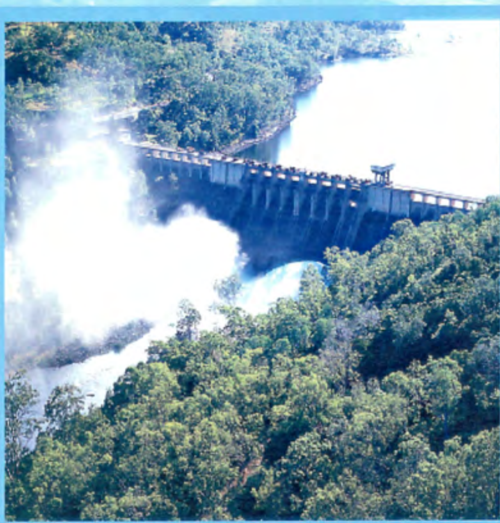
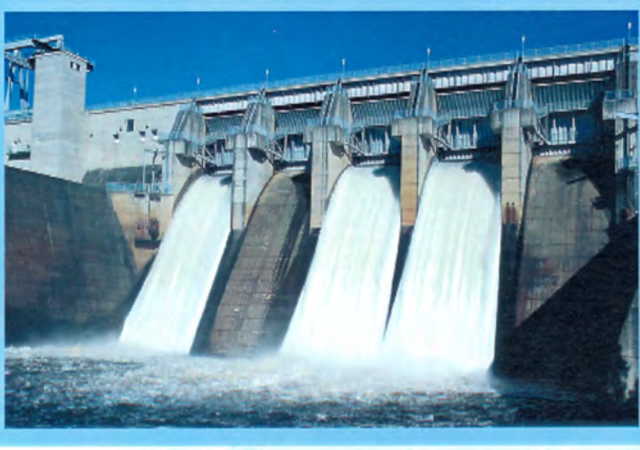
MARCH 1993

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

BRISBANE RIVER AND PINE RIVER FLOOD STUDY :

Report No. 8a





# BRISBANE RIVER FLOOD HYDROLOGY REPORT VOLUME I

Design Flood Estimation

South East Queensland WATER BOARD

NATURAL RESOURCES Queensland

Brisbane River and Pine River Flood Studies

# BRISBANE RIVER FLOOD HYDROLOGY REPORT

# DESIGN FLOOD ESTIMATION FOR SOMERSET DAM AND WIVENHOE DAM

## MAIN REPORT

Volume I
March 1993

# TABLE OF CONTENTS

APPENDICES

# LIST OF TABLES

## List of Figures

# 1.0 INTRODUCTION

A review of the performance during floods of Somerset Dam and Wivenhoe Dam was commissioned by the South East Queensland Water Board, (SEQWB), with the study being undertaken by the Water Resources, (WR). This study included the revision of design floods for the storages, dambreak flood modelling downstream of the storages, and the development of a management model for flood operations of the storages.

This report describes the reassessment of design flood estimates for Somerset Dam and Wivenhoe Dam based upon recent probable maximum precipitation, (PMP), estimates supplied by the Bureau of Meteorology. Design flood estimates for higher probability of exceedance events have also been re-derived using rainfall intensity-frequency-duration procedures outlined in Australian Rainfall and Runoff, (1987).

Flood runoff from a catchment can be estimated from storm rainfall by using a runoff-routing model. The modified and re-calibrated Brisbane Valley runoff-routing models as described by Ayre, Cutler and Ruffini, (1992), have been utilised in conjunction with a storage operation model to reassess the design floods of the storages. Flood frequency techniques have also been used as an independent means of estimating the magnitude of design floods.

# 2.0 SUMMARY OF RESULTS

## 2.1 SOMERSET DAM

The critical design storm scenario for Somerset Dam is the design storm centred over the Stanley River catchment. The 120 hour duration probable maximum flood, (PMF), produces the largest outflow from Somerset Dam under existing normal gate operation procedures.

The 48 hour duration event was selected for the derivation of a full flood frequency of outflows from the dam because it is not possible to estimate 120 hour duration, 100 year average recurrence interval, (ARI), rainfalls using the methods from Australian Rainfall and Runoff, (1987). The 120 hour and 48 hour duration events do produce outflows of similar magnitude however. The results of the design flood re-assessment for Somerset Dam are presented in Table 2.1.

**Table 2.1**
**Somerset Dam Design Flood Estimates**
**Storm Centred over Stanley River Catchment**

| ARI (YEARS) | STORM DURATION (HOURS) | PEAK INFLOW ($m^3/s$) | PEAK OUTFLOW ($m^3/s$) | FLOOD VOLUME (ML) | PEAK LAKE LEVEL (m AHD) |
|---|---|---|---|---|---|
| 10 | 36 | 2 350 | 2 130 | 239 430 | 102.12 |
| 20 | 36 | 2 980 | 2 280 | 300 470 | 102.24 |
| 50 | 36 | 3 810 | 2 410 | 379 380 | 102.68 |
| 100 | 36 | 4 560 | 2 550 | 450 240 | 103.13 |
| 200 | 48 | 3 580 | 2 720 | 557 590 | 103.62 |
| 500 | 48 | 4 350 | 3 070 | 667 080 | 104.49 |
| 1 000 | 48 | 4 940 | 3 400 | 770 000 | 105.19 |
| 10 000 | 48 | 7 070 | 4 620 | 1 101 880 | 107.37 |
| 100 000 | 48 | 9 110 | 6 280 | 1 420 480 | 109.04 * |
| 1 000 000(PMF) | 48 | 11 070 | 8 000 | 1 725 780 | 110.31 * |
| 1 000 000(PMF) | 120b | 9 630 | 8 140 | 2 504 600 | 110.41 * |

Notes:

* Indicates that non-overflow section of the spillway (El 107.46 m AHD) is overtopped.

b Refers to various design rainfall temporal patterns recommended by the Bureau of Meteorology.

The duration of the event that produces the peak outflow from Somerset Dam changes from 48 hours to 36 hours for events of more frequent occurrence, because of the use of temporal patterns from Australian Rainfall and Runoff, (1987), with these events. The difference in temporal patterns affects events up to 1 in 100 years ARI.

An estimate of the magnitude of the flood event which when routed through the storage of Somerset Dam under the existing normal operating procedure just threatens to overtop the non-overflow spillway, (107.46 m AHD), has been made. The rainfall depth associated with this flood equates to approximately 68 % of the PMP. The ARI of this rainfall depth is estimated to be 20 000 years.

For embankment dams this flood is normally referred to as the Imminent Failure Flood, (IFF). However, according to ANCOLD guidelines, (1986), for concrete dams, the IFF can often be a flood for which the stillwater pool level is above the top of the dam or even the parapet. Assessment of the IFF should be based upon the structural stability of the dam with the reservoir at the flood level, and the capability of the downstream foundations to resist the overtopping flow.

Russo, (1988), concludes in his report on the safety of Somerset Dam that structurally, the dam is in excellent condition. He states that the dam can be used to hold back flood waters in extreme events to prevent overtopping of Wivenhoe Dam. These assessments are based upon estimated flood levels which are very similar to, but different from, the levels determined in this reassessment, (refer Section 4.2).

Russo also recommends that to ensure the survival of the portions of two non-overflow monoliths above RL 100.0 m AHD, the reservoir level should not exceed RL 111.7 m AHD. He adds that the structural integrity of the spillway gates would have to be checked for the loads such a reservoir level would impose.

Not withstanding the check on the spillway gates, it would appear that Somerset Dam could withstand being overtopped to a level of 111.7 m AHD. This level is higher than the reservoir level estimated for the PMF, (110.41 m AHD). **It can therefore be concluded that Somerset Dam is structurally sound enough to withstand and safely pass the PMF with existing normal gate operation.**

## 2.2 WIVENHOE DAM

The critical design storm scenario for Wivenhoe Dam is the design storm centred over the whole of the catchment of Wivenhoe Dam. The 48 hour duration PMF produces the largest outflow from Wivenhoe Dam under existing normal gate operation procedures. The results of the design flood re-assessment for Wivenhoe Dam are presented in Table 2.2.

The duration of the event that produces the peak outflow from Wivenhoe Dam changes from 48 hours to 72 hours for events of more frequent occurrence, because of the use of temporal patterns from Australian Rainfall and Runoff, (1987), with these events. Again, this only affects events up to 1 in 100 years ARI.

**Table 2.2**
**Wivenhoe Dam Design Flood Estimates**
**Storm Centred over Wivenhoe Dam Catchment**

| ARI (YEARS) | STORM DURATION (HOURS) | PEAK INFLOW ($m^3/s$) | PEAK OUTFLOW ($m^3/s$) | FLOOD VOLUME (ML) | PEAK LAKE LEVEL (m AHD) |
|---|---|---|---|---|---|
| 10 | 72 | 3 630 | 2 900 | 861 570 | 68.18 |
| 20 | 72 | 4 980 | 3 330 | 1 128 590 | 70.58 |
| 50 | 72 | 7 240 | 3 450 | 1 405 480 | 72.83 |
| 100 | 72 | 9 080 | 6 810 | 1 860 400 | 74.48 |
| 200 | 48 | 11 110 | 7 640 | 1 822 840 | 74.84 |
| 500 | 48 | 12 580 | 9 130 | 2 104 520 | 75.50 |
| 1 000 | 48 | 13 820 | 9 970 | 2 336 350 | 75.99 |
| 10 000 | 48 | 20 770 | 13 490 | 3 593 000 | 78.61 |
| 100 000(PMF) | 48 | 30 670 | 13 490 | 5 333 920 | 81.28 * |
| Note: * Indicates that the embankment crest level, (EL 79.15 m AHD) is overtopped. | | | | | |

Embankment dams when subject to a continuous overtopping flow will normally fail, depending upon the duration of the flow and the likely extent of scouring of the crest. The Imminent Failure Flood, (IFF), for Wivenhoe Dam has therefore been assessed as the flood event which when routed through the storage under the existing storage operating procedure just threatens to overtop the embankment. The embankment crest level, (79.15 m AHD), has been adopted as the critical level in preference to the top of the wave wall, (79.90 m AHD), because the wave wall does not extend over the whole of the embankment.

The estimated magnitude of the rainfall depth associated with the IFF for Wivenhoe Dam is 75 % of the PMP. This rainfall depth has an ARI of approximately 14 300 years.

The peak inflow associated with the IFF of Wivenhoe Dam is estimated to be 21 990 $m^3/s$, whilst the resultant peak outflow from the dam is 14 080 $m^3/s$. The flood volume for the IFF is estimated to be 3 794 180 ML.

By way of comparison, if the top of the wave wall, (79.90 m AHD), is adopted as the critical level, the magnitude of rainfall depth associated with this flood is 81 % of the PMP. This rainfall depth has an ARI of approximately 23 600 years.

The peak inflow associated with this flood is estimated to be 24 060 $m^3/s$, whilst the resultant peak outflow from the dam is 14 920 $m^3/s$. The flood volume for this flood is estimated to be 4 162 020 ML.

## 3.0 BRISBANE VALLEY CATCHMENT DESCRIPTION

The Brisbane Valley has a total catchment area of some 13 570 $km^2$. The valley is bounded by the Great Dividing Range on the west and by a number of smaller coastal ranges to the east and north. Most of the Brisbane River catchment lies to the west of the coastal ranges. Refer to the locality plan in Figure 3.1.

The Brisbane River system consists of the Brisbane River and its six major tributaries. From its headwaters in the Brisbane and Jimna Ranges, the Brisbane River flows in a generally south-easterly direction, before running almost north-easterly into Moreton Bay.

Cooyar Creek, Emu Creek and Cressbrook Creek are the major tributaries of the Upper Brisbane River that flow eastward from the Great Dividing Range. The most northerly of the Upper Brisbane River tributaries is Cooyar Creek. Cooyar Creek has a catchment area of around 1 065 $km^2$ and its catchment is regarded as the driest of the Brisbane River tributaries. Emu Creek, located immediately to the south of Cooyar Creek also flows in a north-easterly direction and it also has a catchment area of about 1 000 $km^2$. The remaining major tributary of the Upper Brisbane River, Cressbrook Creek, has a catchment area of 620 $km^2$.

The Stanley River is the only major tributary of the Brisbane River that flows westwards from the Conondale and D'Aguilar Ranges near the coast. The Stanley River catchment is situated in the steepest and wettest part of the whole Brisbane Valley.

Somerset Dam is situated on the Stanley River some 7 km upstream from its confluence with the Brisbane River. The catchment area of the dam is approximately 1 330 $km^2$. Lake Somerset dominates the Lower Stanley River catchment extending some 40 km upstream and having a surface area at full supply level of about 44 $km^2$.

Somerset Dam is a multi-purpose dam, being used as a water supply for the cities of Brisbane and Ipswich and a number of surrounding shires; in addition it has major flood mitigation capabilities and it is used for recreational activities. It also has minor hydro-electric power generation capabilities.

The dam is a mass gravity concrete structure that has a capacity at full supply level of 369 750 ML with a further 524 000 ML of flood storage available. The spillway is equipped with eight radial sector gates, whilst other outlet works consist of eight low level sluice gates and four fixed dispersion cone valve regulators. Design of the dam commenced in the late 1930,s but construction was not completed until 1959 because of wartime delays.

Wivenhoe Dam is also a multi-purpose dam that has similar functions to that of Somerset Dam, although it is also used in conjunction with Splityard Creek Dam for hydro-electricity generation. Wivenhoe Dam commands over half of the whole Brisbane River catchment, having a catchment area of about 7040 $km^2$, (including the catchment of Somerset Dam). At full supply level the dam has a capacity of 1 150 000 ML with an additional 1 450 000 ML of flood storage available.

Wivenhoe Dam differs from Somerset Dam in that it has a zoned earth and rockfill type embankment. The spillway of Wivenhoe Dam is equipped with five radial gates, whilst low

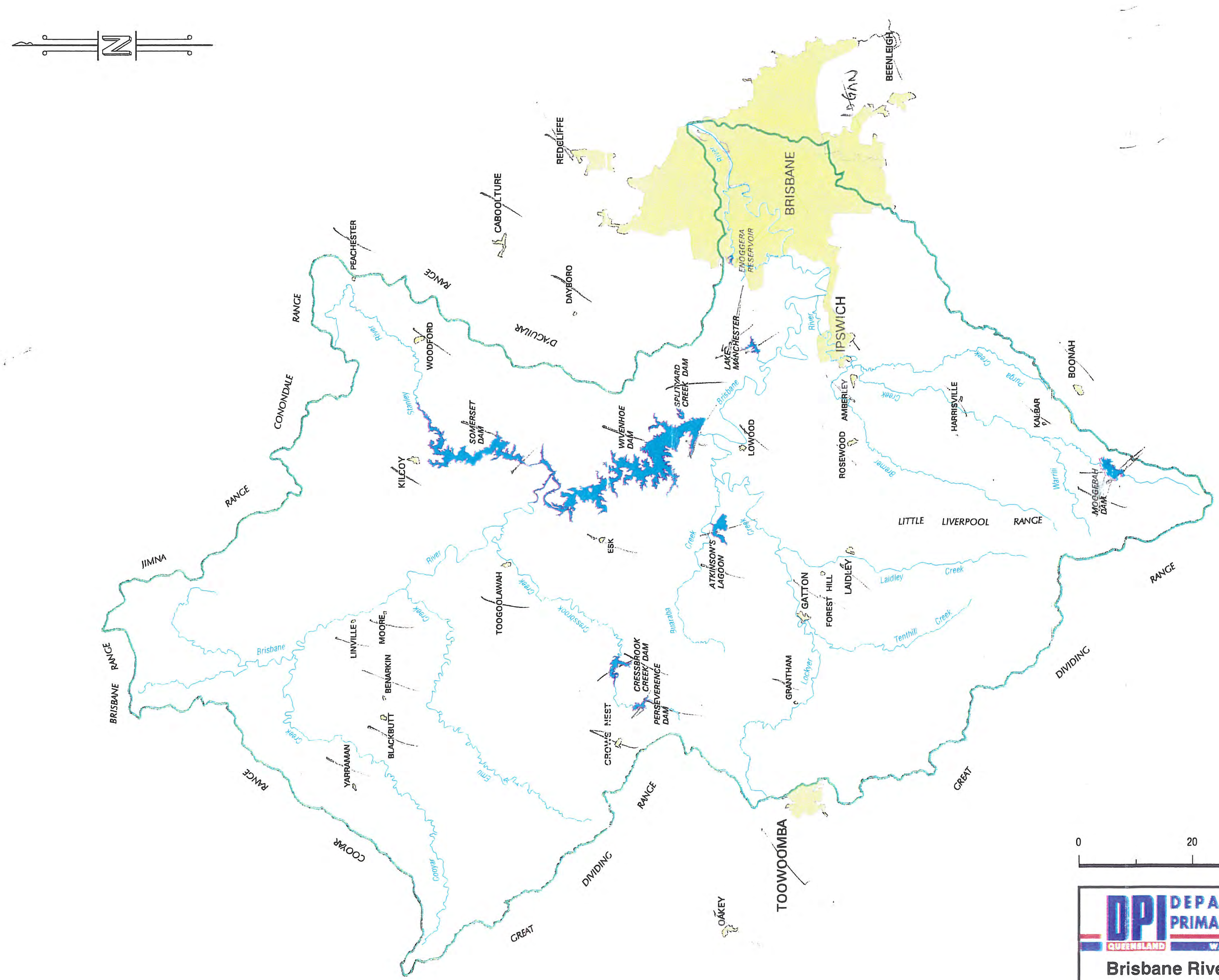
level releases are made through two fixed dispersion cone valve regulators. Construction on Wivenhoe Dam commenced in 1979 and the dam was completed in 1985.
Splityard Creek Dam is located on Pryde Creek and it has a capacity of 28 600 ML. The dam has a catchment area of only 3.6 $km^2$ because it is the upper storage of a pumped storage scheme. The dam is owned and operated by the Queensland Electricity Commission, (QEC).

Lockyer Creek flows east from the Great Dividing Range to join the Brisbane River just downstream of Wivenhoe Dam. Lockyer Creek has a catchment area of about 3 000 $km^2$ making it the largest tributary of the Brisbane River in terms of catchment area.

The last of the major Brisbane River tributaries is the Bremer River. The Bremer River rises in the Little Liverpool Range and its catchment is generally hilly and lightly forested. A major tributary of the Bremer River is Warrill Creek, which flows from the Great Dividing Range in the south to join the Bremer River in its lower reaches just upstream from the outskirts of Ipswich city. Warrill Creek, at its confluence with the Bremer River, has a catchment area of about 900 $km^2$, whereas the Bremer River is only 600 $km^2$ in area at this point.

From its confluence with the Bremer River, the Brisbane River meanders its way to Moreton Bay in a generally north-easterly direction. The city of Brisbane encompasses almost the whole of the lower Brisbane River flood plain from this point.

FIGURE 3.1




KILOMETRES

0 20 40 60 75

DPI DEPARTMENT OF PRIMARY INDUSTRIES
QUEENSLAND WATER RESOURCES

**Brisbane River Flood Study**
**Brisbane River**
# Locality Plan

17 Mar 93

A3-99454

# 4.0 PREVIOUS FLOOD STUDIES

## 4.1 INTRODUCTION

The flood hydrology of Wivenhoe Dam and Somerset Dam has been investigated on a number of previous occasions. The latest investigation concerning Wivenhoe Dam was undertaken by the Water Resources Commission, (WRC), during the construction phase of the dam, (Weeks, 1983 and 1984). The Brisbane City Council, (BCC), reviewed the flood hydrology and safety of Somerset Dam as recently as 1988, (Cossins, 1988 and Russo, 1988). A brief summary of the findings of these and earlier reports is presented in this section.

## 4.2 SOMERSET DAM

Investigations of the damsite at the location of Somerset Dam commenced in the mid-1930's. Original design flood estimates and the design and construction of the Dam were undertaken by the Stanley River Works Board (SRWB), which was formed in November 1934. The engineering branch of the SRWB was absorbed into the Co-ordinator General's Department after the Second World War. The engineering design function of the Co-ordinator General's Department has now been discontinued and as a consequence the original design flood estimates are not readily available. Whilst the actual design calculations are not available, it is known that unit hydrograph techniques were utilised to produce the design flood estimates.

The greatest flood on record at this site was the February 1893 flood event, which had an estimated peak flow of 4 250 $m^3/s$ and a flood volume of 1 130 000 ML, (Russo, 1988). The original design flood estimate for the Somerset Dam storage was derived from the 1893 flood event. The original design flood peak discharge was estimated to be 5 100 $m^3/s$ with a corresponding volume of 1 700 000 ML. The estimated maximum flood level in the reservoir was RL 355 ft, (EL 108.11 m), which is 0.6 metres above the non-overflow crest level.

During the investigation of floods for Wivenhoe Dam conducted in 1977, (Hausler and Porter), the WRC used synthetic unitgraphs based on the Clark-Johnstone method to estimate inflows into Somerset Dam. Synthetic unitgraphs were also derived for the Upper Brisbane River and the residual catchment area below the confluence of the Upper Brisbane River and the Stanley River.

The derived inflow hydrograph for Somerset Dam was routed through the reservoir according to the BCC flood operation procedures that were applicable at that time, (Cossins, 1969). The resultant outflow hydrograph was then combined with the hydrographs of the other catchments in order to determine an inflow hydrograph into the proposed Wivenhoe Dam.

The design rainfalls that were used in conjunction with the synthetic unitgraphs were made up of a combination of PMP estimates supplied by the Bureau of Meteorology and rainfall frequency estimates for higher probability events. The rainfall frequency analysis was also performed by the Bureau.

The PMP estimate for Somerset Dam was calculated by maximising storms 'insitu' without any transposition. The February 1893 storm was selected from a number of events for this determination. The storm was centred over the Stanley River catchment and as a result, PMP rainfalls in the west of the Upper Brisbane River catchment were quite low.

Daily catchment rainfall depths were derived for each of the three major sub-catchments from an annual series frequency analysis of one, two and three day rainfalls. A log-normal distribution was fitted to each of the series.

A summary of the PMP for Somerset Dam is provided in Table 4.1.

**Table 4.1**
**Summary of PMP Estimates for Somerset Dam**
**(Hausler and Porter, 1977)**

| DURATION (HOURS) | RAINFALL DEPTH (mm) |
|---|---|
| 6 | 240 |
| 24 | 620 |
| 48 | 960 |
| 72 | 1 120 |
| 96 | 1 260 |
| 120 | 1 300 |
| 144 | 1 320 |
| 168 | 1 380 |

Six different rainfall temporal patterns were considered for use in the determination of extreme design floods. The temporal pattern associated with the critical peak discharge estimates for all of the sub-catchments was the 'Brisbane Late' pattern. All of the patterns that were considered were derived by the WRC from pluviograph records obtained from the Bureau of Meteorology.

