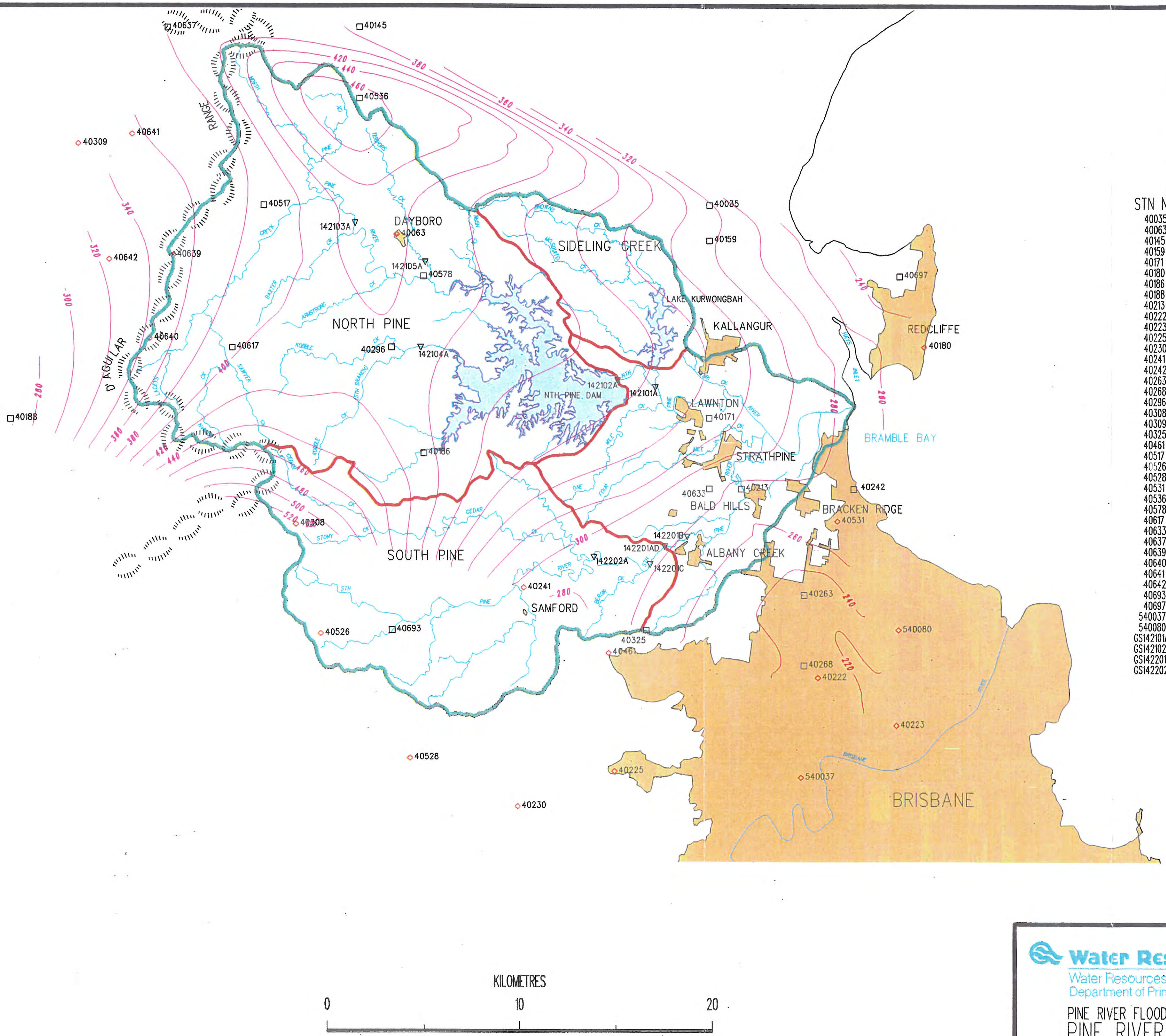
Department of Primary Industries

PINE RIVER FLOOD STUDY
PINE RIVER
ISOHYETS - JUNE 1983

A3-08880

FIGURE 6.8



LEGEND

◇ PLUVIO STATION

□ RAINFALL STATION

▽ GAUGING STATION

STN NO	STN NAME
40035	BURPENGARY
40063	DAYBORO
40145	MOUNT MEE
40159	NARANGBA
40171	PETRIE AUST. PAPER MILL
40180	MARGATE
40186	SAMSONVALE COMPOSITE
40188	SIM JUES CREEK
40213	BALD HILLS
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40641	DAYBORO (BYRON & REEDY CKS NO 4)
40642	DAYBORO (BYRON & REEDY CKS NO 5)
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40697	REDCLIFFE AIRPORT
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GS142101A	NTH PINE RIVER AT YOUNG'S CROSSING
GS142102A	NTH PINE RIVER AT DAMSITE
GS142201AD	STH PINE RIVER AT CASH'S CROSSING
GS142202A	STH PINE RIVER AT DRAPER'S CROSSING

START TIME: 0900 hrs
DATE: 3/4/88
DURATION: 84 hrs

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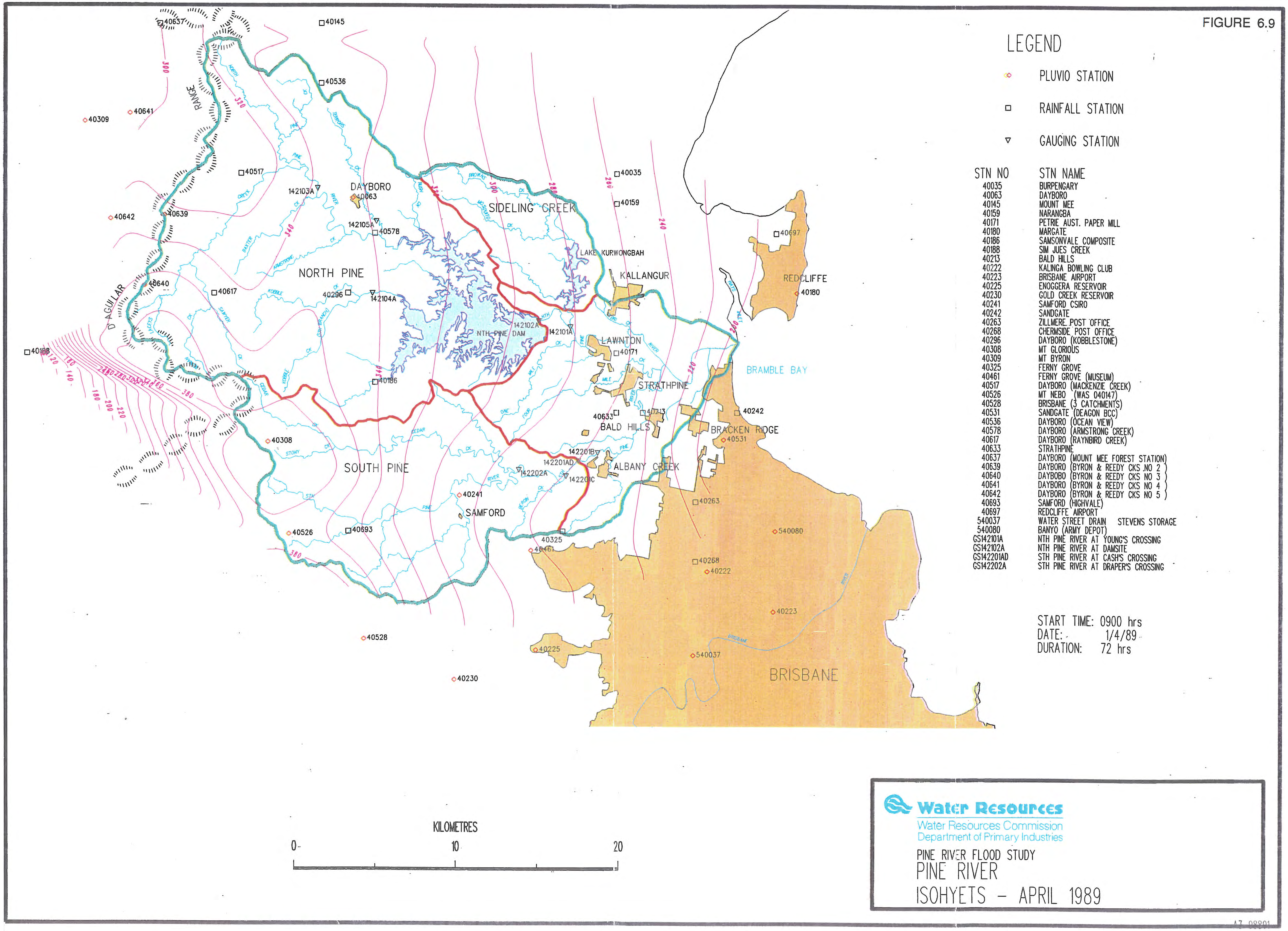
Water Resources Commission
Department of Primary Industries

PINE RIVER FLOOD STUDY
PINE RIVER
ISOHYETS - APRIL 1988

FIGURE 6.9


LEGEND
PLUVIO STATION
RAINFALL STATION
GAUGING STATION
STN NO STN NAME
40035 BURPENGARY
40063 DAYBORO
40145 MOUNT MEE
40159 NARANGBA
40171 PETRIE AUST. PAPER MILL
40180 MARGATE
40186 SAMSONVALE COMPOSITE
40188 SIM JUES CREEK
40213 BALD HILLS
40222 KALINGA BOWLING CLUB
40223 BRISBANE AIRPORT
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540080 BANYO (ARMY DEPOT)
GS142101A NTH PINE RIVER AT YOUNG'S CROSSING
GS142102A NTH PINE RIVER AT DAMSITE
GS142201AD STH PINE RIVER AT CASH'S CROSSING
GS142202A STH PINE RIVER AT DRAPER'S CROSSING
START TIME: 0900 hrs
DATE: 1/4/89
DURATION: 72 hrs
NORTH PINE
SOUTH PINE
SIDELING CREEK
DAYBORO
LAKE KURWONGBAH
KALLANGUR
LAWNTON
STRATHPINE
BALD HILLS
BRACKEN RIDGE
ALBANY CREEK
SAMFORD
REDCLIFFE
BRAMBLE BAY
BRISBANE
NTH PINE DAM
D'AGUILAR RANGE
KILOMETRES
0
10
20
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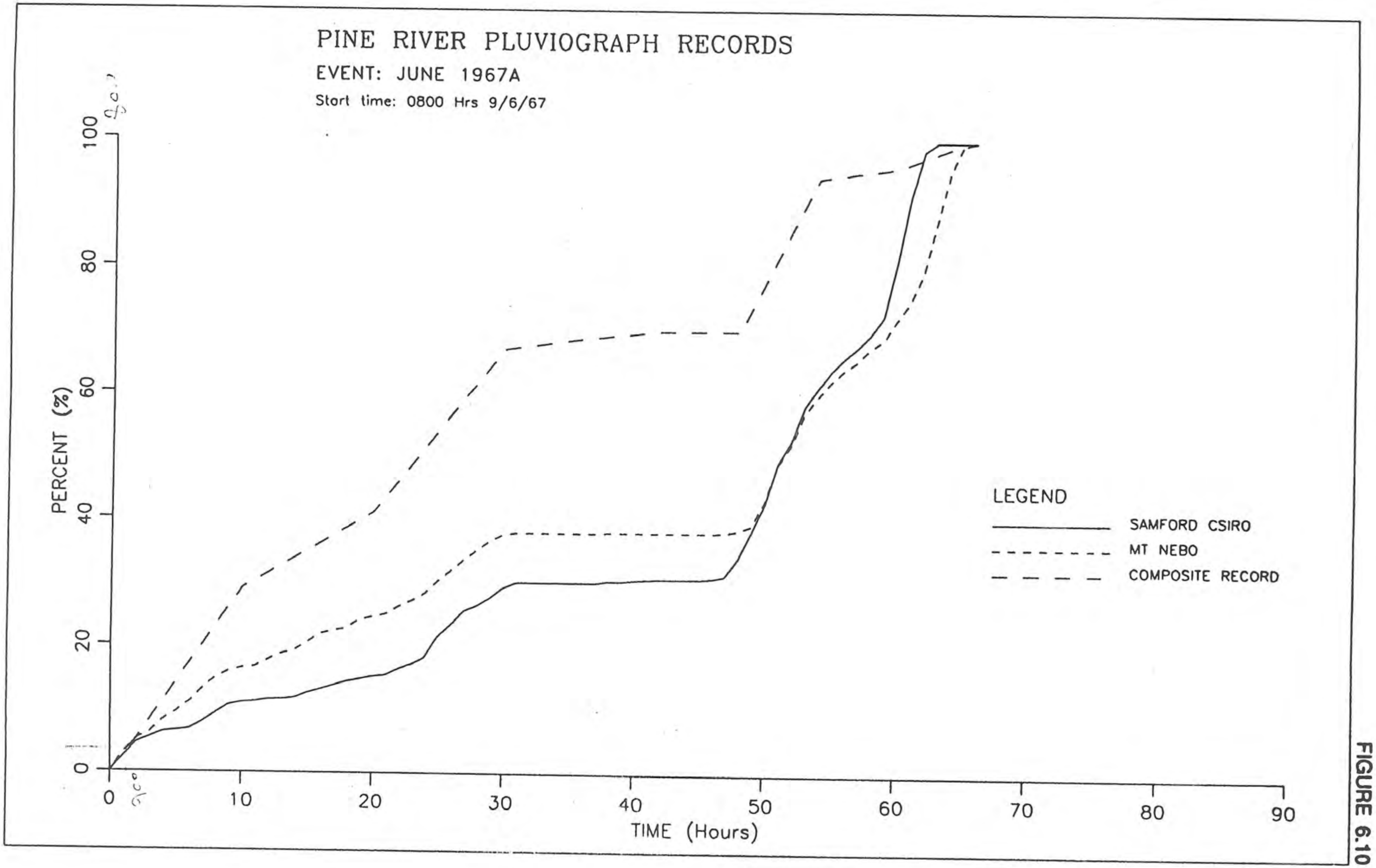
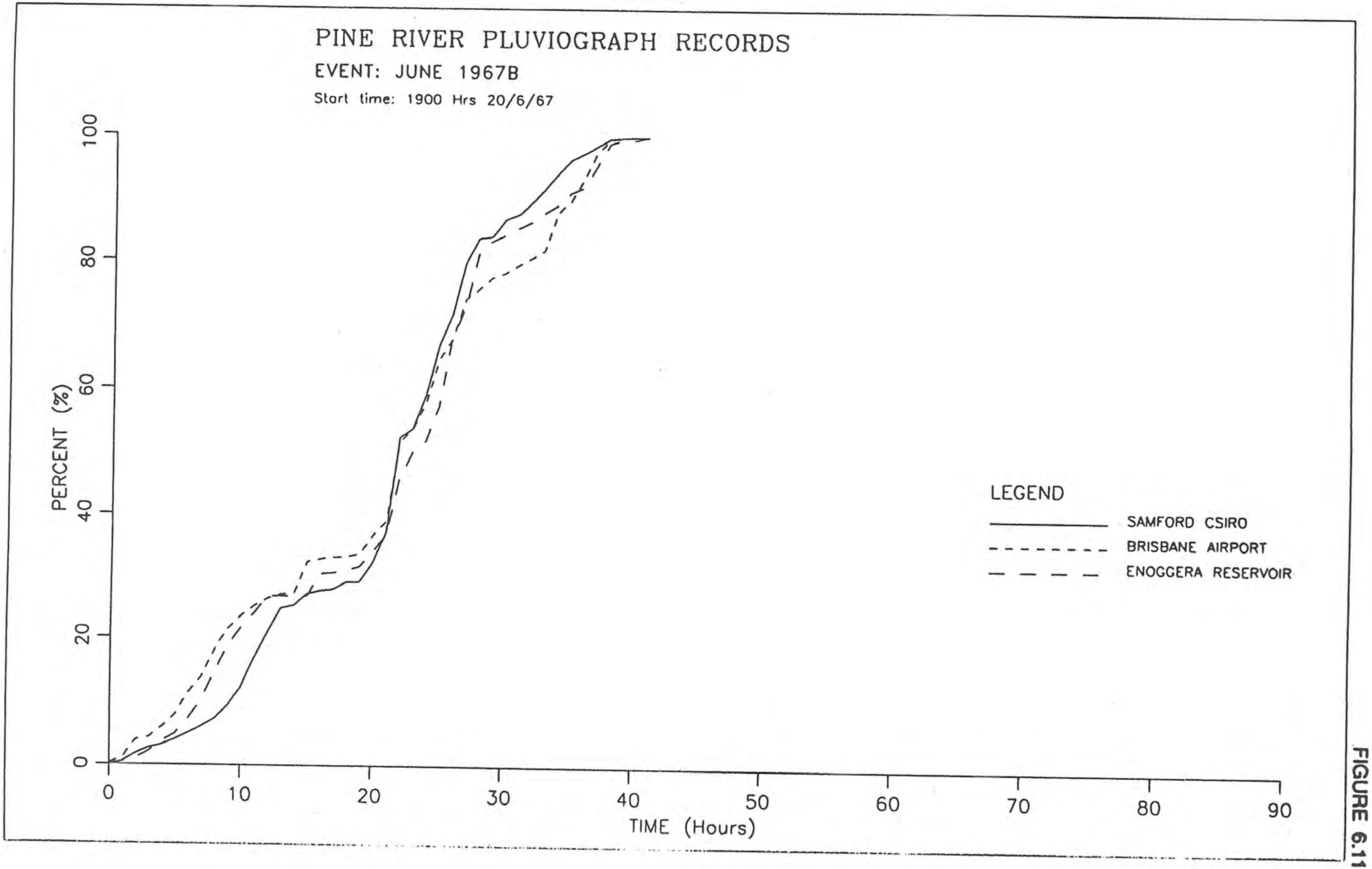
Water Resources Commission
Department of Primary Industries
PINE RIVER FLOOD STUDY
PINE RIVER
ISOHYETS - APRIL 1989



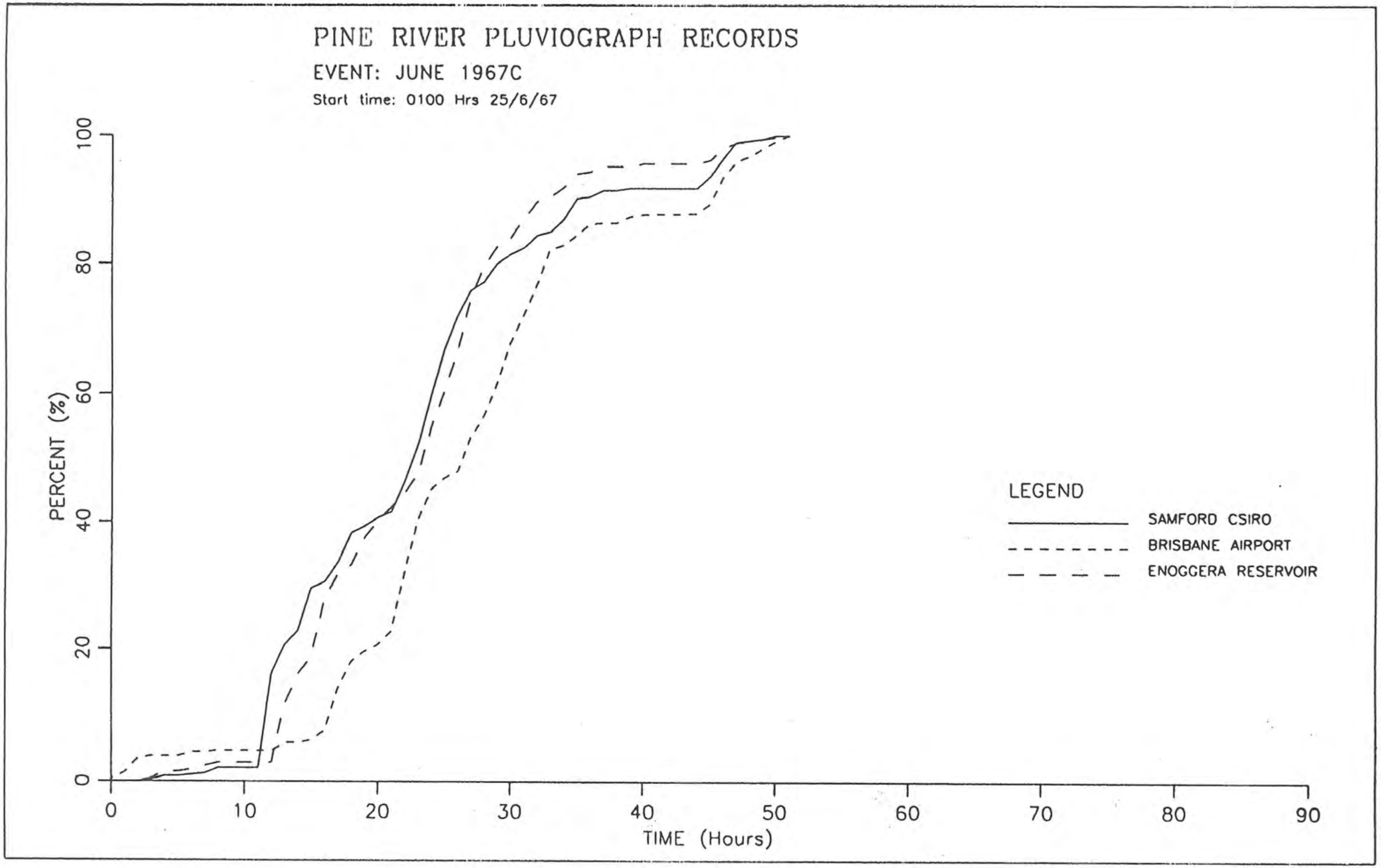
PINE RIVER PLUVIOGRAPH RECORDS
EVENT: JUNE 1967A
Start time: 0800 Hrs 9/6/67
PERCENT (%)
0
20
40
60
80
100
TIME (Hours)
0
10
20
30
40
50
60
70
80
90
LEGEND
SAMFORD CSIRO
MT NEBO
COMPOSITE RECORD

FIGURE 6.10



PINE RIVER PLUVIOGRAPH RECORDS
EVENT: JUNE 1967B
Start time: 1900 Hrs 20/6/67
PERCENT (%)
0
20
40
60
80
100
0
10
20
30
40
50
60
70
80
90
TIME (Hours)
LEGEND
SAMFORD CSIRO
BRISBANE AIRPORT
ENOGGERA RESERVOIR

FIGURE 6.11



PINE RIVER PLUVIOGRAPH RECORDS
EVENT: JUNE 1967C
Start time: 0100 Hrs 25/6/67
PERCENT (%)
0
20
40
60
80
100
0
10
20
30
40
50
60
70
80
90
TIME (Hours)
LEGEND
SAMFORD CSIRO
BRISBANE AIRPORT
ENOGGERA RESERVOIR

FIGURE 6.12

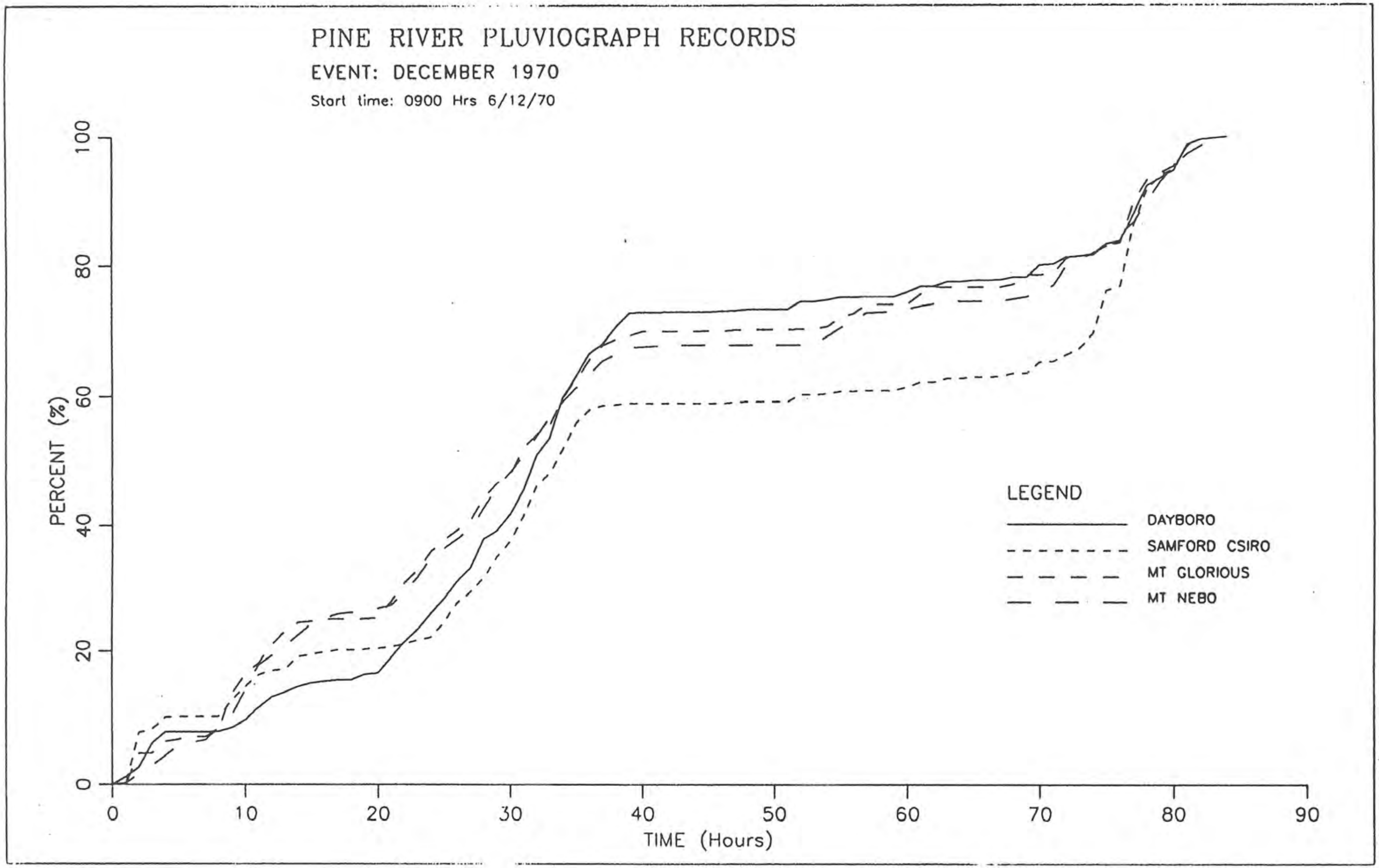
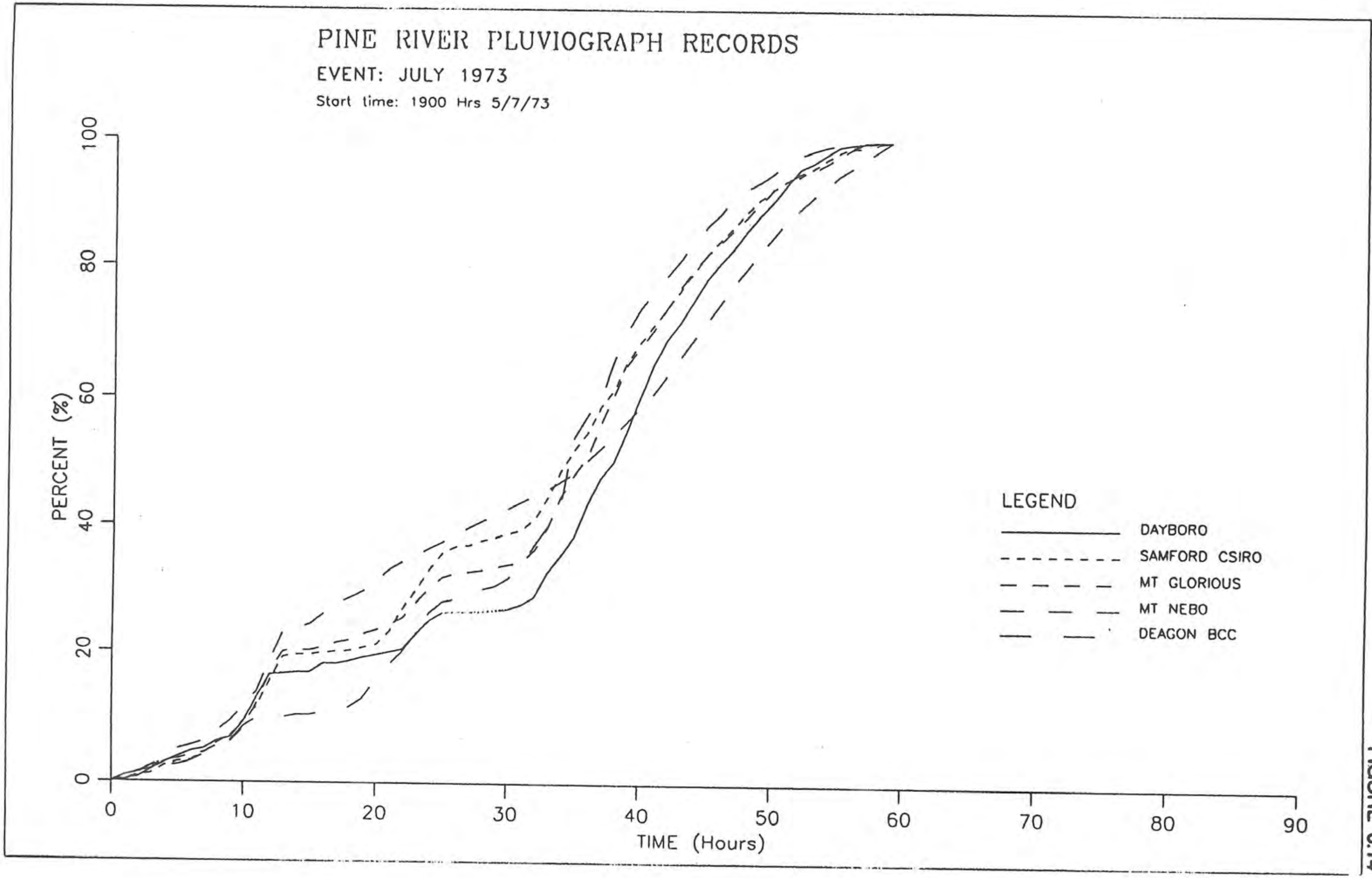


FIGURE 6.13

DOE



PINE RIVER PLUVIOGRAPH RECORDS
EVENT: JULY 1973
Start time: 1900 Hrs 5/7/73
PERCENT (%)
0
20
40
60
80
100
0
10
20
30
40
50
60
70
80
90
TIME (Hours)
LEGEND
DAYBORO
SAMFORD CSIRO
MT GLORIOUS
MT NEBO
DEAGON BCC

FIGURE 6.14

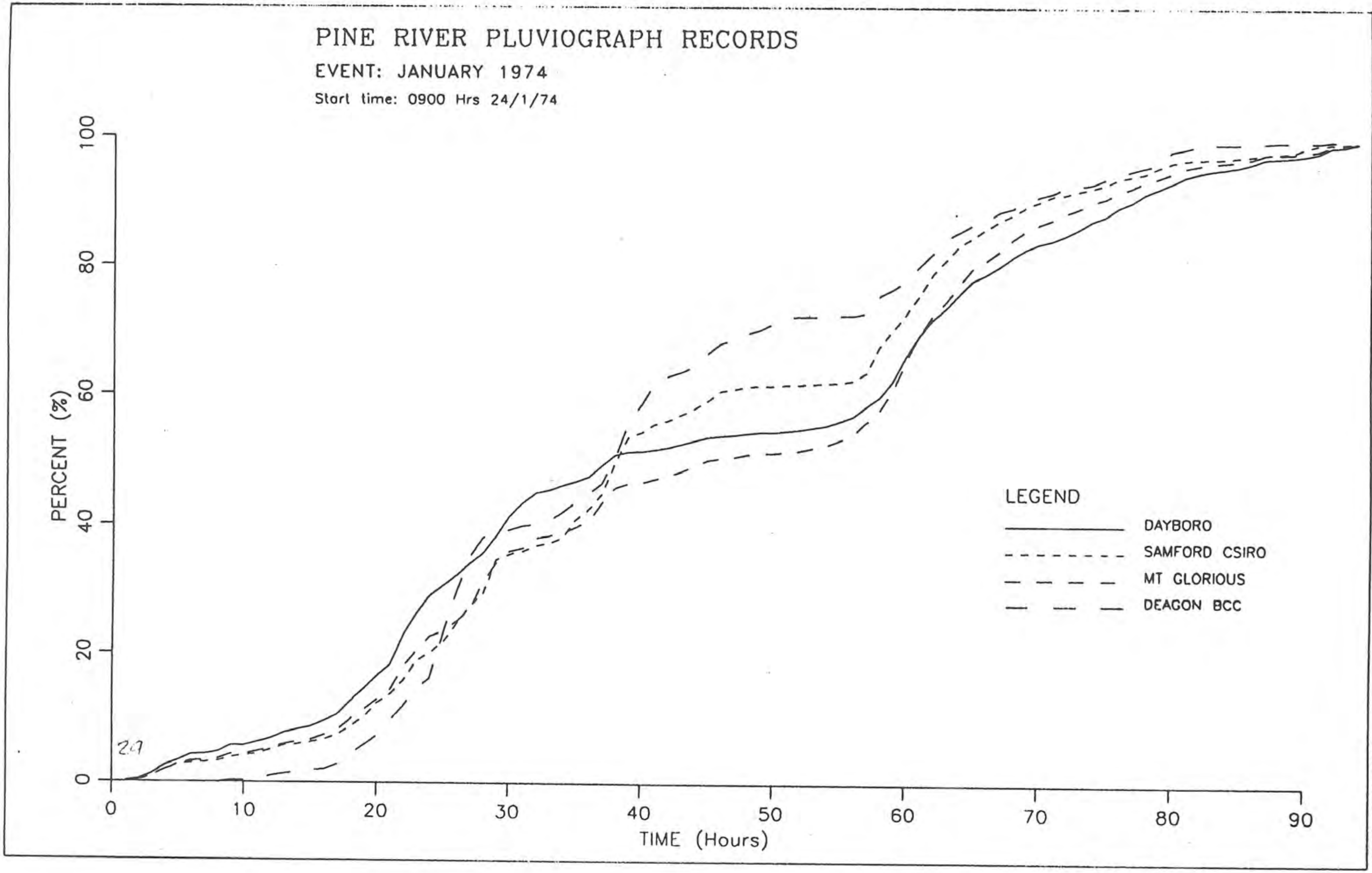

PINE RIVER PLUVIOGRAPH RECORDS
EVENT: JANUARY 1974
Start time: 0900 Hrs 24/1/74
PERCENT (%)
0
20
40
60
80
100
0
10
20
30
40
50
60
70
80
90
TIME (Hours)
LEGEND
DAYBORO
SAMFORD CSIRO
MT GLORIOUS
DEAGON BCC
2.9

FIGURE 6.15

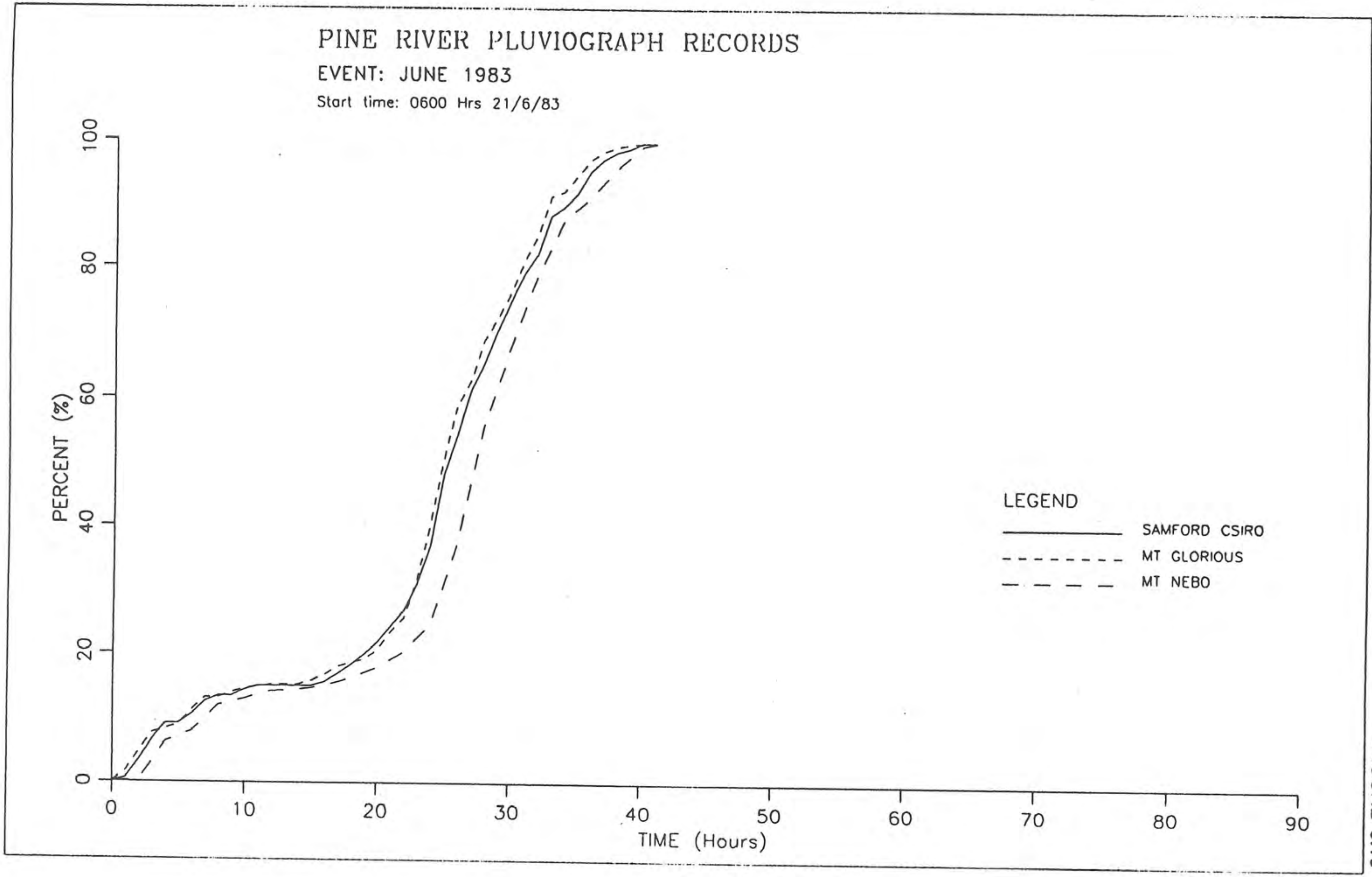

PINE RIVER PLUVIOGRAPH RECORDS
EVENT: JUNE 1983
Start time: 0600 Hrs 21/6/83
PERCENT (%)
0
20
40
60
80
100
TIME (Hours)
0
10
20
30
40
50
60
70
80
90
LEGEND
SAMFORD CSIRO
MT GLORIOUS
MT NEBO

FIGURE 6.16

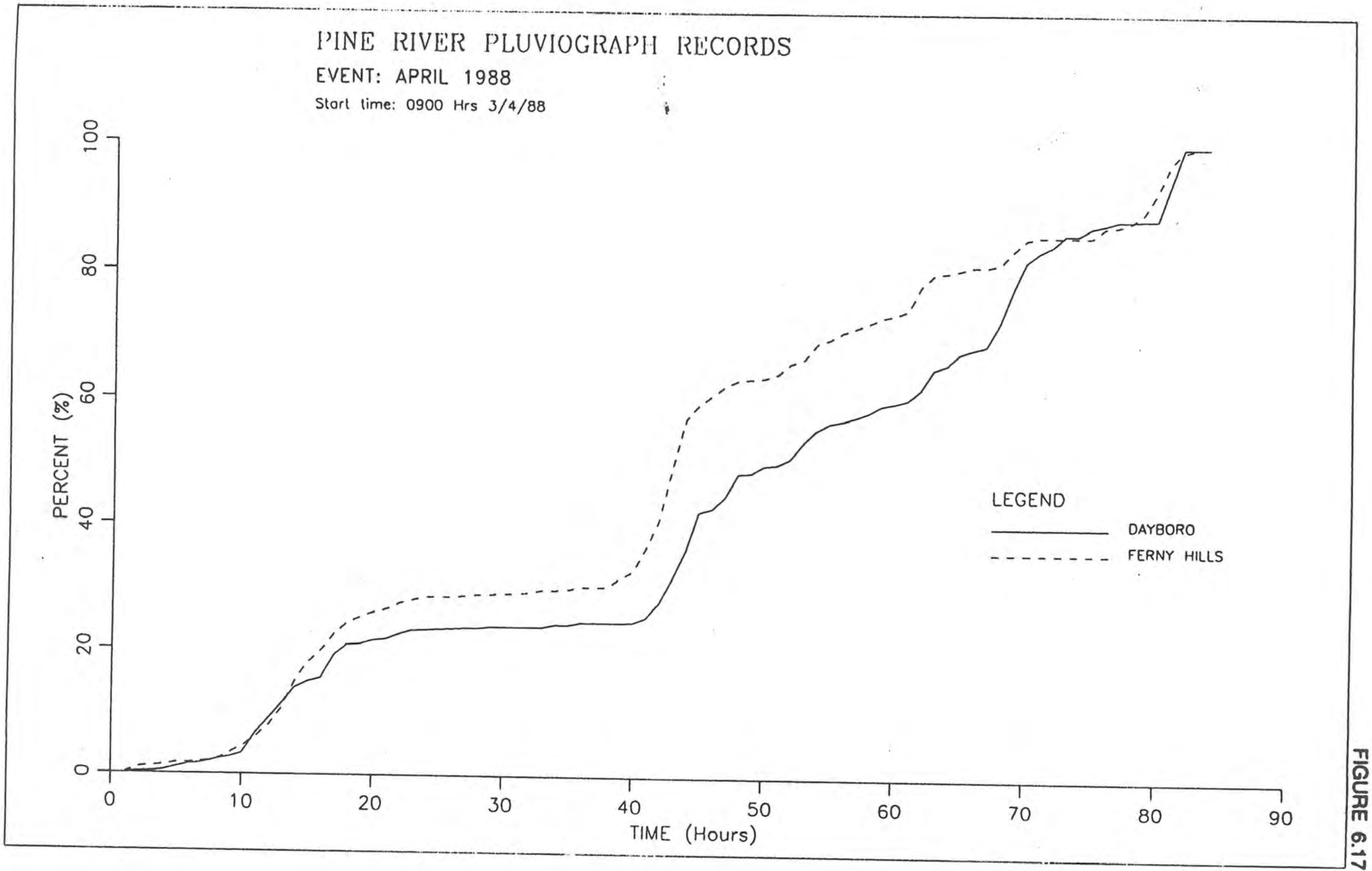
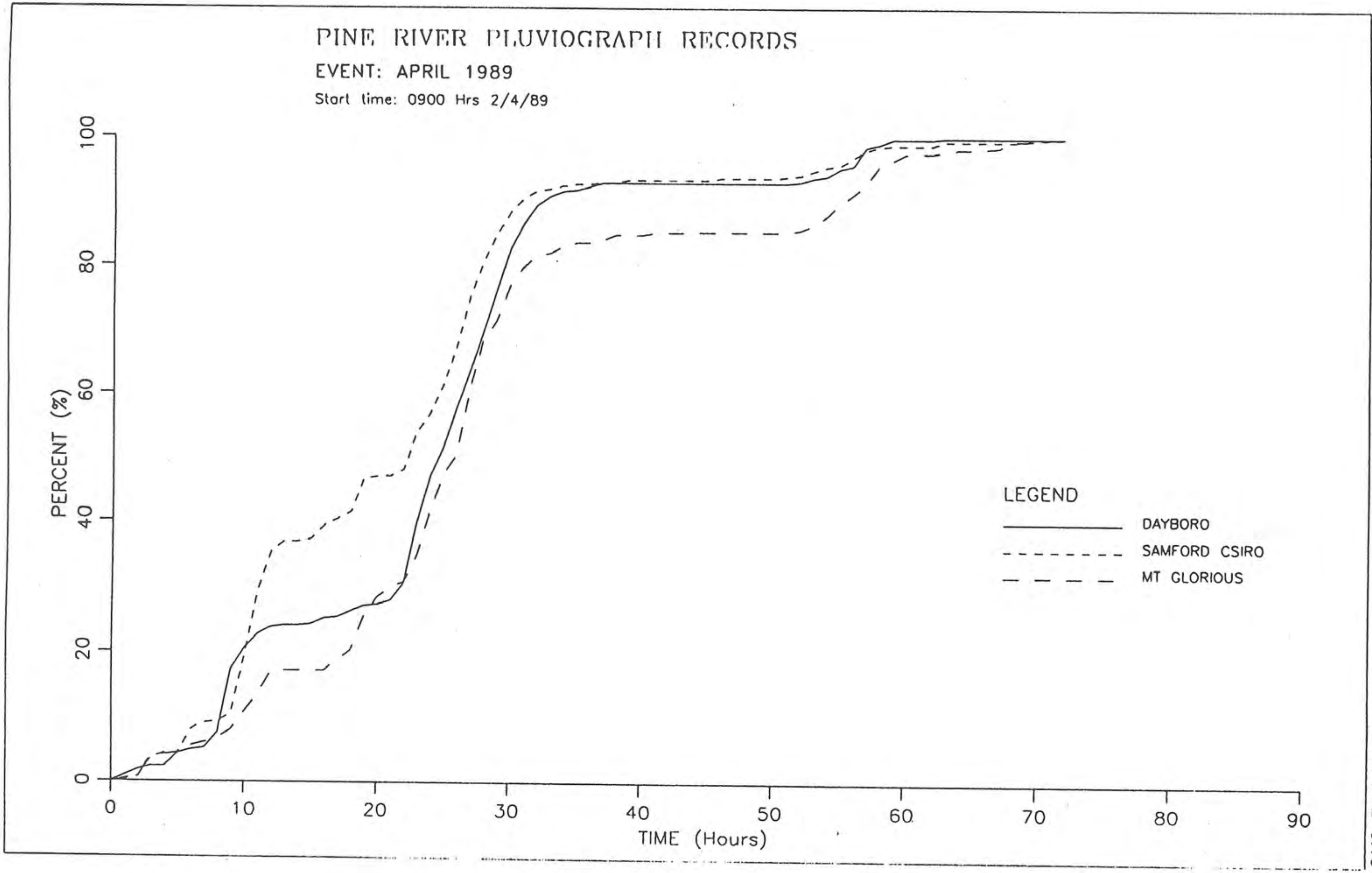

PINE RIVER PLUVIOGRAPH RECORDS
EVENT: APRIL 1988
Start time: 0900 Hrs 3/4/88
PERCENT (%)
0
20
40
60
80
100
TIME (Hours)
0
10
20
30
40
50
60
70
80
90
LEGEND
DAYBORO
FERNY HILLS

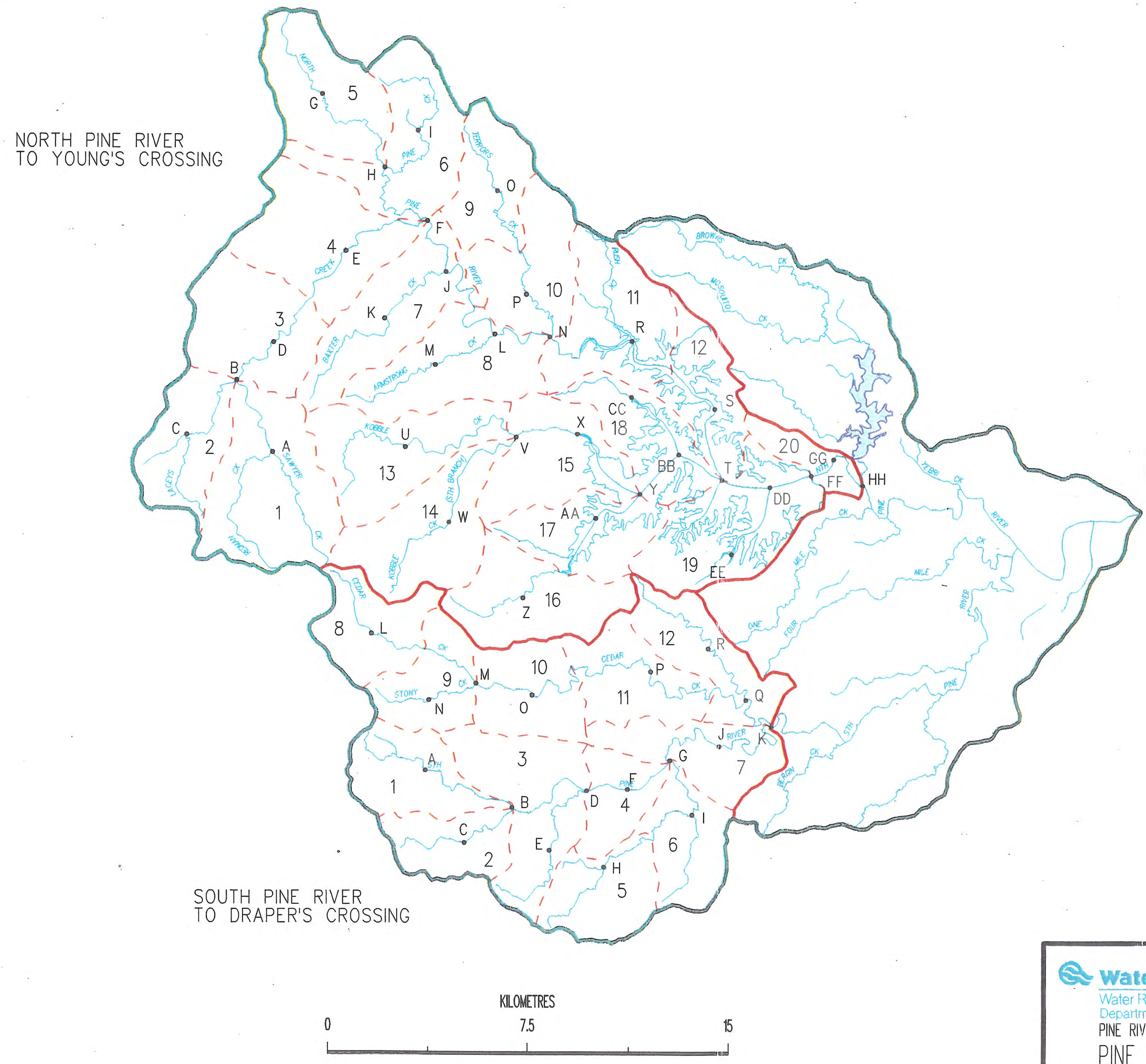
FIGURE 6.17



PINE RIVER PLUVIOGRAPH RECORDS
EVENT: APRIL 1989
Start time: 0900 Hrs 2/4/89
PERCENT (%)
0
20
40
60
80
100
0
10
20
30
40
50
60
70
80
90
TIME (Hours)
LEGEND
DAYBORO
SAMFORD CSIRO
MT GLORIOUS

FIGURE 6.18

FIGURE 6.19



LEGEND

BASIN BOUNDARY

SUB-BASIN BOUNDARY

SUB-AREA BOUNDARY

NODE POINT

Water Resources
Water Resources Commission
Department of Primary Industries
PINE RIVER FLOOD STUDY

PINE RIVER
CALIBRATION MODEL

A3-98892

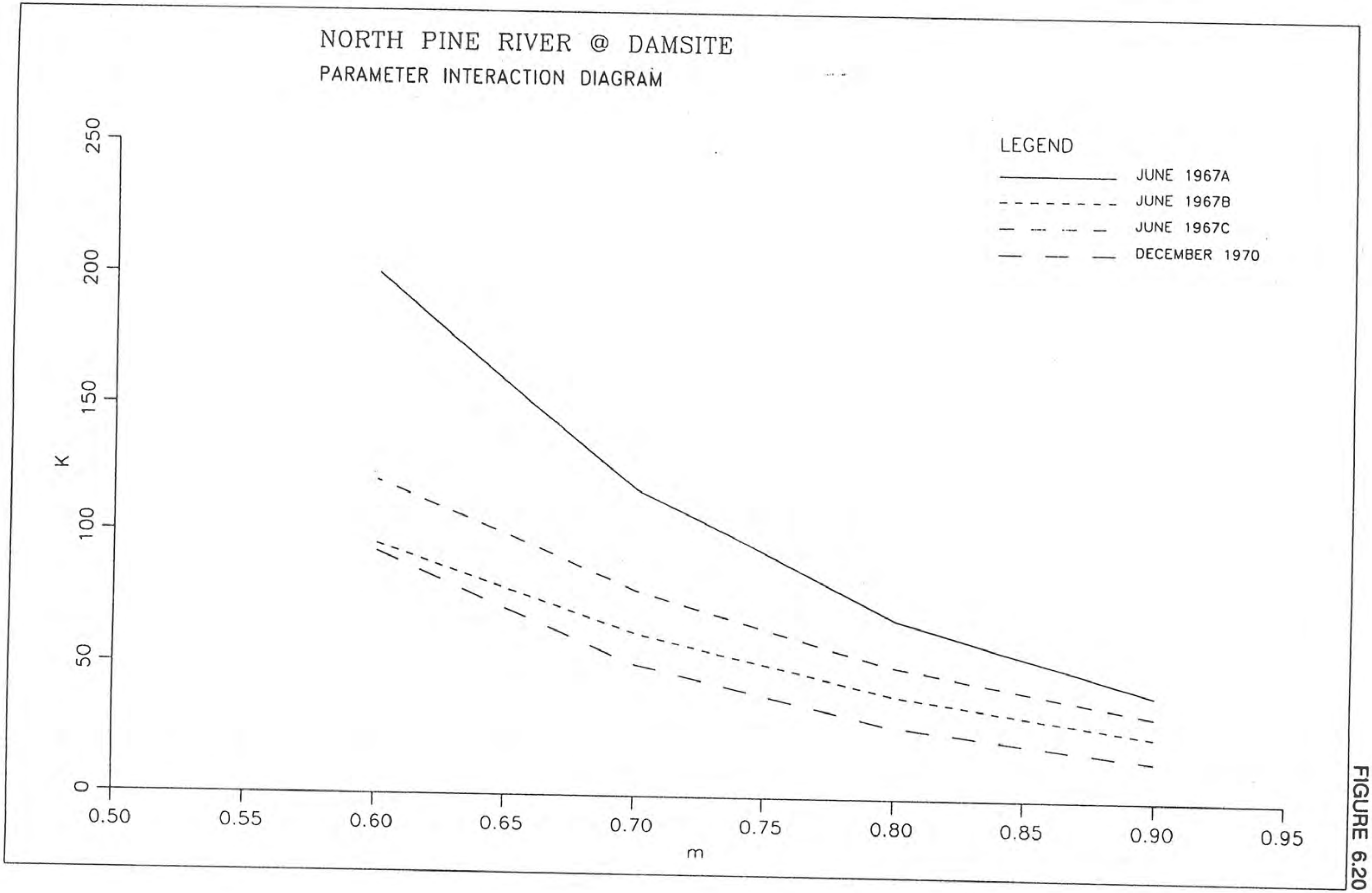
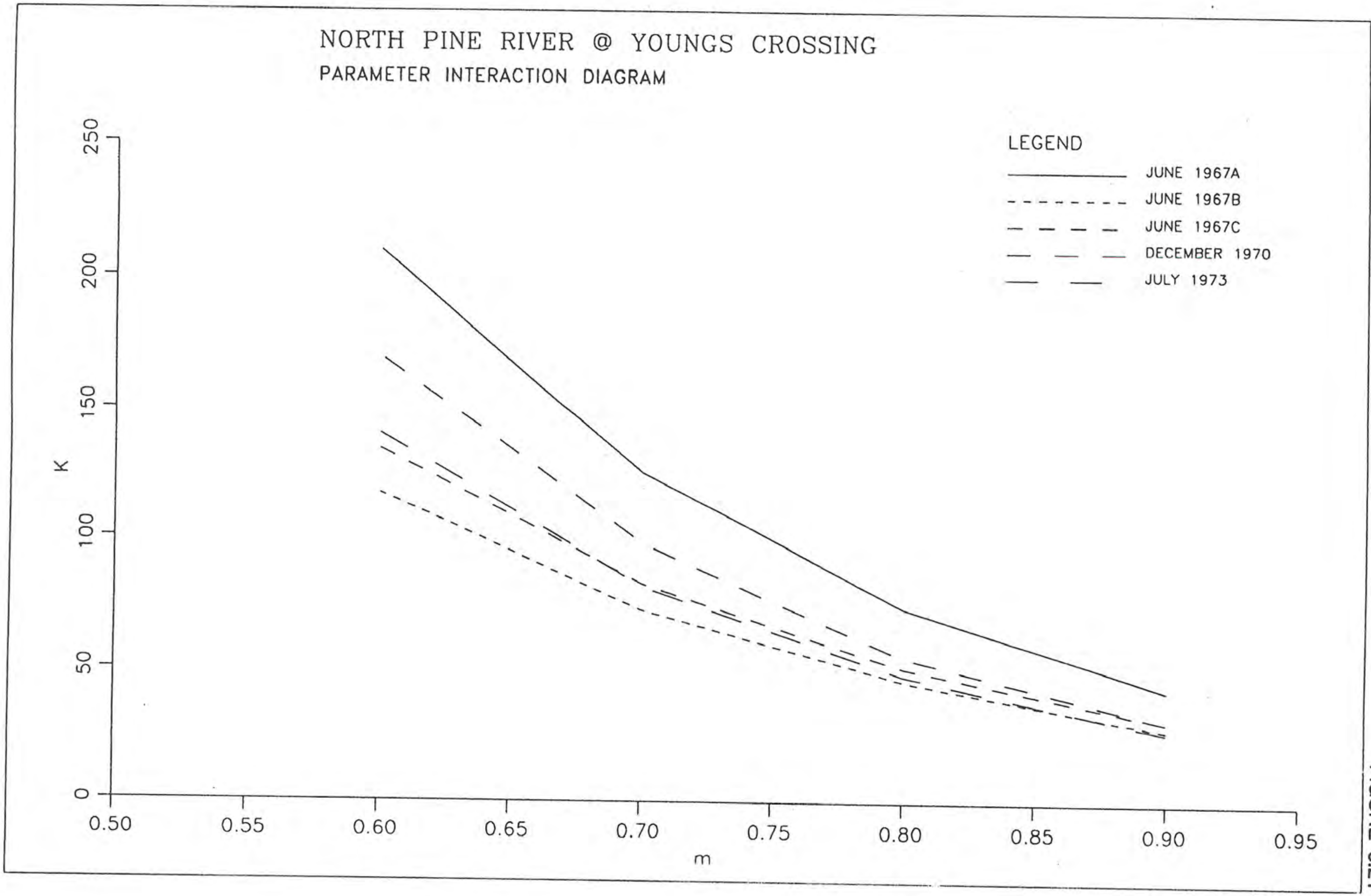

NORTH PINE RIVER @ DAMSITE
PARAMETER INTERACTION DIAGRAM
LEGEND
JUNE 1967A
JUNE 1967B
JUNE 1967C
DECEMBER 1970
K
250
200
150
100
50
0
0.50
0.55
0.60
0.65
0.70
0.75
0.80
0.85
0.90
0.95
m

FIGURE 6.20



NORTH PINE RIVER @ YOUNGS CROSSING
PARAMETER INTERACTION DIAGRAM
LEGEND
JUNE 1967A
JUNE 1967B
JUNE 1967C
DECEMBER 1970
JULY 1973
K
0
50
100
150
200
250
0.50
0.55
0.60
0.65
0.70
0.75
0.80
0.85
0.90
0.95
m

