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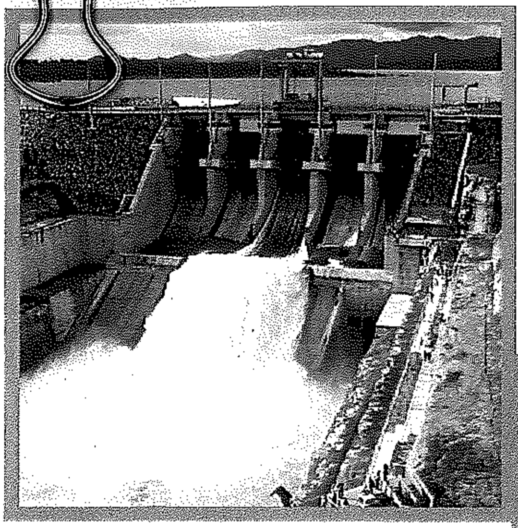
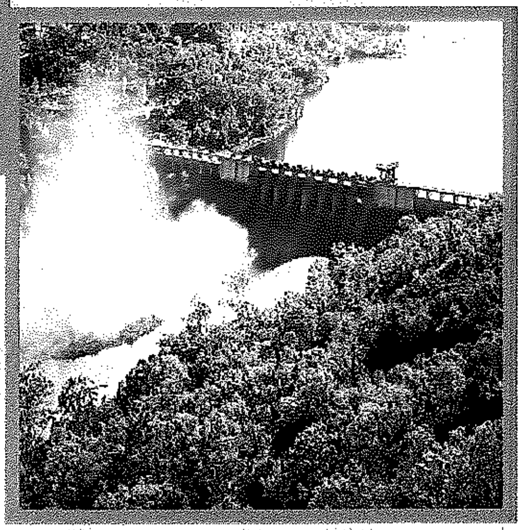
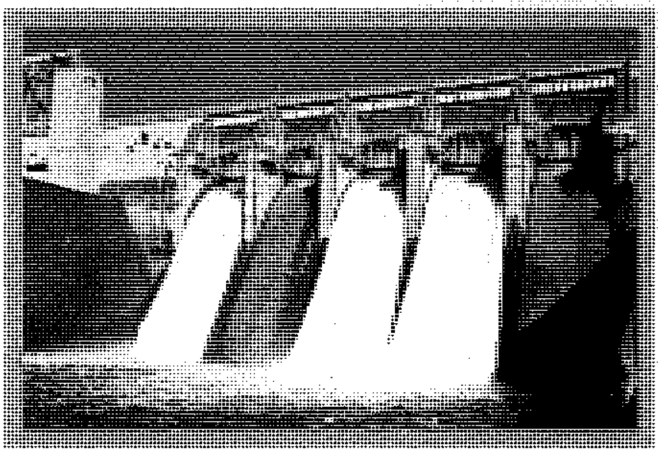
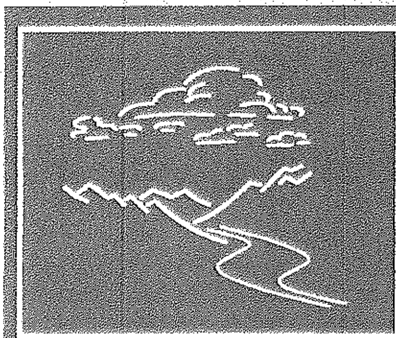
BRISBANE RIVER AND PINE RIVER FLOOD STUDY:









# EXECUTIVE SUMMARY REPORT

JM

QFCI

Date: 16/05/11

Exhibit Number: 401

South East Queensland WATER BOARD

NATURAL RESOURCES Queensland

Brisbane River and Pine River
Flood Studies

# Brisbane River
# Flood Hydrology

Report on

# Downstream Flooding Estimation

August 1993

BRISBANE RIVER DOWNSTREAM FLOODING REPORT

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## 1.0 INTRODUCTION

A review of the performance of Somerset Dam and Wivenhoe Dam during floods, was commissioned by the South East Queensland Water Board, (SEQWB), with the study being undertaken by the Water Resources Business Group of the Department of Primary Industries, (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 flooding downstream of 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 exceedence events have also been re-derived using rainfall intensity-frequency-duration procedures outlined in Australian Rainfall and Runoff, (1987).

This report is supplemental to the report entitled, 'Interim Report on Design Flood Estimation', (1993), and it should be read in conjunction with it. The earlier report details findings of investigations into design floods associated with Somerset Dam and Wivenhoe Dam.

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 flood outflows from the storages and the resultant flooding in the catchments situated downstream of the dams.

A summary of the estimates of the magnitude of flooding that will occur as a consequence of the operation of the storages in accordance with the existing normal gate operation procedures is presented.

This reassessment provides an indication as to the flood mitigation capability of the storages under existing flood operation procedures. These results also provide a reference which may be used as a basis for the comparison of alternative storage operating procedures that may be investigated in the future.

## 2.0 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 2.1.

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.

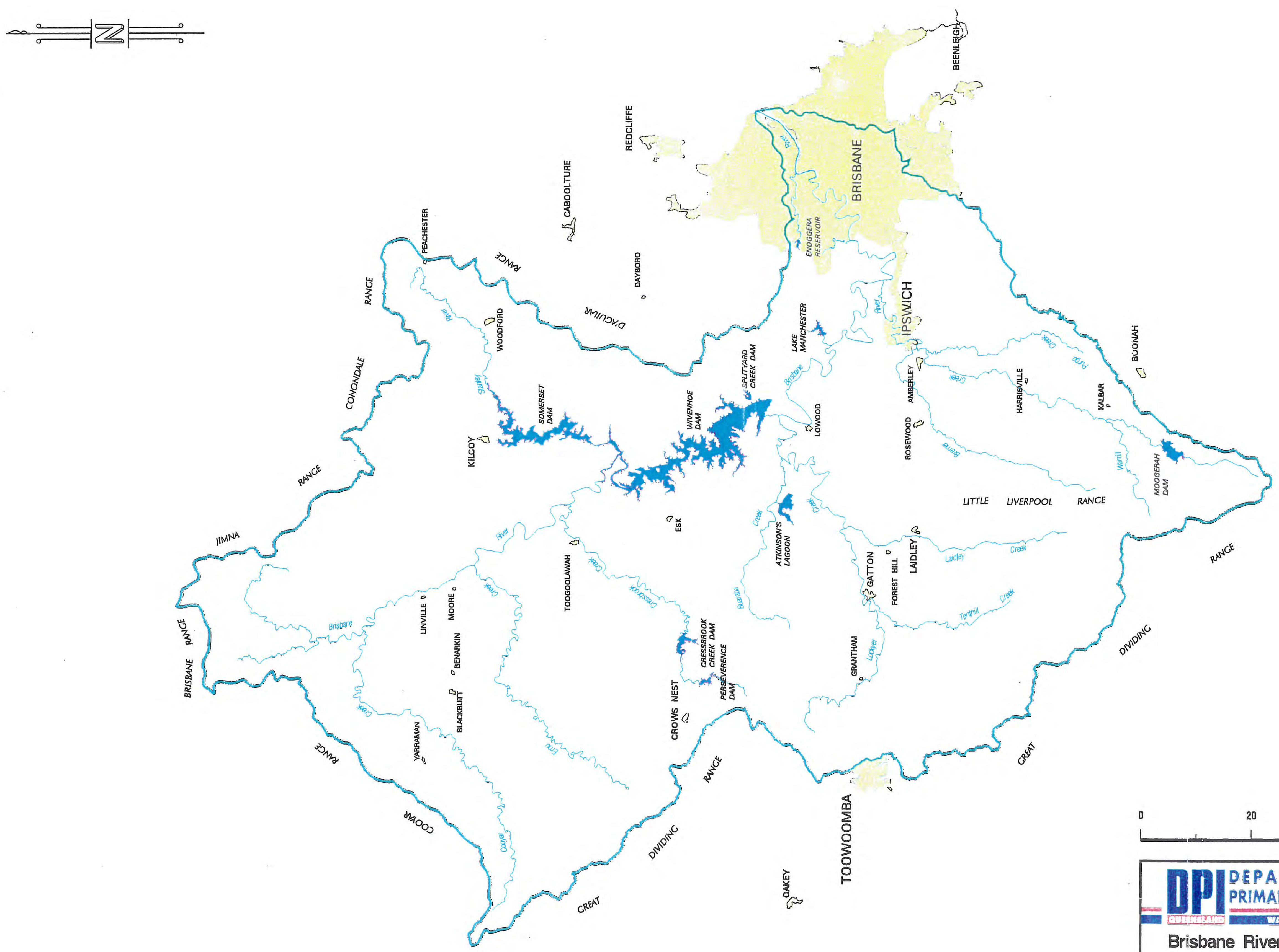
Splityard Creek Dam is located on Pryde Creek and it has a capacity of 28 600 ML and a catchment area of only 3.6 $km^2$. The dam is the upper storage of a pumped storage scheme that is owned and operated by the Queensland Electricity Commission, (QEC).

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 2.1

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DEPARTMENT OF PRIMARY INDUSTRIES
QUEENSLAND WATER RESOURCES

Brisbane River Flood Study

## Brisbane River Locality Plan

24 Aug 93

A3-89454

# 3.0 PREVIOUS FLOOD STUDIES

## 3.1 Introduction

The flood hydrology of the Brisbane River downstream of Wivenhoe Dam and Somerset Dam has been investigated on a previous occasion by the Water Resources Commission, (WRC), and the Brisbane City Council, (BCC). The assessment of downstream flooding was performed as part of the estimation procedure to determine the effects of Wivenhoe Dam on downstream flooding in 1984. Alternative operating rules for the storages were also investigated by the BCC during this study before appropriate operation procedures were adopted.

The 1984 study was the continuation of work performed for the estimation of design floods of the dams conducted in 1983, (Weeks). Since it followed on from the earlier work the same modelling methodology was adopted.

A runoff-routing model of the whole of the Brisbane River catchment below Wivenhoe Dam was developed and calibrated against up to seven historic events. Five different locations within the modelled catchment were considered in the calibration. Model parameters of $k = 270$ and $m = 0.75$ were derived during the calibration process.

The outcomes of this study are summarised in the following sections.

## 3.2 Design Rainfall Estimation

The Bureau of Meteorology provided estimates of the probable maximum precipitation for the catchments under investigation in 1983. These estimates included PMP values for the whole of the catchment of the Brisbane River and estimates for general rainfall that could be expected to accompany a PMP event centred over the catchments of Wivenhoe Dam and Somerset Dam. These values are summarised in Table 3.1.

The method of adjusted United States data, (Brunt, 1958), was used for the six hour duration storm and the generalised method for areas subject to tropical cyclones, (Kennedy, 1982), for the one day event. The twelve hour, two day and three day rainfalls were all derived from the one day estimate. General meteorological considerations were used to derive the design rainfalls for the four, five, six and seven day events.

The same general meteorological considerations were assumed to apply to the rainfall that could be expected to fall concurrently with the PMP event. Temporal patterns that were provided by the Bureau of Meteorology for the 1983 study into design floods of the dams were again used for the PMP and concurrent rainfalls.

**Table 3.1**

**Probable Maximum Precipitation Estimates (mm)**
**Weeks, (1984)**

| Dur (Hours) | STORM CENTRE | | | | |
|---|---|---|---|---|---|
| | Brisbane River Above Mouth | Wivenhoe Catchment | | Somerset Catchment | |
| | | Wivenhoe Dam | Con-Current* | Somerset Dam | Con-Current* |
| 6 | 220 | 260 | 20 | 400 | 140 |
| 12 | 380 | 380 | 30 | 560 | 200 |
| 24 | 560 | 600 | 50 | 840 | 300 |
| 48 | 700 | 1 000 | 90 | 1 380 | 500 |
| 72 | 880 | 1 260 | 110 | 1 760 | 620 |
| 96 | 1 040 | 1 460 | 130 | 2 040 | 720 |
| 120 | 1 080 | 1 520 | 130 | 2 120 | 760 |
| 144 | 1 100 | 1 560 | 140 | 2 160 | 780 |
| 168 | 1 200 | 1 700 | 150 | 2 340 | 840 |

Note: * Rainfall depth over the remainder of the Brisbane River Catchment which could be expected to accompany the Probable Maximum Precipitation over the catchment of the dam.

Design rainfalls for the more frequent events considered in the study were derived by two different methods.

The WRC performed frequency analyses on twenty-one long term daily rainfall stations located within and around the Brisbane River catchment. Log-normal distributions were fitted to annual series of one, two and three day maxima rainfalls of each station. It was assumed that the rainfall for the catchment of a given probability was made up of the individual station rainfalls of that probability all occurring together.

The 100 year ARI mean catchment rainfall estimates for various sub-basins of the lower Brisbane River catchment are shown in Table 3.2.

**Table 3.2**

**100 Year ARI Mean Catchment Rainfall**
**WRC Station Data**
**Weeks, (1984)**

| Catchment | Duration (Hours) | | |
|---|---|---|---|
| | 24 | 48 | 72 |
| Wivenhoe | 217 | 298 | 347 |
| Lockyer | 162 | 222 | 255 |
| Bremer | 205 | 283 | 332 |
| Lower Brisbane | 245 | 334 | 397 |
| Downstream | 197 | 270 | 316 |

Temporal patterns provided by the Bureau of Meteorology for the PMP events were assumed to apply for all cases when the design rainfalls were used to simulate design floods.

The Brisbane City Council, (BCC), adopted a different approach to the estimation of design rainfall depths. The BCC considered frequency analyses of annual series of catchment rainfalls in contrast to the frequency analysis of individual stations.

The BCC only determined 48 hour duration rainfall depths because this was found to be the critical duration for the catchment of Wivenhoe Dam in the 1983 study.

The mean catchment rainfalls for a range of ARI events as estimated by the BCC are presented in Table 3.3.

The 100 year 48 hour duration rainfall totals estimated by the two methods were similar for the Wivenhoe Dam catchment. However, design rainfalls derived from the mean catchment data produced higher estimates of rainfall for rarer events.

The BCC analysis assumed that the rain originated in either the Stanley River catchment or the Upper Brisbane River catchment. Factors were calculated from generalised depth-area rainfall data which were then used to modify the mean catchment rainfall depth to give sub-catchment rainfall depths.

The rainfalls downstream of Wivenhoe Dam were not affected by the assumed position of the storm centre.

**Table 3.3**

**48 Hour Mean Catchment Rainfall**
**BCC Catchment Data**
**Weeks, (1984)**

| Catchment | ARI (Years) | | | |
|---|---|---|---|---|
| | 50 | 100 | 1 000 | 10 000 |
| Wivenhoe Dam | 241 | 304 | 486 | 716 |
| Somerset a | 366 | 420 | 594 | 766 |
| Somerset b | 212 | 243 | 344 | 910 |
| Upper Brisbane a | 159 | 182 | 258 | 333 |
| Upper Brisbane b | 260 | 298 | 422 | 544 |
| Middle Brisbane | 161 | 185 | 262 | 338 |
| Lockyer | 140 | 160 | 227 | 292 |
| Bremer | 113 | 130 | 184 | 237 |
| Lower Brisbane | 58 | 66 | 94 | 121 |

Where (a) Storm centred over Stanley River catchment.
(b) Storm centred over Upper Brisbane catchment.

The temporal pattern for the probable maximum precipitation for the two day duration, as supplied by the Bureau of Meteorology, was used for the analysis of all the BCC storms.

### 3.3 Design Flood Estimation

The 1984 estimates of the design floods for the Port Office Gauge, (Brisbane City), are summarised in this section. In all cases, design rainfall loss rates of 0 mm initial loss and 2.5 mm/hour continuing loss were applied.

The 1984 study produced estimates of flooding at the Port Office Gauge for a number of cases including the following:

(i) without Wivenhoe Dam.
(ii) no release from Wivenhoe Dam.
(iii) normal operation of Somerset Dam and Wivenhoe Dam.

Only 48 hour critical duration events associated with WRC derived design rainfalls are presented in Table 3.4.

**Table 3.4**

**Design Flood Estimates at Port Office Gauge**
**WRC 48 Hour Design Rainfalls**
**Weeks, (1984)**

| **WITHOUT WIVENHOE DAM** | | | |
|---|---|---|---|
| ARI (Years) | Duration (Hours) | Peak Discharge ($m^3/s$) | Flood Volume (ML) |
| 100 | 48 | 11 500 | 2 160 000 |
| 1 000 | 48 | 19 400 | 3 590 000 |
| 10 000 | 48 | 29 600 | 5 270 000 |
| PMF | 48 | 49 400 | 7 787 000 |
| PMF | 144b | 54 400 | 10 610 000 |

Note: (b) Refers to various temporal patterns provided by the Bureau of Meteorology.

| **NO RELEASE FROM WIVENHOE DAM** | | | |
|---|---|---|---|
| ARI (Years) | Duration (Hours) | Peak Discharge ($m^3/s$) | Flood Volume (ML) |
| 100 | 48 | 4 480 | 1 000 000 |
| 1 000 | 48 | 8 520 | 1 690 000 |
| 10 000 | 48 | 13 400 | 2 450 000 |

| NORMAL OPERATION OF SOMERSET AND WIVENHOE DAM | | | |
|---|---|---|---|
| ARI (Years) | Duration (Hours) | Peak Discharge ($m^3/s$) | Flood Volume (ML) |
| 100 | 48 | 5 510 | 2 720 000 |
| 1 000 | 72 | 11 400 | 4 620 000 |
| 10 000 | 72 | 19 100 | 6 030 000 |

The situation with no releases permitted from Wivenhoe Dam was only investigated to study the effect of the dam on downstream flooding. It is not feasible to regard this operating procedure as practicable since the dam would not be capable of holding the large volumes of water produced by the low probability of exceedence flood events.

The BCC routed the inflows for its design floods through the dams using alternative gate operating procedures for Somerset Dam and Wivenhoe Dam. The outflows from Wivenhoe Dam were then routed through the downstream catchment runoff-routing model in order to estimate the impacts of flooding due to the alternative design flood rainfalls. Flow contributions from these catchments were calculated from the storm rainfall depths shown in Table 3.4. Only 48 hour duration rainfall events were modelled.

The results of the BCC estimates are presented in Table 3.5.

**Table 3.5**

**Design Flood Estimates at Port Office Gauge**
**BCC 48 Hour Design Rainfalls**
**Weeks, (1984)**

| NO RELEASE FROM WIVENHOE DAM | | | |
|---|---|---|---|
| ARI (Years) | Operating Rules | | Peak Discharge ($m^3/s$) |
| | Somerset Dam | Wivenhoe Dam | |
| 50 | 5 | - | 410 |
| 100 | 5 | - | 800 |
| 1 000 | 5 | - | 2 670 |
| 10 000 | 5 | - | 4 870 |
| PMF | 5 | - | 10 470 |

| NORMAL OPERATION OF SOMERSET AND WIVENHOE DAM (Storm Centre - Stanley River) | | | |
|---|---|---|---|
| ARI (Years) | Operating Rules | | Peak Discharge ($m^3/s$) |
| | Somerset Dam | Wivenhoe Dam | |
| 50 | 5<br>5 | 2<br>2 | 1 660<br>*1 660 |
| 100 | 5<br>5 | 4<br>4 | 3 440<br>*3 580 |
| 1 000 | 5<br>5 | 4<br>4 | 6 780<br>*4 020 |
| 10 000 | 5<br>5 | 4<br>5 | 11 780<br>*9 700 |

| NORMAL OPERATION OF SOMERSET AND WIVENHOE DAM (Storm Centre - Upper Brisbane River) | | | |
|---|---|---|---|
| ARI (Years) | Operating Rules | | Peak Discharge ($m^3/s$) |
| | Somerset Dam | Wivenhoe Dam | |
| 50 | 5<br>5 | 4<br>4 | 3 540<br>*3 090 |
| 100 | 5<br>5 | 4<br>4 | 3 560<br>*3 580 |
| 1 000 | 5<br>5 | 4<br>5 | 8 430<br>*9 100 |
| 10 000 | 5<br>5 | 4<br>5 | 14 820<br>*13 750 |

Note: * Indicates revised BCC design rainfall estimate.

Somerset Dam operation procedure 5 was later adopted as the most appropriate operating procedure. It is described in the design flood estimation report and later in the 1985 version of the flood operations manual as follows:

'Raise Somerset Dam spillway gates to permit uncontrolled discharge over the spillway once the flood storage between full supply level and spillway crest level has filled. Low level outlets remain closed until the inflow into Wivenhoe Dam has peaked or the level in Somerset Dam exceeds EL 102.2 metres.'

The operating procedures for Wivenhoe Dam all begin by storing all inflow until the lake level exceeds El 67.25 metres. Operating procedures 4 and 5 as stated in the design flood estimation report is as follows:

Procedure 4

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

Procedure 5

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

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

After some minor adjustments, such as increasing the 'control' lake level in Wivenhoe Dam to EL 74.0 m AHD, procedures 4 and 5 were later adopted as appropriate operating procedures for Wivenhoe Dam. They were also renumbered to become operating procedures 3 and 4, respectively, in the manual of flood operation for Somerset Dam and Wivenhoe Dam, (1985).

Procedures 3 and 4 have been subject to some minor adjustments and rewording in the 1992 update of the manual for flood operation for Somerset Dam and Wivenhoe Dam, however the operating philosophy is essentially unchanged. The major changes in the operating procedures for Wivenhoe Dam contained within the 1992 update involve Procedures 1 and 2, which are not used in conjunction with extreme flood events.

The changes incorporated in the 1992 update have not been included in the present modelling as the investigation of appropriate flood operation procedures for the dams is the subject of ongoing work.

Preliminary dambreak hydrographs, associated with the possible failure of Wivenhoe Dam due to overtopping during an extreme flood event, were also routed downstream using the runoff-routing model. Nine floods were investigated which accounted for different failure conditions of the 48 hour and 168 hour duration probable maximum precipitation storms centred over the Wivenhoe Dam catchment.

The peak discharge of the inflow hydrograph for the 48 hour duration storm was 43 000 $m^3/s$, whilst the 168 hour duration event had a peak of 48 000 $m^3/s$. Estimates of peak outflow from Wivenhoe Dam and the peak discharge at the Port Office Gauge are summarised in Table 3.6.

**Table 3.6**

**Peak Discharge Estimates - Wivenhoe Dam Dambreak Weeks, (1984)**

| Case | Duration (Hours) | **Peak Discharge ($m^3/s$)** | |
|---|---|---|---|
| | | Wivenhoe Dam | Port Office |
| 1A-2 | 48 | 61 360 | 51 800 |
| 1B-2 | 168 | 61 630 | 53 300 |
| 2A-2 | 48 | 114 810 | 79 700 |
| 2B-2 | 168 | 114 900 | 83 800 |
| 3A-2 | 48 | 164 950 | 101 700 |
| 3B-2 | 168 | 165 310 | 107 000 |
| 4A-2 | 48 | 213 550 | 120 600 |
| 4B-2 | 168 | 213 940 | 112 700 |
| 6A-2 | 48 | 70 430 | 53 600 |

Weeks indicated that these estimates should be treated with caution because the magnitude of these events is far in excess of any calibration event.

