MURRURUNDI, BLANDFORD AND WILLOW TREE

FLOOD STUDY

DRAFT REPORT

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FOREWORD

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 Management Study	-	evaluates management options for the floodplain in respect of both existing and proposed development.
3.	Floodplain 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 Murrurundi, Blandford and Willow Tree Flood Study constitutes the first stage of the process for this area and has been prepared for Murrurundi Shire Council to define flood behaviour under current conditions.

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ABBREVIATIONS

AEP	Annual Exceedance Probability (%)
AHD	Australian Height Datum
ARI	Average Recurrence Interval (years)
ARR	Australian Rainfall and Runoff, 1987 Edition
BOM	Bureau of Meteorology
DLWC	Department of Land and Water Conservation
DWR	Department of Water Resources, NSW (later part of DLWC)
PW	New South Wales Public Works (later part of DLWC)
SES	State Emergency Service
WRC	Water Resources Commission, NSW (later called DWR)

NOTE: Generally, references to organisations which have changed structure and name over the years are given under their current name (or abbreviation). However, references to documents use the name of the organisation that was current at the time of publication of the relevant document.

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GLOSSARY

annual exceedance probability (AEP)	the probability, or risk, (in percent) of a flood of a given size being exceeded in any given year. A 90% AEP flood has a high probability of being exceeded; it would occur quite often and would be relatively small. A 1% AEP flood has a low probability of being exceeded in any given year; it would be fairly rare but relatively large. Note however, that when the period of exposure to flood risk is longer than one year, the probability of experiencing a flood specified in terms of AEP is greater than the AEP.
average recurrence interval (ARI)	the average period (in years) between the occurrence or exceedance of a flood of given size. Approximately equal to the reciprocal of the AEP.
attenuation	the phenomenon in which a flood hydrograph becomes attenuated, or stretched out, as a flood passes down a river. The peak may become lower and the duration of the flood become longer.
Australian Height Datum (AHD)	a common national reference plane, relative to which survey heights are given. It is approximately at mean sea level.
calibration	the process by which a hydrologic or hydraulic model is adjusted so that it best represents the real world situation that the model is intended to simulate.
catchment	the area draining to a site. It always relates to a particular site, and may include the catchments of tributary streams as well as the main stream.
daily rain gauge	a rain gauge that is read on a daily basis, usually at 9.00 am.
dendritic	tree-like branching.
designated flood	(see flood standard)
development	the erection of a building or the carrying out of work on land; or the use of land or a building or a work; or the subdivision of land.
discharge	the rate of flow of water measured in terms of volume over time. It is to be distinguished from the velocity which is a measure of

	the speed of water rather than how much is moving.
discharge hydrograph	a graph which shows how the discharge changes over time at a particular location.
flood	relatively high stream flow when water overtops the natural or artificial banks of a stream and spreads over adjoining land.
flood hazard	potential for damage to property or persons due to flooding.
flood liable land	land which would be inundated as a result of a designated flood.
floodplain	the portion of a river valley, adjacent to the river channel, which is covered with water when the river floods. It includes the area inundated by all floods up to the probable maximum flood.
floodplain management strategy	a strategy embodying an appropriate selection of options for managing flooding and land use of a floodplain.
floodplain management options	the measures which might be feasible for the management of a floodplain area.
flood standard (or designated flood)	the flood of specified magnitude which is adopted for planning purposes. The selection should be based on an understanding of flood behaviour and the associated flood risk, and take account of social, economic and environmental considerations.
floodways	those areas where a significant volume of water flows during floods. They are often aligned with obvious naturally defined channels. Floodways are areas which, even if partially blocked, would cause a significant redistribution of flood flow, which may in turn adversely affect other areas. They are often, but not necessarily, the areas of deeper flow or the areas where higher velocities occur.
fluvial	of or found in rivers.
hydraulic conveyance	an engineering term combining the effects of a channel cross section area, shape, and boundary roughness on the ability of the channel to convey flow.
hydraulic roughness	a parameter which is used to mathematically express the effect

	on flow of the surface roughness of a channel or floodplain .
hydraulics	the term given to the study of water flow in pipes and channels. In relation to a flood study, it is particularly concerned with the evaluation of flow characteristics such as stage and velocity.
hydrograph	a graph which shows how either the discharge or the stage changes with time at a particular location.
hydrology	the term given to the study of the rainfall and runoff process.
hyetograph	a graph showing rainfall amounts versus time.
isohyet	a line drawn on a map showing all places that received the same amount of rainfall during a particular period. It is analogous to a contour line on a topographic map.
isopleth	the general term for a line drawn on a map showing all places having the same value for some parameter. An isohyet is a special case, but there are not special names used for all parameters.
management plan	a document including, as appropriate, both written and diagrammatic information describing how a particular area of land is to be used and managed to achieve defined objectives.
mathematical / computer models	the mathematical and logical representation of the physical processes involved in hydraulics and hydrology. Often run on computers because of the complexity of the mathematics or the amount of data to be processed.
peak discharge	the maximum discharge occurring during a flood event.
pluviograph	a graph showing the cumulative depth of rainfall versus time. The term is sometimes used as a synonym for pluviometer.
pluviometer	a special rain gauge which records the cumulative depth of rainfall versus time.
precipitation	the general term for the release of water from the atmosphere. It can be in the form of rain, hail, sleet, snow, dew and frost.
probable maximum flood (PMF)	the maximum value of flood that could reasonably be expected to occur at a particular location.

probable maximum precipitation (PMP)	the greatest depth of precipitation considered to be possible from consideration of meteorological processes, applying to a particular location, time of year, storm duration and size of storm area.
probability	a statistical measure of the likelihood, or expected frequency of occurrence, of an event.
rating curve	the relationship between stage and discharge at a particular location on the river valley (for example, at a stream gauging station).
runoff	the amount of precipitation which ends up as streamflow; also known as rainfall excess since it is the amount remaining after accounting for other processes such as evaporation and infiltration.
stage	equivalent to 'water level'. Both are measured with reference to a specified datum.
stage hydrograph	a graph which shows how water level changes with time at a particular location.
stream gauging station	a place on a river or stream at which the stage is routinely measured, either daily or continuously, and where the discharge is measured from time to time so as to develop a relationship between stage and discharge. The relationship is often presented as a table, called a rating table, or as a graph, called a rating curve.
sub-catchment	a subdivision of a catchment. It has all the features of a catchment defined above.
topography	the detailed representation on a map of the natural and artificial features of an area.

EXECUTIVE SUMMARY

The streams investigated in this study included the 12.5 km reach of the Pages River from Murrurundi to Blandford, as well as the 4.5 km section of Borambil Creek in the vicinity of Willow Tree.

The Pages River has its source near the eastern end of the Liverpool Range, from where it flows in a south-easterly direction to join the Hunter River in its headwater region several kilometres upstream of Aberdeen. Borambil Creek rises on the northern side of the Liverpool Range in the Namoi River catchment and flows in a northern direction parallel with the New England Highway. A short distance upstream of Willow Tree it is joined by Chilcotts Creek which drains the foothills of the range to the east. (Figure 1.1)

The study area has been subject to nine significant floods over the past 40 years since the record flood of October 1949. Murrurundi is the most flood prone of the three townships with residential development on both sides of the Pages River vulnerable to flooding. On two recent occasions, in January and October 1996, flow surcharged the right bank and flowed down the New England Highway (Mayne Street). Blandford and Willow Tree are flood prone but to a lesser degree than Murrurundi.

Flood behaviour in the two streams has been modelled 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). A preliminary assessment of flood hazard has also been made using velocity and depth results obtained from the investigation.

Flood behaviour was defined using computer based hydrologic models of the study catchments and hydraulic models of the streams. The hydrologic models were based on the RORB runoff routing program. The Pages River model was calibrated to recorded rainfall and streamflow data. The records at the Blandford stream gauging station on the Pages River and pluviographic data at Scone, Murrurundi, Blandford and Gowrie North were used for this purpose. Four floods were used for model calibration. In the order of investigation they were: January 1996, February 1992, January 1976 and October 1996.

The January 1996 recorded flood peak of 1030 m^3/s is close to the 50 year ARI in terms of peak discharge at Blandford on the basis of the RORB model results. Further upstream at Murrurundi the assessed flood peak was 450 m^3/s, which is near the modelled 100 year ARI peak discharge. The February 1992 and January 1976 floods were somewhat smaller, with recorded peak flows at Blandford of 870 m^3/s and 780 m^3/s respectively. They had an assessed frequency around the 20 year ARI magnitude. In October 1996, rainfall was most intense in the Pages River catchment above Murrurundi, with lesser falls in downstream areas. At Murrurundi, the peak discharge of 380 m^3/s approximated a 50 year ARI event, but at Blandford township just below the junction with Warlands Creek, the peak discharge of 610 m^3/s was less than the design 20 year ARI discharge. For all of these events, there was a consistent set of RORB model parameters which gave reproduction of the recorded hydrographs.

There are no stream gauging stations on Borambil Creek. A formal calibration of the RORB model was not therefore possible. Based on limited flood level data for the recent flood of January 1996, an estimate of the peak discharge was made. This discharge was used to tune the parameters of the RORB model. The October 1996 storm did not produce significant flood flows on this catchment.

The results of testing and calibrating the RORB models of the two streams are set out in Chapter 3 of the report.

A fully dynamic network hydraulic model was adopted for the hydraulic analysis to account for the time varying effects of flows from the tributary streams and the routing effects of the floodplain storage. A one-dimensional link-node model, MIKE 11, was chosen which allowed for the interaction of flows between the channel and the floodplain, flow through culverts and flow over road embankments. Models were set up for both the Pages River and Borambil Creek. The Pages River model extended from upstream of Murrurundi to a point about 1.8 km downstream of Blandford and included Halls Creek, Unnamed Gully and Cohens Gully which join the Pages River in the township, as well as Warlands Creek which joins just downstream of Blandford. The Borambil Creek model commenced upstream of the Hams Bridge at Merriwa Road and extended to a point downstream of the sporting fields. The two models were calibrated and tested using recorded streamflow and flood level data, as available and the results are discussed in Chapter 4.

Design storms were then applied to the RORB models to generate discharge hydrographs within the study area as described in Chapter 5. These hydrographs constituted the inputs to the hydraulic model for the assessment of design flood behaviour.

The hydraulic model was then used to produce water surface profiles, flood contours and flow and velocity distribution for the design events. The results are described in Chapter 6. Water surface profiles along the Pages River and Warlands Creek are shown on Figures 6.1, 6.1a and 6.1b. Flood contours are presented on Figures 6.2 to 6.2b for Murrurundi and on Figures 6.3 to 6.3b for Blandford. Tabulated peak flood levels and the distribution of flows and velocities for each model cross section are shown in Appendix A.

Water surface profiles on the main arm of Borambil Creek are shown on Figure 6.4 and flood contours on the floodplain are presented on Figures 6.5 to 6.5b.

Preliminary delineation of the floodplain into high and low hazard areas are shown on Figures 6.6 to 6.7b for Murrurundi and Blandford, and on Figures 6.8 to 6.8b for Willow Tree. These results were prepared using the velocity - depth criteria set out in the draft Floodplain Management Manual (DLWC, 1995). Hydraulic modelling was also undertaken to allow preliminary hydraulic categorisation of the floodplain into floodway and flood storage areas.

The extent of flooding and the flood hazard delineation shown on the diagrams are approximate only, particularly in the case of Blandford and Willow Tree. The best available contour mapping at these two centres is at 1:25000 scale with 10 m contour spacing. Accurate delineation of these lines would require more detailed survey. For Willow Tree any additional survey should include the establishment of benchmarks to AHD. The cross sectional survey carried out for this flood study adopted a local datum, as there are no reliable AHD survey marks in the township. The local datum is about 100 m above AHD.

At Murrurundi, the survey situation is somewhat better as there is mapping at 1:1000 scale with 1 m contours. However, a flood fringe survey will need to be undertaken to confirm the extent of flooding. This work could be carried out following the Floodplain Management Study and after the Designated Flood Event has been set.

Murrurundi is surrounded on its northern and southern sides by steeply rising hillsides which are drained by several gullies which have contributed to local flooding problems. Two local gullies, Unnamed Gully and Cohens Gully drain the northern side. Halls Creek drains the southern foothills. Flooding in these watercourses as well as overland flooding in the vicinity of Hall Street in Willow Tree are discussed in Chapter 7 of the report.

The models developed in this flood study may be used in the Floodplain Management Study to evaluate potential floodplain management strategies and may also be used by Council to evaluate the effects of development proposals.

1. INTRODUCTION

1.1 Study Area

This study deals with flooding in the townships of Murrurundi, Blandford and Willow Tree, which are situated in the foothills of the Liverpool Range north of the town of Scone (Figure 1.1). The Liverpool Range, a part of the Great Dividing Range with elevations ranging between 600 and 1200 m, forms the northern boundary of the Hunter River catchment and is the watershed between the coastal Hunter River system and inland Namoi system. The range comprises a line of hills formed of basalt which has flowed over the sandstone underlying the Merriwa Plateau and includes volcanic remnants in the form of prominent nobs and peaks.

The Pages River has its source near the eastern end of the range, from where it flows in a south-east direction to join the Hunter River in its headwater region. The Pages River rises near the town of Murrurundi and, with its eastern tributary the Isis River, flows through hills and undulating grassed valleys to join the Hunter River several kilometres upstream of Aberdeen.

Borambil Creek rises on the northern side of the Liverpool Range in the Namoi River catchment and flows in a northerly direction parallel with the New England Highway. A short distance upstream of the township of Willow Tree it is joined by Chilcotts Creek which drains the foothills of the range to the east. Borambil Creek continues past Willow Tree towards Quirindi.

The study area has been subject to nine significant floods over the past 40 years since the record flood of October 1949. Murrurundi is the most flood prone township with residential development on both sides of the Pages River vulnerable to inundation. The most severe flood occurred in January 1996 when one person was drowned on Warlands Creek and properties in Murrurundi and Blandford were inundated. Local runoff from foothills bordering the Pages River at Murrurundi also constitutes a drainage problem.

High flows were also experienced in January 1996 on Borambil Creek. However, damage on this stream was confined to fencing on the floodplain and surcharging of culverts on local tributaries.

Floods are more prevalent in the warmer months, with the highest frequency of occurrence in the January - March period, while spring and early summer have been relatively flood free. Generally major floods in the Hunter Valley have resulted from the occurrence of well developed tropical cyclones with at least several consecutive days of rainfall (W.C.I.C., 1969). Such cyclonic rainfall may arise from ex-tropical systems which originate in the Coral Sea and move south along the coast. Alternatively, deep cyclonic depressions may form over inland tropical Australia which move in a south-easterly direction. On rare occasions, these depressions may penetrate as far south as the upper Hunter River valley, as occurred in February 1955. Local convective storm action is often associated with these events, during which several hours of intense rainfall may be experienced. From inspection of the data, this appears to be the case especially in the higher areas of the upper Hunter catchment. The summer floods of 1955, 1976, 1984, 1991, 1992, and January 1996 in the study area were all typical of this flood producing mechanism.

High intensity, short duration convective thunderstorms may also occur, bringing intense rain for short periods over limited areas. Thunderstorm activity is largely confined to the late spring, summer and early autumn months. The October 1996 storm was typical of this flood producing mechanism.

East coast low-pressure systems are prevalent in the autumn and winter months and produce heavy rain over the lower Hunter Valley. However, these systems tend not to penetrate far inland and rainfalls reduce sharply away from the coast. This mechanism does not appear to have been responsible for flooding in the present study area.

Because of their steep bed slopes, the streams in the study area rise very quickly and flooding is of a "flash flooding" nature. At Murrurundi, the time of rise of the January 1996 flood was approximately four hours and the event lasted about eight hours. A similar situation was experienced on Borambil Creek at Willow Tree.

1.2 Scope of Study

The study objective was to define flood behaviour 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 computer based hydrologic models of the study catchments and hydraulic models of the streams. The hydrologic models used a runoff routing approach to convert storm rainfall to discharge hydrographs within the study area. These hydrographs constituted the boundary inputs to the hydraulic model.

A fully dynamic network hydraulic modelling approach was adopted for the hydraulic analysis to account for the time varying effects of flows from the tributary streams and the routing effects of the floodplain storage. A one-dimensional link-node 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, natural river levees and saddles. This model was used to produce water surface profiles, discharge hydrographs and average velocities of flow for the various floods. From these results, the provisional flood hazard was also determined.

The models are capable of modification in later studies to evaluate future floodplain management strategies and may also be used by Council to evaluate the effects of development proposals as they arise.

1.3 Study Tasks

The flood study had four main strands:

1. **Collection and review of available hydrologic and hydraulic data and previous investigations**. Rainfall and streamflow data were supplied by various organisations including DLWC, Bureau of Meteorology (BOM), and various landowners. This information was used in the calibration and testing of the hydrologic models. Two previous reconnaissance investigations commissioned by Murrurundi Shire Council (Bush 1991, 1996) had identified flooding patterns in the three townships. In these two studies, interviews were held with flood affected residents and flood marks and flow paths identified on plans of the study area. The October 1996 flood occurred during the course of the flood study and was a significant event in the Murrurundi area. Data were collected for this flood and used to confirm model results.

2. **Collection of survey data**. Following a site inspection and review of the data, a brief was prepared for the survey of cross sections of the floodplain of the main streams, the tributaries and bridge waterways. Recorded flood marks and the floor levels of potentially flood prone properties as identified in Bush's investigations, were also levelled.

3. **A hydrologic component** which included preparation and testing of the hydrologic models, estimation of design storms and their application to the models to generate design flood hydrographs. The models were calibrated where possible using recorded flood data. There were some streamflow records and temporal rainfall data on the Pages River catchment. However, there were no equivalent data available on Borambil Creek.

4. **A hydraulic component** which comprised the preparation and testing of the hydraulic models and the definition of the water surface profiles, flows and velocities for the design floods. From site inspection as well as review of the topographic data, it is evident that the floodplains are relatively confined by comparison with many other rivers in NSW where it is difficult to assess the flow pattern for major flood events due to extensive breakouts from the channel into off river storage areas and flood runners. In the case of the Pages River and Borambil Creek, the flow remains essentially one-dimensional over the full range of frequencies investigated.

1.4 Overview of Report

Section 2 contains background information including a description of the catchments, a review of the data base available for the study and a discussion on the history of flooding in the study area, leading to the selection of floods for calibration and testing of the hydrologic and hydraulic models.

Section 3 deals with the hydrology of the catchment. Following a review of the available models, the RORB runoff-routing program was adopted. Models of the study catchments were developed, calibrated and tested for four historic storms.

Section 4 deals with the development of the hydraulic models. The MIKE 11 unsteady flow computer program was used for this purpose. The models were calibrated on the basis of recorded discharge and flood level data for the January and October 1996 floods.

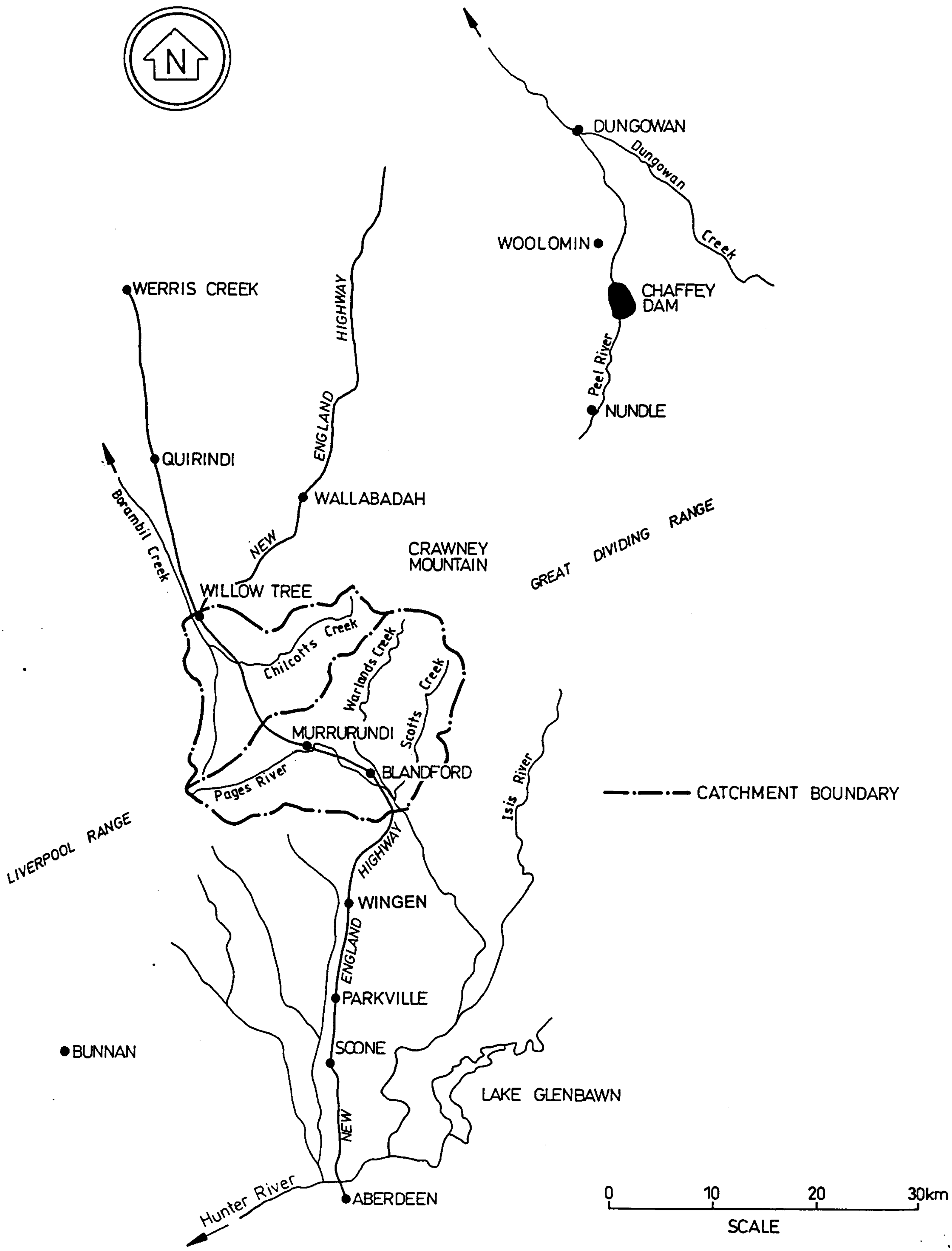
Section 5 describes the computation of design flows using the RORB hydrologic models. This step involved the determination of design storm rainfall depths over the catchment for a range of storm durations, estimation of temporal patterns and conversion of the resulting hyetographs to discharge hydrographs.

Section 6 details the results of the hydraulic modelling of the design floods using MIKE 11. Results are presented as tabulations of peak levels, water surface profiles and plans showing flood contours and flow velocities for the design flood events. Provisional flood hazard estimation was also carried out using these results.

Section 7 deals with flooding in the gullies draining the foothills of the Pages River catchment at Murrurundi and an overland flow sub-catchment of Borambil Creek at Willow Tree.

Section 8 contains a list of references.

Supplementary details are given in the Appendices. **Appendix A** gives details of the flood levels, flow and velocity distribution for the design floods. **Appendix B** describes the pattern of flooding experienced in the three townships in January 1996. Other relevant data including the floor and flood level data collected for the study, have been separately provided to Council. **Appendix C** lists peak flows recorded at the Blandford stream gauging station.



MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 1.1
Location Plan

2. BACKGROUND

2.1 Catchment Description

2.1.1 Pages River

The Pages River (Figure 2.1) rises near Mount Gregson in the Liverpool Range at an elevation of 1176 m and flows in a generally north-easterly direction over a distance of 12 km to Murrurundi, which is at an elevation of 460 m. The catchment area at Murrurundi is 72 km^2 and includes the contributions from two streams, Single Creek and Boyds Creek, which join the right bank of the Pages River several kilometres upstream of the township.

Murrurundi is surrounded on its northern and southern sides by steeply rising hillsides which are drained by several gullies which have contributed to local flooding problems. Two local gullies, Unnamed Gully and Cohens Gully drain the foothills on the northern side and flow through the residential portion of the town before crossing the New England Highway and joining the Pages River (Figure 2.2).

Halls Creek drains the southern foothills and after crossing the Great Northern Railway and Haydon Street joins the right bank of the river just upstream of Mayne Street. Victoria Street gully joins the right bank of Halls Creek on the southern side of the railway. Until recently, flood runoff from this drain crossed the railway embankment and contributed to flooding problems in the residential area on the southern side of Mayne Street between Adelaide and Victoria Streets. A levee was constructed along the northern bank to retain flow in the drain. Drainage from these local gullies is discussed in more detail in Section 7 of the report.

Mayne Street crosses the Pages River at Arnolds Bridge and from this point, flows are conveyed in an easterly direction north of the town for a distance of 5 km to the New England Highway crossing at Benhams Bridge. A short distance downstream of Murrurundi, Campbells Creek, another tributary draining the southern foothills, joins the Pages River. The river continues in a generally easterly direction between the New England Highway and the railway and is joined by Murulla Creek about a kilometre upstream of the Blandford Bridge (Figure 2.3). At Blandford, the Pages River is joined by Warlands Creek which flows in a southerly direction from Wallabadah Rocks (Figure 2.1). This stream drains heavily dissected country to the north of Blandford, falling from an elevation of 960 m to 410 m over a distance of 20 km to the junction. Warlands Creek has a catchment area of 103 km^2.

Downstream of Blandford, the Pages River continues for a further 4 km on the northern side of the New England Highway to the stream gauging station located near the "Manaree" homestead. Just upstream of the gauging station, Scotts Creek joins the left bank. The total gauged catchment amounts to around 300 km^2.

2.1.2 Borambil Creek

Borambil Creek (Figure 2.4) rises on the northern slope of the Liverpool Range and falls from an elevation of 1134 m to 420 m over a distance of 16 km to the junction with Chilcotts Creek. Chilcotts Creek drains the eastern portion of the catchment, commencing near Loders Pinnacle at an elevation of over 1000 m and flowing over a distance of 15 km to cross the New England Highway and Great Northern Railway. From this point, it swings northwards and flows parallel with Borambil Creek before joining that stream about a kilometre upstream of the Merriwa Road. At the junction with Chilcotts Creek, the total catchment area is 163 km^2 of which Borambil Creek contributes 49 km^2.

The Merriwa Road crosses Borambil Creek at the Hams Bridge and from this point flows are conveyed in a north-westerly direction in a more open floodplain past the Willow Tree township (Figure 2.5). Two local tributaries join the right bank of Borambil Creek below Hams Bridge and at the downstream end of the township the total catchment area comprises 182 km^2.

2.2 Hydrologic Data Base

2.2.1 Rainfall

About fifteen daily rainfall stations are situated on the study area (Figure 2.6). A pluviograph has been located at the DLWC's stream gauging station at Blandford since 1986. However it has not recorded rainfall intensities for any of the significant floods identified in this study, apart from the January and October 1996 events.

Several reporting rain gauges are operated by the SES as part of the flood warning system for the Upper Hunter Valley. Three-hourly increments of rainfall depth were available for the January 1996 flood at Murrurundi, Blandford, Blackville and Pine Ridge. (The last two stations are to the west of the study area). Corresponding data were available at Blandford and Murrurundi for the October 1996 flood.

A pluviograph has been operated by the Soil Conservation Service (now DLWC) at Scone since the early 1950's. This station is about 30 km south of Murrurundi. Records at this station were used in the hydrologic analysis, although inspection of the data showed that Scone rainfalls were not always representative of those experienced on the Pages River catchment.

No pluviographs are currently operating in the Borambil Creek catchment. Pluviographs at Gowrie North and Chaffey Dam were operational for portion of the 1970's and data at Gowrie North were supplied by BOM for the January 1976 flood. However, these two stations were closed in 1980.

2.2.2 Stream Gauging

The Blandford gauging station commenced operations in 1960, but only daily readings taken at 09:00 hours are available until 1969. From that date, it was operated as a Bristol pressure recorder but only daily discharges are available on the DLWC's HYDSYS data base for the years until 1983.

Two significant flood events occurred in the 1970's. The first flood occurred in January 1971 and recorded a peak of 8.4 m on the chart. This is equivalent to a peak discharge of 1100 m^3/s. However, a note from the gauge reader on the back of the chart suggests that this reading is not correct and that the true peak was only 7.6 m. This is consistent with reports by the townsfolk that the January 1996 flood at Murrurundi was higher than the January 1971 flood. The January 1996 flood reached a peak level of 8.3 m on the gauge. The second flood reached a peak gauge height of 7.4 m on 23 January 1976. This is equivalent to a peak discharge of 790 m^3/s.

The Muswellbrook office of the DLWC supplied discharge data for the period 1983-1996, including an annual series of flood peaks covering this period, as well as discharge hydrographs for the January 1984, February 1992 and January and October 1996 floods.

There were no data available for another significant flood event at Murrurundi, January 1991, due to failure of the gauge.

It is usual procedure to undertake frequency analyses of recorded flood peaks as a check on flood frequency relations developed using rainfall-runoff routing procedures. Such analyses usually comprise direct analysis of the streamflow records, extension of the record by correlation with nearby gauged catchments, or development of regional flood frequency relationships. The period of reliable records at Blandford is too short for the derivation of an accurate flood frequency relationship.

An annual series flood frequency analysis of the Blandford record was however undertaken, mainly for the sake of completeness. The results are given in Section 3.5 following the RORB model testing.

The Pages River has been gauged at Gundy upstream of the Hunter River junction since 1958, but the catchment area is over 1000 km^2, compared with only 300 km^2 at Blandford. There are several important tributary streams which join the Pages River downstream of Blandford. Rainfall variability may preclude the development of a satisfactory correlation between flood peaks at the two stations and hence extension of the Blandford record. Extension of the record using this approach was not considered cost-effective and was not undertaken.

The Isis River was gauged at the Lower Timor station over the years 1963 to 1982. The catchment area is 307 km^2 and the Isis River drains country similar to the Pages River in topography and climate. However, this station was discontinued prior to the reliable record of flood peaks at Blandford and therefore correlation of the flood peaks is not possible. A similar situation exists for Dart Brook at the Aberdeen station.

Moonan Brook has been gauged at the Moonan site since 1940, but this station is located well to the east of the Pages River and the catchment area is 1000 km^2.

From this review of stream gauging data it was concluded that the development of a flood frequency relationship for the Pages River using a regional approach was not possible.

2.2.3 Previous Investigation of Flooding - January 1996 Flood

A general description of the nature and pattern of flooding in each study area is provided in a reconnaissance study undertaken for Murrurundi Shire Council by Sandra Bush following the January 1996 flood. This study identified a number of peak flood levels which were later surveyed for this study. A summary of Bush's report is given in Appendix B. The report gave a comprehensive description of the pattern of flooding in the three townships and was of considerable assistance in setting up the hydraulic models.

2.3 Selection of Floods for Detailed Investigation

Table 2.1 is a list of significant floods experienced in the study area since the October 1949 event and a summary of hydrologic data available for their analysis. A ranking of these flood events is given, but this is tentative only as several floods predate the establishment of the Blandford stream gauge and as mentioned, there was a failure of the gauge for the January 1991 event.

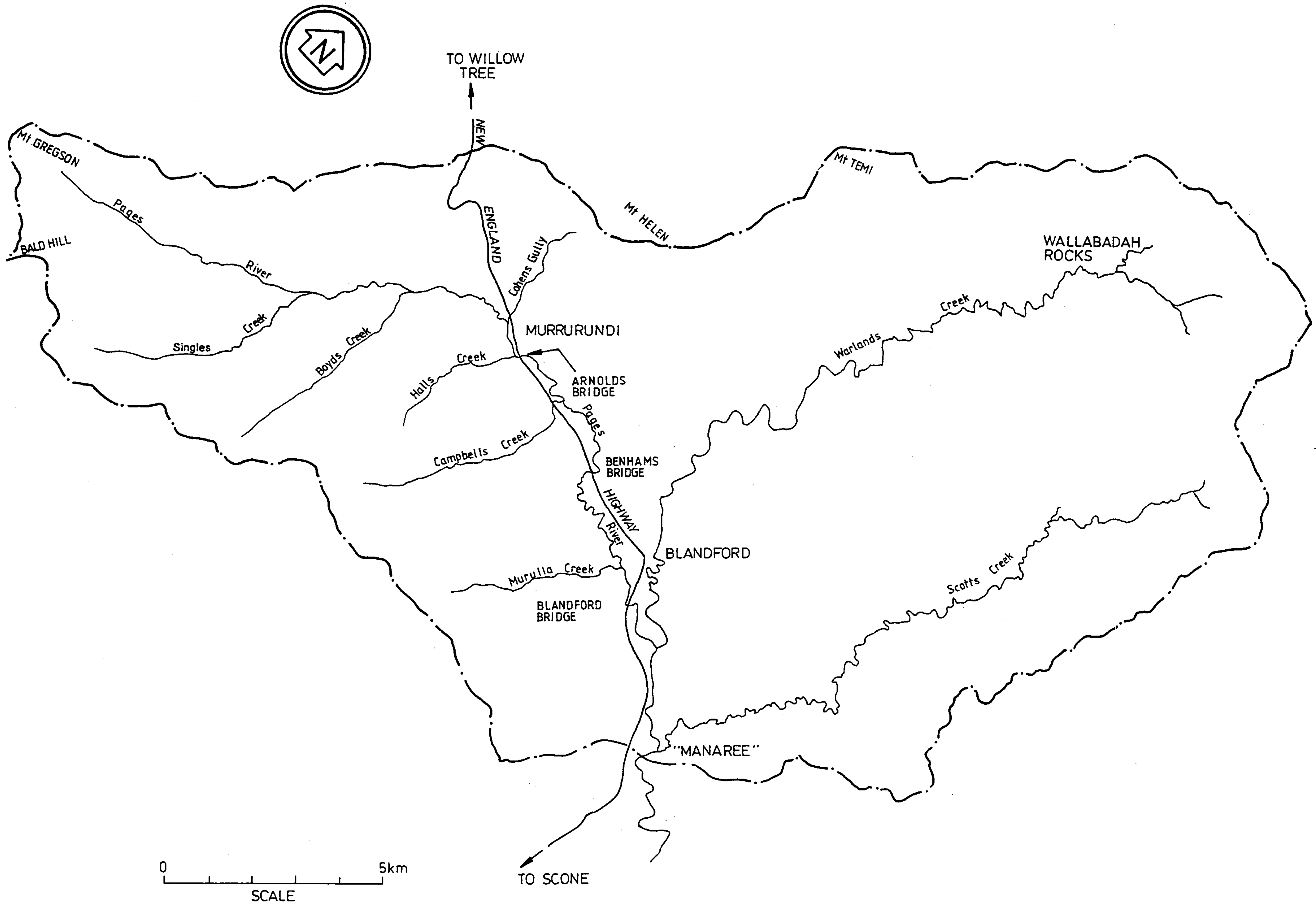
The January 1996 flood had the best data. On the basis of recorded rainfall intensities in the catchment above Murrurundi, it approximated the 100 year ARI flood. Although no recorded rainfall data is available on Warlands Creek catchment, it was clearly a major event in the area downstream of Murrurundi also. It was ranked one in the period 1949 to date and was analysed first. The February 1992 flood only ranked five and there were no Pages River catchment pluviographic data. However, inspection of the Scone trace showed a good correlation between the occurrence of the flood peaks and the intense bursts of rainfall. This flood was analysed next.

The January 1976 flood ranked seven. Pluviographic data were available at Gowrie North and Scone which appeared to be well correlated with the discharge hydrograph derived from the Bristol chart. This flood was the third analysed. In the case of the other floods, it was concluded that there were either insufficient data, or the data were suspect (eg January 1971) to justify model testing.

The October 1996 flood occurred after the catchment modelling had been completed. It was a significant flood and ranked eight in the post-1949 events. Consequently, it was included in the investigation for the purposes of confirming model parameters.

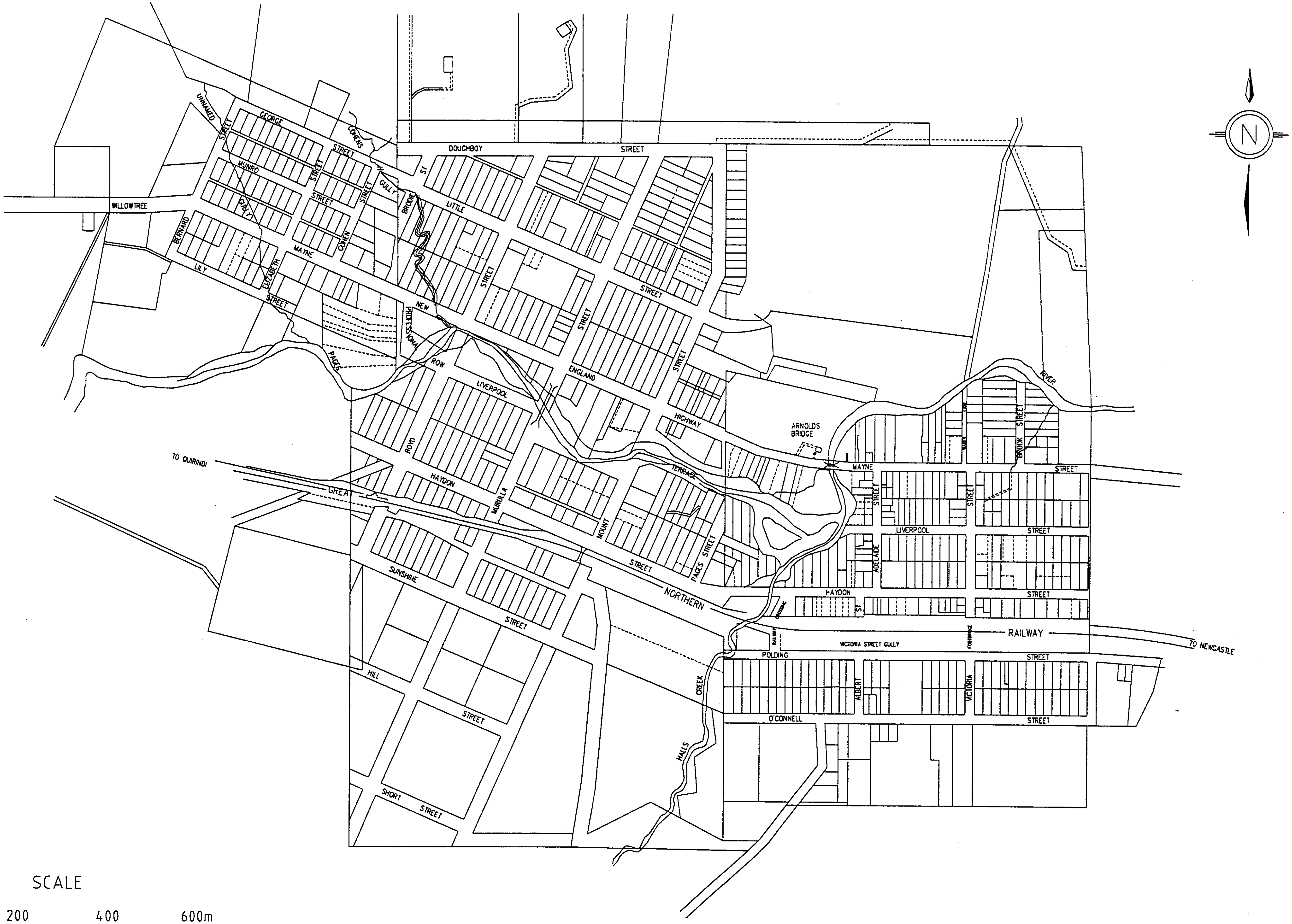
TABLE 2.1
SUMMARY OF AVAILABLE RAINFALL - RUNOFF DATA FOR ANALYSIS OF HISTORIC FLOODS

Date of Flood	Assessed Rank	Blandford G.S.		Rainfall Data		Comments
		Gauge Peak m	Peak Discharge m^3/s	Pluviographic	Daily	
17 October 1949	2	Not Available	Not Available	Not Available	15 Stations	This flood predates the establishment of the Blandford stream gauge and Scone pluviograph
22 February 1955	4	7.92	920	Scone	20 Stations	A 1993 survey of the stream cross-section at the gauging station shows the 1955 peak as 7.92 m
31 January 1971	6	7.62	830	Not Available	Not Collected	Peak was 8.4 m on chart but gauge reader assessed peak at 7.62 m
23 January 1976	7	7.45	780	Scone Gowrie North	20 Stations	Bristol pressure recorder chart obtained from DLWC Parramatta. RORB model testing was undertaken using Scone and Gowrie North pluviographs.
30 January 1984	3	8.0	950	Not available	20 Stations	No data available on temporal distribution of rainfall.
5 January 1991	9	Gauge Failed	Not Available	Scone	23 Stations	The steam gauge failed for this flood and hence the peak discharge and hydrograph shape are not known.
9 February 1992	5	7.74	870	Murrurundi Scone	22 Stations	RORB Model testing was undertaken using Scone and Murrurundi rainfalls. The pluviograph at Blandford failed.
25 January 1996	1	8.32	1030	Murrurundi Blandford Scone	15 Stations	RORB Model calibration was undertaken for this flood. Temporal rainfall data were also available at the Murrurundi P.O. and from SES.
October 1996	8	7.0	680	Murrurundi Blandford	14 Stations	RORB Model calibration was undertaken for this flood.



MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 2.1
Pages River Catchment



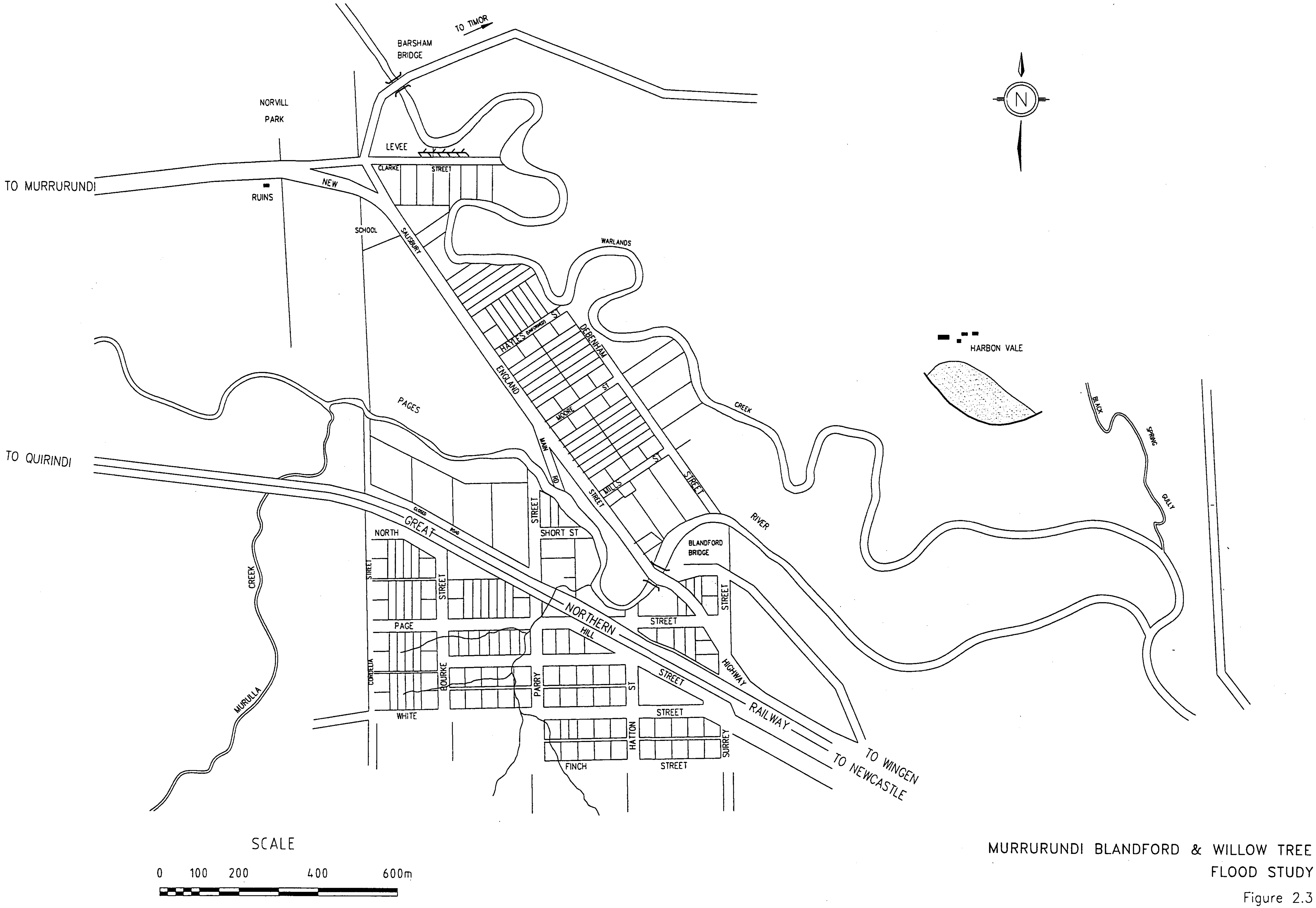
SCALE

0 100 200 400 600m

MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 2.2

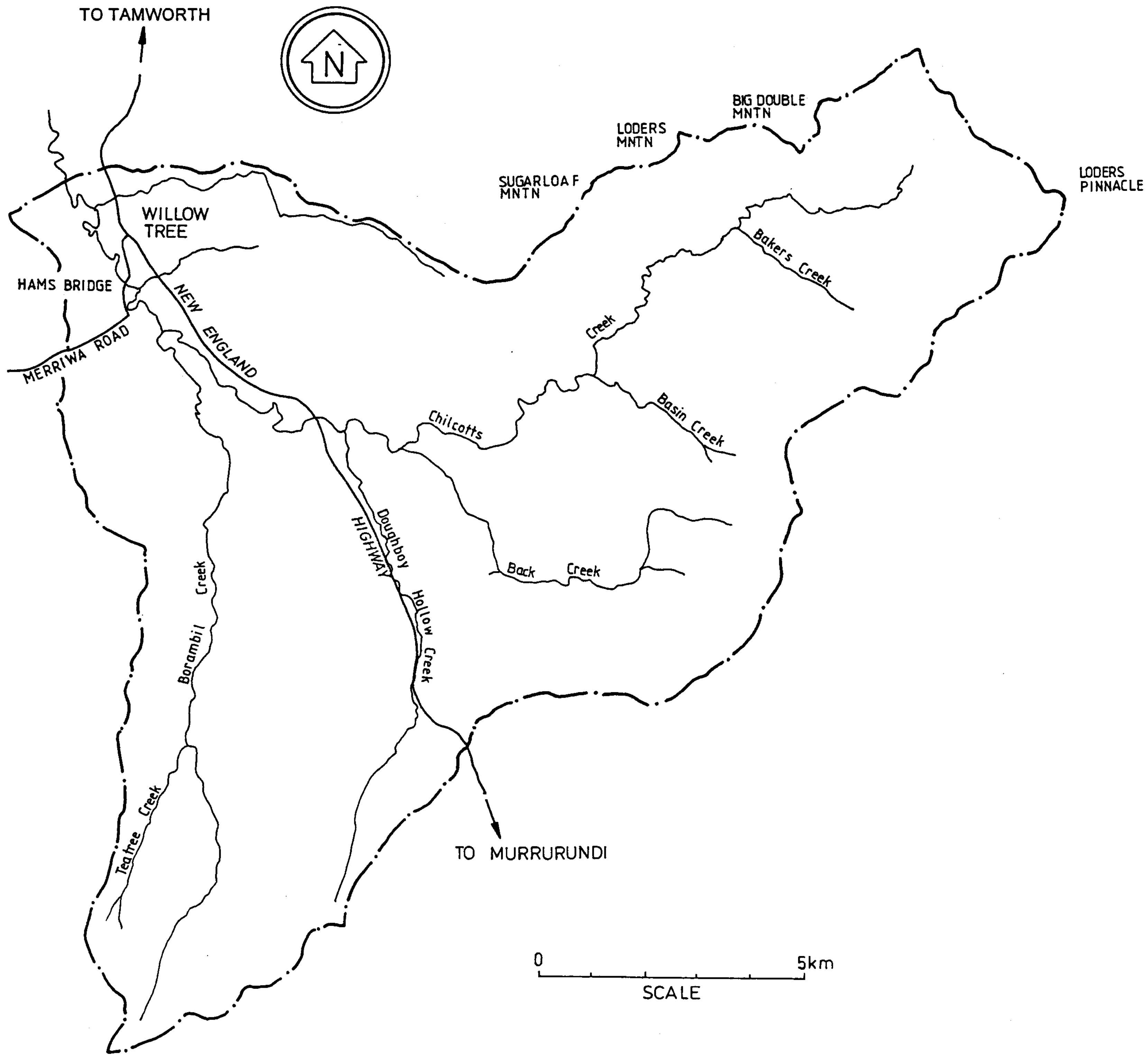
MURRURUNDI TOWNSHIP



MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

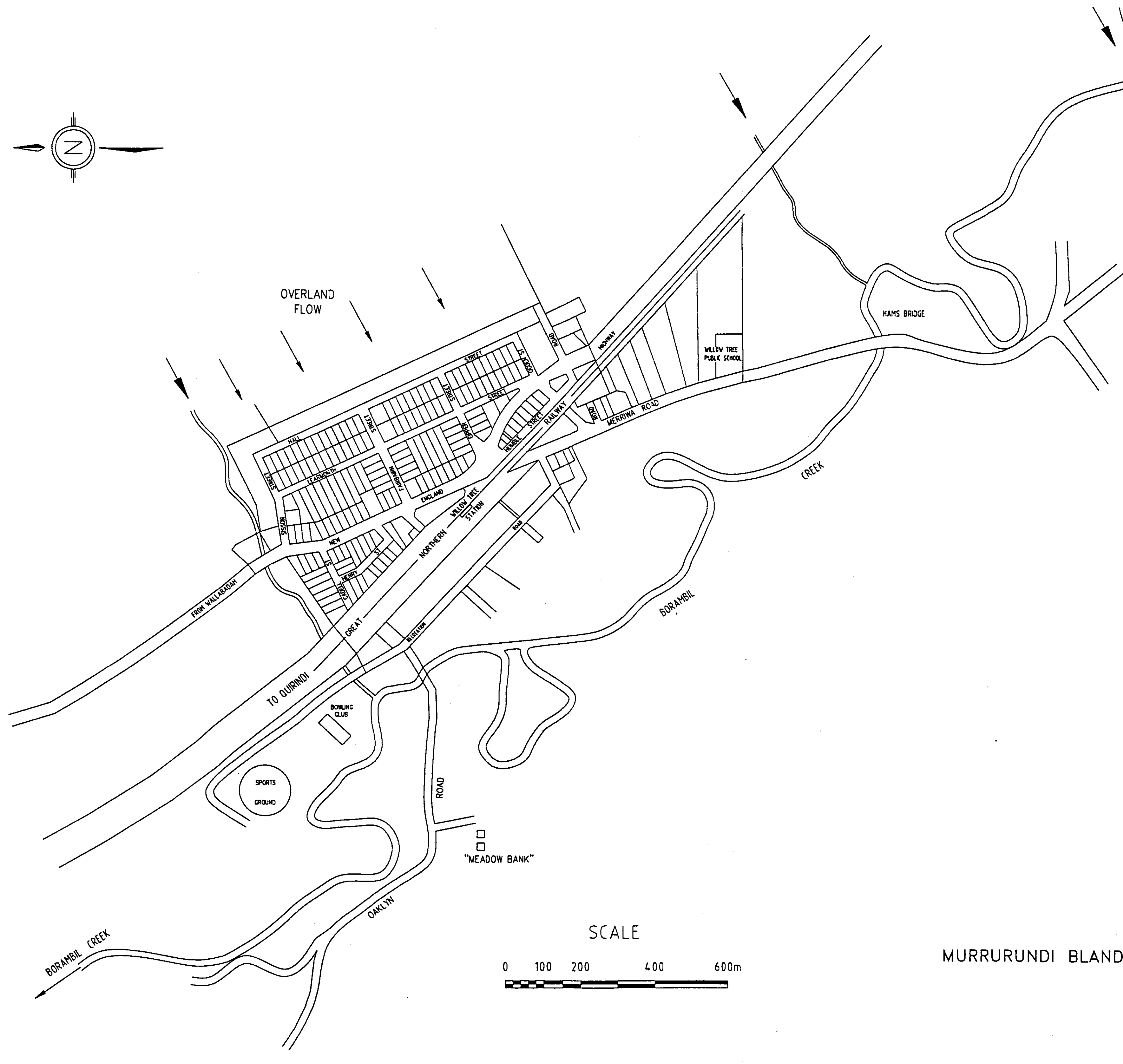
Figure 2.3

BLANDFORD TOWNSHIP



MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

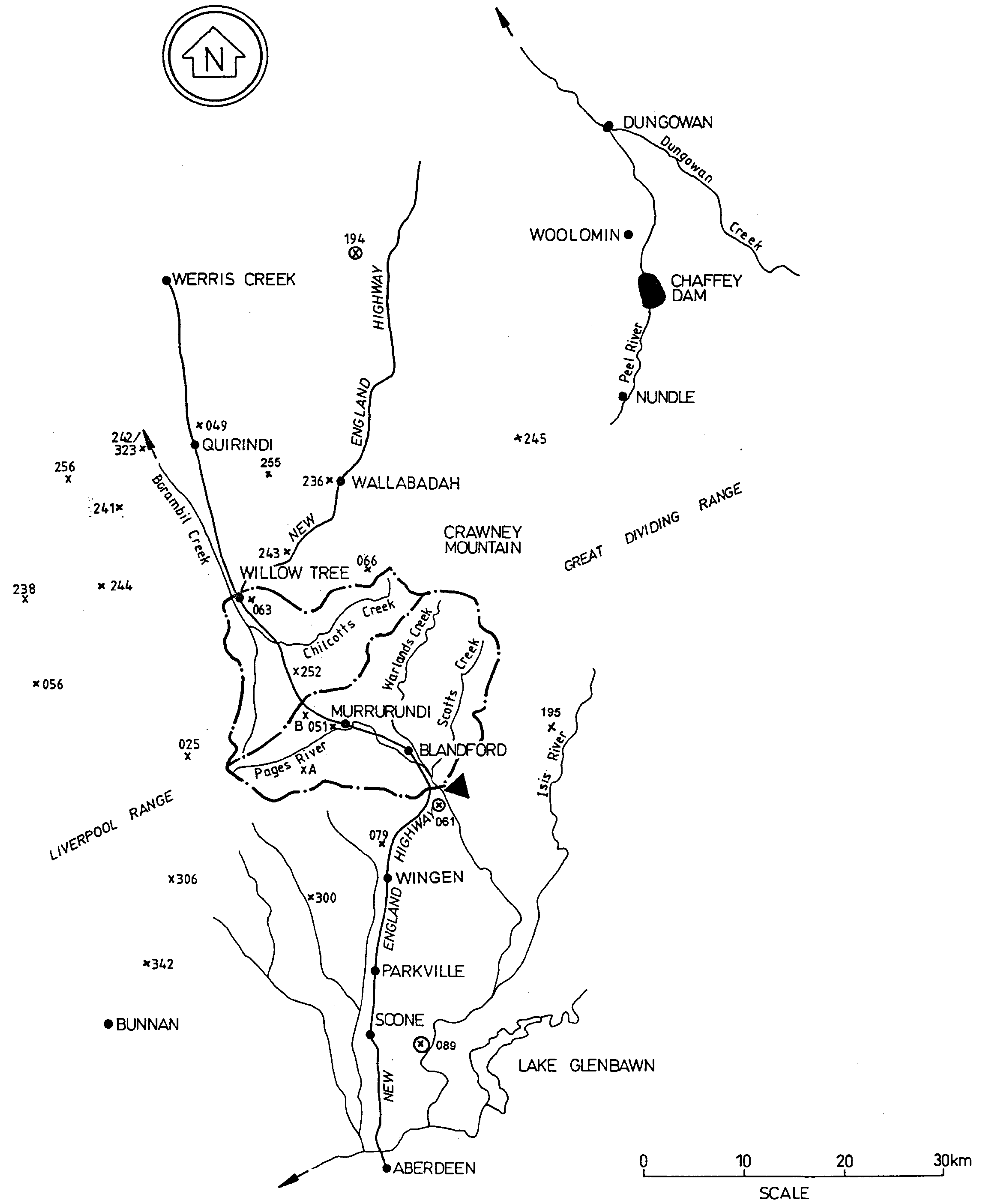
Figure 2.4
Borambil Creek Catchment



MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 2.5

WILLOW TREE TOWNSHIP



LEGEND

- × DAILY RAINFALL STATION
- ⊗ PLUVIOGRAPHIC RAINFALL STATION
- ◀ PAGES RIVER AT BLANDFORD STREAM GAUGING STATION
- —·—·— CATCHMENT BOUNDARY

Station	Lat.	Long.	Location
BOM Daily Rainfall Stations			
055025	31.79	150.67	Willow Tree (Highlands)
055049	31.51	150.68	Quirindi Post Office
055056	31.73	150.51	Willow Tree (Green Hills)
055063	31.65	150.73	Willow Tree Post Office
055066	31.62	150.86	Wallabadah (Woodton)
055236	31.53	150.83	Wallabadah (Seychelles)
055238	31.65	150.50	Pine Ridge (Boondari)
055241	31.57	150.60	Quirindi (Kooyong)
055242	31.52	150.63	Braefield (Hunday)
055243	31.61	150.78	Willow Tree (Pertala)
055244	31.64	150.57	Willow Tree (Cooinda)
055245	31.50	151.03	Nundle (Keeva)
055252	31.72	150.79	Willow Tree (Temi)
055255	31.53	150.76	Quirindi (Springvale)
055256	31.55	150.55	Quirindi (Red Braes)
055323	31.52	150.63	Quirindi (Hunday)
061051	31.77	150.84	Murrurundi Post Office
061079	31.87	150.88	Wingen (Murrulla)
061195	31.77	151.07	Murrurundi (Timor)
061300	31.92	150.80	Parkville (Aroona)
061306	31.90	150.67	Kars Springs (Woodlands)
061342	31.98	150.63	Bunnan (The Cuan)
BOM Pluviographic Rainfall Stations			
055194	31.34	150.85	Gowrie North
061089	32.06	150.93	Scone Soil Conservation Service
210061	31.81	150.93	Blandford Gauging Station
Private Gauges of Local Residents Daily Rainfalls			
A	31.78	150.79	Wykcham Park (Mr Paton)
B	31.76	150.81	Glenalvon (Mr Arnott)

MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 2.6
Rainfall and Stream Gauging Stations

3. HYDROLOGY

3.1 Selection of Hydrologic Model

For hydrologic modelling, the practical choice is between the models known as RAFTS, RORB and WBNM, and any of these would be suitable. Each of these models converts storm rainfall to discharge hydrographs using a procedure known as runoff-routing. There is little to choose technically between these models, and their usage in previous studies in the area, as well as the familiarity of the user with the model, normally decides which is selected.

In the present case, no runoff routing studies were identified in the Murrurundi area which could give a guide to model parameters. There are however, a considerable amount of data in the engineering literature on RORB model parameters for rural catchments. In view of the relative scarcity of recorded data on the study catchments, it was considered that these relationships could be useful for design purposes. Consequently the RORB modelling approach was adopted for this study.

3.2 Brief Review of RORB Model

The RORB program envisages the catchment to be comprised of a series of concentrated storages which represent sub-catchments defined on watershed lines, plus concentrated special storages which represent dams and additional stream routing effects.

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

$$S = kQ^m \qquad \text{....3.1}$$

where
- S = volume of storage
- Q = discharge
- k = a storage delay parameter
- m = a measure of the non-linearity of a catchment. When m is set equal to unity the routing response is linear for the catchment.

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

$$k = k_c.kr \qquad \text{....3.2}$$

where
- k_c = 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

3.3 Model Layout

Figures 3.1 and 3.2 shows the model layouts. The Pages River and its main tributaries are represented by the sub-areas shown in Table 3.1 and Borambil Creek in Table 3.2. The models were set up to represent existing catchment conditions.

To obtain accurate discharge hydrographs at a particular location, RORB requires the catchment upstream to be divided into several sub-areas. There were at least four sub-areas upstream of each of the locations on the main stream where discharge hydrographs were required as input to the hydraulic model. Some of the tributaries which join the main stream downstream of Murrurundi were modelled as a single sub-area. For the purposes of assessing flooding on the main stream, the accurate estimation of peak flow on those tributaries was not as important as determining a reasonable estimate of the temporal distribution of runoff volume for input to the hydraulic model.

3.4 Model Calibration and Testing

3.4.1 Approach

The procedure for the calibration and testing of the RORB models involved the analysis of data from daily rain gauges and pluviometers in and adjacent to the catchment to ascertain the temporal and areal distribution of rainfall.

Isohyetal maps, prepared to cover the durations of the storm events, were used in conjunction with pluviographic data to estimate hyetographs of rainfall input for each sub-catchment of the RORB model.

As mentioned, model calibration and testing was carried out using historic flood data in the following sequence:

- January 1996
- February 1992
- January 1976
- October 1996

TABLE 3.1
DETAILS PAGES RIVER
RORB MODEL

Location	Model Sub areas	Catchment Area km^2
Pages R. u/s Cohens Gully	A-F	64
Cohens Gully	G-H	2.3
Halls Creek	J-K	5.4
Campbells Creek	M	12
Murulla Creek	O	7.4
Warlands Creek	P-U	103
Scotts Creek	X-Z	55
Pages R. at gauging station	A-Z	300

TABLE 3.2
DETAILS BORAMBIL CREEK
RORB MODEL

Location	Model Sub-areas	Catchment Area km^2
Borambil Creek at junction Chilcotts Creek	A-D	49
Chilcotts Creek	E-I	114
Borambil Creek d/s junction with Chilcotts Creek	A-I	163
Borambil Creek d/s Willow Tree	A-J	182

3.4.2 Results of Modelling

3.4.2.1 January 1996 Flood

Heavy rain fell over the two rain days 24 and 25 January 1996. Some of the rain responsible for the flood fell after 09:00 hours on 25 January and is recorded on the rain day 26 January. About 26-30 mm fell over the catchment in the 24 hours prior to 09:00 hours on 24 January. This was sufficient to wet the catchment and produce a very small response at Blandford which resulted in a surface runoff of less than 1 mm.

Heavy rain commenced to fall in the catchment in the early hours of the 25 January. At Blandford the rainfall commenced at 04:00 hours. Over the following five hours to 09:00 hours, 63 mm were experienced, with a maximum hourly fall of 26 mm between 04:00 - 05:00 hours.

At Murrurundi, about 12 mm were experienced to 07:50 hours. However, over the following 90 minutes to 09:20 hours, a further 78 mm were reported to have fallen according to Council records, giving a total depth of 90 mm for the storm. The SES's reporting rain gauge at Murrurundi was interrogated at 23:58 hours on 24 January, and at 08:31 hours and 10:17 hours on 25 January. It reported a total fall of 84 mm, which is consistent with Council data. This rainfall depth at Murrurundi is in excess of the 100 year ARI point rainfall for this locality.

At Blandford, the volume of surface runoff recorded by the stream gauge was assessed at 70 mm. According to the hydrograph supplied by DLWC, the stream rose rapidly from 5 m^3/s at 05:00 hours to a peak of 1030 m^3/s at 10:15 hours, before reducing to 50 m^3/s eight hours later. However, SES records report the peak occurring over an hour later, at 11:38 hours.

The isohyetal map for the rain day 25 January 1996 is shown on Figure 3.3. No rainfall data were available for the important Warlands and Scotts Creek catchments. The isohyets plotted over those catchments are heavily depended on the 10 mm of rainfall reported at Nundle in the Namoi River catchment.

It is understood that flooding was particularly severe on Warlands Creek, where, as mentioned previously, a fatality was experienced and the flood was consistent with the observation of a "wall of water" moving down the drainage system.

It is therefore likely that adoption of the plotted isohyetal pattern would give an underestimate of the rainfall which actually occurred on these eastern tributaries. Consequently in the RORB modelling, the rainfalls in the sub-areas representing Warlands and Scotts Creeks have been adopted which are similar to those recorded in the Murrurundi region.

The best fit of the model is shown on Figure 3.4. The shape of the recorded hydrograph is well reproduced. The modelled peak occurred at 11:00 hours, one hour later than the peak recorded on the DLWC's gauge. However, if it is accepted that the DLWC's clock was slow and the SES records that the peak actually occurred at 11:38 hours are correct, then the fit is very good.

3.4.2.2 February 1992 Flood

This flood occurred as a result of heavy rainfall experienced over the catchment in the three day period 8 to 10 February 1992. Over this period, a total of 214 mm was recorded at Murrurundi. Rainfalls over the study area were heaviest at Murrurundi and over the south-east part of the Pages River catchment. Rainfalls were lighter over the Borambil Creek catchment. At Willow Tree, the three day total was only 121 mm (Figure 3.5).

The Blandford pluviograph failed. However, three hourly rainfall depths were recorded by the SES at Murrurundi. The Scone pluviographic data were also available. Although the total three day rainfall at Scone amounted to only 123 mm, the pattern appeared similar to Murrurundi when compared on a dimensionless percentage of total rainfall versus time basis. Murrurundi and Scone data were used to develop a temporal rainfall pattern for the Pages River catchment.

Two flood peaks were recorded at the Blandford gauging station. The first occurred at 18.00 hours on 8 February and amounted to 480 m^3/s. The second peak of 870 m^3/s was recorded at 09:45 hours on 9 February. The RORB model was run in "fit" mode in an attempt to reproduce the recorded hydrograph, although because of the uncertainty in the definition of the temporal patterns of rainfall over the catchment, it would probably be best to regard the analysis as a "test" of the model rather than a formal calibration.

As far as timing of the peaks is concerned best results were achieved with the same routing parameters as were derived for the January 1996 flood (Figure 3.6). The first modelled peak occurred one model time step (30 minutes) before the observed peak, while the second peaks coincided. The first peak was modelled closely whilst the calculated discharge of the second peak was less than the observed value. The shapes of the hydrographs agree quite well.

Table 3.3 below shows the results for a range of assumed initial loss values. As the model was run in "fit" mode, all of these loss values were compatible with the observed distribution of rainfall and the computed depth of surface runoff of around 92 mm. Best fitting results were achieved with an initial loss of 85 mm and a continuing loss rate of 0.26 mm/h.

TABLE 3.3
RORB MODEL FITTING
8-10 FEBRUARY 1992
k_c = 9.5, m = 0.8

Observed	Modelled Peak Discharge m^3/s for Indicated Loss Values			
Peak Discharges m^3/s	50 mm 1.67 mm/h	80 mm 0.48 mm/h	85 mm 0.26 mm/h	90 mm 0.06 mm/h
480	630	530	480	430
870	620	740	760	780

3.4.2.3 January 1976 Flood

Heavy rainfalls were experienced on the Hunter Valley over the four days 22 to 25 January 1976. On the study area, the heaviest falls were experienced on 23 and 24 January (Figure 3.7). A focus of rain at Murrurundi gave a two day total depth of 229 mm. Falls over the Pages River catchment were generally greater than 150 mm. Lesser falls were experienced further to the north in the Namoi catchment. At Willow Tree (Temi), 168 mm were recorded.

Two major flood peaks were recorded at the Blandford gauge. The Pages River commenced to rise at 05:00 hours on 23 January and reached its first peak of 780 m^3/s at 12:00 hours. A second peak was recorded at 21:00 hours and amounted to 660 m^3/s. A third small peak was recorded at 16:30 hours on 24 January. The total surface runoff from the first two peaks amounted to 130 mm.

The Scone and Gowrie North pluviographs were used to assess the temporal pattern of rainfall over the study area. Their temporal patterns were quite similar, but the total depths of rainfall over the two days differed. The two day rainfall recorded at Scone was smaller than over the study area, amounting to 69 mm on 23 January and 50 mm on 24 January. The total two day fall at Scone was therefore only about 50 percent of the rainfall experienced at Murrurundi.

Table 3.4 and Figure 3.8 show the results of running RORB in the "fit" mode for the Scone pluviographic pattern with the routing parameters found to apply in the previous analyses. The timing of the second peak is well modelled although the discharge is less than recorded. It is not possible to model the first peak. The best results were achieved with an initial loss of 50 mm and a continuing loss rate of 0.42 mm/h.

TABLE 3.4
RORB MODEL FITTING
23 - 24 JANUARY 1976
k_c = 9.5, m = 0.8
(SCONE PLUVIOGRAPHIC PATTERN)

	Modelled Peak Discharge m^3/s for Indicated Loss Values		
Observed Peak Discharges m^3/s	**30 mm 1.36 mm/h**	**40 mm 0.9 mm/h**	**50 mm 0.42 mm/h**
780	660	605	645
660	345	390	430

The two day rainfall at Gowrie North amounted to 90 mm on 23 January and 78 mm on 24 January, giving a total of 168 mm compared with 119 mm at Scone and was more representative of rainfall depths over the study area. Figure 3.9 shows the results achieved running the model in "fit" mode with the Gowrie North pattern. The first peak is closely modelled but the calculated second peak is much less than the observed value. Best results were achieved with an initial loss of 55 mm and a continuing loss rate of 0.18 mm/h.

3.4.2.4 October 1996 Flood

Heavy thunderstorm rainfall fell on the evening of 6 October 1996, resulting in flash flooding on the Pages River catchment. Flows were much smaller on Borambil Creek. During the course of the flood, the stream gauging station at Blandford was damaged. However, the recorded discharge hydrograph which has a peak of 680m^3/s appears plausible. This discharge is equivalent to a 10 year ARI peak at Blandford. At Murrurundi, flow entered the township via a breach in the levee running along the right bank upstream of Arnolds Bridge. Because rainfalls were high in the upper reaches of the catchment above Murrurundi, the frequency of the flood peak was relatively higher at that location, with the modelled peak discharge approximating the 50 year ARI design flow as assessed later in Chapter 5.

The rain responsible for the flood fell after 11:00 hours on 6 October and is recorded on the rain day 7 October. About 25 mm fell over the Pages River catchment in the preceding few days. This was sufficient to wet the catchment and result in a smaller initial loss than for the other floods analysed.

Heavy rain commenced to fall in the catchment in the mid-afternoon of the 6 October. At Blandford the rainfall commenced at 16:00 hours. Over the following two and a half hours to 18:30 hours, 47 mm were experienced, with a maximum hourly fall of 25 mm between 16:15 - 17:15 hours.

At Murrurundi, about 7 mm were experienced to 15:00 hours. However, over the following 3 hours to 18:00 hours, a further 62 mm were reported to have fallen according to Bureau of Meteorology records. In the next 3 hours to 21:00 hours, 3 mm were experienced giving a total depth of 72 mm for the storm.

At Blandford, the volume of surface runoff recorded by the stream gauge was assessed at 31 mm. According to the hydrograph supplied by DLWC, the stream rose rapidly from 6 m^3/s at 17:00 hours to a peak of 680 m^3/s at 20:00 hours, before reducing to 45 m^3/s six hours later.

The isohyetal map for the rain day 7 October 1996 is shown on Figure 3.10. No rainfall data were available for the Warlands and Scotts Creek catchments. The isohyets plotted over these catchments are based on the nearest available stations.

The best fit of the model is shown on Figure 3.11. The shape of the recorded hydrograph is well reproduced. The modelled peak occurred at 20:00 hours, matching the timing of the peak recorded on the DLWC's gauge. However, the peak discharge (620m^3/s) is about 10 per cent less than the recorded peak. This result was achieved with kc = 8.5. Further reduction in kc to 8.0 increased the peak to 650m^3/s, but also increased the peak at Murrurundi to a value which gave excessively high peak flood levels as modelled by MIKE 11 (Section 4.4.4). There is considerable uncertainty regarding the rainfalls actually experienced over Warlands and Scotts Creeks, which enter the Pages River below Murrurundi. It is likely that the rainfalls on these catchments were higher than the values estimated using data from available rain gauges. It was decided to proceed with the hydraulic modelling using hydrographs obtained from the modelling parameters shown on Figure 3.11.

3.4.3 Comments on RORB Modelling - Pages River

3.4.3.1 Model Parameters

The model reproduces the shape of the recorded hydrograph with a consistent set of routing parameters. Best results were achieved with k_c values of 9.5 for the first three floods analysed and 8.5 for the October 1996 event. A value of m equal to 0.8 was adopted for all tests. Discrepancies between recorded and modelled peak flows appear to be due to uncertainties in the definition of temporal rainfall data in the Pages River catchment, particularly on Warlands Creek. Where accurate catchment pluviographic data were available, such as for the January 1996 flood, the model reproduced the peak quite well.

The value of k_c of 9.5 appears quite small when compared with published results and studies on other gauged catchments. Assuming that a relationship like equation 3.3 applies, where k_c is related to the square root of the catchment area, then the constant K equals 0.55 for the Pages River catchment. This value compares with a value of 2.2 for K, as suggested by the RORB manual and a value of 1 for a similar gauged catchment at Currambene Creek on the south coast of NSW (LMCE, 1983).

$$k_c \quad = \quad KA^{0.5} \quad ......3.3$$

Several relationships between k_c and A are presented in ARR, 1987.

For the eastern region of New South Wales a relationship based on data from 29 catchments east of the dividing range derived by Kleemola, 1987 is:

$$k_c \quad = \quad 1.22A^{0.46} \quad ......3.4$$

This equation gives a k_c value of 17 for the Pages River.

A relationship (equation 3.5) was also derived from 86 catchments in Queensland. Most of the available data were 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 3.5 gives a value of k_c of 18 for the Pages River.

$$k_c \quad = \quad 0.88A^{0.53} \quad ......3.5$$

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

3.4.3.2 Rainfall Losses

For two of the floods analysed, initial loss values were: 85 mm for February 1992 and 50 mm for January 1976. For the January and October 1996 floods, the initial losses were 12 mm and 28 mm respectively, apparently reduced by prior rainfall in the catchment. Continuing losses were small for the first two floods 0.26 and 0.42 mm/h compared with 2.74 mm/h and 1.56 mm/h in January and October 1996 respectively.

For design flood estimation (Chapter 5), it is necessary to assess design loss values. The purpose of design loss is to achieve a flood with a given ARI from a design rainfall with the same ARI. Since actual losses vary considerably from event to event, design losses can be viewed as probabilistic or statistical estimates of the most likely value. Walsh et al, 1991 describe the derivation of design losses with different ARIs using the rainfall intensity - frequency - duration, temporal patterns and areal reduction factors published in ARR, 1987.

For design, Walsh et al, 1991 recommended initial loss values ranging between 60 mm and 40 mm for frequencies ranging between 10 and 100 years ARI. All of these values applied for a continuing loss of 2.5 mm/h.

For design flood estimation (Chapter 5), it is a reasonable approach to adopt Walsh et al's recommended initial loss values, along with the continuing loss values and to carry out sensitivity studies around those values.

3.4.3.3 Internal Flows

Discharge hydrographs are required as inputs to the hydraulic model in the main river and its tributaries between Murrurundi and Blandford. Accordingly, it was necessary to assess the consistency of the RORB model flows in this reach.

At Murrurundi, upstream of Cohens Gully, the catchment area is 64 km^2 and the modelled peak discharge of the January 1996 flood is 410 m^3/s. At Blandford the corresponding values are 300 km^2 and 1030 m^3/s. These values suggest that peak flows increase approximately as the square root of the catchment area. This relationship is commonly used in river valley flooding investigations. It is the basis for the Myer maximum formula relating peak flow to catchment area for extreme flood events.

Downstream of the junction with Halls Creek, the modelled January 1996 peak flow is 450 m^3/s. This flow is in close agreement with the value derived from a recorded flood mark and a rating curve prepared at a cross section upstream of Arnolds Bridge (Mayne Street).

These results suggest that the RORB model gives a reasonable estimate of peak flows at locations on the main stream remote from the stream gauging station at the catchment outlet.

Several important tributary streams join the Pages River in the modelled reach. The most important of these is Warlands Creek which has a catchment area of 103 km^2. The modelled January 1996 peak discharge on this stream was 400 m^3/s which compares with a modelled peak discharge of 495 m^3/s on the Pages River just upstream of the junction. The Pages River catchment area at the junction is 114 km^2. These peak discharges are consistent with the sizes of the respective catchments.

From these results, it was concluded that the RORB model could be used to provide inflow hydrographs to the MIKE 11 hydraulic model for the purposes of both model calibration and design flood estimation.

3.4.4 Comments on RORB Modelling - Borambil Creek

Adopting equation 3.3 for the Borambil Creek catchment, together with a value of K equal to 0.55 as for the Pages River catchment, yields a k_c value equal to 7. For loss values found to apply on the Pages River catchment (IL = 12 mm, CL = 2.74 mm/h), this value of k_c along with m = 0.8 gave a peak discharge of 835 m^3/s at Willow Tree for the January 1996 flood.

A rating curve was derived for the stream cross section at the downstream end of the MIKE 11 hydraulic model for use as a boundary condition (Section 4.3). Applying a recorded flood mark for the January 1996 flood to this rating curve gave an estimate of 710 m^3/s for the peak flow, which is in reasonable agreement with the RORB model results.

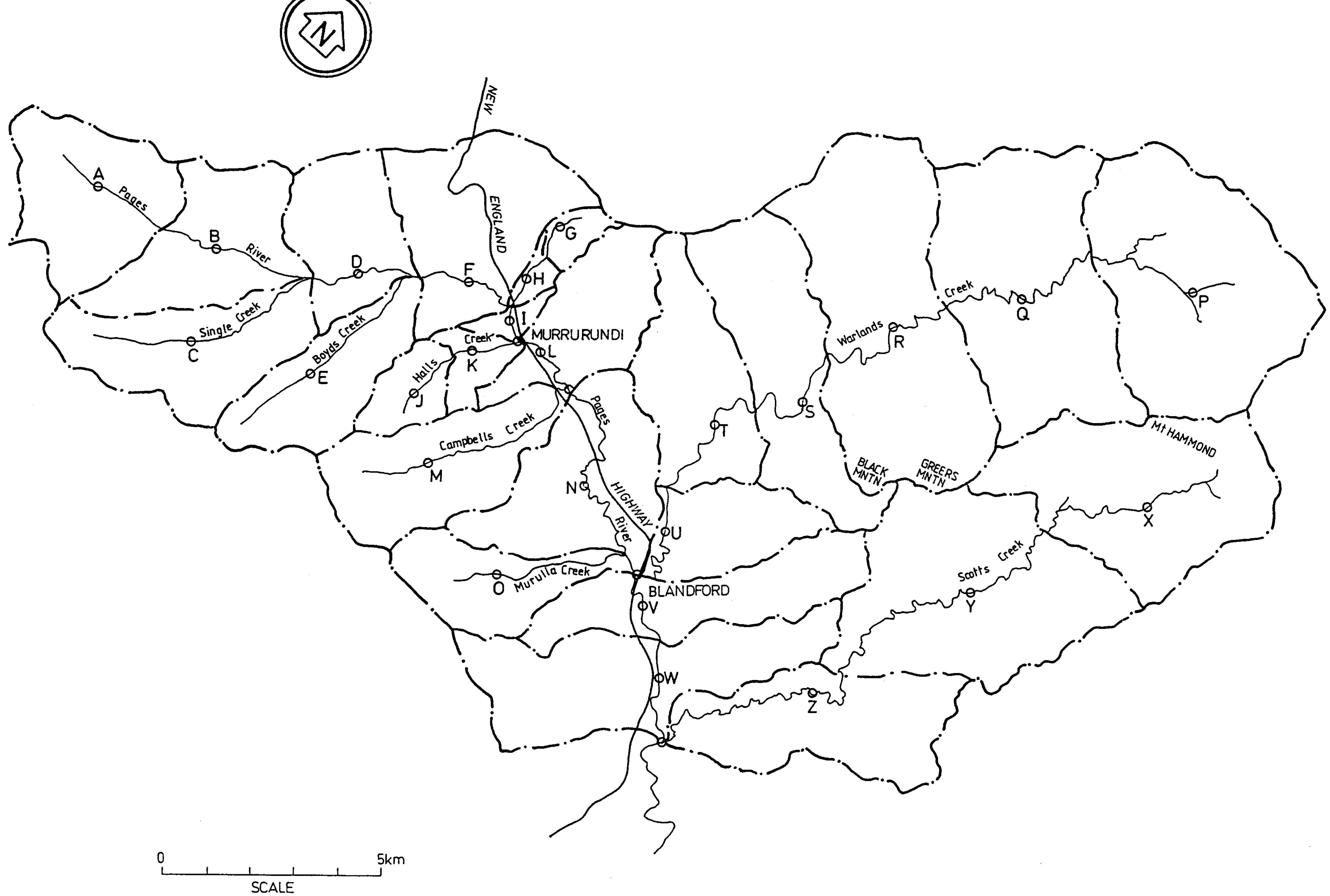
For the purposes of design it was considered reasonable to adopt a value of 7 for k_c, with m equal to 0.8 and loss values according to Walsh et al, 1991.

3.5 Flood Frequency Analysis

The result of an annual series flood frequency analysis at Blandford for the years 1984-96 is shown in Figure 3.12. This period corresponds with the reliable period of record at this station.

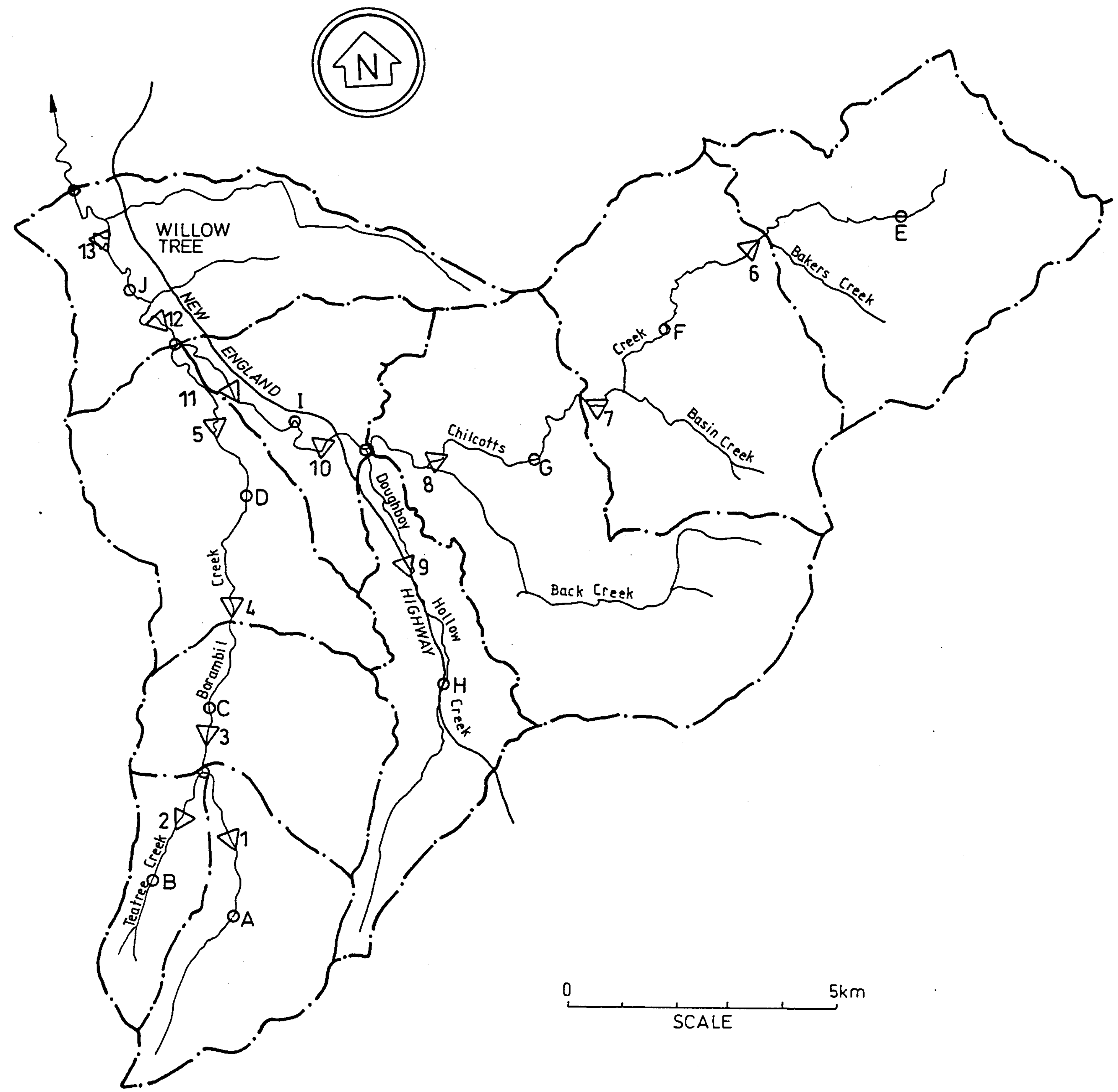
The shape of the fitted frequency curve is heavily influenced by the three recent major flood events 1984, 1992 and 1996 and gives estimates of peak flow which are clearly on the high side. The estimate of the 100 year ARI flood is 5240 m^3/s which compares with 1030 m^3/s for the January 1996 flood. Rainfall intensities over the Pages River catchment for this flood were estimated to be similar to the 100 year ARI. The adopted frequency curve derived by rainfall runoff modelling (Chapter 5) is also shown on Figure 3.12.

The frequency analysis results cannot therefore be used as a check on the results achieved with the RORB model for the design flood estimation (Chapter 5). This conclusion is supported by Section 12.6 of ARR, 1987 which provides quantitative assistance in the choice between flood estimates based on design rainfalls or direct flood frequency analysis.