FIGURE 6.21

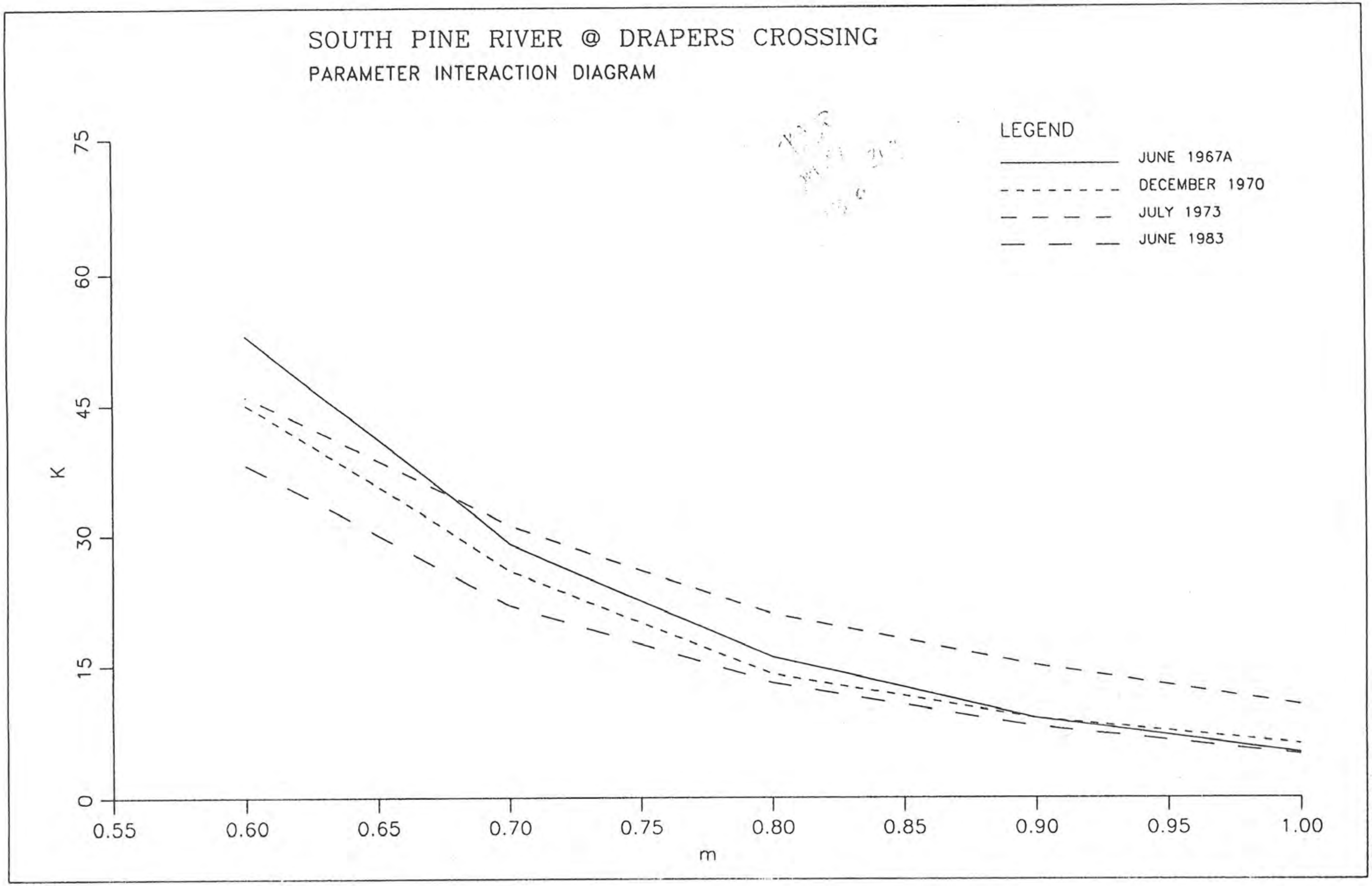

SOUTH PINE RIVER @ DRAPERS CROSSING
PARAMETER INTERACTION DIAGRAM
LEGEND
JUNE 1967A
DECEMBER 1970
JULY 1973
JUNE 1983
K
75
60
45
30
15
0
0.55
0.60
0.65
0.70
0.75
0.80
0.85
0.90
0.95
1.00
m

FIGURE 6.22

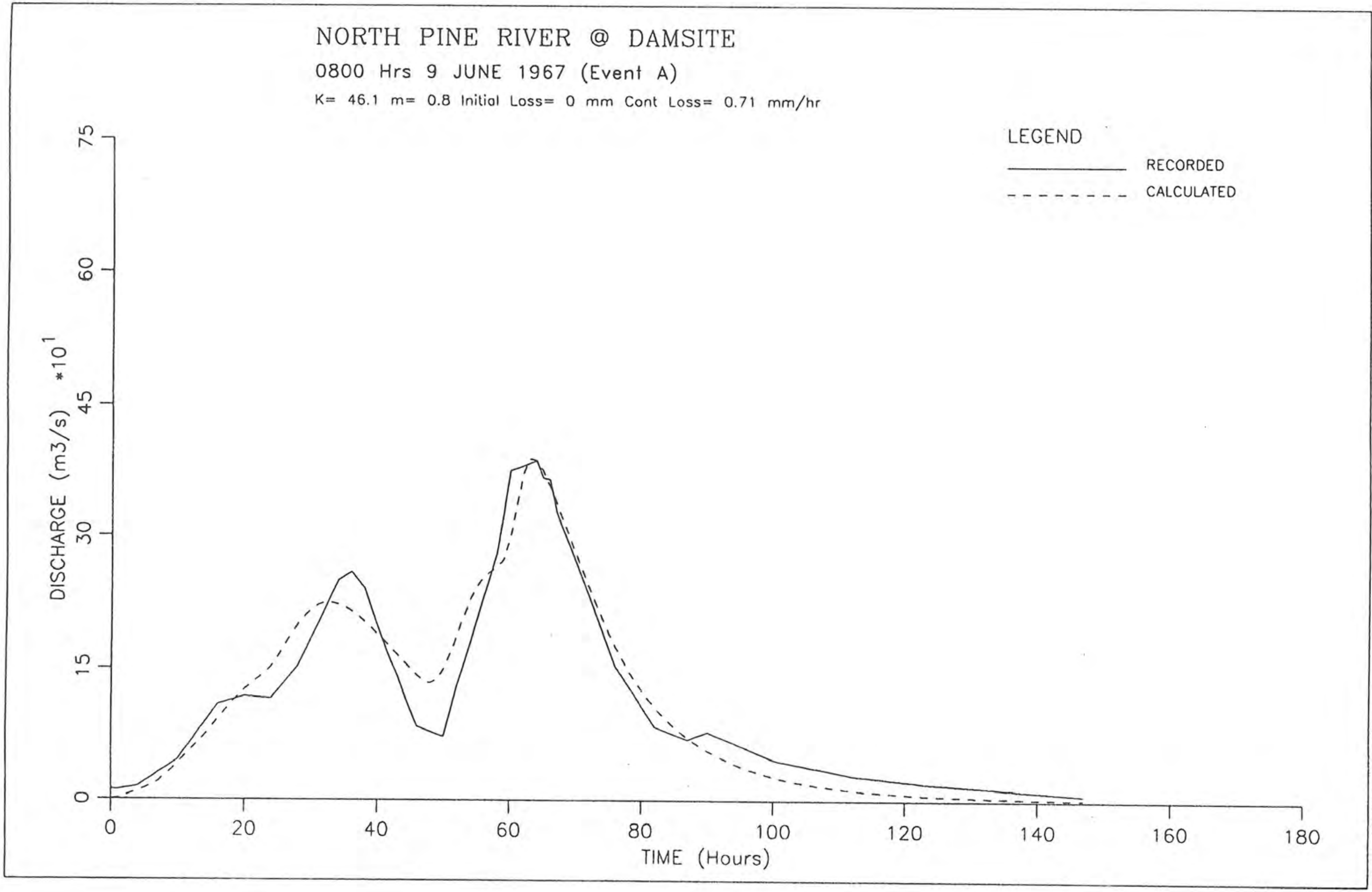

NORTH PINE RIVER @ DAMSITE
0800 Hrs 9 JUNE 1967 (Event A)
K= 46.1 m= 0.8 Initial Loss= 0 mm Cont Loss= 0.71 mm/hr
LEGEND
RECORDED
CALCULATED
DISCHARGE (m3/s) *10^1
75
60
45
30
15
0
0
20
40
60
80
100
120
140
160
180
TIME (Hours)

FIGURE 6.23

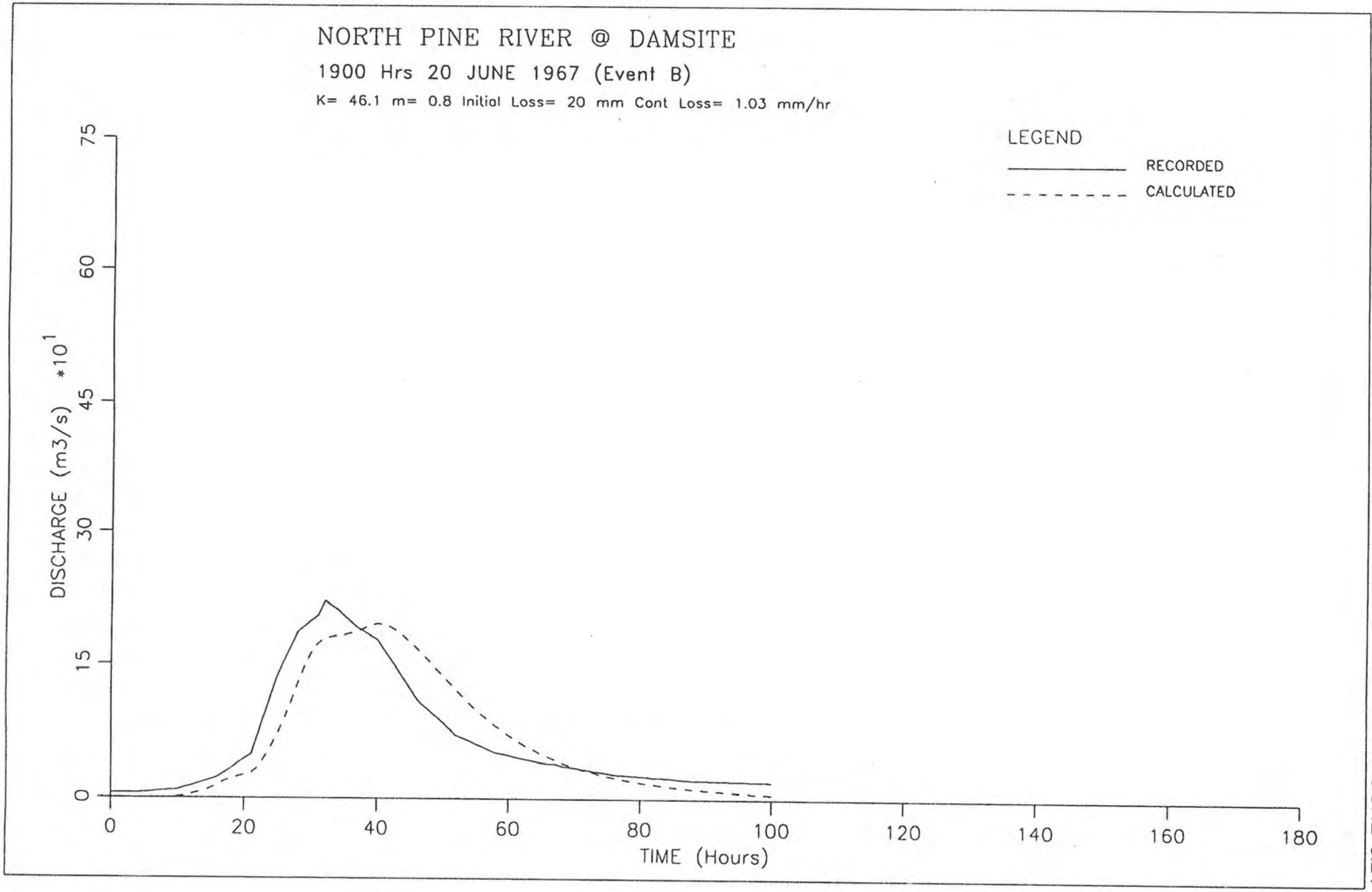
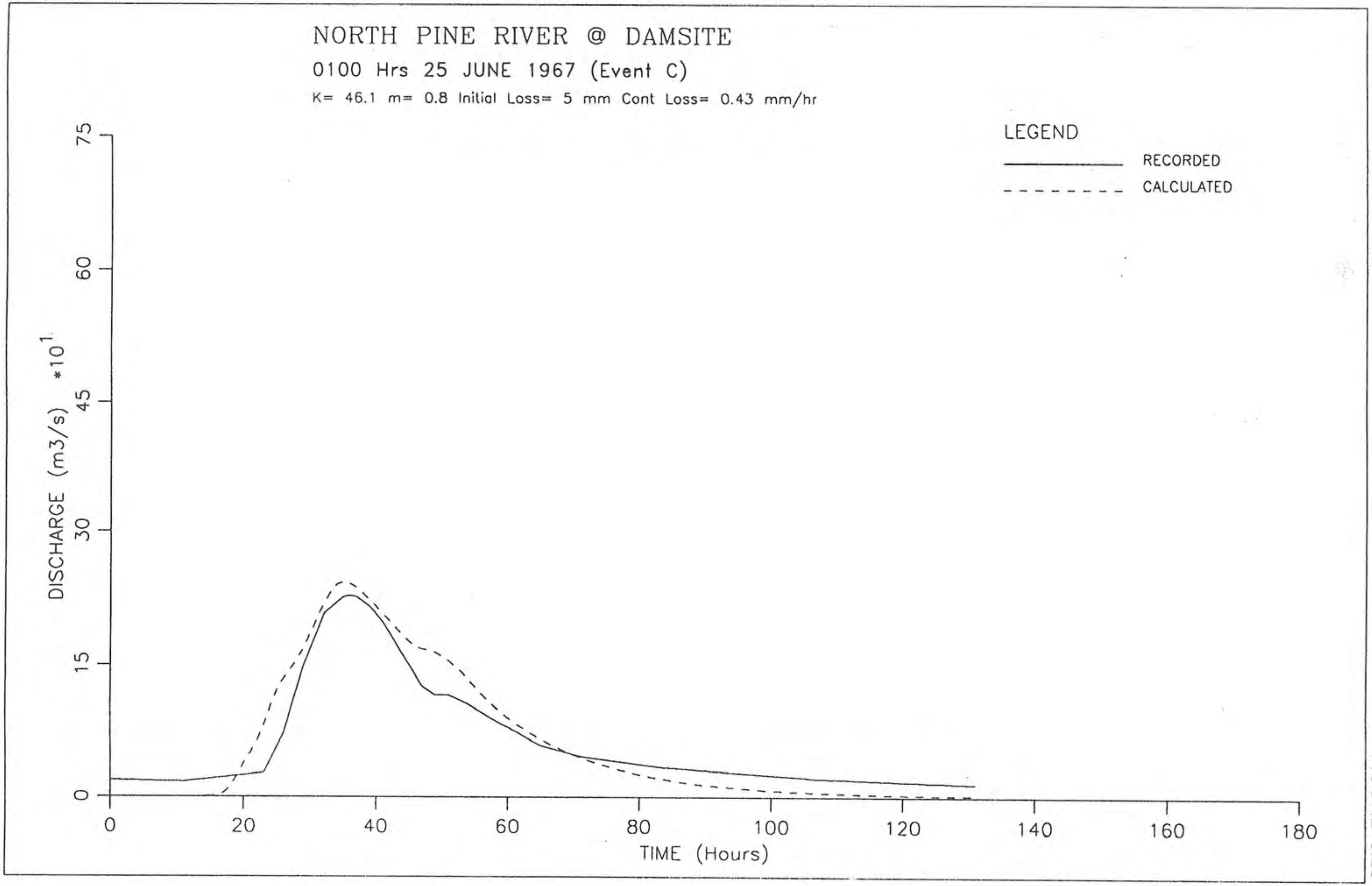

NORTH PINE RIVER @ DAMSITE
1900 Hrs 20 JUNE 1967 (Event B)
K= 46.1 m= 0.8 Initial Loss= 20 mm Cont Loss= 1.03 mm/hr
LEGEND
RECORDED
CALCULATED
DISCHARGE (m3/s) *10^1
0
15
30
45
60
75
0
20
40
60
80
100
120
140
160
180
TIME (Hours)

FIGURE 6.24



NORTH PINE RIVER @ DAMSITE
0100 Hrs 25 JUNE 1967 (Event C)
K= 46.1 m= 0.8 Initial Loss= 5 mm Cont Loss= 0.43 mm/hr
LEGEND
RECORDED
CALCULATED
DISCHARGE (m3/s) *10^1
75
60
45
30
15
0
0
20
40
60
80
100
120
140
160
180
TIME (Hours)

FIGURE 6.25

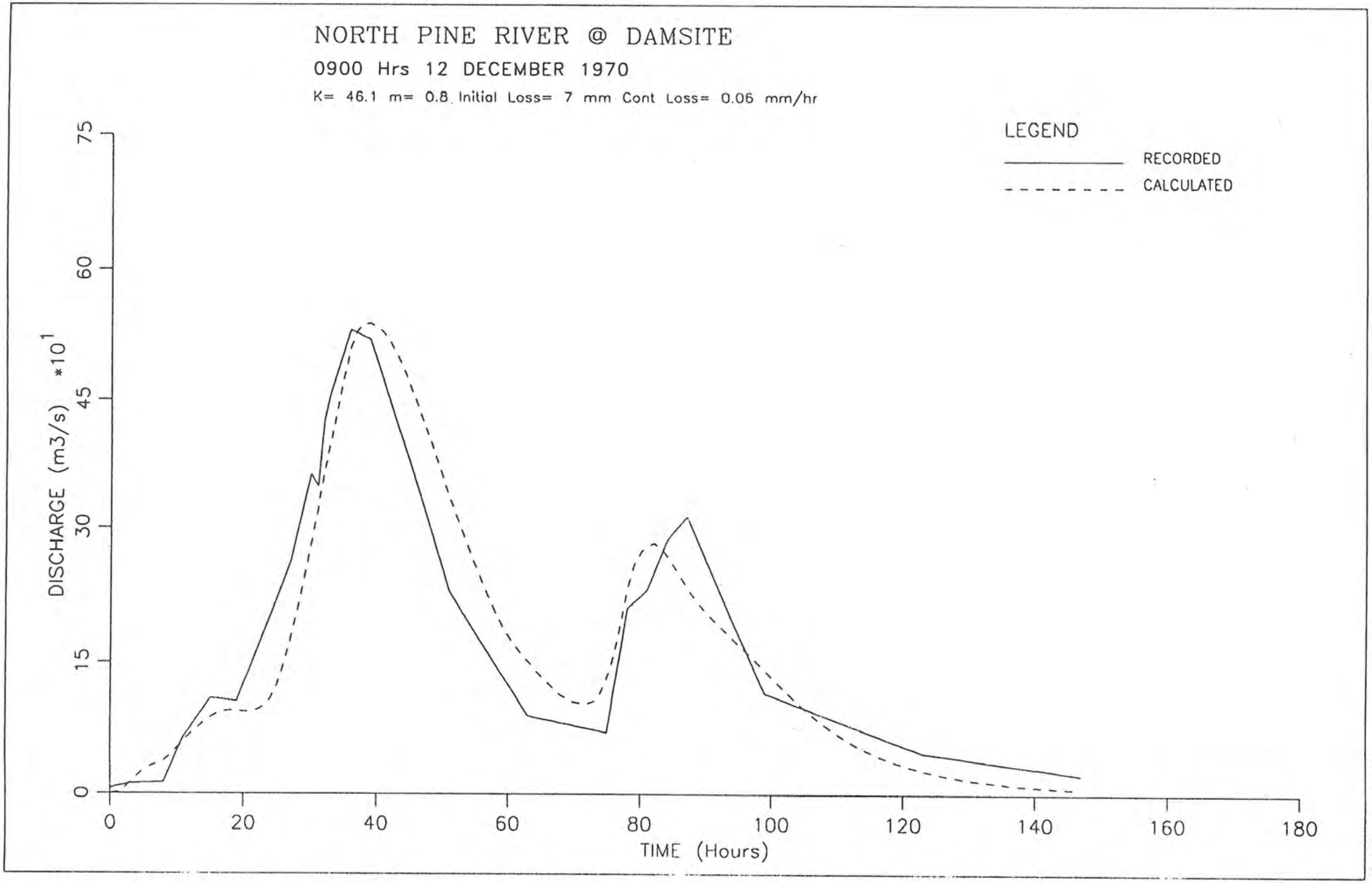
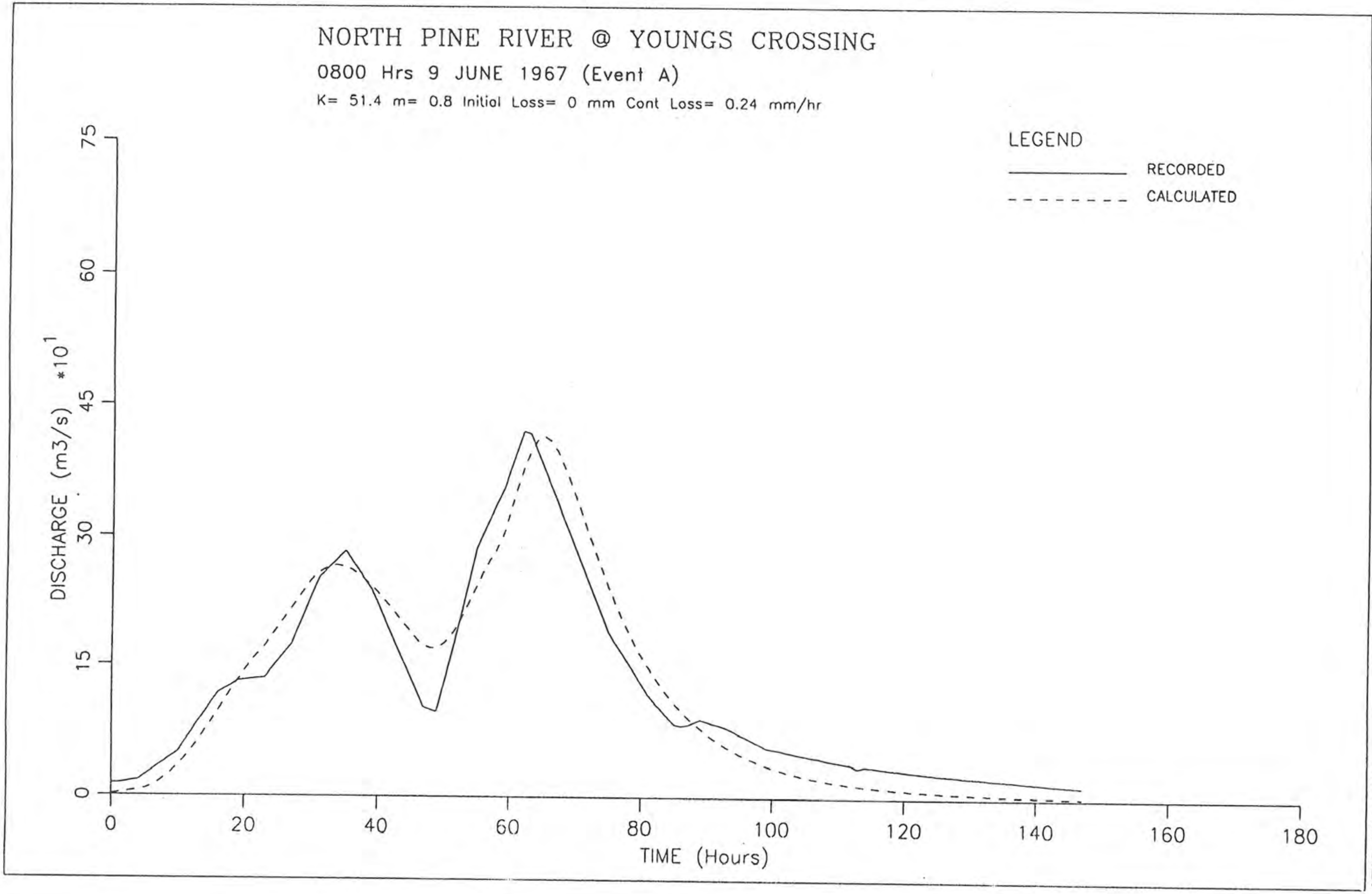

NORTH PINE RIVER @ DAMSITE
0900 Hrs 12 DECEMBER 1970
K= 46.1 m= 0.8 Initial Loss= 7 mm Cont Loss= 0.06 mm/hr
LEGEND
RECORDED
CALCULATED
DISCHARGE (m3/s) *10^1
0
15
30
45
60
75
0
20
40
60
80
100
120
140
160
180
TIME (Hours)

FIGURE 6.26



NORTH PINE RIVER @ YOUNGS CROSSING
0800 Hrs 9 JUNE 1967 (Event A)
K= 51.4 m= 0.8 Initial Loss= 0 mm Cont Loss= 0.24 mm/hr
LEGEND
RECORDED
CALCULATED
DISCHARGE (m3/s) *10^1
0
15
30
45
60
75
0
20
40
60
80
100
120
140
160
180
TIME (Hours)

FIGURE 6.27

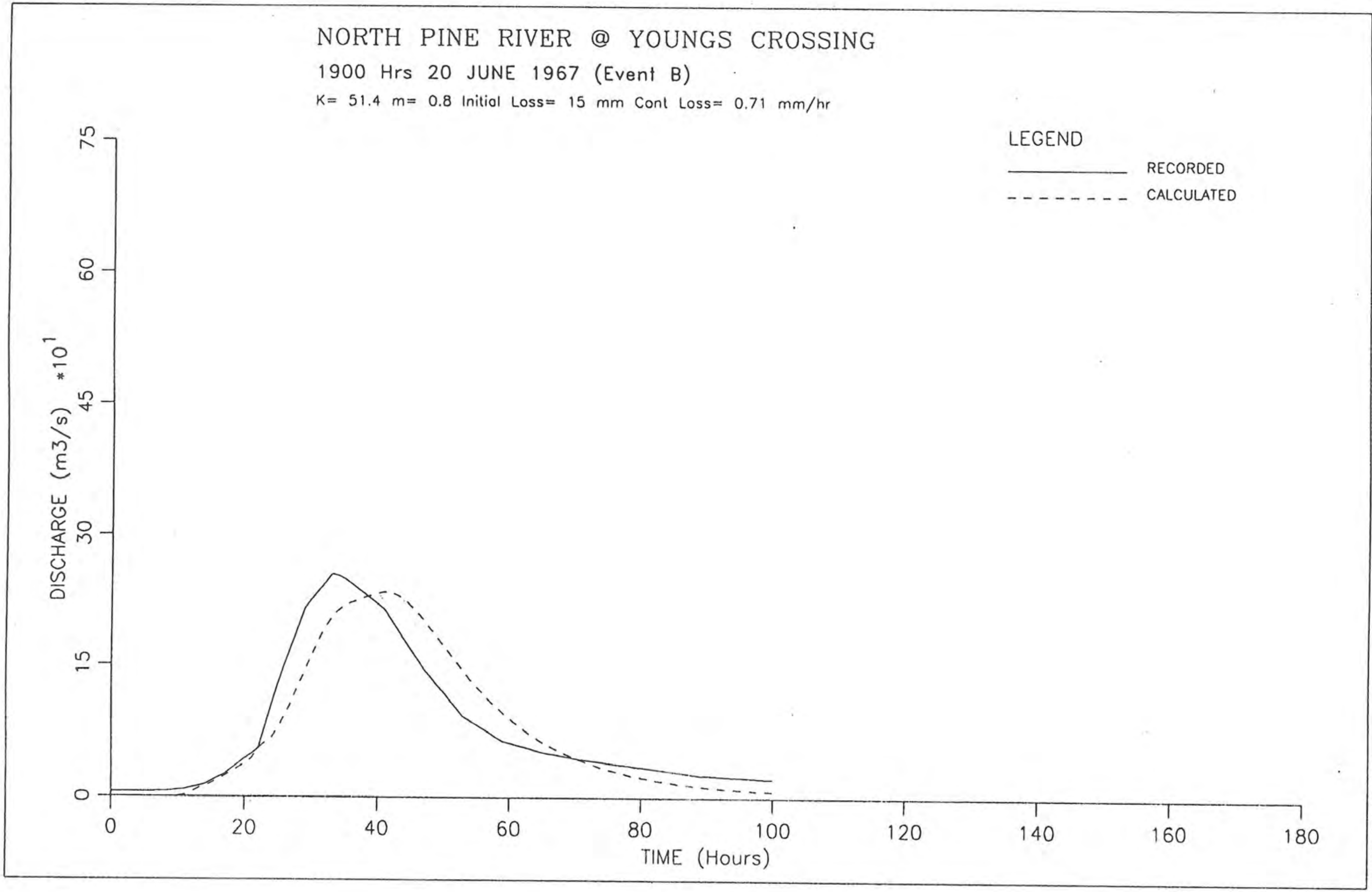

NORTH PINE RIVER @ YOUNGS CROSSING
1900 Hrs 20 JUNE 1967 (Event B)
K= 51.4 m= 0.8 Initial Loss= 15 mm Cont Loss= 0.71 mm/hr
LEGEND
RECORDED
CALCULATED
DISCHARGE (m3/s) *10^1
0
15
30
45
60
75
0
20
40
60
80
100
120
140
160
180
TIME (Hours)

FIGURE 6.28

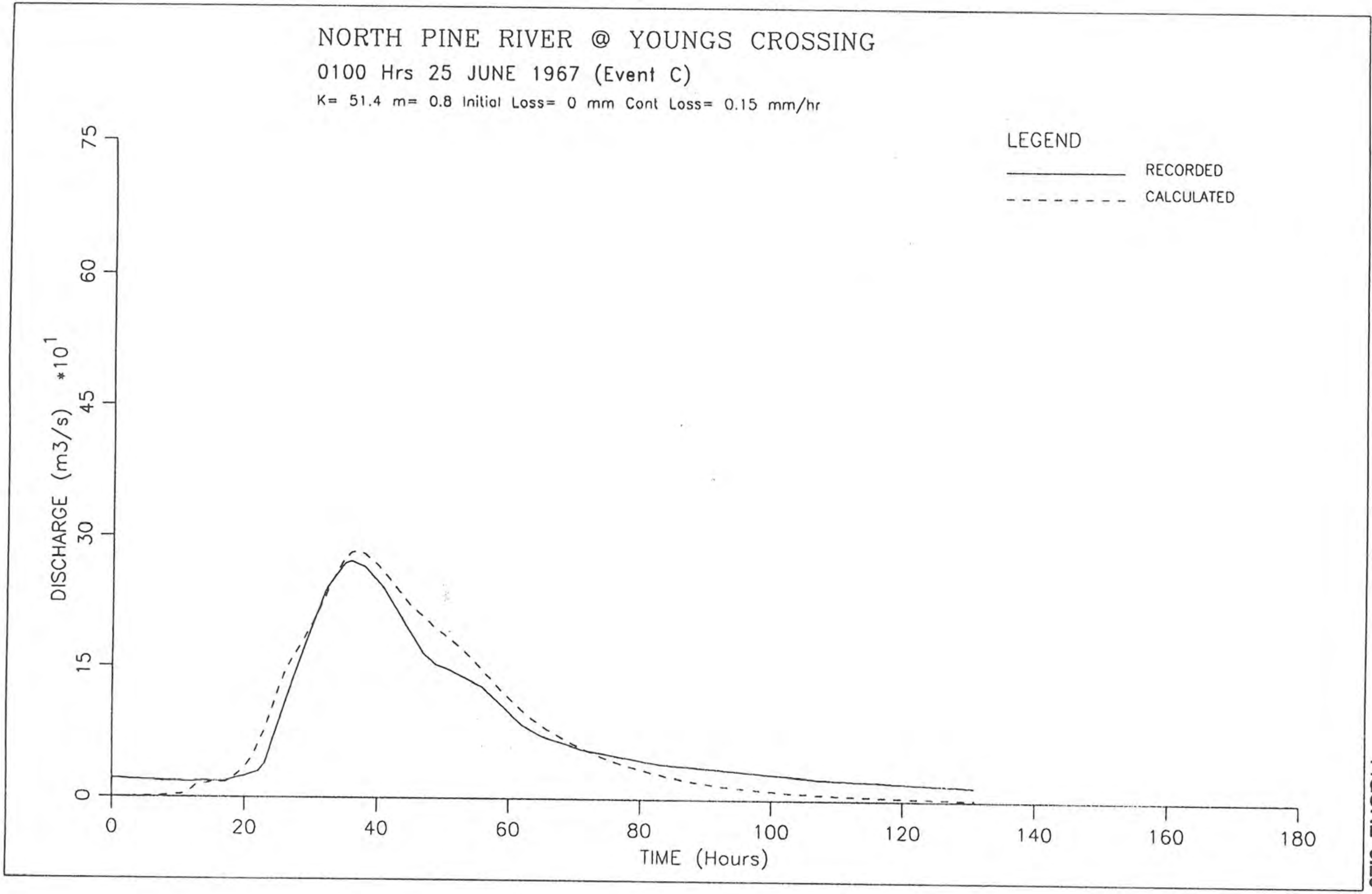

NORTH PINE RIVER @ YOUNGS CROSSING
0100 Hrs 25 JUNE 1967 (Event C)
K= 51.4 m= 0.8 Initial Loss= 0 mm Cont Loss= 0.15 mm/hr
LEGEND
RECORDED
CALCULATED
DISCHARGE (m3/s) *10^1
0
15
30
45
60
75
0
20
40
60
80
100
120
140
160
180
TIME (Hours)

FIGURE 6.29

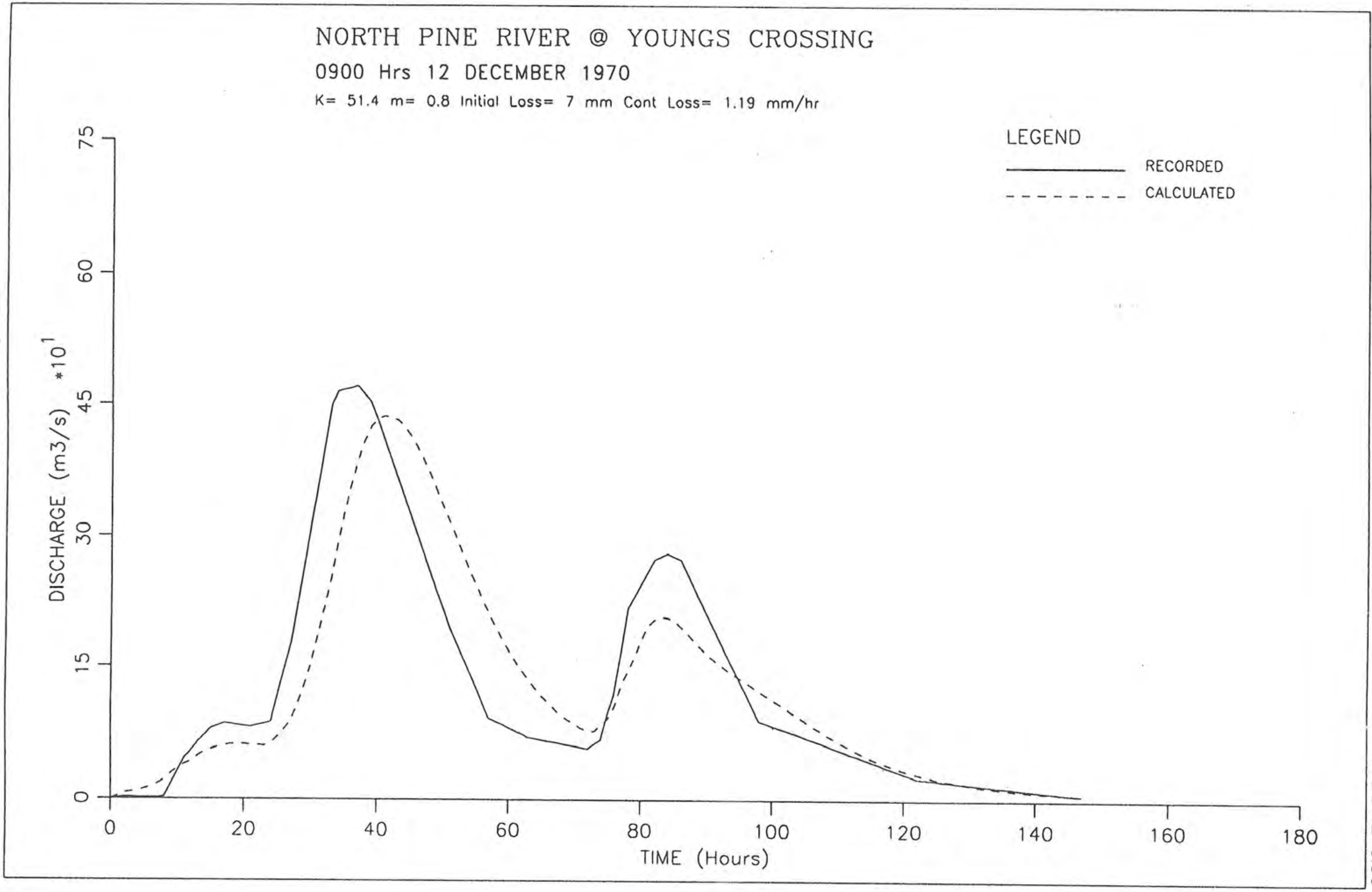

NORTH PINE RIVER @ YOUNGS CROSSING
0900 Hrs 12 DECEMBER 1970
K= 51.4 m= 0.8 Initial Loss= 7 mm Cont Loss= 1.19 mm/hr
LEGEND
RECORDED
CALCULATED
DISCHARGE (m3/s) *10^1
75
60
45
30
15
0
0
20
40
60
80
100
120
140
160
180
TIME (Hours)

FIGURE 6.30

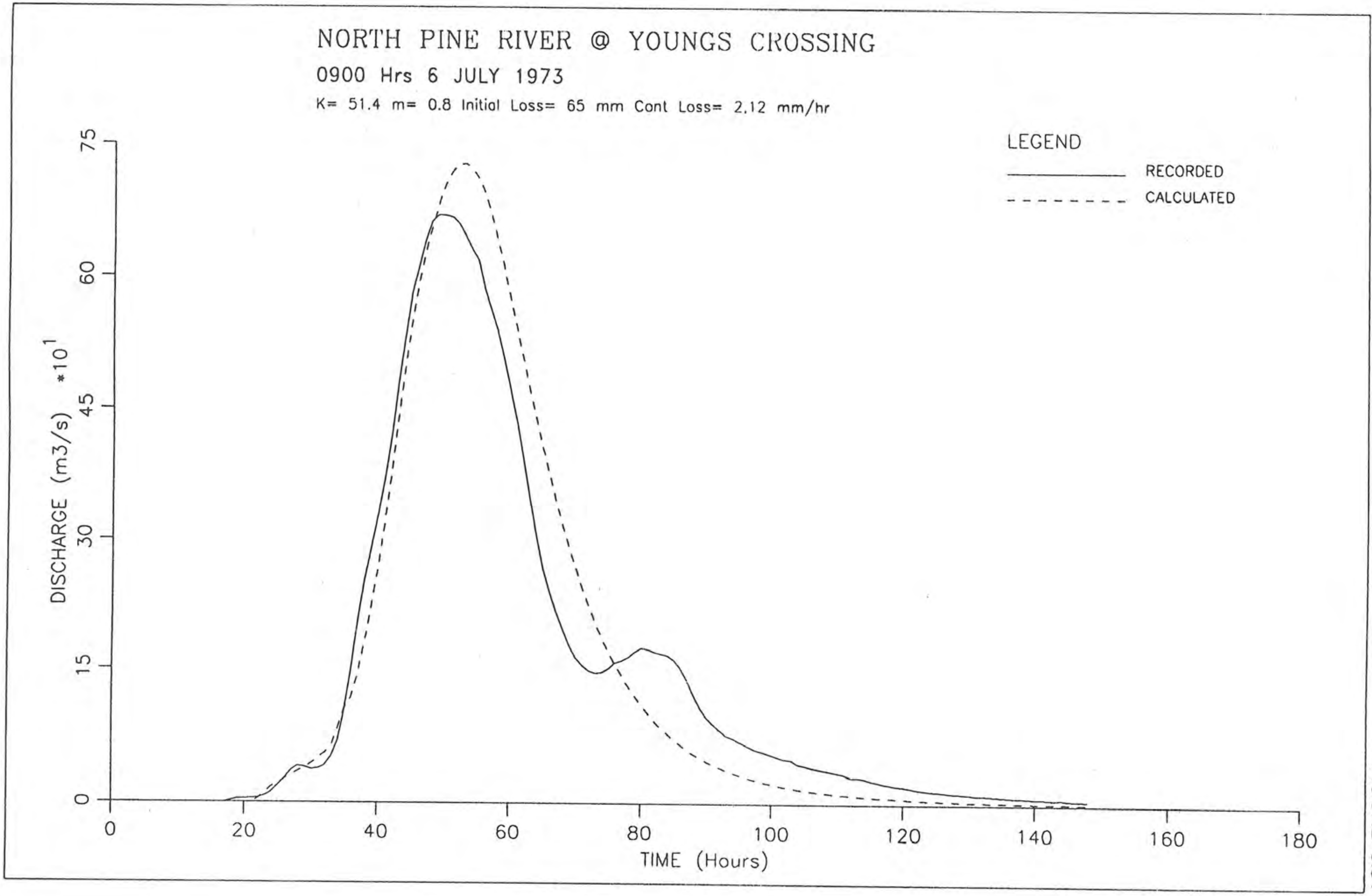

NORTH PINE RIVER @ YOUNGS CROSSING
0900 Hrs 6 JULY 1973
K= 51.4 m= 0.8 Initial Loss= 65 mm Cont Loss= 2.12 mm/hr
LEGEND
RECORDED
CALCULATED
DISCHARGE (m3/s) *10¹
75
60
45
30
15
0
0
20
40
60
80
100
120
140
160
180
TIME (Hours)

FIGURE 6.31

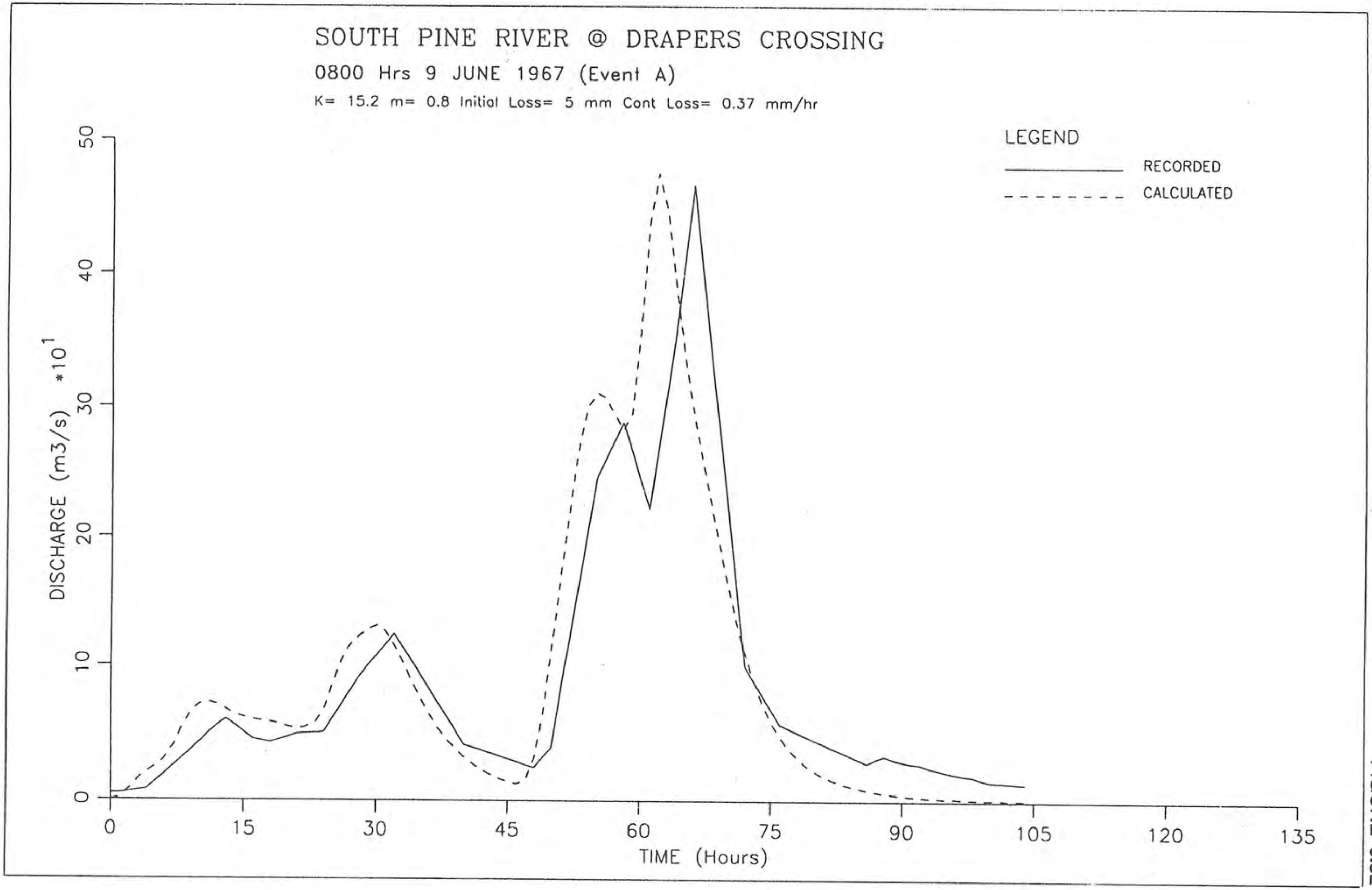

SOUTH PINE RIVER @ DRAPERS CROSSING
0800 Hrs 9 JUNE 1967 (Event A)
K= 15.2 m= 0.8 Initial Loss= 5 mm Cont Loss= 0.37 mm/hr
LEGEND
RECORDED
CALCULATED
DISCHARGE (m3/s) *10
0
10
20
30
40
50
0
15
30
45
60
75
90
105
120
135
TIME (Hours)

FIGURE 6.32

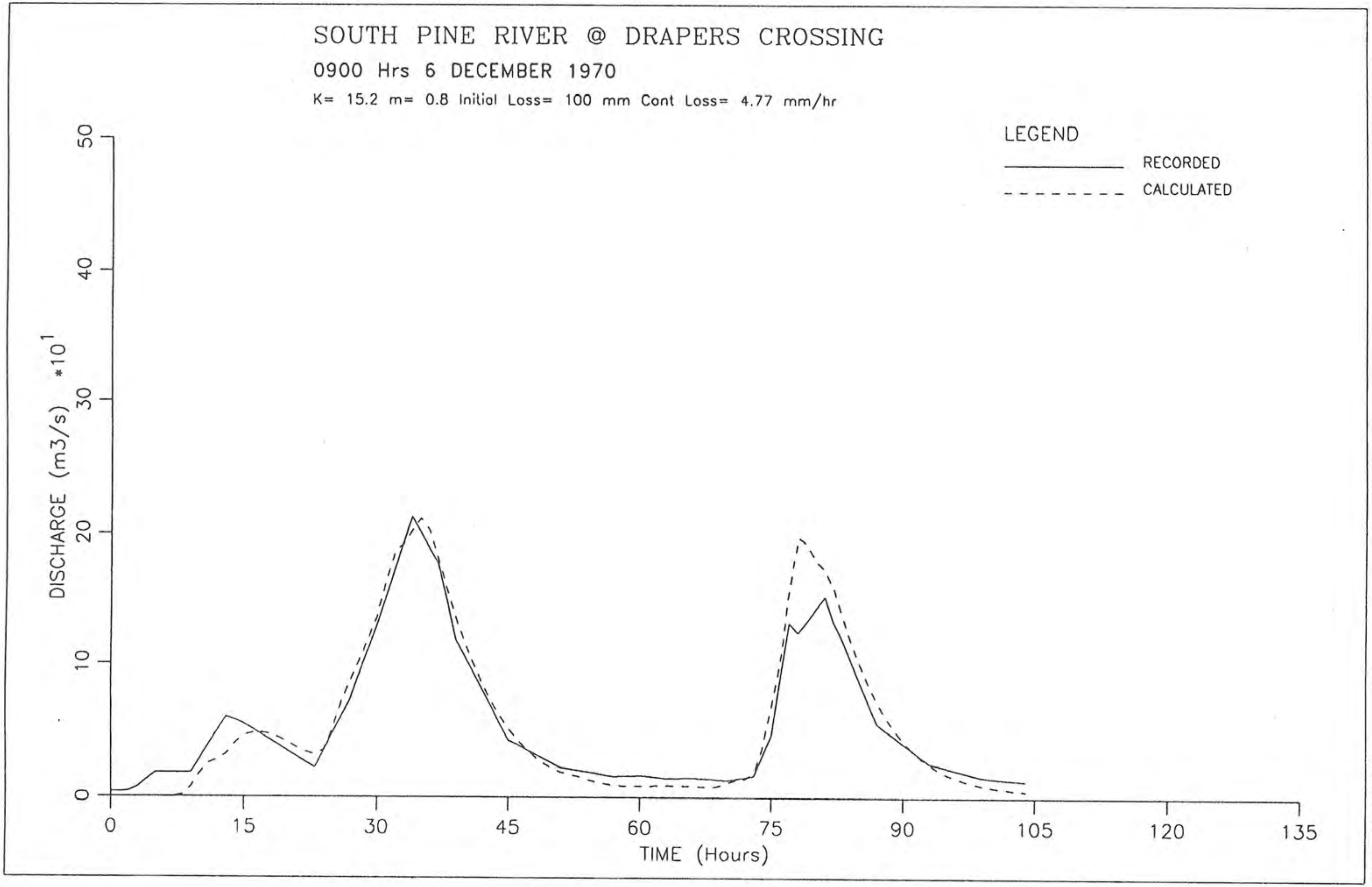
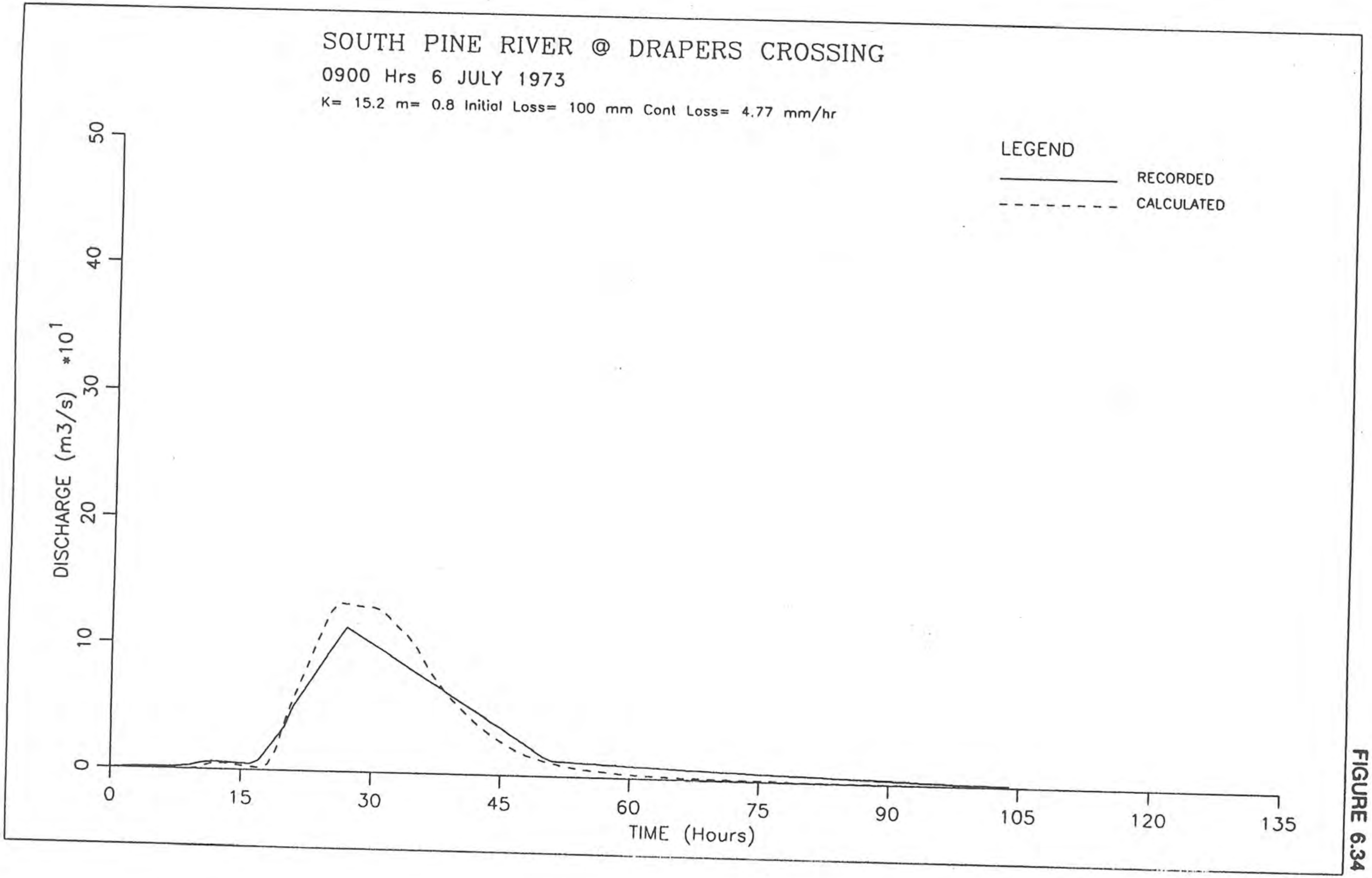

SOUTH PINE RIVER @ DRAPERS CROSSING
0900 Hrs 6 DECEMBER 1970
K= 15.2 m= 0.8 Initial Loss= 100 mm Cont Loss= 4.77 mm/hr
LEGEND
RECORDED
CALCULATED
DISCHARGE (m3/s) *10^1
0
10
20
30
40
50
0
15
30
45
60
75
90
105
120
135
TIME (Hours)

FIGURE 6.33



SOUTH PINE RIVER @ DRAPERS CROSSING
0900 Hrs 6 JULY 1973
K= 15.2 m= 0.8 Initial Loss= 100 mm Cont Loss= 4.77 mm/hr
LEGEND
RECORDED
CALCULATED
DISCHARGE (m3/s) *10[1]
0
10
20
30
40
50
0
15
30
45
60
75
90
105
120
135
TIME (Hours)

FIGURE 6.34

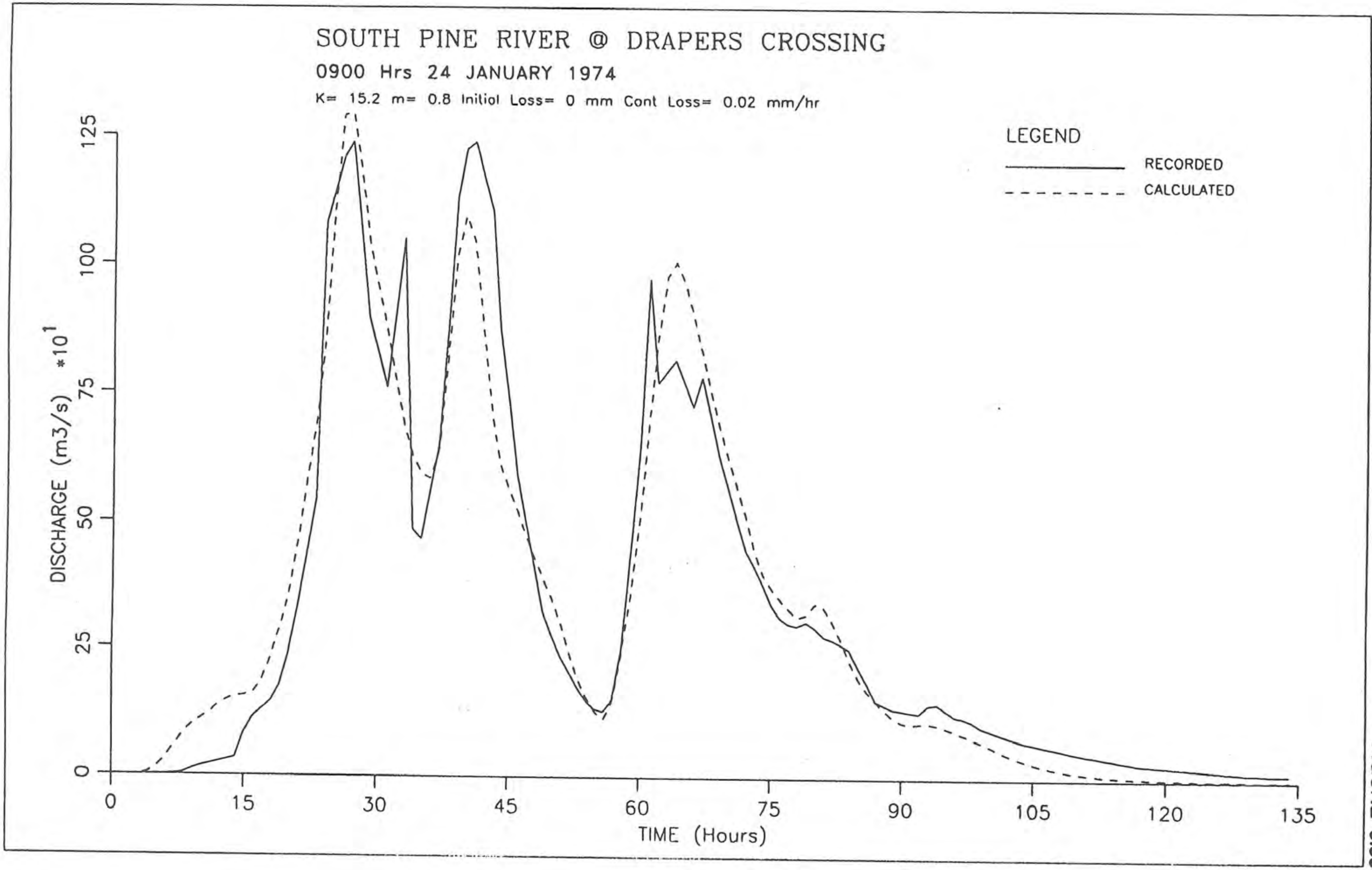
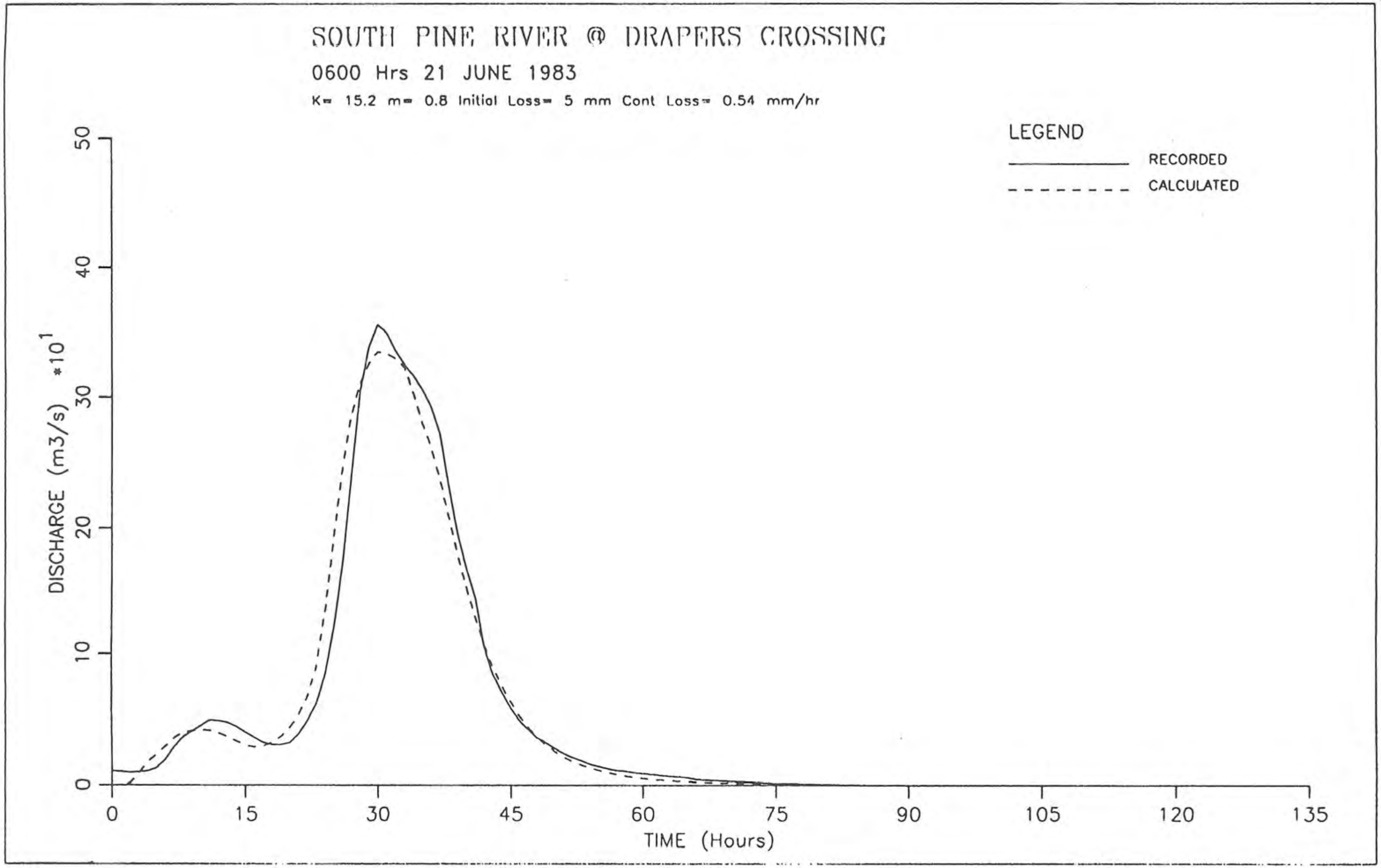

SOUTH PINE RIVER @ DRAPERS CROSSING
0900 Hrs 24 JANUARY 1974
K= 15.2 m= 0.8 Initiol Loss= 0 mm Cont Loss= 0.02 mm/hr
LEGEND
RECORDED
CALCULATED
DISCHARGE (m3/s) *10^1
0
25
50
75
100
125
0
15
30
45
60
75
90
105
120
135
TIME (Hours)

