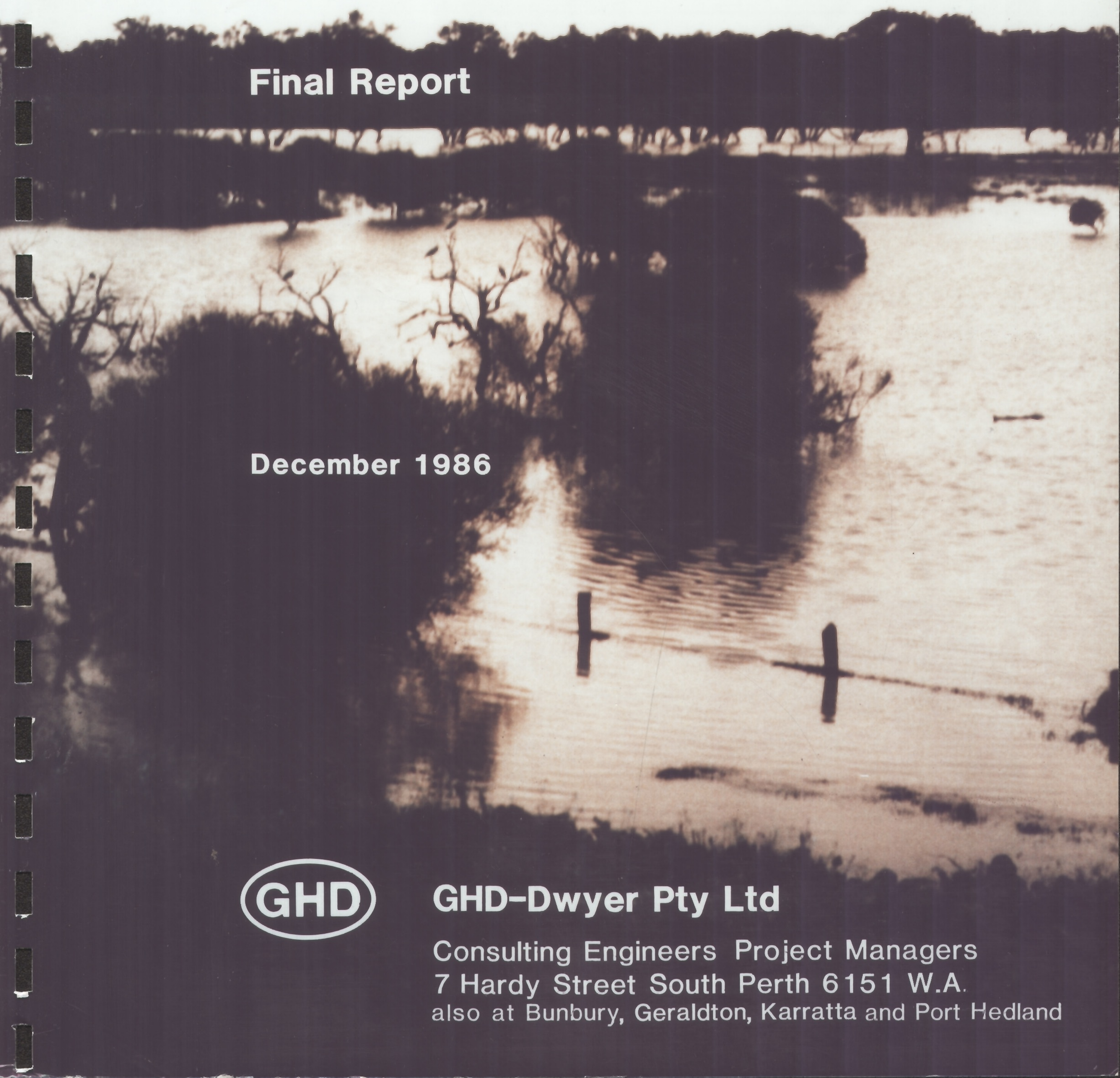
er Authority of Western Australia

Blackadder Creek Flood Study



Final Report

December 1986

GHD

GHD-Dwyer Pty Ltd

Consulting Engineers Project Managers
7 Hardy Street South Perth 6151 W.A.
also at Bunbury, Geraldton, Karratta and Port Hedland

WATER AUTHORITY OF WESTERN AUSTRALIA

REPORT ON

BLACKADDER CREEK FLOOD STUDY

DECEMBER 1986

REFERENCE NO. 3105/001

GHD-DWYER PTY. LTD.
7 HARDY STREET
SOUTH PERTH WA 6151

Offices also at: Bunbury, Geraldton, Port Hedland and Karratha

Cover Photograph: View, looking north, of Blackadder Creek near its confluence with the Swan River. The overbank flooding is due to high water levels in the Swan River. Photograph taken July 1986.

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1.0 INTRODUCTION

1.1 Terms of Reference

By letter of 30 April 1986 the Water Authority of Western Austalia engaged GHD-Dwyer Pty Ltd to undertake a flood study of Blackadder Creek, a tributary of the Swan River. A locality plan of the study area is shown in Figure 1.1.

The brief for this study can be summarised as follows:

- Investigate and study the flood potential of existing developments along Blackadder Creek.
- Identify the magnitude and recurrence interval of historic flood events.
- Prepare a set of plans showing the extent of the 100 year flood.
- Identify sections of the Creek that should be reserved as floodway.
- Identify any practical flood mitigation options.

1.2 Description of the Site Area

Blackadder Creek drains a partly rural, partly urban catchment, approximately 17 sq km in area. The eastern end of the catchment rises steeply towards the Darling Scarp. The rest of the catchment is relatively flat, falling gently away from the foothills towards the Swan River.

The soils in the catchment are generally clayey in nature, though a small pocket of sandy deposits is found along the upper reaches of Blackadder Creek, to the north-west of Swan View.

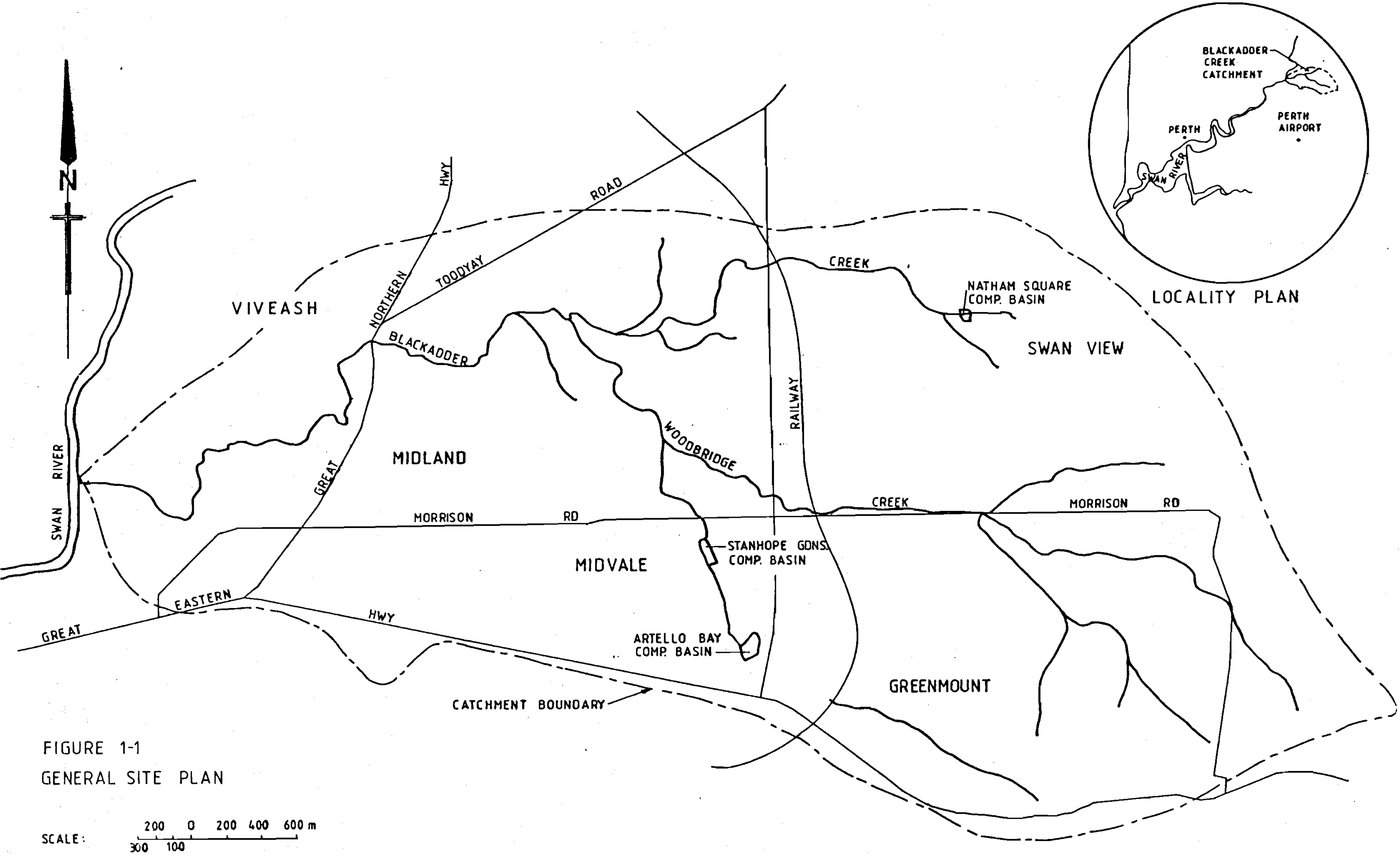

FIGURE 1-1

GENERAL SITE PLAN

BLACKADDER CREEK CATCHMENT

The whole of Blackadder Creek between the Swan River and the Natham Square retarding basin is a declared main drain. Similarly, Woodbridge Creek north of Morrison Road is also a declared main drain.

The main watercourses are generally natural open channels. In built-up areas the channel banks tend to be overgrown with shrubs and small trees, whereas in rural areas the channel is very shallow and flanked by wide flood plains.

1.3 Summary of Tasks Carried Out

Investigations and procedures used in the flood study of Blackadder Creek have included:

1. Simulation of runoff and streamflow routing within the catchment using the RORB computer model. A model was developed for both existing and future (fully urbanised) catchment conditions.

2. Hydraulic backwater modelling for sections of both Blackadder Creek and Woodbridge Creek using the HEC-2 computer model.

3. Derivation of flood frequency curves based on design rainfall for Blackadder Creek outfall and for a point just downstream of the confluence between Blackadder Creek and Woodbridge Creek.

4. Estimation of longitudinal flood profiles along Blackadder Creek for the 25 year, 50 year and 100 year events.

5. Preparation of flood plain maps delineating the extent of the 100 year flood.

6. Evaluation of a minimum floodway requirement along Blackadder Creek through Midland.

Detailed descriptions of the various methods adopted and analyses carried out during the flood study are provided in the following sections of this report.

2.0 DATA AVAILABLE

This chapter describes the available data relevant to the study. A plan showing the location of the main roads and drains referred to in the following sections is shown in Figure 1.1.

2.1 Survey Information

Survey information necessary for the study was supplied mostly by the Water Authority of Western Australia with supplementary material being obtained from the Shire of Swan and the Shire of Mundaring.

The data obtained are summarised as follows:

- General Site Plans:
  - Scale 1:1 000 with cadastral and drainage information;
  - Scale 1:2 000 with cadastral and drainage information;
  - Scale 1:5 000 orthophotomaps;
  - Scale 1:5 000 with cadastral and contour information;
  - Scale 1:10 000 with cadastral and contour information;
  - Aerial photographs (undated, approx late 1970's);
  - Aerial photographs (1963);

- Channel Geometry and Works:
  - Blackadder Creek level book no's. 34246 and 34247;
  - Blackadder Creek bridge/culvert details (recorded on drg. no. 48104-14-1);
  - Woodbridge Creek survey record 12160;

- Miscellaneous Details:
  - Natham Square Compensating Basin;
  - Stanhope Gardens Branch Drain (layout and longitudinal section);
  - Stanhope Gardens Compensating Basin;
  - Artello Bay Road Compensating Basin;
  - Margaret Street re-alignment
  - Improvements to Westrail culvert on Woodbridge Creek.

2.2 Rainfall Data

Rainfall data required for calibration of the runoff-routing model and for estimation of historic flood events were obtained from the Water Authority of Western Australia and the Bureau of Meteorology.

Data from the following daily rainfall stations were used in the study:

- Perth Regional Office (009034)
- Perth Airport (009021)
- Jane Brook (509160)

In addition to the daily records, pluviometer traces from which temporal patterns of storms may be derived were obtained from the above stations for selected storms of interest. The rainfall station located on Woodbridge Creek (509417), although ideally situated, could not be used in the study as all records were of poor quality due to a fault in the pen recorder.

A plot of Intensity - Frequency - Duration curves for the site area was obtained from the Bureau of Meteorology (Figure 2.1). These curves were used in the estimates of probabilistic rainfall events and in the determination of average recurrence intervals for historic flood events.

Information on the temporal pattern of probabilistic rainfall events was also obtained from the Bureau of Meteorology.

2.3 Adjacent Streamflow Data

Adjacent catchments were investigated in the hope that regional procedures could be used to determine flood frequency characteristics for the ungauged Blackadder Creek catchment. Unfortunately, the terrain and vegetation characteristics of the gauged catchments within the region are significantly different from Blackadder Creek and therefore the flow data are of little comparative value.