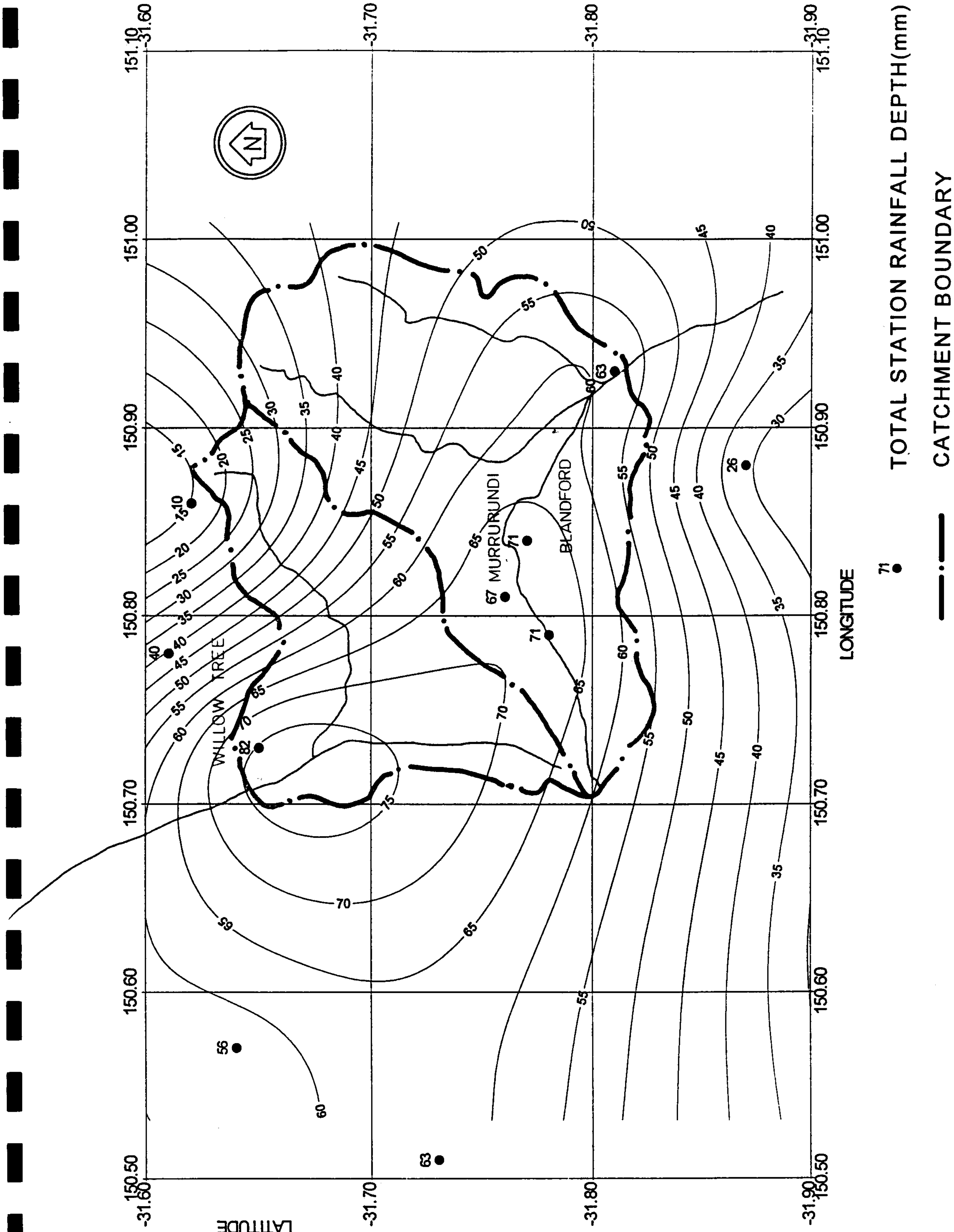
MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 3.1
RORB Hydrologic Model Layout Pages River



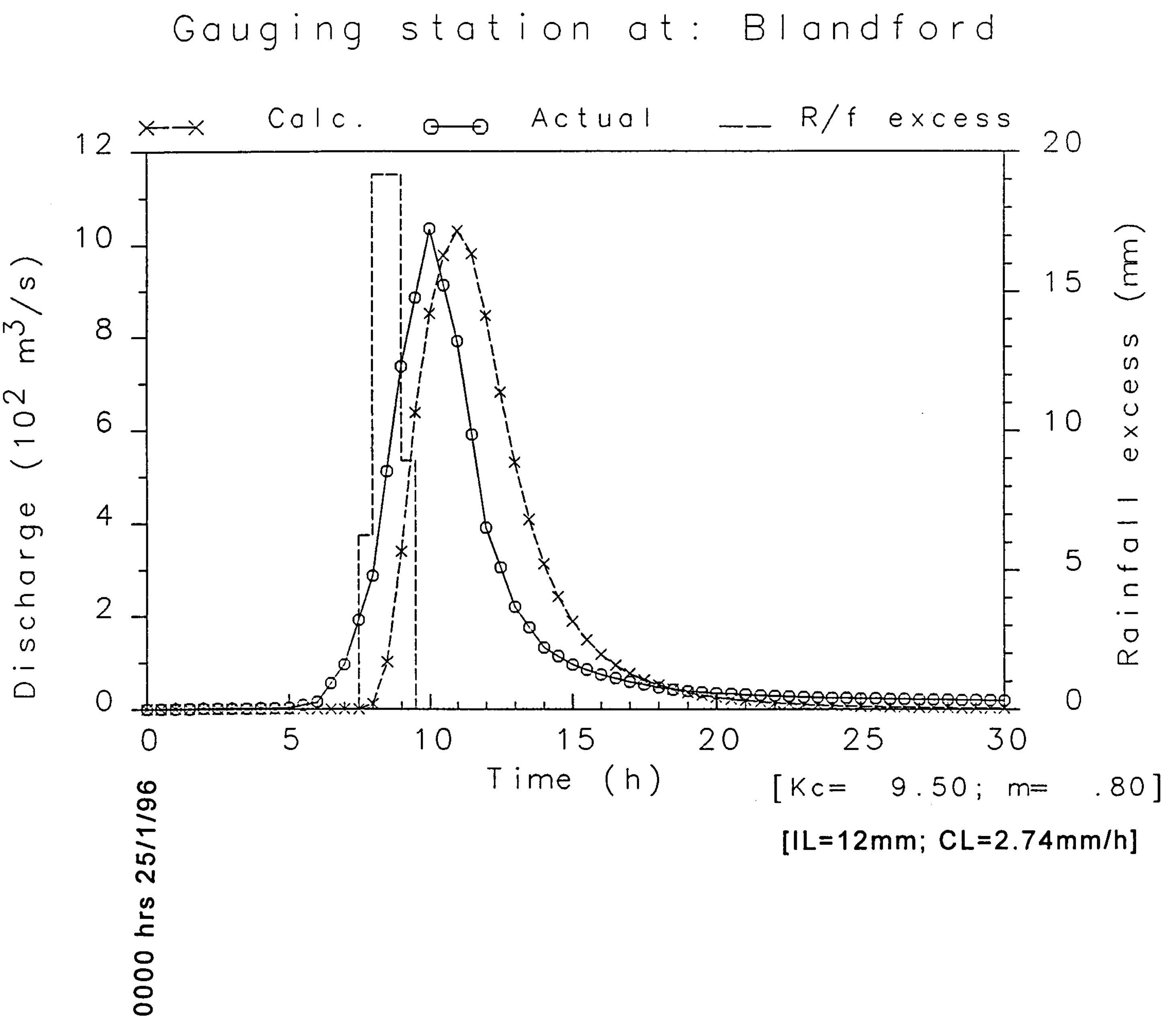
MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 3.2
RORB Hydrologic Model Layout Borambil Creek



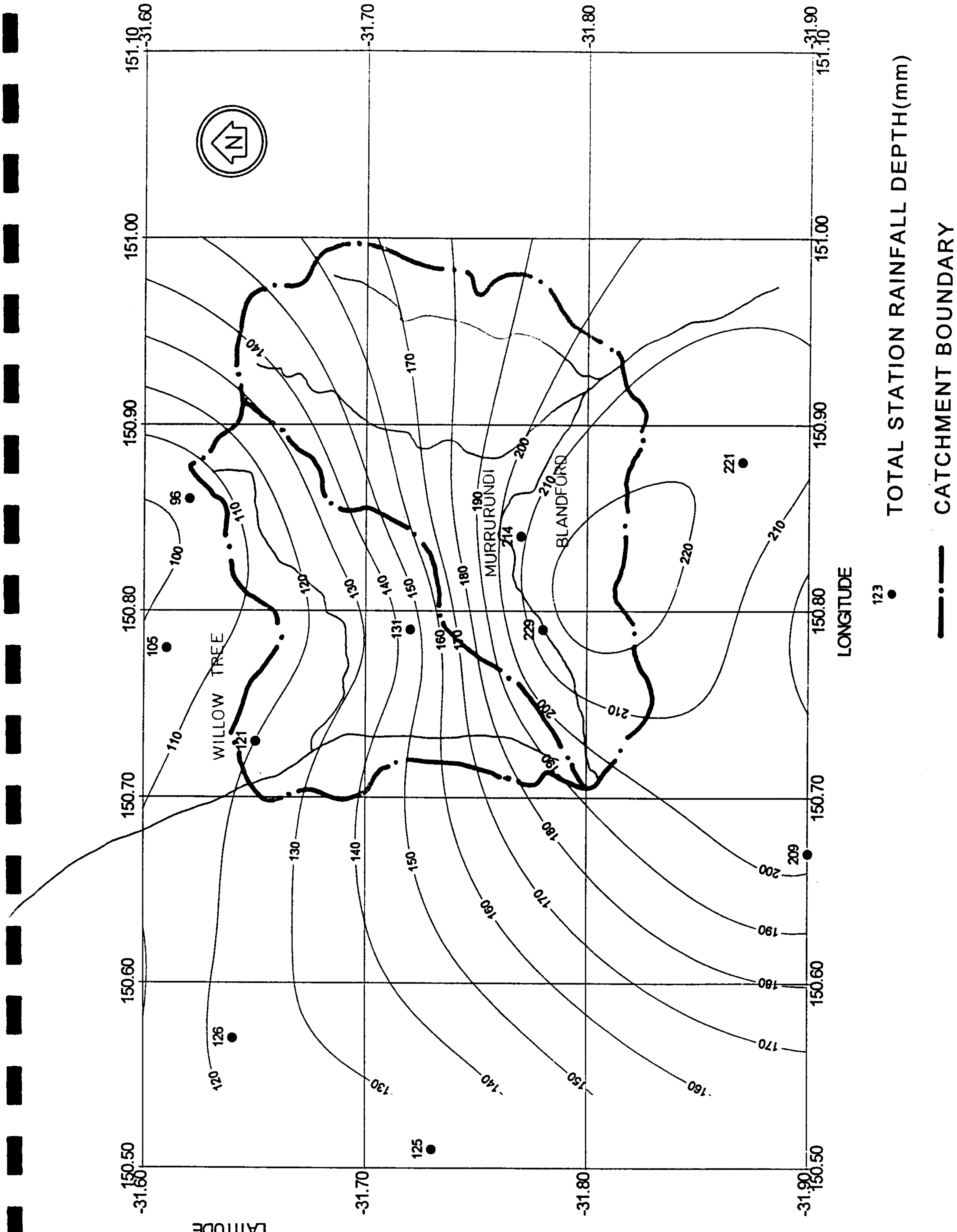
MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 3.3
Isohyetal Map
25 January 1996



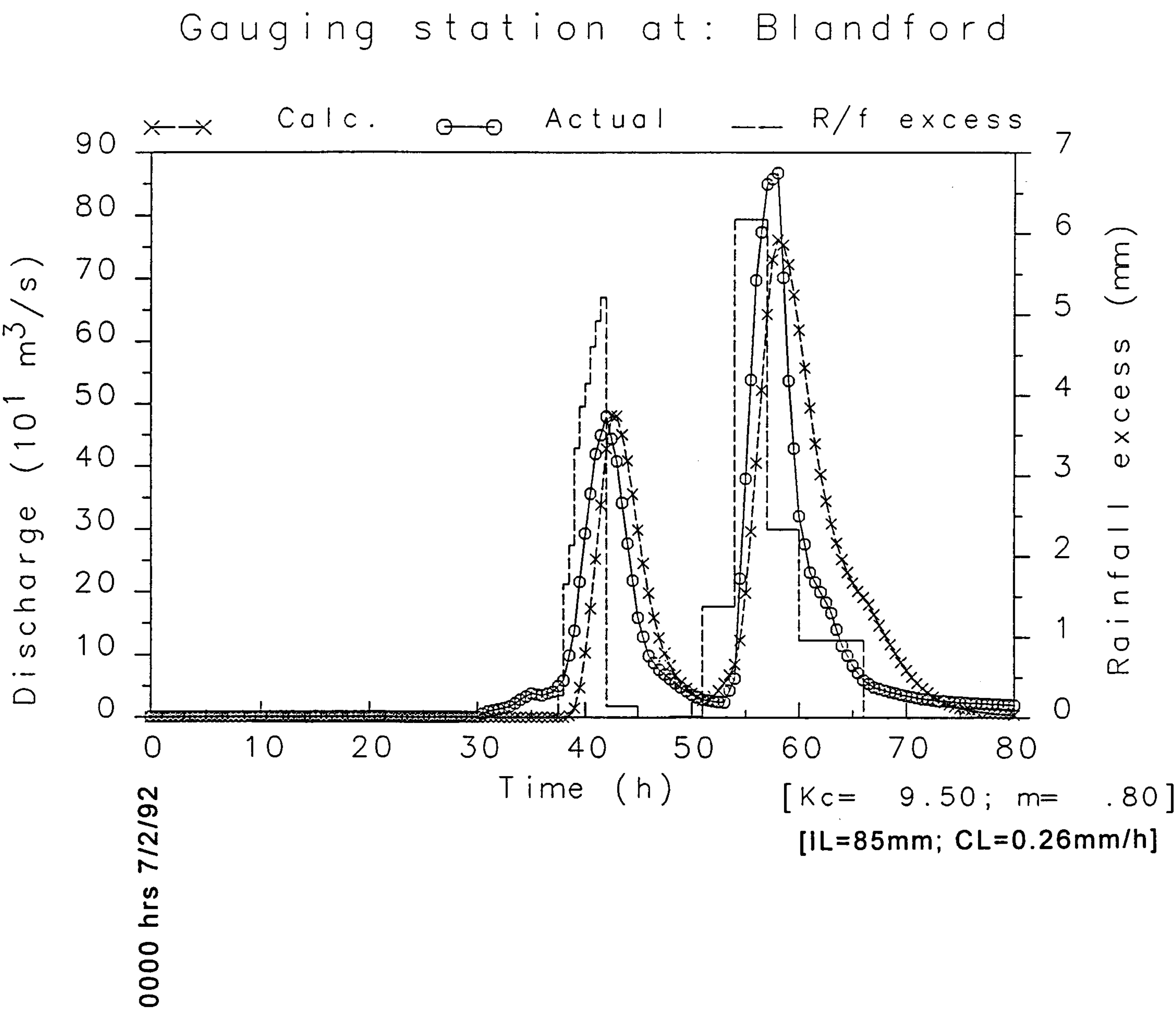
MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 3.4
RORB Model Results
25 January 1996



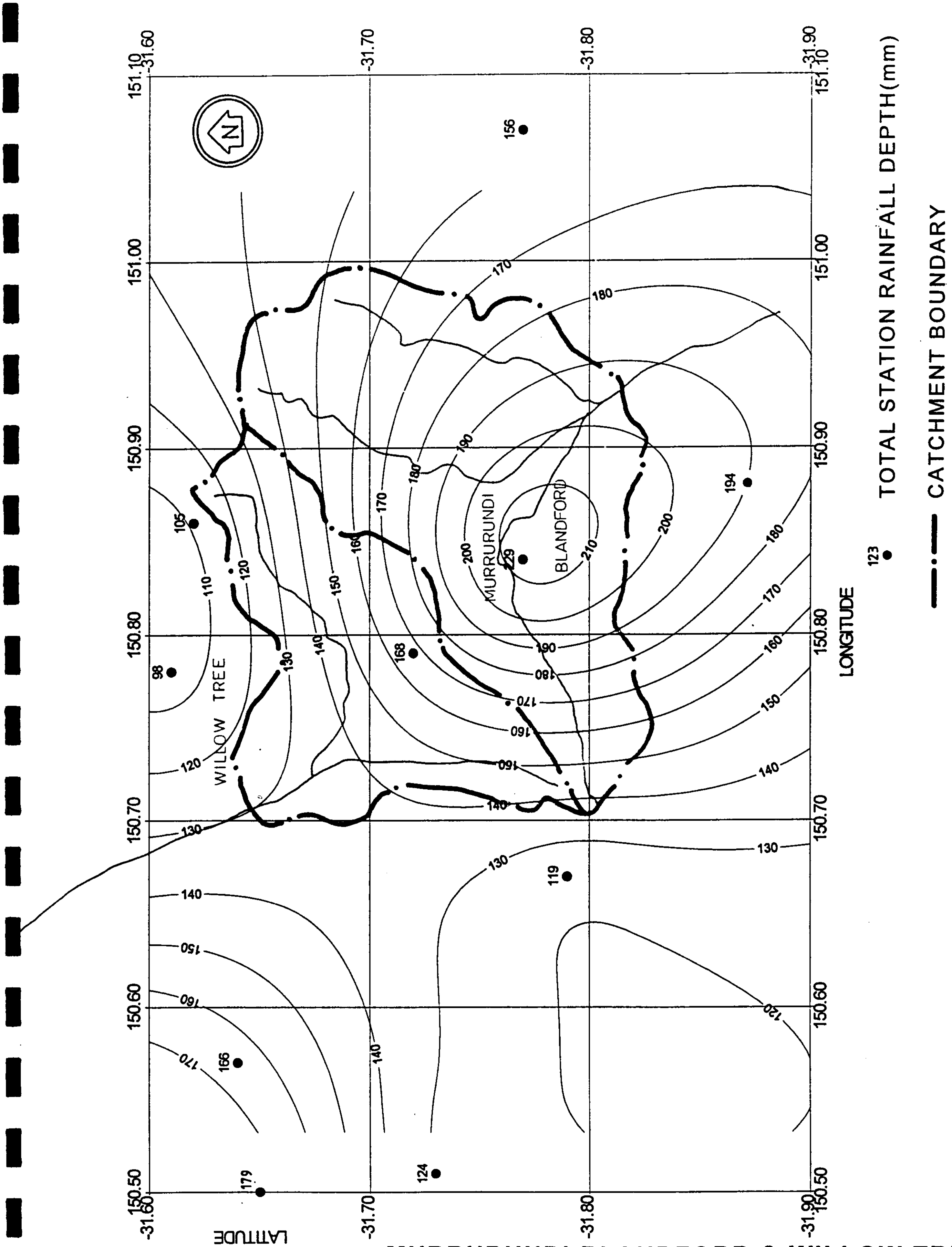
MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 3.5
Isohyetal Map
8-10 February 1992



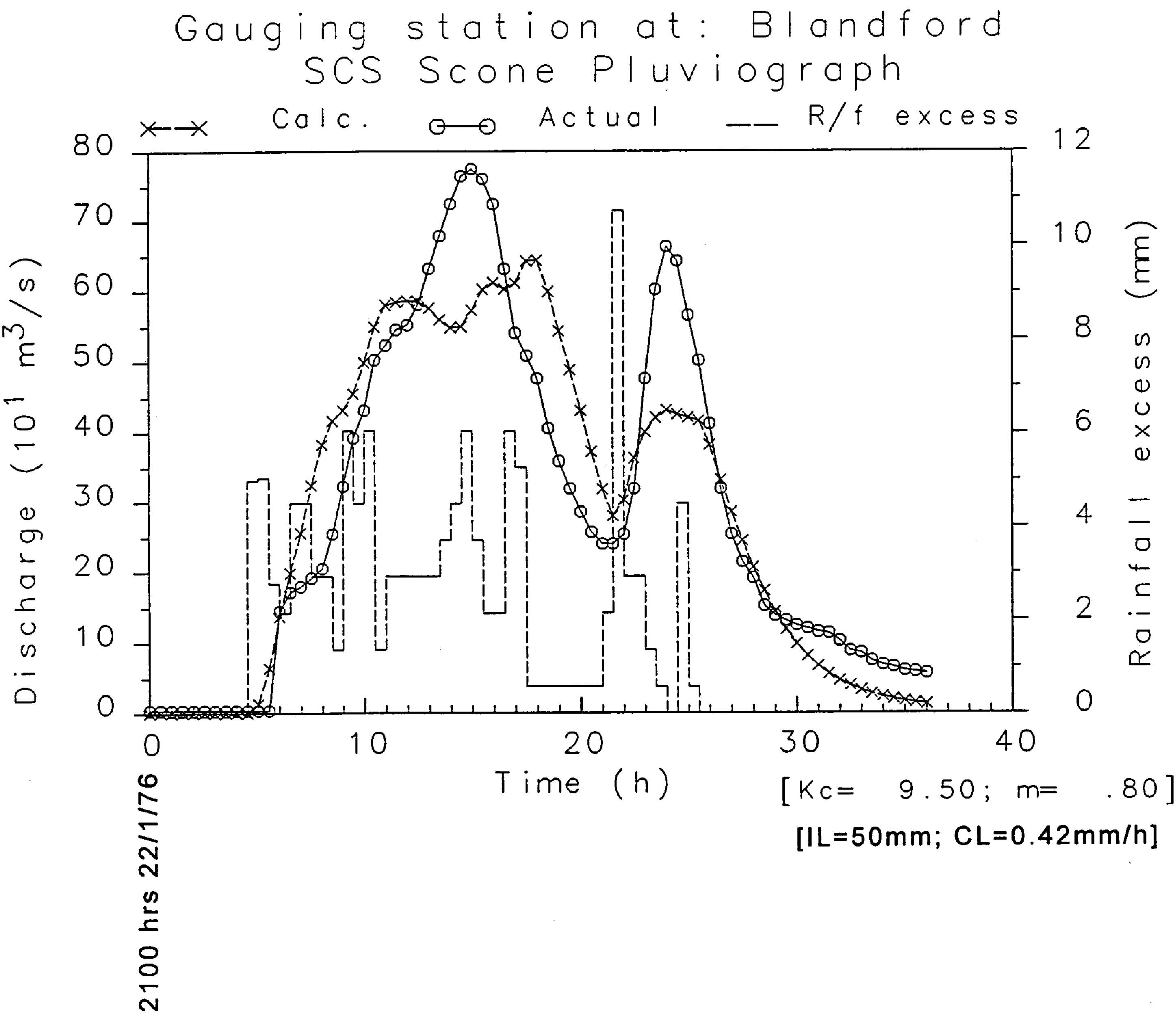
MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 3.6
RORB Model Results
8-10 February 1992



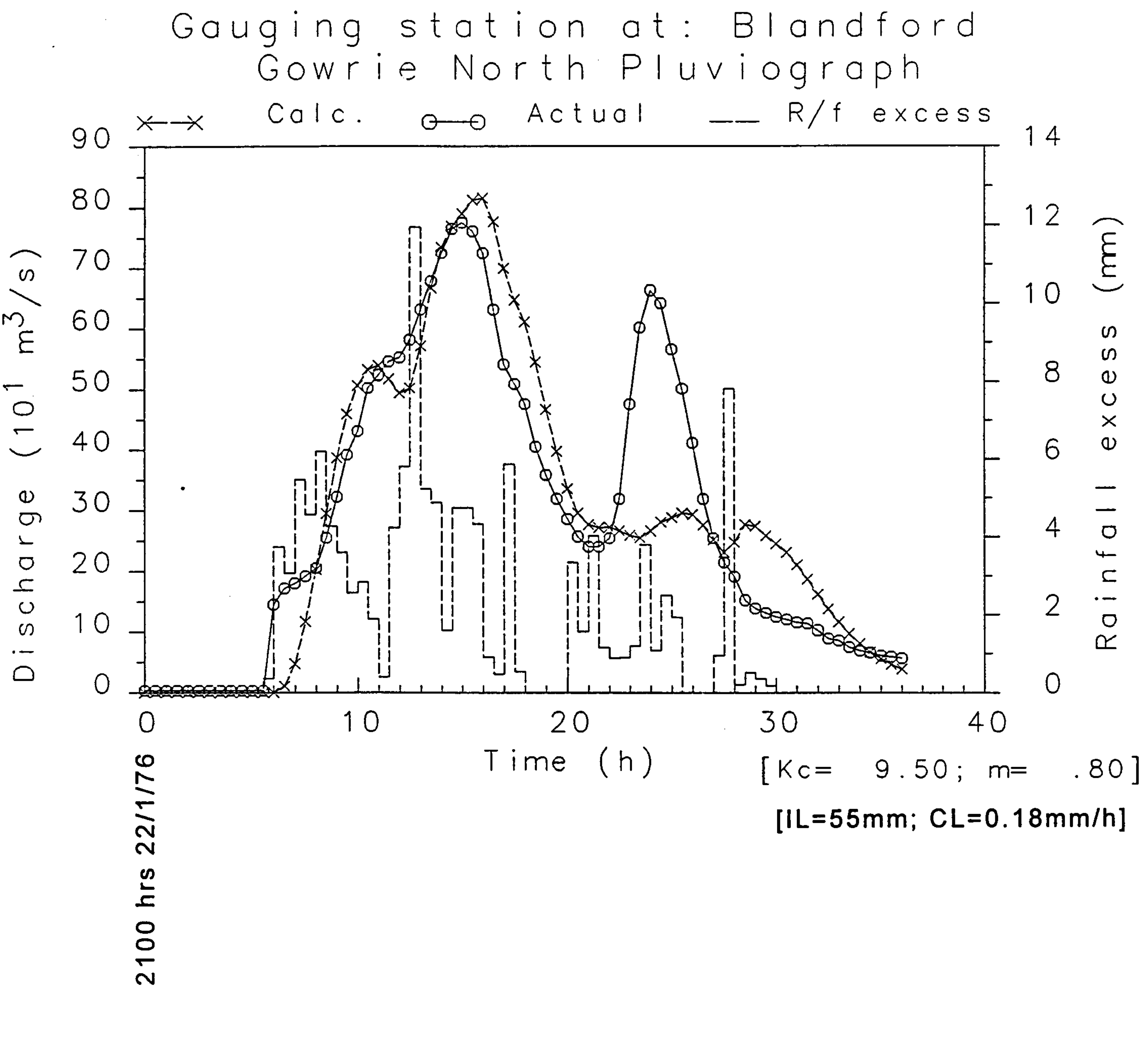
MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 3.7
Isohyetal Map
23-24 January 1976



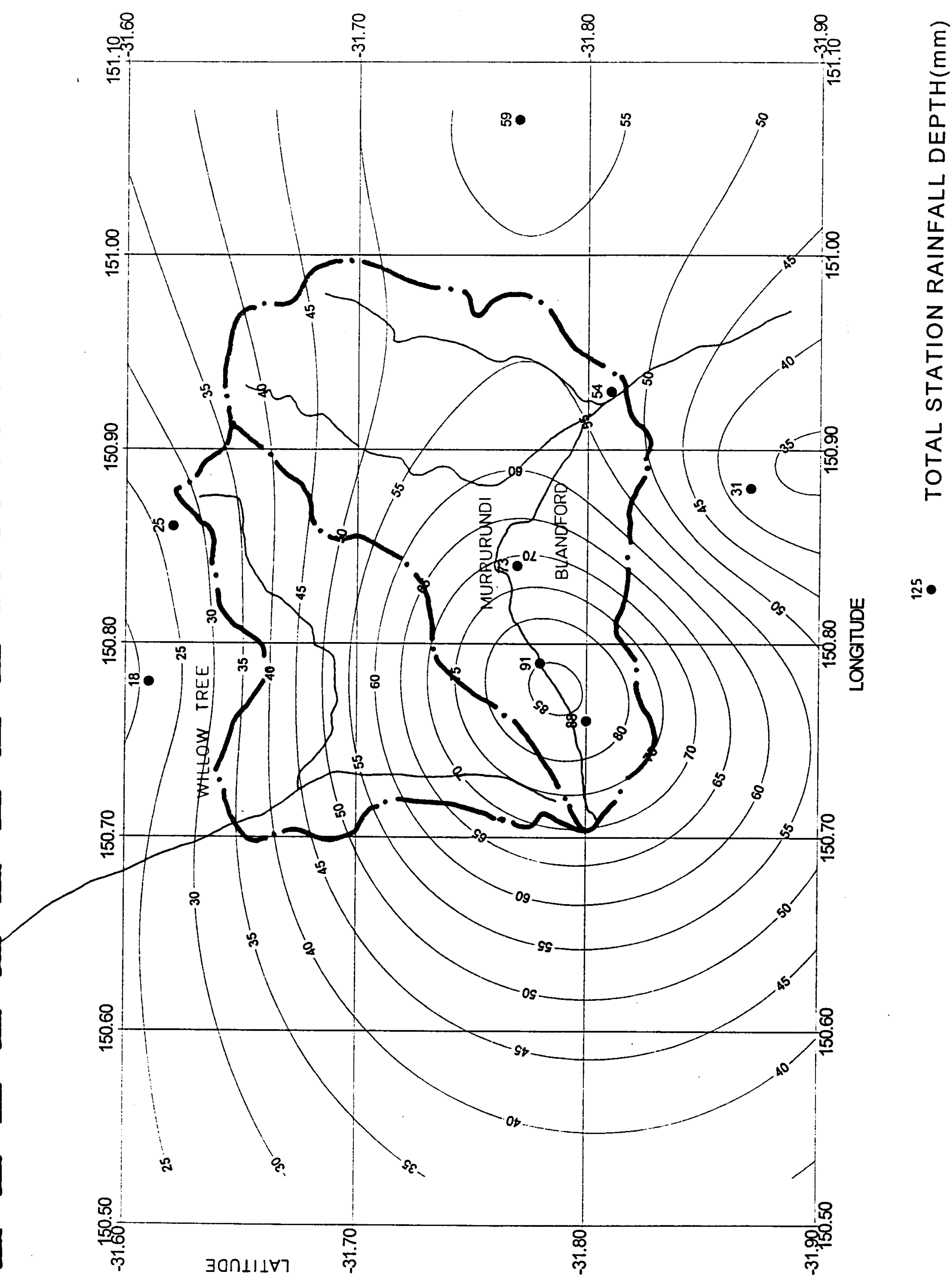
MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 3.8
RORB Model Results
23-24 January 1976



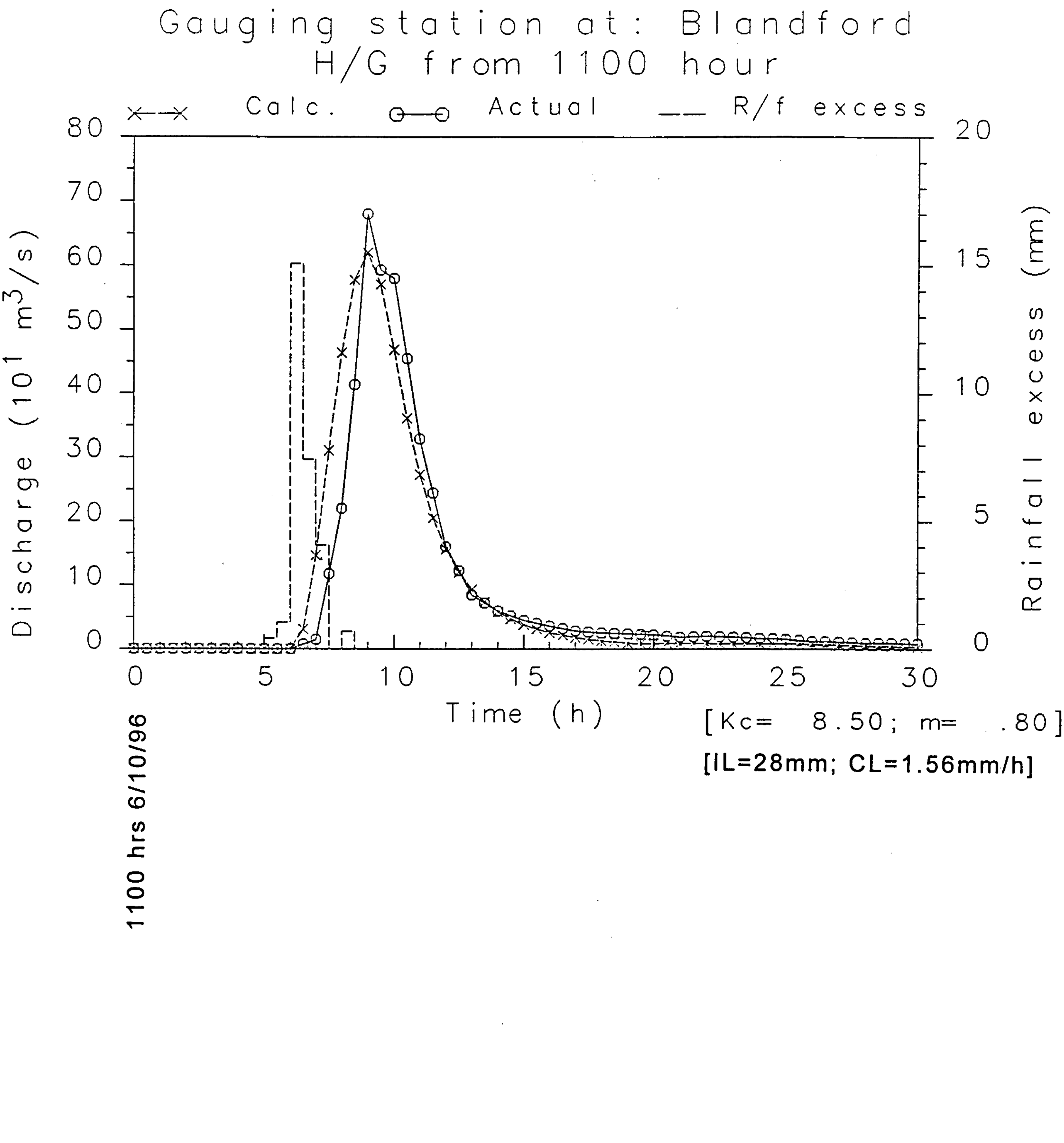
MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 3.9
RORB Model Results
23-24 January 1976



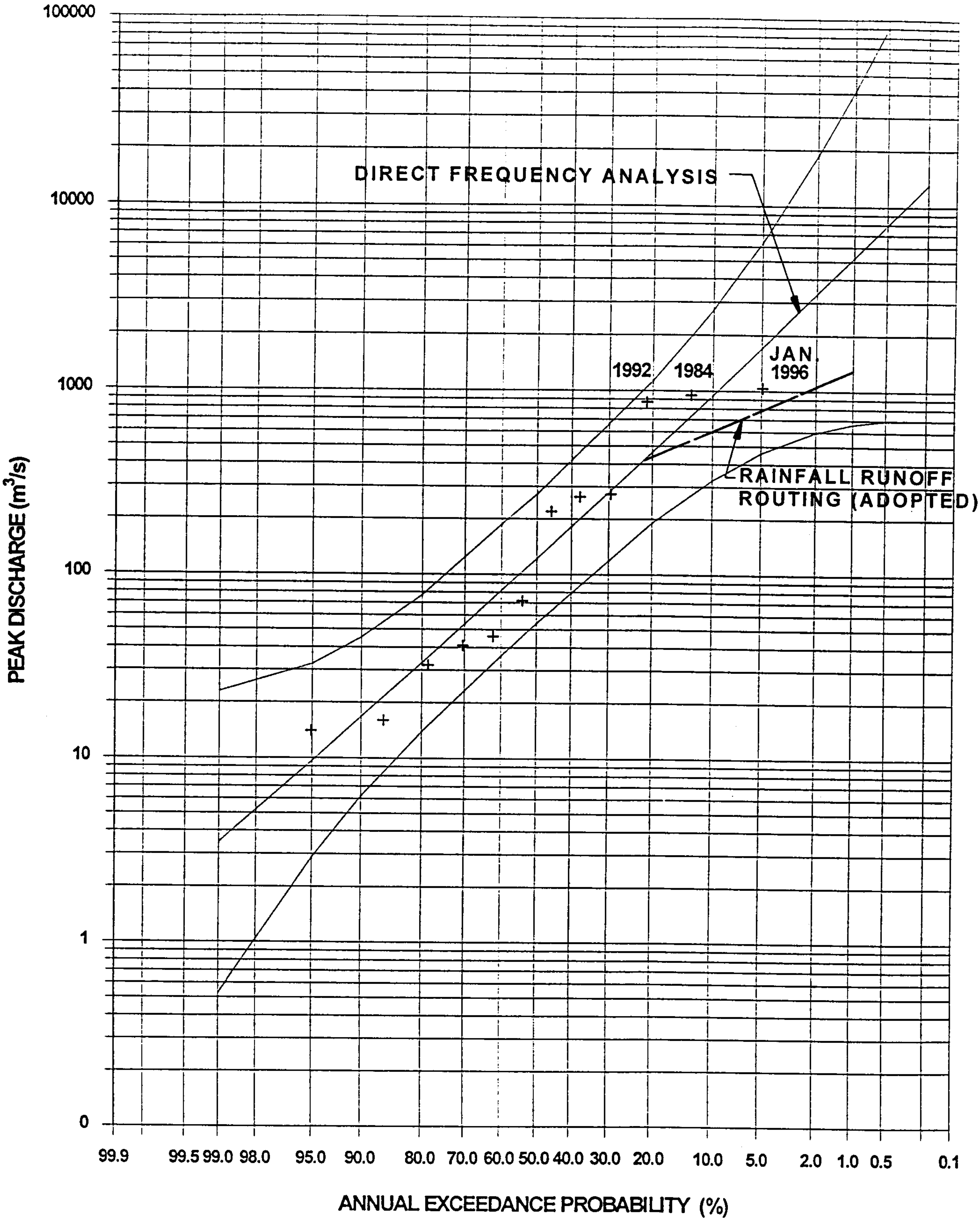
MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 3.10
Isohyetal Map
7 October 1996



MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 3.11
RORB Model Results
6 October 1996



MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 3.12
Annual Series Flood Peaks at Blandford
1984-1996

4. HYDRAULICS

4.1 Model Requirements

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 bridges crossing 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.

Given the extent of modelling on the Pages River, it was not economically feasible to construct a model which considered each and every feature. Rather, a "meso-scale" model was required which considered all of the significant features which affected flow patterns in the main areas of interest (Murrurundi and Blandford). Should it be necessary in later studies (eg. Floodplain Management Study) to focus more closely on individual areas, two approaches could be adopted:

- Additional computational points could be added to the existing model. This approach would be adopted if the area to be investigated covered a reasonably large reach of the model.

- A separate "micro scale" model of the area of interest could be constructed, boundary conditions for which could be obtained from the main model. The approach would be more suited to modelling the effects of small scale modifications, or where the flow pattern at the micro scale level is more of a two dimensional nature rather than the one dimensional type assumed in the main model.

For both approaches further survey would be required to provide the additional geometric detail necessary in a finer scale model.

4.2 Selection of Hydraulic Model

Several commercial modelling systems: MIKE 11, BRANCH, DWOPER and UNET were considered as well as the FPLAIN system developed by LMCE. All of these models are of the link-node type which solve the one-dimensional equations of unsteady flow developed by Saint Venant using an implicit finite difference approach. The flow equations, which are hyperbolic in nature, are linearised over a time step and a set of equations is developed describing the relationship between flows and water surface elevations at an irregularly spaced network of cross sections. These equations are solved simultaneously using a variety of matrix inversion techniques, to describe the behaviour of the system at the end of the time step.

Although the solution procedure is based on one-dimensional flow equations, the models are capable of analysis for situations where the flood flow is conveyed by a number of alternative flow paths in the channel and floodplain. They belong to the family of "quasi two-dimensional" hydraulic models where a prior knowledge of the pattern of flood flows is required in order to set up the various fluvial and weir type linkages which are used to compute the passage of a flood wave through the valley. This is obtained from site inspection together with a knowledge of historic flood behaviour. A geometric model of the channel and floodplain is set up using cross-sections normal to the direction of flow. Information on the elevations of the tops of the banks is required to enable computation of the weir type equations which describe the passage of overbank flow from the channel to the floodplain and vice versa.

BRANCH and DWOPER are public domain software developed by the US Geological Survey and National Weather Service respectively. BRANCH (Schaffranek et al, 1981) may be used to simulate a wide range of flow conditions in channel networks and is computationally very effective. It allows for variable roughness parameters, lateral inflows and distinguishes between effective flow and storage widths in channel cross-sections. However, this program does not include routines for the analysis of bridge waterways and therefore could not be used to accurately model the various crossings on the tributaries.

DWOPER (Fread, 1987) is a model with wide applicability to rivers and floodplains of variable roughness and allows for overbank flow onto the floodplain, off channel storage, flow diversions, local head losses at bridge contractions and flow over embankments. It has been converted to metric units and used previously by LMCE, but does not have the potential for graphical output of results and ease of varying model configuration associated with more modern programs.

UNET (HEC, 1993) is a recently developed network model with all of the features required for the study and has been adopted by the US Army Corps of Engineers as standard software for one dimensional hydraulic analysis. Unfortunately, it has not been converted to metric units. As the conversion process would have delayed completion of the study, it was not considered further.

MIKE 11 is a one-dimensional hydraulic modelling package develop by the Danish Hydraulic Institute which has seen widespread application in Australia in recent years. It contains all of the features required for the analysis and is available to all of the potential model users (although a considerable cost is involved). For reasons of availability and acceptance it has been adopted for the present study.

4.3 Hydraulic Modelling - Murrurundi and Blandford

4.3.1 Model Structure

A schematic layout of the Pages River model is shown on Figures 4.1 and 4.1a. It has some 29 sections along 12.25 km of the main river and includes the important tributaries in the Murrurundi and Blandford areas. 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.

Cross-sections of the channel and floodplain were obtained from a variety of sources. A survey of Murrurundi was undertaken by PWD in the late 1970's for the design of the town's sewerage system. The survey was presented on 1:1000 scale plans at 1 m contours and included numerous spot levels around the top of bank and in the overbank areas. The survey covered all of the developed part of town and allowed a detailed representation of overflow paths from the Pages River as well as defining the various tributary streams and local gullies.

Council, advised that the survey generally represented contemporary conditions in the study area although there had been some deposition of shingle upstream of Arnolds Bridge on the recession of the January 1996 flood. This section which is about 200 m upstream of the bridge was re-surveyed, along with several additional sections upstream and downstream of the township which are outside the extent of the sewerage survey. Field survey was also undertaken on Halls Creek, Unnamed Gully, Cohens Gully and a stormwater drain running along the southern (upstream) side of the railway, which discharges into Halls Creek near the Haydon Street bridge. This stormwater drain, denoted the Victoria Street Gully, is well above the zone of influence of river flooding and was not incorporated in the model. A review of its hydraulic behaviour is given in Section 7.

The RTA supplied a survey of the New England Highway and the Pages River channel in the vicinity of Benhams Bridge, along with a general arrangement of the new bridge. This is a four span structure which replaced a previous three span bridge in 1994.

A detailed cross-sectional survey was carried out by the project surveyors in the Blandford area, where available topographic mapping is limited to a 1:25000 scale map with 10 m contour intervals. The survey covered the channels and floodplains of the Pages River and Warlands Creek. The New England Highway bridge at Blandford and Barsham Bridge over Warlands Creek were also surveyed and their details included in the model.

The field surveys and the model results described in this report for the Pages River are given to Australian Height Datum (AHD). The sewer survey adopted the Murrurundi Sewer Datum (MSD). The connection between these two datums is:

$$\text{Levels to AHD} = \text{Levels to MSD} - 0.1 \text{ m}$$

4.3.2 Boundary Conditions

Discharge hydrographs derived from RORB provided the boundary conditions at the upstream end of the model. Lateral inflow hydrographs were added at various locations to account for runoff from the sub-catchments between Murrurundi and Blandford. In all, a total of 11 hydrographs was applied to the model for each flood event.

The downstream boundary was at cross-section 12.25 km of Figure 4.1a, and comprised a rating curve which was computed using a slope-area approach. Sensitivity analysis was undertaken to confirm that errors in the rating curve did not influence modelling results at Blandford.

The model could in the future be extended a further 2.5 km to the Blandford gauging station if design flood information is required in downstream areas. Additional cross sectional survey would be required, and it would be necessary to confirm the accuracy of the DLWC's rating curve which would then become the downstream boundary condition for design runs, as gaugings have only been carried out to 2.5 m on the gauge. The runoff contribution from Scotts Creek which joins the Pages River about 400 m upstream of the gauging station would need to be included as a lateral inflow to the model.

4.3.3 Model Calibration January 1996 - General

Calibration of the model involved the adjustment of model parameters within physically realistic limits, until agreement was achieved between computed and observed flood behaviour.

The main parameter to be adjusted at each section was the hydraulic roughness, represented by the Manning n coefficient. In most cases relative resistance values were used to take into account the variation in roughness across the total cross section. Where there was floodplain storage and only part of the cross section would be effective for conveying flow, the part of the cross section representative of the storage area was given an arbitrarily high value of relative roughness. A similar situation occurred in urban areas where buildings and fences obstructed the passage of flows. Assessment of areas to be treated in this manner was made on the basis of the local geometry and an understanding of flow patterns, obtained from the reconnaissance investigation of Appendix B (Bush, 1996).

Calibration consisted of adjusting these model features so as to obtain coincidence with the observed peak levels along the modelled reach. Flood marks identified in the flood reconnaissance study were levelled and evaluated for consistency. Levels considered representative of main stream flooding on the Pages River at Murrurundi in January 1996 are shown on the modelled water surface profile of Figure 4.2. Corresponding flood levels on the Pages River and Warlands Creek in the Blandford area are shown on Figures 4.2a and 4.2b.

Calibration was relatively straightforward on the Pages River. The Blandford stream gauge provided the data to allow verification of the modelled travel time of the floodwave as well as its attenuation (although the model terminated a short distance upstream of the gauge). The flood for which quantitative level data were available (January and October 1996) were recent, so that a contemporary model of the floodplain was readily constructed. The floods were also of sufficient magnitude to result in considerable flow on the floodplain and so allow a good assessment to be made of hydraulic roughness and flow patterns representative of the major design flood events. In any sections of the floodplain where few recorded flood levels exist, roughness values were assessed on the basis of experience and by comparison with those adjacent floodplain areas which were calibrated from recorded data. As the model was well calibrated from those data, there was no reason to expect that it would not produce good results in those intervening areas where little information was available.

4.3.4 Model Calibration January 1996 - Results

Modelled peak flows in the Murrurundi area are shown on Figure 4.3 and in the Blandford region on Figure 4.3a. These diagrams show the peak flows in the various arms of the model. It is to be noted that these peaks may not necessarily occur at the same time.

At the upstream end of the model, the peak inflow as derived from the RORB modelling of Section 3.4 was 400 m^3/s and occurred at 09:30 hours on 25 January 1996. At the downstream end the routed peak flow was 870 m^3/s occurring at 11:00 hours. The 90 minutes taken for the floodwave to travel through the modelled reach is equivalent to a wave celerity of 7.3 mins/km. The recorded peak discharge at the Blandford gauging station, about 2.5 km downstream of the model boundary was 1030 m^3/s and occurred at 11:38 hours according to SES records. If the hydraulic model had been continued to the gauge and the wave celerity maintained, then the modelled peak would have arrived at about 11:20 hours, which is in reasonably close agreement with the SES records for the actual flood peak.

Flows from Scotts Creek enter the Pages River just upstream of the gauge. This tributary is ungauged. However, if the Scotts Creek contribution of 150 m^3/s, as estimated by RORB, is added to the 870 m^3/s peak discharge from the hydraulic model, the estimated peak flow downstream of the confluence is 1120 m^3/s, which is very close to the 1030 m^3/s recorded peak at Blandford.

Flooding in the Murrurundi area (Figure 4.3):

Contributions to the peak discharge in the Pages River from the two left bank tributaries Unnamed Gully and Cohens Gully are quite small amounting to 25 m^3/s in a total discharge of 400 m^3/s. The right bank tributary, Halls Creek, is somewhat larger contributing about 50 m^3/s to the flow. It joins the Pages River just upstream of Arnolds Bridge.

Flow velocities in the reach extending from the upstream boundary to the Mount Street pedestrian bridge are in the range 2.4 - 3.5 m/s and average about 3 m/s. In the backwater upstream of Arnolds Bridge, the velocity reduces to 1.6 m/s. The reduction in velocity in the bridge backwater is responsible for the deposition of shingle observed after the flood recession.

About 380 m^3/s are conveyed through Arnolds Bridge waterway which flows under low flow conditions (i.e. water level below underside of deck) at the peak of the flood. This model result agrees with observations indicating that there was about 150 mm of freeboard on the underside of the deck at the flood peak. However, the approach road is submerged, with about 45 m^3/s being conveyed over the New England Highway into the drainage line on the northern side of the road, which is denoted "Runner 1" in the model. This flow eventually rejoins the left bank of the river downstream of the bridge.

About 30 m^3/s surcharges the low levee on the right bank of the Pages River and is conveyed across Adelaide Street and into Mayne Street. This flow path is modelled as "Runner 2". The modelled peak velocity reaches 3.8 m/s near the intersection with Adelaide Street and the peak depth is 370 mm. These values agree with observations of the fast moving flow recorded on video near the peak of the flood. The highest velocities occur in the reach between Adelaide and Victoria Streets. Further downstream, flows are directed by the prevailing grade back to the river.

Flooding in the Blandford area (Figure 4.3a):

The peak discharge at Benhams Bridge is 530 m^3/s which is considerably in excess of the bridge capacity, resulting in surcharge flows crossing the New England Highway. Due to local topographic features, which are not presently incorporated in the model, some of the flow actually travelled in an easterly direction towards Blandford before being deflected across the highway by local access roads which are set above the general level of the floodplain. In the model, the surcharging flow is assumed to cross the highway and rejoin the Pages River in the immediate vicinity of the bridge. Additional survey would be required to more closely model local flooding patterns in the vicinity of Benhams Bridge. This is not required for the present study, as flooding at Benhams Bridge does not significantly influence conditions at Murrurundi and Blandford.

Flooding in the Blandford area mainly arises from breakouts of the Warlands Creek channel which are conveyed along the right overbank in local flood runners. Some of this flow also crosses the New England Highway and flows through the school grounds before joining the Pages River upstream of Blandford Bridge.

The peak inflow to the model on Warlands Creek amounts to 400 m^3/s, of which 270 m^3/s passes through the Barsham bridge waterway and the balance is conveyed across that road via "Runner 3" of the model. About 40 m^3/s leaves Warlands Creek downstream of the bridge via "Runner 4". About 40 m^3/s of the combined flow in these runners passes over the New England Highway and through the school grounds on the southern side of the highway. The modelled depth of flow over the highway is 410 mm. A recorded flood mark at the school indicated that the actual depth of overland flow heading towards the Pages River was about 400 mm.

Flows in the flood runners travel along the right overbank of Warlands Creek and rejoin that stream a short distance downstream of Barsham Bridge. Several side weirs were incorporated in the model to allow for the transfer of flow from Warlands Creek to the Pages River. They are located in the developed area of the township in the vicinity of the Post Office. These weirs did not function during the modelling of the January 1996 flood, which indicates that this area remained flood free. This is understood to have been the case as most of Blandford township is on relatively high ground.

Model results suggest that downstream of the Blandford Bridge, there was a transfer of water across the floodplain, firstly from the Pages River to Warlands Creek (100 m^3/s), and then followed by a reversal of flow (20 m^3/s) about one hour later. Whether this flow reversal actually occurred or is an artefact of the model cannot be determined due to lack of data.