FIGURE 6.35



SOUTH PINE RIVER @ DRAPERS CROSSING
0600 Hrs 21 JUNE 1983
K= 15.2 m= 0.8 Initial Loss= 5 mm Cont Loss= 0.54 mm/hr
LEGEND
RECORDED
CALCULATED
DISCHARGE (m3/s) *10^1
0
10
20
30
40
50
0
15
30
45
60
75
90
105
120
135
TIME (Hours)

FIGURE 6.36

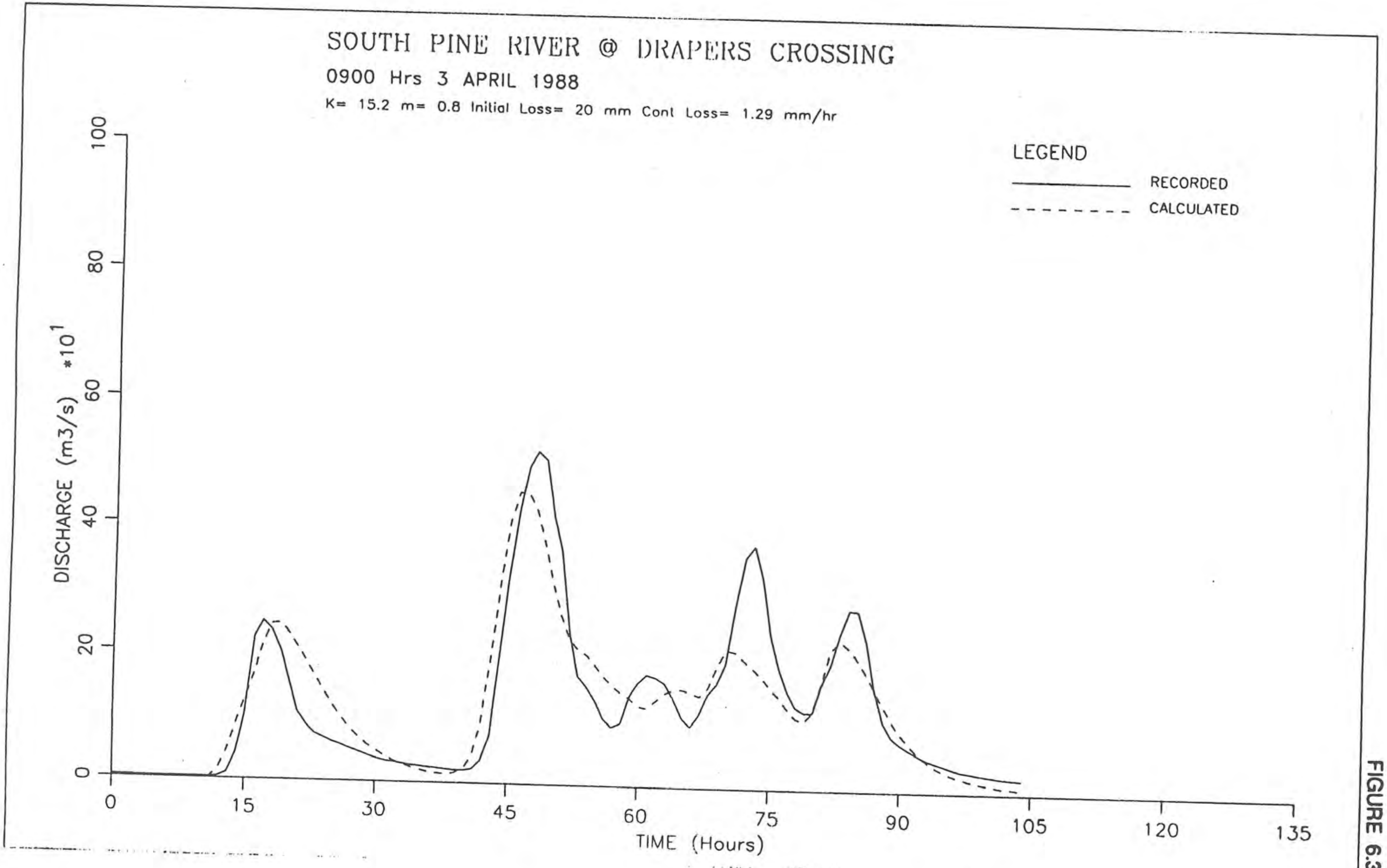

SOUTH PINE RIVER @ DRAPERS CROSSING
0900 Hrs 3 APRIL 1988
K= 15.2 m= 0.8 Initial Loss= 20 mm Cont Loss= 1.29 mm/hr
LEGEND
RECORDED
CALCULATED
DISCHARGE (m3/s) *10^1
0
20
40
60
80
100
0
15
30
45
60
75
90
105
120
135
TIME (Hours)

FIGURE 6.37

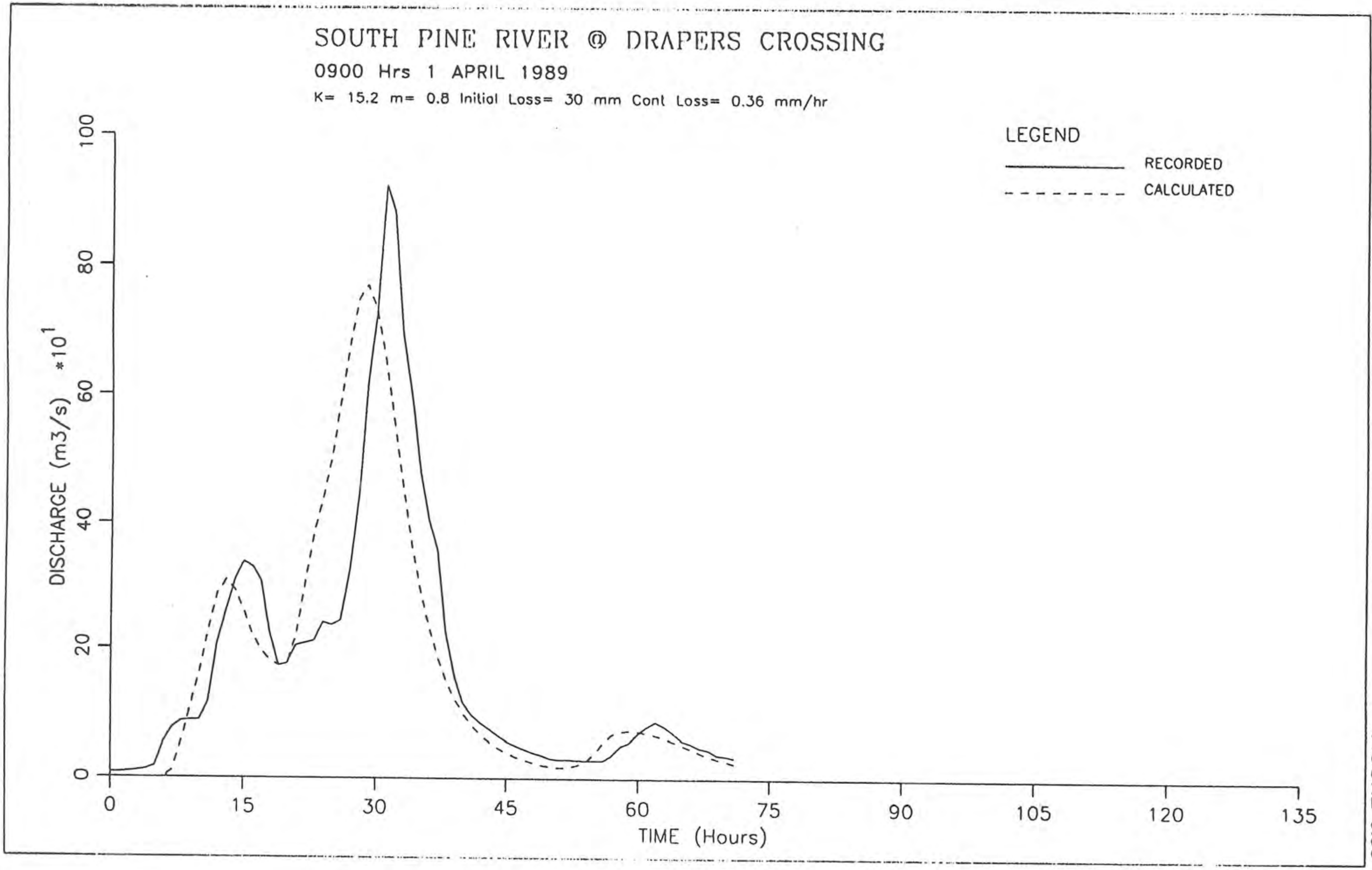

SOUTH PINE RIVER @ DRAPERS CROSSING
0900 Hrs 1 APRIL 1989
K= 15.2 m= 0.8 Initial Loss= 30 mm Cont Loss= 0.36 mm/hr
LEGEND
RECORDED
CALCULATED
DISCHARGE (m3/s) *10^1
0
20
40
60
80
100
0
15
30
45
60
75
90
105
120
135
TIME (Hours)

FIGURE 6.38

AUGUST 1991

…ile was subject to a "FOI" application refer CHQ/ 2034

BRISBANE RIVER AND PINE RIVER FLOOD STUDY :

Report No. 4b

PINE RIVER FLOOD HYDROLOGY REPORT VOLUME II

Design Flood Estimation

South East Queensland WATER BOARD

NATURAL RESOURCES Queensland

Brisbane River and Pine River Flood Studies

PINE RIVER FLOOD HYDROLOGY REPORT

REPORT ON CALIBRATION

Volume II
August 1991

7.0 FLOOD FREQUENCY ANALYSIS

7.1 INTRODUCTION

Flood frequency analyses have been conducted at the four gauging station locations of interest on the North and South Pine Rivers.

The stations located on North Pine River were considered as pre-North Pine Dam cases and as such, records of flows obtained after the start of construction of the Dam are not incorporated in the analysis.

Maximum instantaneous streamflows have been extracted for each year of record from Water Resources Commission records. Annual flood series analyses have been conducted because the study is principally concerned with flood events with average recurrence intervals of greater than 10 years.

7.2 NORTH PINE RIVER AT YOUNG'S CROSSING

Peak annual discharges at this gauging station have been formed into an annual flood series. Records at this station date back to the climatic year of 1914-1915, and extend through to 1977-1978. However, as indicated above, records after 1970-1971 have not been considered. Table 7.1 provides details of the peak annual discharge for 48 years of complete record. A number of incomplete years of record exist, and where this record could not be estimated from correlation with other stations or rainfall records it was ignored in the analysis.

A Log-Pearson type III distribution was fitted to the annual series data, and 95% confidence limits were also determined. Computer program WS06 (Hadgraft) was used to derive the flood frequency estimates. Results of this analysis are presented in Table 7.2. The Cunnane empirical plotting position was adopted in all cases.

TABLE 7.1

NORTH PINE RIVER AT YOUNG'S CROSSING

PEAK ANNUAL DISCHARGES

CLIMATIC YEAR	MONTH	PEAK DISCHARGE (m^3/s)	RANK
1914-15	-	-	Incomplete
15-16	Feb	16	46
16-17	Nov	322	23
17-18	Jan	322	24
18-19	May	16	47
19-20	Jan	299	27
20-21	Apr	459	13
21-22	Dec	230	29
22-23	Apr	3	48
23-24	Mar	48	42
24-25	Mar	223	30
25-26	Jan	28	44
26-27	Jan	539	5
27-28	Apr	795	2
28-29	Apr	347	21
29-30	May	284	28
30-31	-	-	Incomplete
31-32	Dec	88	38
32-33	Apr	203	32
33-34	Feb	620	3
34-35	Feb	71	39
35-36	Oct	20	45
36-37	Mar	348	20
37-38	May	519	8
38-39	-	-	Incomplete
39-40	-	-	Incomplete
40-41	-	-	Incomplete
41-42	-	-	Incomplete
42-43	-	-	Incomplete
43-44	-	-	Incomplete
44-45	-	-	Incomplete
45-46	Mar	376	19
46-47	Feb	585	4
47-48	May	394	17
48-49	Mar	62	41
49-50	Feb	533	7
50-51	Jan	538	6
51-52	Jun	29	43
52-53	Feb	492	10
53-54	Jul	221	31
54-55	Mar	996	1
55-56	Mar	335	22
56-57	Dec	162	34
57-58	Jun	387	18
58-59	Feb	505	9
59-60	Nov	318	25
60-61	Feb	134	36
61-62	Nov	126	37
62-63	Mar	402	16
63-64	Mar	318	26
64-65	Jul	415	15
65-66	Dec	194	33
66-67	Mar	450	14
67-68	Jan	483	11
68-69	Aug	136	35
69-70	Jan	70	40
70-71	Dec	474	12

TABLE 7.2

NORTH PINE RIVER AT YOUNG'S CROSSING

ANNUAL SERIES FLOOD FREQUENCY

Probability Exceedence (%)	Average Recurrence Interval (Years)	Peak Discharge (m^3/s)	95% Confidence Limits (m^3/s)	
			Lower	Upper
50	2	260	168	402
20	5	533	423	671
10	10	679	479	965
5	20	786	447	1383
2	50	883	387	2015
1	100	932	346	2509

Figure 7.1 presents a plot of the fitted distribution of the annual series, together with the observed values. The results obtained in this analysis are substantially lower than the Pre-Dam results presented in Section 4.3 from the Cameron McNamara Study. It should be noted the analysis performed by Cameron McNamara incorporated several large flood events including the January 1974 event that occurred during the construction of North Pine Dam. Estimates of the peak discharges of these large events were synthesised using various techniques since direct streamflow record was not necessarily available. These large events tend to have a major influence on the frequency analysis since they are among the largest on record. It was decided not to include the synthesised peak discharge estimates in this present study because the accuracy of synthesised record is somewhat suspect.

7.3 **NORTH PINE RIVER AT DAMSITE**

There are only 13 complete years of record at this station, commencing in the climatic year 1954-1955. Table 7.3 provides details of the peak annual discharges for the available record. In general, there is good correlation between peak flows at the Damsite and Young's Crossing. Climatic years 1955-56 and 1956-57, however, show little correlation between the two stations. Table 7.4 presents a comparison between the annual peak discharges of the stations on the North Pine River for the period of concurrent record.

TABLE 7.3

NORTH PINE RIVER AT DAMSITE

PEAK ANNUAL DISCHARGES

Climatic Year	Month	Peak Discharge (m^3/s)	Rank
1954-55	Mar	758	1
1955-56	Feb	260	8
1956-57	Jan	52	13
1957-58	Jun	446	4
1958-59	Feb	592	2
1959-60	Nov	295	6
1960-61	Feb	125	11
1961-62	Nov	118	12
1962-63	-	-	Incomplete
1963-64	Mar	279	7
1964-65	-	-	Incomplete
1965-66	Dec	246	9
1966-67	Mar	492	3
1967-68	Jan	439	5
1968-69	Aug	159	10
1969-70	-	-	Incomplete

TABLE 7.4

COMPARISON OF PEAK ANNUAL DISCHARGES ON

NORTH PINE RIVER

Climatic Year	Month	Damsite Peak Discharge (m^3/s)	Young's Crossing Peak Discharge (m^3/s)	Ratio of Peak Discharge (+)
1954-55	Mar	758	996	0.761
55-56	Mar	210 *	335	0.627
56-57	Dec	36 *	162	0.222
57-58	Jun	446	387	1.152
58-59	Feb	592	505	1.172
59-60	Nov	295	318	0.928
60-61	Feb	125	134	0.933
61-62	Nov	118	126	0.937
62-63	Mar	-	402	-
63-64	Mar	279	318	0.877
64-65	Jul	-	415	-
65-66	Dec	246	194	1.268
66-67	Mar	492	450	1.093
67-68	Jan	439	483	0.909
68-69	Aug	159	136	1.169
69-70	Jan	-	70	-

Notes: * These flows are not the peak annual discharges for the climatic year at this station.

+ The ratio indicated is Q damsite/Q youngs crossing.

Results of the annual series flood frequency analysis for the Damsite are presented in Table 7.5 and Figure 7.2, along with the estimated 95% confidence limits. A Log-Pearson type III distribution has been adopted again.

TABLE 7.5

NORTH PINE RIVER AT DAMSITE

ANNUAL SERIES FLOOD FREQUENCY

Probability of Exceedence (%)	Average Recurrence Interval (Years)	Peak Discharge (m^3/s)	95% Confidence Limits (m^3/s)	
			Lower	Upper
50	2	286	181	453
20	5	500	340	737
10	10	640	427	959
5	20	767	462	1273
2	50	918	449	1878
1	100	1022	419	2496

In view of the limited number of years of record available at this station, results of this analysis would be regarded as being appropriate for events of up to 20% probability of exceedence. Extrapolation beyond this range is somewhat less accurate, as is evidenced by the range of the 95% confidence limits.

Interestingly, the results of this analysis compare reasonably well with the results of the annual series for Youngs Crossing. The 50, 2 and 1% probability of exceedence estimates are larger for the Damsite than for Youngs Crossing. This feature illustrates the effect of the influence of small floods on the selected distribution, because the record available at the Damsite contains few years of `small' floods.

7.4 SOUTH PINE RIVER AT CASH'S CROSSING

Streamflow records have been maintained at a number of locations near Cash's Crossing since the climatic year of 1908-09. A combined record based upon the following locations has been formed into an annual series of peak discharges.

Number	Station	AMTD (km)	Area (km^2)	Period
142201A	Cash's Crossing	14.0	179	1909-1917
142201B	Albany Creek	12.7	181	1916-1947
142201C	Fahey's Crossing	15.6	171	1947-1951
142201D	Cash's Crossing	14.0	179	1951-1965

Table 7.6 provides details of the 46 years of complete record available at these locations.

TABLE 7.6

SOUTH PINE RIVER AT CASH'S CROSSING

PEAK ANNUAL DISCHARGES

Climatic Year	Month	Peak Discharge	Rank
1917-18	Jan	48	40
18-19	Mar	24	45
19-20	Jan	71	35
20-21	Jun	66	37
21-22	Dec	155	19
22-23	Dec	1	46
23-24	Jul	31	43
24-25	Mar	109	26
25-26	Jan	70	36
26-27	Jan	252	9
27-28	Apr	376	5
28-29	Apr	139	20
29-30	May	173	15
30-31	Feb	614	1
31-32	Dec	76	34
32-33	Apr	110	25
33-34	Feb	311	7
34-35	Dec	31	44
35-36	Oct	90	30
36-37	Mar	173	16
37-38	Jan	164	18
38-39	Mar	125	21
39-40	Mar	113	24
40-41	Jan	88	31
41-42	Feb	118	23
42-43	Dec	95	28
43-44	Dec	209	13
44-45	Feb	37	41
45-46	Mar	239	10
46-47	Feb	362	6
47-48	May	431	4
48-49	Mar	36	42
49-50	Feb	236	11
50-51	Mar	510	2
51-52	Mar	60	39
52-53	Feb	305	8
53-54	Feb	182	14
54-55	Mar	509	3
55-56	Mar	166	17
56-57	Dec	94	29
57-58	Jun	99	27
58-59	Feb	120	22
59-60	Nov	84	32
60-61	Feb	84	33
61-62	Nov	64	38
62-63	Mar	216	12
63-64	-	-	Incomplete
64-65	-	-	Incomplete

It is evident from the plot of the annual series (Refer Figure 7.3) that a Log-Normal type distribution is more appropriate for this site. Results of the annual series flood frequency analysis are presented in Table 7.7.

TABLE 7.7

SOUTH PINE RIVER AT CASH'S CROSSING

ANNUAL SERIES FLOOD FREQUENCY

Probability of Exceedence (%)	Average Recurrence Interval (Years)	Peak Discharge (m^3/s)	95% Confidence Limits (m^3/s)	
			Lower	Upper
50	2	127	104	150
20	5	236	176	296
10	10	326	222	431
5	20	427	271	582
2	50	567	344	809
1	100	704	406	1003

7.5 SOUTH PINE RIVER AT DRAPER'S CROSSING

This gauging station was opened during the 1965-1966 climatic year and records have been collected to the present date. Some 21 years of complete record are available at this site, which have been incorporated into an annual series of peak discharges. Table 7.8 presents a summary of the annual peak discharges available at this station.

TABLE 7.8

SOUTH PINE RIVERS AT DRAPER'S CROSSING

PEAK ANNUAL DISCHARGES

Climatic Year	Month	Peak Discharge	Rank
1965-66	Dec	32	19
66-67	Jun	468	3
67-68	Jan	302	7
68-69	Aug	86	17
69-70	Jan	166	14
70-71	Jan	310	6
71-72	Apr	563	2
72-73	Oct	123	16
73-74	Jan	1241	1
74-75	Jan	12	21
75-76	Feb	293	8
76-77	Oct	177	13
77-78	Apr	53	18
78-79	Jan	288	9
79-80	May	150	15
80-81	Feb	179	12
81-82	Nov	348	5
82-83	Jun	362	4
83-84	Nov	256	11
84-85	Mar	266	10
85-86	Oct	19	20
86-87	-	-	Incomplete
87-88	-	-	Incomplete
88-89	-	-	Incomplete
89-90	-	-	Incomplete

Results of the annual series flood frequency analysis at Drapers Crossing are presented in Table 7.9. A Log-Pearson type III distribution has been adopted in this analysis. The results are also shown in Figure 7.4.

TABLE 7.9

SOUTH PINE RIVER AT DRAPER'S CROSSING

ANNUAL SERIES FLOOD FREQUENCY

Probability of Exceedence (%)	Average Recurrence Interval (Years)	Peak Discharge (m^3/s)	95% Confidence Limits (m^3/s)	
			Lower	Upper
50	2	201	115	353
20	5	448	294	684
10	10	624	400	974
5	20	787	430	1442
2	50	971	401	2401
1	100	1112	361	3423

7.6 SOUTH PINE RIVER AT CASH'S CROSSING (COMPOSITE ANNUAL SERIES)

A composite annual series has been formed that includes factored peak discharges from the gauging station at Draper's Crossing. This series was compiled as a result of the findings of the individual annual series flood frequency analyses at Cash's Crossing and Draper's Crossing. The estimated peak discharges at the gauging stations were considerably different because of two factors; firstly different distributions were fitted to each series; and secondly the series were from different periods. (Cash's Crossing record extends from 1917-18 to 1963-64, whilst Draper's Crossing record extends from 1965-66 to 1989-90).

The factored Draper's Crossing peak discharges are presented in Table 7.10. The factor for converting peak discharges at Draper's Crossing to Cash's Crossing has been taken to equal the ratio of the respective catchment areas to the power 0.7. This results in a factor of 1.09. This method is the same as that adopted in the report on sand and gravel extraction by Cameron, McNamara and Partners (1978). The composite annual series comprises 67 years of complete record.

TABLE 7.10

FACTORED DRAPER'S CROSSING

PEAK DISCHARGES

Climatic Year	Month	Peak Discharge (m^3/s)
1965-66	Dec	35
66-67	Jun	510
67-68	Jan	329
68-69	Aug	94
69-70	Jan	181
70-71	Jan	338
71-72	Apr	614
72-73	Oct	134
73-74	Jan	1353
74-75	Jan	13
75-76	Feb	319
76-77	Oct	193
77-78	Apr	58
78-79	Jan	314
79-80	May	164
80-81	Feb	195
81-82	Nov	379
82-83	Jun	395
83-84	Nov	279
84-85	Mar	290
85-86	Oct	21
86-87	-	-
87-88	-	-
88-89	-	-
89-90	-	-

The composite factored peak discharge annual series yield appreciably larger estimates than the original Cash's Crossing annual series. This is to be expected since the largest flow on record, January, 1974 occurs in the additional factored flow record, along with several other large events in April 1972 and June, 1967.

Table 7.11 summarised estimates of flood frequency based on the combined record and adopting a Log-Normal distribution. Figure 7.5 shows the fitted distribution and the observed values.

TABLE 7.11

SOUTH PINE RIVER AT CASH'S CROSSING

COMBINED FLOW ANNUAL SERIES

FLOOD FREQUENCY

Probability of Exceedence (%)	Average Recurrence Interval (Years)	Peak Discharge (m^3/s)	95% Confidence Limits (m^3/s)	
			Lower	Upper
50	2	146	116	176
20	5	295	218	372
10	10	425	278	572
5	20	575	346	804
2	50	808	450	1166
1	100	1014	542	1486

By way of comparison, the ten year ARI estimate of peak discharge at Cash's Crossing is similar to the value reported by Cameron McNamara, (1978). The estimates for the 50 and 100 year ARI flood events indicated in Table 7.11 are however, much larger than the corresponding values reported in the Cameron McNamara study. One of the reasons for the differences between the two studies is the length of available record at Draper's Crossing. A number of large floods have occurred in recent years, (April 1988 and April 1989) which have influenced the flood frequency analysis particularly in the range of the less frequent events.

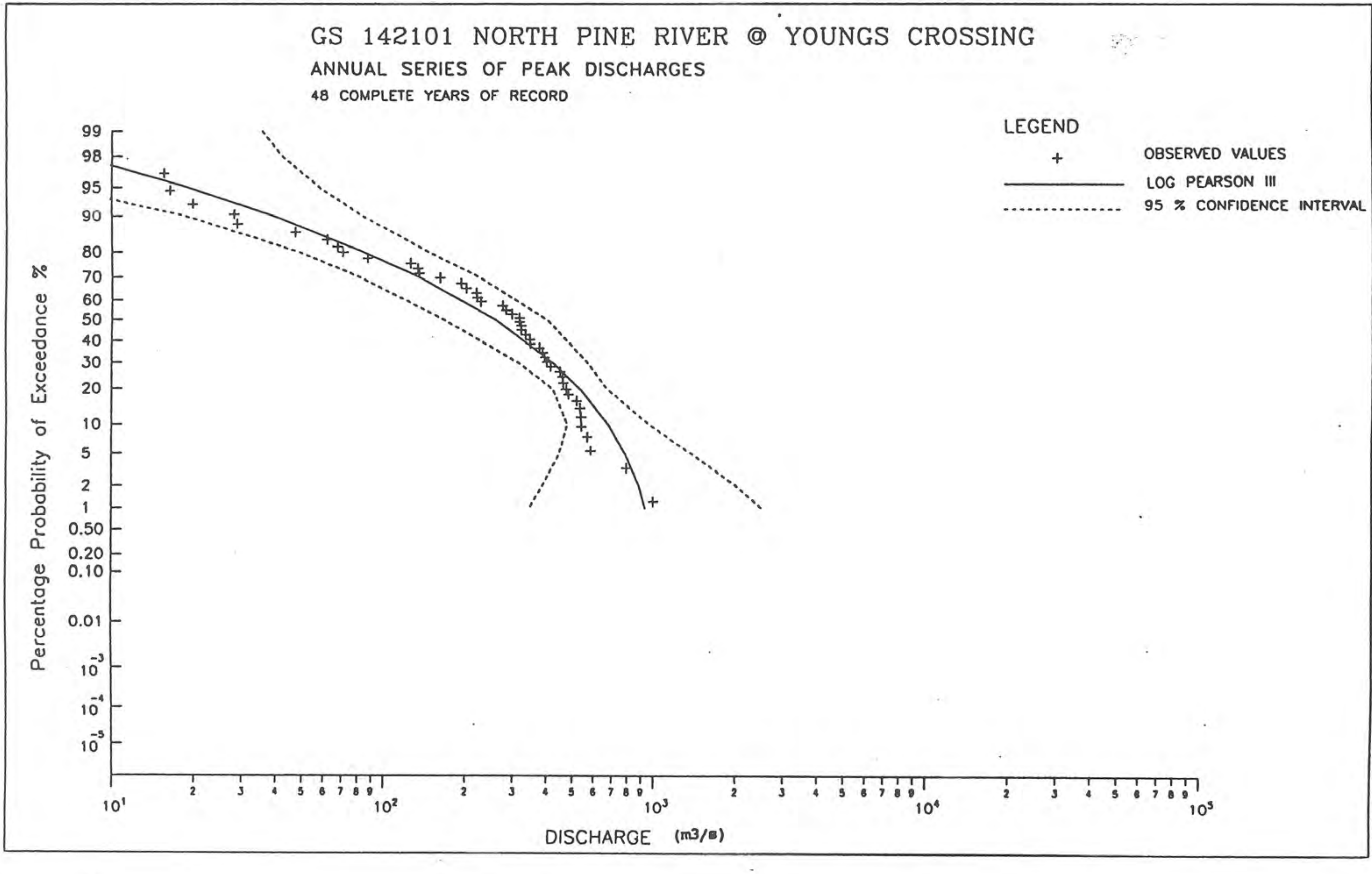

GS 142101 NORTH PINE RIVER @ YOUNGS CROSSING
ANNUAL SERIES OF PEAK DISCHARGES
48 COMPLETE YEARS OF RECORD
LEGEND
OBSERVED VALUES
LOG PEARSON III
95 % CONFIDENCE INTERVAL
Percentage Probability of Exceedance %
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.50
0.20
0.10
0.01
10^{-3}
10^{-4}
10^{-5}
10^1
10^2
10^3
10^4
10^5
DISCHARGE (m3/s)

FIGURE 7.1

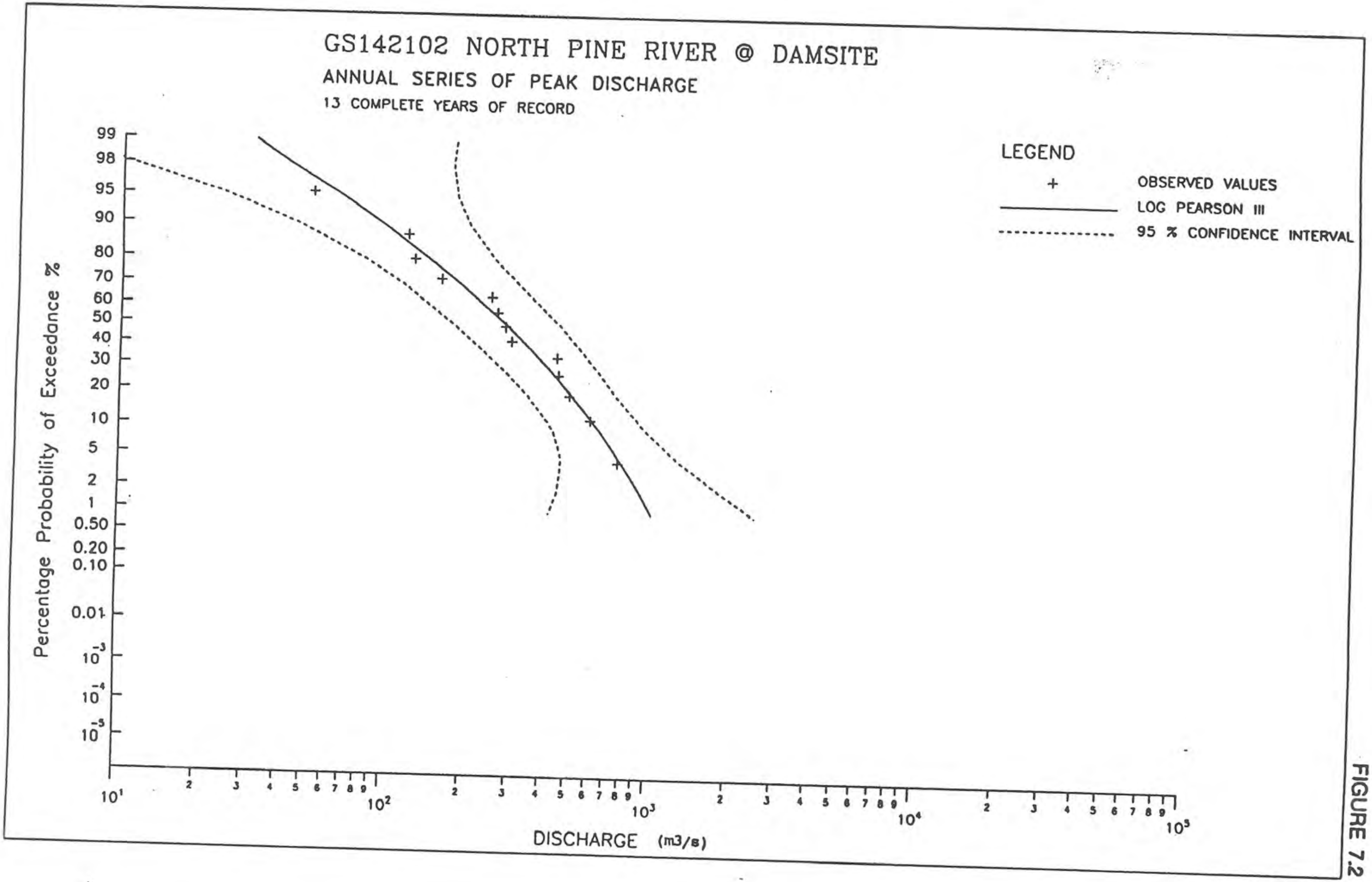

GS142102 NORTH PINE RIVER @ DAMSITE
ANNUAL SERIES OF PEAK DISCHARGE
13 COMPLETE YEARS OF RECORD
LEGEND
OBSERVED VALUES
LOG PEARSON III
95 % CONFIDENCE INTERVAL
Percentage Probability of Exceedance %
DISCHARGE (m3/s)

FIGURE 7.2

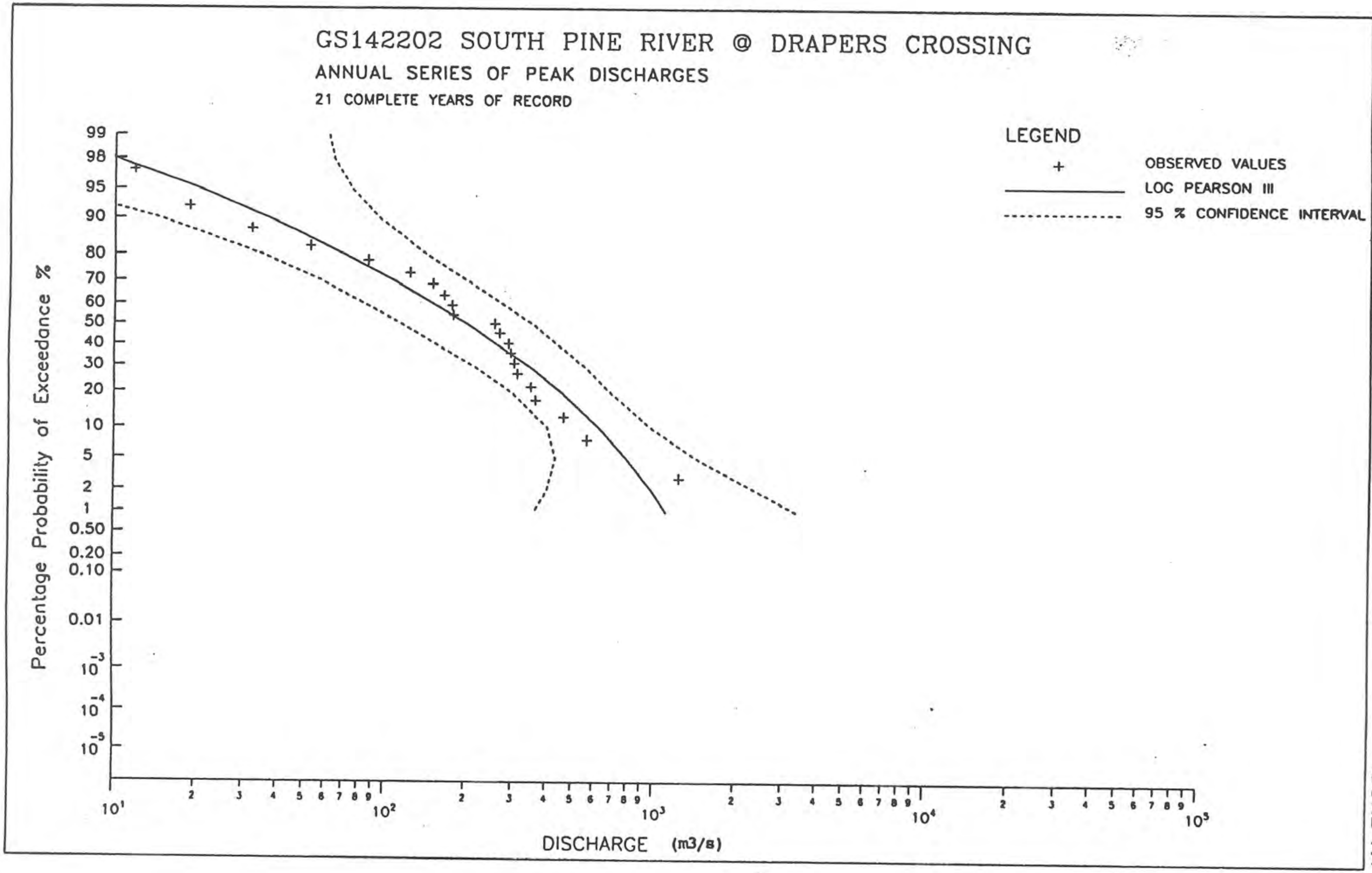

GS142202 SOUTH PINE RIVER @ DRAPERS CROSSING
ANNUAL SERIES OF PEAK DISCHARGES
21 COMPLETE YEARS OF RECORD
LEGEND
OBSERVED VALUES
LOG PEARSON III
95 % CONFIDENCE INTERVAL
Percentage Probability of Exceedance %
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.50
0.20
0.10
0.01
10^{-3}
10^{-4}
10^{-5}
10^{1}
10^{2}
10^{3}
10^{4}
10^{5}
DISCHARGE (m3/s)

FIGURE 7.4

TABLE 8.2

NORTH PINE RIVER AT NORTH PINE DAM

REACH DETAILS

Reach	Reach Length	Reach	Reach Length
A-B	4.0	O-P	4.0
C-B	3.0	P-N	2.0
B-D	2.4	N-Q	3.4
D-E	4.6	R-S	2.0
E-F	3.8	T-U	0.8
G-H	4.6	V-W	5.2
H-I	3.0	X-W	4.4
H-F	3.8	W-Y	1.6
F-J	2.2	Y-Z	1.6
K-J	3.0	AA-BB	2.0
J-L	5.0	CC-DD	1.5
M-L	2.4	EE-FF	1.2
L-N	2.2	GG-HH	1.2

NOTE: All reaches are natural

The storage capacity relationship for North Pine Dam is presented in Figure 8.2.

The normal gate operation procedure rating curve has been incorporated into the runoff-routing model. Figure 8.3 presents the curve. The rating curve is based upon the Water Resources Commission physical model study of the North Pine Dam radial gates (1991), and the existing normal gate operation procedure outlined in the Manual of Operational Procedures for Flood Releases from North Pine Dam (1986).

It should be noted that gate settings associated with normal gate operation have been based upon values presented in the operation manual. Prototype measurements to confirm the reliability of these values are yet to be undertaken and as a consequence the rating curve presented should be regarded as provisional at this time.

The modelled rating curve has been extended so that the headwater range encompasses levels up to the embankment crest level (43.28m AHD). The extension is based upon free flow (or uncontrolled free surface flow) up to a level whereby the water surface profile makes contact with the bottom of the fully open gate (i.e. headwater level of 42.15 m AHD). Sluice gate flow and weir flow equations have been utilised to describe flow under and over the fully open radial gates for headwater levels up to embankment crest level.

The normal gate operation procedure rating curve with one gate out of service has also been incorporated into the runoff routing model. The WRC physical model study derived a rating curve for flow over the top of a closed gate. This rating curve has been incorporated into the overall rating curve for the one gate out of service operation procedure. The rating curve for one gate out of service is presented in Figure 8.4.

8.2.2 **South Pine River at Cash's Crossing**

An additional sub-area has been added to the South Pine River catchment model and the model extended to Cash's Crossing. Cash's Crossing has been selected as the upstream limit of the proposed hydraulic model of the Pine River system. It is proposed to use the design flood estimates for Cash's Crossing as an upstream boundary condition of the hydraulic model.

The modified catchment layout of the South Pine River is shown in Figure 8.1 whilst the catchment characteristics are presented in Table 8.3 and 8.4.

TABLE 8.3

SOUTH PINE RIVER AT CASH'S CROSSING

SUB-AREA DETAILS

Sub-Area	Area (km^2)	Distance to Outlet (km^2)
1	14.1	25.8
2	10.2	24.0
3	20.2	21.0
4	8.9	16.6
5	12.9	21.8
6	12.5	17.6
7	13.5	10.8
8	12.9	28.0
9	11.5	25.6
10	13.0	21.2
11	14.4	13.6
12	14.7	11.4
13	19.9	2.2

Catchment area $A = 178.8\ km^2$
Distance to Centroid of Area $dc = 17.7\ km$

TABLE 8.4

SOUTH PINE RIVER AT CASH'S CROSSING

REACH DETAILS

Reach	Reach Length	Reach	Reach Length
A-B	4.2	L-M	4.6
C-B	2.4	N-M	2.0
B-D	3.6	M-O	2.4
E-D	3.0	O-P	7.6
D-F	1.4	P-Q	5.4
F-G	2.2	R-Q	3.2
H-I	4.2	Q-K	2.0
I-G	3.2	K-S	0.5
G-J	3.6	S-T	3.5
J-K	4.6	T-U	2.2

NOTE: All reaches are natural

To accommodate the additional sub-area and hence change to the catchment characteristics the model parameters derived for Draper's Crossing have been modified so that they may be used with the Cash's Crossing model. The k value has been proportioned according to the ratio of distance to centroid of area of both models. This results in model parameters for the Cash's Crossing model of:

$k = 19.3 \qquad m = 0.8$

8.2.3 Sideling Creek at Lake Kurwongbah

A runoff-routing catchment model has also been developed for Lake Kurwongbah. This model consists of eight sub-areas and has been compiled to determine outflow hydrographs from Sideling Creek Dam. The arrangement of this catchment model and the characteristics of the catchment properties are shown in Figure 8.1 and Tables 8.5 and 8.6.

TABLE 8.5

LAKE KURWONGBAH

SUB-AREA DETAILS

Sub-Area	Area (km^2)	Distance to Outlet (km^2)
1	8.7	13.8
2	7.6	9.0
3	6.0	12.4
4	5.2	8.8
5	8.7	4.8
6	4.4	2.4
7	9.8	4.0
8	2.3	-

Catchment area $A = 52.6\ km^2$
Distance to Centroid of Area $dc = 7.7\ km$

TABLE 8.6

LAKE KURWONGBAH

REACH DETAILS

Reach	Reach Length (km)
A-B	4.8
B-C	2.4
D-E	3.6
E-C	2.2
C-EG	1.8
F-G	1.6
H-I	0.6
J-HK	3.2

Note: All reaches are natural

Storage capacity curves, and a spillway rating curve based on physical model testing have been incorporated into the Lake Kurwongbah runoff-routing model.

Figure 8.5 shows the storage volume/elevation relationship for Lake Kurwongbah, whilst Figure 8.6 illustrates the spillway rating curve.

The spillway rating curve ranges from the spillway crest level of 20.42 m AHD to a level of 23.3 m AHD, a depth of 2.88 metres. Embankment crest level of the dam is at an elevation of 24.08 m AHD. The rating curve was extended to this level by straight line extrapolation. The storage/elevation relationship was also extended to approximately embankment crest level by determining the area enclosed by the 25 m contour above the dam on 1:25 000 topographic maps.

The original data was provided by the Pine Rivers Shire Council. Runoff routing model parameters for the Lake Kurwongbah model can be estimated from either regional formulae or they can be proportioned from adjacent calibrated catchment model parameters such as North Pine River and South Pine River. The latter form of derivation is preferred and further discussion concerning appropriate runoff routing parameters is presented in Section 8.4.2.

8.3 DESIGN RAINFALL ESTIMATES

8.3.1 Average Recurrence Intervals of up to One Hundred Years

Design rainfall intensity-frequency-duration data has been derived for a number of locations within the Pine River catchment. Estimates of this data are based on the procedures outlined in Chapter 2 of the Australian Rainfall and Runoff, (1987). The sites chosen are listed in Table 8.7 along with basic rainfall intensities, skewness values and geographical factors.

TABLE 8.7

LOCATIONS OF IFD ESTIMATES

Location	Rainfall Intensity (mm/hr)						Skewness	Geographical Factors	
	$^{2}i_{1}$	$^{2}i_{12}$	$^{2}i_{72}$	$^{50}i_{1}$	$^{50}i_{12}$	$^{50}i_{72}$		F2	F5O
Dayboro	46.5	9.8	2.9	88.4	19.0	6.8	0.175	4.39	17.27
Deagon	48.0	8.9	2.8	92.9	18.4	6.2	0.130	4.40	17.31
Mt Glorious	46.4	10.2	3.5	88.6	22.3	8.1	0.200	4.39	17.31
Mt Mee	46.3	9.8	3.1	87.5	20.0	7.0	0.200	4.38	17.27
Mt Nebo	46.2	10.0	3.3	90.0	22.1	7.2	0.200	4.39	17.20
Narangba	47.5	9.8	2.9	91.8	18.9	6.6	0.130	4.39	17.32
North Pine Dam	46.6	9.7	2.9	92.2	18.9	6.0	0.150	4.39	17.29
Petrie	47.0	9.4	2.8	94.0	18.9	6.1	0.130	4.39	17.30
Samford	46.6	9.6	2.9	93.7	18.9	6.0	0.175	4.39	17.23

Were $^{2}i_{1}$ = 1 hour duration, 2 year ARI

Summaries of design rainfall intensity-frequency-duration data at each location are provided in Appendix B. Storm rainfall depths are listed, with all values in millimetres.

Adopted design storm rainfall depths are provided in Tables 8.8, 8.9 and 8.10 for each catchment; North Pine River at North Pine Dam; South Pine River at Cash's Crossing; and Sideling Creek at Lake Kurwongbah. All values are in millimetres.

TABLE 8.8

NORTH PINE RIVER AT NORTH PINE DAM

DESIGN CATCHMENT RAINFALL (mm)

Duration (hrs)	Average Storm Recurrence Interval (Years)						
	1	2	5	10	20	50	100
1	36	46	59	67	78	92	104
3	54	70	90	102	119	141	157
6	69	90	117	132	153	182	205
12	89	116	151	175	206	247	280
24	115	151	203	235	267	338	385
48	144	191	263	308	368	453	520
72	161	214	299	353	424	525	606

TABLE 8.9

SOUTH PINE RIVER AT CASH'S CROSSING

DESIGN CATCHMENT RAINFALL (mm)

Duration (hrs)	Average Storm Recurrence Interval (Years)						
	1	2	5	10	20	50	100
1	35	46	60	68	80	96	105
3	53	70	92	105	123	146	167
6	69	90	119	134	160	193	219
12	89	116	155	178	210	253	288
24	116	153	205	236	279	338	385
48	149	196	265	306	363	442	504
72	168	221	300	347	413	503	576

TABLE 8.10

SIDELING CREEK AT LAKE KURWONGBAH
DESIGN CATCHMENT RAINFALL (mm)

Duration (hrs)	Average Storm Recurrence Interval (Years)						
	1	2	5	10	20	50	100
1	36	47	59	69	80	95	106
3	55	71	92	103	120	142	159
6	70	90	117	132	153	181	204
12	89	116	150	169	197	233	261
24	113	149	197	225	264	315	355
48	141	186	252	293	346	421	480
72	155	205	284	333	396	485	557

Design storm temporal patterns of rainfall bursts have been determined in accordance with Chapter 3 of Australian Rainfall and Runoff, (1987). The Pine River catchment is located in Zone 3 of the North-East Coast Division. Design storm temporal patterns for various durations are listed in Table 3.2 of Volume 2 of Australian Rainfall and Runoff (1987). Computer program TEMPAT (Ruffini, 1990) was used to incorporate the design rainfall temporal patters into the runoff-routing models.

8.3.2 Average Recurrence Intervals Above One Hundred Years

Estimates of probable maximum precipitation (PMP) for the Pine River catchment have been obtained from the Bureau of Meteorology, (1991). These estimates have been derived for three catchments; The Pine River above the mouth; North Pine River at North Pine Dam; and Sideling Creek at Lake Kurwongbah. Durations ranging from one hour to seven days have been considered.

The Bureau used several methods to determine the estimates of PMP for each catchment and the range of durations. Bulletin 51 (Bureau of Meterodogy, 1985) was used to derive the PMP estimates for durations of up to six hours for all catchments. The Generalised Tropical Storm Method (GTSM) was utilised for durations of six to ninety six hours. The Pine River catchment lies in the "East Coast Tropical Zone". This zone is influenced by a quasi-stationary trough adjacent to the Queensland coast that appears to enhance heavy rainfall events.

For durations of between five and seven days, the Bureau used the Gordon method of PMP estimation, which is also applicable to the "East Coast Tropical Zone". The report by the Bureau is presented in Appendix C for further information.

Estimates of the PMP for the three catchments mentioned earlier are provided in Tables 8.11, 8.12 and 8.13. Probabilities of exceedance have been assigned to the probable maximum precipitation in accordance with the procedures outlined in Chapter 13 of Australian Rainfall and Runoff. Intermediate rainfall depths for events between the PMP and 100 Year Average Recurrence Interval are also listed in the tables.

TABLE 8.11

NORTH PINE RIVER AT NORTH PINE DAM

DESIGN CATCHMENT RAINFALLS (mm)

Duration (hrs)	Average Storm Recurrence Interval (Years)					
	200	500	1000	10 000	100 000	1 000 000
1	120	135	150	200	240	-
3	175	205	225	295	370	450
6	235	275	305	410	500	590
12	300	360	410	540	680	810
24	450	530	600	810	990	1160
48	600	720	840	1170	1440	1720
72	740	920	1050	1450	1800	2150
96						2490
120						2580
144						2650
168						2920

Note: The ARI assigned to the PMP of one hour duration is 100 000 years.

The ARI assigned to the PMP for all other durations is 1 000 000 years.

TABLE 8.12

PINE RIVER AT BRAMBLE BAY DESIGN CATCHMENT RAINFALLS (mm)

Duration (hrs)	Average Storm Recurrence Interval (Years)					
	200	500	1000	10 000	100 000	1 000 000
1	120	135	150	180	200	-
3	180	205	230	390	390	-
6	235	275	310	520	520	-
12	330	375	420	660	660	750
24	430	490	540	910	910	1080
48	590	610	810	1350	1350	1590
72	680	820	940	1680	1680	1990
96						2350
120						2430
144						2500
168						2700

Note: The ARI assigned to the PMP of 1,3, and 6 hour durations is 100 000 years.

The ARI assigned to the PMP for all other durations is 1 000 000 years.

TABLE 8.13

SIDELING CREEK AT LAKE KURWONGBAH DESIGN CATCHMENT RAINFALLS (mm)

Duration (hrs)	Average Storm Recurrence Interval (Years)					
	200	500	1000	10 000	100 000	1 000 000
1	120	140	160	220	270	320
3	180	215	245	350	460	580
6	235	280	325	470	620	770
12	295	350	410	590	790	980
24	410	490	560	780	1040	1300
48	560	680	780	1120	1460	1890
72	650	780	920	1350	1820	2340
96						2660
120						2760
144						2830
168						3210

Note: The ARI assigned to the PMP for all durations is 1 000 000 years.

8.4 DESIGN FLOOD DISCHARGE ESTIMATES

8.4.1 North Pine River at North Pine Dam

Estimates of design flood discharges have been derived for both pre and post-North Pine Dam. The post-North Pine Dam estimates are of the form of inflow hydrographs to the storage, and outflow hydrographs resulting from the existing normal gate operation procedure. The one gate out of service operation procedure has also been utilised so as to illustrate the impact of possible reductions in release capacities from North Pine Dam.

The pre-North Pine Dam design flood estimates have been derived so that the effect of the storage can be determined and to allow comparisons with results from flood frequency analyses to be made.

Pre-North Pine Dam

Probable maximum flood (PMF) estimates of durations ranging from one hour to seven days have been derived using the natural catchment runoff-routing model, design rainfall depth and temporal pattern estimates and the model calibration parameters of the North Pine River. Table 8.14 presents a summary of peak discharge, time to peak and flood volumes for the PMF's. It should be noted that the results assume the storm is centred over the North Pine Dam catchment and that uniform rainfall occurs within the catchment. There is no storage in the catchment and rainfall losses are as follows:

Initial Loss = 0 mm
Continuing Loss = 1 mm/hr

A continuing loss of 1 mm/hr was used because calibration event loss rates were of this order. A continuing loss rate of 0 mm/hr is too conservative according to Australian Rainfall and Runoff. (1987).

TABLE 8.14

NORTH PINE RIVER AT DAMSITE

PMF ESTIMATES : PRE DAM

Duration (Days)	Duration (Hours)	Flood Volume (ML)	Time to Peak (hrs)	Peak Discharge (m^3/s)
	1	79 270	5.17	1173
	2	119 700	5.17	2004
	3	153 720	5.75	2536
	6	200 370	7.5	3389
	9	237 940	10.25	3925
	12	274 940	13.0	4378
	18	335 280	17.5	4617
1	24	391 920	21.0	4449
2	48	576 800	30.0	3689
3	72	716 930	55.0	3258
4	96	825 920	74.0	3192
5	120A	850 870	68.0	3185
	B	850 860	97.0	3161
6	144A	864 540	65.0	3135
	B	864 600	101.0	3076
	C	864 520	118.0	3232
7	168A	972 180	30.0	3708
	B	949 410	63.0	3422
	C	949 250	151.0	3234

Note: A, B and C refer to different temporal patterns recommended for particular durations.

Results of this runoff-routing model for North Pine River in its natural catchment are presented in Figure 8.7.

It is evident from the table that the North Pine River catchment in its natural state (i.e. Pre-North Pine Dam) has a critical duration of eighteen hours.

Design flood estimates for the North Pine River in its natural state have been derived for a range of average recurrence intervals (ARI), assuming an eighteen hour time of concentration. Initial losses have been adjusted for the two, five and ten year ARI floods, so that the runoff-routing model estimates agree with flood frequency analysis estimates. Continuing losses of 1 mm/hr have been adopted for all events. Table 8.15 summarises the runoff-routing model results for the range of ARI's.

TABLE 8.15

NORTH PINE RIVER AT DAMSITE

RUNOFF-ROUTING DESIGN FLOOD ESTIMATES

PRE-DAM

Average Recurrence Interval (Years)	Initial Loss (mm)	Continuing Loss (mm/hr)	Flood Volume (ML)	Time to Peak (hrs)	Peak Discharge (m^3/s)
2	46	1	23 270	19.5	261
5	19	1	45 900	18.5	534
10	9	1	57 470	18.0	680
20	0	1	74 660	18.0	861
50	0	1	91 190	18.0	1091
100	0	1	100 240	17.5	1272
200	0	1	111 000	19.0	1335
500	0	1	131 700	18.5	1625
1 000	0	1	152 410	18.5	1919
10 000	0	1	211 060	18.0	2766
100 000	0	1	273 170	18.0	3685
1 000 000 (PMF)	0	1	335 280	17.5	4617

Figure 8.8 shows a comparison between flood frequency analysis results and runoff-routing model estimates for the North Pine River in its natural state.

Post-North Pine Dam

PMF estimates of durations ranging from one hour to seven days have been derived considering the North Pine River catchment model incorporating the storage of North Pine Dam. Design rainfall depths, temporal patterns and rainfall losses adopted for the pre-dam scenario have again been utilised, as have the calibration model parameters.

The PMF estimates have been summarised in Table 8.16 and Figure 8.9 for comparative purposes. It should be noted that both inflow and outflow estimates are presented in Figure 8.9.

TABLE 8.16

NORTH PINE RIVER AT NORTH PINE DAM

PMF ESTIMATES OF STORAGE INFLOWS

Duration (Days)	Duration (Hours)	Flood Volume (ML)	Time to Peak (hrs)	Peak Inflow (m^3/s)
	1	79 270	0.75	5 365
	2	125 900	1.42	6 144
	3	148 470	2.25	6 038
	6	199 470	4.25	5 553
	9	238 035	8.0	5 236
	12	275 130	10.5	5 218
	18	335 365	13.5	5 015
1	24	391 870	18.0	4 659
2	48	576 560	24.0	3 751
3	72	716 900	54.0	3 302
4	96	825 960	72.0	3 288
5	120A	850 880	66.0	3 349
	B	850 870	96.0	3 190
6	144A	864 550	58.0	3 204
	B	864 610	100.0	3 079
	C	864 540	115.0	3 317
7	168A	972 130	25.0	3 799
	B	949 380	59.0	3 548
	C	949 200	151.0	3 235

Note: The difference in flood volumes is related to the length of the simulation only.

The effect of the storage is very noticeable for short duration events of up to twenty four hours. The critical duration for peak discharges is only two hours for the catchment with the storage of North Pine Dam considered. Peak discharge increases from 4620 m^3/s to 6140 m^3/s with the inclusion of the storage.

These effects are as expected because of the size and configuration of the storage within the catchment. The storage at Full Supply has a surface area equivalent to 7.2% of the total catchment area with the inundated area confined to the lower reaches of the catchment.

Normal Gate Operation

Estimates of the corresponding PMF outflows from North Pine Dam for normal gate operation procedures are presented in Table 8.17. It has been assumed that North Pine Dam is at Full Supply (201 850 ML) prior to the flood event.

TABLE 8.17

NORTH PINE RIVER AT NORTH PINE DAM

PMF ESTIMATES OF STORAGE OUTFLOWS

NORMAL GATE OPERATION PROCEDURE

Duration (Days)	Duration (Hours)	Flood Volume (ML)	Time to Peak (Hours)	Peak Outflow (m^3/s)	Peak Lake Level (mAHD)
	1	79 270	3.08	1569	40.51
	2	125 900	3.42	2612	40.99
	3	148 470	3.75	3440	41.21
	6	199 470	6.75	3707	41.66
	9	238 035	10.0	3870	41.93
	12	275 130	13.0	4013	42.20
	18	335 365	18.0	4060	42.38
1	24	391 870	23.0	4013	42.27
2	48	576 560	36.0	3608	41.49
3	72	716 900	54.0	3296	41.09
4	96	835 960	72.0	3273	41.08
5	120A	850 880	66.0	3318	41.09
	B	850 870	96.0	3188	41.07
6	144A	864 550	59.0	3192	41.07
	B	864 610	100.0	3079	41.06
	C	864 540	116.0	3302	41.09
7	168A	972 130	34.0	3608	41.49
	B	949 380	62.0	3423	41.18
	C	949 200	151.0	3235	41.08

The estimates of PMF outflows have been summarised in Figure 8.9. It is evident that the critical duration for the normal gate operation procedure outflows from North Pine Dam is 18 hours. A peak discharge of 4060 m^3/s is estimated with a corresponding peak lake level of 42.38 mAHD. The peak lake level anticipated is some 0.72 metres above the radial gate winch switchgear (41.66 mAHD) and approximately 0.92 metres above the maximum flood level predicted in the 1966 Study.

The estimated peak discharge falls within the sluice gate underflow and weir overflow part of the rating curve extension. It is therefore not necessarily as accurate as the physically modelled section of the rating curve. The result is however regarded as the most reliable at this time.

Several other PMF's of durations ranging from six hours up to two days and including the seven day storm result in peak lake levels which would cause inundation of the radial gate winch switchgear.

It has been assumed that the gates can still be operated throughout an event even though the winches may be inundated. Details such as this will have to be confirmed during the review of storage operation.

A full flood frequency of estimated design flood outflows from North Pine Dam is presented in Table 8.18. Normal gate operation procedures were adopted for all events and all events were assumed to have a rainfall duration of 18 hours. North Pine Dam was assumed to be at Full Supply prior to the flood event and rainfall loss parameters that were adopted for the pre-dam scenario have again been utilised.

The water level in North Pine Dam prior to a flood event assumption is significant as it has a large impact on peak outflow estimates. The volume of storage available within the Dam prior to a flood event can substantially reduce and even prevent outflow from the Dam occurring. It is possible an empty storage will be capable of storing a complete flood event thus reducing the downstream 'flood' to no flow.

An analysis of the annual probability of exceedence of various water levels within North Pine Dam will be performed as part of an examination of storage operations of the Dam. This information will be used in conjunction with estimates of design flood inflows to produce estimates of design flood outflows from the Dam.

Results presented in this study should therefore be regarded as 'conservative' given the assumption of Full Supply prior to the flood. This assumption especially affects the higher probability flood estimates whose flood volumes are small in comparison with the storage capacity of North Pine Dam.

TABLE 8.18

NORTH PINE RIVER AT NORTH PINE DAM

RUNOFF ROUTING DESIGN FLOOD OUTFLOW ESTIMATES

NORMAL GATE OPERATION PROCEDURE

Average Recurrence Interval (Years)	Initial Loss (mm)	Continuing Loss (mm/hr)	Flood Volume (ML)	Time to Peak (hrs)	Peak Outflow (m^3/s)	Peak Lake Level (mAHD)
2	46	1	26 010	16.5	356	39.88
5	19	1	49 080	16.5	650	40.02
10	9	1	60 790	17.0	789	40.10
20	0	1	74 900	17.0	969	40.19
50	0	1	91 440	17.0	1 208	40.32
100	0	1	104 200	16.5	1 392	40.42
200	0	1	111 100	17.5	1 423	40.44
500	0	1	131 800	17.0	1 714	40.59
1 000	0	1	152 500	17.0	2 016	40.73
10 000	0	1	211 170	16.0	3 010	41.06
100 000	0	1	273 280	17.5	3 594	41.47
PMF	0	1	335 365	18.0	4 060	42.38

Note: The difference in flood volumes is related to the length of the simulation only.