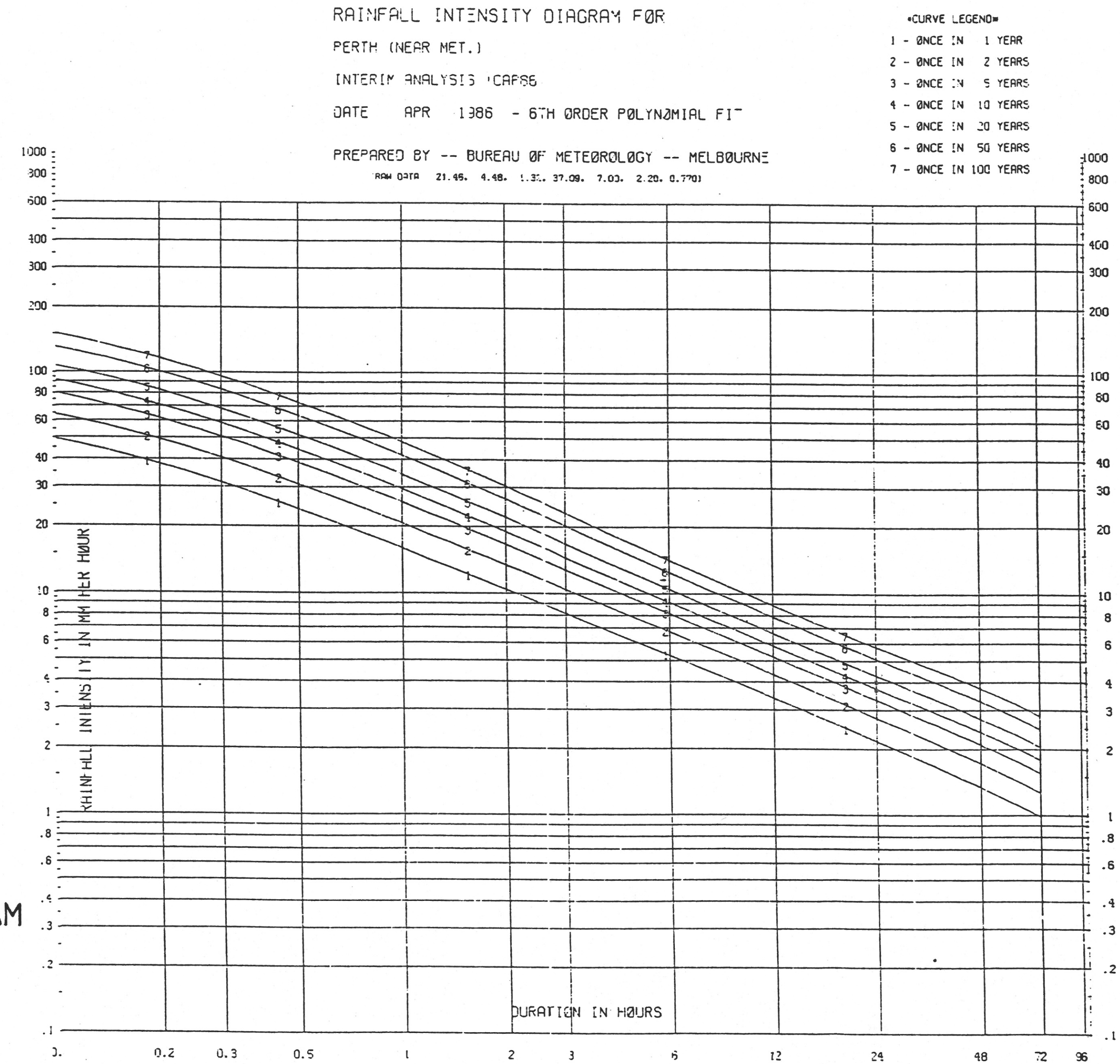

FIGURE 2·1
RAINFALL INTENSITY DIAGRAM FOR BLACKADDER CREEK CATCHMENT.

2.4 Flood Level Data

Three separate sources of flood level information are available. Two of the sources were provided by the Water Authority at the commencement of the study, and the other was obtained by GHD staff during the calibration phase of the work. Other organisations such as Westrail, Main Roads Department and local Shire Councils were also approached for information on flood levels, but no other useful data was obtained.

One set of data provided by the Water Authority contains information on historic flood levels along the lower reaches of Blackadder Creek. This record covers various flood events between the years 1917 and 1974, though the origin, and hence the accuracy of the data is unknown. These stage levels relate to flows originating from between 75% to 100% of the total catchment area and are thus potentially suited for calibration of the hydrologic model. The nature of the stage information is not described and in only two of the cases is it explicitly stated that the measurements relate to high water marks.

The other set of data supplied by the Water Authority contains information obtained from a series of maximum water level recorders installed along the upper reaches of Woodbridge Creek. The area covered by these gauges represents one quarter of the total catchment area and is approximately 50% urbanised.

The most recent significant storm in the catchment occurred in the winter of 1983 and local residents were canvassed for information or photographs relating to this event. Two separate high water marks were obtained from residents in Margaret Street and Elvire Street. These levels were then surveyed by GHD staff and found to match the maximum water level reached by Swan River, as recorded at the stage gauging station at Meadow Street (616004). The maximum water level at Margaret Street was later confirmed by the Water Authority. Therefore the observed high water marks relate to flooding of the Swan River and not Blackadder Creek and thus the levels cannot be used for hydrologic model calibration.

3.0 CATCHMENT HYDROLOGY

3.1 Establishment of Runoff-Routing Model

3.1.1 General

The runoff routing model RORB was selected as the hydrologic modelling tool for this project. Version 3 of the model, released in July 1981 (Ref.1), is the latest version and is available via our in-house computer network.

RORB Version 3 is a general runoff and streamflow routing program used to calculate flood hydrographs from rainfall and other channel inputs. It subtracts losses from rainfall to produce rainfall excess and routes this through catchment storages to produce the hydrograph. It can also be used to design retarding basins and to route floods through channel networks.

The program is operated interactively from a terminal. It can be used both for the calculation of design hydrographs and for model calibration by fitting to rainfall and runoff data of recorded events.

The model is areally distributed, nonlinear, and applicable to both urban and rural catchments. It makes provision for temporal and areal variation of rainfall and losses and can model flows at any number of gauging stations.

Apart from catchment geometry and topographic data, the model requires four parameters to be input. Two of these parameters are catchment parameters, Kc and m. The parameter m describes the degree of linearity of catchment response to excess rainfall while the parameter Kc, the catchment storage coefficient, describes the speed of response of the catchment to excess rainfall, and so influences hydrograph shape. The remaining parameters are related to loss modelling. The RORB program allows catchment losses to be modelled either by the traditional initial loss/continuing loss approach or the initial loss/volumetric runoff coefficient approach.

The latter approach to loss modelling is generally adopted for urban catchments (see for example, Laurenson and Mein, Ref. 1, or Kidd, Ref. 2) and it was adopted for the Blackadder Creek study.

Where the catchment to be modelled is not gauged, a regional analysis of similar, nearby catchments is desirable from which an estimate of the parameters for the catchment of interest may be determined. Various authorities and institutions involved in flood estimation were contacted to establish whether any gauged catchments within the region had been analysed and calibrated using RORB. Enquiries yielded valuable information on the regional estimation work currently being undertaken by the Department of Main Roads (Ref. 6). Also, two urbanised catchments in metropolitan Perth are presently being studied by the Water Authority (Ref. 14). Preliminary results indicate lower Kc and loss parameters than that indicated by available regional estimates, though only frequent storms of low intensity have been avaialable for calibration. This latter data became available after the calibration of this study was completed. Considering the preliminary nature of these findings and the problems inherent in calibrating to small runoff events, further adjustment of the Blackadder Creek calibration results was not considered justifiable.

3.1.2 Model Development

Using the available contour mapping, a catchment model was developed for Blackadder Creek catchment. The catchment was divided into 16 sub-catchments each of approximately the same area. The boundaries of the modelled sub-catchments are based on the stream network and drainage divides of the catchment. The stream network in Blackadder Creek catchment consists of both natural and piped reaches and therefore urban sub-area boundaries were determined with reference to piped stormwater drainage plans obtained from local shire councils.

In order to analyse catchment storage effects, the channel network was divided into 21 non-linear reach storages. Natural reach storages were defined in terms of reach length only, whereas piped sections were defined by both reach length and slope. A schematic illustration of the catchment subdivision and channel network is shown in Figure 3.1.

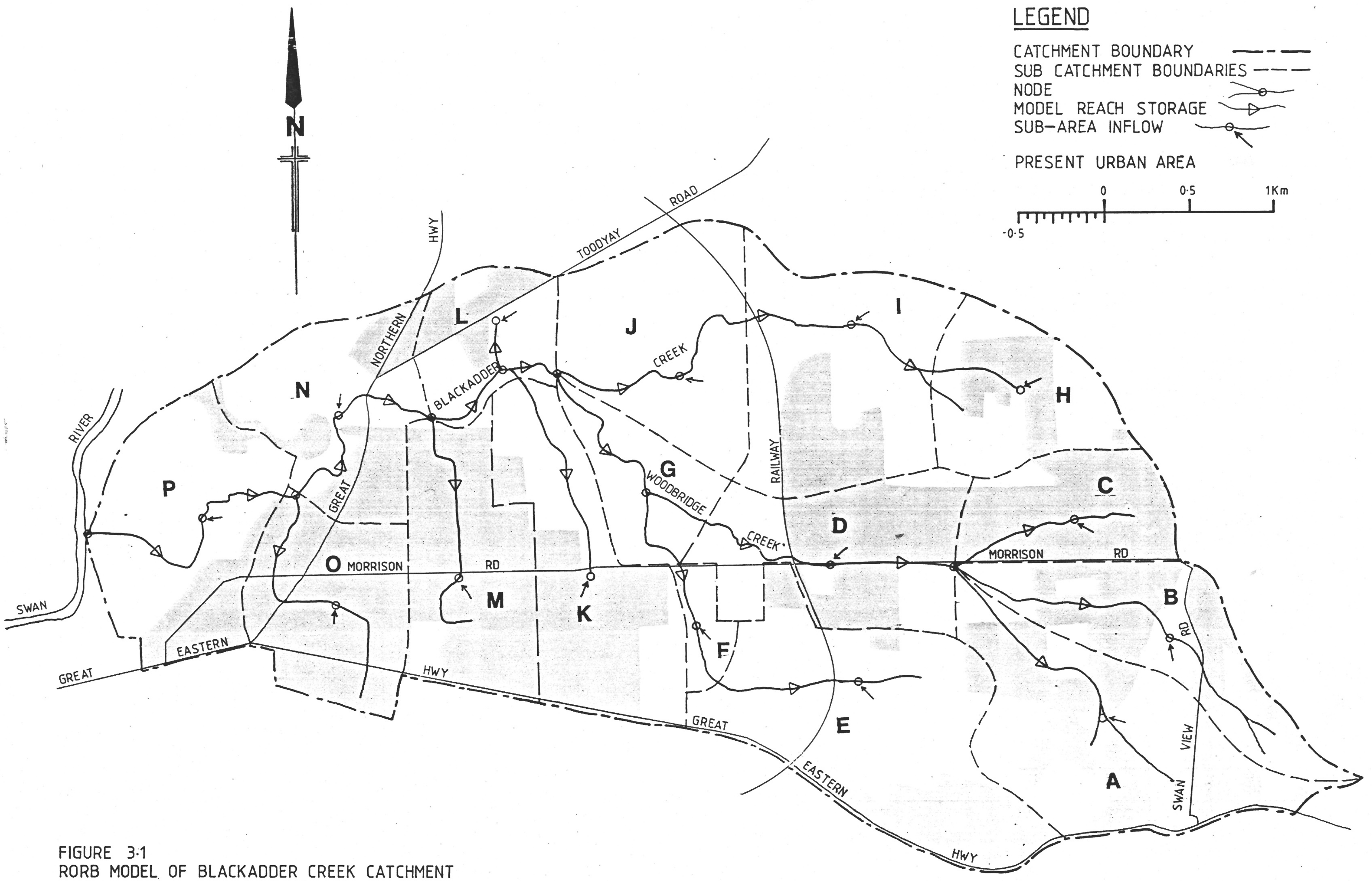

FIGURE 3·1
RORB MODEL OF BLACKADDER CREEK CATCHMENT

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Blackadder Creek catchment is undergoing continual development and thus several versions of the model were created to account for increasing urbanisation. For instance, in 1963 approximately 22% of the catchment was urbanised whereas today the proportion is closer to 45%. Information on the varying degrees of urbanisation was obtained from aerial photographs and old street directories. It is assumed that 35% of urban areas is impervious with a runoff coefficient of 0.9.

The catchment contains three retarding basins with another at the design stage. The basins are excavated below ground level and they do not incorporate berms or relief spillways. They are designed to pass a 1 in 10 year flood, leaving a 0.3m freeboard between top water level and adjacent ground surface.

In more extreme storms, the basins simply fill up to ground level and consequently they have no attenuation effect upon peak flows. As the retarding basins are only effective for storms with return periods of 10 years of less, they are not included in the model when estimating the 25, 50 and 100 year events.

Loss parameters for the catchment were determined by calibrating the model using information derived from the maximum water level recorders located along the middle reaches of Woodbridge Creek. A separate RORB model was established for this purpose. This calibration model is based on the upper 4 sub-areas of the catchment-wide model, however sub-area D has been reduced in size. Doubling the number of sub-areas in this model has negligible effect upon results, and the adopted sub-division was chosen for the sake of convenience. A schematic of the calibration model is shown in Figure 3.2.

3.2 Calibration of Model Parameters

3.2.1 General

Calibration of the model was undertaken using rainfall and maximum level recorder data. Maximum level recorders only provide information on the highest stage reached during flood, without recording the time at which the peak occurred. HEC-2 was used to determine the magnitude of flow corresponding to the peak stage reached by the stream.