4.3.5 Model Calibration October 1996

Flood marks identified in the Murrurundi area during a site inspection carried out shortly after the flood event were levelled by the project surveyors and were evaluated for reliability and consistency. In the Blandford area, the flood was less severe and was mainly confined to the channel and its immediate vicinity. No flood marks were identified on Warlands Creek and the lower reaches of the modelled section of the Pages River.

The flows derived by RORB modelling (Section 3.4.2.4) were applied to the hydraulic model.

Initial model runs were undertaken with the same roughness values derived for January 1996. However, the resulting water surface profile in Murrurundi appeared to be higher than that actually experienced. It appeared that the earlier flood had cleared the channel to some extent and that vegetation had not regrown over the winter of 1996, prior to the occurrence of the October flood. Runs were undertaken with slightly reduced roughness values. A value of n equal to 0.03 in the main channel gave the best correspondence between computed and historic flood levels. The resulting water surface profiles are shown on Figures 4.4 to 4.4b.

Flows on the Pages River in the vicinity of Murrurundi are about 90 per cent of those experienced during the January 1996 event. They increased from 350 m^3/s at the upstream end of the model to 480 m^3/s at Benhams Bridge. Flow on Warlands Creek was only around one third of the January 1996 value, amounting to 140 m^3/s at the junction with the Pages River downstream of Blandford. The combined peak flow in the Pages River below the junction was about 610 m^3/s compared with 870 m^3/s in January 1996.

In Murrurundi, about 16 m^3/s flowed through the breach in the levee and was conveyed along Mayne Street. This flow was about half the value experienced in January 1996 and it is understood that no significant above-floor inundation was experienced. The shop in Adelaide Street which was in the path of floodwaters surcharging the levee had been removed after the January 1996 flood and the allotment was cleared. All of the floodwaters were conveyed through the Arnolds Bridge apart from the small loss through the breached levee. No significant surcharge of the New England Highway on the approaches to the bridge were experienced. Peak water level upstream of the bridge waterway as modelled was about 100 mm lower than in January 1996.

4.3.6 Summary

The MIKE 11 hydraulic model reproduced the observed pattern of flooding for the January 1996 flood along the Pages River and Warlands Creek. Modelling of breakouts from the Pages River and flow through bridge constrictions were closely simulated. Flows generated from the RORB model which were used as input to MIKE 11 appear to be close to actual flows in the study area.

Best modelling results were achieved with the following set of roughness values:

Main channel	0.045
Grassed floodplain	0.060
Roads	0.025
Overbank areas with dense stands of trees	0.18
Built up areas where the flow is blocked by buildings, fences and the like.	0.45

For the October 1996 flood, best results were achieved in the Murrurundi area with a roughness value in the main channel equal to 0.030. This reduced roughness may have been due to clearing out of the channel by the earlier flood. No flood marks were available in the Blandford area. The October 1996 flood was not a major event in the lower reaches of the catchment.

The values found to apply in January 1996 have been adopted as best estimates in the hydraulic modelling of the design floods described in Chapter 6.

4.4 Hydraulic Modelling - Willow Tree

4.4.1 Model Structure

Borambil Creek was modelled over a 4.5 km length extending from approximately 1 km upstream of Hams Bridge to a location 300 m downstream of the Sports Ground. Figure 4.5 shows a schematic layout of the MIKE 11 model. It includes the main stream which is denoted the "Borambil" branch and several flood runners. The waterway opening at Hams Bridge was modelled as 4 irregular openings and Merriwa Road at the bridge was modelled as a weir. The four 1000 x 660 box culverts and the 2 x 900 RCP's at the Oaklyn Road causeway (model section 3.10) convey low flows under the roadway. The capacity of the culverts is small compared to the magnitude of design flood flows and therefore the culverts were not modelled.

A branch named "School" was set up to model flows which surcharge the right bank of Borambil Creek upstream of Hams Bridge and flow towards the school. Merriwa Road acts as the control over flood levels along this branch. To accurately model flow, the road was divided into 3 weirs (weirs 8 to 10). From preliminary runs of the model it was found that during the 1996 flood event the diversion bank to the south of the school was overtopped. Therefore it was incorporated in the model as a flow path.

Flows which surcharge Merriwa Road from the "School" branch enter a branch named "Runner 1". This branch models the wide right floodplain of Borambil Creek downstream of Hams Bridge and extends 1.75 km to the Oaklyn Road causeway. The branch also accepts flows which overtop or outflank the diversion bank at the school and cross Merriwa Road as overland flow.

At the causeway on Borambil Creek the right floodplain narrows while the left floodplain flattens. "Runner 2" models flows which surcharge the left bank of the creek upstream of the causeway in the vicinity of the recently formed cut-off. These flows traverse a short length of floodplain before re-entering the creek 650 m downstream, adjacent to the Sports Ground.

Side weirs were incorporated in both "Runner 1" and "Runner 2" to model surcharging of both the left and right banks of Borambil Creek.

4.4.2 Boundary Conditions

Discharge hydrographs derived from RORB provided the boundary conditions at the upstream end of the model. Three hydrographs comprising Borambil Creek and lateral flows were applied to the model for each flood event.

The downstream boundary condition comprises a rating curve which was computed using the slope-area approach. The HEC-RAS computer program was used to develop the rating curve.

4.4.3 Available Data

Available topographic mapping of the Willow Tree area is limited to 1:25000 scale plans with 10 m contour spacing. Cross sectional survey of Borambil Creek and the adjacent floodplains was undertaken by the project surveyors. However, in the absence of any reliable AHD benchmarks in the township, a local datum was adopted for the survey which is approximately 100 m above AHD. That is, 100 m should be added to the levels given in this study to give an estimate of their AHD equivalent levels.

At State Survey Mark (SSM) 77535 which is situated on the New England Highway near Ch 2.85 km of the MIKE 11 model, the connection with the local datum is:

SSM77535 = 323.53 m Local Datum

The AHD for this mark is not presently known. It is noted as "disturbed" on the State Survey Control Branch's database and consequently, cannot be used to convert the local datum to AHD.

Little flood data is available for Borambil Creek in the vicinity of the township of Willow Tree. For the January 1996 flood event 10 flood marks were noted by Bush (1996). Of these only 3 are representative of main stream flooding. The first is situated at the downstream end of the hydraulic model and was used to confirm the rating curve developed using HEC-RAS. It was found that after running the 1996 flood hydrographs through the MIKE-11 model the flood level at the downstream end of the model was close to that levelled. This would also tend to indicate that the RORB model correctly estimated the peak flood flow. The remaining 2 flood marks are located on the Borambil Creek floodplain downstream of the Oaklyn Road causeway. On the right floodplain one mark was located at the Bowling Club and another at the Sports Ground. On the left floodplain a reported mark was levelled downstream of the "Meadow Bank" homestead on Oaklyn Road. However, this mark appeared to be well above the extent of inundation and was discarded.

A floodmark was also noted in the school grounds at Merriwa Road and was used to give an indication of the depth of inundation experienced during the flood.

The remaining flood marks are located on the eastern side of the railway and represent levels on the small tributaries which drain to Borambil Creek from the east.

4.4.4 Model Calibration

As for the Pages River case, calibration of the model involved the adjustment of model parameters such as Manning n coefficient, within physically realistic limits, until agreement was achieved between computed and observed flood behaviour for the January 1996 flood.

Figure 4.6 shows the reach of Borambil Creek modelled with the computed 1996 flood profile plotted. Recorded flood marks are also shown. Figure 4.7 shows the modelled distribution of peak flow.

The 1996 discharge hydrograph produced by RORB had a peak of 900 m^3/s at the upstream end of the hydraulic model. At Hams Bridge approximately 500 m^3/s is conveyed through the waterway opening. The remainder of the flow, around 400 m^3/s, surcharges the right bank of the creek and flows northward towards the depression to the south of the school. Approximately 350 m^3/s surcharges the roadway at model weirs 8, 9 and 10, while 50 m^3/s overtops the diversion bank at the school and then flows over the Merriwa Road. The computed peak flood levels on each side of the diversion bank differ by around 1 m. This difference in levels probably occurred due to the relatively sharp rise and fall in flood flows and lack of time for flood levels to equalise across the bank. The computed flood level inside the school grounds is about 150 mm below the verandah level of the pre school building which approximates the peak flood level.

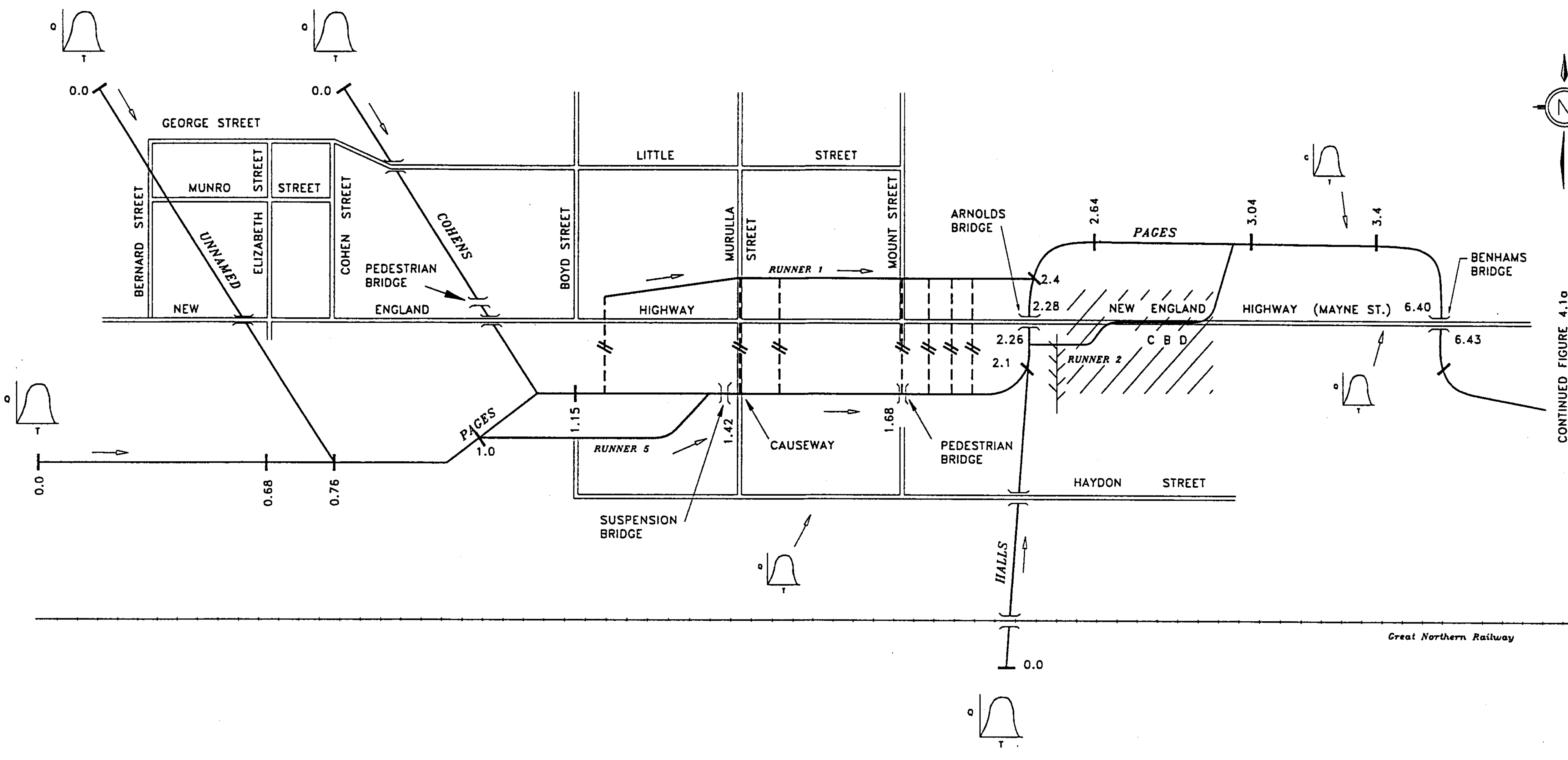
Downstream of Hams Bridge the high ground to the west constrains flows from leaving the left bank of Borambil Creek. Flood flows extend from the creek out over the right floodplain to Merriwa Road. Adjacent to the township of Willow Tree, Borambil Creek changes course and flows closer to the town. The right floodplain narrows upstream of the Oaklyn Road causeway and 220 m^3/s, which is conveyed along Runner 1, returns to Borambil Creek. Surcharging of the left bank occurs in the vicinity of the Oaklyn Road causeway. Flows travel overland along Runner 2 before re-entering the creek adjacent to the Sports Ground. Results from the computer model in the vicinity of the Bowling Club and Sports Ground agree with the extent of flooding observed during the January 1996 event.

The peak discharge at the model outlet amounts to 890 m^3/s, compared with an inflow peak of 900 m^3/s. Local catchment runoff occurs before the arrival of the main flood peak and does not add appreciably to the magnitude of the peak discharge. These results indicate that there is little attenuation of flows resulting from floodplain storage effects.

4.4.5 Summary

Satisfactory results were achieved in calibrating the model to the 1996 event. The model reproduced the observed patterns of flow at Hams Bridge and on the floodplain and it was possible to fit the model to the recorded flood level at the downstream boundary and at the intervening flood marks with realistic values of roughness.

Mannings n values adopted for Borambil Creek and its floodplain were generally 0.045 and 0.06 respectively and these values have been adopted as best estimates in the hydraulic modelling of the design floods described in Chapter 6.



LEGEND

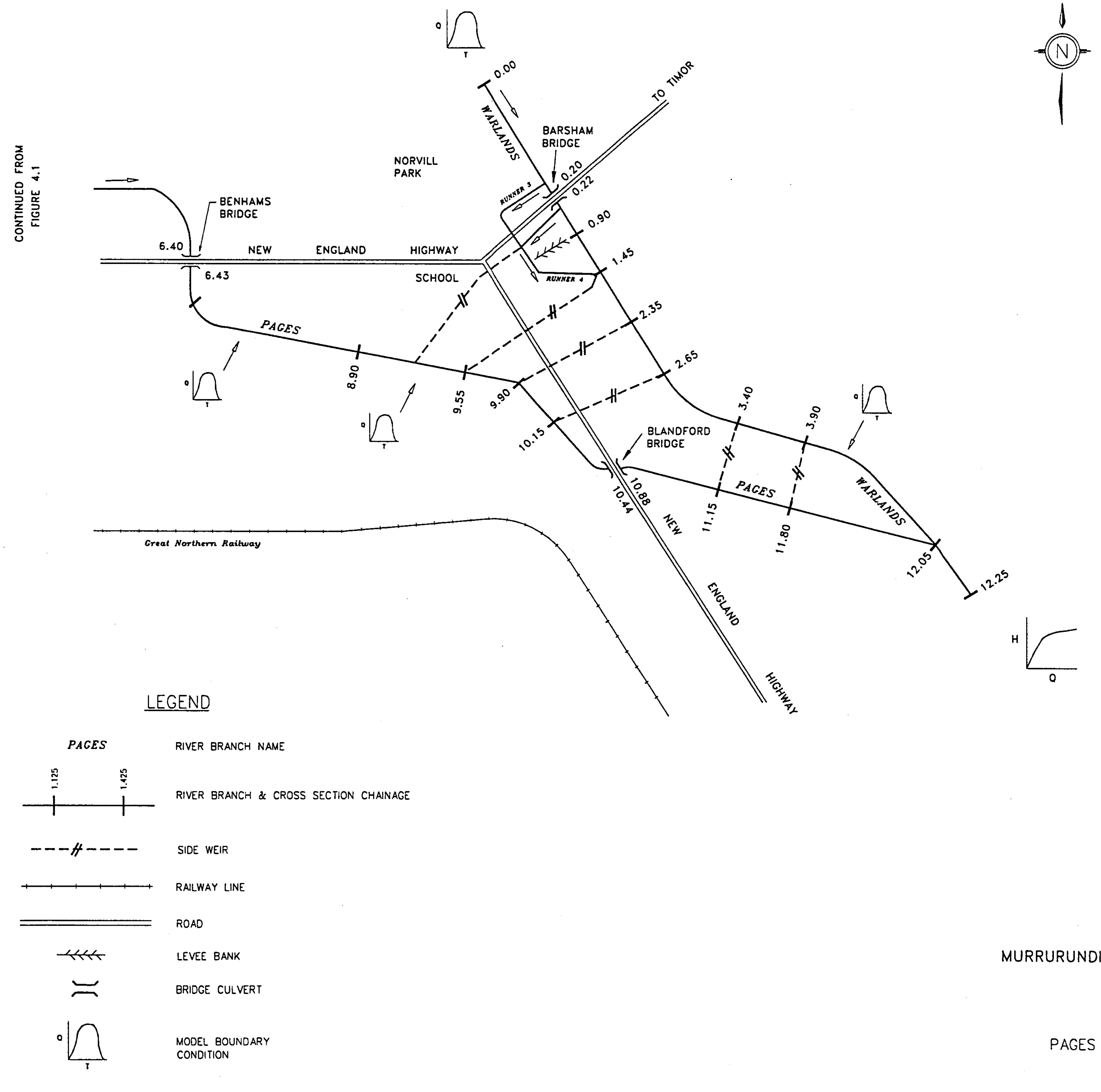
PAGES	RIVER BRANCH NAME
1.125 1.425	RIVER BRANCH & CROSS SECTION CHAINAGE
---//----	SIDE WEIR
	RAILWAY LINE
	ROAD
	LEVEE BANK
	BRIDGE CULVERT
Q T	MODEL BOUNDARY CONDITION

MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 4.1

PAGES RIVER MIKE II HYDRAULIC MODEL
SCHEMATIC LAYOUT

H:\NX476\WORK\FIGURES\FIG4-1.DWG

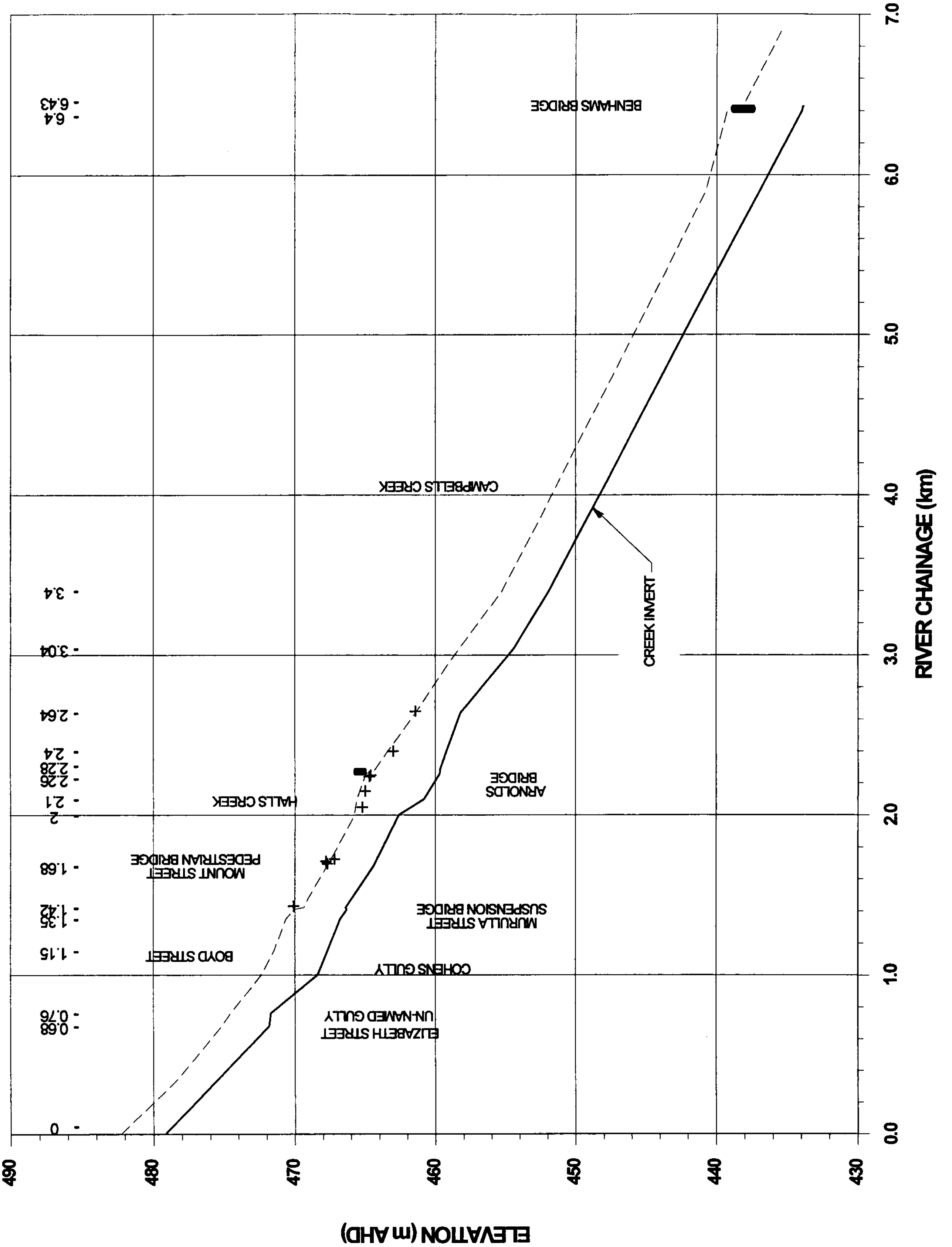


MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 4.1a

PAGES RIVER MIKE II HYDRAULIC MODEL SCHEMATIC LAYOUT

H:\NX476\WORK\FIGURES\FIG4-1.DWG

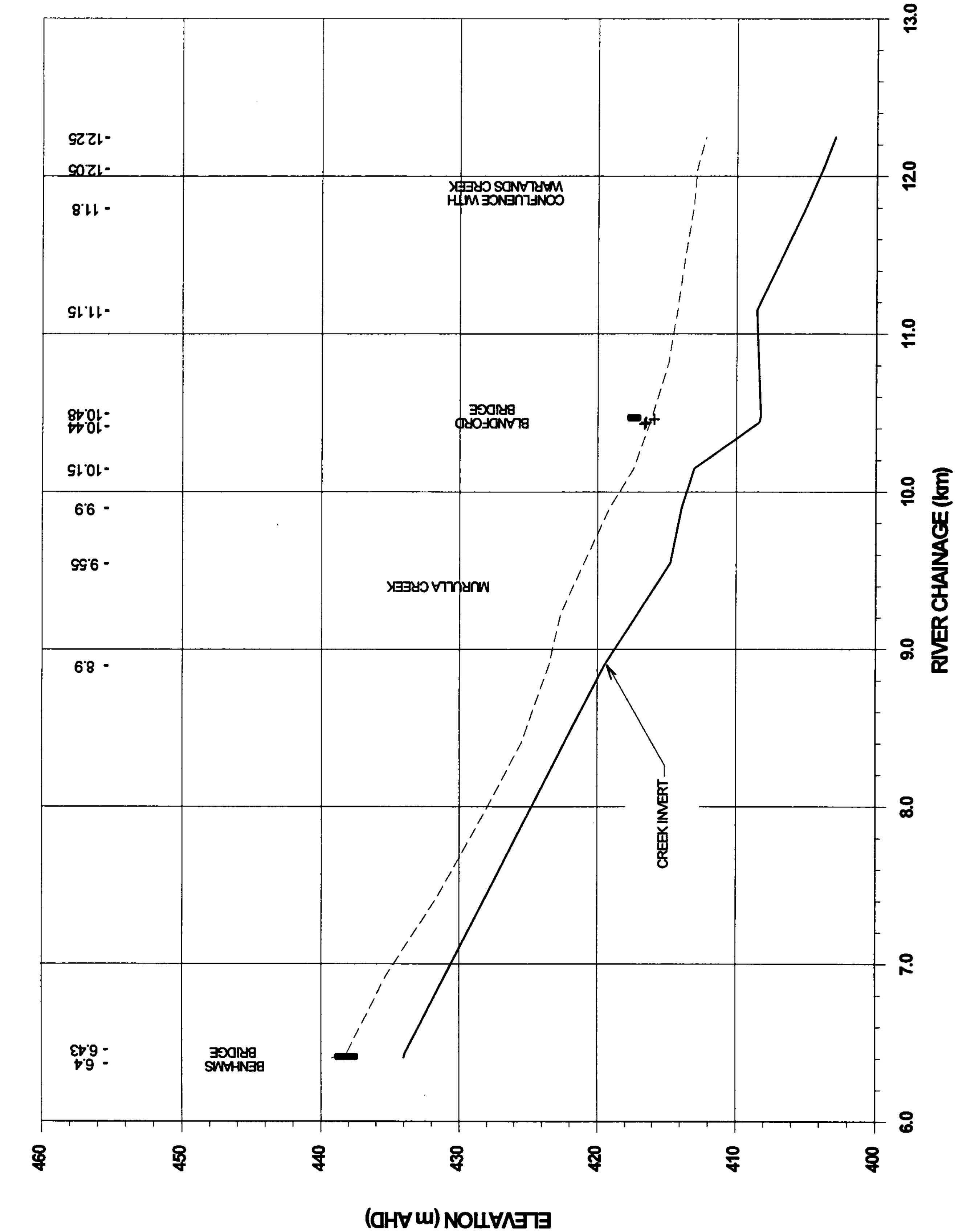


NOTE: -2.64 MIKE 11 SECTION CHAINAGE

MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 4.2

PAGES RIVER
HYDRAULIC MODEL CALIBRATION
WATER SURFACE PROFILE
JANUARY 1996 FLOOD

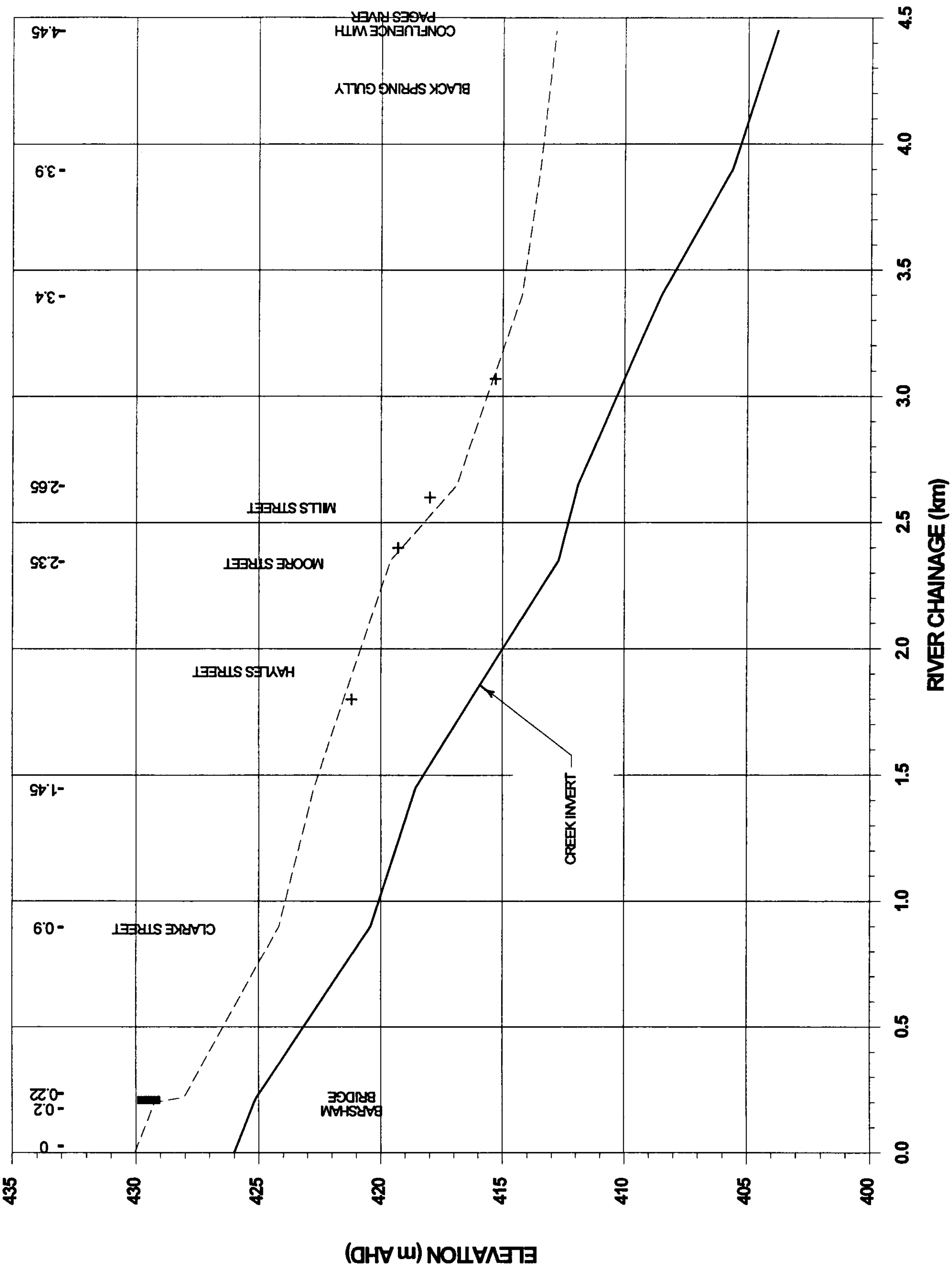


NOTE: - 9.55 MIKE 11 SECTION CHAINAGE

MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 4.2a

PAGES RIVER
HYDRAULIC MODEL CALIBRATION
WATER SURFACE PROFILE
JANUARY 1996 FLOOD

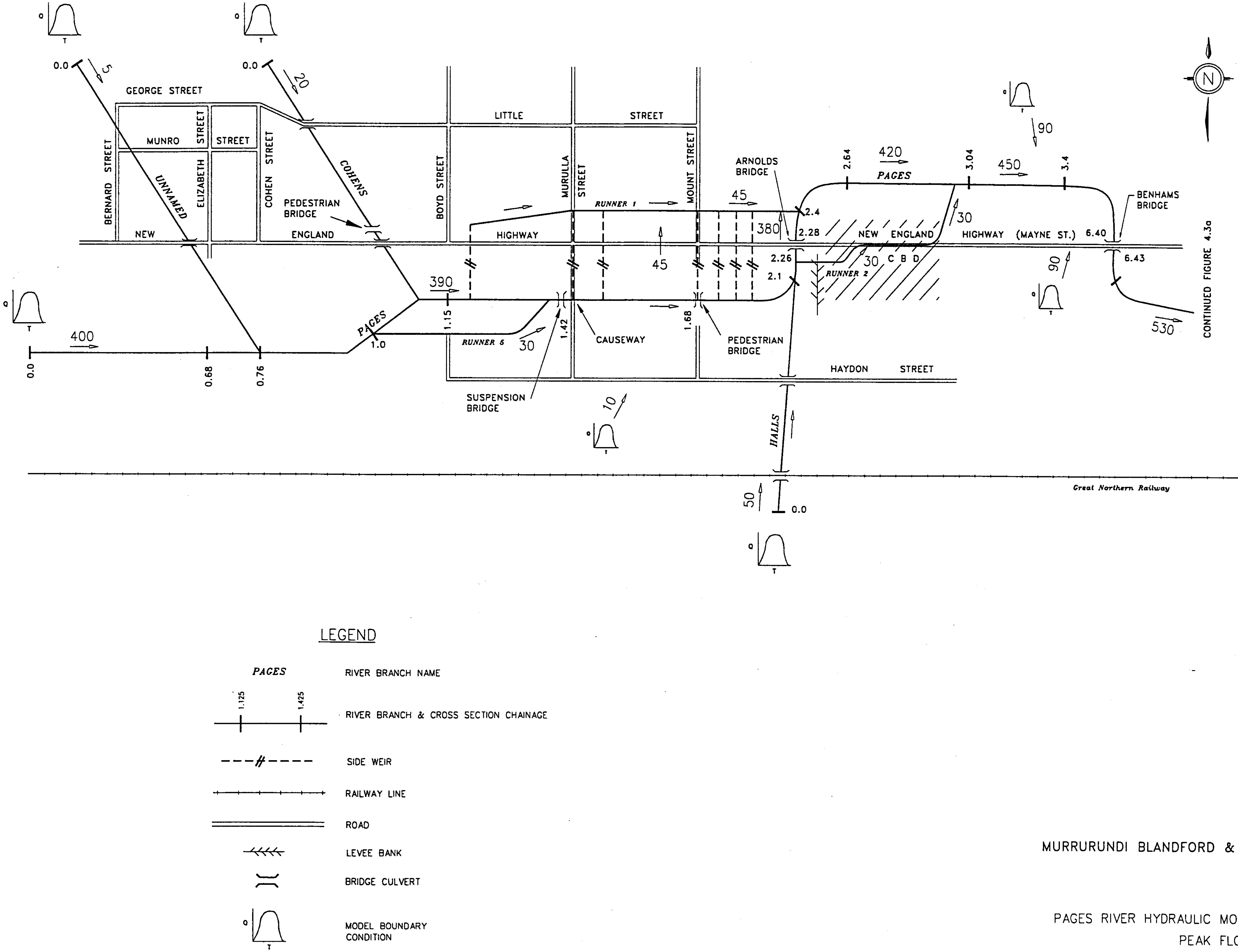


NOTE: - 1.35 MIKE 11 CROSS SECTION CHAINAGE

MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 4.2b

WARLANDS CREEK
HYDRAULIC MODEL CALIBRATION
WATER SURFACE PROFILE
JANUARY 1996 FLOOD

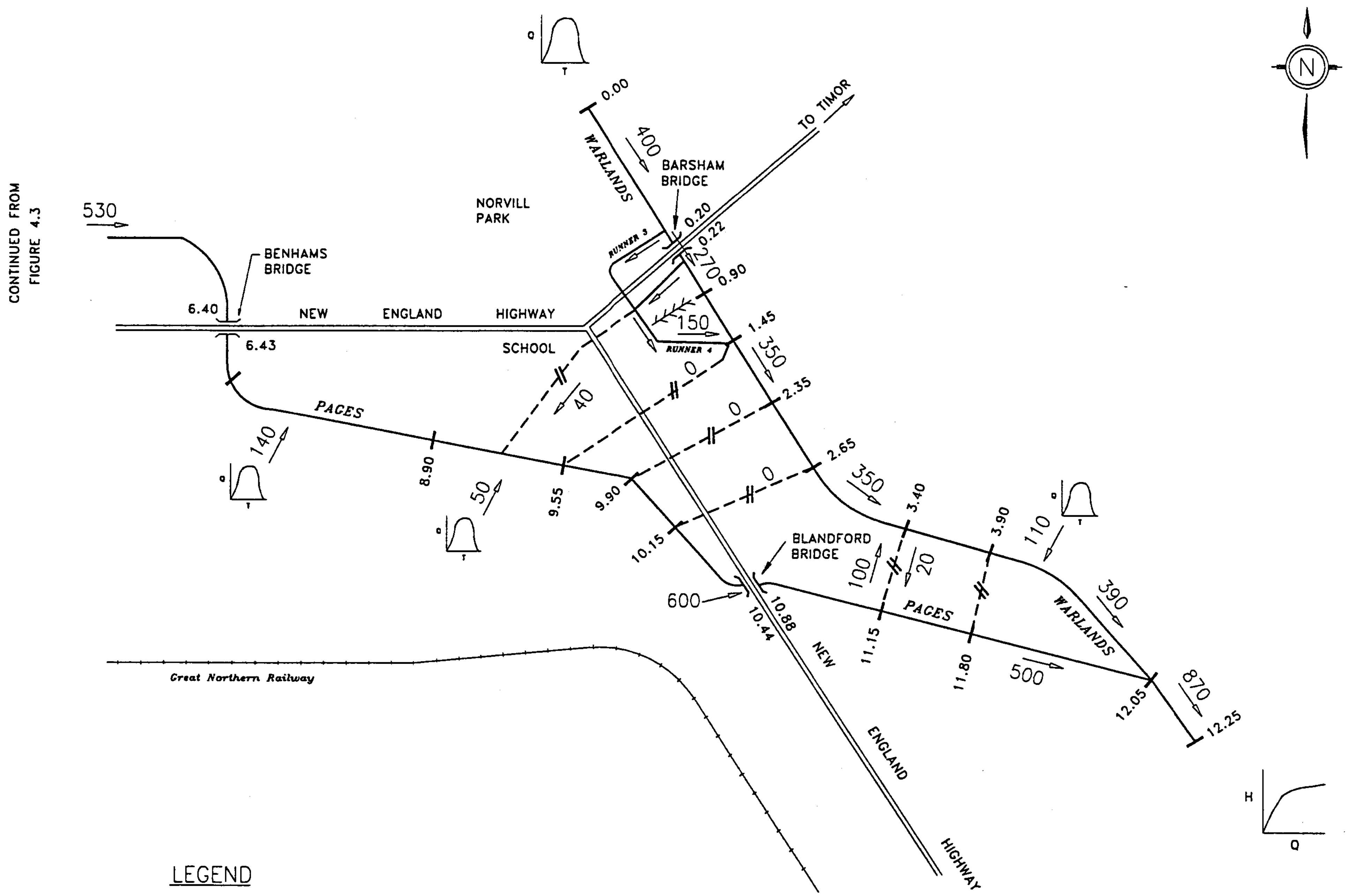


MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 4.3

PAGES RIVER HYDRAULIC MODEL CALIBRATION
PEAK FLOW DISTRIBUTION
JANUARY 1996 FLOOD

H:\NX476\WORK\FIGURES\FIG4-1.DWG



LEGEND

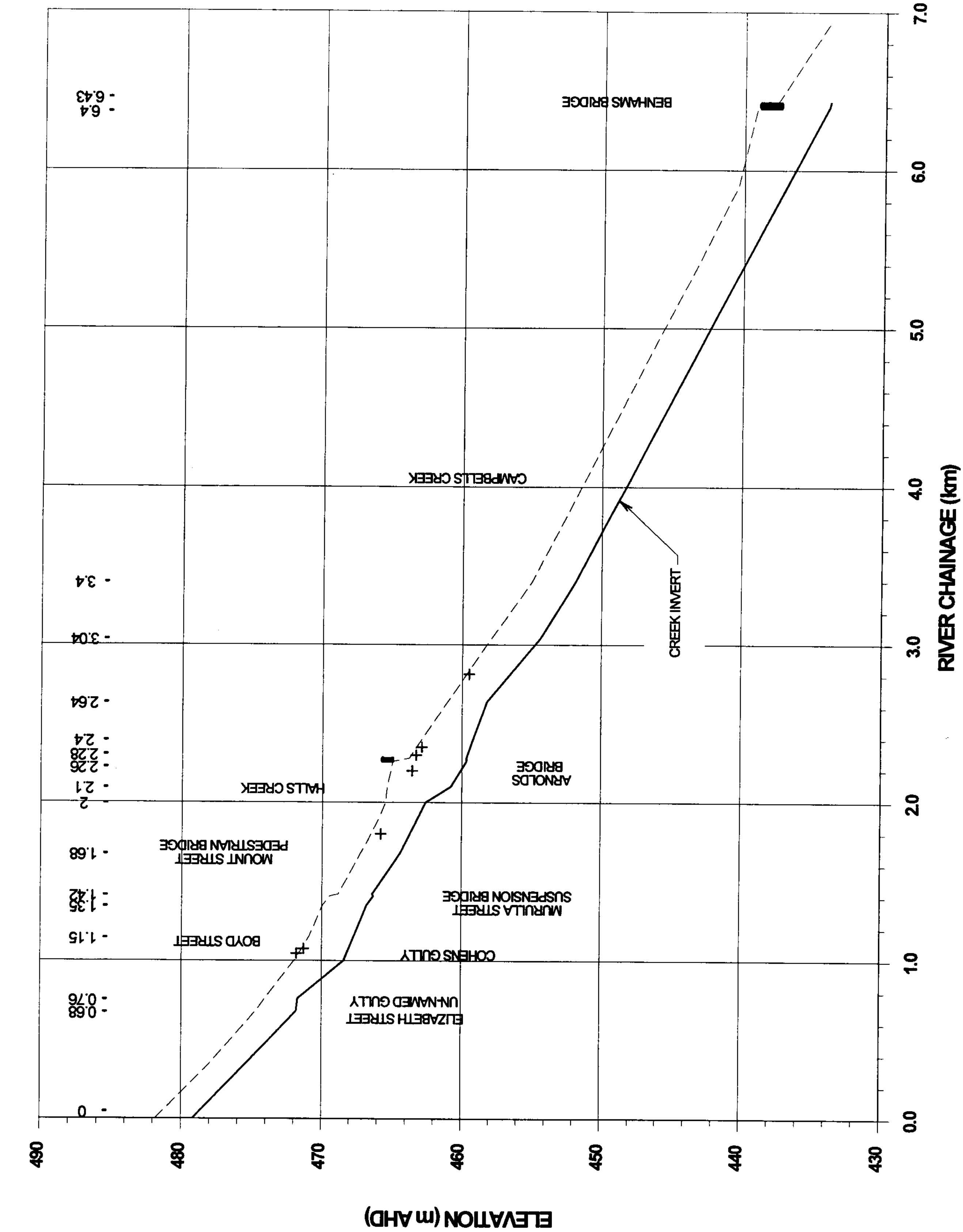
PAGES	RIVER BRANCH NAME
1.125 1.425	RIVER BRANCH & CROSS SECTION CHAINAGE
	SIDE WEIR
	RAILWAY LINE
	ROAD
	LEVEE BANK
	BRIDGE CULVERT
Q T	MODEL BOUNDARY CONDITION