The effect of the existing normal gate operation procedure can be seen by comparing Tables 8.15 and 8.18. In appears that the existing normal gate operation procedure actually leads to increases in peak discharges for events with ARI's less than 100 000 years. The mitigating capability of North Pine Dam only becomes apparent with the most extreme floods, with the PMF reduced from 4620 m^3/s, pre-dam to 4060 m^3/s post-dam.

The estimates of peak outflow are generally higher than values presented in earlier studies. Comparisons with the 10, 50 and 100 ARI estimates presented by Cameron, McNamara, (1978), indicates the greatest difference is associated with the more frequent floods.

One Gate Out Of Service Operation

Table 8.19 provides a summary of the estimates of PMF outflows from North Pine Dam for normal gate operation procedures when one gate is out of service. These outflows correspond to the storage inflows presented in Table 8.16. Once again North Pine Dam is assumed to be at Full Supply prior to the flood event. Figure 8.10 presents a summary of peak inflows and outflows for this case.

TABLE 8.19

NORTH PINE RIVER AT NORTH PINE DAM

PMF ESTIMATES OF STORAGE OUTFLOWS

ONE GATE OUT OF SERVICE

Duration (Days)	Duration (Hours)	Flood Volume (ML)	Time to Peak (hrs)	Peak Outflow (m^3/s)	Peak Lake Level (mAHD)
	1	79 270	3.08	1569	40.51
	2	125 900	3.33	2654	40.96
	3	148 470	4.75	2837	41.32
	6	199 470	7.75	3164	41.96
	9	238 035	11.0	3321	42.37
	12	275 130	13.5	3438	42.76
	18	335 365	19.0	3557	43.14
1	24	391 870	24.0	3585	43.22
2	48	575 560	39.0	3389	42.60
3	72	716 900	56.0	3134	41.90
4	96	825 960	77.0	3029	41.70
5	120A	850 880	79.0	3029	41.70
	B	850 870	99.0	3014	41.67
6	144A	864 550	68.0	2963	41.57
	B	864 610	102.0	2993	41.63
	C	864 540	124.0	3070	41.78
7	168A	972 130	36.0	3358	42.50
	B	949 380	75.0	3230	42.08
	C	949 200	152.0	3176	41.98

The critical duration for North Pine Dam when one gate is out of service increases from 18 hours to 24 hours. The peak outflow is reduced to 3585 m^3/s, whilst the peak lake level increases to 43.22 mAHD or just 60mm below the crest level of the dam. These effects are expected given the reduced capacity of outflow from the spillway.

Storms of durations ranging from six hours up to seven days will result in the radial gate winch switchgear becoming inundated for some period of time.

The 24 hour PMF is very close to being the Imminent Failure Flood for North Pine Dam when one gate is out of service.

A full Flood Frequency of estimated design flood outflows from North Pine Dam is presented in Table 8.20 assuming one gate is out of service. All flood events are assumed to have a 24 hour rainfall duration and rainfall loss parameters are the same as previous. North Pine Dam is assumed to be a Full Supply prior to the flood event.

TABLE 8.20

NORTH PINE RIVER AT NORTH PINE DAM

RUNOFF ROUTING DESIGN FLOOD OUTFLOW ESTIMATES

ONE GATE OUT OF SERVICE

Average Recurrence Interval (Years)	Initial Loss (mm)	Continuing Loss (mm/hr)	Flood Volume (ML)	Time To Peak (hrs)	Peak Outflow (m^3/s)	Peak Lake Level (mAHD)
2	46	1	28 550	21.0	312	39.86
5	19	1	55 270	21.0	544	39.97
10	9	1	69 720	21.0	666	40.04
20	0	1	83 850	20.0	817	40.12
50	0	1	108 320	20.5	1050	40.23
100	0	1	124 530	20.0	1222	40.34
200	0	1	146 990	21.0	1605	40.53
500	0	1	174 600	21.0	1928	40.68
1 000	0	1	198 750	19.0	2259	40.77
10 000	0	1	271 210	22.0	2835	41.32
100 000	0	1	333 320	23.0	3285	42.23
PMF	0	1	391 970	24.0	3585	43.22

The result of operation North Pine Dam when one gate is out of service is that peak outflows are reduced for any given ARI event when compared with the normal gate operation procedure. Peak lake levels are also increased for events with ARI's above 100 years. Because of the difference in critical duration between normal gate operation and one gate out of service operation, flood volumes of corresponding ARI events are also larger for the one gate out of service.

The effect of the rating curves can be seen through a comparison of peak lake levels between normal gate operation and one gate out of service operation. The peak lake levels for one gate out of service are lower than the normal gate operation levels for corresponding ARI events below 100 years. This is so even when the peak outflows for one gate out of service are lower than the normal gate operation outflows. The respective design storm temporal patterns also have an impact upon these results.

The results presented in Table 8.20 cannot be compared directly with the pre-dam results because of the difference in duration.

8.4.2 **Sideling Creek at Lake Kurwongbah**

Estimates of design flood discharges have been derived for both inflows and outflows from Lake Kurwongbah. These estimates are based upon the runoff-routing catchment model described in Section 8.2.3 of this report.

Design rainfall depths and temporal patterns are based on the Bureau of Meteorology report (1991) estimates and estimates obtained from Australian Rainfall and Runoff (1987) as outlined in Section 7.3. The results are for the situation where the storm is centred over Sideling Creek with a uniform rainfall depth distributed throughout the catchment.

Rainfall loss parameters are similar to those adopted for the North Pine Dam investigation.

Two different catchment model parameter sets have been considered in view of comparisons made with the report by John Wilson and Partners (1989). Results are presented for each model parameter set in the following sections.

Model Parameters Proportioned According to South Pine River Parameters

Probable maximum flood (PMF) estimates for durations ranging from one hour to seven days have been derived for the catchment of Lake Kurwongbah. A range of durations has been considered to enable the critical duration of the catchment to be determined.

Runoff-routing model parameters proportioned from the calibration parameters of the South Pine River model to Draper's Crossing are considered appropriate for use with this model. The South Pine River Model parameters have been factored by a ratio of distance to controid of area in order to derive the Lake Kurwongbah parameters. These values provide reasonable agreement with estimates of model parameters obtained from regional relationships applicable to the study area. (Refer to Section 6). The values are:

Adopted Values
K = 8.3
m = 0.8
Weeks
K = 7.2
m = 0.8
McMahon and Muller
K = 9.1
m = 0.8

Tables 8.21 and 8.22 provide a summary of the PMF estimates of inflow and outflow of Lake Kurwongbah for these model parameters. Lake Kurwongbah is assumed to be at Full Supply (15450 ML) prior to the flood.

Figure 8.11 illustrates the relationship between peak inflow and outflow and duration of storm.

TABLE 8.21

SIDELING CREEK AT LAKE KURWONGBAH

PMF ESTIMATES OF STORAGE INFLOWS

Duration (Days)	Duration (Hours)	Flood Volume (ML)	Time to Peak (hrs)	Peak Inflow (m^3/s)
	1	16 760	0.75	2 367
	2	25 120	1.42	2 731
	3	30 300	2.0	2 613
	6	40 150	2.75	2 150
	9	45 500	4.0	1 580
	12	50 870	4.5	1 360
	18	58 960	8.0	1 057
1	24	67 060	11.0	902
2	48	96 800	14.0	672
3	72	119 200	54.0	555
4	96	134 740	72.0	551
5	120A	139 050	66.0	587
	B	139 050	96.0	522
6	144A	141 130	57.0	557
	B	141 140	97.0	502
	C	141 130	115.0	559
7	168A	163 300	25.0	655
	B	159 810	58.0	626
	C	159 810	140.0	543

TABLE 8.22

SIDELING CREEK AT LAKE KURWONGBAH

PMF ESTIMATES OF STORAGE OUTFLOWS

Duration (Days)	Duration (Hours)	Flood Volume (ML)	Time to Peak (hrs)	Peak Outflow (m^3/s)	Peak Lake Level (m AHD)
	1	16 710	2.5	647	22.45
	2	25 060	3.17	850	23.38
	3	29 600	4.0	936	23.88
	6	40 140	6.0	1 054	24.51*
	9	45 500	9.0	995	24.21*
	12	50 860	11.0	949	23.96
	18	58 960	14.5	853	23.40
1	24	67 060	14.0	788	23.02
2	48	96 790	26.0	628	22.40
3	72	119 190	54.0	553	22.21
4	96	134 740	72.0	546	22.20
5	120A	139 050	67.0	562	22.23
	B	139 050	96.0	522	22.15
6	144A	141 130	59.0	542	22.19
	B	141 140	100.0	502	22.11
	C	141 130	116.0	550	22.21
7	168A	163 300	26.0	644	22.45
	B	159 830	59.0	610	22.35
	C	159 810	151.0	543	22.20

Note: Embankment Crest Level of 24.08 mAHD overtopped. The difference in flood volumes is related to the length of simulation only.

The critical duration for inflow hydrographs into Lake Kurwongbah is two hours which results in a peak inflow of 2730 m^3/s. The mitigating effect of the storage is evident from the tables, particularly for events of up to twenty four hours duration. The peak outflow from Lake Kurwongbah is estimated to be 1054 m^3/s, which corresponds to a six hour duration storm.

A complete flood frequency of estimated design flood outflows from Lake Kurwongbah is presented in Table 8.23. All events considered were assumed to have a rainfall duration of six hours and Lake Kurwongbah was assumed to be at Full Supply prior to the flood. Rainfall loss parameters adopted for Lake Kurwongbah are equivalent to the values used in the pre-North Pine Dam determination for events of up to a 10 year return period.

The assumption of initial water level at Full Supply is considered conservative for the higher probability of occurrence design floods. The values of peak outflow presented in Table 8.23 should therefore be regarded as upper limits of the likely magnitude of such events.

TABLE 8.23

SIDELING CREEK AT LAKE KURWONGBAH

RUNOFF-ROUTING DESIGN FLOOD ESTIMATES OUTFLOWS

Average Recurrence Interval (Years)	Initial Loss (mm)	Continuing Loss (mm/hr)	Flood Volume (ML)	Time to Peak (hrs)	Peak Outflow (m^3/s)	Peak Lake Level (mAHD)
2	46	1	2 100	7.5	44	20.69
5	19	1	4 870	6.5	108	21.05
10	9	1	6 170	6.25	147	21.20
20	0	1	7 720	6.0	196	21.36
50	0	1	9 190	5.75	244	21.50
100	0	1	10 400	5.75	288	21.61
200	0	1	12 030	6.0	352	21.77
500	0	1	14 390	6.0	436	21.98
1 000	0	1	16 760	5.75	536	22.18
10 000	0	1	24 380	5.75	761	22.86
100 000	0	1	32 260	6.0	893	23.63
1 000 000 (PMF)	0	1	40 140	6.0	1 054	24.51

Table 8.22 indicates that the spillway is not capable of fully passing the six and nine hour duration peak discharge. It is estimated the main embankment will be overtopped by 0.43 and 0.13 metres respectively for PMF's of durations of this length.

Estimates of lake level and spillway discharge above an elevation of 23.3 m AHD and flow of 840 m^3/s are subject to large error bounds because these values signify the limit of the spillway rating curve for Lake Kurwongbah. The levels and flows quoted above these values are probably conservative because of the straight line extension assumed for the rating curve.

These results differ from the results obtained by John Wilson and Partners, (1989). The rainfall depth estimates and temporal patterns adopted in both studies were obtained from the Bureau of Meteorology. The recommended PMP rainfalls for durations of 2,3,6,12 and 18 hours differ however. The differences in the PMP rainfalls contribute a major part of the differences in the peak outflow estimates.

The type of runoff-routing model and the methods of estimating model parameters also contribute to the differences in peak outflow estimates. John Wilson and Partners utilised the Runoff Analysis and Flow Training System Model (RAFTS) to perform the runoff-routing simulation. The John Wilson and Partners report does not provide details of the adopted model parameters other than the rainfall loss rates. John Wilson and Partners have subsequently advised that the model parameters were derived from a regional relationship for catchments in South East Queensland.

The loss rates used by John Wilson and Partners were an initial loss of nil and a continuing loss of nil. Flood volumes for the John Wilson and Partners' study were therefore higher for the 1, 18 and 24 hour duration events, but were substantially less for the 2,3,6 and 12 hour events, reflecting the lower PMP rainfalls given John Wilson and Partners for these events.

The John Wilson and Partners report indicates that the original design flood estimate of PMF for Lake Kurwongbah in 1965 was derived from procedures in Australian Rainfall and Runoff, (1958). John Wilson and Partners subsequently advised that the Clark-Johnstone synthetic unitgraph method was the procedure that was used together with maximum possible rainfalls for this derivation. The design rainfall was a 12 hour rainfall of 622mm with a loss rate of 2.54mm/hour which resulted in a peak inflow of 999m^3/sec and a peak outflow of 769m^3/sec.

Because of the possible use of streamflow records of North Pine River, Lake Kurwongbah runoff-routing model parameters apportioned from the North Pine calibration model values have also been investigated. The North Pine River runoff-routing model parameters were factored by a ratio of distance to centroid of area in order to determine the Lake Kurwongbah values.

Model Parameters Proportioned According to North Pine River Parameters

A range of durations has again been investigated in the derivation of probable maximum flood estimates using North Pine River apportioned parameters.

The values of runoff-routing parameters are:

$$k = 17.2$$
$$m = 0.8$$

Results of the runoff-routing procedure are presented in Tables 8.24 and 8.25 and Figure 8.12.

TABLE 8.24

SIDELING CREEK AT LAKE KURWONGBAH

PMF ESTIMATES OF STORAGE INFLOWS

RUNOFF-ROUTING PARAMETERS DERIVED FROM NORTH PINE CATCHMENT

Duration		Flood Volume	Time to Peak	Peak Inflow
(Days)	(Hours)	(ML)	(hrs)	(m^3/s)
	1	16 710	0.75	1 390
	2	25 060	1.5	1 665
	3	30 040	2.25	1 729
	6	40 130	4.25	1 654
	9	45 490	6.0	1 320
	12	50 860	8.0	1 170
	18	58 960	8.5	969
1	24	67 050	11.0	866
2	48	96 780	14.0	639
3	72	119 180	54.0	553
4	96	134 730	72.0	548
5	120A	139 050	66.0	570
	B	139 050	96.0	522
6	144A	141 130	57.0	544
	B	141 140	100.0	502
	C	141 130	115.0	553
7	168A	163 300	25.0	650
	B	159 830	58.0	616
	C	159 800	151.0	543

TABLE 8.25

SIDELING CREEK AT LAKE KURWONGBAH

PMF ESTIMATES OF STORAGE OUTFLOWS

RUNOFF-ROUTING PARAMETERS DERIVED FROM NORTH PINE CATCHMENT

Duration (Days)	Duration (Hours)	Flood Volume (ML)	Time to Peak (hrs)	Peak Discharge (m^3/s)	Peak Lake Level (m AHD)
	1	16 590	3.83	430	21.97
	2	24 910	4.0	707	22.64
	3	28 840	4.75	798	23.07
	6	40 100	6.75	910	23.73
	9	45 450	9.5	909	23.72
	12	50 840	12.0	896	23.65
	18	58 960	16.5	829	23.26
1	24	67 040	19.0	771	22.92
2	48	96 750	28.0	625	22.40
3	72	119 180	54.0	550	22.21
4	96	134 730	73.0	539	22.19
5	120A	139 050	67.0	546	22.20
	B	139 050	96.0	521	22.15
6	144A	141 130	59.0	530	22.17
	B	141 140	100.0	502	22.12
	C	141 130	116.0	543	22.20
7	168A	163 300	27.0	636	22.42
	B	159 830	60.0	599	22.33
	C	159 800	151.0	543	22.20

Peak inflow into Lake Kurwongbah using these parameters is 1730 m^3/s for a storm duration of three hours. This compares with the John Wilson and Partners estimate of 1830 m^3/s, also for a three hour duration storm event.

Peak outflow from the storage of Lake Kurwongbah is estimated to be 910 m^3/s with an associated peak lake level of 23.73 m AHD. This is with a storm duration of six hours. These estimates compare favourably with the John Wilson and Partners study results of peak outflow, 884 m^3/s and peak lake level of 23.66 m AHD, from a corresponding storm duration of twelve hours.

The difference in critical storm durations for outflows is not regarded as significant since durations ranging from six to twelve hours in both studies result in peak flows of similar magnitude.

A complete flood frequency of estimated design flood outflows from Lake Kurwongbah is presented in Table 8.26. A six hour storm rainfall duration was adopted and Lake Kurwongbah was again assumed to be at Full Supply prior to the event. Rainfall losses were as assumed in the previous analyses.

TABLE 8.26

SIDELING CREEK AT LAKE KURWONGBAH

RUNOFF-ROUTING DESIGN FLOOD ESTIMATES OUTFLOWS

Average Recurrence Interval (Years)	Initial Loss (mm)	Continuing Loss (mm/hr)	Flood Volume (ML)	Time to Peak (hrs)	Peak Outflow (m^3/s)	Peak Lake Level (mAHD)
2	46	1	2 080	9.5	32	20.62
5	19	1	4 850	8.25	79	20.90
10	9	1	6 150	7.75	105	21.04
20	0	1	7 700	7.25	144	21.19
50	0	1	9 170	7.0	182	21.31
100	0	1	10 380	7.0	214	21.41
200	0	1	12 000	7.0	260	21.54
500	0	1	14 370	6.75	330	21.72
1 000	0	1	16 730	6.75	400	21.89
10 000	0	1	24 340	6.5	638	22.43
100 000	0	1	32 220	6.5	794	23.05
1 000 000 (PMF)	0	1	40 100	6.75	910	23.73

The effect of variation in the runoff-routing parameter has been illustrated in the preceding sections. The most obvious influence of the runoff-routing parameter on the Sideling Creek catchment is on the magnitude of the peak inflow into Lake Kurwongbah. The outflow from the spillway is reasonably similar regardless of the value of the runoff routing model parameter. The implication regarding the adequacy of the spillway is also similar. The spillway is capable of passing floods of the magnitude of the PMF. This finding supports the earlier work of John Wilson and Partners in regard to this matter.

SUMMARY

The value of the runoff-routing k parameter recommended for the catchment of Lake Kurwongbah is 8.3. This value is regarded as being appropriate for the following reasons:

- It is close to values determined by regional relationships for catchments of this size.
- The catchment appears to be more like the South Pine River catchment than the North Pine River in relation to flood runoff characteristics.

Using the above k value, a PMF peak outflow of 1054 m^3/s is estimated. It is estimated an event of this magnitude would overtop the main embankment by a depth of approximately 0.43 metres.

The magnitude of the flood event which when routed through the storage, with the existing spillway, just threatens failure of the dam, is estimated to be 90% of the Probable Maximum Flood. This flood is known as the Imminent Failure Flood. The ARI of the Imminent Failure Flood is estimated as approximately 730 000 years.

These results cannot be regarded as highly reliable, given:

(i) the lack of data on flood runoff from the catchment.

(ii) the extrapolation of the spillway rating curve and storage capacity curve of Lake Kurwongbah.

For more reliable results, continuous records of water levels in Lake Kurwongbah would be necessary. A water level recorder for Lake Kurwongbah has been recommended as part of the proposed Alert Network for the Pine River System. The spillway rating curve requires further definition at its upper end, although this would only be feasible if more physical modelling of the spillway were possible.

8.5 RAINFALL SPATIAL DISTRIBUTION EFFECTS

The effect of the spatial distribution of rainfall has been investigated for probable maximum precipitation events of storms centred over the North Pine Dam catchment. This effect has been investigated on the recommendation of the Bureau of Meteorology (Refer report concerning PMP estimates). Two spatial distribution patterns have been utilised; one for short duration storms (less than 6 hours) and the other for Generalised Tropical Storms (GSTM) for durations of 12 hours and more.

A number of storm pattern orientations have been considered to determine the orientation that produce the most severe flooding for a given duration. The adopted critical storm orientations are presented in Figures 8.13 and 8.14. It should be noted that isohyetal labels vary for each storm duration so as to ensure the mean catchment rainfall equates to the Bureau's estimated PMP depth for that particular duration. Values of isohyetal labels for a range of durations from one hour to seven days are presented in Table 8.27, for the critical pattern orientation.

TABLE 8.27

ISOHYETAL LABELS

DURATION (hrs)	ISOHYET LABEL (mm)									
	A	B	C	D	E	F	G	H	I	J
1	670	510	360	270	220	140	100	50	10	10
2	890	720	550	430	350	230	160	90	20	30
3	1020	830	660	520	430	300	210	130	40	30
6	1220	1020	830	690	570	410	310	210	90	40
9	880	830	750	660	610	550	500	440		
12	1020	960	860	770	700	640	580	510		
18	1250	1170	1050	940	860	780	700	630		
24	1460	1370	1240	1100	1010	920	820	730		
48	2170	2040	1830	1630	1490	1360	1220	1090		
72	2710	2550	2290	2040	1870	1700	1530	1360		
96	3140	2950	2650	2360	2160	1970	1770	1570		
120	3260	3050	2750	2440	2240	2040	1830	1630		
144	3350	3140	2820	2510	2300	2090	1880	1670		
168	3690	3460	3110	2770	2530	2300	2070	1840		

(Labels for spatial distribution pattern orientation 2).

Comparisons between estimates of peak inflows to North Pine Dam for various spatial distribution orientations are presented in Table 8.28. Durations of up to 72 hours are shown since it is apparent the estimates are similar for longer durations. This table clearly illustrates the effect of including spatial variability into the estimation process. Peak inflow estimates are increased significantly for small duration events with the adoption of a spatial distribution pattern orientated so that the storm is centred over the North Pine Dam whilst the pattern encompasses the whole of the Pine River catchment. (As seen in Figure 8.13). It is apparent that with this pattern higher rainfall depths associated with the centre of the storm are located closer to the reservoir of the dam, resulting in less routing of this flow through the catchment.

TABLE 8.28

COMPARISON BETWEEN PEAK INFLOWS TO NORTH PINE DAM

FOR VARIOUS RAINFALL SPATIAL DISTRIBUTION PATTERNS

DURATION (hrs)	MEAN CATCHMENT RAINFALL (mm)	SPATIAL DISTRIBUTION PATTERN		
		UNIFORM	PATTERN ORIENTATION 1	PATTERN ORIENTATION 2
1	240	5365	6063	6959
2	370	6144	7044	7963
3	450	6038	7023	7776
6	590	5553	6434	6846
12	810	5218	5218	5243
18	990	5015	5034	5015
24	1160	4659	4659	4655
48	1720	3757	3752	3751
72	2150	3302	3302	3304

Notes:

(a) Uniform spatial distribution of rainfall indicates the mean catchment rainfall is applied to all of the catchment.

(b) Spatial distribution pattern orientation 1 has the storm centred over the North Pine Dam catchment with the storm pattern encompassing only this catchment.

(c) Spatial distribution pattern orientation 2 also has the storm centred over the North Pine Dam catchment, however, the storm pattern is positioned so as to encompass the whole Pine River catchment.It is also evident that isohyetal patterns of the Generalised Tropical Storm Method do not increase peak discharges significantly. The likely cause for this is that the isohyetal labels are far less variable across a storm pattern when compared to the short duration patterns.

A conclusion can be drawn from these results. It would appear that spatial variability of rainfall is significant for short duration events (less than six hours) but not for durations associated with the GSTM for the North Pine Dam catchments.

Flood volumes remain unaffected as do times to peak discharge with the inclusion of spatial distribution of rainfall.

8.6 CONCURRENT RAINFALL ESTIMATES

Estimates of rainfalls that occur in catchments adjacent to catchments receiving PMP events are required for hydraulic modelling of various dambreak scenarios.

The Bureau of Meteorology have indicated that no definitive method exists for such a determination and as a consequence they have suggested four different methods which may be utilised. The Bureau recommends sensitivity testing of these methods. The four methods of determining concurrent rainfall with GSTM PMP estimates are as follows:

1. The GSTM elliptical distribution is expanded to cover the combined catchment and a scaling used to maintain the PMP depth over the required catchments.

2. The major storms in the area adjacent to the main catchment area are used.

3. The areally adjusted 1:100 year Intensity-Frequency-Duration (IFD) depth and spatial distribution is used over the adjacent area.

4. PMP is calculated for the main catchment and the combined catchment and the difference is distributed as concurrent rainfall.

The joint probabilities associated with these methods vary considerably. They range from method 4 with the lowest joint probability to methods 2 and 3 which have a higher joint probability. The four methods are appropriate for calculating concurrent rainfalls for durations out to 7 days, as the Gordon Method extends the GSTM to this duration.

Method 2 has not been investigated in detail. In this method the spatial distribution of rainfall is obtained from moisture maximised major recorded storms in the area adjacent to the main catchment. Temporal distribution over the adjacent area of the rainfall in these storms is derived from basic data. Only two major storms have been considered during calibration of the runoff-routing model of South Pine River, so relatively little basic data is available to form the basis of this method. Both of these major storms had rainfall durations of between 60 and 90 hours and as a consequence were not considered appropriate for more critical shorter duration events.

The other methods have been utilised to derive estimates of peak flow and flood volume for the catchments of South Pine River and Sideling Creek associated with a PMP event occurring over the North Pine Dam catchment. Tables 8.29, 8.30 and 8.31 provide a summary of the various estimates of concurrent rainfall and peak flow and flood volumes for each of the three methods considered.

Method 4 provides the most extreme estimates for concurrent rainfall for GSTM duration storms. This method also has the lowest joint probability of occurrence. Method 3 provides the lowest flood magnitudes for adjacent catchment estimates for all durations.

TABLE 8.29

CONCURRENT RAINFALL AND FLOW ESTIMATES

PMF NORTH PINE CATCHMENT, ELLIPTICAL DISTRIBUTION ADJACENT CATCHMENTS

METHOD 1

DURATION (hrs)	NORTH PINE DAM		
	MEAN CATCHMENT RAINFALL (mm)	FLOOD VOLUME (ML)	PEAK INFLOW (m^3/s)
1	240	78670	6959
2	370	124910	7963
3	450	152840	7776
6	590	199400	6846
12	810	274830	5243
18	990	334740	5015
24	1160	392120	4655
48	1720	576060	3751
72	2150	716990	3304

DURATION (hrs)	SOUTH PINE RIVER		
	MEAN CATCHMENT RAINFALL (mm)	FLOOD VOLUME (ML)	PEAK FLOW (m^3/s)
1	109	18920	609
2	179	31060	1103
3	232	40950	1475
6	331	58020	1981
12	642	112680	2607
18	784	136910	2255
24	921	160250	2059
48	1363	235030	1540
72	1707	292250	1358

DURATION (hrs)	SIDELING CREEK		
	MEAN CATCHMENT RAINFALL (mm)	FLOOD VOLUME (ML)	PEAK OUTFLOW (m^3/s)
1	124	6410	173
2	201	10420	336
3	261	13510	462
6	367	18940	610
12	677	34900	735
18	827	42490	658
24	927	49820	618
48	1437	72960	476
72	1800	90770	420

South Pine River at Cash's Crossing.
Sideling Creek at Lake Kurwongbah.

TABLE 8.30

CONCURRENT RAINFALL AND FLOW ESTIMATES

PMF NORTH PINE CATCHMENT, 1% AEP EVENT ADJACENT CATCHMENTS

METHOD 3

DURATION (hrs)	NORTH PINE DAM		
	MEAN CATCHMENT RAINFALL (mm)	FLOOD VOLUME (ML)	PEAK INFLOW (m^3/s)
1	240	78340	6063
2	370	124970	7044
3	450	153420	7023
6	590	202610	6434
12	810	275130	5218
18	990	335600	5034
24	1160	391870	4659
48	1720	576630	3752
72	2150	716900	3302

DURATION (hrs)	SOUTH PINE RIVER		
	MEAN CATCHMENT RAINFALL (mm)	FLOOD VOLUME (ML)	PEAK FLOW (m^3/s)
1	88	15170	438
2	120	20650	635
3	152	26510	832
6	204	35340	1069
12	274	46820	1037
18	322	54340	874
24	370	61840	778
48	504	81510	535
72	576	90100	423

DURATION (hrs)	SIDELING CREEK		
	MEAN CATCHMENT RAINFALL (mm)	FLOOD VOLUME (ML)	PEAK OUTFLOW (m^3/s)
1	96	4970	124
2	125	6440	176
3	153	7880	227
6	197	10040	280
12	256	12820	254
18	304	15220	226
24	351	17190	200
48	480	22700	147
72	557	25490	119

South Pine River at Cash's Crossing.
Sideling Creek at Lake Kurwongbah.

TABLE 8.31

CONCURRENT RAINFALL AND FLOW ESTIMATES

PMF EVENTS FOR NORTH PINE AND TOTAL CATCHMENT

METHOD 4

DURATION (hrs)	NORTH PINE DAM		
	MEAN CATCHMENT RAINFALL (mm)	FLOOD VOLUME (ML)	PEAK INFLOW (m^3 /s)
1	240	78670	6959
2	370	124910	7963
3	450	152840	7776
6	590	199400	6846
12	810	274830	5243
18	990	334740	5015
24	1160	392120	4655
48	1720	576060	3751
72	2150	716900	3304

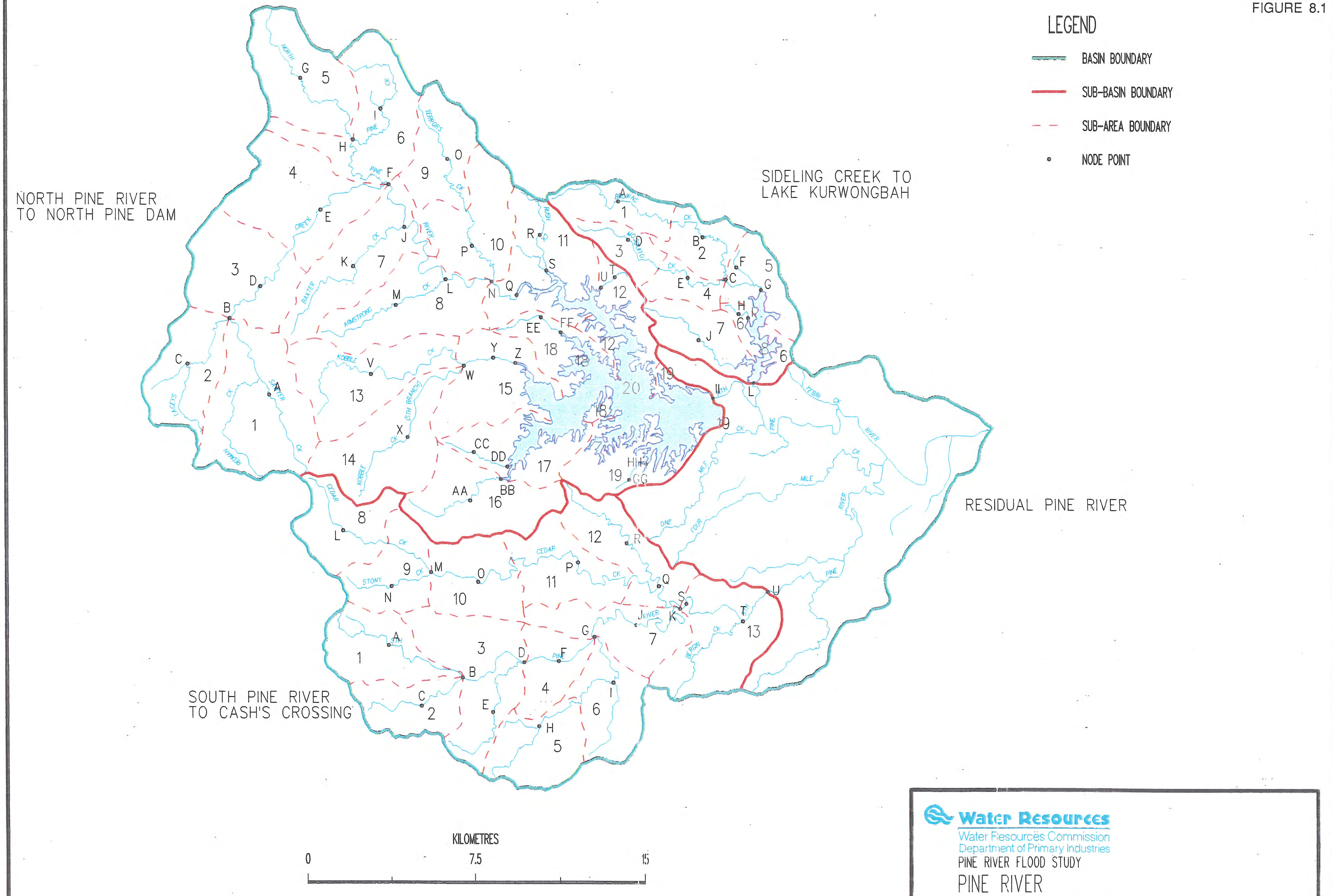
DURATION (hrs)	SOUTH PINE RIVER		
	MEAN CATCHMENT RAINFALL (mm)	FLOOD VOLUME (ML)	PEAK FLOW (m^3 /s)
1	109	18920	604
2	179	31060	1103
3	232	40950	1475
6	331	58020	1981
12	692	121640	2836
18	848	148420	2454
24	997	173990	2240
48	1470	254150	1670
72	1839	315900	1468

DURATION (hrs)	SIDELING CREEK		
	MEAN CATCHMENT RAINFALL (mm)	FLOOD VOLUME (ML)	PEAK OUTFLOW (m^3 /s)
1	124	6410	173
2	201	10420	336
3	261	13510	462
6	367	18940	610
12	687	35470	744
18	841	43250	669
24	989	50710	628
48	1460	74210	484
72	1829	92300	428

South Pine River at Cash's Crossing.
Sideling Creek at Lake Kurwongbah.

Proposed dambreak analyses are concerned with estimating the most severe incremental effect of flooding resulting from a possible failure of the dam under investigation. Because of this, it is believed that estimates of concurrent rainfall based upon the method that provides the lowest flood magnitude for adjacent catchments is more appropriate for this particular study. As a consequence, Method 3 which distributes areally adjusted 1 in 100 year IFD rainfall depths over adjacent catchments will be adopted for the dambreak studies.

FIGURE 8.1



Water Resources
Water Resources Commission
Department of Primary Industries
PINE RIVER FLOOD STUDY

PINE RIVER
DESIGN MODEL (INCORP. NTH PINE DAM)

A3-98893

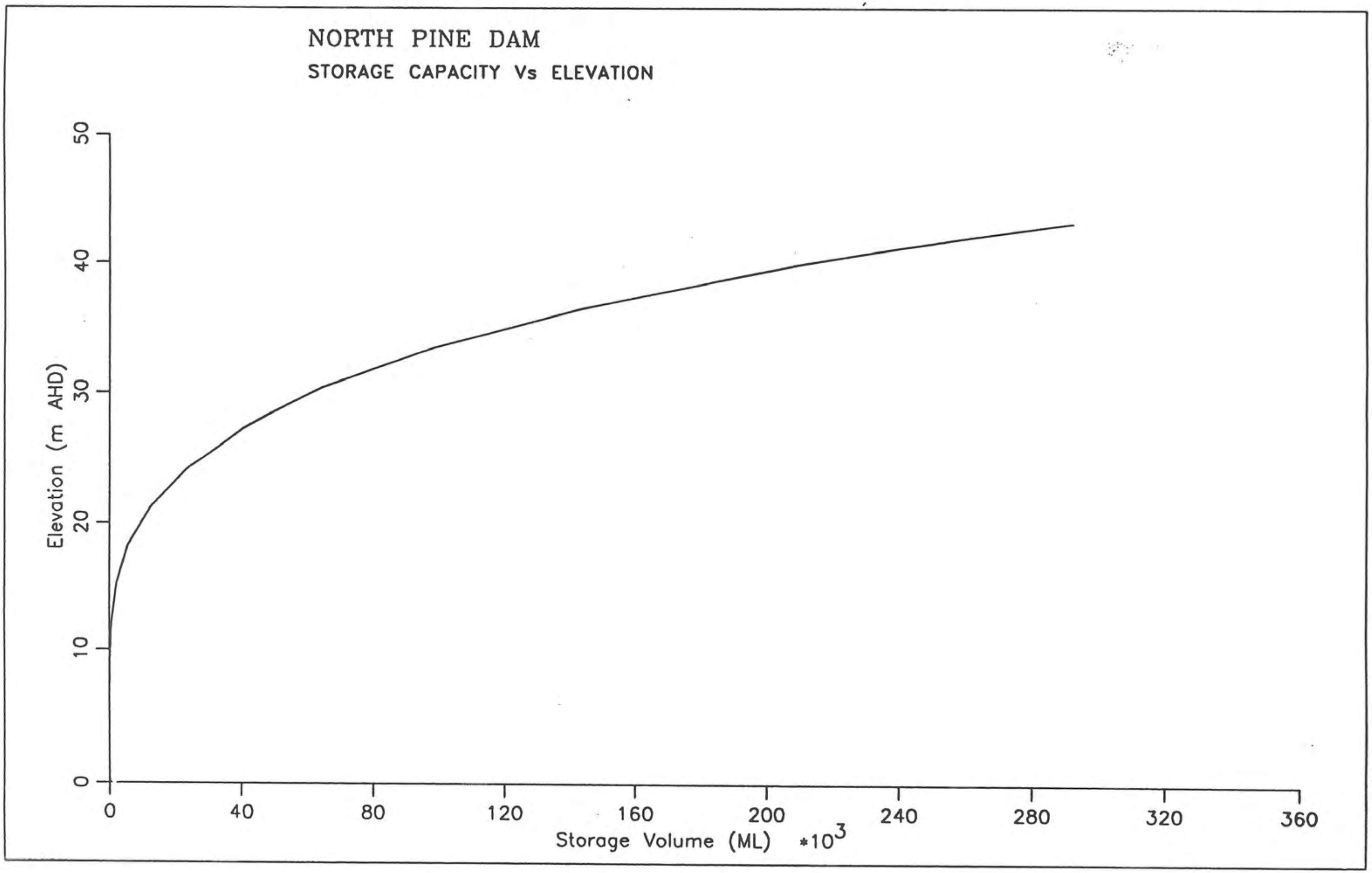

NORTH PINE DAM
STORAGE CAPACITY Vs ELEVATION
Elevation (m AHD)
0
10
20
30
40
50
0
40
80
120
160
200
240
280
320
360
Storage Volume (ML) *10^3

FIGURE 8.2

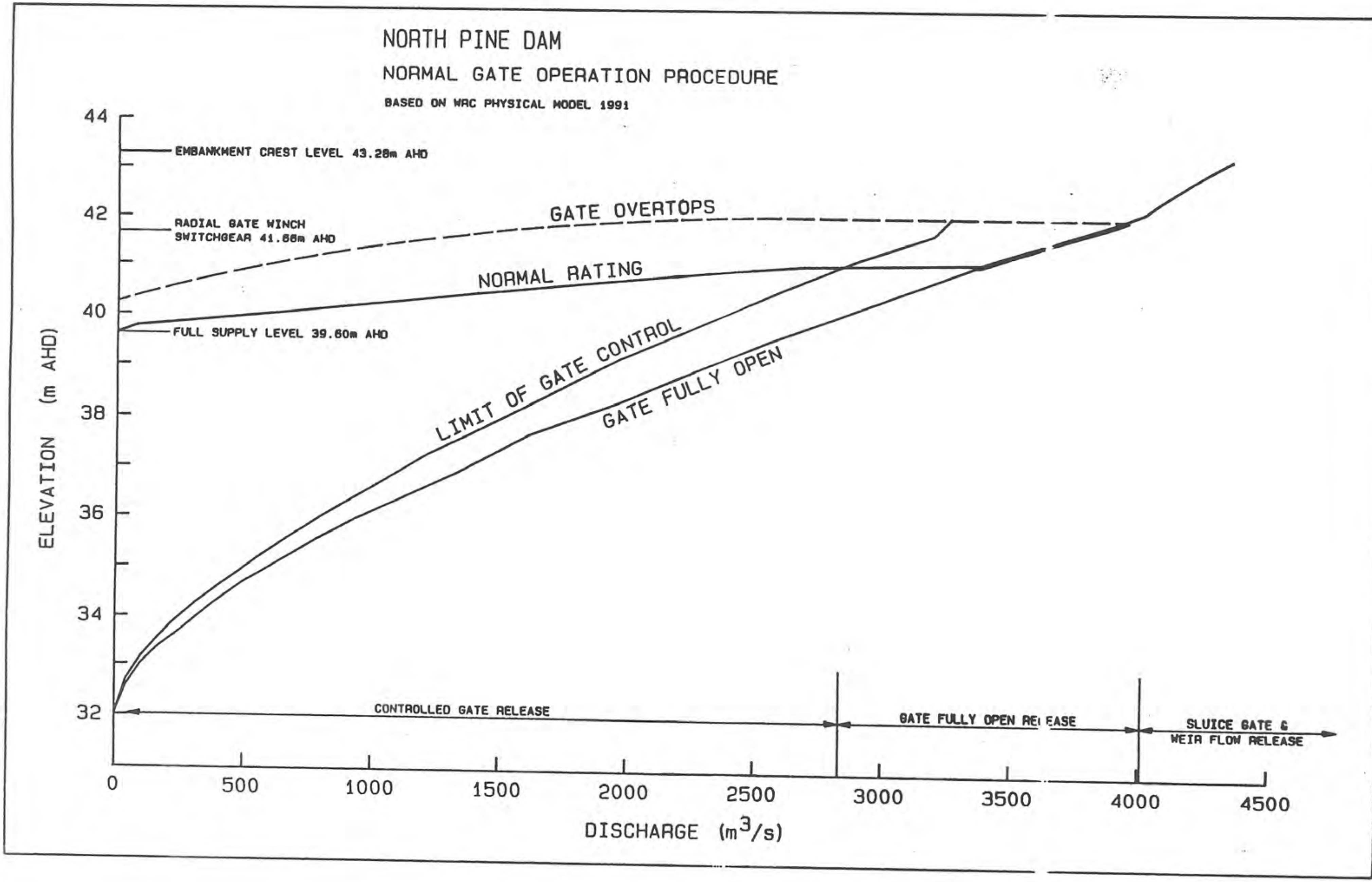

NORTH PINE DAM
NORMAL GATE OPERATION PROCEDURE
BASED ON WRC PHYSICAL MODEL 1991
EMBANKMENT CREST LEVEL 43.28m AHD
RADIAL GATE WINCH
SWITCHGEAR 41.66m AHD
FULL SUPPLY LEVEL 39.60m AHD
GATE OVERTOPS
NORMAL RATING
LIMIT OF GATE CONTROL
GATE FULLY OPEN
CONTROLLED GATE RELEASE
GATE FULLY OPEN RELEASE
SLUICE GATE &
WEIR FLOW RELEASE
ELEVATION (m AHD)
DISCHARGE (m^3/s)
44
42
40
38
36
34
32
0
500
1000
1500
2000
2500
3000
3500
4000
4500

FIGURE 8.3

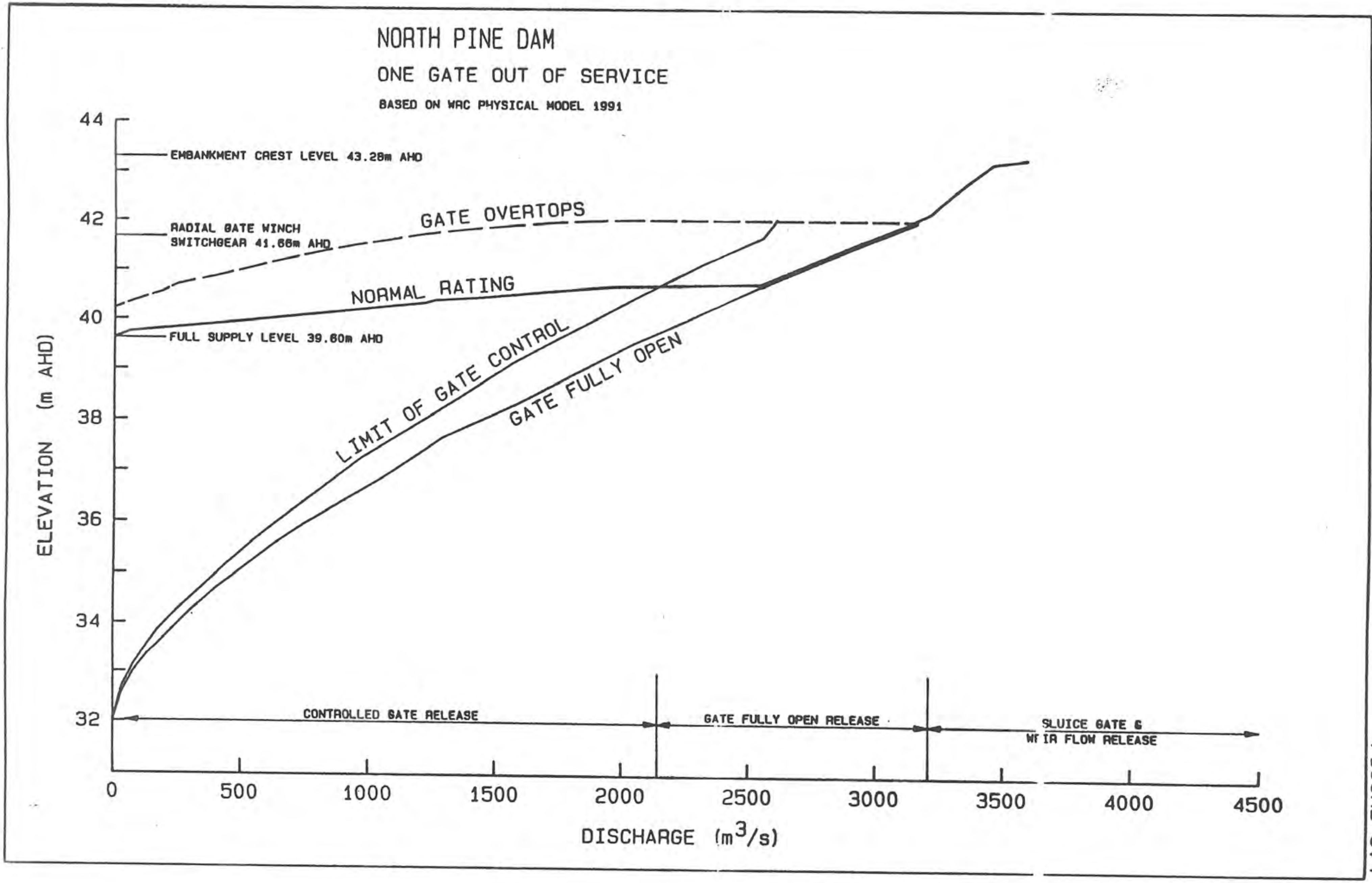
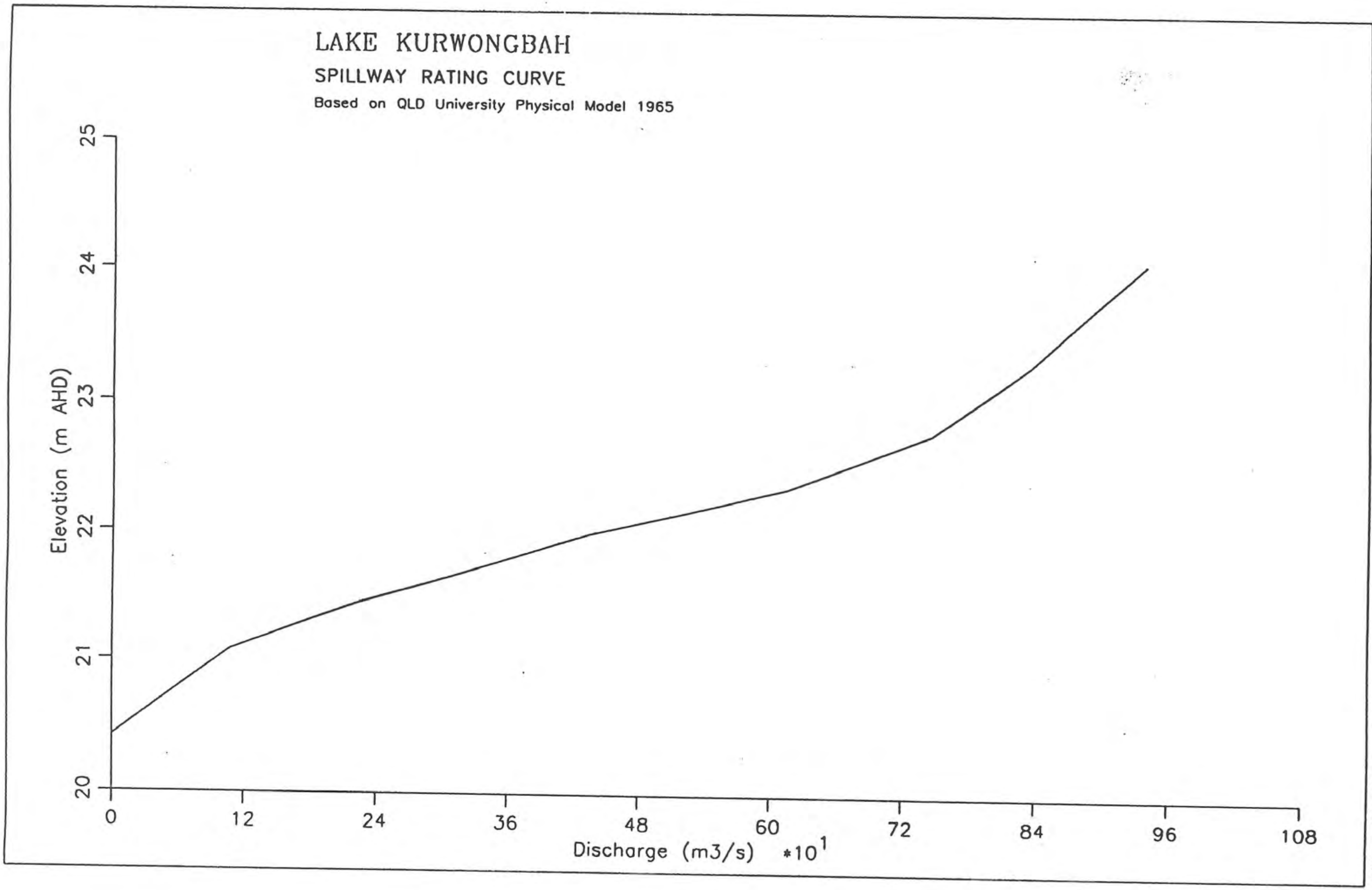

NORTH PINE DAM
ONE GATE OUT OF SERVICE
BASED ON WRC PHYSICAL MODEL 1991
EMBANKMENT CREST LEVEL 43.28m AHD
RADIAL GATE WINCH
SWITCHGEAR 41.66m AHD
GATE OVERTOPS
NORMAL RATING
FULL SUPPLY LEVEL 39.60m AHD
LIMIT OF GATE CONTROL
GATE FULLY OPEN
CONTROLLED GATE RELEASE
GATE FULLY OPEN RELEASE
SLUICE GATE &
WEIR FLOW RELEASE
ELEVATION (m AHD)
44
42
40
38
36
34
32
0
500
1000
1500
2000
2500
3000
3500
4000
4500
DISCHARGE (m3/s)

FIGURE 8.4



LAKE KURWONGBAH
SPILLWAY RATING CURVE
Based on QLD University Physical Model 1965
Elevation (m AHD)
20
21
22
23
24
25
0
12
24
36
48
60
72
84
96
108
Discharge (m3/s) *10[1]

FIGURE 8.6

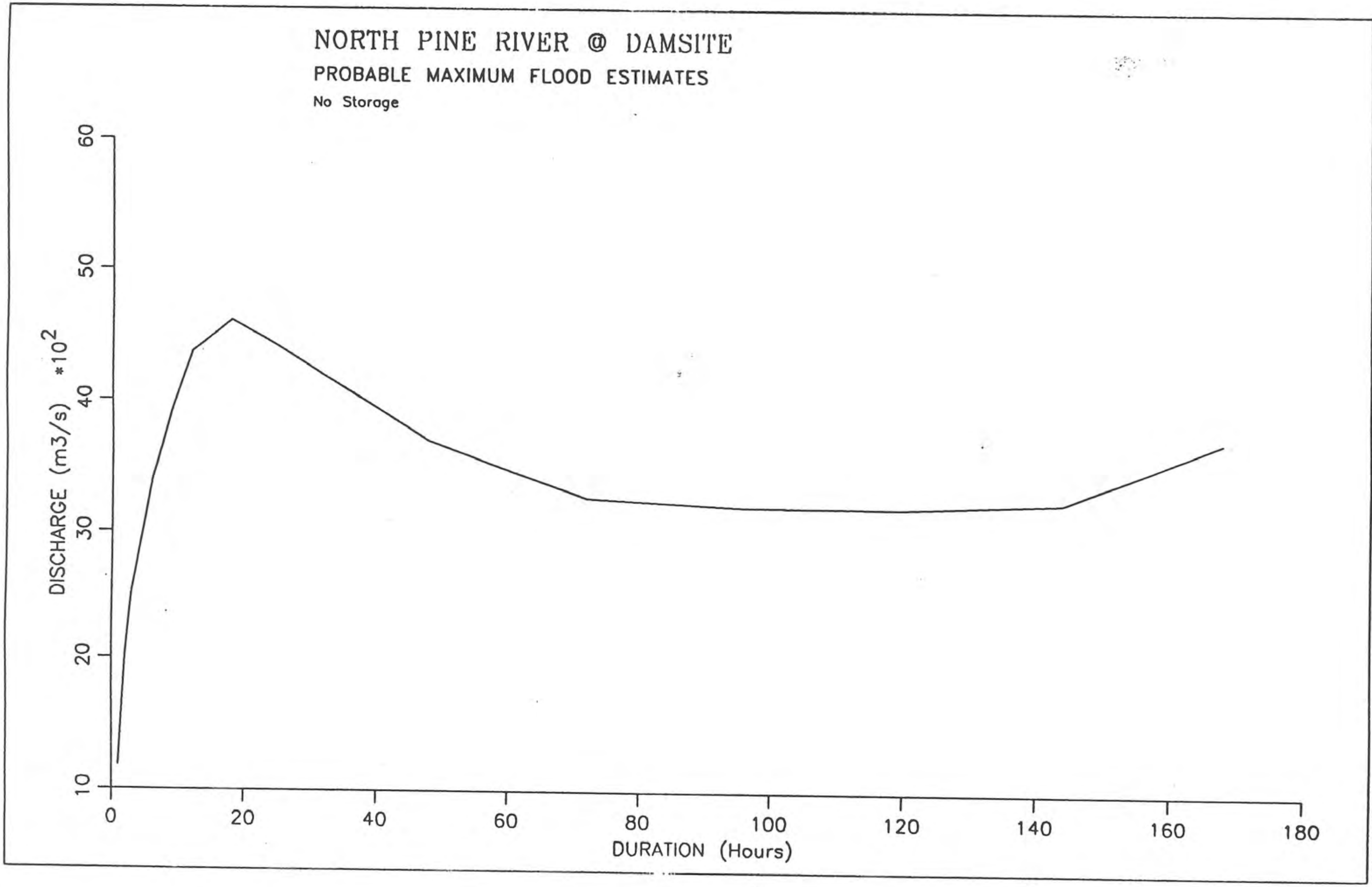

NORTH PINE RIVER @ DAMSITE
PROBABLE MAXIMUM FLOOD ESTIMATES
No Storage
DISCHARGE (m3/s) *10²
60
50
40
30
20
10
0
20
40
60
80
100
120
140
160
180
DURATION (Hours)

FIGURE 8.7

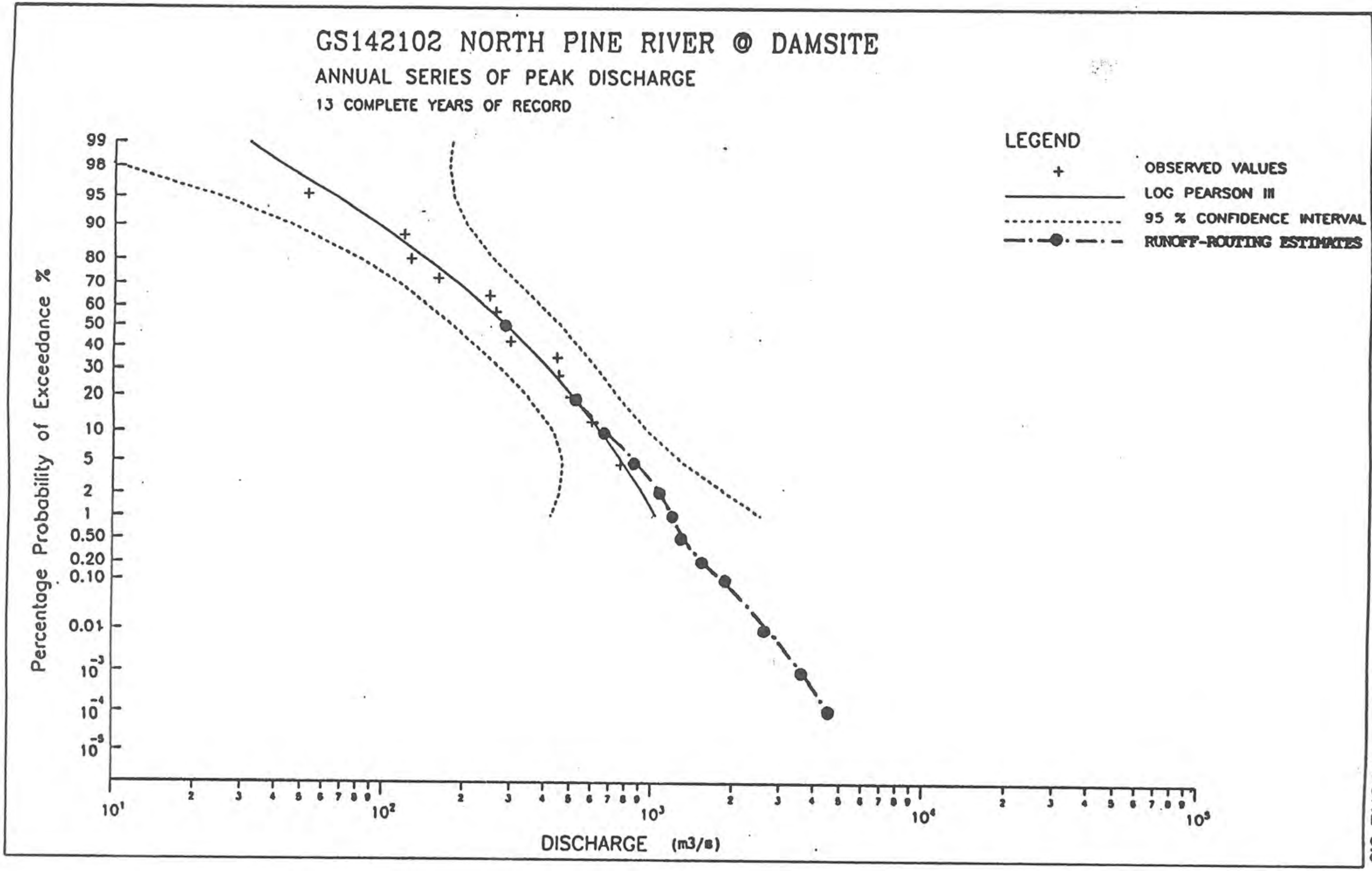
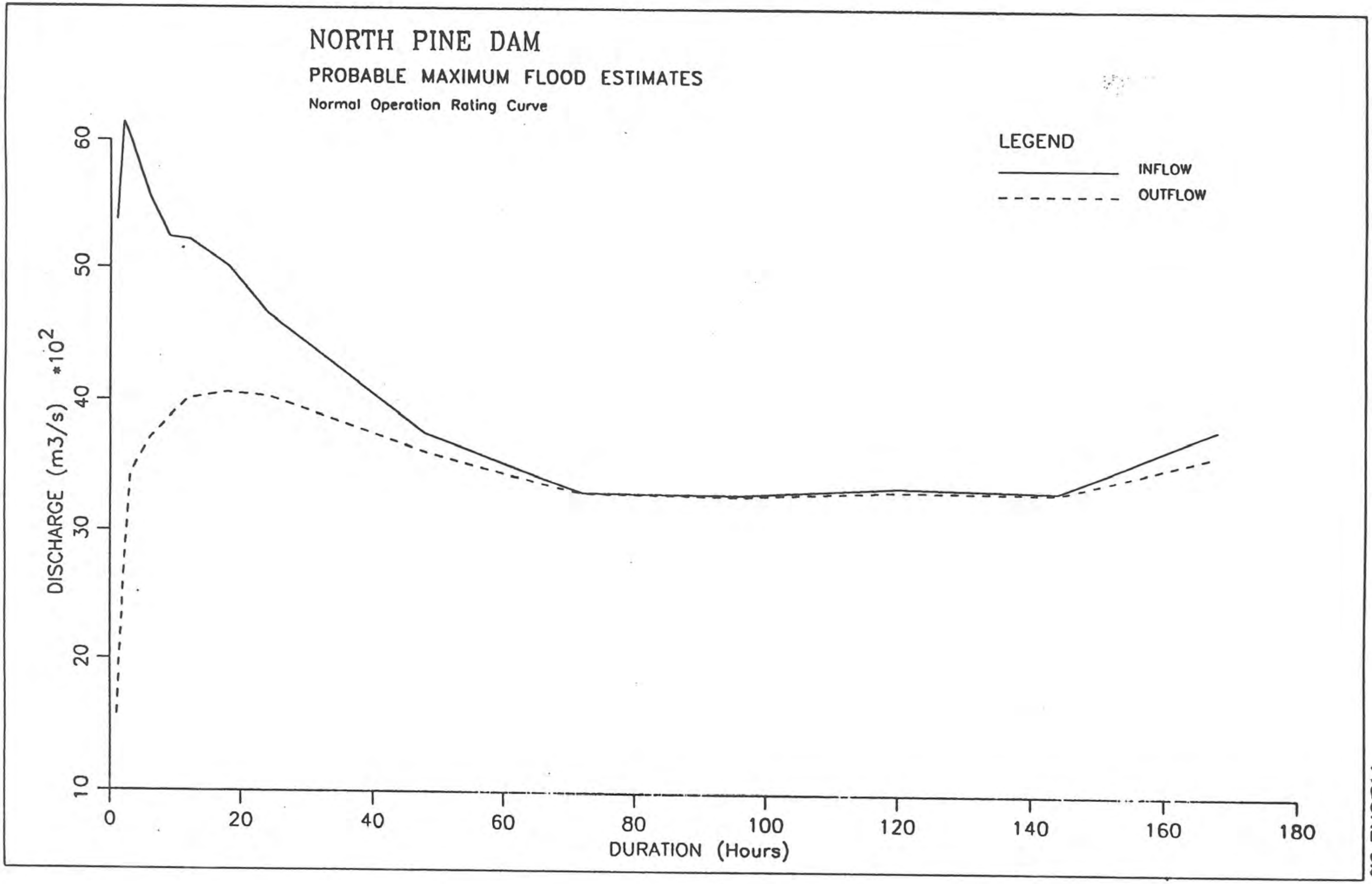

