BLACKJACK CREEK

FLOOD STUDY

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FORWARD

The State Government's Flood Policy is directed at providing solutions to existing flooding problems in developed areas and to ensuring that new development is compatible with the flood hazard and does not create additional flooding problems in other areas.

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

The Policy provides for technical and financial support by the Government through the following four sequential stages:

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

The Blackjack Creek Flood Study constitutes the first stage of the process for this area and has been prepared for Gunnedah Council to define flood behaviour under current conditions.

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NOTE ON FLOOD FREQUENCY

The frequency of floods is generally referred to in terms of their Annual Exceedence Probability (AEP) or Average Recurrence Interval (ARI). For example, for a flood magnitude having 5% AEP, there is a 5% probability that there will be floods of greater magnitude each year. As another example, for a flood having a 5 year ARI, there will be floods of equal or greater magnitude once in 5 years on average. The approximate correspondence between these two systems is:

ANNUAL EXCEEDENCE PROBABILITY (AEP) %	AVERAGE RECURRENCE INTERVAL (ARI) YEARS
0.5	200
1	100
5	20
20	5

Reference is also made in the report to the probable maximum flood (PMF). This flood occurs as a result of the probable maximum precipitation (PMP). The PMP is the result of the optimum combination of the available moisture in the atmosphere and the efficiency of the storm mechanism as regards rainfall production. The PMP is used to estimate PMF discharges using a model which simulates the conversion of rainfall to runoff. The PMF is defined as the limiting value of floods that could reasonably be expected to occur.

ABBREVIATIONS

AEP	Annual Exceedence Probability (%)
AHD	Australian Height Datum
ARI	Average Recurrence Interval (years)
ARR	Australian Rainfall and Runoff, 1987 Edition
BOM	Bureau of Meteorology
DIPNR	Department of Infrastructure Planning and Natural Resources (formerly, the Department of Land and Water Conservation)

1. INTRODUCTION

1.1 Study Background

A comprehensive floodplain risk management plan (FRMP) is to be prepared for Blackjack Creek as part of a Government program to mitigate the impacts of major floods and reduce the hazards in the floodplain. An important first step in the process of preparing an FRMP is the undertaking of the flood study for the stream. The flood study is the formal starting process of defining management measures for flood liable land and represents a detailed technical investigation of flood behaviour.

Mathematical models of the catchment and the floodplain were developed using detailed field surveys and interpreted to present a comprehensive picture of flooding under present day conditions.

The study objective was to define flood behaviour in the streams in terms of flows, levels and flooding behaviour for flood frequencies ranging between 5 and 100 years average recurrence interval (ARI), as well as for the Probable Maximum Flood (PMF).

Flood behaviour was defined using a computer based hydrologic model of the catchment and a hydraulic model of the stream channel and floodplain.

The hydrologic model was a runoff-routing model. As there is no stream flow data available on the Blackjack Creek catchment, model parameters were estimated using relationships derived in similar investigations and published in the engineering literature. Design storms were then applied to the model to generate discharge hydrographs within the study area. Peak flows from those hydrographs constituted the upstream boundary and tributary inflow inputs to the hydraulic model.

A network hydraulic model was adopted for the hydraulic analysis to model flows in the main channels and floodplains. A one-dimensional model was chosen which allowed for the interaction of flows between the channel and the floodplain, flow through culverts and flow over control structures such as road embankments.

Several flood marks had been levelled during the survey of the creek undertaken to provide basic topographic information for the hydraulic model. These marks assisted with the selection of model parameters. The hydraulic model was then used to derive water surface profiles for the flows generated from the hydrologic model, as well as provide an assessment of the flow distribution and average velocities of flow for the design events.

1.2 Study Tasks

The flood study had three main components:

(1) **Review of available hydrologic and hydraulic data and previous investigations.** A brief was prepared for cross sectional survey of Blackjack Creek channel and floodplain. Stewart Surveyors undertook the survey. Several historic flood marks were levelled during the creek survey. Rainfall data for those historic storms were supplied by the Bureau of Meteorology.

Council supplied a contour plan of the catchment from data prepared by the Central Mapping Authority. This information was used to define sub-catchments for the catchment model.

(2) **A hydrologic component,** which included preparation and tuning of the hydrologic model, estimation of design storms and their application to the hydraulic model.

(3) **A hydraulic component,** which comprised the preparation and testing of the hydraulic model and the definition of the water surface profiles, flows and velocities for the design floods.

1.3 Overview of Report

Section 2 contains background information including a description of the catchment, a brief review of the data base available for the study and a discussion on the history of flooding in the catchment.

Section 3 deals with the hydrology of the catchment. The RORB runoff-routing program was adopted for this study.

Section 4 describes the computation of design flows using the RORB hydrologic model. This step involved the determination of design storm rainfall depths over the catchments for a range of storm durations, and conversion of the rainfall hyetographs to discharge hydrographs.

Section 5 deals with the development of the hydraulic model. The HEC-RAS software was used for this purpose. The model was tuned on the basis of available historic flood level data.

Section 6 details the results of the hydraulic modelling of the design floods using HEC-RAS. Results are presented as tabulations of peak levels, water surface profiles and plans showing indicative extents of inundation for each of the design flood events.

Section 7 contains a list of references.

Supplementary details are given in the Appendices. **Appendix A** contains tabulations of flood level, discharge and velocity data for design storm events between 5 and 100 year ARI, as well as the PMF. **Appendix B** contains cross sections of the creek and its floodplain used to develop the hydraulic model and contains a plan showing the hydraulic categorisation of the floodplain for the 100 year ARI flood.

2. BACKGROUND

2.1 Catchment Description

Blackjack Creek drains the catchment to the south of Gunnedah and runs along the western side of the urban area of town, through the area known as the Wandobah Reserve. The stream crosses the Oxley Highway and the railway, before discharging to the floodplain of the Namoi River. **Figure 2.1** is a plan of the Blackjack Creek catchment.

The total catchment area at the Oxley Highway crossing is about 24 km^2. The main arm of Blackjack Creek flows northwards over a distance of 8 km from the catchment boundary to the Highway crossing. The catchment headwaters are quite steep, with natural surface levels falling from RL 670 m at the highest point near the south-west boundary to RL 284 m at Lincoln Street over a distance of 5.5 km and at an average gradient of 7 per cent.

At Lincoln Street the stream flattens, with an average bed slope of 0.78 per cent over the remaining 2.5 km to the Highway bridge. The floodplain in this reach averages about 300 m in width and comprises cleared overbanks on the western side and urban areas on the eastern side. Downstream of Lincoln Street, the stream runs parallel with and close to Wandobah Road.

A levee bank has been constructed on the eastern bank between George Street and Kilcoy Street to contain flows which surcharge the hydraulic capacity of the channel. However, it is likely that in the event of major flooding, the stream would break its banks further upstream and outflank the levee. During those events Wandobah Road would act as a floodway and flooding would extend into the urban area on the eastern side of the road.

The waterway at the Oxley Highway crossing comprises 12 box culverts with a total width of 33.7 m and height of 1.5 m. The channel from the Highway to the railway comprises a grassed trapezoidal floodway of around 30 m width. The railway crossing comprises a three span bridge with each span 8 m wide and about 2.5 m high.

Between the Oxley Highway and the railway culvert, a large rectangular shaped concrete drain joins the right bank of Blackjack Creek. This drain, known locally as Ashfords Watercourse, conveys runoff from the 3.2 km^2 catchment to the east of Blackjack Creek. Council has assessed that peak flows from this catchment could reach 17-18 m^3/s in the event of major flooding. As Ashfords Watercourse is likely to introduce a backwater effect and influence flood levels at the Oxley Highway, contributions to flow from that catchment have been included in the hydraulic modelling of Blackjack Creek, which continues below the Highway to a point downstream of the railway culvert.

2.2 Data Base

There are no stream flow data available on Blackjack Creek to assist with tuning the models. A major storm occurred on 30 January 1984, which was reported by Council to have resulted in inundation of the Blackjack Creek floodplain, with flows extending into the residential area on the eastern side of Wandobah Road and overtopping the Oxley Highway bridge. The peak flood level on the upstream side of the bridge was about 600 mm over the deck. Other significant floods are reported to have occurred in the wet years 1971 and 1976, but there is no quantitative data available for those events.

During the course of the creek survey undertaken for the study by Stewart Surveys, four flood marks were levelled for a flood event, which occurred in 1955. This event occurred in February 1955, when record flooding was experienced in inland NSW. It is understood from Council that whilst the February 1955 rainfalls resulted in major flooding in the Namoi River, local falls on the Blackjack Creek catchment were less severe. This fact was confirmed by a review of rainfall data recorded at the Gunnedah SCS pluviograph.

The Bureau of Meteorology (BOM) supplied pluviographic data for the Gunnedah SCS site, which allowed assessment of the temporal pattern of rainfall experienced on the Blackjack Creek catchment for both the 1984 and 1955 storms (**Figure 2.2**).

Following a site inspection of the catchment, a brief was prepared for a cross sectional survey of the channel and floodplain in the reach between Lincoln Street and the railway bridge. Stewart Surveys undertook the survey.

2.3 Previous Flood Investigations

A Flood Study was carried out by F R Kelly and Associates for the Blackjack Creek catchment in November 1984. The objective of this study was to present concept designs for improvements extending through Wandobah Reserve from Lincoln Street to the Oxley Highway.

F R Kelly and Associates Design assessed discharges using the Pilgrim and McDermott version of the Rational Method, with an allowance for the increase in peak flows resulting from urbanisation of the catchment. Peak flows at the Oxley Highway were estimated to increase from 43 m^3/s for the 5 year ARI flood to 90 m^3/s for the 100 year ARI. The contributing catchment at the Oxley Highway was estimated as 24.5 km^2.

2.4 January 1984 Storm

The January 1984 rainfall intensities as recorded at the Gunnedah SCS indicate that a total depth of 131 mm were experienced over the 24 hours commencing at 0900 hours on 29 January. A total depth of 77 mm had been experienced over the two days 27 and 28 January and consequently, the catchment would have been very wet. Initial and continuing rainfall losses over the storm of the 29 – 30 January would have been low. The cumulative depth of rainfall recorded for that event and the February 1955 storm are plotted on **Figure 2.2**.

Over the three hours of the most intense burst of rainfall on 30 January 1984, a total depth of 82.5 mm were recorded at the Gunnedah SCS pluviograph, compared with 88 mm for the 1 in 100 year rainfall of the same duration. For the 5 hour duration, a total depth of 106 mm was recorded, which exceeds the 1 in 100 year depth of 103 mm.

Storms of between 3 and 6 hours duration would be expected to maximise peak flows on the Blackjack Creek catchment. Consequently, on the basis of recorded rainfall depths, the January 1984 storm was a 1 in 100 year event. In view of the heavy rainfall experienced over the preceeding days, rainfall losses would have been much less than the average loss rates used in design flood estimation. It is likely that the peak discharge experienced would have been considerably in excess of the 100 year ARI design discharge.

2.5 February 1955 Storm

Rainfall records for the Gunnedah area show that over the five day period from 23 February to 27 February 1955 inclusive, a total depth of 176 mm was recorded. Most of the rain was experienced on 24 February, when 92 mm fell.

Cumulative depths of rainfall recorded at the Gunnedah SCS pluviograph station are shown on **Figure 2.2**. The maximum three and six hour depths were 49 and 70 mm respectively, equivalent to around a 1 in 10 year return period.