Rainfall loss rates of 0 mm initial loss and 1 mm/hour continuing loss were adopted for the design events.

The critical duration for inflows into Wivenhoe Dam was determined to be 33 hours which took into account the operation of Somerset Dam. Under normal conditions of operation, discharge from Somerset Dam was limited to a maximum of 1 133 $m^3/s$, until the flood peak from the Upper Brisbane River had passed the junction of the Stanley River.

A summary of the results of the Somerset Dam flood estimates is presented in Table 4.2.

**Table 4.2**
**Summary of Design Flood Estimates for Somerset Dam**
**(Hausler and Porter, 1977)**

| AEP (%) | DURATION (HOURS) | PEAK DISCHARGE ($m^3/s$) | | FLOOD VOLUME (ML) |
|---|---|---|---|---|
| | | INFLOW | OUTFLOW | |
| 0.01 (PMF) | 168 | 8 650 | 5 800 | 1 616 500 |
| 0.1 | 33 | 8 120 | | 632 800 |
| 0.2 | 33 | 7 480 | | 582 000 |
| 1 | 33 | 5 580 | | 429 000 |
| 2 | 33 | 4 780 | | 365 000 |

The duration of the probable maximum flood, (PMF), was governed by the availability of the temporal pattern. The Bureau only provided a seven day temporal pattern for the maximised February 1893 storm.

Design floods for Wivenhoe Dam were revised during the construction phase of the Dam because of changes in the methods the Bureau of Meteorology used to determine PMPs. The revision was performed by the WRC in 1983, (Weeks). Design floods for Somerset Dam were also determined in this revision.

Runoff-routing model techniques were used to derive flood estimates in this revision. The runoff-routing model for Somerset Dam was calibrated to storage records provided by the BCC for five different flood events. Model parameters of k = 94 with m = 0.75 were determined from this calibration.

It should be noted that the Somerset Dam model was also developed using the mean travel distance of the whole catchment to Wivenhoe Dam. This model has calibration parameters of k = 500 and m = 0.75.

The Bureau of Meteorology calculated the PMP for four catchments for this revision. These catchments included:

(a) Area above the junction of the Stanley River and the Brisbane River.

(b) Area above Somerset Dam on the Stanley River.

(c) Area below the junction of the Stanley River and the Brisbane River down to Wivenhoe Dam.

(d) Area above Wivenhoe Dam including Somerset Dam.

The method of adjusted United States data was used for the six hour duration storm and the generalised method for areas subject to tropical cyclones was used for the one day duration storm for all catchments.

The 12 hour, two and three day duration storm rainfalls were derived from the one day value. General meteorological considerations were used to derive storm rainfalls for the four, five, six and seven day duration storms.

A summary of the PMP estimates for Somerset Dam catchment is presented in Table 4.3.

**Table 4.3**
**Summary of PMP Estimates for Somerset Dam**
**(Weeks, 1983)**

| DURATION (HOURS) | RAINFALL DEPTH (mm) |
|---|---|
| 6 | 400 |
| 12 | 560 |
| 24 | 840 |
| 48 | 1 360 |
| 72 | 1 760 |
| 96 | 2 040 |
| 120 | 2 120 |
| 144 | 2 160 |
| 168 | 2 340 |

The WRC performed a rainfall frequency analysis on sixteen rainfall stations located throughout the Wivenhoe Dam catchment in order to determine higher probability design rainfalls. Annual series of one,two and three day maxima were extracted from the record of each station and a log-normal distribution was fitted.

Temporal patterns provided by the Bureau were used with the more extreme design events. Storms with a probability of exceedance of 1 % or less were considered in this category.

Adopted rainfall loss rates differed from the previous study. An initial loss of 0 mm was utilised as before, however a larger continuing loss rate of 2.5 mm/hour was adopted. This value is specified in Australian Rainfall and Runoff, (1977), and is used in design flood analysis by the Water Resources on a routine basis.

A different flood operation procedure was used in the determination of design floods for Somerset Dam in this study.

The policy was developed jointly by the BCC and the WRC, taking into account the presence of Wivenhoe Dam. The normal operation procedure that was adopted is described in the Manual of Operational Procedures for Flood Mitigation for Wivenhoe Dam and Somerset Dam, (1985). An excerpt from the manual is presented below:

> "The spillway gates are to be raised to enable uncontrolled discharge once the flood storage between FSL (EL 99.0 m) and spillway level (EL 100.45 m) has filled. The low level regulators and sluices are to be kept closed until either:
>
> (a) the inflow to Wivenhoe Dam begins to decrease; or
>
> (b) the level in Somerset Dam exceeds EL 102.25 m.
>
> In the case of (a) the opening of the regulators and sluices is not to increase the inflow into Wivenhoe Dam above the peak inflow just passed. In the case of (b) the 'Engineer' shall determine the order of opening of the gates to meet the objectives of the flood operation manual".

A summary of the estimated design floods for Somerset Dam is provided in Table 4.4.

**Table 4.4**
**Summary of Design Flood Estimates for Somerset Dam**
**(Weeks, 1983)**

| AEP (%) | DURATION (HOURS) | PEAK DISCHARGE ($m^3/s$) | | FLOOD VOLUME (ML) |
|---|---|---|---|---|
| | | INFLOW | OUTFLOW | |
| 0.001 (PMF) | 48 | 12 160 | 8 440 | 1 670 000 |
| | 168 | 14 170 | 8 690 | 2 800 000 |
| 0.01 | 48 | 7 780 | 4 900 | 1 077 000 |
| 0.1 | 48 | 5 310 | 3 440 | 738 000 |
| 1 | 24 | 3 250 | 2 570 | 342 000 |

Weeks noted that the peak discharge from the 48 and 168 hour duration events should be the same, because the 168 hour event was made up from two 48 hour storms separated by 72 hours of no rainfall. However due to inaccuracies in the interpretation of the hand drawn temporal patterns supplied by the Bureau of Meteorology, the 168 hour duration PMF produced slightly higher estimates of peak discharge.

Russo, (1988) in his report on the safety review of Somerset Dam makes reference to recent estimates of PMF determined by the BCC.

The peak discharge inflow estimate for Somerset Dam was 12 000 $m^3/s$ which when routed through the dam gave an outflow of 8 000 $m^3/s$. The maximum water level in the reservoir for this flood is EL 110.39 m, which is 2.93 metres above the non-overflow crest level. The ARI of this flood is of the order of 1 in 1 000 000.

Russo does not provide any further detail concerning the revised PMF estimates other than to state that these estimates are based upon the latest method of PMP estimation. The results are very similar to those quoted by Weeks, (1983).

## 4.3 WIVENHOE DAM

The original design flood estimates for Wivenhoe Dam were derived by the WRC in 1977, (Hausler and Porter). As mentioned in the previous section, synthetic unitgraphs based on the Clark-Johnstone method were utilised to determine the design flood estimates. Separate unitgraphs were derived for the Upper Brisbane River catchment, Somerset Dam catchment and the residual catchment area to Wivenhoe Dam. Estimates of inflows into Wivenhoe Dam were made for pre and post-dam scenarios.

PMP rainfalls were derived by the Bureau of Meteorology by maximising the February 1893 storm 'insitu', that is without any transposition. The storm was centred over the Stanley River catchment which resulted in some of the design rainfalls being quite low, particularly in the western part of the Upper Brisbane River catchment. Rainfall-frequency-duration relationships for the higher probability design events for the three sub-catchments were also supplied by the Bureau. A log-normal distribution was fitted to annual series of one, two and three day catchment rainfalls to achieve this.

A summary of the PMP design rainfalls derived for the three sub-catchments and for the overall Wivenhoe Dam catchment is summarised in Table 4.5.

**Table 4.5**
**Summary of PMP Estimates for Wivenhoe Dam**
**(Hausler and Porter, 1977)**

| DURATION (HOURS) | RAINFALL DEPTH (mm) | | | |
|---|---|---|---|---|
| | SUB-CATCHMENT | | | |
| | UPPER BRISBANE | SOMERSET DAM | RESIDUAL | WIVENHOE DAM |
| 6 | 70 | 240 | 100 | 100 |
| 24 | 220 | 620 | 330 | 320 |
| 48 | 340 | 960 | 540 | 480 |
| 72 | 440 | 1 120 | 700 | 600 |
| 96 | 480 | 1 260 | 760 | 660 |
| 120 | 490 | 1 300 | 820 | 680 |
| 144 | 500 | 1 320 | 880 | 700 |
| 168 | 520 | 1 380 | 900 | 740 |

The temporal pattern that was used in conjunction with the extreme design flood rainfalls was the 'Brisbane Late' pattern. This pattern was selected from the six patterns that were considered because it produced the greatest peak estimates for all sub-catchments. All of the patterns considered were derived by the WRC from pluviograph records obtained from the Bureau of Meteorology.

An initial loss rate of 0 mm and a continuing loss of 1 mm/hour were adopted for the range of design floods investigated.

Somerset Dam was assumed to operate in the manner described by Cossins, 1969. (Refer to Section 4.2).

The critical duration for inflows into Wivenhoe Dam was found to be 33 hours, which included the operation procedure of Somerset Dam.

Estimates of design floods for the catchment of Wivenhoe Dam in its natural state are summarised in Table 4.6.

**Table 4.6**
**Summary of Design Flood Estimates**
**Pre-Wivenhoe Dam**
**(Hausler and Porter, 1977)**

| AEP (%) | DURATION (HOURS) | PEAK DISCHARGE ($m^3/s$) | FLOOD VOLUME (ML) |
|---|---|---|---|
| 0.01 (PMF) | 168 | 13 650 | 4 119 570 |
| 0.1 | 33 | 10 230 | 2 279 890 |
| 0.2 | 33 | 8 320 | 2 075 870 |
| 1 | 33 | 6 180 | 1 589 440 |
| 2 | 33 | 5 450 | 1 376 470 |

Estimates of design flood inflows into Wivenhoe Dam are presented in Table 4.7.

**Table 4.7**
**Summary of Design Flood Estimates**
**Inflows to Wivenhoe Dam**
**(Hausler and Porter, 1977)**

| AEP (%) | DURATION (HOURS) | PEAK DISCHARGE ($m^3/s$) | FLOOD VOLUME (ML) |
|---|---|---|---|
| 0.01 (PMF) | 168 | 15 090 | 4 119 570 |
| 0.1 | 33 | 12 360 | 2 279 890 |
| 0.2 | 33 | 10 500 | 2 075 870 |
| 1 | 33 | 8 450 | 1 589 440 |
| 2 | 33 | 7 270 | 1 376 470 |

The duration of the PMF was governed by the availability of a temporal pattern for the maximised February 1893 storm. The Bureau only provided a seven day pattern for the PMF.

The WRC revised design floods for Wivenhoe Dam in 1983, (Weeks), when the dam was in its final phase of construction. The revision of design floods was undertaken because the Bureau of Meteorology changed the manner in which it determined PMP rainfall depths.

Runoff-routing model techniques were utilised in this revision to determine design flood estimates for Somerset Dam and Wivenhoe Dam. Two models were developed. One model was constructed for the catchment of Somerset Dam and the other consisted of the combined Upper Brisbane River and residual area catchments. Details of the Somerset Dam model calibration were presented in the previous section.

The Wivenhoe Dam model, (excluding Somerset Dam), was calibrated to seven separate flood events at six different locations. Model parameters derived from the calibration were k= 140 and m= 0.75.

Design rainfall estimates for high probability events were derived from a frequency analysis of sixteen different rainfall stations located throughout the catchment of Wivenhoe Dam. Log-normal distributions were fitted to the one, two and three day peak annual rainfall series at each of the stations.

The Bureau of Meteorology calculated the PMP for four catchments. These catchments included:

(a) Area above the junction of the Stanley River and the Brisbane River.

(b) Area above Somerset Dam on the Stanley River.

(c) Area below the junction of the Stanley River and the Brisbane River down to Wivenhoe Dam.

(d) Area above Wivenhoe Dam including Somerset Dam.

The method of adjusted United States data was used for the six hour duration storm and the generalised method for areas subject to tropical cyclones was used for the one day duration storm for all catchments. The 12 hour, two and three day duration storm rainfalls were derived from the one day value. General meteorological considerations were used to derive storm rainfalls for the four, five, six and seven day duration storms.

A summary of the PMP estimates for all four the catchments is presented in Table 4.8.

Temporal patterns provided by the Bureau were used with the more extreme design events. Storms with a probability of exceedance of 1 % or less were considered in this category.

**Table 4.8**
**Summary of PMP Estimates for Wivenhoe Dam**
**(Weeks, 1983)**

| DURATION (HOURS) | RAINFALL DEPTH (mm) | | | |
|---|---|---|---|---|
| | SUB-CATCHMENT | | | |
| | UPPER BRISBANE | SOMERSET DAM | RESIDUAL | WIVENHOE DAM |
| 6 | 300 | 400 | 420 | 260 |
| 12 | 420 | 560 | 560 | 380 |
| 24 | 660 | 840 | 820 | 600 |
| 48 | 1 080 | 1 380 | 1 360 | 1 000 |
| 72 | 1 380 | 1 760 | 1 720 | 1 260 |
| 96 | 1 600 | 2 040 | 2 000 | 1 460 |
| 120 | 1 660 | 2 120 | 2 080 | 1 520 |
| 144 | 1 700 | 2 160 | 2 120 | 1 560 |
| 168 | 1 840 | 2 340 | 2 320 | 1 700 |

Rainfall loss rates differed from the previous study. An initial loss of 0 mm was utilised as before, however a larger continuing loss rate of 2.5 mm/hour was adopted. The value of continuing loss rate was somewhat higher than the average values determined from the model calibration but it does correspond to values specified in Australian Rainfall and Runoff, (1977), and it is of an order that is used in design flood analysis on a routine basis by Water Resources.

Design flood estimates were derived for Wivenhoe Dam assuming that the dam did not exist. A second case assuming the dam existed and that it was at normal full supply level prior to the commencement of the flood event was also investigated. In both cases Somerset Dam was assumed to operate according to the flood operation policy described in Section 4.2.

For the determination of PMFs, the storm rainfall was assumed to be centred over the whole of the Wivenhoe Dam catchment. Outflows from Somerset Dam were calculated for this scenario and added to the remainder of the Wivenhoe Dam catchment flows. The results of this analysis are presented in Table 4.9.

**Table 4.9**
**Summary of Design Flood Estimates for Inflows into Wivenhoe Dam from Somerset Dam**
**(Rainfall Centred on Wivenhoe Dam Catchment)**
**(Weeks, 1983)**

| DURATION (HOURS) | PEAK DISCHARGE ($m^3/s$) | | FLOOD VOLUME (ML) |
|---|---|---|---|
| | INFLOW | OUTFLOW | |
| 24 | 7 200 | 4 340 | 717 000 |
| 48 | 8 440 | 5 350* | 1 169 000 |
| 72 | 7 550 | 5 040 | 1 432 000 |
| 96 | 8 110 | 5 120* | 1 627 000 |
| 120 a | 8 370 | 5 270* | 1 660 000 |
| 120 b | 8 550 | 4 920 | 1 660 000 |
| 144 a | 8 340 | 5 190* | 1 630 000 |
| 144 b | 8 710 | 5 390* | 1 630 000 |
| 144 c | 8 890 | 5 320* | 1 630 000 |
| 168 | 9 880 | 5 480* | 1 950 000 |

Notes:

* Indicates that flow occurred over the non-overflow spillway, (EL 107.46 m AHD).

a,b,c Refer to the various design rainfall temporal patterns recommended by the Bureau of Meteorology.

Estimates of design floods derived for Wivenhoe Dam for the pre-dam and post-dam cases are summarised in Tables 4.10 and 4.11. This is for the scenario where the design storm rainfall is centred over the whole of the Wivenhoe Dam catchment.

**Table 4.10**
**Summary of Design Flood Estimates**
**Pre-Wivenhoe Dam**
**(Weeks, 1983)**

| AEP (%) | DURATION (HOURS) | PEAK DISCHARGE ($m^3/s$) | FLOOD VOLUME (ML) |
|---|---|---|---|
| 0.01 (PMF) | 168 | 44 470 | 10 260 000 |
| | 48 | 42 170 | 6 170 000 |
| 0.01 | 48 | 19 070 | 3 045 000 |
| 0.1 | 48 | 12 910 | 2 044 000 |
| 1 | 48 | 8 300 | 1 234 000 |

It should be noted that PMF estimates for durations of 120 (a) hours and 144 (b) provide peak discharges that are greater than the 48 hour duration value, but not as large as the 168 hour value. The (a) and (b) refer to various temporal patterns recommended by the Bureau of Meteorology.

Only durations of up to 72 hours were considered for the higher probability of exceedance design flood events. The 48 hour duration design flood event had the largest peak discharges for the events considered and as a consequence this duration was adopted as the critical duration.

The same note that also applied to the Somerset Dam estimates is also applicable to the Wivenhoe Dam estimates. The peak discharge from the 48 and 168 hour duration events should be the same, because the 168 hour event was made up from two 48 hour storms separated by 72 hours of no rainfall. However due to inaccuracies in the interpretation of the hand drawn temporal patterns supplied by the Bureau of Meteorology, the 168 hour duration PMF produced slightly higher estimates of peak discharge.

**Table 4.11**
**Summary of Design Flood Estimates**
**Inflows to Wivenhoe Dam**
**(Weeks, 1983)**

| AEP (%) | DURATION (HOURS) | PEAK DISCHARGE ($m^3/s$) | FLOOD VOLUME (ML) |
|---|---|---|---|
| 0.01 (PMF) | 168 | 47 840 | 10 260 000 |
| | 48 | 42 970 | 6 170 000 |
| 0.01 | 48 | 19 610 | 3 045 000 |
| 0.1 | 48 | 13 410 | 2 044 000 |
| 1 | 48 | 8 720 | 1 234 000 |

Weeks compared design flood estimates for Somerset Dam and Wivenhoe Dam with the earlier studies and concluded that there appeared to be little difference between the higher probability of exceedance estimates, however there was a dramatic increase associated with the PMF estimates. This increase was attributed to the increase in the PMP estimates and the different techniques used in the design flood derivation. A dramatic increase in design rainfall was noted for the whole of the Wivenhoe Dam catchment and the Upper Brisbane River catchment.

# 5.0 DESIGN FLOOD ESTIMATION TECHNIQUES

## 5.1 INTRODUCTION

This section of the report discusses the design flood estimation techniques that have been utilised in the reassessment of the design floods for the storages. A brief summary of the runoff-routing models that have been used in the derivation of the design floods, including model layouts and model parameters is provided in the following sections.

A model that simulates the existing normal operation of Somerset Dam and Wivenhoe Dam was used in addition to the runoff-routing models of the contributing sub-catchments to determine design floods. The storage routing model is also described in this section of the report.

## 5.2 RUNOFF-ROUTING MODELLING

### 5.2.1 Introduction

The runoff-routing model developed by Mein, Laurenson and McMahon, (1974), and implemented as computer program WT42D, (Shallcross, 1987), was used to perform the simulations. 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 consists of a distribution of concentrated conceptual storages of the catchment that allows rainfall and rainfall losses to vary throughout the catchment. Each storage has a non-linear storage-discharge relation of the form:

$$S = 3600.k.k_1.Q^m$$

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 streamflow, rainfall and pluviographic data or they may be estimated from regional formulae appropriate to the area of interest.

### 5.2.2 Model Layouts

The model layouts that have been adopted are based largely upon the runoff-routing models developed during the re-calibration of the modified Brisbane Valley runoff-routing model, (Ayre, Cutler and Ruffini, 1992).

The whole of the Brisbane River catchment has been divided into 19 different sub-catchments, corresponding to the locations of streamgauging stations used in the calibration.

The sub-division of the catchment in such a manner was undertaken because of the study requirement of including the runoff-routing models in the real time flood management model. The use of individual models that are linked together allows greater scope for the refinement of the calibration parameters of specific models.

A list of the sub-catchment models along with various catchment characteristics is provided in Table 5.1.

**Table 5.1**
**Runoff-Routing Model Characteristics**
**For Sub-catchments of the Brisbane River**

| MODEL NAME | SUB-CATCHMENT | AREA ($KM^2$) | DISTANCE TO CENTROID (KM) |
|---|---|---|---|
| COO | Cooyar Ck @ Damsite | 980 | 28.1 |
| LIN | Brisbane R @ Linville | 1 061 | 23.2 |
| EMU | Emu Ck @ Boat Mountain | 913 | 42.1 |
| GRE | Brisbane R @ Gregors Ck | 973 | 25.0 |
| CREDAM | Cressbrook Ck @ Cressbrook Dam | 317 | 15.9 |
| SOM | Stanley R @ Somerset Dam | 1 328 | 42.6 |
| WIV | Brisbane R @ Wivenhoe Dam | 1 429 | 49.1 |
| HEL | Lockyer Ck @ Helidon | 377 | 23.8 |
| TEN | Tenthill Ck @ Tenthill | 465 | 37.7 |
| LYO | Lockyer Ck @ Lyons Bridge | 1 590 | 53.0 |
| SAVDAM | Brisbane R @ Savages Crossing | 728 | 43.7 |
| MTC | Brisbane R @ Mt Crosby Weir | 358 | 31.3 |
| WAL | Bremer R @ Walloon | 626 | 30.3 |
| KAL | Warrill Ck @ Kalbar | 469 | 21.8 |
| AMB | Warrill Ck @ Amberley | 449 | 25.0 |
| PUR | Purga Ck @ Loamside | 223 | 23.6 |
| IPS | Bremer R @ Ipswich | 265 | 23.4 |
| JIN | Brisbane R @ Jindalee | 390 | 21.0 |
| POG | Brisbane R @ Port Office Gauge | 339 | 36.9 |

The connection between the sub-catchment models is shown in the key plan presented in Figure 5.1. The sub-area layout and characteristics of each of the sub-catchment models is presented in Appendix C. In all cases the reach length has been used to determine the relative delay time.

Computer program WT42D is a modified version of WT42PC which was used in the re-calibration phase of the sub-catchment runoff-routing models. Modifications to WT42PC were made to allow the linked models to be run in batch mode. These modifications enable the program to read hydrographs generated from other sub-catchment models as inflows and allows model parameters to be entered on a single command line, replacing the interactive nature of the original program.

### 5.2.3 Model Parameters

Model parameters as determined through calibration, (refer to Ayre, Cutler and Ruffini, 1992), have been adopted during the reassessment of design floods into Somerset Dam and Wivenhoe Dam. It is anticipated that with the installation of the proposed ALERT network of rainfall and river monitoring stations, a more comprehensive set of data will become available that will allow the calibrations of the runoff-routing models to be refined in the future.