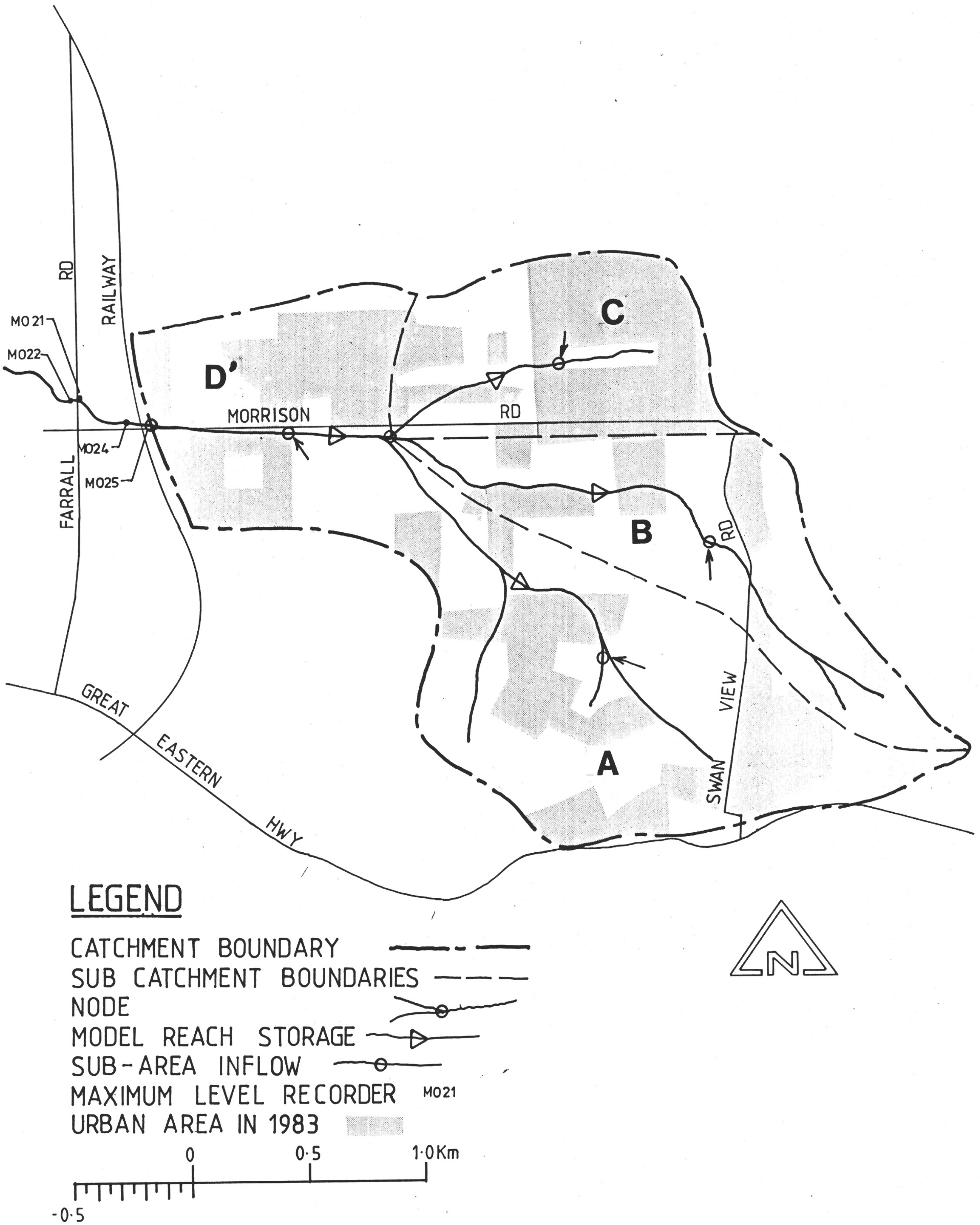

FIGURE 3·2
RORB MODEL OF WOODBRIDGE CREEK
CALIBRATION CATCHMENT
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Thus only partial fitting of the model parameters is possible, where Kc and m are estimated from regional equations and the value of the runoff coefficient is determined from calibration with known peak flow rates.

3.2.2 Regional Estimates of Kc and m

Various studies have been undertaken with the aim of developing regional formulae for estimating model parameters from catchment characteristics for ungauged catchments.

The parameter Kc is often found to be related to the area or mainstream length of a catchment. Laurenson and Mein (Ref. 1) recommend the equation:

$$Kc = 2.2\ A^{0.5}\ (Qp/2)^{0.8-m} \tag{3.1}$$

where A is the catchment area in sq.km and Qp is the peak discharge in m^3/s. It is an empirical equation that generally represents a wide range of fitted data for Australian catchments and is intended to provide an initial estimate of Kc for calibration and for flood estimation in ungauged catchments.

Morris (Ref. 3) derived regional equations relating Kc to catchment area for various states, including Western Australia, following a survey of RORB parameters used by various individuals and organisations throughout Australia. The following regional equation for Western Australia was recommended:

$$Kc = 2.48\ A^{0.47} \tag{3.2}$$

where, as before, A represents catchment area in sq.km. The parameters Kc and m are interdependent and the above equation was derived using a value for m of 0.8.

Flavell et al (Ref. 4) derived regional formulae for estimating the model parameters from catchment characteristics for ungauged catchments in Western Australia. They found that the significant catchment characteristics were catchment area and mainstream length, where the latter is measured from the catchment outlet to the most remote point on the catchment boundary. For the southwest region of the state, the relevant equations are:

$$Kc = 1.49\ L^{0.91} \qquad (3.3)$$

$$Kc = 1.98\ A^{0.50} \qquad (3.4)$$

where L is the mainstream length measured in km, and A is as defined previously. Mainstream length was found to be more significant than area, with Eq (3.3) exhibiting a higher degree of correlation. Both equations are derived with m equal to 0.8.

In the light of the above regional studies undertaken specifically for Western Australia, it was decided to set the value of m equal to 0.8. This is also the value recommended for ungauged catchments in the next edition of "Australian Rainfall and Runoff", as discussed by Pilgrim (Ref. 5).

For comparison purposes, the results of the above regional formulae are shown in Table 3.1. The following variables were used in the equations:

Catchment model	- A	= 17.2 sq.km
	- L	= 9.5 km
Calibration model	- A	= 4.4 sq.km
	- L	= 3.4 km

TABLE 3.1 Comparison of Regional Estimates for Kc

	REGIONAL FORMULAE			
MODEL	Eqn (3.1)	Eqn (3.2)	Eqn (3.3)	Eqn (3.4)
Catchment	9.1	9.4	11.6	8.2
Calibration	4.6	5.0	4.5	4.2

With reference to Table 3.1 and recognizing that Eqn (3.3) has the highest degree of correlation and is most relevant to Blackadder Creek, values for Kc of 10.5 and 4.5 were adopted for the catchment and calibration models, respectively.

To assess the sensitivity of the model, an analysis was undertaken to determine the effect on predicted flows due to changes in the parameter Kc. A base case was established based on simulation of the 100 year event (see Section 3.4 for design data). Evaluation of flows at the catchment outfall yielded the result that a 20% reduction in Kc effected an approximately 10% increase in peak flow, and similarly a 20% increase in Kc results in a 10% decrease in peak flow.

The regional estimates of Kc are within ± 20% of the adopted value. This therefore corresponds to a range in estimates of peak flow of ± 10%.

A sensitivity analysis of the parameter m was not undertaken. As mentioned previously, the parameters Kc and m are interdependent - altering the m value, with the consequent change in Kc, merely alters the shape of the hydrograph without significantly affecting the magnitude of peak discharge. Altering m is sometimes useful in improving a calibration fit but it is less important than varying Kc. To be valid, all regional estimates of Kc must be standardised to the same value of m, and without more comprehensive calibration data, there is no justification for changing m independently of Kc in this analysis.

3.2.3 Selection of Loss Parameters

As mentioned in Section 3.1.1, the initial loss/volumetric runoff coefficient approach was adopted for the study. It is the approach generally recommended for urban or partly urban catchments and regional estimates for the appropriate loss parameters can be obtained.

Loss parameters for the catchment were derived from a combination of calibration results and regional estimates. The model was calibrated to the HEC-2 estimates of flow corresponding to the maximum level recorder data. With initial losses set to zero, volumetric runoff coefficients were determined for 8 storm events such that model predictions matched the flows estimated by HEC-2.

An initial loss of zero was adopted for, without hydrograph information, there is little basis for estimating an initial loss and a conservative value of zero is appropriate. Also, Flavell (Ref. 6) has recently derived regional estimates for proportional losses on the basis that initial losses are set to zero. Values of volumetric runoff coefficients derived from calibration can therefore be directly compared with Flavell's regional estimates, a valuable reference considering the paucity of calibration data available.

The maximum level recorder data represents the only available data suitable for calibration. The recorders were installed in mid-June 1983 and unfortunately no major storm has since occurred in the region. The 8 storm events selected for calibration are among the most significant to have occurred since installation of the recorders, however all but one of these storms have estimated average recurrence intervals of less than one year.

The results of the calibration are given in Table 3.2. It can be seen that the median value of the derived volumetric runoff coefficient (Cv) is around 0.32, though the values of Cv range from 0.22 to 0.75.

By way of comparison, Flavell (Ref. 6) recommends the following equation for the regional estimation of proportional loss for Jarrah Forest, loam soil:

$$PL_2 = 340 \quad 10^{-0.0013C_L} \quad P^{-0.20} \quad L^{0.017} \tag{3.5}$$

where,

PL_2 = proportion loss (%) with 2 year average return period

TABLE 3.2 Calibration Results for Volumetric Runoff Coefficient (Cv)

RUN NO.	DATE OF STORM	PEAK FLOW (m3/s)	Cv
1	16/06/83	1.8	0.34
2	27/06/83	2.0	0.36
3	23/07/83	1.7	0.24
4	29/07/83	1.0	0.22
5	04/08/83	1.8	0.33
6	03/09/83	2.8	0.75
7	17/05/84	1.9	0.31
8	13/06/84	1.6	0.30

CL = clearing of the forest measured as percentage of catchment area

P = average annual rainfall (mm) falling on catchment

L = mainstream length (km) measured from catchment outlet to the most remote point on catchment boundary.

Substituting the following variables:

C_L = 100%
P = 800 mm
L = 3.5 km

yields the result that proportional loss is 68%, or in other words that the volumetric runoff coefficient is equal to 0.32. Equation (3.5) is not very sensitive to mainstream length and the corresponding regional estimate for the catchment-wide value of Cv is 0.31.

The parameter Cv can be viewed as a statistical parameter linking probabilistic estimates of rainfall intensity and peak runoff. As the frequency of the rainfall event decreases, the rainfall intensity, and hence the value of Cv, correspondingly increase. The above regional estimates for Cv are ascribed a two year return period, whereas the flood events used in the calibrations have an annual or greater frequency.

A comparison between the regional estimates and calibration data is illustrated in Figure 3.3. The result for run 6 is omitted from Figure 3.3 as it is not considered to be representative of catchment conditions.

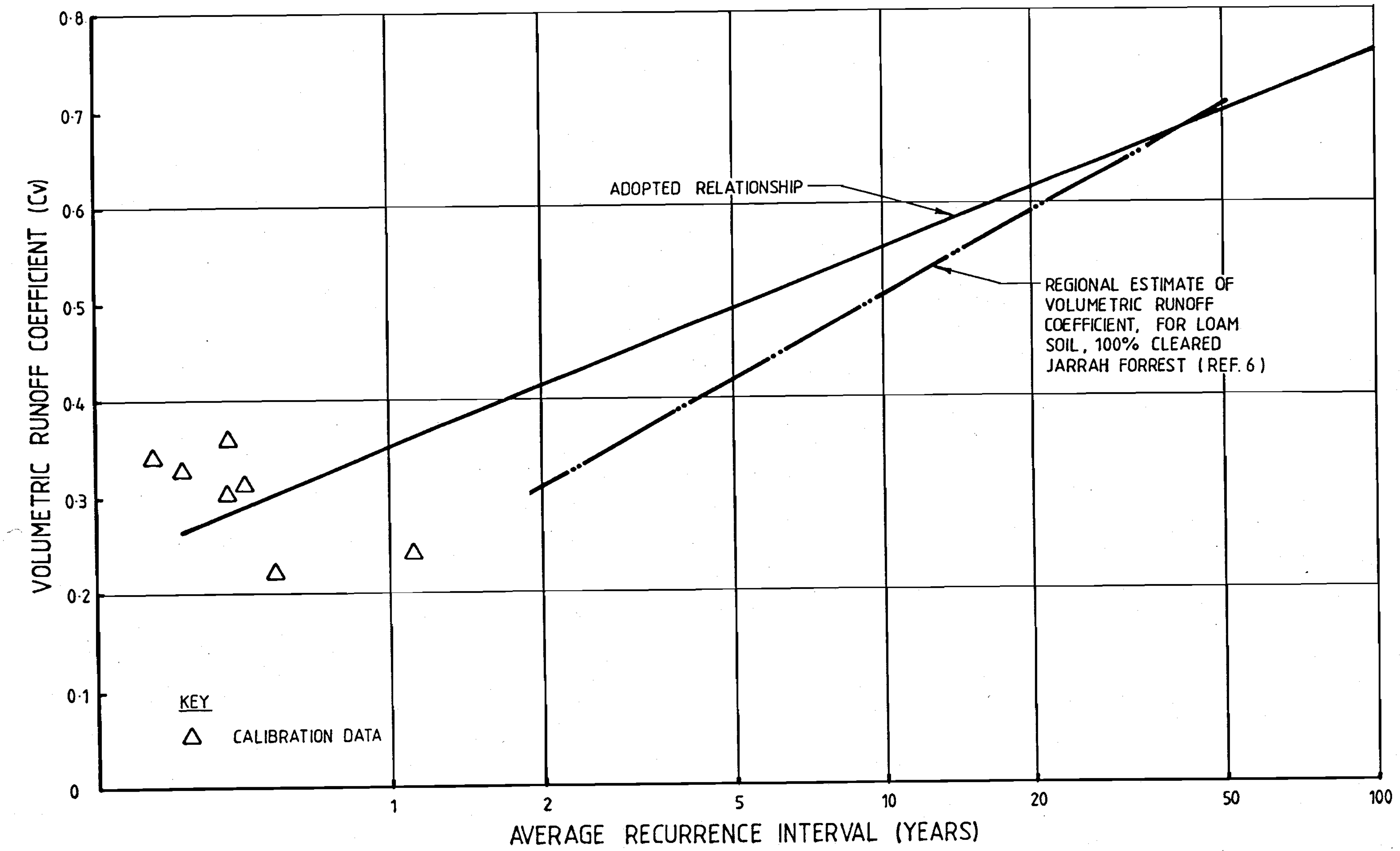

FIGURE 3·3

RELATIONSHIP BETWEEN Cv AND RECURRENCE INTERNAL

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The estimate of return periods for the calibration events is taken from the Intensity-Duration-Frequency curve using a time of concentration of 3 hours. Evaluating return periods from these curves for storms with greater than annual frequency is rather speculative, and so the abscissa location of the data points shown on Figure 3.3 should be considered approximate.

Figure 3.3 illustrates that the runoff coefficients obtained from calibration are slightly higher than the regional estimates, though this can perhaps be explained by differences in clay content of the soils.