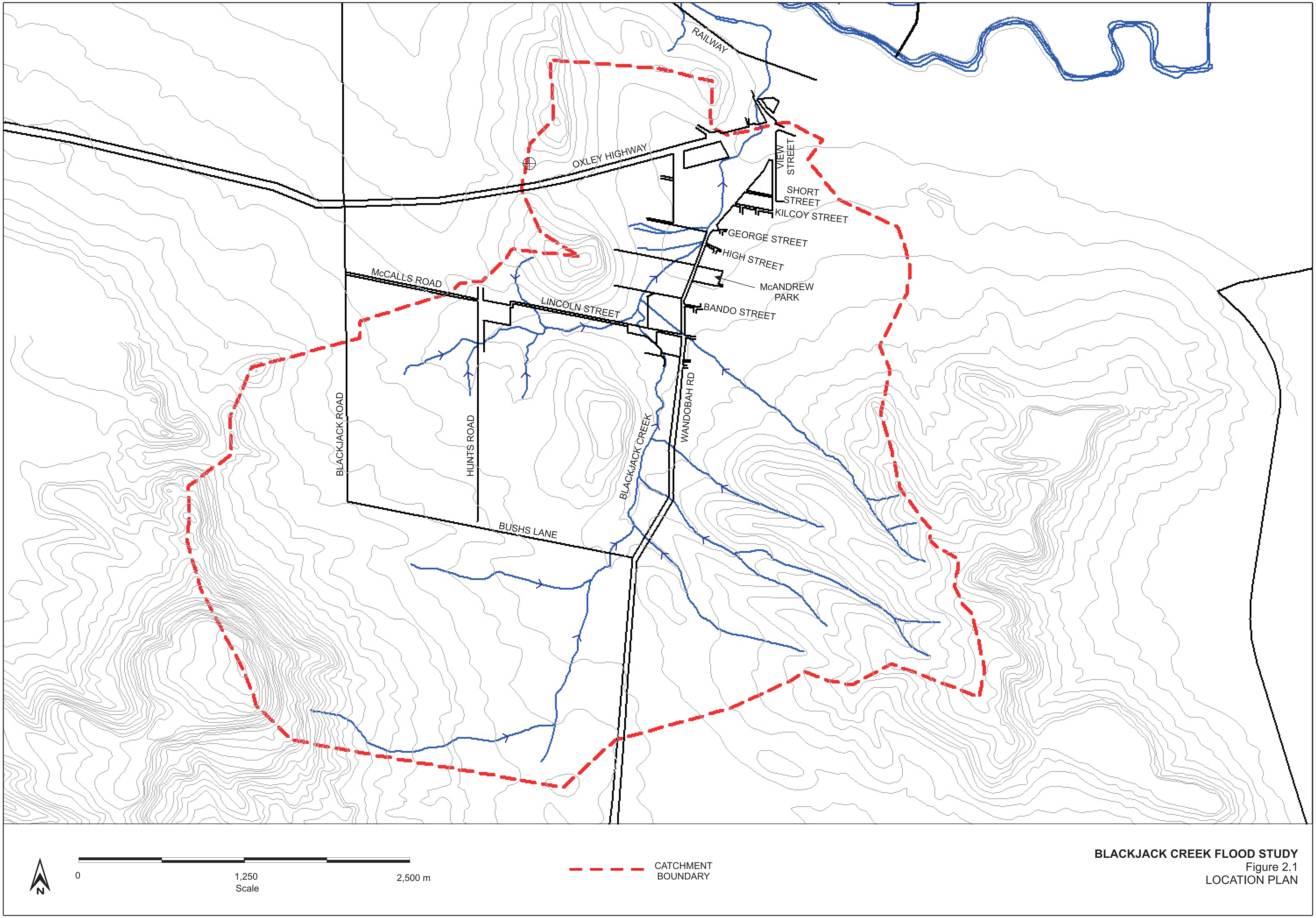

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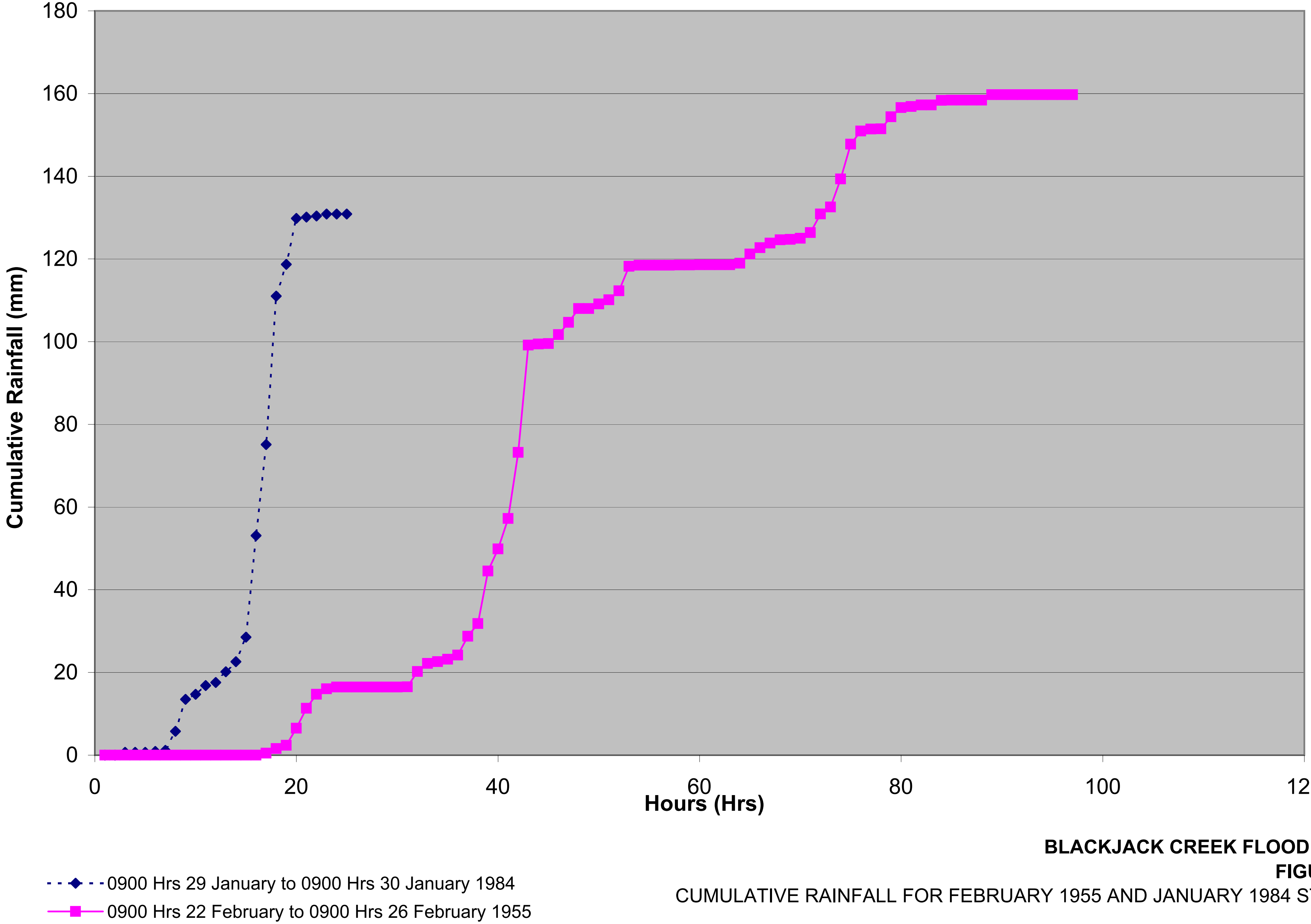
Figure 2.1
LOCATION PLAN



BLACKJACK CREEK FLOOD STUDY

FIGURE 2.2

CUMULATIVE RAINFALL FOR FEBRUARY 1955 AND JANUARY 1984 STORMS

3. HYDROLOGY

3.1 Selection of Hydrologic Model

For hydrologic modelling, the practical choice was between the catchment models known as RAFTS, RORB and WBNM, and any of these would have been suitable. Each of these models converts storm rainfall to discharge hydrographs using a procedure known as runoff-routing. There was little to choose technically between these models, however their usage in previous studies in the catchment, as well as the familiarity of the user with the model, were the determining factors in the selection of the RORB modelling approach.

3.2 Brief Review of RORB Modelling Approach

3.2.1 Model Layout

The catchment is divided into sub-areas bounded by drainage divides as shown on **Figure 3.1**. Rainfall on each sub-area is adjusted to allow for infiltration and other losses. The resulting sub-area rainfall-excess is assumed to enter the channel network at a point near the centroid of the sub-area. There, it is added to any existing flow in the channel, and the combined flow is routed through the sub-area storage by a storage routing procedure based on continuity and a storage discharge relationship (equation 3.1).

The overall catchment storage is represented in the model by a network of such storages arranged like the actual channel network. Each model storage represents the actual storage between two nodes of the model. The nodes represent sub-area inflow points, stream confluences, and other points of interest on the catchment or channel network.

3.2.2 Storage Discharge Relations

All storage elements within the catchment are represented via the storage-discharge equation:

$$S = kQ^{m} \qquad (3.1)$$

where	S	=	volume of storage.
	Q	=	discharge
	k	=	a storage delay parameter.
	m	=	a dimensional empirical coefficient

The factor m in equation 3.1 is a measure of the catchment's non-linearity. When m is set equal to unity the catchment's routing response is linear, that is, the ordinates of the discharge hydrograph increase directly in proportion to the ordinates of the hyetograph of rainfall excess. This is the same assumption used in unit hydrograph theory. A value of m less than unity implies that the peak discharge increases at a proportionally greater rate than the rainfall intensity.

In the absence of more catchment specific data, a value of 0.8 is commonly used for flood estimation.

The storage parameter "k" within the general storage equation is modified to reflect the catchment storage and the reach storage as follows:

$$k = kc.kr \qquad (3.2)$$

where kc = an empirical coefficient applicable to the entire catchment and stream network.

kr = a dimensionless ratio called the relative delay time, applicable to an individual reach storage.

3.2.3 Relative Delay Time

The relative delay time of a storage is calculated in the program as follows:

$$k_{ri} = F\frac{L_i}{d_{av}} \qquad (3.3)$$

where k_{ri} = relative delay time of storage i

Li = length of reach represented by storage i, (km)

d_{av} = average flow distance in channel network

F = A factor depending on the type of the reach (=1 for natural channels)

RORB has been used extensively throughout Australia on a wide range of rural and urban catchments. Calibrated values for kc and m for a large number of regions have been developed and have been used to estimate flows on ungauged catchments.

3.3 Estimation of Model Parameters

3.3.1 Coefficients of Storage Equation

The empirical coefficients kc, and m are the principal parameters of the model. In situations where historic rainfall and runoff data are available, the parameters may be derived in a process of model calibration. However, Blackjack Creek is ungauged and therefore parameters were assessed on the basis of published regional relationships (Pilgrim, 1987).

For the western region of New South Wales, (Pilgrim, 1987) recommends a relationship which was originally derived for flat to undulating areas in the Northern and Western Regions of South Australia. The relation is:

$$kc = CA^{0.57} \quad (3.4)$$

where A = catchment area in km^2

C = ranges between 1.2 and 1.7 for average stream slopes ranging between 1% and 0.2%.

As Blackjack Creek has an average slope approaching 1%, a value of 1.2 should be adopted for C. This results in a value of 7.5 for kc.

For the eastern region of New South Wales, a relationship based on data from 29 catchments east of the dividing range is:

$$kc = 1.22A^{0.46} \quad (3.5)$$

Pilgrim, 1987 states that equation 3.5 should also apply to catchments on the Tablelands and upper Western Slopes of New South Wales. This equation gives a value of 5.3 for kc.