H:\NX476\WORK\FIGURES\FIG4-1.DWG

MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 4.3a

PAGES RIVER HYDRAULIC MODEL CALIBRATION
PEAK FLOW DISTRIBUTION
JANUARY 1996 FLOOD

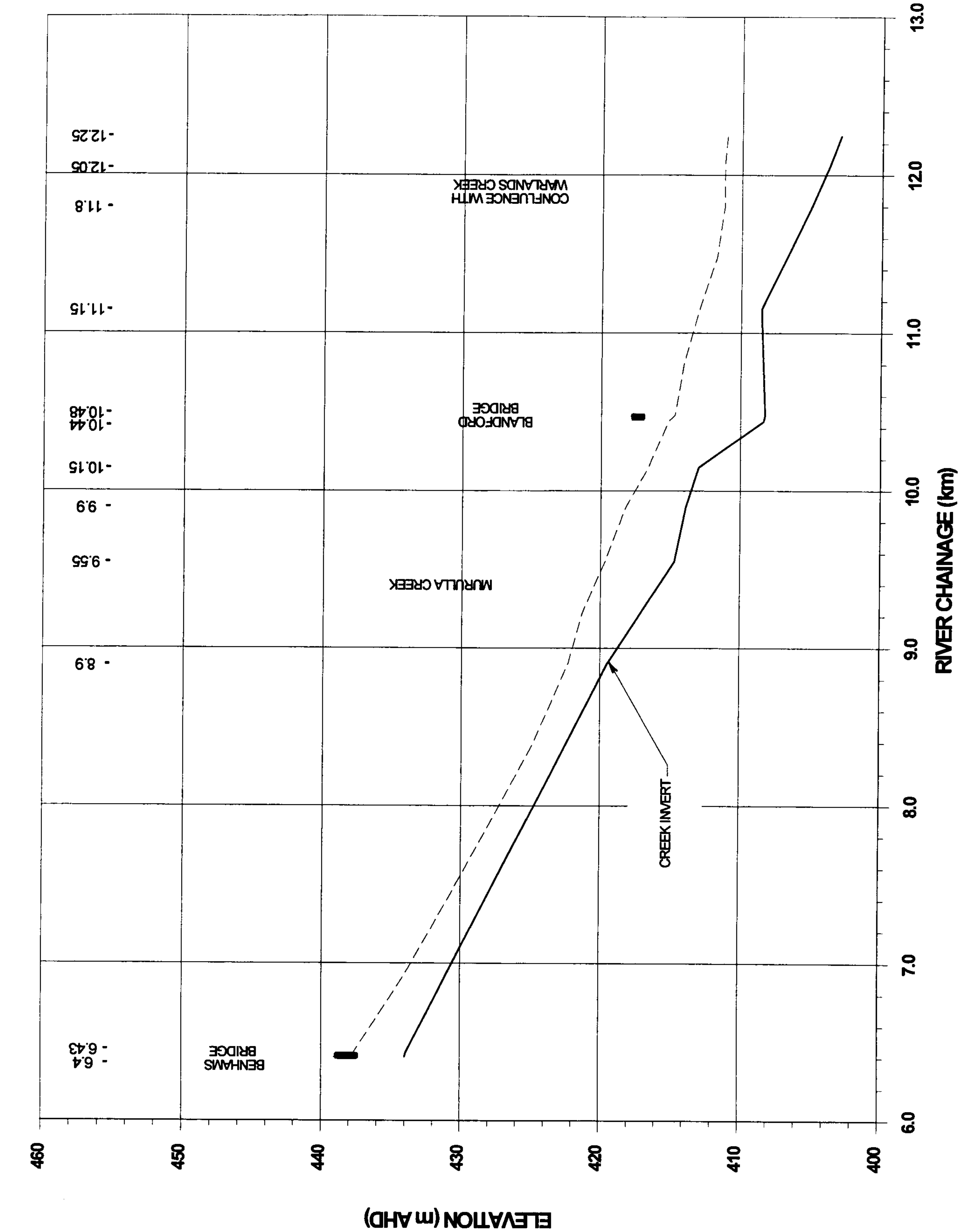


NOTE: - 2.64 MIKE 11 SECTION CHAINAGE

MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 4.4

PAGES RIVER
HYDRAULIC MODEL VERIFICATION
WATER SURFACE PROFILE
OCTOBER 1996 FLOOD

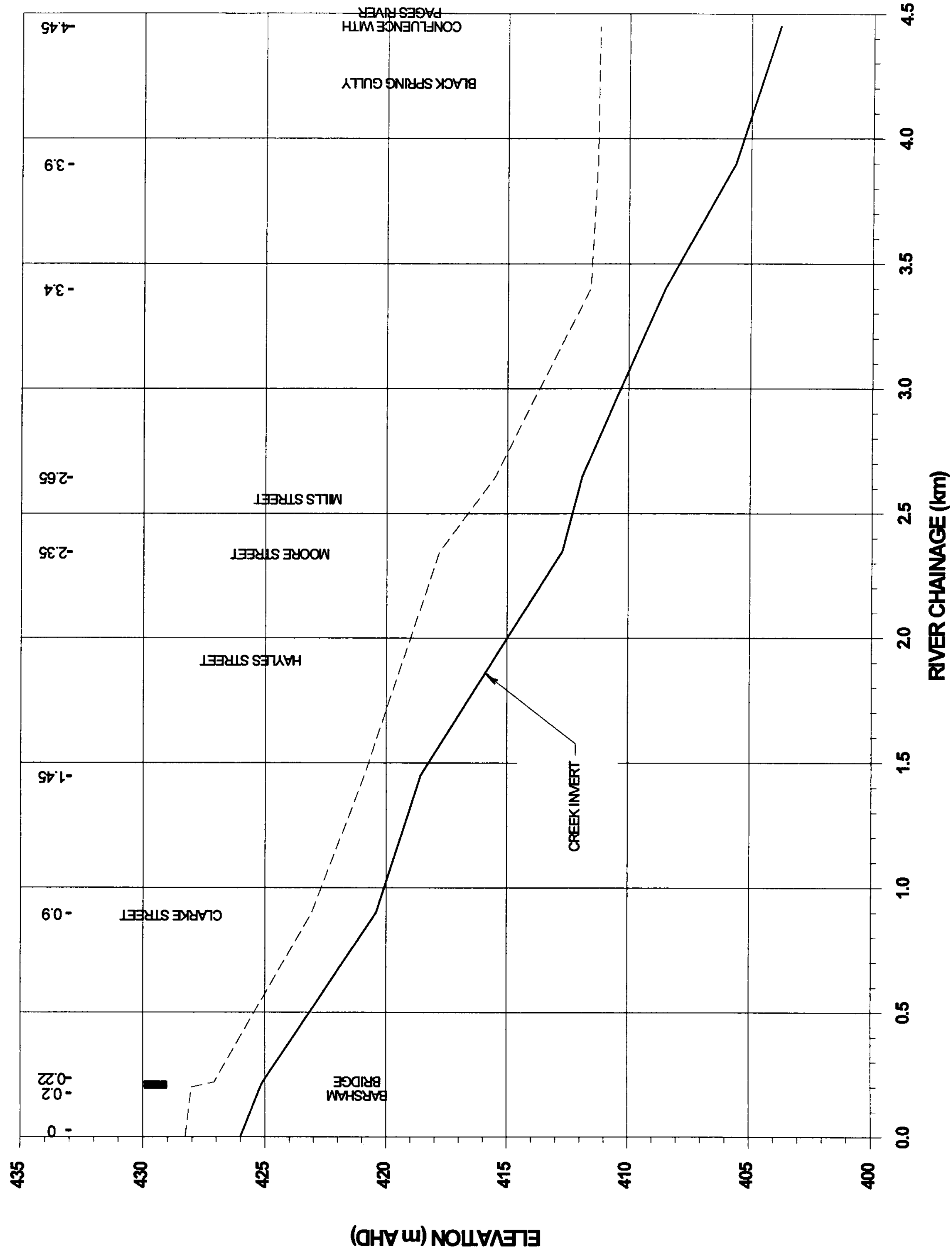


NOTE: -9.55 MIKE 11 SECTION CHAINAGE

MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 4.4a

PAGES RIVER
HYDRAULIC MODEL CALIBRATION
WATER SURFACE PROFILE
OCTOBER 1996 FLOOD

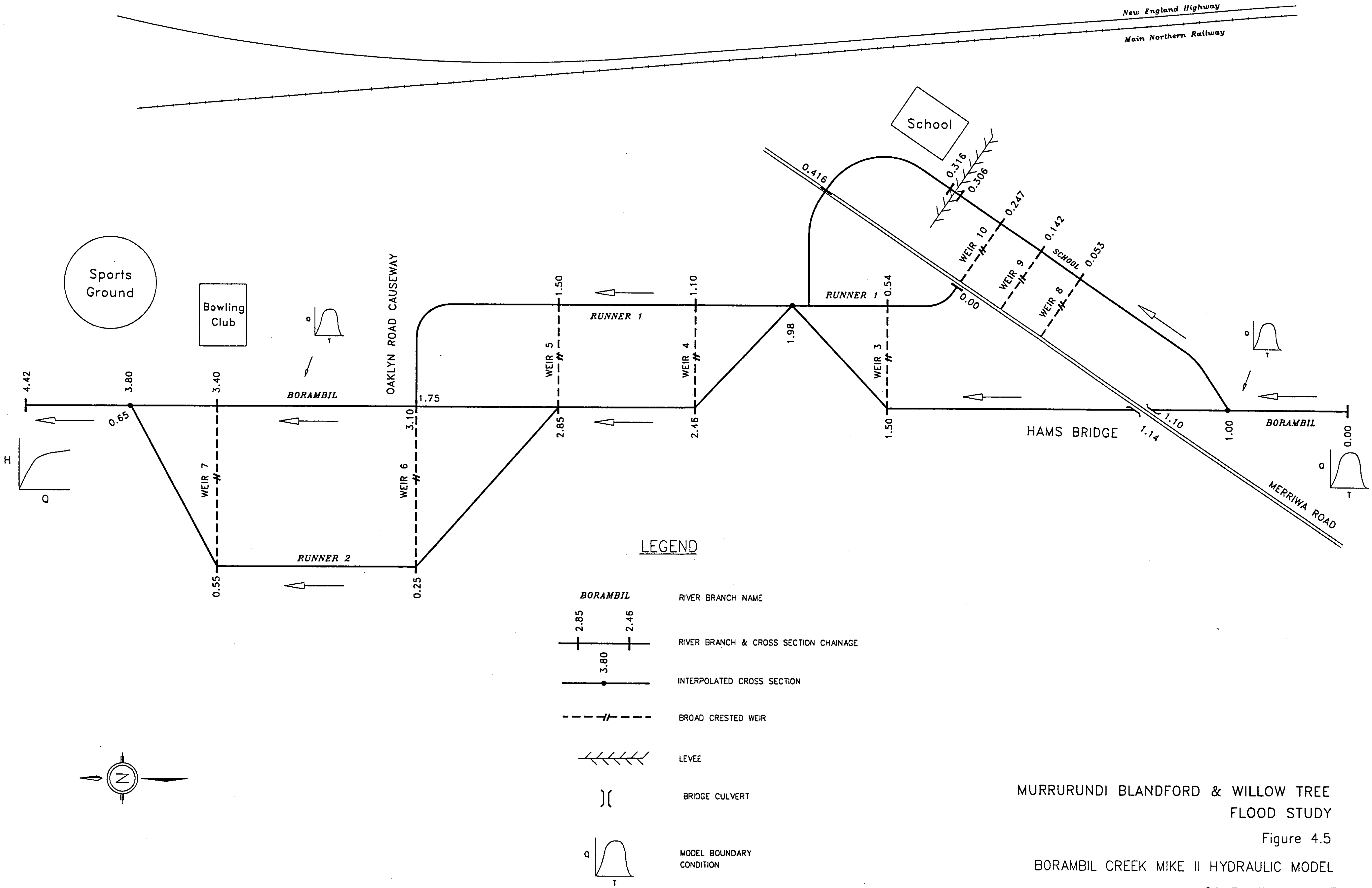


NOTE: - 1.35 MIKE 11 CROSS SECTION CHAINAGE

MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 4.4b

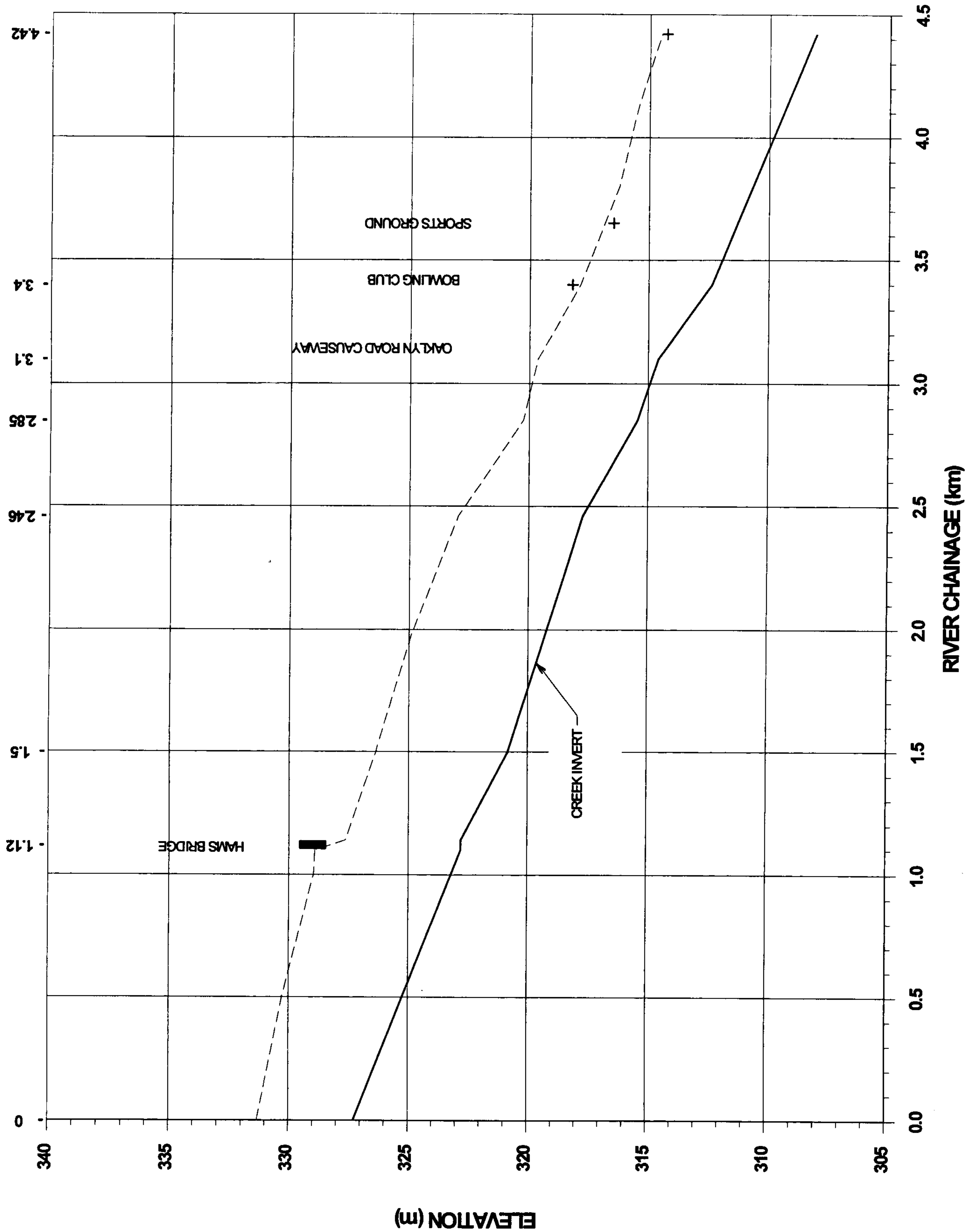
WARLANDS CREEK
HYDRAULIC MODEL CALIBRATION
WATER SURFACE PROFILE
OCTOBER 1996 FLOOD



MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 4.5

BORAMBIL CREEK MIKE II HYDRAULIC MODEL SCHEMATIC LAYOUT

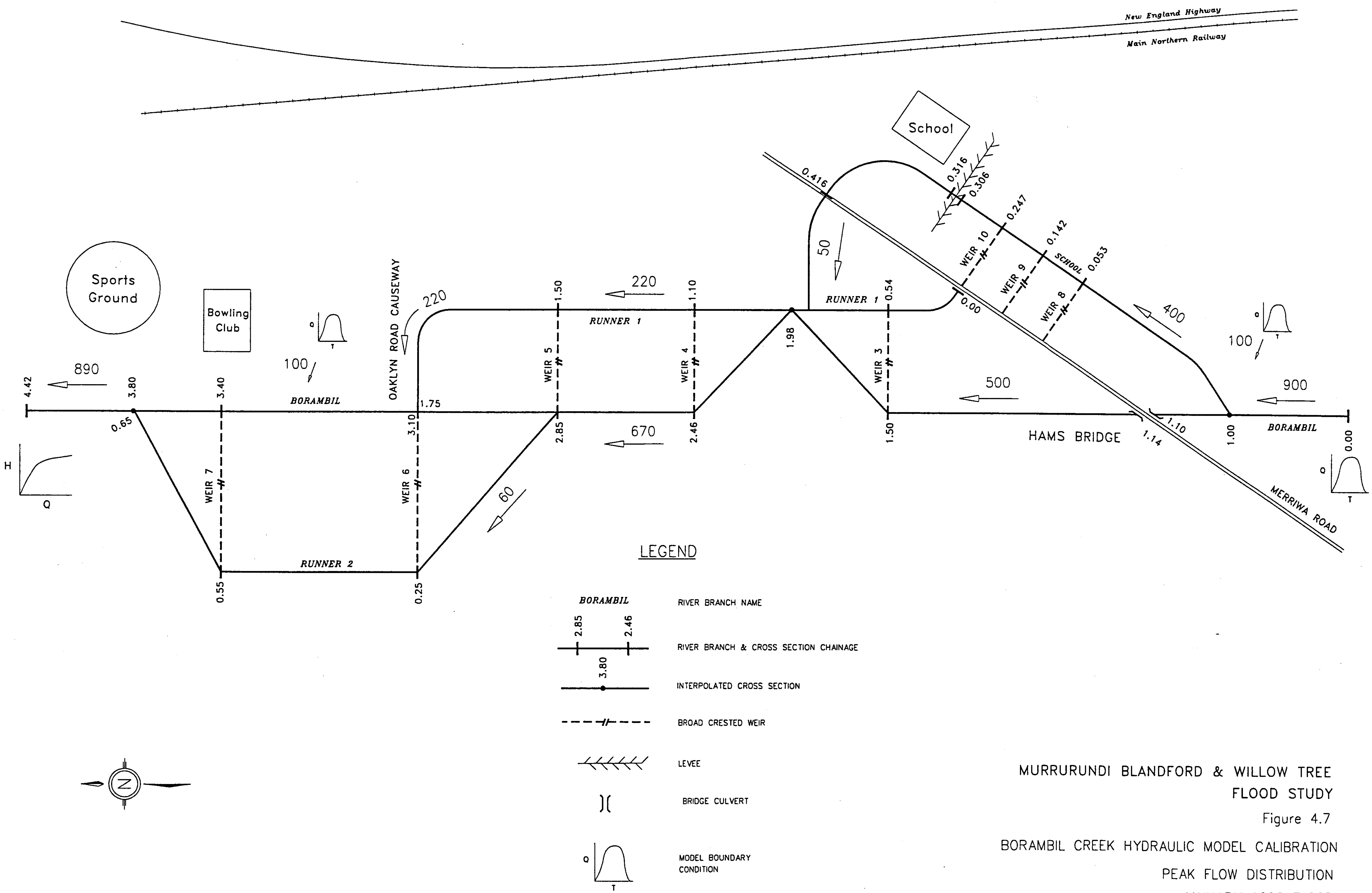


NOTE: - 1.12 MIKE 11 SECTION CHAINAGE

MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 4.6

BORAMBIL CREEK
HYDRAULIC MODEL CALIBRATION
WATER SURFACE PROFILE
JANUARY 1996 FLOOD



MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 4.7

BORAMBIL CREEK HYDRAULIC MODEL CALIBRATION

PEAK FLOW DISTRIBUTION

JANUARY 1996 FLOOD

5. DESIGN FLOOD ESTIMATION

5.1 Design Storms

5.1.1 Rainfall Intensities

Design storms for a range of frequencies up to 100 year ARI were derived using principles given in Chapter 2 of ARR. The procedure adopted was to generate intensity-frequency-duration data over the catchment. The steps involved in this process were:

- Five uniformly spaced points were used for defining the areal distribution of rainfall over the study area. Thiessen weighting was used to determine the area of influence of each point.
- A computer program based on procedures outlined in ARR, calculated the rainfall intensity at each grid point. The intensities derived were for frequencies of 5, 20, 50 and 100 year ARI and durations of 1 to 72 hours.
- For each design frequency and duration, a rainfall depth was calculated at each grid point.
- Finally, rainfall depths at the centroids of each RORB sub-area were estimated using the Thiessen areas. An areal reduction factor was applied to the depths prior to inclusion of the data in the RORB model.

5.1.2 Areal Reduction Factors

The rainfalls derived using the processes outlined in ARR are applicable strictly to a point, however they can be used on areas up to 4 km^2. For larger areas it is not realistic to assume that the same intensity can be maintained over the entire area. A reduction in point values has to be made using an areal reduction factor.

Values for the areal reduction factor (ARF) are presented in Figure 2.6 of ARR. They are based on US data, and cover storm durations to a maximum of 24 hours and catchment areas to a maximum of 1000 km^2. ARR recommends the use of these ARFs for recurrence intervals up to 100 years. The data from which these curves were derived were recorded in frontal rain, rain from decaying storms of tropical origin and from local thunderstorms. From an interpolation of the data to a catchment of 300 km^2, the ARF for a 1 hour storm is around 0.78, increasing to 0.95 for a 24 hour event.

Results from a recent investigation carried out in Australia by Omolayo, 1995 give ARFs for a 24 hour duration and frequencies between 25 and 1000 years ARI for catchments in the Sydney area up to 1000 km^2 in area. They are similar to the values given in ARR. For a catchment of 300 km^2 the ARF for a 24 hour 100 year ARI storm is about 0.94. There is a tendency for ARFs to reduce with increasing ARI.

There is a concern in some sections of the hydrological community in Australia that the U.S. results used for ARR may not be appropriate for Australian conditions (CRC, 1995). This concern was confirmed by recent studies (Nittim, 1989; Avery, 1991) in which the authors found that the values from ARR were generally larger than those from their own studies. Table 5.1 shows typical results for 24 hour duration of rainfall.

TABLE 5.1
AREAL REDUCTION FACTORS
FOR COASTAL CATCHMENTS IN NSW

ARI (Years)	Georges River (1) (360 km^2)	Tweed River (2) (650 km^2)	Bellinger River (2) (640 km^2)	Manning River (2) (6560 km^2)
20	0.94	0.81	0.75	0.74
50	0.90	0.84	0.79	0.74
100	0.88	0.88	0.78	0.74

Source (1) Nittim, 1989
(2) Avery, 1991

For the Manning River catchment, which is adjacent to the Hunter River catchment, Avery, 1991 derived a value of 0.74 for the 24 hour duration. Nittim's values for the Georges River catchment were somewhat higher and approached the values given in ARR. ARFs are clearly a high priority research area in flood estimation and it is understood that the forthcoming revision of ARR will include a treatment of this subject. Derivation of site specific ARFs for the Pages River and Borambil Creek would have required a large amount of data collection and analysis and consumed a considerable proportion of the overall study budget. For this present investigation, design values were adopted which are close to ARR recommendations.

The design ARF's adopted to adjust the point rainfalls are shown in Table 5.2. They show an increase with increase in storm duration.

TABLE 5.2
ADOPTED AREAL REDUCTION FACTORS

Storm Duration	Areal Reduction Factor	
(Hours)	Pages River	Borambil Creek
1	0.78	0.82
3	0.89	0.92
6	0.92	0.93
9	0.93	0.94
12	0.94	0.95
24	0.95	0.96

5.1.3 Temporal Patterns

Temporal patterns for various zones in Australia are presented in ARR. These patterns are used in the conversion of a design rainfall depth with a specific average recurrence interval (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 ARI's up to 500 years 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 and separate patterns are presented in Volume 2 for ARI < 30 years and ARI > 30 years. The second pattern is intended for use with rainfalls with ARI's up to 100 years, and to 500 years in those cases where the design rainfall data in Chapter 2 of ARR are extrapolated to this ARI.

Recent investigations on temporal patterns have been carried out by the Bureau of Meteorology in Melbourne. That work was aimed at the development and application of a procedure for the derivation of design temporal patterns for use with generalised estimates of probable maximum precipitation (PMP), Nathan (1992). These design temporal patterns were derived from the large storm data base for the Inland and Coastal zones which have been compiled as part of the development of the Generalised Southeast Australia Method (GSAM) of estimating PMP. The data base of temporal patterns is related to area rather than being based solely on point rainfall. Design temporal pattern is therefore a function of catchment area.

Studies undertaken by Pilgrim et al (1969) and other investigators have shown that the temporal distributions of heavy rainfalls are more uniform than rainfalls of lesser intensity. That is, temporal patterns become more uniform with increasing ARI. The temporal patterns of observed large events can therefore be transformed into patterns more suited for use with the PMP by "smoothing" the ordinates of the mass curve. This smoothing process converts the spiky variability associated with real storms to the more uniform patterns associated with the extreme events. In the procedure described in Nathan (1992), modification of the "unsmoothed" patterns of observed storms involved smoothing in three directions (i) along the ordinates of the mass curve (ii) across the storm area and (iii) across the storm duration.

While the smoothed patterns are appropriate for PMP, either unsmoothed patterns or ARR patterns should be adopted for the design flood estimation of lesser events. Unsmoothed patterns for durations from 24 hours to 72 hours were supplied by BOM for a recent study of the Upper Nepean River (PWD, 1995) and were applied to a RORB model of that catchment. The ARR patterns incorporated a steep increase in rainfall in the middle of the storm which resulted in higher catchment flows.

It was not practicable to have BOM undertake a similar investigation to derive unsmoothed patterns for the present study area. Although the catchment areas are smaller, with corresponding shorter duration critical storms, it is likely that a similar situation would apply as for the Upper Nepean catchment, with ARR temporal patterns giving larger peak flows.

Accordingly ARR temporal patterns have been adopted herein to derive the design flood events. For computation of the PMF flows, PMP has been estimated using the generalised short duration method (GSDM), as presented in BOM Bulletin 53, 1994.

5.2 Model Parameters

The frequency of the events used in the RORB model calibration process reported in Section 3 are considered to be major floods in the 20-100 year ARI range. For those events there was a constant value for the storage parameter k_c of 9.5 which gave best results for the Pages River. The value of m adopted in the calibration process was 0.80, as recommended in the RORB manual. This value has been retained for design.

Also, in the calibration process, initial loss rates ranged between 12 and 80 mm and continuing loss rates ranged between 2.74 and 0.16 mm/h, but were generally less than the 2.5 mm/h commonly adopted for design. However, there are insufficient data to warrant adoption of loss values other than these recommended by Walsh et al, 1991. The adopted model parameters are summarised in Tables 5.3 and 5.4.

TABLE 5.3
PAGES RIVER RORB MODEL
ADOPTED DESIGN PARAMETERS

Parameters	Average Recurrence Interval (Years)			
	5	20	50	100
IL mm	55	55	50	40
CL mm/h	2.5	2.5	2.5	2.5
k_c	9.5	9.5	9.5	9.5
m	0.80	0.80	0.80	0.80

TABLE 5.4
BORAMBIL CREEK RORB MODEL
ADOPTED DESIGN PARAMETERS

Parameters	Average Recurrence Interval (Years)			
	5	20	50	100
IL mm	55	55	50	40
CL mm/h	2.5	2.5	2.5	2.5
k_c	7.0	7.0	7.0	7.0
m	0.80	0.80	0.80	0.80

5.3 Design Hydrographs

5.3.1 General

The RORB model was run with the above parameters for storms ranging between 6 and 72 hours in duration to obtain design hydrographs for input to the hydraulic model. Peak flows are shown on Tables 5.5 and 5.6.

In general, the long duration storms lasting 48 hours were critical over the study catchments for ARI up to 50 years. These results are in conformity with those achieved by Walsh et al, 1991. In a study of 22 NSW catchments (ranging in areas from 25 to 6560 km^2) those authors found that the critical storm duration was independent of catchment size and in temporal pattern Zone 1, the 36 and 48 hour durations were consistently critical.

For the 100 year ARI, shorter duration storms (9 and 36 hours) were critical in most locations. The higher intensity rainfalls associated with those storms overcame the effects of initial loss.

The recorded January 1996 flood peak of 1030 m^3/s is close to the 50 year ARI in terms of peak discharge at Blandford on the basis of these results. Further upstream at Murrurundi, the January 1996 flood peak was assessed as 450 m^3/s, which is near the modelled 100 year ARI peak discharge.

At Willow Tree the January 1996 flood peak as assessed by RORB was close to the 100 year ARI modelled peak discharge.

A discussion on PMF results is given in Section 5.4.

TABLE 5.5
PAGES RIVER PEAK DISCHARGES (m^3/s)
FROM RORB MODEL

ARI (Years)	LOCATION			
	Murrurundi (Arnolds Bridge)	Warlands Creek At Pages River Confluence	Pages River At Blandford	Blandford Gauging Station
5	170 (48)	150 (30)	200 (30)	440 (30)
20	280 (48)	300 (48)	370 (48)	830 (48)
50	330 (48)	370 (48)	450 (48)	1040 (36)
100	480 (9)	440 (9)	580 (9)	1270 (36)
PMF	4300 (2)	3400 (2)	6100 (3)	10800 (3)

Note: values in brackets are critical storm duration (hours)

TABLE 5.6
BORAMBIL CREEK PEAK DISCHARGES (m^3/s)
FROM RORB MODEL

ARI	LOCATION			
(Years)	Chilcotts Creek At Confluence	Borambil Creek At Confluence	Borambil Creek Hams Bridge	Borambil Creek d/s Willow Tree
5	210 (48)	110 (48)	310 (30)	310 (30)
20	380 (48)	180 (48)	570 (48)	580 (48)
50	460 (48)	220 (48)	690 (48)	700 (48)
100	640 (9)	320 (9)	970 (9)	940 (9)
PMF	6800 (2)	2000 (2)	8900 (2)	8700 (2)

Note: values in brackets are critical storm duration (hours)

5.3.2 Sensitivity of discharges to Loss Values

Runs of the RORB models were undertaken to test the sensitivity of results to variations in loss values. Tables 5.7 and 5.8 show the results for 20 and 100 year ARI rainfalls. For each frequency the design loss values of the previous section were reduced to IL = 20 mm and CL = 1 mm/h. The percentage increases in flows at each location are also given.

On the Pages River, for the 20 year ARI the increase in peak discharge ranges between 14 and 20 percent depending on location. In the Murrurundi area, these increases in discharge generally result in an increase of 100-200 mm in peak flood levels along the Pages River. Similar increases are experienced on Warlands Creek in the vicinity of Blandford.

On Borambil Creek, for the 20 year ARI, the increase in peak discharges range between 14 and 16 per cent. On the floodplain at Willow Tree, the corresponding increase in peak flood levels is generally around 200 mm.

TABLE 5.7
SENSITIVITY OF PAGES RIVER DISCHARGES (m^3/s) TO ASSUMED LOSSES

ARI (Years)	Loss Values	LOCATION			
		Murrurundi (Arnolds Bridge)	**Warlands Creek at Pages River Confluence**	**Pages River at Blandford**	**Blandford Gauging Station**
20	IL 20 mm CL 1.0 mm/h	320	360	430	1000
	IL 55 mm CL 2.5 mm/h (Design Values)	280	300	370	830
	% Increase in Flow	+14	+20	+16	+20
100	IL 20 mm CL 1.0 mm/h	540	550	690	1550 (9) 1410 (36)
	IL 40 mm CL 2.5 mm/h (Design Values)	480	440	580	1230 (9) 1270 (36)
	% Increase in Flow	+13	+25	+20	+26 (9) +11 (36)

Note: values in brackets are critical storm duration (hours).

TABLE 5.8
SENSITIVITY OF BORAMBIL CREEK PEAK DISCHARGES (m^3/s) TO ASSUMED LOSSES

ARI (Years)	Loss Values	LOCATION			
		Chilcotts Creek at Confluence	**Borambil Creek at Confluence**	**Borambil Creek Hams Bridge**	**Borambil Creek d/s Willow Tree**
20	IL 20 mm CL 1.0 mm/h	440	205	660	680
	IL 55 mm CL 2.5 mm/h (Design Values)	380	180	570	580
	% Increase in Flow	+16	+14	+16	+16
100	IL 20 mm CL 1.0 mm/h	740	360	1110	1135
	IL 40 mm CL 2.5 mm/h (Design Values)	640	320	970	940
	% Increase in Flow	+15	+13	+14	+21

5.4 Probable Maximum Flood

Estimates of probable maximum precipitation (PMP) were derived for the study catchments using the Generalised Short Duration Method, GSDM (BOM, 1994). Figure 5.1 shows the envelope of PMP depths, averaged over the catchment, for a range of durations from 1 to 6 hours, for which the GSDM procedure applies. For comparison, the 100 year ARI rainfalls computed from the design storm estimation are also shown. In the case of the 6 hour storm, for example, the PMP on Borambil Creek is 570 mm compared with 105 mm for the 100 year ARI.

Tables 5.5 and 5.6 shows the results of applying PMP hyetographs to the RORB model. For the purpose of this analysis, model parameters were as follows:

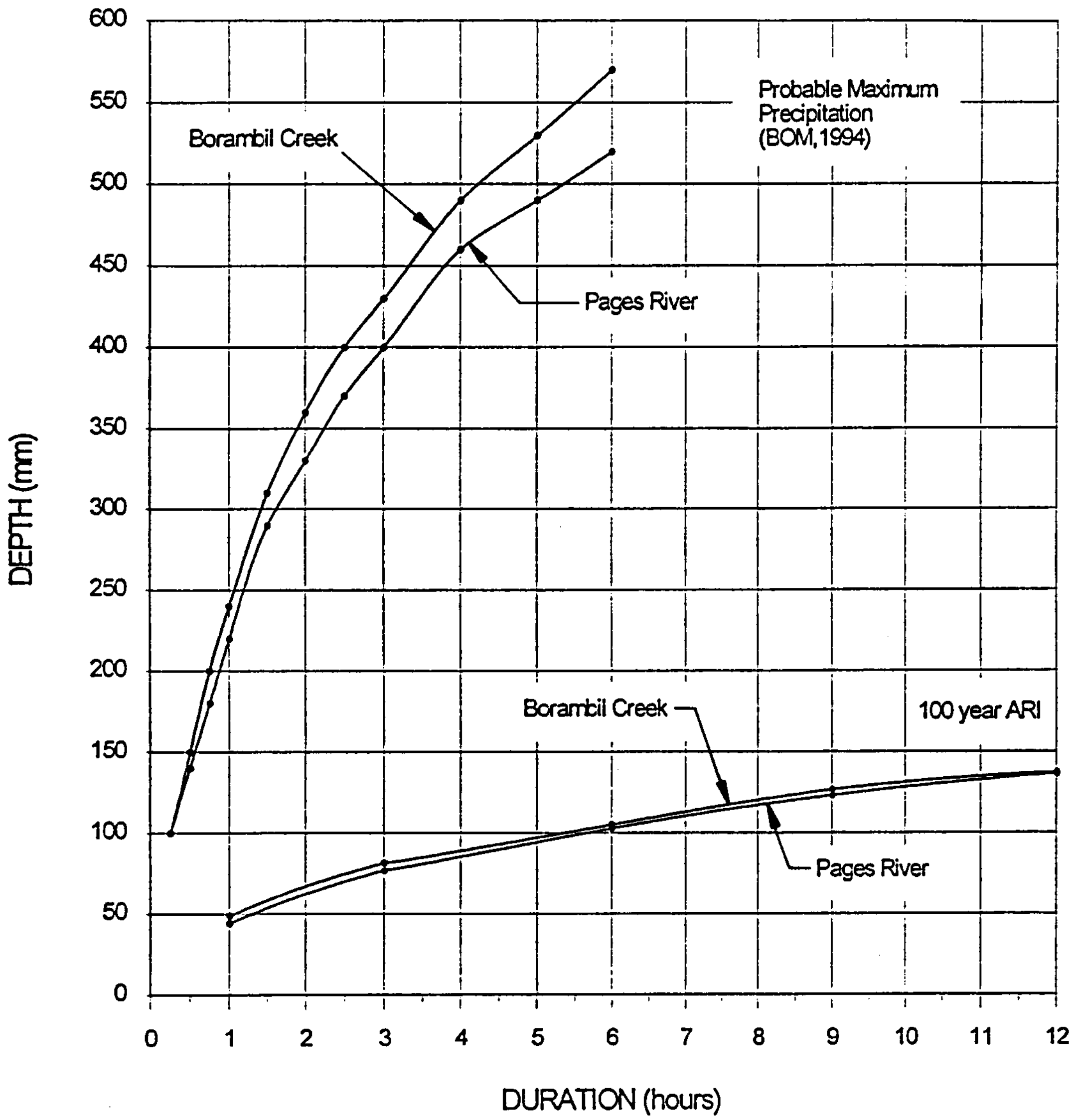
k_c	=	9.5 (Pages River), 7 (Borambil Creek)
m	=	0.80
IL	=	0 mm
CL	=	0 mm/h

It is the usual practice to reduce loss values for PMP analyses, although the magnitude of the PMP is such that resulting peak discharges are not particularly sensitive to losses.

Using the above values, peaks were derived which were 9-10 times 100 year ARI discharges. They appear large when compared with the results of other investigations which commonly show PMP flood peaks four to six times the 100 year ARI value (LMCE, 1994). Recent research suggests that catchments tend to behave in a linear manner for extreme flood events. That is, the exponent m in the storage discharge equation (equation 3.1) equals 1. This effect is due to the reduction in the increase of mean velocity as the floodplain becomes progressively more developed with increasing discharge. The trend to linearity is more pronounced for wide floodplains which are considerably rougher than the main channel and is less for catchments with V-shaped valleys with small flood plains. (Pilgrim, 1986).

Sensitivity runs were undertaken for both the Pages River and Borambil Creek with a linear model (m = 1). However, peak flows at the catchment outlet were less than twice the 100 year ARI design peaks and were clearly too low.

In the case of the PMF, ARR recommends the use of m = 0.8 where most of the valleys in the catchment are V shaped with only small floodplains, as is the case with the present study catchments, unless the value of m by calibration is found to be different. From the RORB model testing of Chapter 3, a value of m = 0.8 was found to be appropriate for major flood events and this value has been retained for the PMF estimation.



MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 5.1
PMP AND 100 YEAR ARI RAINFALLS

6. HYDRAULIC MODELLING OF DESIGN FLOOD

6.1 Introduction

This chapter deals with the derivation of flood behaviour using the MIKE 11 models. Section 6.2 deals with flooding along the Pages River and Section 6.3 covers flooding along Borambil Creek. Modelling was undertaken for the 5, 20, 50 and 100 year ARI events, as well as for the Probable Maximum Flood. Section 6.4 deals with the assessment of flood hazard and hydraulic categorisation and is preliminary only.

It is to be noted that all flood extent and hazard delineation lines are indicative only and based on available mapping which especially in the case of Blandford and Willow Tree is broad scale only.

Results have been presented as follows:

Details	Pages River	Borambil Creek
Water surface profiles	Figures 6.1 - 6.1b	Figure 6.4
Flood contours	Figures 6.2 - 6.3b	Figures 6.5 - 6.5b
Preliminary Hazard	Figures 6.6 - 6.7b	Figures 6.8 - 6.8b

Appendix A tabulates the design flood information in terms of peak levels and flow and velocity distribution across the floodplain.

6.2 Results of Hydraulic Modelling - Pages River

6.2.1 Design Floods

Most of the flow is contained within the river for the 5 year ARI, which increases in discharge from 150 m^3/s at the upstream end of the model to 340 m^3/s at the outlet. None of the flood runners and overflow weirs incorporated in the model in the Murrurundi area function at this magnitude of flooding. Flow velocities in the Pages River range between 1.5 and 2.8 m/s. On Warlands Creek the peak discharge entering the model upstream of Barsham bridge is 140 m^3/s. Most of this flow is conveyed through the bridge waterway, but a small surcharge amounting to 14 m^3/s is conveyed across the bridge approach road (Timor Road) and rejoins Warlands Creek further downstream. No flow crosses the New England Highway and there is no exchange of floodwaters between Warlands Creek and the Pages River downstream of Blandford.

At the 20 year ARI, peak flows at the upstream and downstream model boundaries are 250 and 670 m^3/s respectively. In the Murrurundi area peak water levels are about 600 mm higher than for the 5 year ARI. None of the flood runners and weirs function, indicating that the flow is contained within the immediate confines of the river. Flow velocities increase slightly with the increased depth, ranging between 2 and 3 m/s on the Pages River. On Warlands Creek, the peak discharge on the upstream side of Barsham Bridge is 300 m^3/s of which about 90 m^3/s surcharges the Timor Road. Most of this flow returns to Warlands Creek downstream of the bridge, but a small amount, about 14 m^3/s, passes over the New England Highway and through the school grounds to the Pages River. There are no other exchanges of flow between the two main streams. In the Blandford area, 20 year ARI peak flood, levels are about 1.5 m higher than for the 5 year ARI event but are still contained within the creek system.

At the 100 year ARI, peak inflow to the model amounts to 420 m^3/s and increases to an outflow peak of 1000 m^3/s downstream of Blandford. Warlands Creek contributes 440 m^3/s to the flood peak. These discharges are slightly higher than for the January 1996 event. However, flood levels and the distribution of flood flows are quite similar to this historic event.

In Murrurundi, there is a transfer of flow across the New England Highway from the Pages River to Runner 1. Peak flow returning to the Pages River via this flow path amounts to 60 m^3/s and flow velocities up to 1.5 m/s are experienced. The right bank levee just upstream of Arnolds Bridge is surcharged and the peak discharge conveyed by Runner 2 along Mayne Street amounts to 30 m^3/s at a maximum velocity in the street near the intersection with Adelaide Street of 5 m/s.

Flood behaviour in the Blandford area for the 100 year ARI is similar to that modelled for the January 1996 flood. Flow crossing the New England Highway near Timor Road and heading towards the Pages River amounts to 50 m^3/s and there is an exchange of flows between the two streams downstream of the township similar to that modelled for January 1996.

No flow occurred across the New England Highway on the western end of Arnolds Bridge in Murrurundi for the 50 year event. Runner 2 operated with a small flow of 10 m^3/s due to surcharging of the levee. These model results appear to replicate historic conditions. It is understood that the levee was constructed shortly after the February 1955 flood to prevent Mayne Street operating as a floodway during major flood events. It is understood that no surcharges of the levee have been experienced in the 40 years since its construction, apart from the two experienced during the January and October 1996 floods. (The October 1996 flood occurred when the levee had been breached by the January 1996 event). Since 1955 there have been several other large floods experienced on the Pages River, with the largest experienced in January 1984. At the Blandford gauging station, this flood recorded a peak discharge of 950 m^3/s (Table 2.1) which was equivalent to a 50 year ARI peak discharge (Table 5.5).

6.2.2 Probable Maximum Flood

Peak flood levels upstream of Arnolds Bridge are 3 m higher than for the 100 year ARI and downstream of the bridge, the increase is about 3-4 m depending on location. The total discharge at the bridge site amounts to 4300 m^3/s, of which 700 m^3/s is conveyed through and over the waterway, 3000 m^3/s is conveyed along Runner 1 and 600 m^3/s surcharges the right bank levee and flows along Runner 2.

At the downstream end of the model, peak flow is 10500 m^3/s which is very close to the 10800 m^3/s modelled by RORB. This result shows that there is little attenuation of flow due to the routing effects of the floodplain storage, which are incorporated in the MIKE 11 modelling process. The cross sections extend to the peak water surface level and hence the volume of storage is accurately modelled.

The peak flood level at the downstream end of the model is 5.3 m higher than for the 100 year ARI. Further upstream on the Pages River at Blandford Bridge the increase is 4.5 m. On Warlands Creek, PMF levels are 3.5 m higher near Barsham Bridge and in the range 3-4 m higher in the Blandford township. A large proportion of the flow amounting to 4500 m^3/s is conveyed across the floodplain from Warlands Creek to Pages River downstream of Blandford.

6.2.3 Sensitivity Study

The model was well calibrated with the January 1996 flood which was a major event ranging in magnitude between 50 and 100 year ARI, depending on location along the Pages River. It was considered appropriate to investigate the sensitivity of model results for a lesser flood, in this case the 20 year ARI. The MIKE 11 model was run with the main channel n value increased from 0.045 to 0.055.

In the vicinity of Murrurundi, the average increase in peak levels on the Pages River was 250 - 300 mm and reached a maximum of 350 mm at Mount Street. Further downstream in the Blandford area, the increase averaged about 300 mm and the maximum was 650 mm in the incised channel upstream of Blandford Bridge. This comparatively large increase is due to the high flow velocity in the channel in this area. However, even at the higher flood level resulting from this increase in roughness, the flow would still be contained within the channel.

On Warlands Creek in the Blandford area, the average increase in levels was about 150 mm and reached a maximum of 430 mm at Ch 1.45 km upstream of Hayles Street.

Generally, the extent of flooding did not increase a great deal compared with the previous results and most of the flow was still contained within the immediate vicinity of the channels. The results of this sensitivity study indicated that adjustments to roughness values would not greatly alter model results for the medium flood events when compared with results achieved from the best estimate values.

6.2.4 Recommended Design Flood Values

Figures 6.1 to 6.3b show the recommended design water surface profiles and flood contours. Peak flood levels, flows and velocities are tabulated in Appendix A.

6.3 Results of Hydraulic Modelling - Borambil Creek

6.3.1 Design Floods

Most of the flow is contained within the creek for the 5 year ARI. About 280 m^3/s of the peak discharge at Hams Bridge is conveyed through the waterway of that structure. The balance is conveyed over the right bank upstream of the bridge. This overflow is conveyed along the flood runner towards the school, where it is deflected by the diversion bank and crosses the Merriwa Road and flows over the right floodplain before rejoining Borambil Creek about 200 m downstream of the road.

Just downstream of Hams Bridge the flow is at bankfull with a depth of 4.5 m and a velocity of 1.8 m^3/s. Further downstream, the depth of flow reduces to around 3.5 m with a corresponding increase in velocity to around 3 m/s reflecting the steep gradient and low hydraulic roughness of the stream.

For the 20 year ARI, the peak discharge at Hams Bridge increases to 550 m^3/s, of which 410 m^3/s is conveyed through the bridge and the balance flows over the right bank of Borambil Creek and thence across the Merriwa Road. The peak depth of flow over the stream bank is about 1.5 m and the depth across the road is about 300 mm.

Most of the flow returns to the main stream downstream of the loop in the channel which is located upstream of model section 2.46. At that section 500 m^3/s is conveyed in the channel and 70 m^3/s on the right floodplain. Floodplain flow is about 300 m wide and averages about 500 mm in depth, conveyed at 0.5 m/s velocity.

Further downstream, the right floodplain becomes more confined. At section 2.85, the width of floodplain flow is about 100 m and eventually, at section 3.10 the flow rejoins the main channel.

There is a minor floodrunner on the left floodplain which commences at the cutoff at section 2.85 and continues to section 3.80. At the 20 year ARI, about 10 m^3/s are conveyed as a shallow flow up to 80 m in width.

These floodrunners and the main channel operate as independent streams at the 20 year ARI.

For the 100 year ARI, the peak discharge at Hams Bridge increases to 925 m^3/s. About 525 m^3/s is conveyed through the bridge waterway and the remainder flows over the right bank of Borambil Creek towards the school. The stream bank is submerged to a depth of 2 m. The water surface elevation on the creek side of the diversion bank is 327.4 m and on the school side is 326.5 m. The elevation of the bank increases from 326.4 m at Merriwa Road to 326.9 m about 50 m to the east (upstream) and 327.8 m a further 60 m east. Much of the bank is overtopped and function as a broad crested weir. The depth of flow over Merriwa Road is 600 mm.

Some of the flow crossing the road rejoins Borambil Creek at the loop in the channel. The flow distribution at model section 2.46 is 700 m^3/s in the main channel and 250 m^3/s on the right floodplain. Peak flow velocity in the channel is around 3-4 m/s in the middle reaches of the model, and 1.8 to 2 m/s on the floodplain.

At the 100 year ARI the width of flow is around 400 m at section 2.46, reducing to 200 m in the relatively constricted area at section 3.4 and increasing to around 450 m as the floodplain widens downstream of the Bowling Club.

6.3.2 Probable Maximum Flood

Peak flood levels are generally about 4 m higher than for the 100 year ARI. At the downstream end of the model, the peak discharge is 9380 m^3/s which is close to the 8790 m^3/s peak inflow after allowing for lateral inflow, indicating that there is little attenuation of flow due to floodplain storage effects.

6.3.3 Sensitivity Study

A sensitivity study was undertaken for the 20 year ARI medium flood event, as for the Pages River. Hydraulic roughness of the channel was increased from 0.045 to 0.055. The average increase in flood levels was around 200 mm and reached a maximum of 340 mm locally at Chainage 2.46 km.

From this work it was concluded that the best estimate values of roughness derived from the analysis of the major January 1996 flood could also be adopted for the lesser design flood events.

6.3.4 Recommended Design Flood Values

Figures 6.4 to 6.5b show the recommended design water surface profiles and flood contours. Peak levels, flows and velocities are presented in Appendix A.

6.4 Flood Hazard and Floodway Assessment

For floodplain management purposes it is necessary to subdivide the floodplain into areas that firstly, reflect the impact of development on flood behaviour (i.e. hydraulic effects) and secondly, the impact of flooding on the development (i.e. hazard effects). Sub-division of flood liable land on these two bases are referred to as 'hydraulic categories' and 'hazard categories' respectively.