The adopted model parameter k for each of the sub-catchment models is summarised in Table 5.2. A value of m = 0.8 was adopted for all models because no obvious trends emerged during calibration of the individual models. This value is consistent with recommendations in Australian Rainfall and Runoff, (1987), however for further information refer to the calibration report.

**Table 5.2**
**Adopted Runoff-Routing Model Parameters**

| MODEL NAME | SUB-CATCHMENT | ADOPTED MODEL PARAMETER k |
|---|---|---|
| COO | Cooyar Ck @ Damsite | 43.6 |
| LIN | Brisbane R @ Linville | 20.6 |
| EMU | Emu Ck @ Boat Mountain | 53.0 |
| GRE | Brisbane R @ Gregors Ck | 37.2 |
| CREDAM | Cressbrook Ck @ Cressbrook Dam | 34.3 |
| SOM | Stanley R @ Somerset Dam | 80.7 |
| WIV | Brisbane R @ Wivenhoe Dam | 108.5 |
| HEL | Lockyer Ck @ Helidon | 15.0 |
| TEN | Tenthill Ck @ Tenthill | 19.0 |
| LYO | Lockyer Ck @ Lyons Bridge | 75.0 |
| SAVDAM | Brisbane R @ Savages Crossing | 40.0 |
| MTC | Brisbane R @ Mt Crosby Weir | 47.0 |
| WAL | Bremer R @ Walloon | 44.0 |
| KAL | Warrill Ck @ Kalbar | 34.0 |
| AMB | Warrill Ck @ Amberley | 35.0 |
| PUR | Purga Ck @ Loamside | 49.0 |
| IPS | Bremer R @ Ipswich | 15.7 |
| JIN | Brisbane R @ Jindalee | 20.8 |
| POG | Brisbane R @ Port Office Gauge | 19.3 |

## 5.3 STORAGE ROUTING MODEL

A storage routing model known as '2DAMA' was used as the basis of the reservoir routing for both Somerset Dam and Wivenhoe Dam. This model was specially developed by Hegerty and Weeks, (1985), to enable the routing of floods through Somerset Dam and Wivenhoe Dam using their existing standard gate operating procedures.

The original 2DAMA model has been modified to be capable of running in batch mode like the runoff-routing models. A number of changes to the model's interpretation of the operational procedures were also required so as to ensure the storage model realistically simulated the actual operational procedures. The batch mode version has been renamed, and it is now called 'AUTOPS'.

A brief summary of the details of the dams and the models representation of the storages is provided in this section.

## Somerset Dam

**Type of Dam**

Mass concrete gravity, incorporating gated spillway and stilling basin dissipater.

**Maximum Height**

53 metres (approximately) to crest above lowest cut-off.

**Crest Length**

282 metres (approximately).

**Leakage Control**

Foundation grout curtain and associated drain holes through cut-off; brick drains linked to lower gallery; copper waterstops in vertical joints between monoliths.

**Purpose**

Water supply, flood mitigation, electric power generation.

**Gross Storage at FSL**

369 750 ML

**Flood Storage to Top of Spillway Gates**

524 000 ML

**Spillway Gates**

8 @ 7.93 metre wide by 7.01 metre high radial gates.

**Low Level Sluice Gates**

8 @ 2.44 metre wide by 3.66 metre high upstream sealing caterpillar gates.

**Discharge Regulator Valves**

4 @ 2.3 metre diameter fixed dispersion cone valves with upstream guard gates.

**Power Station**

3.2 MW generation unit, housed integrally within dam wall.

**Storage Operation Control Levels**

| Feature | Elevation (m AHD) |
| --- | --- |
| Invert of Regulator Valves | 68.95 |
| Invert of Sluice Gates | 71.32 |
| Full Supply Level | 99.00 |
| Spillway Crest Level | 100.45 |
| Flood Control Level | 102.25 |
| Top of Closed Spillway Gate | 107.46 |
| Non-overflow Spillway Crest Level | 107.46 |
| Bridge Deck Level | 112.47 |

**Equation for Converting Water Levels into Storage Volumes**

$$SLEVEL = 58.05 + (SVOL/8.27599)^{0.21075} \text{ (m AHD)}$$

**Equation for Flow Over Non-Overflow Spillway Level**

$$QSWALL = 1.7*135.33*(SLEVEL-107.455)^{1.5} \text{ (m}^3\text{/s)}$$

**Equation for Flow Through Each Regulator Valve**

$$QSREG = 12.714*(SLEVEL-70.104)^{0.5} \text{ (m}^3\text{/s)}$$

**Equation for Flow Through Each Sluice Gate**

$$QSSLU = 39.4546*(SLEVEL-73.152)^{0.5} \text{ (m}^3\text{/s)}$$

**Equation for Flow Through Each Spillway Gate**

$$QSSPL = 0.14265*(SLEVEL-100.444)^3 + 14.265*(SLEVEL-100.444)^{1.5} \text{ (m}^3\text{/s)}$$

**Where:**

SLEVEL = Water level in Somerset Dam (m AHD).
SVOL = Storage volume in Somerset Dam (ML).
QSWALL = Discharge over non-overflow spillway ($m^3/s$).
QSREG = Discharge through one regulator valve ($m^3/s$).
QSSLU = Discharge through one sluice gate ($m^3/s$).
QSSPL = Discharge through one spillway gate ($m^3/s$).

A comparison between the actual storage capacity curve for Somerset Dam and the curve that is calculated using the equation is shown in Figure 5.2. The actual storage capacity curve is based on values provided by the BCC which were determined in June 1952. There is a maximum difference of around 150 mm in elevation for the range of storage capacities considered.

Figure 5.3 presents the discharge versus elevation curves for Somerset Dam, based upon the equations contained in '2DAMA'. The curves presented are for the total flow through all 4 regulator valves, 8 sluice gates and 8 spillway gates.

## Wivenhoe Dam

**Type of Dam**

Zoned earthfill and rockfill embankment with central gated spillway and flip bucket and plunge pool dissipater.

**Maximum Height**

59 metres (approximately).

**Crest Length**

2 000 metres (approximately).

**Leakage Control**

Foundation grout curtain with drainage holes.

**Purpose**

Water supply, flood mitigation, electric power generation.

**Gross Storage at FSL**

1 150 825 ML

**Flood Storage to Top of Spillway Gates**

1 450 000 ML

**Spillway Gates**

5 @ 12.00 metre wide by 16.65 metre high radial gates.

**Discharge Regulator Valves**

2 @ 1.5 metre diameter fixed dispersion cone valves with upstream guard gates.

**Power Station**

2 @ 312.5 MW turbines, 2 @ 245 MW pumps, pumped storage, tailbay within reservoir.

**Storage Operation Control Levels**

| Feature | Elevation (m AHD) |
|---|---|
| Invert of Intake Structure | 33.00 |
| Spillway Crest Level | 57.00 |
| Full Supply Level | 67.00 |
| Gate Opening Control Level | 67.25 |
| Gate Closing Control Level | 68.00 |
| Top of Closed Spillway Gate | 73.00 |
| Control Flood Level | 74.00 |
| Design Top Water Level | 77.00 |
| Embankment Crest Level | 79.15 |
| Top of Wave Wall | 79.90 |

**Equation for Converting Water Levels Into Storage Volumes**

$$\text{WLEVEL} = 23.242 + 0.1865*\text{WVOL}^{0.2616} \text{ (m AHD)}$$

**Equation for Flow Over Non-Overflow Embankment Level**

$$\text{QWWALL} = 5300*(\text{WLEVEL}-79.9)^{1.5} \text{ (m}^3\text{/s)}$$

**Equation for Flow Through Spillway when Gates Lifted Clear**

$$\text{DMAX} = (106.1 + (\text{WLEVEL} - 57.00))*(\text{WLEVEL}-57.00)^{1.5} \text{ (m}^3\text{/s)}$$

**Where:**

WLEVEL = Water level in Wivenhoe Dam (m AHD).
WVOL = Storage volume in Wivenhoe Dam (m$^3$/s).

QWWALL = Discharge over non-overflow spillway ($m^3/s$).
DMAX = Discharge through spillway when gates lifted clear ($m^3/s$)

A comparison between the actual and calculated storage capacity curves for Wivenhoe Dam are presented in Figure 5.4. The actual storage capacity curve was obtained from WRC plan number A3-44067 which was revised in May 1984. It is evident from the comparison that the equation does not reproduce the lower portion of the curve very well, however, this particular study is primarily concerned with the upper portion of the curves which are reasonably well matched.

The discharge versus elevation curves based upon the equations for Wivenhoe Dam are presented in Figure 5.5.

The storage routing model determines the sequencing of regulator valve, sluice gate and spillway gate openings and closures from Somerset Dam. However, the model does not determine the spillway gate and regulator valve sequencing for Wivenhoe Dam. The discharge hydrograph for Wivenhoe Dam is determined by considering the various operation procedure objectives, such as ensuring that the combined releases from Wivenhoe Dam and Lockyer Creek is less than 3 500 $m^3/s$. The model assumes that the discharge from Wivenhoe Dam can be achieved subject to maximum gate opening and closing limitations. Therefore, the required outflow from Wivenhoe Dam is determined without reference to actual gate settings, except for the situation when the gates are lifted clear.

The modifications to the storage routing model have entailed the fixing of a number of parameters internally within the model. The following list of parameters have been set within the modified model:

**Initial Storage Levels**

Somerset Dam EL 99.00 or 369 755 ML
Wivenhoe Dam EL 67.00 or 1 150 825 ML

**Time Interval**

1 Hour

**Maximum Number of Sluice Gates to be Opened or Closed in 1 Hour Period**

@ Somerset Dam = 2

**Maximum Number of Regulator Valves to be Opened or Closed in 1 Hour Period**

@ Somerset Dam = 4

**Maximum Rate of Increase or Decrease of Discharge in 1 Hour Period**

@ Wivenhoe Dam = 360 $m^3/s$

The operational procedures incorporated in this model are the recommended procedures described in the manual of flood operations for the dams, (1985). These procedures are summarised below:

**SOMERSET DAM**

The spillway gates are to be raised to enable uncontrolled discharge once the flood storage between full supply level, (FSL = EL 99.0), and spillway crest level, (EL 100.45), has filled. The low level regulator valves and sluice gates are to be kept closed until either:

(a) the inflow to Wivenhoe Dam begins to decrease; or

(b) the level in Somerset Dam exceeds EL 102.25.

In the case of (a) above, the opening of the regulator valves and sluice gates is not to increase the inflow to Wivenhoe Dam above the peak inflow.

**WIVENHOE DAM**

**Procedure 1**

Releases are to be made from Wivenhoe Dam onto the flow of Lockyer Creek such that Fernvale Bridge is not submerged prematurely. If the Lockyer Creek flow is sufficient to submerge Fernvale Bridge, the releases from Wivenhoe Dam are to be regulated to ensure that Mt Crosby Weir Bridge is not submerged.

**Procedure 2**

Releases are to be made from Wivenhoe Dam onto the rising limb of the Lockyer Creek flood, care being taken not to submerge Fernvale Bridge prematurely.
If the flood flow of Lockyer Creek is sufficient to submerge Mt Crosby Weir Bridge, the releases are to be increased such that the combined Lockyer Creek flood flow and Wivenhoe Dam releases does not submerge Mt Crosby Weir Bridge prematurely, and does not exceed the lessor of:

(a) 3 500 $m^3/s$; or

(b) the peak flood flow of Lockyer Creek or the predicted peak flood flow of the Bremer River, whichever is the greater.

**Procedure 3**

Releases are to be made from Wivenhoe Dam onto the rising limb of the Lockyer Creek flood, care being taken not to submerge Fernvale Bridge or Mt Crosby Weir Bridge prematurely. The combined Lockyer Creek flood flow and Wivenhoe Dam releases is not to exceed 3 500 $m^3/s$. The releases are to be regulated such that the total regulated flow at the Brisbane River @

Moggill gauge, (downstream of the Bremer River junction), does not exceed 4 000 $m^3/s$. This value is the upper limit of non-damaging flows for the urban reaches of the Brisbane River.

**Procedure 4**

Releases are to be made from Wivenhoe Dam onto the rising limb of the Lockyer Creek flood, care being taken not to submerge Fernvale Bridge or Mt Crosby Weir Bridge prematurely. The combined flood flow of Lockyer Creek plus releases from Wivenhoe Dam is not to exceed 3 500 $m^3/s$, until the Lockyer Creek flood peak passes the junction with the Brisbane River. The releases are then to be increased until the level behind Wivenhoe Dam begins to fall. The combined flow at Lowood is to be reduced to 3 500 $m^3/s$ as quickly as practicable, and is to remain at this level until the flood storage of Wivenhoe Dam is emptied.

If the lake level behind Wivenhoe Dam exceeds EL 74.0, the releases are to be increased irrespective of the location of the Lockyer Creek flood peak. The time interval between incremental gate openings is to be reduced from ten minutes to five minutes.

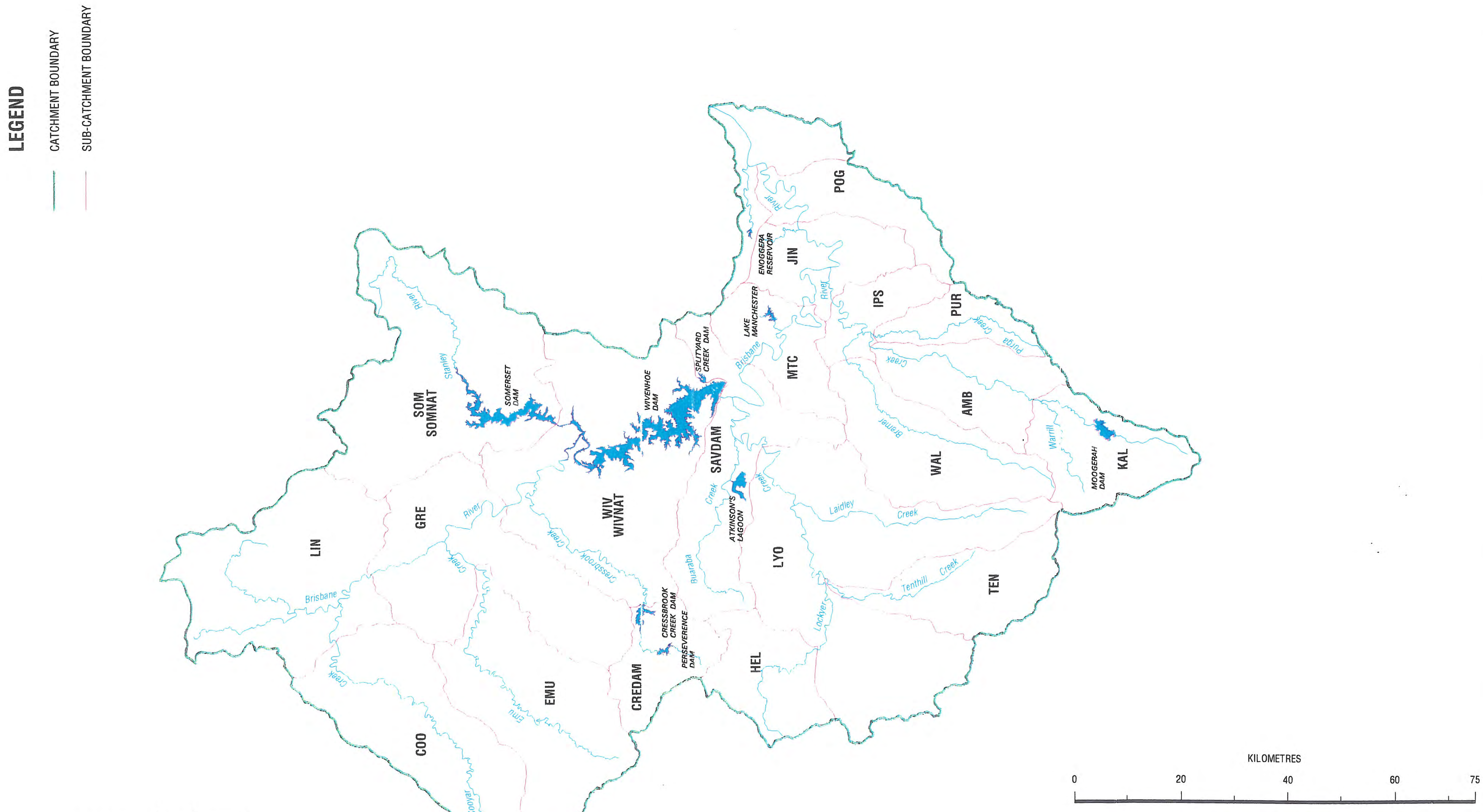
The appropriate operational policy for Wivenhoe Dam is related to estimated peak discharges at six key streamgauge locations.

These locations are:

Brisbane River @ Gregors Creek
Stanley River @ Woodford
Lockyer Creek @ Lyons Bridge
Brisbane River @ Lowood
Bremer River @ Ipswich (David Trumpy Bridge)
Brisbane River @ Moggill

The decision process to determine which of the four operation procedures is most appropriate for Wivenhoe Dam has been incorporated into a policy selection module of the modified storage operation model. This decision process is based upon Table C1 in Appendix C of the manual of operational procedures, (1985).

FIGURE 5.1

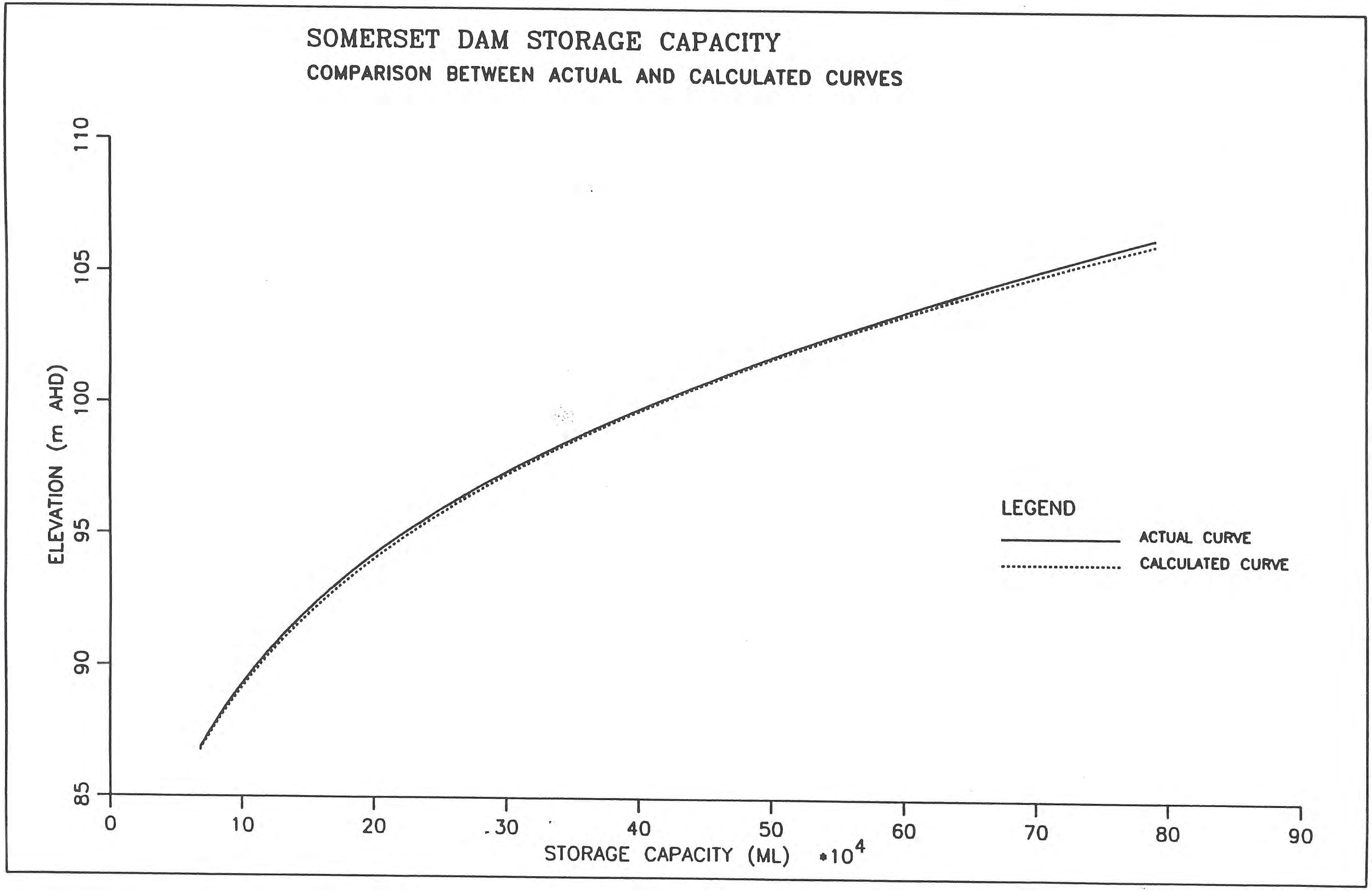



DEPARTMENT OF PRIMARY INDUSTRIES
QUEENSLAND
WATER RESOURCES

**Brisbane River Flood Study**
**Brisbane River**
## Design Model Key Plan

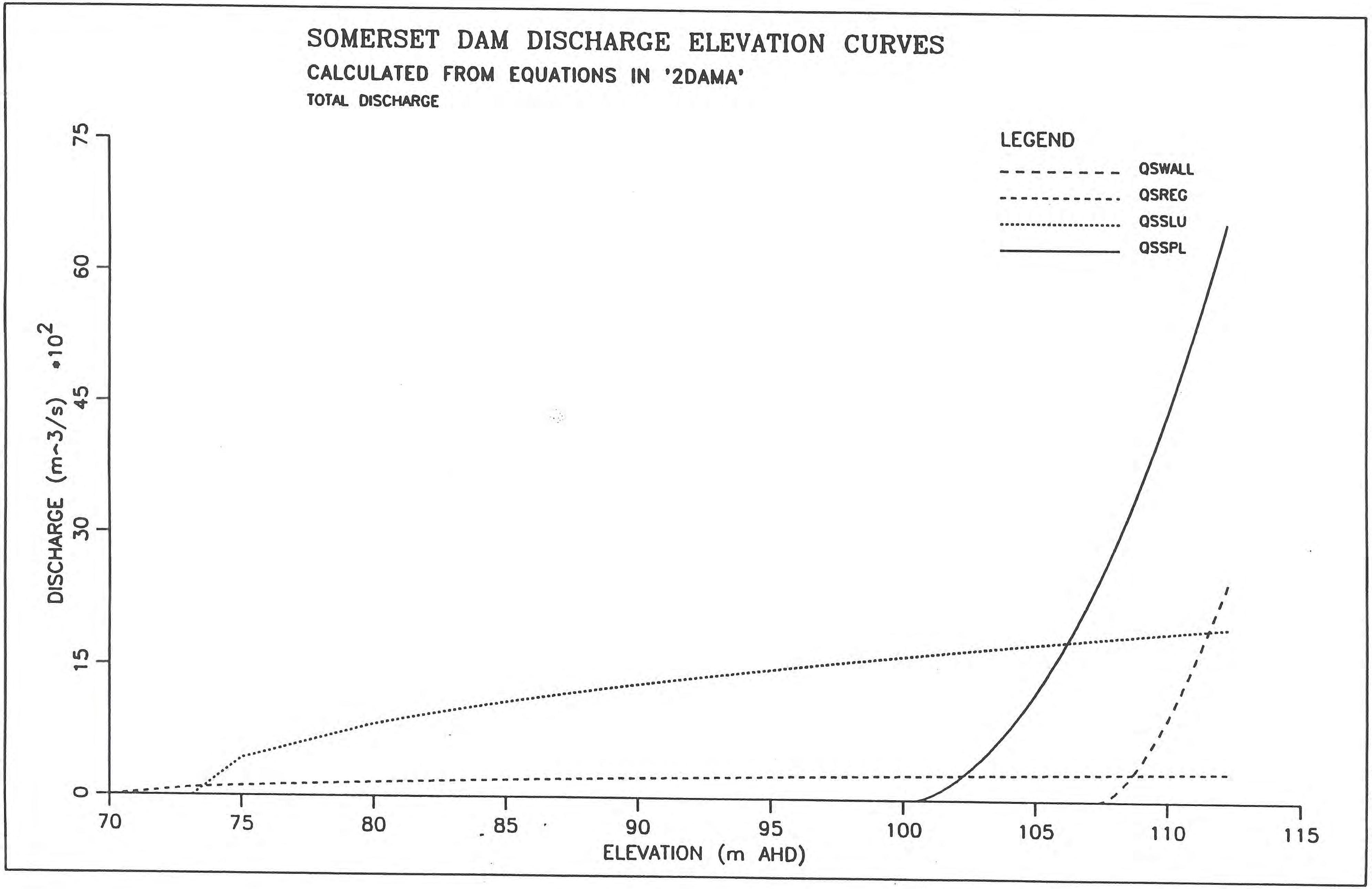
18 Mar 93

A3-101427



SOMERSET DAM STORAGE CAPACITY
COMPARISON BETWEEN ACTUAL AND CALCULATED CURVES
ELEVATION (m AHD)
85
90
95
100
105
110
0
10
20
30
40
50
60
70
80
90
STORAGE CAPACITY (ML) *10^4
LEGEND
ACTUAL CURVE
CALCULATED CURVE