A relationship (equation 3.6) was also derived from 86 catchments in Queensland. Most of the available data was for coastal catchments but values were included for streams west of the Great Dividing Range and near Mt Isa. No regional trends were evident. Equation 3.6 gives a value for kc of 4.8.

$$kc = 0.88A^{0.53} \quad (3.6)$$

All of the above relationships apply for a value of m equal to 0.8.

3.3.2 Rainfall Losses

Walsh et al, 1991 reported on the results of a study into the probabilistic derivation of losses, in particular initial losses, using streamflow data from 22 rural gauged catchments and design rainfalls from Australian Rainfall and Runoff, 1987. The design values of initial loss vary with the ARR 87 rainfall zone, flood frequency and the degree of non linearity assumed in the catchment flood hydrograph (RORB) model.

For rainfall Zone 11 west of the divide, recommended initial loss data are as follows:

TABLE 3.1
AVERAGE DESIGN VALUES OF INITIAL LOSS (mm)

ARI (years)	5	10	20	50	100
Non Linear Model (m = 0.8)	30	30	25	20	15

These values apply for a continuing loss rate of 2.5 mm/h.

3.4 Tuning the RORB Model

This section discusses the sensitivity of flows generated by the RORB model to variations in the RORB model parameters.

Estimates of peak flows for the January 1984 storm generated by varying model parameters are presented in Section 3.4.1.

For comparison purposes, Section 3.4.2 presents peak flows estimated from the Probabilistic Rational Method, which is in common usage for the derivation of flood flows on rural ungauged catchments in NSW.

After consideration of the results of the various approaches, a set of RORB model parameters was selected for the design flood estimation of Chapter 4 of the report. These parameters are presented on **Table 3.4**.

3.4.1 Estimates of January 1984 Flood Flows

Peak flows generated by RORB with the three estimates of kc derived from the formulas in the engineering literature are shown in **Table 3.2**. These flows assume zero rainfall losses in view of the prior saturation of the catchment due to rainfall over the days preceeding 30 January 1984.

TABLE 3.2
SENSITIVITY OF PEAK DISCHARGE
AT OXLEY HIGHWAY
TO RORB PARAMETER kc
JANUARY 1984 STORM

Source	kc	Discharge m^3/s
Lipp (Equation 3.4)	7.5	104
Kleemola (Equation 3.5)	5.3	124
Qld Data (Equation 3.6)	4.8	130

Hydraulic analysis at the Oxley Highway discussed in Chapter 5 showed that adoption of the 130 m^3/s estimate of flow derived using Equation 3.6 in the HEC-RAS modelling of the stream gave reasonable correspondence between the recorded and modelled flood levels on the upstream side of the bridge. Adoption of the flow derived from this equation in the hydraulic modelling also gave correspondence between modelled and observed flood extents in the residential area bordering Wandobah Road.

3.4.2 Estimates of Design Peak Flows from Probabilistic Rational Method

For comparison purposes, the Probabilistic Rational Method (PRM) was also used to provide an estimate of peak flows. This method is recommended for use in eastern New South Wales for rural catchments up to 250 km^2 in area (Pilgrim, 1987).

Steps involved in this method are:

i) Determine the critical rainfall duration as the time of concentration in hours from the equation:

$$tc = 0.76A^{0.38} \qquad (3.7)$$

where A = Catchment area (km^2)

ii) For this duration and the selected frequencies, determine the design rainfall intensities Iy (mm/h).

iii) Compute the runoff coefficient for an ARI of 10 years and adjust via the frequency factor FFy to determine the Y year runoff coefficient Cy.

iv) Compute the design flood magnitude Qy (m^3/s) from the formula:

$$Qy = 0.278 \times FFy \times Iy \times A \qquad (3.8)$$

Using this approach, estimates of Qy were derived for various flood frequencies were prepared. The results are shown on **Table 3.3**.

TABLE 3.3
PEAK DISCHARGES AT OXLEY HIGHWAY
ESTIMATED BY PRM
Values in m^3/s

5 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI
9.4	20	32	45

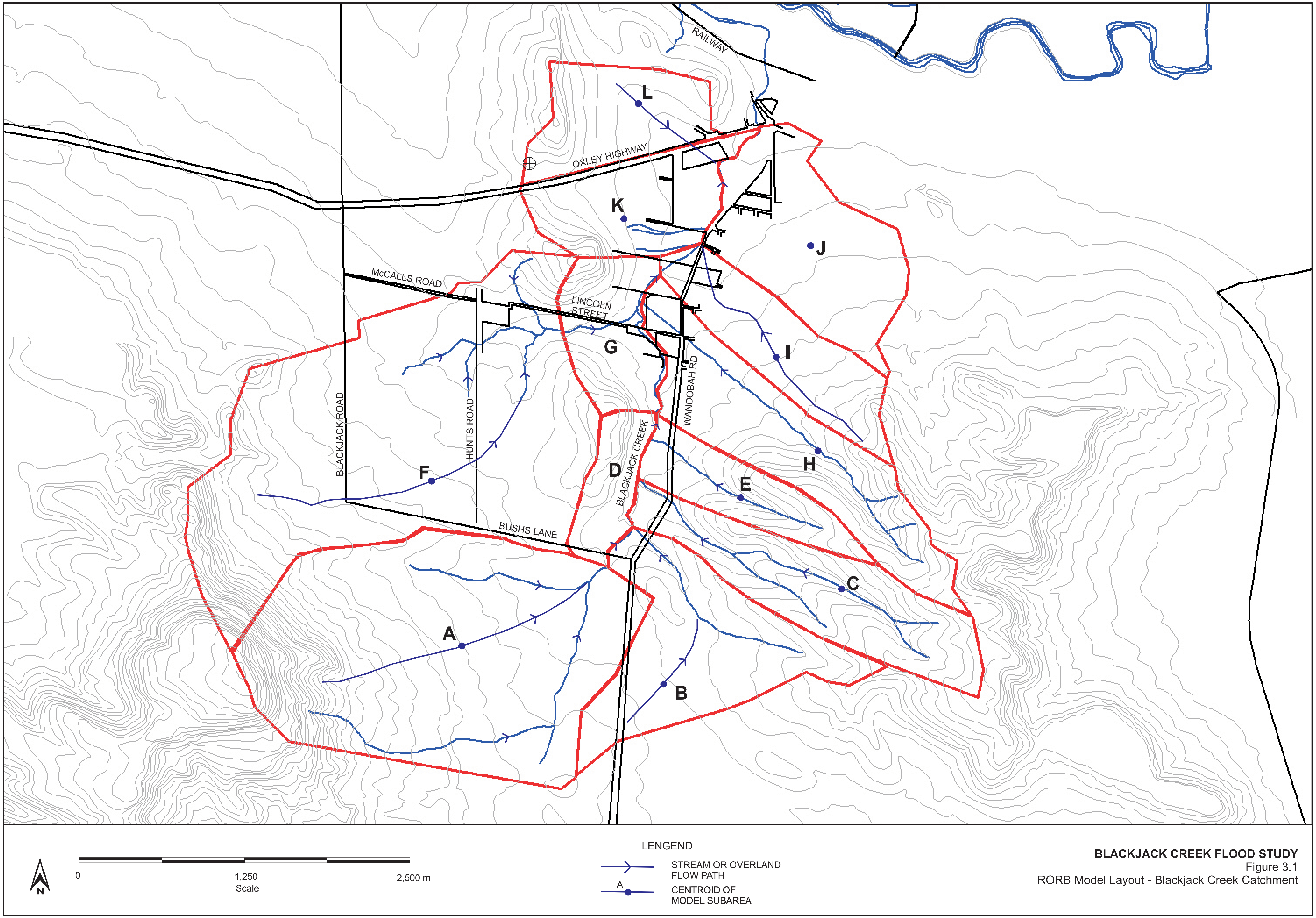
Peak flows estimated by the PRM are less than 50 per cent of flows derived using RORB or in the F R Kelly and Associates' report of November 1984 and appear to be on the low side for this catchment. The Kelly report assessed 1 in 100 year peak flow at the Oxley Highway as 90 m^3/s, after allowing for urbanisation on the eastern side of the catchment. The RORB derived flows are more in agreement with Kelly's results.

3.5 RORB Model Parameters Adopted for Design Flood Estimation

After consideration of the above analyses, the following set of model parameters has been used for design flood estimation. The kc value of 4.8 is based on the Queensland data results summarised as equation 3.6.

TABLE 3.4
DESIGN RORB MODEL PARAMETERS
BLACKJACK CREEK

Parameter	Recurrence Interval year ARI				
	5	20	50	100	PMF
Initial Loss	30	25	20	15	15
Continuing Loss	2.5	2.5	2.5	2.5	2.5
Kc	4.8	4.8	4.8	4.8	2.0
m	0.8	0.8	0.8	0.8	1.0



BLACKJACK CREEK FLOOD STUDY

Figure 3.1

RORB Model Layout - Blackjack Creek Catchment

4. DESIGN FLOOD ESTIMATION

4.1 Design Storms

4.1.1 Rainfall intensity

The procedures used to obtain temporally and spatially accurate and consistent intensity-frequency-duration (IFD) design rainfall curves for the Blackjack Creek catchment are presented in Chapter 2 of ARR (1987). Design storms for frequencies of 5, 20, 50 and 100 year ARI were derived for storm durations ranging between 1 hr and 6 hrs. The procedure adopted was to generate IFD data for each catchment by using the relevant charts in Volume 2 of ARR (1987). These charts included design rainfall isopleths, regional skewness and geographical factors.

4.1.2 Areal Reduction Factors

The rainfalls derived using the processes outlined in ARR (1987) are applicable strictly to a point. In the case of a large catchment of over tens of kilometres square it is not realistic to assume that the same rainfall intensity can be maintained over a large area, an areal reduction factor is typically applied to obtain an intensity that is applicable over the entire area.

However, as the area of the Blackjack Creek catchment is only 24 km^2, the reduction in rainfall intensities would be quite small and accordingly, no reduction in point rainfalls was made for this study.

4.1.3 Temporal Patterns

Temporal patterns for various zones in Australia are presented in ARR (1987). These patterns are used in the conversion of a design rainfall depth with a specific ARI into a design flood of the same frequency. Patterns of average variability are assumed to provide the desired conversion. The patterns may be used for ARIs up to 500 yrs where the design rainfall data is extrapolated to this ARI.

The derivation of temporal patterns for design storms is discussed in Chapter 3 of ARR (1987) and separate patterns are presented in Volume 2 for ARI < 30 years and ARI > 30 years. The second pattern is intended for use for rainfalls with ARIs up to 100 years, and to 500 years in those cases where the design rainfall data in Chapter 2 of ARR (1987) are extrapolated to this ARI.

4.2 Design Hydrographs

The RORB model was run with the above parameters to obtain design hydrographs for input to the hydraulic model. Peak flows at the model outlets for the critical storm duration which ranged between 3 and 6 hours depending on flood frequency are shown on **Table 4.1.**

TABLE 4.1
DESIGN PEAK DISCHARGES
BLACKJACK CREEK

Location	Peak Flow (m^3/s)				
	5 yr ARI	20 yr ARI	50 yr ARI	100 yr ARI	PMF
Main Arm at Lincoln Street	13	33	49	66	380
Tributary at Lincoln Street	9	22	34	45	255
Junction of Main Arm and Tributary d/s Lincoln Street	24	57	82	115	700
High Street	25	58	84	117	705
Short Street	27	64	93	126	770
Oxley Highway	28	65	95	127	770

Note : Flows apply for the critical storm durations of 3 and 6 hours, as appropriate.