A comprehensive analysis of flood hazard can only be made within the framework of a floodplain management plan. It requires an assessment of factors such as flood warning, flood awareness, evacuation problems, etc, in addition to the depth and velocity of floodwaters.

Preliminary high and low hazard categorisations of the floodplain are shown on Figures 6.6 to 6.7b for Murrurundi and Blandford respectively and on Figures 6.8 to 6.8b for Willow Tree. These diagrams have been derived solely from the results of the hydraulic analyses and are provisional only. They have been constructed using the depth - velocity criteria presented in the draft Floodplain Management Manual (FMM) DLWC, 1995.

For the purpose of the FMM, there are three categories of flood liable land:

- floodways
- flood storage
- flood fringe

The manual states that it is not practicable to provide explicit quantitative criteria for defining floodways and flood storage areas, as the nature of each study area is different. The following guidelines are given for delineating these areas:

(1) Floodways are those areas which convey a significant proportion of the total flow and where partial blocking will adversely affect flood behaviour.
(2) Flood storages are those areas outside floodways, which if completely filled would cause peak flood levels or downstream peak discharges to significantly increase.

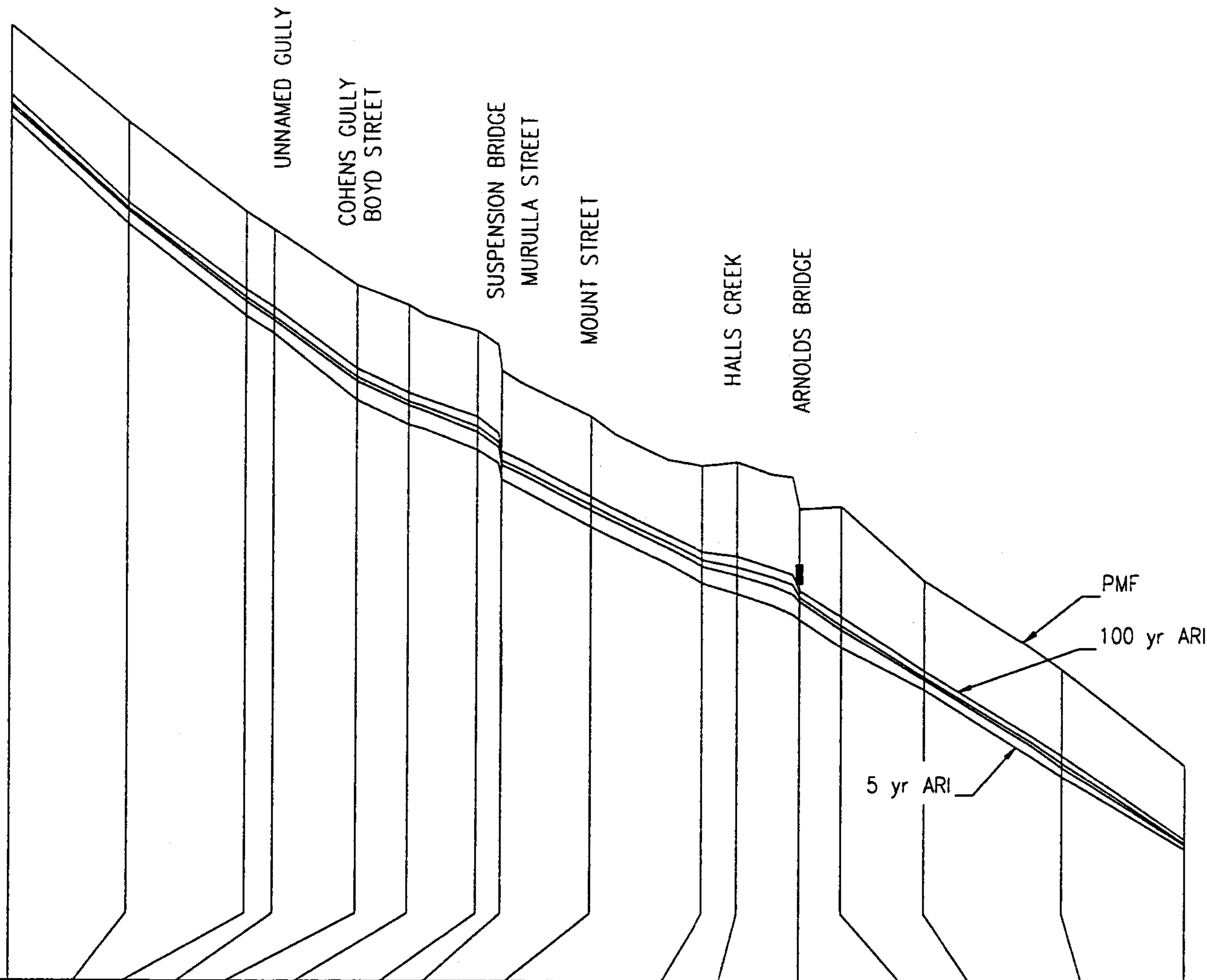
It is to be noted that hydraulic categorisation of the floodplain is usually undertaken during the preparation of the Floodplain Management Study. The hydraulic model of the stream is used for this purpose and investigations are carried out on floods at or around the magnitude of the Designated Flood Event (DFE). Some analysis was undertaken in this present investigation to obtain an estimate of flooding and flood storage areas. A range of floods was investigated, including minor events much smaller than would be considered as a DFE.

In some hydraulic computer programs, for example the HEC-2 program developed by the US Army Corps of Engineers, there is the capacity for automatic reduction in the width of flow along the study reach until a significant increase in peak flood level (e.g. 100 mm) is achieved. This feature is not available for MIKE 11 and the reduction in width has to be undertaken on a trial and error basis. MIKE 11 is a fully dynamic model, in contrast to HEC-2 which is steady state. In some situations, typically in steep streams where velocities are high, a reduction in the width of flow along the study reach results in increases in levels in some areas and reductions in levels in others. This was found to be the case in the present study area, particularly in the case of the medium flood events and consequently the results are necessarily of a preliminary nature and will need to be refined in the Floodplain Management Study.

For the 5 year ARI, the flow is conveyed within the stream banks or their immediate vicinity. Flow velocities in the channel are high, generally in the range 2 to 3.5 m^3/s. There is little overbank flow and development of floodplain storage. The floodway is the main channel, which is closely approximated by the areas denoted as high hazard areas on Figures 6.6, 6.7 and 6.8.

For the 20 year ARI, a trial run of the model was undertaken assuming that the area outside the limit of inundation of the 5 year ARI flood was blocked. Compared with design 20 year ARI flood levels, the blocked levels ranged between - 0.23 m and 0.26 m, but were generally less than 0.1 m higher. Additional reduction in the width of flow would be required to achieve a general increase of 0.1 m in flood levels. As the high hazard area for the 20 year ARI lies within the extent of the 5 year ARI flood, it is considered reasonable to adopt the high hazard area as a preliminary estimate of the floodway for this flood also. (Figures 6.6a, 6.7a, 6.8a).

In the case of the 100 year ARI, however, hydraulic analysis indicated that large increases in flood level would be experienced if flows were constrained to the extent of the 5 year ARI and also if the flood runner on the northern side of the New England Highway at Murrurundi (Runner 1) were blocked. For this flood also, the high hazard area is considered to be a reasonable preliminary estimate of the floodway (Figures 6.6b, 6.7b, 6.8b).



DATUM 450.00

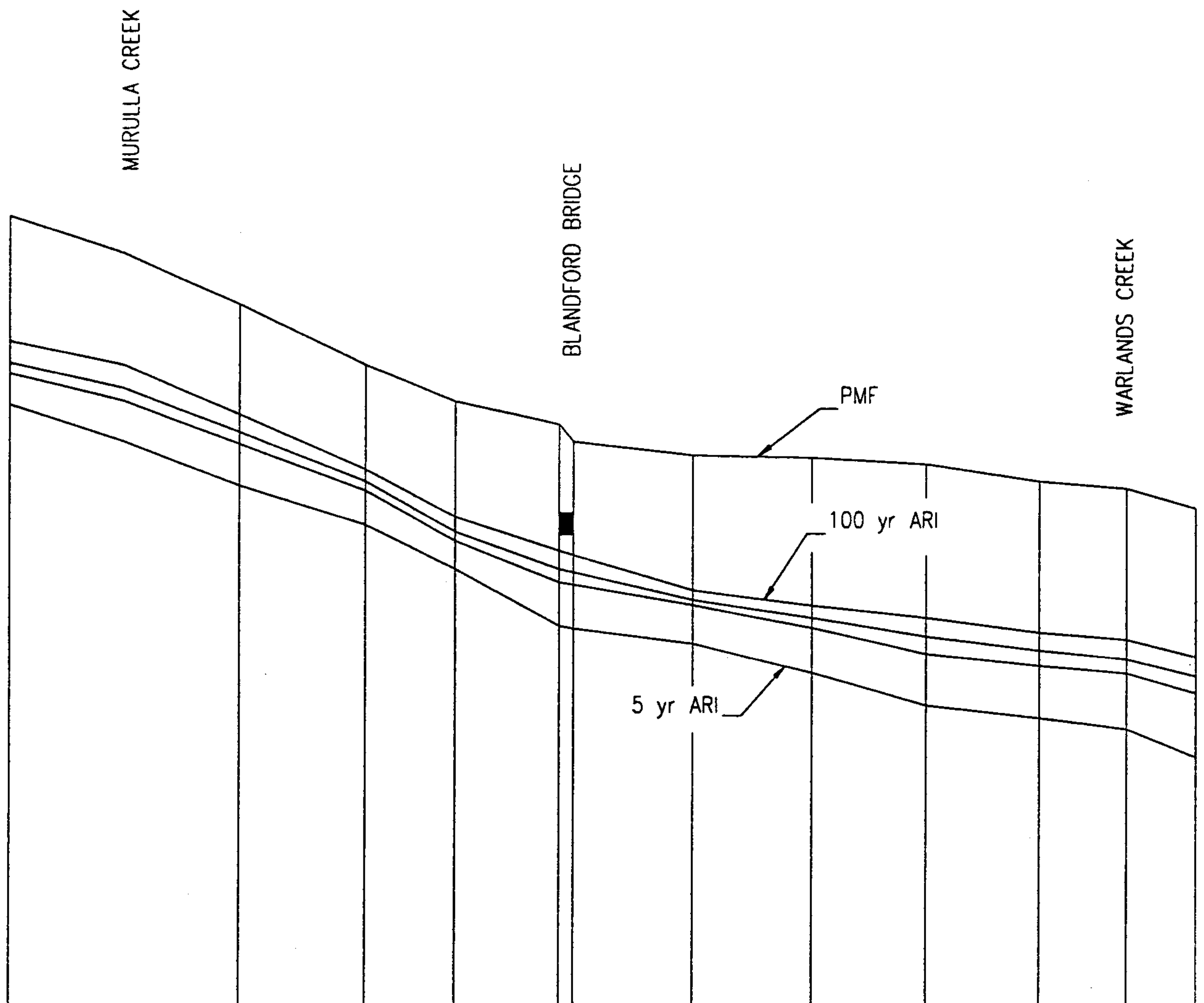
PMF	484.9	481.3	478.1	477.4	475.4	474.7	473.8	473.3	472.3	470.7	468.9	469.0	468.5	467.3	467.4	464.7	461.5	458.0
100 YR ARI	482.3	478.4	475.2	474.6	472.4	471.5	470.7	470.1	469.4	467.8	465.8	465.6	464.9	464.4	463.4	461.4	458.3	455.3
50 YR ARI	482.0	478.2	475.0	474.3	472.1	471.2	470.3	469.7	469.1	467.5	465.5	465.2	464.6	464.1	463.1	461.2	458.1	455.1
20 YR ARI	481.8	478.1	474.8	474.1	471.9	471.1	470.1	469.6	468.9	467.3	465.2	464.9	464.2	464.0	462.9	461.1	457.9	455.1
5 YR ARI	481.5	477.6	474.3	473.6	471.2	470.4	469.4	469.0	468.4	466.7	464.6	464.2	463.5	463.3	462.3	460.7	457.6	454.9
RIVER CHAINAGE	0.00	0.34	0.68	0.76	1.00	1.15	1.35	1.41	1.42	1.68	2.00	2.10	2.26	2.28	2.40	2.64	3.04	3.40

NOTE: LEVELS TO AUSTRALIAN HEIGHT DATUM

MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 6.1

WATER SURFACE PROFILES PAGES RIVER
5 YEAR ARI TO PMF



DATUM 400.00

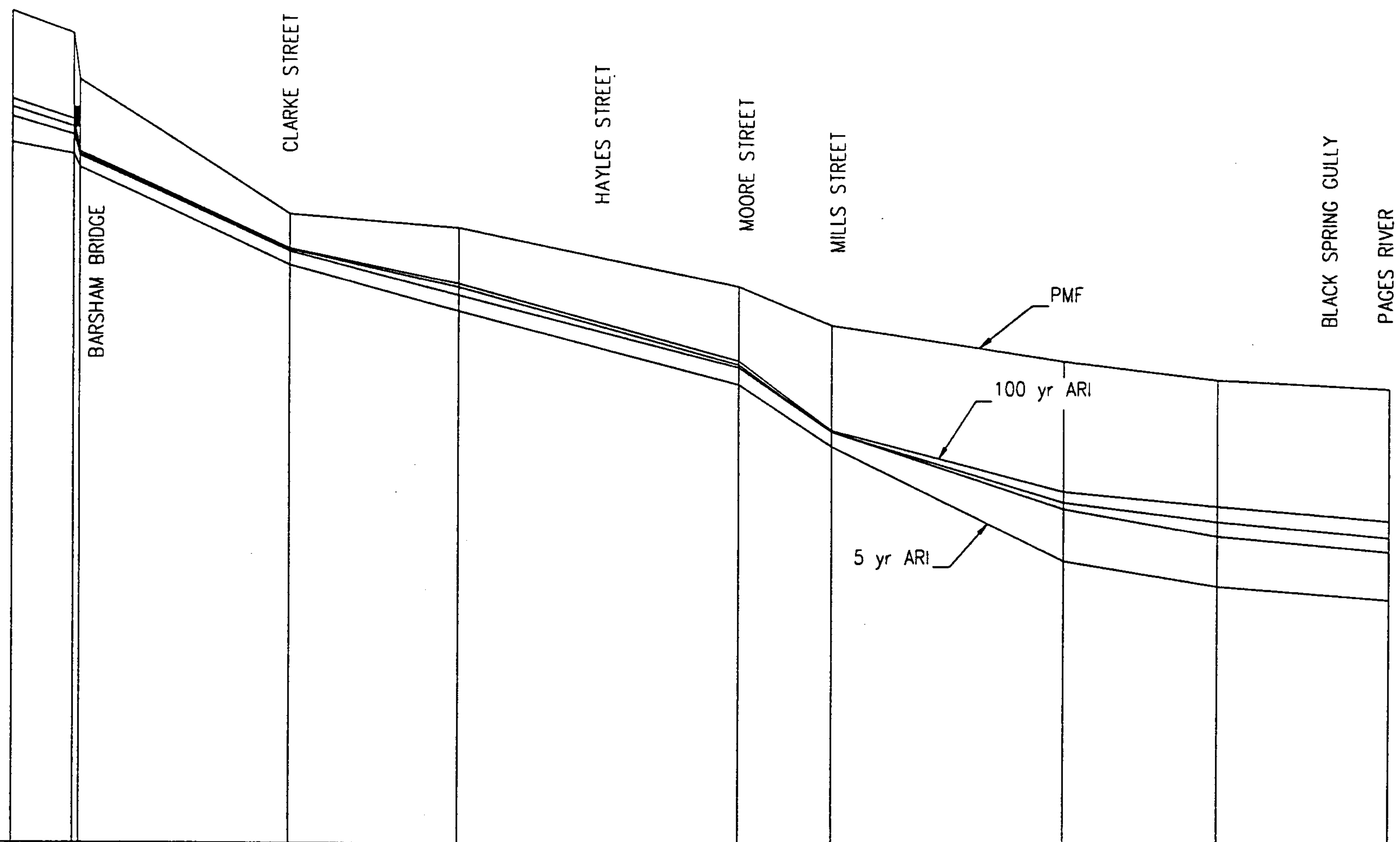
PMF	428.2	425.1	423.0	421.7	420.9	420.3	419.8	419.7	419.5	418.9	418.7	418.0
100 YR ARI	423.8	421.2	419.3	417.6	416.4	416.2	415.0	414.5	414.1	413.5	413.3	412.7
50 YR ARI	423.0	420.6	418.9	417.1	415.7	415.6	414.7	414.0	413.4	412.9	412.6	412.0
20 YR ARI	422.7	420.2	418.5	416.7	415.3	415.2	414.5	413.7	412.8	412.4	412.1	411.4
5 YR ARI	421.6	418.7	417.3	415.7	413.7	413.6	413.1	412.1	410.9	410.5	410.1	409.1
RIVER CHAINAGE	8.90	9.55	9.90	10.15	10.44	10.48	10.81	11.15	11.48	11.80	12.05	12.25

NOTE: LEVELS TO AUSTRALIAN HEIGHT DATUM

MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 6.1a

WATER SURFACE PROFILES PAGES RIVER
5 YEAR ARI TO PMF



DATUM 400.0

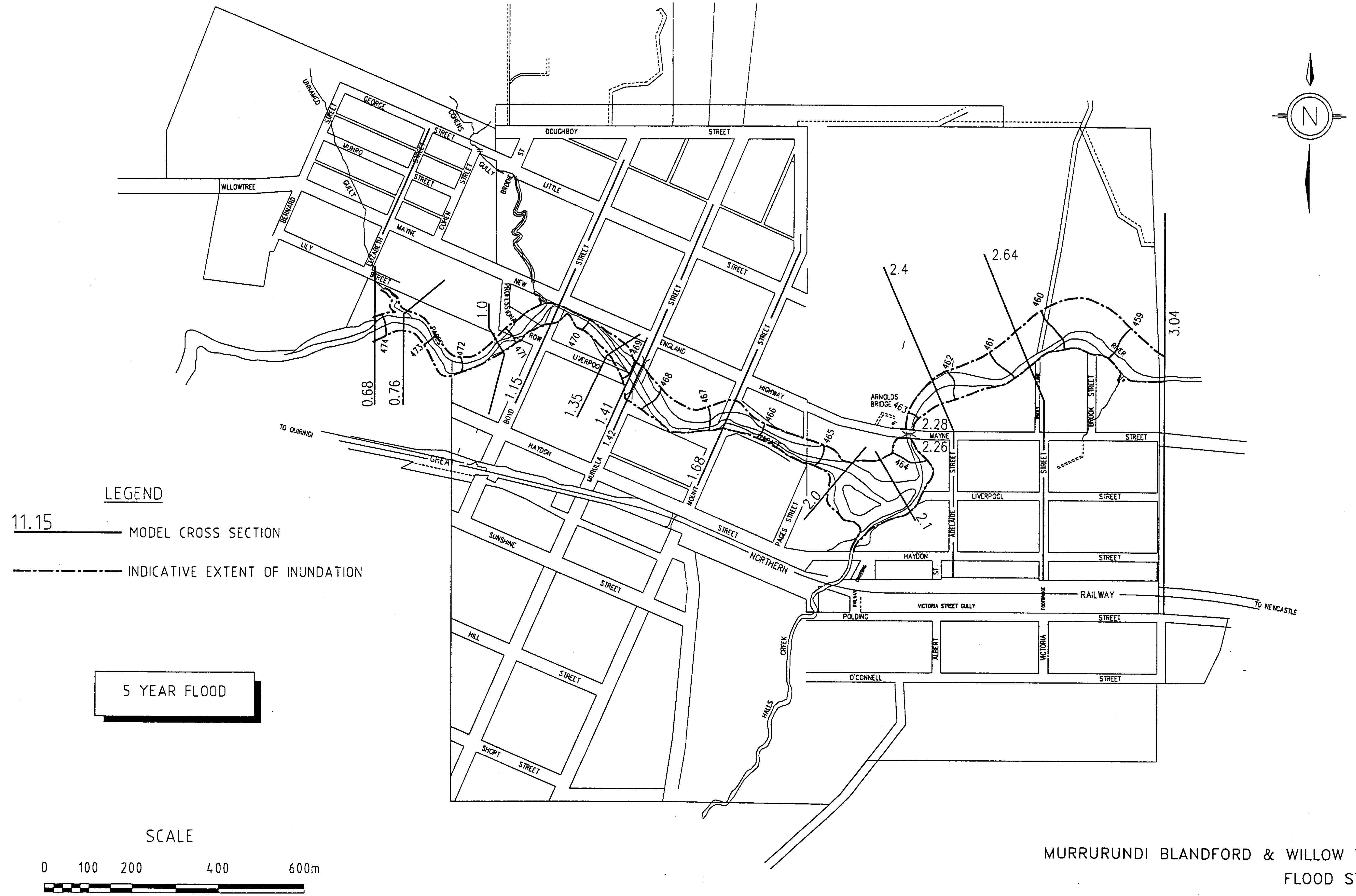
PMF	433.8	432.9	431.0	425.6	425.0	422.7	421.2	419.8	419.0	418.7
100 YR ARI	430.2	429.4	428.1	424.2	422.8	419.7	416.9	414.5	413.9	413.3
50 YR ARI	429.9	429.1	428.0	424.2	422.7	419.6	416.8	414.0	413.3	412.6
20 YR ARI	429.5	428.8	427.9	424.1	422.3	419.4	416.8	413.8	412.7	412.1
5 YR ARI	428.4	428.0	427.4	423.5	421.7	418.8	416.3	411.6	410.6	410.1
RIVER CHAINAGE	0.00	0.20	0.22	0.90	1.45	2.35	2.65	3.40	3.90	4.45

NOTE: LEVELS TO AUSTRALIAN HEIGHT DATUM

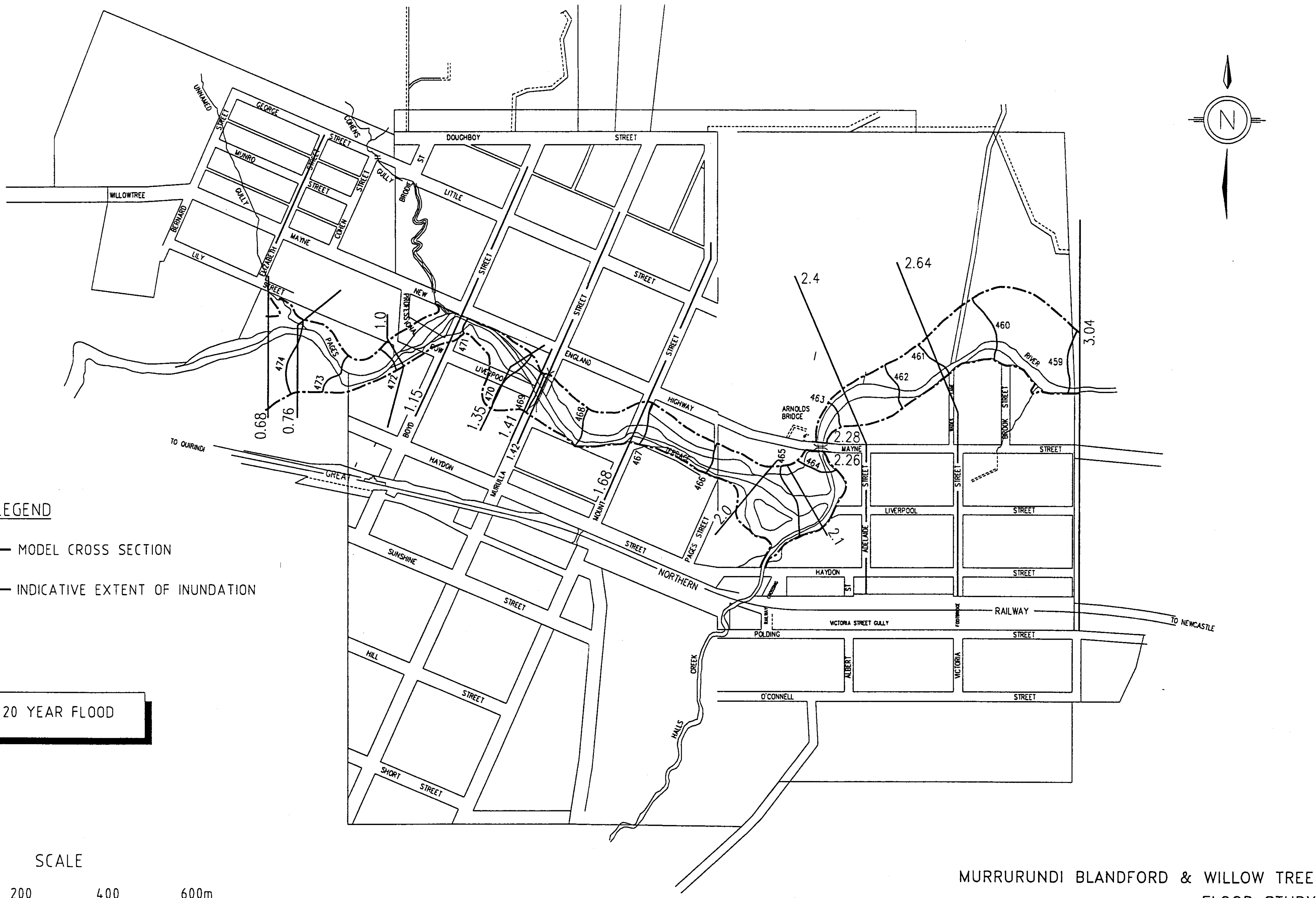
MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 6.1b

WATER SURFACE PROFILES WARLANDS CREEK
5 YEAR ARI TO PMF



MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY
Figure 6.2
5 YEAR ARI FLOOD CONTOURS
MURRURUNDI



LEGEND

11.15 ———— MODEL CROSS SECTION

—·—·—·— INDICATIVE EXTENT OF INUNDATION

20 YEAR FLOOD

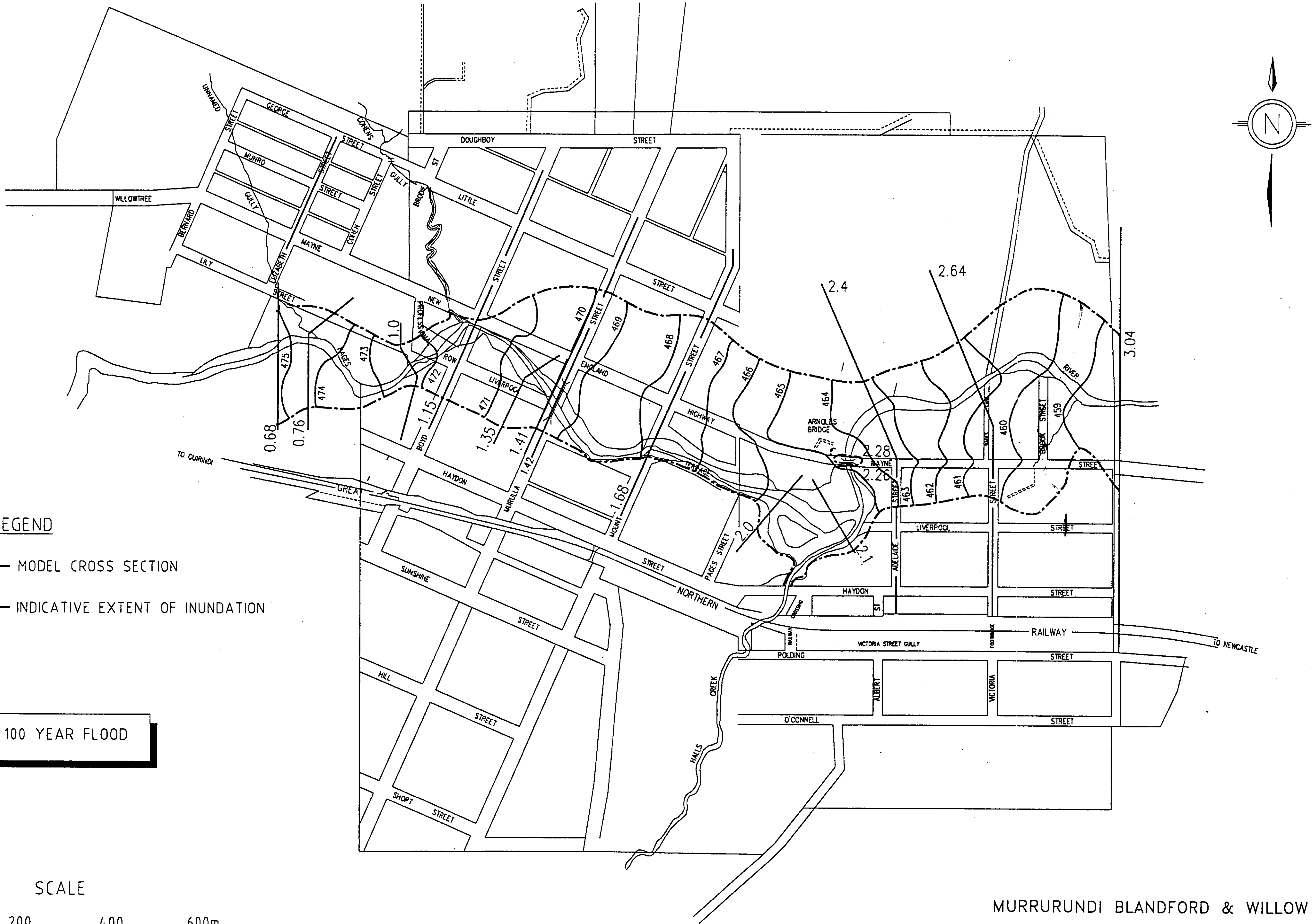
SCALE

0 100 200 400 600m

MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 6.2a

20 YEAR ARI FLOOD CONTOURS
MURRURUNDI



LEGEND

11.15 ——— MODEL CROSS SECTION

—·—·—·— INDICATIVE EXTENT OF INUNDATION

100 YEAR FLOOD

SCALE

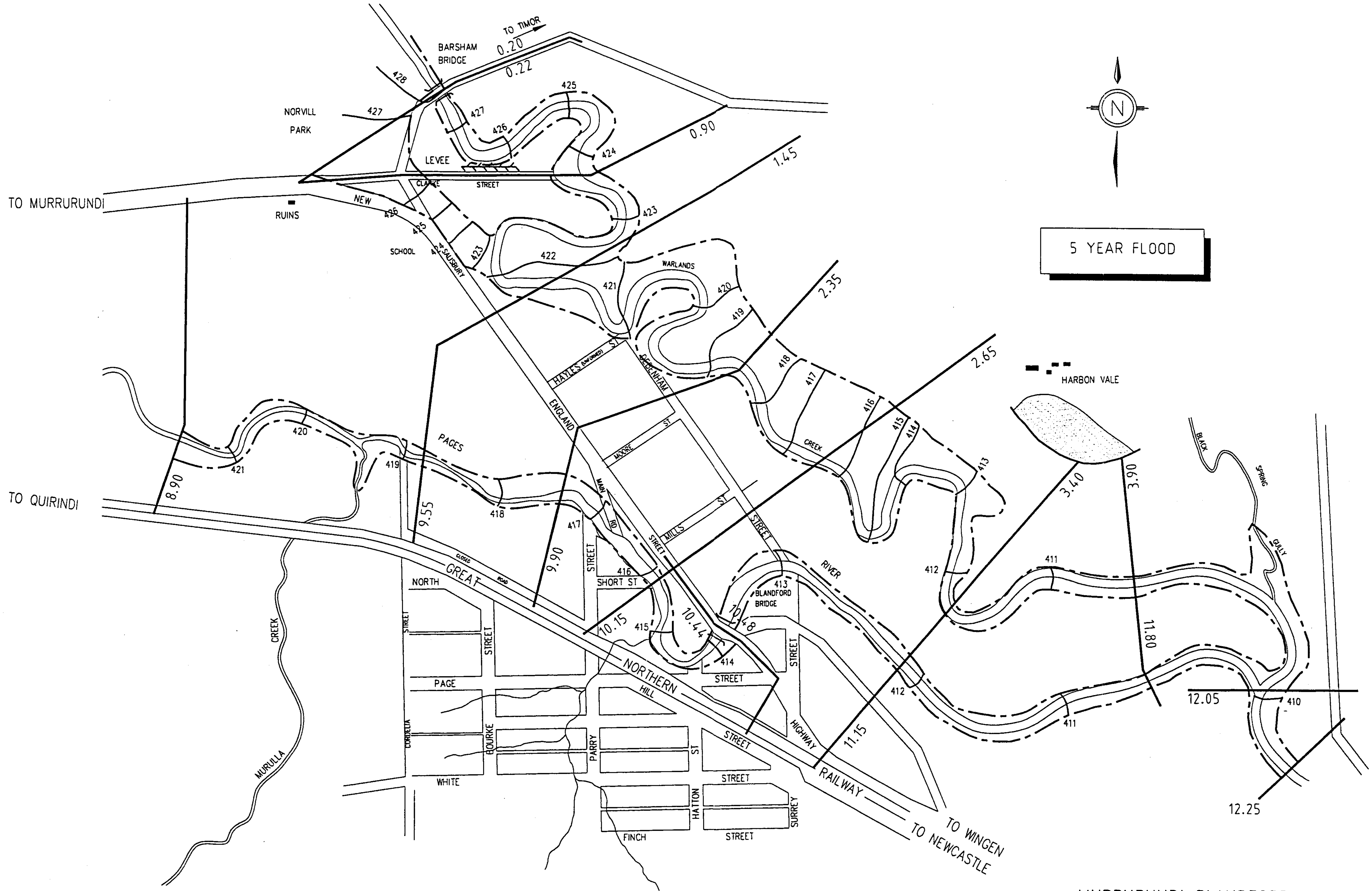
0 100 200 400 600m

MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 6.2b

100 YEAR ARI FLOOD CONTOURS

MURRURUNDI



SCALE

0 100 200 400 600m

LEGEND

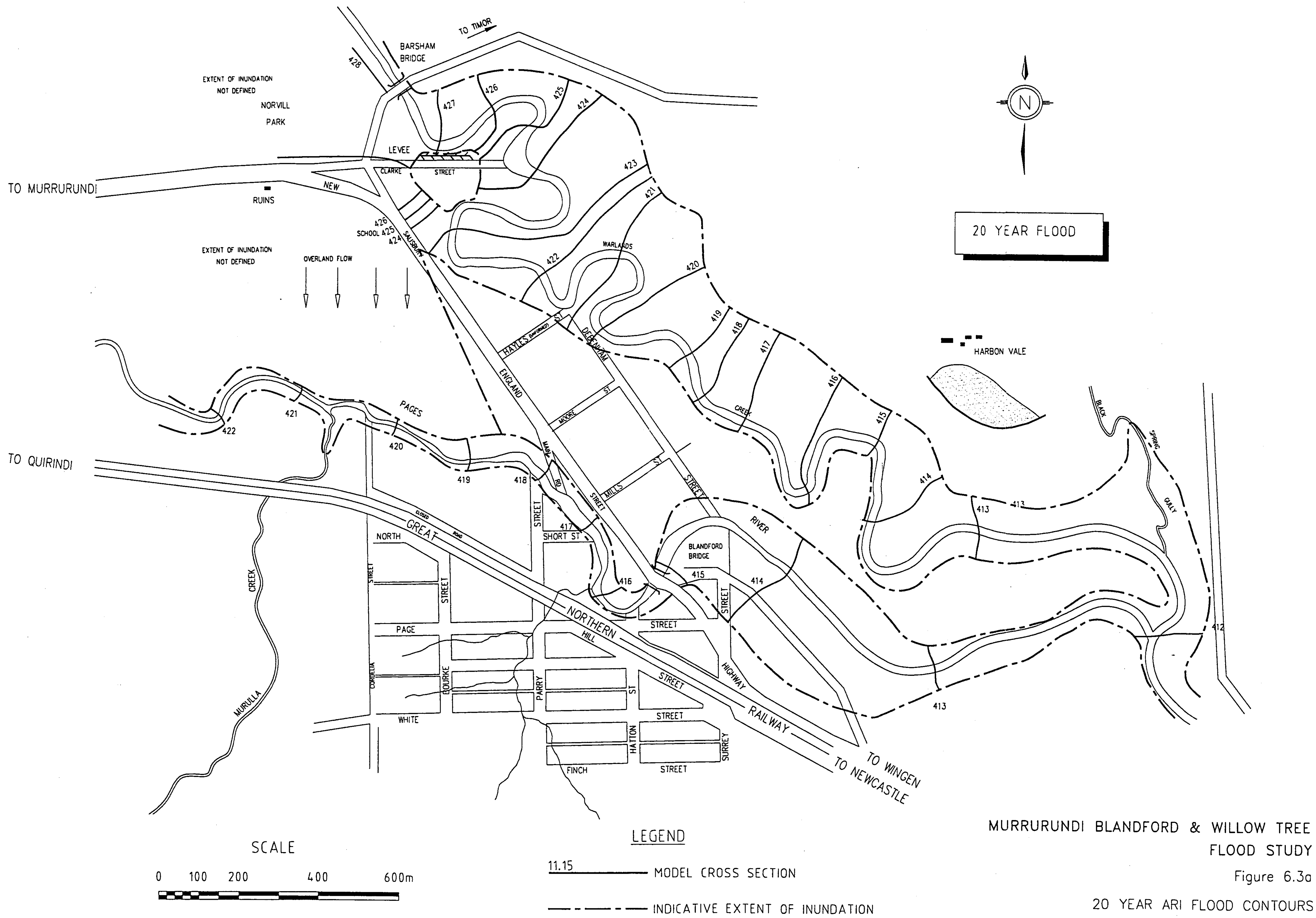
11.15 MODEL CROSS SECTION

INDICATIVE EXTENT OF INUNDATION

MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 6.3

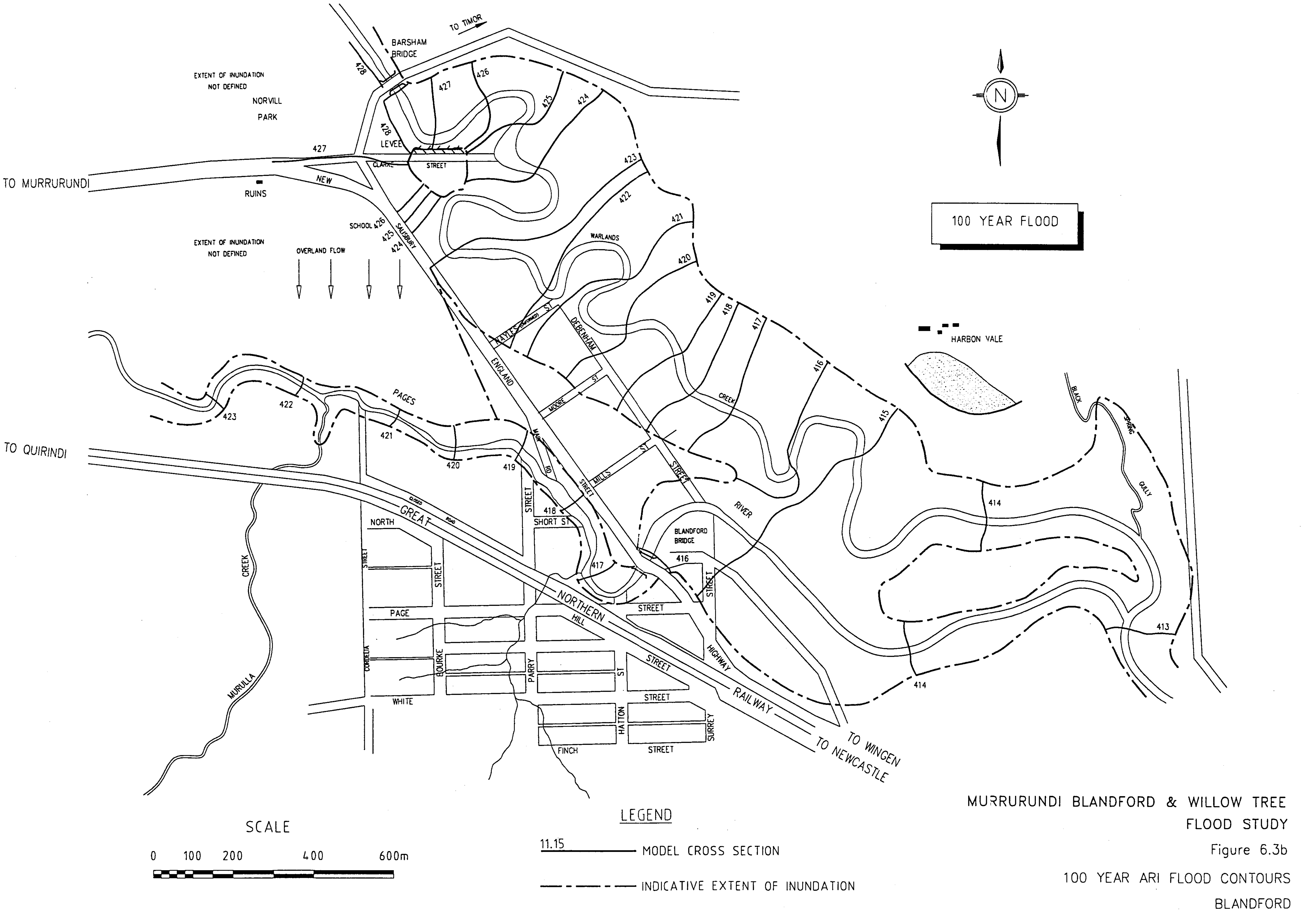
5 YEAR ARI FLOOD CONTOURS
BLANDFORD



MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 6.3a

20 YEAR ARI FLOOD CONTOURS BLANDFORD

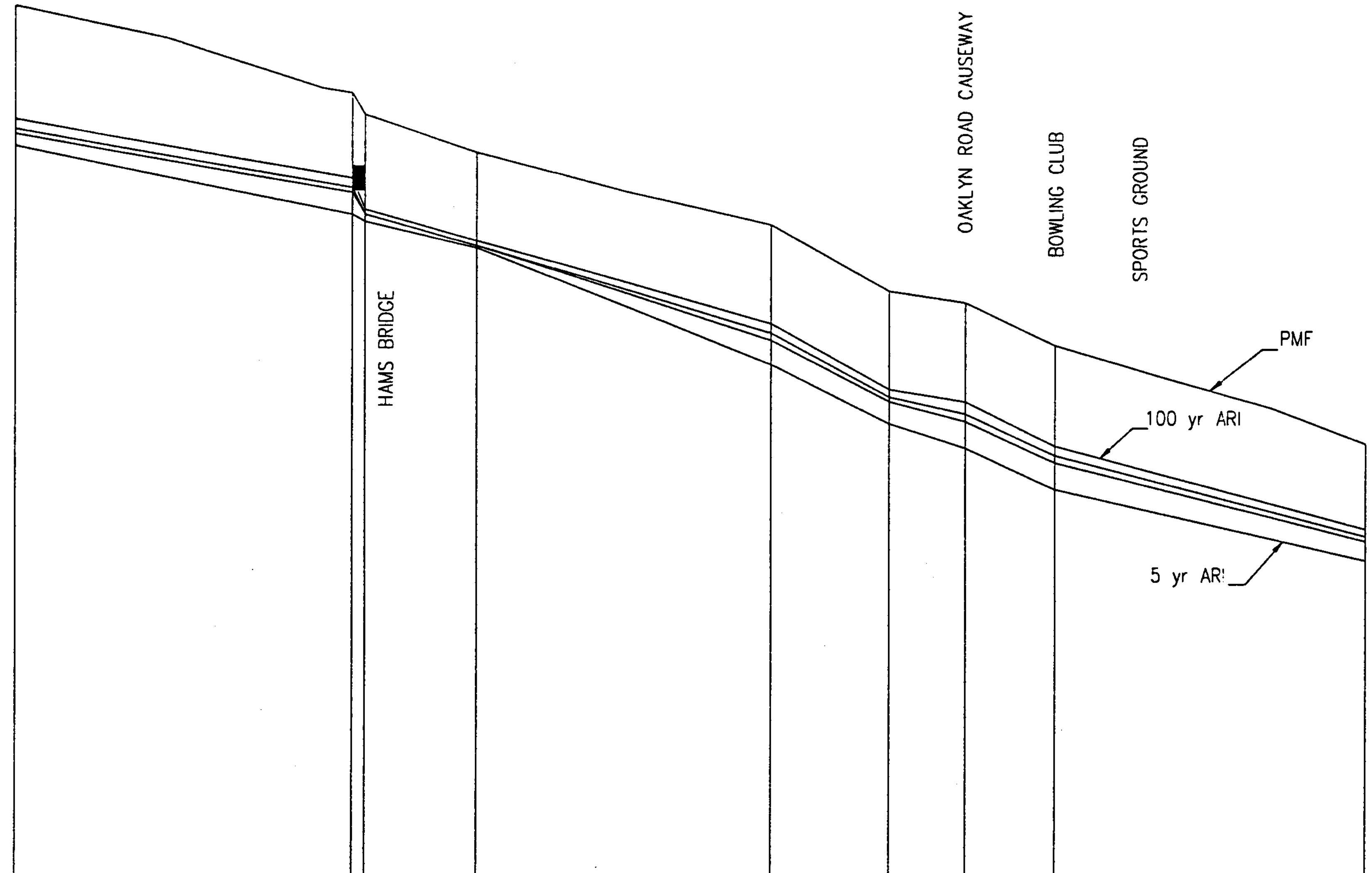


MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 6.3b

100 YEAR ARI FLOOD CONTOURS

BLANDFORD



DATUM 300.00

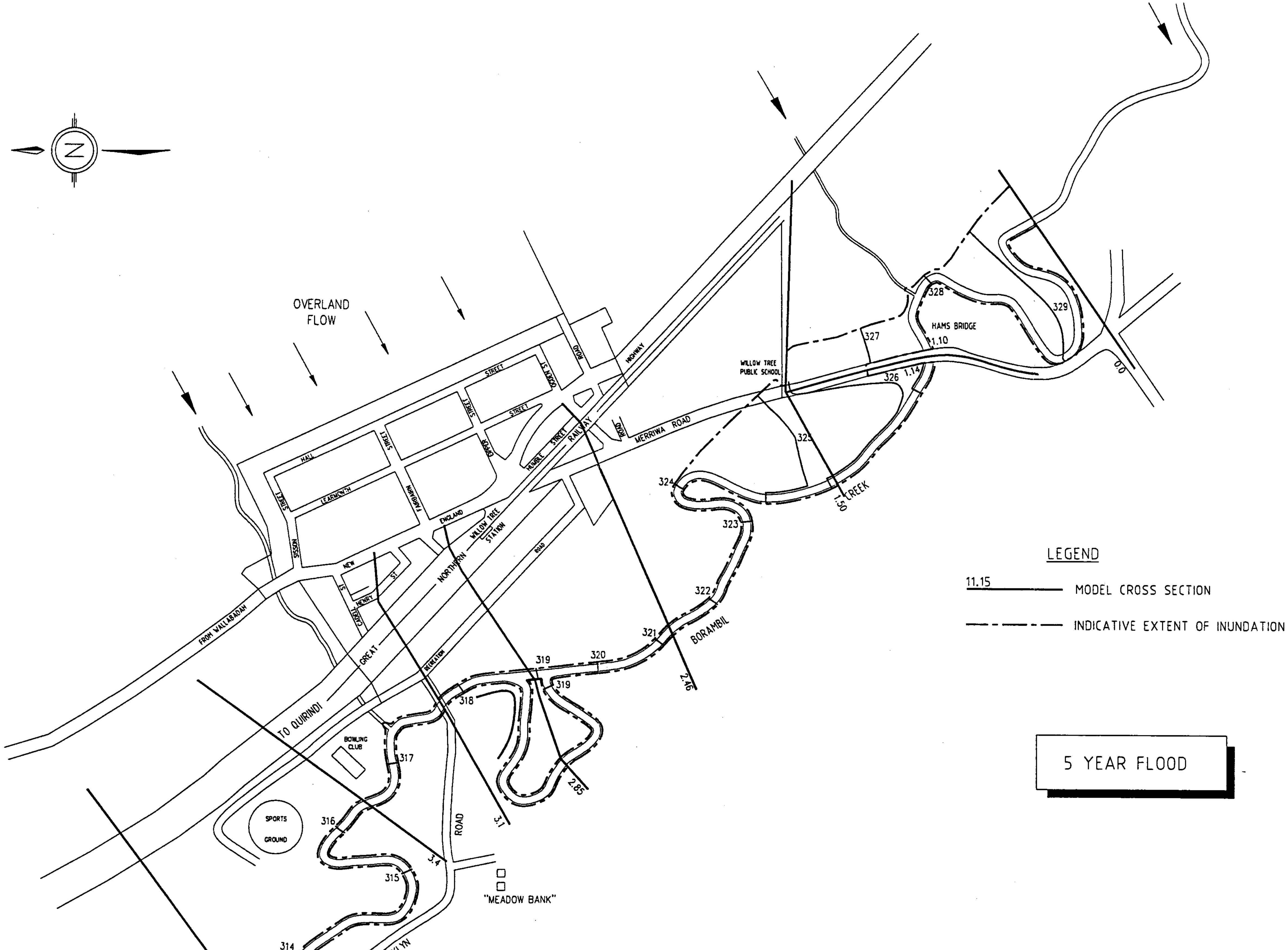
PMF	336.2	332.5	331.6	330.1	327.1	324.4	323.9	322.1	318.1
100 YR ARI	331.4	329.0	327.7	326.4	323.0	320.3	319.8	318.0	314.6
50 YR ARI	331.0	328.6	327.5	326.2	322.6	320.0	319.3	317.6	314.3
20 YR ARI	330.8	328.4	327.5	326.2	322.3	319.8	319.0	317.3	314.1
5 YR ARI	330.3	327.5	327.2	326.1	321.3	318.9	317.9	316.2	313.3
RIVER CHAINAGE	0.00	1.10	1.14	1.50	2.46	2.85	3.10	3.40	4.42