GS142102 NORTH PINE RIVER @ DAMSITE
ANNUAL SERIES OF PEAK DISCHARGE
13 COMPLETE YEARS OF RECORD
LEGEND
OBSERVED VALUES
LOG PEARSON III
95 % CONFIDENCE INTERVAL
RUNOFF-ROUTING ESTIMATES
Percentage Probability of Exceedance %
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.50
0.20
0.10
0.01
10^{-3}
10^{-4}
10^{-5}
10^1
10^2
10^3
10^4
10^5
DISCHARGE (m3/s)

FIGURE 8.8



NORTH PINE DAM
PROBABLE MAXIMUM FLOOD ESTIMATES
Normal Operation Rating Curve
LEGEND
INFLOW
OUTFLOW
DISCHARGE (m3/s) *10^2
60
50
40
30
20
10
0
20
40
60
80
100
120
140
160
180
DURATION (Hours)

FIGURE 8.9

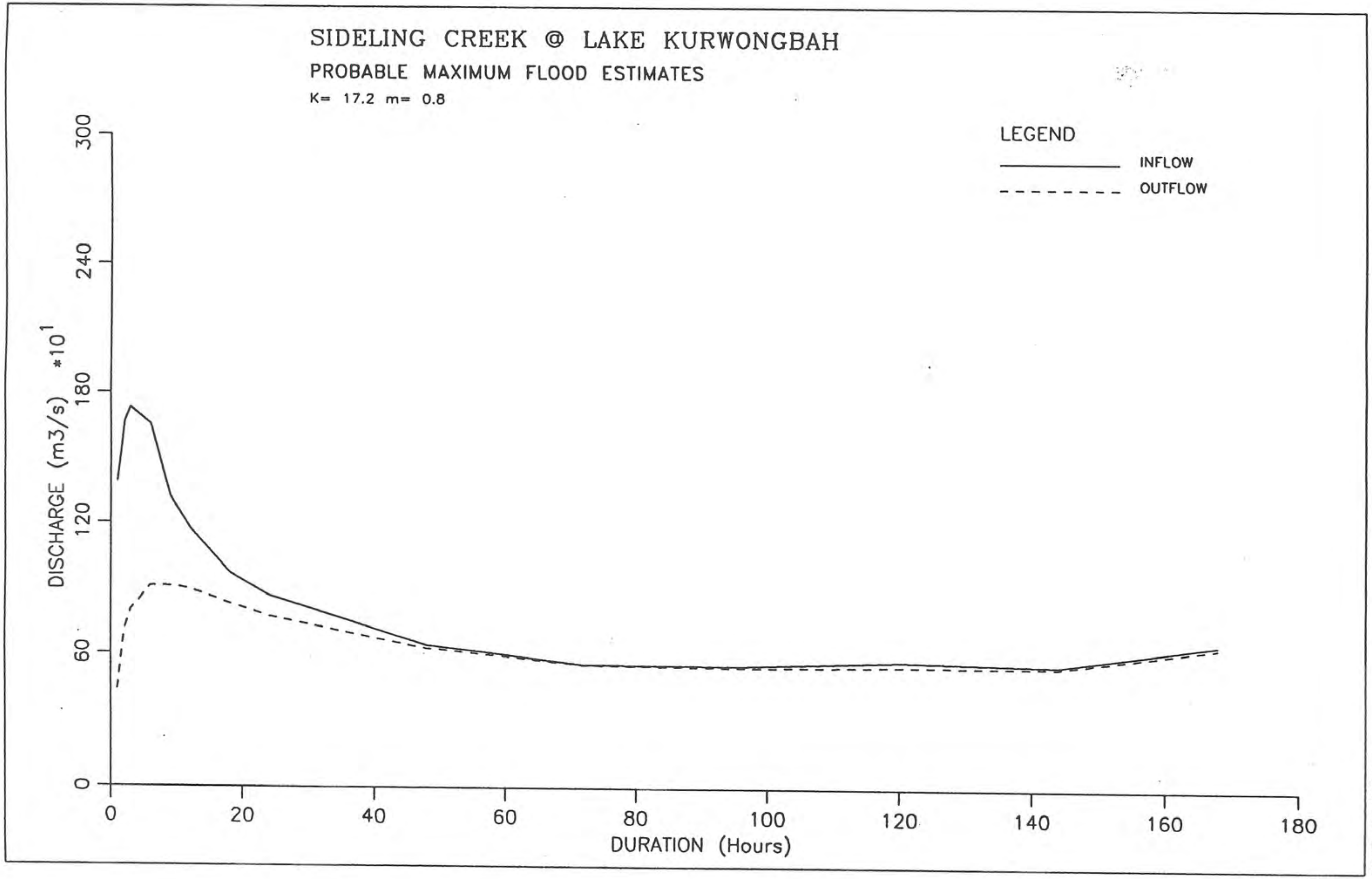
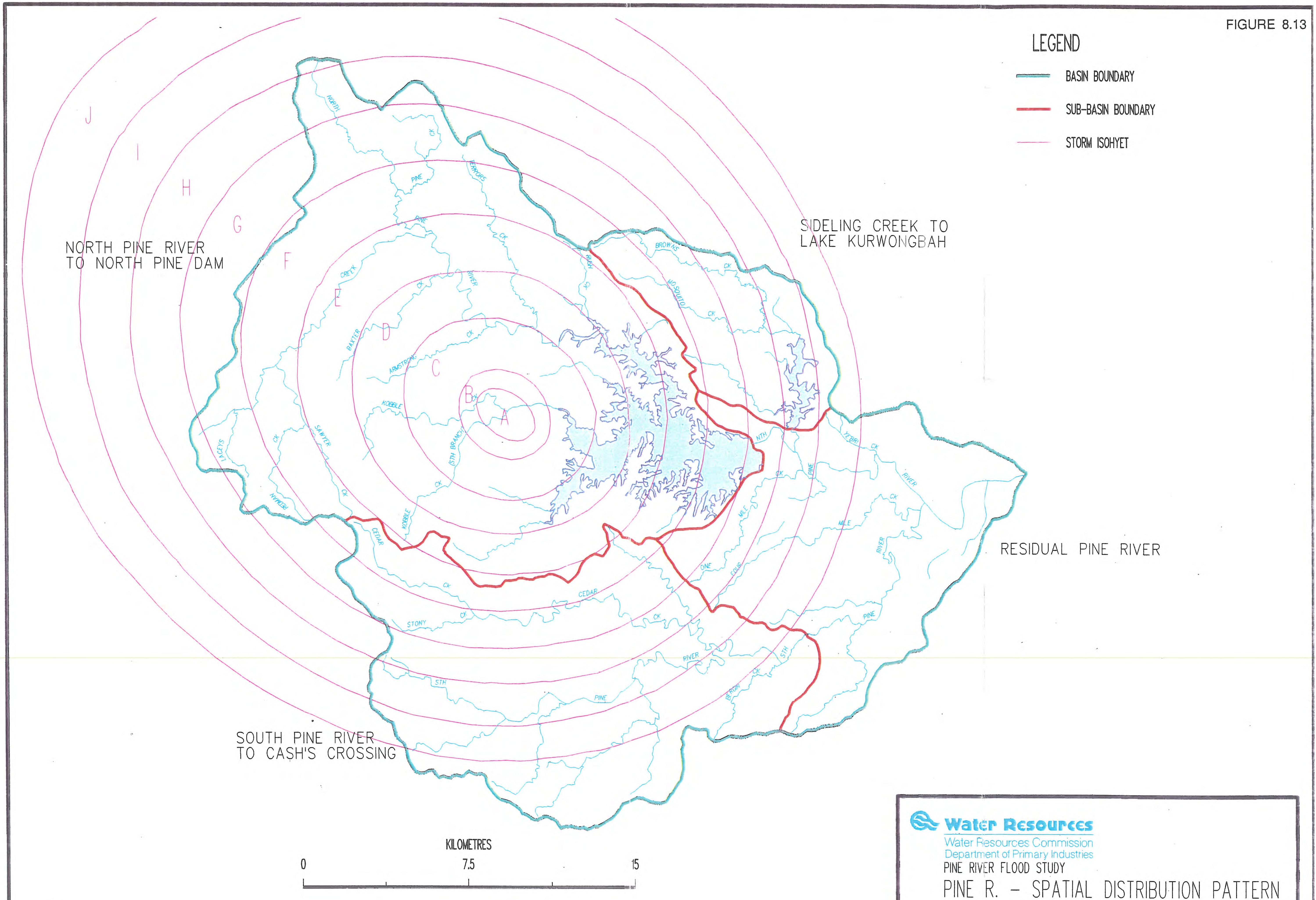

SIDELING CREEK @ LAKE KURWONGBAH
PROBABLE MAXIMUM FLOOD ESTIMATES
K= 17.2 m= 0.8
LEGEND
INFLOW
OUTFLOW
DISCHARGE (m3/s) *10^1
300
240
180
120
60
0
0
20
40
60
80
100
120
140
160
180
DURATION (Hours)

FIGURE 8.12

FIGURE 8.13



Water Resources
Water Resources Commission
Department of Primary Industries
PINE RIVER FLOOD STUDY
PINE R. - SPATIAL DISTRIBUTION PATTERN
SHORT DURATION STORM

A3-98894

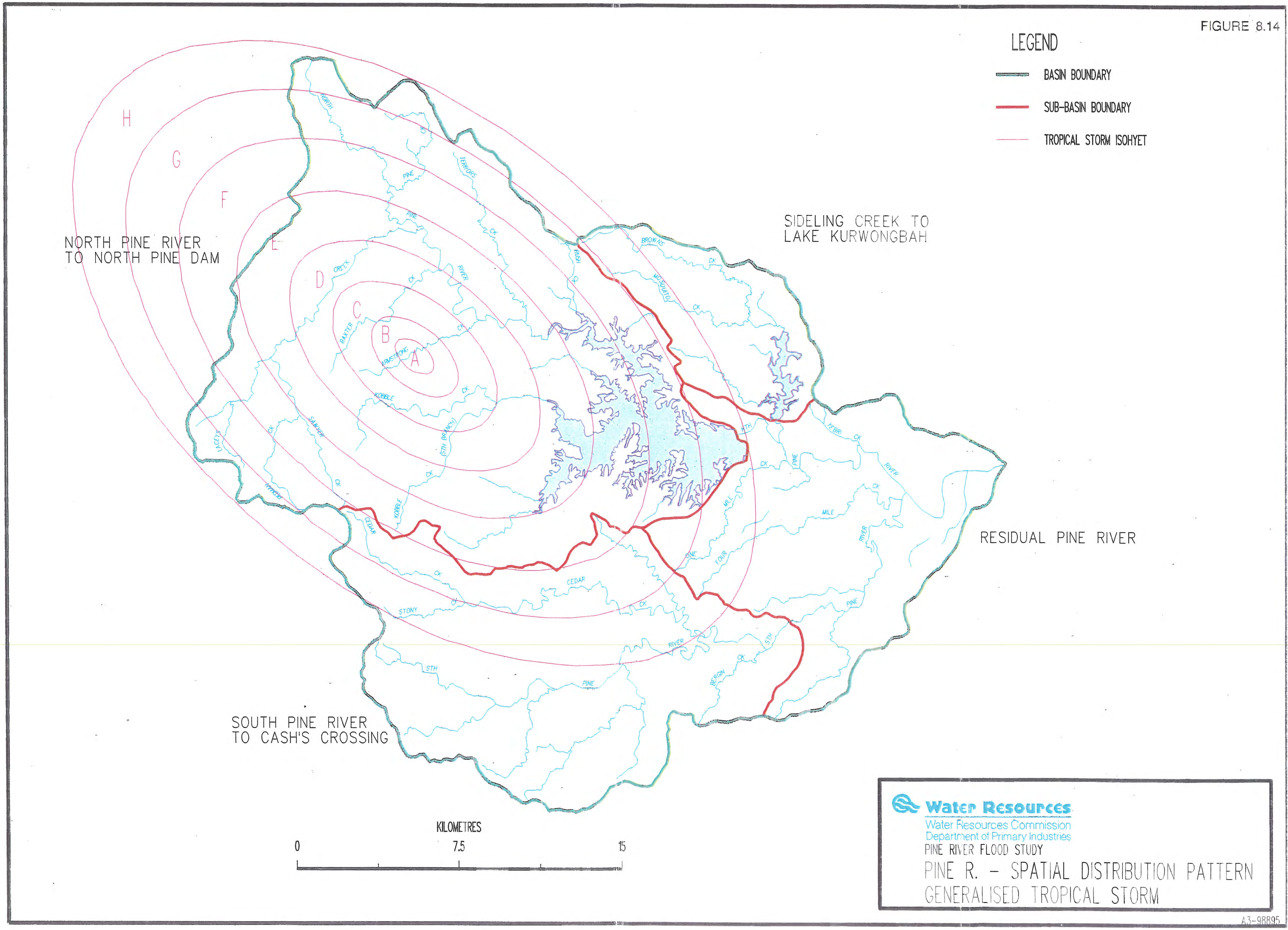


FIGURE 8.14
LEGEND
BASIN BOUNDARY
SUB-BASIN BOUNDARY
TROPICAL STORM ISOHYET
A
B
C
D
E
F
G
H
NORTH PINE RIVER TO NORTH PINE DAM
SIDELING CREEK TO LAKE KURWONGBAH
RESIDUAL PINE RIVER
SOUTH PINE RIVER TO CASH'S CROSSING
KILOMETRES
0
7.5
15
Water Resources
Water Resources Commission
Department of Primary Industries
PINE RIVER FLOOD STUDY
PINE R. - SPATIAL DISTRIBUTION PATTERN
GENERALISED TROPICAL STORM
A3-98895

9.0 HISTORICAL FLOOD ESTIMATES

9.1 INTRODUCTION

Estimates of historical flood hydrographs are required at North Pine Dam, Lake Kurwongbah and South Pine River at Cash's Crossing for inclusion in the calibration of a numerical hydraulic model of the lower North and South Pine Rivers.

The flood events of June 1967, January 1974 and April 1989 have been selected, because these events are some of the largest flood events on record and a substantial amount of flood height data is available for these particular events.

9.2 JUNE 1967

The flood that occurred in early June 1967 was moderate in peak discharge magnitude terms for both North Pine River and South Pine River. Indeed this particular event was not the largest annual flood recorded on North Pine River for the climatic year of 1966-67.

The June 1967 event consisted of two peaks, the first occurring between 4:00 pm and 8:00 pm on the 10th and the second and larger occurring early on the 12th. The recorded peak discharge of 390 m^3/s at the damsite on North Pine River has an average recurrence interval (ARI) of about 3 years (Pre-North Pine Dam). The peak discharge recorded at Draper's Crossing on South Pine River was 470 m^3/s which is equivalent to an ARI of around a 6 years.

Rainfall associated with this event commenced at around 8:00 am on the 9th June and continued for some 66 hours. During this time, the catchment above the damsite on North Pine River received about 200 mm of rainfall. The catchment above Draper's Crossing received 250 mm in the corresponding period.

Since this event occurred prior to the construction of North Pine Dam, the hydrograph recorded at the damsite can be used directly as an input into the hydraulic model. The hydrograph for this event at the damsite is presented in Figure 9.1.

No storage records are available for Lake Kurwongbah for this event so an assumption that the storage was at Full Supply prior to the flood has been made. This assumption appears reasonable given the occurrence of a large flood in the Pine River catchment some 3 months previous in March 1967.

The estimated inflow and outflow hydrographs for Lake Kurwongbah are presented in Figure 9.2.

Assumed rainfall losses for Lake Kurwongbah are equal to those adopted during the calibration of the North Pine catchment model. Likewise the estimated hydrograph for South Pine River at Cash's Crossing is presented in Figure 9.3.

9.3 JANUARY 1974

The Australia Day flood of January 1974 is the largest flood on record for both North Pine River and South Pine River. Unfortunately, there is little useful streamflow data available on North Pine River due to vandal damage of the stream gauge. An estimate of a double peaked inflow hydrograph into North Pine Dam, (which was partially constructed at the time), was made by the Co-ordinator General's Department (COG). This estimate put the first peak discharge at some 2350 m^3/s and the second smaller peak at 1950 m^3/s.

A provisional rating curve was also derived for discharge across the incomplete spillway monoliths and an outflow hydrograph was estimated from recorded storage levels. The COG estimated peak outflow across the incomplete dam was 1210 m^3/s. The Premier's Department has been approached in regard to obtaining copies of the provisional rating curve and recorded storage levels, so that the estimated hydrographs could be reconstructed. Unfortunately, this information was unable to be traced.

A copy of the estimated inflow and outflow hydrographs derived by the COG is available in the Cameron McNamara Report (1978). The Brisbane City Council were able to advise the extent of construction of the spillway monoliths prior to the flood.

A rating curve based upon broad-crested weir relationships was derived using the monolith levels provided by the BCC (the rating curve is shown in Figure 9.4). The estimated COG inflow hydrograph was then routed through a reservoir routing program called 'ROUT' (Ryan, 1990) which incorporated the derived rating curve. The resultant outflow hydrograph was compared with the COG outflow hydrograph to ensure the derived rating curve was similar to the COG provisional rating curve.

An initial water level of 25m AHD was required in the reservoir to ensure the outflow hydrograph could be reproduced.

Once the rating curve and initial water level were established, the inflow hydrograph estimated from the North Pine River runoff routing model was routed through the incomplete storage to determine an outflow hydrograph.

The COG estimated hydrographs were not adopted because it was not possible to determine how they were estimated. It was also discovered that the runoff-routing model of the North Pine River was unable to reproduce the volume of flood runoff and the magnitude of peak discharges of the COG inflow hydrograph using the rainfall records presented in Figure 6.6 with the adopted model parameters.

The magnitude of the COG outflow hydrograph is quite different from the Water Resources Commission estimated peak flow at Young's Crossing for the January 1974 event. The Commission's estimate is based on debris levels recorded at the gauging station. A peak discharge of 821 m^3/s is estimated by the Commission from these records. The Commission's estimate and the COG's estimate disagree markedly given there is only three kilometres of river between the Dam and the streamgauging station, with little opportunity for significant attenuation of the flow.

The inflow hydrograph estimated from the North Pine River runoff-routing model has a peak discharge of 1575 m^3/s. A second but smaller peak of 1320 m^3/s is also predicted. After routing this hydrograph through the reservoir routing program, the resultant outflow hydrograph has a peak discharge of 960 m^3/s. Both estimated hydrographs are shown in Figure 9.5.

The records of this event for Draper's Crossing on the South Pine River are a little more reliable than those of the North Pine. An estimated peak discharge of 1240 m^3/s was recorded at this station. The recorded hydrograph consists of three major peaks with the first and second peaks being of the same order, whilst the third peak was somewhat smaller at 970 m^3/s. The flood frequency analysis results of Draper's Crossing indicates that the January 1974 event has an ARI of greater than 100 years.

Rainfall for this event continued for four days after commencing on January 24th. The North Pine River catchment above North Pine Dam received approximately 800 mm in that time whereas the catchment to Draper's Crossing on South Pine River received closer to 1000 mm.

Estimates of inflow and outflow hydrographs for Lake Kurwongbah have been derived assuming the storage was at Full Supply prior to the flood. This is based on available storage records. These hydrographs are shown in Figure 9.6.

The estimated hydrograph for Cash's Crossing on South Pine River is presented in Figure 9.7. Rainfall loss rates for all models are derived from the South Pine River model calibration.

9.4 **APRIL 1989**

This is the largest flood event to occur in the Pine River catchment since North Pine Dam became fully operational. The recorded hydrograph at Draper's Crossing indicates a series of five peaks starting at midnight on the 1st, with the second and largest occurring at 4:00 pm on the 2nd and the final peak happening at 6:00 pm on the 5th. The largest peak discharge was 920 m^3/s which has an ARI of just under 50 years. Only the first three peaks are simulated since the largest peak occurred early.

Rainfalls associated with this event commenced at around 9.00 am on the 1st April and continued for some 72 hours (although more rain occurred in the following 72 hours resulting in three smaller hydrographs.)

The North and South Pine River catchments received around 330 mm in the first 72 hour period.

Storage records are available for North Pine Dam that consist of time histories of water level in Lake Samsonvale and release rates from the spillway and spillway gate openings.

Actual release rates from North Pine Dam will be used as inputs to the numerical hydraulic model. Figure 9.8 provides a summary of the release rates, together with the estimated inflow to the storage. Normal gate operation procedures were adopted during this event, but some out of sequence gate openings occurred.

Estimates of inflow and outflow hydrographs for Lake Kurwongbah are shown in Figure 9.9. Lake Kurwongbah storage records show that the reservoir was at Full Supply prior to the event.

The estimated hydrograph for Cash's Crossing on South Pine River is presented in Figure 9.10. Rainfall loss rates derived from the South Pine Model were adopted for the Lake Kurwongbah estimates.

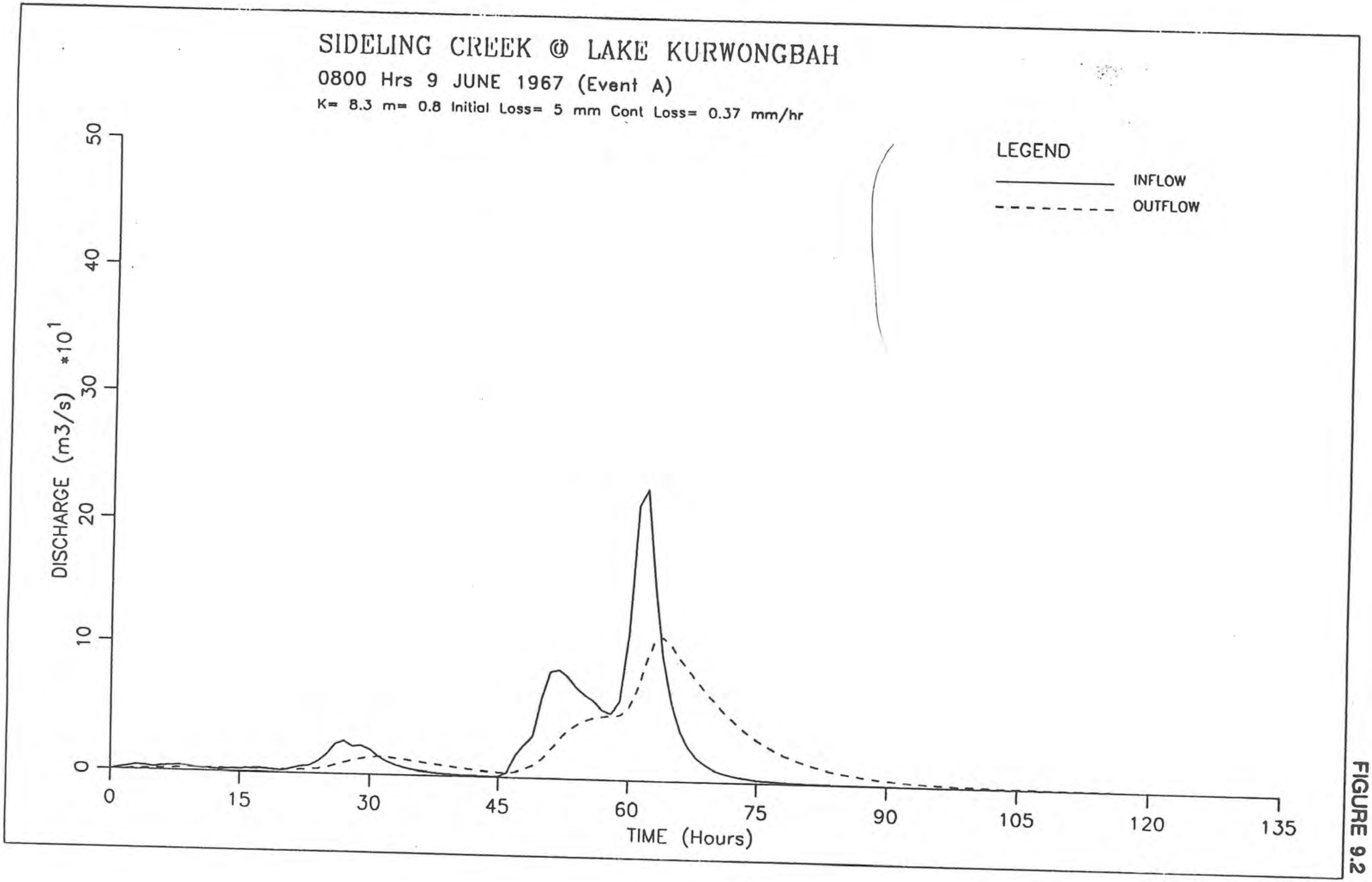

SIDELING CREEK @ LAKE KURWONGBAH
0800 Hrs 9 JUNE 1967 (Event A)
K= 8.3 m= 0.8 Initial Loss= 5 mm Cont Loss= 0.37 mm/hr
LEGEND
INFLOW
OUTFLOW
DISCHARGE (m3/s) *10^1
50
40
30
20
10
0
0
15
30
45
60
75
90
105
120
135
TIME (Hours)

FIGURE 9.2

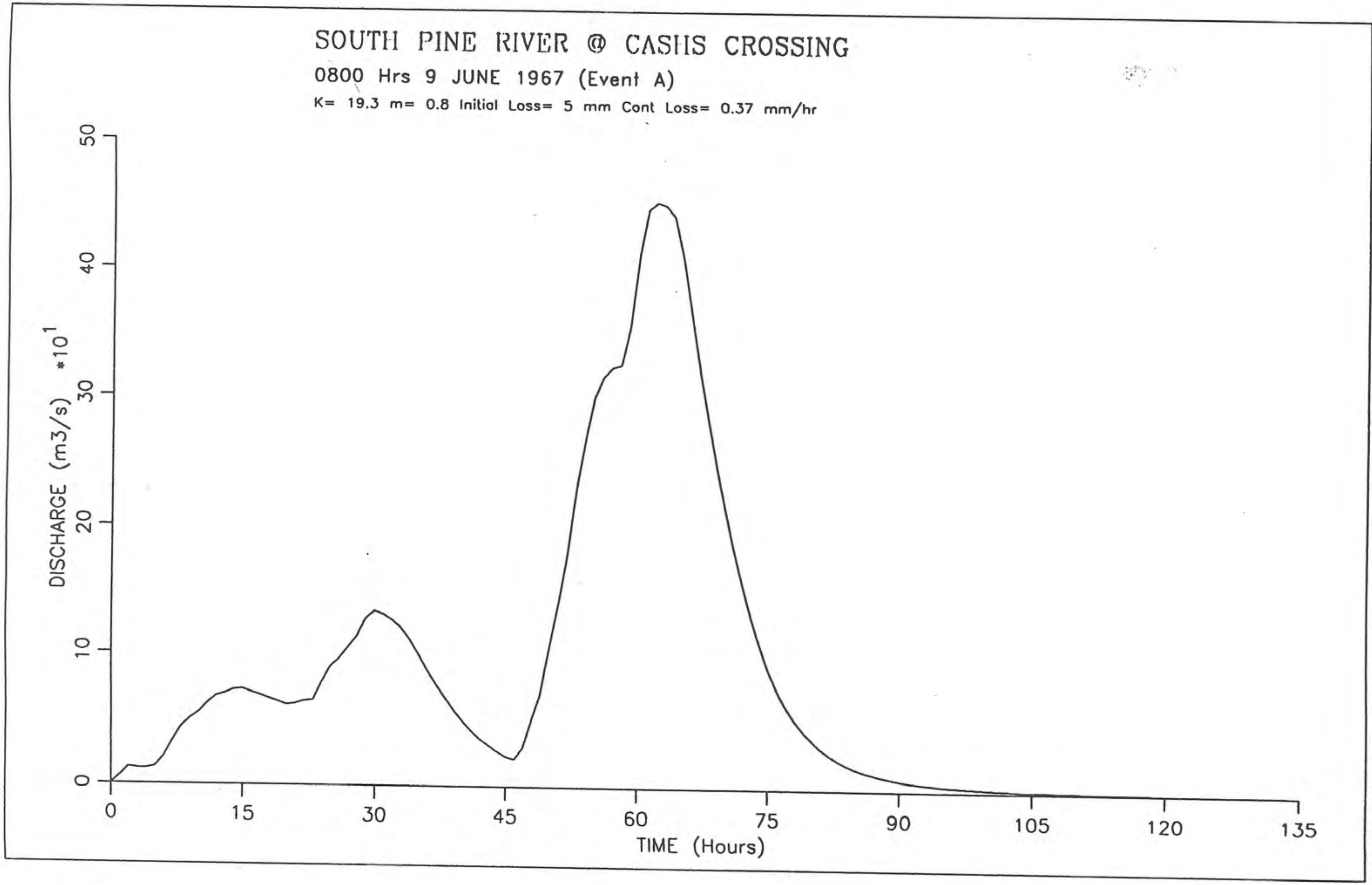

SOUTH PINE RIVER @ CASHS CROSSING
0800 Hrs 9 JUNE 1967 (Event A)
K= 19.3 m= 0.8 Initial Loss= 5 mm Cont Loss= 0.37 mm/hr
DISCHARGE (m3/s) *10^1
0
10
20
30
40
50
0
15
30
45
60
75
90
105
120
135
TIME (Hours)

FIGURE 9.3

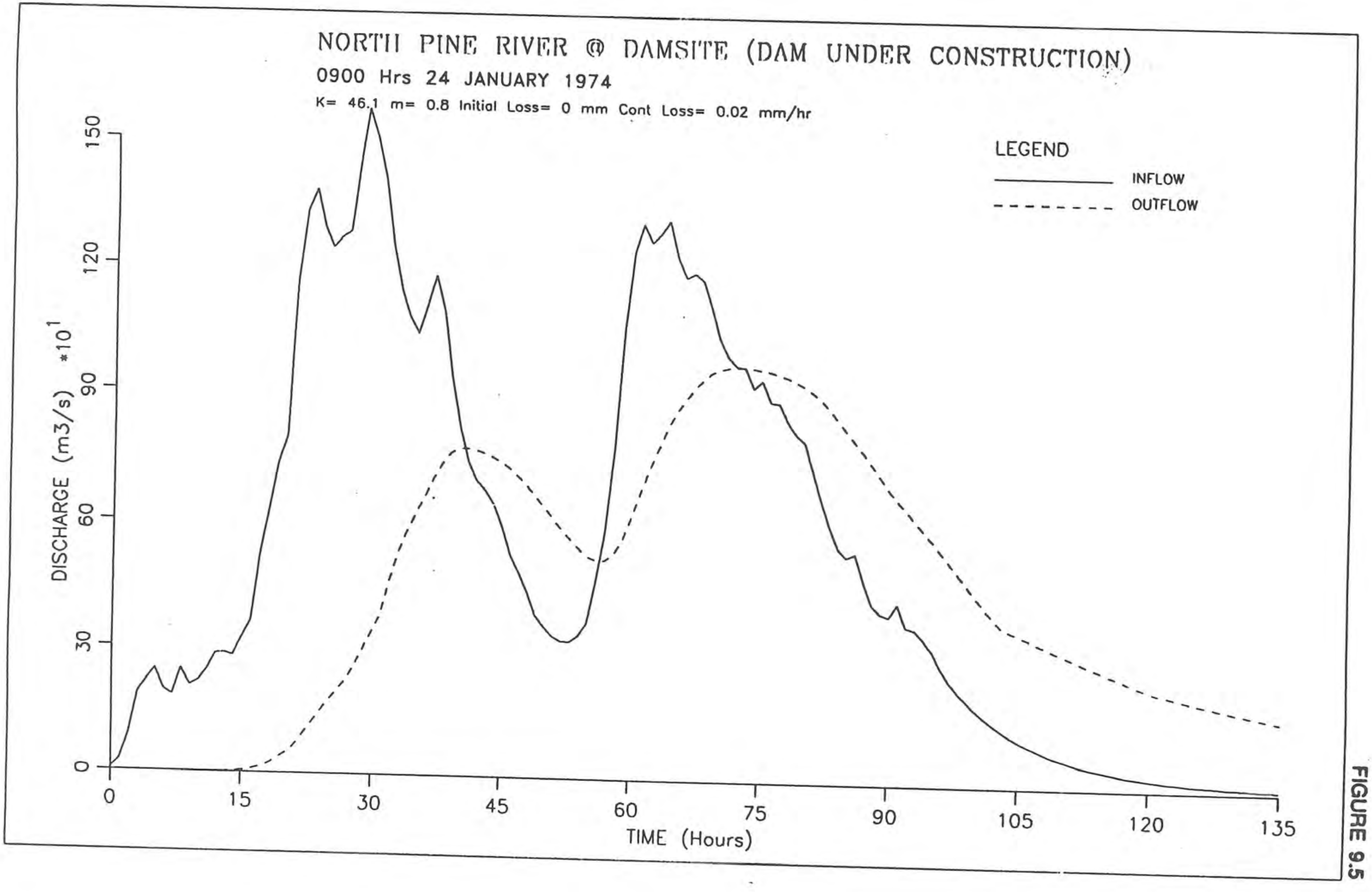

NORTH PINE RIVER @ DAMSITE (DAM UNDER CONSTRUCTION)
0900 Hrs 24 JANUARY 1974
K= 46.1 m= 0.8 Initial Loss= 0 mm Cont Loss= 0.02 mm/hr
LEGEND
INFLOW
OUTFLOW
DISCHARGE (m3/s) *10^1
0
30
60
90
120
150
0
15
30
45
60
75
90
105
120
135
TIME (Hours)

FIGURE 9.5

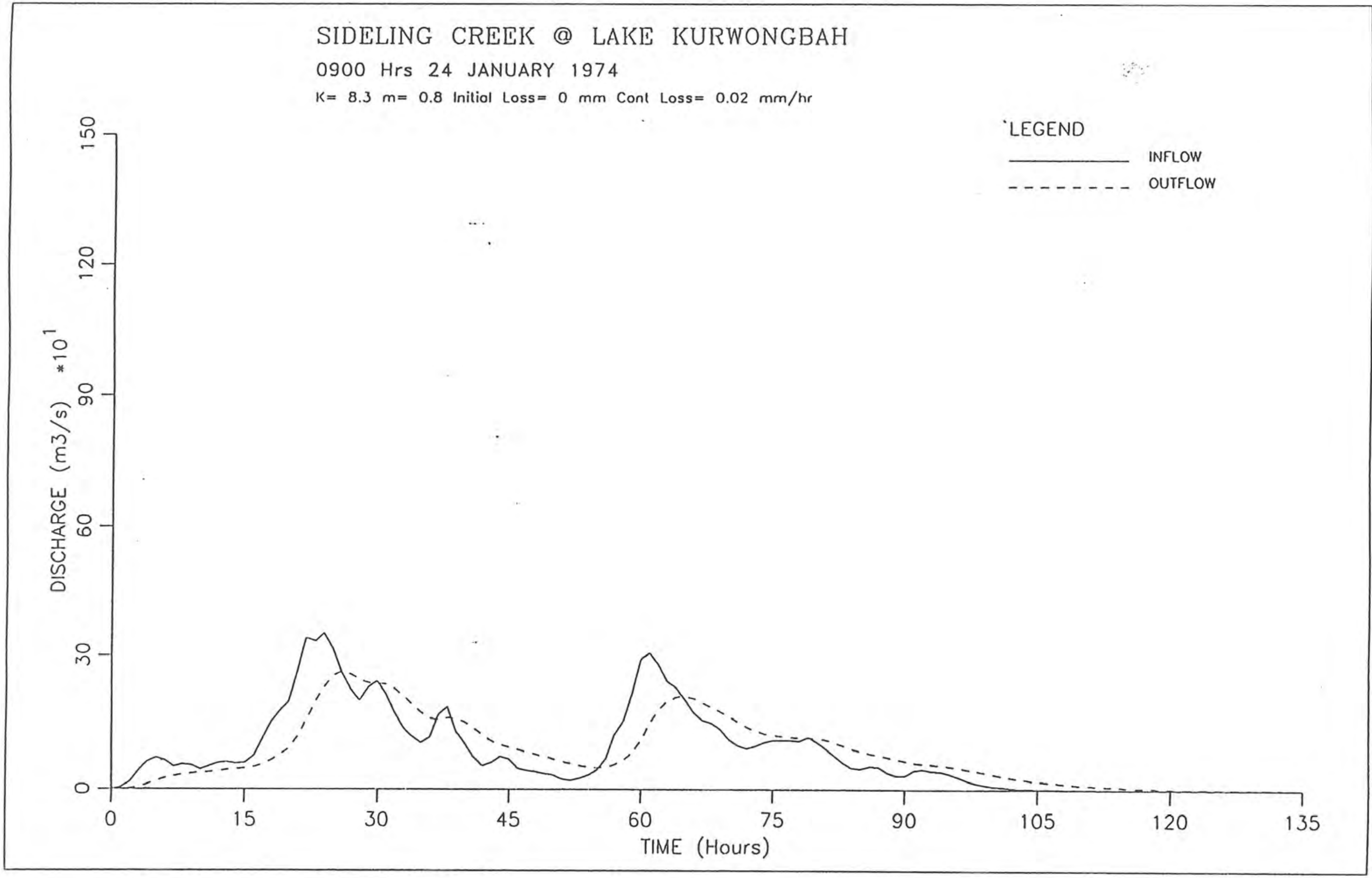
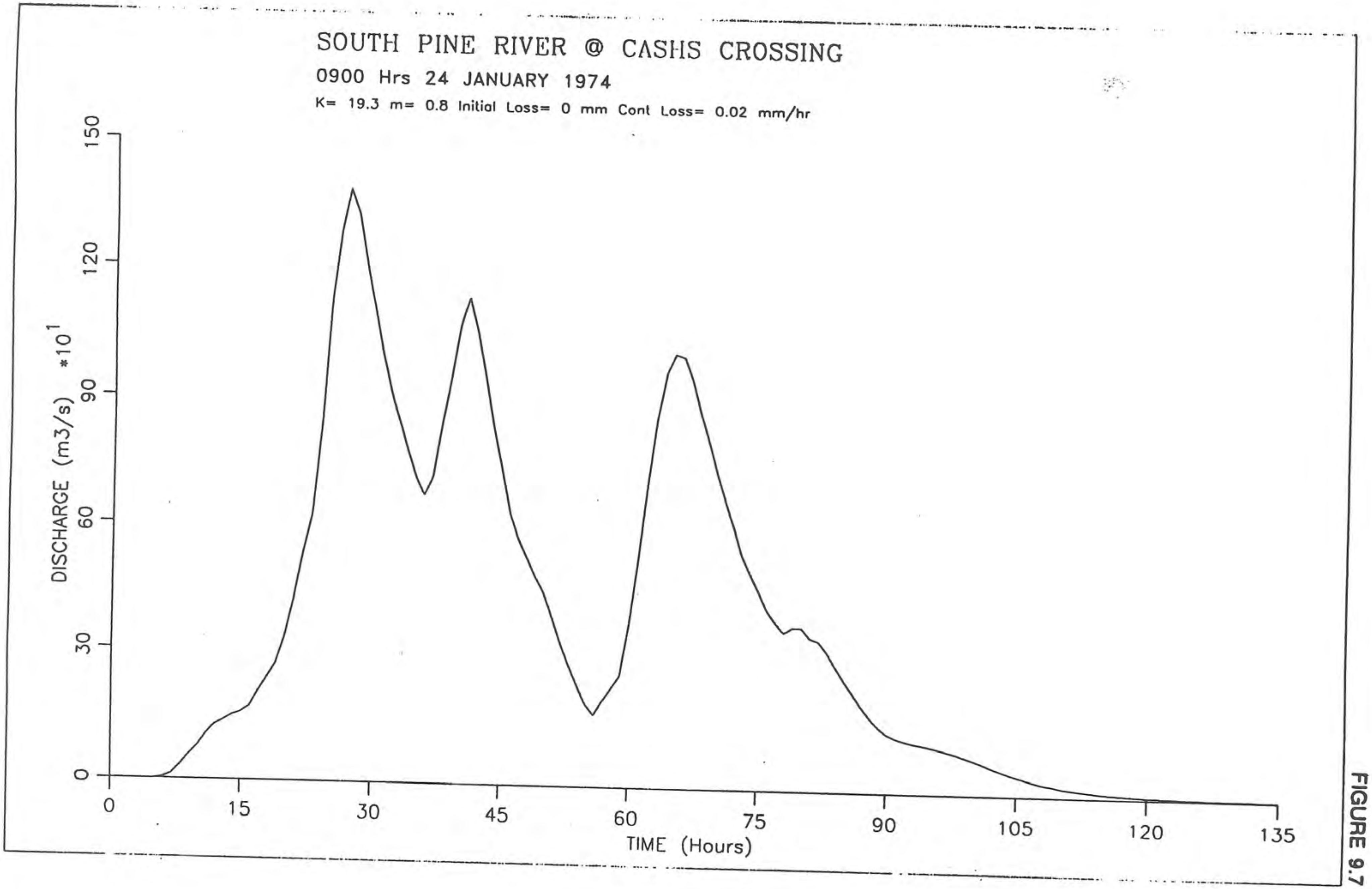
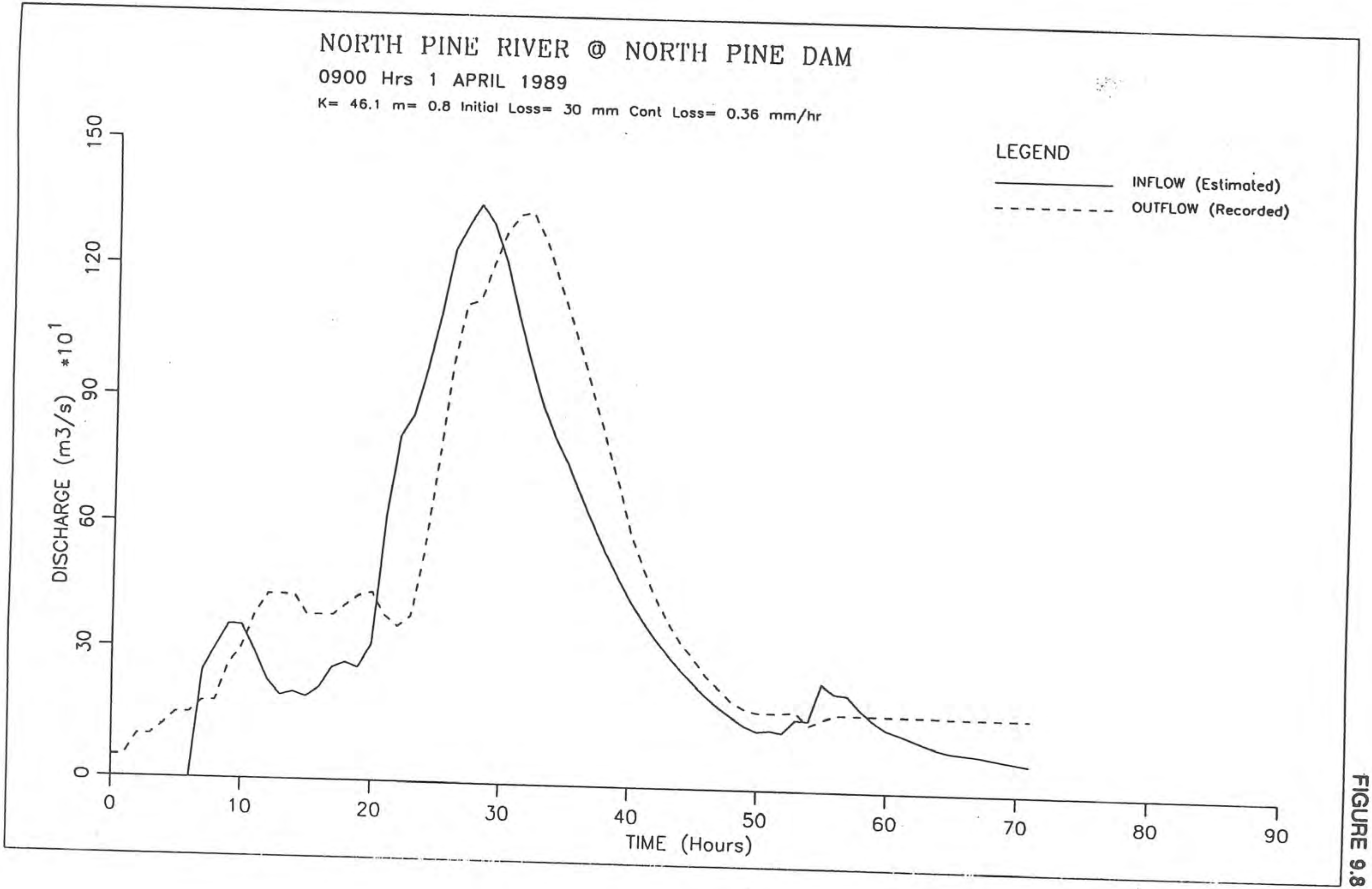

SIDELING CREEK @ LAKE KURWONGBAH
0900 Hrs 24 JANUARY 1974
K= 8.3 m= 0.8 Initial Loss= 0 mm Cont Loss= 0.02 mm/hr
LEGEND
INFLOW
OUTFLOW
DISCHARGE (m3/s) *10^1
0
30
60
90
120
150
0
15
30
45
60
75
90
105
120
135
TIME (Hours)

FIGURE 9.6



SOUTH PINE RIVER @ CASHS CROSSING
0900 Hrs 24 JANUARY 1974
K= 19.3 m= 0.8 Initial Loss= 0 mm Cont Loss= 0.02 mm/hr
DISCHARGE (m3/s) *10^1
0
30
60
90
120
150
TIME (Hours)
0
15
30
45
60
75
90
105
120
135

FIGURE 9.7



NORTH PINE RIVER @ NORTH PINE DAM
0900 Hrs 1 APRIL 1989
K= 46.1 m= 0.8 Initial Loss= 30 mm Cont Loss= 0.36 mm/hr
LEGEND
INFLOW (Estimated)
OUTFLOW (Recorded)
DISCHARGE (m3/s) *10[1]
150
120
90
60
30
0
0
10
20
30
40
50
60
70
80
90
TIME (Hours)

FIGURE 9.8

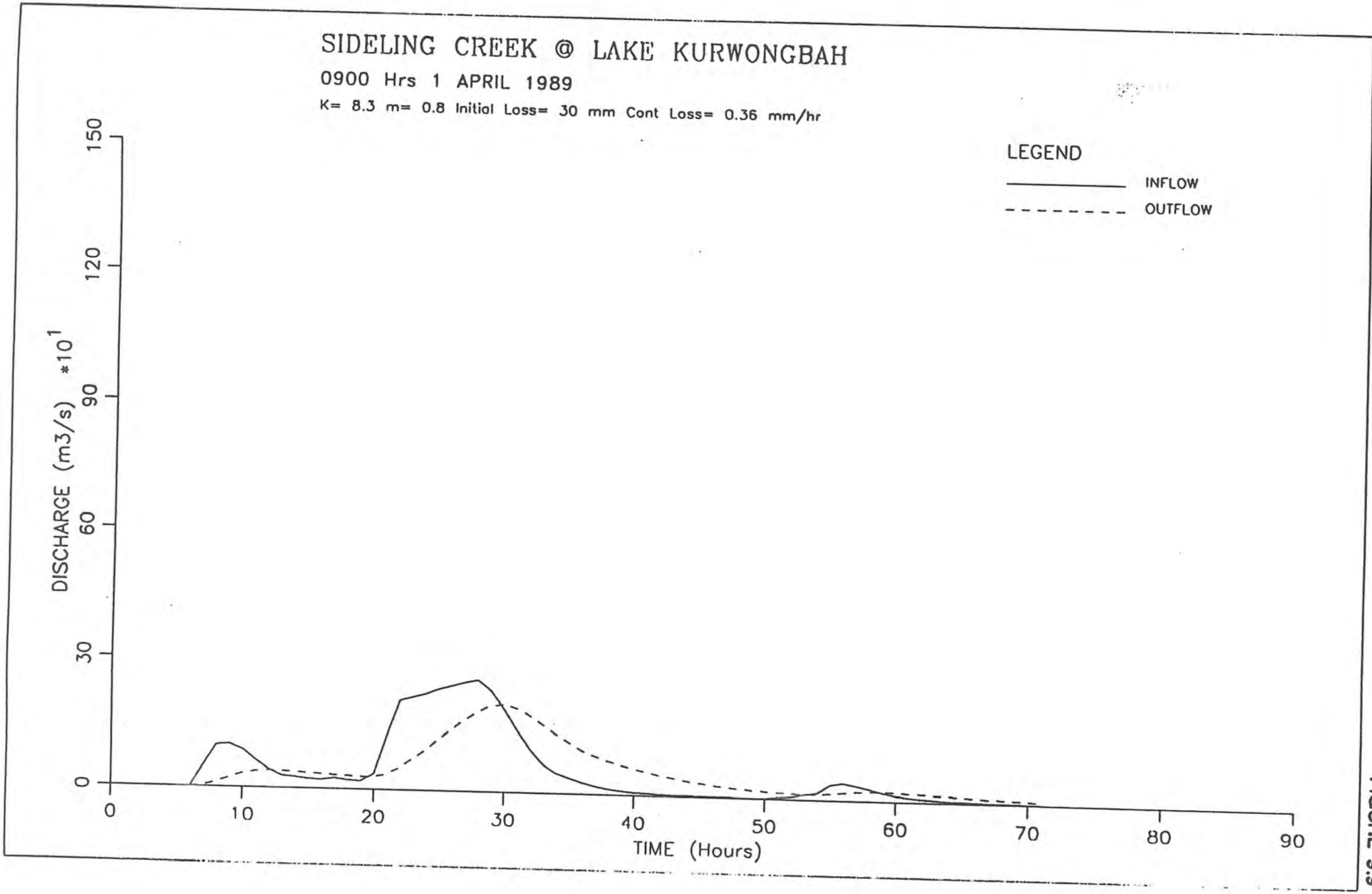

SIDELING CREEK @ LAKE KURWONGBAH
0900 Hrs 1 APRIL 1989
K= 8.3 m= 0.8 Initial Loss= 30 mm Cont Loss= 0.36 mm/hr
LEGEND
INFLOW
OUTFLOW
DISCHARGE (m3/s) *10^1
150
120
90
60
30
0
0
10
20
30
40
50
60
70
80
90
TIME (Hours)

FIGURE 9.9

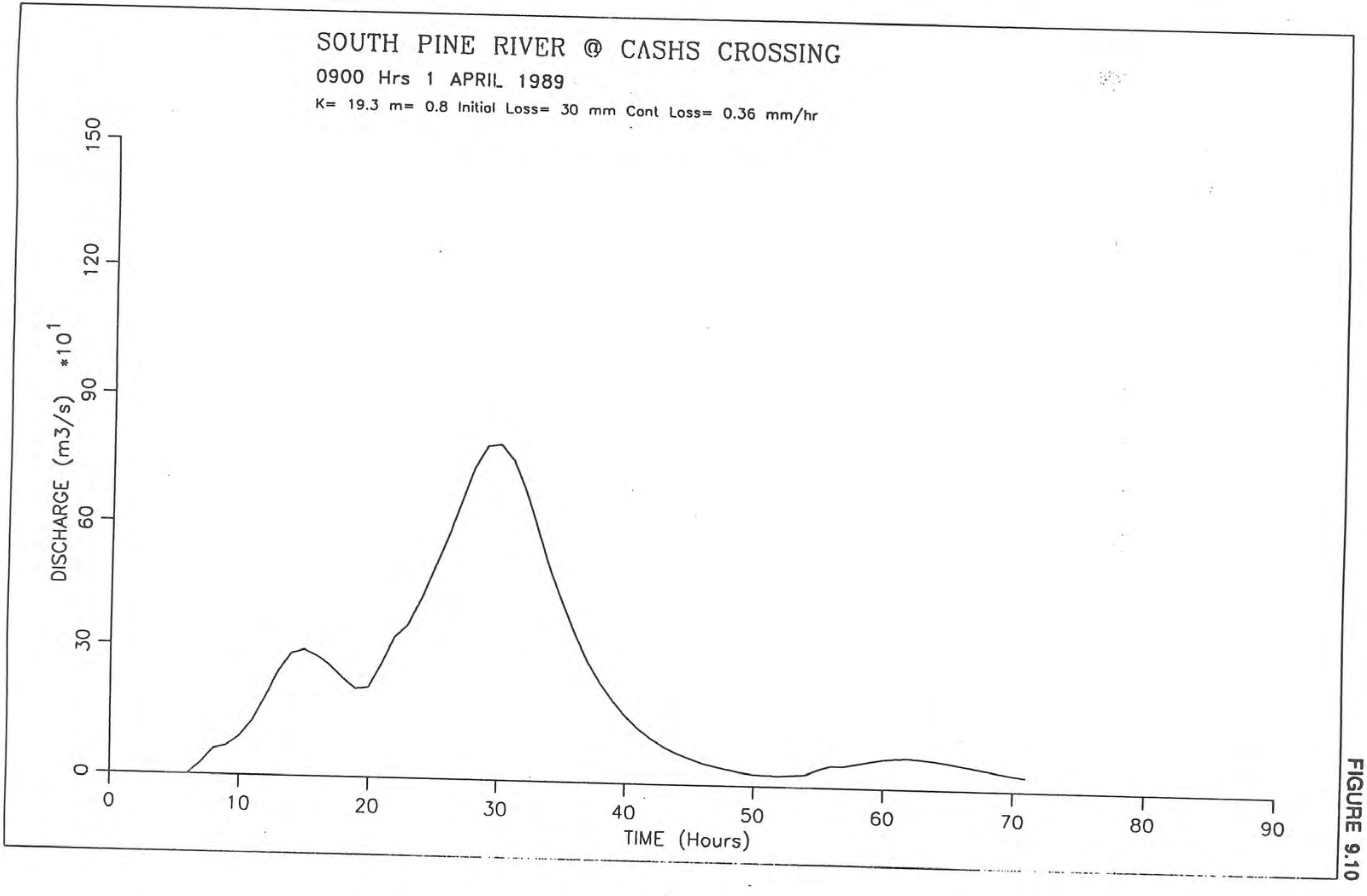

SOUTH PINE RIVER @ CASHS CROSSING
0900 Hrs 1 APRIL 1989
K= 19.3 m= 0.8 Initial Loss= 30 mm Cont Loss= 0.36 mm/hr
DISCHARGE (m3/s) *10
0
30
60
90
120
150
0
10
20
30
40
50
60
70
80
90
TIME (Hours)

FIGURE 9.10

10.0 CONCLUSIONS AND RECOMMENDATIONS

Calibrated runoff-routing models have been developed for North Pine River at North Pine Damsite and South Pine River at Draper's Crossing. These models have been modified and extended to create design models that are capable of being used with a storage routing model of North Pine Dam and a numerical hydraulic model to simulate flooding in the lower reaches of North Pine River and South Pine River.

Adopted runoff routing model parameters for the various catchments are listed below:

	k	m
North Pine River at North Pine Dam	46.1	0.8
North Pine River at Young's Crossing	51.4	0.8
South Pine River at Draper's Crossing	15.2	0.8
South Pine River at Cash's Crossing	19.3	0.8
Sideling Creek at Lake Kurwongbah	8.3	0.8

Historic flood estimates and design flood estimates have been derived for a range of average recurrence intervals from 2 years to the PMF and for a range of durations from 1 hour to 7 days. These estimates have been obtained for inflows and outflows from North Pine Dam, flows at Draper's Crossing and Cash's Crossing on South Pine River, and for inflows and outflows from Lake Kurwongbah on Sideling Creek.

The catchment of North Pine River at North Pine Dam has a critical duration of 18 hours with the catchment in its natural state. The critical duration of this catchment reduces to only two hours when considering peak inflows into North Pine Dam, with the initial water level at Full Supply.

When the existing normal gate operation procedure is used at North Pine Dam the critical duration of outflows from the dam is 18 hours. The existing procedure reduces the Probable Maximum Flood discharge downstream of the dam from 4620 m^3/s (pre-dam) to 4060m^3/s. The resultant peak lake level of 42.38 m AHD however, is sufficient to inundate the radial gate winch switchgear by some 0.72 metres. This level is also approximately 0.92 metres higher than the maximum flood level estimated in 1966.

The full flood frequency of outflows from North Pine Dam indicates that peak discharges downstream of the dam are increased for all but the most extreme flood events.

The assumption that North Pine Dam has a water level equivalent to Full Supply prior to a design flood event is considered conservative. The estimates of the full flood frequency of outflows are therefore regarded as upper limit values, especially for the higher probability events.

The existing gate operation procedure when one gate is out of service increases the critical duration of outflows from North Pine Dam to 24 hours. The reduced discharge capacity of the spillway reduces the PMF discharge downstream of the dam to 3585 m^3/s. The associated increase in peak lake level however, is almost sufficient to overtop the embankment crest level of the dam, and as a consequence it can be regarded as an Imminent Failure Flood.

The critical duration for inflows into Lake Kurwongbah is only two hours. Lake Kurwongbah exhibits flood mitigating capability for floods of durations of less than 24 hours. The peak PMF outflow from Lake Kurwongbah is estimated to be 1054 m^3/s which is associated with a six hour duration storm. This flow is sufficient to cause Lake Kurwongbah to be overtopped by some 0.43 metres. The magnitude of the Imminent Failure Flood for Lake Kurwongbah is estimated to be 90% of the PMF. The ARI of the IFF is estimated to be 730 000 years.

Runoff routing model parameters for Lake Kurwongbah may be derived from either the North Pine River or the South Pine River calibrations. Model parameters based upon the North Pine River values provide estimates of PMF's that concur with previous model studies, whereas parameters apportioned according to the South Pine River model agree with regional formulae estimates. Model parameters based upon the South Pine River calibration have been adopted in this study.

Further refinement of these model parameters is necessary and this can only be achieved by future monitoring of the catchment. A water level recorder is recommended for Lake Kurwongbah as part of the proposed ALERT network of the Pine River system.

The effect of the spatial distribution of PMP rainfalls has been examined for the North Pine Dam catchment. It appears that this aspect is most significant for short storm durations of less than 6 hours and that it only affects peak discharge estimates. Flood volume and estimates of time to peak remain unaltered.

Spatial distribution of PMP rainfalls is required for the determination of concurrent rainfall estimates in adjacent catchments.

The Bureau of Meteorology indicates that no definitive method of determining concurrent rainfall exists. Four possible methods have been suggested by the Bureau and the sensitivity of three of those methods has been investigated.

The method that applies areally adjusted 1:100 year IFD depth and spatial distribution over the adjacent catchment will provide the greatest incremental hazard estimate for failure of North Pine Dam. This method has been adopted for use in the proposed dambreak analyses.