The degree of confidence attributed to these results is largely dependent upon the validity of the regional estimate of the parameter Kc. Accordingly, a sensitivity analysis was undertaken in which the sensitivity of Cv to changes in Kc was examined.

The results of the sensitivity analysis are illustrated in Figure 3.4. The dashed line represents the average response of the parameter Cv to changes in Kc. It can thus be seen that a 10% change in Kc produces approximately a 5% change in Cv. In the calibration model, the variations in regional estimates of Kc ranged from about +10% to -7% around the adopted value. For the median Cv value of 0.32, this implies a range of values from 0.31 to 0.34. If the value of Kc is in error by 50%, the resulting range of Cv values is from 0.24 to 0.4, results which still indicate lower loss rates than that predicted by regional estimates.

The adopted relationship between Cv and return period is shown in Figure 3.3. A less steep gradient is chosen to represent the variation of Cv with return period than that exhibited by the regional estimate. Although the calibration results influenced selection of Cv values for frequent storms, it is necessary to increasingly rely on the regional estimates for less frequent storms so that the upper limit of adopted Cv values realistically reflect catchment processes.

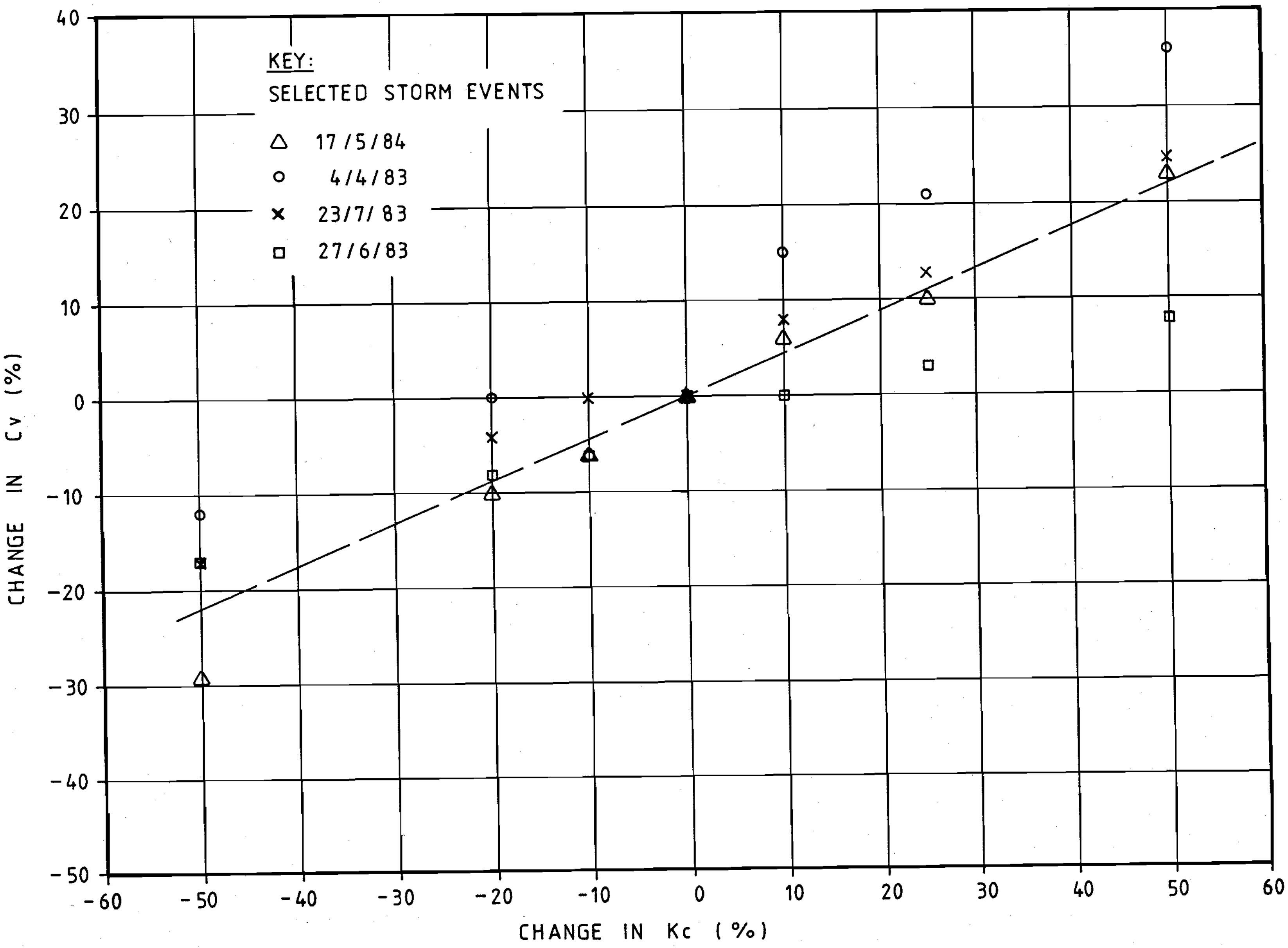

FIGURE 3·4
SENSITIVITY OF Cv TO CHANGES IN Kc

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3.3 Estimation of Historic Events

The locality is known to have been subjected to a number of significant flood events since the 1830's (Ref. 12) but, as is discussed in Section 4.2.1, flood level information for such severe events is related almost certainly to the Swan River and is consequently of little relevance to Blackadder Creek.

Estimation of recurrence intervals for historic events is normally based on a statistical analysis of past floods - however this approach is not possible for Blackadder Creek as there are no appropriate flow gauging records. It was therefore necessary to estimate recurrence intervals using the flood frequency curve shown in Figure 3.5 (the derivation of which is discussed in Section 3.4) and the magnitude of the flows were obtained using RORB and relevant pluviograph information.

RORB simulations were not undertaken for those events for which the associated pluviograph data indicated average recurrence intervals of less than one year. Thus, while the rainfall events of 1963 and 1964 resulted in significant flooding along the Swan River, they were not significant with respect to Blackadder Creek catchment - it was apparent from the pluviograph data that the low rainfall intensities would generate negligible runoff and accordingly a RORB simulation was not carried out.

For some events, local rainfall information was not available and it was necessary to use Perth Regional Office (009034) and Perth Airport (009021) data without reference to the more appropriate information recorded at Jane Brook (509160). To account for any differences between the localities, a correlation analysis of the three rainfall stations was undertaken. A least-squares regression was applied to daily rainfall data for rain days recorded during 1984, and the following results were obtained:

$$RJB = 0.8598 + 0.9309\ RPA \qquad r2 = 0.93$$

$$RJB = 0.7208 + 0.9442\ RRO \qquad r2 = 0.91$$

where RJB is the daily rainfall (mm) recorded at Jane Brook (509160) and RPA and RRO are the corresponding values for Perth Airport (009021) and Perth Regional Office (009034) respectively, and r^2 is the correlation coefficient of the analysis. Where records for Jane Brook were not available, the pluviograph data was factored by the appropriate ratio of daily rainfall estimated from the above correlation formulae.

The magnitude of flow and associated average recurrence interval for significant flood events within Blackadder Creek catchment are shown in Table 3.3.

Also listed are the dates on which significant flooding occurred within the area. A RORB simulation of these events, marked with an asterisk, was not undertaken as the flooding was due to the Swan River and not Blackadder Creek. As can be seen from the estimated average recurrence intervals, the intensity-duration characteristics of these rainfall events were not significant with respect to the relatively small Blackadder Creek catchment.

Only two events of significance have occurred within the Blackadder Creek catchment during the last 30 years. Referring to Table 3.3, the first occurred in February 1955, with an estimated peak flow of 19 m3/s and average recurrence interval of about 8 years. The second event occurred in June 1980, with a corresponding peak flow estimate of 17.5 m3/s and average recurrence interval of 6 years. Although significant flooding occurred in the winter of 1983, the runoff originating from the catchment was not a major contributing factor as the maximum estimated peak flow was only 7.4 m3/s.

The storm event of July 1981 is included in Table 3.3 as the associated minor flooding in Greenmount was cause for concern to the Shire Engineer (Ref. 13). The estimated average recurrence interval for flooding associated with this event for Woodbridge Creek at Morrison Road is about 3 years and the corresponding estimate of flow is 3.0 m3/s. The rainfall was quite short and intense and, as can be seen in Table 3.3, it was not a significant event with respect to the Blackadder Creek catchment.

TABLE 3.3 Analysis of Significant Flood Events

DATE	AVERAGE RECURRENCE INTERVAL (Yr)	ESTIMATE OF PEAK FLOW (m3/s)
Feb 1955	8	19.2
Aug 1958	< 1	*
July 1963	< 1	*
Aug 1963	< 1	*
July 1964	< 1	*
Aug 1964	< 1	*
Jun 1980	6	17.5
July 1981	< 1	5.9
June 1983	1.3	7.2
July 1983	1.3	7.4
May 1984	1	*

Note: * indicates that a RORB simulation was not undertaken.

3.4 Estimation of Probabilistic Floods Under Existing Conditions

3.4.1 Runoff-Routing Approach

The RORB model was used to generate probabilistic floods for the 2, 5, 10, 25, 50 and 100 year events from probabilistic rainfall of the same recurrence intervals. The rainfall intensity-frequency-duration polynomial equations derived for Blackadder Creek were used to determine the design rainfalls.

Rainfall temporal patterns applicable to the site were obtained from the Bureau of Meteorology. These patterns were adjusted to simulate the general flattening of the temporal profile observed for more extreme events. The procedure used to adjust the temporal profile of extreme events was obtained from the Bureau of Meteorology (Ref. 15) and is to be included in the next edition of Australian Rainfall and Runoff.

A study of a range of storm durations was undertaken to determine the critical duration for each design recurrence interval. The durations and corresponding cumulative rainfall totals are shown in Table 3.4.

TABLE 3.4 CUMULATIVE DESIGN RAINFALLS (mm)

RECURRENCE INTERVAL (Yr)	DURATION (Hrs) 6	9	12	30	36
2	40	47	52	71	76
5	49	56	62	86	92
10	55	63	70	98	104
25	66	75	83	117	126
50	75	86	95	132	144
100	85	96	107	150	162

The parameters used in RORB for the generation of probabilistic floods were selected on the basis of the discussion in Section 3.2. The values of Kc, m and initial loss were 10.5, 0.8 and zero, respectively. The values of volumetric runoff coefficient were determined using Figure 3.3, and are shown in Table 3.5.

TABLE 3.5 DESIGN VOLUMETRIC RUNOFF COEFFICIENTS (Cv)

RECURRENCE INTERVAL (Yr)	Cv
2	0.41
5	0.49
10	0.55
25	0.63
50	0.70
100	0.75

An envelope of maximum probabilistic discharges within the study area was determined using the appropriate critical storm durations. Table 3.6 lists the predicted peak discharges at various sites of interest within the catchment for selected recurrence intervals.

Flood frequency curves relating peak flows in Blackadder Creek to average recurrence intervals were developed using information contained in Table 3.6. One curve was developed for the outfall of the creek and the other is applicable to a point just upsteam of the Lloyd Street extension. The two curves are illustrated in Figure 3.5.

3.4.2 Rational Method

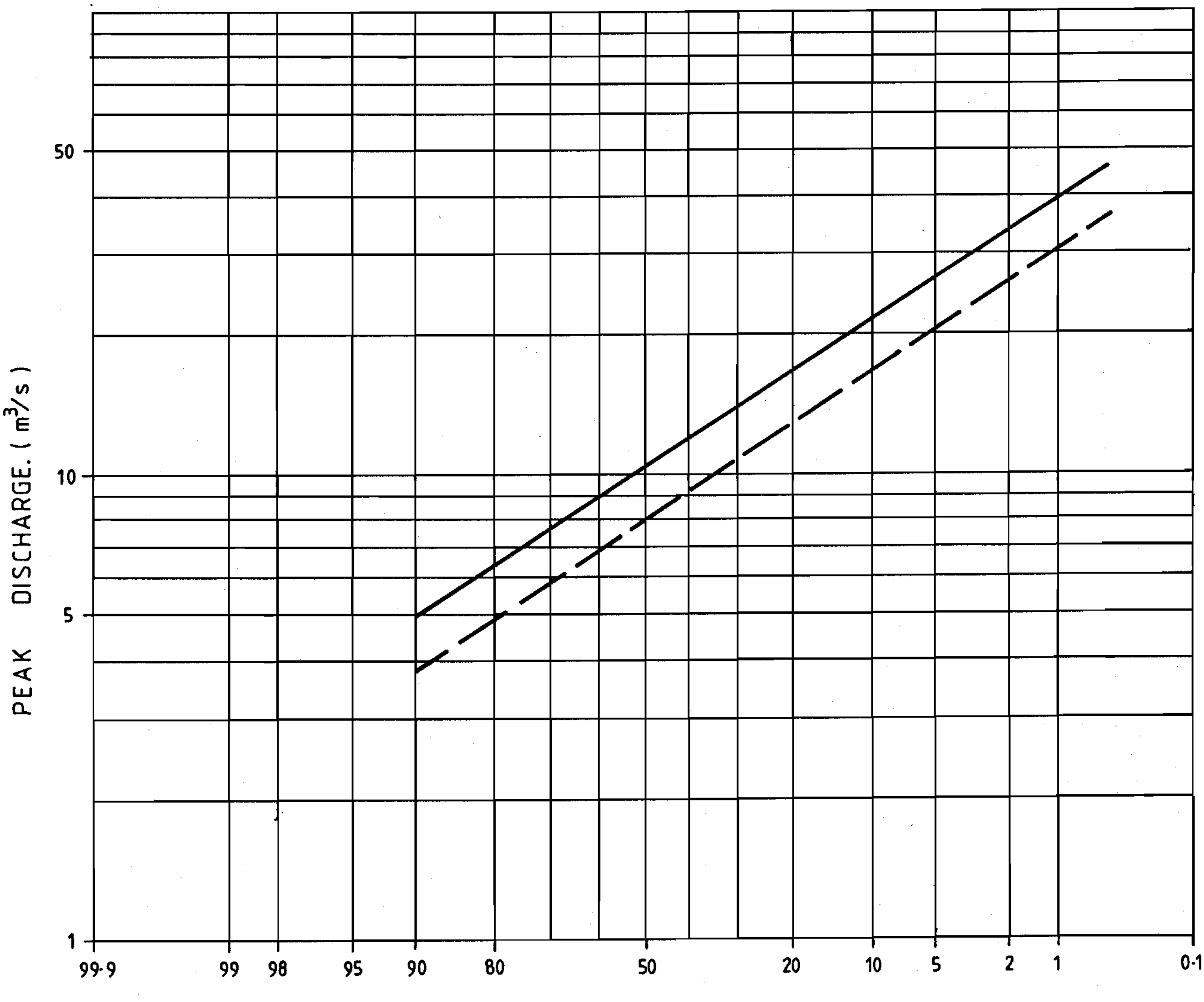
Estimates of peak discharges were prepared using the Rational Method to provide an alternative assessment of probabilistic floods. The method adopted incorporates the regional approach developed by Flavell (Ref. 11).