# 4.0 DESIGN FLOOD ESTIMATION TECHNIQUES

## 4.1 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 and the resultant downstream flooding. 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.

## 4.2 Runoff-Routing Modelling

### 4.2.1 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.

### 4.2.2 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 4.1.

The connection between the sub-catchment models is shown in the key plan presented in Figure 4.1. The sub-area layout and characteristics of each of the sub-catchment models is presented in Appendix D, Volume IV of the 'Interim Report on Design Flood Estimation', (1993). 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.

**Table 4.1**

**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 Creek | 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 | 339 | 36.9 |

### 4.2.3 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 4.2. A value of m = 0.8 was adopted for all models which is in keeping with recommendations in Australian Rainfall and Runoff, (1987). For a more detailed discussion of the selection of the adopted runoff-routing model parameters reference is made to the calibration report.

**Table 4.2**

**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 | 37.2 |
| GRE | Brisbane R @ Gregors Creek | 20.1 |
| 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 | 45.0 |
| 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 | 19.3 |

## 4.3 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 especially developed by Hegerty and Weeks, (1985), to enable the routing of floods through Wivenhoe Dam and Somerset Dam using their 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 below:

**Somerset Dam**

**Type of dam**

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

**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 |
| Control Flood Level | 102.25 |
| Top of Closed Spillway Gates | 107.46 |
| Non-overflow Spillway 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:**

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

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, which was calculated in June 1952 and provided by the BCC, and the curve derived from the equation used in '2DAMA' is presented in Figure 4.2. The maximum discrepancy between the two curves is some 150 mm. The discharge relationships for the total flow through the 4 regulator valves, 8 sluice gates, 8 spillway gates and flow over the non-overflow section of the spillway is shown in Figure 4.3.

__**Wivenhoe Dam**__

**Type of dam**

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

**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 | 33.00 |
| Spillway Crest Level | 57.00 |
| Full Supply Level | 67.00 |
| Gate Opening Level | 67.25 |
| Gate Closure 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:**

$$\text{DMAX} = (106.1 + 1.308*(\text{WLEVEL}-57.0))*(\text{WLEVEL}-57.0)^{1.5}$$

Where

WLEVEL = Water level in Wivenhoe Dam (m AHD).
WVOL = Storage volume in Wivenhoe Dam ($m^3/s$).
QWWALL = Discharge over embankment crest (& wave wall) ($m^3/s$).

DMAX = Discharge through uncontrolled spillway ($m^3/s$).

Figure 4.4 illustrates the comparison between the actual storage capacity curve for Wivenhoe Dam and the curve derived from the equation in '2DAMA'. The actual curve is based on WR drawing A3-44067 which was redrawn in May 1985. The equation produces a curve which is reasonably similar in the range above full supply level.

The discharge relationships used in '2DAMA' for Wivenhoe Dam are presented in Figure 4.5.

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. The model assumes that the discharge from Wivenhoe Dam can be achieved subject to the constraints of the gate opening and closing limitations. Therefore, the required outflow from Wivenhoe Dam is determined without reference to actual gate openings and closures, except for when the spillway gates have been lifted clear.

For the purpose of determining design flood estimates for the storages the storage routing model has been modified. These modifications 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 as follows:

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.

As mentioned in Section 3.3, Procedures 3 and 4 have been subject to some minor adjustments and rewording in the 1992 update of the manual for flood operation for Somerset Dam and Wivenhoe Dam, however the operating philosophy is essentially unchanged. The major changes in the 1992 update, involve Operating Procedures 1 and 2 for Wivenhoe Dam, which are not used in conjunction with extreme flood events. The changes to Procedures 1 and 2 are intended to better reflect the flood operation of the dams in response to small to large freshes.

The changes incorporated in the 1992 update have not been included in the present modelling as the investigation of appropriate flood operation procedures for the dams is the subject of ongoing work.

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
Brisbane River @ Moggill

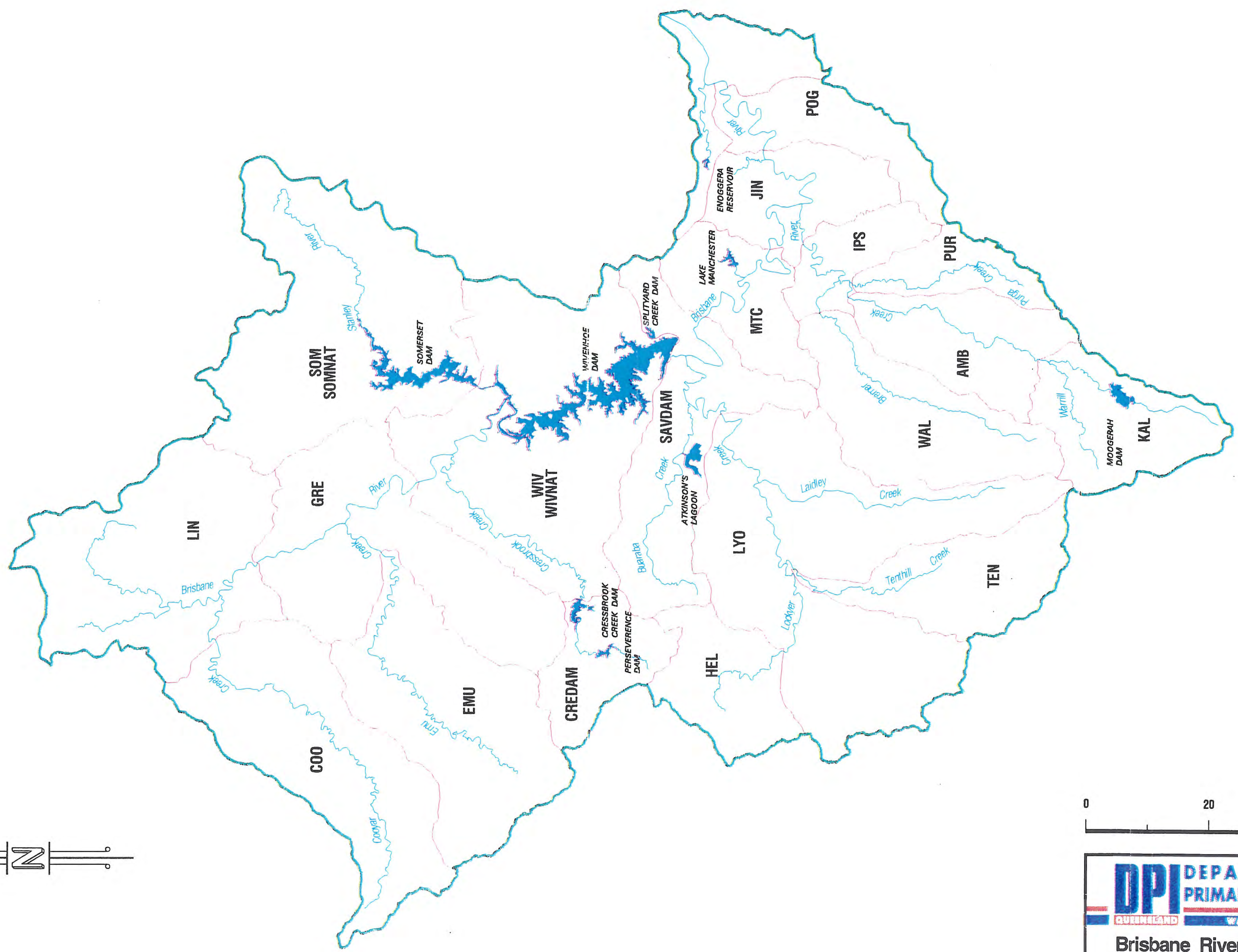
The decision process to determine which of the four operation procedures is most appropriate for Wivenhoe Dam has been incorporated into a policy selection module of the modified storage operation model. This decision process is based upon Table C1 in Appendix C of the manual of operational procedures,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

FIGURE 4.1

**LEGEND**

CATCHMENT BOUNDARY

SUB-CATCHMENT BOUNDARY




KILOMETRES

0 20 40 60 75

**DPI** DEPARTMENT OF PRIMARY INDUSTRIES

QUEENSLAND WATER RESOURCES

Brisbane River Flood Study

**Brisbane River**
**Design Model Key Plan**

24 Aug 93

A3-101427

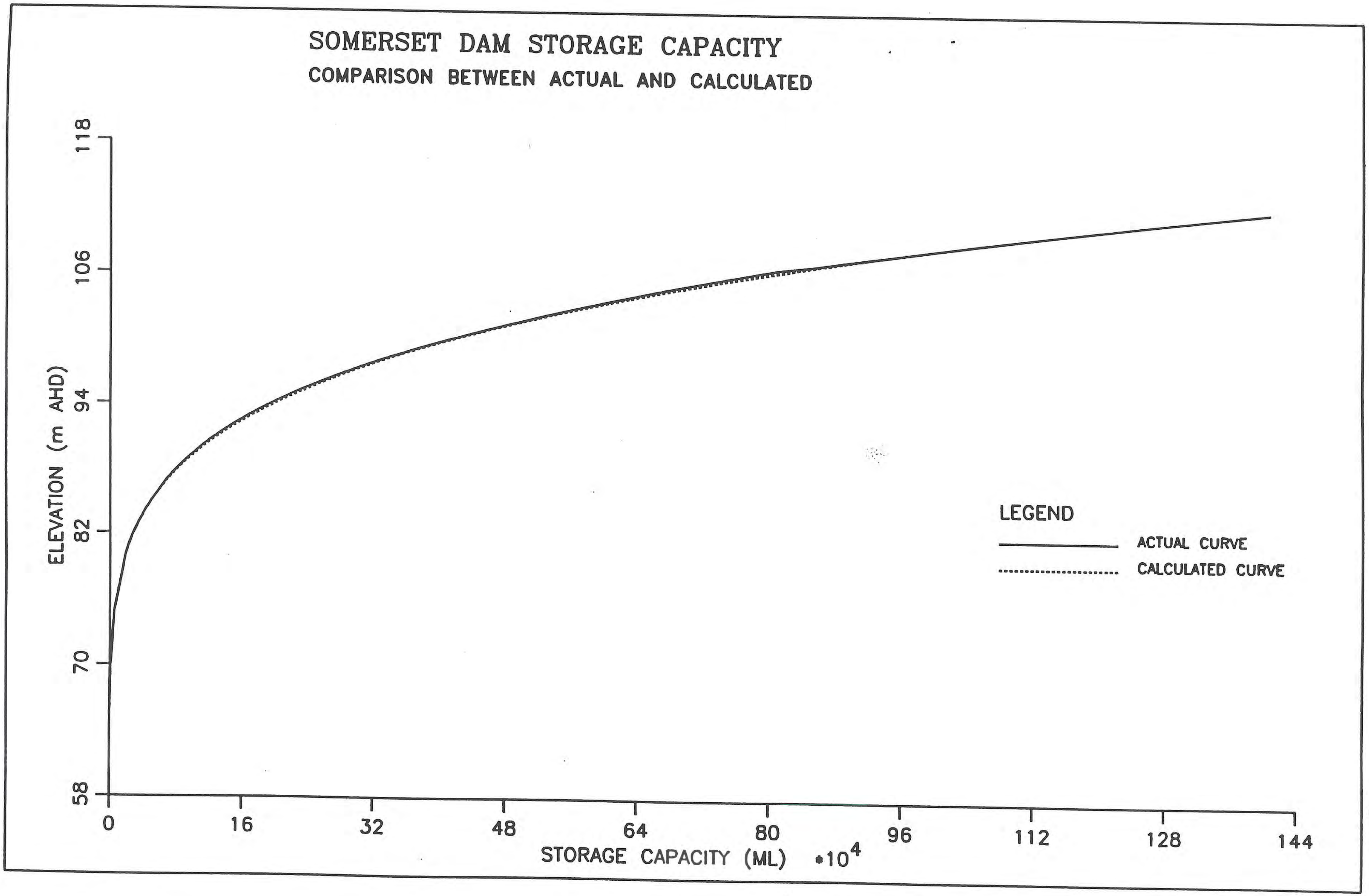

SOMERSET DAM STORAGE CAPACITY
COMPARISON BETWEEN ACTUAL AND CALCULATED
ELEVATION (m AHD)
118
106
94
82
70
58
0
16
32
48
64
80
96
112
128
144
STORAGE CAPACITY (ML) *10^4
LEGEND
ACTUAL CURVE
CALCULATED CURVE

Figure 4.2

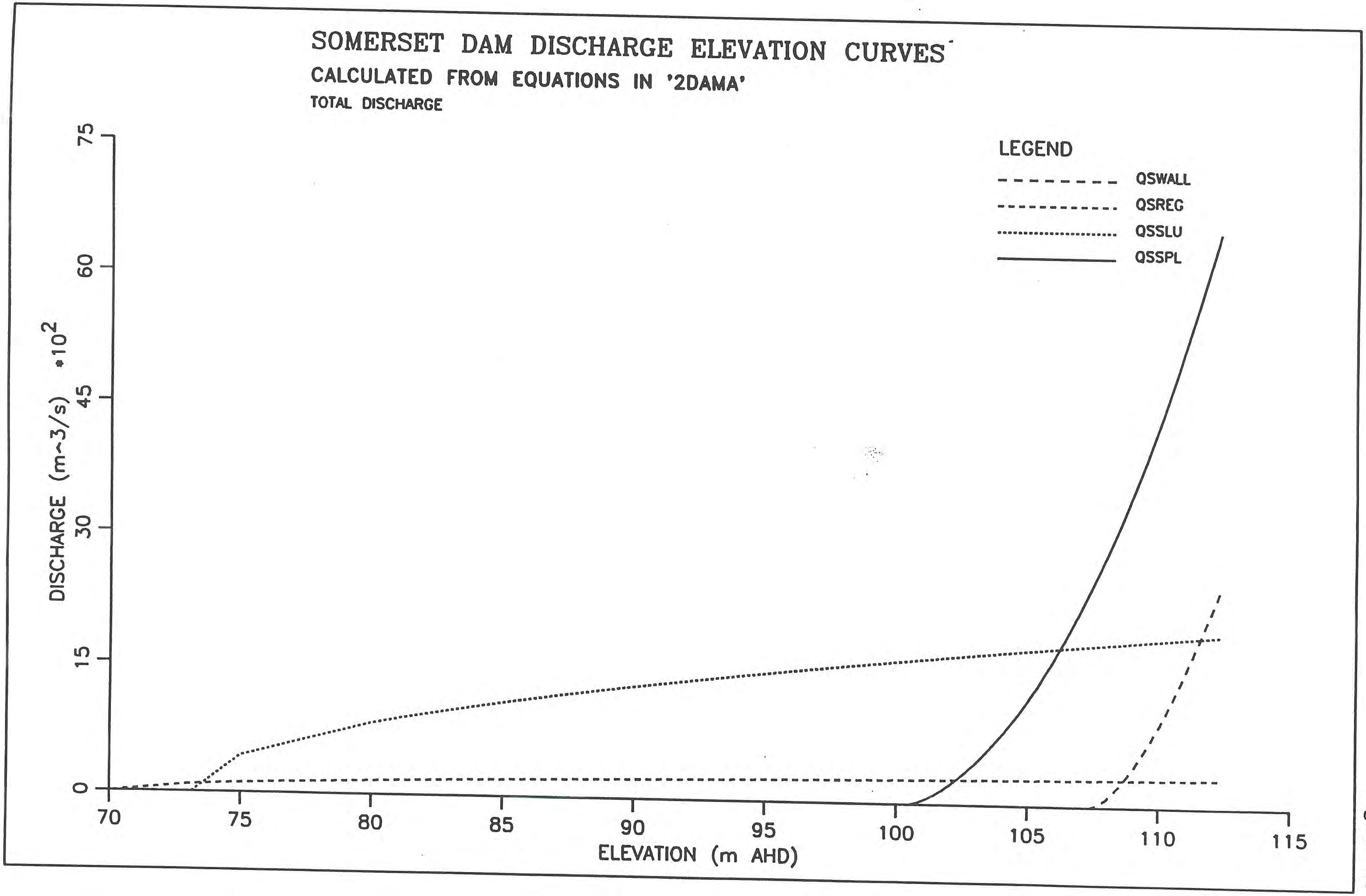

SOMERSET DAM DISCHARGE ELEVATION CURVES
CALCULATED FROM EQUATIONS IN '2DAMA'
TOTAL DISCHARGE
LEGEND
QSWALL
QSREG
QSSLU
QSSPL
DISCHARGE (m~3/s) *10^2
ELEVATION (m AHD)
0
15
30
45
60
75
70
75
80
85
90
95
100
105
110
115

Figure 4.3

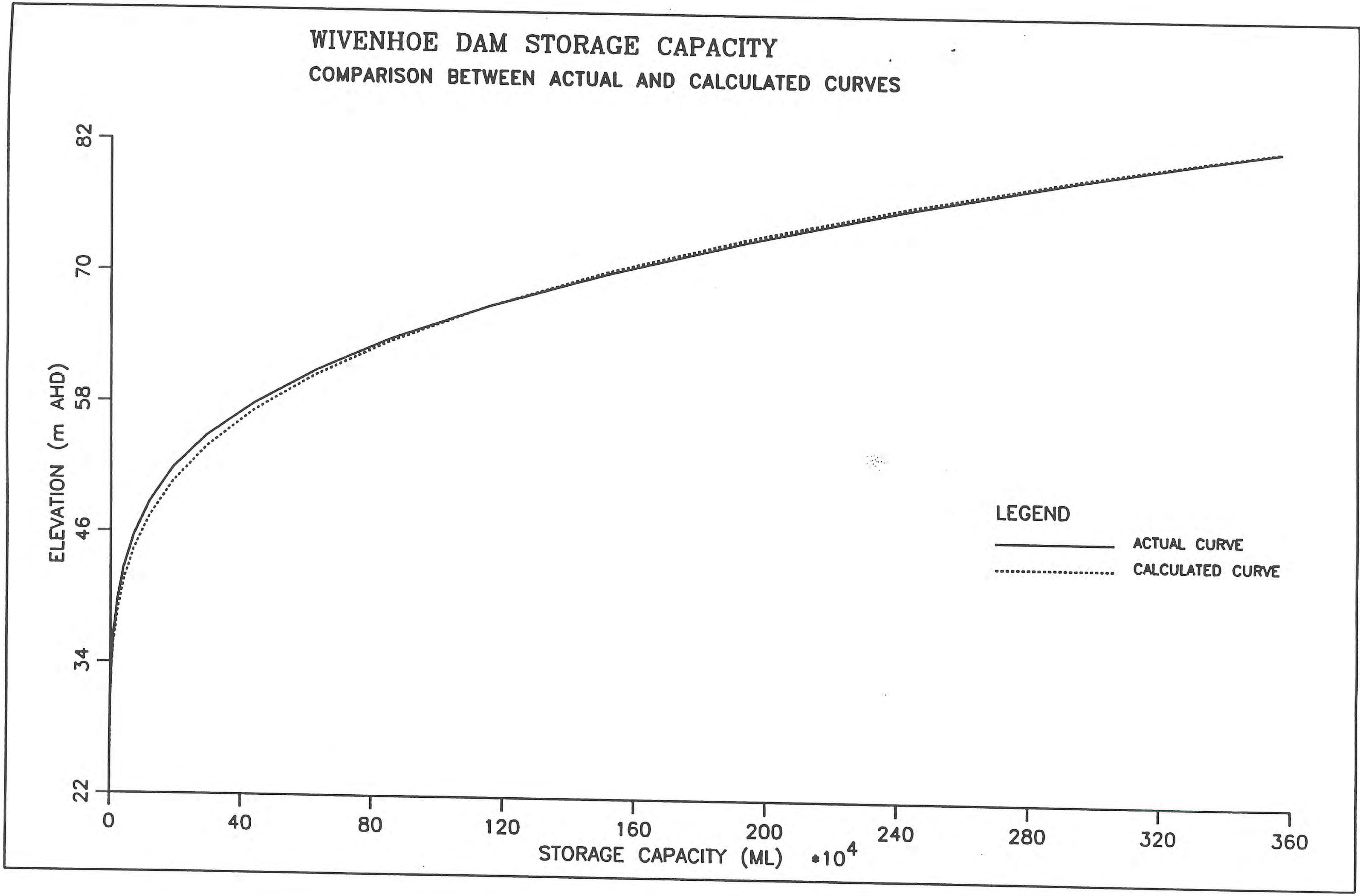

WIVENHOE DAM STORAGE CAPACITY
COMPARISON BETWEEN ACTUAL AND CALCULATED CURVES
ELEVATION (m AHD)
82
70
58
46
34
22
0
40
80
120
160
200
240
280
320
360
STORAGE CAPACITY (ML) *10^4
LEGEND
ACTUAL CURVE
CALCULATED CURVE

Figure 4.4

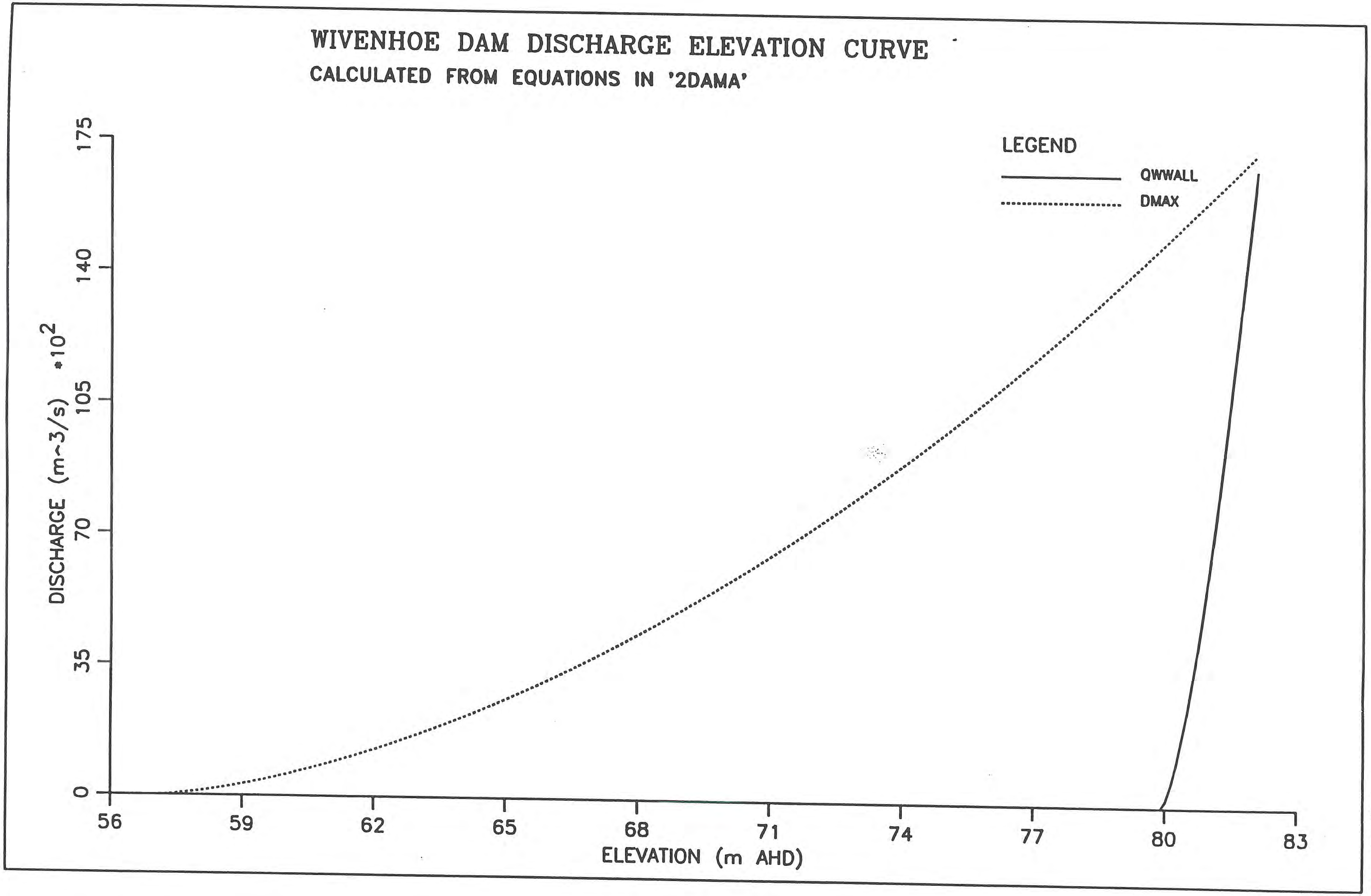

WIVENHOE DAM DISCHARGE ELEVATION CURVE
CALCULATED FROM EQUATIONS IN '2DAMA'
LEGEND
QWWALL
DMAX
DISCHARGE (m~3/s) *10^2
175
140
105
70
35
0
56
59
62
65
68
71
74
77
80
83
ELEVATION (m AHD)

Figure 4.5

# 5.0 DESIGN RAINFALL DATA

## 5.1 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.

## 5.2 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 reproduced in Appendix A, Volume II of the 'Interim Report on Design Flood Estimation', (1993), for further information. These catchments include:

**Table 5.1**

**Locations of PMP Estimates**

| | 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 5.1 and 5.2 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 1000 $km^2$ 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 categorised as 'smooth', the PMP estimates 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 four of the catchments mentioned earlier are provided in Table 5.2. 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 5.2**

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

| Duration (Hours) | Catchment | | | |
|---|---|---|---|---|
| | Lockyer (6) | Bremer (7) | Lower Brisbane (8) | Brisbane (5) |
| 6 | 300 | 340 | 390 | 200 |
| 12 | 540 | 610 | 660 | 370 |
| 24 | 800 | 880 | 950 | 530 |
| 48 | 1 110 | 1 260 | 1 380 | 680 |
| 72 | 1 380 | 1 580 | 1 730 | 830 |
| 96 | 1 590 | 1 830 | 2 010 | 1 010 |
| 120 | 1 650 | 1 900 | 2 080 | 1 050 |
| 144 | 1 690 | 1 940 | 2 140 | 1 070 |
| 168 | 1 890 | 2 140 | 2 350 | 1 160 |
| ARI (Years) | $10^6$ | $10^6$ | $10^6$ | $10^5$ |

Where ARI = Average Recurrence Interval
(6) = Catchment Number in Table 5.1

### 5.3 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 5.3 and shown in Figure 5.3. These sites roughly correspond to the locations of daily rainfall stations that were used in the calibration of the runoff-routing models.