FIGURE 5.2



SOMERSET DAM DISCHARGE ELEVATION CURVES
CALCULATED FROM EQUATIONS IN '2DAMA'
TOTAL DISCHARGE
LEGEND
QSWALL
QSREG
QSSLU
QSSPL
DISCHARGE (m^3/s) *10^2
0
15
30
45
60
75
70
75
80
85
90
95
100
105
110
115
ELEVATION (m AHD)


FIGURE 5.3

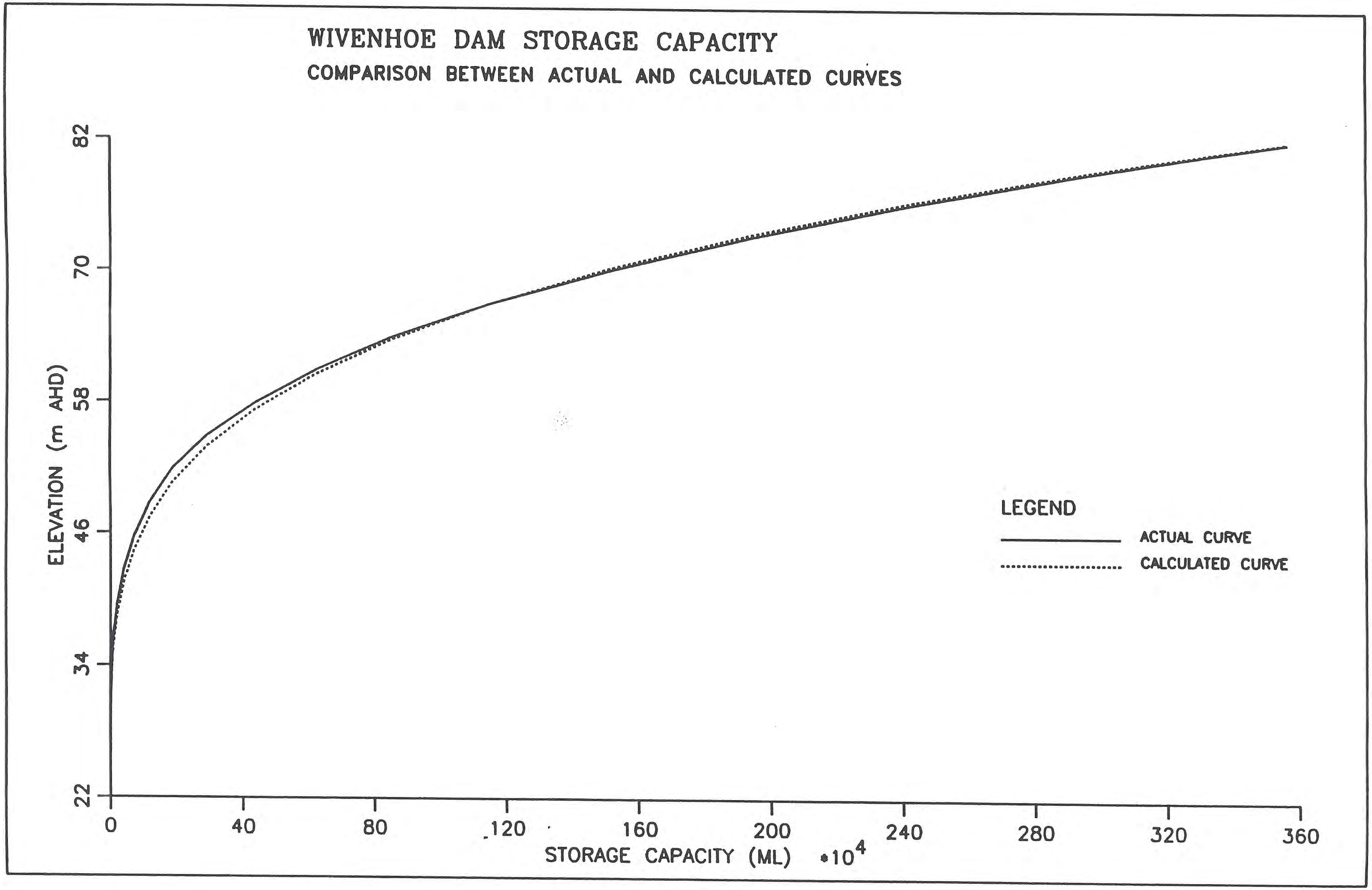

WIVENHOE DAM STORAGE CAPACITY
COMPARISON BETWEEN ACTUAL AND CALCULATED CURVES
ELEVATION (m AHD)
82
70
58
46
34
22
0
40
80
120
160
200
240
280
320
360
STORAGE CAPACITY (ML) *10^4
LEGEND
ACTUAL CURVE
CALCULATED CURVE

FIGURE 5.4

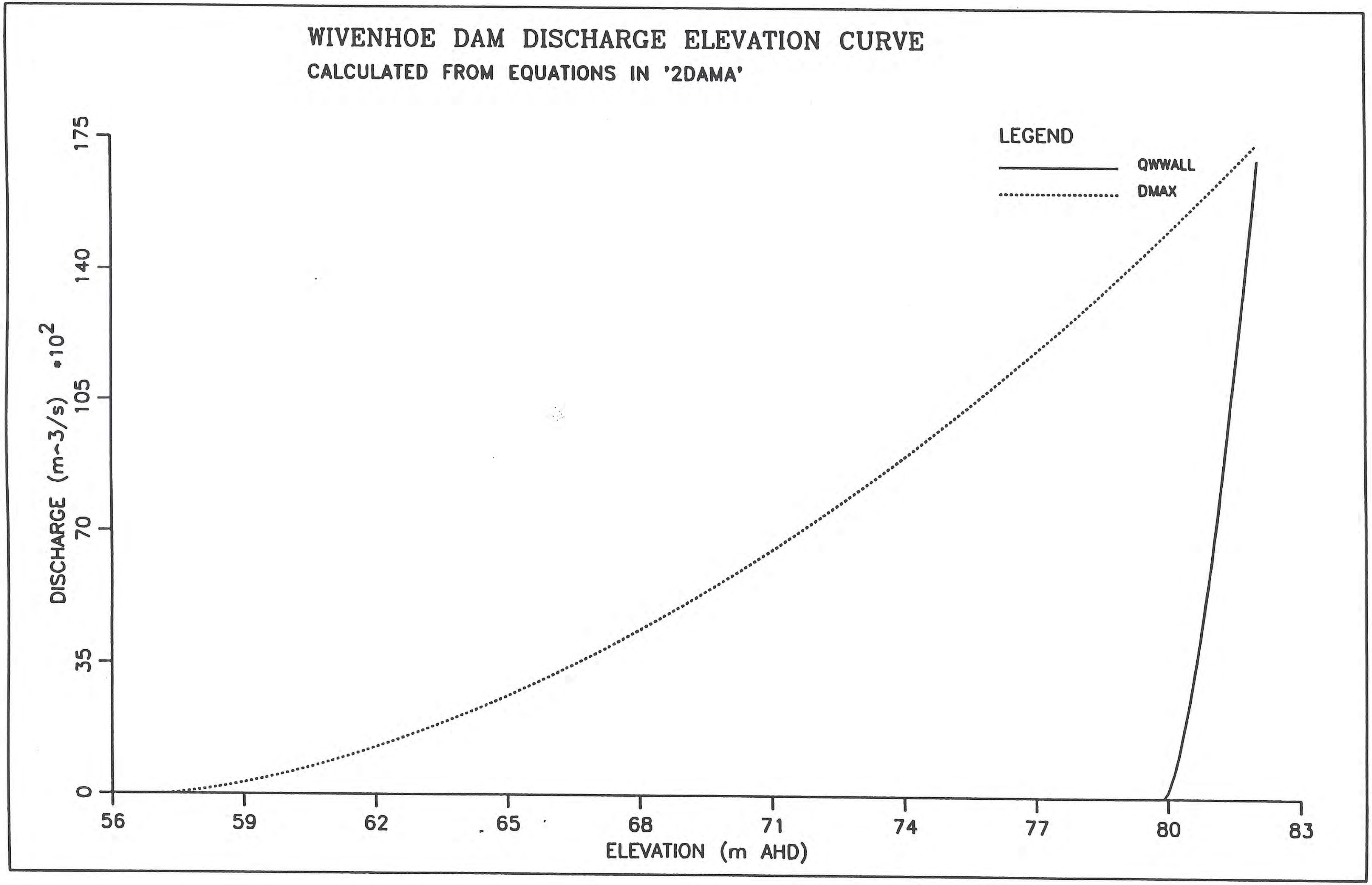

WIVENHOE DAM DISCHARGE ELEVATION CURVE
CALCULATED FROM EQUATIONS IN '2DAMA'
LEGEND
QWWALL
DMAX
DISCHARGE (m^3/s) *10^2
175
140
105
70
35
0
56
59
62
65
68
71
74
77
80
83
ELEVATION (m AHD)

FIGURE 5.5

# 6.0 DESIGN RAINFALL DATA

## 6.1 INTRODUCTION

Design rainfall estimates utilised in this study have been obtained from the Bureau of Meteorology, (1991), and from the procedures outlined in Chapter 2 of Australian Rainfall and Runoff, (1987). The Bureau provided PMP estimates for the catchments of Somerset Dam and Wivenhoe Dam, whilst point intensity-frequency-duration, (IFD), estimates have been converted into catchment rainfall estimates using isohyetal maps of the catchments. The following sections provide more details about the design rainfall derivations.

## 6.2 PROBABLE MAXIMUM PRECIPITATION ESTIMATES

Estimates of PMP for the catchment of the Brisbane River and seven sub-catchments relating to Somerset Dam and Wivenhoe Dam have been obtained from the Bureau of Meteorology, (1991). The report prepared by the Bureau of Meteorology is presented in Appendix A for further information. These catchments include:

| | CATCHMENT DESCRIPTION | CATCHMENT NAME | AREA ($km^2$) |
|---|---|---|---|
| 1 | Stanley River @ Somerset Dam | SOMERSET | 1 335 |
| 2 | Brisbane River @ Stanley River Junction | UPPER BRISBANE | 4 715 |
| 3 | Residual area between Stanley River Junction and Wivenhoe Dam | MIDDLE BRISBANE | 970 |
| 4 | Brisbane River @ Wivenhoe Dam | WIVENHOE | 7 020 |
| 5 | Brisbane River @ Mouth | BRISBANE | 13 560 |
| 6 | Lockyer Creek @ Brisbane River Junction | LOCKYER | 2 990 |
| 7 | Bremer River @ Brisbane River Junction | BREMER | 2 020 |
| 8 | Residual area between Wivenhoe Dam and mouth of Brisbane River (excluding Lockyer and Bremer) | LOWER BRISBANE | 1 530 |

Figures 6.1 and 6.2 illustrate the catchments which were considered as design storm centres that were provided by the Bureau of Meteorology.

The Bureau of Meteorology used a generalised method that is applicable for areas affected by tropical storms for durations from 6 to 96 hours for all of the catchments. The Bulletin 51 method which is appropriate for catchment areas less than 1 000 $km^2$ in area and durations of up to 6 hours was also applied to catchment number 3.

Bulletin 51, (1985), involves the use of enveloping depth-duration-area curves that have been developed for two categories of topography, 'smooth' and 'rough'. 'Rough' areas are defined as areas in which elevation changes of 50 metres or more within 400 metres are common. Catchment number 3 is overshadowed by the rugged terrain of the D'Aguilar Range and although the catchment is only partially rough by definition, it has been classified as 'rough' for the purposes of this study. If the catchment had been classified as 'smooth' then the estimated rainfalls would be lower.

The generalised method known as the Generalised Tropical Storm Method or GTSM has been developed for those parts of Australia affected by storms of tropical origin. The Brisbane River system lies within a region known as the East Coast Tropical Zone, (ECTZ), which is an area that is influenced by a quasistationary easterly trough adjacent to the Queensland coast. This trough appears to enhance heavy rainfall events. The most likely cause of PMP rainfalls in this region is considered to be either the proximity of a tropical cyclone or the slow movement of a low pressure system of tropical origin, which may sometimes interact with a monsoonal trough.

For durations of between five days and seven days, the Bureau of Meteorology used the Gordon Method of PMP estimation. This method was devised by Mr Barry Gordon of the Queensland Regional Office for the previous investigation of Somerset Dam and Wivenhoe Dam in 1983. The method is based upon a method used in the U.S. Hydrometeorological Report No 46, 'Probable Maximum Precipitation, Mekong River Basin', (1970). The method proposes that extreme rainfall affecting an area results from two distinct storms with a short period of little or no rainfall between the major storms.

Temporal patterns associated with the PMP rainfalls were also provided by the Bureau of Meteorology. These patterns include two alternative patterns for the five day storm, three alternatives for the six day storm and three alternatives for the seven day event. All of the patterns are considered by the Bureau of Meteorology to be equally likely, and as a consequence all patterns were considered.

Estimates of the PMP for five of the catchments mentioned earlier are provided in Table 6.1. Probabilities of exceedance have been assigned to the PMP estimates in accordance with the procedures outlined in Chapter 13 of Australian Rainfall and Runoff, (1987).

**Table 6.1**
**Catchment Estimates of PMP**
**(mm Depth)**

| DURATION (HOURS) | RAINFALL DEPTH (mm) | | | | |
|---|---|---|---|---|---|
| | SUB-CATCHMENT | | | | |
| | SOMERSET DAM | UPPER BRISBANE | RESIDUAL AREA | WIVENHOE DAM | BRISBANE RIVER |
| 3 | - | - | 360 | - | - |
| 6 | 390 | 270 | 460 | 240 | 200 |
| 12 | 670 | 490 | 660 | 450 | 370 |
| 24 | 900 | 720 | 940 | 670 | 530 |
| 48 | 1 420 | 980 | 1 390 | 870 | 680 |
| 72 | 1 770 | 1 200 | 1 740 | 1 080 | 830 |
| 96 | 2 090 | 1 390 | 2 060 | 1 250 | 1 010 |
| 120 | 2 170 | 1 440 | 2 130 | 1 300 | 1 050 |
| 144 | 2 220 | 1 480 | 2 190 | 1 330 | 1 070 |
| 168 | 2 410 | 1 670 | 2 360 | 1 480 | 1 160 |
| ARI (Years) | $10^6$ | $10^6$ | $10^6$ | $10^5$ | $10^5$ |
| Where ARI = Average Recurrence Interval | | | | | |

## 6.3 DESIGN INTENSITY-FREQUENCY-DURATION ESTIMATES

Design intensity-frequency-duration data have been derived for a number of locations within and around the Brisbane River catchment. These estimates are based upon the procedures outlined in Chapter 2 of Australian Rainfall and Runoff, (1987). Computer program IFD, (Cantorford, 1988), was used to derive the design rainfall estimates. The locations that were selected are listed in Table 6.2 and shown in Figure 6.3. These sites roughly correspond to the locations of daily rainfall stations that were used in the calibration of the runoff-routing models.

**Table 6.2**
**Design IFD Data Locations**

| STATION NUMBER | STATION NAME |
|---|---|
| 041000 | Acland |
| 041001 | Allora |
| 040004 | Amberley AMO |
| 041005 | Bell |
| 040019 | Benarkin Forestry |
| 040020 | Blackbutt |
| 040024 | Boonah |
| 040214 | Brisbane RO |
| 541032 | Bryn Euryn |
| 040289 | Coalbank |
| 040056 | Coominya |
| 040060 | Cooyar |
| 040382 | Crows Nest |
| 040063 | Dayboro |
| 040531 | Deagon (BCC) |
| 040225 | Enoggera Reservoir |
| 040075 | Esk |
| 040122 | Gallangowan |
| 040083 | Gatton |
| 040091 | Grandchester |
| 041042 | Haden |
| 040094 | Harrisville |
| 040096 | Helidon |
| 040101 | Ipswich Composite |
| 040102 | Jimna |
| 040386 | Kenilworth |
| 040110 | Kilcoy |
| 040111 | Kilkivan |
| 040112 | Kingaroy |
| 040318 | Kirkleagh |
| 040114 | Laidley |
| 040115 | Lake Manchester |
| 040082 | Lawes |
| 040306 | Loganlea |
| 040120 | Lowood |
| 040121 | Maleny |
| 040133 | Monsildale |
| 040135 | Moogerah Dam |
| 040136 | Mooloolah |
| 040137 | Moore |
| 040140 | Mt Brisbane |
| 040142 | Mt Crosby |
| 040308 | Mt Glorious |
| 040247 | Mt Kilcoy |
| 040145 | Mt Mee |
| 040526 | Mt Nebo |
| 040153 | Murphys Creek |
| 040158 | Nanango |
| 040159 | Narangba |
| 040311 | Nukinenda |
| 040169 | Peachester |
| 040171 | Petrie APM |
| 040270 | Ravensbourne |
| 040183 | Rosevale |
| 040184 | Rosewood |
| 040241 | Samford CSIRO |
| 040421 | Spring Bluff |
| 040198 | Tarome |
| 040205 | Toogoolawah |
| 041103 | Toowoomba Composite |
| 040227 | Wacol |
| 040424 | West Haldon |
| 040252 | Woodford |
| 040256 | Wynnum |
| 040258 | Yarraman |

The point estimates of design rainfall were converted into mean catchment rainfalls by drawing isohyetal maps from the point estimates. Sub-catchment rainfalls were estimated from the isohyetal maps by determining the area under each of the isohyetal contours. This method was employed so as to ensure the steep rainfall gradient across some parts of the catchment was properly accounted for. The whole procedure was performed using a Geographical Information

System, (GIS), consequently it is almost completely automated.

The adopted design storm rainfall depths for various Average Recurrence Intervals, (ARI), are summarised in Tables 6.3 to 6.6. Note all values of rainfall depth are millimetres.

**Table 6.3**
**Catchment Estimates of 100 Year ARI Rainfalls**
**(mm Depth)**

| DURATION (HOURS) | RAINFALL DEPTH (mm) | | | | |
|---|---|---|---|---|---|
| | SUB-CATCHMENT | | | | |
| | SOMERSET DAM | UPPER BRISBANE | RESIDUAL AREA | WIVENHOE DAM | BRISBANE RIVER |
| 12 | 267 | 176 | 225 | 200 | 193 |
| 24 | 360 | 231 | 292 | 264 | 255 |
| 36 | 425 | 269 | 338 | 308 | 296 |
| 48 | 475 | 297 | 371 | 341 | 328 |
| 60 | 514 | 319 | 396 | 367 | 352 |
| 72 | 545 | 336 | 416 | 387 | 370 |

**Table 6.4**
**Catchment Estimates of 50 Year ARI Rainfalls**
**(mm Depth)**

| DURATION (HOURS) | RAINFALL DEPTH (mm) | | | | |
|---|---|---|---|---|---|
| | SUB-CATCHMENT | | | | |
| | SOMERSET DAM | UPPER BRISBANE | RESIDUAL AREA | WIVENHOE DAM | BRISBANE RIVER |
| 12 | 234 | 155 | 196 | 176 | 172 |
| 24 | 314 | 203 | 255 | 231 | 225 |
| 36 | 370 | 236 | 295 | 269 | 260 |
| 48 | 413 | 260 | 324 | 298 | 287 |
| 60 | 446 | 278 | 346 | 319 | 307 |
| 72 | 473 | 292 | 362 | 336 | 323 |

**Table 6.5**
**Catchment Estimates of 20 Year ARI Rainfalls**
**(mm Depth)**

| DURATION (HOURS) | RAINFALL DEPTH (mm) | | | | |
|---|---|---|---|---|---|
| | SUB-CATCHMENT | | | | |
| | SOMERSET DAM | UPPER BRISBANE | RESIDUAL AREA | WIVENHOE DAM | BRISBANE RIVER |
| 12 | 192 | 130 | 161 | 146 | 144 |
| 24 | 257 | 169 | 209 | 191 | 186 |
| 36 | 302 | 195 | 241 | 221 | 215 |
| 48 | 336 | 214 | 265 | 244 | 236 |
| 60 | 363 | 228 | 283 | 261 | 252 |
| 72 | 384 | 239 | 297 | 274 | 265 |

**Table 6.6**
**Catchment Estimates of 10 Year ARI Rainfalls**
**(mm Depth)**

| DURATION (HOURS) | RAINFALL DEPTH (mm) | | | | |
|---|---|---|---|---|---|
| | SUB-CATCHMENT | | | | |
| | SOMERSET DAM | UPPER BRISBANE | RESIDUAL AREA | WIVENHOE DAM | BRISBANE RIVER |
| 12 | 163 | 112 | 136 | 125 | 123 |
| 24 | 216 | 144 | 176 | 162 | 159 |
| 36 | 253 | 165 | 203 | 187 | 183 |
| 48 | 281 | 180 | 223 | 205 | 200 |
| 60 | 303 | 192 | 238 | 219 | 213 |
| 72 | 320 | 201 | 250 | 230 | 221 |

Design storm temporal patterns of rainfall bursts have been determined in accordance with Chapter 3 of Australian Rainfall and Runoff, (1987). The Brisbane River catchment is located in Zone 3 of the North-East Coast Division and as a consequence the temporal patterns listed in Table 3.2 of Volume 2 of Australian Rainfall and Runoff, (1987), have been adopted. Computer program WTEMPAT, (Ruffini, 1990) was used to incorporate the design rainfall temporal patterns into the runoff-routing model data files.