11.0 REFERENCES

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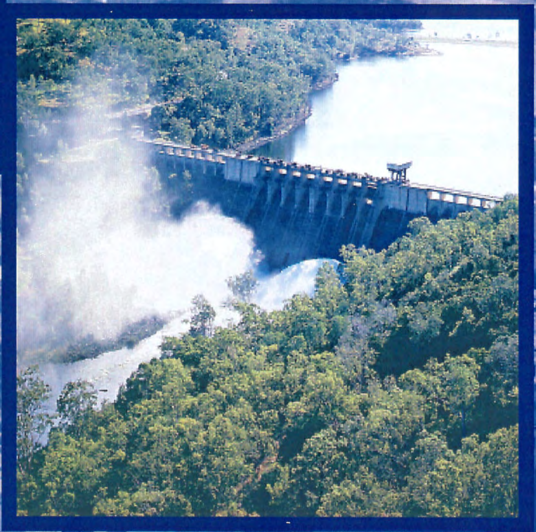
AUGUST 1991

File was subject to a "FOI" application refer CHQ/ 2034

BRISBANE RIVER AND PINE RIVER FLOOD STUDY :

Report No. 4c



PINE RIVER FLOOD HYDROLOGY REPORT VOLUME III

South East Queensland WATER BOARD

NATURAL RESOURCES Queensland

QUEENSLAND GOVERNMENT

Brisbane River and Pine River Flood Studies

PINE RIVER FLOOD HYDROLOGY REPORT

APPENDICES

Volume III
August 1991

APPENDIX A

RUNOFF ROUTING MODEL CATCHMENT DATA FILES AND

RUNOFF ROUTING MODEL RAINFALL DATA FILES

(HISTORIC EVENTS)

A1 NORTH PINE RIVER @ DAMSITE

```
METRIC UNITS.
19 SUBAREAS OF AREA:
25.83 14.67 19.57 22.10 18.53
20.03 19.20 19.43 12.77 15.83
19.40 12.83 20.13 20.07 17.33
17.67 13.90 13.56 22.20
RAIN ON AREA #  1  K1=0.196
STORE HYDROGRAPH.
RAIN ON AREA #  2  K1=0.147
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.118
RAIN ON AREA #  3  K1=0.226
RAIN ON AREA #  4  K1=0.187
STORE HYDROGRAPH.
RAIN ON AREA #  5  K1=0.226
STORE HYDROGRAPH.
RAIN ON AREA #  6  K1=0.147
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.187
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.108
STORE HYDROGRAPH.
RAIN ON AREA #  7  K1=0.147
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.246
STORE HYDROGRAPH.
RAIN ON AREA #  8  K1=0.118
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.108
STORE HYDROGRAPH.
RAIN ON AREA #  9  K1=0.196
RAIN ON AREA # 10  K1=0.098
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.167
RAIN ON AREA # 11  K1=0.196
RAIN ON AREA # 12  K1=0.147
STORE HYDROGRAPH.
RAIN ON AREA # 13  K1=0.255
STORE HYDROGRAPH.
RAIN ON AREA # 14  K1=0.216
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.157
RAIN ON AREA # 15  K1=0.138
STORE HYDROGRAPH.
RAIN ON AREA # 16  K1=0.236
RAIN ON AREA # 17  K1=0.108
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.098
STORE HYDROGRAPH.
RAIN ON AREA # 18  K1=0.138
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.088
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.079
STORE HYDROGRAPH.
RAIN ON AREA # 19  K1=0.147
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.088
P&P HYDROGRAPH. GS 142102 NORTH PINE RIVER @ DAMSITE
END
```

0800 HRS 9 JUNE 1967

FITTING RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 148

RAINFALL IN EACH AREA:

215 190 185 180 200 220 190 190 220 195

180 180 205 220 200 230 210 195 200

STORM DURATION: 66 HRS

PLUVIOGRAPH (SAMFORD):

.023 .022 .008 .008 .002 .003 .011 .014 .013 .004

.002 .003 .001 .002 .008 .006 .006 .006 .004 .004

.002 .010 .008 .011 .034 .018 .022 .008 .013 .016

.008 .000 .000 .000 .000 .000 .000 .003 .000 .002

.001 .001 .000 .000 .000 .001 .004 .028 .042 .044

.067 .033 .053 .030 .028 .021 .017 .023 .028 .085

.103 .072 .014 .000 .000 .000

FOR SUBAREAS:

11 12 15 18 19 -1

PLUVIOGRAPH (MT NEBO):

.029 .021 .010 .020 .012 .016 .022 .016 .011 .005

.003 .011 .008 .007 .014 .013 .005 .004 .013 .005

.004 .012 .010 .011 .020 .017 .017 .016 .014 .010

.003 .000 .000 .000 .000 .000 .000 .001 .000 .000

.000 .000 .000 .000 .000 .000 .001 .004 .007 .039

.060 .030 .047 .027 .025 .019 .015 .021 .014 .032

.028 .045 .078 .080 .045 .003

FOR SUBAREAS:

13 14 16 17 -1

PLUVIOGRAPH (DAILY RAINFALL):

.025 .025 .030 .030 .030 .030 .030 .030 .030 .030

.012 .012 .012 .012 .012 .012 .012 .012 .012 .012

.026 .026 .026 .026 .026 .026 .026 .026 .026 .026

.003 .003 .003 .003 .003 .003 .002 .002 .002 .002

.002 .002 .000 .000 .000 .000 .000 .000 .040 .040

.040 .040 .040 .040 .003 .003 .003 .003 .003 .003

.008 .008 .008 .008 .005 .005

FOR SUBAREAS:

1 2 3 4 5 6 7 8 9 10 -1

LOSS: UNIFORM

RECORDED HYDROGRAPH (GS142102):

10.8	10.6	11.9	13.2	14.5	19.4	24.3	29.2	34.2	39.0
44.0	54.6	65.3	75.8	86.5	97.1	107.8	110.0	112.4	114.6
116.9	116.3	115.7	115.1	114.5	124.0	133.6	143.1	152.7	168.6
184.5	200.4	216.4	232.2	248.2	252.9	257.7	248.1	238.5	216.9
195.3	176.5	157.8	142.1	120.4	101.6	82.9	79.9	77.0	73.9
71.0	100.2	129.4	154.1	178.9	203.7	228.5	253.2	278.0	325.6
373.3	376.1	379.0	381.8	384.7	365.3	364.0	326.6	307.3	287.9
268.6	249.3	230.0	210.6	191.3	171.9	152.6	141.0	129.4	117.8
106.2	94.6	83.0	80.0	77.0	74.0	71.0	68.0	71.3	74.0
76.8	73.6	70.5	67.3	64.2	61.0	57.9	54.7	51.6	48.5
45.3	43.8	42.4	40.9	39.5	38.0	36.6	35.4	33.8	32.3
30.9	29.4	28.0	27.2	26.4	25.6	24.8	24.0	23.2	22.4
21.7	20.8	20.1	19.2	18.5	17.9	17.5	16.9	16.4	15.8
15.4	14.8	14.3	13.8	13.3	12.7	12.3	11.7	11.2	10.6
10.2	9.6	9.1	8.6	8.1	7.5	7.0	6.5		

1900 HRS 20 JUNE 1967

FITTING RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 101

RAINFALL IN EACH AREA:

130 110 105 95 95 105 110 120 105 100

115 125 135 145 130 115 120 125 115

STORM DURATION: 42 HRS

PLUVIOGRAPH (BRISBANE AIRPORT):

.002 .008 .027 .004 .016 .019 .034 .025 .046 .034

.020 .016 .012 .007 .002 .052 .004 .003 .001 .004

.031 .023 .125 .020 .042 .070 .031 .061 .016 .019

.008 .012 .012 .012 .062 .019 .035 .042 .020 .003

.001 .000

FOR SUBAREAS:

1 8 13 14 15 18 19 -1

PLUVIOGRAPH (SAMFORD):

.000 .005 .012 .008 .005 .008 .010 .010 .012 .019

.029 .049 .041 .039 .005 .019 .005 .003 .012 .000

.032 .049 .150 .014 .054 .080 .046 .082 .039 .003

.029 .008 .019 .022 .024 .022 .010 .012 .011 .002

.001 .000

FOR SUBAREAS:

2 3 4 5 6 7 -1

PLUVIOGRAPH (ENOGGERA RESERVOIR):

.000 .004 .006 .010 .016 .010 .024 .027 .047 .043

.029 .024 .027 .000 .000 .033 .004 .002 .004 .006

.024 .027 .092 .041 .016 .056 .109 .045 .088 .019

.010 .012 .010 .015 .012 .020 .008 .035 .035 .006

.000 .004

FOR SUBAREAS:

9 10 11 12 16 17 -1

LOSS: UNIFORM

RECORDED HYDROGRAPH (GS142102):

5.2	5.2	5.1	5.1	5.1	5.7	6.2	6.8	7.3	7.9
8.4	10.8	13.2	15.6	18.0	20.4	22.8	28.0	33.1	38.3
43.4	48.6	70.7	92.8	114.9	137.0	153.8	170.6	187.4	193.6
199.8	206.0	222.7	217.6	212.4	205.7	199.0	192.3	187.9	183.4
179.0	167.8	156.5	145.3	134.0	122.8	111.5	104.7	97.9	91.2
84.4	77.6	70.8	67.6	64.4	61.1	57.9	54.7	51.5	49.9
48.3	46.6	45.0	43.4	41.8	40.6	39.5	39.3	37.1	36.0
34.8	33.6	32.4	31.2	30.0	28.8	27.6	27.1	26.5	26.0
25.4	24.9	24.3	23.8	23.3	22.7	22.2	21.6	21.1	20.9
20.8	20.6	20.4	20.3	20.1	19.9	19.8	19.6	19.4	19.3
19.1									

0100 HRS 25 JUNE 1967

FITTING RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 132

RAINFALL IN EACH AREA:

90 95 95 100 115 110 95 95 105 100

115 120 95 95 105 95 105 110 120

STORM DURATION: 52 HRS

PLUVIOGRAPH (BRISBANE AIRPORT):

.004 .009 .022 .003 .000 .000 .006 .000 .002 .000

.000 .000 .000 .012 .000 .004 .014 .063 .041 .016

.010 .022 .091 .083 .047 .018 .010 .055 .033 .049

.063 .043 .047 .055 .006 .016 .018 .002 .000 .010

.003 .001 .000 .000 .001 .015 .041 .024 .008 .012

.012 .009

FOR SUBAREAS:

4 5 6 9 10 11 12 18 19 -1

PLUVIOGRAPH (SAMFORD):

.000 .000 .000 .003 .005 .000 .002 .003 .008 .000

.000 .000 .142 .044 .023 .067 .010 .030 .047 .010

.013 .008 .047 .057 .077 .072 .052 .039 .013 .028

.015 .010 .020 .005 .020 .033 .003 .010 .000 .003

.000 .000 .000 .000 .000 .018 .028 .025 .003 .003

.005 .000

FOR SUBAREAS:

1 2 3 7 8 14 16 17 -1

PLUVIOGRAPH (ENOGGERA RESERVOIR):

.000 .000 .000 .005 .010 .000 .003 .005 .005 .000

.000 .000 .000 .088 .045 .026 .093 .038 .014 .043

.026 .021 .021 .031 .074 .055 .061 .083 .048 .031

.014 .031 .026 .008 .014 .021 .003 .008 .000 .000

.005 .000 .000 .000 .000 .005 .019 .008 .003 .003

.003 .003

FOR SUBAREAS:

13 15 -1

LOSS: UNIFORM

RECORDED HYDROGRAPH (GS142102):

18.9 18.8 18.6 18.4 18.3 18.1 17.9 17.8 17.6 17.4

17.3 17.1 17.9 18.8 19.6 20.5 21.3 22.2 23.0 23.8

24.7 25.5 26.4 27.2 42.0 56.9 71.7 97.4 123.1 148.8

168.7 188.7 208.6 214.7 220.9 227.0 228.5 227.0 221.6 216.2

207.4 198.5 186.5 174.6 162.6 150.3 137.9 125.6 120.7 115.7

115.7 115.7 112.2 108.7 105.2 100.7 96.2 91.6 87.1 82.6

78.5 74.4 70.3 66.2 62.1 58.0 56.0 54.0 52.0 50.0

48.0 46.0 45.0 44.0 43.0 42.0 41.0 40.0 39.0 38.0

37.0 36.0 35.0 34.0 33.5 32.9 32.4 31.9 31.3 30.8

30.3 29.7 29.2 28.7 28.1 27.6 27.1 26.5 26.0 25.5

24.9 24.4 23.9 23.3 22.8 22.3 21.7 21.2 20.9 20.7

20.4 20.2 19.9 19.6 19.4 19.1 18.8 18.6 18.3 18.1

17.8 17.5 17.3 17.0 16.7 16.5 16.2 16.0 15.7 15.4

15.2 14.9

0900 HRS 12 DECEMBER 1970

FITTING RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 171

RAINFALL IN EACH AREA:

245 245 240 240 295 285 240 240 280 265
285 290 245 250 250 225 235 265 255

STORM DURATION: 84 HRS

PLUVIOGRAPH (DAYBORO):

.011 .014 .036 .017 .000 .000 .000 .001 .005 .011
.019 .016 .007 .009 .005 .003 .002 .000 .008 .002
.023 .026 .019 .026 .026 .028 .020 .045 .012 .026
.038 .052 .026 .063 .036 .034 .014 .026 .021 .001
.000 .001 .000 .000 .000 .001 .001 .002 .000 .000
.000 .012 .000 .002 .004 .000 .001 .000 .000 .007
.009 .000 .007 .000 .002 .000 .001 .004 .000 .018
.001 .011 .001 .004 .016 .004 .043 .044 .012 .012
.040 .008 .002 .002

FOR SUBAREAS:

3 4 5 6 7 8 9 10
13 14 15 16 17 -1

PLUVIOGRAPH (MT GLORIOUS):

.000 .046 .000 .018 .003 .004 .000 .011 .053 .029
.017 .029 .022 .014 .002 .003 .000 .000 .001 .001
.028 .030 .020 .028 .018 .015 .014 .037 .022 .014
.038 .021 .015 .042 .032 .029 .022 .003 .012 .006
.000 .000 .000 .000 .001 .000 .001 .001 .000 .000
.000 .001 .000 .004 .014 .005 .014 .000 .000 .000
.016 .011 .000 .000 .000 .000 .000 .000 .004 .014
.002 .023 .005 .000 .016 .003 .064 .035 .011 .011
.032 .009 .002 .002

FOR SUBAREAS:

1 2 -1

PLUVIOGRAPH (SAMFORD):

.000 .077 .005 .017 .001 .000 .000 .000 .025 .021
.017 .007 .002 .019 .004 .004 .003 .000 .001 .002
.002 .005 .006 .004 .024 .033 .017 .023 .032 .024
.038 .048 .019 .035 .041 .021 .007 .001 .002 .000
.000 .000 .000 .000 .000 .000 .002 .001 .000 .000
.000 .011 .000 .002 .004 .000 .001 .000 .000 .006
.008 .000 .006 .000 .002 .000 .001 .004 .000 .017
.001 .010 .010 .023 .065 .007 .095 .054 .023 .010
.035 .011 .003 .001

FOR SUBAREAS:

11 12 18 19 -1

LOSS: UNIFORM

RECORDED HYDROGRAPHS (GS142102):

6.2 8.0 9.6 11.4 11.6 11.8 12.0 12.3 12.4 29.7
46.9 64.2 75.3 86.5 97.6 108.7 107.8 107.0 106.0 105.1
124.9 144.7 164.5 184.3 204.0 223.9 243.7 263.4 295.9 328.3
360.7 346.4 425.6 458.1 482.1 506.1 530.2 526.4 522.6 518.9
494.8 470.8 446.7 422.6 398.6 374.6 350.4 326.4 302.4 278.3
254.2 230.2 218.4 206.7 195.0 183.3 171.5 159.8 148.0 136.2
124.5 112.8 101.0 89.3 87.7 86.1 84.5 83.0 81.3 79.8
78.2 76.5 75.0 73.4 71.8 70.2 117.3 164.4 211.4 218.1
224.8 231.4 249.5 267.6 285.6 294.4 303.3 312.0 295.6 279.2
262.7 246.3 229.9 213.4 196.9 180.5 164.0 147.6 131.2 114.8
111.9 109.1 106.3 103.5 100.7 97.9 95.0 92.2 89.4 86.5
83.7 81.0 78.1 75.3 72.5 69.6 66.8 64.0 61.1 58.4
55.6 52.8 49.9 47.1 46.1 45.0 44.0 43.0 41.8 40.8
39.8 38.7 37.7 36.7 35.6 34.6 33.5 32.4 31.4 30.4
29.3 28.8 27.3 26.3 25.1 24.1 23.1 22.0 21.6 21.2
20.7 20.3 19.9 19.4 18.9 18.5 18.0 17.6 17.2 16.7
16.3 15.9 15.4 15.0 14.6 14.2 13.7 13.3 12.9 12.4
12.0

A2 NORTH PINE RIVER @ YOUNG'S CROSSING

```
METRIC UNITS.
20 SUBAREAS OF AREA:
25.83 14.67 19.57 22.10 18.53
20.03 19.20 19.43 12.77 15.83
19.40 12.83 20.13 20.07 17.33
17.67 13.90 13.56 22.20  7.60
RAIN ON AREA #  1  K1=0.170
STORE HYDROGRAPH.
RAIN ON AREA #  2  K1=0.128
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.102
RAIN ON AREA #  3  K1=0.196
RAIN ON AREA #  4  K1=0.162
STORE HYDROGRAPH.
RAIN ON AREA #  5  K1=0.196
STORE HYDROGRAPH.
RAIN ON AREA #  6  K1=0.128
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.162
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.094
STORE HYDROGRAPH.
RAIN ON AREA #  7  K1=0.128
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.213
STORE HYDROGRAPH.
RAIN ON AREA #  8  K1=0.102
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.094
STORE HYDROGRAPH.
RAIN ON AREA #  9  K1=0.170
RAIN ON AREA # 10  K1=0.085
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.145
RAIN ON AREA # 11  K1=0.170
RAIN ON AREA # 12  K1=0.128
STORE HYDROGRAPH.
RAIN ON AREA # 13  K1=0.221
STORE HYDROGRAPH.
RAIN ON AREA # 14  K1=0.187
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.136
RAIN ON AREA # 15  K1=0.119
STORE HYDROGRAPH.
RAIN ON AREA # 16  K1=0.204
RAIN ON AREA # 17  K1=0.094
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.085
STORE HYDROGRAPH.
RAIN ON AREA # 18  K1=0.119
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.077
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.068
STORE HYDROGRAPH.
RAIN ON AREA # 19  K1=0.128
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.119
RAIN ON AREA # 20  K1=0.111
P&P HYDROGRAPH.GS 142101 NORTH PINE RIVER @ YOUNGS CROSSING
END
```

0800 HRS 9 JUNE 1967

FITTING RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 148

RAINFALL IN EACH AREA:

215 190 185 180 200 220 190 190 220 195
180 180 205 220 200 230 210 195 200 200

STORM DURATION: 66 HRS

PLUVIOGRAPH (SAMFORD):

.023 .022 .008 .008 .002 .003 .011 .014 .013 .004
.002 .003 .001 .002 .008 .006 .006 .006 .004 .004
.002 .010 .008 .011 .034 .018 .022 .008 .013 .016
.008 .000 .000 .000 .000 .000 .000 .003 .000 .002
.001 .001 .000 .000 .000 .001 .004 .028 .042 .044
.067 .033 .053 .030 .028 .021 .017 .023 .028 .085
.103 .072 .014 .000 .000 .000

FOR SUBAREAS:

11 12 15 18 19 20 -1

PLUVIOGRAPH (MT NEBO):

.029 .021 .010 .020 .012 .016 .022 .016 .011 .005
.003 .011 .008 .007 .014 .013 .005 .004 .013 .005
.004 .012 .010 .011 .020 .017 .017 .016 .014 .010
.003 .000 .000 .000 .000 .000 .000 .001 .000 .000
.000 .000 .000 .000 .000 .000 .001 .004 .007 .039
.060 .030 .047 .027 .025 .019 .015 .021 .014 .032
.028 .045 .078 .080 .045 .003

FOR SUBAREAS:

13 14 16 17 -1

PLUVIOGRAPH (DAILY RAINFALL):

.025 .025 .030 .030 .030 .030 .030 .030 .030 .030
.012 .012 .012 .012 .012 .012 .012 .012 .012 .012
.026 .026 .026 .026 .026 .026 .026 .026 .026 .026
.003 .003 .003 .003 .003 .003 .002 .002 .002 .002
.002 .002 .000 .000 .000 .000 .000 .000 .040 .040
.040 .040 .040 .040 .003 .003 .003 .003 .003 .003
.008 .008 .008 .008 .005 .005

FOR SUBAREAS:

1 2 3 4 5 6 7 8 9 10 -1

LOSS: UNIFORM

RECORDED HYDROGRAPH (GS142101):

14.2	13.7	15.0	16.3	17.6	22.9	28.2	33.4	38.6	43.9
49.2	60.5	71.7	82.9	94.2	105.4	116.7	121.1	125.3	130.6
131.5	132.4	133.3	134.2	144.5	154.8	165.0	175.2	194.3	213.4
232.5	251.6	259.3	267.1	274.9	282.7	271.7	260.7	249.7	238.7
221.6	204.5	187.3	170.3	153.2	136.1	119.0	101.8	99.2	96.6
127.1	157.6	189.9	222.2	254.6	286.9	303.5	320.0	336.6	353.2
375.1	396.9	418.7	415.8	397.1	378.3	359.6	340.7	322.0	303.2
284.5	265.7	246.9	228.1	209.4	190.6	178.3	165.9	153.7	141.4
129.1	116.8	108.2	99.8	91.3	82.8	81.2	82.0	85.2	88.3
86.0	83.8	81.4	79.1	75.4	71.6	67.9	64.1	60.3	56.6
55.1	53.7	52.1	50.6	49.2	47.7	46.2	44.7	43.2	41.7
40.3	38.8	37.9	34.0	36.2	35.3	34.5	33.5	32.7	31.9
31.0	30.2	29.2	28.4	27.8	27.2	26.6	25.9	25.3	24.7
24.1	23.5	22.8	22.2	21.6	21.1	20.5	19.8	19.2	18.6
18.0	17.4	16.7	16.1	15.5	14.9	14.3	13.7		

1900 HRS 20 JUNE 1967

FITTING RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 101

RAINFALL IN EACH AREA:

130 110 105 95 95 105 110 120 105 100

115 125 135 145 130 115 120 125 115 110

STORM DURATION: 42 HRS

PLUVIOGRAPH (BRISBANE AIRPORT):

.002 .008 .027 .004 .016 .019 .034 .025 .046 .034

.020 .016 .012 .007 .002 .052 .004 .003 .001 .004

.031 .023 .125 .020 .042 .070 .031 .061 .016 .019

.008 .012 .012 .012 .062 .019 .035 .042 .020 .003

.001 .000

FOR SUBAREAS:

1 8 13 14 15 -1

PLUVIOGRAPH (SAMFORD):

.000 .005 .012 .008 .005 .008 .010 .010 .012 .019

.029 .049 .041 .039 .005 .019 .005 .003 .012 .000

.032 .049 .150 .014 .054 .080 .046 .082 .039 .003

.029 .008 .019 .022 .024 .022 .010 .012 .011 .002

.001 .000

FOR SUBAREAS:

2 3 4 7 20 -1

PLUVIOGRAPH (ENOGGERA RESERVOIR):

.000 .004 .006 .010 .016 .010 .024 .027 .047 .043

.029 .024 .027 .000 .000 .033 .004 .002 .004 .006

.024 .027 .092 .041 .016 .056 .109 .045 .088 .019

.010 .012 .010 .015 .012 .020 .008 .035 .035 .006

.000 .004

FOR SUBAREAS:

5 6 9 10 11 12 16 17 18 19 -1

LOSS: UNIFORM

RECORDED HYDROGRAPH (GS142101):

5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.8	5.8	6.5
7.2	7.9	9.8	11.7	13.6	17.5	21.3	25.2	30.9	36.7
42.4	48.2	53.9	79.0	104.1	129.2	151.0	172.7	194.5	216.2
226.0	235.8	245.5	255.3	253.6	249.7	243.9	238.0	232.2	226.3
220.5	214.6	203.2	191.8	180.4	169.0	157.6	146.2	136.9	127.7
118.4	109.1	99.9	90.6	86.0	81.4	76.9	72.3	67.7	63.1
61.0	58.9	56.9	54.8	52.7	50.6	49.2	47.8	46.4	45.0
43.6	42.2	41.3	40.4	39.4	38.5	37.6	36.6	35.7	34.7
33.8	32.8	31.9	30.9	30.0	29.0	28.1	27.1	26.2	25.2
24.8	24.4	24.0	23.6	23.2	22.8	22.4	22.0	21.6	21.2
20.8									

0100 HRS 25 JUNE 1967
FITTING RUN
TIME INCREMENTS: 1 HRS
NUMBER OF ORDINATES: 132
RAINFALL IN EACH AREA:
90 95 95 100 115 110 95 95 105 100
115 120 95 95 105 95 105 110 120 130
STORM DURATION: 52 HRS
PLUVIOGRAPH (BRISBANE AIRPORT):
.004 .009 .022 .003 .000 .000 .006 .000 .002 .000
.000 .000 .000 .012 .000 .004 .014 .063 .041 .016
.010 .022 .091 .083 .047 .018 .010 .055 .033 .049
.063 .043 .047 .055 .006 .016 .018 .002 .000 .010
.003 .001 .000 .000 .001 .015 .041 .024 .008 .012
.012 .009
FOR SUBAREAS:
4 5 6 7 8 9 10 11 12 15 18 -1
PLUVIOGRAPH (SAMFORD):
.000 .000 .000 .003 .005 .000 .002 .003 .008 .000
.000 .000 .142 .044 .023 .067 .010 .030 .047 .010
.013 .008 .047 .057 .077 .072 .052 .039 .013 .028
.015 .010 .020 .005 .020 .033 .003 .010 .000 .003
.000 .000 .000 .000 .000 .018 .028 .025 .003 .003
.005 .000
FOR SUBAREAS:
1 2 3 14 16 17 19 20 -1
PLUVIOGRAPH (ENOGGERA RESERVOIR):
.000 .000 .000 .005 .010 .000 .003 .005 .005 .000
.000 .000 .000 .088 .045 .026 .093 .038 .014 .043
.026 .021 .021 .031 .074 .055 .061 .083 .048 .031
.014 .031 .026 .008 .014 .021 .003 .008 .000 .000
.005 .000 .000 .000 .000 .005 .019 .008 .003 .003
.003 .003
FOR SUBAREAS:
13 -1
LOSS: UNIFORM
RECORDED HYDROGRAPH (GS142101):
20.1 19.8 19.4 19.1 18.8 18.5 18.3 18.0 17.8 17.5
17.3 17.0 16.8 17.4 18.0 17.8 17.6 17.4 19.3 21.3
23.2 25.8 28.3 37.5 60.3 83.0 105.8 128.5 151.3 174.0
195.0 216.1 237.1 247.7 258.2 268.8 271.0 267.7 264.4 255.9
247.3 238.8 226.4 214.0 201.6 189.6 177.5 165.5 158.9 152.3
149.3 146.2 142.2 138.2 134.2 130.2 126.2 119.0 111.8 104.7
97.5 90.3 83.1 78.6 74.1 69.6 67.2 64.7 62.3 59.8
57.4 54.9 53.5 52.2 50.8 49.4 48.1 46.7 45.3 44.0
42.6 41.2 39.9 38.5 37.9 37.2 36.6 36.0 35.3 34.7
34.0 33.4 32.8 32.1 31.5 30.9 30.2 29.6 28.9 28.3
27.7 27.0 26.4 25.8 25.1 24.5 23.8 23.2 22.9 22.5
22.2 21.9 21.5 21.2 20.9 20.5 20.2 19.9 19.5 19.2
18.9 18.5 18.2 17.9 17.5 17.2 16.9 16.5 16.2 15.9
15.5 15.2

0900 HRS 12 DECEMBER 1970
FITTING RUN
TIME INCREMENTS: 1 HRS
NUMBER OF ORDINATES: 171
RAINFALL IN EACH AREA:
245 245 240 240 295 285 240 240 280 265
285 290 245 250 250 225 235 265 255 290
STORM DURATION: 84 HRS
PLUVIOGRAPH (DAYBORO):
.011 .014 .036 .017 .000 .000 .000 .001 .005 .011
.019 .016 .007 .009 .005 .003 .002 .000 .008 .002
.023 .026 .019 .026 .026 .028 .020 .045 .012 .026
.038 .052 .026 .063 .036 .034 .014 .026 .021 .001
.000 .001 .000 .000 .000 .001 .001 .002 .000 .000
.000 .012 .000 .002 .004 .000 .001 .000 .000 .007
.009 .000 .007 .000 .002 .000 .001 .004 .000 .018
.001 .011 .001 .004 .016 .004 .043 .044 .012 .012
.040 .008 .002 .002
FOR SUBAREAS:
3 4 5 6 7 8 9 10 11 12
13 15 18 -1
PLUVIOGRAPH (MT GLORIOUS):
.000 .046 .000 .018 .003 .004 .000 .011 .053 .029
.017 .029 .022 .014 .002 .003 .000 .000 .001 .001
.028 .030 .020 .028 .018 .015 .014 .037 .022 .014
.038 .021 .015 .042 .032 .029 .022 .003 .012 .006
.000 .000 .000 .000 .001 .000 .001 .001 .000 .000
.000 .001 .000 .004 .014 .005 .014 .000 .000 .000
.016 .011 .000 .000 .000 .000 .000 .000 .004 .014
.002 .023 .005 .000 .016 .003 .064 .035 .011 .011
.032 .009 .002 .002
FOR SUBAREAS:
1 2 14 16 17 -1
PLUVIOGRAPH (SAMFORD):
.000 .077 .005 .017 .001 .000 .000 .000 .025 .021
.017 .007 .002 .019 .004 .004 .003 .000 .001 .002
.002 .005 .006 .004 .024 .033 .017 .023 .032 .024
.038 .048 .019 .035 .041 .021 .007 .001 .002 .000
.000 .000 .000 .000 .000 .000 .002 .001 .000 .000
.000 .011 .000 .002 .004 .000 .001 .000 .000 .006
.008 .000 .006 .000 .002 .000 .001 .004 .000 .017
.001 .010 .010 .023 .065 .007 .095 .054 .023 .010
.035 .011 .003 .001
FOR SUBAREAS:
19 20 -1
LOSS: UNIFORM
RECORDED HYDROGRAPHS (GS142101):

0.0	1.0	1.8	1.8	1.7	1.6	1.6	1.6	1.9	16.0
30.1	44.1	52.3	60.6	68.7	76.9	80.1	83.2	82.2	81.1
80.0	78.9	80.8	82.7	84.6	115.6	146.6	177.6	222.9	268.0
313.3	358.5	403.7	448.9	463.9	465.8	467.8	469.8	460.7	451.7
430.3	408.9	387.5	366.2	344.8	323.4	302.0	280.6	259.2	237.8
216.5	195.0	177.5	159.9	142.3	124.8	107.2	89.6	86.0	82.3
78.6	75.0	71.3	67.6	66.3	65.0	63.6	62.3	61.1	59.7
58.4	57.1	55.7	60.3	64.9	93.1	116.8	167.0	217.3	231.1
244.9	258.7	272.5	276.1	279.6	276.0	272.4	256.8	241.4	226.0
210.5	195.1	179.7	164.1	148.7	133.3	117.8	102.4	87.0	84.3
81.7	79.2	76.6	73.9	71.3	68.7	66.0	63.5	60.9	58.2
55.6	53.0	50.3	47.8	45.2	42.5	39.9	37.3	34.6	32.1
29.5	26.8	24.2	23.5	22.7	21.9	21.2	20.4	19.7	19.0
18.2	17.4	16.7	15.9	15.2	14.5	13.7	12.9	12.2	11.4
10.7	10.0	9.2	8.4	7.7	6.9	6.2	6.0	5.6	5.4
5.2	4.9	4.6	4.4	4.1	3.8	3.5	3.3	3.0	2.8
2.5	2.3	2.0	1.8	1.5	1.2	1.0	0.7	0.4	0.2
0.0									

0900 HRS 6 JULY 1973
FITTING RUN
TIME INCREMENTS: 1 HRS
NUMBER OF ORDINATES: 160
RAINFALL IN EACH AREA:
335 340 360 370 385 400 365 365 385 375
365 365 350 330 340 300 320 350 335 350
STORM DURATION: 59 HRS
PLUVIOGRAPH (DAYBORO):
.003 .004 .011 .011 .008 .009 .004 .011 .006 .022
.040 .034 .002 .002 .000 .013 .000 .004 .006 .005
.004 .005 .024 .022 .013 .002 .000 .001 .002 .003
.007 .012 .037 .026 .030 .050 .041 .025 .051 .053
.049 .035 .028 .033 .033 .026 .023 .030 .025 .023
.026 .024 .010 .013 .012 .003 .002 .003 .000
FOR SUBAREAS:
3 4 5 6 7 8 9 10 11 12
13 15 18 -1
PLUVIOGRAPH (MT GLORIOUS):
.009 .004 .011 .009 .001 .006 .004 .010 .006 .021
.038 .046 .035 .002 .000 .005 .008 .004 .010 .006
.009 .011 .025 .023 .017 .006 .002 .003 .004 .004
.004 .019 .034 .048 .035 .027 .053 .046 .046 .026
.026 .033 .032 .027 .026 .020 .019 .023 .022 .022
.013 .011 .005 .011 .012 .011 .001 .006 .003
FOR SUBAREAS:
1 2 14 16 17 -1
PLUVIOGRAPH (SAMFORD):
.004 .005 .003 .012 .006 .002 .011 .015 .011 .021
.023 .041 .037 .004 .000 .003 .002 .001 .006 .004
.022 .037 .030 .031 .026 .010 .002 .007 .005 .006
.006 .016 .034 .041 .034 .027 .040 .029 .042 .028
.026 .026 .028 .028 .028 .024 .021 .026 .020 .016
.013 .007 .013 .013 .011 .008 .005 .002 .001
FOR SUBAREAS:
19 -1
PLUVIOGRAPH (DEAGON):
.009 .005 .003 .007 .001 .006 .010 .007 .014 .021
.012 .001 .001 .004 .000 .005 .004 .004 .013 .029
.022 .022 .029 .034 .019 .004 .011 .005 .005 .012
.019 .032 .028 .036 .104 .034 .030 .055 .044 .035
.025 .025 .026 .029 .026 .020 .027 .013 .011 .014
.018 .009 .007 .006 .004 .001 .002 .001 .000
FOR SUBAREAS:
20 -1
LOSS: UNIFORM
RECORDED HYDROGRAPH (GS142101):

0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.0	3.9
4.1	4.3	5.2	7.5	11.6	19.1	26.6	36.0	41.2	40.0
37.5	38.1	41.5	51.6	68.4	101.9	148.8	207.9	254.4	290.7
330.9	372.0	421.6	478.9	530.9	581.0	610.4	641.3	662.2	668.8
668.5	666.6	659.5	645.6	630.8	617.7	584.9	561.4	537.8	503.0
468.6	432.8	389.0	343.9	301.4	264.9	238.4	215.3	198.1	178.6
164.0	154.7	148.7	145.3	146.0	150.6	157.3	160.3	164.6	171.1
175.7	173.9	171.0	168.9	167.1	161.4	150.8	136.7	121.3	106.4
95.8	89.0	82.3	75.7	73.3	69.2	65.9	62.4	60.0	57.7
55.6	53.1	50.4	49.9	45.1	43.6	41.7	40.0	38.3	37.1
35.5	34.0	30.4	30.6	29.2	27.1	25.3	23.6	22.3	21.1
20.3	18.8	17.5	16.6	15.6	15.0	14.3	13.6	12.9	12.4
11.9	11.3	10.9	10.4	10.0	9.5	9.1	8.7	8.3	8.0
7.5	7.2	6.8	6.5	7.0	5.9	5.7	5.3	5.1	4.7
4.5	4.2	3.9	3.6	3.4	3.3	3.0	2.8	2.6	2.4
2.2	2.0	1.8	1.7	1.6	1.3	1.2	1.0	0.9	0.6
0.5	0.3	0.2	0.0						

A3 NORTH PINE RIVER @ NORTH PINE DAM

```
METRIC UNITS.
20 SUBAREAS OF AREA:
25.83 14.67 19.57 22.10 18.53
20.03 19.20 19.43 12.77 15.83
18.00  8.83 20.13 20.07 15.43
17.67 11.20  8.46 12.40 24.90
RAIN ON AREA #  1  K1=0.196
STORE HYDROGRAPH.
RAIN ON AREA #  2  K1=0.147
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.118
RAIN ON AREA #  3  K1=0.226
RAIN ON AREA #  4  K1=0.187
STORE HYDROGRAPH.
RAIN ON AREA #  5  K1=0.226
STORE HYDROGRAPH.
RAIN ON AREA #  6  K1=0.147
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.187
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.108
STORE HYDROGRAPH.
RAIN ON AREA #  7  K1=0.147
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.246
STORE HYDROGRAPH.
RAIN ON AREA #  8  K1=0.118
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.108
STORE HYDROGRAPH.
RAIN ON AREA #  9  K1=0.196
RAIN ON AREA # 10  K1=0.098
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.059
STORE HYDROGRAPH.
RAIN ON AREA # 11  K1=0.098
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 12  K1=0.039
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 13  K1=0.255
STORE HYDROGRAPH.
RAIN ON AREA # 14  K1=0.216
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.079
RAIN ON AREA # 15  K1=0.079
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 16  K1=0.138
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 17  K1=0.074
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 18  K1=0.059
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 19  K1=0.059
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 20  K1=0.0
GET HYDROGRAPH.
P&P HYDROGRAPH. NORTH PINE DAM INFLOW
END
```

Normal Gate Operation Procedure

```
METRIC UNITS.
20 SUBAREAS OF AREA:
25.83 14.67 19.57 22.10 18.53
20.03 19.20 19.43 12.77 15.83
18.00  8.83 20.13 20.07 15.43
17.67 11.20  8.46 12.40 24.90
RAIN ON AREA #  1  K1=0.196
STORE HYDROGRAPH.
RAIN ON AREA #  2  K1=0.147
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.118
RAIN ON AREA #  3  K1=0.226
RAIN ON AREA #  4  K1=0.187
STORE HYDROGRAPH.
RAIN ON AREA #  5  K1=0.226
STORE HYDROGRAPH.
RAIN ON AREA #  6  K1=0.147
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.187
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.108
STORE HYDROGRAPH.
RAIN ON AREA #  7  K1=0.147
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.246
STORE HYDROGRAPH.
RAIN ON AREA #  8  K1=0.118
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.108
STORE HYDROGRAPH.
RAIN ON AREA #  9  K1=0.196
RAIN ON AREA # 10  K1=0.098
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.059
STORE HYDROGRAPH.
RAIN ON AREA # 11  K1=0.098
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 12  K1=0.039
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 13  K1=0.255
STORE HYDROGRAPH.
RAIN ON AREA # 14  K1=0.216
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.079
RAIN ON AREA # 15  K1=0.079
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 16  K1=0.138
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 17  K1=0.074
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 18  K1=0.059
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 19  K1=0.059
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 20  K1=0.0
GET HYDROGRAPH.
DAM ROUTE    VBF=0.0   TABLE NO OF VALUES=34
    0    0
 3135   77
```

4769	237
6412	389
8066	542
9728	691
11399	832
13078	972
14769	1115
16469	1252
18177	1384
19893	1540
21621	1679
23358	1827
25103	1987
26856	2140
28622	2302
30395	2483
32177	2679
33967	3375
35770	3419
37490	3462
39399	3506
41227	3550
46272	3670
51262	3790
56317	3910
61438	4012
66625	4065
71878	4124
77998	4186
83508	4251
89098	4320
91356	4348

P&P HYDROGRAPH. NORTH PINE DAM SPILLWAY FLOW
END

One Gate Out of Service

```
METRIC UNITS.
20 SUBAREAS OF AREA:
25.83 14.67 19.57 22.10 18.53
20.03 19.20 19.43 12.77 15.83
18.00  8.83 20.13 20.07 15.43
17.67 11.20  8.46 12.40 24.90
RAIN ON AREA #  1  K1=0.196
STORE HYDROGRAPH.
RAIN ON AREA #  2  K1=0.147
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.118
RAIN ON AREA #  3  K1=0.226
RAIN ON AREA #  4  K1=0.187
STORE HYDROGRAPH.
RAIN ON AREA #  5  K1=0.226
STORE HYDROGRAPH.
RAIN ON AREA #  6  K1=0.147
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.187
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.108
STORE HYDROGRAPH.
RAIN ON AREA #  7  K1=0.147
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.246
STORE HYDROGRAPH.
RAIN ON AREA #  8  K1=0.118
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.108
STORE HYDROGRAPH.
RAIN ON AREA #  9  K1=0.196
RAIN ON AREA # 10  K1=0.098
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.059
STORE HYDROGRAPH.
RAIN ON AREA # 11  K1=0.098
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 12  K1=0.039
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 13  K1=0.255
STORE HYDROGRAPH.
RAIN ON AREA # 14  K1=0.216
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.079
RAIN ON AREA # 15  K1=0.079
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 16  K1=0.138
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 17  K1=0.074
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 18  K1=0.059
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 19  K1=0.059
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 20  K1=0.0
GET HYDROGRAPH.
DAM ROUTE    VBF=0.0   TABLE NO OF VALUES=36
    0    0
 2809   61
```

4222	189
5536	310
6852	432
8176	550
9506	661
10840	772
12182	884
13529	992
14882	1096
16242	1220
17605	1328
18978	1446
20353	1573
21737	1694
23126	1824
24519	1970
25921	2130
27326	2585
28739	2614
30158	2643
31581	2672
33012	2702
36491	2774
41349	2875
46272	2977
51262	3081
56317	3187
57899	3220
60154	3267
61438	3278
66625	3330
71878	3388
77998	3449
83508	3512
89098	3578
91356	3606

P&P HYDROGRAPH. NORTH PINE DAM SPILLWAY FLOW

END

0900 HRS 24 JANUARY 1974

DESIGN RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 136

RAINFALL IN EACH AREA:

775 675 675 725 825 850 740 775 850 770

760 760 800 850 780 810 725 775 725 725

STORM DURATION: 96 HRS

PLUVIOGRAPH (MT GLORIOUS):

.001 .002 .006 .010 .006 .007 .001 .002 .008 .000

.004 .005 .006 .004 .003 .007 .007 .016 .015 .016

.014 .038 .026 .023 .009 .014 .018 .045 .028 .018

.005 .015 .004 .004 .009 .010 .027 .024 .006 .002

.005 .005 .007 .009 .008 .003 .002 .004 .002 .000

.002 .002 .004 .005 .007 .010 .018 .018 .032 .046

.040 .032 .021 .027 .025 .012 .015 .018 .014 .010

.007 .010 .008 .009 .006 .011 .006 .010 .006 .007

.008 .004 .003 .002 .002 .004 .006 .001 .001 .002

.003 .007 .001 .004 .002 .002

FOR SUBAREAS:

1 2 14 16 -1

PLUVIOGRAPH (SAMFORD):

.000 .001 .007 .010 .009 .002 .001 .002 .006 .003

.002 .007 .006 .002 .003 .004 .007 .012 .013 .024

.013 .022 .031 .014 .017 .028 .027 .021 .052 .009

.004 .009 .004 .008 .028 .015 .024 .048 .039 .007

.013 .005 .010 .009 .012 .016 .003 .005 .001 .000

.002 .000 .001 .001 .001 .003 .012 .042 .025 .026

.029 .033 .022 .022 .013 .014 .012 .010 .012 .008

.009 .003 .005 .004 .006 .010 .003 .004 .010 .007

.004 .001 .001 .001 .002 .002 .003 .001 .001 .008

.005 .001 .001 .001 .002 .002

FOR SUBAREAS:

19 -1

PLUVIOGRAPH (DAYBORO):

.001 .003 .008 .013 .008 .009 .001 .003 .010 .000

.005 .006 .008 .005 .004 .009 .009 .021 .019 .021

.018 .049 .033 .029 .016 .015 .018 .015 .026 .031

.020 .015 .005 .007 .006 .007 .018 .016 .004 .001

.003 .003 .005 .006 .005 .002 .001 .003 .002 .000

.002 .002 .003 .004 .006 .008 .015 .015 .026 .038

.033 .026 .017 .022 .021 .010 .012 .015 .012 .008

.006 .008 .010 .012 .008 .014 .008 .013 .008 .009

.010 .005 .004 .003 .003 .005 .008 .001 .001 .003

.004 .009 .001 .005 .003 .002

FOR SUBAREAS:

3 4 5 6 7 8 9 10 11 12

13 15 17 18 20 -1

LOSS: UNIFORM

0600 HRS 21 JUNE 1983

DESIGN RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 100

RAINFALL IN EACH AREA:

180 180 165 160 210 215 160 155 205 175

180 185 155 160 155 150 155 170 175 175

STORM DURATION: 42 HRS

PLUVIOGRAPH (DAYBORO):

.012 .024 .042 .003 .000 .012 .018 .003 .003 .003

.000 .006 .000 .000 .000 .003 .018 .015 .006 .012

.042 .021 .045 .045 .071 .051 .054 .042 .042 .039

.059 .048 .109 .077 .006 .033 .024 .006 .006 .000

.000 .000

FOR SUBAREAS:

3 4 5 6 7 8 9 10 11 13

15 18 -1

PLUVIOGRAPH (MT GLORIOUS):

.017 .030 .028 .006 .008 .022 .019 .000 .008 .006

.003 .003 .000 .000 .006 .008 .014 .006 .006 .014

.028 .022 .063 .087 .098 .084 .039 .063 .033 .039

.050 .041 .060 .008 .025 .025 .014 .008 .003 .003

.000 .003

FOR SUBAREAS:

1 2 14 16 -1

PLUVIOGRAPH (NORTH PINE DAM):

.008 .018 .026 .006 .000 .022 .033 .006 .006 .006

.000 .010 .003 .000 .000 .005 .015 .020 .010 .005

.041 .020 .046 .064 .074 .056 .036 .023 .033 .028

.051 .079 .049 .090 .023 .041 .023 .008 .008 .005

.003 .000

FOR SUBAREAS:

12 17 19 20 -1

LOSS: UNIFORM

0900 HRS 3 APRIL 1988

DESIGN RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 120

RAINFALL IN EACH AREA:

420 365 365 385 420 435 410 410 440 425
400 380 425 435 410 415 400 395 370 380

STORM DURATION: 84 HRS

PLUVIOGRAPH (DAYBORO):

.000 .002 .001 .002 .005 .005 .002 .005 .004 .006
.032 .024 .024 .025 .010 .006 .036 .018 .001 .006
.002 .008 .006 .001 .001 .001 .001 .000 .002 .000
.000 .000 .000 .004 .000 .004 .000 .000 .000 .001
.008 .024 .041 .043 .059 .005 .019 .037 .001 .012
.002 .010 .027 .020 .011 .004 .006 .008 .012 .004
.006 .017 .031 .007 .018 .007 .005 .036 .051 .044
.016 .010 .017 .000 .012 .005 .006 .000 .001 .000
.056 .057 .000 .000

FOR SUBAREAS:

3 4 5 6 7 8 9 10 11 12
13 15 18 20 -1

PLUVIOGRAPH (MT GLORIOUS):

.002 .002 .005 .002 .006 .006 .010 .005 .023 .016
.014 .023 .073 .033 .016 .011 .009 .005 .004 .003
.005 .003 .007 .002 .000 .003 .003 .001 .004 .016
.005 .001 .001 .004 .000 .002 .002 .000 .001 .008
.006 .027 .023 .030 .043 .036 .009 .046 .009 .006
.022 .005 .019 .016 .026 .005 .007 .007 .008 .008
.006 .008 .028 .013 .014 .010 .008 .007 .031 .051
.015 .020 .007 .001 .006 .013 .000 .001 .005 .006
.005 .045 .005 .001

FOR SUBAREAS:

1 2 14 16 17 -1

PLUVIOGRAPH (SAMFORD):

.000 .005 .000 .002 .000 .003 .002 .013 .009 .011
.013 .021 .035 .044 .033 .014 .019 .016 .006 .003
.003 .011 .003 .002 .003 .003 .000 .006 .003 .000
.002 .002 .002 .000 .000 .000 .000 .002 .011 .008
.028 .041 .057 .073 .041 .006 .027 .000 .003 .006
.003 .025 .024 .025 .006 .008 .014 .014 .013 .006
.013 .043 .006 .003 .003 .003 .003 .014 .016 .017
.019 .002 .000 .000 .002 .011 .002 .008 .011 .027
.046 .021 .000 .000

FOR SUBAREAS:

19 -1

LOSS: UNIFORM

0900 HRS 1 APRIL 1989

DESIGN RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 72

RAINFALL IN EACH AREA:

357 346 329 318 321 335 336 347 343 338

323 306 357 368 339 344 333 322 308 310

STORM DURATION: 72 HRS

PLUVIOGRAPH (MT GLORIOUS):

0.005	0.035	0.002	0.003	0.011	0.005	0.007	0.012
0.027	0.026	0.035	0.003	0	0	0	0.016
0.015	0.053	0.03	0.015	0.008	0.039	0.068	0.056
0.031	0.105	0.08	0.029	0.054	0.027	0.018	0.004
0.011	0.005	0	0.005	0.008	0	0	0.004
0	0	0.001	0	0	0	0	0
0	0	0.003	0.008	0.014	0.019	0.014	0.016
0.03	0.008	0.01	0.001	0	0.004	0.003	0
0.001	0.001	0.011	0	0.003	0	0.001	0

FOR SUBAREAS:

1 2 14 16 17 -1

PLUVIOGRAPH (SAMFORD):

0.007	0.03	0.007	0	0.036	0.01	0.002	0.012
0.086	0.105	0.059	0.015	0	0.003	0.021	0.01
0.01	0.053	0.002	0.001	0.01	0.058	0.03	0.048
0.062	0.076	0.053	0.043	0.033	0.021	0.012	0.003
0.005	0.003	0	0	0.002	0	0.005	0
0	0	0	0	0.003	0	0	0
0	0.002	0.002	0.007	0.007	0.003	0.012	0.012
0.005	0.002	0	0	0	0.007	0	0
0	0	0	0	0.003	0	0.002	0

FOR SUBAREAS:

19 -1

PLUVIOGRAPH (DAYBORO):

0.019	0.005	0	0.02	0.006	0.003	0.023	0.096
0.032	0.023	0.011	0.003	0	0.002	0.009	0.002
0.009	0.009	0.002	0.006	0.026	0.091	0.071	0.046
0.062	0.059	0.063	0.063	0.064	0.04	0.028	0.014
0.008	0.002	0.005	0.006	0	0	0	0
0	0	0	0	0	0	0	0
0	0	0.002	0.006	0.003	0.012	0.005	0.029
0.005	0.008	0	0	0	0.002	0	0
0	0	0	0	0	0	0	0

FOR SUBAREAS:

3 4 5 6 7 8 9 10 11 12 13 15 18 20 -1

LOSS: UNIFORM

A4 SOUTH PINE RIVER @ DRAPER'S CROSSING

```
METRIC UNITS.
12 SUBAREAS OF AREA:
14.07 10.20 20.17  8.87 12.93
12.53 13.53 12.87 11.53 13.03
14.40 14.73
RAIN ON AREA #  1  K1=0.302
STORE HYDROGRAPH.
RAIN ON AREA #  2  K1=0.172
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.258
STORE HYDROGRAPH.
RAIN ON AREA #  3  K1=0.215
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.101
RAIN ON AREA #  4  K1=0.158
STORE HYDROGRAPH.
RAIN ON AREA #  5  K1=0.302
RAIN ON AREA #  6  K1=0.230
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.258
RAIN ON AREA #  7  K1=0.330
STORE HYDROGRAPH.
RAIN ON AREA #  8  K1=0.330
STORE HYDROGRAPH.
RAIN ON AREA #  9  K1=0.144
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.172
RAIN ON AREA # 10  K1=0.546
RAIN ON AREA # 11  K1=0.388
STORE HYDROGRAPH.
RAIN ON AREA # 12  K1=0.230
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.144
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.036
P&P HYDROGRAPH. GS 142202 SOUTH PINE RIVER @ DRAPERS CROSSING
END
```

0800 HRS 9 JUNE 1967

FITTING RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 148

RAINFALL IN EACH AREA:

250 275 240 250 270 255 235 250 245 235
230 225

STORM DURATION: 66 HRS

PLUVIOGRAPH (MT NEBO):

.029 .021 .010 .020 .012 .016 .022 .016 .011 .005
.003 .011 .008 .007 .014 .013 .005 .004 .013 .005
.004 .012 .010 .011 .020 .017 .017 .016 .014 .010
.003 .000 .000 .000 .000 .000 .000 .001 .000 .000
.000 .000 .000 .000 .000 .000 .001 .004 .007 .039
.060 .030 .047 .027 .025 .019 .015 .021 .014 .032
.028 .045 .078 .080 .045 .003

FOR SUBAREAS:

1 2 8 9 -1

PLUVIOGRAPH (SAMFORD):

.023 .022 .008 .008 .002 .003 .011 .014 .013 .004
.002 .003 .001 .002 .008 .006 .006 .006 .004 .004
.002 .010 .008 .011 .034 .018 .022 .008 .013 .016
.008 .000 .000 .000 .000 .000 .000 .003 .000 .002
.001 .001 .000 .000 .000 .001 .004 .028 .042 .044
.067 .033 .053 .030 .028 .021 .017 .023 .028 .085
.103 .072 .014 .000 .000 .000

FOR SUBAREAS:

3 4 5 6 7 10 11 12 -1

LOSS: UNIFORM

RECORDED HYDROGRAPH (GS142202):

4.7	4.7	5.7	6.6	7.6	13.3	19.1	24.9	30.7	37.0
43.3	49.6	54.9	60.0	55.4	50.8	45.7	44.3	42.9	45.1
47.1	49.4	49.7	50.1	50.4	60.4	70.4	80.3	90.3	98.8
107.2	115.7	124.2	113.9	103.6	93.3	83.0	72.7	62.3	52.0
41.7	39.6	37.5	35.3	33.2	31.1	28.9	26.7	24.6	32.2
39.8	80.6	121.4	162.5	203.4	244.5	258.8	273.1	287.4	265.2
243.1	220.9	264.4	307.9	351.5	408.8	466.0	404.8	343.7	282.5
221.2	160.1	98.9	88.5	78.0	67.6	57.1	54.2	51.3	48.4
45.5	42.7	39.8	36.9	34.0	31.2	28.2	31.5	33.7	31.2
29.7	28.3	27.8	25.3	23.7	22.2	20.7	19.3	18.8	16.3
14.8	14.4	13.9	13.4	13.0	12.6	12.2	11.7	11.3	18.3
25.2	21.1	16.9	16.2	15.4	14.7	14.0	13.2	12.4	11.7
10.9	9.5	8.7	8.0	7.8	7.5	7.3	7.1	6.9	6.7
6.6	6.4	6.2	5.9	5.7	5.6	5.3	5.1	4.9	4.7
4.5	4.2	4.1	3.9	3.7	3.5	3.3	3.1		

0900 HRS 6 DECEMBER 1970

FITTING RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 165

RAINFALL IN EACH AREA:

215 200 210 235 220 245 260 260 235 220
235 255

STORM DURATION: 84 HRS

PLUVIOGRAPH (MT NEBO):

.002 .013 .013 .015 .015 .004 .004 .018 .018 .038
.038 .015 .015 .018 .018 .008 .008 .002 .002 .005
.005 .023 .023 .030 .015 .014 .014 .033 .033 .023
.023 .030 .030 .024 .024 .021 .021 .010 .010 .001
.001 .001 .001 .000 .000 .000 .000 .000 .000 .000
.000 .000 .000 .013 .013 .011 .011 .001 .001 .004
.004 .004 .004 .000 .000 .000 .000 .003 .003 .009
.009 .030 .008 .011 .011 .019 .019 .032 .031 .022
.022 .012 .012 .002

FOR SUBAREAS:

1 2 3 -1

PLUVIOGRAPH (MT GLORIOUS):

.000 .046 .000 .018 .003 .004 .000 .011 .053 .029
.017 .029 .022 .014 .002 .003 .000 .000 .001 .001
.028 .030 .020 .028 .018 .015 .014 .037 .022 .014
.038 .021 .015 .042 .032 .029 .022 .003 .012 .006
.000 .000 .000 .000 .001 .000 .001 .001 .000 .000
.000 .001 .000 .004 .014 .005 .014 .000 .000 .000
.016 .011 .000 .000 .000 .000 .000 .000 .004 .014
.002 .023 .005 .000 .016 .003 .064 .035 .011 .011
.032 .009 .002 .002

FOR SUBAREAS:

8 9 10 -1

PLUVIOGRAPH (SAMFORD):

.000 .077 .005 .017 .001 .000 .000 .000 .025 .021
.017 .007 .002 .019 .004 .004 .003 .000 .001 .002
.002 .005 .006 .004 .024 .033 .017 .023 .032 .024
.038 .048 .019 .035 .041 .021 .007 .001 .002 .000
.000 .000 .000 .000 .000 .000 .002 .001 .000 .000
.000 .011 .000 .002 .004 .000 .001 .000 .000 .006
.008 .000 .006 .000 .002 .000 .001 .004 .000 .017
.001 .010 .010 .023 .065 .007 .095 .054 .023 .010
.035 .011 .003 .001

FOR SUBAREAS:

4 5 6 7 11 12 -1

LOSS: UNIFORM

RECORDED HYDROGRAPH (GS142202):

3.6	3.7	3.9	7.5	12.6	17.7	17.7	17.7	17.7	17.7
28.1	38.5	48.7	59.1	56.5	53.8	49.8	45.7	41.7	37.6
33.5	29.4	25.4	21.3	34.0	46.6	59.3	71.9	91.1	110.4
129.7	150.4	171.2	191.9	212.6	200.5	188.3	176.2	147.7	119.1
106.2	93.4	80.5	67.6	54.7	41.8	38.5	35.1	31.8	28.4
25.1	21.7	20.6	19.6	18.5	17.5	16.5	15.5	15.7	15.8
15.9	15.3	14.6	14.0	14.1	14.3	14.4	14.0	13.5	13.1
12.7	13.7	14.8	15.8	30.5	45.2	88.5	131.8	123.8	133.0
142.2	151.4	132.2	118.9	102.6	86.4	70.1	53.9	49.1	44.3
39.4	34.6	29.7	24.9	23.3	21.6	20.0	18.3	16.7	15.0
14.3	13.7	13.1	12.5	11.9	11.3	10.7	10.1	9.4	8.8
8.2	7.6	7.4	7.1	6.9	6.6	6.3	6.0	5.8	5.6
5.3	5.1	4.8	4.6	4.4	4.3	4.1	4.0	3.9	3.8
3.6	3.5	3.1	3.0	3.0	2.9	2.8	2.7	2.5	2.3
2.2	2.1	1.9	1.8	1.7	1.6	1.4	1.2	1.1	1.1
1.0	1.0	0.9	0.9	0.8	0.7	0.6	0.6	0.5	0.4
0.4	0.3	0.3	0.1	0.1	0.0				

0900 HRS 6 JULY 1973

FITTING RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 120

RAINFALL IN EACH AREA:

260 220 245 270 250 275 305 305 290 285

300 315

STORM DURATION: 59 HRS

PLUVIOGRAPH (MT NEBO):

.011 .004 .004 .015 .015 .006 .006 .015 .015 .023

.023 .048 .047 .006 .006 .017 .017 .010 .010 .017

.017 .011 .011 .010 .010 .011 .011 .010 .010 .010

.010 .010 .010 .013 .013 .019 .019 .022 .022 .022

.022 .027 .027 .026 .026 .029 .029 .026 .026 .026

.026 .019 .019 .017 .017 .013 .013 .013 .013

FOR SUBAREAS:

1 2 3 -1

PLUVIOGRAPH (MT GLORIOUS):

.009 .004 .011 .009 .001 .006 .004 .010 .006 .021

.038 .046 .035 .002 .000 .005 .008 .004 .010 .006

.009 .011 .025 .023 .017 .006 .002 .003 .004 .004

.004 .019 .034 .048 .035 .027 .053 .046 .046 .026

.026 .033 .032 .027 .026 .020 .019 .023 .022 .022

.013 .011 .005 .011 .012 .011 .001 .006 .003

FOR SUBAREAS:

8 9 10 -1

PLUVIOGRAPH (SAMFORD):

.004 .005 .003 .012 .006 .002 .011 .015 .011 .021

.023 .041 .037 .004 .000 .003 .002 .001 .006 .004

.022 .037 .030 .031 .026 .010 .002 .007 .005 .006

.006 .016 .034 .041 .034 .027 .040 .029 .042 .028

.026 .026 .028 .028 .028 .024 .021 .026 .020 .016

.013 .007 .013 .013 .011 .008 .005 .002 .001

FOR SUBAREAS:

4 5 6 7 11 12 -1

LOSS: UNIFORM

RECORDED HYDROGRAPH (GS142202):