TABLE 3.6 PROBABILISTIC DISCHARGE ESTIMATES FOR CATCHMENT UNDER EXISTING CONDITIONS

LOCATION	RECURRENCE INTERVAL (Yr)	TIME TO PEAK (hr)	PEAK DISCHARGE (m^3/s)
B C upstream of W C	25	3.0	8.1
	50	3.0	10.4
	100	3.0	12.8
W C outfall	25	5.0	13.2
	50	4.5	16.8
	100	4.5	20.7
B C Upstream of Lloyd Street	2	5.0	7.9
	5	4.5	12.0
	10	4.5	15.6
	25	4.5	19.4
	50	4.5	25.0
	100	4.5	30.7
B C at Great Northern Hwy	25	8.0	22.1
	50	5.0	27.9
	100	5.0	34.4
B C Outfall	2	12.0	10.4
	5	10.0	15.6
	10	10.0	20.1
	25	10.0	25.0
	50	10.0	31.6
	100	10.0	38.5

NOTE: B C denotes Blackadder Creek

W C denotes Woodbridge Creek



——— BLACKADDER CREEK OUTFALL.

— — BLACKADDER CREEK UPSTREAM OF LLOYD STREET.

FIGURE 3·5

FLOOD FREQUENCY CURVES BASED ON DESIGN RAINFALLS

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The variables used in this method are:

A = catchment area = 17.2 sq km
L = mainstream length = 9.5km
S = equivalent uniform slope = 4.9m/km
CL = clearing = 100%

Flavell's regional estimate for the time of concentration of the catchment is given as:

$$T_c = 2.31\ A^{0.54} = 10.7 \text{ hrs}$$

This value of 10.7 hours is similar to the time to peak of 10 hours predicted by RORB.

The regional estimate of runoff coefficients is dependent upon mainstream length and slope, clearing, soil type, and the recurrence interval of the event being considered. Flavell provides formulae and graphs for the estimation of this parameter for up to a 50 year recurrence interval, and so it was necessary to extrapolate his recommendations for the 100 year event.

Two broad soil classes can be accommodated in the assessment of runoff coefficients : sand soils / lateritic soils, and loamy soils / red earths. The latter group was considered to be the most appropriate to the pervious areas of Blackadder Creek catchment. The final runoff coefficient used for the overall catchment includes a proportional component for impermeable areas, for which a runoff coefficient of 0.9 was adopted.

Table 3.7 summarises the results and the major parameters used by the rational method in the estimation of peak discharges, and it is seen that there is neglible difference between the rational method and RORB estimates of flow.

TABLE 3.7 RATIONAL ESTIMATE OF PROBABILISTIC DISCHARGES

RECURRENCE INTERVAL (Yr)	RAINFALL INTENSITY (mm/hr)	PERVIOUS RUNOFF COEFFICIENT	PEAK DISCHARGE (m^3/s)	RORB ESTIMATE (m^3/s)
25	7.2	0.59	22	25.0
50	8.5	0.74	31	31.0
100	9.6	0.82	38	38.5

The rational method does not account for the attenuation effects of channel storage nor the temporal and spatial variations of rainfall intensity. The former limitations of this approach causes over-estimation of peak discharges while the latter results in under-estimation. In general, the effect of neglecting variations in rainfall intensity is usually less than that of channel storage and thus the rational method tends to yield conservative results. The degree to which this occurs is dependent upon the characteristics of the catchment concerned.

Too great an emphasis should not be placed upon the comparison between the RORB and rational estimates of peak discharge, though the close agreement shown in Table 3.7 does give further confidence in the RORB results.

3.5 Estimation of Probabilistic Floods Under Future Conditions

The RORB model representing existing catchment conditions was altered to account for future development. Using the available town planning information (Refs. 16 and 17) future urbanisation within the catchment was estimated to be approximately 75%. To provide an upper limit of future peak discharges a RORB model was also developed to simulate runoff within the catchment under fully developed (100% urban) conditions.

TABLE 3.8 PROBABILISTIC FLOOD ESTIMATES FOR CATCHMENT UNDER FUTURE CONDITIONS

LOCATION	RECURRENCE INTERVAL (Yr)	GROWTH FACTORS RELATIVE TO EXISTING CONDITIONS 75% URBAN	100% URBAN
Blackadder Creek, upstream of Lloyd Street	5	1.11	1.19
	100	1.04	1.05
Blackadder Creek Outfall	5	1.09	1.16
	100	1.03	1.05

The growth factors are considerably lower for the 100 year event than the 5 year event because of the relative differences between the pervious and impermeable area volumetric runoff coefficients. In the 100 year event the runoff coefficient for pervious areas is 0.83 times that of impermeable areas, whereas for the 5 year event this fraction reduces to 0.54. Thus, the more extreme the flood event, the less significant the effects of urbanisation, a phenomenon that is generally observed in practice (see, for example, Ref. 18).

The growth factors are greater for a point just upstream of Lloyd Street than for the outfall. This reflects the fact that most of the remaining development will occur in the upper reaches of the catchment.

4.0 HYDRAULIC MODELLING

4.1 Establishment of Hydraulic Model

4.1.1 General

The hydraulic model used in this study was the HEC-2 backwater program developed in the USA by the US Army Corps of Engineers (Ref. 9). This model is a computerised application of Bernoulli's theorem utilizing Manning's formula for the friction head loss between cross-sections and adopting the Standard Step Method of calculation.

Each section is divided into three major segments: the main channel, and the left and right overbanks. A value of Manning's "n" is assigned to each segment, and a weighted value of conveyance is then calculated by the model for each cross-section. The friction loss in a reach between cross-sections is then obtained by averaging the conveyances at each end of the reach. By starting calculations with a known water level at the downstream end of the reach, the water surface elevation at each successive cross-section is computed in an iterative manner, by applying the above principles, with calculations proceeding in the upstream direction.

Losses such as transition losses occurring at expansions or contractions in the waterway are computed as a proportion of the difference in velocity head between successive cross-sections. This proportion is a function of the severity of the transition. Losses in head through bridge openings (other than transition losses) are evaluated using special subroutines which take account of the effect of piers on the flow and allow for weir flow over a road embankment where this occurs.

The parameters that require calibration in the model are:

At each cross-section	-	Manning's "n" - left bank
	-	Manning's "n" - right bank
	-	Manning's "n" - main channel
Between cross-sections	-	expansion transition coefficient
	-	contraction transition coefficient

At bridges - pier shape coefficient
- weir flow coefficient
- orifice flow coefficient

4.1.2 Model Development

Boundary geometry for the analysis of flow in the catchment is specified in HEC-2 in terms of ground surface profiles, or cross-sections, and the length of reach between them. This information was obtained from the survey data listed in section 2.1.

Two HEC-2 models were developed. The main model simulates flow along Blackadder Creek from its outfall to just upstream of the railway line, and also along the lower reaches of Woodbridge Creek. A smaller model was also developed to simulate flow between the two pairs of maximum level recorders located on Woodbridge Creek, between Farrall Road and the railway line. The latter model was used to determine channel flows for calibration of the RORB model. It was considered necessary because the presence of several culverts and variable channel geometry precludes the accurate application of slope-area techniques.

The locations of the cross-sections used in the main HEC-2 model were set at the time of the original survey. Extra sections were incorporated in the model, primarily to define flow at control structures such as bridges and culverts, though additional sections were also required to model the lower reaches of Woodbridge Creek. A total of 79 cross-sections was required to represent the channel network and control structures. Figure 4.1 illustates the locations of the major cross-sections used in the model.

The Blackadder Creek survey data obtained from the Water Authority was received in survey log-book format. This information was entered into GHD's HP9845B computer and plotted out on a HP7580B plotter. The resultant plots were a useful aid in selecting realistic flow boundaries for the model. Using both contour plans and the section plots, areas of ineffective flow in the model were chosen to characterise the flow carrying capacity of the stream and its adjacent flood plains.

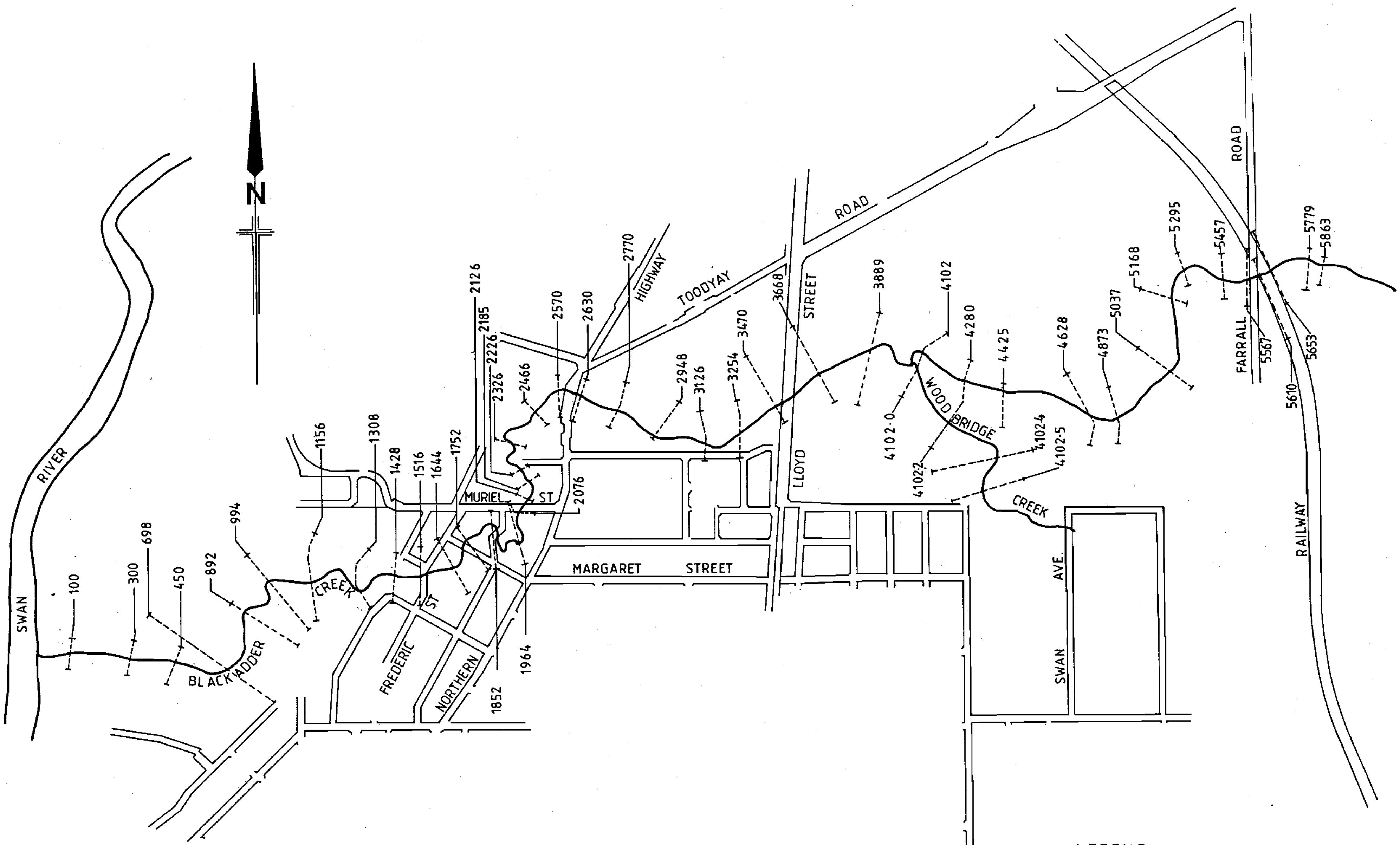

FIGURE 4·1
LOCATION OF MAJOR CROSS SECTIONS
USED IN HEC-2 MODEL OF
BLACKADDER CREEK CATCHMENT.

LEGEND:

├--------┤	LOCATION OF SECTION
— 2226	HEC-2 SECTION NO.

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Survey data necessary for the establishment of the smaller Woodbridge Creek model was obtained from the Shire of Mundaring and from investigation of the site area. In total 17 cross sections were required to model the reach between the maximum level recorders and these are illustrated in Figure 4.2. Cross-sections were located where changes occur in slope and channel geometry, and at the upstream and downstream reaches adjacent to culverts.

4.1.3 Model Calibration

A detailed initial estimate of loss parameters for use in HEC-2 was obtained from a visual assesssment of the catchment. Features and structures at significant locations within the site area were photographed for later reference. Preliminary estimates of other coefficients such as transition loss parameters were made by drawing upon experience gained on similar streams elsewhere.

Calibration of the Woodbridge Creek model was based upon the maximum level recorder data. Calibration proceeded in an iterative manner until the best match of observed versus simulated levels was achieved. Transition and energy loss parameters were adjusted to achieve consistency between the four levels recorded by the maximum level recorders. Appendix A lists the energy loss coefficients used in the HEC-2 model of Woodbridge Creek.

Direct calibration of the Blackadder Creek model was not possible due to the absence of any meaningful observed levels. A list of historic flood levels was provided by the Water Authority but, as is discussed more fully in section 4.2.1, the data is not of sufficient quality or relevance to Blackadder Creek. Adjustment of the initial estimates of loss parameters were therefore based on the calibration results of the Woodbridge Creek model in conjunction with the comparisons noted during the field investigation. These refinements were made using modelling experience gained on similar streams elsewhere, and with regard to estimates recommended in Australian Rainfall and Runoff (Ref. 7), Chow (Ref. 8) and the US Army Corps of Engineers (Ref. 9). A summary of the energy loss coefficients used in the HEC-2 model of Blackadder Creek is given in Appendix B.