**Table 5.3**

**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 the procedure was almost completely automated.

The adopted design storm rainfall depths for various Average Recurrence Intervals, (ARI), are summarised in Tables 5.4 to 5.7. Note all values of rainfall depth are millimetres.

**Table 5.4**
**Catchment Estimates of 100 Year ARI Rainfalls**
**(mm Depth)**

| Duration (Hours) | Catchment | | | |
|---|---|---|---|---|
| | Lockyer (6) | Bremer (7) | Lower Brisbane (8) | Brisbane (5) |
| 12 | 173 | 175 | 228 | 193 |
| 24 | 223 | 238 | 299 | 255 |
| 36 | 254 | 278 | 347 | 296 |
| 48 | 282 | 307 | 383 | 328 |
| 60 | 301 | 330 | 411 | 352 |
| 72 | 315 | 348 | 432 | 370 |

Note: (6) Numbers in brackets refer to Catchment Description given in Table 5.1

**Table 5.5**

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

| Duration (Hours) | Catchment | | | |
|---|---|---|---|---|
| | Lockyer (6) | Bremer (7) | Lower Brisbane (8) | Brisbane (5) |
| 12 | 154 | 161 | 201 | 172 |
| 24 | 197 | 211 | 263 | 225 |
| 36 | 227 | 245 | 306 | 260 |
| 48 | 248 | 270 | 337 | 287 |
| 60 | 264 | 290 | 361 | 307 |
| 72 | 275 | 304 | 379 | 323 |

Note: (6) Numbers in brackets refer to Catchment Description given in Table 5.1

**Table 5.6**

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

| Duration (Hours) | Catchment | | | |
|---|---|---|---|---|
| | Lockyer (6) | Bremer (7) | Lower Brisbane (8) | Brisbane (5) |
| 12 | 131 | 136 | 167 | 144 |
| 24 | 165 | 178 | 218 | 186 |
| 36 | 189 | 204 | 253 | 215 |
| 48 | 205 | 224 | 278 | 236 |
| 60 | 217 | 239 | 298 | 252 |
| 72 | 226 | 250 | 312 | 265 |

Note: (6) Numbers in brackets refer to Catchment Description given in Table 5.1

**Table 5.7**

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

| Duration (Hours) | Catchment | | | |
|---|---|---|---|---|
| | Lockyer (6) | Bremer (7) | Lower Brisbane (8) | Brisbane (5) |
| 12 | 113 | 119 | 143 | 123 |
| 24 | 142 | 153 | 186 | 159 |
| 36 | 161 | 175 | 215 | 183 |
| 48 | 174 | 191 | 237 | 200 |
| 60 | 184 | 203 | 252 | 213 |
| 72 | 193 | 212 | 264 | 221 |

Note: (6) Numbers in brackets refer to Catchment Description given in Table 5.1

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.

## 5.4 **Areal Reduction Factors and Design Initial Losses**

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. 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 shown in Raudkivi, (1979). This equation can be used to extend the curves to the size of catchment and duration of storm that is of interest in this particular study. The equation predicts that when the area exceeds 1 000 $km^2$ for durations of 24, 48, and 72 hours, the areal reduction factor would be 0.912, 0.945 and 0.959 respectively. 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 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 5.3 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. Sequences of representative daily catchment rainfalls were derived from daily rainfall station records of a number of stations located within and around the catchments of interest. The sequences of daily catchment rainfalls were determined by combining individual station records using regression weighted coefficients.

The regression weights were determined by the Bureau of Meteorology in 1975 during a previous examination of catchment rainfalls of the Brisbane River. Detailed isohyetal maps of 18 major historical events were constructed first, using all available daily rainfall information. Estimates of the various catchment rainfalls for each of the events were then made based upon these isohyetal maps.

These estimates were then compared against estimates based upon combinations of various daily rainfall stations which were factored by varying weighting factors. Comparisons were performed on all 18 historical events and a least square calculation was then performed on each of the weighted combinations of daily rainfall stations to produce the smallest error over all of the events. The combination of weighted stations which produced the smallest error was then adopted to form the representative catchment daily rainfall sequence.

The resulting comparison of IFD data and depth-area-duration data showed little consistency. There was an underlying trend for the ratios derived in this comparison to be less than the areal reduction factors predicted by the equation, but the significance of this finding is debatable given the lack of consistency in the results.

**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 estimates.

An alternative procedure 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 the runoff-routing methods produced similar peak discharge values to those of the flood frequency estimates.

Walsh, Pilgrim and Cordery, (1991), also used this type of technique to investigate appropriate design loss rates for a range of catchments in Eastern and Western New South Wales.

This procedure depends upon the quality and length of streamflow record on which the flood frequency analysis is based and 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 are that the durations of the storms that cause the peak discharges that form the annual series are not the always the same and that the antecedent conditions of catchments are not consistent. One of the basic assumptions associated with design flood estimation which is based on design rainfalls, is that 1 in 10 year ARI rainfalls produce 1 in 10 year ARI floods. The reality is that this assumption is not always upheld, especially for higher probability of exceedence events, and therefore it is difficult to compare the two distinct methods directly.

However, despite these drawbacks, the procedure has been utilised on six unregulated catchments in the Brisbane River Valley. In all cases small initial loss rates for the 10 year and 20 year ARI events were derived. Table 5.8 presents the initial loss rates that were determined from this procedure.

These estimates are lower than the values derived by Walsh, Pilgrim and Cordery, (1991), for rainfall zone I (east of Great Divide) of New South Wales. The design initial loss rates reported for Eastern New South Wales are 60 mm, (+/- 20 mm) for the 10 year ARI event and 55 mm, (+/- 30 mm) for the 20 year ARI event. These estimates were, however, for design events with critical durations of between 36 and 48 hours, which are much longer than the critical durations of the Brisbane River catchments. Generally, the critical durations for the Brisbane catchments are less than 24 hours in duration.

**Table 5.8**

**Design Initial Losses for Brisbane River Catchments (mm)**

| Catchment | Area ($km^2$) | ARI (Years) | |
|---|---|---|---|
| | | 10 | 20 |
| Brisbane R @ Gregors Ck | 3 885 | 32.5 | 21.0 |
| Stanley R @ Somerset DS | 1 335 | 32.5 | 37.5 |
| Lockyer Ck @ Lyons Br | 2 540 | 17.5 | 4.0 |
| Bremer R @ Walloon | 620 | 0.0 | 0.0 |
| Warrill Ck @ Amberley | 920 | 5.0 | 0.0 |
| Purga Ck @ Loamside | 215 | 0.0 | 0.0 |

Note: Continuing loss = 2.5 mm/hour in all cases.

In most cases the initial loss rates approached zero as the ARI of the event increased. Because the initial loss rates were small and because they approached zero for lower probability of exceedence events, a conservative assumption of zero initial loss rate has been adopted for all design flood derivations of events with an ARI of greater than 1 in 20 years.

This conservative approach of adopting no initial loss and applying no areal reduction factor is considered appropriate because the primary objective of the study was to reassess design floods that affect the operation of Somerset Dam and Wivenhoe Dam. These floods are associated with lower probability of exceedence rainfall events where rainfall loss rates are likely to be of less significance.

For the 1 in 10 year and 1 in 20 year ARI events, initial loss rates have been considered for comparative purposes because of the implications of the estimates in the urban reaches of the lower Brisbane and Bremer Rivers.

By means of comparison with the design event initial loss rates, a summary of the initial and continuing loss rates utilised during calibration of the runoff-routing models is presented in Table 5.9.

**Table 5.9**

**Initial Losses Derived During Calibration of Brisbane River Catchments**

| Catchment | Area ($km^2$) | Loss Rates | | | |
|---|---|---|---|---|---|
| | | Mean | | Median | |
| | | IL | CL | IL | CL |
| Brisbane R @ Gregors Ck | 3 885 | 38.9 | 2.0 | 30.0 | 1.5 |
| Stanley R @ Somerset DS | 1 335 | 10.0 | 0.2 | 10.0 | 0.2 |
| Lockyer Ck @ Lyons Br | 2 540 | 44.0 | 4.4 | 18.0 | 2.6 |
| Bremer R @ Walloon | 620 | 46.3 | 2.6 | 57.5 | 2.5 |
| Warrill Ck @ Amberley | 920 | 17.5 | 1.7 | 15.0 | 2.0 |
| Purga Ck @ Loamside | 215 | 32.0 | 3.9 | 20.0 | 3.6 |

Note: IL = Initial Loss (mm)
CL = Continuing Loss (mm/hour)

These values tend to be of the same order as the design loss rates shown in Table 5.8, with the exception of the Bremer River catchments, whose calibration initial loss rates are higher than the corresponding design initial loss rates.

FIGURE 5.1

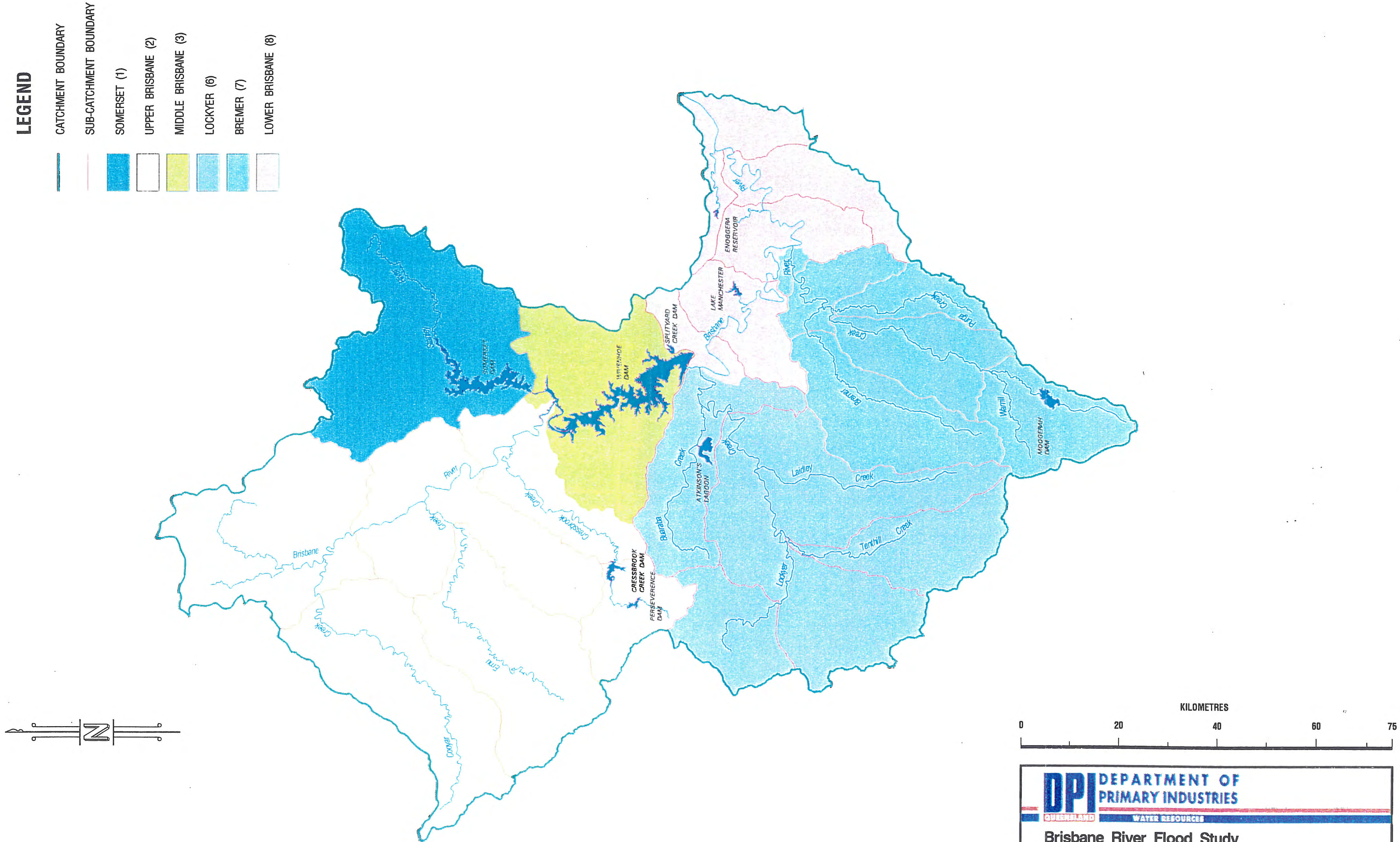



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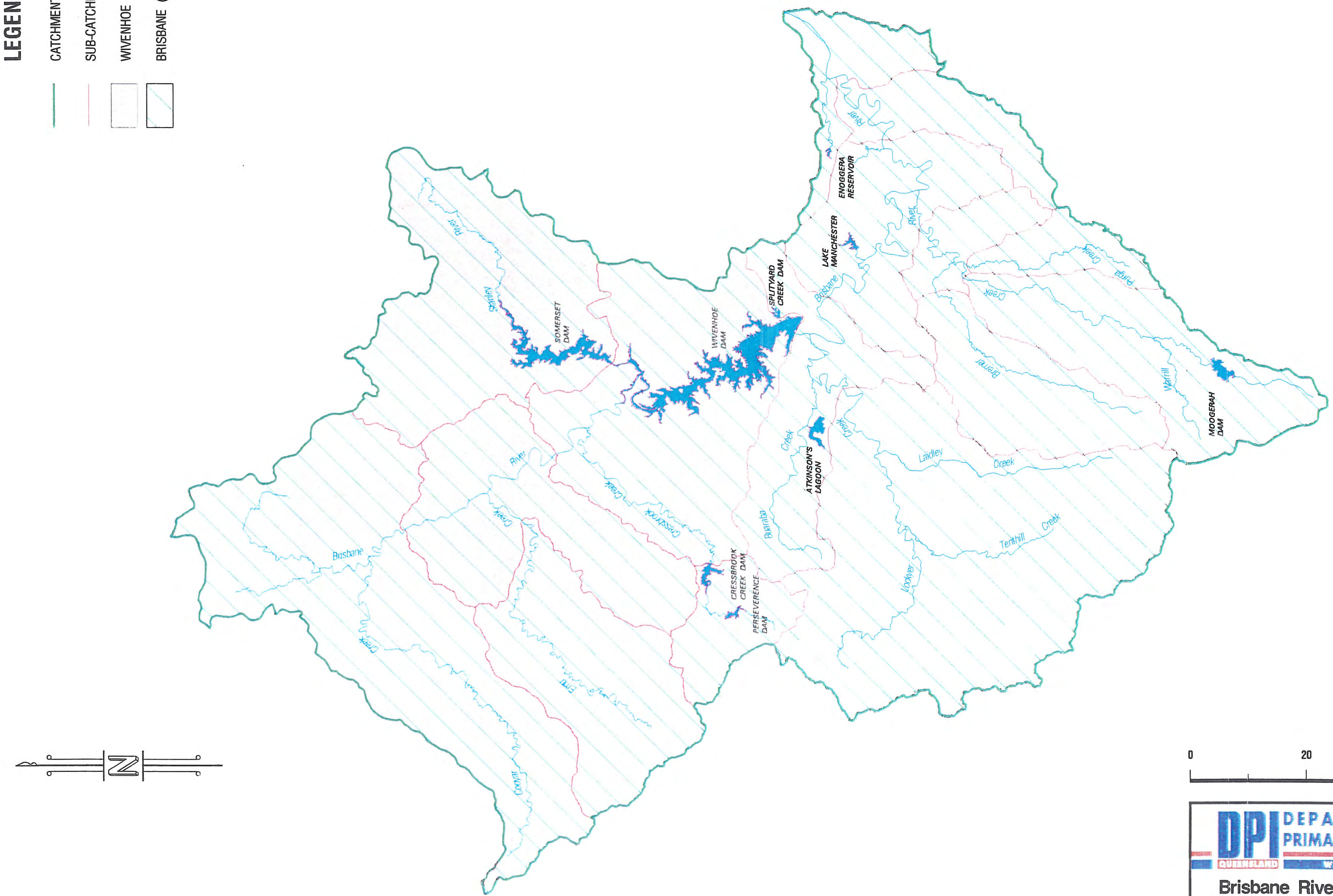
FIGURE 5.2

## LEGEND

- CATCHMENT BOUNDARY
- SUB-CATCHMENT BOUNDARY
- WIVENHOE (4)
- BRISBANE (5)




KILOMETRES

0 20 40 60 75

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Brisbane River Flood Study

**Brisbane River**
**Design Storm Centres**

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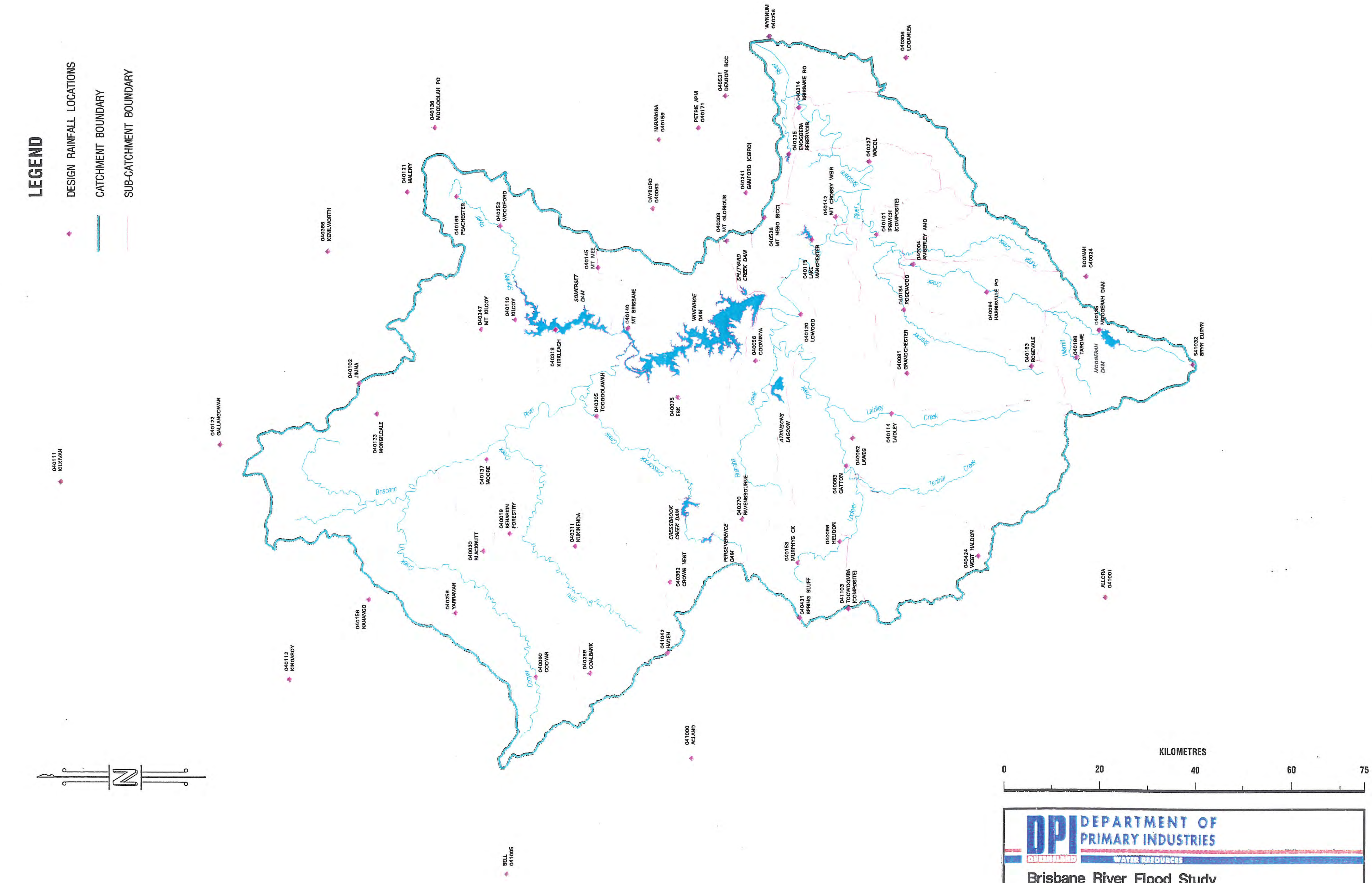
FIGURE 5.3

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Brisbane River Flood Study

## Brisbane River Rainfall Locations

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# 6.0 RESULTS OF DESIGN FLOOD ESTIMATION

## 6.1 Introduction

A range of design flood scenarios were considered during the reassessment of design floods for both Somerset Dam and Wivenhoe Dam and the resultant downstream flooding. Design storms were centred over seven different catchments and the resulting flood hydrographs at the Port Office Gauge determined. The design storm centres considered were:

1. Somerset Dam Catchment : SOMERSET
2. Upper Brisbane River Catchment: UPPER BRISBANE
3. Middle Brisbane Catchment : MIDDLE BRISBANE
4. Wivenhoe Dam Catchment : WIVENHOE
5. Brisbane River Catchment : BRISBANE
6. Lockyer Creek Catchment : LOCKYER
7. Bremer River Catchment : BREMER

These catchments are shown in Figures 5.1 and 5.2.