4.3 Probable Maximum Flood

Estimates of probable maximum precipitation were made using the Generalised Short Duration Method (GSDM) as described in the Bureau of Meteorology's Bulletin 53 (BOM, 1994). This method is appropriate for estimating extreme rainfall depths for catchments up to 1000 km^2 in area and storm durations up to 6 hours.

The steps involved in assessing PMP for the Blackjack Creek catchment are briefly as follows:

- Calculate PMP for a given duration and catchment area using depth-duration-area envelope curves derived from the highest recorded US and Australian rainfalls.

- Adjust the PMP estimate according to the percentages of the catchment which are meteorologically rough and smooth, and also according to elevation adjustment and moisture adjustment factors.

- Assess the design spatial distribution of rainfall using the distribution for convective storms based on US and world data, but modified in the light of Australian experience.

- Derive storm hyetographs using the temporal distribution contained in Bulletin 53, which is based on pluviographic traces recorded in major Australian storms.

The design flows derived for events up to the 100 year ARI were based on the assumption that the catchment behaved in a non-linear manner. A value of 0.8 was adopted for the exponent m of the catchment's storage-discharge equation (Equation 3.1). While there is evidence of non-linear response (i.e. a value of m not equal to unity) over the range of observed floods in most natural catchments, it is unclear whether this effect persists to the PMF. At that magnitude of flooding, the routing response depends on the relative efficiency of the drainage system and the amount of storage on the catchment.

The V-shaped valleys of the headwaters of Blackjack Creek have comparatively small overbank areas and therefore a theoretical value of 0.75 - 0.8 for the exponent m of the storage versus discharge relationship used by RORB in the rainfall-runoff routing process for each sub-area of the model. This indicates that the headwaters of the creeks should continue to behave in a non-linear manner for extreme floods.

On the other hand, the storage in the lower floodplains may have the effect of increasing the value of m for extreme flood events. Also, the flow resistance in extreme floods may be increased by debris, erosive processes and increased turbulence and all of these influences may promote linear behaviour.

A sensitivity analysis of the PMF was undertaken with the RORB models for both catchments run in a linear manner. The coefficient kc in the storage versus discharge relationship was first adjusted to ensure that the magnitude of peak flow at the 100 year ARI level was unchanged when used with the new value of m equal to 1. RORB model parameters for the linear model are shown on **Table 3.4**.

Design storms were derived for durations ranging between 1 and 6 hours and applied to the model using the linear model parameters. One in 100 year rainfall losses were adopted for the PMF.

The 3 hour storm was found to be critical. Peak flows are shown in **Table 4.1** and are around six to seven times the magnitude of the 100 year ARI peaks. This multiple is generally in agreement with the results of other investigations on small catchments.

5. HYDRAULICS

5.1 Selection of Hydraulic Model

5.1.1 General

A model was required which could route flows through main streams and their tributaries, and produce time series of flows, velocities and water surface elevations at nominated locations. The model was to be capable of analysing hydraulic conditions at the culvert and bridge crossings of the streams, and capable of adjustment so that it could analyse the effects of possible modifications such as levees, channel enlargement, adjustments to bridge waterways or future land use changes on the floodplain, all of which could influence flooding behaviour.

Few commercially available hydrodynamic models contain all the features required for this present study. One however, HEC-RAS, has the required capabilities and is readily available to all potential model users at minimal cost.

5.1.2 Brief Review of HEC-RAS Modelling Approach

HEC-RAS is a one-dimensional hydraulic modelling package developed by the Hydrologic Engineering Centre of the US Army Corps of Engineers and has seen widespread application in Australia in recent years.

The momentum equation of open channel flow is solved numerically between user defined grid arrangements (more typically, cross section locations) for given boundary conditions. Typically, a peak discharge comprises the upstream boundary and the downstream boundary is either a rating curve (stage versus discharge relationship) or the assumption of uniform flow (friction slope equals the bed slope of the stream).

5.2 Blackjack Creek Model Layout

5.2.1 Model Structure

The model consisted of cross sections derived from ground survey. The choice of section locations depended on the need to accurately represent features on the floodplain which influence hydraulic behaviour (e.g. bridge constrictions, changes in channel and floodplain dimensions, weir controls) as well as supplying adequate flood information in existing urban areas. The locations of the cross sections are shown on **Figure 5.1** which shows the extent of inundation modelled for the January 1984 flood. Each cross section is denoted as a river station "RS" in the hydraulic model.

5.2.2 Boundary Conditions

Peak flows derived from RORB provided the boundary conditions at the upstream end of the model. The flow was increased along the modelled reach to account for runoff from the various sub-catchments.

A rectangular concrete drain joins the right bank of Blackjack Creek and conveys flows from a catchment to the east of Blackjack Creek. This drain is known as Ashfords Watercourse. Council advised that this drain is estimated to convey peak flows of 17 – 18 m^3/s during major flood events. The Ashfords Watercourse catchment is outside the study area for this present investigation.

However, to account for potential backwater effects, the flow was increased downstream of the Oxley Highway to allow for inflows from Ashfords Watercourse. The adopted contributions from the Ashfords Watercourse ranged from 9 m^3/s for the 5 year ARI flood, up to 20 m^3/s for the 100 year ARI.

5.3 Tuning Hydraulic Model of Blackjack Creek

5.3.1 General

The main physical parameter for HEC-RAS is hydraulic roughness. There are other parameters such as contraction and expansion head loss coefficients which are of a hydraulic nature but which do not greatly affect computed flood levels in relatively slow moving streams such as Blackjack Creek.

There are very limited historic flood level data available to assist with calibration of the model. Accordingly, roughness was estimated from site inspection, past experience and values contained in the engineering literature (Arcement and Schneider, 1984; Cowan, 1956; Barnes, 1967).

5.3.2 Roughness Values for Stream Channel

Although several factors affect the selection of an "n" value for the channel, the most important factors are the type and size of the materials that compose the bed and banks of the channel as well as its shape. Cowan, 1956 developed a procedure for estimating the effects of these factors.

In this procedure, the value of n may be computed by the following equation:

$$n = (nb + n1 + n2 + n3 + n4)\ m \quad \ldots\ldots\ldots \quad 5.1$$

where	nb	=	a base value of n for a straight, uniform, smooth channel in natural materials
	n1	=	a value added to correct for the effects of surface irregularities
	n2	=	a value for variations in shape and size of the channel cross section
	n3	=	a value for obstructions to flow
	n4	=	a value for vegetation and flow conditions
	and m	=	a correction factor for meandering of the channel

5.3.3 Roughness Values for Floodplain

It is usually necessary to determine roughness values for channels and floodplains separately. The fabric of a floodplain can be quite different from that of a channel. The physical shape of a floodplain is different and the vegetation covering a floodplain is typically different from that found in a channel.

Cowan's procedure was altered by Arcement and Schneider, 1984 to assess n values for a floodplain, using equation 5.1, where:

nb	=	a base value of n for the floodplain's natural bare soil surface, with no vegetation cover
n1	=	a value to correct for the effects of surface irregularities on the floodplain
n2	=	a value for variations in shape and size of the floodplain cross section
n3	=	a value for obstructions on the floodplain
n4	=	a value for vegetation on the floodplain
m	=	a correction factor for the sinuosity of the floodplain

Arcement and Schneider, 1984 also present photographs of densely vegetated floodplains for which roughness coefficients have been verified from historic flood data. These photographs were used together with application of equation 5.1 for estimating floodplain roughness.

TABLE 5.1
"BEST ESTIMATE" OF HYDRAULIC ROUGHNESS VALUES
Blackjack CREEK

Location	Channel	Floodplain
Downstream Oxley Hwy	0.05	0.06
Short St. to Oxley Hwy	0.05-0.06	0.06
High Street to Lincoln Street	0.07	0.06-0.07
Upstream Lincoln Street	0.07	0.06

5.4 Results for January 1984 Flood

The RORB model extends as far as the Oxley Highway, while the HEC-RAS model continues a further 450 m downstream, incorporating the Railway culvert and the downstream grassed floodway.

The RORB modelling of the January 1984 storm gave a peak discharge of 130 m^3/s at the Oxley Highway. The January 1984 flood was estimated to be an event somewhat larger than a 1 in 100 year event as far as peak discharges were concerned. Assuming that Ashfords Watercourse contributed a peak discharge of 20 m^3/s, the peak discharge in the modelled reach of Blackjack Creek downstream of Oxley Highway would be 150 m^3/s.

The extent of inundation and water surface profiles as modelled by HEC-RAS with the above flows are shown on **Figures 5.1 and 5.2**. Two sets of water surface profiles are shown on **Figure 5.2**: the best estimate of hydraulic roughness and also with hydraulic roughness values for the channel and floodplain increased by 30 per cent.

At the Oxley Highway, the recorded peak flood level was RL 267.7 m, which agrees well with the modelled value of RL 267.68 m for hydraulic roughness values increased by 30 per cent above the best estimate values. For the best estimate of roughness, the modelled level was 200 mm lower, at RL 267.48 m.

Further upstream at McAndrew Park (RS 8), the modelled peak water level was RL 276.6 m for the 30 per cent increased roughness, compared with RL 276.51 for the best estimate of roughness. There is no recorded flood level information available at this location but Council advised that flooding extended across the width of the park and that the downstream residential area north of High street was inundated. This information supports a recorded peak flood level of RL 276.6 m at RS 8.

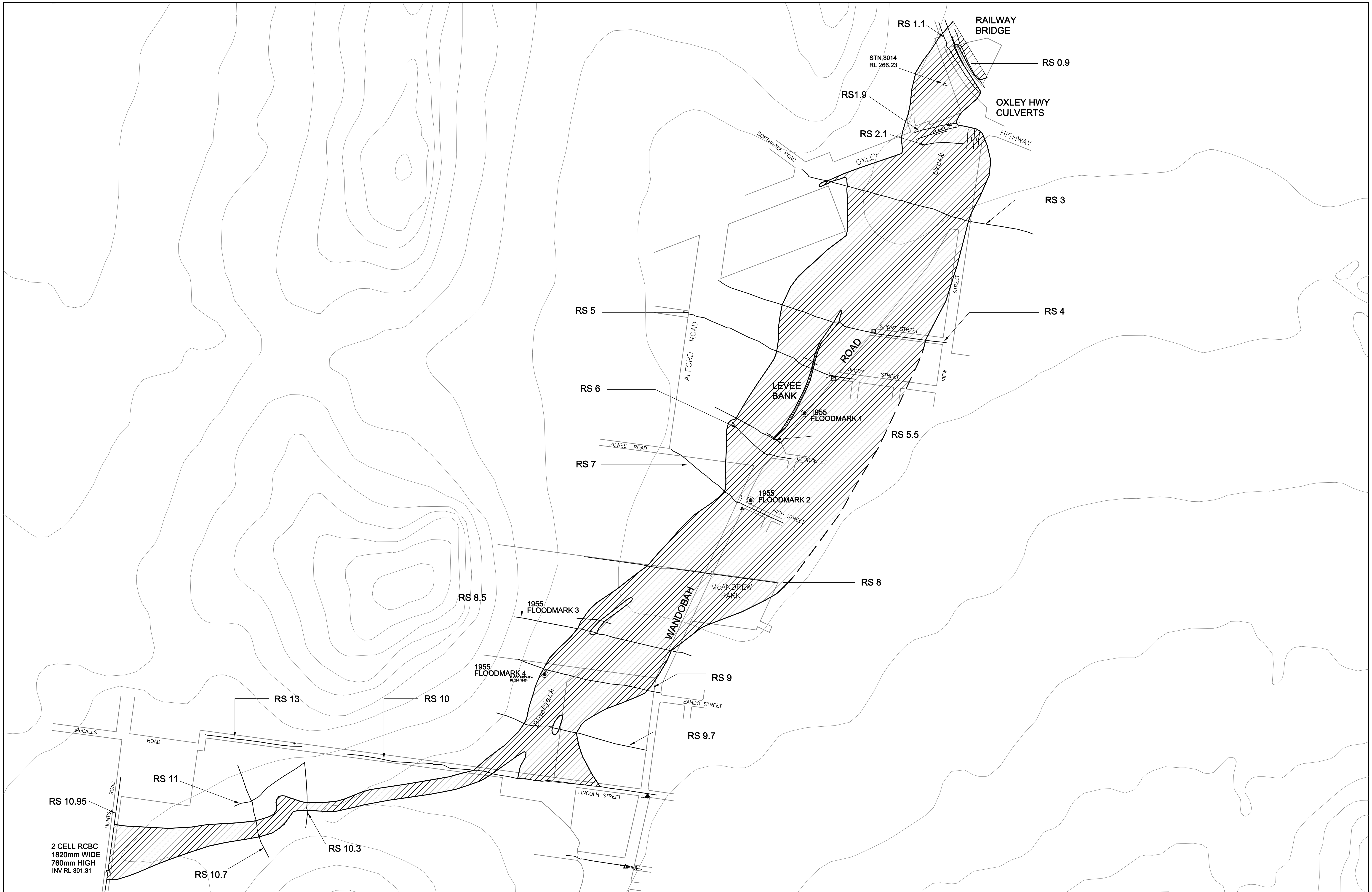
5.5 Results for February 1955 Flood

Hydrologic Modelling of the February 1955 storm using the rainfall intensities recorded at the Gunnedah SCS gave peak flows which increased from 66 m^3/s downstream of Lincoln Street, to 77 m^3/s at the Oxley Highway.