## 6.4 AREAL REDUCTION FACTORS

The rainfall IFD values derived above are applicable strictly only to a point, but they may be taken to represent IFD values over small areas. For larger areas it is not realistic to assume that the same intensity can be maintained over the entire area, thus some reduction is usually made.

Unfortunately, little work has been done on this topic in Australia, so Australian Rainfall and Runoff, (1987), recommends the use of depth-area ratios that have been derived overseas, in particular the United States. However, the research performed overseas is also limited, as the curves derived from studies conducted on the East and West Coasts of the United States only extend to areas of 1 000 $km^2$ and for durations of up 24 hours. Refer to Figure 2.6 in Australian Rainfall and Runoff, (1987). This range does not cover the catchment sizes of interest in this study or the durations of storms that are most critical to the storage operation. Hence, areal reduction factors applicable to catchments larger than 1 000 $km^2$ in area and storm durations longer than 24 hours have to be derived by alternative means or conservatively, not employed.

An equation describing the family of areal reduction factor curves presented in Australian Rainfall and Runoff, (1987), is presented in Raudkivi, (1979). This equation can be used to extend the curves to the size of catchment and duration of interest required for the study. The equation predicts that the areal reduction factors for durations of 24, 48 and 72 hours approach 0.912, 0.945, and 0.959 respectively, as the catchment area exceeds 1 000 $km^2$. However, there remains a doubt about the appropriateness of the US derived data on which this equation is based.

An attempt was made to estimate areal reduction factors for the Brisbane River catchment by comparing design rainfalls derived from IFD data with design rainfalls derived from a depth-area-duration analysis. This procedure is similar to that reported by Nittim, (1989). The catchment IFD data derived in a manner described in Section 6.3 of this report were utilised for this comparison. The depth-area-duration data were derived from previous analyses concerning station rainfalls that were conducted by the WRC and the Bureau of Meteorology and which were reported by Hausler and Porter, (1977), and Weeks, (1983 and 1984).

In the depth-area-duration analysis, a log-normal distribution was applied to annual series of one, two and three day catchment rainfall maxima which were compiled from daily rainfall station records of a number of stations located within and around the catchments of interest. The catchment daily rainfall sequences, (of over 75 years in length), were formed by combining long term daily rainfall records using regression weighted coefficients.

The regression weights were determined by comparing estimates of catchment rainfall derived in two ways. Detailed isohyetal maps of 18 major historic events were first constructed using all available daily rainfall information. Estimates of the various catchment rainfall for these events were then made from these maps. Combinations of various daily rainfall stations were then considered, together with varying weighting factors, and estimates of catchment rainfall made for all 18 events. A least square error calculation was then performed comparing the two different estimates and the weighted combination of daily rainfall stations that produced the smallest error over the whole 18 events was selected to form the representative catchment daily rainfall sequence.

The resulting comparison of IFD data and depth-area-duration showed little consistency. There was an underlying trend for the ratios derived from this comparison to be less than the areal reduction factors presented in Australian Rainfall and Runoff, (1987), but the significance of this finding is debatable given the lack of consistency in the results used in the comparison.

Because the results of the comparison did not reveal any specific conclusion no areal reduction factor has been applied to the design rainfalls. This obviously introduces a degree of conservatism into the design storm rainfall derivation.

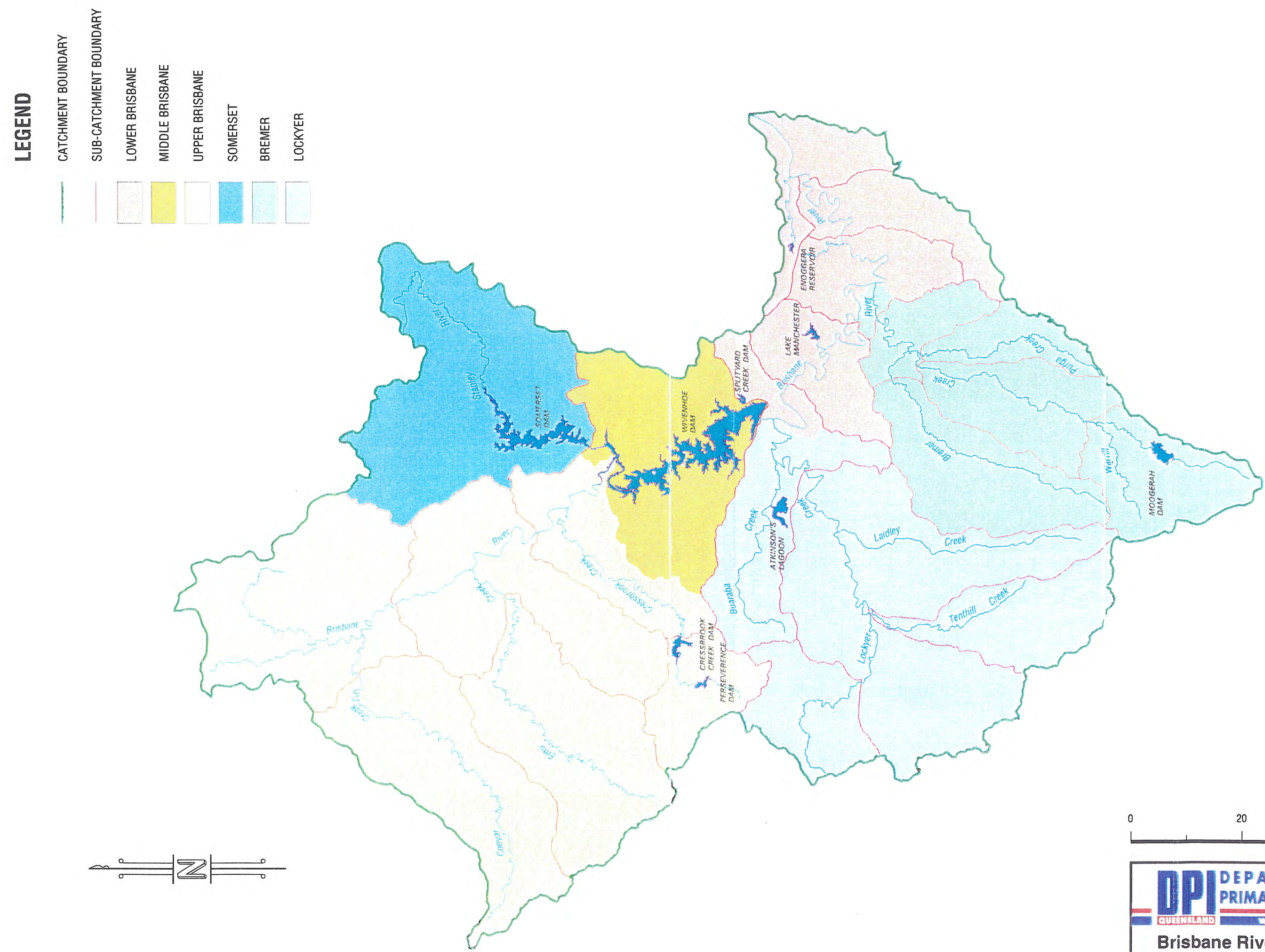
An alternative to applying areal reduction factors to design rainfall estimates was adopted in the reassessment of design floods for North Pine Dam, (refer Ayre, Cutler, and Ruffini, 1991). Runoff-routing model estimates of peak discharge were compared to peak discharge estimates derived from flood frequency techniques in order to establish appropriate initial loss rates so as to ensure both methods produced similar peak discharge values.

The appropriateness of this procedure depends upon the quality and length of streamflow record on which the flood frequency analysis is based and to which the runoff-routing model is calibrated. The resulting initial loss rates may vary according to the ARI of the design rainfall and in some instances the peak discharge estimates derived from runoff-routing techniques may be smaller than the flood frequency estimates implying that no initial loss rate is applicable. Uncertainties in the results of the flood frequency analyses and the runoff-routing modelling methods may mean that the initial loss rates so derived are a product of the uncertainties in the data, rather than any physical phenomena.

Some of the problems inherent in this procedure is that the durations of the storms associated with the peak discharges that form the annual series, are not the always the same and that the antecedent condition of the catchment for these events are not always consistent. Therefore, design floods of 1 in 10 years ARI do not necessarily result from 1 in 10 year ARI design rainfalls. This is usually one of the basic assumptions made in conjunction with design flood estimates based upon design rainfall.

However, the procedure was applied to four catchments in the Brisbane River Valley, (refer to Appendix B for the flood frequency estimates), which resulted in small initial loss rates being derived for the 10 year ARI event, (ie around 30 mm). In all cases the initial loss rates approached zero as the ARI of the event increased. Because the initial loss rates were small and because they tended to approach zero for lower probability events, a conservative assumption of zero initial loss rate has been adopted for all subsequent design flood derivations.

The approach of adopting no initial loss and not applying areal reduction factors is considered appropriate because the primary objective of the study is to reassess design floods that affect the operation of Somerset Dam and Wivenhoe Dam. These floods are associated with lower probability of exceedance rainfall events where rainfall loss rates are likely to be of less significance.




KILOMETRES

0 20 40 60 75

**DPI** DEPARTMENT OF PRIMARY INDUSTRIES
QUEENSLAND WATER RESOURCES

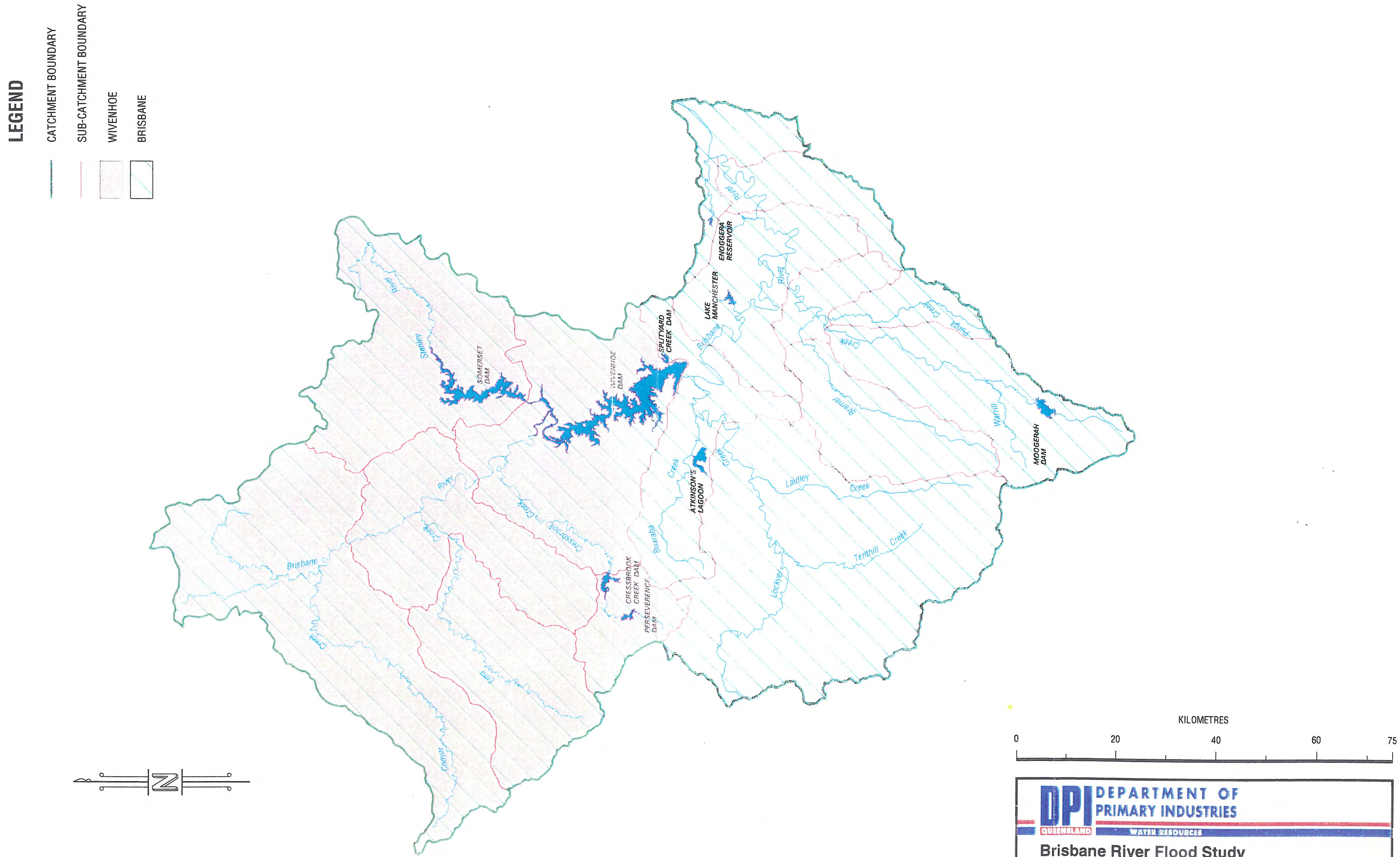
**Brisbane River Flood Study**
**Brisbane River**
## Design Storm Centres

Sh 1 of 2

17 Mar 93

A3-101428

FIGURE 6.2




DEPARTMENT OF PRIMARY INDUSTRIES
QUEENSLAND WATER RESOURCES

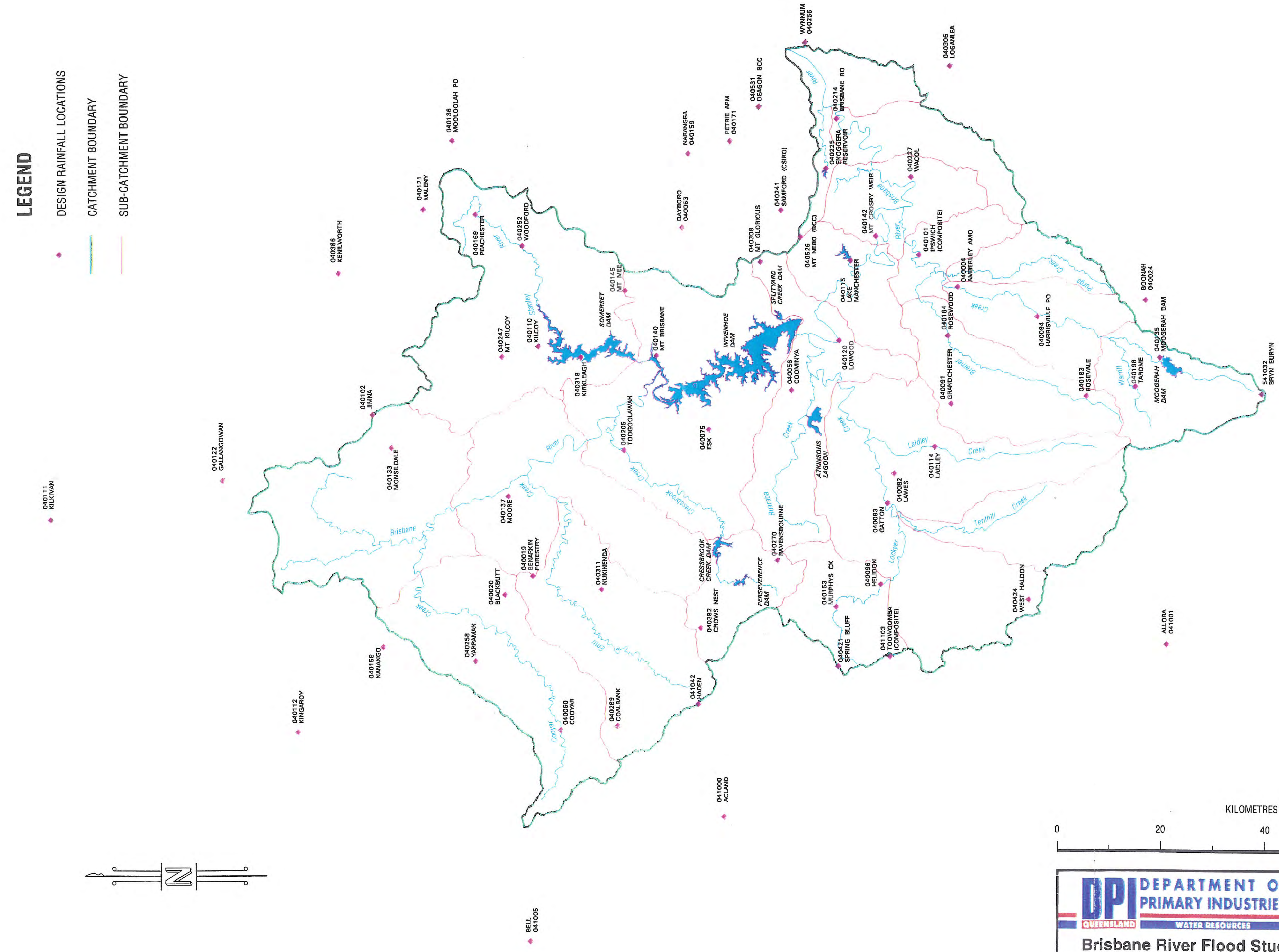
Brisbane River Flood Study
Brisbane River
Design Storm Centres

Sh 2 of 2

17 Mar 93

A3-101429

FIGURE 6.3




KILOMETRES

0 20 40 60 75

DEPARTMENT OF PRIMARY INDUSTRIES
QUEENSLAND WATER RESOURCES

**Brisbane River Flood Study**
**Brisbane River**
**Rainfall Locations**

17 Mar 93 A3-101426

# 7.0 RESULTS OF DESIGN FLOOD ESTIMATION

## 7.1 INTRODUCTION

A range of design flood scenarios were considered during the reassessment of design floods for both Somerset Dam and Wivenhoe Dam. Design storms were centred over five different catchments and the resulting storage inflow and outflow hydrographs determined using the runoff-routing models and storage operation model described earlier in this report. The five design storm centres considered were:

1. Somerset Dam Catchment
2. Upper Brisbane River Catchment
3. Middle Brisbane River Catchment
4. Wivenhoe Dam Catchment
5. Brisbane River Catchment

These catchments are shown in Figures 6.1 and 6.2.

Rainfall falling in adjacent catchments was accounted for in accordance with 'method 3' of the Bureau of Meteorology's report on the calculation of concurrent rainfall over an area when the PMP storm occurs over an adjacent catchment, (1991). The Bureau of Meteorology's report is included as Appendix A.

This method involves the use of areally adjusted 100 year ARI IFD depth and spatial distribution estimates. It produces the lowest flood magnitudes for adjacent catchments of all the methods suggested by the Bureau for estimating concurrent rainfalls.

This method was adopted because proposed dambreak analyses of Somerset Dam and Wivenhoe Dam are concerned with estimating the most severe incremental effect of flooding resulting from a possible failure of the storages under investigation. Because of this, it is believed that estimates of concurrent rainfall based upon the method that provides the lowest flood magnitudes for adjacent catchments is most appropriate for this particular application.

For the estimation of concurrent rainfalls of the higher probability of exceedance events, (ie 100 year ARI and less), a different approach was adopted. Adjacent catchments were assigned a rainfall depth sufficient to produce a rainfall depth of the same probability when averaged over the whole Brisbane River catchment, as the probability of the event over the particular sub-catchment.

## 7.2 NATURAL CATCHMENT (NO DAMS EFFECTIVE)

### 7.2.1 Introduction

An estimate of the magnitude of floods for the Brisbane River catchment without the storages of Somerset Dam and Wivenhoe Dam in place, has been made for the purpose of determining the overall mitigation effect of the dams.

The runoff-routing model layouts of the catchments, (SOM and WIV), were modified so as to represent the catchments in their natural state. The adopted model parameters were not changed, because in the original formulation of the model layouts, the distance to centroid of the various sub-catchments were based upon the natural catchment configuration.

### 7.2.2 Stanley River to Somerset Damsite

Design flood estimates for the Stanley River to Somerset Damsite were calculated for both the probable maximum precipitation, (PMP), and 100 year ARI events with the design storm centred over the subject catchment.

In all cases of the runoff-routing modelling an initial loss of 0 mm and a continuing loss of 2.5 mm/hour was adopted. The continuing loss rate is higher than the values obtained during calibration of the Somerset Dam runoff-routing model, which was typically 0.3 mm/hour. The value of 2.5 mm/hour is however, a value commonly used in design flood estimation and it is the same as what had been adopted in previous flood studies of the dam.

The rainfall depths provided in Table 6.1 and Table 6.3 were utilised for the assessment of PMP storms. Table 7.1 provides a summary of the probable maximum flood, (PMF), design storm estimates for the Stanley River catchment to Somerset Dam in its natural state.

**Table 7.1**
**Stanley River at Somerset Damsite (Natural Catchment)**
**PMF Design Estimates**
**Storm Centred over Stanley River**

| STORM DURATION (HOURS) | PEAK FLOW ($m^3/s$) | FLOOD VOLUME (ML) |
|---|---|---|
| 12 | 9 880 | 850 220 |
| 24 | 12 180 | 1 222 220 |
| 48 | 10 710 | 1 726 680 |
| 72 | 9 470 | 2 113 280 |
| 96 | 9 370 | 2 457 750 |
| 120 a | 9 410 | 2 504 540 |
| 120 b | 9 350 | 2 504 520 |
| 144 a | 9 170 | 2 486 650 |
| 144 b | 9 190 | 2 469 550 |
| 144 c | 9 550 | 2 488 760 |
| 168 | 10 640 | 2 859 340 |

Note: a b c Refer to various temporal patterns provided by the Bureau of Meteorology.