0.0	0.4	0.8	1.1	1.1	1.2	1.2	1.4	2.0	2.8
4.3	5.7	6.1	5.8	5.4	5.0	4.5	7.3	16.3	25.3
34.3	50.6	61.3	72.0	82.7	93.3	104.0	114.7	110.4	106.1
101.7	97.4	93.1	88.8	84.4	80.1	75.8	71.5	67.2	62.8
58.5	54.2	49.9	45.5	41.2	36.9	32.3	28.3	23.9	19.6
15.3	12.2	11.9	11.6	11.4	11.1	10.9	10.5	10.3	10.0
9.8	9.4	9.2	8.9	8.7	8.4	8.1	7.8	7.6	7.3
7.0	6.7	6.5	6.2	6.0	5.6	5.5	5.3	5.2	4.9
4.8	4.6	4.5	4.4	4.1	4.0	3.8	3.7	3.6	3.3
3.2	3.0	2.9	2.7	2.5	2.4	2.2	2.1	1.8	1.7
1.6	1.6	1.5	1.3	1.3	1.2	1.1	1.0	0.9	0.8
0.8	0.7	0.5	0.5	0.4	0.3	0.2	0.1	0.0	0.0

0900 HRS 24 JANUARY 1974

FITTING RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 162

RAINFALL IN EACH AREA:

1150 1050 990 975 980 975 930 1110 1130 960
870 760

STORM DURATION: 96 HRS

PLUVIOGRAPH (MT GLORIOUS):

.001 .002 .006 .010 .006 .007 .001 .002 .008 .000
.004 .005 .006 .004 .003 .007 .007 .016 .015 .016
.014 .038 .026 .023 .009 .014 .018 .045 .028 .018
.005 .015 .004 .004 .009 .010 .027 .024 .006 .002
.005 .005 .007 .009 .008 .003 .002 .004 .002 .000
.002 .002 .004 .005 .007 .010 .018 .018 .032 .046
.040 .032 .021 .027 .025 .012 .015 .018 .014 .010
.007 .010 .008 .009 .006 .011 .006 .010 .006 .007
.008 .004 .003 .002 .002 .004 .006 .001 .001 .002
.003 .007 .001 .004 .002 .002

FOR SUBAREAS:

1 8 9 -1

PLUVIOGRAPH (SAMFORD):

.000 .001 .007 .010 .009 .002 .001 .002 .006 .003
.002 .007 .006 .002 .003 .004 .007 .012 .013 .024
.013 .022 .031 .014 .017 .028 .027 .021 .052 .009
.004 .009 .004 .008 .028 .015 .024 .048 .039 .007
.013 .005 .010 .009 .012 .016 .003 .005 .001 .000
.002 .000 .001 .001 .001 .003 .012 .042 .025 .026
.029 .033 .022 .022 .013 .014 .012 .010 .012 .008
.009 .003 .005 .004 .006 .010 .003 .004 .010 .007
.004 .001 .001 .001 .002 .002 .003 .001 .001 .008
.005 .001 .001 .001 .002 .002

FOR SUBAREAS:

2 3 4 5 10 -1

PLUVIOGRAPH (DEAGON BCC):

.000 .000 .000 .000 .000 .000 .000 .000 .003 .000
.001 .006 .003 .003 .003 .001 .008 .015 .014 .014
.021 .024 .028 .015 .075 .064 .053 .030 .006 .006
.006 .002 .006 .013 .015 .014 .015 .040 .054 .027
.035 .010 .006 .010 .020 .015 .006 .010 .007 .010
.009 .001 .000 .001 .001 .000 .004 .028 .010 .013
.023 .019 .023 .011 .011 .007 .013 .006 .009 .009
.006 .008 .002 .003 .009 .007 .005 .006 .004 .021
.004 .005 .002 .000 .000 .001 .001 .001 .000 .000
.000 .002 .001 .001 .000 .000

FOR SUBAREAS:

6 7 11 12 -1

LOSS: UNIFORM

RECORDED HYDROGRAPH (GS142202):

0.0	0.0	0.0	0.0	0.3	0.5	0.8	1.6	3.6	11.2
16.4	21.2	25.2	29.0	33.0	80.5	112.4	126.4	141.4	173.7
236.3	330.0	437.2	544.4	1078.3	1143.3	1208.3	1238.3	1069.1	899.8
829.5	759.1	904.6	1050.0	484.9	465.7	557.1	648.4	875.7	1129.4
1224.9	1238.7	1170.7	1102.7	875.7	730.3	585.0	497.3	409.5	321.8
276.1	236.1	203.3	171.7	145.7	131.1	125.9	147.1	251.0	446.4
648.2	972.9	770.1	793.2	816.3	769.5	722.7	781.6	704.2	626.7
566.5	506.4	446.4	415.4	380.0	339.7	313.6	301.2	296.6	304.8
293.4	275.7	269.4	261.0	250.7	217.6	185.1	149.8	142.1	134.4
131.4	128.5	125.5	141.6	144.0	132.5	121.1	117.6	110.6	100.2
93.9	88.2	82.7	77.1	71.5	67.9	64.4	60.8	57.2	53.6
50.0	47.1	44.1	41.2	38.2	35.3	32.3	29.3	28.1	26.8
25.6	24.3	23.1	21.8	20.6	19.2	18.0	16.7	15.5	14.2
13.8	13.4	12.9	12.5	12.1	11.7	11.3	10.9	10.4	10.0
9.6	9.1	8.7	8.3	7.9	7.5	7.1	6.7	6.5	6.2
6.0	5.8	5.6	5.4	5.2	4.9	4.7	4.4	4.2	4.0
3.8	3.6								

0600 HRS 21 JUNE 1983

FITTING RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 86

RAINFALL IN EACH AREA:

165 160 145 155 160 165 165 185 170 140

155 165

STORM DURATION: 42 HRS

PLUVIOGRAPH (MT GLORIOUS):

.017 .030 .028 .006 .008 .022 .019 .000 .008 .006

.003 .003 .000 .000 .006 .008 .014 .006 .006 .014

.028 .022 .063 .087 .098 .084 .039 .063 .033 .039

.050 .041 .060 .008 .025 .025 .014 .008 .003 .003

.000 .003

FOR SUBAREAS:

8 9 -1

PLUVIOGRAPH (SAMFORD):

.006 .028 .031 .025 .000 .015 .019 .009 .000 .009

.006 .000 .000 .000 .000 .006 .012 .015 .016 .019

.025 .025 .043 .059 .111 .059 .071 .043 .049 .046

.043 .031 .059 .015 .022 .034 .019 .012 .006 .009

.000 .003

FOR SUBAREAS:

3 4 5 6 7 10 11 12 -1

PLUVIOGRAPH (MT NEBO):

.001 .001 .030 .030 .008 .008 .020 .020 .005 .005

.006 .006 .001 .001 .003 .003 .006 .006 .008 .008

.013 .013 .020 .020 .064 .064 .090 .090 .064 .064

.054 .054 .044 .044 .018 .018 .023 .023 .018 .018

.004 .004

FOR SUBAREAS:

1 2 -1

LOSS: UNIFORM

RECORDED HYDROGRAPH (GS142202):

10.3	9.9	9.3	9.8	10.5	12.5	18.6	28.0	35.6	40.0
44.2	48.2	48.0	46.8	43.5	39.4	35.4	31.8	30.1	30.0
31.3	37.8	47.3	60.4	82.7	119.9	173.8	241.0	303.3	337.5
355.6	349.5	336.0	325.3	316.6	305.8	292.2	271.4	230.0	194.0
167.3	144.6	105.5	83.3	70.3	57.5	48.2	41.5	35.3	31.7
27.5	23.5	20.8	18.4	15.6	14.0	12.3	11.0	10.4	9.4
8.8	8.1	7.5	6.8	6.2	5.5	4.4	4.1	3.6	3.3
3.0	2.7	2.3	2.0	1.7	1.5	1.4	1.2	1.1	0.9
0.7	0.5	0.4	0.3	0.2	0.1				

0900 HRS 3 APRIL 1988

FITTING RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 106

RAINFALL IN EACH AREA:

470 425 390 325 320 295 310 500 480 405
345 340

STORM DURATION: 84 HRS

PLUVIOGRAPH (DAYBORO):

.000 .002 .001 .002 .005 .005 .002 .005 .004 .006
.032 .024 .024 .025 .010 .006 .036 .018 .001 .006
.002 .008 .006 .001 .001 .001 .001 .000 .002 .000
.000 .000 .000 .004 .000 .004 .000 .000 .000 .001
.008 .024 .041 .043 .059 .005 .019 .037 .001 .012
.002 .010 .027 .020 .011 .004 .006 .008 .012 .004
.006 .017 .031 .007 .018 .007 .005 .036 .051 .044
.016 .010 .017 .000 .012 .005 .006 .000 .001 .000
.056 .057 .000 .000

FOR SUBAREAS:

8 9 10 11 12 -1

PLUVIOGRAPH (FERNY HILLS):

.000 .009 .002 .002 .004 .000 .002 .002 .011 .011
.015 .018 .029 .041 .031 .020 .026 .018 .009 .008
.005 .010 .006 .005 .000 .000 .002 .002 .000 .002
.000 .002 .004 .000 .002 .004 .000 .000 .015 .011
.038 .044 .082 .076 .025 .015 .015 .009 .002 .002
.007 .016 .007 .025 .007 .011 .007 .007 .009 .005
.007 .038 .018 .002 .005 .005 .000 .004 .022 .018
.004 .000 .000 .000 .000 .015 .002 .007 .015 .033
.045 .018 .005 .000

FOR SUBAREAS:

1 2 3 4 5 6 7 -1

LOSS: UNIFORM

RECORDED HYDROGRAPH (GS142202):

2.5	2.5	2.4	2.3	2.3	2.3	2.3	2.2	2.2	2.2
2.2	2.3	3.1	9.2	38.7	98.8	223.7	248.1	233.6	203.1
154.3	104.6	86.0	71.7	65.7	59.6	55.1	50.0	45.1	40.5
35.7	32.8	30.7	28.7	26.7	25.1	23.9	22.4	21.3	20.1
19.7	22.0	34.4	73.6	191.9	328.7	426.7	494.7	520.0	506.7
416.6	362.7	239.0	170.2	153.1	132.6	104.9	91.6	97.4	139.0
161.3	173.9	169.9	161.0	137.9	108.2	94.4	112.5	144.8	161.0
193.9	286.5	360.2	378.0	329.2	241.9	190.0	153.3	127.1	119.1
119.3	163.8	195.0	243.7	282.0	280.4	231.7	156.7	108.3	84.2
73.0	65.6	58.1	51.6	46.6	42.5	38.3	34.2	31.7	29.5
27.4	25.3	24.1	22.8	21.6	20.4				

0900 HRS 1 APRIL 1989

FITTING RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 72

RAINFALL IN EACH AREA:

380 372 343 312 324 292 289 382 372 340
310 298

STORM DURATION: 72 HRS

PLUVIOGRAPH (MT GLORIOUS):

0.005	0.035	0.002	0.003	0.011	0.005	0.007	0.012
0.027	0.026	0.035	0.003	0	0	0	0.016
0.015	0.053	0.03	0.015	0.008	0.039	0.068	0.056
0.031	0.105	0.08	0.029	0.054	0.027	0.018	0.004
0.011	0.005	0	0.005	0.008	0	0	0.004
0	0	0.001	0	0	0	0	0
0	0	0.003	0.008	0.014	0.019	0.014	0.016
0.03	0.008	0.01	0.001	0	0.004	0.003	0
0.001	0.001	0.011	0	0.003	0	0.001	0

FOR SUBAREAS:

1 8 -1

PLUVIOGRAPH (SAMFORD):

0.007	0.03	0.007	0	0.036	0.01	0.002	0.012
0.086	0.105	0.059	0.015	0	0.003	0.021	0.01
0.01	0.053	0.002	0.001	0.01	0.058	0.03	0.048
0.062	0.076	0.053	0.043	0.033	0.021	0.012	0.003
0.005	0.003	0	0	0.002	0	0.005	0
0	0	0	0	0.003	0	0	0
0	0.002	0.002	0.007	0.007	0.003	0.012	0.012
0.005	0.002	0	0	0	0.007	0	0
0	0	0	0	0.003	0	0.002	0

FOR SUBAREAS:

2 3 4 5 6 -1

PLUVIOGRAPH (DAYBORO):

0.019	0.005	0	0.02	0.006	0.003	0.023	0.096
0.032	0.023	0.011	0.003	0	0.002	0.009	0.002
0.009	0.009	0.002	0.006	0.026	0.091	0.071	0.046
0.062	0.059	0.063	0.063	0.064	0.04	0.028	0.014
0.008	0.002	0.005	0.006	0	0	0	0
0	0	0	0	0	0	0	0
0	0	0.002	0.006	0.003	0.012	0.005	0.029
0.005	0.008	0	0	0	0.002	0	0
0	0	0	0	0	0	0	0

FOR SUBAREAS:

7 9 10 11 12 -1

LOSS: UNIFORM

RECORDED HYDROGRAPH (GS142202):

7.0 7.0 7.8 9.0 11.0 16.6 53.2 76.3 86.2 87.0 87.7 113.5
206.2 260.8 311.7 339.0 330.4 307.7 223.7 173.7 176.0
204.3 208.1 212.0 243.0 237.8 246.0 327.0 444.1 617.7
723.1 922.8 882.5 692.4 596.8 478.5 403.2 358.3 225.7
158.2 115.0 96.9 85.0 75.0 65.9 56.5 50.0 45.0 39.3 35.0
30.0 29.0 29.0 28.0 27.0 27.1 27.5 35.3 50.0 55.4 70.0
80.0 87.5 81.0 71.0 58.8 54.0 47.0 45.0 36.6 35.2 32.0

A5 SOUTH PINE RIVER @ CASH'S CROSSING

```
METRIC UNITS.
13 SUBAREAS OF AREA:
14.07 10.20 20.17  8.87 12.93
12.53 13.53 12.87 11.53 13.03
14.40 14.73 19.90
RAIN ON AREA #  1  K1=0.237
STORE HYDROGRAPH.
RAIN ON AREA #  2  K1=0.136
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.203
STORE HYDROGRAPH.
RAIN ON AREA #  3  K1=0.169
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.079
RAIN ON AREA #  4  K1=0.124
STORE HYDROGRAPH.
RAIN ON AREA #  5  K1=0.237
RAIN ON AREA #  6  K1=0.181
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.203
RAIN ON AREA #  7  K1=0.260
STORE HYDROGRAPH.
RAIN ON AREA #  8  K1=0.260
STORE HYDROGRAPH.
RAIN ON AREA #  9  K1=0.113
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.136
RAIN ON AREA # 10  K1=0.429
RAIN ON AREA # 11  K1=0.305
STORE HYDROGRAPH.
RAIN ON AREA # 12  K1=0.181
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.113
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.028
P&P HYDROGRAPH. GS 142202 SOUTH PINE RIVER @ DRAPERS CROSSING
ROUTE HYDROGRAPH   K1=0.198
RAIN ON AREA # 13  K1=0.124
P&P HYDROGRAPH. GS 142201 SOUTH PINE RIVER @ CASHS CROSSING
END
```

0800 HRS 9 JUNE 1967

DESIGN RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 148

RAINFALL IN EACH AREA:

250 275 240 250 270 255 235 250 245 235

230 225 220

STORM DURATION: 66 HRS

PLUVIOGRAPH (MT NEBO):

.029 .021 .010 .020 .012 .016 .022 .016 .011 .005

.003 .011 .008 .007 .014 .013 .005 .004 .013 .005

.004 .012 .010 .011 .020 .017 .017 .016 .014 .010

.003 .000 .000 .000 .000 .000 .000 .001 .000 .000

.000 .000 .000 .000 .000 .000 .001 .004 .007 .039

.060 .030 .047 .027 .025 .019 .015 .021 .014 .032

.028 .045 .078 .080 .045 .003

FOR SUBAREAS:

1 2 8 9 -1

PLUVIOGRAPH (SAMFORD):

.023 .022 .008 .008 .002 .003 .011 .014 .013 .004

.002 .003 .001 .002 .008 .006 .006 .006 .004 .004

.002 .010 .008 .011 .034 .018 .022 .008 .013 .016

.008 .000 .000 .000 .000 .000 .000 .003 .000 .002

.001 .001 .000 .000 .000 .001 .004 .028 .042 .044

.067 .033 .053 .030 .028 .021 .017 .023 .028 .085

.103 .072 .014 .000 .000 .000

FOR SUBAREAS:

3 4 5 6 7 10 11 12 13 -1

LOSS: UNIFORM

0900 HRS 6 DECEMBER 1970

DESIGN RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 165

RAINFALL IN EACH AREA:

215 200 210 235 220 245 260 260 235 220
235 255 260

STORM DURATION: 84 HRS

PLUVIOGRAPH (MT NEBO):

.002 .013 .013 .015 .015 .004 .004 .018 .018 .038
.038 .015 .015 .018 .018 .008 .008 .002 .002 .005
.005 .023 .023 .030 .015 .014 .014 .033 .033 .023
.023 .030 .030 .024 .024 .021 .021 .010 .010 .001
.001 .001 .001 .000 .000 .000 .000 .000 .000 .000
.000 .000 .000 .013 .013 .011 .011 .001 .001 .004
.004 .004 .004 .000 .000 .000 .000 .003 .003 .009
.009 .030 .008 .011 .011 .019 .019 .032 .031 .022
.022 .012 .012 .002

FOR SUBAREAS:

1 2 3 -1

PLUVIOGRAPH (MT GLORIOUS):

.000 .046 .000 .018 .003 .004 .000 .011 .053 .029
.017 .029 .022 .014 .002 .003 .000 .000 .001 .001
.028 .030 .020 .028 .018 .015 .014 .037 .022 .014
.038 .021 .015 .042 .032 .029 .022 .003 .012 .006
.000 .000 .000 .000 .001 .000 .001 .001 .000 .000
.000 .001 .000 .004 .014 .005 .014 .000 .000 .000
.016 .011 .000 .000 .000 .000 .000 .000 .004 .014
.002 .023 .005 .000 .016 .003 .064 .035 .011 .011
.032 .009 .002 .002

FOR SUBAREAS:

8 9 10 -1

PLUVIOGRAPH (SAMFORD):

.000 .077 .005 .017 .001 .000 .000 .000 .025 .021
.017 .007 .002 .019 .004 .004 .003 .000 .001 .002
.002 .005 .006 .004 .024 .033 .017 .023 .032 .024
.038 .048 .019 .035 .041 .021 .007 .001 .002 .000
.000 .000 .000 .000 .000 .000 .002 .001 .000 .000
.000 .011 .000 .002 .004 .000 .001 .000 .000 .006
.008 .000 .006 .000 .002 .000 .001 .004 .000 .017
.001 .010 .010 .023 .065 .007 .095 .054 .023 .010
.035 .011 .003 .001

FOR SUBAREAS:

4 5 6 7 11 12 13 -1

LOSS: UNIFORM

0900 HRS 6 JULY 1973

DESIGN RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 120

RAINFALL IN EACH AREA:

260 220 245 270 250 275 305 305 290 285
300 315 320

STORM DURATION: 59 HRS

PLUVIOGRAPH (MT NEBO):

.011 .004 .004 .015 .015 .006 .006 .015 .015 .023
.023 .048 .047 .006 .006 .017 .017 .010 .010 .017
.017 .011 .011 .010 .010 .011 .011 .010 .010 .010
.010 .010 .010 .013 .013 .019 .019 .022 .022 .022
.022 .027 .027 .026 .026 .029 .029 .026 .026 .026
.026 .019 .019 .017 .017 .013 .013 .013 .013

FOR SUBAREAS:

1 2 3 -1

PLUVIOGRAPH (MT GLORIOUS):

.009 .004 .011 .009 .001 .006 .004 .010 .006 .021
.038 .046 .035 .002 .000 .005 .008 .004 .010 .006
.009 .011 .025 .023 .017 .006 .002 .003 .004 .004
.004 .019 .034 .048 .035 .027 .053 .046 .046 .026
.026 .033 .032 .027 .026 .020 .019 .023 .022 .022
.013 .011 .005 .011 .012 .011 .001 .006 .003

FOR SUBAREAS:

8 9 10 -1

PLUVIOGRAPH (SAMFORD):

.004 .005 .003 .012 .006 .002 .011 .015 .011 .021
.023 .041 .037 .004 .000 .003 .002 .001 .006 .004
.022 .037 .030 .031 .026 .010 .002 .007 .005 .006
.006 .016 .034 .041 .034 .027 .040 .029 .042 .028
.026 .026 .028 .028 .028 .024 .021 .026 .020 .016
.013 .007 .013 .013 .011 .008 .005 .002 .001

FOR SUBAREAS:

4 5 6 7 11 12 13 -1

LOSS: UNIFORM

0900 HRS 24 JANUARY 1974

DESIGN RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 192

RAINFALL IN EACH AREA:

1150 1050 990 975 980 975 930 1110 1130 960
870 760 760

STORM DURATION: 96 HRS

PLUVIOGRAPH (MT GLORIOUS):

.001 .002 .006 .010 .006 .007 .001 .002 .008 .000
.004 .005 .006 .004 .003 .007 .007 .016 .015 .016
.014 .038 .026 .023 .009 .014 .018 .045 .028 .018
.005 .015 .004 .004 .009 .010 .027 .024 .006 .002
.005 .005 .007 .009 .008 .003 .002 .004 .002 .000
.002 .002 .004 .005 .007 .010 .018 .018 .032 .046
.040 .032 .021 .027 .025 .012 .015 .018 .014 .010
.007 .010 .008 .009 .006 .011 .006 .010 .006 .007
.008 .004 .003 .002 .002 .004 .006 .001 .001 .002
.003 .007 .001 .004 .002 .002

FOR SUBAREAS:

1 8 9 -1

PLUVIOGRAPH (SAMFORD):

.000 .001 .007 .010 .009 .002 .001 .002 .006 .003
.002 .007 .006 .002 .003 .004 .007 .012 .013 .024
.013 .022 .031 .014 .017 .028 .027 .021 .052 .009
.004 .009 .004 .008 .028 .015 .024 .048 .039 .007
.013 .005 .010 .009 .012 .016 .003 .005 .001 .000
.002 .000 .001 .001 .001 .003 .012 .042 .025 .026
.029 .033 .022 .022 .013 .014 .012 .010 .012 .008
.009 .003 .005 .004 .006 .010 .003 .004 .010 .007
.004 .001 .001 .001 .002 .002 .003 .001 .001 .008
.005 .001 .001 .001 .002 .002

FOR SUBAREAS:

2 3 4 5 10 -1

PLUVIOGRAPH (DEAGON BCC):

.000 .000 .000 .000 .000 .000 .000 .000 .003 .000
.001 .006 .003 .003 .003 .001 .008 .015 .014 .014
.021 .024 .028 .015 .075 .064 .053 .030 .006 .006
.006 .002 .006 .013 .015 .014 .015 .040 .054 .027
.035 .010 .006 .010 .020 .015 .006 .010 .007 .010
.009 .001 .000 .001 .001 .000 .004 .028 .010 .013
.023 .019 .023 .011 .011 .007 .013 .006 .009 .009
.006 .008 .002 .003 .009 .007 .005 .006 .004 .021
.004 .005 .002 .000 .000 .001 .001 .001 .000 .000
.000 .002 .001 .001 .000 .000

FOR SUBAREAS:

6 7 11 12 13 -1

LOSS: UNIFORM

0600 HRS 21 JUNE 1983

DESIGN RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 86

RAINFALL IN EACH AREA:

165 160 145 155 160 165 165 185 170 140

155 165 170

STORM DURATION: 42 HRS

PLUVIOGRAPH (MT GLORIOUS):

.017 .030 .028 .006 .008 .022 .019 .000 .008 .006

.003 .003 .000 .000 .006 .008 .014 .006 .006 .014

.028 .022 .063 .087 .098 .084 .039 .063 .033 .039

.050 .041 .060 .008 .025 .025 .014 .008 .003 .003

.000 .003

FOR SUBAREAS:

8 9 -1

PLUVIOGRAPH (SAMFORD):

.006 .028 .031 .025 .000 .015 .019 .009 .000 .009

.006 .000 .000 .000 .000 .006 .012 .015 .016 .019

.025 .025 .043 .059 .111 .059 .071 .043 .049 .046

.043 .031 .059 .015 .022 .034 .019 .012 .006 .009

.000 .003

FOR SUBAREAS:

3 4 5 6 7 10 11 12 13 -1

PLUVIOGRAPH (MT NEBO):

.001 .001 .030 .030 .008 .008 .020 .020 .005 .005

.006 .006 .001 .001 .003 .003 .006 .006 .008 .008

.013 .013 .020 .020 .064 .064 .090 .090 .064 .064

.054 .054 .044 .044 .018 .018 .023 .023 .018 .018

.004 .004

FOR SUBAREAS:

1 2 -1

LOSS: UNIFORM

0900 HRS 3 APRIL 1988

DESIGN RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 106

RAINFALL IN EACH AREA:

470 425 390 325 320 295 310 500 480 405

345 340 330

STORM DURATION: 84 HRS

PLUVIOGRAPH (DAYBORO):

.000 .002 .001 .002 .005 .005 .002 .005 .004 .006

.032 .024 .024 .025 .010 .006 .036 .018 .001 .006

.002 .008 .006 .001 .001 .001 .001 .000 .002 .000

.000 .000 .000 .004 .000 .004 .000 .000 .000 .001

.008 .024 .041 .043 .059 .005 .019 .037 .001 .012

.002 .010 .027 .020 .011 .004 .006 .008 .012 .004

.006 .017 .031 .007 .018 .007 .005 .036 .051 .044

.016 .010 .017 .000 .012 .005 .006 .000 .001 .000

.056 .057 .000 .000

FOR SUBAREAS:

8 9 10 11 12 13 -1

PLUVIOGRAPH (FERNY HILLS):

.000 .009 .002 .002 .004 .000 .002 .002 .011 .011

.015 .018 .029 .041 .031 .020 .026 .018 .009 .008

.005 .010 .006 .005 .000 .000 .002 .002 .000 .002

.000 .002 .004 .000 .002 .004 .000 .000 .015 .011

.038 .044 .082 .076 .025 .015 .015 .009 .002 .002

.007 .016 .007 .025 .007 .011 .007 .007 .009 .005

.007 .038 .018 .002 .005 .005 .000 .004 .022 .018

.004 .000 .000 .000 .000 .015 .002 .007 .015 .033

.045 .018 .005 .000

FOR SUBAREAS:

1 2 3 4 5 6 7 -1

LOSS: UNIFORM

0900 HRS 1 APRIL 1989

DESIGN RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 72

RAINFALL IN EACH AREA:

380 372 343 312 324 292 289 382 372 340

310 298 267

STORM DURATION: 72 HRS

PLUVIOGRAPH (MT GLORIOUS):

.005	.035	.002	.003	.011	.005	.007	.012
.027	.026	.035	.003	0	0	0	.016
.015	.053	.030	.015	.008	.039	.068	.056
.031	.105	.080	.029	.054	.027	.018	.004
.011	.005	0	.005	.008	0	0	.004
0	0	.001	0	0	0	0	0
0	0	.003	.008	.014	.019	.014	.016
.030	.008	.010	.001	0	.004	.003	0
.001	.001	.011	0	.003	0	.001	0

FOR SUBAREAS:

1 8 -1

PLUVIOGRAPH (SAMFORD):

.007	.030	.007	0	.036	.010	.002	.012
.086	.105	.059	.015	0	.003	.021	.010
.010	.053	.002	.001	.010	.058	.030	.048
.062	.076	.053	.043	.033	.021	.012	.003
.005	.003	0	0	.002	0	.005	0
0	0	0	0	.003	0	0	0
0	.002	.002	.007	.007	.003	.012	.012
.005	.002	0	0	0	.007	0	0
0	0	0	0	.003	0	.002	0

FOR SUBAREAS:

2 3 4 5 6 -1

PLUVIOGRAPH (DAYBORO):

.019	.005	0	.020	.006	.003	.023	.096
.032	.023	.011	.003	0	.002	.009	.002
.009	.009	.002	.006	.026	.091	.071	.046
.062	.059	.063	.063	.064	.040	.028	.014
.008	.002	.005	.006	0	0	0	0
0	0	0	0	0	0	0	0
0	0	.002	.006	.003	.012	.005	.029
.005	.008	0	0	0	.002	0	0
0	0	0	0	0	0	0	0

FOR SUBAREAS:

7 9 10 11 12 13 -1

LOSS: UNIFORM

A6 SIDELING CREEK @ LAKE KURWONGABAH

```
METRIC UNITS.
8 SUBAREAS OF AREA:
8.70 7.60 5.95 5.15 8.65 4.40 9.80 2.30
RAIN ON AREA # 1   K1=0.621
RAIN ON AREA # 2   K1=0.310
STORE HYDROGRAPH.
RAIN ON AREA # 3   K1=0.466
RAIN ON AREA # 4   K1=0.285
GET HYDROGRAPH.
ROUTE HYDROGRAPH   K1=0.233
STORE HYDROGRAPH.
RAIN ON AREA # 5   K1=0.207
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 6   K1=0.078
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 7   K1=0.414
GET HYDROGRAPH.
STORE HYDROGRAPH.
RAIN ON AREA # 8   K1=0.0
GET HYDROGRAPH.
DAM ROUTE          VBF=0     TABLE NO OF VALUES=17
    0   0
  337  13
 1390  54
 2443  95
 3496 161
 4643 243
 5844 343
 7045 443
 8245 571
 9446 664
10647 741
12001 785
13477 827
14952 862
16427 896
17903 929
18345 940
P&P HYDROGRAPH.LAKE KURWONGBAH SPILLWAY FLOW
END
```

0800 HRS 9 JUNE 1967

DESIGN RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 148

RAINFALL IN EACH AREA:

185 160 160 165 160 180 180 180

STORM DURATION: 66 HRS

PLUVIOGRAPH (ENOGGERA RES):

.004 .008 .006 .009 .000 .002 .007 .006 .006 .003

.001 .002 .002 .001 .007 .005 .006 .006 .003 .004

.003 .009 .008 .007 .015 .016 .024 .013 .008 .016

.005 .000 .000 .000 .000 .000 .001 .002 .001 .001

.003 .002 .000 .001 .001 .000 .013 .031 .019 .026

.063 .046 .037 .029 .025 .026 .025 .018 .025 .041

.107 .176 .062 .006 .001 .000

FOR SUBAREAS:

1 2 3 4 5 6 7 8 -1

LOSS: UNIFORM

0900 HRS 12 DECEMBER 1970

DESIGN RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 171

RAINFALL IN EACH AREA:

320 340 310 320 345 320 315 320

STORM DURATION: 84 HRS

PLUVIOGRAPH (DAYBORO):

.011 .014 .036 .017 .000 .000 .000 .001 .005 .011
.019 .016 .007 .009 .005 .003 .002 .000 .008 .002
.023 .026 .019 .026 .026 .028 .020 .045 .012 .026
.038 .052 .026 .063 .036 .034 .014 .026 .021 .001
.000 .001 .000 .000 .000 .001 .001 .002 .000 .000
.000 .012 .000 .002 .004 .000 .001 .000 .000 .007
.009 .000 .007 .000 .002 .000 .001 .004 .000 .018
.001 .011 .001 .004 .016 .004 .043 .044 .012 .012
.040 .008 .002 .002

FOR SUBAREAS:

1 2 3 4 5 6 7 8 -1

LOSS: UNIFORM

0900 HRS 6 JULY 1973

DESIGN RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 80

RAINFALL IN EACH AREA:

400 400 385 385 390 365 365 370

STORM DURATION: 59 HRS

PLUVIOGRAPH (DAYBORO):

.003 .004 .011 .011 .008 .009 .004 .011 .006 .022

.040 .034 .002 .002 .000 .013 .000 .004 .006 .005

.004 .005 .024 .022 .013 .002 .000 .001 .002 .003

.007 .012 .037 .026 .030 .050 .041 .025 .051 .053

.049 .035 .028 .033 .033 .026 .023 .030 .025 .023

.026 .024 .010 .013 .012 .003 .002 .003 .000

FOR SUBAREAS:

1 2 3 4 5 7 -1

PLUVIOGRAPH (DEAGON):

.009 .005 .003 .007 .001 .006 .010 .007 .014 .021

.012 .001 .001 .004 .000 .005 .004 .004 .013 .029

.022 .022 .029 .034 .019 .004 .011 .005 .005 .012

.019 .032 .028 .036 .104 .034 .030 .055 .044 .035

.025 .025 .026 .029 .026 .020 .027 .013 .011 .014

.018 .009 .007 .006 .004 .001 .002 .001 .000

FOR SUBAREAS:

6 8 -1

LOSS: UNIFORM

```
0900 HRS 24 JANUARY 1974
DESIGN RUN
TIME INCREMENTS: 1 HRS
NUMBER OF ORDINATES: 136
RAINFALL IN EACH AREA:
800 780 790 770 725 720 725 720
STORM DURATION: 96 HRS
PLUVIOGRAPH (DAYBORO):
.001 .003 .008 .013 .008 .009 .001 .003 .010 .000
.005 .006 .008 .005 .004 .009 .009 .021 .019 .021
.018 .049 .033 .029 .016 .015 .018 .015 .026 .031
.020 .015 .005 .007 .006 .007 .018 .016 .004 .001
.003 .003 .005 .006 .005 .002 .001 .003 .002 .000
.002 .002 .003 .004 .006 .008 .015 .015 .026 .038
.033 .026 .017 .022 .021 .010 .012 .015 .012 .008
.006 .008 .010 .012 .008 .014 .008 .013 .008 .009
.010 .005 .004 .003 .003 .005 .008 .001 .001 .003
.004 .009 .001 .005 .003 .002
FOR SUBAREAS:
 1 2 3 4 5 7 -1
PLUVIOGRAPH (DEAGON BCC):
.000 .000 .000 .000 .000 .000 .000 .000 .003 .000
.001 .006 .003 .003 .003 .001 .008 .015 .014 .014
.021 .024 .028 .015 .075 .064 .053 .030 .006 .006
.006 .002 .006 .013 .015 .014 .015 .040 .054 .027
.035 .010 .006 .010 .020 .015 .006 .010 .007 .010
.009 .001 .000 .001 .001 .000 .004 .028 .010 .013
.023 .019 .023 .011 .011 .007 .013 .006 .009 .009
.006 .008 .002 .003 .009 .007 .005 .006 .004 .021
.004 .005 .002 .000 .000 .001 .001 .001 .000 .000
.000 .002 .001 .001 .000 .000
FOR SUBAREAS:
 6 8 -1
LOSS: UNIFORM
```

0600 HRS 21 JUNE 1983

DESIGN RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 80

RAINFALL IN EACH AREA:

210 215 205 205 215 205 200 205

STORM DURATION: 42 HRS

PLUVIOGRAPH (DAYBORO):

.012 .024 .042 .003 .000 .012 .018 .003 .003 .003

.000 .006 .000 .000 .000 .003 .018 .015 .006 .012

.042 .021 .045 .045 .071 .051 .054 .042 .042 .039

.059 .048 .109 .077 .006 .033 .024 .006 .006 .000

.000 .000

FOR SUBAREAS:

1 3 -1

PLUVIOGRAPH (NORTH PINE DAM):

.008 .018 .026 .006 .000 .022 .033 .006 .006 .006

.000 .010 .003 .000 .000 .005 .015 .020 .010 .005

.041 .020 .046 .064 .074 .056 .036 .023 .033 .028

.051 .079 .049 .090 .023 .041 .023 .008 .008 .005

.003 .000

FOR SUBAREAS:

2 3 4 5 6 7 8 -1

LOSS: UNIFORM

0900 HRS APRIL 1988

DESIGN RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 120

RAINFALL IN EACH AREA:

395 360 395 375 340 345 365 350

STORM DURATION: 84 HRS

PLUVIOGRAPH (DAYBORO):

.000 .002 .001 .002 .005 .005 .002 .005 .004 .006
.032 .024 .024 .025 .010 .006 .036 .018 .001 .006
.002 .008 .006 .001 .001 .001 .001 .000 .002 .000
.000 .000 .000 .004 .000 .004 .000 .000 .000 .001
.008 .024 .041 .043 .059 .005 .019 .037 .001 .012
.002 .010 .027 .020 .011 .004 .006 .008 .012 .004
.006 .017 .031 .007 .018 .007 .005 .036 .051 .044
.016 .010 .017 .000 .012 .005 .006 .000 .001 .000
.056 .057 .000 .000

FOR SUBAREAS:

1 2 3 4 5 7 -1

PLUVIOGRAPH (DEAGON BCC):

.005 .000 .002 .000 .000 .000 .000 .000 .002 .002
.002 .016 .017 .024 .021 .049 .063 .030 .023 .035
.016 .005 .023 .019 .000 .010 .009 .002 .000 .000
.000 .000 .005 .002 .002 .000 .000 .000 .000 .007
.042 .028 .030 .143 .007 .002 .031 .016 .003 .005
.000 .010 .009 .031 .002 .002 .012 .007 .007 .000
.003 .042 .000 .000 .002 .012 .000 .002 .009 .003
.000 .002 .000 .017 .021 .054 .045 .010 .000 .000
.000 .000 .000 .000

FOR SUBAREAS:

6 8 -1

LOSS: UNIFORM

0900 HRS 1 APRIL 1989

DESIGN RUN

TIME INCREMENTS: 1 HRS

NUMBER OF ORDINATES: 72

RAINFALL IN EACH AREA:

305 286 304 291 274 275 289 275

STORM DURATION: 72 HRS

PLUVIOGRAPH (DAYBORO):

0.019	0.005	0	0.02	0.006	0.003	0.023	0.096
0.032	0.023	0.011	0.003	0	0.002	0.009	0.002
0.009	0.009	0.002	0.006	0.026	0.091	0.071	0.046
0.062	0.059	0.063	0.063	0.064	0.04	0.028	0.014
0.008	0.002	0.005	0.006	0	0	0	0
0	0	0	0	0	0	0	0
0	0	0.002	0.006	0.003	0.012	0.005	0.029
0.005	0.008	0	0	0	0.002	0	0
0	0	0	0	0	0	0	0

FOR SUBAREAS:

1 2 3 4 5 6 7 8 -1

LOSS: UNIFORM

APPENDIX B

DESIGN STORM RAINFALL DEPTH ESTIMATES

DAYBORO STORM RAINFALL DEPTH ESTIMATES

Duration (hrs)	Average Storm Recurrence Interval (Years)						
	1	2	5	10	20	50	100
1	36	46	59	67	77	92	103
3	54	70	90	102	118	140	157
6	70	90	117	132	153	182	204
12	90	116	151	171	199	236	266
24	114	149	199	229	270	326	370
48	140	186	256	300	358	440	505
72	154	206	289	342	413	512	593

DEAGON STORM RAINFALL DEPTH ESTIMATES

Duration (hrs)	Average Storm Recurrence Interval (Years)						
	1	2	5	10	20	50	100
1	37	48	62	70	81	95	107
3	53	68	89	102	118	141	159
6	65	85	112	128	150	179	202
12	81	106	141	161	189	227	257
24	106	139	186	215	253	306	347
48	135	178	241	280	331	402	458
72	151	199	274	318	378	461	526

MT GLORIOUS STORM RAINFALL DEPTH ESTIMATES

Duration (hrs)	Average Storm Recurrence Interval (Years)						
	1	2	5	10	20	50	100
1	35	46	59	67	78	92	104
3	54	71	94	107	126	152	172
6	70	92	124	143	170	206	235
12	91	121	165	192	229	280	322
24	123	163	225	263	315	388	447
48	162	216	300	353	425	525	607
72	186	248	347	410	494	613	710

MT MEE STORM RAINFALL DEPTH ESTIMATES

Duration (hrs)	Average Storm Recurrence Interval (Years)						
	1	2	5	10	20	50	100
1	35	45	57	65	75	88	99
3	51	66	87	100	117	141	160
6	64	84	113	131	155	188	215
12	80	106	146	171	205	252	290
24	105	139	192	224	268	330	380
48	133	177	244	287	343	423	487
72	150	199	275	323	386	476	549

MT NEBO STORM RAINFALL DEPTH ESTIMATES

Duration (hrs)	Average Storm Recurrence Interval (Years)						
	1	2	5	10	20	50	100
1	35	46	59	67	79	94	106
3	53	70	93	107	126	152	173
6	69	91	123	142	169	206	235
12	89	118	162	190	227	278	320
24	119	158	216	252	301	369	424
48	156	205	281	328	391	479	550
72	177	234	320	372	444	543	624

NORTH PINE DAM STORM RAINFALL DEPTH ESTIMATES

Duration (hrs)	Average Storm Recurrence Interval (Years)						
	1	2	5	10	20	50	100
1	36	46	60	69	80	95	107
3	54	70	91	103	120	143	160
6	69	90	117	132	153	182	205
12	89	115	150	170	197	234	263
24	114	148	194	221	258	308	347
48	143	186	246	282	330	396	447
72	158	207	275	316	371	446	506

NARANGBA STORM RAINFALL DEPTH ESTIMATES

Duration (hrs)	Average Storm Recurrence Interval (Years)						
	1	2	5	10	20	50	100
1	36	47	61	69	80	94	106
3	55	71	92	103	120	142	159
6	70	91	117	132	153	181	203
12	90	117	151	170	197	233	261
24	114	150	198	227	265	318	359
48	142	187	254	295	349	424	484
72	156	207	287	336	400	490	563

SAMFORD STORM RAINFALL DEPTH ESTIMATES

Duration (hrs)	Average Storm Recurrence Interval (Years)						
	1	2	5	10	20	50	100
1	35	46	61	69	81	97	110
3	53	69	91	103	121	145	163
6	68	89	116	132	154	184	208
12	88	114	149	169	197	235	265
24	113	147	194	221	258	310	350
48	142	185	246	282	331	398	451
72	158	207	275	315	372	449	510

PETRIE STORM RAINFALL DEPTH ESTIMATES

Duration (hrs)	Average Storm Recurrence Interval (Years)						
	1	2	5	10	20	50	100
1	36	47	61	70	81	97	109
3	53	69	91	103	120	143	161
6	68	88	115	131	152	181	204
12	86	112	147	166	194	231	259
24	110	144	191	218	256	307	347
48	137	180	242	279	330	398	453
72	152	199	271	315	373	453	516

APPENDIX C

BUREAU OF METEOROLOGY REPORT ON

PINE RIVER PMP ESTIMATES

PROBABLE MAXIMUM PRECIPITATION STUDIES

FOR THE PINE RIVER CATCHMENT,

THE NORTH PINE DAM AND THE

SIDELING CREEK DAM CATCHMENTS

Bureau of Meteorology

Queensland Regional Office

February 1991

PMP STUDIES FOR THE PINE RIVER, THE NORTH PINE DAM AND SIDELING CREEK DAM CATCHMENTS

Table of Contents

PMP STUDIES FOR THE PINE RIVER, THE NORTH PINE DAM AND SIDELING CREEK DAM CATCHMENTS

1. INTRODUCTION

The Water Resources Commission, Brisbane, have requested the Bureau of Meteorology to provide estimates of probable maximum precipitation (PMP) for the catchments:

a) The Pine River above the mouth - 708 km^2,
b) The North Pine Dam - 347 km^2,
c) The Sideling Creek Dam - 53 km^2.

Estimates of PMP for durations from 1 hour to 7 days have been provided. A location map is shown at Figure 1.

2. METHODS FOR ESTIMATING PMP

The size and location of the subject catchments determines the range of methods of PMP estimation that can be utilised in this study. The Bulletin 51 Method can be applied to small areas (less than 1000 sq kms) and for durations up to 6 hours. A generalised method has been developed for durations from 6 to 96 hours for areas that may be affected by tropical storms. The Gordon Method extends the duration of the tropical storm methodology out to 7 days.

All three methods are applicable to the subject catchments and have been employed in these studies.

3. BULLETIN 51 METHOD OF PMP ESTIMATION for durations up to 6 hours

Procedures used in this method have been published by the Bureau of Meteorology (1985) and involve the use of enveloping depth-duration-area curves. Sets of curves have been provided for two categories of topography - "smooth" and "rough", the latter being defined as areas in which elevation changes of 50 metres or more within 400 metres are common. By this standard the subject catchments are partially rough, however, "if a catchment includes a smooth or flat area of land within 20km of generally rough terrain the whole catchment should be classified as rough." (The Institution of Engineers, 1987). "Rugged terrain, as well as triggering more convective storms than flat terrain, tends to hold them in place, thus increasing rainfall over a given area." The subject catchments are overshadowed by the rugged terrain of the D'Aguilar Range, accordingly, their topography must be classified as rough.

Values applicable to the subject catchments were extracted from the curves for durations up to 6 hours. As the curves are standardised to the moisture content of a saturated atmospheric column with a surface dewpoint of 28°C and a surface pressure of 1000 hPa, adjustments were made for the highest expected moisture content over the subject catchment.

The extreme 24-hour persisting dewpoint temperature for the catchments were extracted from maps prepared by the Bureau of Meteorology, and the values derived from the depth-duration-area curves adjusted by the ratio of precipitable water at the extreme 24-hour persisting dewpoint temperature to the precipitable water at the standardised dewpoint of 28°C.

The extreme 24-hour persisting dewpoint temperature across the three catchments is 25.25°C, which allows for a Moisture Adjustment Factor of 0.79.

No adjustment for elevation is considered necessary as the mean elevation of the catchment is less than 1500 metres.

The derived PMP values for durations up to 6 hours are listed in Tables 1, 2 and 3. These estimates of PMP have been based on a number of theoretical assumptions concerning the storm mechanisms. The depth-duration-area values used are those derived from major short duration, limited area rainfall events both in the United States and in Australia. In view of the assumptions made the PMP values given should be regarded as approximate upper limits until an adequate Australian data base exists.

4. GENERALISED TROPICAL STORM METHOD (GTSM) OF PMP ESTIMATION for durations of 6 to 96 hours for the ECTZ

4.1 Method

A generalised method (the Generalised Tropical Storm Method or GTSM) for estimating Probable Maximum Precipitation (PMP) has been developed for those parts of Australia affected by storms of tropical origin (Kennedy (1982) and (Kennedy and Hart, 1984)). These parts of Australia have been sub-divided into two regions, namely the "East Coast Tropical Zone" (ECTZ, formerly called the "Queensland Coastal Zone") being the area of influence of a quasistationary easterly trough adjacent to the Queensland coast which appears to enhance heavy rainfall events, and the "Remaining Tropical Zone" (RTZ). In these areas of Australia the meteorological situation most likely to produce the PMP is considered to be either the proximity of a tropical cyclone or the slow movement of a low pressure system of tropical origin, sometimes interacting with a monsoonal trough.

For the two regions of Australia where the GTSM is applicable, the 6-, 12-, 24-, 48- and 72-hour (plus the 96-hour for the ECTZ only) rainfall values associated with heavy rainfall events have been extracted. These values were adjusted to remove the differences in observed storm depths caused by differing moisture content, distance from the coast, height of any intervening barrier and topography. The highest adjusted values of each storm were then enveloped to obtain depth-duration-area (DDA) curves. These curves provide the base PMP values for durations up to 96 hours for a catchment which is flat, is at the coast and has no significant barrier between it and its moisture source. The enveloping curves are standardised to a moisture index (the precipitable water) corresponding to a dew point temperature of 28°C.

Base values obtained from the DDA curves must therefore be adjusted for the conditions applying to the subject catchments which lie within the ECTZ. Each of the adjustments is discussed in the following sections.

4.2 Adjustment for Moisture Content of the Air

The moisture content adjustment allows for the difference between the standard extreme dew point of 28° C and the extreme persisting dew point temperature for the area in question. The extreme 24 hour persisting dew point temperature is obtained from maps prepared by the Bureau of Meteorology. The moisture adjustment factor is the ratio of the precipitable water value at this dew point temperature to that at the standard dew point temperature of 28° C.

The extreme 24 hour persisting dewpoint temperature for each of the subject catchments is 25.25° C. The moisture adjustment factor is therefore 0.79.

4.3 Adjustment for Distance from the Coast

This adjustment is made to allow for a decrease in moisture content of the air with distance from the moisture source, and is based on reduction factors obtained from a comparison of Australian rainfall data with a study of 60 United States storms (United States Weather Bureau, 1966). All plausible inflow directions are examined and from these the critical inflow path was determined by calculating the distance and barrier adjustments for each direction and choosing the direction which gives the highest combined adjustment factor. The mean distance of the catchment from the coast is measured along the critical inflow path.

The mean distance of the subject catchments from the coast, along their respective critical inflow paths is 25 kms or less which gives a distance adjustment factor of 1.00 for each catchment.

4.4 Adjustment for Height of Barrier

A significant topographical barrier between the moisture source and the catchment may also reduce the moisture content of the air which reaches the catchment. The adjustment factor is calculated as the ratio of the mixing ratios (specific humidities) of the air at the surface (assumed to be 1000 hPa) and the air at the height of the barrier.

The direction of low level inflow which would produce the heaviest rains from consideration of both synoptic influences and the height of intervening barrier is along the critical inflow path.

The critical inflow direction is considered to be east and the effective barrier height is less than 50 metres for each cathchment. The resulting adjustment factors are 1.00.

4.5 Adjustment for Topography

The recommended method of adjusting for the effect of topography on a broad scale is that given by the United States Weather Bureau (1965, 1966, 1969)and reproduced by WMO (1973 and 1986). Average percentage changes can be applied to whole areas of the catchment.

Application of this method to each of the subject catchments give adjustments of well less than 1%, accordingly the topographical adjustment factor for each catchment is 1.00.

4.6 The 6 hour to 96 Hour Derived PMP Values

The products of the adjustment factors derived for each catchment above has been applied to the base PMP values obtained from the DDA curves and these derived PMP values are listed in Tables 1, 2 and 3.

5. GORDON METHOD OF PMP ESTIMATION
For durations of 5 to 7 days for the ECTZ

5.1 Introduction

This method was devised by Barry Gordon of the Qld Regional Office for the report:
"Probable Maximum Precipitation Study, Brisbane River
- Somerset Dam - Wivenhoe Dam Catchments, Queensland",
B. of M., Qld R.O., September 1983.

It was based on a method used in the U.S. Hydrometeorological Report No. 46, "Probable Maximum Precipitation, Mekong River Basin", (1970).

It has since been used in several reports by both Qld and NSW Regional Offices.

5.2 Situation 1

In this situation it is proposed that the extreme rainfall event affecting an area would result from two distinct storms with a short period of little or no rainfall between the major storm events. It is further proposed that one of the two storms would be the extreme [PMP] storm event. Two possibilities were considered.

(a) Two 3-day storms separated by one day of no significant rainfall; and

(b) Two 2-day storms separated by three days of little or no rainfall.

To arrive at the best estimate using this method, it is necessary to consider how soon before or after an extreme storm a second significant storm occur and how much rain could the smaller storm produce. Little data are available in Australia on the occurrence of two major storms within a 7 day period. However, in some past overseas studies (eg U.S. Weather Bureau 1970), values of the lesser storm up to 70 per cent of the major storm have been assumed, based on a limited study of storms. Additionally, a minimum period of 3 to 4 days between storms was chosen. Of the two possibilities under consideration, (a) was discarded as it was not considered meteorologically feasible and so (b) was selected to derive a 7 day PMP estimate.

The temporal pattern associated with the second storm is assumed to be the same as that for the 48 hour PMP for the relevant zone.

For the ECTZ:
From the 48 hour PMP temporal pattern the proportion of rain falling in the first day is 52%.

For the RTZ:
According to the 48 hour temporal pattern the proportion of rain that will occur on the first day is 70%.

ECTZ

5dPMP = 2dPMP + 3d nil rain
6dPMP = 2dPMP + 3d nil rain
+ [0.52 * 0.70] * 2dPMP
= 1.36 * 2dPMP
7dPMP = 2dPMP + 3d nil rain
+ 0.70 * 2dPMP
= 1.70 * 2dPMP

RTZ

4dPMP = 2dPMP + 2d nil rain
5dPMP = 2dPMP + 3d nil rain
6dPMP = 2dPMP + 3d nil rain
+ [0.70 * 0.70} * 2dPMP
= 1.49 * 2dPMP
7dPMP = 2dPMP + 3d nil rain
+ 0.70 * 2dPMP
= 1.70 * 2dPMP

In the above table please read 4dPMP as 4-day PMP, etc.

5.3 Situation 2

In this method it is proposed that the 7-day extreme rainfall event would result from 7 days of general rain with the 4-day PMP storm occurring within the period. To determine the remaining 3-day "general" rainfall, the storms associated with the highest 24 hour point rainfalls to have occurred in Queensland were selected for preliminary examination. For the most significant of these storms, the highest 7- day rainfall totals were extracted. From these totals the highest 4-day totals were taken and the remaining 3-day rainfalls expressed as ratios of the 4-day rainfalls.

For the ECTZ:
Using this procedure a representative ratio of 0.08 was selected. This ratio can be applied to the 4-day PMP rainfall to derive the 3 day "general" rainfall. The two rainfall totals were then summed to give the 7-day PMP estimate. The general rain is distributed over the three days in the following ratios: 0.45, 0.33 and 0.22.

For the RTZ:
The 7-day PMP is comprised of 3-day PMP and 4 days of general rain. The ratio of the general rain to the 3-day PMP is 0.25 and this is distributed in the following proportions: 0.64, 0.16, 0.12 & 0.08.

ETCZ

5dPMP = 4dPMP + [0.45 * 0.08] * 4dPMP
= 1.04 * 4dPMP
6dPMP = 4dPMP + [0.78 * 0.08] * 4dPMP
= 1.0624 * 4dPMP
7dPMP = 4dPMP + 0.08 * 4dPMP
= 1.08 * 4dPMP

RTZ

4dPMP = 3dPMP + [0.64 * 0.25] * 3dPMP
= 1.16 * 3dPMP
5dPMP = 3dPMP + [0.80 * 0.25] * 3dPMP
= 1.20 * 3dPMP
6dPMP = 3dPMP + [0.92 * 0.25] * 3dPMP
= 1.24 * 3dPMP
7dPMP = 3dPMP + 0.25 * 3dPMP
= 1.25 * 3dPMP

5.4 The 5 to 7 day Derived PMP Values

Recall that the subject catchments lie within the "East Coast Tropical Zone" accordingly the ECTZ relationships above have been utilised to provide the 5 to 7 day PMP values which are listed at Tables 1, 2 and 3.

6. TEMPORAL PATTERNS

Generalised temporal patterns have been developed for all durations and are shown as Figures 2 to 6.

The temporal pattern at Figure 2 is applicable for all durations up to and including 6 hours.

Two patterns are provided for the five day storm and three for the six day storms. For their respective durations, the patterns are considered to be equally likely and the most critical should be selected for use.

Three patterns are provided at Figure 6 for the seven day storm. The "stepped" curve ie the one which includes three days of nil rainfall is the temporal pattern associated with the synoptic situation which would produce the PMP event over the catchment, ie as described in Section 5 Situation 1. For the sake of completeness two temporal patterns associated with the synoptic situation described in Section 5 Situation 2 are also shown.

7. SPATIAL DISTRIBUTION DIAGRAMS

Instructions for the use of the spatial distribution diagram for the Bulletin 51 method, the spatial distribution diagram for short duration PMP together with a worked example for the Pykes Creek Catchment have been provided, at Appendix 1, for the clients information and guidance.

Similar instructions and diagrams are provided at Appendix 2 for the Generalised Tropical Storm Method. These diagrams are suitable for application to the Gordon Method of PMP estimation and distribution.

8. RAINFALL CONCURRENT WITH GTSM PMP ESTIMATES

There is no definitive method for deriving concurrent rainfall when the PMP storm occurs over an adjacent catchment. Sensitivity testing of several methods is recommended.

With the GTSM, four methods have been widely used. These are:

1) the GTSM elliptical distribution is expanded to cover the combined catchment and a scaling used to maintain the PMP depth over the required catchment.

2) the major recorded storms in the area adjacent to the main catchment are used.

3) the areally adjusted 1:100 year IFD depth and spatial distribution is used over the adjacent area.

4) PMP is calculated for the main catchment and the combined catchment and the difference is distributed as concurrent rainfall.

Instructions for their use are attached at Appendix 3.

The joint probabilities associated with these methods vary considerably. They range from method 4 with the lowest joint probability to methods 2 and 3 which have a higher joint probability.

Sensitivity tsting combined with a subjective understanding of the relative joint probabilities are needed to select the most appropriate method.

Since the Gordon Method extends the GTSM out to 7 days it is appropriate that the four methods suggested above for calculating concurrent rainfall be applied to PMP Estimates for durations out to 7 days.

TABLE 1

PMP VALUES (mm) for the PINE RIVER CATCHMENT (708 sq kms)
ABOVE THE MOUTH

Duration	Derived Values (rounded to the nearest 10mm)					PMP Estimate
	Bulletin 51	GTSM	"Best Fit" Transition	Gordon Methods Situation 1	Gordon Methods Situation 2	
1 hour	200	na	na	na	na	200
2 hours	320	na	na	na	na	320
3 hours	390	na	na	na	na	390
6 hours	500	450	520	na	na	520
12 hours	na	750	750	na	na	750
18 hours	na	na	920	na	na	920
24 hours	na	1080	na	na	na	1080
2 days	na	1590	na	na	na	1590
3 days	na	1990	na	na	na	1990
4 days	na	2350	na	na	na	2350
5 days	na	na	na	1590	2430	2430
6 days	na	na	na	2170	2500	2500
7 days	na	na	na	2700	2540	2700

Notes 1. na denotes that this method is not applicable for these durations.

2. The rainfall data sets from which the GTSM was derived is much more extensive for 24 hours than for either 6 or 12 hours duration, accordingly greater confidence can be assigned to the base PMP values for 24 hours duration than for either of the 6 and 12 hour durations.

3. The two rainfall depth-duration series, as derived from the Bulletin 51 and GTSM methods, are frequently discontinuous between 6 hours and 24 hours. When this discontinuity occurs a "best fit" curve is applied to remove the point of inflection and provide a smoothed transition between the two series. An interpolated value for 12 hours is then available from the fitted transition curve.

4. The higher derived (or interpolated) value, for each duration, is recommended as the PMP Estimate for that duration.

TABLE 2

PMP VALUES (mm) for the NORTH PINE DAM (347 sq kms) LAKE SAMSONVALE

Duration	Derived Values (rounded to the nearest 10mm)					PMP Estimate
	Bulletin 51	GTSM	"Best Fit" Transition	Gordon Methods Situation 1	Situation 2	
1 hour	240	na	na	na	na	240
2 hours	370	na	na	na	na	370
3 hours	450	na	na	na	na	450
6 hours	580	500	590	na	na	590
12 hours	na	810	810	na	na	810
18 hours	na	na	990	na	na	990
24 hours	na	1160	na	na	na	1160
2 days	na	1720	na	na	na	1720
3 days	na	2150	na	na	na	2150
4 days	na	2490	na	na	na	2490
5 days	na	na	na	1720	2580	2580
6 days	na	na	na	2350	2650	2650
7 days	na	na	na	2920	2690	2920

Notes 1. na denotes that this method is not applicable for these durations.

2. The rainfall data sets from which the GTSM was derived is much more extensive for 24 hours than for either 6 or 12 hours duration, accordingly greater confidence can be assigned to the base PMP values for 24 hours duration than for either of the 6 and 12 hour durations.

3. The two rainfall depth-duration series, as derived from the Bulletin 51 and GTSM methods, are frequently discontinuous between 6 hours and 24 hours. When this discontinuity occurs a "best fit" curve is applied to remove the point of inflection and provide a smoothed transition between the two series. An interpolated value for 12 hours is then available from the fitted transition curve.

4. The higher derived (or interpolated) value, for each duration, is recommended as the PMP Estimate for that duration.

TABLE 3

PMP VALUES (mm) for the SIDELING CREEK DAM (53 sq kms)
LAKE KURWONGBAH

Duration	Derived Values (rounded to the nearest 10mm)					PMP Estimate
	Bulletin 51	GTSM	"Best Fit" Transition	Gordon Methods Situation 1	Gordon Methods Situation 2	
1 hour	320	na	na	na	na	320
2 hours	480	na	na	na	na	480
3 hours	580	na	na	na	na	580
6 hours	770	620	na	na	na	770
12 hours	na	930	980	na	na	980
18 hours	na	na	1140	na	na	1140
24 hours	na	1300	na	na	na	1300
2 days	na	1890	na	na	na	1890
3 days	na	2340	na	na	na	2340
4 days	na	2660	na	na	na	2660
5 days	na	na	na	1890	2760	2760
6 days	na	na	na	2580	2830	2830
7 days	na	na	na	~~3100~~ 3210	2870	~~3100~~ 3210

Notes 1. na denotes that this method is not applicable for these durations.

2. The rainfall data sets from which the GTSM was derived is much more extensive for 24 hours than for either 6 or 12 hours duration, accordingly greater confidence can be assigned to the base PMP values for 24 hours duration than for either of the 6 and 12 hour durations.

3. The two rainfall depth-duration series, as derived from the Bulletin 51 and GTSM methods, are frequently discontinuous between 6 hours and 24 hours. When this discontinuity occurs a "best fit" curve is applied to remove the point of inflection and provide a smoothed transition between the two series. An interpolated value for 12 hours is then available from the fitted transition curve.