NOTE: LEVELS TO A LOCAL DATUM
SSM 77535 AT NEW ENGLAND HIGHWAY = RL 323.53 (LOCAL DATUM)

MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 6.4

WATER SURFACE PROFILES BORAMBIL CREEK
5YEAR ARI TO PMF

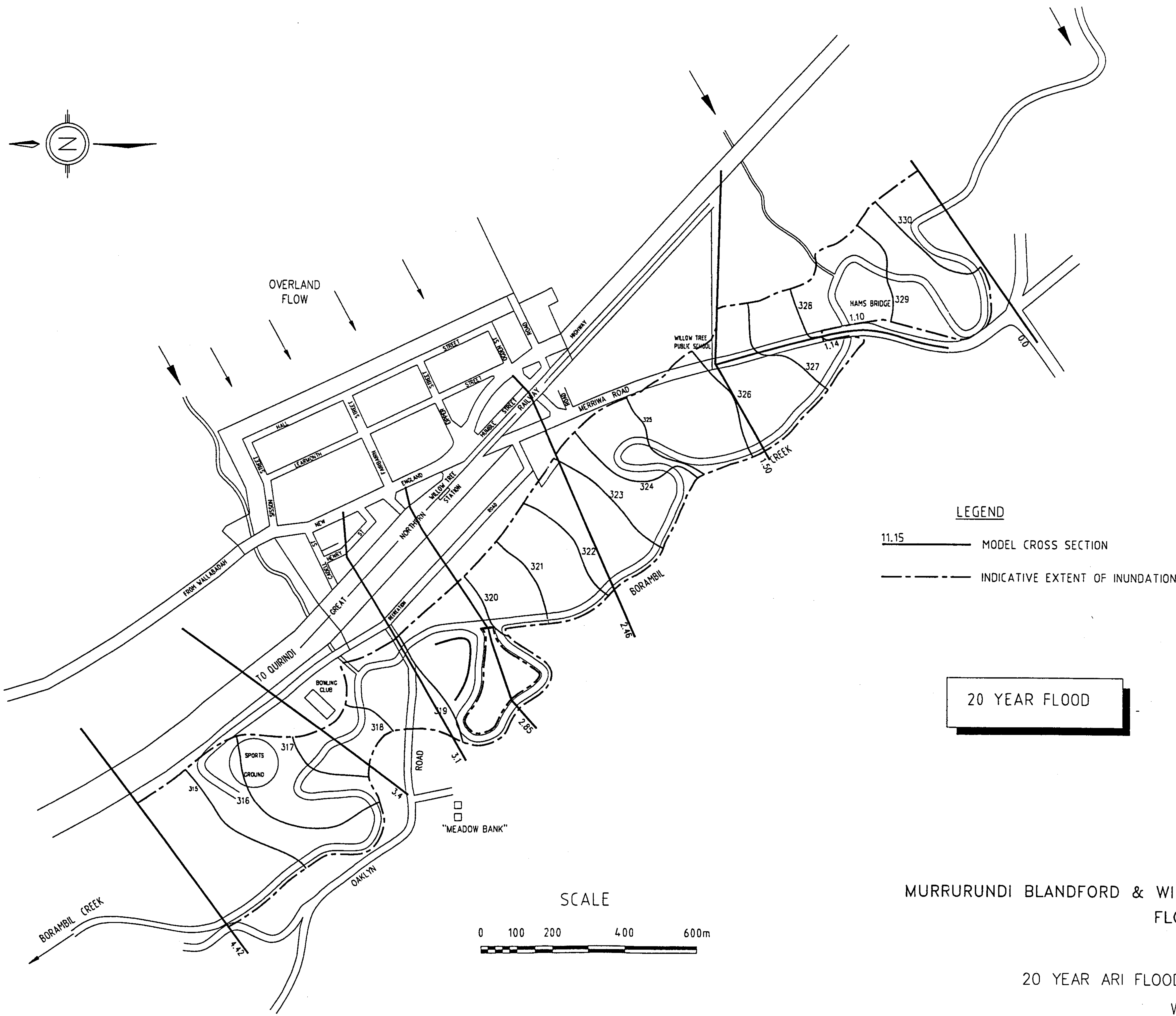


MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 6.5

5 YEAR ARI FLOOD CONTOURS

WILLOW TREE

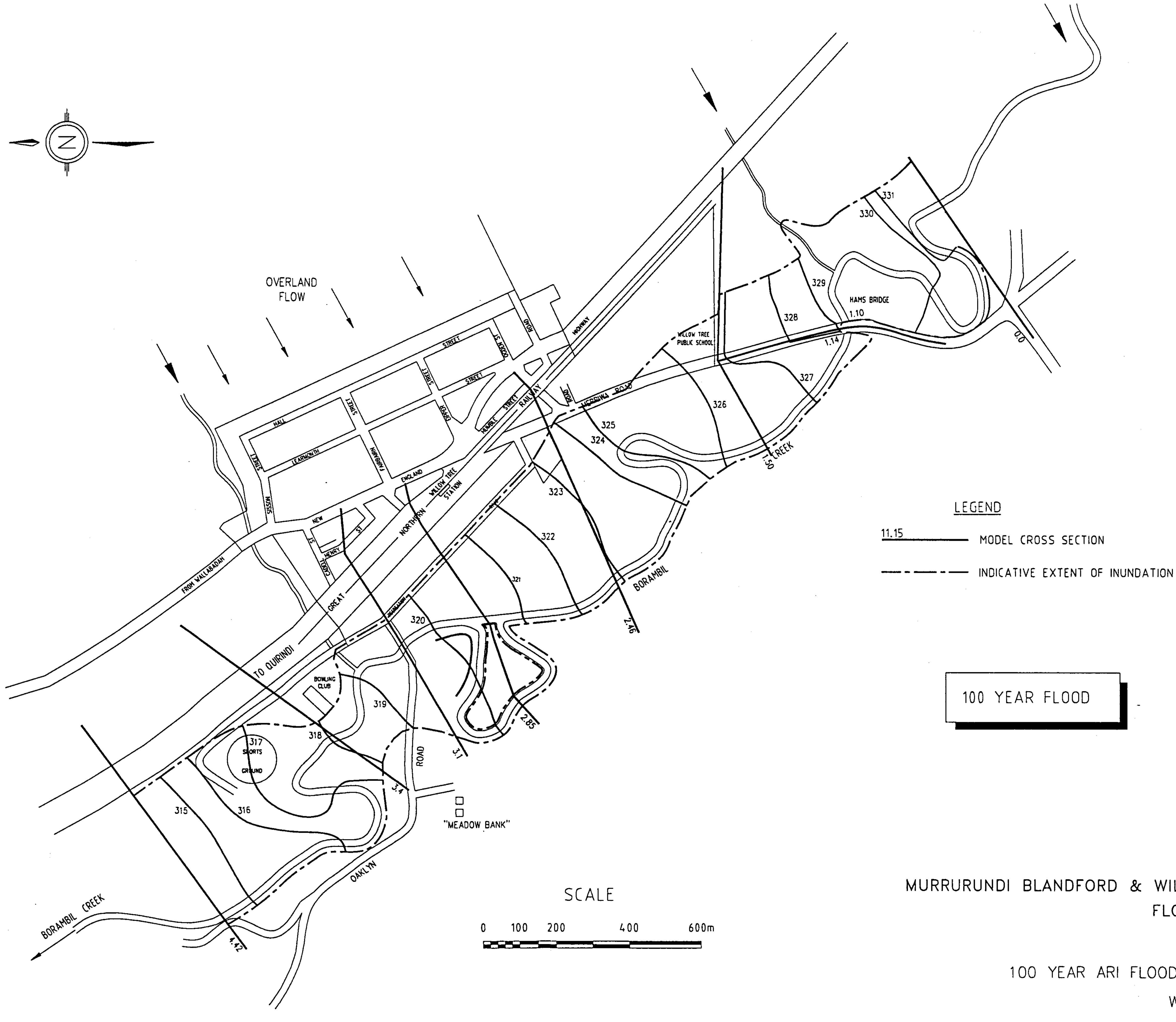


MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 6.5a

20 YEAR ARI FLOOD CONTOURS

WILLOW TREE

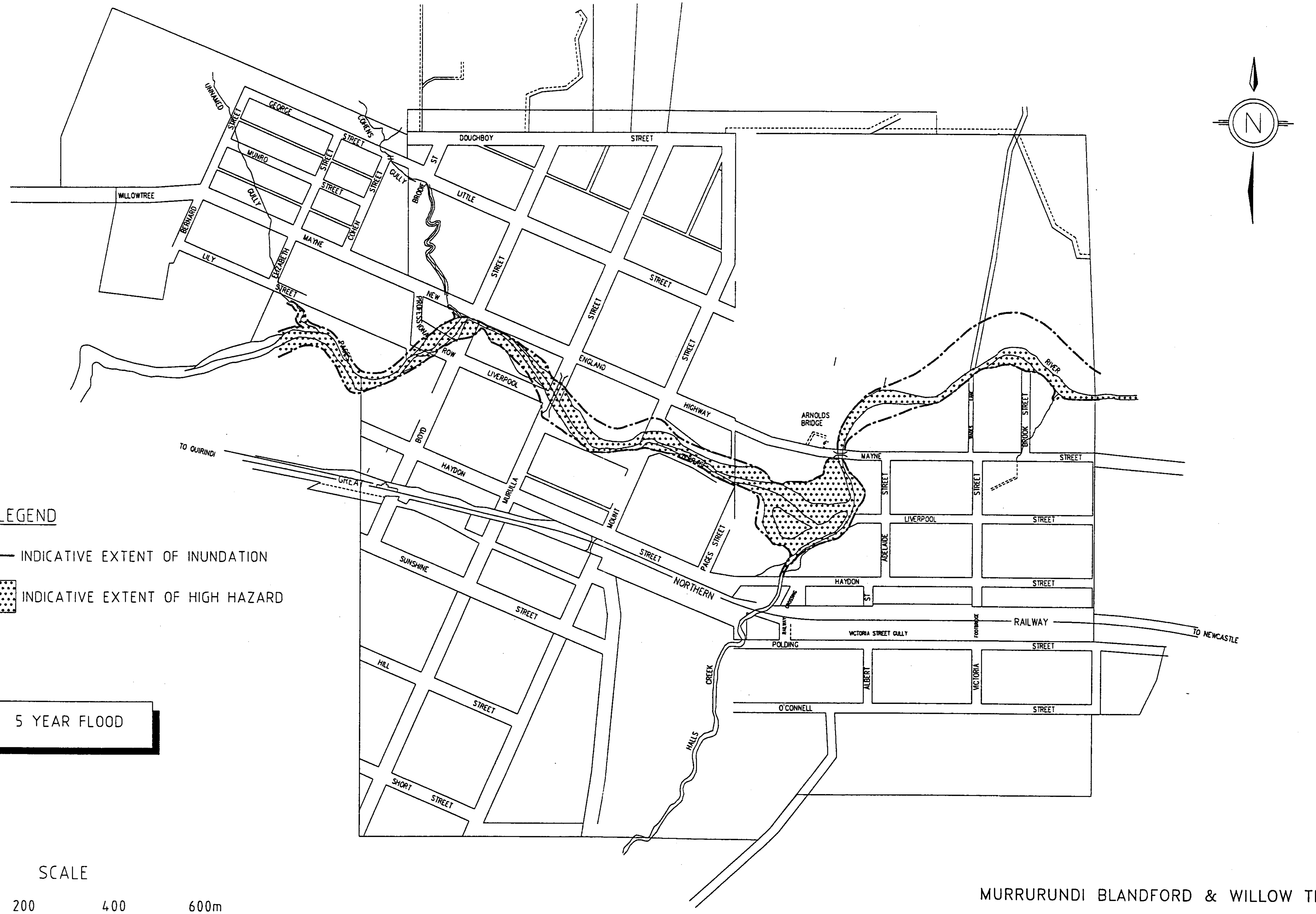


MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 6.5b

100 YEAR ARI FLOOD CONTOURS

WILLOW TREE



LEGEND

INDICATIVE EXTENT OF INUNDATION

INDICATIVE EXTENT OF HIGH HAZARD

5 YEAR FLOOD

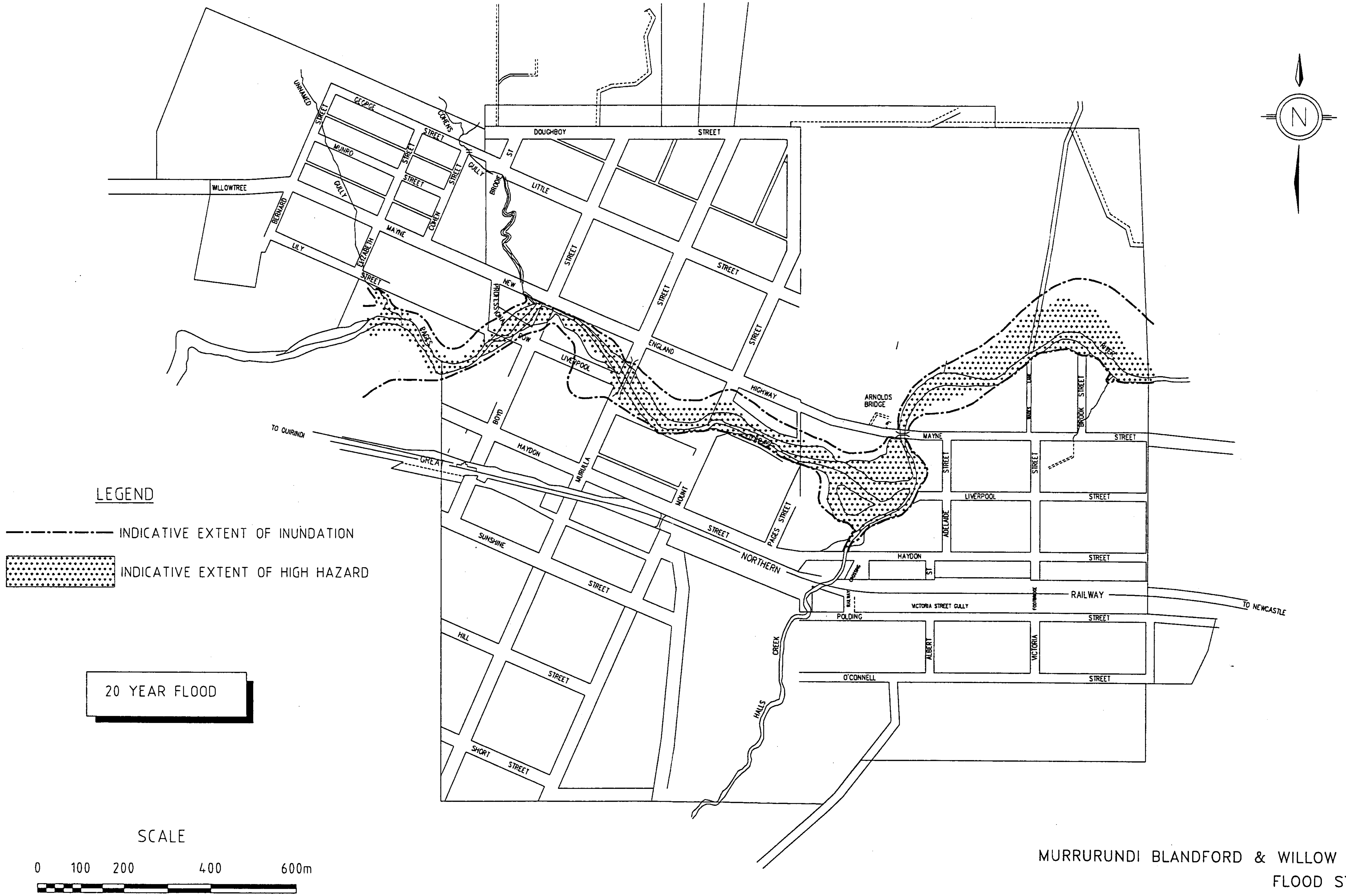
SCALE

0 100 200 400 600m

MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 6.6

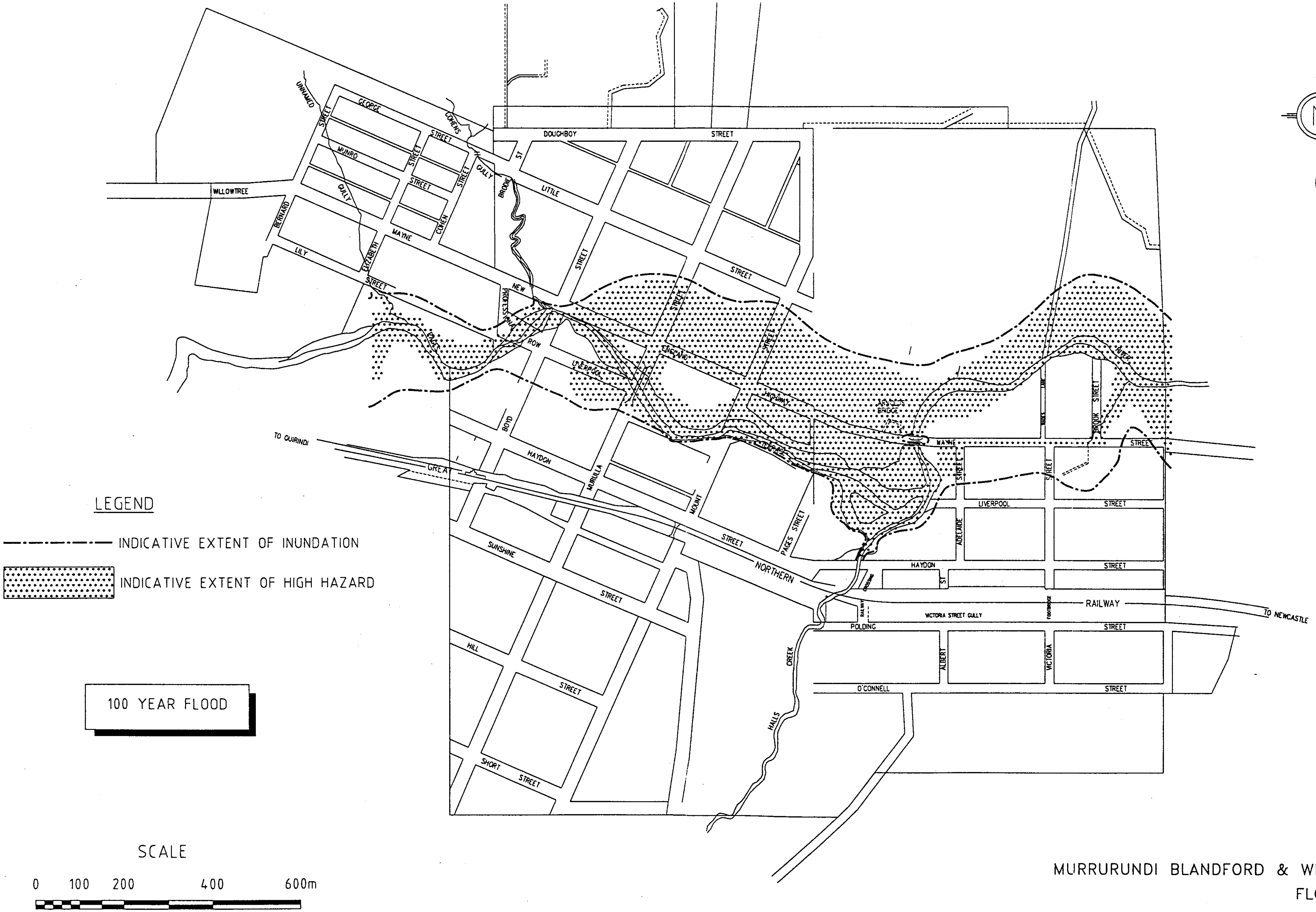
5 YEAR ARI PROVISIONAL HIGH HAZARD
MURRURUNDI



MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 6.6a

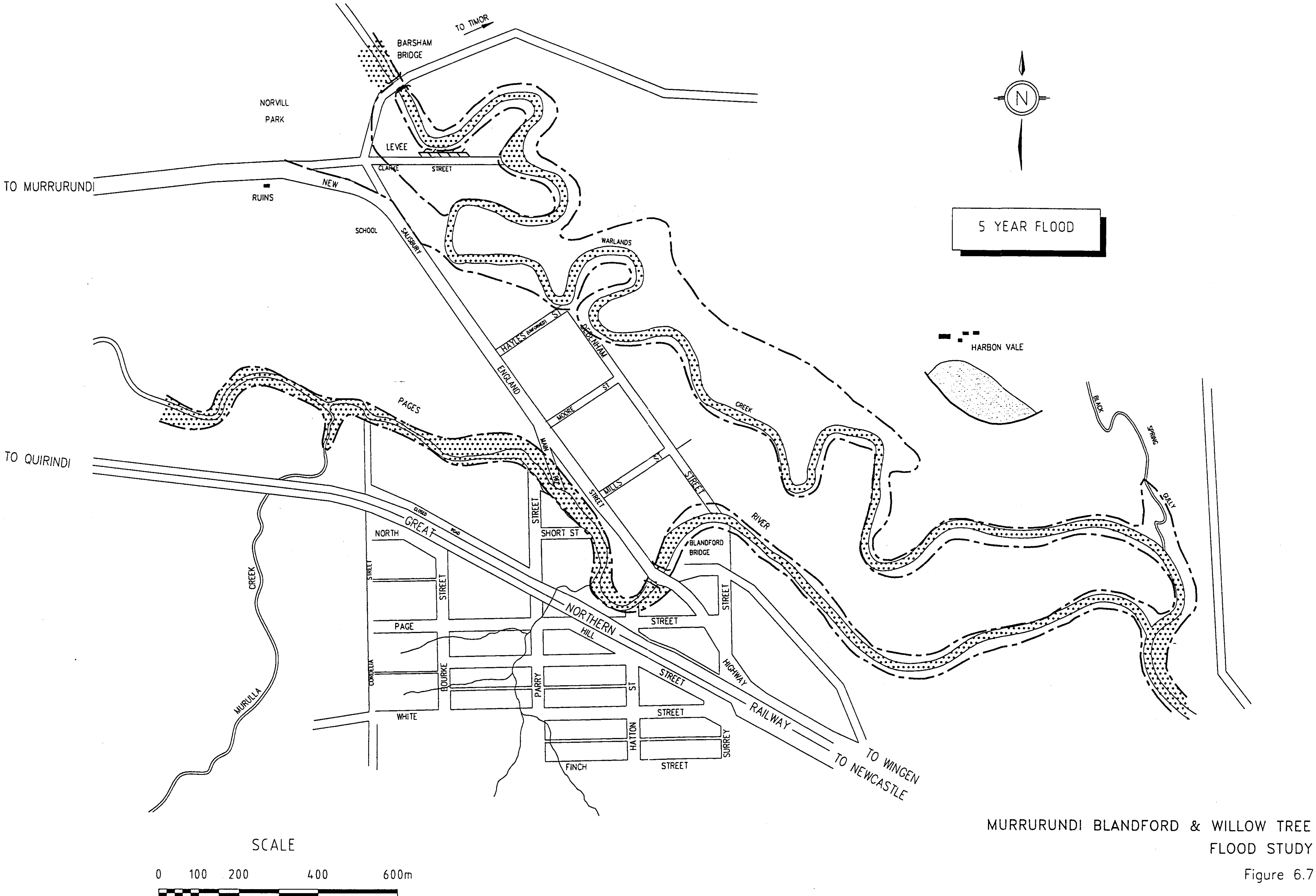
20 YEAR ARI PROVISIONAL HIGH HAZARD
MURRURUNDI



MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 6.6b

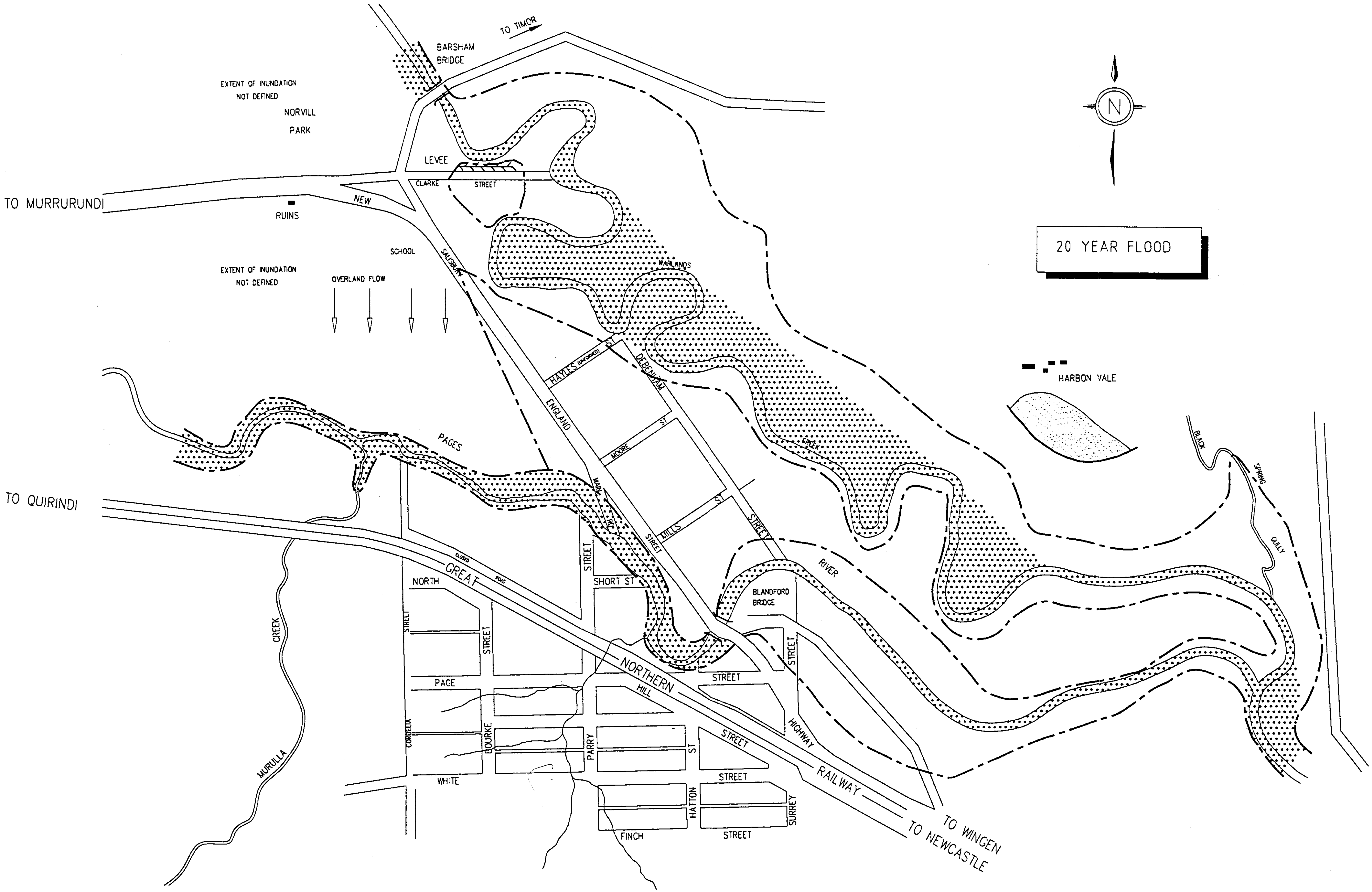
100 YEAR ARI PROVISIONAL HIGH HAZARD
MURRURUNDI



MURRURUNDI BLANDFORD & WILLOW TREE FLOOD STUDY

Figure 6.7

5 YEAR ARI PROVISIONAL HIGH HAZARD BLANDFORD



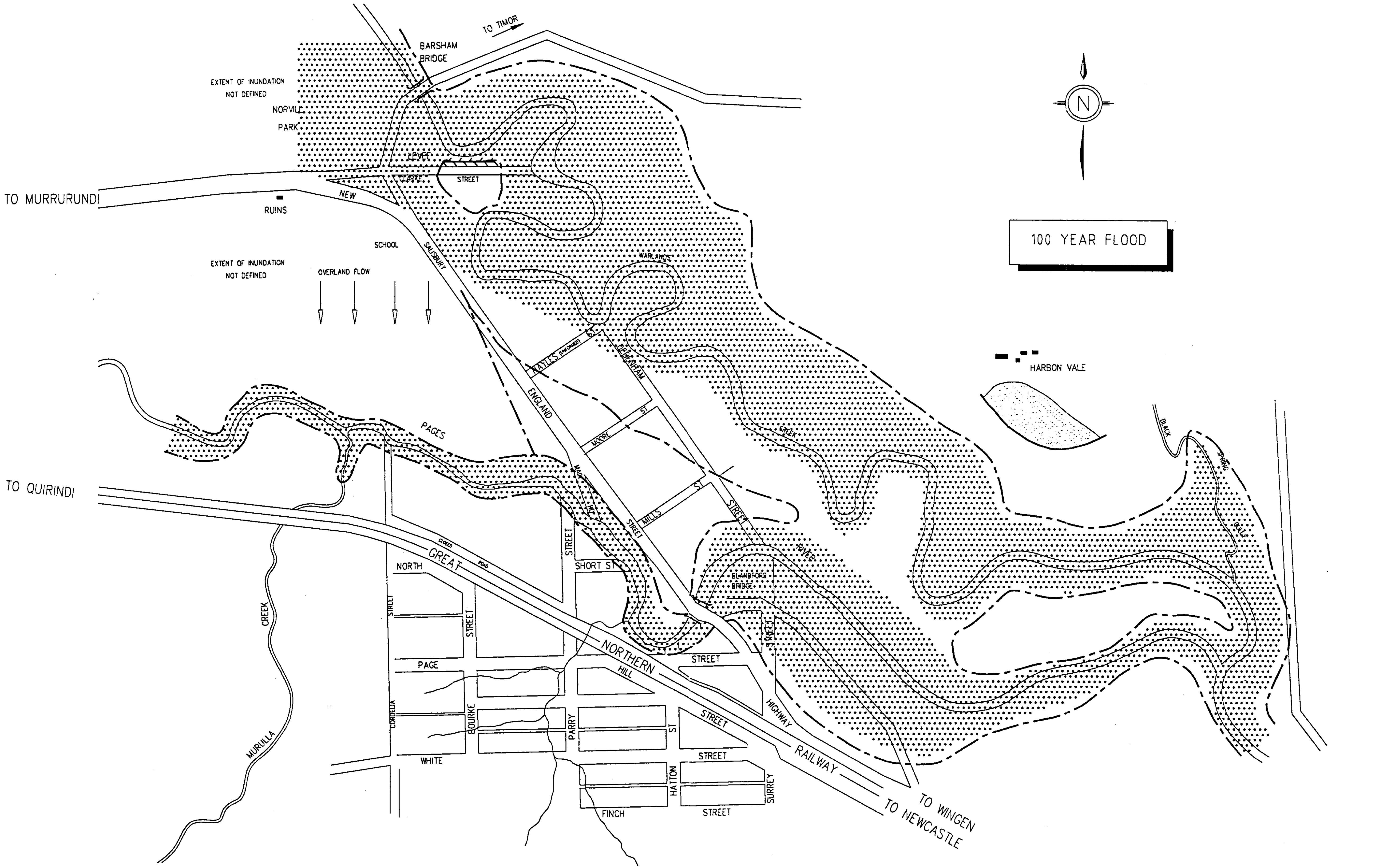
SCALE

0 100 200 400 600m

MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 6.7a

20 YEAR ARI PROVISIONAL HIGH HAZARD
BLANDFORD



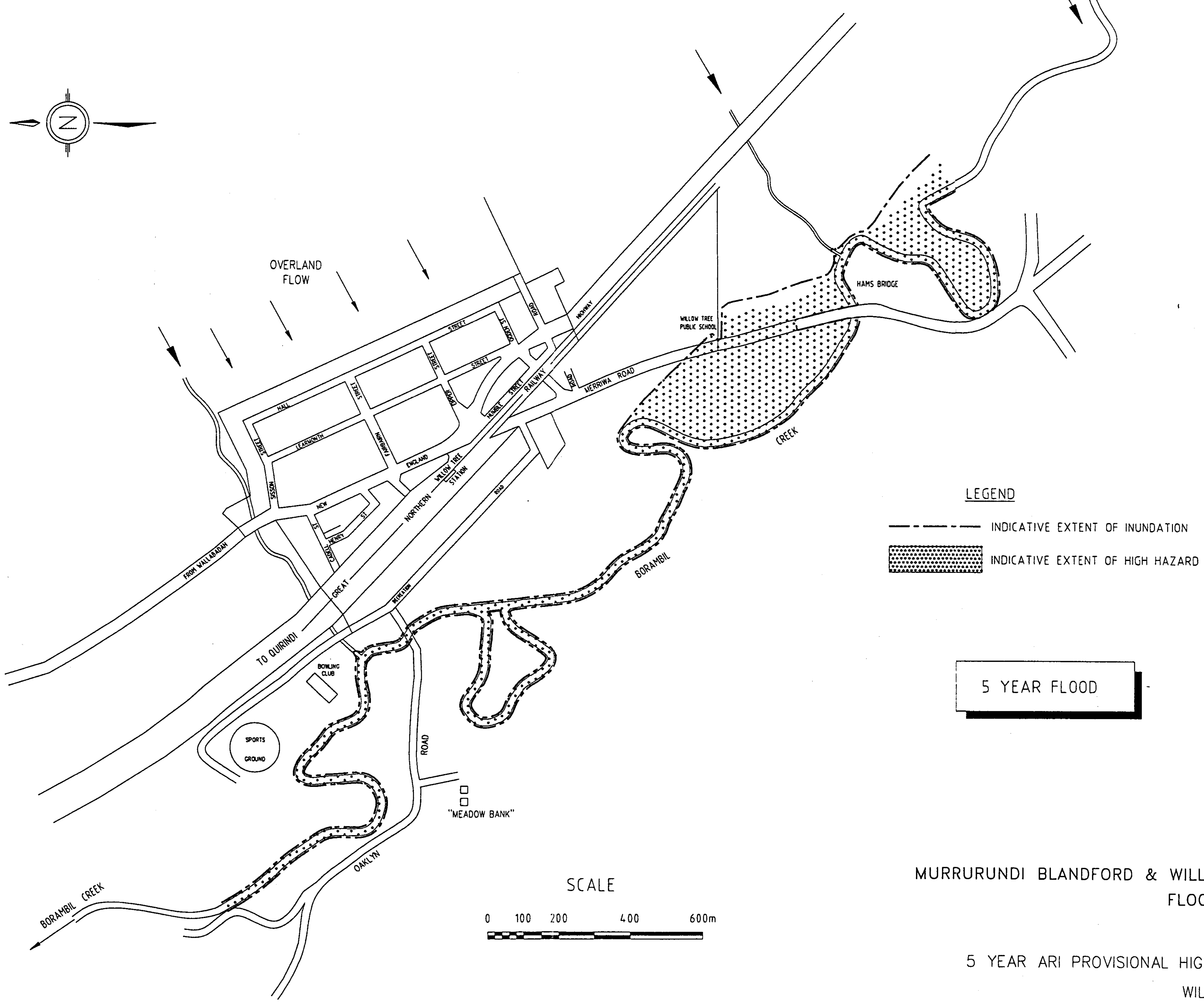
SCALE

0 100 200 400 600m

MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 6.7b

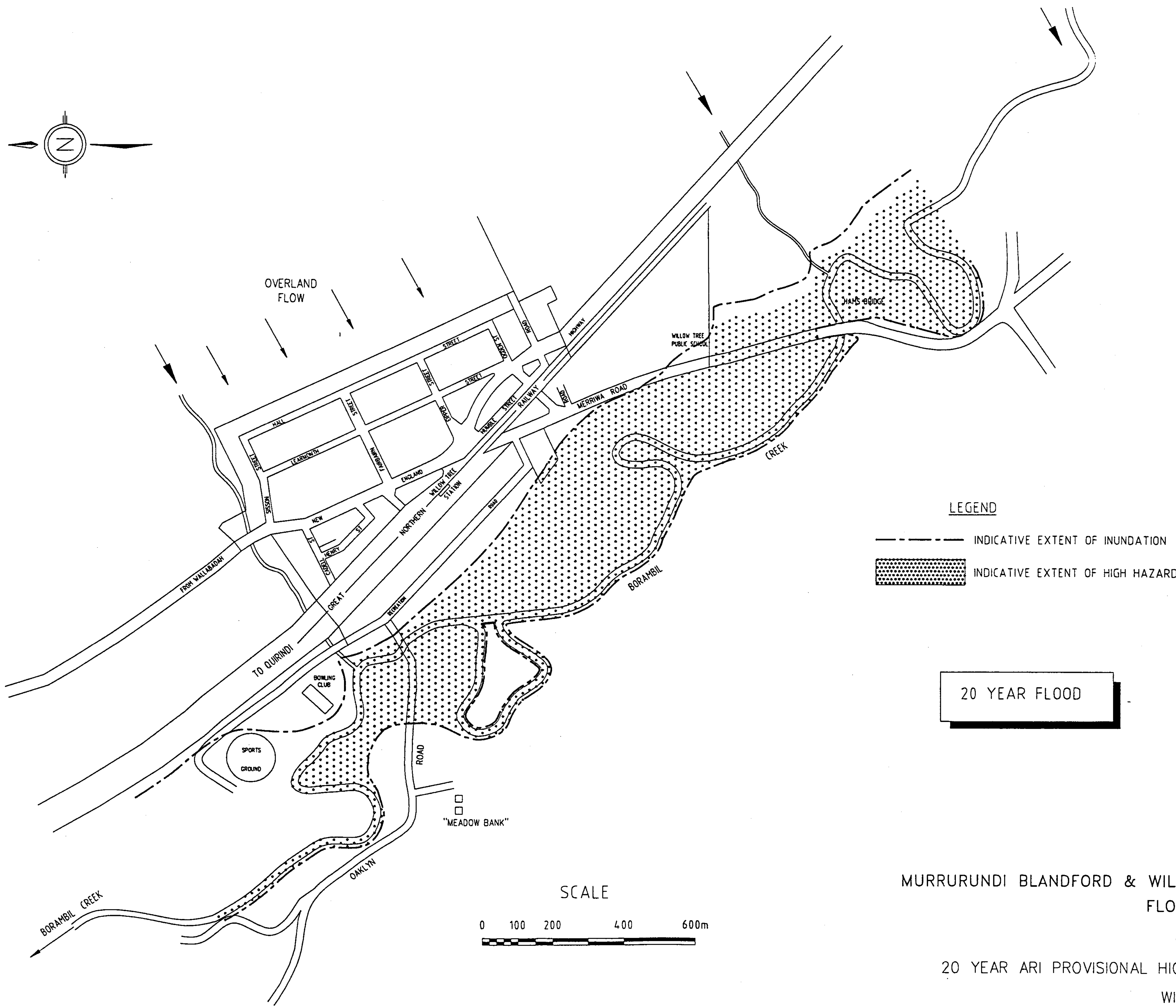
100 YEAR ARI PROVISIONAL HIGH HAZARD
BLANDFORD



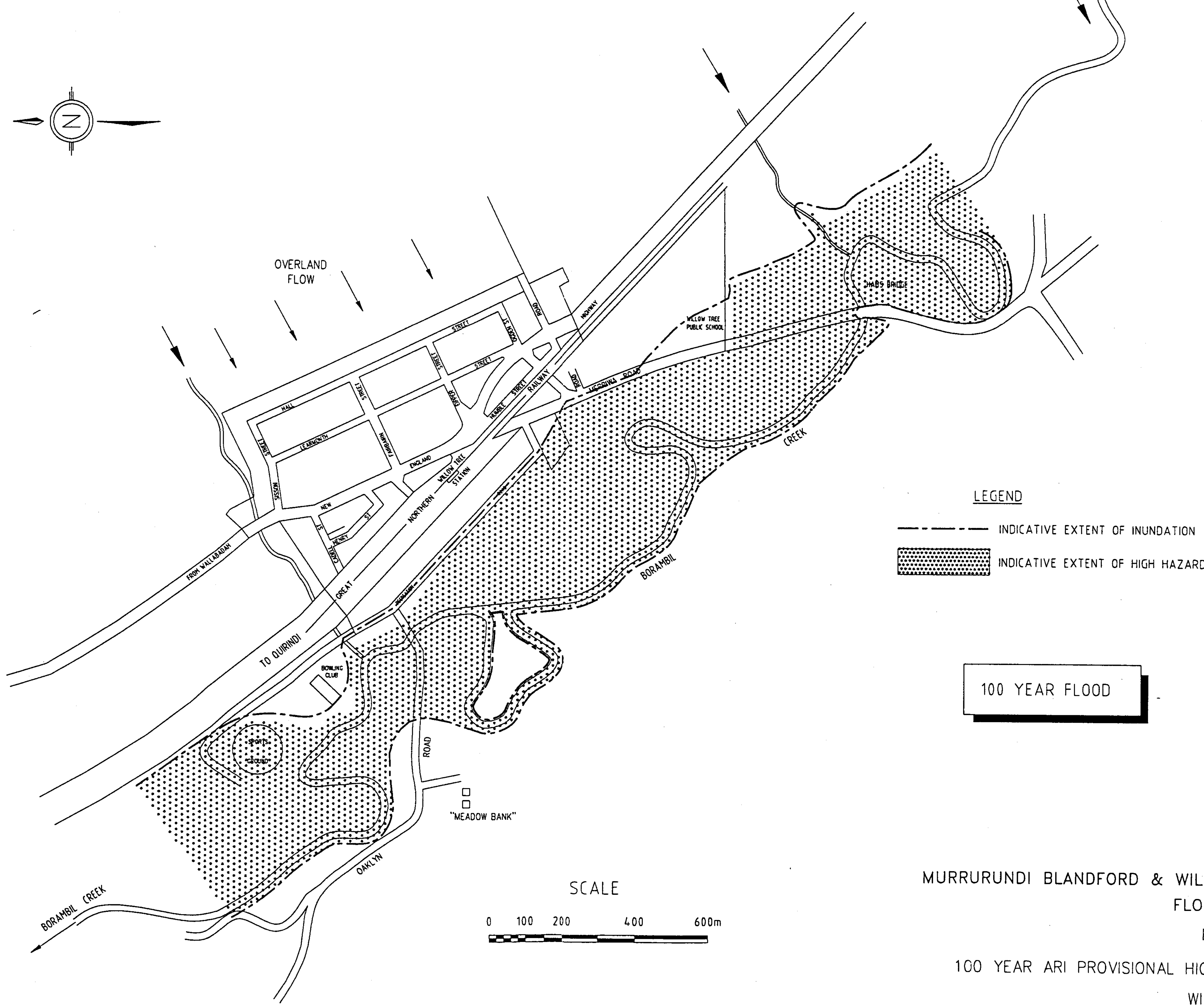
MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 6.8

5 YEAR ARI PROVISIONAL HIGH HAZARD
WILLOW TREE



MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY
Figure 6.8a
20 YEAR ARI PROVISIONAL HIGH HAZARD
WILLOW TREE



MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure 6.8b

100 YEAR ARI PROVISIONAL HIGH HAZARD
WILLOW TREE

7. FLOODING IN THE LOCAL GULLIES

7.1 General

This chapter deals with flooding in the local gullies which drain the foothills surrounding Murrurundi. Cohens Gully and Unnamed Gully drain the area on the northern side of Mayne Street and flow to the Pages River. Victoria Street Gully drains the foothills on the southern side of the township and drains to Halls Creek on the southern side of Polding Street. Problems in a drainage line on the eastern side of Willow Tree are also covered.