The Lower Brisbane River catchment was not considered because it was felt that the flood response from such a catchment would not be of comparable magnitude to the other catchments. This is because of its proximity to the Port Office Gauge and the fact that the flood hydrographs emanating from the remainder of the catchment would be lagged some considerable time behind the flood response of the Lower Brisbane River catchment.

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). 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 exceedence 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 subject catchment.

## 6.2 **Brisbane River @ Port Office Gauge**

Design flood estimates for the Port Office Gauge have been derived for two cases:

(i) No dams effective.
(ii) Somerset Dam and Wivenhoe Dam effective.

### 6.2.1 No Dams Effective

The no dams effective case was simulated so that the effect of the flood mitigation of the dams could be demonstrated. Only design storm scenarios involving the catchments of Somerset Dam, Wivenhoe Dam and the Brisbane River were considered for the no dam effective simulations.

Probable Maximum Flood, (PMF), estimates for the Port Office Gauge for the no dams effective are presented in Table 6.1. In all cases, an initial loss of 0 mm and a continuing loss rate of 2.5 mm/hour was adopted. The rainfall depths provided in Table 5.2 and Table 5.4 were utilised for the assessment of the PMF storms.

**Table 6.1**

**Critical Design Flood Estimates**
**Brisbane River @ Port Office Gauge**
**For PMF Events on Sub-Catchments**
**No Dams Effective**

| Storm Centre | | Storm Duration (Hours) | Peak Discharge ($m^3/s$) | Flood Volume (ML) |
|---|---|---|---|---|
| 1 | Somerset | 24 | 24 510 | 3 549 900 |
| 4 | Wivenhoe | 24 | 43 990 | 5 504 210 |
| 5 | Brisbane | 24 | 42 490 | 6 235 130 |

Figure 6.1 presents the estimated PMF hydrographs of a number of locations along the Brisbane River for the 24 hour design storm scenario centred over the whole of the Brisbane River for the 'No Dams Effective' case.

A critical duration of 24 hours was derived for all of the PMF scenarios considered for the Port Office Gauge for the no dams effective case. The peak discharge of 43 990 $m^3/s$ at the Port Office is lower than the estimate of 54 400 $m^3/s$ that Weeks obtained in the 1984 study. The peak discharge that Weeks obtained was for a PMF event with a duration of 144 hours without Wivenhoe Dam effective. (Refer to Section 3.3).

100 Year ARI flood event estimates for the Port Office Gauge for the no dams effective are presented in Table 6.2. In all cases, an initial loss of 0 mm and a continuing loss rate of 2.5 mm/hour was adopted. The rainfall depths provided in Table 5.4 were utilised for the assessment of the 100 Year ARI flood events.

**Table 6.2**

**Critical Design Flood Estimates**
**Brisbane River @ Port Office Gauge**
**For 100 Year ARI Flood Events on Sub-Catchments**
**No Dams Effective**

| Storm Centre | | Storm Duration (Hours) | Peak Discharge ($m^3/s$) | Flood Volume (ML) |
|---|---|---|---|---|
| 1 | Somerset | 24 | 14 950 | 2 642 750 |
| 4 | Wivenhoe | 24 | 15 070 | 2 641 210 |
| 5 | Brisbane | 24 | 14 910 | 2 632 020 |

Figure 6.2 presents the estimated 100 Year ARI hydrographs of a number of locations along the Brisbane River for the 24 hour design storm scenario centred over the whole of the Brisbane River for the 'No Dams Effective' case.

A critical duration of 24 hours was derived for all of the 100 Year ARI scenarios considered for the Port Office Gauge for the no dams effective case. The peak discharge of 15 070 $m^3/s$ at the Port Office is higher than the estimate of 11 500 $m^3/s$ that Weeks obtained in the 1984 study. The peak discharge Weeks obtained was for a 100 Year ARI event centred over the catchment of Wivenhoe Dam with a duration of 48 hours and without Wivenhoe Dam effective. (Refer to Section 3.3).

Estimates of peak discharge for events with ARI's higher than 1 in 100 years for the Port Office Gauge can also be compared against the estimates of peak discharge obtained for Moggill. These estimates were derived from a flood frequency analysis of the BCC's 'No Dams' annual series data. (Refer to Volume III of the Interim Report on Design Flood Estimation', 1993). A Log-Pearson Type III distribution was adopted in the frequency analysis.

Table 6.3 presents a summary of the peak discharge estimates for the Port Office Gauge for the higher probability of exceedence events. The estimates of peak discharge derived from the BCC annual series data flood frequency analysis have been factored by the a ratio of catchment areas raised to the power of 0.7, to account for the difference in catchment areas between Moggill and the Port Office Gauge. The value of this factor is only 1.016.

The runoff-routing model estimates are based upon 24 hour design rainfalls centred over the whole of the Brisbane River, even though these estimates produce marginally smaller peaks than the corresponding Wivenhoe Dam catchment based estimates. This scenario was adopted because of ease of calculation, and because it is consistant with critical post-dam scenarios.

**Table 6.3**

**Comparison of Design Flood Estimates**
**Brisbane River @ Port Office Gauge**
**For Storm Centred Over the Brisbane River**
**No Dams Effective**

| ARI (Years) | Peak Discharge ($m^3/s$) | | | |
|---|---|---|---|---|
| | BCC Flood Frequency | No Initial Loss* | Variable Initial Loss* | Uniform Initial Loss* |
| 100 | 18 500 | 14 910 | 14 910 | 14 910 |
| 50 | 13 370 | 12 340 | 12 340 | 12 340 |
| 20 | 8 070 | 9 300 | 7 870 | 8 040 |
| 10 | 5 060 | 7 330 | 5 140 | 5 620 |

Notes:
(*) Continuing Loss = 2.5 mm/hour for all ARIs.
No IL = 0 mm over whole catchment for all ARIs.
Variable IL = Initial Loss Rates as per Table 5.8.
Uniform IL = 22.9 mm over whole catchment for 10 Yr ARI.
Uniform IL = 14.9 mm over whole catchment for 20 Yr ARI.

The comparison illustrates the effect of design initial loss rates in equating independently derived estimates of peak discharges. The runoff-routing model estimates derived without the inclusion of the design initial losses appear to 'overestimate' peak discharge for the more frequently occurring design flood events.

The adopted full range of flood frequencies of peak discharges at the Port Office Gauge for the 'No Dams Effective' case is presented in Table 6.4. The estimates are based upon design storm rainfalls centred over the whole of the Brisbane River catchment with a duration of 24 hours. Variable design initial losses have been applied to the 10 year and 20 year ARI events.

Figure 6.3 presents the the adopted flood frequency distribution for the Brisbane River at the Port Office Gauge for the 'No Dams Effective' case. Also shown for comparative purposes is the BCC Log-Pearson Type III flood frequency distribution derived for Moggill that has been adjusted to account for the difference in catchment areas between Moggill and the Port Office Gauge.

**Table 6.4**

**Design Flood Frequency Estimates**
**Brisbane River @ Port Office Gauge**
**For 24 Hour Storm Centred Over the Brisbane River**
**No Dams Effective**

| ARI (Years) | Peak Discharge ($m^3/s$) | Flood Volume (ML) |
|---|---|---|
| 10 | *5 140 | 1 168 710 |
| 20 | *7 870 | 1 629 130 |
| 50 | 12 340 | 2 243 810 |
| 100 | 14 910 | 2 632 020 |
| 200 | 17 830 | 2 984 460 |
| 500 | 21 660 | 3 515 200 |
| 1 000 | 25 080 | 3 979 590 |
| 10 000 | 35 680 | 5 372 710 |
| 100 000 (PMF) | 42 920 | 6 235 130 |

Note: (*) Variable Design Initial Loss Applied.

Figure 6.4 presents a range of design flood hydrographs for the Brisbane River at the Port Office Gauge for the 24 hour duration design storm scenario centred over the whole of the Brisbane River catchment with 'No Dams Effective'.

### 6.2.2 Somerset Dam and Wivenhoe Dam Effective

Design flood estimates for the Port Office Gauge were also calculated assuming that Somerset Dam and Wivenhoe Dam were effective and operating in accordance with existing normal gate operation procedures. Operational procedure 4 for Wivenhoe Dam, (Refer to Section 4.3), was utilised for all cases involving the PMF events. The dams were assumed to be at full supply level prior to the event.

In all cases involving the PMF an initial loss of 0 mm and a continuing loss rate of 2.5 mm/hour was adopted. The rainfall depths provided in Table 5.2 and Table 5.4 were utilised for the assessment of the probable maximum flood storms. Table 6.5 provides a summary of the PMF design storm scenarios.

**Table 6.5**

**Critical Design Flood Estimates**
**Brisbane River @ Port Office Gauge**
**For PMF Events on Sub-Catchments**
**Somerset Dam and Wivenhoe Dam Effective**

| Storm Centre | | Storm Duration (Hours) | Peak Discharge ($m^3/s$) | Flood Volume (ML) |
|---|---|---|---|---|
| 1 | Somerset | 96 | 14 270 | 4 803 960 |
| 2 | Upper Brisbane | 48 | 25 550 | 5 891 290 |
| 3 | Middle Brisbane | 48 | 14 430 | 3 827 230 |
| 4 | Wivenhoe | 48 | 29 220 | 6 612 280 |
| 5 | Brisbane | 120a | 31 950 | 10 318 610 |
| 6 | Lockyer | 24 | 23 450 | 4 266 570 |
| 7 | Bremer | 24 | 16 960 | 3 060 290 |

Note: (a) Refers to various temporal patterns provided by the Bureau of Meteorology.

The results presented in Table 6.5 indicate that the critical storm event in terms of peak discharge at the Port Office Gauge is the one associated with a 120(a) hour storm centred over the whole of the Brisbane River catchment.

Figure 6.5 presents the estimated PMF event hydrographs for various locations downstream of Wivenhoe Dam for the 120 hour duration design storm scenario centred over the whole of the Brisbane River.

It should be noted that this particular event is of sufficient magnitude to overtop Wivenhoe Dam. Several of the events presented in the table would overtop either Somerset Dam or Wivenhoe Dam. For this reason, the performance of the dams corresponding to the events shown in Table 6.5 is summarised in Table 6.6.

**Table 6.6**

**PMF Events**
**Storage Performance Summary**

| Storm Centre | | SOMERSET DAM | | | | |
|---|---|---|---|---|---|---|
| | | Storm Dur (Hrs) | Peak Inflow ($m^3/s$) | Peak Out flow ($m^3/s$) | Flood Volume (ML) | Peak Lake level (mAHD) |
| 1 | Somerset | 96 | 9 910 | 8 000 | 2 457 600 | 110.32# |
| 2 | Upper Brisbane | 48 | 1 790 | 2 310 | 276 150 | 102.38 |
| 3 | Middle Brisbane | 48 | 1 790 | 2 310 | 276 150 | 102.38 |
| 4 | Wivenhoe | 48 | 6 390 | 4 210 | 995 680 | 106.71 |
| 5 | Brisbane | 120a | 4 290 | 3 250 | 1 094 190 | 104.88 |
| 6 | Lockyer | 24 | 2 700 | 2 330 | 258 740 | 102.43 |
| 7 | Bremer | 24 | 2 700 | 2 330 | 258 740 | 102.43 |

Where: # Non-overflow spillway level (107.46 m AHD) overtopped.

| Storm Centre | | WIVENHOE DAM | | | | |
|---|---|---|---|---|---|---|
| | | Storm Dur (Hrs) | Peak Inflow ($m^3/s$) | Peak Out flow ($m^3/s$) | Flood Volume (ML) | Peak Lake Level (mAHD) |
| 1 | Somerset | 96 | 12 890 | 10 850 | 3 598 600 | 76.00 |
| 2 | Upper Brisbane | 48 | 28 910 | 21 900 | 4 594 030 | 80.96* |
| 3 | Middle Brisbane | 48 | 15 490 | 11 130 | 2 529 970 | 76.28 |
| 4 | Wivenhoe | 48 | 30 670 | 25 040 | 5 315 030 | 81.28* |
| 5 | Brisbane | 120a | 20 720 | 17 250 | 5 490 260 | 80.39* |
| 6 | Lockyer | 24 | 12 270 | 5 760 | 1 411 920 | 74.60 |
| 7 | Bremer | 24 | 12 270 | 5 760 | 1 436 490 | 74.52 |

Where: (*) Embankment crest level (79.15 m AHD) and top of wave wall (79.90 m AHD) overtopped.

(a) Refers to various temporal patterns provided by the Bureau of Meteorology.

Somerset Dam being a mass concrete gravity dam is able to withstand some overtopping and according to Russo, (1988), it is structurally capable of handling the estimated PMF floods without failure. Wivenhoe Dam, on the other hand, is a zoned earth and rockfill dam and it is unlikely that it could withstand being overtopped to the extent that has been estimated. Wivenhoe Dam is therefore most likely to fail when subjected to PMF floods of the magnitude that have been estimated. The flooding resulting from such a failure will be the subject of a future report.

100 year ARI flood events for the same catchments have also been assessed using the same assumptions as for the PMF events. In all cases an initial loss of 0 mm and a continuing loss rate of 2.5 mm/hour was adopted. The rainfall depths provided in Table 5.4 were utilised for the assessment of the 100 Year ARI storms. Table 6.7 provides a summary of the 100 year ARI design storm scenarios.

**Table 6.7**

**Critical Design Flood Estimates**
**Brisbane River @ Port Office Gauge**
**For 100 Year ARI Flood Events on Sub-Catchments**
**Somerset Dam and Wivenhoe Dam Effective**

| Storm Centre | | Storm Duration (Hours) | Peak Discharge ($m^3/s$) | Flood Volume (ML) |
|---|---|---|---|---|
| 1 | Somerset | 24 | 8 550 | 2 705 820 |
| 2 | Upper Brisbane | 24 | 9 380 | 2 695 570 |
| 3 | Middle Brisbane | 24 | 8 820 | 2 670 570 |
| 4 | Wivenhoe | 24 | 8 620 | 2 705 170 |
| 5 | Brisbane | 24 | 9 120 | 2 696 300 |
| 6 | Lockyer | 24 | 8 740 | 2 694 320 |
| 7 | Bremer | 24 | 8 700 | 2 698 870 |

It is obvious from the Table 6.7 that the results are very similar for all assumed storm centres. These results are to be expected because of the adopted concurrent rainfall scenario for this particular event. In essence, the whole of the Brisbane River is assumed to be subjected to an equivalent 100 year ARI event which is why the resultant flooding is so alike. The variation in estimates is caused by the catchment on which the storm is centred being subject to a slightly different rainfall depth than the adjacent catchments.

For ease of calculation and consistancy with the PMF estimates, the adopted critical peak discharge estimate for the Port Office Gauge is the one associated with the storm centred over the whole of the Brisbane River catchment.

Figure 6.6 presents the 100 year ARI event hydrographs for various locations downstream of Wivenhoe Dam, for the 24 hour duration design storm scenario centred over the whole of the Brisbane River catchment.

The performance of the dams corresponding to the events shown in Table 6.7 is summarised in Table 6.8.

**Table 6.8**

**100 Year ARI Events**
**Storage Performance Summary**

| Storm Centre | | **SOMERSET DAM** | | | | |
|---|---|---|---|---|---|---|
| | | Storm Dur (Hrs) | Peak Inflow ($m^3/s$) | Peak Out flow ($m^3/s$) | Flood Volume (ML) | Peak Lake level (mAHD) |
| 1 | Somerset | 24 | 4 780 | 2 550 | 398 460 | 103.13 |
| 2 | Upper Brisbane | 24 | 3 330 | 2 370 | 279 350 | 102.57 |
| 3 | Middle Brisbane | 24 | 3 080 | 2 320 | 258 720 | 102.42 |
| 4 | Wivenhoe | 24 | 3 260 | 2 350 | 274 190 | 102.51 |
| 5 | Brisbane | 24 | 3 130 | 2 340 | 262 590 | 102.47 |
| 6 | Lockyer | 24 | 3 260 | 2 350 | 274 190 | 102.51 |
| 7 | Bremer | 24 | 3 170 | 2 350 | 266 450 | 102.51 |

| Storm Centre | | WIVENHOE DAM | | | | |
|---|---|---|---|---|---|---|
| | | Storm Dur (Hrs) | Peak Inflow ($m^3/s$) | Peak Out flow ($m^3/s$) | Flood Volume (ML) | Peak Lake level (mAHD) |
| 1 | Somerset | 24 | 11 710 | 5 900 | 1 534 430 | 74.52 |
| 2 | Upper Brisbane | 24 | 10 430 | 4 570 | 1 378 230 | 74.36 |
| 3 | Middle Brisbane | 24 | 11 230 | 5 760 | 1 450 120 | 74.53 |
| 4 | Wivenhoe | 24 | 11 720 | 5 780 | 1 521 570 | 74.59 |
| 5 | Brisbane | 24 | 11 150 | 5 080 | 1 460 330 | 74.52 |
| 6 | Lockyer | 24 | 11 720 | 5 560 | 1 521 310 | 74.45 |
| 7 | Bremer | 24 | 11 300 | 5 760 | 1 481 210 | 74.53 |

A critical duration of 24 hours was derived for all of the 100 Year ARI scenarios considered for the Port Office Gauge for the Dams effective case. The peak discharge of 9 120 $m^3/s$ at the Port Office is higher than the estimate of 5 510 $m^3/s$ that Weeks obtained in the 1984 study. The peak discharge that Weeks obtained was for a 100 Year ARI event centred over the catchment of Wivenhoe Dam with a duration of 48 hours, with Wivenhoe Dam effective and releasing up to 3 500 $m^3/s$. (Refer to Section 3.3).

Estimates of the 100 Year ARI event peak discharge for the Port Office Gauge can also be compared against estimates of peak discharge obtained for Moggill. These estimates were derived from a flood frequency analysis of the BCC's 'Somerset Dam and Wivenhoe Dam Effective' annual series data. (Refer to Volume III of the Interim Report on Design Flood Estimation', 1993).

These estimates are reproduced in Table 6.9. The estimates of peak discharge derived from the BCC annual series data flood frequency analysis have been factored by the a ratio of catchment areas raised to the power of 0.7, to account for the difference in catchment area between Moggill and the Port Office Gauge. The value of this factor is only 1.016.

**Table 6.9**

**Flood Frequency Estimates**
**Brisbane River at Port Office Gauge**
**BCC - Log-Pearson Type III Distribution**
**Somerset Dam & Wivenhoe Dam Effective**

| ARI (Years) | Peak Discharge ($m^3/s$) |
|---|---|
| 10 | 2 880 |
| 20 | 4 710 |
| 50 | 7 890 |
| 100 | 10 900 |

In addition to the flood frequency results for Moggill there are also estimates available of the probable frequency of peak discharge for the Brisbane River at the Port Office Gauge from 'Revision A - Brisbane River Flood Plain Map Series Sheets 1 - 18', which were compiled by the Survey Office. These estimates are summarised in Table 6.10 and are based upon the BCC's annual series mentioned earlier. The BCC have advised that the Boughton empirical distribution was adopted in preference to the Log-Pearson Type III for the derivation of these estimates, hence the differences between peak discharge estimates shown in Tables 6.9 and 6.10.

**Table 6.10**
**Flood Frequency Estimates**
**Brisbane River at Port Office Gauge**
**BCC - Boughton Empirical Distribution**
**Somerset Dam & Wivenhoe Dam Effective**

| ARI (Years) | Gauge Height (m) | Peak Discharge ($m^3/s$) |
|---|---|---|
| 25 | 2.0 | 4 040 |
| 125 | 4.0 | 7 150 |
| 700 | 6.0 | 10 150 |
| 3 500 | 8.0 | 13 200 |
| > 10 000 | 10.0 | 16 500 |

Table 6.11 presents a summary of the peak discharge estimates for the Port Office Gauge for the higher probability of exceedence events derived from the runoff-routing modelling. These estimates have been compared against the flood frequency results derived from the flood frequency analysis using the

Log-Pearson Type III distribution.

**Table 6.11**

**Comparison of Design Flood Estimates**
**Brisbane River @ Port Office Gauge**
**For 24 Hour Storm Centred Over the Brisbane River**
**Somerset Dam & Wivenhoe Dam Effective**

| ARI (Years) | Peak Discharge ($m^3/s$) | | | |
|---|---|---|---|---|
| | BCC Flood Frequency | No Initial Loss* | Variable Initial Loss* | Uniform Initial Loss* |
| 100 | 10 900 | 9 120 | 9 120 | 9 120 |
| 50 | 7 890 | 7 990 | 7 990 | 7 990 |
| 20 | 4 710 | 6 600 | 6 460 | 6 090 |
| 10 | 2 880 | 5 710 | 5 270 | 5 160 |

Note: Wivenhoe Dam Operational Procedure 4 adopted.

Operational procedure 4 for Wivenhoe Dam, (Refer to Section 4.3), has been adopted in all of the cases shown in Table 6.11.

It should be noted that greater flood mitigation can be achieved for the higher probability of exceedence events if the other operational procedures were adopted, although the operating selection policy described in the flood operations manual indicates that these procedures are not necessarily appropriate. Tables 6.12 to 6.14 provide a summary of the peak flows for the other operational procedures for comparative purposes.

**Table 6.12**

**Comparison of Design Flood Estimates**
**Brisbane River @ Port Office Gauge**
**For 24 Hour Storm Centred Over the Brisbane River**
**Somerset Dam & Wivenhoe Dam Effective**

| ARI (Years) | Peak Discharge ($m^3$/s) | | | |
|---|---|---|---|---|
| | BCC Flood Frequency | No Initial Loss* | Variable Initial Loss* | Uniform Initial Loss* |
| 100 | 10 900 | 9 120 | 9 120 | 9 120 |
| 50 | 7 890 | 7 990 | 7 990 | 7 990 |
| 20 | 4 710 | 6 600+ | 6 460+ | 6 090+ |
| 10 | 2 880 | 5 710+ | 5 240+ | 5 160+ |

Note: (+) Wivenhoe Dam Operational Procedure 3 adopted for 10 Year an 20 Year ARI events.