Table 7.2 provides a summary of the 100 Year ARI event design storm estimates for the Stanley River catchment to Somerset Dam in its natural state.

**Table 7.2**
**Stanley River at Somerset Damsite (Natural Catchment)**
**100 Year ARI Design Estimates**
**Storm Centred over Stanley River**

| STORM DURATION (HOURS) | PEAK FLOW ($m^3/s$) | FLOOD VOLUME (ML) |
|---|---|---|
| 12 | 2 970 | 314 800 |
| 24 | 3 220 | 400 340 |
| 36 | 2 770 | 452 480 |
| 48 | 2 670 | 481 400 |
| 60 | 2 390 | 499 850 |
| 72 | 2 500 | 534 370 |

The critical duration for the catchment of the Stanley River to Somerset Dam in its natural state, appears to be around 24 hours.

### 7.2.3 Brisbane River to Wivenhoe Damsite

Design flood estimates for the Brisbane River to Wivenhoe Damsite were calculated for both the probable maximum precipitation, (PMP), and 100 year ARI events with the design storm centred over the subject catchment.

In all cases of the runoff-routing modelling an initial loss of 0 mm and a continuing loss of 2.5 mm/hour was adopted. The continuing loss rate is similar to the values obtained during calibration of the runoff-routing models making up the Wivenhoe Dam catchment. The value of 2.5 mm/hour is a value commonly used in design flood estimation and it is the same as what had been adopted in previous flood studies of the dam.

The rainfall depths provided in Table 6.1 and Table 6.3 were utilised for the assessment of PMP storms. Table 7.3 provides a summary of the probable maximum flood, (PMF), design storm estimates for the Brisbane River catchment to Wivenhoe Damsite in its natural state.

**Table 7.3**
**Brisbane River at Wivenhoe Damsite (Natural Catchment)**
**PMF Design Estimates**
**Storm Centred over Wivenhoe Dam Catchment**

| STORM DURATION (HOURS) | PEAK FLOW ($m^3/s$) | FLOOD VOLUME (ML) |
|---|---|---|
| 12 | 31 730 | 2 952 430 |
| 24 | 39 090 | 4 288 160 |
| 48 | 31 650 | 5 271 160 |
| 72 | 28 160 | 6 329 880 |
| 96 | 27 130 | 7 100 080 |
| 120 a | 27 330 | 7 160 630 |
| 120 b | 27 290 | 7 160 460 |
| 144 a | 26 790 | 7 099 690 |
| 144 b | 26 740 | 7 002 460 |
| 144 c | 27 710 | 7 108 680 |
| 168 | 31 340 | 8 599 310 |

Note: a b c Refer to various temporal patterns provided by the Bureau of Meteorology

Table 7.4 provides a summary of the 100 Year ARI event design storm estimates for the Brisbane River catchment to Wivenhoe Damsite in its natural state.

**Table 7.4**
**Brisbane River at Wivenhoe Damsite (Natural Catchment)**
**100 Year ARI Design Estimates**
**Storm Centred over Wivenhoe Dam Catchment**

| STORM DURATION (HOURS) | PEAK FLOW ($m^3/s$) | FLOOD VOLUME (ML) |
|---|---|---|
| 12 | 10 670 | 1 194 760 |
| 24 | 10 530 | 1 457 650 |
| 36 | 9 380 | 1 601 780 |
| 48 | 8 600 | 1 657 000 |
| 60 | 7 570 | 1 685 200 |
| 72 | 8 350 | 1 796 740 |

The critical duration for the catchment of the Brisbane River to Wivenhoe Dam in its natural state, appears to be around 24 hours.

## 7.3 SOMERSET DAM

Design flood estimates for Somerset Dam were calculated assuming the dam was at its normal full supply level, (FSL), prior to the event and the estimated inflow hydrographs were routed through the storage using the operation procedure outlined in Section 5.3.

In all cases of the runoff-routing modelling an initial loss of 0 mm and a continuing loss of 2.5 mm/hour was adopted. The continuing loss rate is higher than the values obtained during calibration of the Somerset Dam runoff-routing model, which was typically 0.3 mm/hour. The value of 2.5 mm/hour is however, a value commonly used in design flood estimation and it is the same as what had been adopted in previous flood studies of the dam.

The rainfall depths provided in Table 6.1 and Table 6.3 were utilised for the assessment of PMP storms. Table 7.5 provides a summary of the probable maximum flood, (PMF), design storm scenarios for critical design storms centred on catchments that affect Somerset Dam.

**Table 7.5**
**Somerset Dam PMF Design Estimates**
**Critical Design Storms Centred Over Different Catchments**

| CATCHMENT OF STORM CENTRE | STORM DURATION (HOURS) | PEAK INFLOW ($m^3/s$) | PEAK OUTFLOW ($m^3/s$) | FLOOD VOLUME( ML) | PEAK LAKE LEVEL (m AHD) |
|---|---|---|---|---|---|
| 1. Somerset Dam | 120 b | 9 630 | 8 140 | 2 504 600 | 110.41 * |
| 2. Upper Brisbane | 24 | 2 700 | 2 330 | 258 740 | 102.43 |
| 3. Middle Brisbane | 24 | 2 700 | 2 330 | 258 740 | 102.43 |
| 4. Wivenhoe Dam | 48 | 6 390 | 4 210 | 995 680 | 106.71 |
| 5. Brisbane River | 24 | 6 850 | 3 390 | 623 500 | 105.17 |

Notes:

* Indicates that the non-overflow section of the spillway, (EL 107.46 m AHD) is overtopped.

a b c Refers to various temporal patterns provided by the Bureau of Meteorology.

It is evident from Table 7.5 that the critical design storm scenario for Somerset Dam is the design storm centred over the catchment of the dam. The inflow and outflow hydrographs for this particular event are presented in Figure 7.1, and a time series of the predicted water level in Somerset Dam is presented in Figure 7.2. A summary of the complete range of PMF durations for this particular storm centre scenario is presented in Table 7.6.

It was noted during the simulation of the PMF events over the Somerset Dam catchment that the 168 hour durations with early and late peaks, (168 b and c), produced the largest peak outflows and associated peak lake levels.

A check on these temporal patterns revealed that a 72 hour period embedded within these patterns produced a larger rainfall depth than that yielded by the 72 hour temporal pattern provided by the Bureau. (Refer to Figures 7.3 and 7.4).

Since the 72 hour pattern supposedly produces the critical depth for that duration, the results obtained from the combination of the 168 b and c temporal patterns and associated PMP depths have been disregarded because the resulting rainfall depths are invalid.

**Table 7.6**
**Somerset Dam PMF Design Estimates**
**Storm Centred Over Somerset Dam Catchment**

| STORM DURATION (HOURS) | PEAK INFLOW ($m^3/s$) | PEAK OUTFLOW ($m^3/s$) | FLOOD VOLUME (ML) | PEAK LAKE LEVEL (m AHD) |
|---|---|---|---|---|
| 12 | 14 990 | 4 650 | 847 000 | 107.42 |
| 24 | 13 810 | 6 700 | 1 220 400 | 109.38 * |
| 36 | 12 030 | 7 610 | 1 499 690 | 110.05 * |
| 48 | 11 070 | 8 000 | 1 725 800 | 110.31 * |
| 72 | 9 810 | 7 920 | 2 112 800 | 110.26 * |
| 96 | 9 910 | 8 000 | 2 457 600 | 110.32 * |
| 120 a | 10 030 | 8 080 | 2 504 300 | 110.37 * |
| 120 b | 9 630 | 8 140 | 2 504 600 | 110.41 * |
| 144 a | 9 540 | 7 930 | 2 486 300 | 110.27 * |
| 144 b | 9 290 | 8 060 | 2 469 600 | 110.35 * |
| 144 c | 9 930 | 8 130 | 2 488 800 | 110.40 * |
| 168 a | 11 060 | 7 600 | 2 858 300 | 110.04 * |

Notes:

* Indicates that the non-overflow section of the spillway (EL 107.46 m AHD) is overtopped.

a b c Refers to various temporal patterns provided by the Bureau of Meteorology.

A full flood frequency range of the critical design storm scenario has been estimated for Somerset Dam. However, because it is not possible to estimate IFD rainfalls for durations in excess of 72 hours with the procedures outlined in Australian Rainfall and Runoff, (1987), the 48 hour duration storm rainfall has been adopted for this exercise. It should be noted from Table 7.6 that the 48 hour duration event is only slightly smaller than the 120 b hour duration event and as a consequence the results should be similar for both durations. Table 7.7 provides the full flood frequency estimates for Somerset Dam.

The effect of using different temporal patterns for various ARI's is evident in Table 7.7. The PMP temporal patterns provided by the Bureau of Meteorology, (which were used for events with an ARI greater than 100 years), and the temporal patterns provided in Australian Rainfall and Runoff, (1987), (which were used for events with an ARI of 100 years or less), are quite different. The Bureau of Meteorology's PMP temporal pattern for the 48 hour duration is relatively uniform, whilst the temporal pattern in Australian Rainfall and Runoff, (1987), has two distinct bursts incorporated within it. Refer to Figure 7.5 for a comparison between the two temporal patterns. This leads to the peak inflows of the 100 year ARI event being larger than the 200 year ARI event, but this is not so for the outflows.

**Table 7.7**
**Somerset Dam Design Flood Estimates**
**Storm centred over Somerset Dam Catchment**
**48 Hour Duration**

| ARI (YEARS) | PEAK INFLOW ($m^3/s$) | PEAK OUTFLOW ($m^3/s$) | FLOOD VOLUME (ML) | PEAK LAKE LEVEL (m AHD) |
|---|---|---|---|---|
| 10 | 2 340 | 1 950 | 246 040 | 102.24 |
| 20 | 2 940 | 2 290 | 311 540 | 102.30 |
| 50 | 3 350 | 2 360 | 400 920 | 102 52 |
| 100 | 4 030 | 2 480 | 479 300 | 102.92 |
| 200 | 3 580 | 2 720 | 557 590 | 103.62 |
| 500 | 4 350 | 3 070 | 667 080 | 104.49 |
| 1 000 | 4 940 | 3 400 | 770 000 | 105.19 |
| 10 000 | 7 070 | 4 620 | 1 101 880 | 107.37 |
| 100 000 | 9 110 | 6 280 | 1 420 480 | 109.04 * |
| 1 000 000 (PMF) | 11 070 | 8 000 | 1 725 780 | 110.31 * |
| Notes: * Indicates that the non-overflow spillway level, (EL 107.46 m AHD), is overtopped. | | | | |

A further check on the 100 year ARI events reveals that the 36 hour storm duration produces the largest outflows from Somerset Dam under the current operating procedure. In light of this finding the higher probability of exceedance flood events have been estimated for storm durations of 36 hours.

The results of this assessment are provided in Table 7.8.

**Table 7.8**
**Somerset Dam Design Flood Estimates**
**Storm centred over Somerset Dam Catchment**
**36 Hour Duration**

| ARI (YEARS) | PEAK INFLOW ($m^3/s$) | PEAK OUTFLOW ($m^3/s$) | FLOOD VOLUME (ML) | PEAK LAKE LEVEL (m AHD) |
|---|---|---|---|---|
| 10 | 2 350 | 2 130 | 239 430 | 102.12 |
| 20 | 2 980 | 2 280 | 300 470 | 102.24 |
| 50 | 3 810 | 2 410 | 379 380 | 102.68 |
| 100 | 4 560 | 2 550 | 450 240 | 103.13 |

Estimates of the magnitude of the design floods that occur in conjunction with the critical design floods of the dams for various locations throughout the Brisbane River catchment have been made. It should be noted that these estimates are not the actual design floods of these sites but rather, they are the critical Somerset Dam design floods associated with these areas. Table 7.9 provides a summary of these estimates.

An estimate of the magnitude of the flood event which when routed through the storage under the existing storage operating procedure just threatens to overtop the non-overflow spillway has been made. The rainfall depth associated with this flood equates to approximately 68 % of the PMP. The ARI of this rainfall depth is estimated to be 20 000 years.

For embankment dams this flood is normally referred to as the Imminent Failure Flood, (IFF). However, according to ANCOLD guidelines, (1986), for concrete dams, the IFF can often be a flood for which the stillwater pool level is above the top of the dam or even the parapet. Assessment of the IFF should be based upon the structural stability of the dam with the reservoir at the flood level, and the capability of the downstream foundations to resist the overtopping flow.

Russo, (1988), concludes in his report on the safety of Somerset Dam that structurally, the dam is in excellent condition. He states that the dam can be used to hold back flood waters in extreme events to prevent overtopping of Wivenhoe Dam. These assessments are based upon estimated flood levels which are very similar to, but different from, the levels determined in this reassessment, (refer Section 3.2).

Russo also recommends that to ensure the survival of the portions of two non-overflow monoliths above EL 100.0 m AHD, the reservoir level should not exceed EL 111.7 m AHD. He adds that the structural integrity of the spillway gates would have to be checked for the loads such a reservoir level would impose.

**Table 7.9**
**Somerset Dam Design Flood Estimates**
**Storm centred over Somerset Dam Catchment**
**Estimated Floods at Various Locations**

| SITE | 100 YEAR ARI (36 HOUR) | | PMF (120 b HOUR) | |
|---|---|---|---|---|
| | PEAK FLOW ($m^3/s$) | FLOOD VOLUME (ML) | PEAK FLOW ($m^3/s$) | FLOOD VOLUME (ML) |
| Cooyar Ck @ Damsite | 1 870 | 198 470 | 900 | 186 300 |
| Brisbane R @ Linville | 4 690 | 414 220 | 1 890 | 388 660 |
| Emu Ck @ Boat Mountain | 1 880 | 185 540 | 860 | 174 110 |
| Brisbane R @ Gregors Ck | 7 860 | 797 950 | 3 640 | 748 380 |
| Cressbrk Ck @ Cressbrk Dam | 400 | 64 370 | 300 | 60 350 |
| Stanley R @ Somerset Dam | 2 550 | 450 240 | 8 140 | 2 504 600 |
| Brisbane R @ Wivenhoe Dam | 5 460 | 1 675 020 | 10 820 | 3 630 630 |
| Lockyer Ck @ Helidon | 1 310 | 76 470 | 360 | 71 890 |
| Tenthill Ck @ Tenthill | 1 650 | 93 300 | 440 | 88 680 |
| Lockyer Ck @ Lyons Bridge | 3 080 | 494 530 | 2 080 | 463 670 |
| Brisbane R @ Savages Xing | 7 330 | 2 317 180 | 13 070 | 4 232 820 |
| Brisbane R @ Mt Crosby Weir | 6 680 | 2 389 810 | 13 090 | 4 301 020 |
| Bremer R @ Walloon | 1 060 | 127 140 | 570 | 119 340 |
| Warrill Ck @ Kalbar | 870 | 95 260 | 440 | 89 380 |
| Warrill Ck @ Amberley | 1 310 | 186 500 | 800 | 174 900 |
| Purga Ck @ Loamside | 330 | 45 240 | 190 | 42 460 |
| Bremer R @ Ipswich | 2 640 | 405 400 | 1 720 | 380 300 |
| Brisbane R @ Jindalee | 7 780 | 2 873 730 | 14 150 | 4 755 410 |
| Brisbane R @ Port Office | 8 170 | 2 941 820 | 14 200 | 4 819 900 |

Not withstanding the check on the spillway gates, it would appear that Somerset Dam could withstand being overtopped to a level of 111.7 m AHD. This level is higher than the reservoir level estimated for the PMF, (110.41 m AHD). It can therefore be concluded that Somerset Dam is structurally sound enough to withstand and safely pass the PMF.

## 7.4 WIVENHOE DAM

Design flood estimates for Wivenhoe Dam were calculated assuming the dam was at its normal full supply level, (FSL), prior to the event and that the estimated inflow hydrographs were routed through the storage using operation procedure 4. Procedure 4, which is outlined in Section 5.3, is the operation procedure used for floods of the largest magnitude. The estimated inflow hydrographs allow for the effect of Somerset Dam.

In all cases an initial loss of 0 mm and a continuing loss of 2.5 mm/hour was adopted. The continuing loss rate is similar to the values obtained during model calibration and it is the same as what had previously been adopted. The rainfall depths provided in Table 6.1 and Table 6.3 were utilised for the assessment of PMP storms. Table 7.10 provides a summary of the PMF design storm scenarios for design storms centred on different catchments that affect Wivenhoe Dam.

It is evident from Table 7.10 that the critical design storm scenario for Wivenhoe Dam is the design storm centred over the catchment of the dam. The inflow and outflow hydrographs for this particular event are presented in Figure 7.6 and a time series of the predicted water level in Wivenhoe Dam is presented in Figure 7.7.

**Table 7.10**
**Wivenhoe Dam PMF Design Estimates**
**Critical Design Storms**

| CATCHMENT OF STORM CENTRE | STORM DURATION (HOURS) | PEAK INFLOW ($m^3/s$) | PEAK OUTFLOW ($m^3/s$) | FLOOD VOLUME (ML) | PEAK LAKE LEVEL (m AHD) |
|---|---|---|---|---|---|
| 1. Somerset Dam | 48 | 15 090 | 11 260 | 2 974 400 | 76.43 |
| 2. Upper Brisbane | 48 | 28 910 | 21 900 | 4 612 580 | 80.96 * |
| 3. Middle Brisbane | 48 | 15 490 | 11 130 | 2 555 310 | 76.28 |
| 4. Wivenhoe Dam | 48 | 30 670 | 25 040 | 5 333 920 | 81.28 |
| 5. Brisbane River | 120 a | 20 720 | 17 250 | 5 506 010 | 80.39 |
| Note: * Indicates that the embankment crest level, (EL 79.15 m AHD), is overtopped. | | | | | |

A summary of the complete range of PMF durations for this particular scenario is presented in Table 7.11.

A similar problem exists relating to the early and late 168 hours temporal patterns as was discovered with Somerset Dam and so these results have also been disregarded. Refer to Section 7.3.

**Table 7.11**
**Wivenhoe Dam PMF Design Estimates**
**Storm Centred over Wivenhoe Dam Catchment**

| STORM DURATION (HOURS) | PEAK INFLOW ($m^3/s$) | PEAK OUTFLOW ($m^3/s$) | FLOOD VOLUME (ML) | PEAK LAKE LEVEL (m AHD) |
|---|---|---|---|---|
| 12 | 34 420 | 13 230 | 3 011 580 | 78.56 |
| 24 | 39 880 | 23 710 | 4 349 350 | 81.15 * |
| 36 | 30 670 | 25 040 | 5 333 920 | 81.28 * |
| 72 | 27 290 | 24 830 | 6 394 180 | 81.26 * |
| 96 | 26 550 | 24 380 | 7 164 920 | 81.21 * |
| 120 a | 26 680 | 24 730 | 7 225 670 | 81.25 * |
| 120 b | 26 490 | 24 650 | 7 223 730 | 81.24 * |
| 144 a | 25 850 | 23 950 | 7 164 150 | 81.17 * |
| 144 b | 25 980 | 24 430 | 7 065 620 | 81.22 * |
| 144 c | 27 010 | 24 750 | 7 165 940 | 81.25 * |
| 168 a | 30 490 | 24 080 | 8 650 650 | 81.18 * |

Notes:
* Indicates that the embankment crest level, (EL 79.15 m AHD), is overtopped.

a b c Refer to different temporal patterns.

A full flood frequency range of the critical design storm scenario has been estimated for Wivenhoe Dam. The 48 hour duration has been adopted as the critical storm duration. Table 7.12 provides the full flood frequency estimates for Wivenhoe Dam.

**Table 7.12**
**Wivenhoe Dam Design Flood Estimates**
**Storm centred over Wivenhoe Dam Catchment**
**48 Hour Duration**

| ARI (YEARS) | PEAK INFLOW ($m^3/s$) | PEAK OUTFLOW ($m^3/s$) | FLOOD VOLUME (ML) | PEAK LAKE LEVEL (m AHD) |
|---|---|---|---|---|
| 10 | 4 230 | 3 240 | 906 210 | 69.51 |
| 20 | 5 280 | 3 410 | 1 141 270 | 71.40 |
| 50 | 7 640 | 3 470 | 1 445 360 | 73.97 |
| 100 | 9 250 | 6 490 | 1 718 680 | 74.57 |
| 200 | 11 110 | 7 640 | 1 822 840 | 74.84 |
| 500 | 12 580 | 9 130 | 2 104 520 | 75.50 |
| 1 000 | 13 820 | 9 970 | 2 336 350 | 75.99 |
| 10 000 | 20 770 | 13 490 | 3 593 000 | 78.61 |
| 100 000 (PMF) | 30 670 | 25 040 | 5 333 920 | 81.28 * |

Notes: * Indicates that the embankment crest level, (EL 79.15 m AHD), is overtopped.

The effect of using different temporal patterns for events above the 100 year ARI event is not as pronounced in the case of Wivenhoe Dam as compared to Somerset Dam. The reason for this lies with the releases from Somerset Dam. Under the current operation procedure the release from Somerset Dam is similar in shape regardless of the shape of the inflow hydrograph. And as the Stanley River catchment is a significant contributor to the runoff into Wivenhoe Dam the releases from Somerset Dam have a major impact on the time distribution of inflows into Wivenhoe Dam.

The 100 year ARI temporal patterns do have a similar impression on the critical duration of the higher probability of exceedance events however. In keeping with the Somerset Dam experience, a further check on the 100 year ARI events revealed that the 72 hour storm duration produces the largest outflows from Wivenhoe Dam under the current operating procedures. In light of this finding, the higher probability of exceedance flood events have been estimated for storm durations of 72 hours. The results of this assessment are provided in Table 7.13.

Interestingly, whilst the peak outflow from Wivenhoe Dam occurs in association with the 72 hour storm duration, peak inflows and peak lake levels are a consequence of the 24 hour duration flood event.

**Table 7.13**
**Wivenhoe Dam Design Flood Estimates**
**Storm centred over the Wivenhoe Dam Catchment**
**72 Hour Duration**

| ARI (YEARS) | OPS | PEAK INFLOW ($m^3/s$) | PEAK OUTFLOW ($m^3/s$) | FLOOD VOLUME (ML) | PEAK LAKE LEVEL (m AHD) |
|---|---|---|---|---|---|
| 10 | 3 | 3 630 | 2 900 | 861 570 | 68.18 |
| 20 | 4 | 4 980 | 3 330 | 1 128 590 | 70.58 |
| 50 | 4 | 7 240 | 3 450 | 1 405 480 | 72.83 |
| 100 | 4 | 9 080 | 6 810 | 1 860 400 | 74.48 |

Where OPS = Wivenhoe Dam Operating Procedure. (Refer Section 5.3).