Figure 5.3 shows the peak water surface profiles along Blackjack Creek for both the best estimate roughness values and with roughness values increased by 30 per cent.

The roughness conditions and other topographic features on the floodplain existing at the time of the February 1955 flood are uncertain, as the event occurred 50 year ago. It is likely that the catchment was in an essentially rural state and that consequently, the flows generated by RORB which were estimated with 2005 urbanisation, may be on the high side compared with the flows which actually occurred in February 1955.

Comparison of the modelled water surface profiles with the recorded flood marks indicates a correspondence only with floodmark 2, which is located on the downstream side of High Street. Floodmark 1 between Kilcoy and George streets is about 0.5 m below the modelled profile and the two floodmarks 3 and 4 near the upstream end of the model are above the water surface profile computed in **Section 6** for the Probable Maximum Flood discharge and are not considered reliable.



SCALE 1:10000 @ A3

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N

RS 5 — Cross Section Location and HEC-RAS Model River Station Number

January 1984 Flood Extent

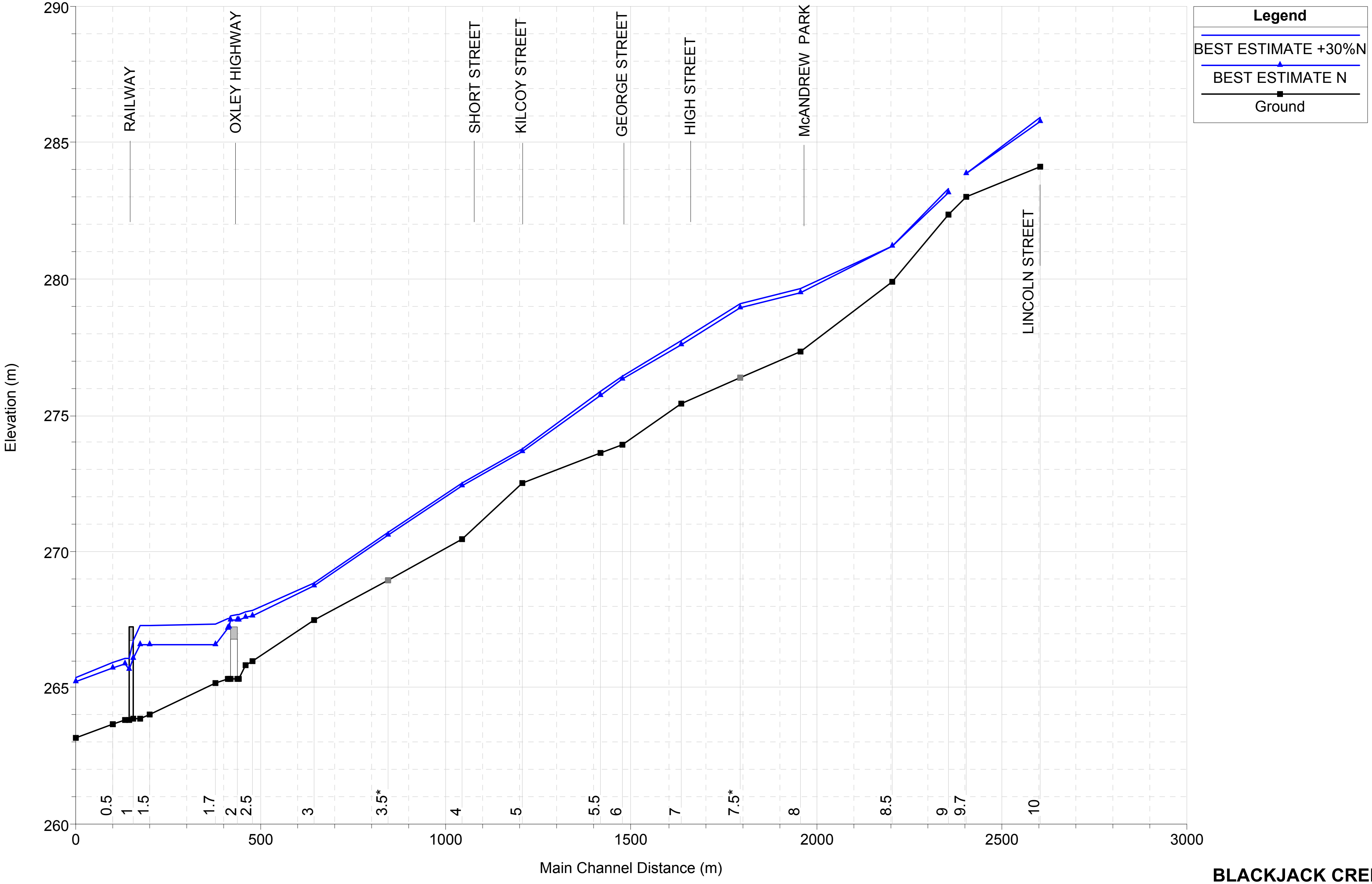
NOTE: CROSS SECTION LOCATIONS EXTRACTED FROM STEWART SURVEYS CONSULTING SURVEYORS (NOV/04).

BLACKJACK CREEK FLOOD STUDY

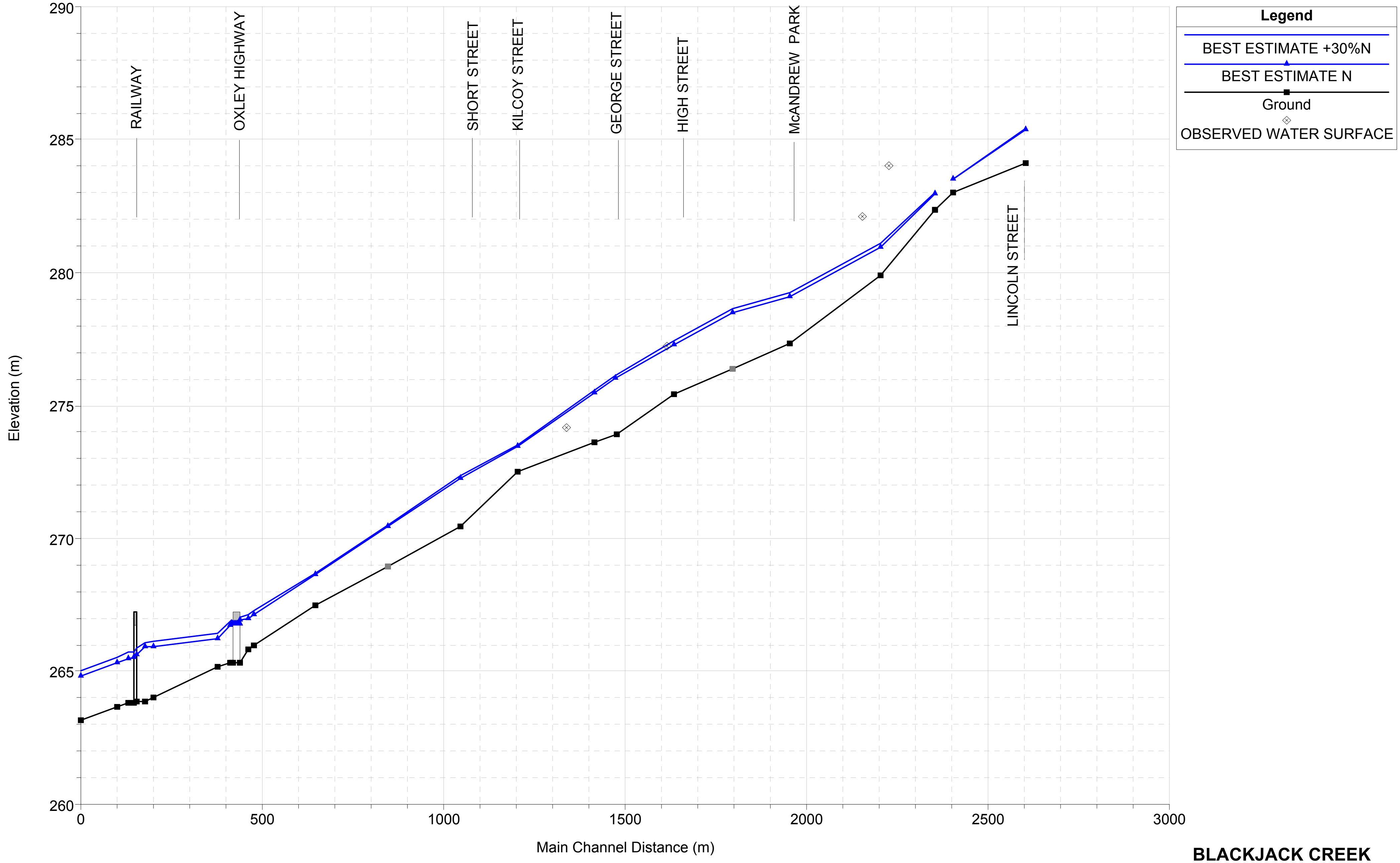
Figure 5.1

EXTENT OF FLOODING

JANUARY 1984 FLOOD



BLACKJACK CREEK
FLOOD STUDY
Figure 5.2
WATER SURFACE PROFILE
JANUARY 1984 FLOOD



BLACKJACK CREEK
FLOOD STUDY
Figure 5.3
WATER SURFACE PROFILE
FEBRUARY 1955 FLOOD

6. HYDRAULIC MODELLING OF DESIGN FLOODS

6.1 Introduction

This Chapter deals with the derivation of flood behaviour along Blackjack Creek using the HEC-RAS hydraulic model. The flows generated by RORB and presented in **Table 4.1** have been used, in conjunction with hydraulic roughness values increased by 30 per cent above the best estimate values shown in **Table 5.1** for design flood levels up to the 100 year ARI. For the PMF, the best estimate values of roughness have been adopted, as hydraulic roughness tends to reduce with increasing discharge.

6.2 Results of Hydraulic Modelling

6.2.1 Presentation of Results

Water surface profiles for the 5 to 100 year ARI and PMF design events are shown in **Figure 6.1**. **Figure 6.2** shows the indicative extents of inundation for the 5, and 100 year ARI floods, as well as the PMF.