**Table 6.13**

**Comparison of Design Flood Estimates**
**Brisbane River @ Port Office Gauge**
**For 24 Hour Storm Centred Over the Brisbane River**
**Somerset Dam & Wivenhoe Dam Effective**

| ARI (Years) | Peak Discharge ($m^3$/s) | | | |
|---|---|---|---|---|
| | BCC Flood Frequency | No Initial Loss* | Variable Initial Loss* | Uniform Initial Loss* |
| 100 | 10 900 | 9 120 | 9 120 | 9 120 |
| 50 | 7 890 | 7 990 | 7 990 | 7 990 |
| 20 | 4 710 | 4 710+ | 4 580+ | 4 100+ |
| 10 | 2 880 | 3 760+ | 3 270+ | 3 060+ |

Note: (+) Wivenhoe Dam Operational Procedure 2 adopted for 10 Year and 20 Year ARI events.

**Table 6.14**

**Comparison of Design Flood Estimates**
**Brisbane River @ Port Office Gauge**
**For 24 Hour Storm Centred Over the Brisbane River**
**Somerset Dam & Wivenhoe Dam Effective**

| ARI (Years) | Peak Discharge ($m^3/s$) | | | |
|---|---|---|---|---|
| | BCC Flood Frequency | No Initial Loss* | Variable Initial Loss* | Uniform Initial Loss* |
| 100 | 10 900 | 9 120 | 9 120 | 9 120 |
| 50 | 7 890 | 7 990 | 7 990 | 7 990 |
| 20 | 4 710 | 4 510+ | 4 390+ | 3 950+ |
| 10 | 2 880 | 3 620+ | 3 050+ | 2 840+ |

Note: (+) Wivenhoe Dam Operational Procedure 1 adopted for 10 Year and 20 Year ARI events.

General notes pertaining to Tables 6.11 to 6.14:

(*) Continuing Loss = 2.5 mm/hour for all ARIs.
No IL = 0 mm over whole catchment for all ARIs.
Variable IL = Initial Loss Rates as per Table 5.8.
Uniform IL = 22.9 mm over whole catchment for 10 Yr ARI.
Uniform IL = 14.9 mm over whole catchment for 20 Yr ARI.
IL = Initial Loss

The comparisons illustrate the effect of the operational procedure and the design initial loss rates in equating independently derived estimates of peak discharges. Runoff-routing model estimates derived without the inclusion of the design initial loss rates appear to 'overestimate' peak discharge for the higher probability of exceedence design flood events.

A full range of flood frequencies of peak discharges for the Brisbane River at the Port Office Gauge is presented in Table 6.15. The estimates are based upon a design storm scenario centred over the whole of the Brisbane River catchment with a duration of 24 hours.

**Table 6.15**

**Design Flood Frequency Estimates**
**Brisbane River @ Port Office Gauge**
**For 24 Hour Storm Centred Over the Brisbane River**
**Somerset Dam & Wivenhoe Dam Effective**

| ARI (Years) | Peak Discharge ($m^3/s$) | Flood Volume (ML) |
|---|---|---|
| 10 | *3 050 | 1 243 760 |
| 20 | *4 390 | 1 670 010 |
| 50 | 7 990 | 2 284 180 |
| 100 | 9 120 | 2 696 300 |
| 200 | 10 240 | 3 025 700 |
| 500 | 11 950 | 3 556 000 |
| 1 000 | 13 550 | 4 023 320 |
| 10 000 | 18 950 | 5 420 340 |
| 100 000 (PMF) | 21 980 | 6 282 800 |

Note: (*) Variable Design Initial Loss Rates Applied, and Wivenhoe Dam Operation Procedure 1 Adopted.

Whilst, the 24 hour storm duration event results in the largest magnitude peak discharges for floods up to the 100 Year ARI event, it should be noted that the 120 hour storm duration yields the largest magnitude PMF peak discharges. This causes some problems as there are no standard procedures available for estimating 120 hour duration, 100 Year ARI storm rainfalls, which are required for interpolation of rainfall depths of ARI's greater than 100 years.

For the purpose of estimating the 120 hour duration 100 Year ARI storm rainfalls, the ratio of the 24 hour 100 year ARI rainfall to the 24 hour PMP rainfall was adopted and applied to the 120 hour PMP rainfall depth in order to estimate the 120 hour 100 Year ARI rainfall. This resulted in a 120 hour duration depth of 505 mm being derived. Intermediate event rainfalls were then estimated in the usual fashion using the techniques outlined in Chapter 13 of Australian Rainfall and Runoff, (1987).

The 120 (a) hour duration temporal pattern was adopted for all events, including the 100 Year ARI event.

A full range of flood frequencies of peak discharges for the Brisbane River at the Port Office Gauge is presented in Table 6.16. The estimates are based upon a design storm scenario centred over the whole of the Brisbane River catchment with a duration of 120 hours.

**Table 6.16**

**Design Flood Frequency Estimates**
**Brisbane River @ Port Office Gauge**
**For 120 (a) Hour Storm Centred Over the Brisbane River**
**Somerset Dam & Wivenhoe Dam Effective**

| ARI (Years) | Peak Discharge ($m^3/s$) | Flood Volume (ML) |
|---|---|---|
| 100 | 7 800 | 3 293 590 |
| 200 | 11 840 | 4 118 910 |
| 500 | 17 510 | 5 266 990 |
| 1 000 | 20 100 | 5 968 430 |
| 10 000 | 27 560 | 8 630 790 |
| 100 000 (PMF) | 31 950 | 10 318 610 |

Figure 6.7 presents the flood frequency distributions for the two durations of design storm for Brisbane River at the Port Office Gauge for the Somerset Dam and Wivenhoe Dam Effective case. Also shown for comparative purposes, are the frequency distributions derived by the BCC in earlier studies.

Figure 6.8 presents a range of design flood hydrographs for the Brisbane River at the Port Office Gauge for the 24 hour duration design storm scenario centred over the whole of the Brisbane River catchment for the 'Somerset Dam and Wivenhoe Dam Effective' case. Likewise, Figure 6.9 presents a range of design flood hydrographs for the Brisbane River at the Port Office Gauge for the 120(a) hour duration design storm scenario centred over the whole of the Brisbane River catchment for the 'Somerset Dam and Wivenhoe Dam Effective' case.

## 6.3 Lockyer Creek @ Lyons Bridge

Design flood estimates for the Lockyer Creek catchment at Lyons Bridge were calculated so that an estimate of the magnitude of floods from this sub-basin could be made. This analysis was performed as a check on the capability of the existing operation procedures of the dams to cope with large floods centred in adjacent catchments.

Estimates of flood hydrographs from the Lockyer Creek catchment also provide an indication of the magnitude of flooding that could result from an essentially unregulated part of the Brisbane River catchment.

Design flood estimates for Lockyer Creek were calculated assuming that the dams were at full supply level prior to the event and that the existing normal operation procedures of the dams were utilised. The PMP storm rainfall was assumed to be centred over the Lockyer Creek catchment with 100 year ARI rainfall falling in the remaining adjacent catchments.

In all cases an initial loss of 0 mm and a continuing loss rate of 2.5 mm/hour was adopted. The rainfall depths provided in Table 5.2 and Table 5.4 were utilised for the assessment of the probable maximum flood, (PMF), storms. Table 6.17 provides a summary of the PMF design storm scenarios.

**Table 6.17**

**Critical Design Flood Estimates**
**Lockyer Creek at Lyons Bridge**
**PMF Storm Centred Over Lockyer Creek**

| Duration (Hours) | Peak Discharge ($m^3/s$) | Flood Volume (ML) |
|---|---|---|
| 12 | 15 510 | 1 240 240 |
| 24 | 18 480 | 1 799 600 |
| 48 | 15 010 | 2 407 060 |
| 72 | 13 200 | 2 919 660 |

The critical duration for the catchment at Lyons Bridge was found to be the 24 hour event.

Likewise an assessment of the 100 year ARI event was undertaken with the results summarised in Table 6.18. The critical duration for the 100 year ARI event appears to be 18 hours. However, for consistancy with the PMF, 24 hours has been adopted as the critical duration so that a full flood frequency could be determined.

**Table 6.18**

**Critical Design Flood Estimates**
**Lockyer Creek at Lyons Bridge**
**100 Year ARI Storm Centred Over Lockyer Creek**

| Duration (Hours) | Peak Discharge ($m^3/s$) | Flood Volume (ML) |
|---|---|---|
| 12 | 3 370 | 347 700 |
| 24 | 3 220 | 407 200 |
| 48 | 2 360 | 443 470 |
| 72 | 2 200 | 465 700 |

Table 6.19 presents the full range of flood frequencies for storms centred over the Lockyer Creek catchment with a duration of 24 hours. Shown in this table are the associated outflows from Wivenhoe Dam and the resultant peak discharge at the Port Office Gauge.

**Table 6.19**

**Design Flood Frequency Estimates**
**24 Hour Storm Centred Over Lockyer Creek**

| ARI (Yrs) | Peak Discharge ($m^3/s$) | | | Flood Volume (ML) | | |
|---|---|---|---|---|---|---|
| | Lyons Bridge | Wivenhoe Dam | Port Office | Lyons Bridge | Wivenhoe Dam | Port Office |
| * 10 | 1 660 | 3 320 | 5 680 | 226 460 | 851 540 | 1 497 670 |
| * 20 | 2 100 | 3 420 | 6 490 | 277 910 | 1 021 190 | 1 815 430 |
| * 50 | 2 680 | 3 470 | 7 650 | 346 590 | 1 302 770 | 2 300 120 |
| * 100 | 3 220 | 5 560 | 8 740 | 407 200 | 1 521 310 | 2 694 320 |
| 200 | 4 000 | 5 050 | 8 910 | 462 050 | 1 411 180 | 2 612 640 |
| 500 | 4 990 | 5 040 | 9 500 | 559 330 | 1 437 750 | 2 758 730 |
| 1 000 | 6 240 | 5 760 | 10 300 | 680 920 | 1 411 740 | 2 883 730 |
| 10 000 | 12 010 | 5 760 | 15 870 | 1 215 940 | 1 438 470 | 3 571 050 |
| 100 000 | 16 320 | 5 760 | 20 890 | 1 605 050 | 1 412 800 | 4 026 980 |
| PMF | 18 480 | 5 760 | 23 450 | 1 799 600 | 1 411 920 | 4 266 570 |

Note: * Temporal patterns from Australian Rainfall and Runoff, (1987), used.

The resultant peak discharges for the Port Office Gauge for the more frequently occurring events are greatly affected by the actual operational procedure adopted for Wivenhoe Dam and whether or not design initial loss rates are utilised for these events. In Table 6.19, operational procedure 4 was adopted for Wivenhoe Dam and no design initial loss was applied for all cases.

A comparison between the peak discharge estimates for Lockyer Creek at Lyons Bridge based upon runoff-routing model techniques and flood frequency techniques is possible.

A Log-Pearson Type III distribution was fitted to the annual series of peak discharges for the stream gauge located at Lyons Bridge and the results of the analysis were presented in Volume III the report on design flood estimates for Somerset Dam and Wivenhoe Dam, (Ayre, Cutler and Ruffini, 1993). Table 6.10 provides a summary of these results and compares the runoff-routing model estimates. It should be noted that the runoff-routing model estimates shown in Table 6.20 are for an 18 hour duration event and not 24 hour as in Table 6.19.

**Table 6.20**

**Comparison of Peak Discharge Estimates**
**Lockyer Creek at Lyons Bridge**

| ARI (Years) | Peak Discharge ($m^3/s$) | | |
|---|---|---|---|
| | Runoff-Routing | Flood Frequency (1) | Flood Frequency (2) |
| 10 | 1 760 | 1 450 | 930 |
| 20 | 2 220 | 2 190 | 1 350 |
| 50 | 2 870 | 3 220 | 1 920 |
| 100 | 3 430 | 3 990 | 2 350 |

Where: (1) Based upon individual station record, 25 years.
(2) Based upon composite station record, 82 years.

The comparison in Table 6.20 shows that the runoff-routing model estimates agree reasonably well with the 10 and 20 year ARI estimates obtained from the flood frequency analysis of the individual station record. As mentioned in Section 5.4, design initial losses of 17.5 mm for the 10 Year ARI event and 4 mm for the 20 Year ARI event are required to equate the estimates obtained from the two techniques.

The corresponding 50 and 100 year ARI runoff-routing estimates are much lower than the values derived from the flood frequency techniques. However, because of the short period of record available at this station, the estimates obtained from the flood frequency analysis for events greater than the 10 year ARI should be viewed cautiously.

It should also be stressed that this site has a highest streamflow measurement of only 595 $m^3/s$ and that the flows under discussion are in the extrapolated range of the rating curve. Overbank flooding is extensive at this site and as a consequence the high flow portion of the rating curve is only fair.

The quality of record of the composite record is even more suspect than the individual station record which means the estimates of peak discharge obtained from the flood frequency analysis of this record are even less reliable.

The stations used to formulate the composite record include Brightveiw Weir and Tarampa which have poor definition of the high flow portions of their respective rating curves. The results of the composite record frequency analysis do indicate that the period from 1910 to 1990 is drier than the period 1965 to 1990.

Overall, when these factors are taken into account, the comparison illustrates that the runoff-routing model estimates appear reasonable and not too dissimilar from the flood frequency estimates.

Tables 6.21 to 6.23 provide peak discharge estimates of the 10 Year ARI and 20 Year ARI design flood events for the other operational procedures for Wivenhoe Dam and show a comparsion between the application of design initial loss rates.

**Table 6.21**

**Comparison of Design Flood Estimates**
**24 Hour Storm Centred Over Lockyer Creek**
**Wivenhoe Dam Procedure 3**

| Loss | ARI (Yrs) | Peak Discharge ($m^3/s$) | | |
|---|---|---|---|---|
| | | Lyons Bridge | Wivenhoe Dam | Port Office Gauge |
| No IL | 10<br>20 | 1 660<br>2 100 | 3 320<br>3 420 | 5 680<br>6 490 |
| Var IL | 10<br>20 | 1 310<br>2 010 | 3 040<br>3 360 | 5 170<br>6 330 |
| Uni IL | 10<br>20 | 1 220<br>1 760 | 3 140<br>3 370 | 5 050<br>6 060 |

**Table 6.22**

**Comparison of Design Flood Estimates**
**24 Hour Storm Centred Over Lockyer Creek**
**Wivenhoe Dam Procedure 2**

| Loss | ARI (Yrs) | Peak Discharge ($m^3/s$) | | |
|---|---|---|---|---|
| | | Lyons Bridge | Wivenhoe Dam | Port Office Gauge |
| No IL | 10<br>20 | 1 660<br>2 100 | 1 750<br>2 220 | 3 640<br>4 520 |
| Var IL | 10<br>20 | 1 310<br>2 010 | 1 580<br>2 210 | 3 160<br>4 380 |
| Uni IL | 10<br>20 | 1 220<br>1 760 | 1 590<br>1 950 | 2 930<br>3 990 |

**Table 6.23**

**Comparison of Design Flood Estimates**
**24 Hour Storm Centred over Lockyer Creek**
**Wivenhoe Dam Procedure 1**

| Loss | ARI (Yrs) | Peak Discharge ($m^3/s$) | | |
|---|---|---|---|---|
| | | Lyons Bridge | Wivenhoe Dam | Port Office Gauge |
| No IL | 10<br>20 | 1 660<br>2 100 | 1 600<br>1 600 | 3 340<br>4 250 |
| Var IL | 10<br>20 | 1 310<br>2 010 | 1 590<br>1 600 | 3 160<br>4 130 |
| Uni IL | 10<br>20 | 1 220<br>1 760 | 1 590<br>1 600 | 2 550<br>3 640 |

General Notes pertaining to Tables 6.21 to 6.23:

Continuing Loss = 2.5 mm/hour for all ARIs.
No IL = 0 mm over whole catchment for all ARIs.
Variable IL = Initial Loss Rates as per Table 5.8.
Uniform IL = 22.9 mm over whole catchment for 10 Yr ARI.
Uniform IL = 14.9 mm over whole catchment for 20 Yr ARI.

Figure 6.10 presents the flood frequency distribution for Lockyer Creek at Lyons Bridge based on the runoff-routing model estimates for the 24 hour duration design storm centred over Lockyer Creek. Also shown for comparative purposes, are the frequency distributions derived from flood frequency techniques.

Figure 6.11 presents a range of estimated design flood hydrographs for Lockyer Creek at Lyons Bridge.

### 6.4 Bremer River @ David Trumpy Bridge

Design flood estimates for the Bremer River catchment at the David Trumpy Bridge, (Ipswich), were calculated so that an estimate of the magnitude of floods from this sub-basin could be made. This analysis was performed as a check on the capability of the existing operation procedures of the dams to cope with large floods centred in adjacent catchments.

Estimates of flood hydrographs from the Bremer River catchment also provide an indication of the magnitude of flooding that could result from the unregulated part of the Brisbane River catchment.

Design flood estimates for Bremer River were calculated assuming that the dams were at full supply level prior to the event and that the existing normal operation procedures of the dams were utilised. The PMP storm rainfall was assumed to be centred over the Bremer River catchment with 100 year ARI rainfall falling in the remaining adjacent catchments.

In all cases an initial loss of 0 mm and a continuing loss rate of 2.5 mm/hour was adopted. The rainfall depths provided in Table 5.2 and Table 5.4 were utilised for the assessment of the probable maximum flood, (PMF), storms. Table 6.24 provides a summary of the PMF design storm scenarios.

**Table 6.24**

**Critical Design Flood Estimates**
**Bremer River at David Trumpy Bridge**
**PMF Storm Centred Over Bremer River**

| Duration (Hours) | Peak Discharge ($m^3/s$) | Flood Volume (ML) |
|---|---|---|
| 12 | 15 180 | 1 156 370 |
| 24 | 17 450 | 1 635 260 |
| 48 | 13 360 | 2 101 700 |
| 72 | 12 740 | 2 919 660 |

The critical duration for the catchment at the David Trumpy Bridge was found to be the 24 hour event.

Likewise an assessment of the 100 year ARI event was undertaken with the results summarised in Table 6.25. The critical duration for the 100 year ARI event is also 24 hours. A full flood frequency has been determined for the Bremer River catchment with 24 hours being adopted as the critical duration.

**Table 6.25**

**Critical Design Flood Estimates**
**Bremer River at David Trumpy Bridge**
**100 Year ARI Storm Centred Over Bremer River**

| Duration (Hours) | Peak Discharge ($m^3/s$) | Flood Volume (ML) |
|---|---|---|
| 12 | 2 840 | 289 030 |
| 24 | 3 010 | 362 880 |
| 48 | 2 320 | 408 170 |
| 72 | 2 190 | 440 120 |

A comparison between the estimates obtained from the runoff-routing modelling can be made with estimates presented for the Ipswich City Gauge that are shown on 'Revision A - Brisbane River Flood Plain Map Series Sheets 1 - 18', which were compiled by the BCC.

These estimates are summarised in Table 6.26 and are based on a regional flood frequency analysis of 10 stations in the Brisbane River Valley. It is believed that the Boughton Empirical distribution was used as the basis of the regional flood frequency analysis.

**Table 6.26**

**Flood Frequency Estimates**
**Bremer River at Ipswich City Gauge**
**BCC - Boughton Empirical Distribution**
**Somerset Dam & Wivenhoe Dam Effective**

| ARI (Years) | Gauge Height (m) | Peak Discharge ($m^3/s$) |
|---|---|---|
| 11 | 8.5 | 1 100 |
| 28 | 12.0 | 1 620 |
| 60 | 14.5 | 2 100 |
| 110 | 16.4 | 2 540 |
| 200 | 18.0 | 3 050 |

Table 6.27 presents a range of flood frequencies for storms centred over the Bremer River catchment with a duration of 24 hours. Shown in this table are the associated outflows from Wivenhoe Dam and the resultant peak discharge at the Port Office Gauge.

**Table 6.27**

**Design Flood Frequency Estimates**
**Bremer River @ David Trumpy Bridge**
**24 Hour Storm Centred Over Bremer River**

| ARI (Yrs) | Peak Discharge ($m^3/s$) | | | Flood Volume (ML) | | |
|---|---|---|---|---|---|---|
| | David Trumpy Bridge | Wivenhoe Dam | Port Office | David Trumpy Bridge | Wivenhoe Dam | Port Office |
| * 10 | 1 580 | 3 330 | 5 625 | 205 810 | 822 950 | 1 494 130 |
| * 20 | 1 990 | 3 420 | 6 520 | 251 990 | 995 160 | 1 823 200 |
| * 50 | 2 520 | 3 470 | 7 650 | 310 760 | 1 242 700 | 2 280 380 |
| * 100 | 3 010 | 5 760 | 8 700 | 362 880 | 1 481 210 | 2 698 870 |
| 200 | 3 820 | 5 040 | 9 240 | 418 780 | 1 437 640 | 2 763 360 |
| 500 | 4 660 | 5 040 | 9 940 | 498 550 | 1 437 640 | 2 763 360 |
| 1 000 | 5 520 | 5 040 | 10 640 | 578 320 | 1 437 640 | 2 843 140 |
| 10 000 | 9 250 | 5 040 | 13 810 | 917 340 | 1 437 640 | 3 182 210 |
| 100 000 | 13 540 | 5 040 | 17 700 | 1 296 240 | 1 437 640 | 3 561 170 |
| PMF | 17 450 | 5 040 | 21 390 | 1 635 260 | 1 437 640 | 3 900 240 |

Note: * Temporal patterns from Australian Rainfall and Runoff, (1987), used.

Tables 6.28 to 6.30 provide peak discharge estimates of the 10 Year ARI and 20 Year ARI design flood events for the other operational procedures for Wivenhoe Dam and show a comparsion between the application of design initial loss rates.