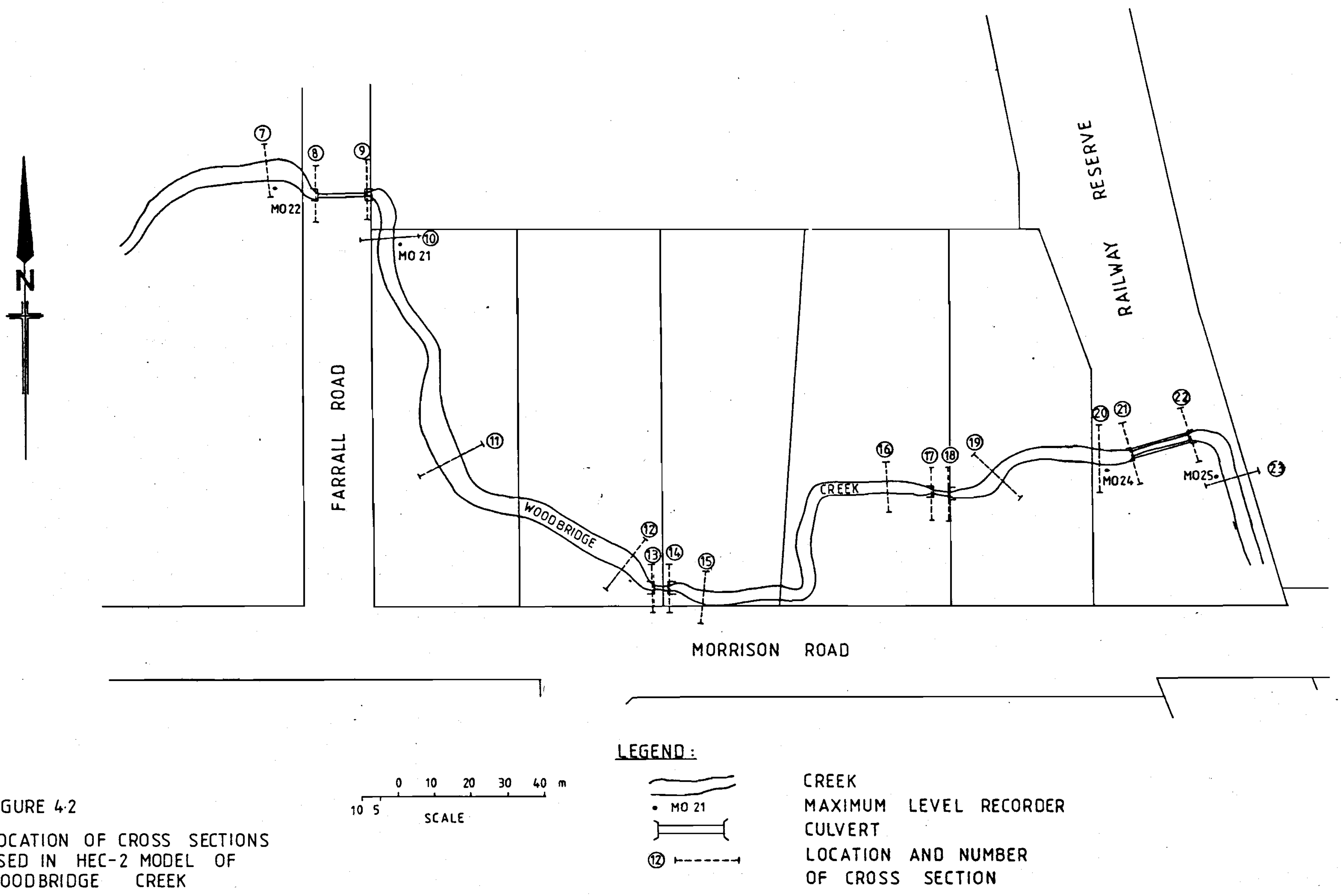

FIGURE 4.2

LOCATION OF CROSS SECTIONS USED IN HEC-2 MODEL OF WOODBRIDGE CREEK

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4.2 Estimation of Historic Flows

As discussed in section 2.4, only two sets of flood level data are available for calibration purposes. This section describes the analysis undertaken to determine the magnitude of flow corresponding to the recorded flood levels. The main technique used was to run the HEC-2 models iteratively with different trial values of flow in an attempt to match predicted levels with recorded stage data. The results were then checked using regional and other analytical methods.

The results of this analysis are used in the calibration of RORB, where the model parameters are adjusted till predictions match the values of flow estimated for the event by HEC-2.

4.2.1 Estimation Based on Historic Flood Level Information

The flow profile along the lower reaches of Blackadder Creek is dependent upon tailwater levels in the Swan River. This backwater effect, in conjunction with the afflux created by the various culverts and bridges, means that a direct slope-area technique could not be used to evaluate the magnitude of flows associated with the stage data. The HEC-2 model is ideally suited to solving such a problem and, by setting the starting elevation of the model equal to the Swan River level, an estimate of flow can be found by trial and error.

Table 4.1 summarises the historic flood level information available for use in the HEC-2 model. A careful examination of the data was made before any simulations were attempted and it was found that most of the events lacked data of sufficient quality to warrant the use of HEC-2. A simulation was only attempted if the following criteria were met:

(i) The tailwater is known and more than one upstream level is available for estimation of flow;

(ii) The data is consistent, i.e. all hydraulic gradients and depths of flow must be positive.

TABLE 4.1 SUMMARY OF HISTORIC FLOOD LEVEL INFORMATION

DATE	WATER LEVEL (mAHD) 1142(1) 100(2)	2354 300	1141 1428	2114 1566	1140 2570	7233 3126	4680 3580	SIMULATION ATTEMPTED	COMMENTS
1917		3.06						No	Insufficient Data
1926		5.56						No	Insufficient Data
05/07/26					3.66(5)			No	Inconsistant Data
07/07/26						5.79(5)	8.29(5)	No	Insufficient Data
1945	5.0 (3)	4.89(4)						No	Inconsistent Data
1946				4.58				No	Insufficient Data
26/08/85			0.88(6)		3.02(6)			No	Inconsistent Data
19/07/54	0.74				3.23(6)			No	Inconsistent Data
26/08/54			1.13					No	Insufficient Data
18/02/55	4.04	4.59(4)	4.09					Yes	Inconsistent Results
18/08/55					4.20(5)			No	Insufficient Data
17/10/55				4.31				No	Insufficient Data
28/07/58	4.37	4.42(4)	4.73					Yes	Inconsistent Results
05/07/63	3.24		3.32		3.76			Yes	Inconsistent Results
25/08/63	4.66		4.70		4.23			Yes	Inconsistent Results
17/07/64	4.18(3)				3.99(5)		8.77(5)	Yes	Inconsistent Results
06/08/64	3.83		3.88		3.88(4)			Yes	Inconsistent Results
02/07/65							8.77	No	Insufficient Data
16/08/66							8.18	No	Insufficient Data
18/08/66						5.35		No	Insufficient Data

NOTES:
1. Water Authority Schedule No.
2. Relevant HEC-2 Section No.
3. Level estimated from Water Authority Schedule 1139
4. Levels recorded as high water marks
5. Within bank flow with depth less than 1.0m.
6. Depth of flow less than 300mm, or level below channel invent.

In Table 4.1 the comments "insufficient data" and "inconsistent data" refer to the data failing to meet the above criteria.

It can be seen that six simulations were attempted for events occurring between the years 1955 and 1964. Where known, the HEC-2 model was adjusted to take account of original channel works.

In all simulations, inconsistent results were obtained. The differences between flows estimated by matching the different levels were greater than one order of magnitude. Typical results indicate flows in excess of 50 m^3/s to match the downstream level, and below 2 m^3/s for the upstream level. Regardless of how well the model reflects channel conditions for the event considered, these results are inconsistent, and are incompatible with the magnitude of the rainfall events considered.

The results of this simulation are not surprising considering the nature of the information available. The most likely factor responsible for the failure of these simulations is that the rainfall intensity-duration characteristics for all but the 1955 event indicate insignificant or negligible flooding for a catchment the size of Blackadder Creek. The estimated average recurrence intervals for some of these events for runoff from Blackadder Creek catchment are shown in Table 3.3. It is therefore likely that the recorded levels are associated with the Swan River flooding and have no relation to flows originating from Blackadder Creek. Furthermore, such an approach assumes that the recorded levels are high water marks - if this is not the case then application of a steady-state flow model is invalid. Without further information on the nature and timing of these flood levels it is not possible to use such information for the estimation of historic flows.

4.2.2 Estimation based on Maximum Level Recorder Data

The information obtained from the maximum level recorders were converted into their corresponding values of flow using HEC-2 and the results were confirmed using hydraulic charts published by the US Bureau of Public Roads (Ref. 10).

Two pairs of maximum level recorders were chosen for analysis. One pair of recorders, M021 and M022, are located on either side of the Farrall Road culvert, and the other pair, M024 and M025, are located on either side of the railway culvert. These recorders were chosen for calibration for the following reasons:

(i) Approximately the same flow should pass through both culverts because the intervening 400m length of catchment has negligible contribution to channel flows;

(ii) The recorders are located at the most downstream end of the gauged sub-catchment and thus the area is more representative of the whole catchment;

(iii) Results from two pairs of recorders can be checked for consistency;

(iv) Using results from the most downstream recorder as a starting elevation, the HEC-2 model can be used confidently because simulations can be matched to three upstream levels.

Table 4.2 shows the results for eight simulations, for storm events that occurred during 1983 and 1984. These results were confirmed using hydraulic charts (Ref. 10), though a check was not possible for simulations 1, 3 and 4 as the affluxes are below the range covered by the nomograms.

TABLE 4.2 SIMULATION RESULTS FOR MAXIMUM LEVEL RECORDER DATA

SIMULATION NO.	DATE	FLOW ESTIMATE (m^3/s) HEC-2
1	16/06/83	1.8(2)
2	27/06/83	2.0
3	23/07/83	1.7(2)
4	29/07/83	1.0(2)
5	04/08/83	1.8(2)
6	03/09/83	2.8(1)
7	17/05/84	1.9
8	13/06/84	1.6(1)

NOTES: (1) Levels matched for both M021/22 and M024/25 for flow shown but using two separate runs with starting elevations at the downstream end of each culvert. Inability to match all levels in a single run most probably due to debris located between the two pairs of recorders.

(2) Data available for M024/25 only, thus no cross-check possible using M021/22.

4.3 Simulation of Probabilistic Floods Under Existing Conditions

4.3.1 Selection of Swan River Tailwater Level

The flood profile along the downstream reach of Blackadder Creek is very dependent upon tailwater levels in the Swan River and thus the downstream boundary of HEC-2 has to be chosen carefully.

Because of the large differences in the times to peak of the Swan River and Blackadder Creek and the large distance between the centroids of the two catchments, it is reasonable to assume that, at any given instant of time, flows in Blackadder Creek are independent of flows in the Swan River. It is therefore appropriate to consider the likelihood of a flood event in terms of the joint probabilities of the concurrence of extreme flows in both streams.

Information on Swan River levels is at present restricted to the 25, 50 and 100 year events. For a 1% joint probability it is therefore only possible to examine the 4, 2 and 1 year floods originating from Blackadder Creek catchment. As the object of this study is analysis of extreme Blackadder Creek flows this approach is rather limited.

The 100 year Swan River level is approximately 4.0m above the flood plain adjacent to the outfall of Blackadder Creek, and extends about one third the way along the modelled section of the catchment. By inspection it is apparent that the upper limit of the 100 year flood profile along the lower reaches of Blackadder Creek is due to the concurrence of the 100 year flood in Swan River and a one year flood in Blackadder Creek. To evaluate the 100 year event along the length of Blackadder Creek it is also necessary to examine extreme flows originating from within the catchment.

Analysis of the Swan River hydrograph recorded at Meadow Street bridge revealed that the peak flows in Blackadder Creek associated with the significant flood events of June and July 1983 occurred about 72 hours before the Swan River reached its maximum level. At the times that Blackadder Creek reached its peak, levels in the Swan River were dominated by tidal influences.

It is therefore reasonable to assume that tailwater levels in Blackadder Creek during extreme events is dependent upon tidal levels in the Swan River. Accordingly an analysis of high tides was undertaken so that the sensitivity of tailwater levels could be evaluated.

Tides theoretically follow a cyclic pattern which repeats itself approximately once every 18.6 years. However, varying meteorological factors such as barometric pressure and wind speed introduce a stochastic element into this periodicity which makes it suited to extreme value analysis.

A 19 year series of annual maximum high tides was abstracted from data supplied by the Fremantle Port Authority. The data was fitted to various frequency distributions to determine the appropriate form. The best fit (by eye) was obtained using the two parameter log-Gumbel distribution. The analysis can be summarised as:

$$HTF = 0.3048\ (\mathrm{Antilog}\ (a + b.y)) - 0.716$$

where, HTF = high tide level at Fremantle (m AHD)

y = Gumbel reduced variate = $-\ln(-\ln((T-1)/T))$

T = average recurrence interval

a = location parameter = 0.695

b = scale parameter = 0.023

and the other constants are used to convert the result to m AHD.

Substituting appropriate values of T into the above equation reveals a small probabilistic tidal range, where the difference between the 2 year and 100 year high tides is only 0.4m. It is therefore apparent that selection of an appropriate probabilistic tidal level is not particularly important and a mean value (where T = 2.3 years) was adopted.

The relationship between the tide level at Fremantle and at the Swan River / Blackadder Creek confluence was determined from regression analysis.

The tidal trace recorded at Fremantle for June and July 1983 was compared to the hydrograph recorded at Meadow Street bridge on the Swan River. A regression analysis was undertaken for all the recorded high tide levels and the following relationship was obtained:

$$TW = 1.31 + 0.32\ (TF) \qquad r^2 = 0.92$$

where, TW = tidal level at Swan River / Blackadder Creek confluence
TF = tide level at Fremantle
$r2$ = correlation coefficient

This equation allows for the 0.4m level difference between Meadow Street bridge and Blackadder Creek outfall, as estimated from the historic flood level information summarised in Table 4.1.