7.2 Victoria Street Gully

This gully is located on the southern side of town above the railway line. Figure 2.2 shows the location of this drainage line. An open drain runs along the western side of Victoria Street and crosses Polding Street in 600 and 450 mm diameter pipes. From this point, flows are conveyed in an open drain running in a westerly direction parallel with Polding Street to a 900 mm diameter Armco pipe. Between this point and the outfall to Halls Creek, part of the drain has been built over by an extension of the railway buildings. Flow is conveyed beneath the built up area in a 900 RCP. A 900 RCP then conveys flows beneath the railway crossing to Halls Creek.

The limited capacity of the drain has resulted in frequent surcharging over the years. Surcharge flows traverse the railway line and result in nuisance flooding in the residential areas fronting Haydon Street and in low lying areas between Haydon and Mayne Streets. In an effort to alleviate the problem, a levee was constructed on the northern side of the drain and was extended 30 m eastwards by Council after the January 1996 flood. The levee had been outflanked by this flood. The levee presently commences on the downstream side of the crossing at Polding Street and continues westwards for a distance of 180 m. However, it has not been tied into high ground near the railway overpass on the prolongation of Victoria Street. From site inspection, it appears that flows which surcharge the Polding Street crossing could outflank the levee and continue over the railway line. The surcharge location would be some distance to the east of the point where it used to occur prior to the levee construction.

The total catchment area of Victoria Street Gully increases from 13 ha at the Polding Street crossing, to 28 ha at the confluence with Halls Creek. One hundred year ARI peak flows, estimated by the Probabilistic Rational Method (PRM) increase from 1.2 m^3/s at Polding Street to 2.2 m^3/s at Halls Creek.

The hydraulic capacity at the Polding Street crossing is around 1.0 m^3/s assuming both pipes are functioning under inlet control. This is slightly less than the 100 year ARI peak flow, but is still adequate for a road culvert. As mentioned, outflanking the levee could occur in a major storm due to flows exceeding the capacity of the culvert, or if runoff cannot enter the open drain further to the south (upstream) and is forced to flow northwards along Victoria Street as overland flow.

The capacity of the 900 mm diameter Armco pipe, which is situated about 220 m upstream of the Halls Creek confluence, is about 1.6 m^3/s, compared with a 100 year ARI peak flow at that location of around 1.7 m^3/s. Further downstream, the capacity of the 900 RCP in the built over area is 1.3 m^3/s, compared with 2.2 m^3/s for the 100 year ARI peak flow. Flow which surcharges this pipe would fill the gully upstream and would then be directed onto the railway area. Depending on the duration of the storm, the surcharge could have sufficient volume to eventually reach Haydon Street.

Recommended action to reduce the probability of overflows in order of priority is as follows:

- Extend the existing levee eastwards to tie into high ground near the railway overpass;
- Investigate the path by which flows arrive at the upstream end of the Polding Street culvert and if necessary, formalise the channel to reduce overland flows along Victoria Street;
- Consider upgrading the 900 mm diameter pipe running beneath the built up area upstream of the Halls Creek confluence or alternatively, creating an escape path for surcharging flows by providing a drainage swale along the northern side of Polding Street. (Natural surface levels may however, preclude construction of an escape path);
- Amplify the capacity of the 900 RCP railway crossing, either by duplication or replacement with a larger line (minimum 1200 mm diameter).

7.3 Cohens Gully

Peak flows on Cohens Gully increase from 6.5 m^3/s at the 5 year ARI to 21 m^3/s for the 100 year ARI flood. The bridge at George Street has a waterway area of only 2.4 m^2 and is likely to be overtopped for comparatively small floods.

Between George Street and Mayne Street, the hydraulic modelling of Chapter 6 showed that the 100 year ARI flow would be contained within the creek except in the backwater of the road and pedestrian bridges at Mayne Street. Flood levels within the backwater are controlled by the level of the road. Model results showed that the road would not be surcharged due to flows from Cohens Gully, indicating that these two bridges have an adequate hydraulic capacity.

Some problems may arise further upstream in George Street where flows which overtop the bridge in that street could be directed down Cohen Street acting as a floodway. The risk of flooding could be reduced by upgrading the bridge.

Recommended action is to carry out a feasibility study to investigate the replacement of the existing bridge at George Street by a box culvert design which matches the capacity of the downstream channel of Cohens Gully. This will reduce the likelihood of George Street acting as an informal floodway, but could increase flows at Mayne Street.

The bridges at Mayne Street are considered to have adequate hydraulic capacity for their purpose, but further upstream there are several residential developments which could be flood liable in the event of a major flood. Channel improvements on Cohens Gully could be considered as a method for reducing the flood risk.

7.4 Unnamed Gully

Flood peaks on Unnamed Gully increase from 1.6 m^3/s, for the 5 year ARI to 5 m^3/s for the 100 year ARI flood. Council has recently excavated a drain along the route of Unnamed Gully extending for about 80 m downstream of Munro Street. The excavated drain is about 1.5 m deep and can contain the 100 year ARI discharge. However, between the present termination point of the drain and Mayne Street, the drainage line is indistinct and of low hydraulic capacity, resulting in the risk of flooding in surrounding residential allotments. Whilst the constraints of existing development and landform limit the opportunities for upgrading the capacity of the drain in this area, Council's Engineering Department has prepared a design for a drain which can accommodate a 20 year ARI flood of 2.4 m^3/s. The drain proposed is of open earth trapezoidal section , 3.6 m wide at the bottom and of minimum depth 0.5 m over most of its length. This is a reasonable approach, given the prevailing site conditions.

The hydraulics of the entrance to the Mayne Street culvert are quite poor. Approaching flow has to turn through a 90° bend to be conveyed through the waterway. The constriction imposed at the inlet limits the culvert capacity and could exacerbate flooding upstream.

Downstream of Mayne Street, flows exceed the creek capacity and are conveyed as wide sheet flow to the junction with the Pages River. Apart from one residence on the right bank of Unnamed Gully there are no potentially flood affected properties downstream of Mayne Street.

Due to the presence of the development referred to above, enlargement of the drainage line downstream of Munro Street does not appear feasible. Investigation should be undertaken of the feasibility of providing detention storage to reduce peak flows on Unnamed Gully. Storage sites upstream of Bernard Street or on the excavated section of drain should be considered.

7.5 Hall Street Drain at Willow Tree

Flooding problems have been experienced on residential land at Willow Tree, in the block bounded by Learmonth, Sisson, Hall and Fairbairn Streets, due to uncontrolled overland flow from the catchment above Hall Street. There is an existing open earth catch drain along the east side of Hall Street, but it is partly ineffective north of Fairbairn Street, due to inadequate cross sectional area and grade over part of its length, which includes a low spot at one location. Runoff is collected by the drain, then spilled across Hall Street into a shallow table drain along the western side of the street, and thence through residential land between Hall and Learmonth Streets. The catchment which generates this overflow is about 2.3 ha in area and the peak flow therefore is estimated at 0.23 m^3/s for the 100 year ARI at the problem location. The peak flow from the drain at its northern end is estimated at 0.39 m^3/s for the 100 year ARI.

The drain has adequate capacity for the estimated peak flow (for the 100 year ARI) over most of its length, and the overflow problem can be resolved by re-grading and re-forming the drain over the section between 160 and 300 m from Sisson Street, i.e. over a length of 140 m. This can be carried out at minimal cost and is to be arranged by Council's Engineering Department.

The total catchment of the culvert on the New England Highway just north of Sisson Street is 12.6 km^2, including the catchment of the drain referred to above, which is less than 0.5% of the total. Modification of the Hall Street catch drain as proposed and the consequent redirection of runoff should have negligible effect upon the watercourse west of Sisson Street and the Highway culvert.

Problems have been experienced at the culvert on the New England Highway. Surcharge of the culvert may occur during high flows, due mainly to debris blocking the entrance. The estimated 100 year ARI discharge on this catchment is 10.9 m^3/s and it is considered that the existing culvert which comprises a 2 cell 2140 mm x 2130 mm reinforced concrete structure, should be adequate.

The risk of blockage could be reduced by constructing a debris trap a short distance upstream of the culvert entrance.

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APPENDIX A

FLOOD LEVELS
FLOW AND VELOCITY DISTRIBUTIONS
DESIGN FLOODS

PAGES RIVER
5 year ARI

LOCATION	RIVER CHAINAGE	PEAK WATER LEVEL	DISCHARGE (m^3/s)			VELOCITY (m/s)		
			LEFT BANK	CHANNEL	RIGHT BANK	LEFT BANK	CHANNEL	RIGHT BANK
	0	481.5	0	150	0	0.0	3.1	0.0
ELIZABETH STREET	0.68	474.3	1	146	3	0.4	2.1	0.4
	0.76	473.6	18	130	3	1.2	2.7	0.8
	1	471.2	1	151	0	0.4	3.1	0.0
BOYD STREET	1.15	470.4	0	158	0	0.0	1.8	0.0
	1.35	469.4	NA	161	0	NA	1.8	0.0
SUSPENSION BRIDGE	1.41	469.0	0	161	0	0.0	2.2	0.0
MURRULA STREET	1.42	468.4	0	161	0	0.0	2.0	0.0
MOUNT STREET	1.68	466.7	0	162	0	0.0	2.2	0.0
	2.00	464.6	0	153	8	0.0	2.1	0.6
	2.10	464.2	45	109	19	0.9	2.0	0.7
ARNOLDS BRIDGE	2.26	463.5	NA	174	NA	NA	2.2	NA
	2.28	463.3	NA	174	NA	NA	2.5	NA
	2.40	462.3	0	144	32	0.0	2.6	0.9
WADES LANE	2.64	460.7	38	138	0	1.0	2.7	0.0
	3.04	457.6	9	181	0	0.7	3.5	0.0
	3.40	454.9	2	159	26	0.3	2.8	0.5

PAGES RIVER Cont.
5 year ARI

LOCATION	RIVER CHAINAGE	PEAK WATER LEVEL	DISCHARGE (m^3/s)			VELOCITY (m/s)		
			LEFT BANK	CHANNEL	RIGHT BANK	LEFT BANK	CHANNEL	RIGHT BANK
BENHAMS BRIDGE	6.40	437.1	0	194	1	0.0	2.1	0.4
	6.43	436.6	0	194	0	0.0	2.6	0.1
	8.9	421.6	0	203	0	0.0	1.6	0.0
	9.55	418.7	0	211	0	0.0	1.9	0.0
	9.9	417.3	0	210	0	0.0	2.5	0.0
	10.15	415.7	0	209	0	0.0	3.4	0.0
BLANDFORD BRIDGE	10.44	413.7	0	209	0	0.0	1.8	0.0
	10.48	413.6	0	209	0	0.0	1.9	0.0
	11.15	412.1	0	209	0	0.0	2.1	0.0
	11.8	410.5	0	207	0	0.0	1.7	0.0
WARLANDS CREEK	12.05	410.1	0	341	0	0.0	3.6	0.0
	12.25	409.1	0	341	0	0.0	3.8	0.0

WARLANDS CREEK
5 year ARI

LOCATION	RIVER CHAINAGE	PEAK WATER LEVEL	DISCHARGE (m^3/s)			VELOCITY (m/s)		
			LEFT BANK	CHANNEL	RIGHT BANK	LEFT BANK	CHANNEL	RIGHT BANK
BARSHAM BRIDGE	0.2	428.0	0	122	15	0.0	1.3	0.5
	0.22	427.4	0	122	15	0.0	1.8	0.5
CLARKE STREET	0.9	423.5	2	121	14	0.6	1.9	0.5
	1.45	421.7	15	104	15	0.5	1.3	0.6
MOORE STREET	2.35	418.8	3	131	0	0.2	1.0	0.0
MILLS STREET	2.65	416.3	29	104	0	0.5	1.4	0.0
	3.4	411.6	0	133	0	0.0	2.4	0.0
	3.9	410.6	0	136	0	0.0	1.6	0.0
PAGES RIVER								

BORAMBIL CREEK
5 year ARI

LOCATION	RIVER CHAINAGE	PEAK WATER LEVEL	DISCHARGE (m^3/s)			VELOCITY (m/s)		
			LEFT BANK	CHANNEL	RIGHT BANK	LEFT BANK	CHANNEL	RIGHT BANK
	0	330.3	136	156	17	0.8	1.9	0.5
HAMS BRIDGE	1.1	327.5	0	278	36	0.0	1.4	1.0
	1.14	327.2	0	278	35	0.0	1.8	0.7
	1.5	326.1	0	263	50	0.0	3.2	1.4
	2.46	321.3	0	311	0	0.0	3.8	0.0
	2.85	318.9	0	313	0	0.0	2.4	0.0
OAKLYN ROAD	3.1	317.9	0	314	0	0.0	2.4	0.0
	3.4	316.2	0	319	0	0.0	2.9	0.0
	4.42	313.3	0	326	0	0.0	2.8	0.0

NOTE: LEVELS ARE TO A LOCAL DATUM
SSM 77535 AT NEW ENGLAND HIGHWAY = RL 323.53 (LOCAL DATUM)

PAGES RIVER
20 year ARI

LOCATION	RIVER CHAINAGE	PEAK WATER LEVEL	DISCHARGE (m^3/s)			VELOCITY (m/s)		
			LEFT BANK	CHANNEL	RIGHT BANK	LEFT BANK	CHANNEL	RIGHT BANK
	0	481.8	5	245	2	0.5	3.8	0.4
ELIZABETH STREET	0.68	474.8	11	207	30	0.7	2.2	0.6
	0.76	474.1	43	191	15	1.4	3.1	0.9
	1	471.9	17	234	0	0.9	3.2	0.1
BOYD STREET	1.15	471.1	0	259	0	0.0	1.9	0.2
	1.35	470.1	NA	250	13	NA	2.0	0.4
SUSPENSION BRIDGE	1.41	469.6	0	263	0	0.0	2.6	0.0
MURRULA STREET	1.42	468.9	0	263	0	0.0	2.2	0.0
MOUNT STREET	1.68	467.3	0	266	0	0.0	2.2	0.0
	2.00	465.2	2	226	38	0.3	2.2	0.9
	2.10	464.9	93	146	47	1.0	2.0	0.9
ARNOLDS BRIDGE	2.26	464.2	NA	286	NA	NA	2.6	NA
	2.28	464.0	NA	286	NA	NA	2.9	NA
	2.40	462.9	0	202	84	0.1	2.8	1.2
WADES LANE	2.64	461.1	87	199	0	1.4	3.3	0.0
	3.04	457.9	44	267	1	1.1	4.4	0.4
	3.40	455.1	13	221	79	0.6	3.5	0.8

PAGES RIVER Cont.
20 year ARI

LOCATION	RIVER CHAINAGE	PEAK WATER LEVEL	DISCHARGE (m^3/s)			VELOCITY (m/s)		
			LEFT BANK	CHANNEL	RIGHT BANK	LEFT BANK	CHANNEL	RIGHT BANK
BENHAMS BRIDGE	6.40	438.4	48	263	47	0.6	1.7	0.6
	6.43	437.7	17	316	20	0.6	2.5	0.6
	8.9	422.7	0	379	0	0.0	1.9	0.0
	9.55	420.2	0	417	0	0.0	2.7	0.0
	9.9	418.5	26	391	0	0.8	2.9	0.0
	10.15	416.7	0	417	0	0.0	3.4	0.0
BLANDFORD BRIDGE	10.44	415.3	0	417	0	0.1	2.2	0.0
	10.48	415.2	0	417	0	0.0	2.2	0.0
	11.15	413.7	27	359	24	0.4	1.8	0.4
	11.8	412.4	0	402	0	0.0	1.7	0.0
WARLANDS CREEK	12.05	412.1	0	677	0	0.0	2.9	0.0
	12.25	411.4	0	677	0	0.0	4.2	0.0

WARLANDS CREEK
20 year ARI

LOCATION	RIVER CHAINAGE	PEAK WATER LEVEL	DISCHARGE (m^3/s)			VELOCITY (m/s)		
			LEFT BANK	CHANNEL	RIGHT BANK	LEFT BANK	CHANNEL	RIGHT BANK
BARSHAM BRIDGE	0.2	428.7	0	206	88	0.0	2.3	0.7
	0.22	427.9	0	206	88	0.0	2.3	0.7
CLARKE STREET	0.9	424.1	32	152	108	0.5	1.8	1.5
	1.45	422.3	72	160	46	0.7	1.6	0.9
MOORE STREET	2.35	419.4	34	230	12	0.5	1.5	0.3
MILLS STREET	2.65	416.8	117	164	0	0.8	1.9	0.1
	3.4	413.8	79	197	0	0.5	1.7	0.0
	3.9	412.7	7	283	0	0.4	1.6	0.0
PAGES RIVER								

BORAMBIL CREEK
20 year ARI

LOCATION	RIVER CHAINAGE	PEAK WATER LEVEL	DISCHARGE (m^3/s)			VELOCITY (m/s)		
			LEFT BANK	CHANNEL	RIGHT BANK	LEFT BANK	CHANNEL	RIGHT BANK
	0	330.8	278	220	59	1.1	2.3	0.7
HAMS BRIDGE	1.1	328.4	0	412	168	0.0	1.3	1.8
	1.14	327.5	0	412	154	0.0	1.8	1.0
	1.5	326.2	0	305	262	0.0	4.4	2.0
	2.46	322.3	0	504	74	0.0	4.3	2.4
	2.85	319.8	0	494	73	0.0	2.9	1.6
OAKLYN ROAD	3.1	319.0	11	566	0	2.5	2.8	0.0
	3.4	317.3	10	577	0	2.8	3.5	0.0
	4.42	314.1	0	437	147	0.0	3.5	0.8

NOTE: LEVELS ARE TO A LOCAL DATUM
SSM 77535 AT NEW ENGLAND HIGHWAY = RL 323.53 (LOCAL DATUM)

PAGES RIVER
50 year ARI

LOCATION	RIVER CHAINAGE	PEAK WATER LEVEL	DISCHARGE (m^3/s)			VELOCITY (m/s)		
			LEFT BANK	CHANNEL	RIGHT BANK	LEFT BANK	CHANNEL	RIGHT BANK
	0	482.0	17	273	6	0.7	3.8	0.6
ELIZABETH STREET	0.68	475.0	17	229	47	0.7	2.2	0.7
	0.76	474.3	56	213	25	1.4	3.3	0.9
	1	472.1	27	264	0	1.0	3.4	0.3
BOYD STREET	1.15	471.2	0	300	6	0.0	2.0	0.6
	1.35	470.3	NA	278	29	NA	2.0	0.4
SUSPENSION BRIDGE	1.41	469.7	3	304	0	0.0	2.8	0.0
MURRULA STREET	1.42	469.1	3	304	0	0.0	2.3	0.0
MOUNT STREET	1.68	467.5	0	307	0	0.0	2.3	0.0
	2.00	465.5	6	249	52	0.5	2.1	1.0
	2.10	465.2	114	159	59	1.1	2.0	1.0
ARNOLDS BRIDGE	2.26	464.6	NA	322	NA	NA	2.6	NA
	2.28	464.1	NA	322	NA	NA	3.0	NA
	2.40	463.1	2	219	102	0.3	2.9	1.3
WADES LANE	2.64	461.2	104	219	0	1.5	3.5	0.0
	3.04	458.1	71	287	5	1.2	4.5	0.5
	3.40	455.1	19	240	105	0.7	3.7	0.9

PAGES RIVER Cont.
50 year ARI

LOCATION	RIVER CHAINAGE	PEAK WATER LEVEL	DISCHARGE (m^3/s)			VELOCITY (m/s)		
			LEFT BANK	CHANNEL	RIGHT BANK	LEFT BANK	CHANNEL	RIGHT BANK
BENHAMS BRIDGE	6.40	438.8	69	279	65	0.6	1.6	0.7
	6.43	437.9	32	348	33	0.7	2.6	0.8
	8.9	423.0	0	439	0	0.0	2.0	0.0
	9.55	420.6	0	499	0	0.0	3.0	0.0
	9.9	418.9	48	449	0	0.9	3.0	0.0
	10.15	417.1	0	496	0	0.0	3.3	0.0
BLANDFORD BRIDGE	10.44	415.7	3	493	1	0.4	2.3	0.2
	10.48	415.6	2	494	0	0.3	2.3	0.2
	11.15	414.0	52	366	53	0.5	1.7	0.4
	11.8	412.9	0	470	0	0.0	1.7	0.0
WARLANDS CREEK	12.05	412.6	0	808	0	0.0	2.7	0.0
	12.25	412.0	0	808	0	0.0	4.0	0.0

WARLANDS CREEK
50 year ARI

LOCATION	RIVER CHAINAGE	PEAK WATER LEVEL	DISCHARGE (m^3/s)			VELOCITY (m/s)		
			LEFT BANK	CHANNEL	RIGHT BANK	LEFT BANK	CHANNEL	RIGHT BANK
BARSHAM BRIDGE	0.2	429.1	0	247	119	0.0	2.5	0.8
	0.22	428.0	0	247	119	0.0	2.6	0.8
CLARKE STREET	0.9	424.2	42	158	164	0.6	1.8	1.7
	1.45	422.7	106	155	58	0.6	1.4	0.7
MOORE STREET	2.35	419.6	46	254	19	0.5	1.6	0.4
MILLS STREET	2.65	416.8	132	186	0	0.9	2.1	0.1
	3.4	414.0	129	199	0	0.6	1.6	0.0
	3.9	413.3	26	316	0	0.5	1.5	0.0
PAGES RIVER								

BORAMBIL CREEK
50 year ARI

LOCATION	RIVER CHAINAGE	PEAK WATER LEVEL	DISCHARGE (m^3/s)			VELOCITY (m/s)		
			LEFT BANK	CHANNEL	RIGHT BANK	LEFT BANK	CHANNEL	RIGHT BANK
	0	331.0	342	242	83	1.1	2.4	0.8
HAMS BRIDGE	1.1	328.6	0	446	251	0.0	1.2	2.1
	1.14	327.5	0	446	229	0.0	1.9	1.1
	1.5	326.2	0	317	358	0.0	4.7	1.4
	2.46	322.6	0	589	105	0.0	4.4	3.2
	2.85	320.0	0	560	106	0.0	3.2	1.7
OAKLYN ROAD	3.1	319.3	20	678	0	1.7	2.7	0.0
	3.4	317.6	20	695	0	2.9	3.9	0.0
	4.42	314.3	0	475	239	0.0	3.6	0.9

NOTE: LEVELS ARE TO A LOCAL DATUM
SSM 77535 AT NEW ENGLAND HIGHWAY = RL 323.53 (LOCAL DATUM)

PAGES RIVER
100 year ARI

LOCATION	RIVER CHAINAGE	PEAK WATER LEVEL	DISCHARGE (m^3/s)			VELOCITY (m/s)		
			LEFT BANK	CHANNEL	RIGHT BANK	LEFT BANK	CHANNEL	RIGHT BANK
	0	482.3	66	323	25	1.1	3.6	1.0
ELIZABETH STREET	0.68	475.2	33	285	92	0.9	2.4	0.9
	0.76	474.6	92	257	63	1.4	3.4	1.0
	1	472.4	52	326	3	1.3	3.6	0.5
BOYD STREET	1.15	471.5	0	391	39	0.0	2.3	0.5
	1.35	470.7	NA	345	72	NA	2.2	0.7
SUSPENSION BRIDGE	1.41	470.1	13	390	0	0.0	3.0	0.0
MURRULA STREET	1.42	469.4	13	390	0	0.0	2.5	0.0
MOUNT STREET	1.68	467.8	38	386	0	0.0	2.4	0.0
	2.00	465.8	14	285	74	0.7	2.2	1.1
	2.10	465.6	146	182	80	1.1	2.0	1.0
ARNOLDS BRIDGE	2.26	464.9	NA	380	NA	NA	2.7	NA
	2.28	464.4	NA	380	NA	NA	3.3	NA
	2.40	463.4	12	271	155	0.6	3.2	1.5
WADES LANE	2.64	461.4	160	277	0	1.8	4.0	0.0
	3.04	458.3	135	332	32	1.5	4.7	0.9
	3.40	455.3	40	280	176	1.0	3.9	1.2

PAGES RIVER Cont.
100 year ARI

LOCATION	RIVER CHAINAGE	PEAK WATER LEVEL	DISCHARGE (m^3/s)			VELOCITY (m/s)		
			LEFT BANK	CHANNEL	RIGHT BANK	LEFT BANK	CHANNEL	RIGHT BANK
BENHAMS BRIDGE	6.40	439.3	113	347	105	0.8	1.7	0.8
	6.43	438.2	60	445	60	1.0	3.1	1.0
	8.9	423.8	0	599	0	0.0	2.2	0.0
	9.55	421.2	0	678	0	0.0	3.7	0.0
	9.9	419.3	104	574	0	1.1	3.3	0.0
	10.15	417.6	0	678	0	0.0	3.3	0.0
BLANDFORD BRIDGE	10.44	416.4	13	654	11	0.6	2.6	0.5
	10.48	416.2	10	660	7	0.6	2.7	0.5
	11.15	414.5	87	370	92	0.5	1.5	0.5
	11.8	413.5	0	539	0	0.0	2.0	0.0
WARLANDS CREEK	12.05	413.3	0	1002	0	0.0	2.4	0.0
	12.25	412.7	29	973	0	0.5	3.8	0.0

WARLANDS CREEK
100 year ARI

LOCATION	RIVER CHAINAGE	PEAK WATER LEVEL	DISCHARGE (m^3/s)			VELOCITY (m/s)		
			LEFT BANK	CHANNEL	RIGHT BANK	LEFT BANK	CHANNEL	RIGHT BANK
BARSHAM BRIDGE	0.2	429.4	0	286	148	0.0	2.8	0.9
	0.22	428.1	0	286	148	0.0	2.9	0.9
CLARKE STREET	0.9	424.2	51	166	218	0.6	1.8	1.9
	1.45	422.8	141	170	72	0.6	1.5	0.8
MOORE STREET	2.35	419.7	63	282	34	0.6	1.7	0.4
MILLS STREET	2.65	416.9	164	215	0	1.1	2.4	0.2
	3.4	414.5	235	220	0	0.7	1.6	0.0
	3.9	413.9	75	387	0	0.6	1.6	0.0
PAGES RIVER								

BORAMBIL CREEK
100 year ARI

LOCATION	RIVER CHAINAGE	PEAK WATER LEVEL	DISCHARGE (m^3/s)			VELOCITY (m/s)		
			LEFT BANK	CHANNEL	RIGHT BANK	LEFT BANK	CHANNEL	RIGHT BANK
	0	331.4	492	287	147	1.2	2.5	1.0
HAMS BRIDGE	1.1	329.0	0	530	426	0.0	1.2	2.6
	1.14	327.7	0	530	371	0.0	2.0	1.4
	1.5	326.4	0	359	541	0.0	3.5	2.5
	2.46	323.0	0	700	256	0.0	4.8	2.0
	2.85	320.3	0	633	254	0.0	3.5	1.9
OAKLYN ROAD	3.1	319.8	81	873	0	3.2	3.0	0.0
	3.4	318.0	81	900	0	3.0	4.3	0.0
	4.42	314.6	0	555	426	0.0	3.9	1.1

NOTE: LEVELS ARE TO A LOCAL DATUM
SSM 77535 AT NEW ENGLAND HIGHWAY = RL 323.53 (LOCAL DATUM)

APPENDIX B

RECONNAISSANCE INVESTIGATION

FLOOD OF 25 JANUARY 1996

The following information was obtained from the reconnaissance study commissioned by Council following the January 1996 flood (Bush 1996). A similar investigation was carried out after the smaller January 1991 flood (Bush 1991).

B.1 Murrurundi

Of those properties inspected in Murrurundi, 21 premises had been inundated by floodwater to varying degrees and many others had sheds and other outbuildings affected by flooding. Those areas subject to flooding were predominantly located adjacent to the Pages River, however a number of residences were also flooded as a result of the river breaking its banks and flowing across roads and from local stormwater run-off.

On the western side of Arnolds Bridge, the river broke its banks in a number of locations (illustrated in Figure B.1 and flooded rural lands, the New England Highway, Wilson Memorial Park and some residences adjacent to the river. Run-off from the New England Highway contributed to the flooding of the premises at 118 Mayne Street.

On the eastern side of the Bridge, a total of 12 business, 3 residences (including the flat behind Seckold's Butchery) and the rear of a studio were inundated as a result of the Pages River breaking its banks behind 19 Adelaide Street. The studio and dwelling at the western end of Mayne Street (in the vicinity of Brook Street) were affected by flooding due mainly to the inability of the stormwater gully in that locality to dispose of run-off in conjunction with the flow of water from the commercial centre.

In Haydon Street, run-off from stormwater also became a concern in the low-lying areas adjacent to the Great Northern railway line. A number of dwellings located on the southern side of Haydon Street between the railway crossing and Adelaide Street were flooded.

Murravale was also flooded as a result of stormwater flowing across Haydon Street from the railway line and all residents were evacuated. It is noted that Council have since completed works in Haydon Street to prevent future stormwater problems in these localities.

B.2 Murrurundi to Blandford

Areas affected by the flooding between Murrurundi and Blandford are situated on the floodplain adjacent to critical bends in the Pages River. The narrow channel of the Pages River, in conjunction with the fast flow of the floodwaters, resulted in extensive damage to rural fences, creek crossings, pumps and floodgates.

Immediately to the east of Murrurundi, Campbells Creek exacerbated the flood problem by impeding the movement of water under Campbells Creek Bridge on the New England Highway. Water crossed the highway, approximately 100 m east of Campbells Creek Bridge. This water flowed over the highway to join the Pages River to the north.

Emirates Park sustained the majority of damage in this area. Approximately 3 km of fencing was destroyed.

B.3 Blandford

Warlands Creek broke its banks on the north-western side of the Warlands Creek Bridge and inundated land in the vicinity of Norvill Park and the Blandford School. Although traffic was cut in this area, the school buildings, school residence and new dwelling recently constructed in Clarke Street were not inundated by flood waters. The residence opposite the school however, was severely flooded.

Once over the highway the water flowed towards the lucerne paddock to the west of the school yard and continued flowing over the flats behind the dwelling located to the south-east of the school residence and joined the Pages River further downstream.

As indicated on Figure B.2, Warlands Creek also broke its banks on the north-west of Debenham Street. These breakouts, in conjunction with flow from the New England Highway, flowed over the flats behind Sipple's dwelling and inundated properties in Moore, Mills and Debenham Streets.

Whilst many properties received damage to outbuildings, 1 Moore Street is the only occupied residence which was identified as being inundated by flooding. Unfortunately, not all residents were at home during the flood survey in Blandford and therefore more research is required to determine whether additional dwellings were flooded in this locality.

Downstream of Blandford, approximately 100 m from the intersection of the Pages River and Warlands Creek, the Bridge on Haydons Lane was demolished by floodwaters.

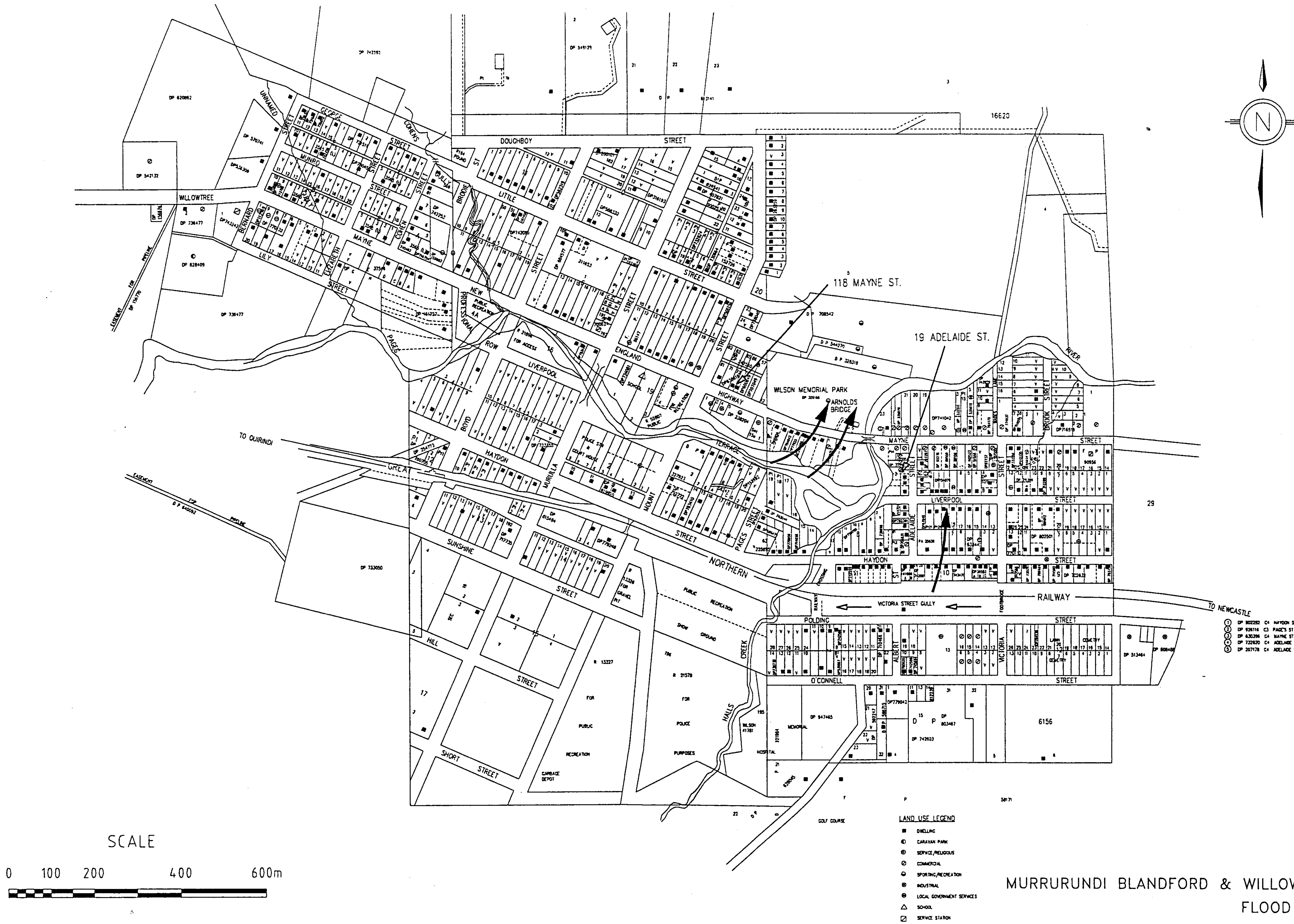
B.4 Willow Tree

The extent of flooding in Willow Tree was difficult to determine given that the area of land affected is located mainly within the floodway. This land is low-lying and is virtually free of structures, apart from rural fences, floodgates, flooding crossings and buildings used for agricultural purposes. Very few flood levels were recorded in Willow Tree, as virtually all fences in the path of the flood were severely damaged.

In general terms, the flood water followed the natural floodway associated with Borambil and Chilcotts Creeks and broke its banks in a number of places including the southern side of Hams Bridge, behind "Meadow Bank" and behind the Bowling Club and Sports Ground. A significant amount of damage was caused to the fencing in these localities. On the south-eastern side of Hams Bridge, water from Chilcotts Creek is reported to have extended to as far as the railway line.

The school residence was inundated by the floodwaters, however a number of properties received water through outbuildings, as well as significant damage to fences. The severe damage to the fencing surrounding the residence left no evidence from which to gain an indication of the flood level in this area.

Other damage at Willow Tree occurred to the Willow Tree Park and residences in Cadell Street, located at the northern end of the town. Inundation at these localities resulted from a blocked stormwater culvert on the northern end of the Community Hall on the corner of the New England Highway and Sisson Street. Some levels were recorded from flood marks at Willow Tree Park.

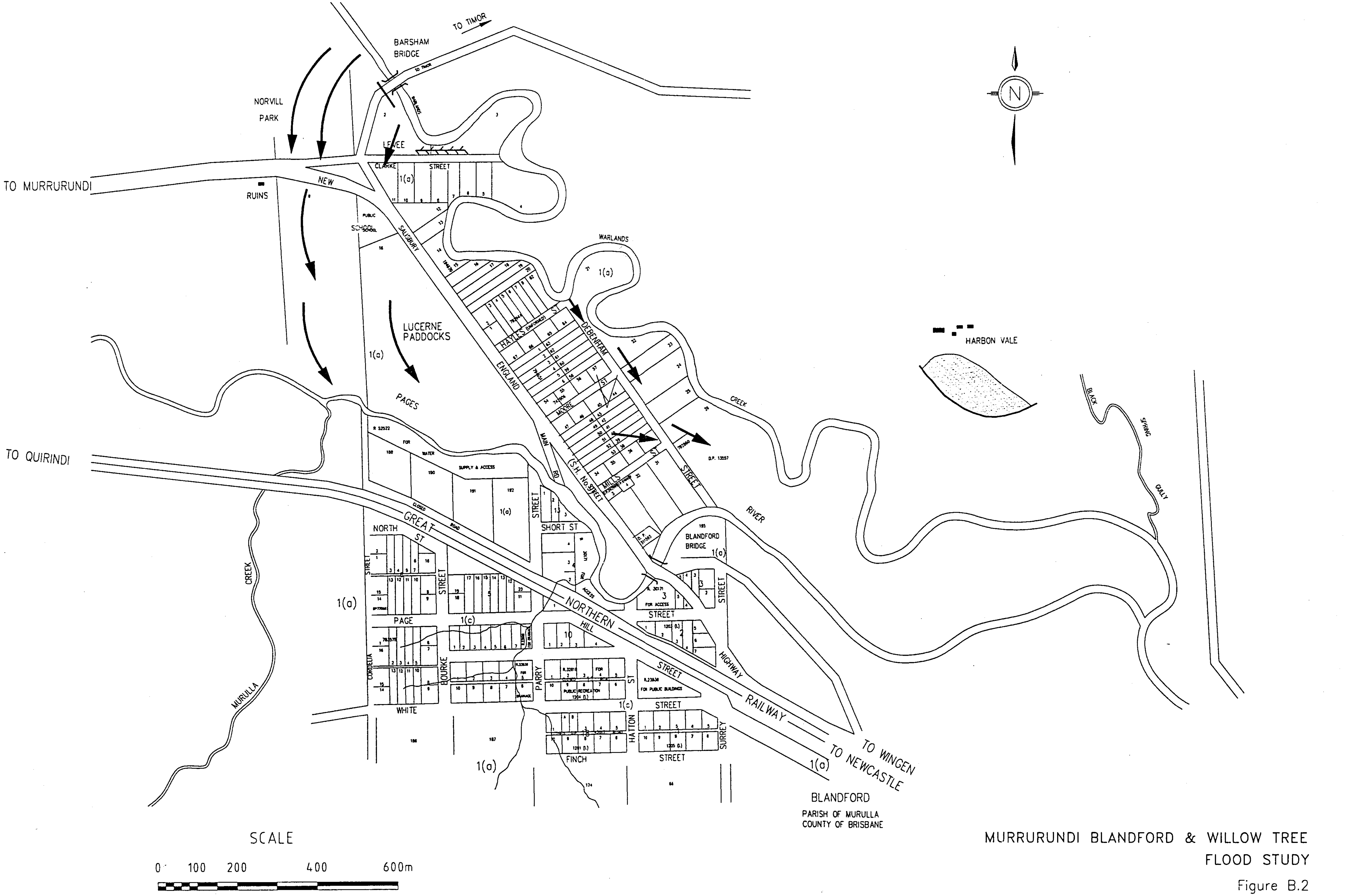


MURRURUNDI BLANDFORD & WILLOW TREE

FLOOD STUDY

Figure B.1

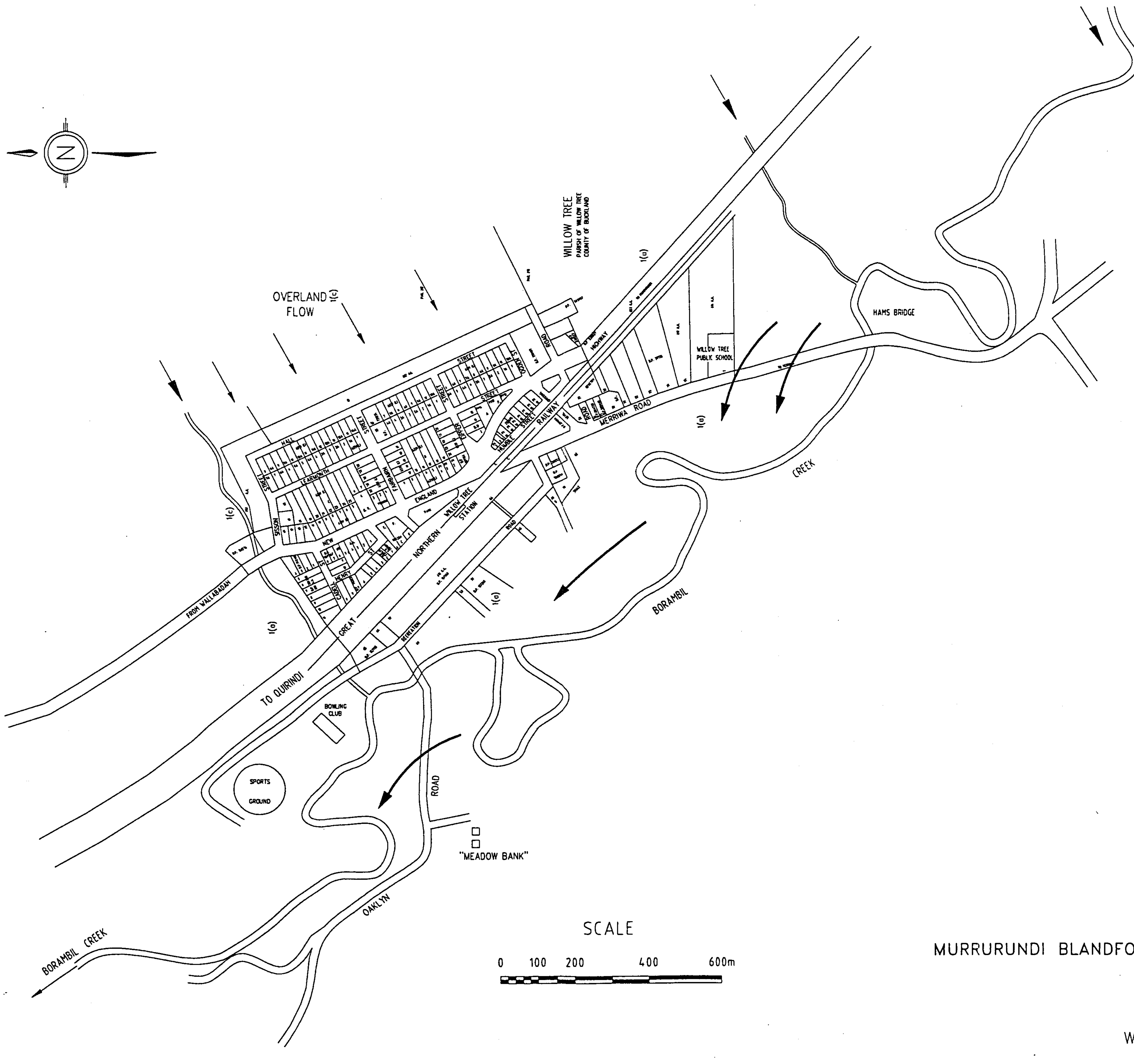
MURRURUNDI TOWNSHIP



MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure B.2

BLANDFORD TOWNSHIP



MURRURUNDI BLANDFORD & WILLOW TREE
FLOOD STUDY

Figure B.3

WILLOW TREE TOWNSHIP

APPENDIX C

ANNUAL PEAK FLOWS
BLANDFORD
GAUGING STATION

TABLE C1

ANNUAL PEAK FLOWS
BLANDFORD GAUGING STATION

Date	Time (24 hr)	Stage (m)	Discharge (m^3/s)
30.01.1984	205	8.01	948
07.08.1985	314	2.55	72
25.07.1986	205	1.81	32
15.08.1987	1912	2.01	41
06.07.1988	43	2.10	46
13.04.1989	512	4.66	273
1990		No Data	No Data
14.07.1991	1220	1.35	14
09.02.1992	945	7.74	867
18.10.1993	1945	4.22	220
20.11.1994	2145	1.36	16
10.12.1995	1800	4.58	264
25.01.1996	1015	8.32	1036

Note: 06.10.1996 2000 7.00 681