**Table 6.28**

**Comparison of Design Flood Estimates**
**24 Hour Storm Centred Over Bremer River**
**Wivenhoe Dam Procedure 3**

| Loss | ARI (Yrs) | Peak Discharge ($m^3/s$) | | |
|---|---|---|---|---|
| | | David Trumpy Bridge | Wivenhoe Dam | Port Office Gauge |
| No IL | 10<br>20 | 1 580<br>1 990 | 3 320<br>3 420 | 5 630<br>6 520 |
| Var IL | 10<br>20 | 1 490<br>1 990 | 2 910<br>3 370 | 5 170<br>6 380 |
| Uni IL | 10<br>20 | 1 230<br>1 720 | 3 080<br>3 370 | 5 080<br>6 100 |

**Table 6.29**

**Comparison of Design Flood Estimates**
**24 Hour Storm Centred Over Bremer River**
**Wivenhoe Dam Procedure 2**

| Loss | ARI (Yrs) | Peak Discharge ($m^3/s$) | | |
|---|---|---|---|---|
| | | David Trumpy Bridge | Wivenhoe Dam | Port Office Gauge |
| No IL | 10<br>20 | 1 580<br>1 990 | 1 990<br>2 550 | 3 670<br>4 620 |
| Var IL | 10<br>20 | 1 490<br>1 990 | 1 590<br>2 440 | 3 180<br>4 490 |
| Uni IL | 10<br>20 | 1 230<br>1 720 | 1 590<br>2 190 | 2 970<br>4 090 |

**Table 6.30**

**Comparison of Design Flood Estimates**
**24 Hour Storm Centred Over Bremer River**
**Wivenhoe Dam Procedure 1**

| Loss | ARI (Yrs) | Peak Discharge ($m^3/s$) | | |
|---|---|---|---|---|
| | | David Trumpy Bridge | Wivenhoe Dam | Port Office Gauge |
| No IL | 10<br>20 | 1 580<br>1 990 | 1 600<br>1 600 | 3 540<br>4 440 |
| Var IL | 10<br>20 | 1 490<br>1 990 | 1 590<br>1 600 | 2 980<br>4 320 |
| Uni IL | 10<br>20 | 1 230<br>1 720 | 1 590<br>1 600 | 2 770<br>3 940 |

General Notes pertaining to Tables 6.28 to 6.30:

Continuing Loss = 2.5 mm/hour for all ARIs.
No IL = 0 mm over whole catchment for all ARIs.
Variable IL = Initial Loss Rates as per Table 5.8.
Uniform IL = 22.9 mm over whole catchment for 10 Yr ARI.
Uniform IL = 14.9 mm over whole catchment for 20 Yr ARI.

Figure 6.12 presents the flood frequency distribution for the Bremer River at the David Trumpy Bridge, based on the runoff-routing model estimates for the 24 hour duration design storm centred over the Bremer River. Also shown for comparative purposes, is the frequency distribution derived by the BCC in earlier studies.

Figure 6.13 presents a range of estimated design flood hydrographs for the Bremer River at the David Trumpy Bridge. As with the BCC estimates, no allowance has been made for backwater effects from the Brisbane River on the peak discharge estimates. This issue will be the addressed in a future report regarding the hydraulic modelling of the Brisbane-Bremer River system.

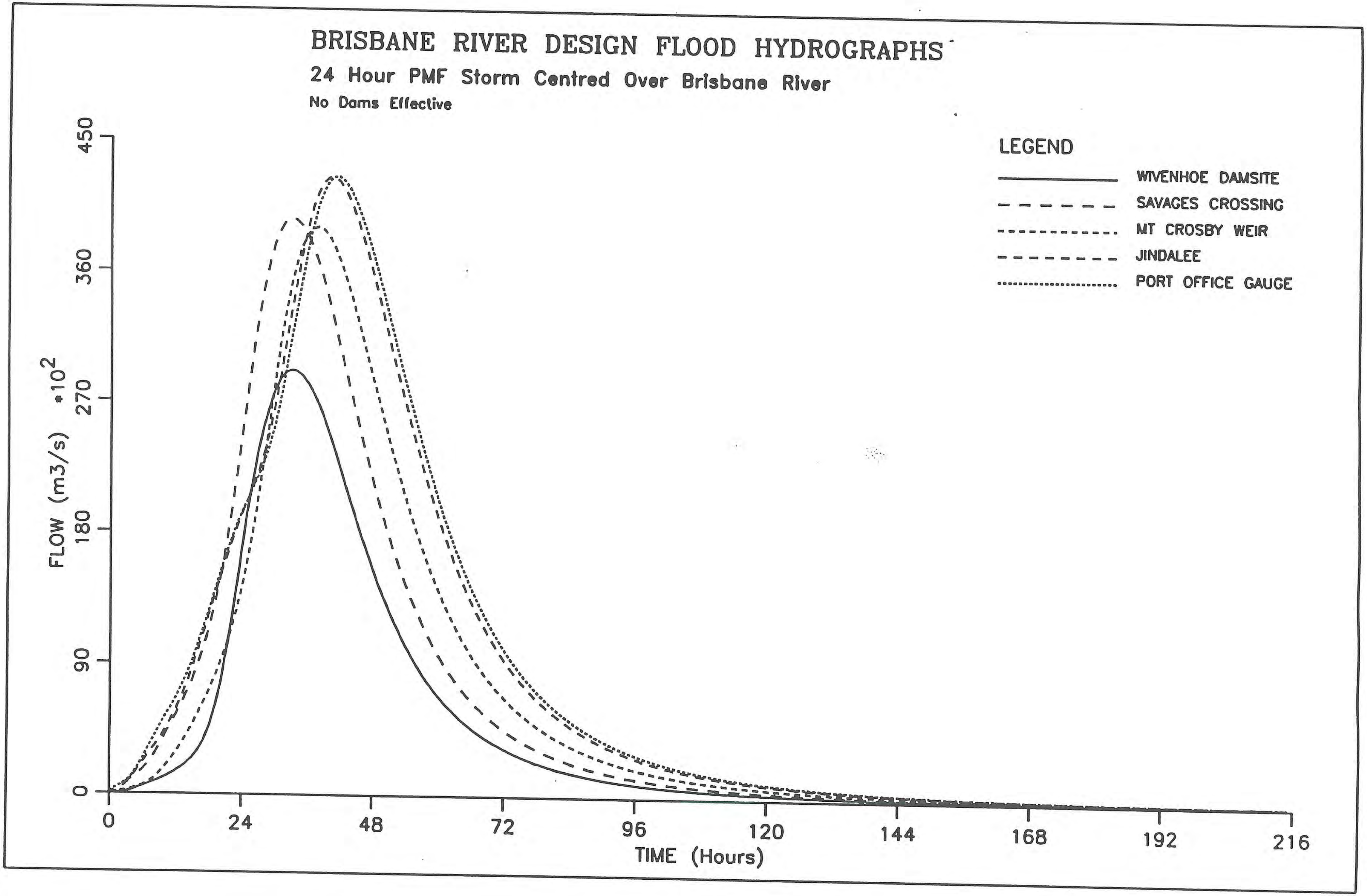

BRISBANE RIVER DESIGN FLOOD HYDROGRAPHS
24 Hour PMF Storm Centred Over Brisbane River
No Dams Effective
LEGEND
WIVENHOE DAMSITE
SAVAGES CROSSING
MT CROSBY WEIR
JINDALEE
PORT OFFICE GAUGE
FLOW (m3/s) *10^2
0
90
180
270
360
450
0
24
48
72
96
120
144
168
192
216
TIME (Hours)

Figure 6.1

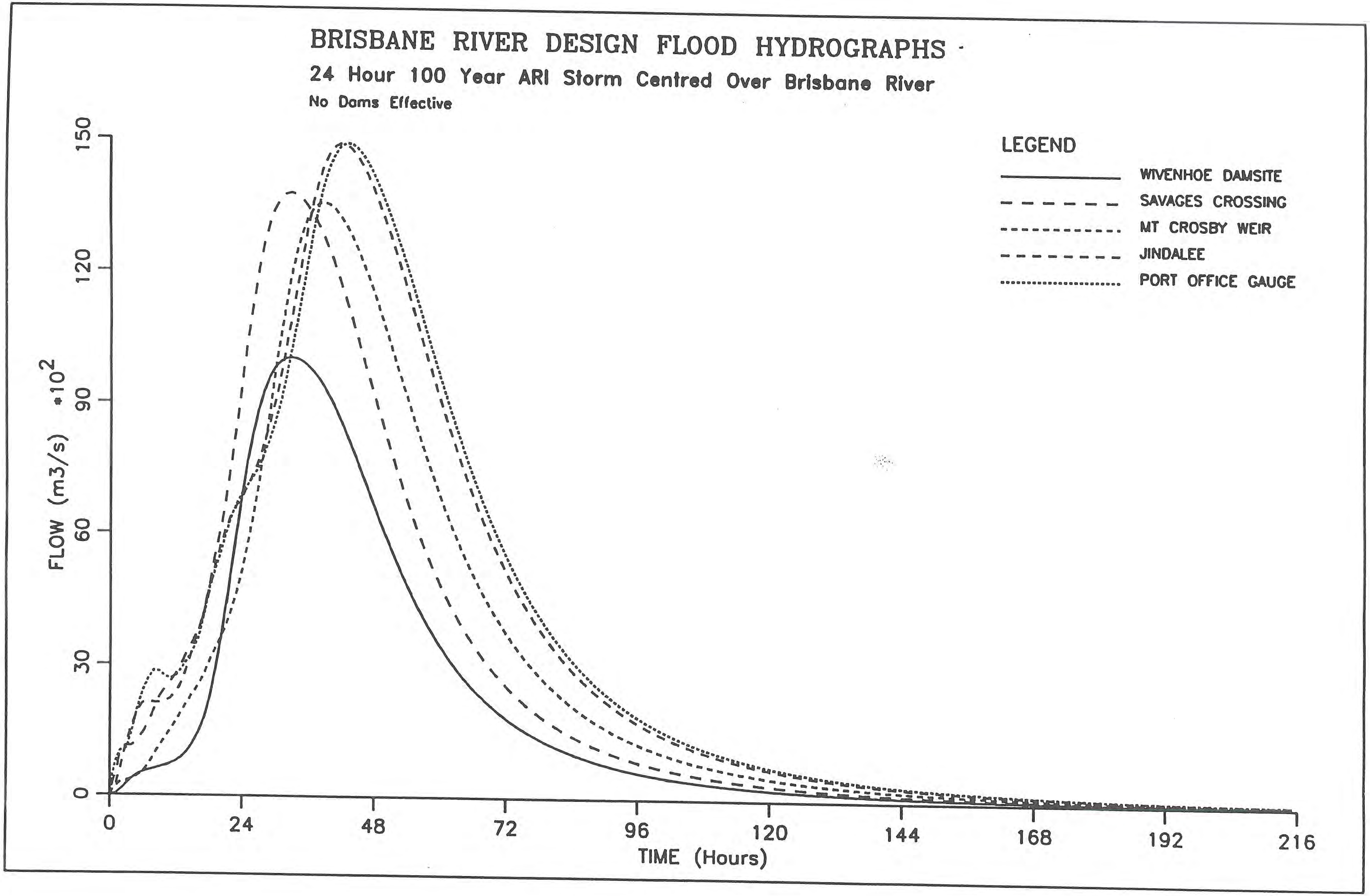
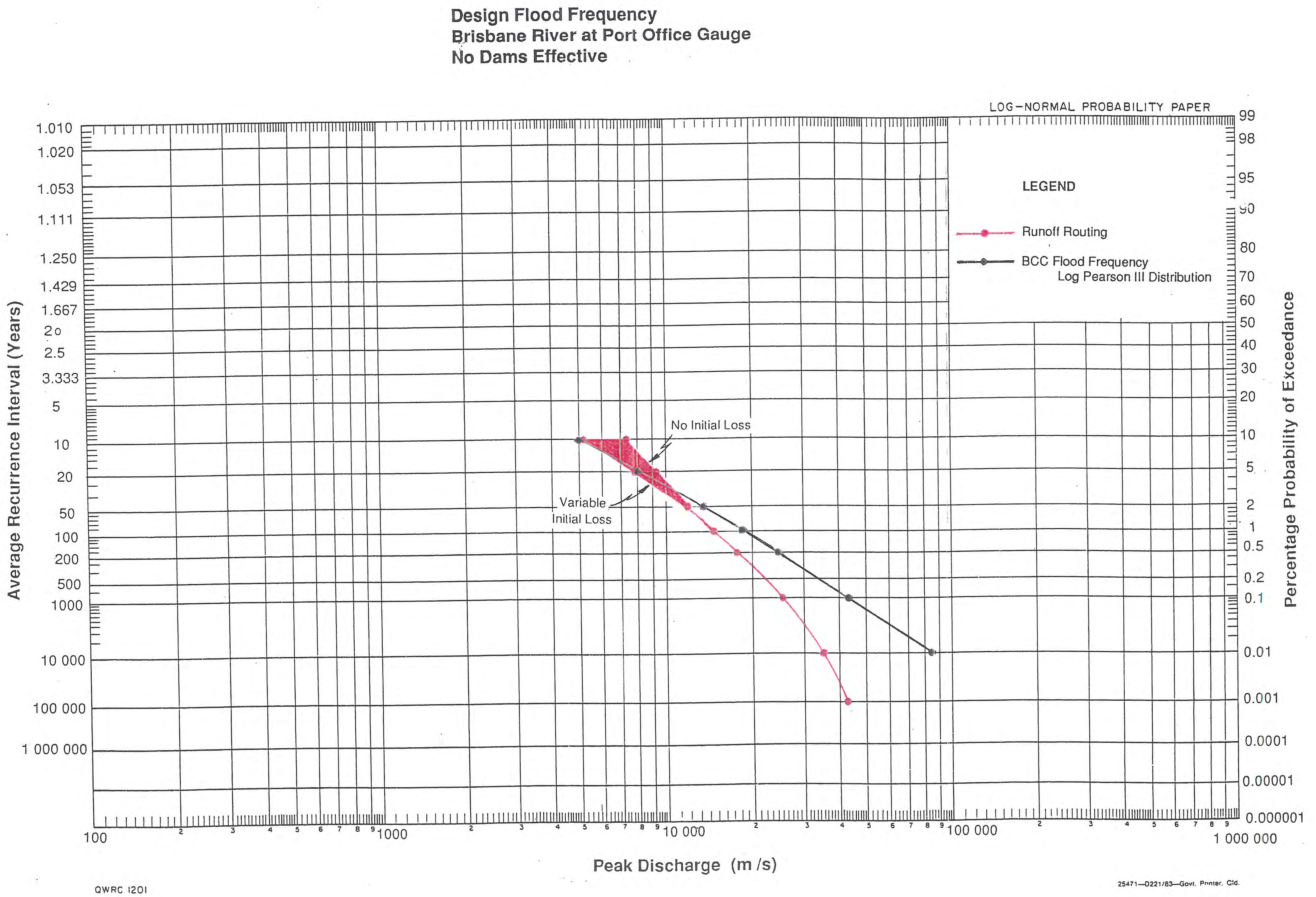

BRISBANE RIVER DESIGN FLOOD HYDROGRAPHS
24 Hour 100 Year ARI Storm Centred Over Brisbane River
No Dams Effective
LEGEND
WIVENHOE DAMSITE
SAVAGES CROSSING
MT CROSBY WEIR
JINDALEE
PORT OFFICE GAUGE
FLOW (m3/s) *10^2
0
30
60
90
120
150
TIME (Hours)
0
24
48
72
96
120
144
168
192
216

Figure 6.2

Figure 6.3


Design Flood Frequency
Brisbane River at Port Office Gauge
No Dams Effective
LOG-NORMAL PROBABILITY PAPER
LEGEND
Runoff Routing
BCC Flood Frequency
Log Pearson III Distribution
No Initial Loss
Variable
Initial Loss
Average Recurrence Interval (Years)
Percentage Probability of Exceedance
Peak Discharge (m /s)
100
1000
10 000
100 000
1 000 000
QWRC 1201
25471—D221/83—Govt. Printer, Qld.

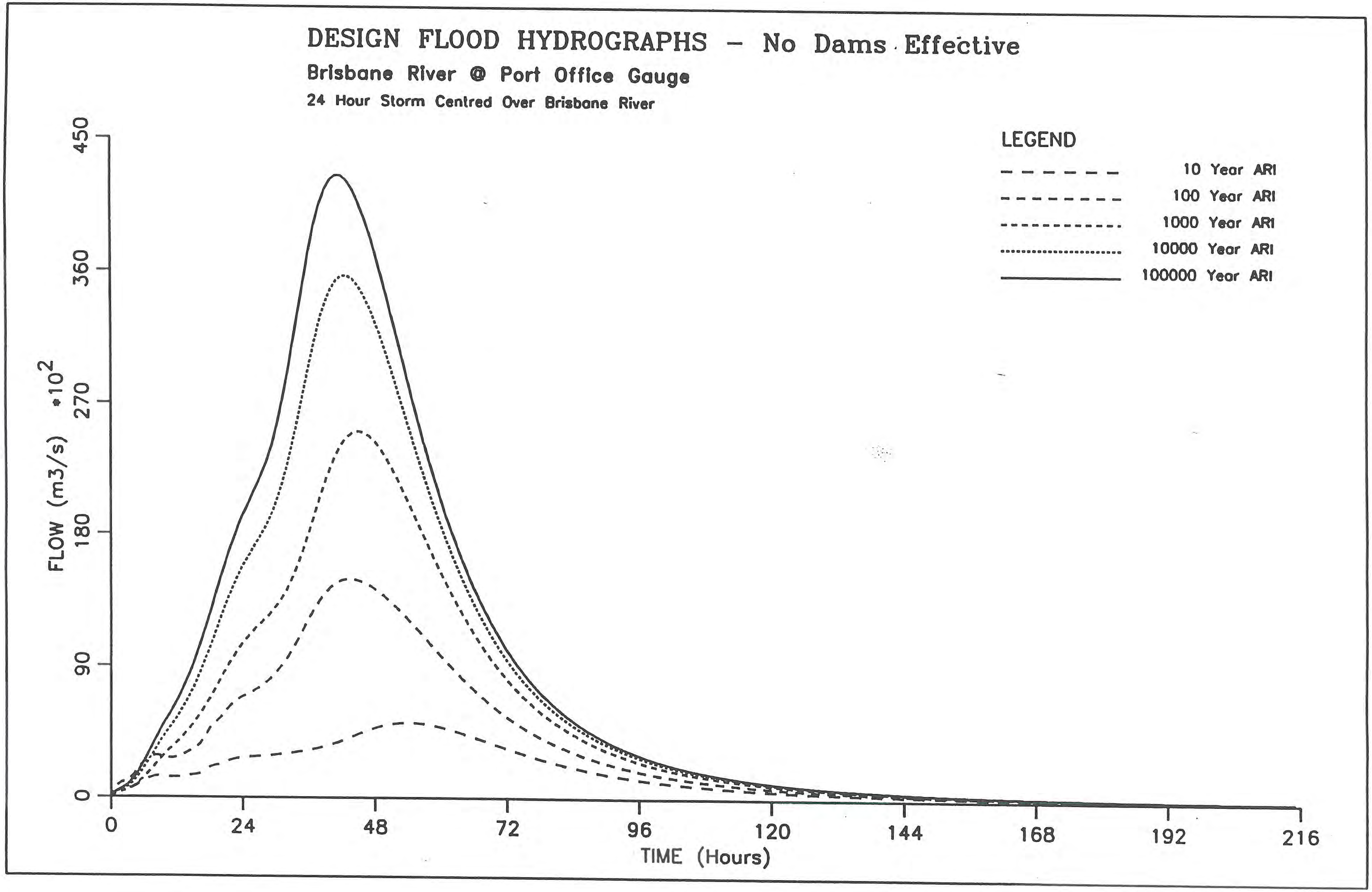

DESIGN FLOOD HYDROGRAPHS – No Dams Effective
Brisbane River @ Port Office Gauge
24 Hour Storm Centred Over Brisbane River
LEGEND
10 Year ARI
100 Year ARI
1000 Year ARI
10000 Year ARI
100000 Year ARI
FLOW (m3/s) *10^2
0
90
180
270
360
450
0
24
48
72
96
120
144
168
192
216
TIME (Hours)

Figure 6.4

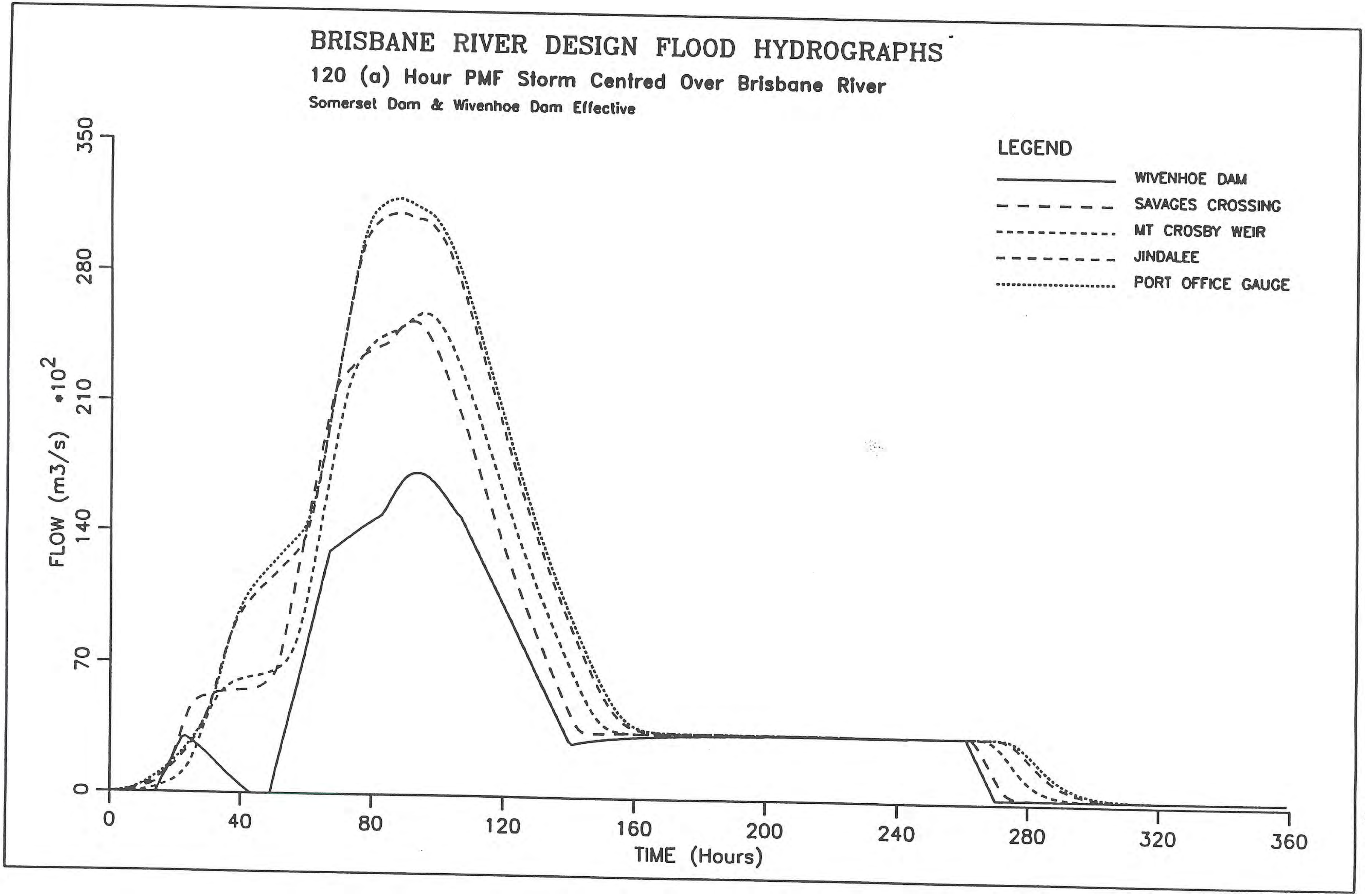

BRISBANE RIVER DESIGN FLOOD HYDROGRAPHS
120 (a) Hour PMF Storm Centred Over Brisbane River
Somerset Dam & Wivenhoe Dam Effective
LEGEND
WIVENHOE DAM
SAVAGES CROSSING
MT CROSBY WEIR
JINDALEE
PORT OFFICE GAUGE
FLOW (m3/s) *10^2
0
70
140
210
280
350
TIME (Hours)
0
40
80
120
160
200
240
280
320
360