Peak water surface elevations and the average flow and velocity distributions are tabulated in **Appendix A**.

Uncertainties associated with numerical hydraulic modelling are such that water levels are usually rounded off to the nearest 100 mm. However, in the present study water surface profiles along the steeper reaches of the creek do not show large differences in elevation, indicating that large increases in flow result in relatively small increases in water level. Consequently, the results have generally been presented to two decimal places (i.e. to the nearest 10 mm), to highlight differences in the model results for the various floods.

6.2.2 Discussion of Results

Design floods of up to 6 hour duration were critical for generating peak water levels within the study area for the modelled flood events, reducing to 3 hours for the PMF event.

Peak water levels do not vary greatly for the various design storm events. There is a difference in levels between the 5 year ARI level and the 100 year ARI level ranging between 890 and 400 mm, depending on the location along the stream. The PMF flood levels are between 0.8 and 1.5 m higher than the 100 year ARI levels.

The relatively small difference in flood levels between the 5 and 100 year ARI events is characteristic of flows in wide, shallow floodplains where the flow is slow moving, with flow velocities generally no greater than 1 m/s.

Flows up to the 5 year ARI magnitude would be conveyed within the channel and its immediate vicinity. The levee on the right bank between George Street and Kilcoy Street would have about 500 mm of freeboard against overtopping.

At the 20 year ARI, however, floodwaters extend over a width of floodplain up to 400 m downstream of High Street. Floodwaters break the right bank of the creek upstream of

McAndrew Park, with flows entering the residential area north of High street. The protective levee would be outflanked and Wandobah Road would act as a floodway. Further to the east, the residential area would mainly function as a storage area because of the blocking effect of buildings, inter-allotment fences and other obstructions on flows.

The bridge at Oxley Highway would convey flows up to the 20 year ARI. However, larger floods, in combination with backwater influences resulting from the entry of flows from Ashfords Watercourse, would result in overtopping of the roadway.

6.3 Sensitivity Studies

6.3.1 Variation in Hydraulic Roughness

Hydraulic roughness along the creeks was estimated from site inspection, past experience and values shown in Arcement and Schneider, 1984. The hydraulic model allows for variations in roughness across the waterway section by multiplying the base roughness by specified relative roughness factors which apply to the channel and floodplains.

Prior to adopting peak water levels for the design floods, model runs were undertaken to test the sensitivity of results to variations in hydraulic roughness.

The sensitivity of results to variations in hydraulic roughness confirmed that the 500 mm of freeboard on design flood levels which is commonly adopted for planning purposes would be appropriate for the Blackjack Creek floodplain.

6.4 Floodway and Flood Hazard Areas

6.4.1 Floodways

According to the Floodplain Management Manual (NSW Government, 2001), the floodplain may be subdivided into the following:

- Floodways;
- Flood storage; and
- Flood fringe

Floodways are those areas where a significant volume of water flows during floods and are often aligned with obvious natural channels. They are areas that, even if partially blocked, would cause a significant increase in flood level and/or a significant redistribution of flow, which may in turn adversely affect other areas. They are often, but not necessarily, areas with deeper flow, or areas where higher velocities occur.

Flood Storage areas are those parts of the floodplain that are important for the temporary storage of floodwaters during the passage of a flood. If the capacity of a flood storage area is substantially reduced by, for example, the construction of levees or by landfill, flood levels in nearby areas may rise and the peak discharge downstream may be increased. Substantial reduction of the capacity of a flood storage area can also cause a significant redistribution of flood flows.

Flood Fringe is the remaining area of land affected by flooding, after floodway and flood storage areas have been defined. Development in flood fringe areas would not have any significant effect on the pattern of flood flows and/or flood levels (NSW Government, 2001).

The notion of hydraulic categories is subjective, and to a large degree can reflect the opinion of the assessor, particularly with what is considered to be a significant impact

A procedure in common use for the definition of the floodway for a particular flood event is to adopt the extent of inundation of a lesser flood event as its floodway extent. For example, the 100 year ARI floodway is often defined as the extent of the 20 year ARI flood. The remaining flooded area between the extent of inundation of the 20 year ARI flood and the 100 year ARI event is often adopted as the flood storage and flood fringe areas.

This pragmatic categorisation of the hydraulic areas of the floodplain has considerable merit and is easily understood. It has provisionally been adopted for Blackjack Creek except on the eastern floodplain in the residential area between McAndrew Park and Short Street.

As mentioned above, the residential area in that zone mainly functions as a storage area. Consequently, the 100 year ARI floodway is assumed to continue across Wandobah Road (which is hydraulically efficient and conveys most of the overbank flow) and terminate at the boundary of the residential allotments on the eastern side of that road. The remaining area further to the east inundated at the 100 year ARI is assumed to be a storage area.

Figure B1 in **Appendix B** shows the 100 ARI floodway defined according to the above principles.

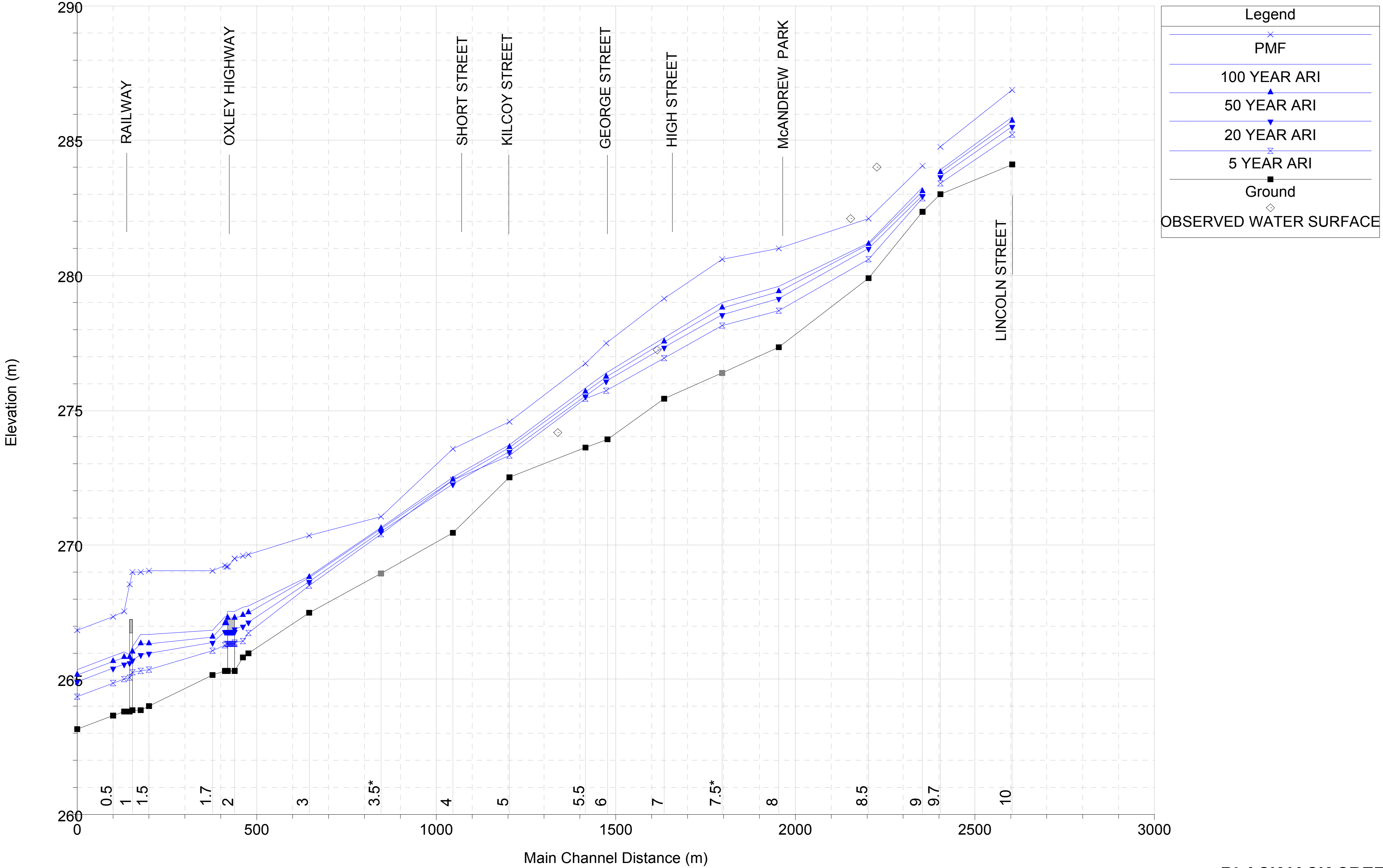
6.4.2 Flood Hazard

Flood hazard categories may be assigned to flood affected areas in accordance with the procedures outlined in the Floodplain Management Manual.

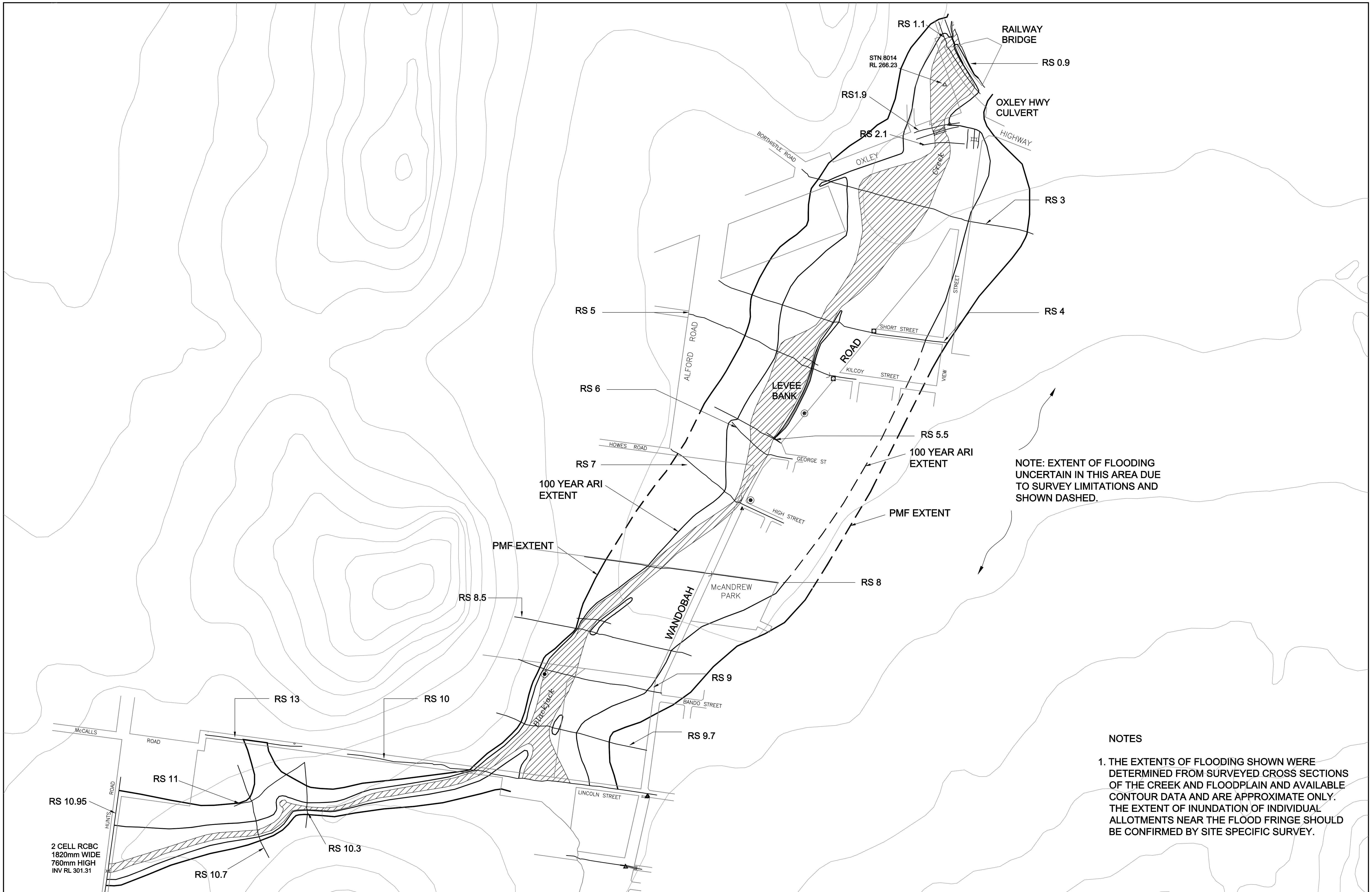
Flood prone areas may be provisionally categorised into *Low Hazard* and *High Hazard* areas depending on the depth of inundation and flow velocity. Flood depths as high as 0.8 m in the absence of any significant flow velocity represent Low Hazard conditions. Similarly, areas of flow velocities up to 2.0 m/s but with minimal flood depth also represent Low Hazard conditions.