From the frequency analysis the mean level of extreme high tides is 0.84m. This corresponds to a tailwater level at Blackadder Creek of approximately 1.58m AHD, and this was the level chosen as the starting elevation for all probabilistic flood profiles. In practice however, flood levels upstream of Margaret Street are largely insensitive to variations in this starting level.

4.3.2 Modelling the 25 Year, 50 Year and 100 Year Flood Events

Peak flows predicted by RORB for the 25 year, 50 year and 100 year events under existing conditions were used in the HEC-2 model to predict flood profiles. Flood profiles were not evaluated for the creek under future conditions for, as discussed in Section 3.5, the effects of increased urbanisation on extreme events are not significant.

To reduce computing costs, the maximum flood profile for each probabilistic event of interest was determined using the envelope of maximum flows encompassing all critical storm durations for each given recurrence interval. A single HEC-2 model thus incorporates the results of many RORB simulations. This approach is possible as only a steady state analysis is required to predict maximum flood profiles.

Longitudinal profiles for the 25 year, 50 year and 100 year events under existing conditions are shown in Figure 4.3. The corresponding flow velocities for these events are given in Appendix C.

Significant affluxes occur upstream of the culverts located at Margaret Street, Muriel Street, Lloyd Street and the Farrall Road/Railway line crossing. Table 4.3 summarises the affluxes and depths of weir flow appropriate to each recurrence interval for these locations.

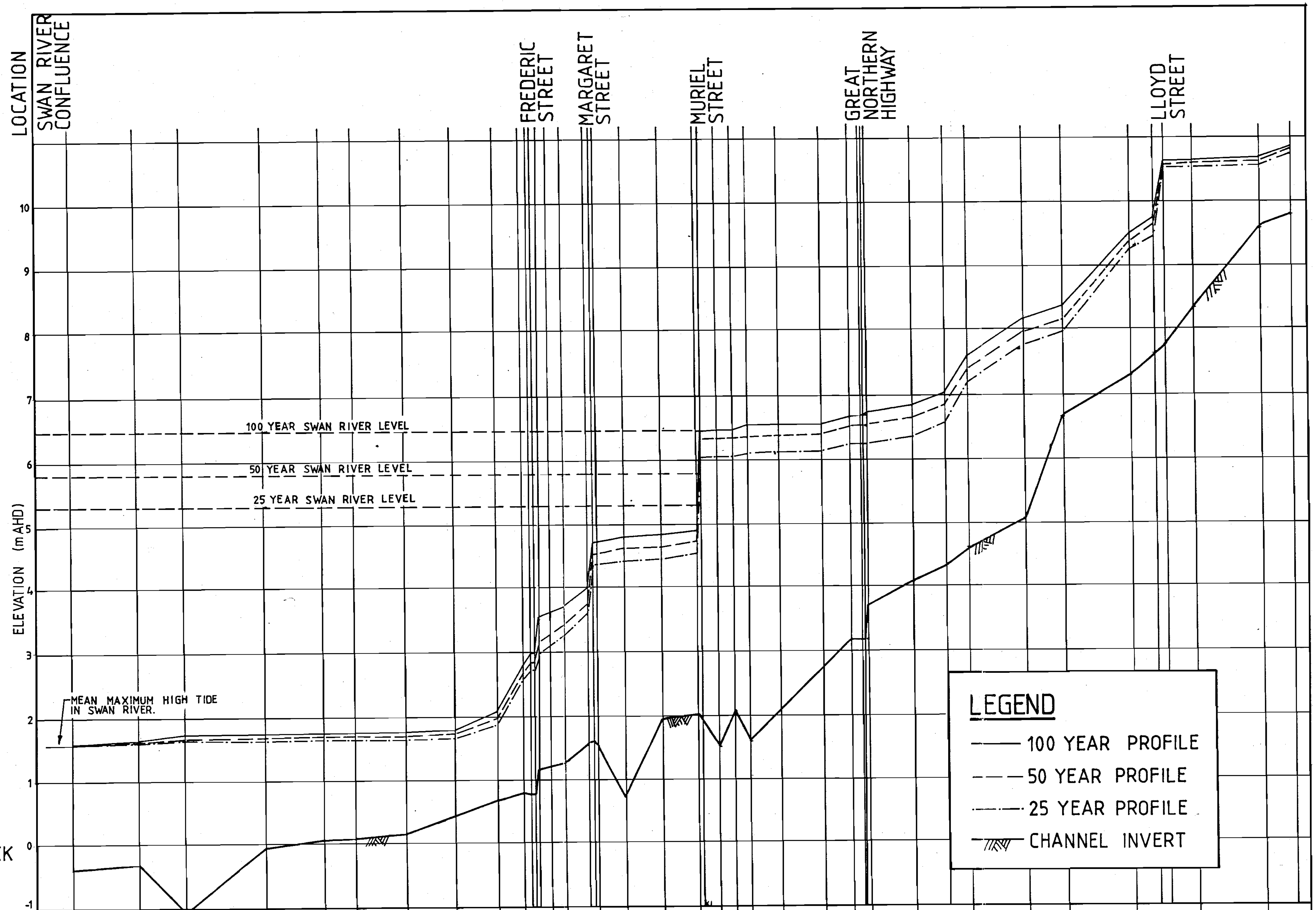
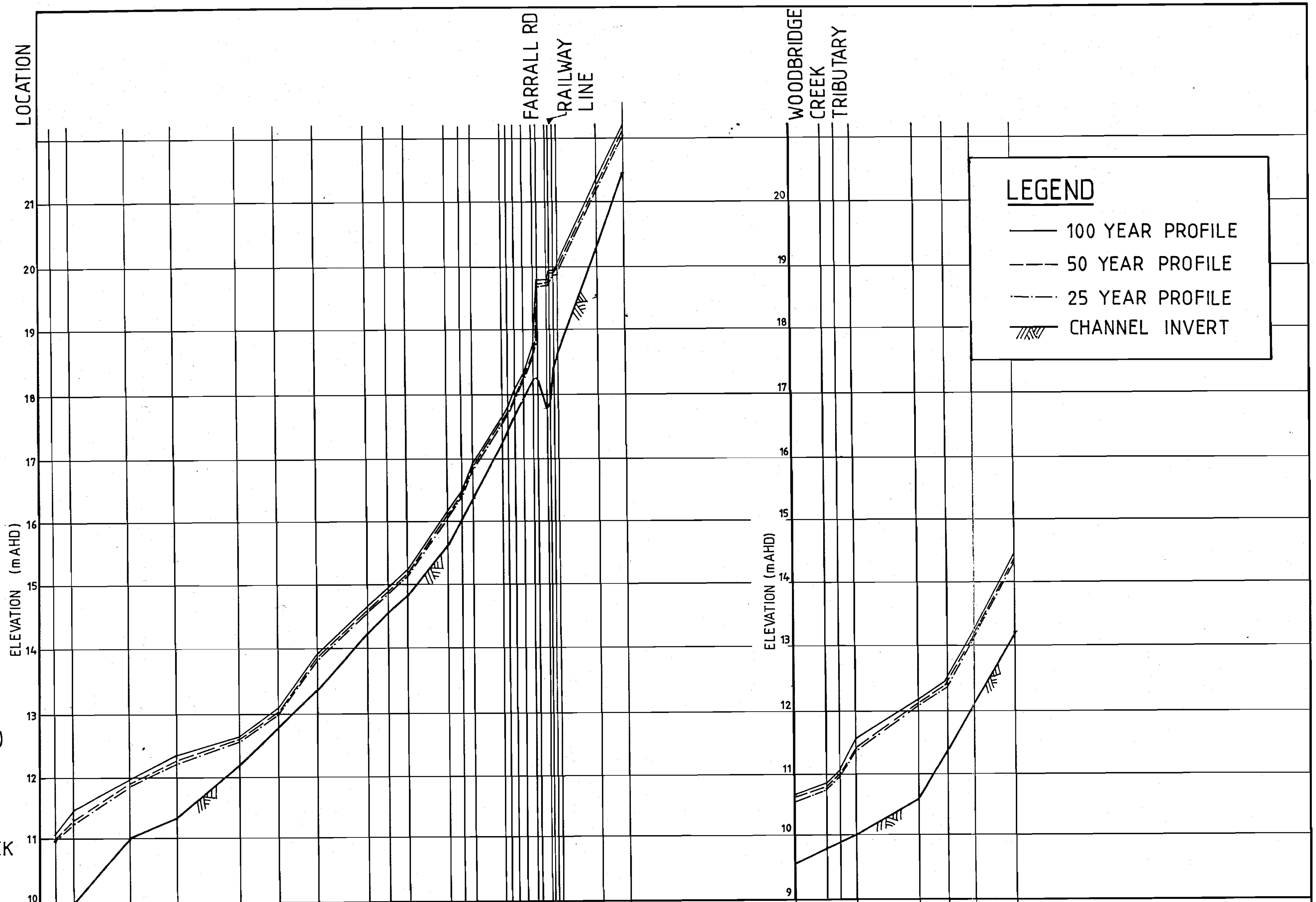


FIGURE 4·3
PROBABILISTIC
FLOOD PROFILES
UNDER EXISTING
CONDITIONS.
BLACKADDER CREEK
CATCHMENT

SECTION N°. (≡ CHAINAGE,m)	100	300	450	698	892	994	1156	1308	1428	1516	1566	1644	1752	1852	1964	2076
25 YEAR WATER LEVEL	1·58	1·61	1·65	1·66	1·66	1·67	1·67	1·69	1·88	2·52	2·98	3·25	4·40	4·46	4·48	4·55
50 YEAR WATER LEVEL	1·58	1·63	1·68	1·70	1·71	1·71	1·72	1·74	1·98	2·66	3·16	3·41	4·57	4·64	4·66	4·73
100 YEAR WATER LEVEL	1·58	1·66	1·73	1·75	1·76	1·76	1·77	1·80	2·08	2·79	3·55	3·70	4·74	4·82	4·85	4·92

SECTION N°. (≡ CHAINAGE,m)	2126	2185	2226	2326	2466	2570	2630	2770	2948	3126	3254	3470	3668	3889
25 YEAR WATER LEVEL	6·09	6·08	6·12	6·13	6·14	6·23	6·28	6·36	7·19	7·74	7·94	9·25	10·57	10·59
50 YEAR WATER LEVEL	6·35	6·35	6·39	6·40	6·41	6·51	6·56	6·64	7·40	7·97	8·16	9 37	10·62	10·64
100 YEAR WATER LEVEL	6·49	6·48	6·54	6·55	6·57	6·68	6·75	6·84	7·59	8·17	8·37	9·48	10·67	10·70

FIGURE 4·3 (CONT)
PROBABILISTIC
FLOOD PROFILES
UNDER EXISTING
CONDITIONS
BLACKADDER CREEK
CATCHMENT



25 YEAR WATER LEVEL	11·29	11·89	12·23	12·58	13·89	14·59	15·21	16·10	17·58	18·81	19·74	19·88	21·27	22·11	10·59	11·40	12·11	12·41	13·23	14·33
50 YEAR WATER LEVEL	11·34	11·95	12·28	12·62	13·91	14·62	15·24	16·12	17·62	18·85	19·77	19·91	21·31	22·15	10·64	11·45	12·18	12·46	13·24	14·38
100 YEAR WATER LEVEL	11·39	12·00	12·32	12·65	13·94	14·64	15·27	16·14	17·64	18·89	19·80	19·94	21·36	22·19	10·70	11·50	12·23	12·49	13·26	14·43
SECTION N°. (≡CHAINAGE,m)	4102	4280	4425	4628	4873	5037	5168	5295	5457	5567	5610	5635	5779	5863	3889	4102·1	4102·2	4102·3	4102·4	4102·5

TABLE 4.3 EFFECTS OF BRIDGES UPON FLOOD PROFILES

LOCATION	CULVERT SIZE	HEADWALL HEIGHT (m)	RECURRENCE INTERVAL (Yr)	AFFLUX (m)	DEPTH OF WEIR FLOW (m)
Frederic Street	Trapezoidal Bridge Opening	0.6	25	0.26	0
			50	0.29	0
			100	0.55	0
Margaret Street	2 No. 1.39 dia.	1.0	25	0.81	0.68
			50	0.81	0.85
			100	0.76	1.02
Muriel Street	2 No. 1.83 dia.	2.2	25	1.54	0.08
			50	1.62	0.34
			100	1.57	0.47
Lloyd Street	3 No. 1.22 dia.	1.4	25	1.07	0.11
			50	0.97	0.15
			100	0.89	0.19
Farrall Rd/ Railway Line	1 No. 0.74 dia.	1.1	25	1.07	0.14
			50	1.06	0.17
			100	1.05	0.20

Figure 4.3 also indicates the 25 year, 50 year and 100 year Swan River flood levels. It is apparent that the 100 year maximum flood profile along the lower reaches of Blackadder Creek is determined solely by Swan River levels, as is discussed in Section 4.3.1.

4.3.3 Extent of the 100 Year Flood

The 100 year flood extent was derived using 1:2 000 contour plans, cross- sectional survey information, and the water surface profiles shown in Figure 4.3. The extent of flooding is shown on the attached Flood Plain Maps (Drawing Nos. AQ38-1-1 and AQ38-1-2 (Sheets 1 to 7)). Flooding from both the Swan River and Blackadder Creek is plotted on the plans to illustrate the relative contribution of each to the envelope of flood extent.

The contours are based upon aerial photography dated April 1974 and consequently do not account for any localised land-fill placed within the last 12 years. Also, information on the location and level of buildings is not available. Where the extent of flooding is shown to inundate developed lots, further survey will be required to establish both the extent of flooding and which buildings within the inundated area would be flooded above floor level.

The HEC-2 cross-sections used in the study are also plotted on the flood plain maps. These can be used in conjuction with the flood level information of Figure 4.3 to precisely locate the extent of flooding within a particular area of interest, as further survey data becomes available.

The main cause for concern with flooding in the study area is due to the Swan River. The flood plain maps were used in conjunction with 1:5 000 scale orthophotomaps to determine areas of existing development that appeared to be at risk from flooding. The following list summarises those developed lots that are partly subject to inundation in the 100 year event:

- Harper Street, Lots 43 - 58
  Lots 27 - 31

- William Street/Morrison Road, Lots 8 - 18

- Charles Street, Lots 165 - 167

- Frederic Street, Lots 122 - 134
  Lots 142 - 148

- Great Northern Highway, Lots 6 - 9

- Dudley Street, Lots 48 - 50
  Lots 68 - 70
  Lots 73 - 76
  Lots 82 - 113

Elsewhere, the extent of flooding is limited to currently undeveloped areas of floodplain. The extent of flooding in areas of future development can be easily controlled by appropriate choice of fill levels and channel improvement works. The actual floodway required will depend on the nature of future development and the relevant flows that can be estimated from Figure 3.5 and Table 3.8.