Figure 6.5

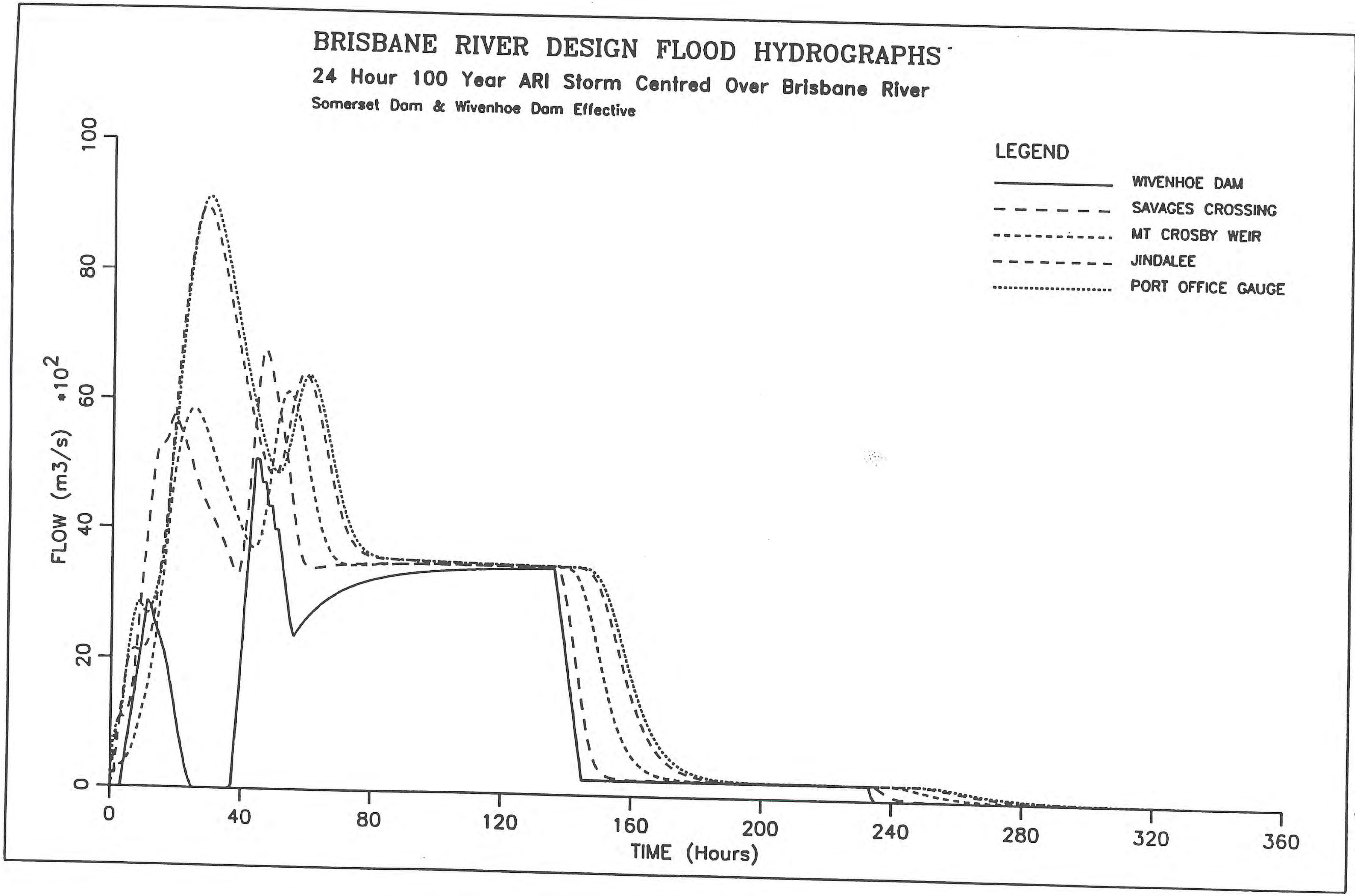
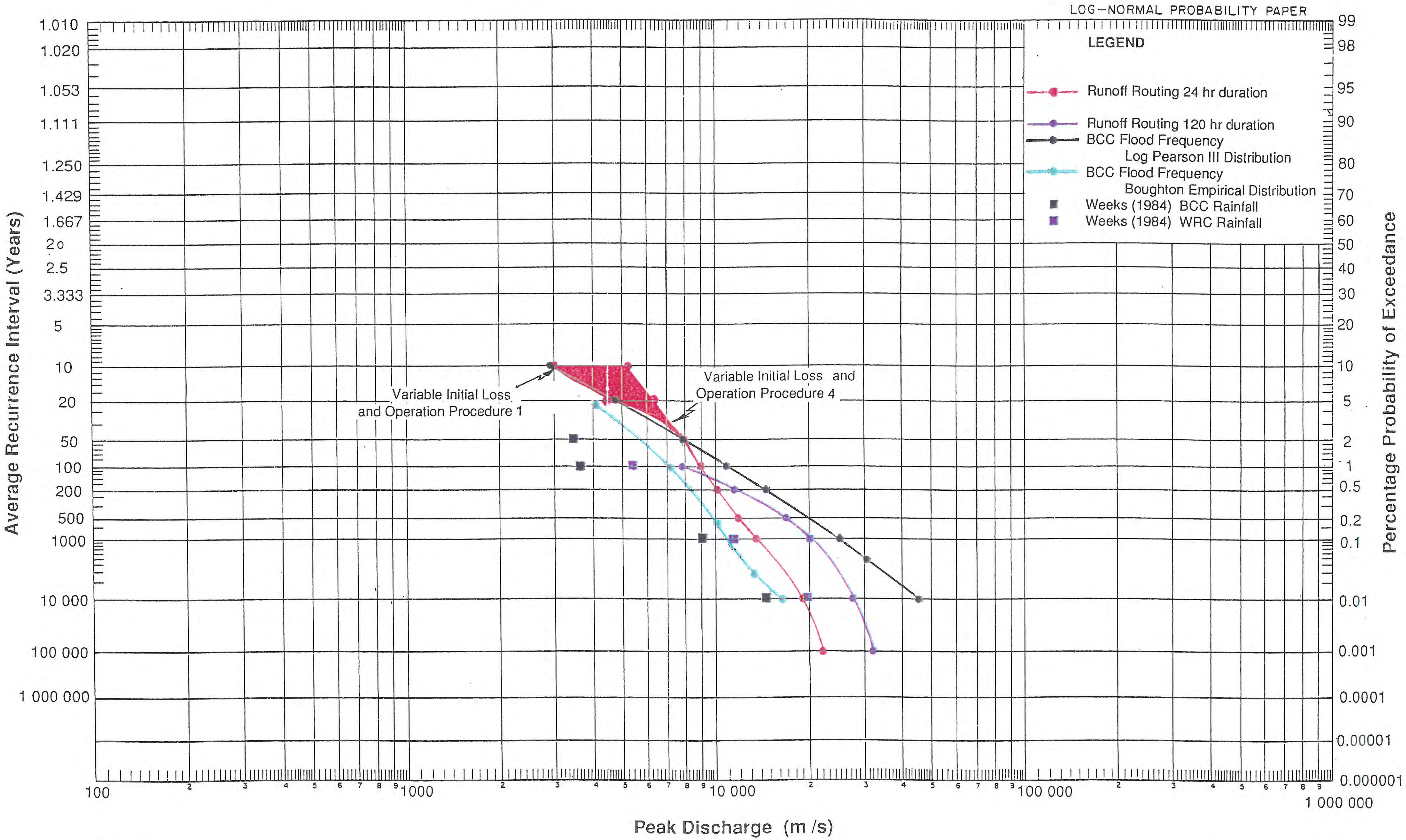

BRISBANE RIVER DESIGN FLOOD HYDROGRAPHS
24 Hour 100 Year ARI Storm Centred Over Brisbane River
Somerset Dam & Wivenhoe Dam Effective
LEGEND
WIVENHOE DAM
SAVAGES CROSSING
MT CROSBY WEIR
JINDALEE
PORT OFFICE GAUGE
FLOW (m3/s) *10²
TIME (Hours)
0
20
40
60
80
100
0
40
80
120
160
200
240
280
320
360

Figure 6.6

## Design Flood Frequency
## Brisbane River at Port Office Gauge
## Somerset Dam & Wivenhoe Dam Effective



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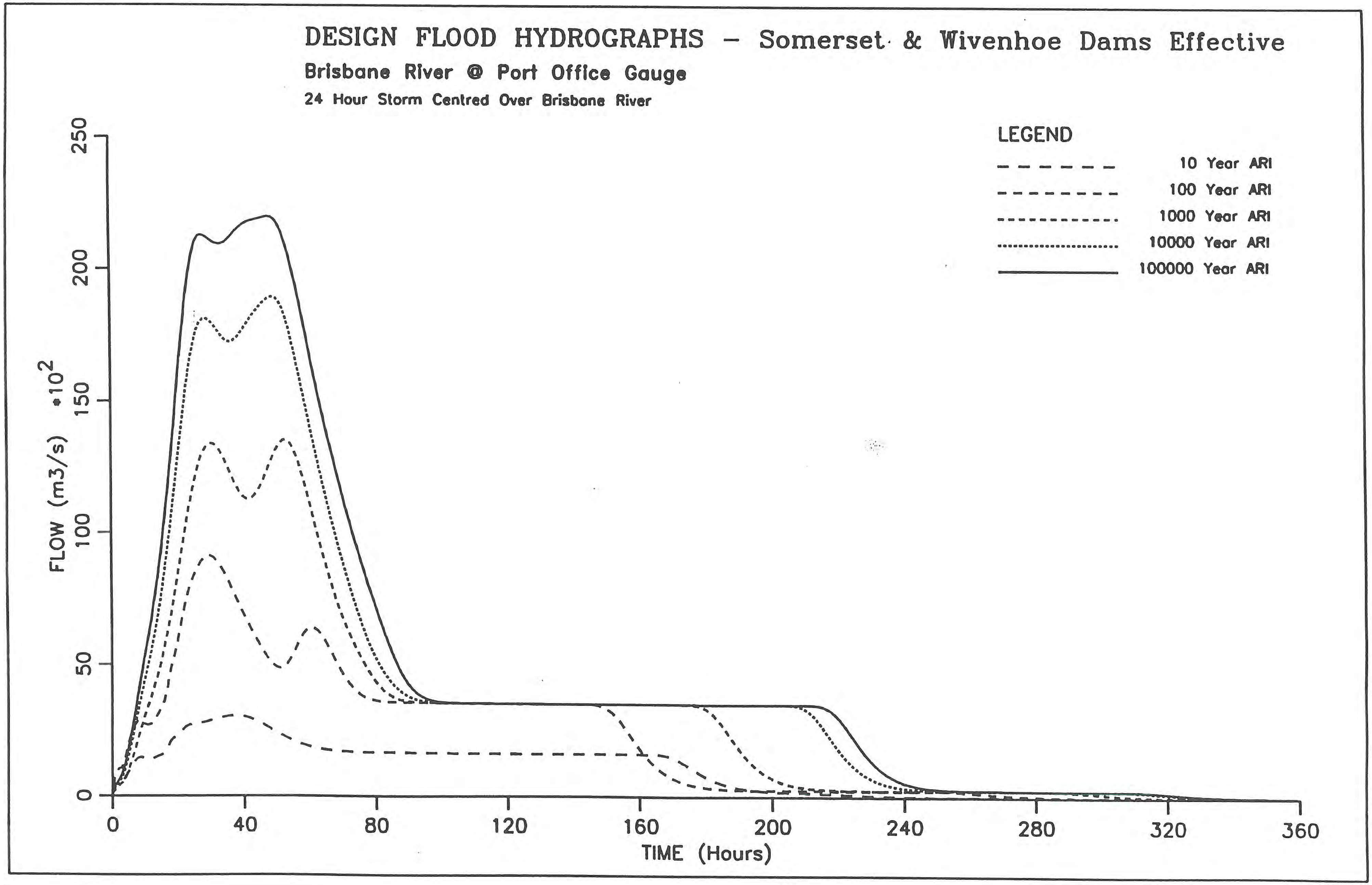

DESIGN FLOOD HYDROGRAPHS - Somerset & Wivenhoe Dams Effective
Brisbane River @ Port Office Gauge
24 Hour Storm Centred Over Brisbane River
LEGEND
10 Year ARI
100 Year ARI
1000 Year ARI
10000 Year ARI
100000 Year ARI
FLOW (m3/s) *10^2
0
50
100
150
200
250
0
40
80
120
160
200
240
280
320
360
TIME (Hours)
Figure 6.8

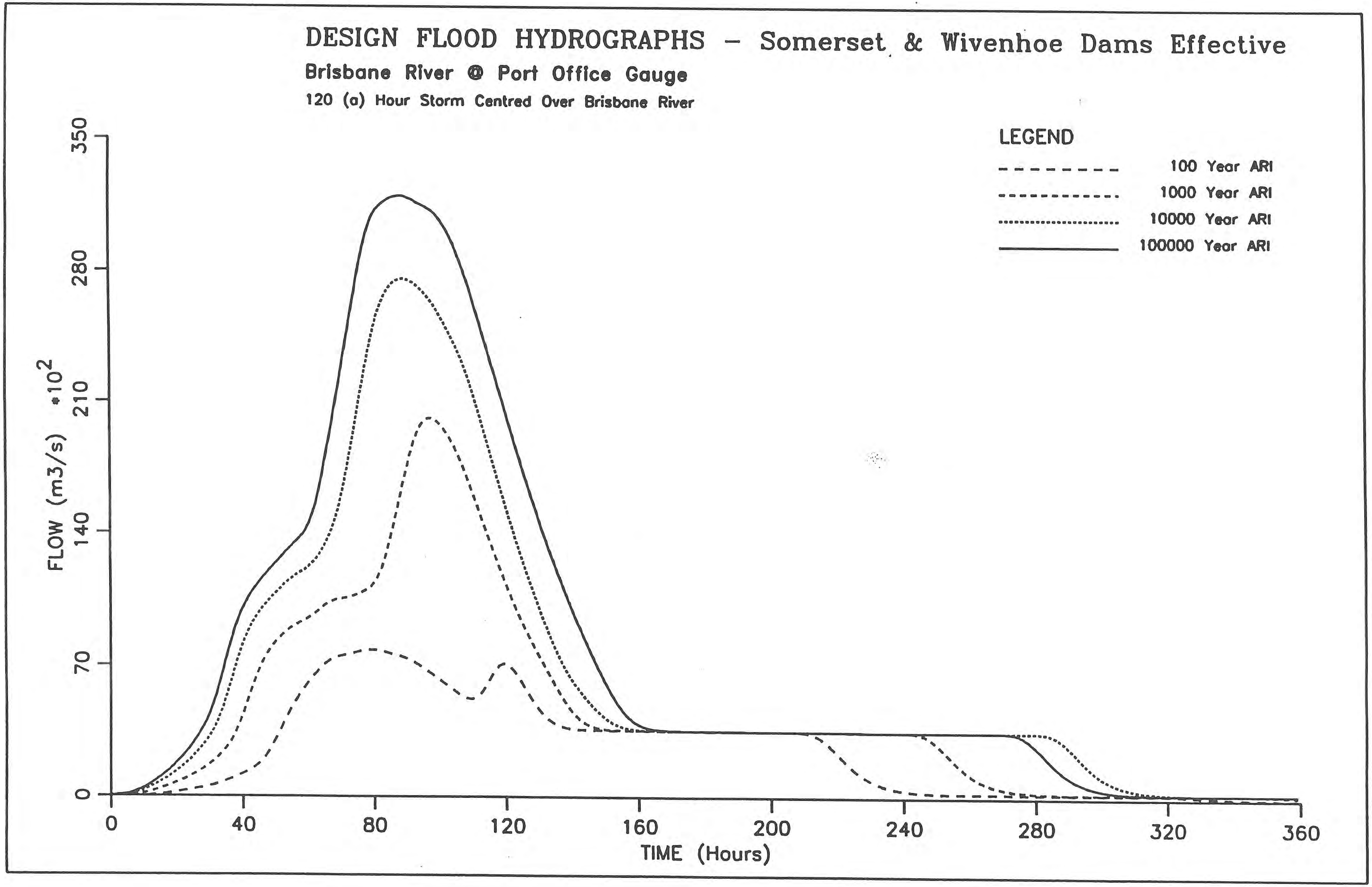
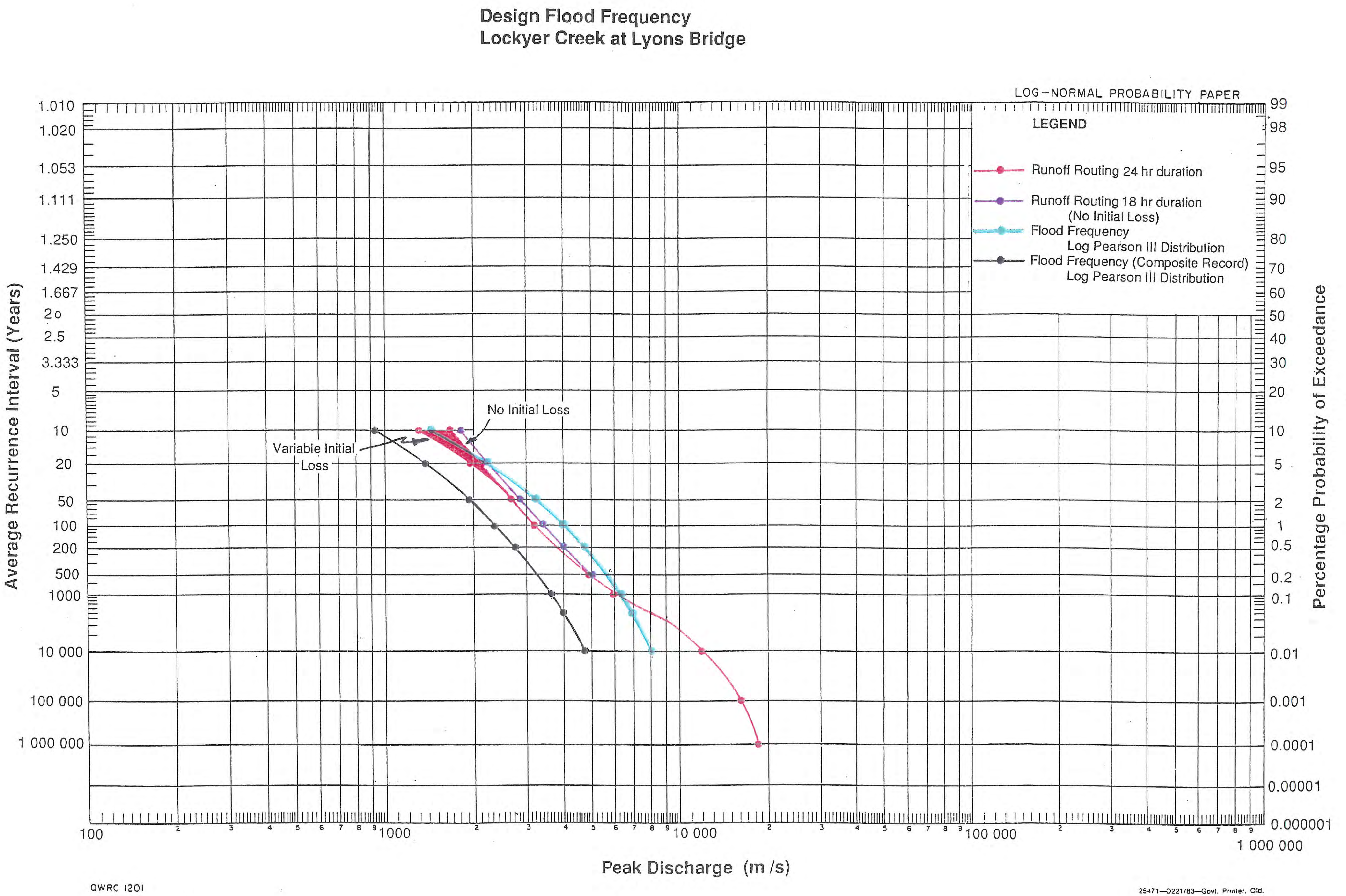

DESIGN FLOOD HYDROGRAPHS - Somerset & Wivenhoe Dams Effective
Brisbane River @ Port Office Gauge
120 (a) Hour Storm Centred Over Brisbane River
LEGEND
100 Year ARI
1000 Year ARI
10000 Year ARI
100000 Year ARI
FLOW (m3/s) *10^2
0
70
140
210
280
350
TIME (Hours)
0
40
80
120
160
200
240
280
320
360