The High Hazard zone for the 100 year ARI flood is shown on **Figure B2**. This zone includes the channel and the overbank areas where the flooding is deepest. Portion of the residential area on the eastern side of Wandobah Road is provisionally located in a High Hazard zone on the basis of depth of inundation only.

As noted in the Floodplain Management Manual, other considerations such as rate of rise of floodwaters and access to high ground for evacuation from the floodplain should also be taken into consideration before a final determination of Flood Hazard can be made. These factors are normally taken into account in the *Floodplain Risk Management Study* for the catchment, which is the next stage in the flood management process for the area.



BLACKJACK CREEK
FLOOD STUDY
Figure 6.1
WATER SURFACE PROFILE
5, 20, 50, 100 YEAR ARI AND PMF



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0 200 SCALE 500m

NOTE: CROSS SECTION LOCATIONS EXTRACTED FROM STEWART SURVEYS CONSULTING SURVEYORS (NOV/04).

RS 5 — Cross Section Location and HEC-RAS Model River Station Number

5 year ARI Flood Extent

100 year ARI Flood Extent

PMF Flood Extent

BLACKJACK CREEK FLOOD STUDY

Figure 6.2

EXTENTS OF FLOODING

5, 100 YEAR ARI AND PMF

7. REFERENCES

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APPENDIX A

FLOOD LEVEL, FLOW AND VELOCITY DISTRIBUTION TABULATIONS - DESIGN FLOODS

Reach	River Station	Peak Water Level	5 yr ARI					
			Flow (m^3/s)			Velocity (m/s)		
			Left	Channel	Right	Left	Channel	Right
Blackjack upper	**10**	285.22	0	8.95	4.45	0	0.41	0.21
Blackjack upper	**9.7**	283.41	0	13.4	0	0	1.3	0
Blackjack lower	**9**	282.88	0	3.39	20.6	0.07	0.28	0.29
Blackjack lower	**8.5**	280.62	0	24	0	0	1.41	0
Blackjack lower	**8**	278.68	0	24	0	0	0.67	0
Blackjack lower	**7**	276.93	0	24.5	0	0	1.47	0
Blackjack lower	**6**	275.71	4.58	19.72	0.2	0.36	0.84	0.2
Blackjack lower	**5.5**	275.41	1.23	23.27	0	0.25	0.8	0
Blackjack lower	**5**	273.32	19.36	5.14	0	0.8	1.03	0
Blackjack lower	**4**	272.42	14.91	12.09	0	0.29	0.42	0
Blackjack lower	**3**	268.48	10.16	12.59	4.24	0.37	0.72	0.38
Blackjack lower	**2.5**	266.74	0	27	0	0	1.05	0
Blackjack lower	**2.1**	266.42	0	27	0	0	1.42	0
Blackjack lower	**2.05**	266.37	0	27	0	0	0.74	0
Blackjack lower	**2**	Oxley Highway						
Blackjack lower	**1.95**	266.33	0	31	0	0	0.88	0
Blackjack lower	**1.9**	266.3	0	31	0	0	1.02	0
Blackjack lower	**1.7**	266.06	0	31	0	0	1.1	0
Blackjack lower	**1.5**	265.38	0	30.09	1.41	0	0.73	0.14
Blackjack lower	**1.1**	265.34	0	28.26	3.24	0	0.63	0.16
Blackjack lower	**1**	Railway Bridge						
Blackjack lower	**0.9**	265.03	0.79	30.71	0	0.3	1.04	0
Blackjack lower	**0.5**	264.87	0.85	30.65	0	0.31	1.03	0
Blackjack lower	**0**	264.35	0.76	30.74	0	0.3	1.05	0

Reach	River Station	Peak Water Level	20 yr ARI					
			Flow (m^3/s)			Velocity (m/s)		
			Left	Channel	Right	Left	Channel	Right
Blackjack upper	**10**	285.55	0.01	14.56	17.93	0.07	0.43	0.3
Blackjack upper	**9.7**	283.66	3.1	51.3	0	1.56	1.89	0
Blackjack lower	**9**	282.98	0.05	7.87	48.58	0.19	0.51	0.51
Blackjack lower	**8.5**	281.03	0	54.07	2.43	0	1.78	0.86
Blackjack lower	**8**	279.14	0	42.26	14.24	0	0.8	0.15
Blackjack lower	**7**	277.34	21.71	33.12	3.37	0.98	1.33	0.07
Blackjack lower	**6**	276.06	17.61	35.46	5.14	0.45	1.14	0.13
Blackjack lower	**5.5**	275.54	5.76	47.34	5.1	0.55	1.39	0.84
Blackjack lower	**5**	273.47	23.26	4.96	29.98	0.6	0.75	0.55
Blackjack lower	**4**	272.28	22.15	22.59	18.87	0.6	0.93	0.28
Blackjack lower	**3**	268.66	30.97	21.65	10.98	0.56	0.95	0.51
Blackjack lower	**2.5**	267.16	0.04	63.54	0.02	0.14	1.37	0.13
Blackjack lower	**2.1**	266.98	0.02	63.56	0.01	0.13	1.39	0.11
Blackjack lower	**2.05**	266.9	0	63.6	0	0	1.15	0
Blackjack lower	**2**	Oxley Highway						
Blackjack lower	**1.95**	266.81	0	72.6	0	0	1.39	0
Blackjack lower	**1.9**	266.77	0	72.6	0	0	1.46	0
Blackjack lower	**1.7**	266.36	0	72.6	0	0	1.8	0
Blackjack lower	**1.5**	265.97	0	46.05	27.55	0	0.69	0.33
Blackjack lower	**1.1**	265.95	0	43.5	30.1	0.03	0.6	0.3
Blackjack lower	**1**	Railway Bridge						
Blackjack lower	**0.9**	265.59	7.77	65.83	0	0.71	1.38	0
Blackjack lower	**0.5**	265.42	7.86	65.74	0	0.7	1.36	0
Blackjack lower	**0**	264.9	7.68	65.92	0	0.71	1.39	0

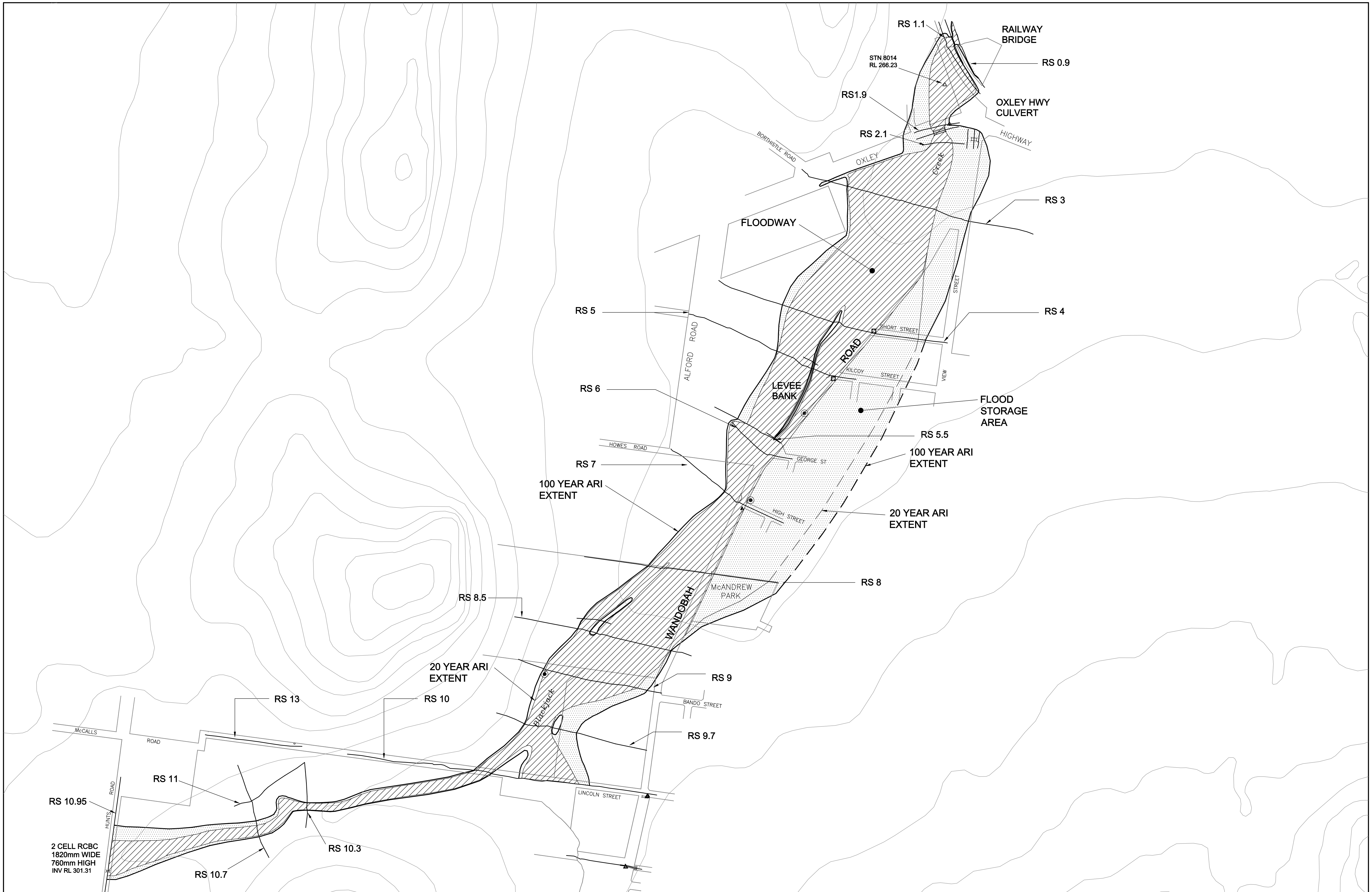
Reach	River Station	Peak Water Level	50 yr ARI					
			Flow (m^3/s)			Velocity (m/s)		
			Left	Channel	Right	Left	Channel	Right
Blackjack upper	**10**	285.72	0.12	19.66	28.93	0.13	0.49	0.35
Blackjack upper	**9.7**	283.8	5.53	76.29	1.17	1.76	2.05	0.58
Blackjack lower	**9**	283.13	0.23	11.63	73.14	0.26	0.56	0.56
Blackjack lower	**8.5**	281.16	0	79.4	5.6	0	2.26	1.19
Blackjack lower	**8**	279.37	0	50.22	34.78	0	0.81	0.21
Blackjack lower	**7**	277.52	35.18	44.4	7.42	1.2	1.5	0.1
Blackjack lower	**6**	276.24	34.92	43.2	8.89	0.59	1.23	0.16
Blackjack lower	**5.5**	275.68	13.43	63.75	9.82	0.71	1.6	1.08
Blackjack lower	**5**	273.6	35.82	6.69	44.49	0.69	0.82	0.62
Blackjack lower	**4**	272.4	33.72	27.96	32.32	0.68	0.99	0.33
Blackjack lower	**3**	268.77	50.27	27.72	16	0.68	1.06	0.5
Blackjack lower	**2.5**	267.48	2.77	89.24	1.99	0.42	1.41	0.35
Blackjack lower	**2.1**	267.39	3.53	87.75	2.72	0.43	1.31	0.35
Blackjack lower	**2.05**	267.29	0.01	93.99	0.01	0.05	1.36	0.06
Blackjack lower	**2**	Oxley Highway						
Blackjack lower	**1.95**	267.09	0	107	0	0	1.72	0
Blackjack lower	**1.9**	267.06	0.16	106.84	0	0.19	1.69	0
Blackjack lower	**1.7**	266.59	0	107	0	0	2.13	0
Blackjack lower	**1.5**	266.34	0.21	57.52	50.26	0.13	0.69	0.38
Blackjack lower	**1.1**	266.32	0.38	55.28	52.34	0.15	0.62	0.35
Blackjack lower	**1**	Railway Bridge						
Blackjack lower	**0.9**	265.85	13.14	86.81	8.04	0.85	1.52	0.35
Blackjack lower	**0.5**	265.69	13.17	86.29	8.54	0.84	1.5	0.35
Blackjack lower	**0**	265.17	13.13	87.11	7.76	0.86	1.53	0.35