Estimates of the magnitude of the design floods that occur in conjunction with the critical design floods of the dams for various locations throughout the Brisbane River catchment have been made. It should be noted that these estimates are not the actual design floods of these sites but rather, they are the critical Wivenhoe Dam design floods associated with these areas. Table 7.14 provides a summary of these estimates.

Embankment dams when subject to a continuous overtopping flow will normally fail, depending upon the duration of the flow and the likely extent of scouring of the crest. The Imminent Failure Flood, (IFF), for Wivenhoe Dam has therefore been assessed as the flood event which when routed through the storage under the existing storage operating procedure just threatens to overtop the embankment. The embankment crest level, (EL 79.15 m AHD), has been adopted as the critical level in preference to the top of the wave wall because the wave wall does not extend over the whole of the embankment.

The estimated magnitude of the rainfall depth associated with the IFF for Wivenhoe Dam is 75 % of the PMP. This rainfall depth has an ARI of approximately 14 300 years.

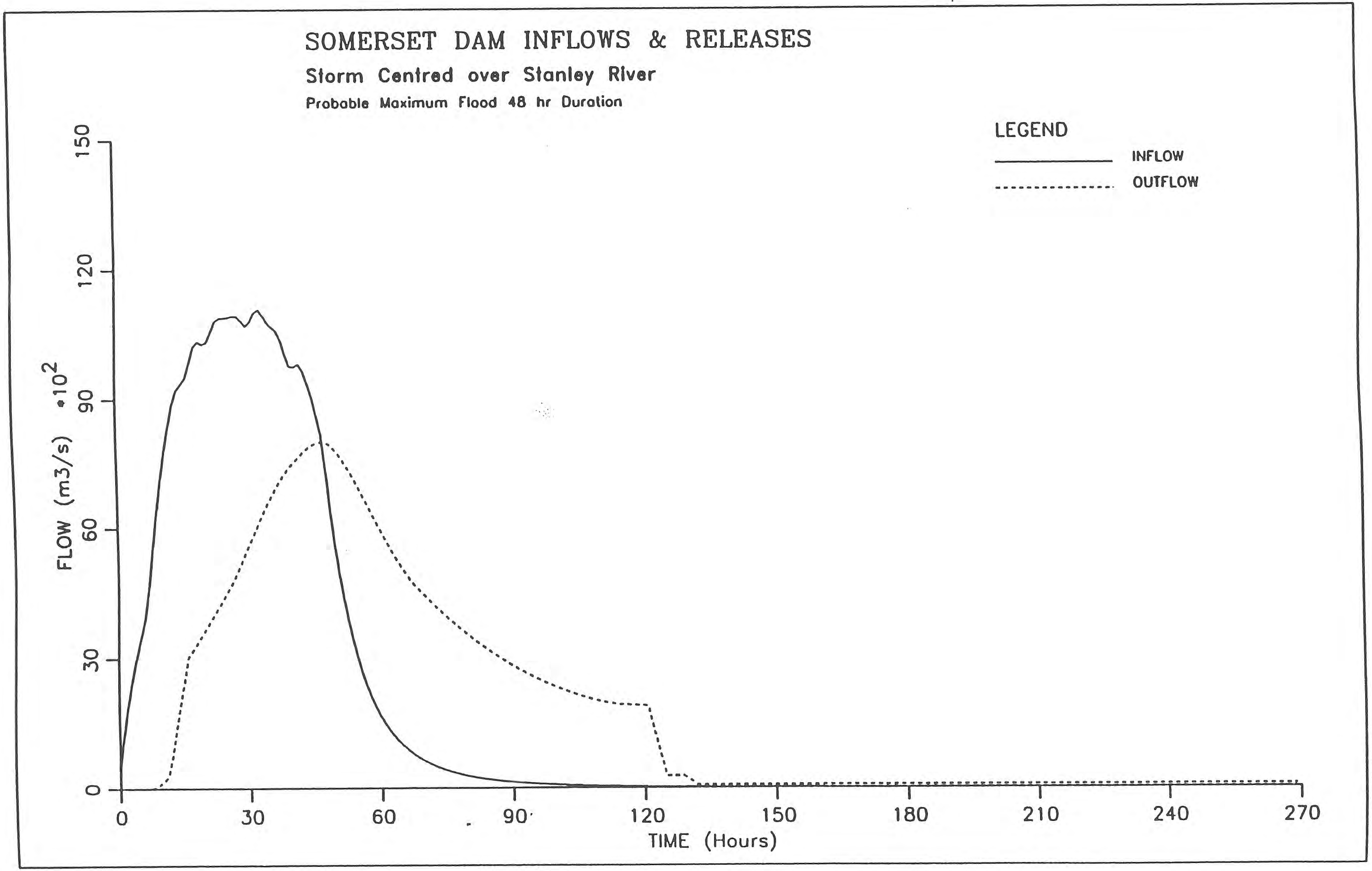
The peak inflow associated with the IFF of Wivenhoe Dam is estimated to be 21 990 $m^3/s$, whilst the resultant peak outflow from the dam is 14 080 $m^3/s$. The flood volume for the IFF is estimated to be 3 794 180 ML.

**Table 7.14**
**Wivenhoe Dam Design Flood Estimates**
**Storm centred over Wivenhoe Dam Catchment**
**Estimated Floods at Various Locations**

| SITE | 100 YEAR ARI (72 HOUR) | | PMF (48 HOUR) | |
|---|---|---|---|---|
| | PEAK FLOW ($m^3/s$) | FLOOD VOLUME (ML) | PEAK FLOW ($m^3/s$) | FLOOD VOLUME (ML) |
| Cooyar Ck @ Damsite | 1 750 | 249 600 | 4 750 | 732 520 |
| Brisbane R @ Linville | 4 510 | 520 830 | 9 950 | 1 528 230 |
| Emu Ck @ Boat Mountain | 1 810 | 233 300 | 4 490 | 684 620 |
| Brisbane R @ Gregors Ck | 7 460 | 1 003 120 | 19 110 | 2 942 820 |
| Cressbrk Ck @ Cressbrk Dam | 610 | 80 860 | 1 420 | 237 340 |
| Stanley R @ Somerset Dam | 1 980 | 338 600 | 4 210 | 995 680 |
| Brisbane R @ Wivenhoe Dam | 6 810 | 1 860 400 | 25 040 | 5 333 920 |
| Lockyer Ck @ Helidon | 1 430 | 84 790 | 560 | 78 390 |
| Tenthill Ck @ Tenthill | 1 810 | 103 900 | 700 | 96 590 |
| Lockyer Ck @ Lyons Bridge | 2 620 | 547 640 | 3 060 | 505 670 |
| Brisbane R @ Savages Xing | 8 580 | 2 547 210 | 28 290 | 5 971 780 |
| Brisbane R @ Mt Crosby Weir | 7 930 | 2 627 860 | 27 940 | 6 046 220 |
| Bremer R @ Walloon | 900 | 140 860 | 850 | 130 130 |
| Warrill Ck @ Kalbar | 720 | 105 530 | 650 | 97 470 |
| Warrill Ck @ Amberley | 1 090 | 206 550 | 1 210 | 190 730 |
| Purga Ck @ Loamside | 270 | 50 120 | 286 | 46 300 |
| Bremer R @ Ipswich | 2 240 | 449 020 | 2 580 | 414 700 |
| Brisbane R @ Jindalee | 8 590 | 3 164 720 | 29 200 | 6 541 910 |
| Brisbane R @ Port Office | 8 580 | 3 240 750 | 29 220 | 6 612 280 |

By way of comparison, if the top of the wave wall, (EL 79.0 m AHD), is adopted as critical level, the magnitude of rainfall depth associated with this flood is 81 % of the PMP. This rainfall depth has an ARI of approximately 23 600 years.

The peak inflow associated with this flood is estimated to be 24 060 $m^3/s$, whilst the resultant peak outflow from the dam is 14 920 $m^3/s$. The flood volume for this flood is estimated to be 4 162 020 ML.



SOMERSET DAM INFLOWS & RELEASES
Storm Centred over Stanley River
Probable Maximum Flood 48 hr Duration
LEGEND
INFLOW
OUTFLOW
FLOW (m3/s) *10²
0
30
60
90
120
150
0
30
60
90
120
150
180
210
240
270
TIME (Hours)


FIGURE 7.1

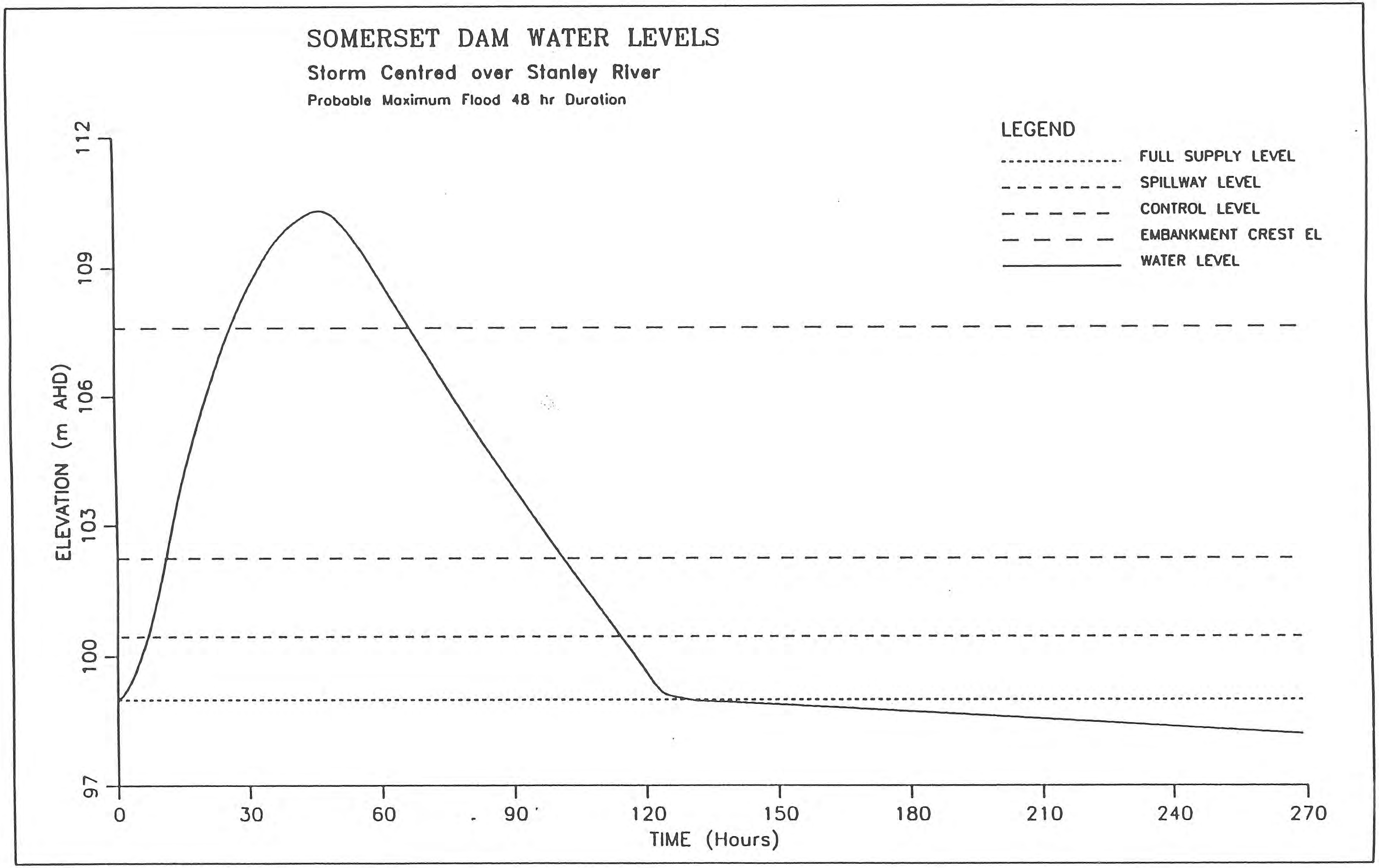

SOMERSET DAM WATER LEVELS
Storm Centred over Stanley River
Probable Maximum Flood 48 hr Duration
LEGEND
FULL SUPPLY LEVEL
SPILLWAY LEVEL
CONTROL LEVEL
EMBANKMENT CREST EL
WATER LEVEL
ELEVATION (m AHD)
97
100
103
106
109
112
0
30
60
90
120
150
180
210
240
270
TIME (Hours)

FIGURE 7.2

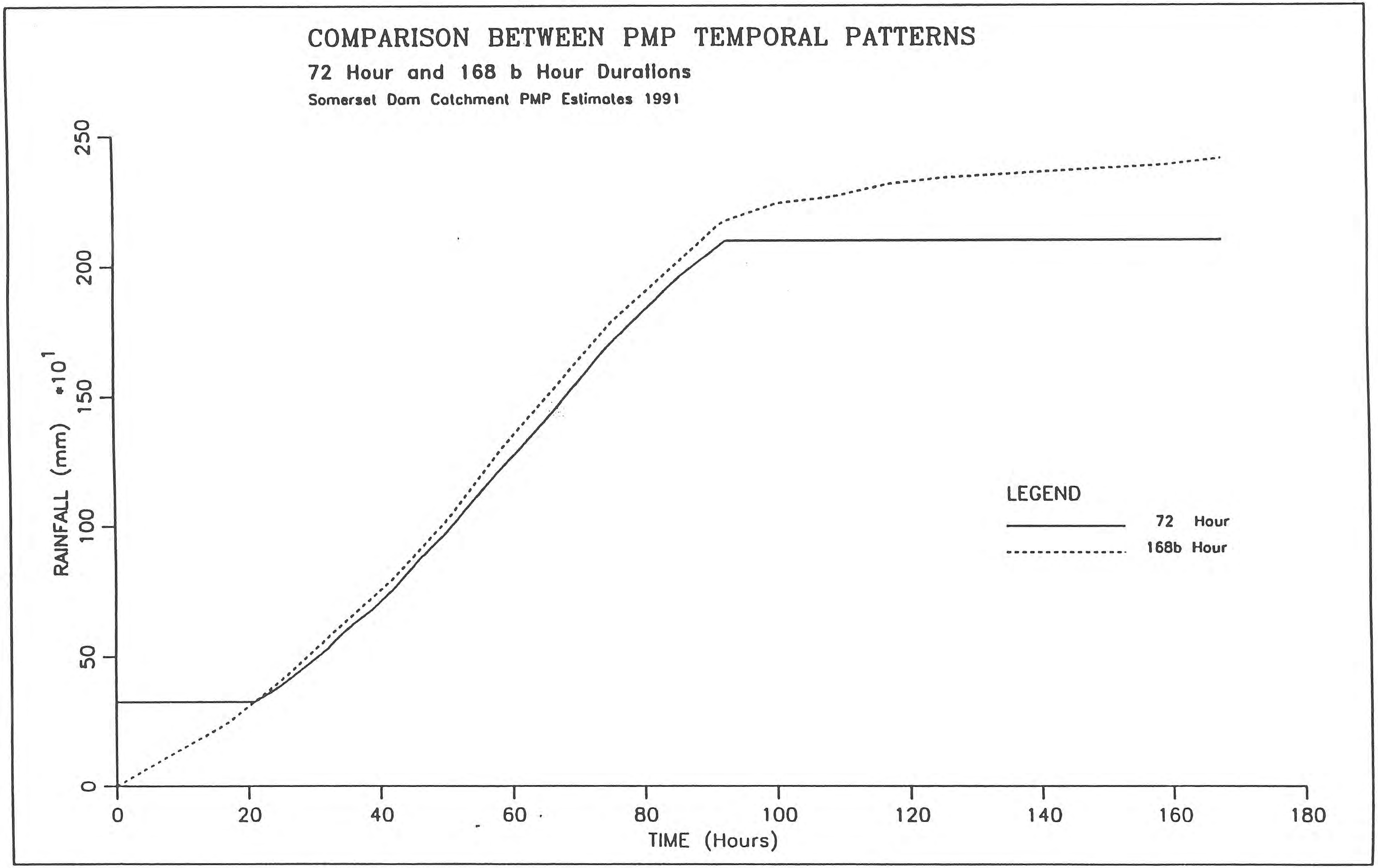

COMPARISON BETWEEN PMP TEMPORAL PATTERNS
72 Hour and 168 b Hour Durations
Somerset Dam Catchment PMP Estimates 1991
RAINFALL (mm) *10^1
0
50
100
150
200
250
0
20
40
60
80
100
120
140
160
180
TIME (Hours)
LEGEND
72 Hour
168b Hour

FIGURE 7.3

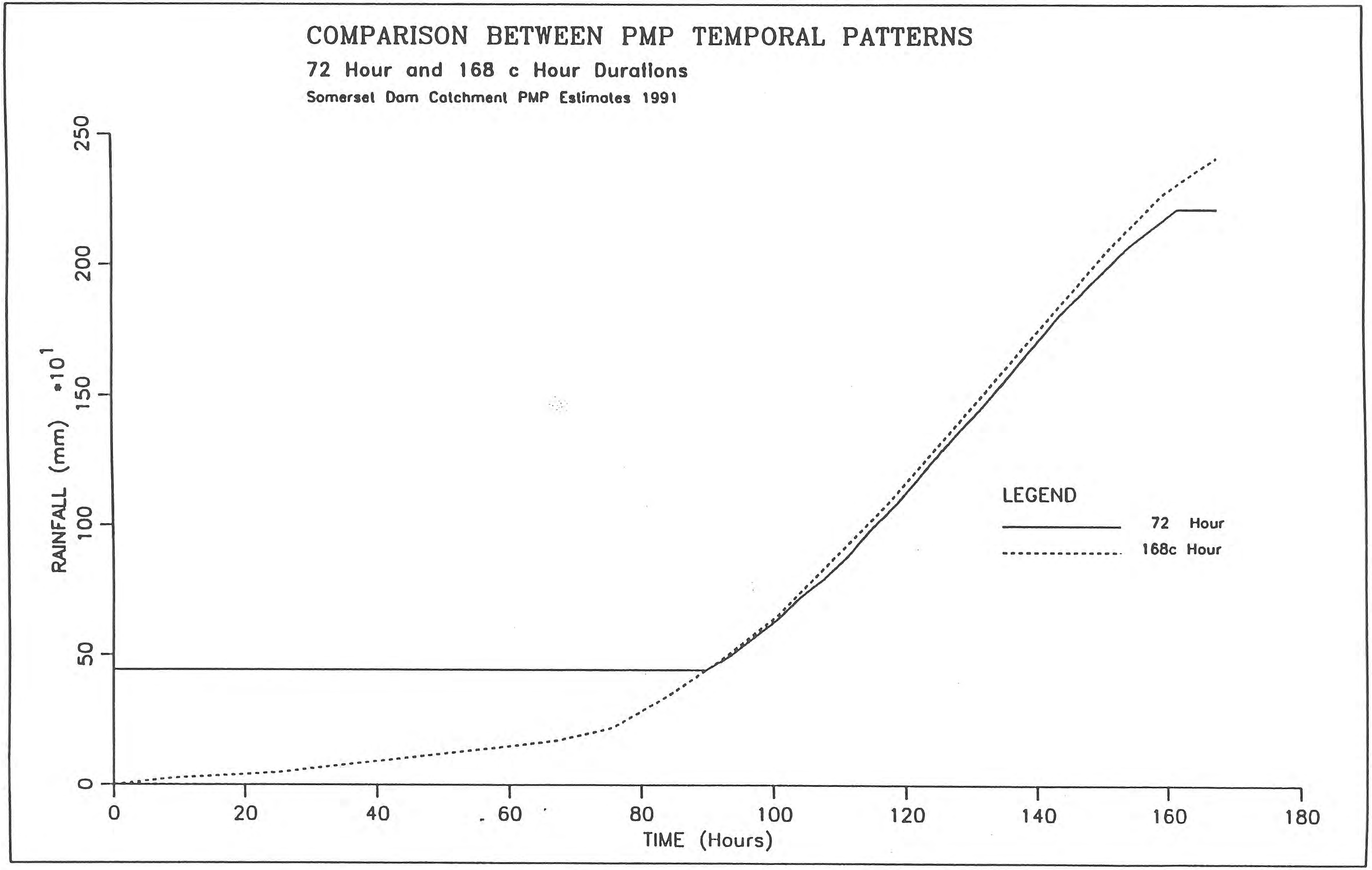

COMPARISON BETWEEN PMP TEMPORAL PATTERNS
72 Hour and 168 c Hour Durations
Somerset Dam Catchment PMP Estimates 1991
RAINFALL (mm) *10¹
0
50
100
150
200
250
0
20
40
60
80
100
120
140
160
180
TIME (Hours)
LEGEND
72 Hour
168c Hour