Figure 6.9


Design Flood Frequency
Lockyer Creek at Lyons Bridge
LOG-NORMAL PROBABILITY PAPER
LEGEND
Runoff Routing 24 hr duration
Runoff Routing 18 hr duration (No Initial Loss)
Flood Frequency Log Pearson III Distribution
Flood Frequency (Composite Record) Log Pearson III Distribution
No Initial Loss
Variable Initial Loss
Average Recurrence Interval (Years)
Percentage Probability of Exceedance
Peak Discharge (m /s)
100
1000
10 000
100 000
1 000 000
QWRC 1201
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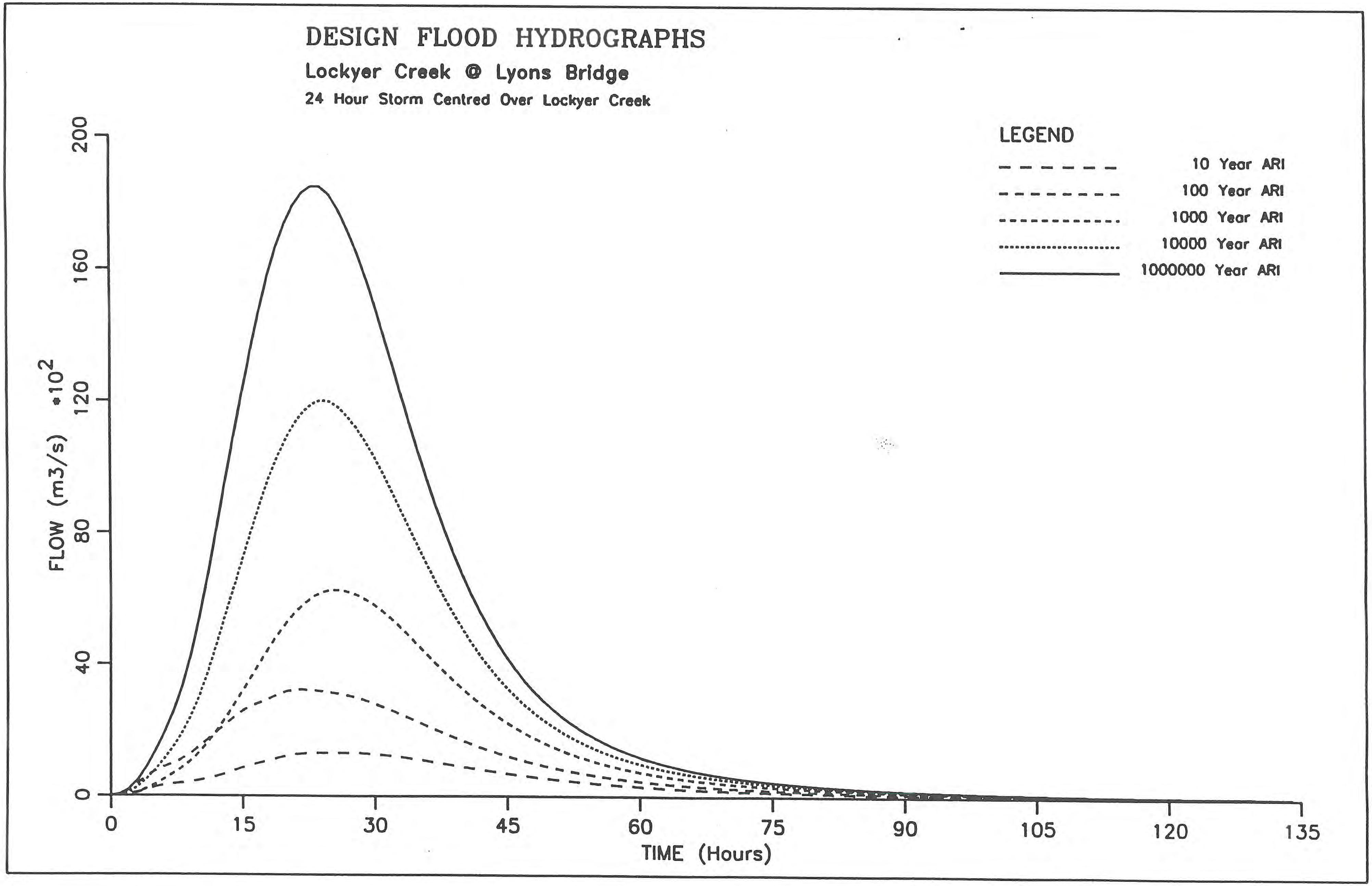
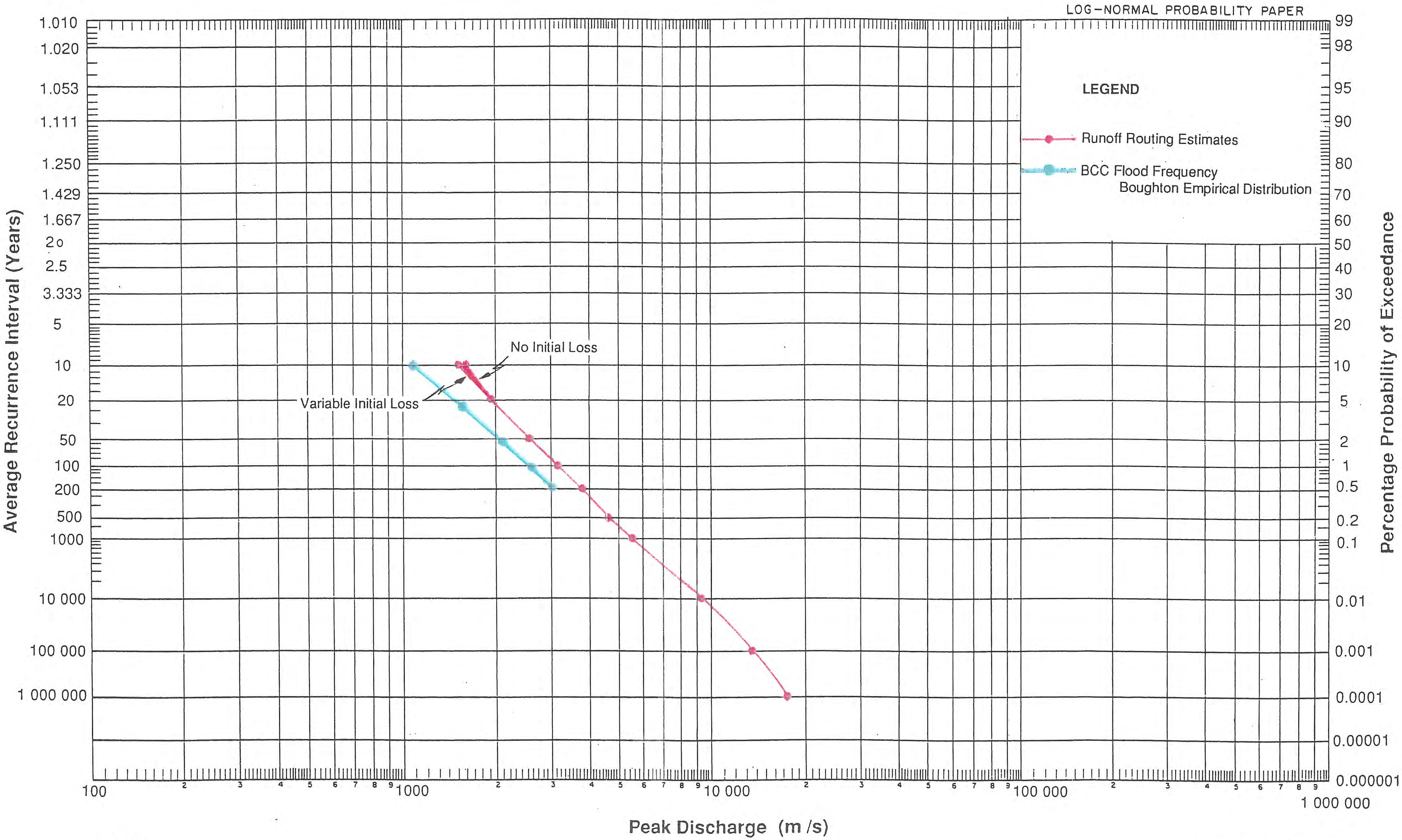

DESIGN FLOOD HYDROGRAPHS
Lockyer Creek @ Lyons Bridge
24 Hour Storm Centred Over Lockyer Creek
LEGEND
10 Year ARI
100 Year ARI
1000 Year ARI
10000 Year ARI
1000000 Year ARI
FLOW (m3/s) *10^2
0
40
80
120
160
200
0
15
30
45
60
75
90
105
120
135
TIME (Hours)

Figure 6.11

Figure 6.12

## Design Flood Frequency
## Bremer River at David Trumpy Bridge

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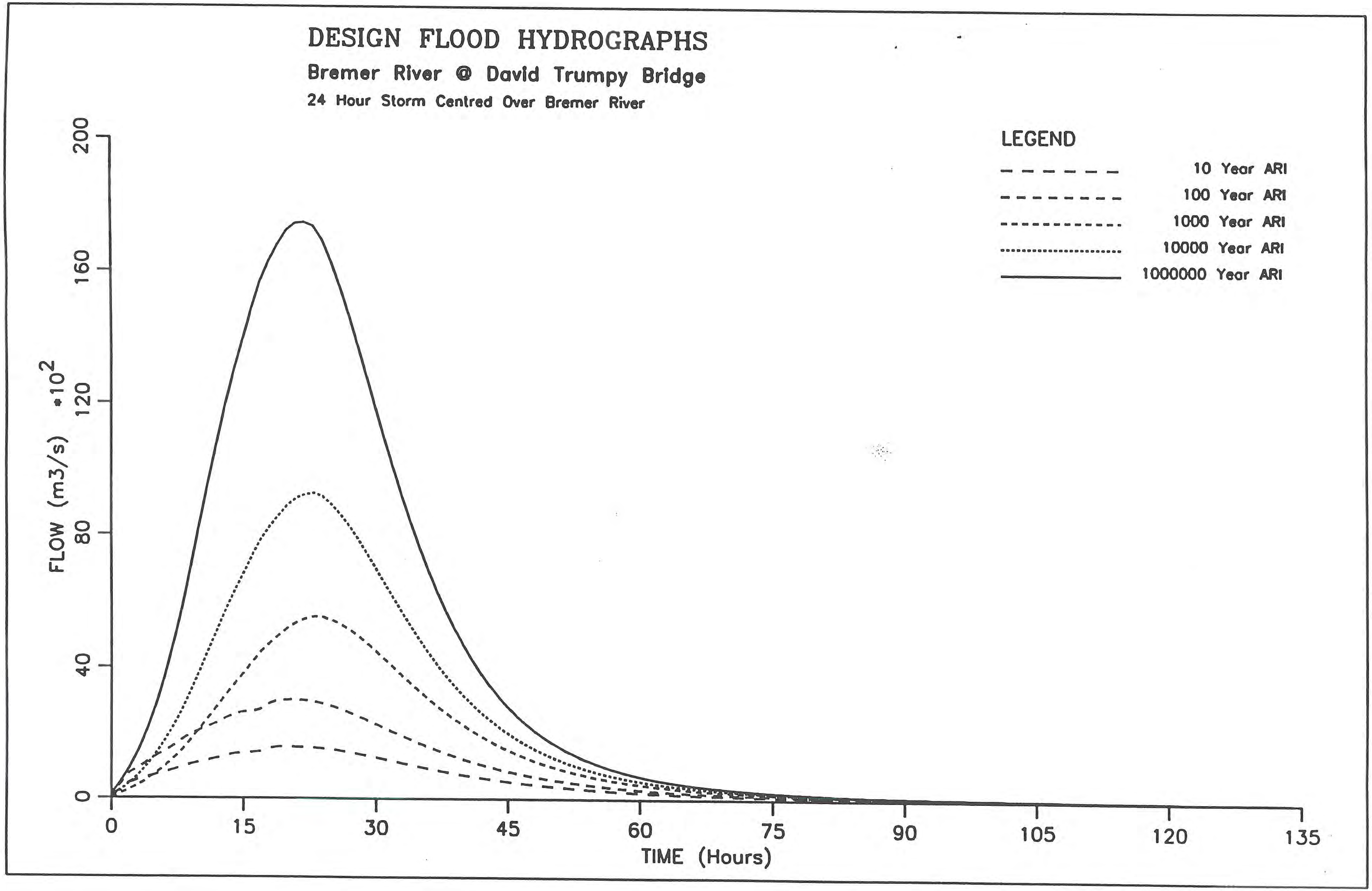


DESIGN FLOOD HYDROGRAPHS
Bremer River @ David Trumpy Bridge
24 Hour Storm Centred Over Bremer River
LEGEND
10 Year ARI
100 Year ARI
1000 Year ARI
10000 Year ARI
1000000 Year ARI
FLOW (m3/s) *10^2
0
40
80
120
160
200
TIME (Hours)
0
15
30
45
60
75
90
105
120
135


Figure 6.13

# 7.0 SUMMARY AND CONCLUSIONS

## 7.1 General

The design floods for the catchment downstream of 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).

Comparisons between estimates of PMP rainfalls used in each of the studies for the subject catchments indicate that the PMP rainfalls are not very different and neither are the associated temporal patterns.

For example, the 24 hour duration PMP estimates for a storm centred over the whole of the Brisbane River catchment has decreased from 560 mm to 530 mm, and the 144 hour duration PMP estimates have also decreased from 1 100 mm to 1 070 mm. Storms centred over the Wivenhoe Dam catchment have actually increased from 600 mm to 670 mm for the 24 hour duration event, but decreased from 1 560 mm to 1 330 mm for the 144 hour duration. Table 7.1 provides a comparison between PMP estimates provided by the Bureau of Meteorology for the previous study and the current study.

**Table 7.1**

**Comparison Between PMP Estimates (mm Depth)**

| Dur (Hrs) | Wivenhoe | | Somerset | | Brisbane | |
|---|---|---|---|---|---|---|
| | 1983 | 1991 | 1983 | 1991 | 1983 | 1991 |
| 6 | 260 | 240 | 400 | 390 | 220 | 200 |
| 12 | 380 | 450 | 560 | 670 | 380 | 370 |
| 24 | 600 | 670 | 840 | 980 | 560 | 530 |
| 48 | 1 000 | 870 | 1 380 | 1 420 | 700 | 680 |
| 72 | 1 260 | 1 080 | 1 760 | 1 770 | 880 | 830 |
| 96 | 1 460 | 1 250 | 2 040 | 2 090 | 1 040 | 1 010 |
| 120 | 1 520 | 1 300 | 2 120 | 2 170 | 1 080 | 1 050 |
| 144 | 1 560 | 1 330 | 2 160 | 2 220 | 1 100 | 1 070 |
| 168 | 1 700 | 1 480 | 2 340 | 2 410 | 1 200 | 1 160 |

There are however, major differences in the design rainfall depth estimates and their associated temporal patterns for the more frequent events of 100 year ARIs and less. Weeks, (1984), used design rainfall estimates based upon frequency analyses of individual daily rainfall stations. The present study utilised the IFD procedures and temporal patterns presented in Australian Rainfall and Runoff, (1987).

The differences in design rainfall estimates are very significant. Table 7.2 provides a summary of some of mean 100 year ARI catchment estimates of design rainfalls that were used in the two studies.

**Table 7.2**

**Comparison Between Design Rainfall Estimates**
**100 Year ARI**
**(mm Depth)**

| Dur (Hrs) | Wivenhoe | | Lockyer | | Bremer | |
|---|---|---|---|---|---|---|
| | 1984 | 1993 | 1984 | 1993 | 1984 | 1993 |
| 24 | 217 | 264 | 162 | 223 | 205 | 238 |
| 48 | 298 | 341 | 222 | 282 | 283 | 307 |
| 72 | 347 | 387 | 255 | 315 | 332 | 348 |

The temporal patterns that were utilised in the two studies for the more frequent events are also quite dissimilar. In 1984, PMP temporal patterns that were provided by the Bureau of Meteorology were adopted for use with the 100 year ARI event. These PMP patterns are more uniform than the temporal patterns shown in Australian Rainfall and Runoff, (1987), which are used in the present study.

There is also a significant difference in the handling of concurrent rainfalls between the two studies. In the 1984 study, rainfall falling on the remainder of the catchment in association with a PMP event was provided by the Bureau of Meteorology.

In the present study, the concurrent rainfall has been assessed in accordance with the methodology outlined in the 1991 report by the Bureau of Meteorology. In essence, this has meant that a 100 year ARI rainfall depth has been applied to the remainder of the catchment. A comparison between the concurrent rainfalls adopted in each study is presented in Table 7.3.

**Table 7.3**

**Comparison Between Concurrent* Rainfall Estimates (mm Depth)**

| Duration (Hours) | Storm Centre | | | |
|---|---|---|---|---|
| | Wivenhoe | | Somerset | |
| | 1984 | 1993 | 1984 | 1993 |
| 6 | 20 | 138 | 140 | 143 |
| 12 | 30 | 186 | 200 | 185 |
| 24 | 50 | 246 | 300 | 244 |
| 48 | 90 | 314 | 500 | 312 |
| 72 | 110 | 353 | 620 | 351 |
| 96 | 130 | 409 | 720 | 408 |
| 120 | 130 | 426 | 760 | 424 |
| 144 | 140 | 435 | 780 | 433 |
| 168 | 150 | 533 | 840 | 530 |

Note: * Rainfall depth over remainder of the Brisbane River catchment that can be expected to accompany the Probable Maximum Precipitation over the catchment of the dam.

## 7.2 Impact of the Dams

The flood mitigation effect of the dams is reflected by the reduction in peak discharge estimated to occur at the Port Office Gauge for the probable maximum floods considered. In the no dams effective case, a peak discharge of 43 990 $m^3/s$ is estimated at the Port Office Gauge, for a 24 hour duration event centred over the catchment of Wivenhoe Dam.

The corresponding peak discharge associated with the case where both dams are effective and operating in accordance with existing normal gate operation procedures, is estimated to be 31 950 $m^3/s$, which results from the 120(a) hour duration storm event centred over the whole of the Brisbane River catchment.

The estimate for no dams effective is substantially smaller than the value derived by Weeks, (1984). Weeks estimated that for the case of 'Without Wivenhoe Dam', a peak discharge of 54 400 $m^3/s$ would occur at the Port Office Gauge for an event with a duration of 144(b) hours. Somerset Dam was assumed to be operational and the storm was assumed to be centred over

Wivenhoe Dam in this scenario. Weeks did not simulate the PMF for the case where both dams are effective, although in an earlier report, (1983), PMF inflows to Wivenhoe Dam were estimated.

The mitigation capability of the dams is illustrated in Figure 7.1. This figure shows the pre and post-dams flood frequency curves based on the results of the runoff-routing modelling undertaken. It should be noted that the post-dams frequency curve is conservative because of the assumption that both dams are at full supply level prior to the onset of the flood event. This is especially so for higher probability of exceedence events.

As stated in Section 7.1, there appears to be little difference between the PMP estimates that were utilised in the 1983/84 study and the present study. The differences in the PMF estimates are therefore due to the different scenarios considered, (ie 'Without Wivenhoe Dam' as opposed to 'No Dams Effective'), the adopted runoff-routing model parameters, and the concurrent rainfall depths assumed to be associated with the PMP event.

There are also significant differences between the two studies in regard to estimates of design floods of more frequent occurrence for the case where both dams are effective and operating in accordance with existing normal gate operation procedures.

In the 1984 study, Weeks estimated that the 48 hour, 100 year ARI event with the storm centred over the whole of the Brisbane River catchment would produce a peak discharge of 5 510 $m^3/s$ at the Port Office Gauge. The corresponding volume of this flood was estimated to be 2 720 000 ML. The present study estimates that a 24 hour, 100 year ARI flood centred over the whole Brisbane River catchment would produce a peak discharge of around 9 120 $m^3/s$, with a corresponding flood volume of 2 696 300 ML.

In the 1984 report, Weeks also indicated that the peak discharge from Wivenhoe Dam would be 3 500 $m^3/s$ for a 100 year ARI storm centred above the dam. The present study shows that Wivenhoe Dam may release up to 5 780 $m^3/s$ during the course of a 100 year ARI event.

The reason for the difference lies with the temporal variation of rainfalls, the magnitudes of concurrent rainfall, and the operational procedure utilised in the two studies.

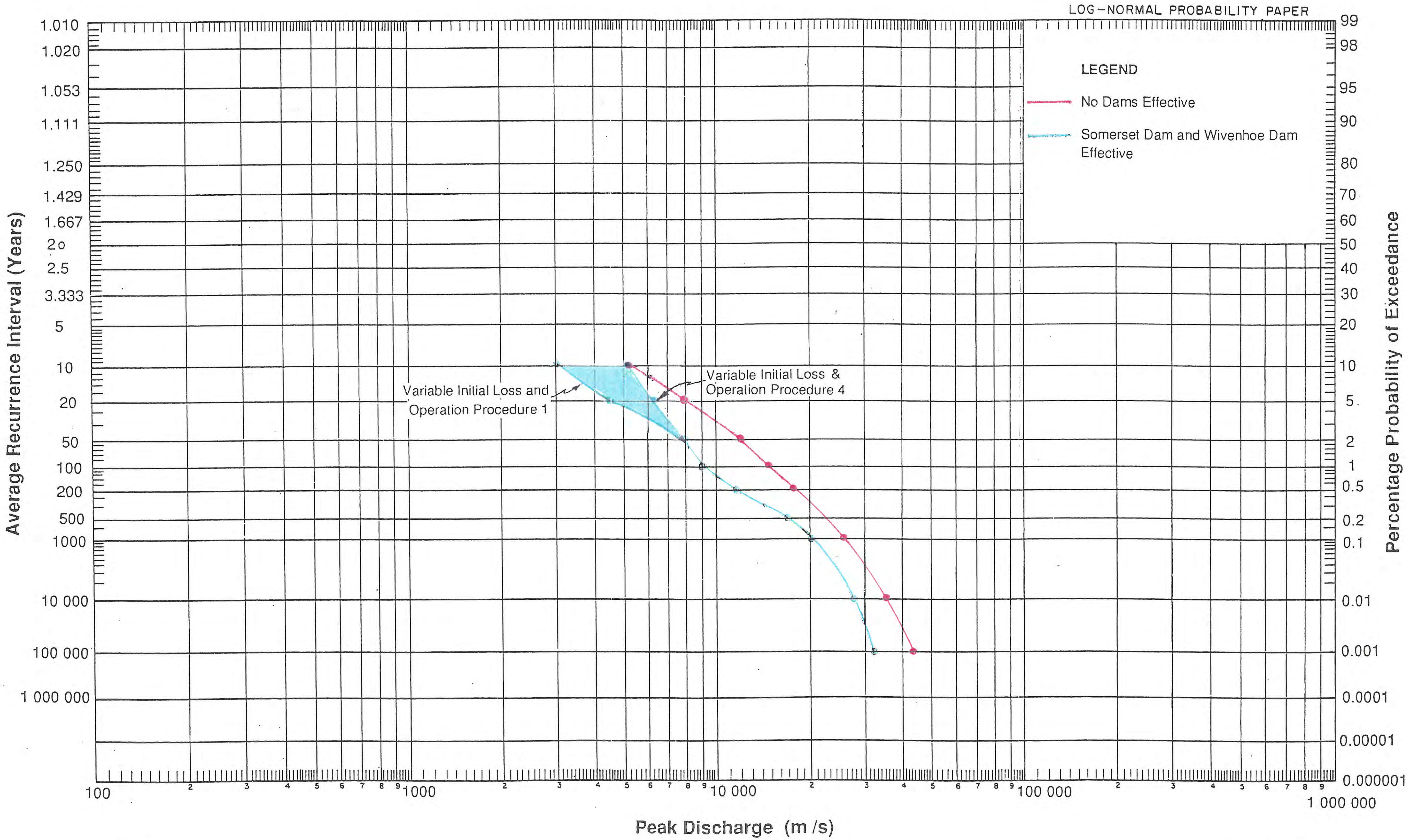
The overall results indicate that although Somerset Dam and Wivenhoe Dam provide substantial mitigation effects there is still the opportunity for severe flooding to occur in Brisbane and Ipswich. In particular, the results illustrate that floods which originate in catchments located downstream of the dams can cause major flooding in urban areas. This is evidenced by the magnitude of peak discharge that results at the Port Office

Gauge from storms centred over Lockyer Creek and the Bremer River.

The existing normal flood operation procedures, whilst reducing the impact of flooding to some extent, will not completely remove the risk of damaging flooding in Brisbane and Ipswich. The existing normal flood operation procedures of the storages will be refined and/or enhanced in future studies.

This can be accomplished through rigourous testing of possible operation procedures using the Real Time Flood Operations Model. The Real Time Flood Operations Model will be capable of utilising design rainfall data, historical rainfall data or real time data that is captured from proposed ALERT rain and river stations located throughout the Brisbane Valley catchment, in this assessment.

## Design Flood Frequency
## Brisbane River at Port Office Gauge
## Comparison Pre-Dams to Post-Dams

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