4. The higher derived (or interpolated) value, for each duration, is recommended as the PMP Estimate for that duration.

7. REFERENCES

BUREAU OF METEOROLOGY	1985	"The Estimation of Probable Maximum Precipitation in Australia for Short Durations and Small Areas." Bulletin 51.
INSTITUTION OF ENGINEERS, (AUSTRALIA) 1987	1987	"Australian Rainfall and Runoff. A guide to Flood Estimation."
KENNEDY: M.R. & HART: T.L.	1984	"The Estimation of Probable Maximum Precipitation in Australia." Civ. Engg. Trans. Inst. Engrs. Aust. Vol. CE 26 No 1.
UNITED STATES WEATHER BUREAU	1965	"Probable Maximum and Precipitation over the Tennessee River Basin above Chattanooga" Hydrometeorological Report No 41
UNITED STATES WEATHER BUREAU	1966	"Probable Maximum Precipitation, North-west States". Hydrometeorological Report No 43.
UNITED STATES WEATHER BUREAU	1969	"Probable Maximum and TVA Precipitation for Tennessee River Basins up to 3000 sq. miles in Area and Durations to 72 Hours." Hydrometeorological Report No 45.
UNITED STATES WEATHER BUREAU	1970	"Probable Maximum Precipitation Mekong River Basin." Hydrometeorological Report No 46.

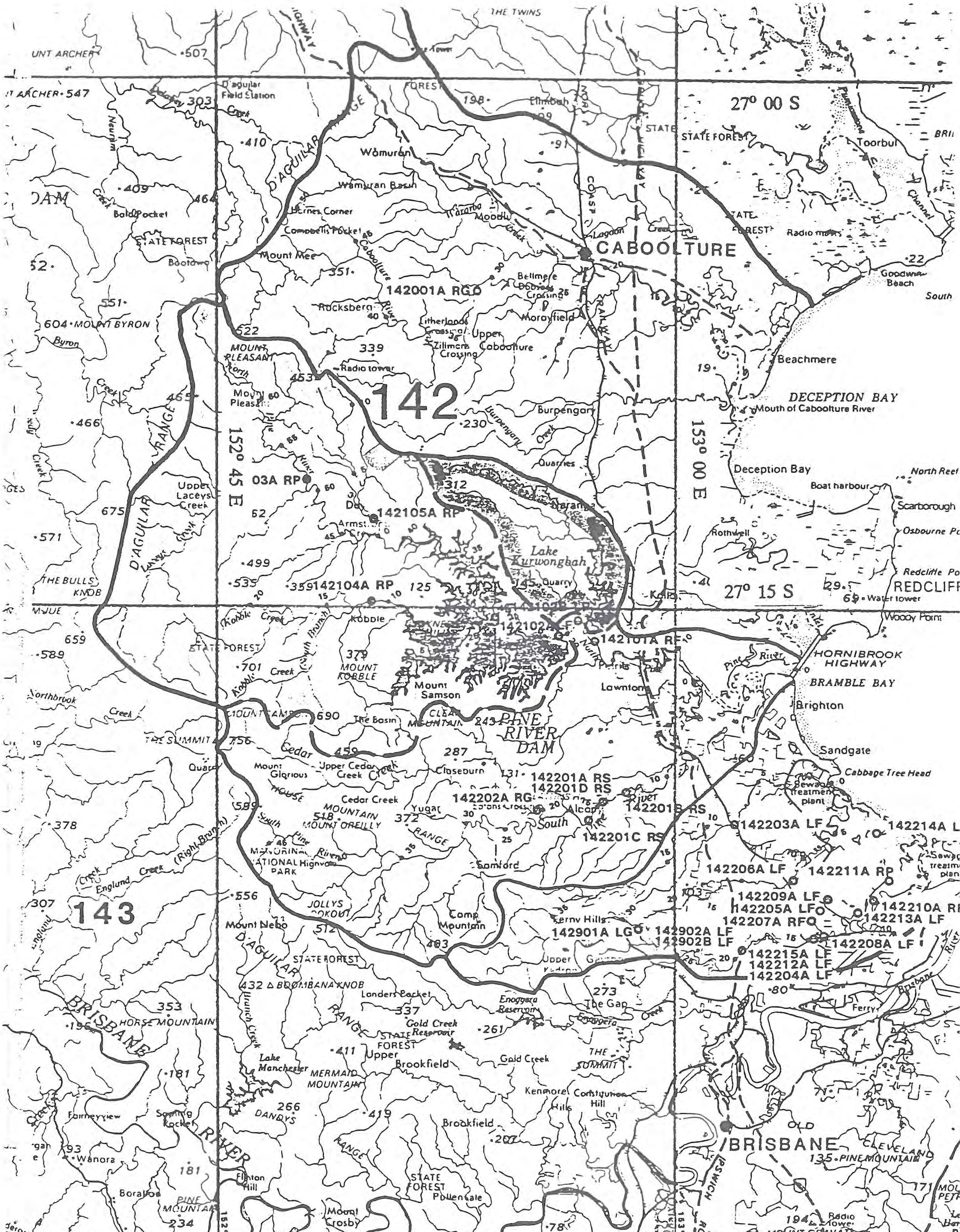


Figure 1: CATCHMENT LOCATIONS

for the a) Pine River Catchment
b) North Pine Dam Catchment

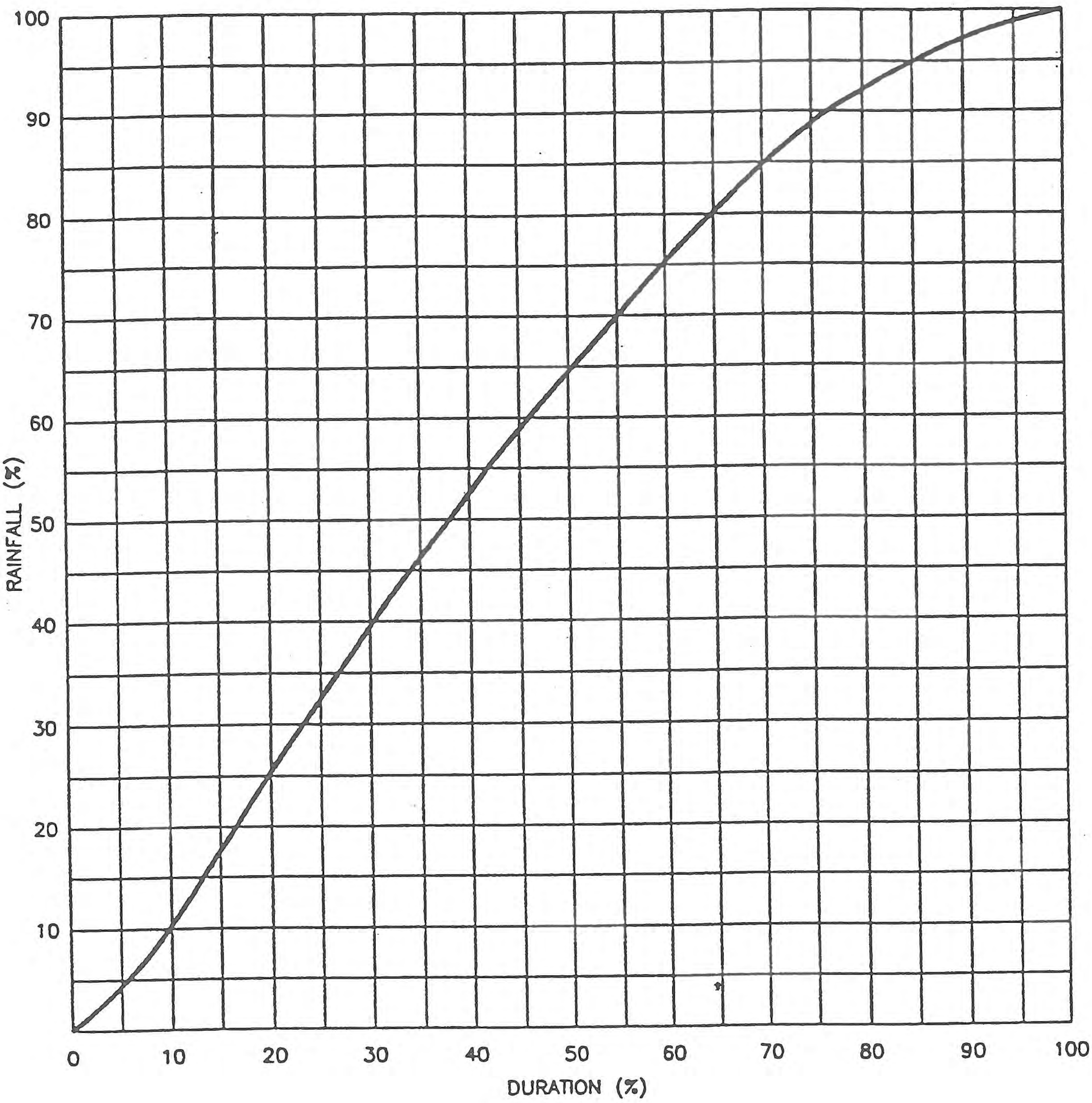


Figure 2: Design Temporal Pattern of short duration PMP

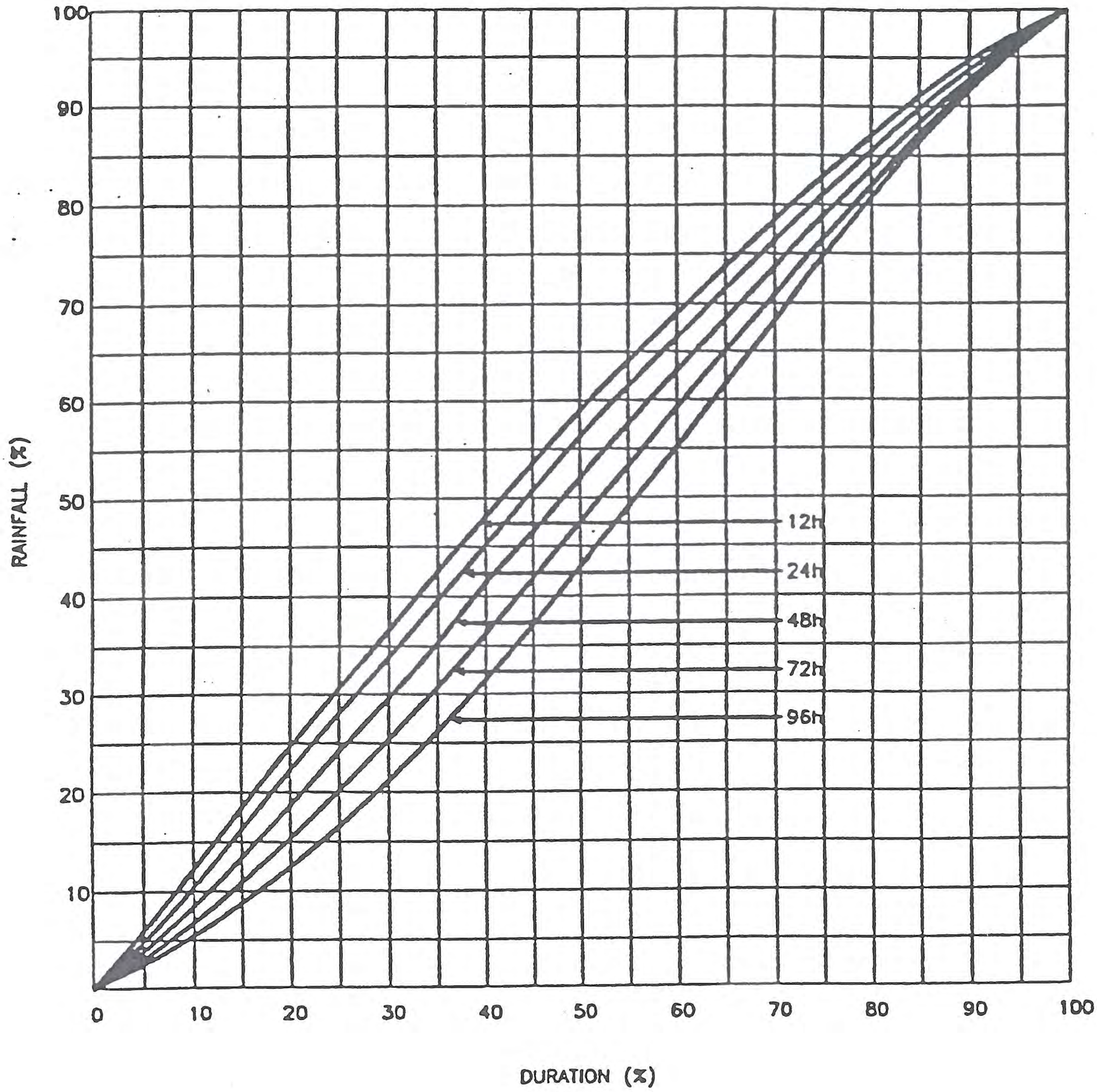


Figure 3 : Design Temporal Patterns of Generalised Tropical Storm PMP for the "East Coast Tropical Zone" for storm durations of 12, 24, 48, 72 and 96 hours.

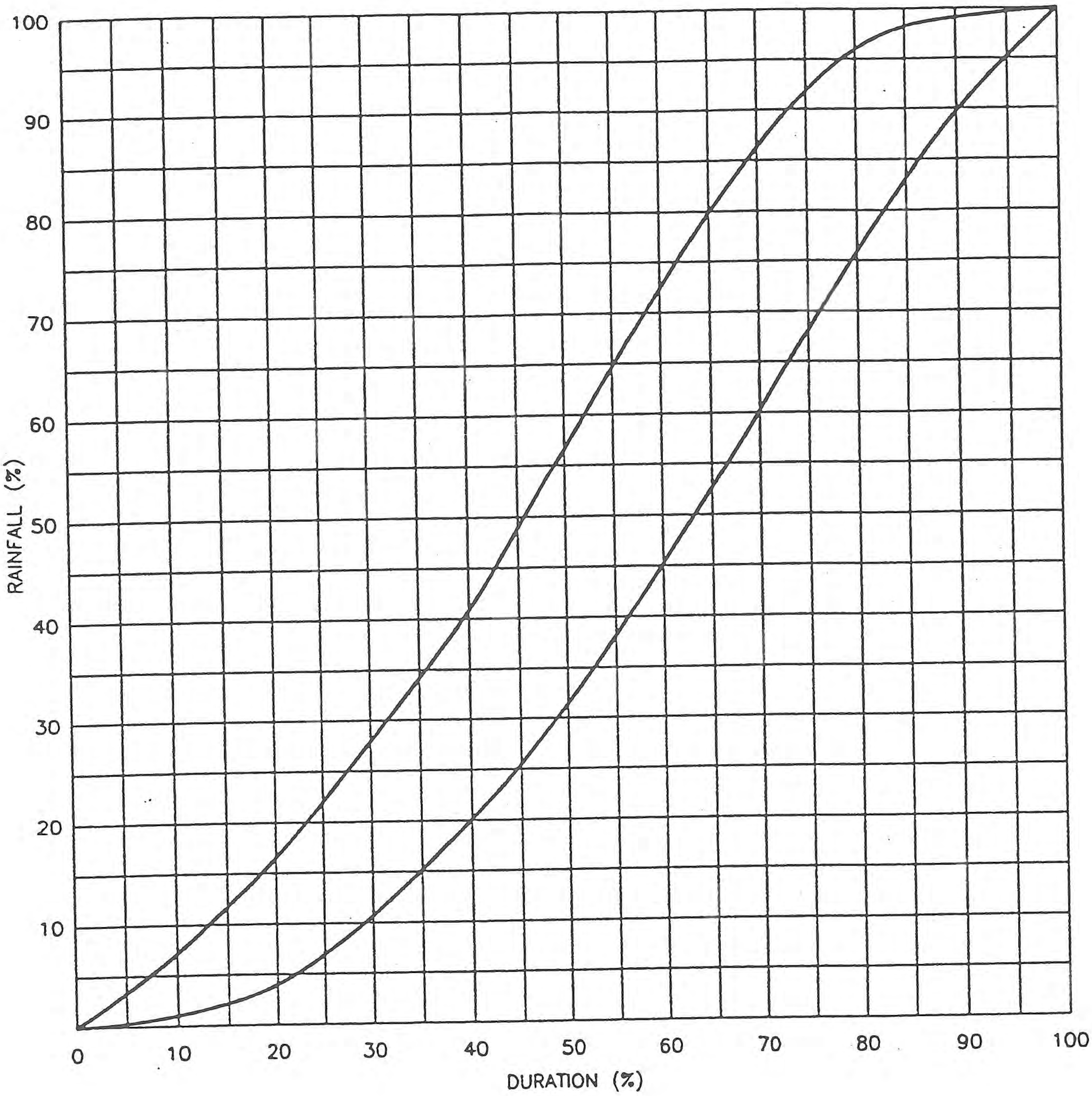


Figure 4 : Design Temporal Patterns of PMP for the "East Coast Tropical Zone" for a duration of 120 hours

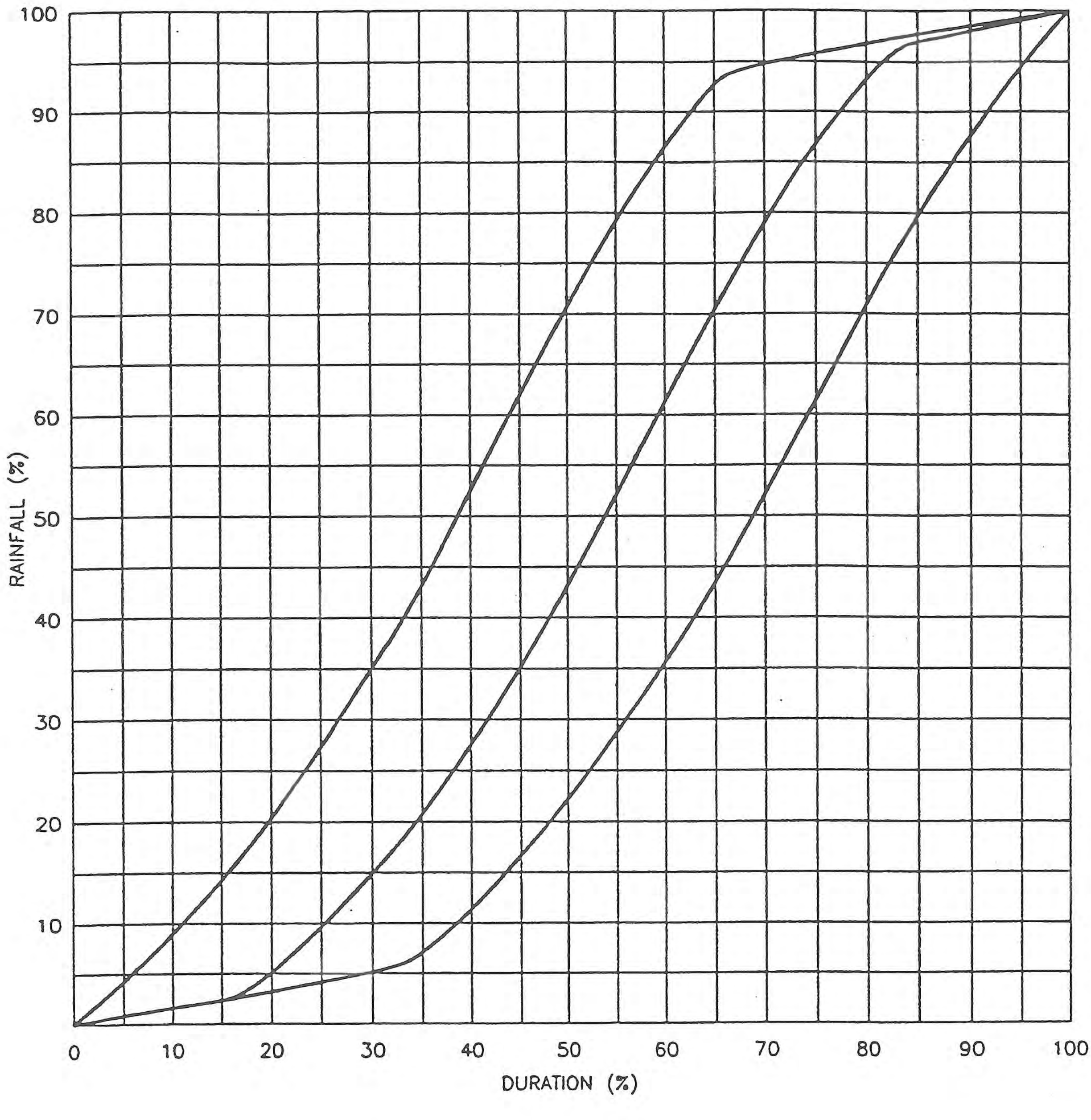


Figure 5 : Design Temporal Patterns of PMP for the "East Coast Tropical Zone" for a duration of 144 hours

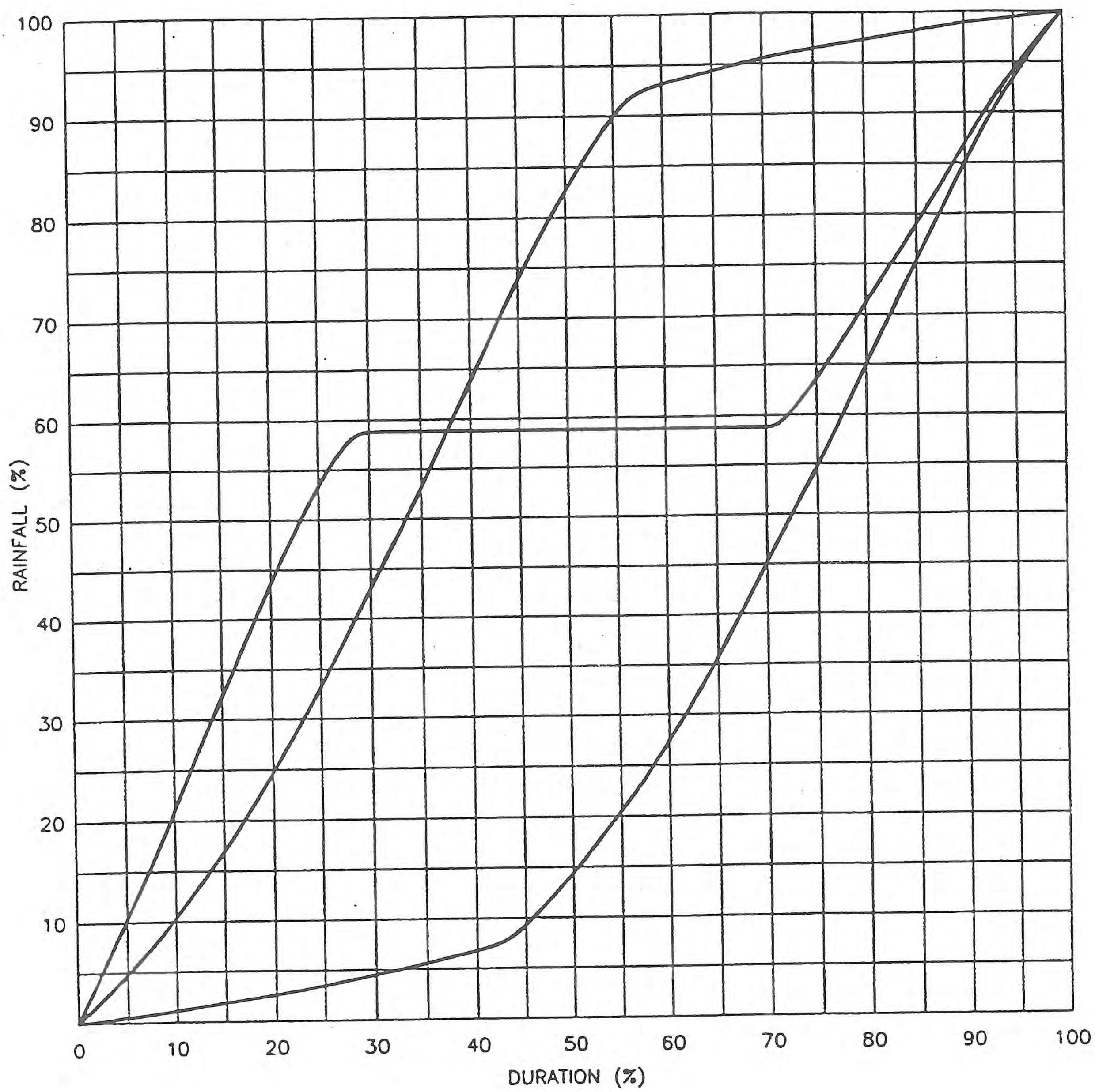


Figure 6 : Design Temporal Patterns of PMP for the "East Coast Tropical Zone" for a duration of 168 hours

INSTRUCTIONS FOR THE USE OF THE "BULLETIN 51"

PMP SPATIAL DISTRIBUTION DIAGRAM

TABLE 1

ISOHYETAL LABELS FOR SPATIAL DISTRIBUTION OF "BULLETIN 51" (AS HOURLY INCREMENTS IN PERCENT OF PMP)

Isohyet	Area Enclosed (km^2)	Hourly Increments of PMP (Percent)					
		1st	2nd	3rd	4th	5th	6th
A	2	100	19	10	6	5	4
B	16	76	19	10	6	5	4
C	65	54	19	10	6	5	4
D	153	40	17	9	6	5	4
E	246	32	14	8	5	4	4
F	433	21	10	7	4	3	3
G	635	14	7	5	4	3	3
H	847	8	4	4	3	3	3
I	1114	1	2	2	2	2	2
J	1396	1	3	0	0	0	0

To obtain the spatial distribution of PMP proceed as follows:

Step 1. Obtain a mean x-hour, y-km^2 PMP depth for the catchment to use as the basis for calculating the isohyetal labels. This mean depth can be any of the 1 to 6 hour PMP values for the catchment.

Step 2. Alter the scale of the spatial distribution diagram to match that of the outline of the catchment.

Step 3. Centre (approximately) and rotate the isohyetal distribution to provide the best fit between the shape of the catchment and that of the isohyets.

Step 4. Obtain labels from Table 1 for isohyets up to the minimum size to enclose the catchment completely. The labels for the isohyets are obtained by adding up the percentages given in Table 1 up to the duration required (eg for 4 hours, A = 100 + 19 + 10 + 6 = 135).

Step 5. Multiply the mean PMP value by the isohyetal percentages from Step 4 to obtain initial isohyetal labels in millimetres.

Step 6. Calculate the mean initial rainfall depth over the catchment by planimetering or other procedures. The rainfall values assigned to the area between each pair of successive ellipses should be a weighted average of the rainfall values of the two isohyets. The formula used is:

$$\textbf{Mean Initial PMP Depth} = \frac{\sum_{i=1}^{n} (\text{area}_i * \text{initial rainfall depth}_i)}{\text{Total catchment area}}$$

where n is the number of sub-areas of the catchment between successive isohyets;

area_i is the value of a sub-area of the catchment between the isohyets i and i-1;

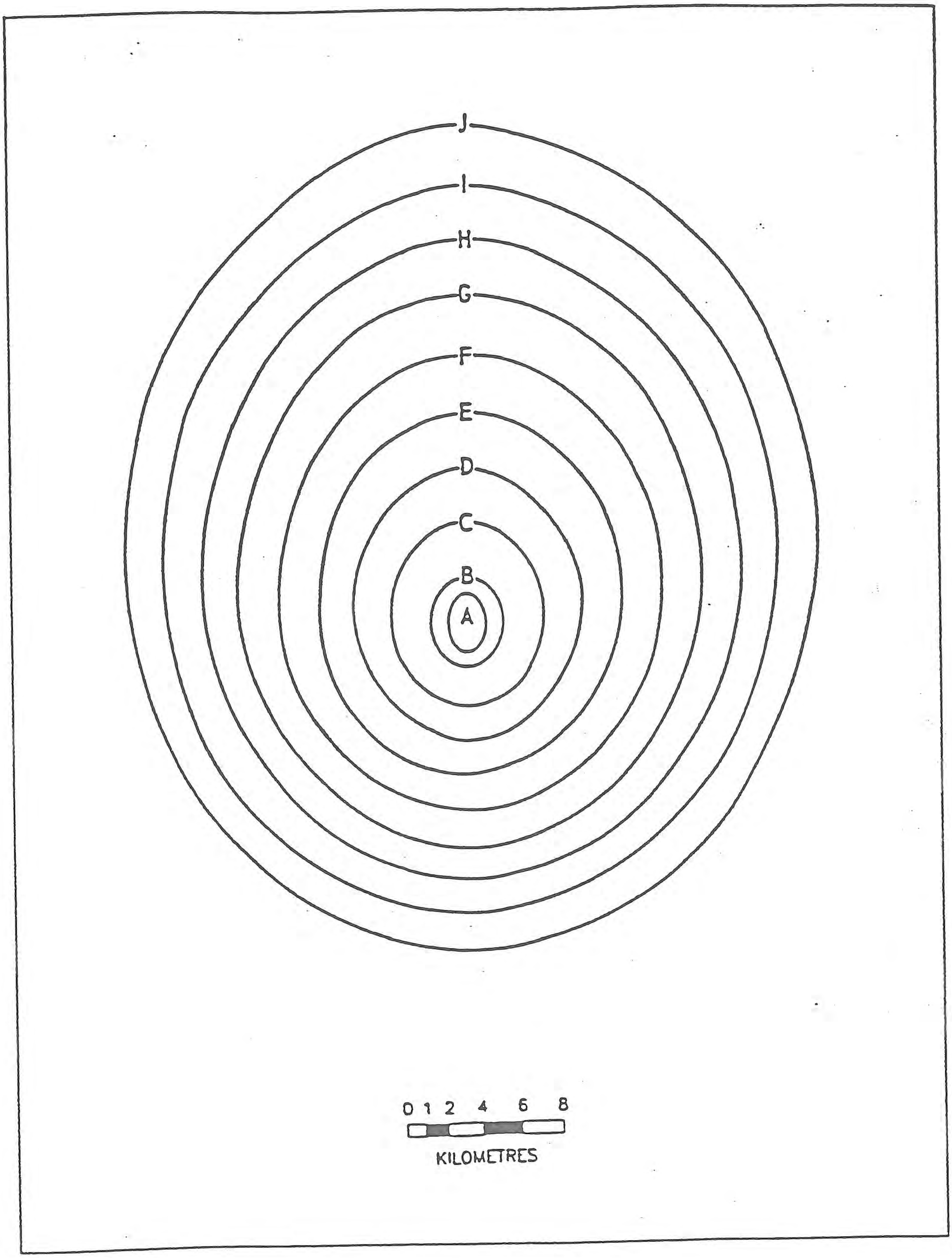
initial rainfall depth_i is the weighted average of the initial rainfall values in mm of the labels of the isohyets i and i-1;

Step 7. Scale the values for the sub-areas to obtain the correct mean PMP depth over the catchment, using the formula:

$$\textbf{Scaling Factor} = \frac{\text{Mean PMP depth from Step 1}}{\text{Mean Initial PMP Depth from Step 6}}$$

Step 8. Obtain the final isohyetal labels in millimetres by using the scaling factor of Step 7 and the values obtained in Step 5.

Step 9. Repeat the above steps for the other durations for which the PMP is required.



Bulletin 51 Spatial Distribution of

Short Duration PMP.

EXAMPLE: PYKES CREEK CATCHMENT

Pykes Creek is located in southern Victoria. The area of this catchment is 120 km^2 and the maximum duration for which the Bulletin 51 method can be used here is 4 hours. The terrain in the catchment is classified as "rough". In this example only the distribution of the 4-hour PMP will be derived.

Step 1. The unadjusted 4-hour PMP is 753 mm. The moisture adjustment factor is 0.55. The 4-hour PMP is thus 753x0.55 = 410 mm (rounded to the nearest 10 mm).

Step 2. The spatial distribution diagram and the catchment outline were obtained at the same scale.

Step 3. The pattern was placed over the catchment with the isohyetal centre near the centroid of the catchment. It is necessary to use isohyets A to E to enclose the catchment.

The position of the pattern was adjusted slightly and the pattern was rotated to get the estimated optimum depth over the catchment. The position of the pattern with respect to the catchment is shown in Figure 1.

Step 4. The labels for isohyets A to E were obtained as percentages of the mean PMP by adding the incremental values up to 4 hours from Table 1. These labels are given in Table 2 as percentages of the mean PMP depth.

TABLE 2

ISOHYETAL PERCENT OF PMP LABELS FOR SPATIAL DISTRIBUTION OF 4-HOUR PMP FOR PYKES CREEK CATCHMENT

Isohyet	Area Enclosed (km^2)	4-Hour PMP Isohyet Label (Per Cent)
A	2	135
B	16	111
C	65	89
D	153	72
E	246	59

Step 5. Initial isohyetal labels in millimetres were obtained by multiplying the percentages by the mean PMP depth. These values are given in Table 3.

TABLE 3

ISOHYETAL INITIAL DEPTH LABELS FOR SPATIAL DISTRIBUTION OF 4-HOUR PMP FOR PYKES CREEK CATCHMENT

Isohyet	Area Enclosed (km^2)	Initial 4-Hour PMP Isohyet Label (mm)
A	2	554
B	16	455
C	65	365
D	153	295
E	246	242

Step 6. The magnitudes of the sub-areas of the catchment between successive pairs of isohyets were determined. The rainfall over each sub-area was obtained by taking a weighted average of the values of the two adjacent isohyets. For the central isohyet, (A), assume that the rainfall depth increases to a maximum at its central point. The mean initial PMP depth over the catchment is calculated from the sum of the products of area and rainfall as given in Table 4.

TABLE 4

CALCULATION OF MEAN AREAL RAINFALL FOR SPATIAL DISTRIBUTION OF PMP FOR PYKES CREEK CATCHMENT

Isohyet	Incremental Area (km^2)	Mean Initial Areal Rainfall Value (mm)	Area * Rainfall
A	2.0	560	1120
A to B	14.0	505	7070
B to C	49.0	410	20090
C to D	38.2	335	12797
D to E	16.8	275	4620
A to E	120.00		45697

The mean initial PMP depth (mean catchment rainfall) is the sum of the products of sub-area and rainfall divided by the total area. This is 45697/120 = 381 mm.

Step 7. The rainfall values given in Table 4 for the sub-areas are therefore scaled up by the factor 410/381 = 1.076 to give the correct mean PMP depth over the catchment. The adjusted mean rainfall values for the sub-areas are given in Table 5. This procedure distributes the mean initial PMP depth over the catchment correctly.

TABLE 5

SCALED SUB-AREA RAINFALL VALUES OF 4-HOUR PMP FOR PYKES CREEK CATCHMENT

Isohyet	Incremental Area (km^2)	Mean Areal Rainfall Value (mm)
A	2.0	603
A to B	14.0	543
B to C	49.0	441
C to D	38.2	360
D to E	16.8	296

This is the final spatial distribution of the mean PMP over the catchment.

Step 8. The final isohyetal labels in millimetres are given in Table 6. These were obtained by multiplying the values in table 3 by 1.076.

TABLE 6

FINAL ISOHYETAL LABELS FOR SPATIAL DISTRIBUTION OF 4-HOUR PMP FOR PYKES CREEK CATCHMENT

Isohyet	Area Enclosed (km^2)	Isohyetal Label (mm)
A	2	600
B	16	490
C	65	390
D	153	320
E	246	260

(Values rounded to the nearest 10 mm)

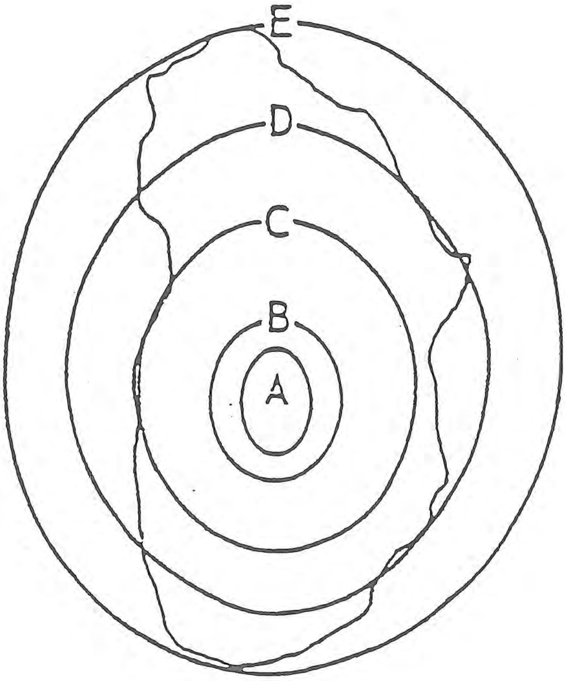


FIGURE 1. ISOHYETAL DISTRIBUTION OF PMP OVER THE PYKES CREEK CATCHMENT

INSTRUCTIONS FOR THE USE OF THE GTSM PMP SPATIAL DISTRIBUTION DIAGRAMS

1. Select the appropriate distribution diagram according to whether the area of the catchment is above or below 2000 km^2.

2. Expand or contract the scale of the isohyetal pattern until the outermost isohyet just touches the catchment. Adjust the positioning of the pattern to get an (estimated) highest PMP depth over the catchment. This depends on the shape of the catchment as well as the position of the pattern.

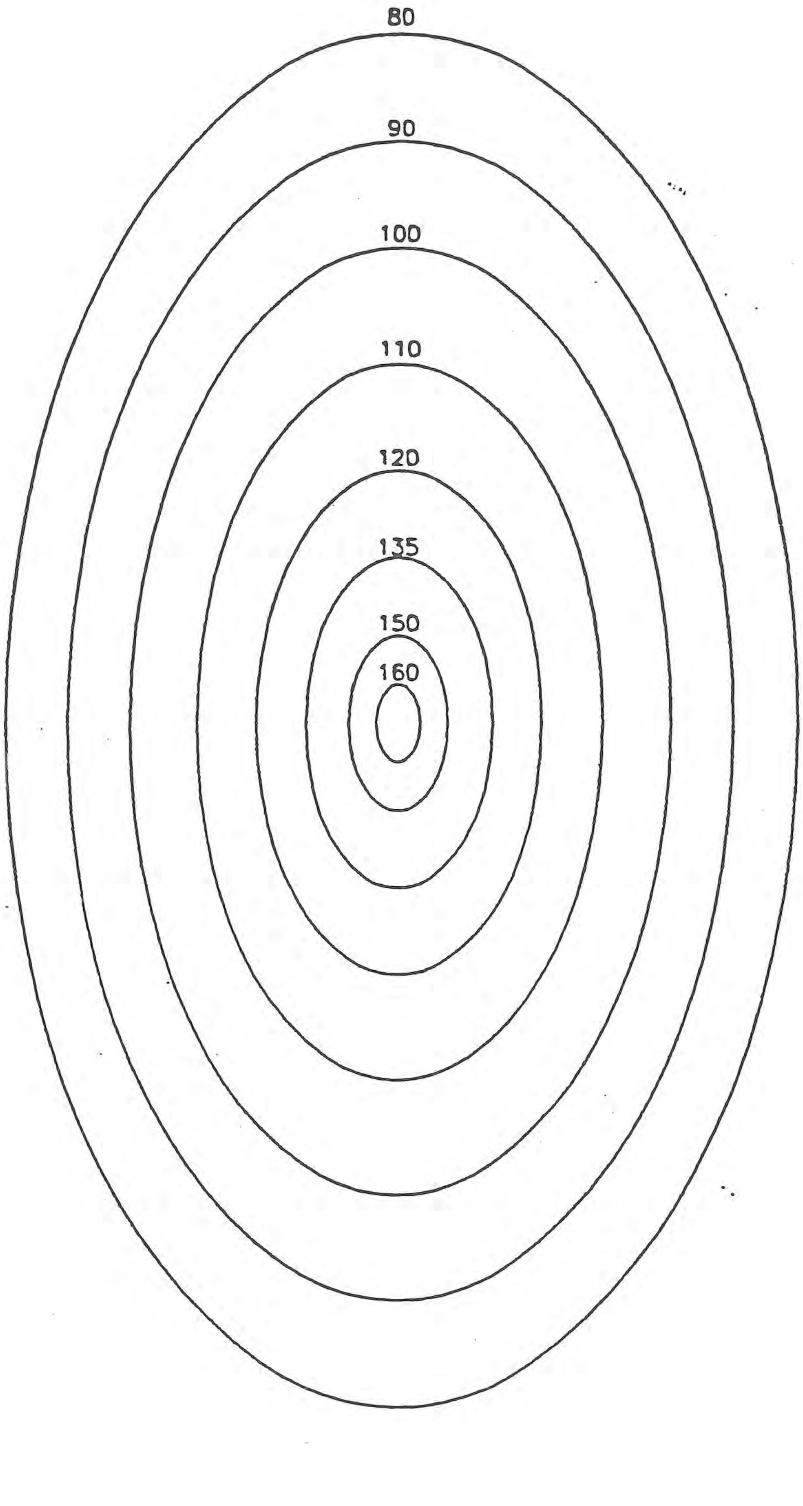
3. Calculate the area of the catchment within the central isohyet, and then between each adjacent pair of isohyets until all these areas have been calculated. A planimeter or other means are suitable methods of doing this.

4. Multiply the percentage assigned to the label on each isohyet by the mean PMP depth for that duration. This gives isohyet labels in millimeters.

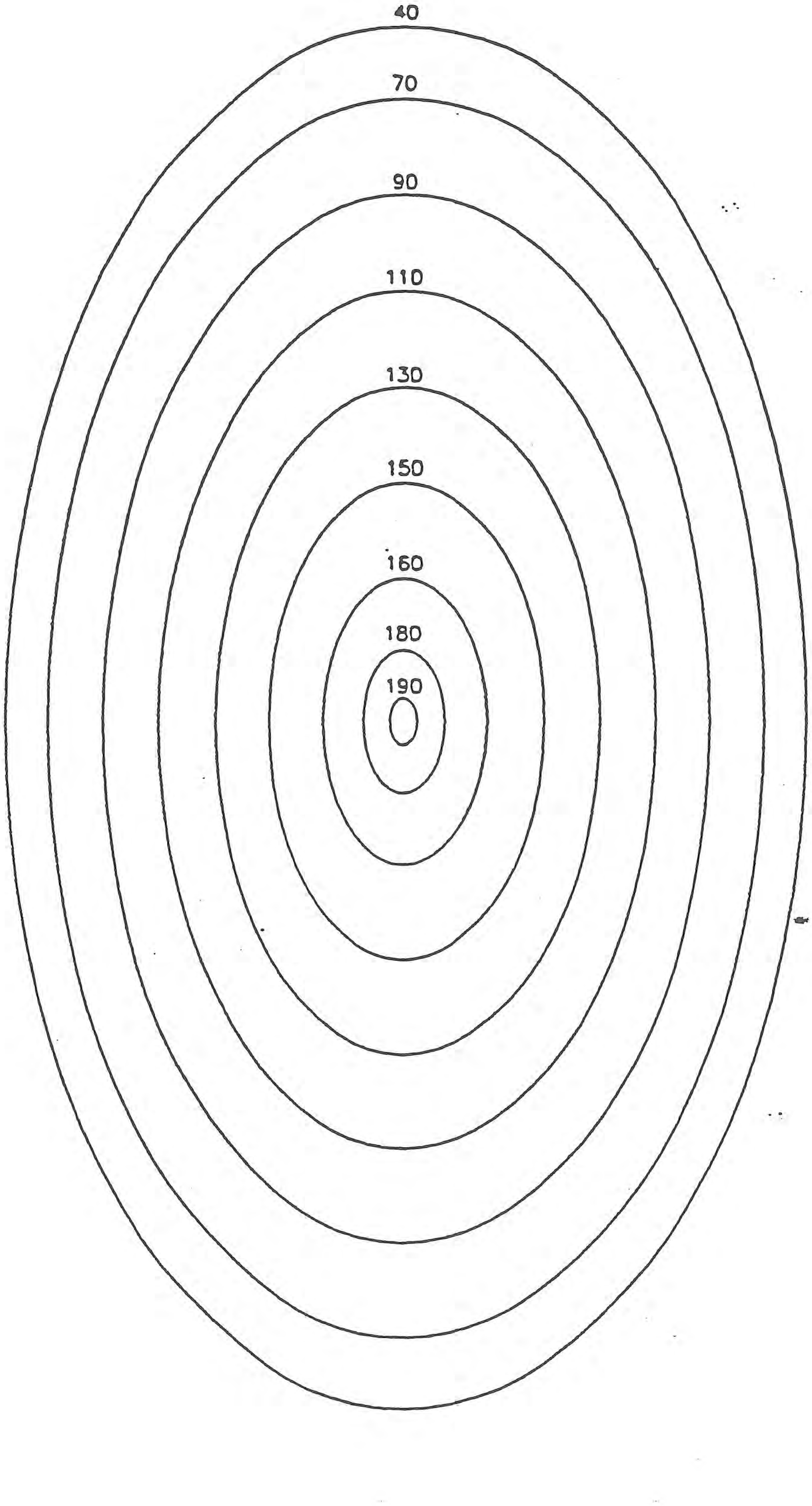
5. Multiply these areas by an estimate of the mean rainfall value over that part of the catchment contained in the annulus between each successive pair of isohyets. This will generally not be the arithmetic mean because of the usually irregular shape of the catchment boundary. For the central isohyet a mean value has to be estimated. This will not be critical.

6. The sum of all the above products is divided by the total catchment area to obtain the calculated mean catchment PMP depth. This will usually not be equal to the true PMP depth. The ratio of the actual PMP to the calculated PMP values is then calculated.

7. The values of the isohyetal labels are all multiplied by this ratio (ie a constant scaling factor) to ensure that the isohyetal pattern gives the correct mean PMP depth.



Generalised Tropical Storm Method Design Isohyetal Pattern
for the Distribution of PMP over Areas <= 2000 sq km
(% of mean catchment value)



Generalised Tropical Storm Method Design Isohyetal Pattern for distribution of PMP for Areas >2000 sq km.
(% of mean catchment value)

APPENDIX 3

CALCULATION OF CONCURRENT RAINFALL OVER AN AREA WHEN THE PMP STORM OCCURS OVER AN ADJACENT CATCHMENT

METHOD 1

In this method the PMP is calculated for the specified catchment using the GTSM. The design isohyetal pattern is centred over the catchment for which the PMP is required. The pattern is expanded or contracted to cover the adjacent area for which the concurrent rainfall is required. A scaling factor is calculated to give the correct mean PMP depth over the smaller catchment. It is then used to obtain the isohyetal values over the larger area.

1. Calculate the PMP using the GTSM for the smaller catchment for the range of durations required.

2. Select the design spatial distribution according to whether the area of the total or larger catchment is less than or greater than 2000 km^2.

3. Centre the isohyets over the smaller catchment for which the PMP is required and enlarge or reduce the spatial distribution until the outermost isohyet just encloses the larger catchment, as shown in Figure 1.

4. Multiply the mean PMP value for the smaller catchment by the isohyetal percentages for isohyets 1 to m, which encloses the smaller catchment, to obtain initial labels in millimetres.

5. Obtain, by planimetering or other means, the values of the sub-areas of the smaller catchment between successive isohyets and also inside the central isohyet.

6. Assign each of the areas obtained in step 5 a weighted mean rainfall value using the values for the isohyets. Estimate a rainfall value for the area enclosed by the central isohyet.

7. Calculate a mean initial PMP depth over the smaller catchment using the formula:

$$\text{Mean Initial PMP Depth} = \frac{\sum_{i=1}^{m} (\text{Area}_i \times \text{Initial Rainfall Depth}_i)}{\text{Catchment Area}}$$

where m is the number of sub-areas of the catchment between successive isohyets;

Area_i is the value of the sub-area of the catchment lying between the isohyets i and i-1;

$\text{Initial Rainfall Depth}_i$ is the weighted value of the labels on isohyets i and i-1.

8. Scale the weighted mean rainfall values for the sub-areas to obtain the correct mean PMP depth over the catchment using the formula:

$$\text{Scaling Factor} = \frac{\text{Mean PMP Depth from Step 1}}{\text{Mean Initial PMP Depth from Step 7}}$$

9. Obtain the final isohyetal values in millimetres for the smaller catchment by using the scaling factor obtained in Step 6 and the isohyetal values obtained in Step 4.

   To obtain the spatial distribution of the concurrent rainfall over the larger catchment when the PMP occurs over the smaller catchment, proceed as follows :

10. Obtain the ratio of the final value in mm for isohyet m to the initial percentage value of isohyet m.

11. Apply this ratio to all isohyets from m+1 to the final isohyet, which encloses the larger catchment, to obtain the spatial distribution over the larger catchment area.

12. Repeat the above procedure for other durations. This can be done by using the same scaling factors.

METHOD 2

The PMP values for the specified catchment are obtained using the GTSM. The design spatial distribution is fitted to this catchment only. The spatial distribution of rainfall accompanying the PMP storm is obtained from moisture maximised major recorded storms in the area adjacent to the main catchment.

1. Calculate the PMP using the GTSM for the smaller catchment for the range of durations required.

2. Derive the spatial distributions of these PMP depths using the appropriate design isohyetal patterns.

3. Obtain the spatial distributions of the most severe storms over the adjacent catchments.

4. Maximise these storms in situ using the ratio of the precipitable water values for the extreme and storm dew points.

5. Use the spatial distribution of the largest of these maximised storms over the adjacent areas with the PMP and design spatial distribution for the smaller catchment.

6. Derive the temporal distribution over the adjacent area of the rainfall in these storms from basic data.

METHOD. 3

In this method the PMP values for the specified catchment are calculated using the GTSM. The accompanying rainfall on the adjacent areas is represented by 100-year fields of design rainfall intensity based on information given in "Australian Rainfall and Runoff" (ARR), published by the Institution of Engineers, Australia, 1987.

1. Using the GTSM, calculate the PMP for the smaller catchment for the range of durations required.

2. Obtain the spatial distribution of the point 12-hour, 50-year and 72-hour, 50-year rainfall intensities from Volume 2 of ARR.

3. The design spatial distribution of rainfall accompanying the PMP will be based on the 12-hour, 50-year field for durations of up to 36 hours and on the 72-hour, 50-year field for durations from 48 to 120 hours.

4. Estimate the position of the centroid of the area adjacent to the catchment and derive the point 50- and 100- year intensities at this point for each of the durations for which PMP is required.

5. Obtain a point-to-area reduction factor, for the adjacent area, for each duration, using the curves given in Chapter 2 or ARR. In most cases an extrapolation will have to be made. Apply these factors to both the 50-year and 100-year intensities of Step 4.

6. Calculate the ratios of the 100-year intensities for the required durations up to 36 hours to the 12-hour, 50-year value.

7. Apply these ratios to the 12-hour, 50-year field of rainfall intensities (Step 2) to obtain the 100-year field for each of the required durations up to 36 hours. Convert the intensity fields to fields of rainfall totals for each duration by multiplying by the duration (in hours).

8. Repeat Steps 6 and 7 for the durations between 48 hours and 120 hours, using the 72-hour, 50-year value.

9. Use the GTSM temporal distributions as the design temporal distributions of the rainfall over the areas adjacent to the catchment.

METHOD 4

In this method, PMP storms are assumed to occur over the smaller catchment and over the total area simultaneously. The spatial pattern is centred over the smaller catchment but expanded to encompass the larger area. The rainfall over the adjacent area is the difference between the two PMP values.

1. Calculate the PMP using the GTSM for the smaller catchment for the range of durations required.

2. Calculate the PMP using the GTSM for the larger catchment for the range of durations required.

3. Select a spatial distribution according to whether the area of the total or larger catchment is less than or greater than 2000 km^2.

4. Centre the isohyets over the smaller catchment for which the PMP is required and enlarge or reduce the spatial distribution until the outermost isohyet just encloses the larger catchment, as shown in Figure 1.

5. Multiply the mean PMP value for the smaller catchment by the isohyetal percentages for isohyets 1 to m, which encloses the smaller catchment, to obtain initial isohyetal labels in millimetres.

6. Obtain, by planimetering or other means, the values of the sub-areas of the smaller catchment between successive isohyets and also the area inside the central isohyet.

7. Assign each of the areas obtained in Step 6 a weighted mean rainfall figure using the values for the isohyets. Estimate a rainfall value for the area enclosed by the central isohyet.

8. Calculate a mean initial PMP depth over the smaller catchment using the formula:

$$\text{Mean Initial PMP Depth} = \frac{\sum_{i=1}^{m} (\text{Area}_i \times \text{Initial Rainfall Depth}_i)}{\text{Total Catchment Area}}$$

where m is the number of sub-areas of the catchment between successive isohyets;

Area_i is the value of the sub-area of the catchment lying between the isohyets i and i-1;

Initial Rainfall Depth_i is the weighted value of the labels on isohyets i and i-1.

9. Scale the weighted mean rainfall values for the sub-areas to obtain the correct mean PMP depth over the catchment using the formula:

$$\text{Scaling Factor} = \frac{\text{Mean PMP Depth from Step 1}}{\text{Mean Initial PMP Depth from Step 8}}$$

10. Obtain the final isohyetal values in millimetres for the smaller catchment by using the scaling factor obtained in Step 9 and the isohyetal values obtained in Step 5.

To obtain the spatial distribution of the concurrent rainfall over the larger catchment when PMP occurs over the smaller catchment proceed as follows:

11. Using the mean PMP depth obtained in Step 2 obtain values in millimetres for the labels of the isohyets m to the final isohyet. Use the values obtained in Step 10 for the isohyets 1 to m.

12. Repeat Steps 6 to 10 for the larger catchment but leave the values for isohyets 1 to m as calculated in Step 10.

13. If the value of the isohyet m+1 exceeds the value previously obtained for isohyet m interpolate an extra isohyet between isohyets m and m+1 with a value equal to that for isohyet m.

14. Repeat the above procedure for other durations. This can be done by just using the same scaling factors.

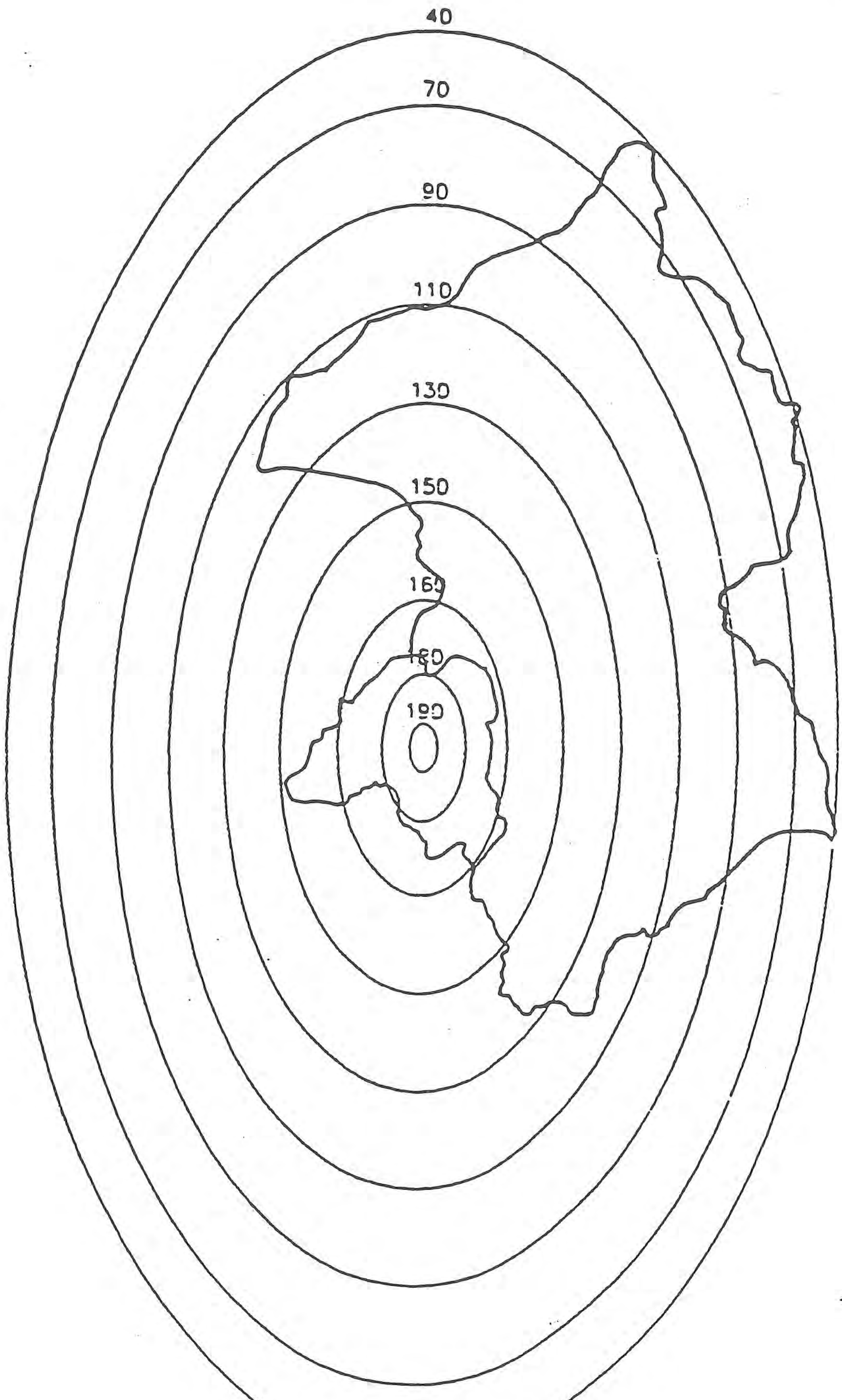


FIGURE 1 Positioning of Spatial Distribution to Obtain Rainfall over the Adjacent Area when the PMP Occurs over the Smaller Catchment

APPENDIX D

HISTORIC FLOOD EVENT SUMMARY

Summary JUNE 1967

Start time:	0800 hrs	9 June 1967
Finish time:	1200 hrs	15 June 1967
Rainfall Duration:	66 hours	
Simulation Duration:	148 hours	

North Pine River at Damsite (Pre-Dam)

Flood Volume		56 400 ML		
Peak Discharge (a)	258 m^3/s	2100 hrs	10 June 1967	
	(b)	385 m^3/s	0100 hrs	12 June 1967

Sideling Creek at Lake Kurwongbah

Flood Volume		7 230 ML		
Peak Inflow	(a)	25 m^3/s	1200 hrs	10 June 1967
	(b)	230 m^3/s	2300 hrs	11 June 1967
Peak Outflow	(a)	14 m^3/s	1600 hrs	10 June 1967
	(b)	111 m^3/s	0100 hrs	12 June 1967
Peak Lake Level	(a)	20.50 m AHD		
	(b)	21.06 m AHD		

South Pine River at Cash's Crossing

Flood Volume		39 160 ML		
Peak discharge (a)	134 m^3/s	1500 hrs	10 June 1967	
	(b)	453 m^3/s	2300 hrs	11 June 1967

Summary JANUARY 1974

Start time:	0900 hrs	24 January 1974
Finish time:	0900 hrs	1 February 1974
Rainfall Duration:	96 hours	
Simulation Duration:	192 hours	

North Pine River at Damsite (Dam under construction)

Flood Volume			265 370 ML	
Peak Inflow	(a)	1382 m^3/s	0900 hrs	25 January 1974
	(b)	1574 m^3/s	1500 hrs	25 January 1974
	(c)	1323 m^3/s	0200 hrs	27 January 1974
Peak Outflow	(a)	775 m^3/s	0200 hrs	26 January 1974
	(b)	974 m^3/s	1000 hrs	27 January 1974
Peak Lake Level	(a)	33.93 m AHD		
	(b)	34.59 m AHD		

Sideling Creek at Lake Kurwongbah

Flood Volume		39 760 ML		
Peak Inflow	(a)	352 m^3/s	1000 hrs	25 January 1974
	(b)	306 m^3/s	2300 hrs	26 January 1974
Peak Outflow	(a)	264 m^3/s	1200 hrs	25 January 1974
	(b)	209 m^3/s	0300 hrs	27 January 1974
Peak Lake Level	(a)	21.55 m AHD		
	(b)	21.40 m AHD		

South Pine River at Cash's Crossing

Flood Volume			171 400 ML	
Peak discharge	(a)	1383 m^3/s	1300 hrs	25 January 1974
	(b)	1132 m^3/s	0300 hrs	26 January 1974
	(c)	1013 m^3/s	0300 hrs	27 January 1974

Summary APRIL 1989

Start time:	0900 hrs	1 April
Finish time:	0900 hrs	4 April
Rainfall Duration:	0072 hrs	
Simulation Duration:	144 hours	

North Pine River at North Pine Dam

Flood Volume		97 020 ML		
Peak Outflow	(a)	460 m^3/s	0445 hrs	2 April 1989
	(b)	1338 m^3/s	1615 hrs	2 April 1989
Peak Lake	(a)	39.93 m AHD		
Level	(b)	40.45 m AHD		

Sideling Creek at Lake Kurwongbah

Flood Volume		12 870 ML		
Peak Inflow	(a)	100 m^3/s	1800 hrs	1 April 1989
	(b)	256 m^3/s	1300 hrs	2 April 1989
Peak Outflow	(a)	41 m^3/s	2100 hrs	1 April 1989
	(b)	201 m^3/s	1500 hrs	2 April 1989
Peak Lake	(a)	20.67 m AHD		
Level	(b)	21.37 m AHD		

South Pine River at Cash's Crossing

Flood Volume		49 400 ML		
Peak Dis-	(a)	292 m^3/s	2400 hrs	1 April 1989
charge	(b)	790 m^3/s	1400 hrs	2 April 1989