FIGURE 7.4

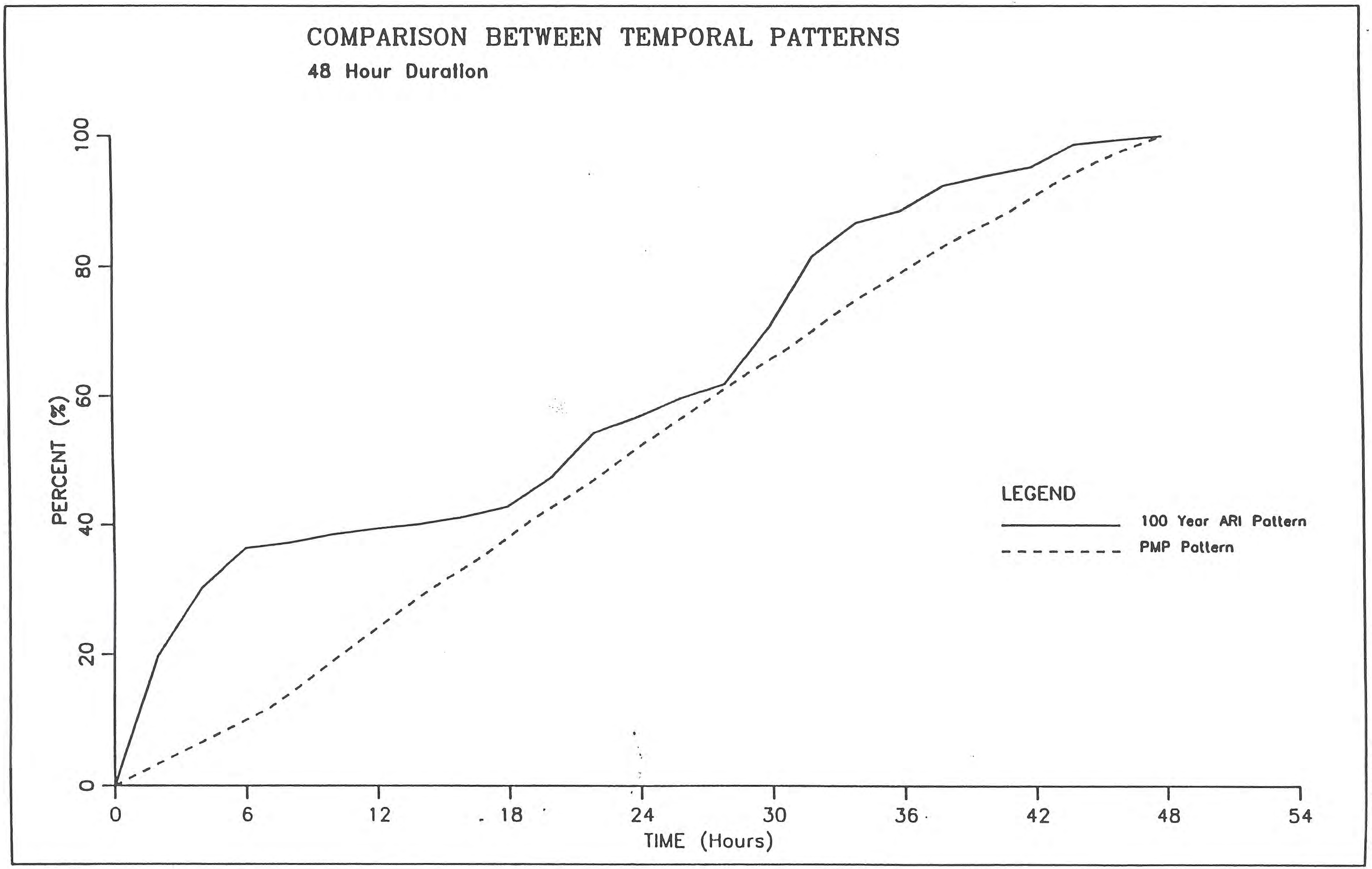

COMPARISON BETWEEN TEMPORAL PATTERNS
48 Hour Duration
PERCENT (%)
0
20
40
60
80
100
0
6
12
18
24
30
36
42
48
54
TIME (Hours)
LEGEND
100 Year ARI Pattern
PMP Pattern

FIGURE 7.5

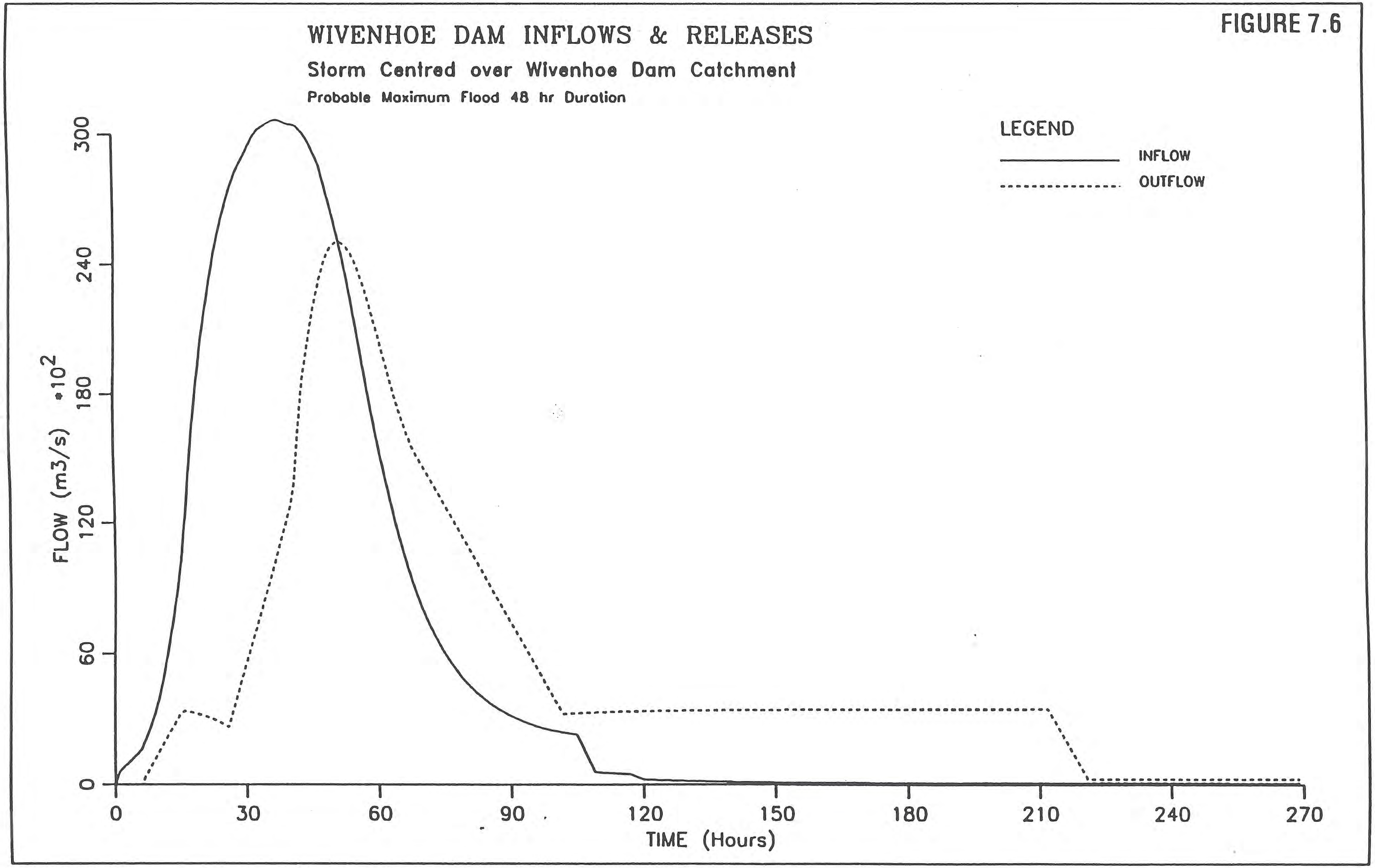


FIGURE 7.6
WIVENHOE DAM INFLOWS & RELEASES
Storm Centred over Wivenhoe Dam Catchment
Probable Maximum Flood 48 hr Duration
LEGEND
INFLOW
OUTFLOW
FLOW (m3/s) *10^2
0
60
120
180
240
300
0
30
60
90
120
150
180
210
240
270
TIME (Hours)

FIGURE 7.6

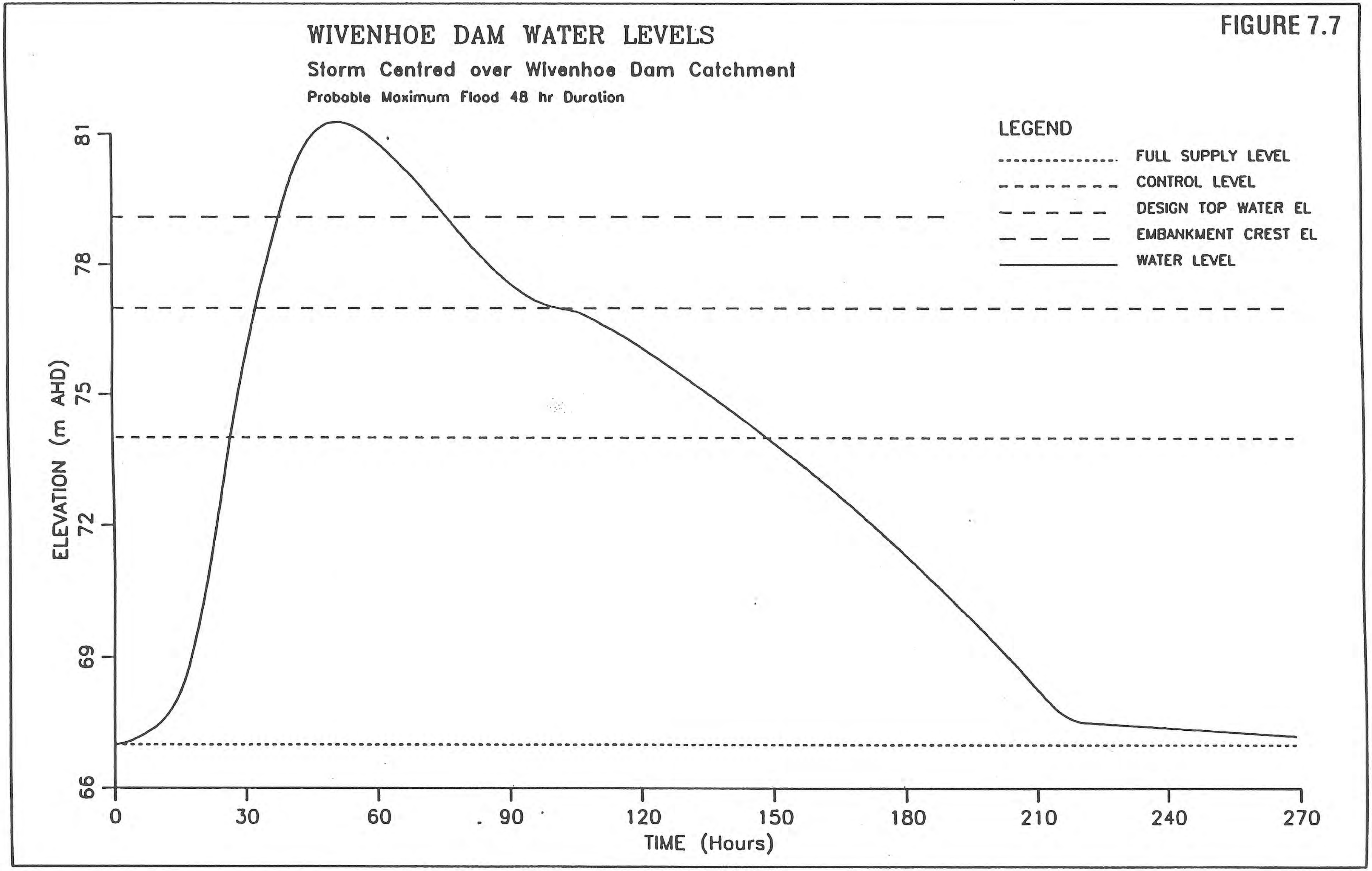


FIGURE 7.7
WIVENHOE DAM WATER LEVELS
Storm Centred over Wivenhoe Dam Catchment
Probable Maximum Flood 48 hr Duration
LEGEND
FULL SUPPLY LEVEL
CONTROL LEVEL
DESIGN TOP WATER EL
EMBANKMENT CREST EL
WATER LEVEL
ELEVATION (m AHD)
81
78
75
72
69
66
0
30
60
90
120
150
180
210
240
270
TIME (Hours)

FIGURE 7.7

# 8.0 SUMMARY AND CONCLUSIONS

## 8.1 GENERAL

The design floods for Somerset Dam and Wivenhoe Dam have been reassessed in view of updated estimates of the probable maximum precipitation and the recent changes to procedures for estimating intensity-frequency-duration data in Australian Rainfall and Runoff, (1987).

In general, the revised PMP values for the catchment of Somerset Dam have increased by between 5 to 20 % over the estimates of PMP that were used in 1983 for durations greater than 6 hours in length. There is however, a small reduction in the 6 hour duration PMP depth which is associated with a change in the way in which the estimate was derived. Table 8.1 provides a comparison between the 1983 and 1991 PMP estimates derived by the Bureau of Meteorology.

**Table 8.1**
**Comparison of PMP Estimates**

| DUR (HRS) | PMP ESTIMATE (mm) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | SOMERSET | | UPPER BRISBANE | | MIDDLE BRISBANE | | WIVENHOE | |
| | 1983 | 1991 | 1983 | 1991 | 1983 | 1991 | 1983 | 1991 |
| 6 | 400 | 390 | 300 | 270 | 420 | 460 | 260 | 240 |
| 12 | 560 | 670 | 420 | 490 | 560 | 660 | 380 | 450 |
| 24 | 840 | 900 | 660 | 720 | 820 | 940 | 600 | 670 |
| 48 | 1 360 | 1 420 | 1 080 | 980 | 1 360 | 1 390 | 1 000 | 870 |
| 72 | 1 760 | 1 770 | 1 380 | 1 200 | 1 720 | 1 740 | 1 260 | 1 080 |
| 96 | 2 040 | 2 090 | 1 600 | 1 390 | 2 000 | 2 060 | 1 460 | 1 250 |
| 120 | 2 120 | 2 170 | 1 660 | 1 440 | 2 080 | 2 130 | 1 520 | 1 300 |
| 144 | 2 160 | 2 220 | 1 700 | 1 480 | 2 120 | 2 190 | 1 560 | 1 330 |
| 168 | 2 340 | 2 410 | 1 840 | 1 670 | 2 320 | 2 360 | 1 700 | 1 480 |

The same conclusions can be drawn for the residual area catchment, (Middle Brisbane), PMP estimates that were made for Somerset Dam, although the 6 hour duration increased by a small amount for this catchment.

The PMP estimates for the larger catchments of the Upper Brisbane and the Wivenhoe Dam catchment exhibit different behaviour than for the two smaller catchments. The 6 hour duration estimates are smaller as per the estimates for Somerset Dam. However, small duration estimates, (ie from 6 hours up to 24 hours) are larger by up to 20 % and the longer duration estimates are all smaller in magnitude by up to 15 %.

The IFD data derived from Australian Rainfall and Runoff, (1987), produce larger depth estimates for corresponding ARI events than the estimates that were derived from individual station analyses as determined by Weeks, (1983, 1984). The mean catchment rainfalls derived from the various methods differ by between 10 to 20 %.

Another major difference between the higher probability rainfalls are the associated temporal patterns. Weeks utilised the temporal patterns associated with the PMP estimates provided by the Bureau of Meteorology, whereas the present study used the patterns presented in the 1987 edition of Australian Rainfall and Runoff. The time distribution of corresponding temporal patterns are dissimilar as the Australian Rainfall and Runoff, (1987), patterns tend to be multi-burst in nature whereas the PMP patterns are more uniform.

## 8.2 SOMERSET DAM

The catchment of Somerset Dam in its natural state, (ie. without the dam), is estimated to have a critical duration of 24 hours. The estimated magnitude of the PMF at this site is of the order of 12 180 $m^3$/s. By comparison, the critical duration of the same catchment with Somerset Dam in place and operating in accordance with existing normal gate operation procedures is 120 hours. The peak outflow from the dam for the PMF event of this duration is 8 140 $m^3$/s. The mitigation effect of the dam for this extreme event is obvious from this comparison. Comparisons between the 100 Year ARI events shows that the peak flow at Somerset Dam reduces from 3 220 $m^3$/s for a 24 hour duration event for the catchment in its natural state to 2 550 $m^3$/s for a 36 hour duration event post-dam.

The design floods estimated for Somerset Dam from the runoff-routing and storage routing modelling are quite similar to the estimates derived by Weeks, (1983), despite the differences in rainfall estimates and runoff-routing model parameters. It should be noted though that the annual exceedance probability of the PMF is different from that of the previous study. The current PMF has an estimated ARI of 1 in 1 000 000 years.

A comparison between the corresponding 48 hour duration design floods for Somerset Dam for various ARIs between 100 years and the PMF, shows that the estimated flood volumes have all increased as is expected because of the increased rainfalls. The peak inflows have decreased by a small amount, as have the associated outflows from the dam. This would be mainly due to the different runoff-routing model parameters adopted in the two studies, although the temporal variation of the estimated runoff would also account for some of the difference.

It would appear from the design flood assessment that under the current storage operation procedure, assuming normal operation, the spillway of Somerset Dam is capable of passing the 20 000 year ARI event without overtopping of the non-overflow embankment level. The rainfall depth of this event is equivalent to 67 % of the PMP. The PMF will overtop the non-overflow embankment by some 2.85 metres, however the dam is structurally sound and is capable of withstanding being overtopped by this amount, (refer Russo, 1988). The magnitude of the IFF of Somerset Dam is therefore greater than the magnitude of the PMF.

Direct comparisons between design flood estimates derived from the runoff-routing modelling techniques outlined in this report, and flood frequency analyses are of limited value, because of the short length of homogeneous record on which to base the flood frequency determinations. However, a summary of the flood frequency analyses that have been undertaken for this study can be found in Appendix B of this report. It was concluded that little credence can be assigned to the results of the flood frequency analyses especially in regard to the more extreme design floods, because of the short length of record.

## 8.3 WIVENHOE DAM

The catchment of Wivenhoe Dam in its natural state, is estimated to have a critical duration of around 24 hours. The PMF at this site is estimated to be 39 090 $m^3/s$, whereas the 100 Year ARI event is estimated to have a peak discharge of over 10 670 $m^3/s$. Comparing estimates of peak discharge for outflows from Wivenhoe Dam under existing normal gate operation procedures for corresponding events reveals that the critical duration for the PMF event increases to 48 hours and produces a peak outflow from the dam of 25 040 $m^3/s$, whilst the peak outflow for the 100 Year ARI event is
6 810 $m^3/s$ for a 72 hour duration storm.

The design floods estimated for Wivenhoe Dam are different from the those obtained by Weeks. A comparison between the corresponding 48 hour duration design floods for various ARIs between 100 years and the PMF shows that the flood volumes are all less as is expected because of the decreased rainfalls. The peak inflow has also been reduced substantially for the PMF event, but for higher probability of exceedance events, the peak inflows have increased. The differences in estimates between the two studies are due largely to the differences in design rainfall estimates and temporal patterns and also the differing runoff-routing model model parameters.

Based upon the design flood reassessment, the IFF of Wivenhoe Dam, under current normal storage operation procedures, has an ARI of 14 300 years. The rainfall depth associated with the IFF is equivalent to 75 % of the PMP. The PMF will overtop the main embankment crest level by 2.13 metres. Embankment dams, like Wivenhoe Dam, are likely to fail under these circumstances.

ANCOLD guidelines recommend that for dams classified in the high incremental flood hazard category, the annual exceedance probability of the Recommended Design Flood, (RDF), should be between the PMF and 1 in 10 000.

The design flood modelling with the temporal patterns provided in the 1987 edition of Australian Rainfall and Runoff illustrated some shortfalls in the storage routing model and the existing storage operation procedures. The multi-peaked hydrographs derived for some of the higher probability of exceedance events caused the storage routing model to make releases from the storages which were not consistent with the existing operating policies. The model was modified so as to overcome these problems, but it became evident during the modelling that there is scope for further investigation into the storage operation procedures and that the storage operation model needs to be refined.

## 9.0 REFERENCES

Ayre, R.A., Cutler, P., and Ruffini, J.L., (1991), 'Pine River Flood Hydrology Report'. Water Resources Commission, QDPI, Hydrology report 14200?.PR/?, Brisbane.

Ayre, R.A., Cutler, P., and Ruffini, J.L., (1992), 'Brisbane River Flood Hydrology Runoff-Routing Model Calibration'. Water Resources Commission, QDPI, Hydrology Report 14300?.PR/?, Brisbane.

Brisbane and Area Water Board, (1985), 'Manual of Operational Procedures for Flood Mitigation for Wivenhoe Dam and Somerset Dam'. Brisbane and Area Water Board, Brisbane.

Bureau of Meteorology, (1985), 'The Estimation of Probable Maximum Precipitation in Australia for Short Durations and Small Areas'. Bureau of Meteorology, Canberra.

Bureau of Meteorology, (1991), 'Probable Maximum Precipitation Studies for the Brisbane River Catchment, and Sub-Catchments of the Brisbane River System'. Queensland Regional Office, Brisbane.

Cantorford, R.P., (1988), 'Intensity-Frequency-Duration Design Rainfall Program, Version 2.0'. Bureau of Meteorology, Canberra.

Cossins, G., (1969), 'Hydrology without Tears and Other Stories'. Queensland Division Technical Papers, I.E.Aust, Volume 10 Number 7, Brisbane.

Cossins, G., (1988), 'Report to the Brisbane and Area Water Board on Safety of the Board's Dams'. Brisbane and Area Water Board, Brisbane.

Hadgraft, R., (1981), 'Computer Program WS06 Frequency Analysis Documentation and Users Manual'. Water Resources Commission, QDPI, Brisbane.

Hausler, G., and Porter, N., (1977), 'Report on the Hydrology of Wivenhoe Dam'. Water Resources Commission, QDPI, Hydrology Report 143005.PR, Brisbane.

Hegerty, K.L., and Weeks, W.D., (1985), 'Hydrology Report for Manual of Operational Procedures for Flood Mitigation for Wivenhoe Dam and Somerset Dam'. Brisbane and Area Water Board, Brisbane.

Hydsys, (1992), ' HYDSYS Time Series Data Management Users Manual Release 4.0'. Hydsys Pty Ltd, Canberra.

Institution of Engineers, Australia., (1977), 'Australian Rainfall and Runoff, Flood Analysis and Design'. I.E.Aust., Canberra.

Institution of Engineers, Australia., (1987), ' Australian Rainfall and Runoff, A Guide to Flood Estimation'. I.E.Aust., Canberra.

Mein, R.G., Laurenson E.M., and McMahon, T.A., (1974), 'Simple Non-linear Method for Flood Estimation'., J.Hyd.Div, ASCE, Vol 100, No. HY11, pp 1507-1518.

Nittim, R., (1989), 'Areal Distribution of Design Rainfall'. Hydrology and Water Resources Symposium, Christchurch, New Zealand.

Raudkivi, A, J., (1979), 'Hydrology: An Advanced Introduction to Hydrological Processes and Modelling.' Permagon Press, Oxford.

Russo, R., (1988), 'Safety Review of Somerset Dam'. Brisbane City Council, Brisbane.

Ruffini, J.L., (1990), 'Tempat Users Guide'. Water Resources Commission, QDPI, Brisbane.

Shallcross, W., (1987), 'Flood Estimation by Runoff-routing Program WT42'. Water Resources Commission, QDPI, Brisbane.

Weeks, W.D., (1983), 'Wivenhoe Dam Design Flood Report'. Water Resources Commission, QDPI, Hydrology Report 143005.PR/3, Brisbane.

Weeks, W.D., (1984), 'Wivenhoe Dam Report on Downstream Flooding'. Water Resources Commission, QDPI, Hydrology Report 143005.PR/4, Brisbane.