Reach	River Station	Peak Water Level	100 yr ARI					
			Flow (m^3/s)			Velocity (m/s)		
			Left	Channel	Right	Left	Channel	Right
Blackjack upper	**10**	285.86	0.39	25.25	40.36	0.18	0.55	0.38
Blackjack upper	**9.7**	283.91	7.79	97.62	5.59	1.86	2.14	0.83
Blackjack lower	**9**	283.28	0.58	15.65	98.77	0.3	0.60	0.58
Blackjack lower	**8.5**	281.23	0	105.68	9.32	0	2.79	1.45
Blackjack lower	**8**	279.57	0	57.29	57.71	0	0.82	0.24
Blackjack lower	**7**	277.69	49.15	55.58	12.27	1.37	1.64	0.13
Blackjack lower	**6**	276.39	53.52	50.69	12.79	0.71	1.32	0.19
Blackjack lower	**5.5**	275.81	23.45	78.53	15.02	0.83	1.74	1.25
Blackjack lower	**5**	273.71	49.52	8.44	59.04	0.78	0.89	0.69
Blackjack lower	**4**	272.51	45.53	33.85	46.62	0.73	1.06	0.37
Blackjack lower	**3**	268.87	68.08	32.32	25.61	0.75	1.11	0.58
Blackjack lower	**2.5**	267.75	7.53	110.94	7.54	0.55	1.44	0.49
Blackjack lower	**2.1**	267.67	8.83	107.77	9.4	0.54	1.32	0.5
Blackjack lower	**2.05**	267.55	2.12	121.55	2.33	0.24	1.55	0.29
Blackjack lower	**2**	Oxley Highway						
Blackjack lower	**1.95**	267.34	0.1	143.79	0.11	0.14	2.02	0.15
Blackjack lower	**1.9**	267.33	3.56	140.22	0.21	0.37	1.87	0.24
Blackjack lower	**1.7**	266.85	0.11	143.89		0.23	2.34	
Blackjack lower	**1.5**	266.68	0.99	69.53	74.48	0.2	0.70	0.41
Blackjack lower	**1.1**	266.67	1.25	67.48	76.27	0.2	0.65	0.39
Blackjack lower	**1**	Railway Bridge						
Blackjack lower	**0.9**	266.05	17.96	105.32	21.72	0.94	1.64	0.49
Blackjack lower	**0.5**	265.89	17.96	104.64	22.4	0.93	1.62	0.49
Blackjack lower	**0**	265.37	17.95	105.79	21.25	0.95	1.66	0.49

Reach	River Station	Peak Water Level	PMF					
			Flow (m^3/s)			Velocity (m/s)		
			Left	Channel	Right	Left	Channel	Right
Blackjack upper	**10**	286.91	124.88	128.09	383.04	0.89	1.51	1.16
Blackjack upper	**9.7**	284.77	42.3	390.64	203.06	2.6	3.4	2.06
Blackjack lower	**9**	284.09	12.68	83.44	602.88	0.95	1.53	1.48
Blackjack lower	**8.5**	282.13		241.32	457.68		3.18	2.09
Blackjack lower	**8**	281.03	96.63	183.96	418.41	0.75	1.4	0.57
Blackjack lower	**7**	279.14	333.86	262.86	108.27	3.44	3.69	0.38
Blackjack lower	**6**	277.49	433.17	187.16	84.67	2.19	2.99	0.55
Blackjack lower	**5.5**	276.76	335.06	267.64	102.3	2.06	3.18	2.91
Blackjack lower	**5**	274.57	329.9	44.3	330.8	1.81	2.09	1.66
Blackjack lower	**4**	273.57	298.95	130.63	339.42	1.29	1.88	0.75
Blackjack lower	**3**	270.33	369.73	85.69	313.58	0.9	1.18	0.75
Blackjack lower	**2.5**	269.67	126.37	487.01	155.62	1.73	2.74	1.71
Blackjack lower	**2.1**	269.62	128.04	482.91	158.05	1.67	2.63	1.66
Blackjack lower	**2.05**	269.49	184.63	445.16	139.21	1.66	3.02	1.67
Blackjack lower	**2**	Oxley Highway						
Blackjack lower	**1.95**	269.19	195.1	533.78	148.12	2.05	3.9	2.06
Blackjack lower	**1.9**	269.23	201.44	549.57	125.99	1.94	3.38	1.77
Blackjack lower	**1.7**	269.05	201.48	549.49	126.03	1.94	3.38	1.77
Blackjack lower	**1.5**	269.03	36.74	299.14	541.12	0.54	1.46	1.03
Blackjack lower	**1.1**	269.01	40.09	295.7	541.21	0.54	1.41	1
Blackjack lower	**1**	Railway Bridge						
Blackjack lower	**0.9**	267.52	96.76	366.65	413.6	1.92	3.16	1.66
Blackjack lower	**0.5**	267.36	96.66	365.47	414.87	1.91	3.14	1.65
Blackjack lower	**0**	266.83	96.89	368.47	411.64	1.94	3.19	1.67

APPENDIX B

PRELIMINARY HYDRAULIC AND HAZARD CATEGORISATION 100 YEAR ARI.



N

0 200 500m
SCALE

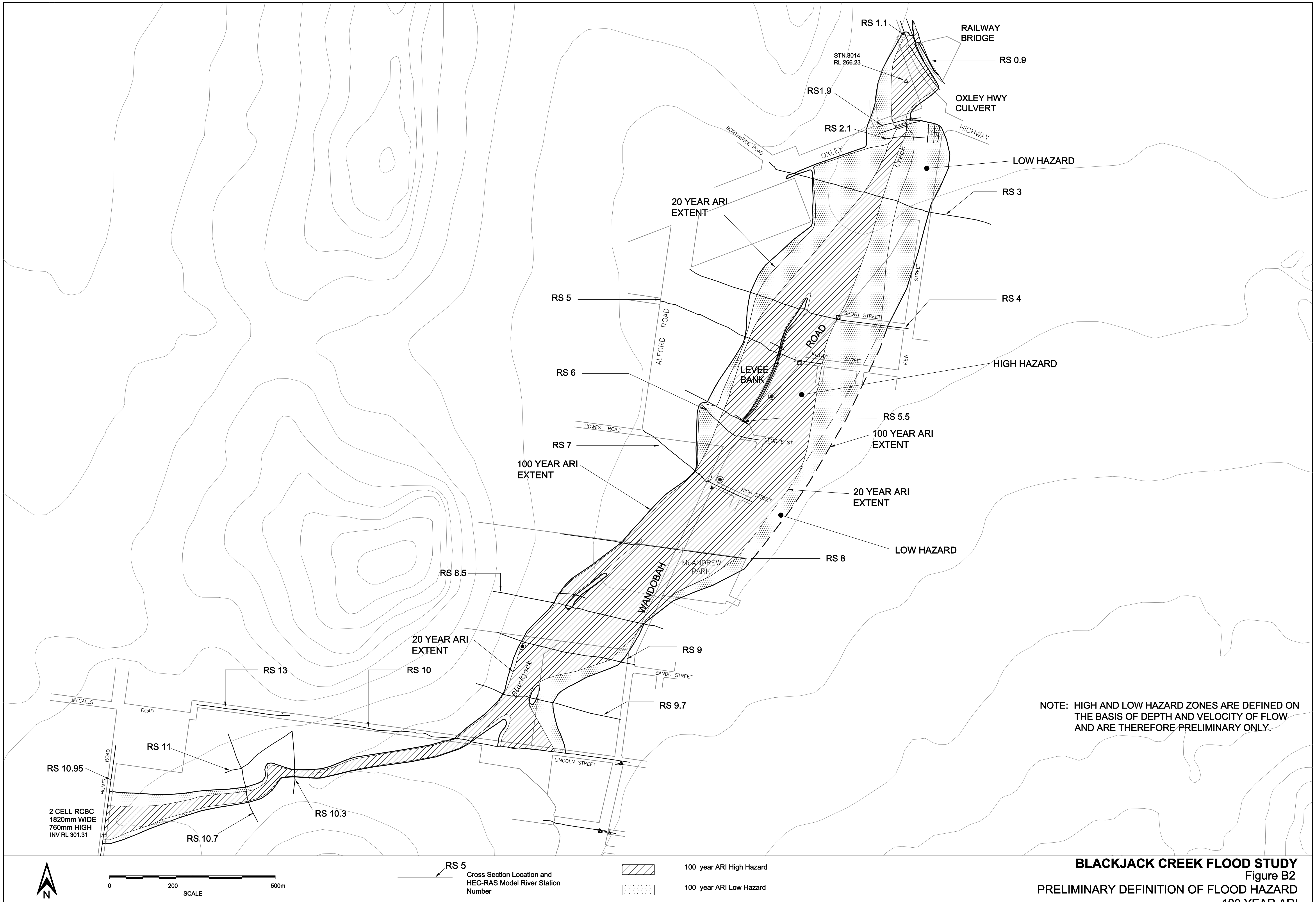
RS 5
Cross Section Location and HEC-RAS Model River Station Number

100 year ARI Floodway Extent

100 year ARI Flood Storage Area

BLACKJACK CREEK FLOOD STUDY
Figure B1
EXTENT OF 100 YEAR ARI FLOODWAY

NOTE: CROSS SECTION LOCATIONS EXTRACTED FROM STEWART SURVEYS CONSULTING SURVEYORS (NOV/04).



NOTE: CROSS SECTION LOCATIONS EXTRACTED FROM STEWART SURVEYS CONSULTING SURVEYORS (NOV/04).

BLACKJACK CREEK FLOOD STUDY
Figure B2
PRELIMINARY DEFINITION OF FLOOD HAZARD
100 YEAR ARI