5.0 FLOOD MITIGATION MEASURES

5.1 General

As discussed in sections 4.3.2 and 4.3.3, the major cause of flooding is the Swan River. Therefore, with respect to the extreme flood events considered in this study, there are no requirements for flood mitigation measures below Swan River flood levels.

From Figure 4.3, it can be seen that the Swan River flood levels extend up to Muriel Street, a distance of approximately 2km from Blackadder Creek outfall. Although the affluxes created at both Muriel Street and Margaret Street culverts are very large, it is difficult to justify upgrading these structures as any improvements would not mitigate flooding from the Swan River.

With reference to the Flood Plain Maps, it is apparent that there are no existing developments at risk from flooding upstream of Muriel Street. There is therefore no immediate requirement for flood mitigation measures along the upper reaches of Blackadder Creek.

5.2 Floodway Requirements

An evaluation of a minimum floodway requirement was undertaken for the section of Blackadder Creek that passes through Midland. Low lying land adjacent to the creek is in places poorly drained and local residents are keen to fill these areas to obtain year-round use of their land.

Flood encroachments were added to the HEC-2 model to simulate a reduced waterway area. A minimum floodway width was determined by trial and error such that increases above existing water levels were restricted to within 150mm.

The increases in water level associated with decreasing floodway widths are shown in Table 5.1. The values given in Table 5.1 should be regarded as only indicative of the increases as the imposed flood encroachments do not allow for the sloping batter of adjacent fill.

TABLE 5.1 INCREASE IN WATER LEVEL ABOVE EXISTING CONDITIONS FOR SELECTED FLOODWAY WIDTHS

SECTION NO.	INCREASE IN WATER LEVEL (mm)		
	40m FLOODWAY	30m FLOODWAY	25m FLOODWAY
1426	0	0	10
1516	0	30	70
1644	20	80	120
1752	0	20	20
1852	20	50	70
1964	30	70	90
2076	50	60	160
2126	10	10	20
2185	10	30	40
2226	0	20	40
2326	0	30	50
2466	10	50	80
2570	10	50	80
2630	0	50	80
2770	10	70	120
2948	0	0	0

The effects of not modelling the batter slope of the encroachment vary depending upon the geometry of the cross-sections concerned, however the actual flood profile would be between 0 and 40mm above the levels indicated.

From Appendix C it can be seen that existing flow velocities are quite low, with overbank velocities less than 0.5 m/s and channel velocities (other than at bridge sections) less than 1.0 m/s. The increase in flow velocities due to the reduction in floodway width is not significant, and is generally less than 10%.

Table 5.1 indicates that adoption of a 30m wide floodway would ensure that increases in water levels would be limited to within 150mm of existing conditions. The associated increase in flow velocities is minor, and scour protection measures need not be taken.

The location of the 30m wide floodway is shown on the drawings. It will be noted from the drawings that some existing sections of the creek have a width of less than 30m at 100 year flood level. Obviously the channel will need to remain as it is, with no further encroachment.

6.0 CONCLUSIONS AND RECOMMENDATIONS

The investigations into the flood potential of Blackadder Creek have resulted in the following conclusions and recommendations:

1. Figures have been prepared that show the longitudinal water surface profiles for the 25 year, 50 year and 100 year floods.

2. A series of 1:2000 maps has been developed that delineates the 100 year flood extent for Blackadder Creek between the Swan River and just upstream of the railway at Wexcombe.

3. The flood extent maps and longitudinal profiles should prove useful to both State and Local Government bodies in developing strategies for future development of the area.

4. The main cause for concern with flooding in the study area is due to the Swan River. Several developed lots in Midland are subject to inundation in the 100 year event, but further survey is required to determine if any buildings would be flooded above floor level.

5. No existing developments are at risk of flooding from Blackadder Creek and no flood mitigation measures are considered necessary at present other than periodic maintenance to ensure vegetal growth in the waterway does not constitute a significant obstruction to flows.

6. A minimum floodway width of 30m is considered appropriate to the section of Blackadder Creek that passes through Midland. A floodway of this width would limit the rise in flood profile to within 150mm of existing levels and it would not significantly increase flow velocities.

7. The Water Authority should ensure that existing floodways in areas that will be subject to future development are preserved as floodways.

This report is dated December 1986.

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APPENDIX A

Energy Loss Coefficients

Used in HEC-2 Model of

Woodbridge Creek

TABLE 1A ENERGY LOSS COEFFICIENTS USED IN HEC-2 MODEL OF WOODBRIDGE CREEK

SECTION NO.	CHANNEL "n"	CONTRACTION COEFFICIENT	EXPANSION COEFFICIENT
7	0.036	0.1	0.3
8	0.036	0.6	0.8
9	0.036	0.6	0.8
10	0.036	0.6	0.8
11	0.036	0.1	0.3
12	0.036	0.1	0.3
13	0.036	0.5	0.7
14	0.036	0.5	0.7
15	0.036	0.5	0.7
16	0.036	0.1	0.3
17	0.036	0.5	0.7
18	0.036	0.5	0.7
19	0.036	0.5	0.7
20	0.036	0.1	0.3
21	0.036	0.5	0.7
22	0.036	0.5	0.7
23	0.036	0.5	0.7

Note: Only within-bank flows considered.

APPENDIX B

Energy Loss Coefficients

Used in HEC-2 Model of

Blackadder Creek

TABLE B1 ENERGY LOSS COEFFICIENTS USED IN HEC-2 MODEL OF BLACKADDER CREEK

SECTION NO.	CHANNEL "n"	LEFT O/B "n"	RIGHT O/B "n"	CONTRACTION COEFFICIENT	EXPANSION COEFFICENT
100	0.03	0.055	0.055	0.1	0.3
300	0.03	0.055	0.055	0.1	0.3
450	0.048	0.140	0.120	0.1	0.3
698	0.048	0.140	0.120	0.1	0.3
892	0.032	0.036	0.036	0.1	0.3
994	0.032	0.036	0.036	0.1	0.3
1156	0.032	0.036	0.036	0.1	0.3
1308	0.075	0.09	0.09	0.1	0.3
1428	0.075	0.09	0.09	0.1	0.3
1516	0.075	0.14	0.14	0.1	0.3
1543	0.075	0.14	0.14	0.3	0.5
1550	0.075	0.14	0.14	0.3	0.5
1566	0.075	0.14	0.14	0.3	0.5
1644	0.075	0.14	0.14	0.1	0.3
1727	0.075	0.14	0.14	0.6	0.8
1737	0.075	0.14	0.14	0.6	0.8
1752	0.075	0.14	0.14	0.6	0.8
1852	0.075	0.14	0.14	0.1	0.3
1964	0.075	0.14	0.14	0.1	0.3
2076	0.075	0.14	0.14	0.6	0.8
2083	0.085	0.12	0.12	0.6	0.8
2126	0.085	0.12	0.12	0.6	0.8
2185	0.085	0.12	0.12	0.1	0.3
2226	0.085	0.12	0.12	0.1	0.3
2326	0.085	0.12	0.12	0.1	0.3
2466	0.085	0.12	0.12	0.1	0.3
2570	0.085	0.12	0.12	0.1	0.3
2602	0.085	0.12	0.12	0.3	0.5
2620	0.085	0.17	0.17	0.3	0.5
2630	0.085	0.12	0.12	0.3	0.5
2770	0.045	0.060	0.055	0.1	0.3

SECTION NO.	CHANNEL "n"	LEFT O/B "n"	RIGHT O/B "n"	CONTRACTION COEFFICIENT	EXPANSION COEFFICENT
2859	0.045	0.060	0.055	0.1	0.3
3126	0.045	0.060	0.055	0.1	0.3
3254	0.045	0.060	0.055	0.1	0.3
3470	0.045	0.060	0.055	0.1	0.3
3550	0.045	0.060	0.055	0.3	0.5
3580	0.045	0.060	0.055	0.3	0.5
3668	0.045	0.060	0.055	0.3	0.5
3889	0.045	0.045	0.045	0.1	0.3
3989	0.045	0.045	0.045	0.1	0.3
4014	0.045	0.045	0.045	0.1	0.3
4046	0.045	0.045	0.045	0.1	0.3
4074	0.045	0.045	0.045	0.1	0.3
4102	0.045	0.045	0.045	0.1	0.3
4280	0.045	0.045	0.045	0.1	0.3
4425	0.045	0.045	0.045	0.1	0.3
4628	0.045	0.045	0.045	0.1	0.3
4750	0.045	0.045	0.045	0.1	0.3
4873	0.045	0.045	0.045	0.1	0.3
5037	0.055	0.045	0.045	0.1	0.3
5100	0.055	0.045	0.045	0.1	0.3
5168	0.055	0.045	0.045	0.1	0.3
5295	0.055	0.045	0.045	0.1	0.3
5340	0.055	0.045	0.045	0.1	0.3
5380	0.055	0.045	0.045	0.1	0.3
5457	0.055	0.045	0.045	0.1	0.3
5485	0.055	0.045	0.045	0.1	0.3
5512	0.055	0.045	0.045	0.1	0.3
5538	0.055	0.045	0.045	0.1	0.3
5567	0.055	0.045	0.045	0.4	0.7
5577	0.055	0.045	0.045	0.4	0.7
5610	0.050	0.065	0.065	0.4	0.7
5620	0.050	0.065	0.065	0.4	0.7
5635	0.050	0.065	0.065	0.4	0.7

SECTION NO.	CHANNEL "n"	LEFT O/B "n"	RIGHT O/B "n"	CONTRACTION COEFFICIENT	EXPANSION COEFFICENT
5644	0.050	0.065	0.065	0.1	0.3
5649	0.050	0.065	0.065	0.1	0.3
5653	0.050	0.065	0.065	0.1	0.3
5670	0.050	0.065	0.065	0.1	0.3
5685	0.050	0.065	0.065	0.1	0.3
5700	0.050	0.065	0.065	0.1	0.3
5716	0.050	0.065	0.065	0.1	0.3
5779	0.050	0.065	0.065	0.1	0.3
5863	0.050	0.065	0.065	0.1	0.3
4102.1	0.045	0.045	0.045	0.1	0.3
4102.2	0.045	0.045	0.045	0.1	0.3
4102.3	0.045	0.045	0.045	0.1	0.3
4102.4	0.045	0.045	0.045	0.1	0.3
4102.5	0.045	0.045	0.045	0.1	0.3

APPENDIX C

Blackadder Creek

Overbank and Channel Velocities

25, 50 and 100 Year Events

TABLE C1 BLACKADDER CREEK VELOCITIES - 25 YEAR EVENT

SECTION NO.	LOCATION	FLOW VELOCITY (m/s) LEFT O/B	CHANNEL	RIGHT O/B
300	Outfall	0.06	0.60	0.21
994		0.13	0.20	0.11
1428	Elvire Street	0.83	1.24	0.82
1550	Frederic Street Bridge	-	2.34	-
1644		0.19	1.04	0.33
1752	D/S of Margaret Street	0.18	0.84	0.38
1964		0.19	0.46	0.19
2076	D/S of Muriel Street	-	2.21	-
2226		0.20	0.34	0.18
2466		0.38	0.70	0.38
2630	Gt Northern Hwy Bridge	0.27	0.45	0.21
2770		0.95	1.83	1.10
3254		0.90	2.18	1.43
3550	D/S of Lloyd Street	1.03	1.84	0.70
3889		0.51	0.68	0.13
4425		0.37	0.70	0.33
4873		0.32	0.79	0.52
5295		0.45	0.61	0.35
5567	D/S of Farrall Road	0.89	1.15	0.88
5779		0.34	1.28	0.52

TABLE C2 BLACKADDER CREEK VELOCITIES - 50 YEAR EVENT

SECTION NO.	LOCATION	FLOW VELOCITY (m/s) LEFT O/B	CHANNEL	RIGHT O/B
300	Outfall	0.08	0.74	0.26
994		0.16	0.24	0.13
1428	Elvire Street	0.93	1.38	0.91
1550	Frederic Street Bridge	-	2.78	-
1644		0.22	1.11	0.37
1752	D/S of Margaret Street	0.20	0.92	0.42
1964		0.22	0.52	0.22
2076	D/S of Muriel Street	-	2.56	-
2226		0.22	0.39	0.21
2466		0.41	0.76	0.42
2630	Gt Northern Hwy Bridge	0.29	0.48	0.21
2770		0.95	1.85	0.95
3254		0.90	2.15	1.45
3550	D/S of Lloyd Street	0.82	2.02	0.84
3889		0.57	0.75	0.21
4425		0.40	0.73	0.36
4873		0.38	0.82	0.55
5295		0.48	0.62	0.40
5567	D/S of Farrall Road	1.03	1.33	0.97
5779		0.42	1.37	0.55

TABLE C3 BLACKADDER CREEK VELOCITIES - 100 YEAR EVENT

SECTION NO.	LOCATION	FLOW VELOCITY (m/s) LEFT O/B	CHANNEL	RIGHT O/B
300	Outfall	0.10	0.87	0.31
994		0.19	0.28	0.15
1428	Elvire Street	1.01	1.49	0.98
1550	Frederic Street Bridge	-	3.00	-
1644		0.26	1.03	0.37
1752	D/S of Margaret Street	0.35	1.72	0.80
1964		0.25	0.58	0.24
2076	D/S of Muriel Street	-	2.92	-
2226		0.26	0.45	0.25
2466		0.46	0.85	0.47
2630	Gt Northern Hwy Bridge	0.32	0.53	0.22
2770		1.00	1.96	0.92
3254		0.89	2.11	1.25
3550	D/S of Lloyd Street	0.70	2.16	0.95
3889		0.62	0.80	0.28
4425		0.42	0.76	0.38
4873		0.42	0.85	0.58
5295		0.52	0.65	0.44
5567	D/S of Farrall Road	1.13	1.46	1.01
5779		0.12	0.18	0.18